Owner-Directed Facility Seismic Evaluation

Benton County Courthouse
120 NW 4th Avenue, Corvallis, Oregon

Prepared for
Benton County

Prepared by

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TABLE OF CONTENTS

Executive Summary, Background and Scope of Work .....2 – 7

Photo Plan .....8 – 12

Photos.....13 – 32

Definitions.....33 – 35

Basic Performance Objective

Structural and Non-structural plans ..... B1 – B5

Performance Evaluation..... B6 – B11

Demand Capacity Ratio ..... B12 – B13

Cost Benefit Table ..... B14 – B17

Cost Tables..... B18 – B30

Enhanced Performance Objective

Structural and Non-structural plans .....E1 – E5

Performance Evaluation.....E6 – E11

Demand Capacity Ratio .....E12 – E13

Cost Benefit Table .....E14 – E17

Cost Tables.....E18 – E33

Limited Performance Objective

Structural plans .....L1 – L5

Performance Evaluation.....L6 – L11

Demand Capacity Ratio L12

Cost Benefit Table .....L13 – L16

Cost Tables.....L17 – L33

Appendix

BPOE Summary Data Sheet and Checklists.....APPENDIX – A

BPOE Supporting Calculations..... APPENDIX – B

EPOE Summary Data Sheet and Checklists ..... APPENDIX – C

EPOE Supporting Calculations.....APPENDIX – D

LPOE Data Sheets..... APPENDIX – E

LPOE Supporting Calculations.....APPENDIX – F

Supplemental Report Table of Contents.....(See after Appendix F)

BENTON COUNTY COURTHOUSE EXECUTIVE SUMMARY

Overview See the Supplemental Report for additional information and clarification.

Benton County has contracted Miller Consulting Engineers to seismically evaluate the Benton County Courthouse using the ASCE 41-13 standard. As the building sits today, the vertical system of the building consists of wood framing that bears on the unreinforced masonry (URM) walls and the lateral system is dependent on the friction from the framing bearing on the URM walls. The friction capacity is sufficient to transfer the lateral loads for wind loading to the existing lateral force-resisting system, but in the event of a seismic event, the friction capacity will be lost as a result of vertical ground movement and the building will be unable to transfer the lateral loads to the existing lateral force-resisting system. Subsequently, the building elements will begin to be damaged during the seismic event and the building may collapse depending on the seismic hazard's direction (where the earthquake originated), intensity (the severity of the ground movement) and duration (how long the event lasts).

The ASCE 41-13 standard has been developed so that buildings meeting the Basic Performance Objective (BPOE) are expected to experience little damage from relatively frequent, moderate earthquakes but significantly more damage and potential economic loss from the most severe and infrequent earthquakes. In order to seismically evaluate the building, the ASCE 41-13 uses site-specific information that is generated from the U.S. Geological Survey (USGS). The USGS information provides the spectral acceleration for different return periods in which the BPOE-1E seismic hazard has a 225-year return period and the BPOE-2E seismic hazard has a 975-year return period. This information is then modified into a design acceleration considering the building's structural system and other factors such as the soil conditions at the site.

We evaluated the building using the BPOE-1E seismic hazard, which is a considered a moderate earthquake and used the Life Safety and Immediate Occupancy Building Performance Level standards with the following goals.

- **Basic Performance Objective for Existing Buildings.** The goal of this objective is to allow the occupants to safely exit the structure after an earthquake. Continued occupancy might not be likely before repair work is completed.

- **Enhanced Performance Objective for Existing Buildings.** The goal of this objective is to allow the building to be immediately operable after an earthquake. It is expected that the building will develop cracks and items that are damaged will remain in place.

We also evaluated the building using the BPOE-2E seismic hazard, which is a considered a severe earthquake and used the Collapse Prevention Building Performance Level standards with the following goal.

- **Limited Performance Objective for Existing Buildings.** The goal of this objective is to prevent the building from collapsing during a major earthquake. It is expected that the building will be severely damaged and should not continue to be occupied following the earthquake.

This two tier method of evaluation produces an envelope of results for the building using the performance levels and seismic hazards to provide feedback to where the building is deficient and to what degree the building is deficient. The evaluation identified the following items that need to be addressed considering each of the performance objectives as part of the proposed seismic upgrade:

- Strengthening the roof and floor diaphragms to transfer lateral loads to the walls
- Providing connections between the walls and diaphragm for in-plane and out-of-plane loads
- Providing connections at the roof and floor diaphragm to account for plan irregularities
- Providing concrete shear walls on the lower floors to support the in-plane loading
- Providing structural steel horizontal framing members at the stairway openings to support the unreinforced masonry walls for out-of-plane loads
- Providing structural steel support columns for the secondary support of the trusses and beams that are independent of the unreinforced masonry walls

When considering the basic and enhanced performance objectives, there are additional items that need to be addressed including the following:

- Addressing the non-structural concerns identified in the evaluation including bracing the fire suppression lines, bracing the gas line, bracing the suspended gypsum ceilings, bracing the in-line mechanical equipment and anchoring the non-compliant shelves.

Lastly, there are additional items that need to be addressed when considering the enhanced and limited performance objective including the following:

- Providing bonded shear walls to reinforce the unreinforced masonry walls at the fourth floor bell lounge for out-of-plane loads
- Providing more concrete shear walls for many of the unreinforced masonry walls to support the in-plane loading

Depending on what performance objective is selected, we anticipate that the proposed upgrade work will take approximately 12 to 24 months to complete and will require the building to be vacated during construction. It is intended that the above repairs will be completed in such a way that the new construction will be encapsulated within a wall cavity or masked in some manner to preserve the historical and architectural features where possible. The following is our preliminary estimate of probable construction cost to upgrade the building:

Performance Objective	Construction	Contingency	Total Cost	Cost per sq. foot
Basic	$4,312,271	$1,078,068	$6,737,923	$204
Enhanced	$5,824,570	$1,456,143	$9,100,891	$275
Limited	$6,966,813	$1,741,703	$10,885,645	$329

The above costs only consider the structural upgrades and corresponding work as required by the structural upgrades; the other concerns with the building such as HVAC, plumbing, ADA access or other tenant improvement costs are not included in this cost. The above cost estimate is based on the work being completed as a whole and not broken into phases to allow for the continued occupancy of the building during construction. Furthermore, these estimates are limited to the repairs of the detailed items and do not include relocation costs of staff or other soft costs. Within our opinion of probable construction cost estimate, we have included the contractor's overhead and profit, as well as a 25% construction contingency as further detailed in the cost portion of this report. See the Supplemental Report for additional cost information and clarification.

Background

The Benton County Courthouse is located at 120 NW Fourth Avenue in Corvallis, Oregon, and is Oregon's oldest active courthouse. The building was originally constructed around 1889 with one major renovation that was completed circa 1978 that added the third and fourth floors, updated all the interior spaces, and added central air conditioning in the courtrooms. The current size of the building is approximately 33,052 square feet. Original construction documents were

not available for our review, but the plans from the remodel from Cy Stadsvold (1977) were reviewed, of which the basic floor plan has changed very little since the remodel was completed. Also, we received and reviewed documents from K & D Engineering (1985), Endex Engineering (2001), Hennebery Eddy Architects (2008), KPFF (2012) and Pillar Consulting Group (2015), which were used in our evaluation as well. We also conducted three visual site observations at the building observing all the accessible spaces including the clock tower, the elevator shaft, the attic spaces, the third floor cavity areas and above the restrooms on the second floor.

The courthouse building is a five-story structure comprised of three primary areas on each floor, which consist of a central core area and an east and west wing area to the east and west of the core area. The basement level is a partially exposed story with the concrete floor slab that is approximately 4 feet below the exterior grade, but has an ADA-accessible ramp on the west side of the building, as well as an employee entry access on the east side of the building. The first floor of the courthouse is approximately 6 feet above the exterior grade and houses the main public access entry on the east side of the building. The building houses the county courts, the judge's offices, and the district attorney's offices and support staff, as well as several other county departments including the elections office and the licensing office.

The exterior walls of the building consist of 30" unreinforced stone walls at the basement, 20" unreinforced brick walls with stucco exterior finish at the first floor, 16" unreinforced brick walls with stucco exterior finish at the second floor, and 12" unreinforced brick walls with stucco exterior finish at the third and fourth floors. The interior load bearing walls of the building typically consist of 12" unreinforced brick walls while the non-load bearing interior walls typically consist of light wood framing with plaster finish. The clock tower is a light wood-framed structure with board sheathing that sits atop the 12" unreinforced brick walls of the fourth floor bell lounge room in the east wing; the clock tower walls consist of battered light wood framing with four primary support columns that support each section of the tower and extend from the lower section floor (bell level) to the top of the middle section (clock level). The upper section (flag level) has steep 3x rafters that are supported at the floor with a tension ring and at the peak by a compression ring.

The roof framing of the building consists of 2x roof rafters at 24" o.c. typically that bear on the exterior walls with straight tongue-and-groove lumber roof sheathing (the lumber is installed

perpendicular to the framing). The rafters are notched and toenailed to a 4 x 12 wood bearing plate that was set in a bed of mortar on top of the brick walls that also has small diameter bolts (3/8" diameter) spaced at 10' o.c. that appear to be embedded into the top of the brick wall. The bolts have wood spacers for unknown reasons of varying length where the nut bears on top of the spacer. The core area roof is framed differently than the wings and has heavy timber trusses at 12' o.c. that bear on heavy steel plates on the exterior brick walls, which the trusses support the roof and ceiling framing. However, the heavy steel plates at the truss bearing locations do not appear to be anchored or connected to the top of the brick wall in any way.

The floor framing of the building consists typically of 3x floor joists spaced at 16" o.c. with straight tongue-and-groove lumber floor sheathing observed directly above the joists. It was reported that the floors either have another layer of ¾" finished wood flooring or a layer of ¾" plywood sheathing above the base layer. The floor joists have varying bearing conditions, which include the following: bearing on a wood bearing plate at a ledge where the bearing wall thicknesses change; embedded in the bearing wall where the wall thickness does not change; or bearing on a wood bearing plate on top of the wall, which only occurs at the clock tower. In each of these bearing conditions, there was no positive connection observed from the floor structure to the wall in order to transfer in-plane or out-of-plane loads from the diaphragm to the walls or vice versa. Where the floor joists are parallel with the walls, there is a joist in close proximity to the wall, but there was no positive connection observed from the joist to the wall either.

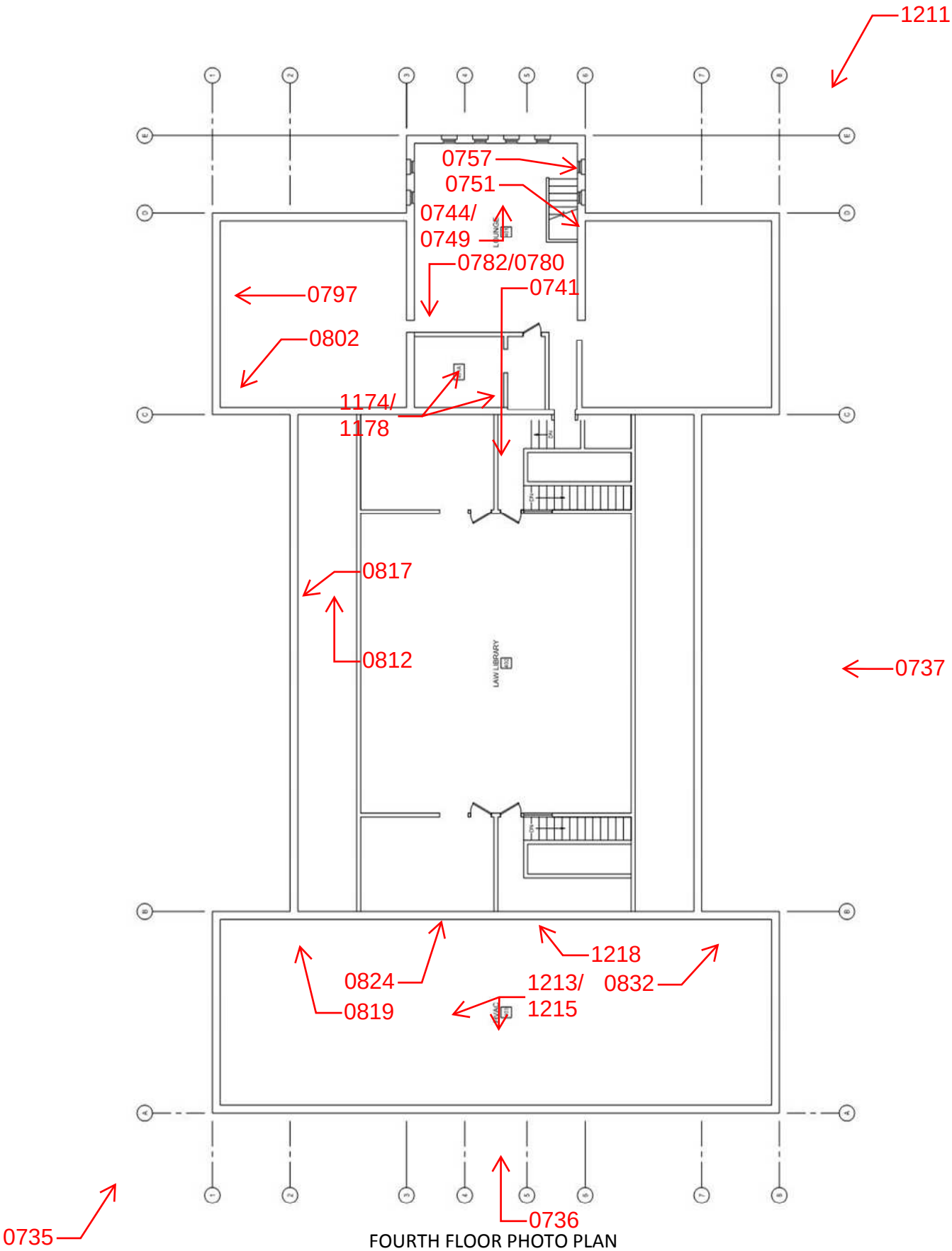
Scope of Work

Miller Consulting Engineers, Inc. was contracted by Benton County to provide an ASCE 41-13 evaluation of the county courthouse located in Corvallis, Oregon, using three criteria found in the *ASCE 41-13* document. The first criterion was considered the Basic Performance Objective (BPOE) and utilizes the Basic Safety Objective for Life-Safety (3-C). The second criterion was considered the Enhanced Performance Objective (EPOE) and utilizes an immediate occupancy structural performance level (S-1) and a non-structural performance level (N-B), which is position retention of non-structural components. And the third criterion was considered the Limited Performance Objective (LPOE) and utilizes a collapse prevention structural performance level (5-C) and a non-structural performance level (N-D), in which non-structural components are not evaluated. Both the BPOE and the EPOE use the BSE-1E seismic hazard level, which is an earthquake that has the probability of exceedance of 20% in 50 years or a mean return period

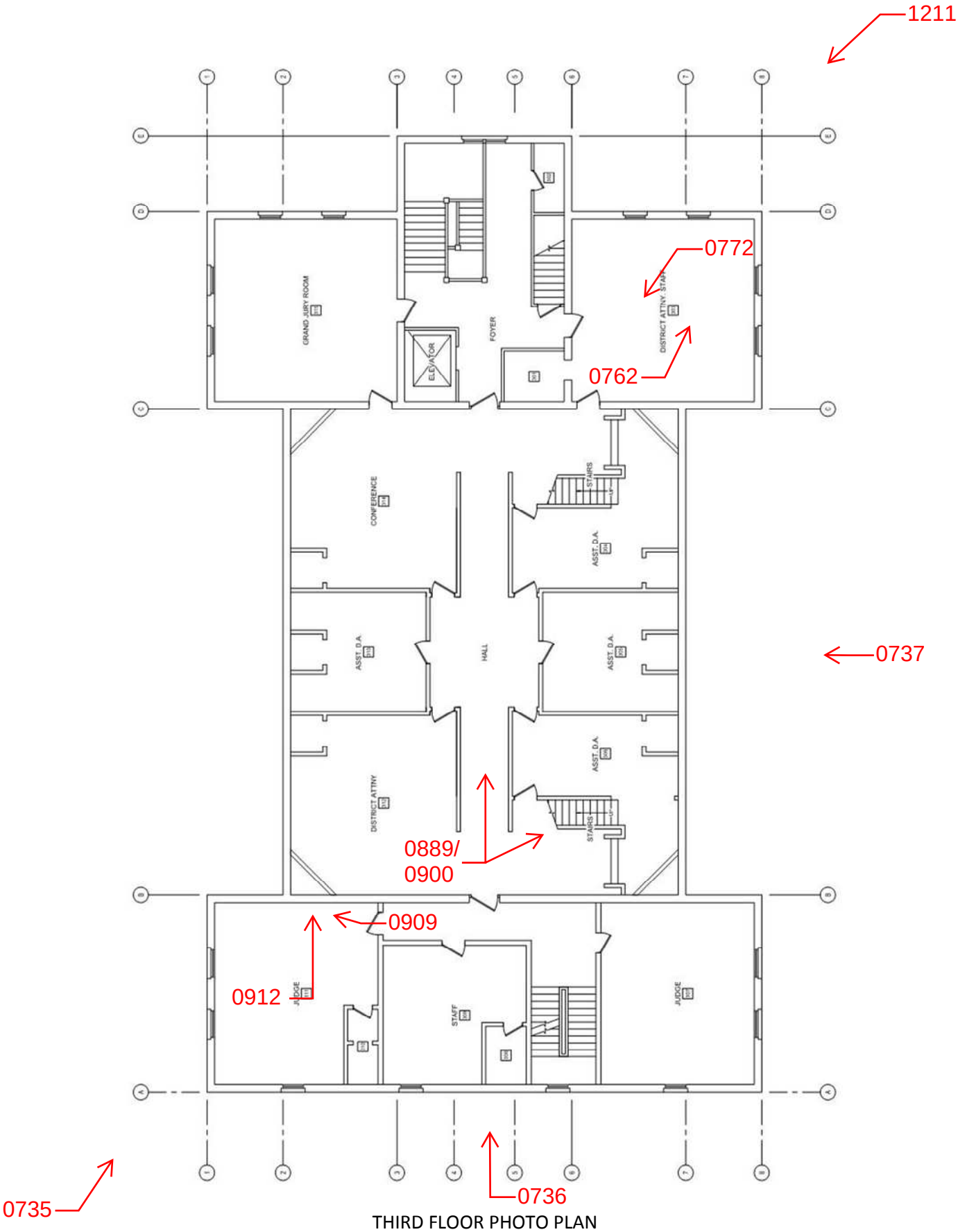
of 225 years between occurrences. The LPOE uses the BSE-2E seismic hazard level, which is an earthquake that has the probability of exceedance of 5% in 50 years or a mean return period of 975 years between occurrences.

For each of the evaluations, the non-structural reviews exclude building contents and furnishings. Furthermore, soils information was based on information obtained from the Oregon Department of Geology and Mineral Information (DOGAMI) website, which provided the site classification, soil rupture information, and liquefaction information for this study.

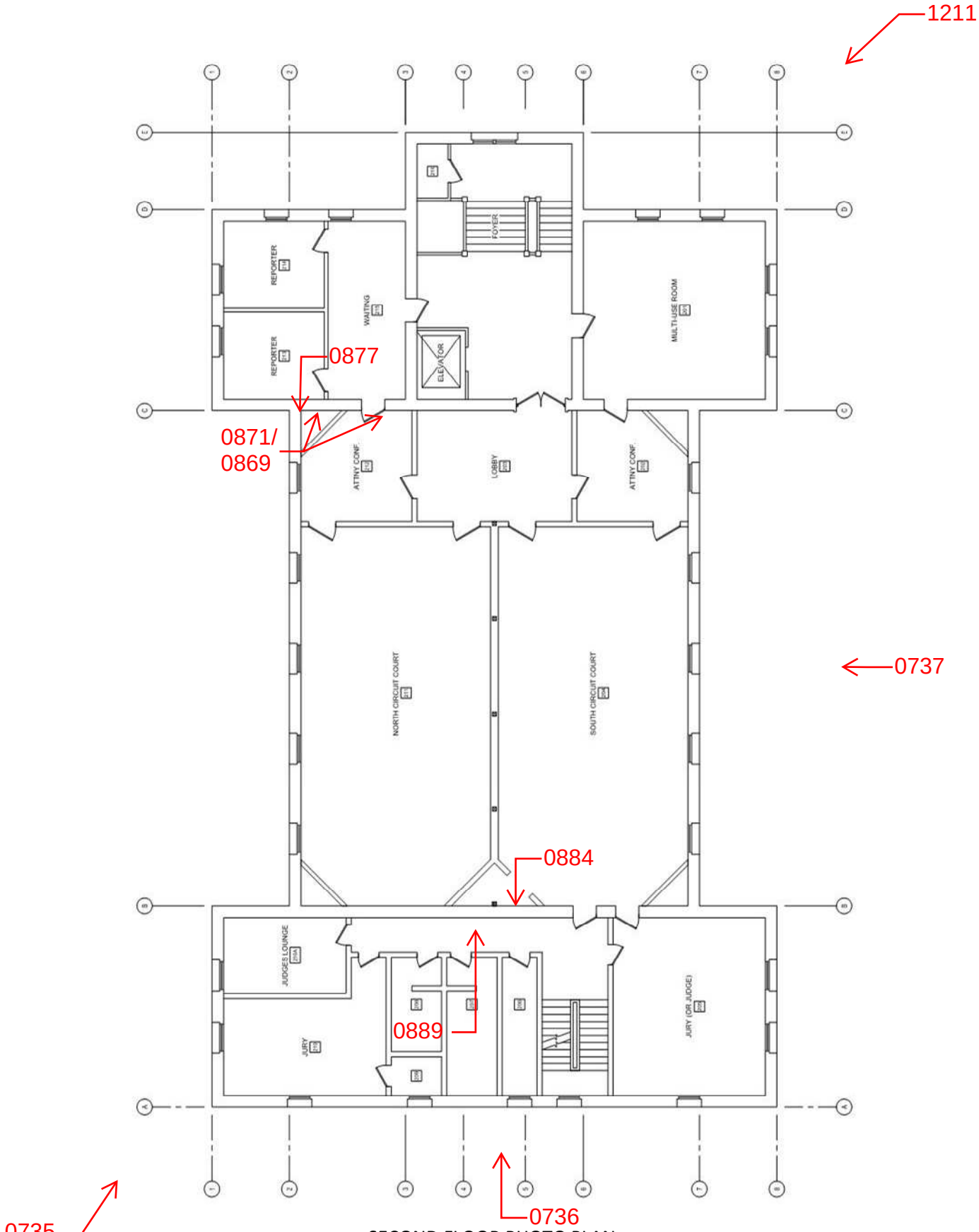
Miller Consulting Engineers, Inc. (MCE) also provided an engineer's preliminary estimate of probable construction costs to correct deficiencies found based on the evaluations. These estimates are limited to the repairs of the items and do not include relocation costs of staff or other soft costs.



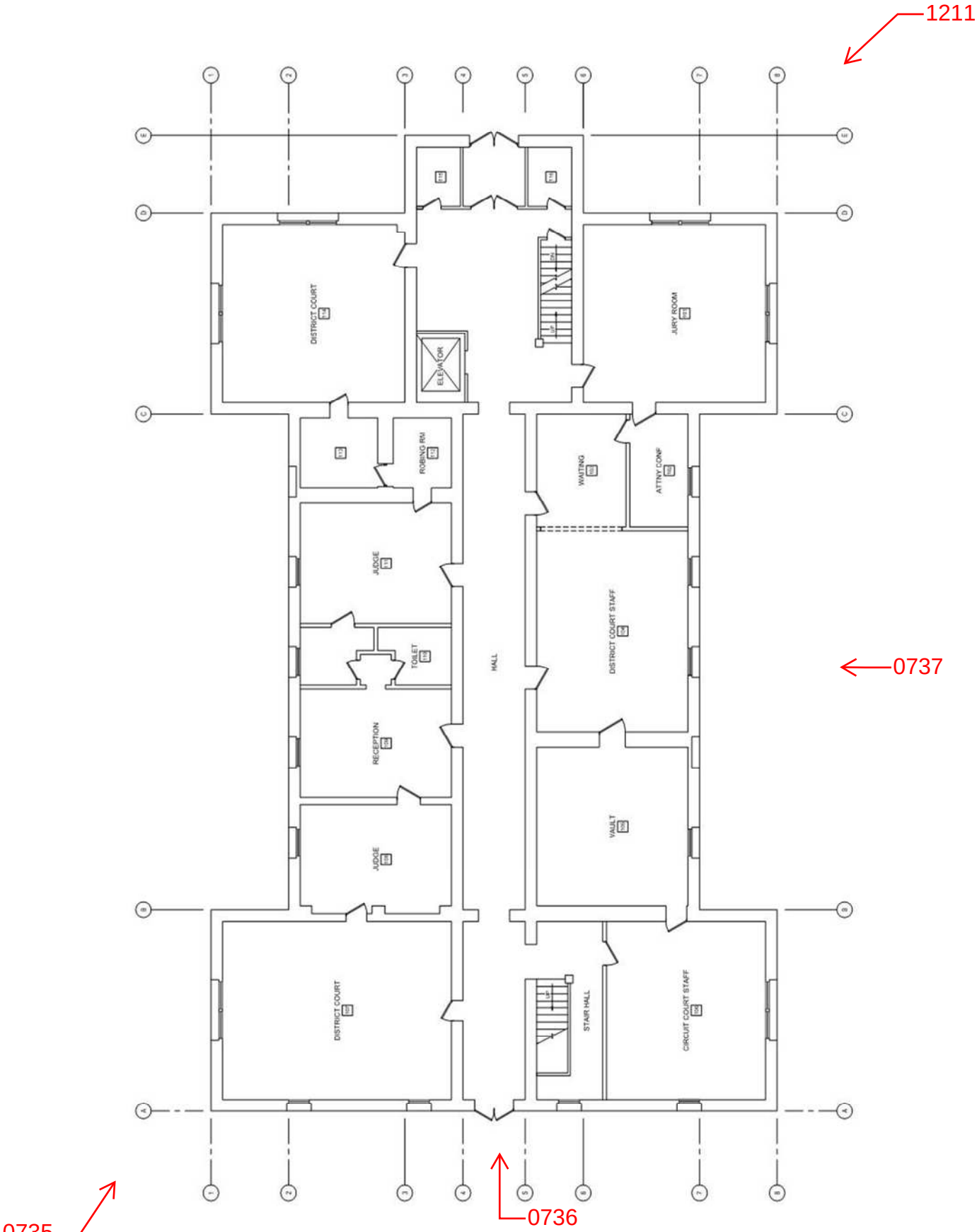
FOURTH FLOOR PHOTO PLAN



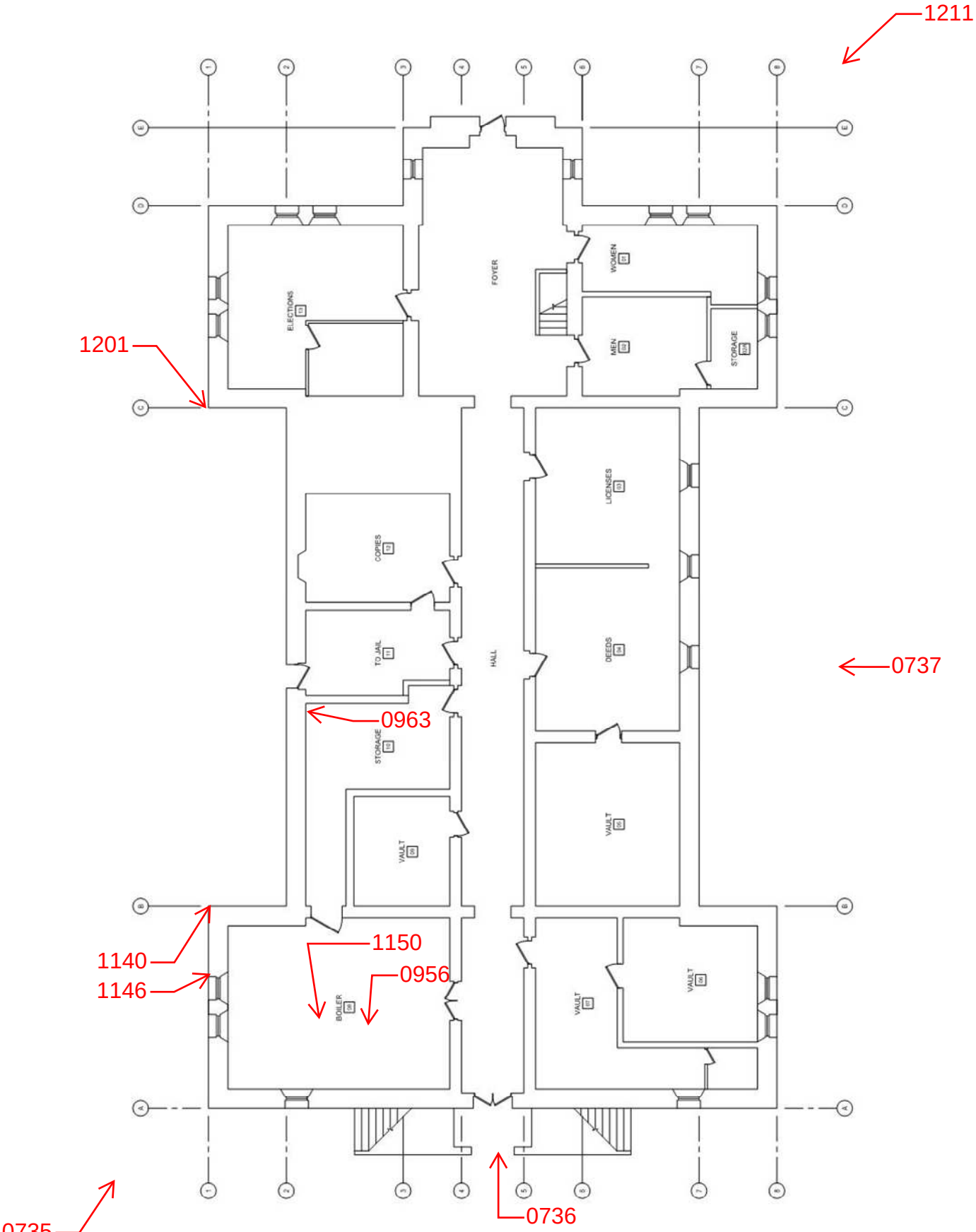
THIRD FLOOR PHOTO PLAN



SECOND FLOOR PHOTO PLAN



FIRST FLOOR PHOTO PLAN



BASEMENT FLOOR PHOTO PLAN

PHOTO 1211 – East elevation of the building looking toward the southeast corner

PHOTO 0737 – South elevation of the building looking toward the north

PHOTO 0736 – West elevation of the building looking toward the east

PHOTO 0735 – North elevation of the building looking toward the northwest corner

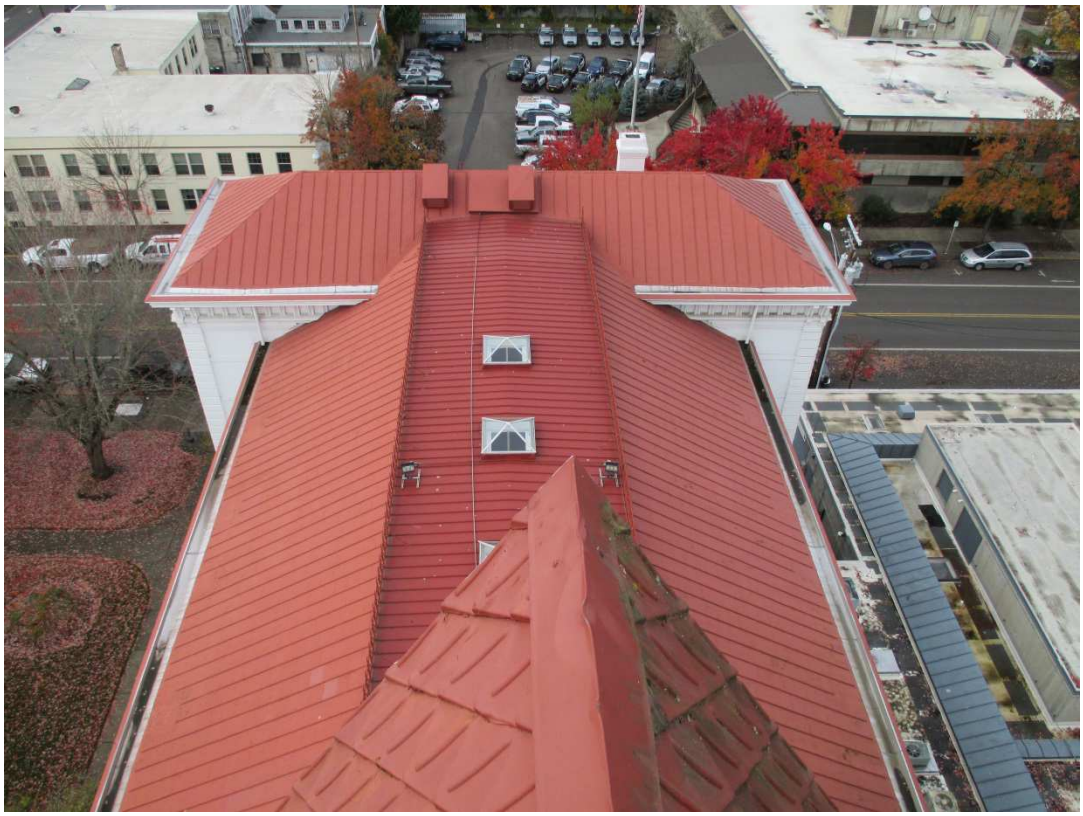


PHOTO 0741 – West wing and core roof area taken from flag level of the clock tower looking west

PHOTO 0744 – Compression ring at the top of the clock tower (flag level)

PHOTO 0749 – Light wood framing of the clock tower showing the main support columns

PHOTO 0757 – Light wood framing of the clock tower bearing on top of the bell lounge brick walls

PHOTO 0802 – Roof and ceiling framing in the northeast attic area of the east wing

PHOTO 0797 – Visible wall anchor at the ceiling joists in the northeast attic area of the east wing

PHOTO 0812 – Heavy timber truss, roof framing and fourth floor framing visible in the core area

PHOTO 0817 – Heavy steel plate visible at the heavy timber truss bearing point in the core area

PHOTO 0819 – Roof and ceiling framing in the northwest attic area of the west wing

PHOTO 0824 – Small anchor visible in the wood bearing plate in the west wing attic area

PHOTO 1218 – Visible wall anchor at the ceiling joists in the west wing attic area

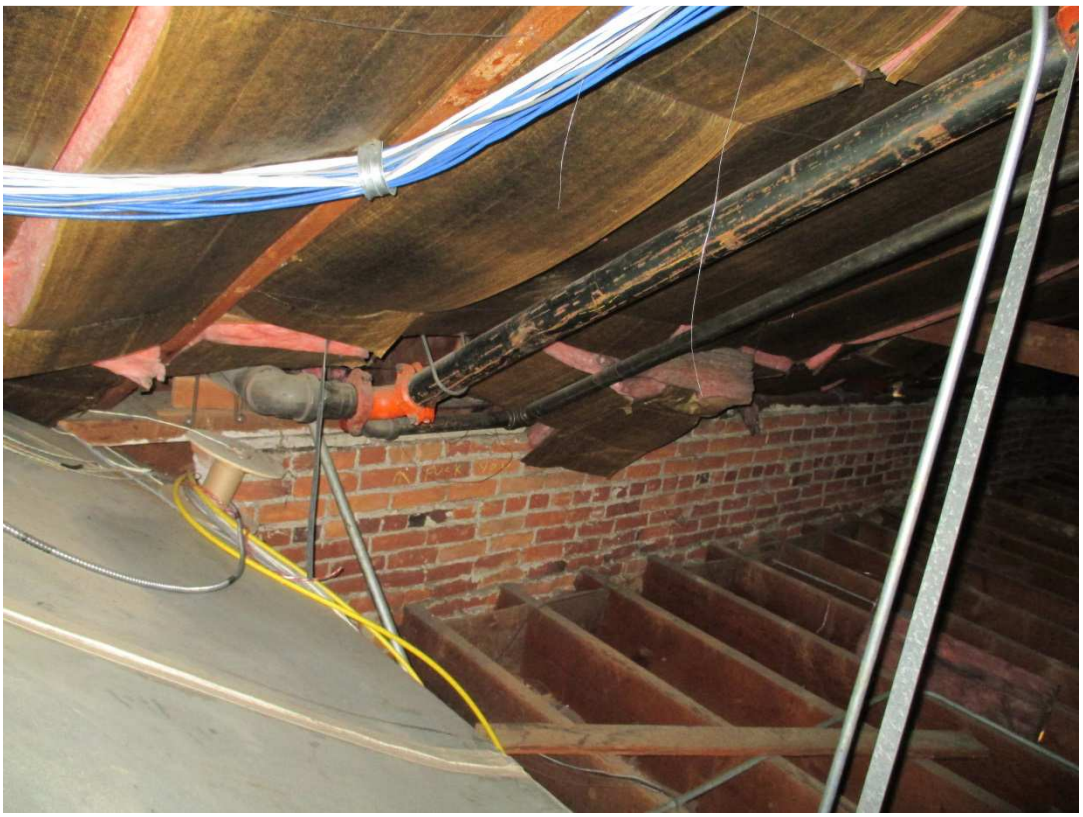


PHOTO 0832 – Roof and ceiling framing in the southwest attic area of the west wing

PHOTO 0782 – West wall of the 4th floor Bell Lounge room with crack gauge visible on the right side

PHOTO 0780 – Close-up of the crack gauge with no movement indicated

PHOTO 1178 – Elevator control panel anchored to the south wall of the elevator room

PHOTO 1174 – Elevator lift equipment anchored to the floor of the elevator room

PHOTO 0772 – Hanging light fixtures from the suspended gypsum ceiling in Courtroom #3

PHOTO 0862 – Unbraced suspended gypsum ceiling in Courtroom #3 looking southeast

PHOTO 0871 – Third floor framing visible from the HVAC opening in Room 212 looking east

PHOTO 0869 – Third floor framing visible from the HVAC opening in Room 212 looking south

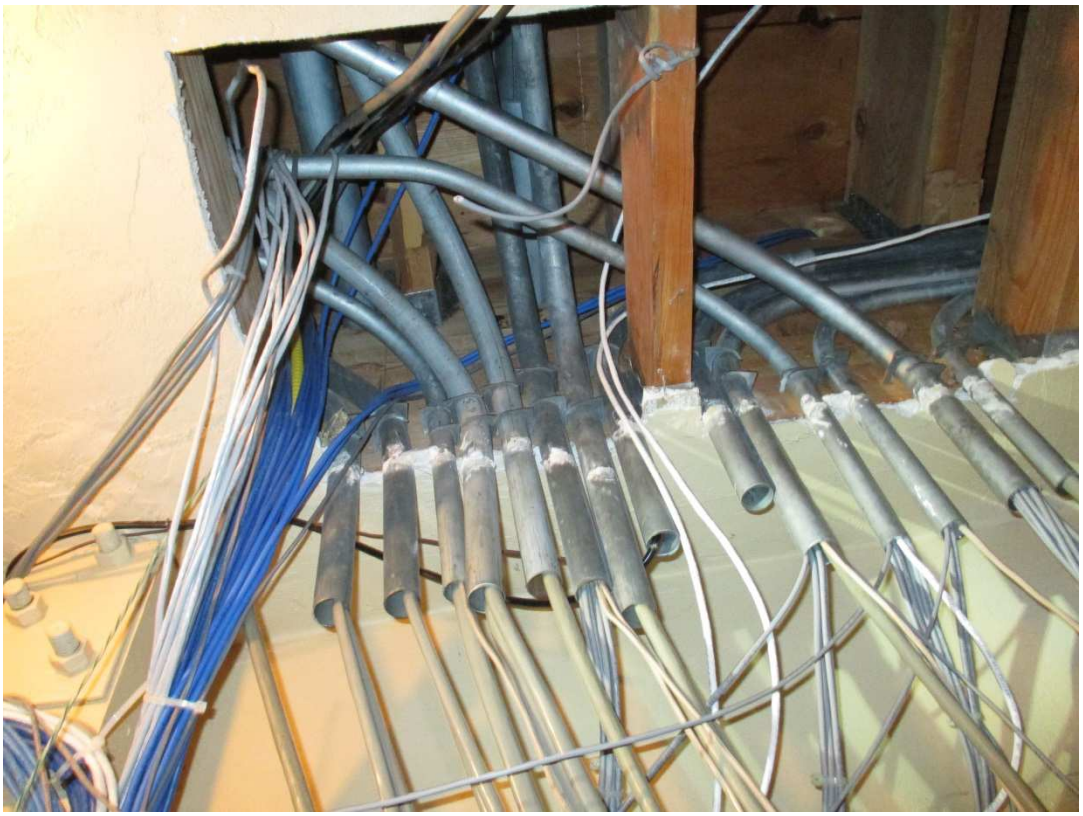


PHOTO 0884 – Third floor framing visible from the IT closet in Room 204 looking west

PHOTO 0889 – Third floor framing above Room 207 with the east brick wall of the west wing visible

PHOTO 0878 – Duct space in Room 207 with the north and east brick walls of the core visible

PHOTO 0963 –First floor supported on a wood bearing plate at the wall step in Storage Room 10

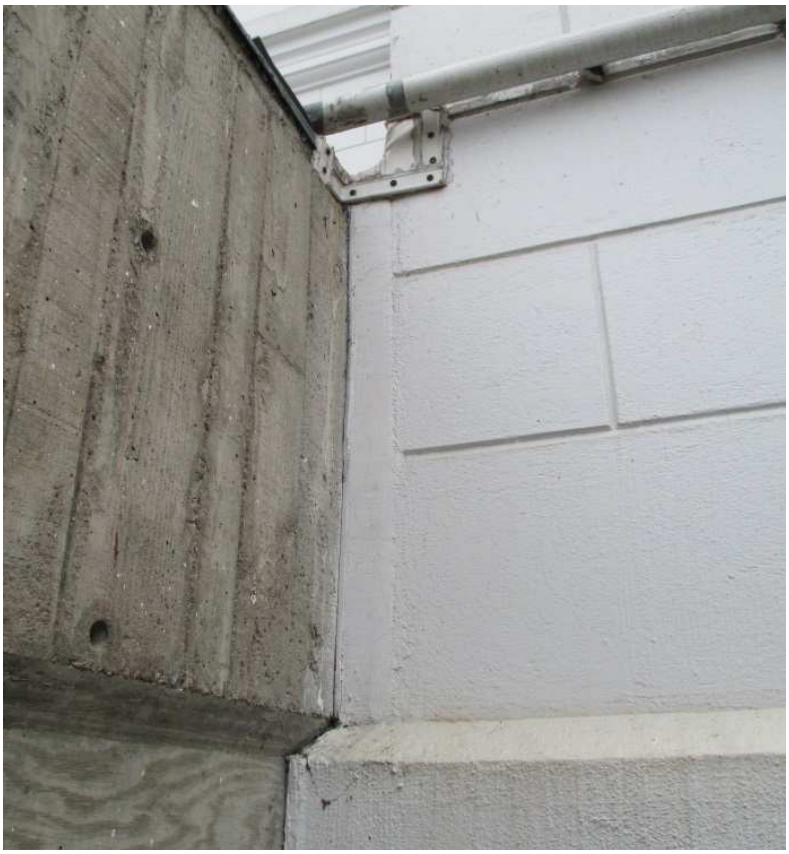


PHOTO 1140 – Interface between the jail and the courthouse at the west side of the jail

PHOTO 1201 – Interface between the jail and the courthouse at the east side of the jail

PHOTO 0889 – Unanchored tall contents located on the third floor at the west end of the hallway

PHOTO 0900 – Unanchored tall contents located on the third floor adjacent to the southwest staircase

PHOTO 0912 – Unanchored tall contents located on the east wall of Room 311

PHOTO 0909 – Unanchored tall contents located on the east wall of Room 311

PHOTO 0956 – Unanchored boiler sitting on concrete slab on grade in the Boiler Room

PHOTO 1150 – Unbraced gas line and suspended in-line HVAC equipment in the Boiler Room

PHOTO 1213 – Unanchored HVAC equipment in the HAVC Attic 403

PHOTO 1215 – Unanchored HVAC equipment in the HAVC Attic 403

PHOTO 0751 – Wood deterioration observed along the south wall of the clock level in the tower

PHOTO 1146 – Soft mortar observed at the exposed wall at the northwest corner of the west wing

A list of definitions is included below to assist the reader with the technical terms used throughout this report.

Definitions

Clerestory: A vertical step in the roof containing windows.

CMU Wall: A Concrete Masonry Unit wall is constructed of modular hollow concrete blocks that are attached together with mortar.

Collector: A component that transfers lateral load from the building diaphragm to the lateral force resisting system.

Concrete Tilt Panel: A concrete wall that was constructed flat on the ground and then tilted into place.

Continuity Tie: A structural tie, such as a strap, to ensure transfer of loads from one member to another aligned member.

Diagonal Sheathing: A type of roof sheathing where 1x wood members are nailed at a 45 degree angle across roof or wall framing.

Diaphragm: A roof or floor that supports walls.

Dynamic Landslide: A landslide caused by an earthquake.

Essential Facilities (2012 IBC): Buildings and other structures that are intended to remain operational in the event of extreme environmental loading from flood, wind, snow, or earthquakes. The word operational in this definition is the distinguishing link between the ASCE 41-13 and the 2012 IBC.

Glazing: The glass portion of a window.

Glulam Beam: An engineered wood beam constructed of many layers of wood glued together.

Immediate Occupancy (ASCE 41-13): Overall damage is light; no permanent drift; the structure substantially retains original strength and stiffness; and continued occupancy is likely. Equipment is generally secure but might not operate due to mechanical failure or lack of utilities. There may be some cracking of facades, partitions, and ceilings, as well as structural elements. All systems important to normal operation may not be functional.

Immediate Occupancy Structural Performance Level (S-1) (ASCE 41-13): The postearthquake damage state in which a structure remains safe to occupy and essentially retains its preearthquake strength and stiffness.

Life-Safety Structural Non-Structural Performance Level (N-C) (ASCE 41-13): Performance criteria in which non-structural components may be damaged, but the consequential damage does not pose a life-safety threat.

Life-Safety Structural Performance (ASCE 41-13): Postearthquake damage to a structure is such that the building retains capacity against onset of partial or full collapse.

Liquefaction: The rapid shaking of the soil during an earthquake, which results in the soil acting like a liquid rather than a solid, reducing the ability of the soil to carry the weight of the above structure.

Load Path: A path through which seismic forces travel through the building to the foundation.

Non-Structural Component: A permanently installed covering, mechanical or electrical component that does not support the primary structure.

Open Web Joist: A structural steel member composed of a top and bottom component with diagonal and vertical members linked in between.

Operational (ASCE 41-13): Overall damage is very light; no permanent drift; the structure retains substantial original strength and stiffness. The occupancy is continuous and use of the structure is highly likely. Negligible damage occurs to the equipment and power and other utilities are available, possibly from standby sources. There is minor cracking of facades, partitions, ceilings, and structural elements. All systems important to normal operation are functional.

Out-of-plane (forces/wall): Loads that act on a wall that typically are trying to push or pull the wall away from the floor or roof.

Performance Objective: One or more pairings of the selected hazard level with the acceptable or desirable structural and non-structural performance level.

Pilaster: A supporting column whose partial girth protrudes from the wall.

Position Retention Non-structural Performance Level N-B (ASCE 41-13): Non-structural components might be damaged to the extent that they cannot immediately function, but are secured in place so that damage caused by falling, toppling, or breaking of utility connections is avoided. Building access and life-safety systems, including doors, stairways, elevators, emergency lighting, fire alarms and fire suppression systems, generally remain available.

Post-Installed Anchor: A bolt that is installed in hardened concrete.

Primary Structure: A portion of a structure that is used to support gravity, snow, rain or wind and earthquake loads.

R&R: Remove and replace.

Seismic Evaluation: A study to evaluate a building's capacity to resist loads from an earthquake.

Shear Wall: A wall that resists seismic forces applied to it that are transferred from the roof and/or floor levels.

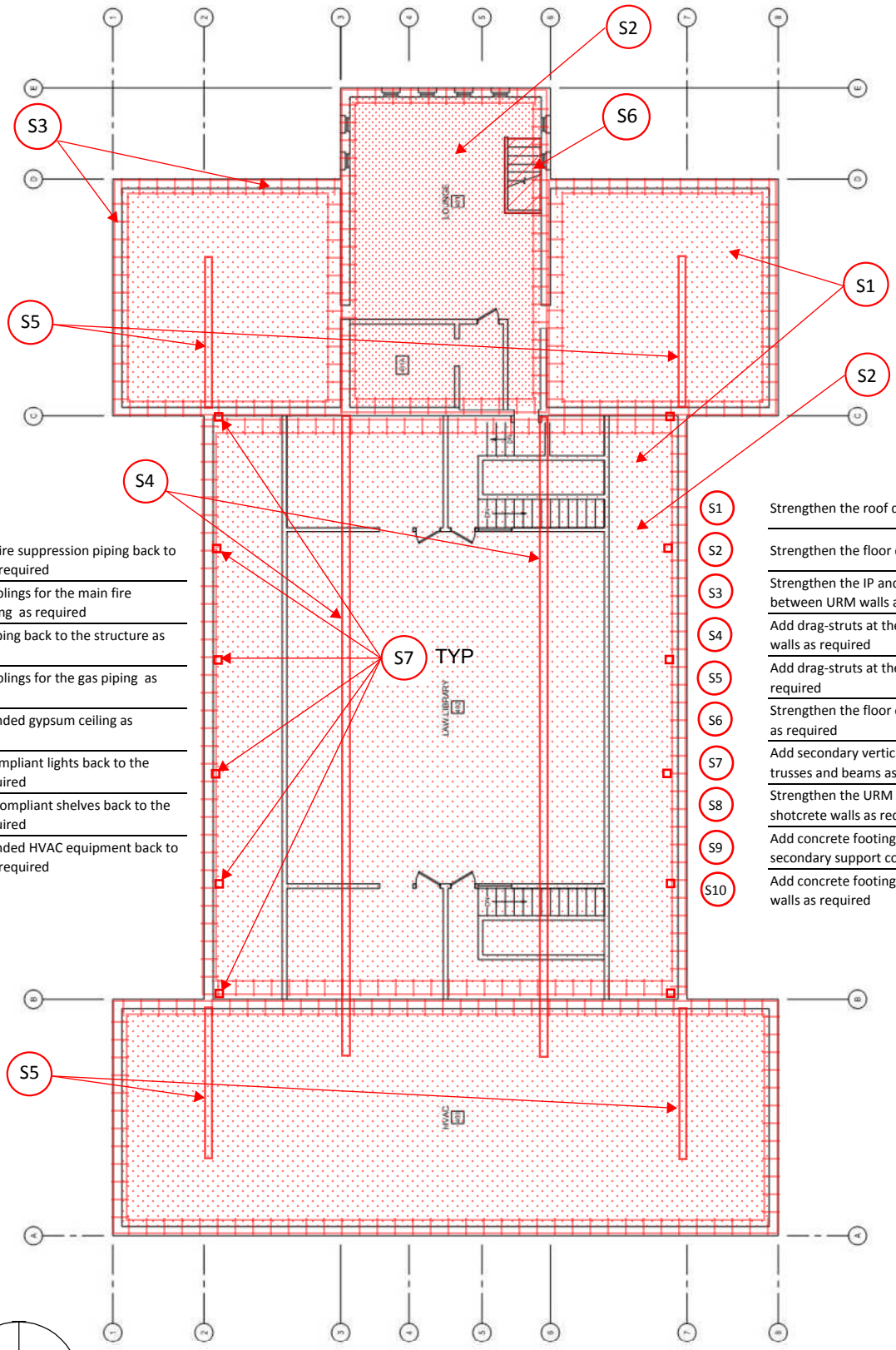
Ship-lap: A type of roof sheathing where 1x wood members are spaced over roof framing and overlap with adjoining members.

Special Concentrically Braced Frame: A steel frame that resists lateral loads through its diagonal members.

Straight Sheathing: A type of roof sheathing where 1x wood members are nailed perpendicular to roof or wall framing.

Surface Fault Rupture: Where a fault line opens up creating a crack in the soil.

BASIC PERFORMANCE OBJECTIVE



N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required

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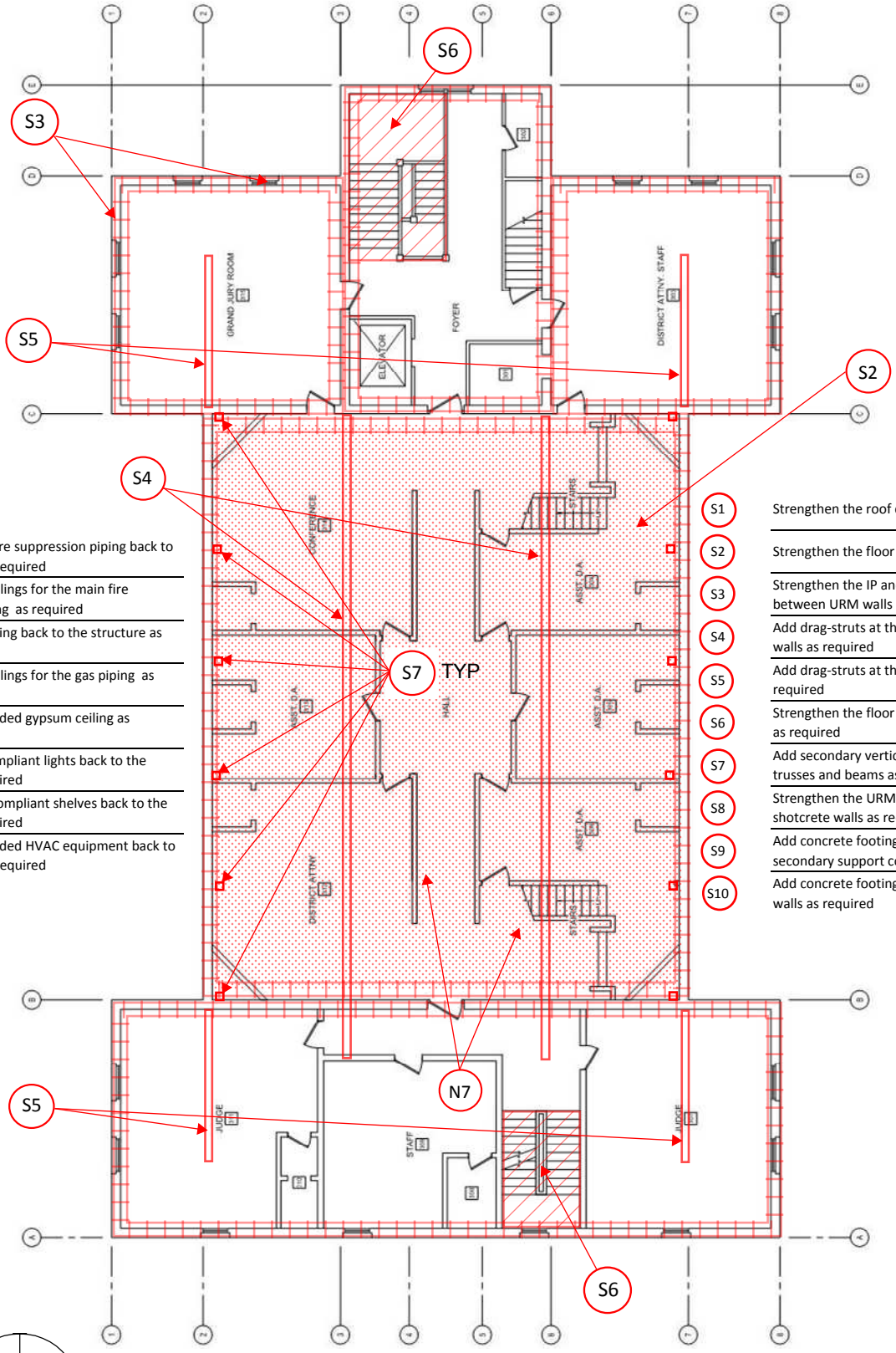
FOURTH FLOOR PLAN



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N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required

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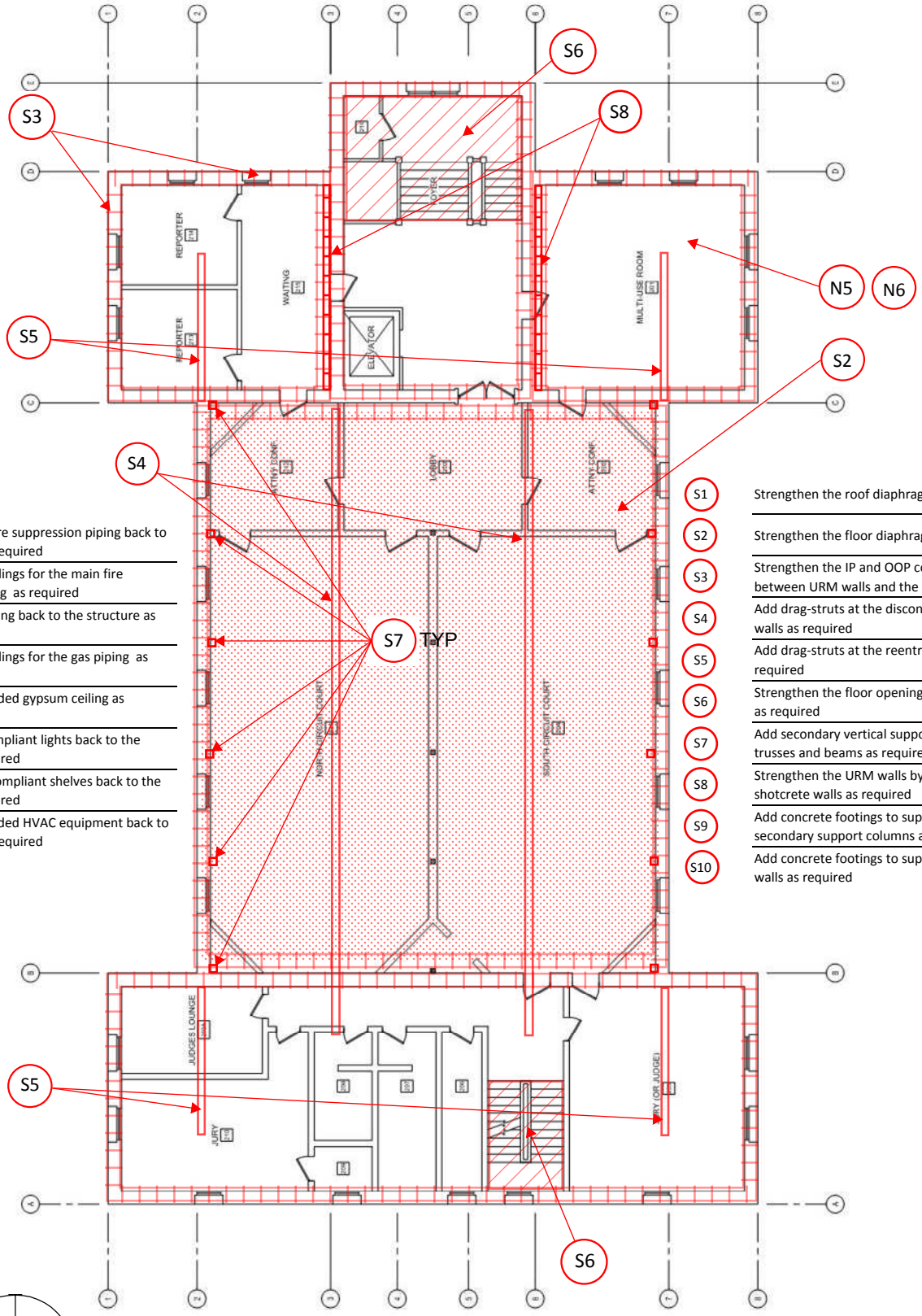
THIRD FLOOR PLAN



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N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required

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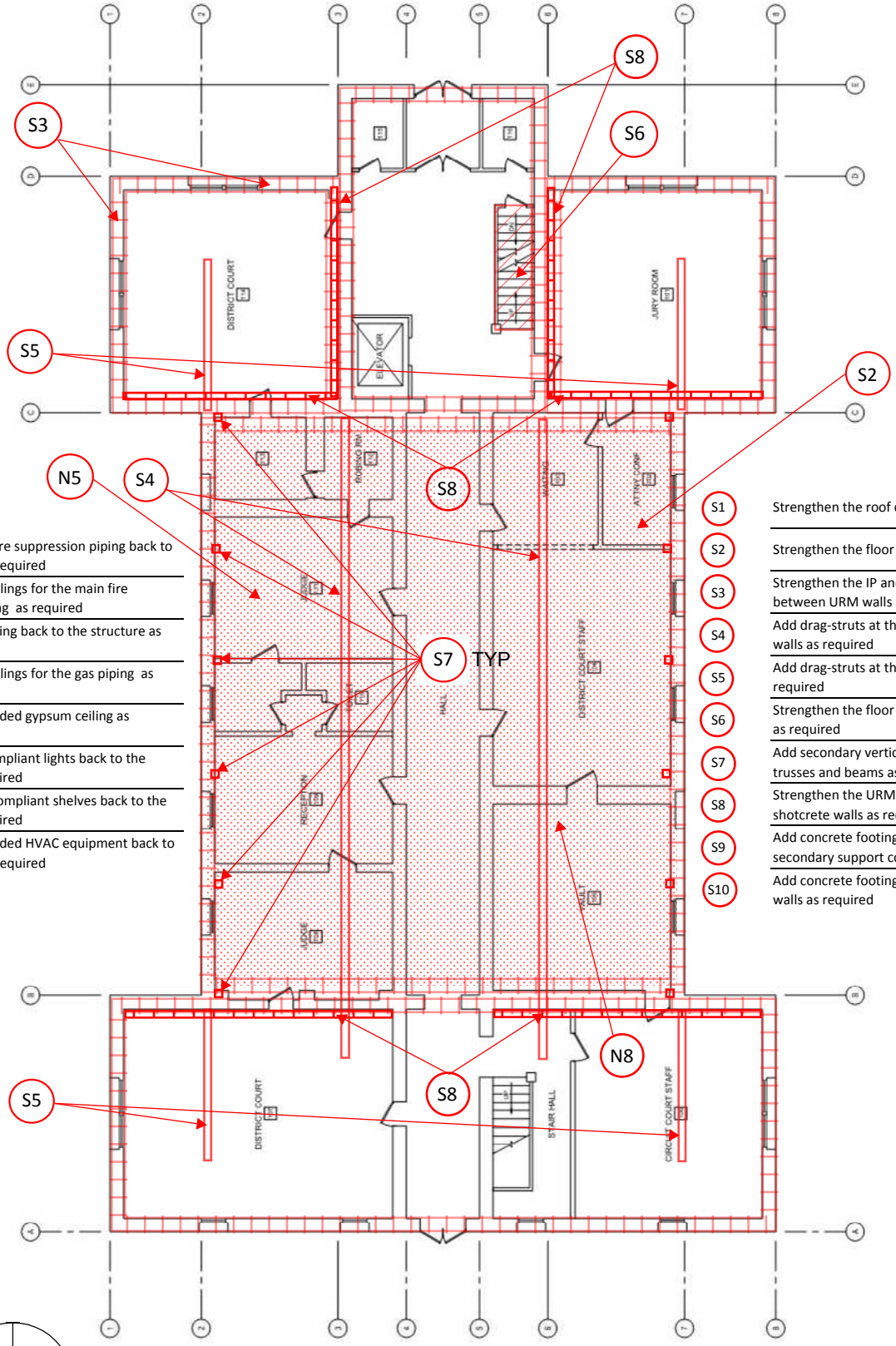
SECOND FLOOR PLAN



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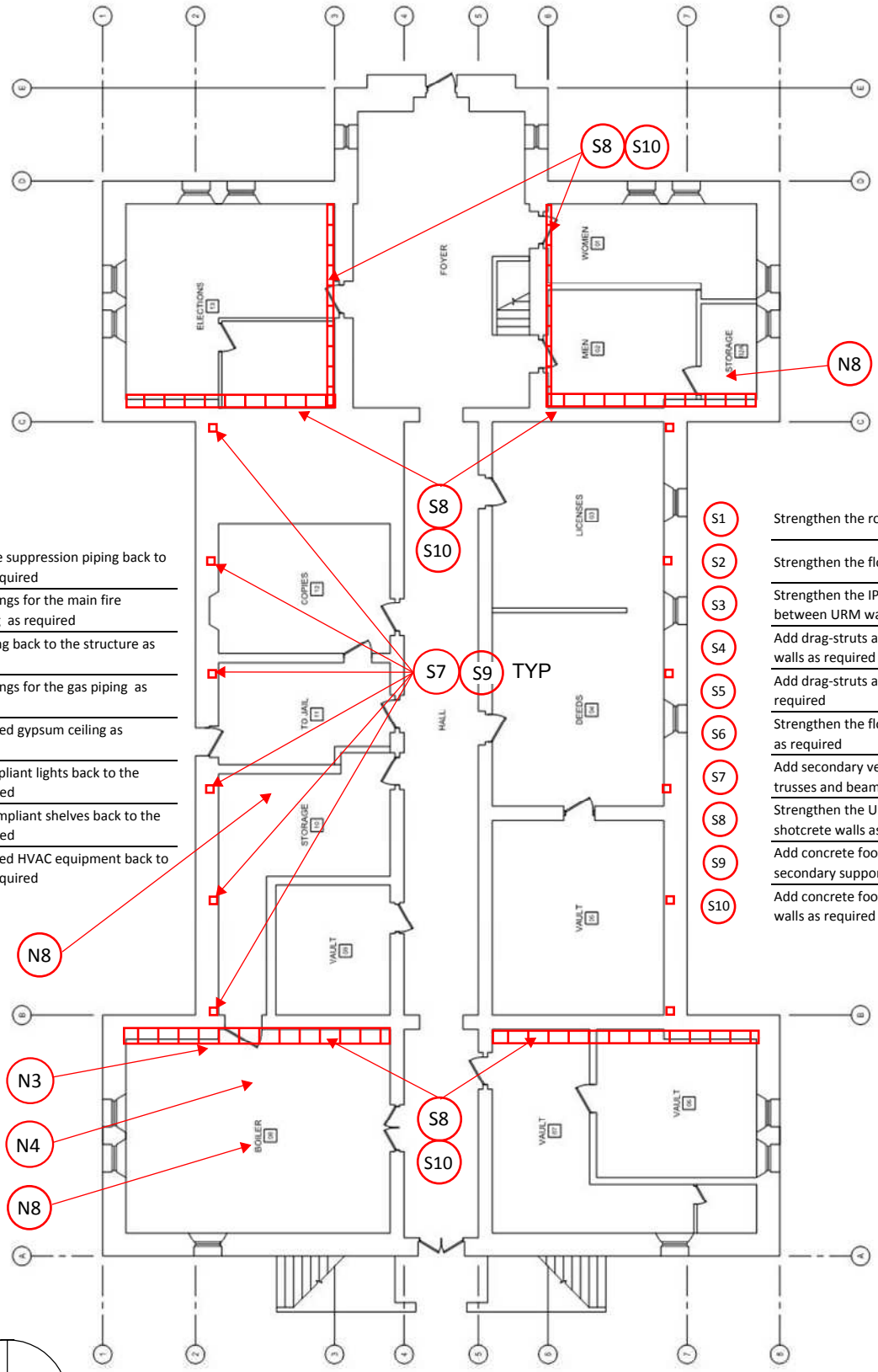
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required

N

FIRST FLOOR PLAN



N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required

N

BASEMENT FLOOR PLAN

Basic Performance Objective

The goal of this objective is to allow the occupants to safely exit the structure after an earthquake. Continued occupancy might not be likely before repair work is completed. This objective is similar to the current building code for most structures. This evaluation uses the BSE-1E seismic hazard level, which is an earthquake that has the probability of exceedance of 20% in 50 years or a mean return period of 225 years between occurrences. The Basic Performance Objective (BPOE) is based on the Life Safety standard and has checklists that provide a systematic approach to the evaluation of the building considering the system modification factors for the materials in the building.

The courthouse building is a five-story structure comprised of three primary areas on each floor, which consist of a central core area and an east and west wing area to the east and west of the core area. Also, the clock tower is located at the center of the east wing and protrudes to the east 10 feet from the remainder of the east wing. There are several unknown structural conditions, which include the condition of the unreinforced masonry walls considering the tensile and shear strength of the masonry, if there is an isolation joint between the courthouse and the adjacent jail building and how the existing anchors at the ceiling of the fourth floor are attached to the ceiling diaphragm.

There is a concern with the unknown condition of the isolation joint; if there is no isolation joint between the jail and the courthouse, there is a potential pounding concern between the buildings. There are a few options to address this concern, one of which is to cut in a new isolation gap into the jail building and provide support for the jail building as required. The other option is to tie the courthouse to the jail building. We anticipate that the preliminary estimate of probable construction cost for the new isolation gap could be approximately $120,000 and could cause some disruption along the south wall of the jail. Tying the buildings together is a more complicated concern and more investigation and design will be required to provide the specifics and the probable construction cost estimate. However, this concern would be covered by the contingency we have used in our preliminary estimate of probable construction cost. The following is the structural evaluation of the courthouse building.

Roof Level Evaluation

The roof framing of the building consists of 2x roof rafters at 24” o.c. typically that bear on the exterior walls with straight tongue-and-groove lumber roof sheathing (the lumber is installed perpendicular to the framing). The rafters are notched and toenailed to a 4 x 12 wood bearing plate that was set in a bed of mortar on top of the brick walls that also has small diameter bolts (3/8” diameter) spaced at 10’ o.c. that appear to be embedded into the top of the brick wall. The bolts have wood spacers of varying length where the nut bears on top of the spacer. The core

area roof is framed differently than the wings and has heavy timber trusses at 12' o.c. that bear on heavy steel plates on the exterior brick walls, of which the trusses support the roof and ceiling framing. However, the heavy steel plates at the truss bearing locations do not appear to be anchored or connected to the top of the brick wall in any way.

Considering the straight sheathing, the west wing exceeds the allowable aspect ratios, as allowed by the checklists, and based on our analysis has a demand-to-capacity ratio (DCR) of 2.2, which is to say that there is 2.2 times more demand (or load) than the capacity of the sheathing. At the core roof area the straight sheathing also spans greater than 24 feet, as allowed by the checklists, and based on our analysis has a DCR of 8.2.

As far as anchorage is concerned, the standard states that when anchors are nonexistent, mitigation with connections needed to anchor the walls to the diaphragm is necessary to meet the performance goals. This concern applies both to the connection of the roof diaphragm, as well as the connection at the heavy timber trusses. Furthermore, the roof lacks connections capable of transferring the loads as a result of the building irregularities and offset shear walls (i.e., gridlines 3 and 6 are offset to the east).

Roof Level Recommendations

To address the noncompliant concerns at the roof level, we recommend the following: For the straight sheathing, overlay the existing sheathing with new plywood sheathing. While addressing the sheathing, add anchors and blocking for both the in-plane and out-of-plane loading concerns. For the missing connections, add custom structural steel drag struts from the offset shear walls and at the reentrant corners. Add connections from the heavy timber trusses to the heavy steel plates and then to the top of the unreinforced masonry walls. Lastly, provide structural steel support columns for the secondary support of the trusses that are independent of the unreinforced masonry walls.

Fourth Floor Evaluation

The fourth floor is predominately located within the original attic space of the heavy timber trusses and was finished as part of the 1978 renovation. The fourth floor bell lounge room is an original finished space that is at the base of the clock tower and is slightly above the remaining fourth floor level. The renovated fourth floor space has light wood floors that span between the bottom chords of the heavy timber trusses, but the floor diaphragm does not extend out to the north and south walls of the core area. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 1.4. In addition, the floor opening at the stairway at the bell lounge room is more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.6 and the unreinforced masonry walls at the stairway opening has a DCR of 3.3.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Fourth Floor Recommendations

To address the noncompliant concerns on the fourth floor (most of the repairs at the fourth floor are similar to the repairs as required at the roof level), we recommend including the strengthening of the floor diaphragm and the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls.

Third Floor Evaluation

The third floor layout covers the footprint of the building including the area within the core, as well as the east and west wings of the building. The third floor core area was added as part of the 1978 renovation when a floor was framed in the upper 10 feet of the second floor. The floor is supported by glulam beams that span in the north-south direction with floor joists spanning in between the glulam beams. The area in the east and west wings is part of the original finished space of the building and is level with the remaining third floor level. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 2.2. In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.2 and the unreinforced masonry walls at the stairway opening has a DCR of 2.0.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Third Floor Recommendations

To address the noncompliant concerns on the third floor (most of the repairs at the third floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. Also, continue to provide structural steel support columns to the secondary support of the trusses, as well as the glulam beams of the third floor that are independent of the unreinforced masonry walls.

Second Floor Evaluation

The second floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 2.2. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 3 and 6 in the east wing are not adequate to resist the in-plane loading from the diaphragm and have a DCR of 2.1 and 2.2 respectively. In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.7 and the unreinforced masonry walls at the stairway opening has a DCR of 4.3.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Second Floor Recommendations

To address the noncompliant concerns on the second floor (most of the repairs at the second floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls.

First Floor Evaluation

The first floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 1.92. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 3, 6, B and C are not adequate to resist the in-plane loading from the diaphragm and have the following DCRs: GL 3 = 1.6, GL 6 = 1.7, GL B = 2.1 and GL C = 2.0. The floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.2 and the unreinforced masonry walls at the stairway opening has a DCR of 2.0.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm

as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

First Floor Recommendations

To address the noncompliant concerns on the first floor (most of the repairs at the first floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls. In addition, the proximity of the jail building creates a potential pounding concern, which, if it is determined to be too close, a seismic isolation joist should be added between the courthouse and the adjacent jail.

Basement Evaluation

The basement layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the east wing of the building, the unreinforced masonry wall at gridline 6 is not adequate to resist the in-plane loading from the diaphragm and has a DCR of 6.4.

Basement Recommendations

To address the noncompliant concerns on the basement, we recommend the following: For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls. The concrete shear walls and the independent support columns will both require new foundation supports, which will need to be placed without impacting the existing foundation system of the building. To address this concern, we used deep foundation elements, such as grouted micropiles to provide vertical support to the heavily loaded foundation elements.

Nonstructural Evaluation

On all levels of the courthouse, a number of non-structural deficiencies were observed, including unbraced fire suppression lines, missing flexible pipe connections at the fire suppression lines, unbraced gas lines, missing flexible pipe connections at the gas lines, noncompliant suspended gypsum board ceiling bracing, and unbraced light fixtures that are supported by the suspended gypsum board ceiling. There are also some noncompliant tall slender contents (book shelves, vertical storage cabinets, etc.) located on the first and third floors of the building. There are a few

unknown non-structural conditions, which include whether the appendages including the statue at the east wall are reinforced and whether the fall-prone equipment weighing more than 20 pounds are braced.

Limitations

The information contained in this report is for the exclusive use of Benton County. Miller Consulting Engineers, Inc. assumes no responsibility or liability for any use of this report by other parties. This report relates solely to the stated purpose of this investigation; no representations concerning other aspects (if any) of the circumstance, structure or site are included. The conclusions (if any) are based on the above stated visual structural observations, and no destructive testing or monitoring was performed. Specific construction details exceed the scope of this report. No guarantee or warranty, expressed or implied, is provided.

Opinions of Probable Construction Cost

In providing opinions of probable construction cost, the Client understands that the Consultant has no control over the cost or availability of labor, equipment or materials, or over market conditions of the Contractor's method of pricing, and that the Consultant's opinions of probable construction costs are made on the basis of the Consultant's professional judgment and experience. Within our opinion of probable construction cost estimate, we have included the contractor's overhead and profit, as well as a 25% construction contingency to account for some differences in the field during construction. The Consultant makes no warranty, express or implied, that the bids or the negotiated cost of the Work will not vary from the Consultant's opinion of probable construction cost.

Opinions of probable construction cost only include the cost to perform the retrofit work, cost to provide temporary storage for displaced material, removal of contents, a contained work environment with ventilation, and protection of fragile finishes. Items excluded from the opinions of probable construction cost that should also be considered include, but are not limited to, staff relocation, bonding, insurance, and permits.

DEFICIENCIES - BASIC PERFORMANCE OBJECTIVES

DCR: Structural Items		Repair Solutions:
2.2**	Overstressed URM shear walls at the second floor at gridlines 3 and 6	Reinforce URM shear walls by adding concrete shotcrete walls.
2.1**	Overstressed URM shear walls at the first floor at gridlines 3, 6, B and C	Reinforce URM shear walls by adding concrete shotcrete walls.
6.4**	Overstressed URM shear walls at the basement at gridlines 3 and 6	Reinforce URM shear walls by adding concrete shotcrete walls.
10*	Missing/overstressed out-of-plane anchorage at URM walls to floor/roof	Install out-of-plane anchorage from URM walls to wood diaphragms.
10*	Transfer to shear walls is inadequate	Add continuous angles from roofs and floors to shear walls.
10*	Missing connection between heavy timber trusses and the URM walls	Install anchors from URM walls to trusses
10*	Missing drag strut connections at gridlines 3 & 6 from roof to URM walls	Install anchors from URM walls to the wood roof diaphragm
10*	Missing drag strut connections at gridlines 3 & 6 from floor to URM walls	Install anchors from URM walls to the wood floor diaphragms
1.6**	Openings at the URM walls do not have drag struts for in-plane loads	Strap wood diaphragm back to URM wall.
4.3**	URM walls at the openings do not have adequate out-of-plane capacity	Add horizontal girts at the openings to support the URM walls
10*	Missing diaphragm cross ties across the wood diaphragms	Install straps ties at the floor joists across the floor diaphragm
1.5**	Straight sheathed wood roof diaphragm aspect ratio too large	Add plywood overlay sheathing over the west wing roof area
5.4**	Straight sheathed wood roof diaphragm span > 24'	Add plywood overlay sheathing over the core roof area
2.2**	Unblocked wood structural panel diaphragm span > 40'	Remove floor finishes and add blocking and nailing to the sheathing as required
10*	Missing independent secondary support columns for the heavy timber trusses	Install secondary support columns for the heavy timber trusses

* DCR ranking was limited to 10 to items that have no capacity
** DCR varies from floor to floor or gridline to gridline

DCR: Non-structural Items		Repair Solutions:
10*	Main fire suppression piping is inadequately braced.	Brace main fire suppression piping back to the structure.
10*	Fire suppression piping does not have flexible couplings per the NFPA-13	Install flexible couplings as required by the NFPA-13
10*	Natural gas piping inadequately braced.	Brace gas piping back to the structure.
10*	No flexible connections at gas piping to equipment.	Install flexible connections to equipment to gas piping.
10*	Suspended gypsum ceiling is not braced.	Brace ceiling with guy wires and posts every 144 sq. ft.
10*	The lights in Courtroom #3 are not independently supported	Install connections for the lights independent of the suspended ceiling
10*	Noncompliant shelves are located on the 1st and 3rd floors	Brace the shelves back to the structure.
10*	The suspended HVAC inline air handlers are not braced.	Install connections to equipment back to the structure

* DCR ranking was limited to 10 to items that have no capacity
** DCR varies from floor to floor or gridline to gridline

COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - BPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 22
Structural Items:							
Roof Level							
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	5.4	6,600	13.08	17
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	8
Bell Tower Lower Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	1.2	828	22.34	22
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	10
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	3.3	100	6.97	13
Fourth Floor Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	1.4	3,392	14.34	19
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	6
Third Floor Level							
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	2.1	3,392	10.71	16
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	12
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	2	200	20.75	21
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$23,800	10	3,392	0.70	4
Second Floor Level							
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.9	3,392	13.42	18
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$597,000	10	7,420	8.05	14
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	20
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$30,200	10	6,784	0.45	2
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$59,900	2.2	22,260	1.22	5
First Floor Level							
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$493,000	10	7,420	6.64	11
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2.8	104	9.27	15

COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 22
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$33,300	10	10,176	0.33	1
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$301,800	2.1	29,680	4.84	9
Basement Level							
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,000	10	13,568	0.51	3
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$524,448	6.4	37,100	2.21	7

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - BPOE		
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TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 22
Structural Items:							
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$33,300	10	10,176	0.33	1
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$30,200	10	6,784	0.45	2
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,000	10	13,568	0.51	3
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$23,800	10	3,392	0.70	4
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$59,900	2.2	22,260	1.22	5
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	6
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$524,448	6.4	37,100	2.21	7
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	8
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$301,800	2.1	29,680	4.84	9
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	10
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$493,000	10	7,420	6.64	11
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	12
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	3.3	100	6.97	13
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$597,000	10	7,420	8.05	14
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2.8	104	9.27	15
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	2.1	3,392	10.71	16
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	5.4	6,600	13.08	17
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.9	3,392	13.42	18
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	1.4	3,392	14.34	19
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	20
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	2	200	20.75	21

COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 22
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	1.2	828	22.34	22

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - BPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Mobilization	Job Trailer	12	Month	$500	$6,000	
	Supervisor	52	Week	$4,125	$214,500	
Construction Aids						
Lift 500# capacity	Provide (2) Boom lifts	12	Month	$8,600	$103,200	
Temporary Storage	Provide (2) 40' container for storage as required	12	Month	$300	$3,600	
Ventilation	Negative Air Equipment per (4) zones	52	Week	$2,000	$104,000	
						$431,300
Strengthen West, East & Core Roofs	Remove (E) tile roofing	6600	S.F.	$4	$26,400	
	Add framing angles from rafters to top PL and anchors to TOW @ 4' o.c.	470	L.F.	$50	$23,500	
	Add perimeter steel PL and anchors in masonry	470	L.F.	$223	$104,810	
	Add Blocking between rafters@ anchors	470	L.F.	$25	$11,750	
	Add Plywood	6600	S.F.	$6	$39,600	
Cross ties	Install steel PLs on each side of the rafters across the west/east wings (18 per wing)	36	Ea.	$1,000	$36,000	
	Install steel PLs on each side of the rafters splices at the core area (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
Heavy timber truss connections	Install steel angles to the sides of the truss and welded to steel PL	12	Ea.	$4,000	$48,000	
	Add (4) anchors from the steel PL to the URM wall	12	Ea.	$2,000	$24,000	
Roof Drag- Strut	Custom steel strut (32k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Re-entrant corners	Custom drag-strut (7k load) at the corners of the core area	4	Ea.	$10,000	$40,000	
New Roofing	Add new tile roofing to match existing	6600	S.F.	$30	$198,000	
Strengthen Bell Tower Floor	Temporary work area	828	S.F.	$2	$1,242	
	Out of plane ties @ 4' o.c.	30	Ea.	$400	$11,800	
	Install straps at E&W walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between rafters@ anchors	118	L.F.	$25	$2,950	
	Add perimeter angle and anchors in masonry	118	L.F.	$223	$26,314	
	Re-nail the plywood floor sheathing or add as req'd	828	S.F.	$6	$4,968	
	Replace the floor finishes	828	S.F.	$5	$4,140	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
						$703,770
Strengthen 4th Floor Core Area	Remove the contents from the floor	2368	S.F.	$10	$23,680	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	2368	S.F.	$3	$7,104	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at N&S walls for subdiaphragm	32	Ea.	$1,000	$32,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Extend to the N&S and re-nail the plywood sheathing	3392	S.F.	$6	$20,352	
	Replace the floor finishes	2368	S.F.	$5	$11,840	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
						$207,496

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Strengthen 3rd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing	3392	S.F.	$3	$10,176	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard beam)	26	Ea.	$1,000	$26,000	
Re-entrant corners	Custom drag-strut (11k load) at the corners of the core area	4	Ea.	$15,000	$60,000	
Add independent HSS column supports	Demo (E) 3FL wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Install HSS 4 x 4 column from truss to 3FL beam	12	Ea.	$1,500	$18,000	
	(N) gypsum and finish	480	S.F.	$8	$3,840	
Strengthen 3rd Floor East Wing Area	Remove contents from the URM walls (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Drag- Strut	Custom steel strut (49k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$45,000	$90,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Floor Opening Strengthening	Custom steel girt (10' span) for OOP support of the E URM wall at the stairway	1	Ea.	$1,440	$1,440	
	Custom steel girt (18' span) for OOP support of the N URM wall at the stairway	1	Ea.	$2,592	$2,592	
	Install straps at ea. end of the stairway opening	4	Ea.	$500	$2,000	
Strengthen 3rd Floor West Wing Area	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	792	S.F.	$5	$3,960	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
						$608,688
Strengthen 2nd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing or add as req'd	3392	S.F.	$6	$20,352	
	Replace the floor finishes	3392	S.F.	$5	$16,960	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	
Re-entrant corners	Custom drag-strut (17k load) at the corners of the core area	4	Ea.	$25,000	$100,000	
Add independent HSS column supports	Demo (E) 2FL wall finishes at (12) locations	720	S.F.	$4	$2,880	
	Install HSS 4 x 4 column from 3FL to 2FL beam	12	Ea.	$1,800	$21,600	
	(N) gypsum and finish	720	S.F.	$8	$5,760	
Strengthen 2nd Floor East Wing Area	Remove contents around the perimeter (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Floor Opening Strengthening	Custom steel girt (14' span) for OOP support of the N/S URM wall at the stairs	2	Ea.	$2,016	$4,032	
	Custom steel girt (20' span) for OOP support of the E URM wall at the	1	Ea.	$2,880	$2,880	
	Install straps at ea. End of the stairway opening	6	Ea.	$500	$3,000	
Drag- Strut	Custom steel strut (77k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$75,000	$150,000	
GL 3 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$12,778	$12,778	
	(N) gypsum and finish	345	S.F.	$8	$2,760	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 6 URM Strengthening	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$12,778	$12,778	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
Strengthen 2nd Floor West Wing Area	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	198	S.F.	$5	$990	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. End of the stairway opening	2	Ea.	$500	$1,000	
						$776,126
Strengthen 1st Floor Core Area	Remove contents around the perimeter (4' wide)	936	S.F.	$10	$9,360	
	Temporary work area	936	S.F.	$2	$1,404	
	Remove (E) floor finishes	936	S.F.	$3	$2,808	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Replace the floor finishes	936	S.F.	$5	$4,680	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	
Re-entrant corners	Custom drag-strut (12k load) at the corners of the core area	4	Ea.	$18,000	$72,000	
Add independent HSS column supports	Demo (E) 1FL wall finishes at (12) locations	792	S.F.	$4	$3,168	
	Install HSS 4 x 4 column from 2FL to 1FL beam	12	Ea.	$1,980	$23,760	
	(N) gypsum and finish	792	S.F.	$8	$6,336	
Strengthen 1st Floor East Wing Area	Remove contents around the perimeter (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Floor Opening Strengthening	Custom steel girt (12' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,728	$1,728	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
Drag- Strut	Custom steel strut (33k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	
	Remove contents from drag strut path (4' wide)	256	S.F.	$10	$2,560	
	Temporary work area	256	S.F.	$2	$384	
	Remove (E) floor finishes	256	S.F.	$3	$768	
	Replace the floor finishes	256	S.F.	$5	$1,280	
GL 3 URM Strengthening	Demo (E) wall finishes	380	S.F.	$4	$1,518	
	Install shear anchors at URM wall @ 3 SF o.c.	127	Ea.	$100	$12,650	
	Build 23' x 16.5' x 8" shotcrete wall	1	Ea.	$21,083	$21,083	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 6 URM Strengthening	(N) gypsum and finish	380	S.F.	$8	$3,036	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Demo (E) wall finishes	380	S.F.	$4	$1,518	
	Install shear anchors at URM wall @ 3 SF o.c.	127	Ea.	$100	$12,650	
	Build 23' x 16.5' x 8" shotcrete wall	1	Ea.	$21,083	$21,083	
GL C URM Strengthening	(N) gypsum and finish	380	S.F.	$8	$3,036	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Demo (E) wall finishes	776	S.F.	$4	$3,102	
	Install shear anchors at URM wall @ 3 SF o.c.	259	Ea.	$100	$25,850	
	Build 23.5' x 16.5' x 8" shotcrete wall	2	Ea.	$14,361	$28,722	
	Custom drag-strut (42k load) tieing new walls	1	Ea.	$45,000	$45,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
Strengthen 1st Floor West Wing Area	(N) gypsum and finish	856	S.F.	$8	$6,844	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	198	S.F.	$5	$990	
GL B URM Strengthening	Demo (E) wall finishes	974	S.F.	$4	$3,894	
	Install shear anchors at URM wall @ 3 SF o.c.	325	Ea.	$100	$32,450	
	Build 29.5' x 16.5' x 8" shotcrete wall	2	Ea.	$18,028	$36,056	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	1054	S.F.	$8	$8,428	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
						$830,346
Strengthen Basement Area	Temporary work area for HSS columns	960	S.F.	$2	$1,440	
Add independent HSS column supports	Demo (E) basement wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Demo (E) slab for new footings (12) locations	480	S.F.	$10	$4,800	
	Excavate for new footings (12) locations	192	S.F.	$30	$5,760	
	Add concrete foundation (12) locations	192	S.F.	$100	$19,200	
	Install HSS 4 x 4 column from 1FL to footing	12	Ea.	$2,700	$32,400	
	(N) gypsum and finish	480	S.F.	$8	$3,840	
GL 3 URM Strengthening	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	
	Excavate for new footings	92	S.F.	$40	$3,680	
	Install (2) micropiles @ ea. end of the (N) SW	1	Ea.	$36,000	$36,000	
	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 10" shotcrete wall	1	Ea.	$10,648	$10,648	
	(N) gypsum and finish	230	S.F.	$8	$1,840	
GL 6 URM Strengthening	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	
	Excavate for new footings	92	S.F.	$40	$3,680	
	Install (2) micropiles @ ea. end of the (N) SW	1	Ea.	$36,000	$36,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL B URM Strengthening	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 10" shotcrete wall	1	Ea.	$10,648	$10,648	
	(N) gypsum and finish	230	S.F.	$8	$1,840	
	Temporary work area	354	S.F.	$2	$531	
	Demo (E) wall finishes	590	S.F.	$4	$2,360	
	Demo (E) slab for new footings	236	S.F.	$10	$2,360	
	Excavate for new footings	236	S.F.	$40	$9,440	
	Install (2) micropiles @ ea. end of the (N) SW	2	Ea.	$36,000	$72,000	
	Add concrete foundation	236	S.F.	$100	$23,600	
	Install shear anchors at URM wall @ 3 SF o.c.	197	Ea.	$100	$19,667	
	Build 29.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$24,583	$49,167	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	32	S.F.	$4	$128	
	(N) gypsum and finish	622	S.F.	$8	$4,976	
GL C URM Strengthening	Temporary work area	282	S.F.	$2	$423	
	Demo (E) wall finishes	470	S.F.	$4	$1,880	
	Demo (E) slab for new footings	188	S.F.	$10	$1,880	
	Excavate for new footings	188	S.F.	$40	$7,520	
	Install (2) micropiles @ ea. end of the (N) SW	2	Ea.	$36,000	$72,000	
	Add concrete foundation	188	S.F.	$100	$18,800	
	Install shear anchors at URM wall @ 3 SF o.c.	157	Ea.	$100	$15,667	
	Build 23.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$19,583	$39,167	
	Custom drag-strut (18k load) tieing new walls	1	Ea.	$26,000	$26,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	550	S.F.	$8	$4,400	
						$593,808

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
					Sub-total	**$4,151,534**
					Construction Contingency (25%)	**$1,037,884**
					Contractor Overhead and Profit (25%)	**$1,297,354**
					Total Seismic Construction Cost	**$6,486,772**

ESTIMATE OF PROBABLE CONSTRUCTION COST
NON-STRUCTURAL

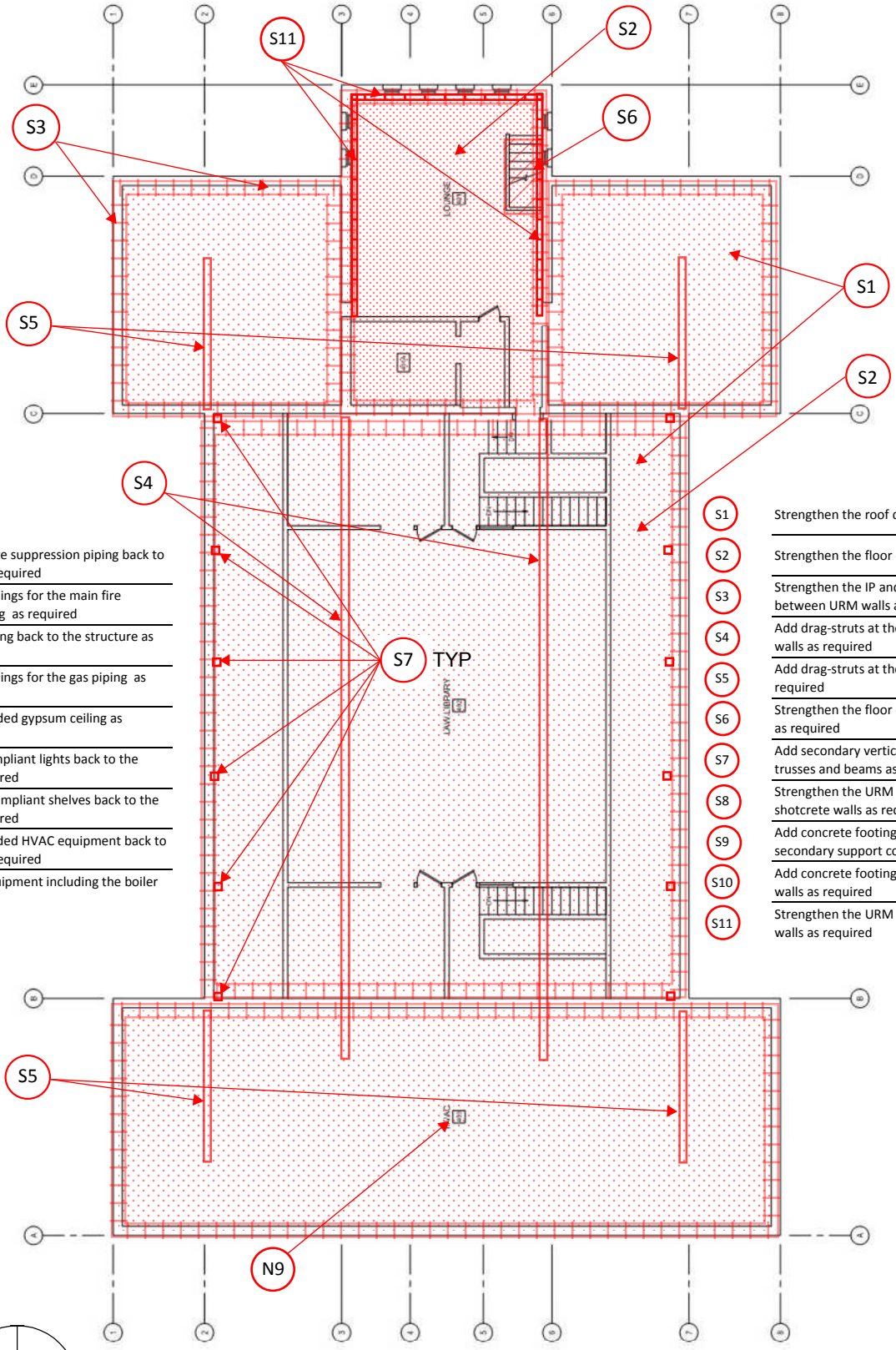
PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - BPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Mobilization	Job Trailer	2	Month	$500	$1,000	
	Supervisor	9	Week	$4,125	$37,125	
Construction Aids						
Lift 500# capacity	(1) Scissor lift	2	Month	$4,300	$8,600	
Temporary Storage	Provide (1) 40' container for storage as required	2	Month	$500	$1,000	
Ventalation	Negative Air Equipment per each zone	9	Week	$500	$4,500	
						$52,225
Fire suppresion piping	Brace to NFPA standards	120	Each	$300	$36,000	
	Add Flex Connection	30	Each	$430	$12,900	
Gas piping	Brace to FM standards	6	Each	$300	$1,800	
	Add Flex Connection	2	Each	$430	$860	
Brace Courtroom #3 suspended ceiling	Remove contents	540.5	S.F.	$10	$5,405	
	Demo (E) ceiling	540.5	S.F.	$6	$3,243	
	Temporary work area	540.5	S.F.	$2	$811	
	Temporary flooring	540.5	S.F.	$2	$1,081	
	Add seismic bracing	540.5	S.F.	$16	$8,648	
	(N) metal suspended ceiling	540.5	S.F.	$5	$2,703	
	(N) gypsum and finish	540.5	S.F.	$12	$6,486	
Brace Room #111 suspended ceiling	Remove contents	310	S.F.	$10	$3,100	
	Demo (E) ceiling	310	S.F.	$6	$1,860	
	Temporary work area	310	S.F.	$2	$465	
	Temporary flooring	310	S.F.	$2	$620	
	Add seismic bracing	310	S.F.	$16	$4,960	
	(N) metal suspended ceiling	310	S.F.	$5	$1,550	
	(N) gypsum and finish	310	S.F.	$12	$3,720	
Brace Courtroom #3 suspended lights	Add independent braces/safety chains	5	Each	$300	$1,500	
Brace NC tall contents on 1FL and 3FL	Add independent braces/safety chains	29	Each	$200	$5,800	
Brace suspended inline HVAC equipment	Add independent braces/safety chains	5	Each	$1,000	$5,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
NON-STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
						$108,511
					Sub-total	**$160,736**
					Construction Contingency (25%)	**$40,184**
					Contractor Overhead and Profit (25%)	**$50,230**
					Total Seismic Construction Cost	**$251,150**

ENHANCED PERFORMANCE OBJECTIVE

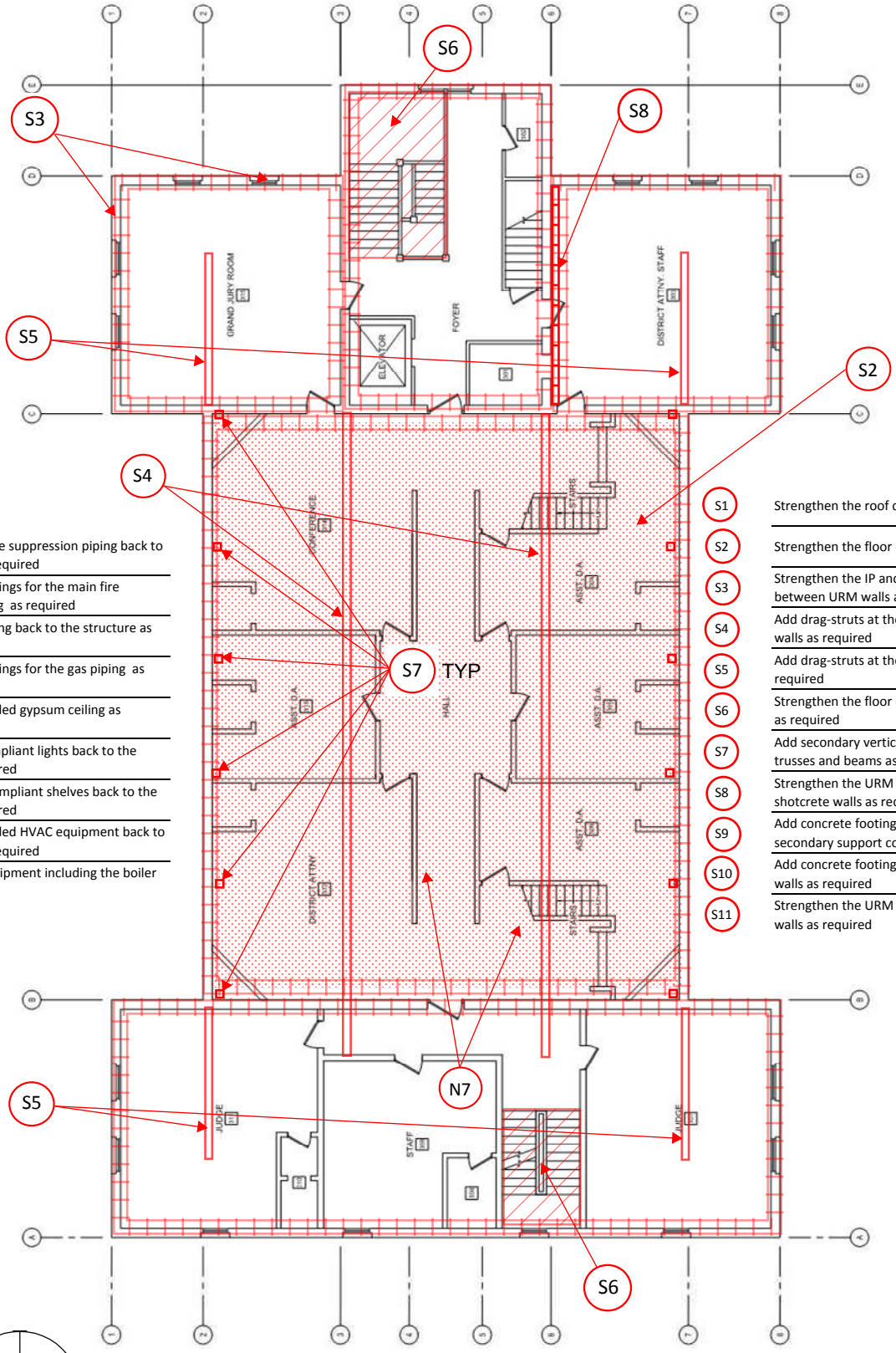


N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required
N9	Anchor heavy equipment including the boiler and attic HVAC

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

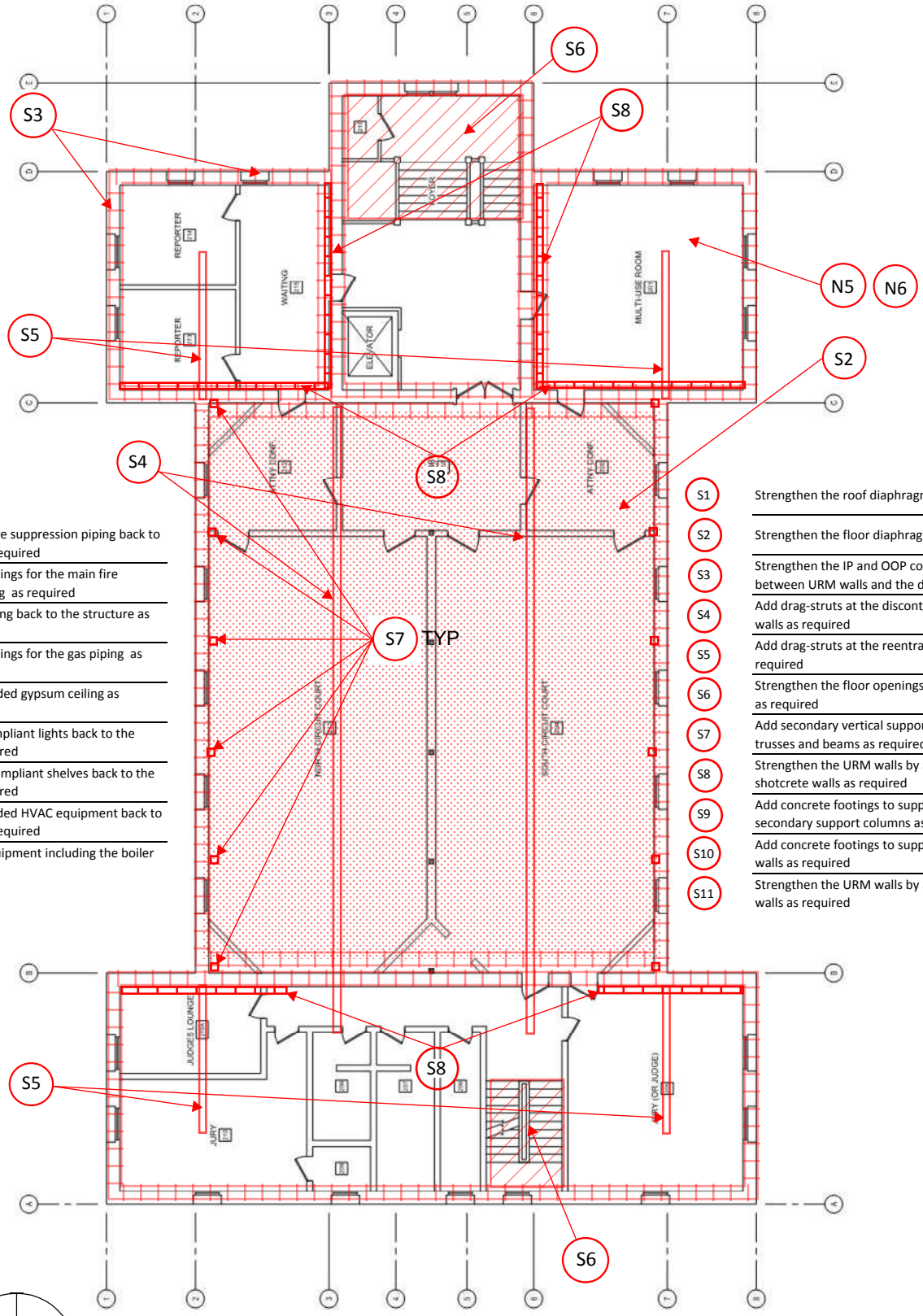
FOURTH FLOOR PLAN



N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required
N9	Anchor heavy equipment including the boiler and attic HVAC

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

THIRD FLOOR PLAN



N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required
N9	Anchor heavy equipment including the boiler and attic HVAC

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

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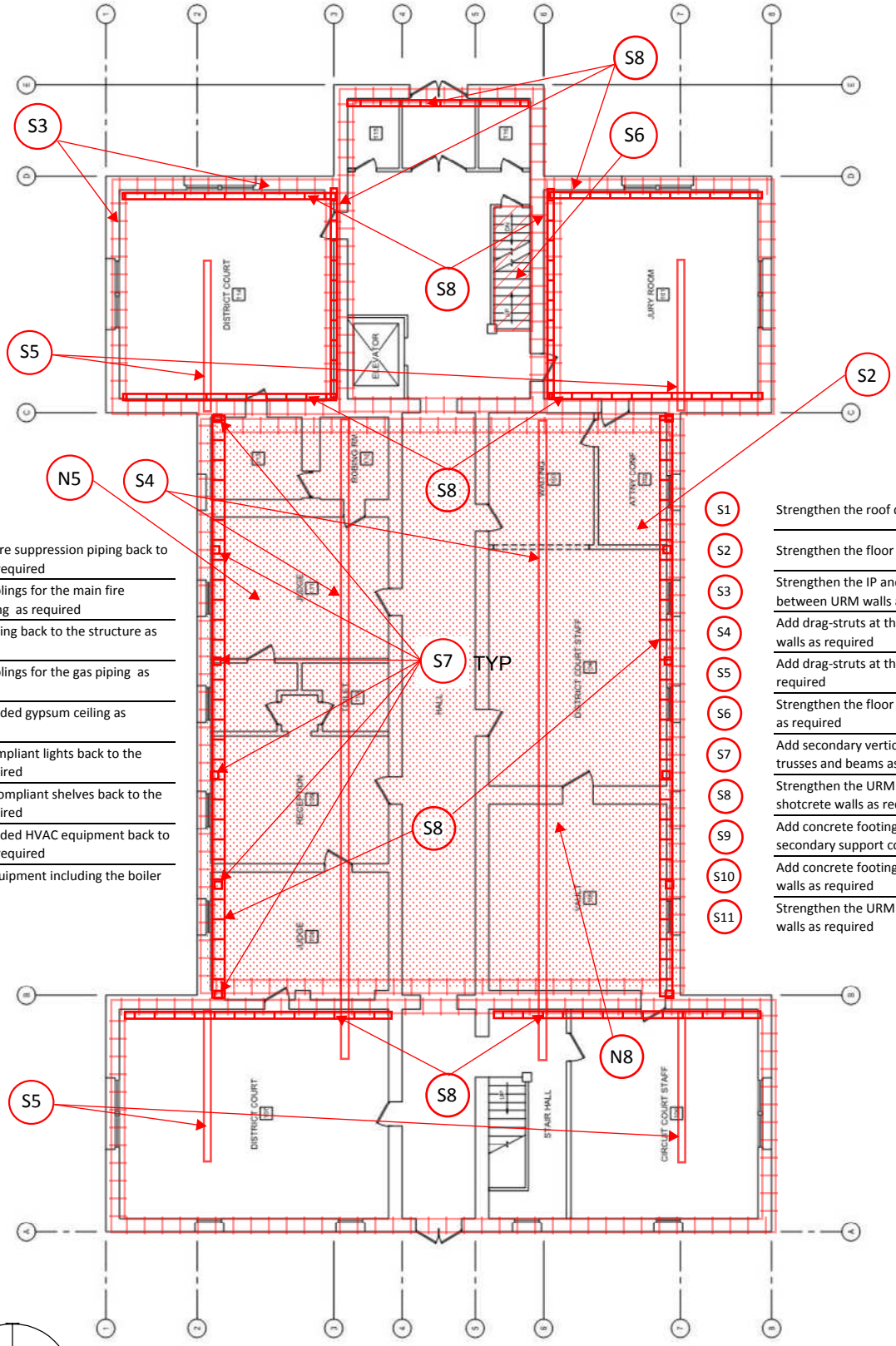
SECOND FLOOR PLAN



N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required
N9	Anchor heavy equipment including the boiler and attic HVAC

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

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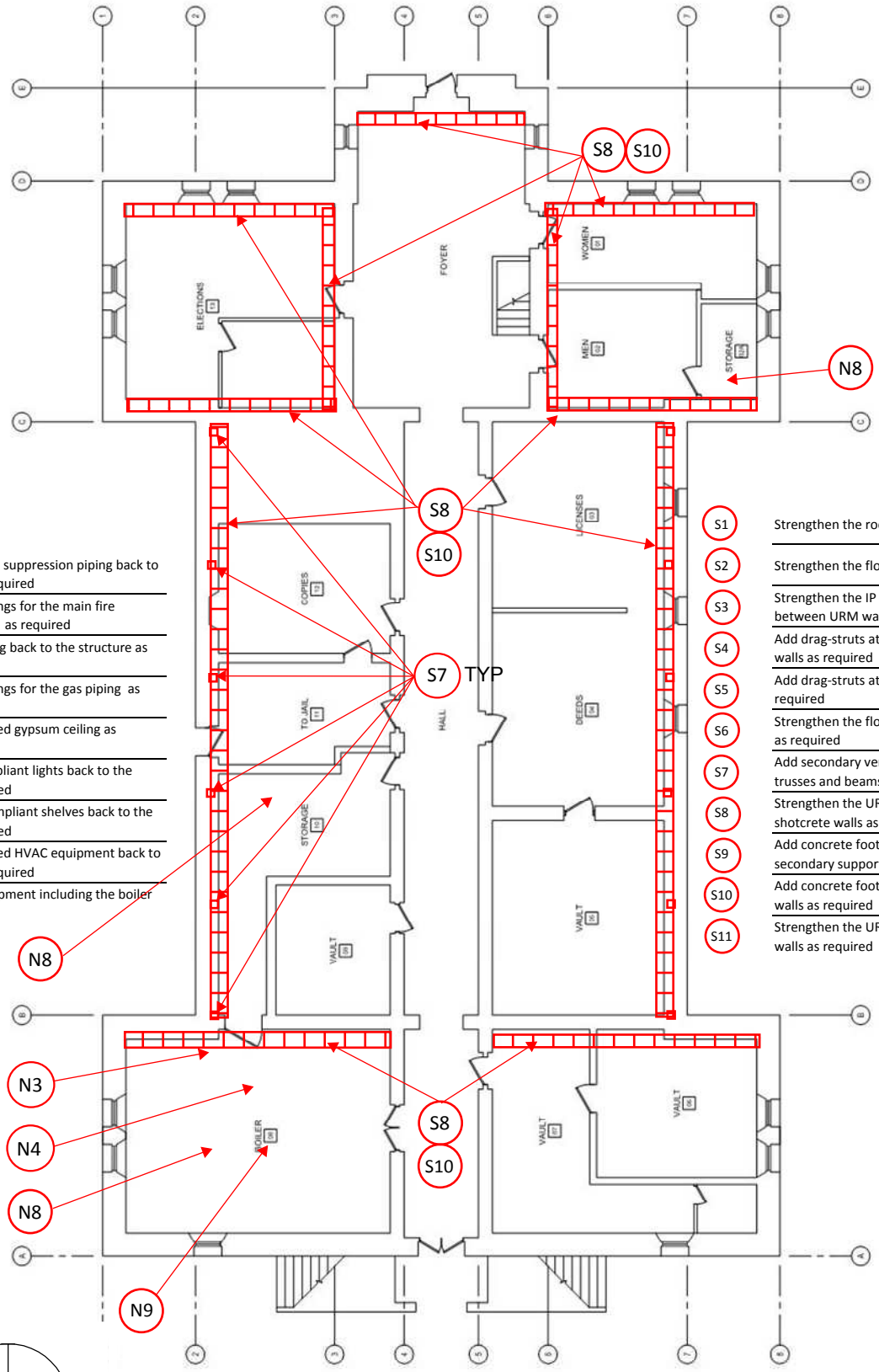
FIRST FLOOR PLAN



N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required
N9	Anchor heavy equipment including the boiler and attic HVAC

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

BASEMENT FLOOR PLAN

Enhanced Performance Objective

The goal of this objective is to allow the building to be immediately operable after an earthquake. It is expected that the building will develop cracks and items that are damaged will remain in place. This objective is similar to current building code for buildings and other structures that represent a substantial hazard to human life in the event of failure. This evaluation uses the BSE-1E seismic hazard level, which is an earthquake that has the probability of exceedance of 20% in 50 years or a mean return period of 225 years between occurrences. The Enhanced Performance Objective (EPOE) is based on the Immediate Occupancy standard checklists that provide a systematic approach to the evaluation of the building considering the system modification factors for the materials in the building.

The courthouse building is a five-story structure comprised of three primary areas on each floor, which consist of a central core area and an east and west wing area to the east and west of the core area. Also, the clock tower is located at the center of the east wing and protrudes to the east 10 feet from the remainder of the east wing. There are several unknown structural conditions, which include the condition of the unreinforced masonry walls considering the tensile and shear strength of the masonry; if there is an isolation joint between the courthouse; and the adjacent jail building and how the existing anchors at the ceiling of the fourth floor are attached to the ceiling diaphragm.

There is a concern with the unknown condition of the isolation joint; if there is no isolation joint between the jail and the courthouse, there is a potential pounding concern between the buildings. There are a few options to address this concern, one of which is to cut in a new isolation gap into the jail building and provide support for the jail building as required. The other option is to tie the courthouse to the jail building. We anticipate that the preliminary estimate of probable construction cost for the new isolation gap could be approximately $120,000 and could cause some disruption along the south wall of the jail. Tying the buildings together is a more complicated concern and more investigation and design will be required to provide the specifics and the probable construction cost estimate. However, this concern would be covered by the contingency we have used in our preliminary estimate of probable construction cost. The following is the structural evaluation of the courthouse building.

Roof Level Evaluation

The roof framing of the building consists of 2x roof rafters at 24” o.c. typically that bear on the exterior walls with straight tongue-and-groove lumber roof sheathing (the lumber is installed perpendicular to the framing). The rafters are notched and toenailed to a 4 x 12 wood bearing plate that was set in a bed of mortar on top of the brick walls that also has small diameter bolts (3/8” diameter) spaced at 10’ o.c. that appear to be embedded into the top of the brick wall. The

bolts have wood spacers of varying length where the nut bears on top of the spacer. The core area roof is framed differently than the wings and has heavy timber trusses at 12' o.c. that bear on heavy steel plates on the exterior brick walls, of which the trusses support the roof and ceiling framing. However, the heavy steel plates at the truss bearing locations do not appear to be anchored or connected to the top of the brick wall in any way.

Considering the straight sheathing, the west wing exceeds the allowable aspect ratios as allowed by the checklists and based on our analysis has a demand-to-capacity ratio (DCR) of 2.2, which is to say that there is 2.2 times more demand (or load) than the capacity of the sheathing. At the core roof area, the straight sheathing also spans greater than 24 feet as allowed by the checklists and based on our analysis has a DCR of 8.2.

As far as anchorage is concerned, the standard states that when anchors are non-existent, mitigation with connections needed to anchor the walls to the diaphragm is necessary to meet the performance goals. This concern applies both to the connection of the roof diaphragm, as well as the connection at the heavy timber trusses. Furthermore, the roof lacks connections capable of transferring the loads as a result of the building irregularities and offset shear walls (i.e., gridlines 3 and 6 are offset to the east).

Roof Level Recommendations

To address the noncompliant concerns at the roof level, we recommend the following: For the straight sheathing, overlay the existing sheathing with new plywood sheathing. While addressing the sheathing, add anchors and blocking for both the in-plane and out-of-plane loading concerns. For the missing connections, add custom structural steel drag struts from the offset shear walls and at the reentrant corners. Add connections from the heavy timber trusses to the heavy steel plates and then to the top of the unreinforced masonry walls. Lastly, provide structural steel support columns for the secondary support of the trusses that are independent of the unreinforced masonry walls.

Fourth Floor Evaluation

The fourth floor is predominately located within the original attic space of the heavy timber trusses and was finished as part of the 1978 renovation. The fourth floor bell lounge room is an original finished space that is at the base of the clock tower and is slightly above the remaining fourth floor level. The renovated fourth floor space has light wood floors that span between the bottom chords of the heavy timber trusses, but the floor diaphragm does not extend out to the north and south walls of the core area. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 1.4. In addition, the floor opening at the stairway at the bell lounge room is more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.6 and the unreinforced masonry walls at the stairway opening has a DCR of 3.3.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls. For the inadequate lateral force-resisting elements (the unreinforced masonry walls), add bonded shear walls to resist the in-plane loading as required by code.

Fourth Floor Recommendations

To address the noncompliant concerns on the fourth floor (most of the repairs at the fourth floor are similar to the repairs as required at the roof level), we recommend strengthening the floor diaphragm and the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls.

Third Floor Evaluation

The third floor layout covers the footprint of the building including the area within the core, as well as the east and west wings of the building. The third floor core area was added as part of the 1978 renovation when a floor was framed in the upper 10 feet of the second floor. The floor is supported by glulam beams that span in the north-south direction with floor joists spanning in between the glulam beams. The area in the east and west wings is part of the original finished space of the building and is level with the remaining third floor level. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 2.2. In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.2 and the unreinforced masonry walls at the stairway opening has a DCR of 2.0.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Third Floor Recommendations

To address the noncompliant concerns on the third floor (most of the repairs at the third floor are similar to the repairs as required at the roof level), we recommend the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. Also, continue to provide structural steel support columns to the secondary support of the

trusses, as well as the glulam beams of the third floor that are independent of the unreinforced masonry walls.

Second Floor Evaluation

The second floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 2.2. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 3 and 6 in the east wing are not adequate to resist the in-plane loading from the diaphragm and have a DCR of 2.1 and 2.2, respectively. In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.7 and the unreinforced masonry walls at the stairway opening has a DCR of 4.3.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Second Floor Recommendations

To address the noncompliant concerns on the second floor (most of the repairs at the second floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls.

First Floor Evaluation

The first floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 1.92. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 3, 6, B and C are not adequate to resist the in-plane loading from the diaphragm and have the following DCRs: GL 3 = 1.6, GL 6 = 1.7, GL B = 2.1 and GL C = 2.0. The floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.2 and the unreinforced masonry walls at the stairway opening has a DCR of 2.0.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

First Floor Recommendations

To address the noncompliant concerns on the first floor (most of the repairs at the first floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls. In addition, the proximity of the jail building creates a potential pounding concern, which, if it is determined to be too close, a seismic isolation joist should be added between the courthouse and the adjacent jail.

Basement Evaluation

The basement layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the east wing of the building, the unreinforced masonry wall at gridline 6 is not adequate to resist the in-plane loading from the diaphragm and has a DCR of 6.4.

Basement Recommendations

To address the noncompliant concerns on the basement, we recommend the following: For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls. The concrete shear walls and the independent support columns will both require new foundation supports, which will need to be placed without impacting the existing foundation system of the building. To address this concern, we used deep foundation elements, such as grouted micropiles to provide vertical support to the heavily loaded foundation elements.

Nonstructural Evaluation

On all levels of the courthouse, a number of non-structural deficiencies were observed, including unbraced fire suppression lines, missing flexible pipe connections at the fire suppression lines, unbraced gas lines, missing flexible pipe connections at the gas lines, noncompliant suspended gypsum board ceiling bracing, and unbraced light fixtures that are supported by the suspended

gypsum board ceiling. There are also some noncompliant tall slender contents (book shelves, vertical storage cabinets, etc.) located on the first and third floors of the building. There are a few unknown non-structural conditions, which include whether the appendages including the statue at the east wall are reinforced and whether the fall-prone equipment weighing more than 20 pounds are braced.

Limitations

The information contained in this report is for the exclusive use of Benton County. Miller Consulting Engineers, Inc. assumes no responsibility or liability for any use of this report by other parties. This report relates solely to the stated purpose of this investigation; no representations concerning other aspects (if any) of the circumstance, structure or site are included. The conclusions (if any) are based on the above stated visual structural observations, and no destructive testing or monitoring was performed. Specific construction details exceed the scope of this report. No guarantee or warranty, expressed or implied, is provided.

Opinions of Probable Construction Cost

In providing opinions of probable construction cost, the Client understands that the Consultant has no control over the cost or availability of labor, equipment or materials, or over market conditions of the Contractor's method of pricing, and that the Consultant's opinions of probable construction costs are made on the basis of the Consultant's professional judgment and experience. Within our opinion of probable construction cost estimate, we have included the contractor's overhead and profit, as well as a 25% construction contingency to account for some differences in the field during construction. The Consultant makes no warranty, express or implied, that the bids or the negotiated cost of the Work will not vary from the Consultant's opinion of probable construction cost.

Opinions of probable construction cost only include the cost to perform the retrofit work, cost to provide temporary storage for displaced material, removal of contents, a contained work environment with ventilation, and protection of fragile finishes. Items excluded from the opinions of probable construction cost that should also be considered include, but are not limited to, staff relocation, bonding, insurance, and permits.

DEFICIENCIES - ENHANCED PERFORMANCE OBJECTIVES

DCR: Structural Items		Repair Solutions:
1.8**	Overstressed URM shear walls at the third floor at gridlines 6	Reinforce URM shear walls by adding concrete shotcrete walls.
3.8**	Overstressed URM shear walls at the second floor at gridlines 3, 6, B and C	Reinforce URM shear walls by adding concrete shotcrete walls.
5.1**	Overstressed URM shear walls at the first floor at gridlines 2, 3, 6, 7, B -E	Reinforce URM shear walls by adding concrete shotcrete walls.
17**	Overstressed URM shear walls at the basement at gridlines 3, 6, B and C	Reinforce URM shear walls by adding concrete shotcrete walls.
10*	Missing/overstressed out-of-plane anchorage at URM walls to floor/roof	Install out-of-plane anchorage from URM walls to wood diaphragms.
10*	Transfer to shear walls is inadequate	Add continuous angles from roofs and floors to shear walls.
10*	Missing connection between heavy timber trusses and the URM walls	Install anchors from URM walls to trusses
10*	Missing drag strut connections at gridlines 3 & 6 from roof to URM walls	Install anchors from URM walls to the wood roof diaphragm
10*	Missing drag strut connections at gridlines 3 & 6 from floor to URM walls	Install anchors from URM walls to the wood floor diaphragms
1.2	URM walls at the bell lounge do not have adequate out-of-plane capacity	Strengthen the URM walls for out-of-plane load capacity
2.2**	Openings at the URM walls do not have drag struts for in-plane loads	Strap wood diaphragm back to URM wall.
6.7**	URM walls at the openings do not have adequate out-of-plane capacity	Add horizontal girts at the openings to support the URM walls
10*	Missing diaphragm cross ties across the wood diaphragms	Install straps ties at the floor joists across the floor diaphragm
2.2**	Straight sheathed wood roof diaphragm aspect ratio too large	Add plywood overlay sheathing over the west wing roof area
8.1**	Straight sheathed wood roof diaphragm span > 24'	Add plywood overlay sheathing over the core roof area
3.6**	Unblocked wood structural panel diaphragm span > 40'	Remove floor finishes and add blocking and nailing to the sheathing as required
10*	Missing independent secondary support columns for the heavy timber trusses	Install secondary support columns for the heavy timber trusses

* DCR ranking was limited to 10 to items that have no capacity
** DCR varies from floor to floor or gridline to gridline

DCR: Non-structural Items		Repair Solutions:
10*	Main fire suppression piping is inadequately braced.	Brace main fire suppression piping back to the structure.
10*	Fire suppression piping does not have flexible couplings per the NFPA-13	Install flexible couplings as required by the NFPA-13
10*	Natural gas piping inadequately braced.	Brace gas piping back to the structure.
10*	No flexible connections at gas piping to equipment.	Install flexible connections to equipment to gas piping.
10*	Suspended gypsum ceiling is not braced.	Brace ceiling with guy wires and posts every 144 sq. ft.
10*	The lights in Courtroom #3 are not independently supported	Install connections for the lights independent of the suspended ceiling
10*	Lens covers on lighting are not attached with safety devices.	Install additional safety connections on lens covers.
10*	Noncompliant shelves are located on the 1st and 3rd floors	Brace the shelves back to the structure.
10*	The suspended HVAC inline air handlers are not braced.	Install connections to equipment back to the structure
10*	The boiler is unanchored at the base	Install anchors into the concrete slab on grade

* DCR ranking was limited to 10 to items that have no capacity
** DCR varies from floor to floor or gridline to gridline

COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - EPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Structural Items:							
Roof Level							
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	8.1	6,600	8.72	18
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	8
Bell Tower Lower Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	2.2	828	12.19	20
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	12
Insufficient OOP URM capacity	4th Floor	Build bonded shear wall	$66,100	1.3	1,613	31.52	25
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	5.1	100	4.51	10
Fourth Floor Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	2.3	3,392	8.73	19
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	7
Third Floor Level							
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	3.6	3,392	6.25	13
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	15
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	3.1	200	13.39	22
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$25,700	10	3,392	0.76	4
Insufficient IP shear wall capacity	3rd Floor	Build shotcrete walls	$28,500	2.5	14,840	0.77	5
Second Floor Level							

COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	3.2	3,392	7.97	17
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$553,000	10	7,420	7.45	16
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	23
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$33,100	10	6,784	0.49	2
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$275,400	3.9	22,260	3.17	9
First Floor Level							
Insufficient floor diaphragm	1st Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.4	3,392	18.22	24
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$485,000	10	7,420	6.54	14
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2	104	12.98	21
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$36,400	10	10,176	0.36	1
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$730,500	5	29,680	4.92	11
Basement Level							
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,400	10	13,568	0.51	3
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$1,100,700	17.2	37,100	1.72	6

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - EPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Structural Items:							
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$36,400	10	10,176	0.36	1
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$33,100	10	6,784	0.49	2
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,400	10	13,568	0.51	3
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$25,700	10	3,392	0.76	4
Insufficient IP shear wall capacity	3rd Floor	Build shotcrete walls	$28,500	2.5	14,840	0.77	5
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$1,100,700	17.2	37,100	1.72	6
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	7
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	8
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$275,400	3.9	22,260	3.17	9
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	5.1	100	4.51	10
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$730,500	5	29,680	4.92	11
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	12
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	3.6	3,392	6.25	13
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$485,000	10	7,420	6.54	14
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	15
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$553,000	10	7,420	7.45	16

COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	3.2	3,392	7.97	17
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	8.1	6,600	8.72	18
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	2.3	3,392	8.73	19
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	2.2	828	12.19	20
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2	104	12.98	21
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	3.1	200	13.39	22
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	23
Insufficient floor diaphragm	1st Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.4	3,392	18.22	24
Insufficient OOP URM capacity	4th Floor	Build bonded shear wall	$66,100	1.3	1,613	31.52	25

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - EPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Mobilization	Job Trailer	16	Month	$500	$8,000	
	Supervisor	69	Week	$4,125	$284,625	
Construction Aids						
Lift 500# capacity	Provide (2) boom lifts	16	Month	$8,600	$137,600	
Temporary Storage	Provide (2) 40' container for storage as required	16	Month	$300	$4,800	
Ventalation	Negative Air Equipment per (4) zones	69	Week	$2,000	$138,000	
						$573,025
Strengthen West, East & Core Roofs	Remove (E) tile roofing	6600	S.F.	$4	$26,400	
	Add framing angles from rafters to top PL and anchors to TOW @ 4' o.c.	470	L.F.	$50	$23,500	
	Add perimeter steel PL and anchors in masonry	470	L.F.	$223	$104,810	
	Add Blocking between rafters@ anchors	470	L.F.	$25	$11,750	
	Add Plywood	6600	S.F.	$6	$39,600	
Cross ties	Install steel PLs on each side of the rafters across the west/east wings (18 per wing)	36	Ea.	$1,000	$36,000	
	Install steel PLs on each side of the rafters splaices at the core area (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
Heavy timber truss connections	Install steel angles to the sides of the truss and welded to steel PL	12	Ea	$4,000	$48,000	
	Add (4) anchors from the steel PL to the URM wall	12	Ea	$2,000	$24,000	
Roof Drag- Strut	Custom steel strut (32k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Re-entrant corners	Custom drag-strut (7k load) at the corners of the core area	4	Ea.	$10,000	$40,000	
New Roofing	Add new tile roofing to match existing	6600	S.F.	$30	$198,000	
Strengthen Bell Tower	Temporary work area	828	S.F.	$2	$1,242	
	Out of plane ties @ 4' o.c.	30	Ea.	$400	$11,800	
	Install straps at E&W walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between rafters@ anchors	118	L.F.	$25	$2,950	
	Add perimeter angle and anchors in masonry	118	L.F.	$223	$26,314	
	Re-nail the plywood floor sheathing or add as req'd	828	S.F.	$6	$4,968	
	Replace the floor finishes	828	S.F.	$5	$4,140	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
Strengthen Bell Lounge Level URM Walls	Demo (E) wall finishes	1613	S.F.	$4	$6,452	
	Build 23' x 13.67 bonded shear wall	629	S.F.	$25	$15,721	
	Build 36' x 13.67 bonded shear wall	984	S.F.	$25	$24,606	
	(N) gypsum and finish	1613	S.F.	$12	$19,357	
						$769,905
Strengthen 4th Floor Core Area	Remove the contents from the floor	2368	S.F.	$10	$23,680	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	2368	S.F.	$3	$7,104	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at N&S walls for subdiaphragm	32	Ea.	$1,000	$32,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Extend to the N&S and re-nail the plywood sheathing	3392	S.F.	$6	$20,352	
	Replace the floor finishes	2368	S.F.	$5	$11,840	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
						$207,496
Strengthen 3rd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing	3392	S.F.	$3	$10,176	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard beam)	26	Ea.	$1,000	$26,000	
Re-entrant corners	Custom drag-strut (11k load) at the corners of the core area	4	Ea.	$15,000	$60,000	
Add independent HSS column supports	Demo (E) 3FL wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Install HSS 4 x 4 column from truss to 3FL beam	12	Ea.	$1,500	$18,000	
	(N) gypsum and finish	480	S.F.	$12	$5,760	
Strengthen 3rd Floor East Wing Area	Remove contents from the URM walls (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Drag- Strut	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
	Custom steel strut (49k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$45,000	$90,000	
Floor Opening Strengthening	Custom steel girt (10' span) for OOP support of the E URM wall at the stairway	1	Ea.	$1,440	$1,440	
	Custom steel girt (18' span) for OOP support of the N URM wall at the stairway	1	Ea.	$2,592	$2,592	
	Install straps at ea. end of the stairway opening	4	Ea.	$500	$2,000	
GL 6 URM Strengthening	Demo (E) wall finishes	328	S.F.	$4	$1,311	
	Install shear anchors at URM wall @ 3 SF o.c.	109	Ea.	$100	$10,925	
	Build 23' x 14.25' x 8" shotcrete wall	1	Ea.	$12,139	$12,139	
	(N) gypsum and finish	328	S.F.	$8	$2,622	
	Re-build 3' tall	69	S.F.	$22	$1,518	
Strengthen 3rd Floor West Wing Area	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	792	S.F.	$5	$3,960	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
						$639,123

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Strengthen 2nd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing or add as req'd	3392	S.F.	$6	$20,352	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	
Re-entrant corners	Custom drag-strut (17k load) at the corners of the core area	4	Ea.	$25,000	$100,000	
Add independent HSS column supports	Demo (E) 2FL wall finishes at (12) locations	720	S.F.	$4	$2,880	
	Install HSS 4 x 4 column from 3FL to 2FL beam	12	Ea.	$1,800	$21,600	
	(N) gypsum and finish	720	S.F.	$12	$8,640	
Strengthen 2nd Floor East Wing Area	Remove contents around the perimeter (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Floor Opening Strengthening	Custom steel girt (14' span) for OOP support of the N/S URM wall at the stairs	2	Ea.	$2,016	$4,032	
	Custom steel girt (20' span) for OOP support of the E URM wall at the	1	Ea.	$2,880	$2,880	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Install straps at ea. End of the stairway opening	6	Ea.	$500	$3,000	
Drag- Strut	Custom steel strut (77k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$75,000	$150,000	
GL 3 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$12,778	$12,778	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL 6 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$19,167	$19,167	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL C URM Strengthening	Demo (E) wall finishes	705	S.F.	$4	$2,820	
	Install shear anchors at URM wall @ 3 SF o.c.	235	Ea.	$100	$23,500	
	Build 23.5' x 15' x 8" shotcrete wall	2	Ea.	$13,056	$26,111	
	Custom drag-strut (42k load) tieing new walls	1	Ea.	$45,000	$45,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	785	S.F.	$8	$6,280	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
Strengthen 2nd Floor West Wing Area	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$50	$9,900	
	Replace the floor finishes	198	S.F.	$5	$990	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. End of the stairway opening	2	Ea.	$500	$1,000	
GL B URM Strengthening	Demo (E) wall finishes	885	S.F.	$4	$3,540	
	Install shear anchors at URM wall @ 3 SF o.c.	295	Ea.	$100	$29,500	
	Build 29.5' x 15' x 8" shotcrete wall	2	Ea.	$16,389	$32,778	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	965	S.F.	$8	$7,720	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
						$960,233
Strengthen 1st Floor Core Area	Remove contents around the perimeter	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing or add as req'd	3392	S.F.	$6	$20,352	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Re-entrant corners	Custom drag-strut (12k load) at the corners of the core area	4	Ea.	$18,000	$72,000	
Add independent HSS column supports	Demo (E) 1FL wall finishes at (12) locations	792	S.F.	$4	$3,168	
	Install HSS 4 x 4 column from 2FL to 1FL beam	12	Ea.	$1,980	$23,760	
	(N) gypsum and finish	792	S.F.	$12	$9,504	
GL 2 URM Strengthening	Demo (E) wall finishes	1040	S.F.	$4	$4,158	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Install shear anchors at URM wall @ 3 SF o.c.	347	Ea.	$100	$34,650	
	Build 63' x 16.5' x 8" shotcrete wall	1	Ea.	$57,750	$57,750	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	2079	S.F.	$8	$16,632	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
GL 7 URM Strengthening	Demo (E) wall finishes	1040	S.F.	$4	$4,158	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Install shear anchors at URM wall @ 3 SF o.c.	347	Ea.	$100	$34,650	
	Build 63' x 16.5' x 8" shotcrete wall	1	Ea.	$57,750	$57,750	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	2079	S.F.	$8	$16,632	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
Strengthen 1st Floor East Wing Area	Remove contents around the perimeter (4' wide)	1898	S.F.	$10	$18,980	
	Temporary work area	1898	S.F.	$2	$2,847	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Floor Opening Strengthening	Custom steel girt (12' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,728	$1,728	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
Drag- Strut	Custom steel strut (33k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	
	Remove contents from drag strut path (4' wide)	256	S.F.	$10	$2,560	
	Temporary work area	256	S.F.	$2	$384	
	Remove (E) floor finishes	256	S.F.	$3	$768	
	Replace the floor finishes	256	S.F.	$5	$1,280	
GL 3 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$19,167	$19,167	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL 6 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$25,556	$25,556	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL C URM Strengthening	Demo (E) wall finishes	776	S.F.	$4	$3,102	
	Install shear anchors at URM wall @ 3 SF o.c.	259	Ea.	$100	$25,850	
	Build 23.5' x 16.5' x 8" shotcrete wall	2	Ea.	$21,542	$43,083	
	Custom drag-strut (42k load) tieing new walls	1	Ea.	$60,000	$60,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	856	S.F.	$8	$6,844	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL D URM Strengthening	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
	Demo (E) wall finishes	776	S.F.	$4	$3,102	
	Install shear anchors at URM wall @ 3 SF o.c.	259	Ea.	$100	$25,850	
	Build 23.5' x 16.5' x 8" shotcrete wall	2	Ea.	$14,361	$28,722	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	856	S.F.	$8	$6,844	
GL E URM Strengthening	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
	Demo (E) wall finishes	330	S.F.	$4	$1,320	
	Install shear anchors at URM wall @ 3 SF o.c.	110	Ea.	$100	$11,000	
	Build 20' x 16.5' x 8" shotcrete wall	1	Ea.	$12,222	$12,222	
	(N) gypsum and finish	330	S.F.	$8	$2,640	
Strengthen 1st Floor West Wing Area	Re-build 3' tall wainscotting	60	S.F.	$22	$1,320	
	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	198	S.F.	$5	$990	
GL B URM Strengthening	Demo (E) wall finishes	974	S.F.	$4	$3,894	
	Install shear anchors at URM wall @ 3 SF o.c.	325	Ea.	$100	$32,450	
	Build 29.5' x 16.5' x 8" shotcrete wall	2	Ea.	$27,042	$54,083	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	1054	S.F.	$8	$8,428	
	Re-build 3' tall wainscotting	177	S.F.	$22	$3,894	
						$1,340,780
Strengthen Basement Area	Temporary work area for HSS columns	960	S.F.	$2	$1,440	
Add independent HSS column supports	Demo (E) basement wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Demo (E) slab for new footings (12) locations	480	S.F.	$10	$4,800	
	Excavate for new footings (12) locations	192	S.F.	$30	$5,760	
	Add concrete foundation (12) locations	192	S.F.	$100	$19,200	
	Install HSS 4 x 4 column from 1FL to footing	12	Ea.	$2,700	$32,400	
	(N) gypsum and finish	480	S.F.	$8	$3,840	
GL 2 URM Strengthening	Temporary work area	378	S.F.	$2	$567	
	Demo (E) wall finishes	630	S.F.	$4	$2,520	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Demo (E) slab for new footings	252	S.F.	$10	$2,520	
	Excavate for new footings	252	S.F.	$40	$10,080	
	Add concrete foundation	252	S.F.	$100	$25,200	
	Install shear anchors at URM wall @ 3 SF o.c.	210	Ea.	$100	$21,000	
	Build 63' x 10' x 10" shotcrete wall	1	Ea.	$58,333	$58,333	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	1670	S.F.	$8	$13,356	
GL 3 URM Strengthening	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	
	Excavate for new footings	92	S.F.	$40	$3,680	
	Install (2) micropiles @ ea. end of the (N) SW	1	Ea.	$36,000	$36,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 6 URM Strengthening	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 10" shotcrete wall	1	Ea.	$21,296	$21,296	
	(N) gypsum and finish	230	S.F.	$8	$1,840	
	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	
	Excavate for new footings	92	S.F.	$40	$3,680	
	Install (2) micropiles @ ea. end of the (N) SW	1	Ea.	$36,000	$36,000	
GL 7 URM Strengthening	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 10" shotcrete wall	1	Ea.	$26,620	$26,620	
	(N) gypsum and finish	230	S.F.	$8	$1,840	
	Temporary work area	378	S.F.	$2	$567	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Demo (E) wall finishes	630	S.F.	$4	$2,520	
	Demo (E) slab for new footings	252	S.F.	$10	$2,520	
	Excavate for new footings	252	S.F.	$40	$10,080	
GL B URM Strengthening	Add concrete foundation	252	S.F.	$100	$25,200	
	Install shear anchors at URM wall @ 3 SF o.c.	210	Ea.	$100	$21,000	
	Build 63' x 10' x 10" shotcrete wall	1	Ea.	$58,333	$58,333	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	1670	S.F.	$8	$13,356	
	Temporary work area	354	S.F.	$2	$531	
	Demo (E) wall finishes	590	S.F.	$4	$2,360	
	Demo (E) slab for new footings	236	S.F.	$10	$2,360	
	Excavate for new footings	236	S.F.	$40	$9,440	
	Install (2) micropiles @ ea. end of the (N) SW	2	Ea.	$36,000	$72,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL C URM Strengthening	Add concrete foundation	236	S.F.	$100	$23,600	
	Install shear anchors at URM wall @ 3 SF o.c.	197	Ea.	$100	$19,667	
	Build 29.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$32,778	$65,556	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	32	S.F.	$4	$128	
	(N) gypsum and finish	622	S.F.	$8	$4,976	
	Temporary work area	282	S.F.	$2	$423	
	Demo (E) wall finishes	470	S.F.	$4	$1,880	
	Demo (E) slab for new footings	188	S.F.	$10	$1,880	
	Excavate for new footings	188	S.F.	$40	$7,520	
	Install (2) micropiles @ ea. end of the (N) SW	2	Ea.	$36,000	$72,000	
GL D URM Strengthening	Add concrete foundation	188	S.F.	$100	$18,800	
	Install shear anchors at URM wall @ 3 SF o.c.	157	Ea.	$100	$15,667	
	Build 23.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$26,111	$52,222	
	Custom drag-strut (18k load) tieing new walls	1	Ea.	$26,000	$26,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	550	S.F.	$8	$4,400	
	Temporary work area	276	S.F.	$2	$414	
	Demo adjoining (E) walls and finishes	920	S.F.	$6	$5,520	
	Demo (E) wall finishes	460	S.F.	$4	$1,840	
	Demo (E) slab for new footings	184	S.F.	$10	$1,840	
	Excavate for new footings	184	S.F.	$40	$7,360	
	Install (1) micropiles @ ea. end of the (N) SW	2	Ea.	$18,000	$36,000	
	Add concrete foundation	184	S.F.	$100	$18,400	
	Install shear anchors at URM wall @ 3 SF o.c.	153	Ea.	$100	$15,333	
	Build 23.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$25,556	$51,111	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL E URM Strengthening	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	Re-build adjoining walls	460	S.F.	$6	$2,760	
	(N) gypsum and finish	1460	S.F.	$8	$11,680	
	Temporary work area	120	S.F.	$2	$180	
	Demo adjoining (E) walls and finishes	200	S.F.	$6	$1,200	
	Demo (E) wall finishes	200	S.F.	$4	$800	
	Demo (E) slab for new footings	80	S.F.	$10	$800	
	Excavate for new footings	80	S.F.	$40	$3,200	
	Install (1) micropiles @ ea. end of the (N) SW	1	Ea.	$18,000	$18,000	
	Add concrete foundation	80	S.F.	$100	$8,000	
	Install shear anchors at URM wall @ 3 SF o.c.	67	Ea.	$100	$6,667	
	Build 20' x 10' x 12" (avg thick) shotcrete wall	1	Ea.	$22,222	$22,222	
	Re-build adjoining walls	200	S.F.	$6	$1,200	
	(N) gypsum and finish	400	S.F.	$8	$3,200	
						$1,170,021
					Sub-total	**$5,660,584**
					Construction Contingency (25%)	**$1,415,146**
					Contractor Overhead and Profit (25%)	**$1,768,933**
					Total Seismic Construction Cost	**$8,844,663**

ESTIMATE OF PROBABLE CONSTRUCTION COST
NON-STRUCTURAL

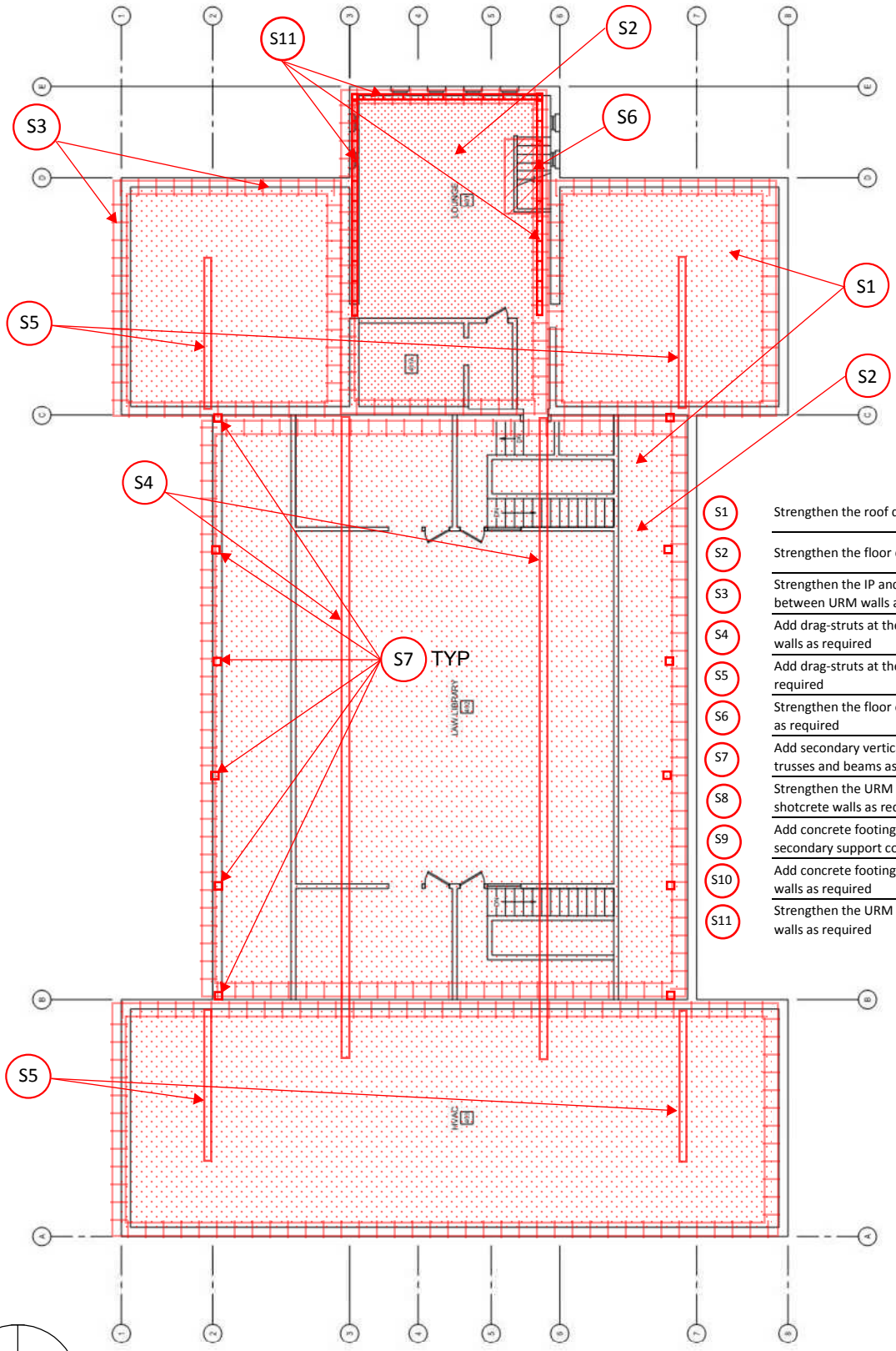
PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - EPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Mobilization	Job Trailer	2	Month	$500	$1,000	
	Supervisor	9	Week	$4,125	$37,125	
Construction Aids						
Lift 500# capacity	(1) Scissor lift	2	Month	$4,300	$8,600	
Temporary Storage	Provide (1) 40' container for storage as required	2	Month	$500	$1,000	
Ventalation	Negative Air Equipment per each zone	9	Week	$500	$4,500	
						$52,225
Fire suppresion piping	Brace to NFPA standards	120	Each	$300	$36,000	
	Add Flex Connection	30	Each	$430	$12,900	
Gas piping	Brace to FM standards	6	Each	$300	$1,800	
	Add Flex Connection	2	Each	$430	$860	
Brace Courtroom #3 suspended ceiling	Remove contents	540.5	S.F.	$10	$5,405	
	Demo (E) ceiling	540.5	S.F.	$6	$3,243	
	Temporary work area	540.5	S.F.	$2	$811	
	Temporary flooring	540.5	S.F.	$2	$1,081	
	Add seismic bracing	540.5	S.F.	$16	$8,648	
	(N) metal suspended ceiling	540.5	S.F.	$5	$2,703	
	(N) gypsum and finish	540.5	S.F.	$12	$6,486	
Brace Room #111 suspended ceiling	Remove contents	310	S.F.	$10	$3,100	
	Demo (E) ceiling	310	S.F.	$6	$1,860	
	Temporary work area	310	S.F.	$2	$465	
	Temporary flooring	310	S.F.	$2	$620	
	Add seismic bracing	310	S.F.	$16	$4,960	
	(N) metal suspended ceiling	310	S.F.	$5	$1,550	
	(N) gypsum and finish	310	S.F.	$12	$3,720	
Brace Courtroom #3 suspended lights	Add independent braces/safety chains	5	Each	$300	$1,500	
Secure lens covers for Courtroom #3 suspended lights	Add clips to secure the lens covers of the lights	5	Each	$150	$750	

ESTIMATE OF PROBABLE CONSTRUCTION COST
NON-STRUCTURAL

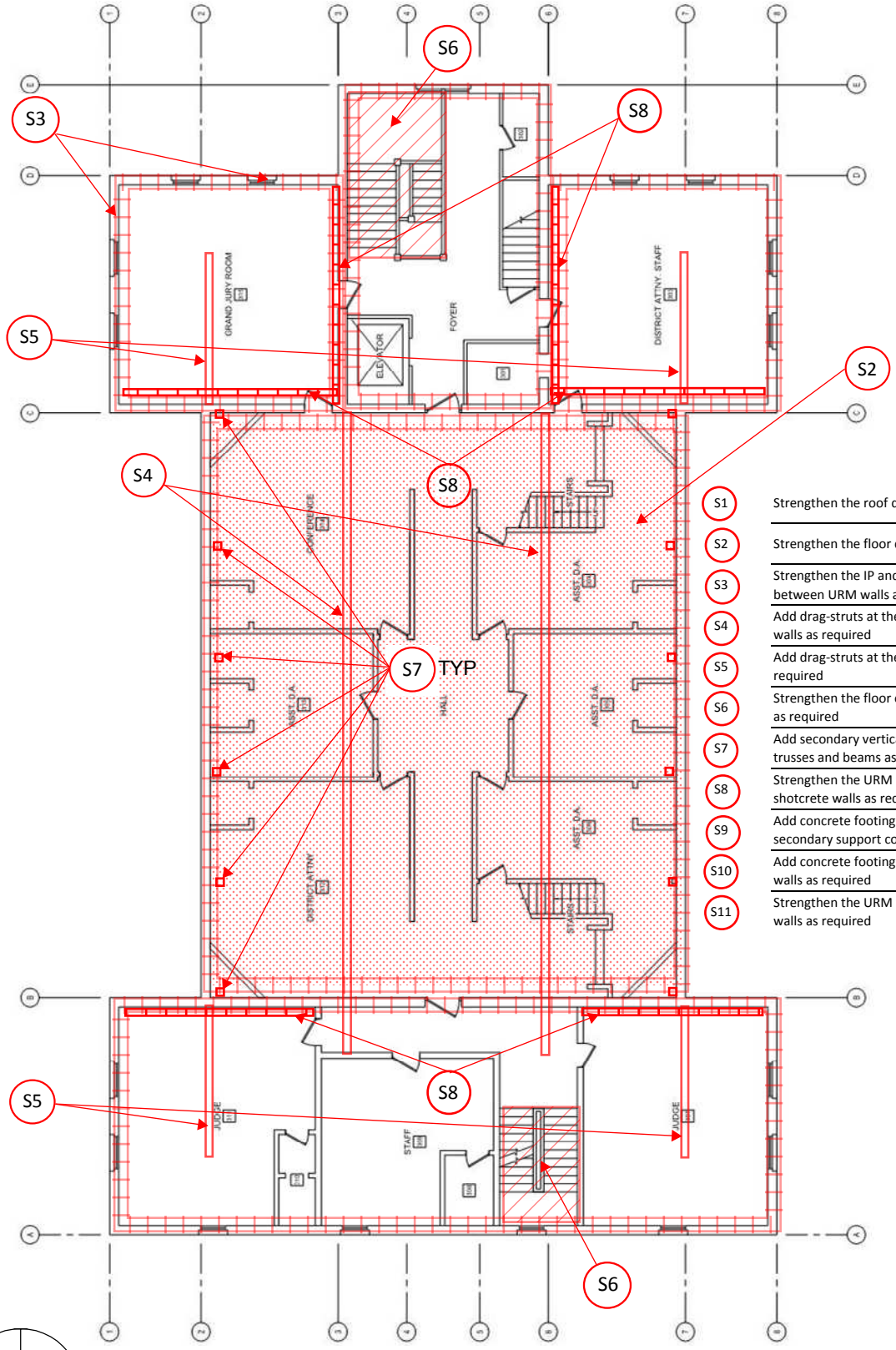
SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Brace NC tall contents on 1FL and 3FL	Add independent braces/safety chains	29	Each	$200	$5,800	
Brace suspended inline HVAC equipment	Add independent braces/safety chains	5	Each	$1,000	$5,000	
Anchor the gas boiler in the boiler room	Add concrete anchors at each corner	1	Each	$500	$500	
Anchor the HVAC unit in the attic HVAC room	Add concrete anchors at each corner	1	Each	$2,000	$2,000	
						$111,761
					Sub-total	**$163,986**
					Construction Contingency (25%)	**$40,997**
					Contractor Overhead and Profit (25%)	**$51,246**
					Total Seismic Construction Cost	**$256,229**

LIMITED PERFORMANCE OBJECTIVE



S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

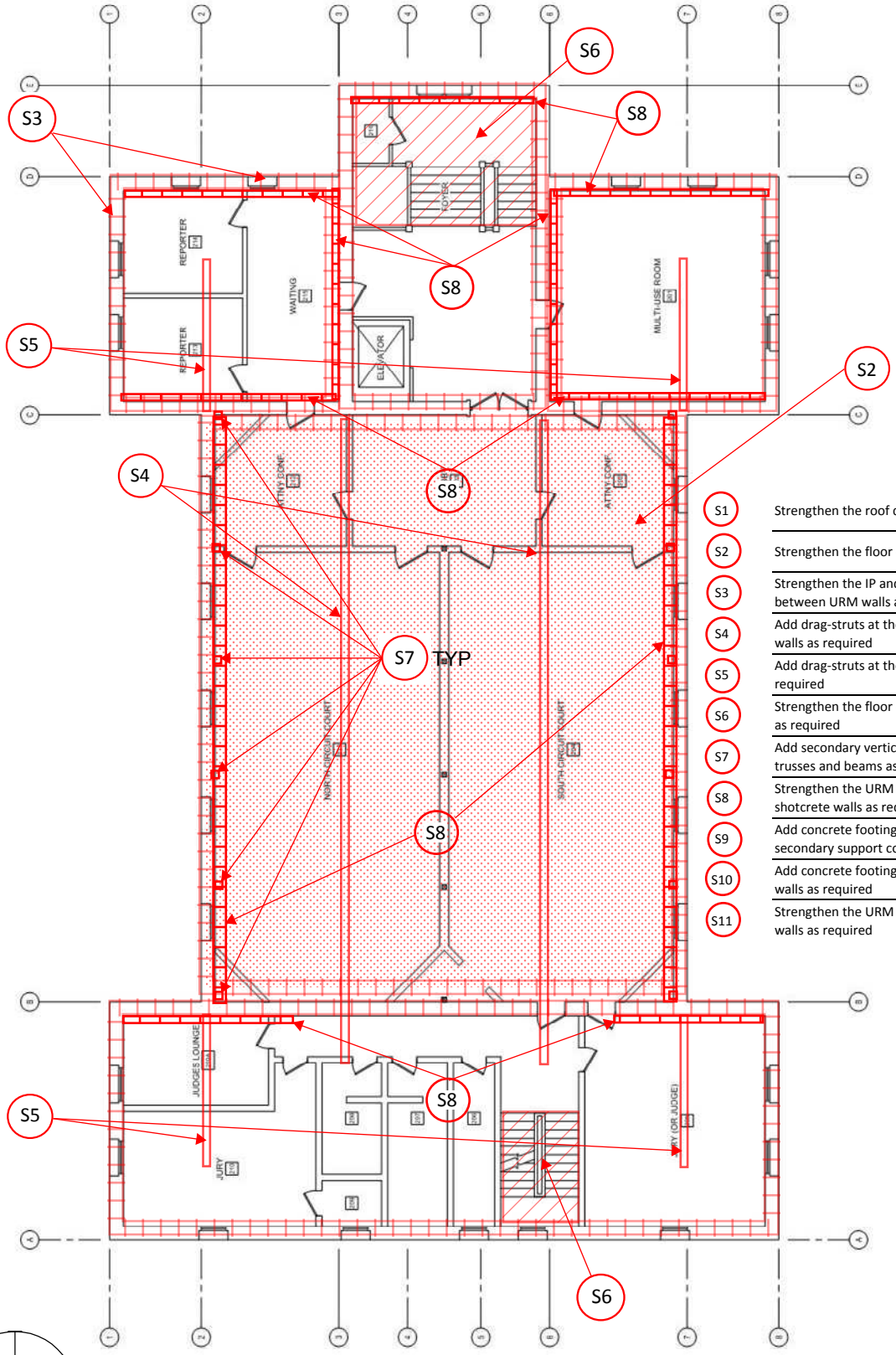
FOURTH FLOOR PLAN



S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

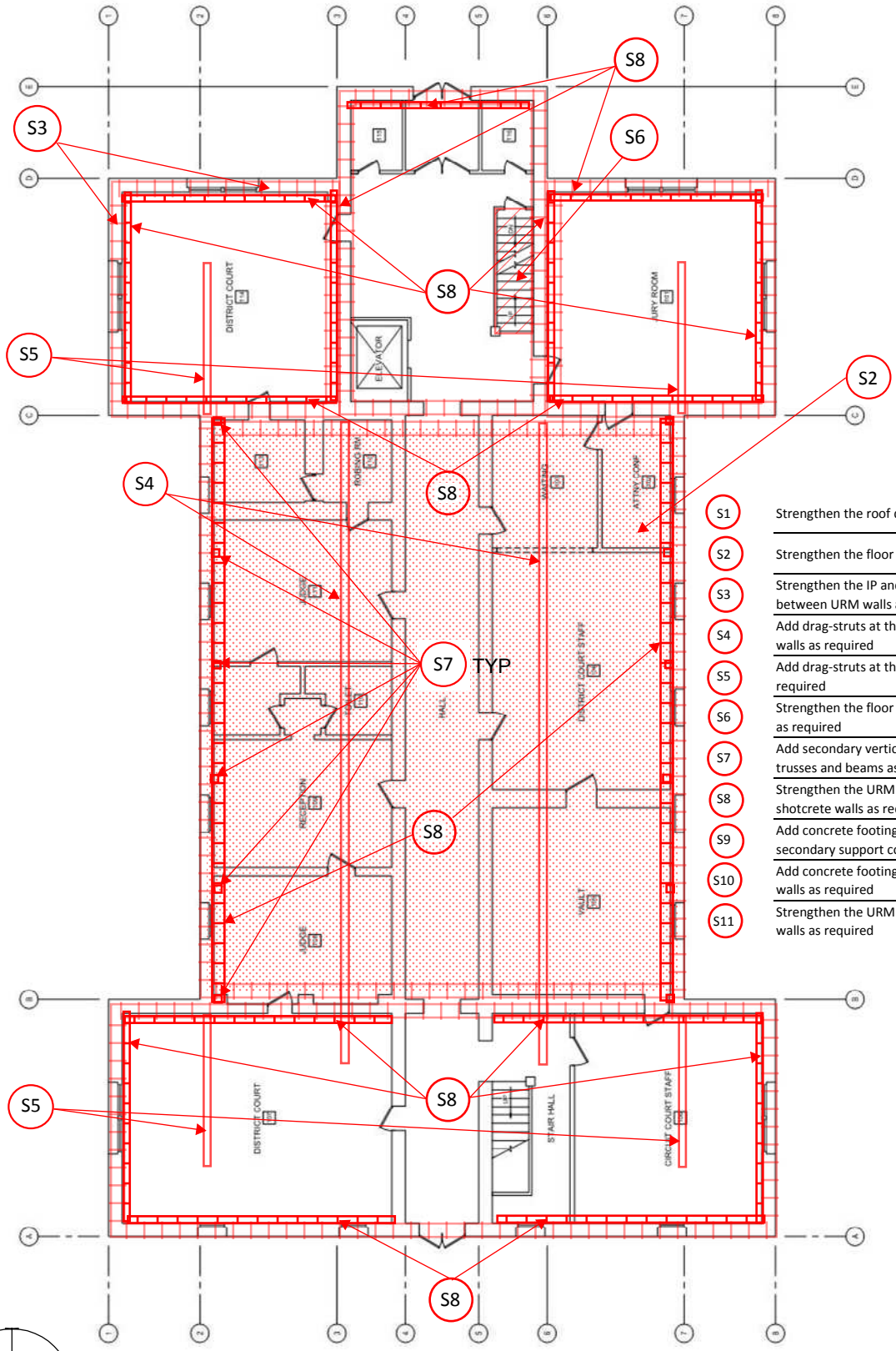
THIRD FLOOR PLAN



S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

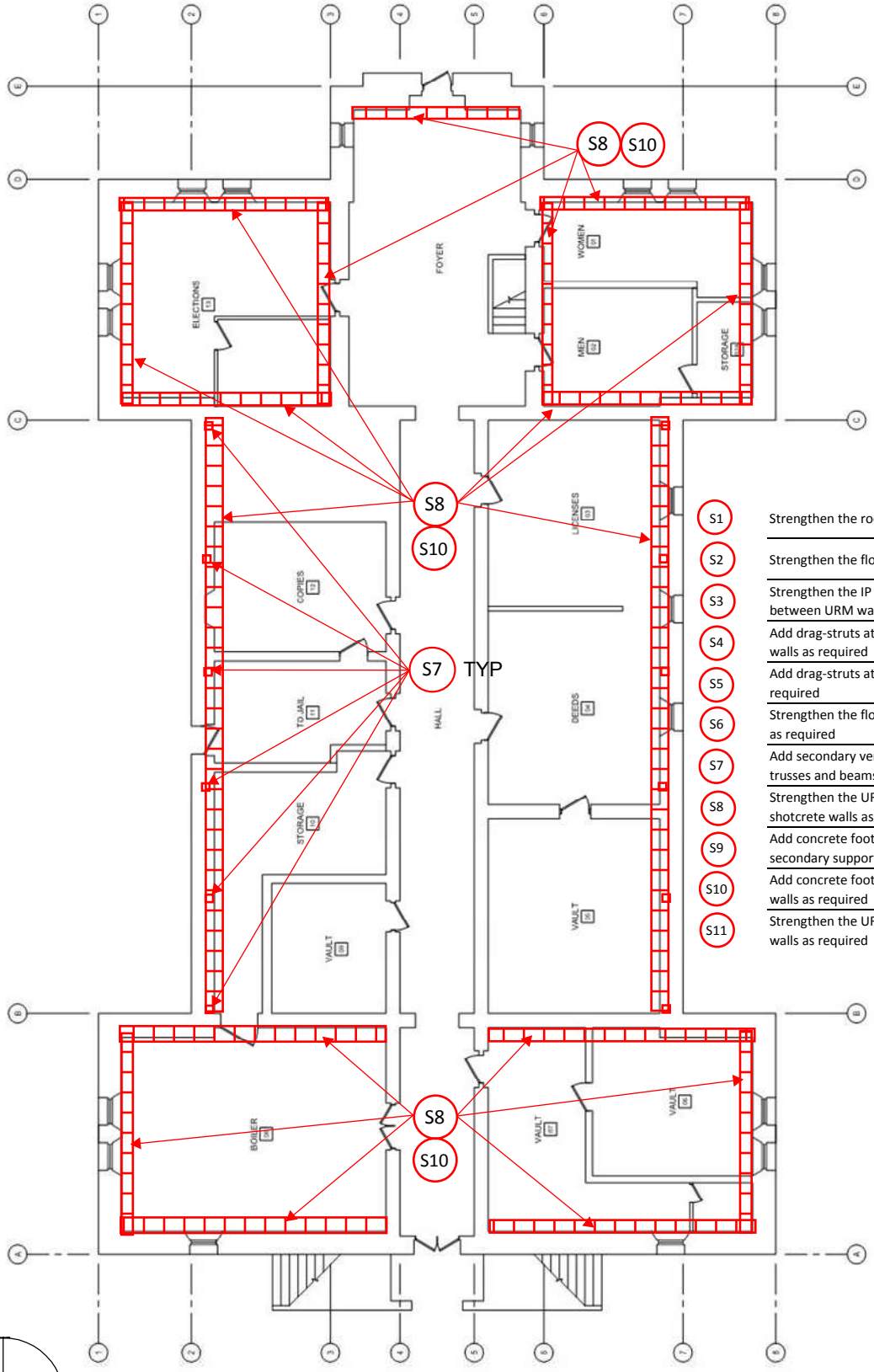
SECOND FLOOR PLAN



S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

FIRST FLOOR PLAN



S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

BASEMENT FLOOR PLAN

Limited Performance Objective

The goal of this objective is to prevent the building from collapsing during a major earthquake. It is expected that the building will be severely damaged and should not continue to be occupied following the earthquake. This objective represents significantly more damage and greater life safety risk when compared to the current building code. This evaluation uses the BSE-2E seismic hazard level, which is an earthquake that has the probability of exceedance of 5% in 50 years or a mean return period of 975 years between occurrences. The Limited Performance Objective (LPOE) is based on the Collapse Prevention standard which requires the use of a Tier III analysis, which, in this study, a Linear Static Procedure was used for the evaluation of the building considering the system modification factors for the materials in the building.

The courthouse building is a five-story structure comprised of three primary areas on each floor, which consist of a central core area and an east and west wing area to the east and west of the core area. Also, the clock tower is located at the center of the east wing and protrudes to the east 10 feet from the remainder of the east wing. There are several unknown structural conditions, which include the condition of the unreinforced masonry walls considering the tensile and shear strength of the masonry, if there is an isolation joint between the courthouse and the adjacent jail building and how the existing anchors at the ceiling of the fourth floor are attached to the ceiling diaphragm.

There is a concern with the unknown condition of the isolation joint; if there is no isolation joint between the jail and the courthouse, there is a potential pounding concern between the buildings. There are a few options to address this concern, one of which is to cut in a new isolation gap into the jail building and provide support for the jail building as required. The other option is to tie the courthouse to the jail building. We anticipate that the preliminary estimate of probable construction cost for the new isolation gap could be approximately $120,000 and could cause some disruption along the south wall of the jail. Tying the buildings together is a more complicated concern and more investigation and design will be required to provide the specifics and the probable construction cost estimate. However, this concern would be covered by the contingency we have used in our preliminary estimate of probable construction cost. The following is the structural evaluation of the courthouse building.

Roof Level Evaluation

The roof framing of the building consists of 2x roof rafters at 24” o.c. typically that bear on the exterior walls with straight tongue-and-groove lumber roof sheathing (the lumber is installed perpendicular to the framing). The rafters are notched and toenailed to a 4 x 12 wood bearing plate that was set in a bed of mortar on top of the brick walls that also has small diameter bolts (3/8” diameter) spaced at 10’ o.c. that appear to be embedded into the top of the brick wall. The

bolts have wood spacers for unknown reasons of varying length where the nut bears on top of the spacer. The core area roof is framed differently than the wings and has heavy timber trusses at 12' o.c. that bear on heavy steel plates on the exterior brick walls, of which the trusses support the roof and ceiling framing. However, the heavy steel plates at the truss bearing locations do not appear to be anchored or connected to the top of the brick wall in any way.

Considering the straight sheathing, the west wing exceeds the allowable aspect ratios, as allowed by the checklists, and based on our analysis has a demand-to-capacity ratio (DCR) of 2.2, which is to say that there is 2.2 times more demand (or load) than the capacity of the sheathing. At the core roof area, the straight sheathing also spans greater than 24 feet, as allowed by the checklists, and based on our analysis has a DCR of 8.2.

As far as anchorage is concerned, the standard states that when anchors are nonexistent, mitigation with connections needed to anchor the walls to the diaphragm is necessary to meet the performance goals. This concern applies both to the connection of the roof diaphragm, as well as the connection at the heavy timber trusses. Furthermore, the roof lacks connections capable of transferring the loads as a result of the building irregularities and offset shear walls (i.e., gridlines 3 and 6 are offset to the east).

Roof Level Recommendations

To address the noncompliant concerns at the roof level, we recommend the following: For the straight sheathing, overlay the existing sheathing with new plywood sheathing. While addressing the sheathing, add anchors and blocking for both the in-plane and out-of-plane loading concerns. For the missing connections, add custom structural steel drag struts from the offset shear walls and at the reentrant corners. Add connections from the heavy timber trusses to the heavy steel plates and then to the top of the unreinforced masonry walls. Lastly, provide structural steel support columns for the secondary support of the trusses that are independent of the unreinforced masonry walls.

Fourth Floor Evaluation

The fourth floor is predominately located within the original attic space of the heavy timber trusses and was finished as part of the 1978 renovation. The fourth floor bell lounge room is an original finished space that is at the base of the clock tower and is slightly above the remaining fourth floor level. The renovated fourth floor space has light wood floors that span between the bottom chords of the heavy timber trusses, but the floor diaphragm does not extend out to the north and south walls of the core area. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 1.4. In addition, the floor opening at the stairway at the bell lounge room is more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.6 and the unreinforced masonry walls at the stairway opening has a DCR of 3.3.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Fourth Floor Recommendations

To address the noncompliant concerns on the fourth floor (most of the repairs at the fourth floor are similar to the repairs as required at the roof level), we recommend including the strengthening of the floor diaphragm and the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls.

Third Floor Evaluation

The third floor layout covers the footprint of the building including the area within the core, as well as the east and west wings of the building. The third floor core area was added as part of the 1978 renovation when a floor was framed in the upper 10 feet of the second floor. The floor is supported by glulam beams that span in the north-south direction with floor joists spanning in between the glulam beams. The area in the east and west wings is part of the original finished space of the building and is level with the remaining third floor level. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 2.2. In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.2 and the unreinforced masonry walls at the stairway opening has a DCR of 2.0.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Third Floor Recommendations

To address the noncompliant concerns on the third floor (most of the repairs at the third floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. Also, continue to provide structural steel support columns to the secondary support of the trusses, as well as the glulam beams of the third floor that are independent of the unreinforced masonry walls.

Second Floor Evaluation

The second floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 2.2. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 3 and 6 in the east wing are not adequate to resist the in-plane loading from the diaphragm and have a DCR of 2.1 and 2.2, respectively. In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.7 and the unreinforced masonry walls at the stairway opening has a DCR of 4.3.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Second Floor Recommendations

To address the noncompliant concerns on the second floor (most of the repairs at the second floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls.

First Floor Evaluation

The first floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 1.92. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 3, 6, B and C are not adequate to resist the in-plane loading from the diaphragm and have the following DCRs: GL 3 = 1.6, GL 6 = 1.7, GL B = 2.1 and GL C = 2.0. The floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.2 and the unreinforced masonry walls at the stairway opening has a DCR of 2.0.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm,

as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

First Floor Recommendations

To address the noncompliant concerns on the first floor (most of the repairs at the first floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls. In addition, the proximity of the jail building creates a potential pounding concern, which, if it is determined to be too close, a seismic isolation joist should be added between the courthouse and the adjacent jail.

Basement Evaluation

The basement layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the east wing of the building, the unreinforced masonry wall at gridline 6 is not adequate to resist the in-plane loading from the diaphragm and has a DCR of 6.4.

Basement Recommendations

To address the noncompliant concerns on the basement, we recommend the following: For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls. The concrete shear walls and the independent support columns will both require new foundation supports, which will need to be placed without impacting the existing foundation system of the building. To address this concern, we used deep foundation elements, such as grouted micropiles to provide vertical support to the heavily loaded foundation elements.

Limitations

The information contained in this report is for the exclusive use of Benton County. Miller Consulting Engineers, Inc. assumes no responsibility or liability for any use of this report by other parties. This report relates solely to the stated purpose of this investigation; no representations concerning other aspects (if any) of the circumstance, structure or site are included. The conclusions (if any) are based on the above stated visual structural observations,

and no destructive testing or monitoring was performed. Specific construction details exceed the scope of this report. No guarantee or warranty, expressed or implied, is provided.

Opinions of Probable Construction Cost

In providing opinions of probable construction cost, the Client understands that the Consultant has no control over the cost or availability of labor, equipment or materials, or over market conditions of the Contractor's method of pricing, and that the Consultant's opinions of probable construction costs are made on the basis of the Consultant's professional judgment and experience. Within our opinion of probable construction cost estimate, we have included the contractor's overhead and profit, as well as a 25% construction contingency to account for some differences in the field during construction. The Consultant makes no warranty, express or implied, that the bids or the negotiated cost of the Work will not vary from the Consultant's opinion of probable construction cost.

Opinions of probable construction cost only include the cost to perform the retrofit work, cost to provide temporary storage for displaced material, removal of contents, a contained work environment with ventilation, and protection of fragile finishes. Items excluded from the opinions of probable construction cost that should also be considered include, but are not limited to, staff relocation, bonding, insurance, and permits.

DEFICIENCIES - LIMITED PERFORMANCE OBJECTIVES

DCR: Structural Items		Repair Solutions:
15.1**	Overstressed URM shear walls at the third floor at gridlines 3, 6, B and C	Reinforce URM shear walls by adding concrete shotcrete walls.
8.2**	Overstressed URM shear walls at the second floor at GLs 2, 3, 6, 7 and B-E	Reinforce URM shear walls by adding concrete shotcrete walls.
4.3**	Overstressed URM shear walls at the first floor at gridlines 3, 6, B and C	Reinforce URM shear walls by adding concrete shotcrete walls.
10.4**	Overstressed URM shear walls at the basement at gridlines 3 and 6	Reinforce URM shear walls by adding concrete shotcrete walls.
10*	Missing/overstressed out-of-plane anchorage at URM walls to floor/roof	Install out-of-plane anchorage from URM walls to wood diaphragms.
10*	Transfer to shear walls is inadequate	Add continuous angles from roofs and floors to shear walls.
10*	Missing connection between heavy timber trusses and the URM walls	Install anchors from URM walls to trusses
10*	Missing drag strut connections at gridlines 3 & 6 from roof to URM walls	Install anchors from URM walls to the wood roof diaphragm
10*	Missing drag strut connections at gridlines 3 & 6 from floor to URM walls	Install anchors from URM walls to the wood floor diaphragms
1.7	URM walls at the bell lounge do not have adequate out-of-plane capacity	Strengthen the URM walls for out-of-plane load capacity
2.6**	Openings at the URM walls do not have drag struts for in-plane loads	Strap wood diaphragm back to URM wall.
7.9**	URM walls at the openings do not have adequate out-of-plane capacity	Add horizontal girts at the openings to support the URM walls
10*	Missing diaphragm cross ties across the wood diaphragms	Install straps ties at the floor joists across the floor diaphragm
2.0**	Straight sheathed wood roof diaphragm aspect ratio too large	Add plywood overlay sheathing over the west wing roof area
9.8**	Straight sheathed wood roof diaphragm span > 24'	Add plywood overlay sheathing over the core roof area
4.3**	Unblocked wood structural panel diaphragm span > 40'	Remove floor finishes and add blocking and nailing to the sheathing as required
10*	Missing independent secondary support columns for the heavy timber trusses	Install secondary support columns for the heavy timber trusses

* DCR ranking was limited to 10 to items that have no capacity
** DCR varies from floor to floor or gridline to gridline

COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - LPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Structural Items:							
Roof Level							
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	8.1	6,600	8.72	18
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	6
Bell Tower Lower Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	2.2	828	12.19	20
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	10
Insufficient OOP URM capacity	4th Floor	Build bonded shear wall	$66,100	1.3	1,613	31.52	25
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	5.1	100	4.51	8
Fourth Floor Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	2.3	3,392	8.73	19
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	5
Third Floor Level							
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	3.6	3,392	6.25	13
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	15
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	3.1	200	13.39	22
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$25,700	10	3,392	0.76	4
Insufficient IP shear wall capacity	3rd Floor	Build shotcrete walls	$172,300	2.5	14,840	4.64	9
Second Floor Level							

COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	3.2	3,392	7.97	17
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$553,000	10	7,420	7.45	16
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	23
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$33,100	10	6,784	0.49	2
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$534,300	3.9	22,260	6.15	12
First Floor Level							
Insufficient floor diaphragm	1st Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.7	3,392	15.00	24
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$485,000	10	7,420	6.54	14
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2	104	12.98	21
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$36,400	10	10,176	0.36	1
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$884,100	5	29,680	5.96	11
Basement Level							
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,400	10	13,568	0.51	3
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$1,790,000	17.2	37,100	2.81	7

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - LPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Structural Items:							
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$36,400	10	10,176	0.36	1
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$33,100	10	6,784	0.49	2
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,400	10	13,568	0.51	3
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$25,700	10	3,392	0.76	4
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	5
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	6
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$1,790,000	17.2	37,100	2.81	7
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	5.1	100	4.51	8
Insufficient IP shear wall capacity	3rd Floor	Build shotcrete walls	$172,300	2.5	14,840	4.64	9
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	10
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$884,100	5	29,680	5.96	11
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$534,300	3.9	22,260	6.15	12
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	3.6	3,392	6.25	13
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$485,000	10	7,420	6.54	14
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	15
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$553,000	10	7,420	7.45	16

COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	3.2	3,392	7.97	17
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	8.1	6,600	8.72	18
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	2.3	3,392	8.73	19
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	2.2	828	12.19	20
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2	104	12.98	21
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	3.1	200	13.39	22
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	23
Insufficient floor diaphragm	1st Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.7	3,392	15.00	24
Insufficient OOP URM capacity	4th Floor	Build bonded shear wall	$66,100	1.3	1,613	31.52	25

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - LPOE		
CLIENT:	Benton County	DATE:	26-Feb-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Mobilization	Job Trailer	18	Month	$500	$9,000	
	Supervisor	78	Week	$4,125	$321,750	
Construction Aids						
Lift 500# capacity	Provide (2) boom lifts	18	Month	$8,600	$154,800	
Temporary Storage	Provide (2) 40' container for storage as required	18	Month	$300	$5,400	
Ventalation	Negative Air Equipment per (4) zones	78	Week	$2,000	$156,000	
						$646,950
Strengthen West, East & Core Roofs	Remove (E) tile roofing	6600	S.F.	$4	$26,400	
	Add framing angles from rafters to top PL and anchors to TOW @ 4' o.c.	470	L.F.	$50	$23,500	
	Add perimeter steel PL and anchors in masonry	470	L.F.	$223	$104,810	
	Add Blocking between rafters@ anchors	470	L.F.	$25	$11,750	
	Add Plywood	6600	S.F.	$6	$39,600	
Cross ties	Install steel PLs on each side of the rafters across the west/east wings (18 per wing)	36	Ea.	$1,000	$36,000	
	Install steel PLs on each side of the rafters splaices at the core area (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
Heavy timber truss connections	Install steel angles to the sides of the truss and welded to steel PL	12	Ea	$4,000	$48,000	
	Add (4) anchors from the steel PL to the URM wall	12	Ea	$2,000	$24,000	
Roof Drag- Strut	Custom steel strut (32k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Re-entrant corners	Custom drag-strut (7k load) at the corners of the core area	4	Ea.	$10,000	$40,000	
New Roofing	Add new tile roofing to match existing	6600	S.F.	$30	$198,000	
Strengthen Bell Tower	Temporary work area	828	S.F.	$2	$1,242	
	Out of plane ties @ 4' o.c.	30	Ea.	$400	$11,800	
	Install straps at E&W walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between rafters@ anchors	118	L.F.	$25	$2,950	
	Add perimeter angle and anchors in masonry	118	L.F.	$223	$26,314	
	Re-nail the plywood floor sheathing or add as req'd	828	S.F.	$6	$4,968	
	Replace the floor finishes	828	S.F.	$5	$4,140	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
Strengthen Bell Lounge Level URM Walls	Demo (E) wall finishes	1613	S.F.	$4	$6,452	
	Build 23' x 13.67 bonded shear wall	629	S.F.	$25	$15,721	
	Build 36' x 13.67 bonded shear wall	984	S.F.	$25	$24,606	
	(N) gypsum and finish	1613	S.F.	$8	$12,904	
						$763,453
Strengthen 4th Floor Core Area	Remove the contents from the floor	2368	S.F.	$10	$23,680	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes (along E&W walls)	2368	S.F.	$3	$7,104	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at N&S walls for subdiaphragm	32	Ea.	$1,000	$32,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Extend plywood sheathing to the N&S	3392	S.F.	$6	$20,352	
	Replace the floor finishes	2368	S.F.	$5	$11,840	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
						$207,496
Strengthen 3rd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing	3392	S.F.	$3	$10,176	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard beam)	26	Ea.	$1,000	$26,000	
Re-entrant corners	Custom drag-strut (11k load) at the corners of the core area	4	Ea.	$15,000	$60,000	
Add independent HSS column supports	Demo (E) 3FL wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Install HSS 4 x 4 column from truss to 3FL beam	12	Ea.	$1,500	$18,000	
	(N) gypsum and finish	480	S.F.	$8	$3,840	
GL 3 URM Strengthening	Demo (E) wall finishes	328	S.F.	$4	$1,311	
	Install shear anchors at URM wall @ 3 SF o.c.	109	Ea.	$100	$10,925	
	Build 23' x 14.25' x 8" shotcrete wall	1	Ea.	$12,139	$12,139	
	(N) gypsum and finish	328	S.F.	$8	$2,622	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL 6 URM Strengthening	Demo (E) wall finishes	328	S.F.	$4	$1,311	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL C URM Strengthening	Install shear anchors at URM wall @ 3 SF o.c.	109	Ea.	$100	$10,925	
	Build 23' x 14.25' x 8" shotcrete wall	1	Ea.	$12,139	$12,139	
	(N) gypsum and finish	328	S.F.	$8	$2,622	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Demo (E) wall finishes	670	S.F.	$4	$2,679	
Strengthen 3rd Floor East Wing Area	Install shear anchors at URM wall @ 3 SF o.c.	223	Ea.	$100	$22,325	
	Build 23' x 14.25' x 8" shotcrete wall	2	Ea.	$12,403	$24,806	
	(N) gypsum and finish	670	S.F.	$8	$5,358	
	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	
	Remove contents from the URM walls (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
Drag- Strut	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
	Custom steel strut (49k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$45,000	$90,000	
Floor Opening Strengthening	Custom steel girt (10' span) for OOP support of the E URM wall at the stairway	1	Ea.	$1,440	$1,440	
	Custom steel girt (18' span) for OOP support of the N URM wall at the stairway	1	Ea.	$2,592	$2,592	
	Install straps at ea. end of the stairway opening	4	Ea.	$500	$2,000	
Strengthen 3rd Floor West Wing Area	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	792	S.F.	$5	$3,960	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
GL B URM Strengthening	Demo (E) wall finishes	656	S.F.	$4	$2,622	
	Install shear anchors at URM wall @ 3 SF o.c.	219	Ea.	$100	$21,850	
	Build 23' x 14.25' x 8" shotcrete wall	2	Ea.	$12,139	$24,278	
	(N) gypsum and finish	656	S.F.	$8	$5,244	
	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	
						$780,951
Strengthen 2nd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing or add as req'd	3392	S.F.	$6	$20,352	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Re-entrant corners	Custom drag-strut (17k load) at the corners of the core area	4	Ea.	$25,000	$100,000	
Add independent HSS column supports	Demo (E) 2FL wall finishes at (12) locations	720	S.F.	$4	$2,880	
	Install HSS 4 x 4 column from 3FL to 2FL beam	12	Ea.	$1,800	$21,600	
	(N) gypsum and finish	720	S.F.	$12	$8,640	
GL 2 URM Strengthening	Demo (E) wall finishes	945	S.F.	$4	$3,780	
	Demo adjoining (E) walls and finishes	945	S.F.	$6	$5,670	
	Install shear anchors at URM wall @ 3 SF o.c.	315	Ea.	$100	$31,500	
	Build 63' x 15' x 8" shotcrete wall	1	Ea.	$35,000	$35,000	
	Re-build adjoining walls	945	S.F.	$6	$5,670	
	(N) gypsum and finish	1890	S.F.	$8	$15,120	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
GL 7 URM Strengthening	Demo (E) wall finishes	945	S.F.	$4	$3,780	
	Demo adjoining (E) walls and finishes	945	S.F.	$6	$5,670	
	Install shear anchors at URM wall @ 3 SF o.c.	315	Ea.	$100	$31,500	
	Build 63' x 16.5' x 8" shotcrete wall	1	Ea.	$35,000	$35,000	
	Re-build adjoining walls	945	S.F.	$6	$5,670	
	(N) gypsum and finish	1890	S.F.	$8	$15,120	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
Strengthen 2nd Floor East Wing Area	Remove contents around the perimeter (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Floor Opening Strengthening	Custom steel girt (14' span) for OOP support of the N/S URM wall at the stairs	2	Ea.	$2,016	$4,032	
	Custom steel girt (20' span) for OOP support of the E URM wall at the	1	Ea.	$2,880	$2,880	
	Install straps at ea. End of the stairway opening	6	Ea.	$500	$3,000	
Drag- Strut	Custom steel strut (77k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$75,000	$150,000	
GL 3 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$19,167	$19,167	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL 6 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$19,167	$19,167	
	(N) gypsum and finish	345	S.F.	$12	$4,140	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL C URM Strengthening	Demo (E) wall finishes	690	S.F.	$4	$2,760	
	Install shear anchors at URM wall @ 3 SF o.c.	230	Ea.	$100	$23,000	
	Build 23' x 15' x 8" shotcrete wall	2	Ea.	$19,167	$38,333	
	(N) gypsum and finish	690	S.F.	$8	$5,520	
	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	
GL D URM Strengthening	Demo (E) wall finishes	705	S.F.	$4	$2,820	
	Install shear anchors at URM wall @ 3 SF o.c.	235	Ea.	$100	$23,500	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL E URM Strengthening	Build 23.5' x 15' x 8" shotcrete wall	2	Ea.	$13,056	$26,111	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	785	S.F.	$8	$6,280	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
	Demo (E) wall finishes	300	S.F.	$4	$1,200	
	Install shear anchors at URM wall @ 3 SF o.c.	100	Ea.	$100	$10,000	
Strengthen 2nd Floor West Wing Area	Build 20' x 15' x 8" shotcrete wall	1	Ea.	$11,111	$11,111	
	(N) gypsum and finish	300	S.F.	$8	$2,400	
	Re-build 3' tall wainscotting	60	S.F.	$22	$1,320	
	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	198	S.F.	$5	$990	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. End of the stairway opening	2	Ea.	$500	$1,000	
GL B URM Strengthening	Demo (E) wall finishes	690	S.F.	$4	$2,760	
	Install shear anchors at URM wall @ 3 SF o.c.	230	Ea.	$100	$23,000	
	Build 23' x 15' x 8" shotcrete wall	2	Ea.	$19,167	$38,333	
	(N) gypsum and finish	690	S.F.	$8	$5,520	
	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
						$1,253,422
Strengthen 1st Floor Core Area	Remove contents around the perimeter	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing or add as req'd	3392	S.F.	$6	$20,352	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	
Re-entrant corners	Custom drag-strut (12k load) at the corners of the core area	4	Ea.	$18,000	$72,000	
Add independent HSS column supports	Demo (E) 1FL wall finishes at (12) locations	792	S.F.	$4	$3,168	
	Install HSS 4 x 4 column from 2FL to 1FL beam	12	Ea.	$1,980	$23,760	
	(N) gypsum and finish	792	S.F.	$8	$6,336	
GL 1 URM Strengthening	Demo (E) wall finishes	660	S.F.	$4	$2,640	
	Install shear anchors at URM wall @ 3 SF o.c.	220	Ea.	$100	$22,000	
	Build 20' x 16.5' x 8" shotcrete wall	2	Ea.	$12,222	$24,444	
	(N) gypsum and finish	660	S.F.	$8	$5,280	
	Re-build 3' tall wainscotting	120	S.F.	$22	$2,640	
GL 2 URM Strengthening	Demo (E) wall finishes	1040	S.F.	$4	$4,158	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Install shear anchors at URM wall @ 3 SF o.c.	347	Ea.	$100	$34,650	
	Build 63' x 16.5' x 8" shotcrete wall	1	Ea.	$57,750	$57,750	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 7 URM Strengthening	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	2079	S.F.	$8	$16,632	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
	Demo (E) wall finishes	1040	S.F.	$4	$4,158	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Install shear anchors at URM wall @ 3 SF o.c.	347	Ea.	$100	$34,650	
	Build 63' x 16.5' x 8" shotcrete wall	1	Ea.	$57,750	$57,750	
GL 8 URM Strengthening	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	2079	S.F.	$12	$24,948	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
	Demo (E) wall finishes	660	S.F.	$4	$2,640	
	Install shear anchors at URM wall @ 3 SF o.c.	220	Ea.	$100	$22,000	
	Build 20' x 16.5' x 8" shotcrete wall	2	Ea.	$12,222	$24,444	
Strengthen 1st Floor East Wing Area	(N) gypsum and finish	660	S.F.	$8	$5,280	
	Re-build 3' tall wainscotting	120	S.F.	$22	$2,640	
	Remove contents around the perimeter (4' wide)	1898	S.F.	$10	$18,980	
	Temporary work area	1898	S.F.	$2	$2,847	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Floor Opening Strengthening	Custom steel girt (12' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,728	$1,728	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Drag- Strut	Custom steel strut (33k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	
	Remove contents from drag strut path (4' wide)	256	S.F.	$10	$2,560	
	Temporary work area	256	S.F.	$2	$384	
	Remove (E) floor finishes	256	S.F.	$3	$768	
	Replace the floor finishes	256	S.F.	$5	$1,280	
GL 3 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$25,556	$25,556	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL 6 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 8" shotcrete wall	1	Ea.	$25,556	$25,556	
	(N) gypsum and finish	345	S.F.	$12	$4,140	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL C URM Strengthening	Demo (E) wall finishes	776	S.F.	$4	$3,102	
	Install shear anchors at URM wall @ 3 SF o.c.	259	Ea.	$100	$25,850	
	Build 23.5' x 16.5' x 8" shotcrete wall	2	Ea.	$21,542	$43,083	
	Custom drag-strut (42k load) tieing new walls	1	Ea.	$45,000	$45,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	856	S.F.	$8	$6,844	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
GL D URM Strengthening	Demo (E) wall finishes	776	S.F.	$4	$3,102	
	Install shear anchors at URM wall @ 3 SF o.c.	259	Ea.	$100	$25,850	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL E URM Strengthening	Build 23.5' x 16.5' x 8" shotcrete wall	2	Ea.	$21,542	$43,083	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	856	S.F.	$8	$6,844	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
	Demo (E) wall finishes	330	S.F.	$4	$1,320	
	Install shear anchors at URM wall @ 3 SF o.c.	110	Ea.	$100	$11,000	
Strengthen 1st Floor West Wing Area	Build 20' x 16.5' x 8" shotcrete wall	1	Ea.	$18,333	$18,333	
	(N) gypsum and finish	330	S.F.	$8	$2,640	
	Re-build 3' tall wainscotting	60	S.F.	$22	$1,320	
	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$50	$9,900	
	Replace the floor finishes	198	S.F.	$5	$990	
GL B URM Strengthening	Demo (E) wall finishes	974	S.F.	$4	$3,894	
	Install shear anchors at URM wall @ 3 SF o.c.	325	Ea.	$100	$32,450	
	Build 29.5' x 16.5' x 8" shotcrete wall	2	Ea.	$36,056	$72,111	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	1054	S.F.	$8	$8,428	
	Re-build 3' tall wainscotting	177	S.F.	$22	$3,894	
						$1,456,952

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Strengthen Basement Area	Temporary work area for HSS columns	960	S.F.	$2	$1,440	
Add independent HSS column supports	Demo (E) basement wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Demo (E) slab for new footings (12) locations	480	S.F.	$10	$4,800	
	Excavate for new footings (12) locations	192	S.F.	$30	$5,760	
	Add concrete foundation (12) locations	192	S.F.	$100	$19,200	
	Install HSS 4 x 4 column from 1FL to footing	12	Ea.	$2,700	$32,400	
	(N) gypsum and finish	480	S.F.	$8	$3,840	
GL 1 URM Strengthening	Temporary work area	240	S.F.	$2	$360	
	Demo (E) wall finishes	400	S.F.	$4	$1,600	
	Demo (E) slab for new footings	160	S.F.	$10	$1,600	
	Excavate for new footings	160	S.F.	$40	$6,400	
	Install (1) micropiles @ ea. end of the (N) SW	2	Ea.	$18,000	$36,000	
	Add concrete foundation	160	S.F.	$100	$16,000	
	Install shear anchors at URM wall @ 3 SF o.c.	133	Ea.	$100	$13,333	
	Build 20' x 10' x 10" shotcrete wall	1	Ea.	$13,889	$13,889	
	(N) gypsum and finish	400	S.F.	$8	$3,200	
GL 2 URM Strengthening	Temporary work area	378	S.F.	$2	$567	
	Demo (E) wall finishes	630	S.F.	$4	$2,520	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Demo (E) slab for new footings	252	S.F.	$10	$2,520	
	Excavate for new footings	252	S.F.	$40	$10,080	
	Install (3) micropiles @ ea. end of the (N) SW	1	Ea.	$48,000	$48,000	
	Add concrete foundation	252	S.F.	$100	$25,200	
	Install shear anchors at URM wall @ 3 SF o.c.	210	Ea.	$100	$21,000	
	Build 63' x 10' x 10" shotcrete wall	1	Ea.	$58,333	$58,333	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 3 URM Strengthening	(N) gypsum and finish	1670	S.F.	$8	$13,356	
	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	
	Excavate for new footings	92	S.F.	$40	$3,680	
	Install (3) 500k micropiles @ ea. end of the (N) SW	1	Ea.	$72,000	$72,000	
	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 10" shotcrete wall	1	Ea.	$26,620	$26,620	
GL 6 URM Strengthening	(N) gypsum and finish	230	S.F.	$8	$1,840	
	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	
	Excavate for new footings	92	S.F.	$40	$3,680	
	Install (3) 500k micropiles @ ea. end of the (N) SW	1	Ea.	$72,000	$72,000	
	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 10" shotcrete wall	1	Ea.	$26,620	$26,620	
GL 7 URM Strengthening	(N) gypsum and finish	230	S.F.	$8	$1,840	
	Temporary work area	378	S.F.	$2	$567	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Demo (E) wall finishes	630	S.F.	$4	$2,520	
	Demo (E) slab for new footings	252	S.F.	$10	$2,520	
	Excavate for new footings	252	S.F.	$40	$10,080	
	Install (3) micropiles @ ea. end of the (N) SW	1	Ea.	$48,000	$48,000	
	Add concrete foundation	252	S.F.	$100	$25,200	
	Install shear anchors at URM wall @ 3 SF o.c.	210	Ea.	$100	$21,000	
	Build 63' x 10' x 10" shotcrete wall	1	Ea.	$58,333	$58,333	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 8 URM Strengthening	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	1670	S.F.	$8	$13,356	
	Temporary work area	240	S.F.	$2	$360	
	Demo (E) wall finishes	400	S.F.	$4	$1,600	
	Demo (E) slab for new footings	160	S.F.	$10	$1,600	
	Excavate for new footings	160	S.F.	$40	$6,400	
	Install (1) micropiles @ ea. end of the (N) SW	2	Ea.	$18,000	$36,000	
	Add concrete foundation	160	S.F.	$100	$16,000	
	Install shear anchors at URM wall @ 3 SF o.c.	133	Ea.	$100	$13,333	
	Build 20' x 10' x 10" shotcrete wall	1	Ea.	$13,889	$13,889	
GL A URM Strengthening	(N) gypsum and finish	400	S.F.	$8	$3,200	
	Temporary work area	354	S.F.	$2	$531	
	Demo (E) wall finishes	590	S.F.	$4	$2,360	
	Demo (E) slab for new footings	236	S.F.	$10	$2,360	
	Excavate for new footings	236	S.F.	$40	$9,440	
	Install (1) micropiles @ ea. end of the (N) SW	2	Ea.	$18,000	$36,000	
	Add concrete foundation	236	S.F.	$100	$23,600	
	Install shear anchors at URM wall @ 3 SF o.c.	197	Ea.	$100	$19,667	
	Build 29.5' x 10' x 10" shotcrete wall	2	Ea.	$20,486	$40,972	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	32	S.F.	$4	$128	
GL B URM Strengthening	(N) gypsum and finish	622	S.F.	$8	$4,976	
	Temporary work area	354	S.F.	$2	$531	
	Demo (E) wall finishes	590	S.F.	$4	$2,360	
	Demo (E) slab for new footings	236	S.F.	$10	$2,360	
	Excavate for new footings	236	S.F.	$40	$9,440	
	Install (3) 500k micropiles @ ea. end of the (N) SW	2	Ea.	$60,000	$120,000	
	Add concrete foundation	236	S.F.	$100	$23,600	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL C URM Strengthening	Install shear anchors at URM wall @ 3 SF o.c.	197	Ea.	$100	$19,667	
	Build 29.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$40,972	$81,944	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	32	S.F.	$4	$128	
	(N) gypsum and finish	622	S.F.	$8	$4,976	
	Temporary work area	282	S.F.	$2	$423	
	Demo (E) wall finishes	470	S.F.	$4	$1,880	
	Demo (E) slab for new footings	188	S.F.	$10	$1,880	
	Excavate for new footings	188	S.F.	$40	$7,520	
	Install (2) 500k micropiles @ ea. end of the (N) SW	2	Ea.	$60,000	$120,000	
	Add concrete foundation	188	S.F.	$100	$18,800	
GL D URM Strengthening	Install shear anchors at URM wall @ 3 SF o.c.	157	Ea.	$100	$15,667	
	Build 23.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$32,639	$65,278	
	Custom drag-strut (18k load) tieing new walls	1	Ea.	$26,000	$26,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	550	S.F.	$8	$4,400	
	Temporary work area	276	S.F.	$2	$414	
	Demo adjoining (E) walls and finishes	920	S.F.	$6	$5,520	
	Demo (E) wall finishes	460	S.F.	$4	$1,840	
	Demo (E) slab for new footings	184	S.F.	$10	$1,840	
	Excavate for new footings	184	S.F.	$40	$7,360	
	Install (2) micropiles @ ea. end of the (N) SW	2	Ea.	$36,000	$72,000	
	Add concrete foundation	184	S.F.	$100	$18,400	
	Install shear anchors at URM wall @ 3 SF o.c.	153	Ea.	$100	$15,333	
	Build 23.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$25,556	$51,111	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL E URM Strengthening	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	Re-build adjoining walls	460	S.F.	$6	$2,760	
	(N) gypsum and finish	1460	S.F.	$8	$11,680	
	Temporary work area	120	S.F.	$2	$180	
	Demo adjoining (E) walls and finishes	200	S.F.	$6	$1,200	
	Demo (E) wall finishes	200	S.F.	$4	$800	
	Demo (E) slab for new footings	80	S.F.	$10	$800	
	Excavate for new footings	80	S.F.	$40	$3,200	
	Install (2) micropiles @ ea. end of the (N) SW	1	Ea.	$36,000	$36,000	
	Add concrete foundation	80	S.F.	$100	$8,000	
	Install shear anchors at URM wall @ 3 SF o.c.	67	Ea.	$100	$6,667	
	Build 20' x 10' x 12" (avg thick) shotcrete wall	1	Ea.	$22,222	$22,222	
	Re-build adjoining walls	200	S.F.	$6	$1,200	
	(N) gypsum and finish	400	S.F.	$8	$3,200	
						$1,857,588
					Sub-total	**$6,966,813**
					Construction Contingency (25%)	**$1,741,703**
					Contractor Overhead and Profit (25%)	**$2,177,129**
					Total Seismic Construction Cost	**$10,885,645**

APPENDIX - A

SUMMARY DATA SHEET

BUILDING DATA

Building Name: Benton County Courthouse Date: January 29, 2016

Building Address: 120 NW 4th Street, Corvallis, Oregon

Latitude: 44.56528 N Longitude: 123.26237 W By:

Year Built: 1887 Year(s) Remodeled: 1977 Original Design Code: N/A

Area (sf): 33,052 Length (ft): Varies Width (ft): Varies

No. of Stories: Four Story Height: Varies Total Height: 59'-6"(Tower:115'-5")

USE ☐ Industrial ☒ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☒ Other: Assembly

CONSTRUCTION DATA

Gravity Load Structural System: Unreinforced masonry bearing walls with interior steel supports

Exterior Transverse Walls: Unreinforced masonry shear walls Openings? Punched openings

Exterior Longitudinal Walls: Unreinforced masonry shear walls Openings? Punched openings

Roof Materials/Framing: Timber trusses and 2x rafters with T&G straight sheathing

Intermediate Floors/Framing: Timber 3 x 16 floor joists with T&G straight sheathing

Ground Floor: Concrete slab on grade

Columns: Wood and steel columns Foundation: Unknown foundation

General Condition of Structure: Well maintained structure in good condition

Levels Below Grade? N/A

Special Features and Comments: The building houses the Benton County Courts

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Masonry shear wall	Masonry shear wall
Vertical Elements:	Bearing wall	Bearing wall
Diaphragms:	Flexible wood sheathing	Flexible wood sheathing
Connections:	Unknown	Unknown

EVALUATION DATA

BSE-1N Spectral Response Accelerations: S_{Ds} = 0.712 S_{D1} = 0.480

Soil Factors: Class = D F_a = 1.12 F_v = 1.53

BSE-1E Spectral Response Accelerations: S_{XS} = 0.36 S_{X1} = 0.22

Level of Seismicity: High Performance Level: Life Safety

Building Period: T = 0.43

Spectral Acceleration: S_a = 0.36

Modification Factor: $C_mC_1C_2$ = 1.0 Building Weight: W = 7,344,016 lb

Pseudo Lateral Force: V= 0.36

$C_mC_1C_2S_aW$= 2,608,595 lb

BUILDING CLASSIFICATION:

REQUIRED TIER 1 CHECKLISTS	**Yes**	**No**
Basic Configuration Checklist	☒	☐
Building Type URM **Structural Checklist**	☒	☐
Nonstructural Component Checklist	☒	☐

FURTHER EVALUATION REQUIREMENT:

Table 2-1. Basic Performance Objective for Existing Buildings (BPOE)

	Tier 1[a]	Tier 2[a]	Tier 3	
Risk Category	**BSE-1E**	**BSE-1E**	**BSE-1E**	**BSE-2E**
I & II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Nonstructural Performance Not Considered (5-D)
III	See footnote *b* for Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Nonstructural Performance Not Considered (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Nonstructural Performance Not Considered (3-D)

[a]For Tier 1 and 2 assessments, seismic performance for the BSE-2E is not explicitly evaluated.
[b]For Risk Category III, the Tier 1 screening checklists shall be based on the Life Safety Performance Level (S-3), except that checklist statements using the Quick Check procedures of Section 4.5.3 shall be based on MS-factors and other limits that are an average of the values for Life Safety and Immediate Occupancy.

Per USGS Sds = 0.712, so the courthouse site is considered in a region with high level of seismicity.

Table 2-5. Level of Seismicity Definitions

Level of Seismicity[a]	S_{DS}	S_{D1}
Very low	<0.167 g	<0.067 g
Low	≥0.167 g <0.33 g	≥0.067 g <0.133 g
Moderate	≥0.33 g <0.50 g	≥0.133 g <0.20 g
High	≥ 0.50 g	≥0.20 g

[a]The higher level of seismicity defined by S_{DS} or S_{D1} shall govern.

C2.5 LEVEL OF SEISMICITY

The Levels of Seismicity in this standard have been adjusted to match the Seismic Design Categories in ASCE 7 as follows:

SDC A: Very Low
SDC B: Low
SDC C: Moderate
SDC D–F: High

Therefore, the parameters S_{DS} and S_{D1} correspond to the parameters at the BSE-1N level.

The courthouse does not meet the Benchmark Building Criteria

Table 4-6. Benchmark Buildings

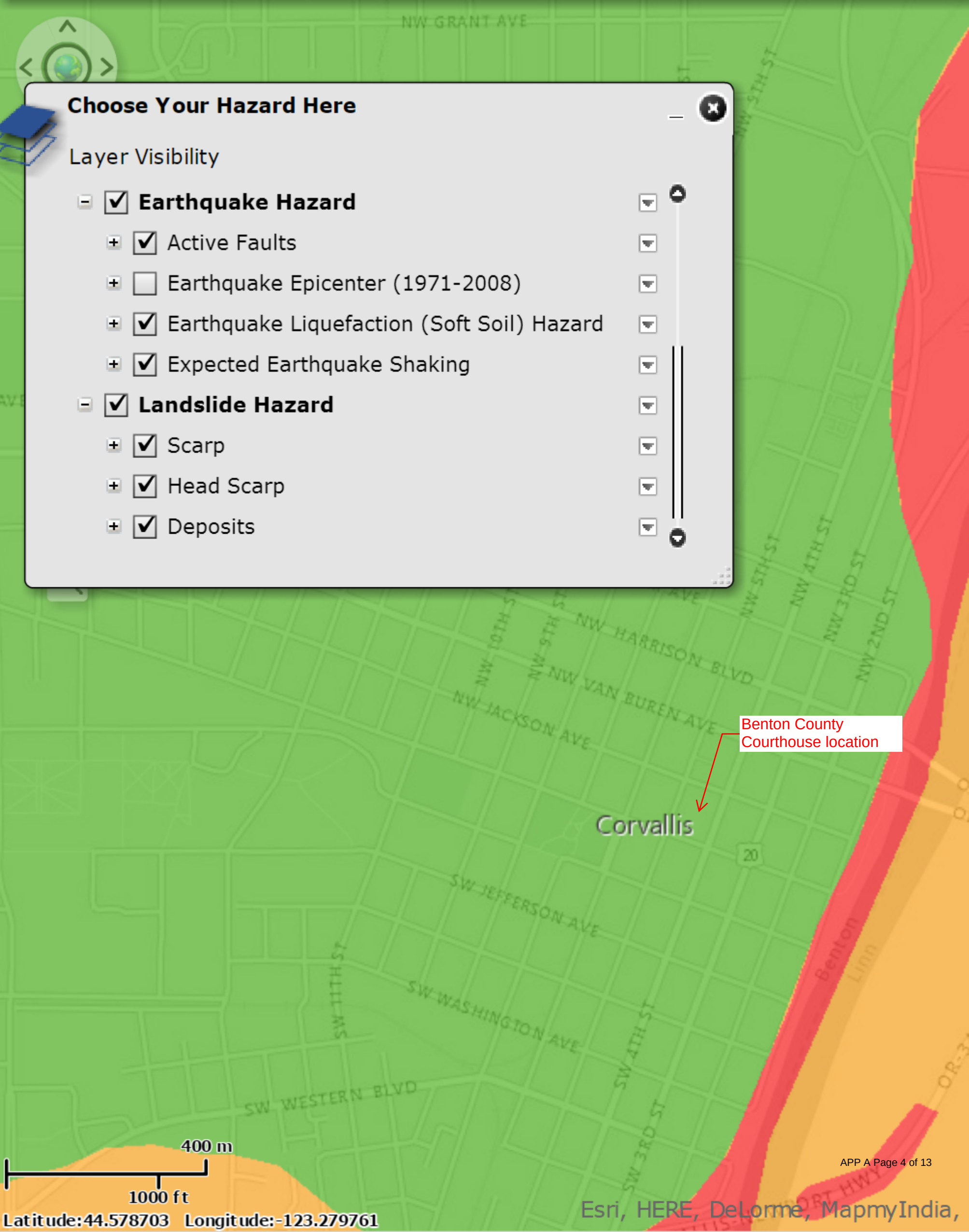
Building Type[a,b]	Building Seismic Design Provisions				Seismic Evaluation or Retrofit Provisions		
	NBC[LS] SBC[LS]	UBC[LS]	IBC[LS]	NEHRP[LS]	FEMA 178[LS]	FEMA 310 (1998e)/ ASCE 31[LS,IO]	FEMA 356 (2000)/ ASCE 41[LS,IO][c,d]
Wood frame, wood shear panels (Types W1 & W2)	1993	1976	2000	1985	[e]	1998	2000
Wood frame, wood shear panels (Type W1a)	[e]	1997	2000	1997	[e]	1998	2000
Steel moment-resisting frame (Types S1 & S1a)	[e]	1994[f]	2000	1997	[e]	1998	2000
Steel concentrically braced frame (Types S2 & S2a)	[e]	1997	2000	[e]	[e]	1998	2000
Steel eccentrically braced frame (Types S2 & S2a)	[e]	1988[f]	2000	1997	[e]	[e]	2000
Buckling-restrained braced frame (Types S2 & S2a)	[e]	[e]	2006	[e]	[e]	[e]	2000
Light metal frame (Type S3)	[e]	[e]	2000	[e]	1992	1998	2000
Steel frame w/ concrete shear walls (Type S4)	1993	1994[?]	2000	1985	[e]	1998	2000
Steel frame with URM infill (Types S5 & S5a)	[e]	[e]	2000	[e]	[e]	1998	2000
Steel plate shear wall (Type S6)	[e]	[e]	2006	[e]	[e]	[e]	2000
Reinforced concrete moment-resisting frame (Type C1)[g]	1993	1994	2000	1997	[e]	1998	2000
Reinforced concrete shear walls (Types C2 & C2a)	1993	1994	2000	1985	[e]	1998	2000
Concrete frame with URM infill (Types C3 & C3a)	[e]	[e]	2000	[e]	[e]	1998	2000
Tilt-up concrete (Types PC1 & PC1a)	[e]	1997	2000	[e]	[e]	1998	2000
Precast concrete frame (Types PC2 & PC2a)	[e]	[e]	2000	[e]	1992	1998	2000
Reinforced masonry (Type RM1)	[e]	1997	2000	[e]	[e]	1998	2000
Reinforced masonry (Type RM2)	1993	1994[?]	2000	1985	[e]	1998	2000
Unreinforced masonry (Type URM)[h]	[e]	1991[i]	2000	[e]	1992	1998	2000
Unreinforced masonry (Type URMa)	[e]	[e]	2000	[e]	[e]	1998	2000
Seismic isolation or passive dissipation	[e]	1991	2000	[e]	[e]	[e]	2000

[a]Building type refers to one of the common building types defined in Table 3-1.
[b]Buildings on hillside sites shall not be considered Benchmark Buildings.
[c]LS: S-3 Structural Performance Level for the BSE-1.
[d]IO: S-1 Structural Performance Level for the BSE-1.
[e]No benchmark year; buildings shall be evaluated using this standard.
[f]Steel moment-resisting frames and eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
[g]Flat slab concrete moment frames shall not be considered Benchmark Buildings.
[h]URM buildings evaluated or retrofitted and shown to be acceptable using Special Procedure (the ABK Methodology, 1984) may be considered benchmark buildings subject to the limitation of Section 15.2.
[i]Refers to the GSREB or its predecessor, the Uniform Code of Building Conservation (UCBC), or its successor, IEBC Appendix Chapter A1.
[LS]Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life Safety Performance Level may be considered Benchmark Buildings.
[IO]Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Immediate Occupancy Performance Level may be considered Benchmark Buildings.

Table 4-7. Checklists Required for a Tier 1 Screening

Level of Seismicity[b]	Level of Building Performance[c]	Required Checklists[a]					
		Very Low Seismicity Checklist (Sec 16.1.1)	Basic Configuration Checklist (Sec. 16.1.2)	Life Safety Checklist (Sec. 16.2LS through 16.15LS)	Immediate Occupancy Checklist (Sec. 16.2IO through 16.15IO)	Life Safety Nonstructural Checklist (Sec. 16.17)	Position Retention Nonstructural Checklist (Sec. 16.17)
Very low	LS	X					
Very low	IO		X		X		X
Low	LS		X	X		X	
Low	IO		X		X		X
Moderate	LS		X	X		X	
Moderate	IO		X		X		X
High	LS		X	X		X	
High	IO		X		X		X

[a]An X designates the checklist that must be completed for a Tier 1 screening as a function of the level of seismicity and level of performance.
[b]Defined in Section 2.5.
[c]LS = Life Safety Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).


Choose Your Hazard Here
Layer Visibility
Earthquake Hazard
Active Faults
Earthquake Epicenter (1971-2008)
Earthquake Liquefaction (Soft Soil) Hazard
Expected Earthquake Shaking
Landslide Hazard
Scarp
Head Scarp
Deposits
NW GRANT AVE
NW HARRISON BLVD
NW VAN BUREN AVE
NW JACKSON AVE
Corvallis
Benton County Courthouse location
SW JEFFERSON AVE
SW WASHINGTON AVE
SW WESTERN BLVD
SW 11TH ST
SW 4TH ST
SW 3RD ST
NW 2ND
Benton
Linn
20
400 m
1000 ft
Latitude:44.578703 Longitude:-123.279761
Esri, HERE, DeLorme, MapmyIndia,

Project: Benton County Courthouse

Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers

Date: January 29, 2016

16.1.2LS LIFE SAFETY BASIC CONFIGURATION CHECKLIST

Low Seismicity

Building System

General

C NC N/A U LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

C NC N/A U ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

C NC N/A U MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

Building Configuration

C NC N/A U WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)

C NC N/A U SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)

C NC N/A U VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)

C NC N/A U GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)

C NC N/A U MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

C NC N/A U TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Geologic Site Hazards

Based on information from the DOGAMI Statewide Geohazard Online Map

C NC N/A U LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)

C NC N/A U SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

C NC N/A U SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Foundation Configuration

C NC N/A U OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

C NC N/A U TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

Project: Benton County Courthouse Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers Date: January 29, 2016

16.16LS LIFE SAFETY STRUCTURAL CHECKLIST FOR BUILDING TYPES URM: UNREINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS AND URMA: UNREINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

Low and Moderate Seismicity

The following shear walls are NC: 1st floor at gridlines 3, 6 B and C; and basement at gridline 6

Seismic-Force-Resisting System

C NC N/A U REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)

C NC N/A U SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 30 lb/in.^{2} for clay units and 70 lb/in.^{2} for concrete units. (Commentary: Sec. A.3.2.5.1. Tier 2: Sec. 5.5.3.1.1)

Connections

The masonry walls have anchors at 8' o.c. at the ceiling of the upper level; no anchors were observed at the other floors

C NC N/A U WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

C NC N/A U WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)

C NC N/A U TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)

C NC N/A U GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Seismic-Force-Resisting System

C NC N/A U PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following (Commentary: Sec. A.3.2.5.2. Tier 2: Sec. 5.5.3.1.2):

Top story of multi-story building	9
First story of multi-story building	15
All other conditions	13

C NC N/A U MASONRY LAYUP: Filled collar joints of multi-wythe masonry walls have negligible voids. (Commentary: Sec. A.3.2.5.3. Tier 2: Sec. 5.5.3.4.1)

Diaphragms (Stiff or Flexible)

The bell tower lounge walls do not meet the proportions but based on analysis are compliant

C NC N/A U OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)

C NC N/A U OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)

Openings are > 8 feet at: 4th GL6; 3rd GL A, E & 6; 2nd GL A, 3 & 6; and 1st GL 3 & 6.

Flexible Diaphragms

The west roof diaphragm has a 2.75 : 1 ratio in the E-W direction

C NC N/A U CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) Cross ties were not observed

C NC N/A U STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

C NC N/A U SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

The west roof diaphragm has straight sheathing with 71.5 span in the E-W direction

The core floor diaphragm span is 51.5' x 65.5' with a 1.27 : 1 ratio in the N-S direction (Vact > Vall)

C NC N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

C NC N/A U OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

Connections

The (E) anchors are installed tight, but the connection from the joist to the diaphragm is unknown

C NC N/A U STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)

C NC N/A U BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads. (Commentary: Sec. A.5.4.5. Tier 2: Sec. 5.7.4.4)

The trusses do not have secondary supports and there is not a Tier 2 path to evaluate the load path

Project: Benton County Courthouse Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers Date: January 29, 2016

16.17 NONSTRUCTURAL CHECKLIST

Life Safety Systems

One transverse brace was observed in Storage Room #10, but does not meet NFPA-13, Sect. 9.3.5.1 bracing requirements

C NC N/A U LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)

C NC N/A U LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)

C NC N/A U LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4) Observed 1" clearance per NFPA-13, Sect. 9.3.4.2

C NC N/A U LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)

Hazardous Materials

The natural gas line does not have seismic bracing or flexible couplings

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)

C NC N/A U LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)

Partitions

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)

Ceilings

The Category C suspended gypsum ceilings are not braced at 144 ft2 and do not have a sufficient gap to allow for relative displacement in accordance with 13.6.4.3

C NC N/A U LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 ft^2 and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)

Light Fixtures

The Courtroom #3 lights are not independently supported

C NC N/A U LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)

C NC N/A U LS-not required; PR-H. PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft and, if rigidly supported, are free to move with the structure to which they are attached without damaging adjoining components. (Commentary: A.7.3.3. Tier 2: Sec. 13.7.9)

C NC N/A U LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)

Cladding and Glazing

C NC N/A U LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. CLADDING ISOLATION: For steel or concrete moment frame buildings, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02. (Commentary: Sec. A.7.4.3. Tier 2: Section 13.6.1)

The glazing for the original and renovated skylights are laminated wire glass annealed material

C NC N/A U LS-MH; PR-MH. MULTI-STORY PANELS: For multi-story panels attached at more than one floor level, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicty, 0.02. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 ft^2 in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)

Masonry Veneer

C NC N/A U LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. STUD TRACKS: For veneer with metal stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)

C NC N/A U LS-not required; PR-MH. OPENINGS: For veneer with metal stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)

Parapets, Cornices, Ornamentation, and Appendages

C NC N/A U LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)

C NC N/A U LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)

C NC N/A U LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)

C NC N/A U LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)

The statue and sloped cornices at the east wall elevation are anchored back to the structure less than 6' o.c.; it is unknown whether they are reinforced.

Masonry Chimneys

The west chimney h/w = 2.1; the URM chimneys are built integrally with the exterior walls

C NC N/A U LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)

C NC N/A U LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)

Stairs

Each of the exterior walls at the stairway openings is non-compliant

C NC N/A U LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)

C NC N/A U LS-LMH; PR-LMH. STAIR DETAILS: In moment frame structures, the connection between the stairs and the structure does not rely on shallow anchors in concrete. Alternatively, the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.5.3.1 without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: 13.6.8)

Contents and Furnishings

Non-compliant contents are located on the 1st and 3rd Floors with aspect ratios as high as 8.4 : 1

C NC N/A U LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)

C NC N/A U LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)

C NC N/A U LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)

C NC N/A U LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)

C NC N/A U LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)

C NC N/A U LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)

Mechanical and Electrical Equipment

The suspended HVAC equipment in the boiler room, storage room 10, storage room 02 and vault room 105 are not seismically

C NC N/A U LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)

C NC N/A U LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)

C NC N/A U LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)

C NC N/A U LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)

C NC N/A U LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)

Piping

C NC N/A U LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)

Ducts

C NC N/A U LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)

Elevators

C NC N/A U LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)

C NC N/A U LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)

C	NC	N/A	U	LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)

APPENDIX - B

Building Code: 2014 Oregon Structural Specialty Code

Soils Report: No **Soils Report by:** N/A **Dated:** N/A

Soil Bearing: 1500 PSF **Retaining Walls:** No

Equivalent Fluid Pressure (active): N/A PCF **Passive bearing:** N/A PCF **Friction:** N/A

Structural System: Building Structure

Vertical System: Wood framed roof / masonry walls **Lateral Sys:** Flexible Diaphragm / Concrete or Masonry Shearwalls

Basic Design Loads:

Element	Roof	Floor	Corridor	
Load Type	Dead	Dead	Dead	
Value (PSF)	20	20	20	
Load Type	Snow	Floor Live	Corridor Live	
Value (PSF)	25	50	100	
Deflection Criteria	L/240	L/360	L/360	

Lateral Design Parameters:

Wind Design: Other: ASCE 41-13 **Wind Speed (3 sec Gust):** 120 MPH

Exposure B

Importance Factors I_W = (ice) I_E = (seismic) I_S = (snow) I_i = (ice) **Risk Cat:** II

Seismic Design

Seismic design parameters are based on published values from the USGS web site.

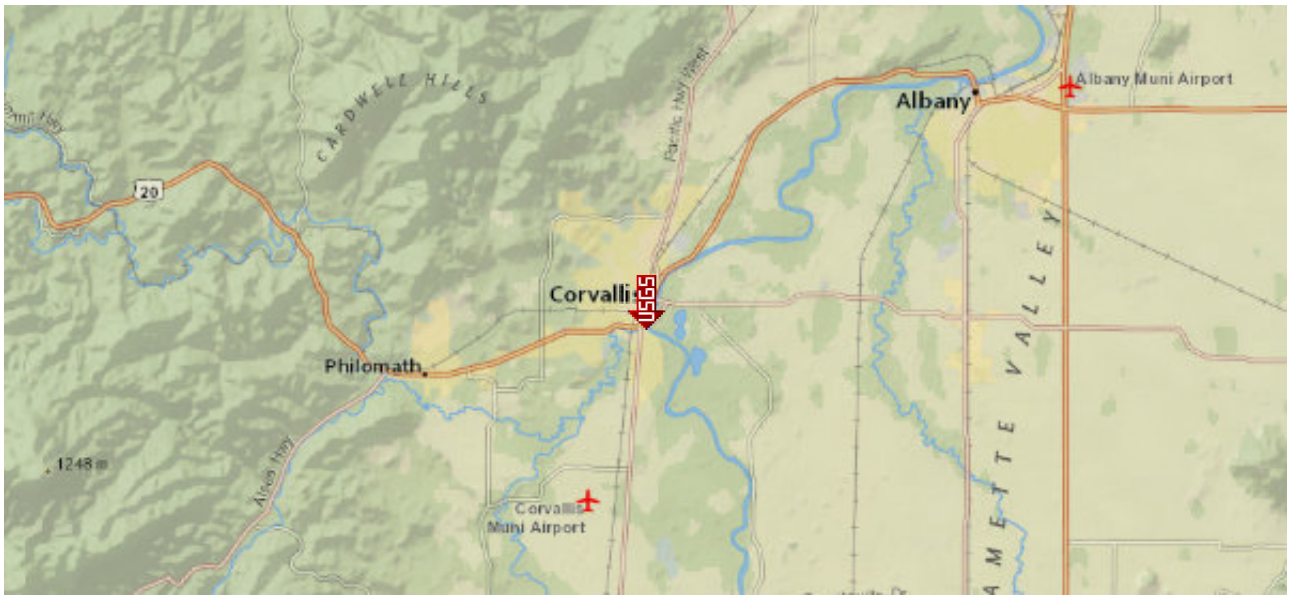
Design Summary:

The following calculations are the Tier I and II analysis of the courthouse building based on the ASCE 41-13 standard for evaluating an existing building considering seismic loading.

USGS Design Maps Summary Report

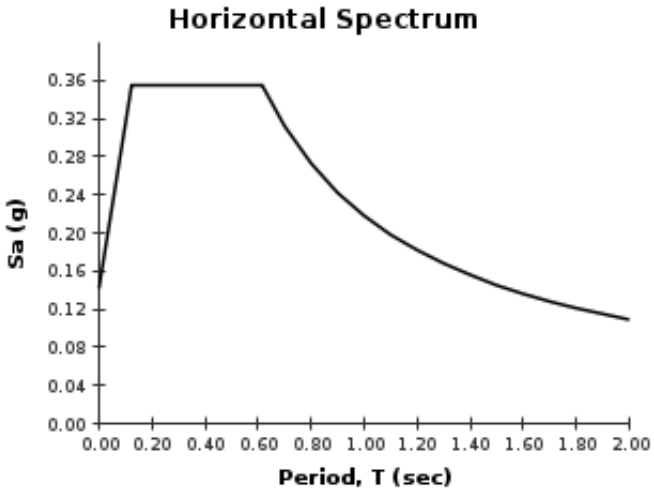
User–Specified Input

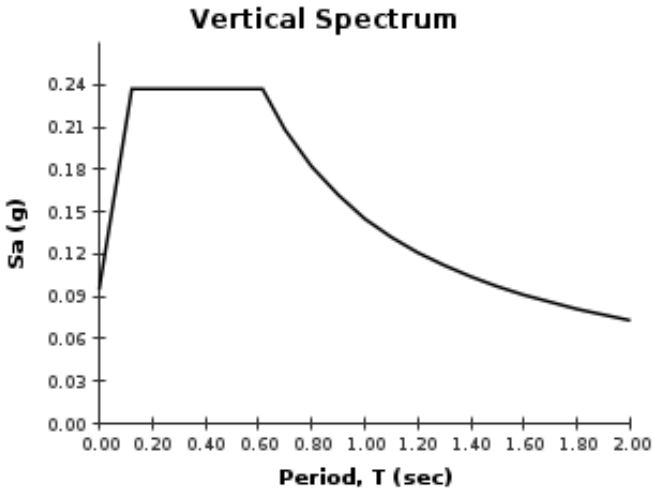
Report Title	Benton County Courthouse Thu December 10, 2015 17:17:07 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-1E (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

$S_{S,20/50}$	0.222 g	$S_{XS,BSE-1E}$	0.355 g
$S_{1,20/50}$	0.091 g	$S_{X1,BSE-1E}$	0.218 g





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USGS Design Maps Summary Report

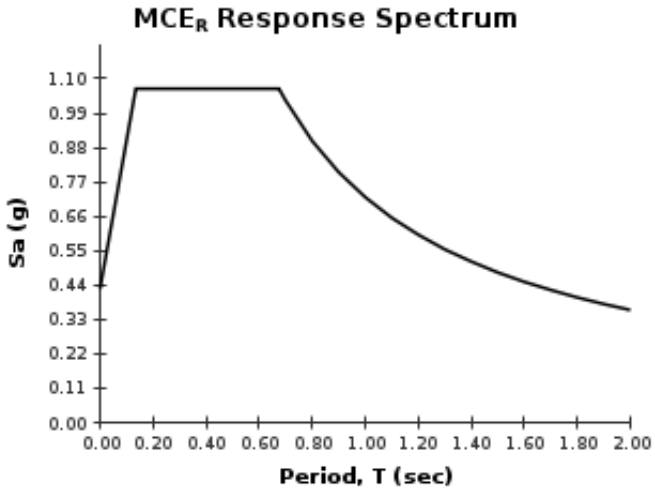
User–Specified Input

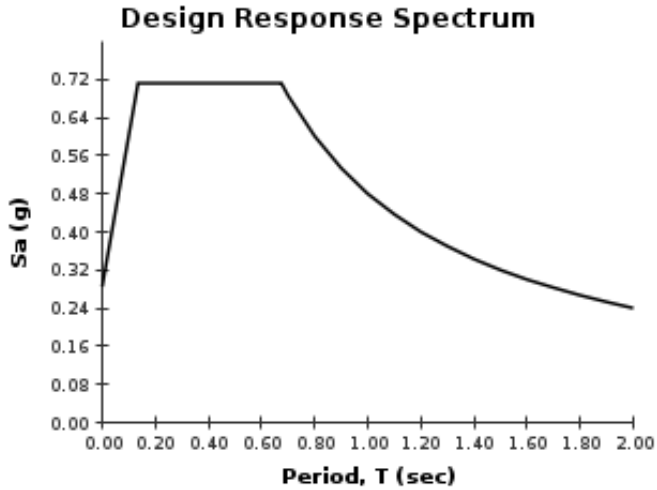
Report Title	Benton County Couthouse Wed November 18, 2015 23:04:50 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.956 g	S_{MS} =	1.068 g	S_{DS} =	0.712 g
S_1 =	0.472 g	S_{M1} =	0.721 g	S_{D1} =	0.480 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





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Benton County Courthouse BPOE Calculations:

Building Weights:

Roof Weights =	20	psf
Wood Floor =	20	psf
Light Framed =	8	psf
8" CMU =	80	psf
12" brick =	120	psf
18" brick =	180	psf
20" brick =	200	psf
30" stone =	350	psf

Building Geometries:

West =	1752	ft^2
Center =	3373	ft^2
East =	1752	ft^2
Total sq footage =	6877	ft^2
Total width =	114.50	ft
Total length =	71.50	ft
Total height =	59.50	ft

Roofs:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

Partial Floors:

Upper Foyer=	14513	lb
Speaker Floor=	9138	lb
Bell Floor=	4102	lb
Top Floor=	995	lb
Sub Total =	28747	lb

Floors:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

East Section

Width =	24.50	ft
Length =	71.50	ft
Bell Tower Lounge Height =	13.67	ft
3rd Floor Height =	14.25	ft
2nd Floor Height =	15.00	ft
1st Floor Height =	16.50	ft
Basement Floor Height =	10.50	ft

Center Section

Width =	65.50	ft
Length =	51.50	ft
4th Floor Height =	8.50	ft
3rd Floor Height =	9.00	ft

West Section

Width =	24.50	ft
Length =	71.50	ft

4th East-West Walls: (starting at north wall)

Gridline 8 =	11760	lb
GL 7 =	2180	lb
GL 6 =	42445	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	42445	lb
GL 2 =	2180	lb
GL 1 =	11760	lb
Sub Total =	112770	lb

4th North-South Walls: (starting at west wall)

Gridline A =	17160	lb
GL B =	17160	lb
GL C =	38451	lb
GL D =	12000	lb
GL E =	26451	lb
Sub Total =	111223	lb
Total =	**375763**	**lb**

3rd East-West Walls: (starting at north wall)

Gridline 8 =	41895	lb
GL 7 =	35370	lb
GL 6 =	86692	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	86692	lb
GL 2 =	35370	lb
GL 1 =	41895	lb
Sub Total =	327913	lb

3rd North-South Walls: (starting at west wall)

Gridline A =	61133	lb
GL B =	61133	lb
GL C =	96775	lb
GL D =	42750	lb
GL E =	54025	lb
Sub Total =	315815	lb
Total =	**795776**	**lb**

2nd East-West Walls: (starting at north wall)

Gridline 8 =	128993	lb
GL 7 =	172429	lb
GL 6 =	90821	lb

2nd North-South Walls: (starting at west wall)

Gridline A =	188224	lb
GL B =	188224	lb
GL C =	188224	lb

GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	90821	lb
GL 2 =	172429	lb
GL 1 =	128993	lb
Sub Total =	784485	lb

GL D =	131625	lb
GL E =	56599	lb
Sub Total =	752895	lb
Total =	**1674915**	**lb**

1st East-West Walls: (starting at north wall)

Gridline 8 =	154350	lb
GL 7 =	206325	lb
GL 6 =	108675	lb
GL 5 =	108075	lb
GL 4 =	108075	lb
GL 3 =	108675	lb
GL 2 =	206325	lb
GL 1 =	154350	lb
Sub Total =	1154850	lb

1st North-South Walls: (starting at west wall)

Gridline A =	225225	lb
GL B =	225225	lb
GL C =	225225	lb
GL D =	157500	lb
GL E =	67725	lb
Sub Total =	900900	lb
Total =	**2193285**	**lb**

Basement East-West Walls: (starting at north wall)

Gridline 8 =	162803	lb
GL 7 =	217624	lb
GL 6 =	114626	lb
GL 5 =	159165	lb
GL 4 =	159165	lb
GL 3 =	114626	lb
GL 2 =	217624	lb
GL 1 =	162803	lb
Sub Total =	1308435	lb

Basement North-South Walls: (starting at west wall)

Gridline A =	237559	lb
GL B =	191595	lb
GL C =	191595	lb
GL D =	166125	lb
GL E =	71434	lb
Sub Total =	858308	lb
Total =	**2304278**	**lb**

4.5.2.1 - Pseudo Seismic Force - for use with quick check proceedures

S_s=	22.20%	
S_1 =	9.10%	
F_a =	1.60	Table 2-3, page 46
F_v =	2.40	Table 2-4, page 46
S_{XS} =	0.36	eqn. 2-1, pg. 46
S_{X1} =	0.22	eqn. 2-2, pg. 46
C_t =	0.02	eqn. 4-5, pg. 69
h_n =	59.50	ft (roof height)
β =	0.75	eqn. 4-5, pg. 69
T =	0.428	eqn. 4-5, pg. 69
Sa =	0.36	eqn. 4-4, pg. 68

Evaluation: Life Safety

Immediate Occupancy Responses:	
S_s=	22.20%
S_1 =	9.10%
Life Safety:	
S_s=	22.20%
S_1 =	9.10%
Collapse Prevention:	
S_s=	68.50%
S_1 =	33.40%

Total Diaphragm Weight

C =	1	table 4-8, pg. 68
W_1 =	2304278	lb (lowest floor)
W_2 =	2193285	lb
W_3 =	1674915	lb
W_4 =	795776	lb
W_5 =	375763	lb

Heights of floors to base:

h_1 =	10.50	ft
h_2 =	27.00	ft
h_3 =	42.00	ft
h_4 =	51.00	ft
h_5 =	59.50	ft

W_{total} =	7344016	lb
V_{total} =	2608595	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

k =	1.00		
F_1 =	291251	lb	0.36
F_2 =	712855	lb	0.92
F_3 =	846808	lb	1.42
F_4 =	488544	lb	1.73
F_5 =	269137	lb	2.02
Σ =	2608595	lb	

7.4.1.3.1 - Pseudo Seismic Force for LSP (linear static proceedure)

Sa =	0.4	
C_1C_2 =	1	from Table 7-3
C_m =	1	from Table 7-4
V =	0.36	*W

Diaphragm Weights East-West Walls:

W_1 =	995843	lb (lowest floor)
W_2 =	1038435	lb
W_3 =	890430	lb
W_4 =	467863	lb
W_5 =	262992	lb
W_{total} =	3655563	lb
V_{total} =	1298456	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	117651	lb
F_2 =	315471	lb
F_3 =	420791	lb
F_4 =	268476	lb
F_5 =	176067	lb
Σ =	1298456	lb

Diaphragm Weights North-South Walls:

W_1 =	1445970	lb (lowest floor)
W_2 =	1292385	lb
W_3 =	922020	lb
W_4 =	479961	lb
W_5 =	264540	lb
W_{total} =	4404875	lb
V_{total} =	1564612	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	184119	lb
F_2 =	423160	lb
F_3 =	469612	lb
F_4 =	296842	lb
F_5 =	190879	lb
Σ =	1564612	lb

URM Shear Wall Loads/Diaphragm Loads for Life Safety Evaluation:

Ms =	1.5	URM Wall, from Table 4-9, pg. 70
M_1 =	2.0	Straight Diaphragm (chorder), from Table 12-3
M_2 =	1.5	Straight Diaphragm (unchorder), from Table 12-4
M_3 =	2.5	Wood Diaphragm (chorder), from Table 12-3
M_4 =	2.0	Wood Diaphragm (unchorded), from Table 12-3

4th Floor / Roof	Shear (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	16489	49	224	2	30	OK	126	120	NG
GL 7 =	31555	66	321	2	30	OK	235	120	NG
GL 6 =	60017	20	2052	9	30	OK	521	450	NG
GL 5 =	0	0							
GL 4 =	0	0							
GL 3 =	60017	20	2052	9	30	OK	521	450	NG
GL 2 =	31555	66	321	2	30	OK	235	120	NG
GL 1 =	16489	49	224	2	30	OK	126	120	NG
Gridline A =	24876	72	232	2	30	OK	175	120	NG
GL B =	75088	72	700	5	30	OK	650	120	NG
GL C =	82650	72	771	4	30	OK	413	120	NG

GL D =	30709	50	409	3	30	OK	175	120	NG
GL E =	17061	15	784	4	30	OK	178	300	OK
	216123	230385	268	279					

3rd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	33656	50145	38	880	6	30	OK	192	300	OK
GL 7 =	59500	91055	66	927	6	30	OK	287	450	OK
GL 6 =	99320	159337	24	4426	20	30	OK	274	450	OK
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	99320	159337	32	3286	15	30	OK	274	450	OK
GL 2 =	59500	91055	66	927	6	30	OK	287	450	OK
GL 1 =	33656	50145	38	880	6	30	OK	192	300	OK
Gridline A =	50922	75798	58	871	6	30	OK	163	450	OK
GL B =	129007	204094	67	2021	14	30	OK	606	450	NG
GL C =	141667	224317	62	2432	11	30	OK	514	450	NG
GL D =	56314	87023	39	1507	10	30	OK	163	450	OK
GL E =	31111	48172	16	2007	9	30	OK	222	450	OK
	384951	409020	263	241						

2nd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	75244	125389	38	2200	10	30	OK	300	300	NG
GL 7 =	134811	225866	47	3192	15	30	OK	449	450	OK
GL 6 =	168651	327988	33	6693	31	30	NG	430	450	OK
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	168651	327988	33	6693	31	30	NG	430	450	OK
GL 2 =	134811	225866	47	3192	15	30	OK	449	450	OK
GL 1 =	75244	125389	38	2200	10	30	OK	300	300	NG
Gridline A =	113064	188862	55	2289	11	30	OK	258	450	OK
GL B =	236596	440690	63	4688	22	30	OK	959	450	NG
GL C =	236596	460913	59	5230	24	30	OK	813	450	NG
GL D =	111820	198843	39	3399	16	30	OK	258	450	OK
GL E =	38964	87136	16	3631	17	30	OK	351	450	OK
	757414	737040	236	231						

1st Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	76886	202275	36	3720	17	30	OK	225	300	OK
GL 7 =	128439	354305	47	5026	23	30	OK	337	450	OK
GL 6 =	84929	412917	29	9386	43	30	NG	185	450	OK
GL 5 =	72583	72583	80	606	3	30	OK	152	450	OK
GL 4 =	72583	72583	81	595	3	30	OK	152	450	OK
GL 3 =	84929	412917	33	8426	39	30	NG	185	450	OK
GL 2 =	128439	354305	47	5026	23	30	OK	337	450	OK
GL 1 =	76886	202275	36	3720	17	30	OK	225	300	OK
Gridline A =	121636	310498	50	4140	19	30	OK	233	450	OK
GL B =	232949	673640	62	7205	33	30	NG	865	450	NG
GL C =	232949	693863	62	7461	35	30	NG	733	450	NG
GL D =	114575	313418	36	5751	27	30	OK	233	450	OK
GL E =	41050	128187	16	5233	24	30	OK	316	450	OK
	725674	743160	390	227						

Basement Floor	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	66055	268330	36	4935	14	30	OK	84	300	OK
GL 7 =	97868	452174	59	5095	14	30	OK	126	450	OK
GL 6 =	57993	470910	24	12903	36	30	NG	200	450	OK
GL 5 =	69288	141871	78	1207	3	30	OK	57	450	OK
GL 4 =	69288	141871	69	1366	4	30	OK	57	450	OK
GL 3 =	57993	470910	29	10950	30	30	NG	200	450	OK
GL 2 =	97868	452174	62	4882	14	30	OK	126	450	OK
GL 1 =	66055	268330	36	4935	14	30	OK	84	300	OK
Gridline A =	102497	412995	60	4602	13	30	OK	101	450	OK
GL B =	134603	808243	63	8553	24	30	OK	376	450	OK
GL C =	134603	828466	67	8223	23	30	OK	319	450	OK
GL D =	84518	397936	39	6745	19	30	OK	101	450	OK
GL E =	32768	160954	16	6706	19	30	OK	138	450	OK
	582407	488990	394	245						

Foundation Design	M_{OT} (ft*lb)	M_R (ft*lb)	Uplift (lb)	250k MP
Gridline 8 =	4246573.8	3589560	25269.76136	0.1
GL 7 =	14843486	21075390	-239688.6043	-1.0
GL 6 =	19537828	8106480	439667.2159	1.8
GL 5 =	2616319.5	10076625	-286934.827	-1.1
GL 4 =	2616319.5	10076625	-286934.827	-1.1
GL 3 =	19537828	8106480	439667.2159	1.8
GL 2 =	14843486	21075390	-239688.6043	-1.0
GL 1 =	4246573.8	3589560	25269.76136	0.1
				0.0
Gridline A =	12979732	27146047.5	-544858.2894	-2.2
GL B =	14141469	4163040	356372.4571	1.4
GL C =	14684241	4163040	375757.192	1.5
GL D =	6588812.2	4163040	86634.72319	0.3
GL E =	5952716.2	4228380	66320.62431	0.3

Section 16.1.2 Basic Configuration - Life Safety (Tier 1 evaluations)

Life Safety

Adjacent Building, Sect. 5.4.1.1

Taller Building =	59.50	ft
Shorter Building =	10	ft
Clear Distance =	4.80	inch

Weak Story, Sect. 5.4.2.1

Basement SW (NS) =	245.33	ft	108% of story above
1st Floor SW (NS) =	226.99	ft	98% of story above
2nd Floor SW (NS) =	231.42	ft	96% of story above
3rd Floor SW (NS) =	241.33	ft	86% of story above
4th Floor SW (NS) =	279.00	ft	
Basement SW (EW) =	394.00	ft	101% of story above
1st Floor SW (EW) =	389.66	ft	165% of story above
2nd Floor SW (EW) =	235.67	ft	89% of story above
3rd Floor SW (EW) =	263.33	ft	98% of story above
4th Floor SW (EW) =	268.00	ft	

Change in Building Mass, Sect. 5.4.2.5

Basement Mass =	2304278	lbs	105% of story above
1st Floor Mass =	2193285	lbs	131% of story above
2nd Floor Mass =	1674915	lbs	210% of story above
3rd Floor Mass =	795776	lbs	212% of story above
4th Floor Mass =	375763	lbs	

Foundation - Overturning, Sect. 5.4.3.3

Basement SW (NS) =	71.5	ft	
b/h Ratio (NS) =	1.20		
Basement SW (EW) =	113.5	ft	
b/h Ratio (EW) =	1.91		
0.6 x Sa =	0.51		**< MIN b/h ratio of 1.2 OK**

Section 16.16 URM Bearing Walls - Life Safety Evaluation

Wall Shear Stress Check - Shear Stress in Shear Walls, Sect. 4.5.3.3

Tier 1 Quick Check - Shear Wall loading see page

Tier 2 - Shear Wall Capacities - Sect. 7.5.1.5

m =	3	from Table 11-3 for Life Safety
K =	0.75	from 11.2.4
F_m =	600	PSI, from Table 11-2(a)
F_t =	45	PSI, from Table 11-2(a)
F_v =	78.3837	PSI per shear area, from TMS 402, Section 3.2.4
E_m =	420000	PSI, per TMS 402, Section 1.8.2.2
G_m =	168000	PSI, per TMS 402, Section 1.8.2.2

Check 2nd Floor Gridline 3:

Total Wall Load=	327988	lb			
Wall height =	15.00	ft	15.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	11.5	ft	21.25	ft	
Pier thickness =	1.5	ft	1.5	ft	
L / h_{eff} =	0.77	for (11-12)	1.42		
Trib DL P_D =	55262.1	lb	102114.8	lb	
Pier SW P_W =	31050	lb	57375	lb	
Wall Rigidity =	204166		545641		
Load to Pier =	89308.3	7765.939	238679.6	11231.98	327987.9
Shear Stress=	36	psi	52	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)	49	psi	
In-plane V_{bjs1} =	120781	lb, 11.3.2.2.2 (11-9)	223182	lb	
In-plane V_{bjs2} =	27631	lb, 11.3.2.2.2 (11-10)	51057	lb	
Rocking V_r =	48843	lb, 11.3.2.2.1 (11-8)	166773	lb	
Lower Bound V_{tc} =	49780	lb, 11.3.2.2.3 (11-11)	169973	lb	
Lower Bound V_{dt} =	137595	lb, 11.3.2.2.4 (11-12)	292242	lb	
Lower Bound P_{CL} =	1013472	lb, 11.3.2.2.5 (11-13)	1872720	lb	
Controlling Q_{CE} =	27631	lb	51057	lb	
Allowable Stress=	25	psi **NG**	25	psi **NG**	
DCR =	1.44	2359.864	DCR = 2.08	5825.906	

Check 2nd Floor Gridline 6:

Total Wall Load=	327988	lb			
Wall height =	15.00	ft	15.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	9.5	ft	23.33	ft	
Pier thickness =	1.5	ft	1.5	ft	
L / h_{eff} =	0.63	for (11-12)	1.56		
Trib DL P_D =	45651.3	lb	112110	lb	
Pier SW P_W =	25650	lb	62991	lb	
Wall Rigidity =	140188		618528		
Load to Pier =	60602.5	6379.212	267385.4	11461.01	327987.9
Shear Stress=	30	psi	53	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)	49	psi	

In-plane V_{bjs1} =	99776	lb, 11.3.2.2.2 (11-9)	245028	lb	
In-plane V_{bjs2} =	22826	lb, 11.3.2.2.2 (11-10)	56055	lb	
Rocking V_r =	33331	lb, 11.3.2.2.1 (11-8)	201019	lb	
Lower Bound V_{tc} =	33971	lb, 11.3.2.2.3 (11-11)	204876	lb	
Lower Bound V_{dt} =	113665	lb, 11.3.2.2.4 (11-12)	320848	lb	
Lower Bound P_{CL} =	837216	lb, 11.3.2.2.5 (11-13)	2056026	lb	
Controlling Q_{CE} =	22826	lb	56055	lb	
Allowable Stress=	25	psi **NG**	25	psi	**NG**
DCR =	1.18	973.1371	DCR = 2.12	6054.936	

Check 1st Floor Gridline 3:

Total Wall Load=	412917	lb			
Wall height =	15.58	ft	15.58	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	13.5	ft	19.25	ft	
Pier thickness =	1.83	ft	1.83	ft	
L / h_{eff} =	0.87	for (11-12)	1.24		
Trib DL P_D =	102398	lb	146011.3	lb	
Pier SW P_W =	37859.4	lb	53984.7	lb	
Wall Rigidity =	332170		579610		
Load to Pier =	150430	11142.94	262487.6	13635.72	
Shear Stress=	42	psi	52	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1			
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)	52	psi	
In-plane V_{bjs1} =	184606	lb, 11.3.2.2.2 (11-9)	263234	lb	
In-plane V_{bjs2} =	51199	lb, 11.3.2.2.2 (11-10)	73006	lb	
Rocking V_r =	54689	lb, 11.3.2.2.1 (11-8)	111198	lb	
Lower Bound V_{tc} =	55062	lb, 11.3.2.2.3 (11-11)	111955	lb	
Lower Bound V_{dt} =	213055	lb, 11.3.2.2.4 (11-12)	349196	lb	
Lower Bound P_{CL} =	1451468	lb, 11.3.2.2.5 (11-13)	2069686	lb	
Controlling Q_{CE} =	51199	lb	73006	lb	
Allowable Stress=	32	psi **NG**	32	psi	**NG**
DCR =	1.31	2609.811	DCR = 1.60	5102.594	

Check 1st Floor Gridline 6:

Total Wall Load=	412917	lb
Wall height =	15.58	ft
Pier height =	8.00	ft
Pier length =	29.25	ft
Pier thickness =	1.83	ft
L / h_{eff} =	1.88	for (11-12)
Trib DL P_D =	221861	lb
Pier SW P_W =	82028.7	lb
Wall Rigidity =	1003921	
Load to Pier =	412917	14116.83
Shear Stress=	54	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)
In-plane V_{bjs1} =	399979	lb, 11.3.2.2.2 (11-9)
In-plane V_{bjs2} =	110931	lb, 11.3.2.2.2 (11-10)
Rocking V_r =	444172	lb, 11.3.2.2.1 (11-8)
Lower Bound V_{tc} =	447197	lb, 11.3.2.2.3 (11-11)
Lower Bound V_{dt} =	530597	lb, 11.3.2.2.4 (11-12)
Lower Bound P_{CL} =	3144848	lb, 11.3.2.2.5 (11-13)
Controlling Q_{CE} =	110931	lb
Allowable Stress=	32	psi **NG**
DCR =	1.65	5583.703

Check 1st Floor Gridline B:

Total Wall Load=	673640	lb						
Wall height =	16.50	ft	16.50	ft	16.50	ft	16.50	ft
Pier height =	8.00	ft	9.00	ft	8.00	ft	9.00	ft
Pier length =	11.5	ft	14.25	ft	16.83	ft	19.75	ft

Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.70	for (11-12)		0.86			1.02			1.20	
Trib DL P_D =	62438.1	lb		77368.95	lb		91376.8	lb		107230.7	lb
Pier SW P_W =	34155	lb		42322.5	lb		49985.1	lb		58657.5	lb
Wall Rigidity =	249083			297138			475040			506405	
Load to Pier =	109835	9550.912		131026.1	9194.817		209473.9	12446.46		223304.3	11306.55
Shear Stress=	36	psi		35	psi		47	psi		43	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1			1			1	
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi
In-plane V_{bjs1} =	144862	lb, 11.3.2.2.2 (11-9)		179503	lb		212002	lb		248785	lb
In-plane V_{bjs2} =	31219	lb, 11.3.2.2.2 (11-10)		38684	lb		45688	lb		53615	lb
Rocking V_r =	30295	lb, 11.3.2.2.1 (11-8)		46516	lb		64885	lb		89353	lb
Lower Bound V_{tc} =	31107	lb, 11.3.2.2.3 (11-11)		47763	lb		66623	lb		91747	lb
Lower Bound V_{dt} =	138397	lb, 11.3.2.2.4 (11-12)		210889	lb		273704	lb		321192	lb
Lower Bound P_{CL} =	1236436	lb, 11.3.2.2.5 (11-13)		1532105	lb		1809497	lb		2123444	lb
Controlling Q_{CE} =	30295	lb		38684	lb		45688	lb		53615	lb
Allowable Stress=	22	psi	**NG**	23	psi	**NG**	23	psi	**NG**	23	**NG**
DCR =	1.61	3623.608	DCR =	1.51	3086.742	DCR =	2.04	6338.385	DCR =	1.85	5198.473

Check 1st Floor Gridline C:

Total Wall Load=	693863	lb									
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft
Pier height =	8.00	ft		9.00	ft		8.00	ft		9.00	ft
Pier length =	14.25	ft		15.25	ft		15.5	ft		17	ft
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.86	for (11-12)		0.92			0.94			1.03	
Trib DL P_D =	77369	lb		82798.35	lb		84155.7	lb		92299.8	lb
Pier SW P_W =	42322.5	lb		45292.5	lb		46035	lb		50490	lb
Wall Rigidity =	364047			334521			417652			400927	
Load to Pier =	161643	11343.37		152992.4	10032.29		185444.8	11964.18		178018.6	10471.68
Shear Stress=	43	psi		38	psi		45	psi		40	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1			1			1	
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi
In-plane V_{bjs1} =	179503	lb, 11.3.2.2.2 (11-9)		192100	lb		195249	lb		214144	lb
In-plane V_{bjs2} =	38684	lb, 11.3.2.2.2 (11-10)		41399	lb		42078	lb		46150	lb
Rocking V_r =	46516	lb, 11.3.2.2.1 (11-8)		53274	lb		55035	lb		66203	lb
Lower Bound V_{tc} =	47763	lb, 11.3.2.2.3 (11-11)		54701	lb		56509	lb		67976	lb
Lower Bound V_{dt} =	210889	lb, 11.3.2.2.4 (11-12)		243049	lb		252075	lb		276469	lb
Lower Bound P_{CL} =	1532105	lb, 11.3.2.2.5 (11-13)		1639621	lb		1666500	lb		1827775	lb
Controlling Q_{CE} =	38684	lb		41399	lb		42078	lb		46150	lb
Allowable Stress=	23	psi	**NG**	23	psi	**NG**	23	psi	**NG**	23	**NG**
DCR =	1.86	5235.299	DCR =	1.64	3924.212	DCR =	1.96	5856.109	DCR =	1.71	4363.608

Check Basement Gridline 3:

Total Wall Load=	470910	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		5.00	ft		8.00	ft		8.00	ft
Pier length =	3.75	ft		6.75	ft		8.33	ft		11.25	ft
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.38	for (11-12)		0.68			0.83			1.13	
Trib DL P_D =	40782.8	lb		73408.95	lb		90592.08	lb		122348.3	lb
Pier SW P_W =	13125	lb		23625	lb		14994	lb		20250	lb
Wall Rigidity =	81936			301931			129304			238981	
Load to Pier =	51299.1	13679.75		189033.8	28005.01		80954.82	9718.466		149622.2	13299.75
Shear Stress=	38	psi		78	psi		37	psi		50	psi
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5	
LB Bed-Joint V_{mL} =	53	psi, 11.2.3.6.3 (11-6)		53	psi		58	psi		58	psi
In-plane V_{bjs1} =	71016	lb, 11.3.2.2.2 (11-9)		127829	lb		127613	lb		172347	lb
In-plane V_{bjs2} =	20391	lb, 11.3.2.2.2 (11-10)		36704	lb		45296	lb		61174	lb
Rocking V_r =	9097	lb, 11.3.2.2.1 (11-8)		29474	lb		39579	lb		72190	lb

Lower Bound V_{tc} =	9147	lb, 11.3.2.2.3 (11-11)		29635	lb		38940	lb		71025	lb
Lower Bound V_{dt} =	69700	lb, 11.3.2.2.4 (11-12)		154282	lb		172509	lb		232980	lb
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		895610	lb		1209557	lb
Controlling Q_{CE} =	9097	lb		29474	lb		38940	lb		61174	lb
Allowable Stress=	15	psi	**NG**	27	psi	**NG**	40	psi	**OK**	46	**NG**
DCR =	2.51	8221.589	DCR =	2.85	18180.33	DCR =	0.92	-799.5952	DCR =	1.09	1064.926

Check Basement Gridline 6:

Total Wall Load=	470910	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		6.50	ft		8.00	ft		8.00	ft
Pier length =	3.75	ft		6.75	ft		5.5	ft		9.33	ft
Pier thickness =	2.5	ft		2.5	ft		1.5	ft		1.5	ft
L / h_{eff} =	0.38	for (11-12)		0.68			0.55			0.93	
Trib DL P_D =	40047.8	lb		72085.95	lb		58736.7	lb		99638.8	lb
Pier SW P_W =	13125	lb		23625	lb		9900	lb		16794	lb
Wall Rigidity =	81936			175608			39509			135040	
Load to Pier =	127740	34064.01		191383.7	28353.14		61594.43	11198.99		210529.8	22564.82
Shear Stress=	95	psi		79	psi		52	psi		104	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1			1			1	
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)		52	psi		62	psi		62	psi
In-plane V_{bjs1} =	70649	lb, 11.3.2.2.2 (11-9)		127168	lb		73918	lb		125392	lb
In-plane V_{bjs2} =	20024	lb, 11.3.2.2.2 (11-10)		36043	lb		29368	lb		49819	lb
Rocking V_r =	8973	lb, 11.3.2.2.1 (11-8)		29072	lb		16988	lb		48884	lb
Lower Bound V_{tc} =	9035	lb, 11.3.2.2.3 (11-11)		29273	lb		16279	lb		46844	lb
Lower Bound V_{dt} =	69212	lb, 11.3.2.2.4 (11-12)		153202	lb		102652	lb		174135	lb
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		484704	lb		822234	lb
Controlling Q_{CE} =	8973	lb		29072	lb		16279	lb		46844	lb
Allowable Stress=	15	psi	**NG**	27	psi	**NG**	31	psi	**NG**	52	**NG**
DCR =	6.33	28680.27	DCR =	2.93	18662.4	DCR =	1.68	4539.542	DCR =	2.00	11267.98

Wall Anchorage - Check shear capacity of anchorage, Sect. 5.7.1.1

Tier 1 - Quick Check, Sect. 4.5.3.7

Wall thickness =	1.5	ft	
Wall W =	180	psf	
X =	1.3	from Sect 4.5.3.7	
S_x =	0.36	from 4.5.2.1 (2-1)	
A_p =	54.68	sf, area to anchor	
T_c =	4544.83	lb	**Tc < Ta OK**
m =	3	from Table 12-3 for Life Safety	
Anchor Capacity =	5845.5	lb, per TMS 3.1.6.3.1.2	

Tier 2 - Sect. 5.7.1.1 and 7.2.11.1

Height =	13.67	ft	
Wall thickness =	1.5	ft	
Ratio =	9.11		
X =	1.3	from Table 7-2	
K_a =	1.515	Eq 7-11	
K_h =	1	Eq 7-12	
F_p =	38.75	psf, from 7.2.11.1 (7-9)	
F_{pMIN} =	70.90	psf, from 7.2.11.1 (7-10)	
T_c =	3876.92	lb	**Tc < Ta OK**
DCR =	0.66		

Proportions - Check Height to Thickness Rations, Sect. 5.5.3.1.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

Top story height =	9.00	ft			
Wall thickness =	1	ft	Ratio:	9.0	**OK**
Bell Lounge h =	13.67	ft			

Wall thickness =	1.33	ft	Ratio:	10.3	**NG**
First story height =	10.00	ft			
Wall thickness =	1.5	ft	Ratio:	6.7	**OK**
Other story height =	15.58	ft			
Wall thickness =	1.5	ft	Ratio:	10.4	**OK**

Tier 2 - Bell Tower Lounge - Sect. 11.3.3.2

Height =	13.67	ft	
Wall thickness =	1.33	ft	
Ratio =	10.28		
X =	1.3	from Table 7-2	
F_p =	73.70	psf, from 7.2.11.2 (7-13)	
OOP Moment =	1721.46	ft-lbs	
F_t =	40.55	psi	**< Ft = 45 psi OK**
DCR =	0.90		

For Collapse Prevention - Bell Tower Lounge - Sect. 11.3.3.2

S_{X1} =	0.22	from 4.5.2.1	
Allowable h/t =	14	**per Table 11-5, OK**	

Diaphragms - Openings at Shear Walls, Sect. 5.6.1.3

Bell Tower Lounge Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	8.50	ft	
Diaphragm V =	521	plf, from page	
Revised Dia. V =	691.39	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.54		

Wall OOP, Sect. 11.3.3.2

Height =	27.92	ft	
Wall thickness =	1.5	ft	
Ratio =	18.61		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	8098.97	ft-lbs	
F_t =	149.98	psi	**> Ft = 45 psi NG**
DCR =	3.33		

3rd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	16.75	ft	
Diaphragm V =	274	plf, from page	
Revised Dia. V =	532.77	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.18		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	4914.93	ft-lbs	
F_t =	91.02	psi	**> Ft = 45 psi NG**
DCR =	2.02		

Diaphragm strength, Sect. 12.5.2.1

GL E Opening =	9.75	ft	
Diaphragm V =	222	plf, from page	
Revised Dia. V =	432.77	plf	**OK**
Dia. Capacity =	450	plf, from page	
DCR =	0.96		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	4914.93	ft-lbs	
F_t =	91.02	psi	**> Ft = 45 psi NG**
DCR =	2.02		

3rd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft	
Diaphragm V =	163	plf, from page	
Revised Dia. V =	185.44	plf	**OK**
Dia. Capacity =	450	plf, from page	
DCR =	0.41		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	4914.93	ft-lbs	
F_t =	91.02	psi	**> Ft = 45 psi NG**
DCR =	2.02		

2nd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 & 6 Opening =	14.25	ft	
Diaphragm V =	430	plf, from page	
Revised Dia. V =	731.94	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.63		

Wall OOP, Sect. 11.3.3.2

Height =	31.50	ft	
Wall thickness =	1.5	ft	
Ratio =	21.00		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	10309.1	ft-lbs	
F_t =	190.91	psi	**> Ft = 45 psi NG**
DCR =	4.24		

2nd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft	
Diaphragm V =	258	plf, from page	
Revised Dia. V =	293.38	plf	**OK**
Dia. Capacity =	450	plf, from page	
DCR =	0.65		

Wall OOP, Sect. 11.3.3.2

Height =	15.00	ft	
Wall thickness =	1.5	ft	
Ratio =	10.00		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	2337.66	ft-lbs	
F_t =	43.29	psi	**< Ft = 45 psi OK**
DCR =	0.96		

1st Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	9.00	ft	
Diaphragm V =	185	plf, from page	
Revised Dia. V =	250.72	plf	**OK**
Dia. Capacity =	450	plf, from page	
DCR =	0.56		

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft	
Wall thickness =	1.5	ft	
Ratio =	17.00		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	6755.84	ft-lbs	
F_t =	125.11	psi	**> Ft = 45 psi NG**
DCR =	2.78		

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	13.33	ft	
Diaphragm V =	185	plf, from page	
Revised Dia. V =	302.00	plf	**OK**
Dia. Capacity =	450	plf, from page	
DCR =	0.67		

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft	
Wall thickness =	1.5	ft	
Ratio =	17.00		
X =	1.3	from Table 7-2	
F_p =	83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	6755.837	ft-lbs	
F_t =	125.11	psi	**> Ft = 45 psi NG**
DCR =	2.78		

Stair Enclosures - Check Height to Thickness Rations, Sect. A.7.10.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

3rd Floor Opng =	21.75	ft			
Wall thickness =	1	ft	Ratio:	21.8	**> 12, NG**
2nd Floor Opng =	31.50	ft			
Wall thickness =	1.5	ft	Ratio:	21.0	**> 12, NG**
First story height =	25.50	ft			
Wall thickness =	1.5	ft	Ratio:	17.0	**> 12, NG**

Tier 2 - 3rd Floor Opening - Sect. Sect. 11.3.3.2				**2rd Floor Opening**			**1st Floor Opening**		
Height =	21.75	ft		31.50	ft		25.50	ft	
Wall thickness =	1	ft		1.5	ft		1.5	ft	
Ratio =	21.75			21.00			17.00		
X =	1.3	from Table 7-2		1.3	from Table 7-2		1.3	from Table 7-2	
F_p =	55.41	psf, from 7.2.11.2 (7-13)		83.12	psf, from 7.2.11.2 (7-13)		83.12	psf, from 7.2.11.2 (7-13)	
OOP Moment =	3276.62	ft-lbs		10309.08	ft-lbs		6755.837	ft-lbs	
F_t =	136.53	psi	**> Ft = 45 psi NG**	190.91	psi	**> Ft = 45 psi NG**	125.11	psi	**> Ft = 45 psi NG**
DCR =	3.03			4.24			2.78		

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS AT ROOF **Shape Capacity = 0.49 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	30.21	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.49	
Tr/Tc =		
	0.49	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS LOWER **Shape Capacity = 0.68 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	41.34	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.68	
Tr/Tc =		
	0.68	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Concrete Shear Wall Design per ACI 318-05

Wall Mark 2FL GL 3

Wall Properties

t_{wall} =	8	in
h_{wall} =	15	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	7505	plf, dead laod	DL =	0.9	w_{du} =	6755	plf
w_{wall} =	1500	plf, wall weight	LL =	0.0	w_{wallu} =	1350	plf
w_s =	100	plf, snow load	SL =	0.0	w_{su} =	0	plf
w_l =	320	plf, live load	W =	1.6	w_{lu} =	0	plf
			E =	1.4			

Load Factors

Lateral Loading

C_s =	0.36		
V_{seis} =	327998	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	10.19318	0.78	2.35	51%	Controls
1	8	9.931818	0.81	2.27	49%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		20.125	ft	4.62	100%	

P_{du} =	69161	lb/pier
P_{wallu} =	13822	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	82983	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	97.85455	in	(ACI 11.10.4)	
V_u =	233893	lb/pier		
$\phi V_{n,max}$ =	415162	lb > 233893 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	69194	lb < 233893 NG	(ACI 21.7.2.1)	
ϕV_n =	354363	lb < 233893 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	138387	lb < 233893 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	196442	lb		
$\phi V_c/2$ =	73666	lb < 233893 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0343 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	24.46364	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0025	(EQ. 11-32)	$l_w/3$ =	40.77273	in
R'qd A_v =	0.163	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.163 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	1890963	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	82983	lb
$M_{u(max)}$ =	1890963	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	2124681	lb > 82983 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	97.85455	in	$0.10f'_cA_g$ =	489272.7	lb > 82983 OK
R_n =	329	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0057		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.47	in^2	$A_{s,min}$ =	2.77	in^2
Try	8	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0076	
A_s =	4.8	in^2	ρ_{act} =	0.0061	> 0.0035 OK
a =	8.47	in			
c =	9.97	in			
ε_t =	0.0265	> 0.004 OK (Tension Controls)			
Mn =	2246862	ft-lb			
ϕM_n =	2022176	ft-lb > 1890963 OK			

<u>Boundary Elements</u>

A_g =	979	in^2	
S_g =	19949	in^3	
P_u =	82983	lb	
M_u =	1890963	ft-lb	
σ =	1222	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1222 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0061	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 2FL GL 6

Wall Properties

t_{wall} =	8	in
h_{wall} =	15	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	7505	plf, dead laod	DL =	0.9	w_{du} =	6755	plf
w_{wall} =	1500	plf, wall weight	LL =	0.0	w_{wallu} =	1350	plf
w_s =	100	plf, snow load	SL =	0.0	w_{su} =	0	plf
w_l =	320	plf, live load	W =	1.6	w_{lu} =	0	plf
			E =	1.4			

Load Factors

Lateral Loading

C_s =	0.36		
V_{seis} =	327988	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	12.02273	0.67	2.91	63%	Controls
1	8	8.102273	0.99	1.70	37%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		20.125	ft	4.61	100%	

P_{du} =	85838	lb/pier
P_{wallu} =	17155	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	102994	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	115.4182	in	(ACI 11.10.4)	
V_u =	290286	lb/pier		
$\phi V_{n,max}$ =	489678	lb > 290286 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	81613	lb < 290286 NG	(ACI 21.7.2.1)	
ϕV_n =	417966	lb < 290286 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	163226	lb < 290286 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	231701	lb		
$\phi V_c/2$ =	86888	lb < 290286 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0370 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 16"o.c.

A =	0.31	in^2
s_2 =	16	in
A_v =	0.23	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	28.85455	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0024	< 0.0025 NG		**16" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0025	(EQ. 11-32)	$l_w/3$ =	48.09091	in
R'qd A_v =	0.160	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 16 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	16	in		**16" < 18" OK**	
A_v =	0.2325	> 0.16 OK			
$\rho_{l,actual}$ =	0.0024	< 0.0025 NG, need more steel			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	2345527	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	102994	lb
$M_{u(max)}$ =	2345527	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	2482979	lb > 102994 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	115.4182	in	$0.10f'_cA_g$ =	577090.9	lb > 102994 OK
R_n =	293	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0051		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.68	in^2	$A_{s,min}$ =	3.26	in^2
Try	8	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0068	
A_s =	4.8	in^2	ρ_{act} =	0.0052	> 0.0035 OK
a =	8.47	in			
c =	9.97	in			
ε_t =	0.0317	> 0.004 OK (Tension Controls)			
Mn =	2668389	ft-lb			
ϕM_n =	2401550	ft-lb > 2345527 OK			

<u>Boundary Elements</u>

A_g =	1154	in^2	
S_g =	27753	in^3	
P_u =	102994	lb	
M_u =	2345527	ft-lb	
σ =	1103	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1103 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0052	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL 3

Wall Properties

t_{wall} =	8	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1650	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1485	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	412917	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	424	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	17.5	0.46	4.54	100%	Controls
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		17.5	ft	4.54	100%	

P_{du} =	163633	lb/pier
P_{wallu} =	25988	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	189620	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	168	in	(ACI 11.10.4)	
V_u =	578678	lb/pier		
$\phi V_{n,max}$ =	712764	lb > 578678 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	118794	lb < 578678 NG	(ACI 21.7.2.1)	
ϕV_n =	608382	lb < 578678 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	237588	lb < 578678 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	190070	lb		
$\phi V_c/2$ =	71276	lb < 578678 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0577 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	42	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	70	in
R'qd A_v =	0.164	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.164 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	4666250	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	189620	lb
$M_{u(max)}$ =	4666250	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3635928	lb > 189620 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	168	in	$0.10f'_cA_g$ =	840000	lb > 189620 OK
R_n =	276	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0048		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	6.39	in^2	$A_{s,min}$ =	4.75	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0063	
A_s =	7.8	in^2	ρ_{act} =	0.0058	> 0.0035 OK
a =	13.76	in			
c =	16.19	in			
ε_t =	0.0281	> 0.004 OK (Tension Controls)			
Mn =	6283588	ft-lb			
ϕM_n =	5655229	ft-lb > 4666250 OK			

Boundary Elements

A_g =	1680	in^2	
S_g =	58800	in^3	
P_u =	189620	lb	
M_u =	4666250	ft-lb	
σ =	1065	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1065 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0058	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL 6

Wall Properties

t_{wall} =	8	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1650	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1485	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	412917	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	424	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	18.33	0.44	4.79	100%	Controls
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		18.33	ft	4.79	100%	

P_{du} =	171394	lb/pier
P_{wallu} =	27220	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	198614	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	175.968	in	(ACI 11.10.4)	
V_u =	578678	lb/pier		
$\phi V_{n,max}$ =	746569	lb > 578678 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	124428	lb < 578678 NG	(ACI 21.7.2.1)	
ϕV_n =	637236	lb < 578678 NG	(ACI EQ. 21-7)	
$2A_{cv}$sqrt(f_c') =	248856	lb < 578678 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	199085	lb		
$\phi V_c/2$ =	74657	lb < 578678 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0542 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	43.992	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	73.32	in
R'qd A_v =	0.164	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.164 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	4668222	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	198614	lb
$M_{u(max)}$ =	4668222	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3798475	lb > 198614 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	175.968	in	$0.10f'_cA_g$ =	879840	lb > 198614 OK
R_n =	251	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0043		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	6.08	in^2	$A_{s,min}$ =	4.98	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0058	
A_s =	7.8	in^2	ρ_{act} =	0.0055	> 0.0035 OK
a =	13.76	in			
c =	16.19	in			
ε_t =	0.0296	> 0.004 OK (Tension Controls)			
Mn =	6594340	ft-lb			
ϕM_n =	5934906	ft-lb > 4668222 OK			

Boundary Elements

A_g =	1760	in^2	
S_g =	64510	in^3	
P_u =	198614	lb	
M_u =	4668222	ft-lb	
σ =	981	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 981 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0055	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL B

Wall Properties

t_{wall} =	10	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	2062.5	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1856	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	673640	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	530	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	16.25	0.49	5.22	32%	Controls
1	8	15.33	0.52	4.88	30%	
1	8	10.67	0.75	3.12	19%	
1	8	10.25	0.78	2.96	18%	
total wall length =		52.5	ft	16.18	100%	

P_{du} =	158401	lb/pier
P_{wallu} =	31446	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	189847	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	156	in	(ACI 11.10.4)	
V_u =	304554	lb/pier		
$\phi V_{n,max}$ =	827315	lb > 304554 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	137886	lb < 304554 NG	(ACI 21.7.2.1)	
ϕV_n =	706157	lb < 304554 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	275772	lb < 304554 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	394407	lb		
$\phi V_c/2$ =	147903	lb < 304554 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0198 in^2/in/ft (EQ. 11-31)

Try #5 at 12"o.c.

A =	0.31	in^2
s_2 =	12	in
A_v =	0.31	in^2/ft
A_g =	120	in^2

Max spacing

$l_w/5$ =	39	in
$3t_{wall}$ =	30	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**12" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	65	in
R'qd A_v =	0.205	in^2/ft	$3t_{wall}$ =	30	in
Try #5 at 12 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	12	in		**12" < 18" OK**	
A_v =	0.31	> 0.205 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	30	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	120	in^2		**16" < 18" OK**	
ρ_t =	0.0019	< 0.002 NG, more steel required			

Vertical Steel - (ACI 14.3.2)

Try #5 at 15 o.c.		
A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	120	in^2
ρ_t =	0.0021	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2482780	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	189847	lb
$M_{u(max)}$ =	2482780	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	4122504	lb > 189847 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	156	in	$0.10f'_cA_g$ =	975000	lb > 189847 OK
R_n =	136	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0023		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	3.60	in^2	$A_{s,min}$ =	5.52	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0031	
A_s =	5.4	in^2	ρ_{act} =	0.0035	> 0.0031 OK
a =	7.62	in			
c =	8.97	in			
ε_t =	0.0492	> 0.004 OK (Tension Controls)			
Mn =	4109082	ft-lb			
ϕM_n =	3698174	ft-lb > 2482780 OK			

Boundary Elements

A_g =	1950	in^2	
S_g =	63375	in^3	
P_u =	189847	lb	
M_u =	2482780	ft-lb	
σ =	567	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 567 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0035	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL C

Wall Properties

t_{wall} =	10	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	2062.5	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1856	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	673640	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	530	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	13.75	0.58	4.29	37%	Controls
1	8	12.67	0.63	3.88	34%	
1	8	7.67	1.04	1.95	17%	
1	8	6.33	1.26	1.43	12%	
total wall length =		40.42	ft	11.56	100%	

P_{du} =	140238	lb/pier
P_{wallu} =	27840	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	168079	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	132	in	(ACI 11.10.4)	
V_u =	350216	lb/pier		
$\phi V_{n,max}$ =	700036	lb > 350216 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	116673	lb < 350216 NG	(ACI 21.7.2.1)	
ϕV_n =	597518	lb < 350216 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	233345	lb < 350216 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	333729	lb		
$\phi V_c/2$ =	125149	lb < 350216 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0354 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 12"o.c.

A =	0.31	in^2
s_2 =	12	in
A_v =	0.31	in^2/ft
A_g =	120	in^2

Max spacing

$l_w/5$ =	33	in
$3t_{wall}$ =	30	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**12" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	55	in
R'qd A_v =	0.204	in^2/ft	$3t_{wall}$ =	30	in
Try #5 at 12 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	12	in		**12" < 18" OK**	
A_v =	0.31	> 0.204 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	30	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	120	in^2		**16" < 18" OK**	
ρ_t =	0.0019	< 0.002 NG, more steel required			

Vertical Steel - (ACI 14.3.2)

Try #5 at 15 o.c.		
A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	120	in^2
ρ_t =	0.0021	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2840364	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	168079	lb
$M_{u(max)}$ =	2840364	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3510504	lb > 168079 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	132	in	$0.10f'_cA_g$ =	825000	lb > 168079 OK
R_n =	217	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0037		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.91	in^2	$A_{s,min}$ =	4.67	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0050	
A_s =	5.4	in^2	ρ_{act} =	0.0041	> 0.0035 OK
a =	7.62	in			
c =	8.97	in			
ε_t =	0.0412	> 0.004 OK (Tension Controls)			
Mn =	3461082	ft-lb			
ϕM_n =	3114974	ft-lb > 2840364 OK			

Boundary Elements

A_g =	1650	in^2	
S_g =	45375	in^3	
P_u =	168079	lb	
M_u =	2840364	ft-lb	
σ =	853	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 853 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0041	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL 3

Wall Properties

t_{wall} =	12	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1500	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1350	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	470910	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	9.5	0.84	3.20	43%	Controls
1	8	8.25	0.97	2.62	35%	
1	8	6	1.33	1.57	21%	
0	0	0	0.00	0.00	0%	
total wall length =		23.75	ft	7.39	100%	

P_{du} =	96228	lb/pier
P_{wallu} =	13893	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	110121	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	91.2	in	(ACI 11.10.4)	
V_u =	285908	lb/pier		
$\phi V_{n,max}$ =	580393	lb > 285908 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	96732	lb < 285908 NG	(ACI 21.7.2.1)	
ϕV_n =	495397	lb < 285908 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	193464	lb < 285908 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	273139	lb		
$\phi V_c/2$ =	102427	lb < 285908 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0414 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	144	in^2

Max spacing

$l_w/5$ =	22.8	in
$3t_{wall}$ =	36	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0017	< 0.0025 NG		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0019	(EQ. 11-32)	$l_w/3$ =	38	in
R'qd A_v =	0.240	in^2/ft	$3t_{wall}$ =	36	in
Try #5 at 9 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	9	in		**9" < 18"**	**OK**
A_v =	0.413333	> 0.24 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 9 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	36	in
s_2 =	9	in	18 =	18	in
A_v =	0.41	in^2/ft	s_{max} =	18	in
A_g =	144	in^2		**9" < 18"**	**OK**
ρ_t =	0.0029	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	144	in^2
ρ_t =	0.0014	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	2305909	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	110121	lb
$M_{u(max)}$ =	2305909	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3031560	lb > 110121 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	91.2	in	$0.10f'_cA_g$ =	684000	lb > 110121 OK
R_n =	308	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0053		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	5.84	in^2	$A_{s,min}$ =	3.87	in^2
Try	15	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0071	
A_s =	9	in^2	ρ_{act} =	0.0082	> 0.0035 OK
a =	10.59	in			
c =	12.46	in			
ε_t =	0.0190	> 0.004 OK (Tension Controls)			
Mn =	3865765	ft-lb			
ϕM_n =	3479188	ft-lb > 2305909 OK			

<u>Boundary Elements</u>

A_g =	1368	in^2	
S_g =	25992	in^3	
P_u =	110121	lb	
M_u =	2305909	ft-lb	
σ =	1145	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1145 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0082	> 0.0067 Provide Boundary Ties	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL 6

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1875	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	470910	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	9	0.89	3.71	53%	Controls
1	8	6.75	1.19	2.39	34%	
1	8	4	2.00	0.89	13%	
0	0	0	0.00	0.00	0%	
total wall length =		19.75	ft	7.00	100%	

P_{du} =	97917	lb/pier
P_{wallu} =	17671	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	115588	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	86.4	in	(ACI 11.10.4)	
V_u =	349919	lb/pier		
$\phi V_{n,max}$ =	687308	lb > 349919 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	114551	lb < 349919 NG	(ACI 21.7.2.1)	
ϕV_n =	586654	lb < 349919 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	229103	lb < 349919 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	319246	lb		
$\phi V_c/2$ =	119717	lb < 349919 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0546 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	21.6	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0014	< 0.0025 NG	**15" < 18" OK**		

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0017	(EQ. 11-32)	$l_w/3$ =	36	in
R'qd A_v =	0.300	in^2/ft	$3t_{wall}$ =	45	in
Try #5 at 8 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	8	in	**8" < 18"**	**OK**	
A_v =	0.465	> 0.3 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 9 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	45	in
s_2 =	9	in	18 =	18	in
A_v =	0.41	in^2/ft	s_{max} =	18	in
A_g =	180	in^2	**9" < 18"**	**OK**	
ρ_t =	0.0023	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 9 o.c.		
A =	0.31	in^2
s_2 =	9	in
A_v =	0.41	in^2/ft
A_g =	180	in^2
ρ_t =	0.0023	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2820793	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	115588	lb
$M_{u(max)}$ =	2820793	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3513528	lb > 115588 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	86.4	in	$0.10f'_cA_g$ =	810000	lb > 115588 OK
R_n =	336	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0058		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	7.57	in^2	$A_{s,min}$ =	4.58	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0078	
A_s =	7.8	in^2	ρ_{act} =	0.0060	> 0.0035 OK
a =	7.34	in			
c =	8.64	in			
ε_t =	0.0270	> 0.004 OK (Tension Controls)			
Mn =	3226447	ft-lb			
ϕM_n =	2903802	ft-lb > 2820793 OK			

Boundary Elements

A_g =	1620	in^2	
S_g =	29160	in^3	
P_u =	115588	lb	
M_u =	2820793	ft-lb	
σ =	1232	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1232 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0060	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL B

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1875	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	808243	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	28	0.29	14.20	58%	Controls
1	8	13.5	0.59	6.29	26%	
1	8	9.75	0.82	4.15	17%	
0	0	0	0.00	0.00	0%	
total wall length =		51.25	ft	24.64	100%	

P_{du} =	276123	lb/pier
P_{wallu} =	49833	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	325955	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	268.8	in	(ACI 11.10.4)	
V_u =	652387	lb/pier		
$\phi V_{n,max}$ =	2138291	lb > 652387 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	356382	lb < 652387 NG	(ACI 21.7.2.1)	
ϕV_n =	1825145	lb > 652387 OK	(ACI EQ. 21-7)	
$2A_{cv}$sqrt(f_c') =	712764	lb > 652387 OK	(ACI 21.7.2.2)	**Min. One Curtain of ReStl**
V_c =	570211	lb		
$\phi V_c/2$ =	213829	lb < 652387 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0186 $in^2/in/ft$ (EQ. 11-31)

Try #6 at 10"o.c.

A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	67.2	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0029	> 0.0025 OK		**10" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0030	(EQ. 11-32)	$l_w/3$ =	112	in
R'qd A_v =	0.356	in^2/ft	$3t_{wall}$ =	45	in
Try #6 at 10 o.c.			18 =	18	in
A =	0.44	in^2	s_{max} =	18	in
s =	10	in		**10" < 18" OK**	
A_v =	0.528	> 0.356 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #6 at 10 o.c.			**Max spacing**		
A =	0.44	in^2	$3t_{wall}$ =	45	in
s_2 =	10	in	18 =	18	in
A_v =	0.53	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**10" < 18" OK**	
ρ_t =	0.0029	> 0.0025 OK			

Vertical Steel - (ACI 14.3.2)

Try #6 at 12 o.c.		
A =	0.44	in^2
s_2 =	12	in
A_v =	0.44	in^2/ft
A_g =	180	in^2
ρ_t =	0.0024	> 0.0015 OK

BENDING - ACI Chapter 10

M_u =	5291587	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	325955	lb
$M_{u(max)}$ =	5291587	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	10426104	lb > 325955 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	268.8	in	$0.10f'_cA_g$ =	2520000	lb > 325955 OK
R_n =	65	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0011		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.41	in^2	$A_{s,min}$ =	14.26	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0015	
A_s =	5.4	in^2	ρ_{act} =	0.0013	< 0.0015 NG, more steel required
a =	5.08	in			
c =	5.98	in			
ε_t =	0.1319	> 0.004 OK (Tension Controls)			
Mn =	7188988	ft-lb			
ϕM_n =	6470089	ft-lb > 5291587 OK			

Boundary Elements

A_g =	5040	in^2	
S_g =	282240	in^3	
P_u =	325955	lb	
M_u =	5291587	ft-lb	
σ =	290	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 290 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0013	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL C

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1875	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	808243	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	22.5	0.36	11.24	50%	Controls
1	8	22.25	0.36	11.11	50%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		44.75	ft	22.35	100%	

P_{du} =	210480	lb/pier
P_{wallu} =	37986	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	248466	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	216	in	(ACI 11.10.4)	
V_u =	569529	lb/pier		
$\phi V_{n,max}$ =	1718269	lb > 569529 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	286378	lb < 569529 NG	(ACI 21.7.2.1)	
ϕV_n =	1466635	lb > 569529 OK	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	572756	lb > 569529 OK	(ACI 21.7.2.2)	**Min. One Curtain of ReStl**
V_c =	458205	lb		
$\phi V_c/2$ =	171827	lb < 569529 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0232 in^2/in/ft (EQ. 11-31)

Try #6 at 10"o.c.

A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	54	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0029	> 0.0025 OK		**10" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0029	(EQ. 11-32)	$l_w/3$ =	90	in
R'qd A_v =	0.353	in^2/ft	$3t_{wall}$ =	45	in
Try #6 at 10 o.c.			18 =	18	in
A =	0.44	in^2	s_{max} =	18	in
s =	10	in		**10" < 18" OK**	
A_v =	0.528	> 0.353 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #6 at 10 o.c.			**Max spacing**		
A =	0.44	in^2	$3t_{wall}$ =	45	in
s_2 =	10	in	18 =	18	in
A_v =	0.53	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**10" < 18" OK**	
ρ_t =	0.0029	> 0.0025 OK			

Vertical Steel - (ACI 14.3.2)

Try #6 at 10 o.c.		
A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2
ρ_t =	0.0029	> 0.0015 OK

BENDING - ACI Chapter 10

M_u =	4614264	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	248466	lb
$M_{u(max)}$ =	4614264	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	8406504	lb > 248466 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	216	in	$0.10f'_cA_g$ =	2025000	lb > 248466 OK
R_n =	88	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0015		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.80	in^2	$A_{s,min}$ =	11.46	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0020	
A_s =	5.4	in^2	ρ_{act} =	0.0017	< 0.002 NG, more steel required
a =	5.08	in			
c =	5.98	in			
ε_t =	0.1054	> 0.004 OK (Tension Controls)			
Mn =	5763388	ft-lb			
ϕM_n =	5187049	ft-lb > 4614264 OK			

Boundary Elements

A_g =	4050	in^2	
S_g =	182250	in^3	
P_u =	248466	lb	
M_u =	4614264	ft-lb	
σ =	365	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 365 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0017	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Current Date: 2/25/2016 1:17 AM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 3.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	41.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 3

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00
3	15.00	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	9.50	0.00	3.00	7.00
Lower left	17.50	10.00	3.00	10.00
Lower left	10.00	26.50	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
3	EQ	Horizontal	327998.00	0.00	0.00
2	EQ	Horizontal	84930.00	0.00	0.00
1	EQ	Horizontal	57990.00	0.00	0.00

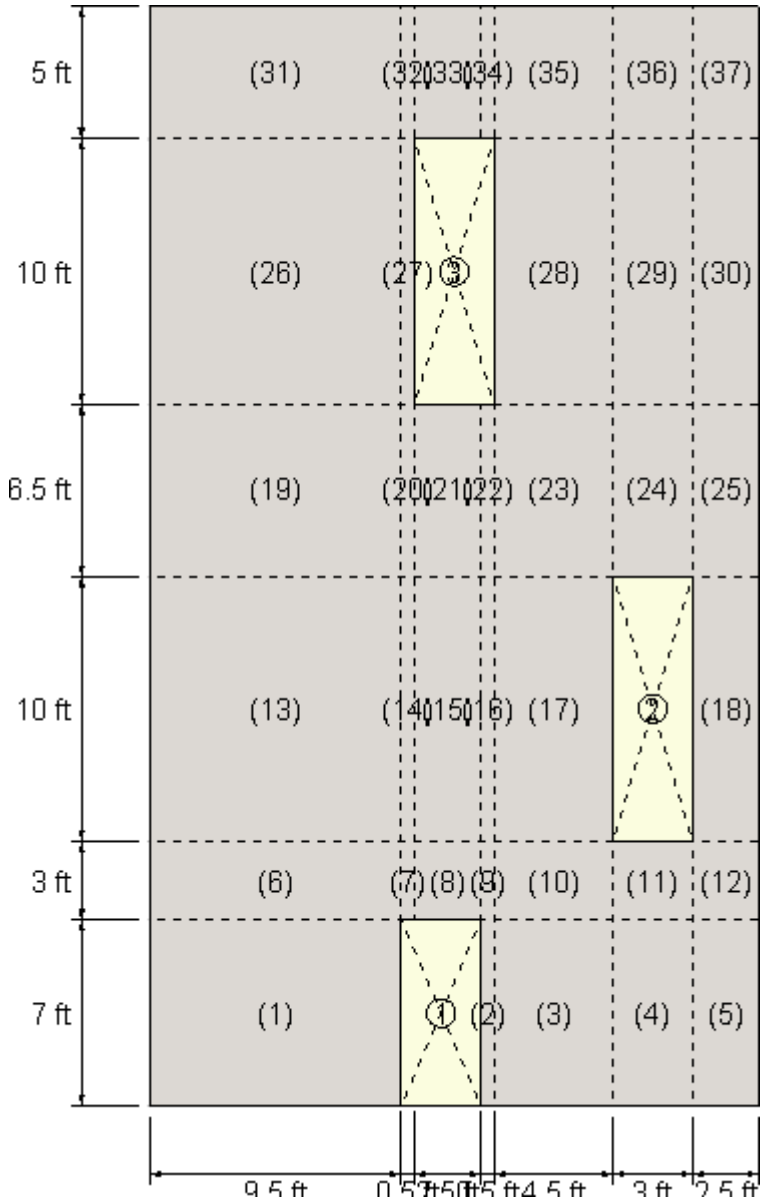
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
3	DL	Vertical	3360.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	0.50	7.00
3	13.00	0.00	4.50	7.00
4	17.50	0.00	3.00	7.00
5	20.50	0.00	2.50	7.00
6	0.00	7.00	9.50	3.00
7	9.50	7.00	0.50	3.00
8	10.00	7.00	2.50	3.00
9	12.50	7.00	0.50	3.00
10	13.00	7.00	4.50	3.00
11	17.50	7.00	3.00	3.00
12	20.50	7.00	2.50	3.00
13	0.00	10.00	9.50	10.00
14	9.50	10.00	0.50	10.00
15	10.00	10.00	2.50	10.00
16	12.50	10.00	0.50	10.00

17	13.00	10.00	4.50	10.00
18	20.50	10.00	2.50	10.00
19	0.00	20.00	9.50	6.50
20	9.50	20.00	0.50	6.50
21	10.00	20.00	2.50	6.50
22	12.50	20.00	0.50	6.50
23	13.00	20.00	4.50	6.50
24	17.50	20.00	3.00	6.50
25	20.50	20.00	2.50	6.50
26	0.00	26.50	9.50	10.00
27	9.50	26.50	0.50	10.00
28	13.00	26.50	4.50	10.00
29	17.50	26.50	3.00	10.00
30	20.50	26.50	2.50	10.00
31	0.00	36.50	9.50	5.00
32	9.50	36.50	0.50	5.00
33	10.00	36.50	2.50	5.00
34	12.50	36.50	0.50	5.00
35	13.00	36.50	4.50	5.00
36	17.50	36.50	3.00	5.00
37	20.50	36.50	2.50	5.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	12-#9	10.00	43.69
2	1-#6	14.00	23.24
3	4-#6	14.00	23.24
4	3-#6	14.00	23.24
5	2-#6	14.00	23.24
6	12-#9	10.00	43.69
7	1-#6	14.00	23.24
8	2-#6	14.00	23.24
9	1-#6	14.00	23.24
10	4-#6	14.00	23.24
11	3-#6	14.00	23.24
12	2-#6	14.00	23.24
13	7-#9	18.00	43.69
14	1-#8	16.00	38.73
15	2-#8	16.00	38.73
16	1-#8	16.00	38.73
17	4-#8	16.00	38.73
18	2-#5	18.00	19.36
19	7-#9	18.00	43.69
20	1-#8	16.00	38.73
21	2-#8	16.00	38.73
22	1-#8	16.00	38.73
23	4-#8	16.00	38.73
24	2-#5	18.00	19.36
25	2-#5	18.00	19.36
26	8-#6	14.00	23.24
27	1-#6	14.00	23.24
28	4-#5	14.00	19.36
29	3-#5	14.00	19.36
30	2-#5	14.00	19.36
31	8-#6	14.00	23.24
32	1-#6	14.00	23.24
33	2-#5	14.00	19.36

34	1-#5	14.00	19.36
35	4-#5	14.00	19.36
36	3-#5	14.00	19.36
37	2-#5	14.00	19.36

Intermediate results for axial-bending

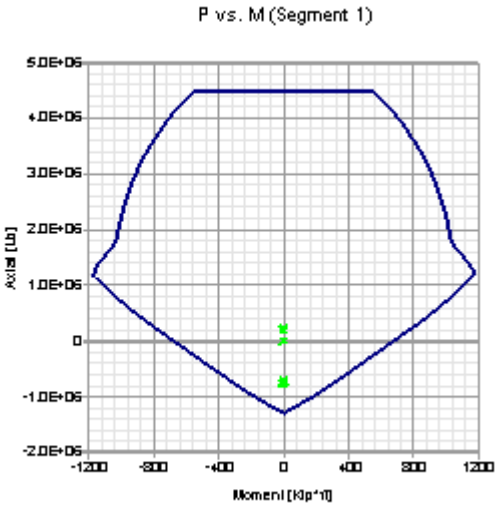
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	2.19	13.06
2	DC1 (Top)	1.82	13.25
3	DC1 (Top)	1.29	13.25
4	DC1 (Top)	1.27	13.25
5	DC1 (Top)	1.21	13.25
6	DC1 (Top)	2.17	12.94
7	DC1 (Top)	1.71	13.13
8	DC1 (Top)	1.13	13.13
9	DC1 (Top)	1.73	13.13
10	DC1 (Top)	1.34	13.13
11	DC1 (Top)	1.18	13.13
12	DC1 (Top)	1.31	13.13
13	DC1 (Top)	1.58	8.06
14	DC1 (Top)	2.34	8.13
15	DC1 (Top)	1.46	8.13
16	DC1 (Top)	2.36	8.13
17	DC1 (Top)	1.56	8.13
18	DC1 (Top)	1.01	8.31
19	DC1 (Top)	1.58	8.06
20	DC1 (Top)	2.35	8.13
21	DC1 (Top)	1.38	8.13
22	DC1 (Top)	2.28	8.13
23	DC1 (Top)	1.52	8.13
24	DC1 (Top)	0.79	8.31
25	DC1 (Top)	0.90	8.31
26	DC1 (Top)	1.05	6.25
27	DC1 (Top)	1.55	6.25
28	DC1 (Top)	0.88	6.31
29	DC1 (Top)	0.93	6.31
30	DC1 (Top)	0.77	6.31
31	DC1 (Top)	1.03	6.25
32	DC1 (Top)	1.52	6.25
33	DC1 (Top)	0.76	6.31
34	DC1 (Top)	1.29	6.31
35	DC1 (Top)	0.84	6.31
36	DC1 (Top)	0.91	6.31
37	DC1 (Top)	0.76	6.31

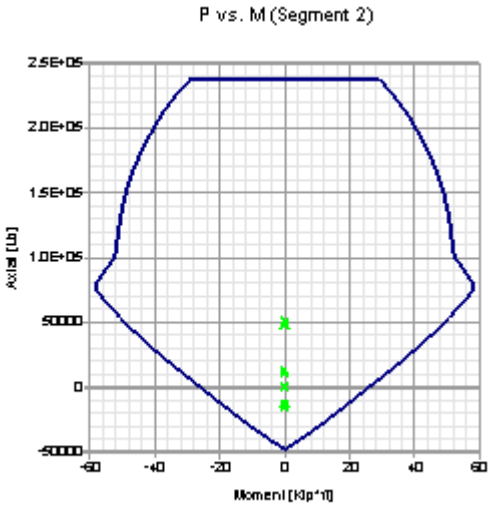
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	196336.50	0.00	771.10	0.00
2	DC1 (Top)	13511.45	0.00	32.58	0.00
3	DC1 (Top)	96222.32	0.00	162.32	0.00
4	DC1 (Top)	47900.73	0.00	107.81	0.00
5	DC1 (Top)	49754.27	0.00	82.74	0.00
6	DC1 (Top)	173847.30	0.00	754.12	0.00
7	DC1 (Top)	7411.47	0.00	29.35	0.00
8	DC1 (Top)	33625.52	0.00	74.27	0.00

9	DC1 (Top)	8628.80	0.00	29.95	0.00
10	DC1 (Top)	97167.51	0.00	163.05	0.00
11	DC1 (Top)	23454.13	0.00	95.06	0.00
12	DC1 (Top)	58513.21	0.00	87.55	0.00
13	DC1 (Top)	109443.00	0.00	290.00	0.00
14	DC1 (Top)	5701.29	0.00	27.72	0.00
15	DC1 (Top)	29099.90	0.00	68.65	0.00
16	DC1 (Top)	6122.53	0.00	27.83	0.00
17	DC1 (Top)	65914.50	0.00	137.06	0.00
18	DC1 (Top)	43014.94	0.00	41.64	0.00
19	DC1 (Top)	107326.60	0.00	289.39	0.00
20	DC1 (Top)	5980.71	0.00	27.80	0.00
21	DC1 (Top)	11632.51	0.00	63.55	0.00
22	DC1 (Top)	4394.59	0.00	27.35	0.00
23	DC1 (Top)	53636.53	0.00	133.52	0.00
24	DC1 (Top)	31272.20	0.00	38.44	0.00
25	DC1 (Top)	27831.40	0.00	36.58	0.00
26	DC1 (Top)	56444.18	0.00	128.32	0.00
27	DC1 (Top)	3538.71	0.00	12.68	0.00
28	DC1 (Top)	29902.82	0.00	50.09	0.00
29	DC1 (Top)	16872.29	0.00	35.42	0.00
30	DC1 (Top)	12762.66	0.00	24.61	0.00
31	DC1 (Top)	45639.82	0.00	125.63	0.00
32	DC1 (Top)	2374.27	0.00	12.44	0.00
33	DC1 (Top)	11859.14	0.00	24.35	0.00
34	DC1 (Top)	2379.63	0.00	9.58	0.00
35	DC1 (Top)	21727.12	0.00	47.75	0.00
36	DC1 (Top)	14372.06	0.00	34.80	0.00
37	DC1 (Top)	11926.93	0.00	24.37	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	216777.00	4471272.00	0.05
2	D2 (Top)	50682.07	236346.20	0.21
3	D2 (Top)	401981.50	2138785.00	0.19
4	D2 (Bottom)	371917.10	1425079.00	0.26
5	D2 (Bottom)	501030.40	1188732.00	0.42
6	D1 (Bottom)	196334.90	4471272.00	0.04
7	DC1 (Bottom)	7543.46	236346.20	0.03
8	D2 (Top)	51427.67	1188732.00	0.04
9	D2 (Bottom)	50681.58	236346.20	0.21
10	D2 (Top)	478757.80	2138785.00	0.22
11	D2 (Bottom)	275152.30	1425079.00	0.19
12	D2 (Top)	374595.00	1188732.00	0.32
13	D1 (Bottom)	128535.00	2986152.00	0.04
14	D2 (Top)	6169.06	154929.80	0.04
15	D2 (Top)	45936.00	787219.70	0.06
16	D2 (Bottom)	25335.92	154929.80	0.16
17	D2 (Bottom)	474710.40	1415319.00	0.34
18	D2 (Bottom)	373166.80	792311.50	0.47
19	DC1 (Bottom)	109443.00	2986152.00	0.04
20	D2 (Top)	42576.12	154929.80	0.27
21	D2 (Top)	52188.64	787219.70	0.07
22	D2 (Bottom)	11918.36	154929.80	0.08
23	D2 (Bottom)	151741.30	1415319.00	0.11
24	D2 (Top)	128289.80	951431.50	0.13
25	D2 (Bottom)	338107.70	792311.50	0.43
26	DC1 (Bottom)	69366.26	2399954.00	0.03
27	D2 (Bottom)	55943.86	124962.20	0.45
28	D2 (Top)	142396.30	1139087.00	0.13
29	D2 (Bottom)	117760.70	758843.30	0.16
30	D2 (Bottom)	187223.80	633191.50	0.30
31	D2 (Top)	127969.20	2399954.00	0.05
32	DC1 (Bottom)	3515.11	124962.20	0.03
33	DC1 (Top)	11859.14	633191.50	0.02
34	D2 (Bottom)	42286.45	125651.80	0.34
35	D2 (Bottom)	142295.00	1139087.00	0.12
36	D2 (Bottom)	25685.95	758843.30	0.03
37	D2 (Bottom)	13870.79	633191.50	0.02

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	771236.50	1296000.00	0.60
2	D3 (Bottom)	14811.33	47520.00	0.31
3	DC1 (Top)	0.00	190080.00	0.00
4	DC1 (Top)	0.00	142560.00	0.00
5	DC1 (Top)	0.00	95040.00	0.00
6	D3 (Bottom)	754700.80	1296000.00	0.58
7	D3 (Bottom)	21837.08	47520.00	0.46
8	DC1 (Bottom)	1753.25	95040.00	0.02
9	DC1 (Top)	0.00	47520.00	0.00
10	DC1 (Top)	0.00	190080.00	0.00
11	DC1 (Top)	0.00	142560.00	0.00
12	DC1 (Top)	0.00	95040.00	0.00
13	D3 (Bottom)	720928.20	756000.00	0.95

14	D3 (Bottom)	7456.77	85320.00	0.09
15	DC1 (Top)	0.00	170640.00	0.00
16	DC1 (Top)	0.00	85320.00	0.00
17	DC1 (Top)	0.00	341280.00	0.00
18	DC1 (Top)	0.00	66960.00	0.00
19	D3 (Bottom)	380051.20	756000.00	0.50
20	DC1 (Top)	0.00	85320.00	0.00
21	DC1 (Top)	0.00	170640.00	0.00
22	D3 (Top)	33526.12	85320.00	0.39
23	D3 (Top)	38289.84	341280.00	0.11
24	DC1 (Top)	0.00	66960.00	0.00
25	DC1 (Top)	0.00	66960.00	0.00
26	D3 (Bottom)	214493.40	380160.00	0.56
27	D3 (Top)	31360.71	47520.00	0.66
28	D3 (Bottom)	57351.15	133920.00	0.43
29	DC1 (Top)	0.00	100440.00	0.00
30	DC1 (Top)	0.00	66960.00	0.00
31	D3 (Bottom)	68969.52	380160.00	0.18
32	D3 (Bottom)	31136.23	47520.00	0.66
33	D2 (Bottom)	35111.83	66960.00	0.52
34	DC1 (Top)	0.00	33480.00	0.00
35	DC1 (Top)	0.00	133920.00	0.00
36	DC1 (Top)	0.00	100440.00	0.00
37	DC1 (Top)	0.00	66960.00	0.00

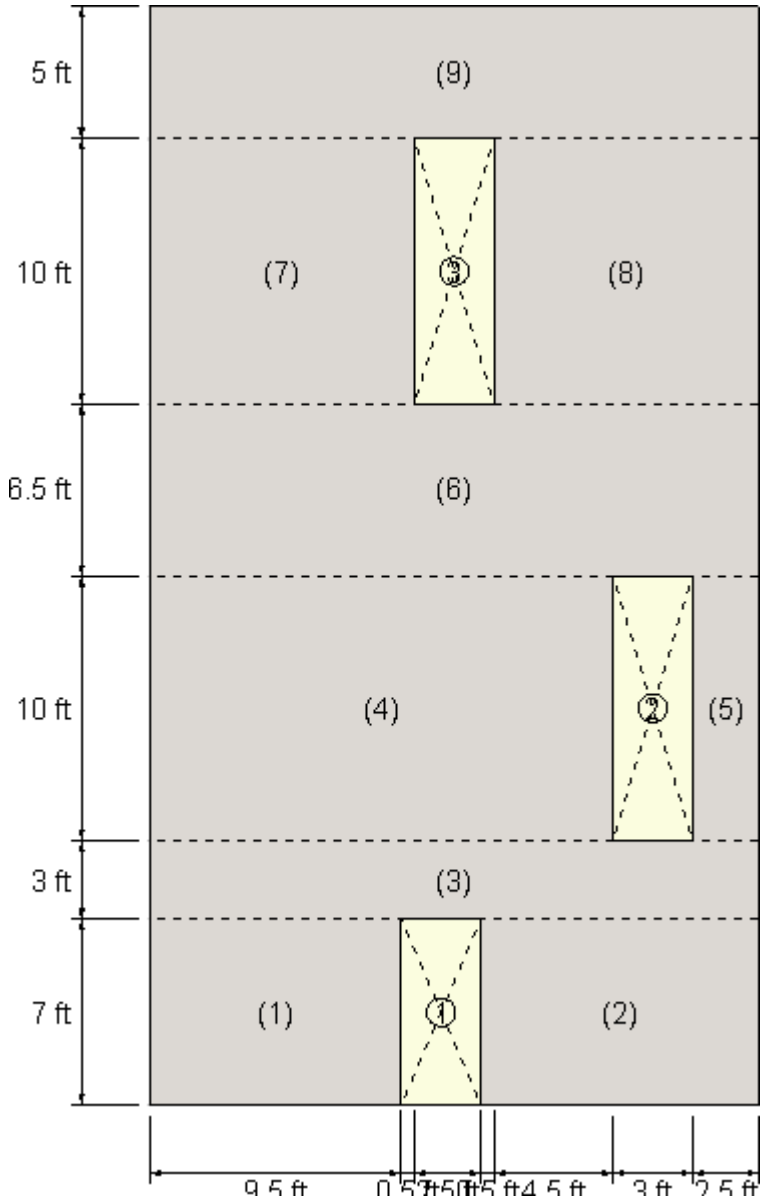
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	173000.820	0.00
2	DC1 (Top)	0.000	9237.065	0.00
3	DC1 (Top)	0.000	83133.588	0.00
4	DC1 (Top)	0.000	55422.392	0.00
5	DC1 (Top)	0.000	46185.326	0.00
6	DC1 (Top)	0.000	171345.120	0.00
7	DC1 (Top)	0.000	9149.923	0.00
8	DC1 (Top)	0.000	45749.616	0.00
9	DC1 (Top)	0.000	9149.923	0.00
10	DC1 (Top)	0.000	82349.308	0.00
11	DC1 (Top)	0.000	54899.539	0.00
12	DC1 (Top)	0.000	45749.616	0.00
13	DC1 (Top)	0.000	106772.810	0.00
14	DC1 (Top)	0.000	5664.238	0.00
15	DC1 (Top)	0.000	28321.191	0.00
16	DC1 (Top)	0.000	5664.238	0.00
17	DC1 (Top)	0.000	50978.143	0.00
18	DC1 (Top)	0.000	28974.757	0.00
19	DC1 (Top)	0.000	106772.810	0.00
20	DC1 (Top)	0.000	5664.238	0.00
21	DC1 (Top)	0.000	28321.191	0.00
22	DC1 (Top)	0.000	5664.238	0.00
23	DC1 (Top)	0.000	50978.143	0.00
24	DC1 (Top)	0.000	34769.708	0.00
25	DC1 (Top)	0.000	28974.757	0.00
26	DC1 (Top)	0.000	82785.019	0.00
27	DC1 (Top)	0.000	4357.106	0.00
28	DC1 (Top)	0.000	39606.096	0.00
29	DC1 (Top)	0.000	26404.064	0.00

30	DC1 (Top)	0.000	22003.387	0.00
31	DC1 (Top)	0.000	82785.019	0.00
32	DC1 (Top)	0.000	4357.106	0.00
33	DC1 (Top)	0.000	22003.387	0.00
34	DC1 (Top)	0.000	4400.677	0.00
35	DC1 (Top)	0.000	39606.096	0.00
36	DC1 (Top)	0.000	26404.064	0.00
37	DC1 (Top)	0.000	22003.387	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	10.50	7.00
3	0.00	7.00	23.00	3.00
4	0.00	10.00	17.50	10.00
5	20.50	10.00	2.50	10.00
6	0.00	20.00	23.00	6.50
7	0.00	26.50	10.00	10.00
8	13.00	26.50	10.00	10.00
9	0.00	36.50	23.00	5.00

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	12-#9	10.00	43.69	6-#5	16.00	25.17
2	1-#6	14.00	23.24	6-#5	16.00	25.17
	4-#6	14.00	23.24	6-#5	16.00	25.17
	3-#6	14.00	23.24	6-#5	16.00	25.17
	2-#6	14.00	23.24	6-#5	16.00	25.17
3	12-#9	10.00	43.69	2-#6	18.00	30.21
	1-#6	14.00	43.69	2-#6	18.00	30.21
	2-#6	14.00	43.69	2-#6	18.00	30.21
	1-#6	14.00	43.69	2-#6	18.00	30.21
	4-#6	14.00	43.69	2-#6	18.00	30.21
	3-#6	14.00	43.69	2-#6	18.00	30.21
	2-#6	14.00	43.69	2-#6	18.00	30.21
4	7-#9	18.00	43.69	7-#5	18.00	25.17
	1-#8	16.00	43.69	7-#5	18.00	25.17
	2-#8	16.00	43.69	7-#5	18.00	25.17
	1-#8	16.00	43.69	7-#5	18.00	25.17
	4-#8	16.00	43.69	7-#5	18.00	25.17
5	2-#5	18.00	19.36	7-#5	18.00	25.17
6	7-#9	18.00	43.69	5-#5	18.00	25.17
	1-#8	16.00	43.69	5-#5	18.00	25.17
	2-#8	16.00	43.69	5-#5	18.00	25.17
	1-#8	16.00	43.69	5-#5	18.00	25.17
	4-#8	16.00	43.69	5-#5	18.00	25.17
	2-#5	18.00	43.69	5-#5	18.00	25.17
	2-#5	18.00	43.69	5-#5	18.00	25.17
7	8-#6	14.00	23.24	7-#5	18.00	25.17
	1-#6	14.00	23.24	7-#5	18.00	25.17
8	4-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
	2-#5	14.00	19.36	7-#5	18.00	25.17
9	8-#6	14.00	23.24	4-#5	18.00	25.17
	1-#6	14.00	23.24	4-#5	18.00	25.17
	2-#5	14.00	23.24	4-#5	18.00	25.17
	1-#5	14.00	23.24	4-#5	18.00	25.17
	4-#5	14.00	23.24	4-#5	18.00	25.17
	3-#5	14.00	23.24	4-#5	18.00	25.17
	2-#5	14.00	23.24	4-#5	18.00	25.17

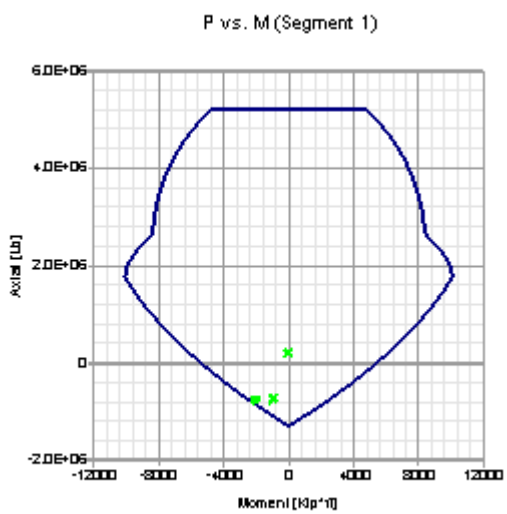
Intermediate results for axial-bending

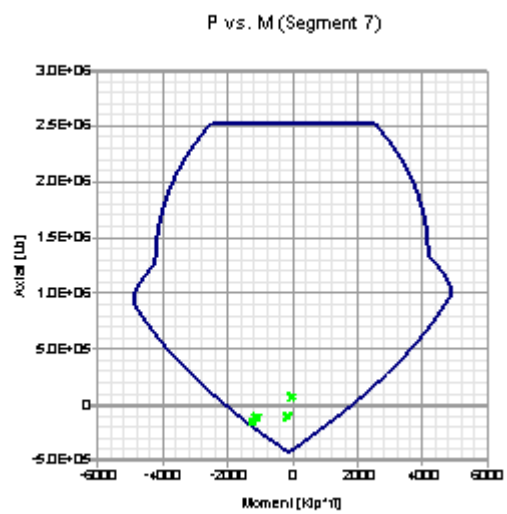
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	6.22	91.20
2	D3 (Bottom)	26.86	100.80
3	D3 (Bottom)	32.83	220.80
4	D3 (Bottom)	24.95	168.00
5	D2 (Bottom)	14.17	24.00
6	D3 (Bottom)	36.06	220.80
7	D3 (Bottom)	8.08	96.00
8	D3 (Bottom)	17.48	96.00
9	D3 (Bottom)	28.13	220.80

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	-771276.30	-2130.29	2403.00	0.89
2	D3 (Bottom)	1059704.00	-2188.94	6568.01	0.33
3	D3 (Bottom)	286510.00	-13162.87	16566.86	0.79
4	D3 (Bottom)	-209026.70	-7147.00	9757.08	0.73
5	D2 (Bottom)	373173.90	-44.07	354.78	0.12
6	D3 (Bottom)	177526.40	-7641.68	15236.73	0.50
7	D3 (Bottom)	-159506.10	-1241.97	1398.19	0.89
8	D3 (Bottom)	239555.20	-1167.44	2374.35	0.49
9	D3 (Bottom)	74015.34	-1604.91	8828.22	0.18

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	216786.30	5220072.00	0.04
2	D2 (Max)	1117333.00	4988942.00	0.22
3	D1 (Max)	415625.30	1.088529E07	0.04
4	D1 (Bottom)	245479.60	6329719.00	0.04
5	D2 (Max)	375149.00	792311.50	0.47
6	D1 (Bottom)	266300.50	8150838.00	0.03
7	DC1 (Bottom)	74787.95	2524916.00	0.03
8	D2 (Bottom)	255451.20	2531122.00	0.10
9	D2 (Top)	185997.80	5814881.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Max)	781391.10	1296000.00	0.60
2	DC1 (Top)	0.00	475200.00	0.00
3	DC1 (Top)	0.00	1913760.00	0.00
4	D3 (Bottom)	209026.70	1438560.00	0.15
5	DC1 (Top)	0.00	66960.00	0.00
6	DC1 (Top)	0.00	1572480.00	0.00
7	D3 (Bottom)	159506.10	427680.00	0.37
8	DC1 (Top)	0.00	301320.00	0.00
9	DC1 (Top)	0.00	829440.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	197782.340	216867.990	0.91
2	D2 (Top)	284603.910	627333.760	0.45
3	D3 (Bottom)	487438.430	1009002.600	0.48
4	D3 (Bottom)	400797.630	505706.360	0.79
5	D3 (Bottom)	9417.372	89752.509	0.10
6	D3 (Max)	418831.650	788173.350	0.53
7	D3 (Bottom)	132984.030	259985.580	0.51
8	D3 (Top)	200482.740	323318.950	0.62
9	D3 (Bottom)	362154.100	691983.570	0.52

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 1:11 AM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 6.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : N. G.

Design code	:	ACI 318-05

Geometry:

Total height	:	41.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 3

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00
3	15.00	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	4.00	0.00	3.00	7.00
Lower left	17.00	0.00	3.00	7.00
Lower left	2.00	10.00	3.00	10.00
Lower left	8.00	26.50	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ

D3	Yes	0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
3	EQ	Horizontal	327998.00	0.00	0.00
2	EQ	Horizontal	84930.00	0.00	0.00
1	EQ	Horizontal	57990.00	0.00	0.00

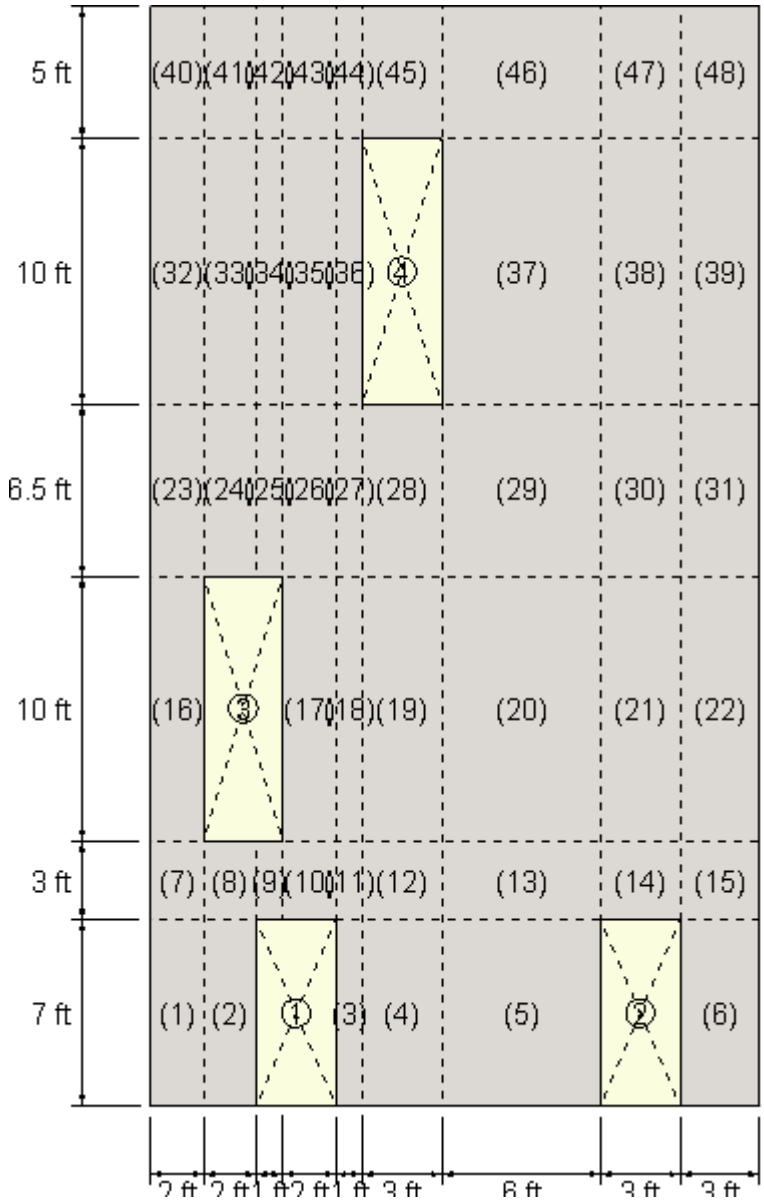
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
3	DL	Vertical	3360.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : N. G.
- Insufficient axial tension strength (Segment 1)



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.00	7.00
2	2.00	0.00	2.00	7.00
3	7.00	0.00	1.00	7.00
4	8.00	0.00	3.00	7.00
5	11.00	0.00	6.00	7.00
6	20.00	0.00	3.00	7.00
7	0.00	7.00	2.00	3.00
8	2.00	7.00	2.00	3.00
9	4.00	7.00	1.00	3.00
10	5.00	7.00	2.00	3.00
11	7.00	7.00	1.00	3.00
12	8.00	7.00	3.00	3.00
13	11.00	7.00	6.00	3.00
14	17.00	7.00	3.00	3.00
15	20.00	7.00	3.00	3.00
16	0.00	10.00	2.00	10.00

17	5.00	10.00	2.00	10.00
18	7.00	10.00	1.00	10.00
19	8.00	10.00	3.00	10.00
20	11.00	10.00	6.00	10.00
21	17.00	10.00	3.00	10.00
22	20.00	10.00	3.00	10.00
23	0.00	20.00	2.00	6.50
24	2.00	20.00	2.00	6.50
25	4.00	20.00	1.00	6.50
26	5.00	20.00	2.00	6.50
27	7.00	20.00	1.00	6.50
28	8.00	20.00	3.00	6.50
29	11.00	20.00	6.00	6.50
30	17.00	20.00	3.00	6.50
31	20.00	20.00	3.00	6.50
32	0.00	26.50	2.00	10.00
33	2.00	26.50	2.00	10.00
34	4.00	26.50	1.00	10.00
35	5.00	26.50	2.00	10.00
36	7.00	26.50	1.00	10.00
37	11.00	26.50	6.00	10.00
38	17.00	26.50	3.00	10.00
39	20.00	26.50	3.00	10.00
40	0.00	36.50	2.00	5.00
41	2.00	36.50	2.00	5.00
42	4.00	36.50	1.00	5.00
43	5.00	36.50	2.00	5.00
44	7.00	36.50	1.00	5.00
45	8.00	36.50	3.00	5.00
46	11.00	36.50	6.00	5.00
47	17.00	36.50	3.00	5.00
48	20.00	36.50	3.00	5.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	3-#10	10.00	49.19
2	3-#10	10.00	49.19
3	1-#10	10.00	49.19
4	2-#9	18.00	43.69
5	7-#6	10.00	23.24
6	3-#6	12.00	23.24
7	3-#10	10.00	49.19
8	3-#10	10.00	49.19
9	1-#9	18.00	43.69
10	3-#8	10.00	38.73
11	1-#10	10.00	49.19
12	2-#9	18.00	43.69
13	7-#6	10.00	23.24
14	3-#6	12.00	23.24
15	3-#6	12.00	23.24
16	3-#9	10.00	43.69
17	3-#8	10.00	38.73
18	1-#6	18.00	23.24
19	3-#5	14.00	19.36
20	5-#5	14.00	19.36
21	3-#5	14.00	19.36
22	3-#5	14.00	19.36

23	3-#9	10.00	43.69
24	2-#6	18.00	23.24
25	1-#5	14.00	19.36
26	3-#8	10.00	38.73
27	1-#6	18.00	23.24
28	3-#5	14.00	19.36
29	5-#5	14.00	19.36
30	3-#5	14.00	19.36
31	3-#5	14.00	19.36
32	2-#8	18.00	38.73
33	2-#7	18.00	33.89
34	1-#7	18.00	33.89
35	2-#7	18.00	33.89
36	1-#7	18.00	33.89
37	5-#5	14.00	19.36
38	3-#5	14.00	19.36
39	3-#5	14.00	19.36
40	2-#8	18.00	38.73
41	2-#7	18.00	33.89
42	1-#7	18.00	33.89
43	2-#7	18.00	33.89
44	1-#7	18.00	33.89
45	3-#5	14.00	19.36
46	5-#5	14.00	19.36
47	3-#5	14.00	19.36
48	3-#5	14.00	19.36

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	3.15	12.86
2	DC1 (Top)	3.18	12.86
3	DC1 (Top)	2.38	12.99
4	DC1 (Top)	1.67	13.06
5	DC1 (Top)	1.46	13.25
6	DC1 (Top)	1.38	13.25
7	DC1 (Top)	3.28	12.86
8	DC1 (Top)	2.63	12.86
9	DC1 (Top)	1.82	12.94
10	DC1 (Top)	2.10	13.00
11	DC1 (Top)	2.19	12.86
12	DC1 (Top)	1.69	12.94
13	DC1 (Top)	1.46	13.13
14	DC1 (Top)	1.27	13.13
15	DC1 (Top)	1.35	13.13
16	DC1 (Top)	2.43	8.06
17	DC1 (Top)	1.97	8.13
18	DC1 (Top)	1.25	8.25
19	DC1 (Top)	1.03	8.31
20	DC1 (Top)	0.93	8.31
21	DC1 (Top)	1.02	8.31
22	DC1 (Top)	1.01	8.31
23	DC1 (Top)	2.22	8.06
24	DC1 (Top)	1.21	8.25
25	DC1 (Top)	1.01	8.31
26	DC1 (Top)	1.91	8.13
27	DC1 (Top)	1.24	8.25
28	DC1 (Top)	0.91	8.31
29	DC1 (Top)	0.93	8.31
30	DC1 (Top)	1.00	8.31
31	DC1 (Top)	0.99	8.31

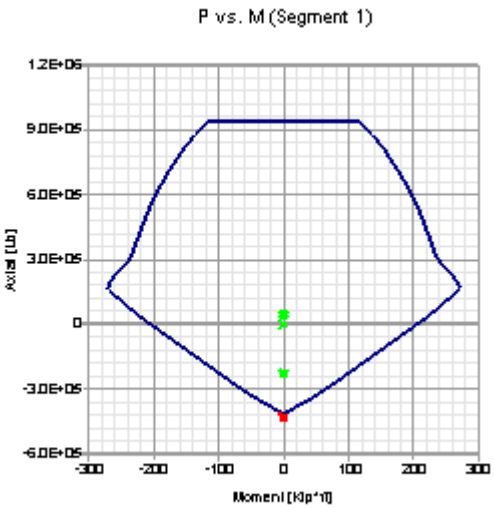
32	DC1 (Top)	1.52	6.13
33	DC1 (Top)	1.33	6.19
34	DC1 (Top)	1.34	6.19
35	DC1 (Top)	1.34	6.19
36	DC1 (Top)	1.36	6.19
37	DC1 (Top)	0.84	6.31
38	DC1 (Top)	0.93	6.31
39	DC1 (Top)	0.92	6.31
40	DC1 (Top)	1.51	6.13
41	DC1 (Top)	1.32	6.19
42	DC1 (Top)	1.32	6.19
43	DC1 (Top)	1.32	6.19
44	DC1 (Top)	1.32	6.19
45	DC1 (Top)	0.91	6.31
46	DC1 (Top)	0.79	6.31
47	DC1 (Top)	0.91	6.31
48	DC1 (Top)	0.91	6.31

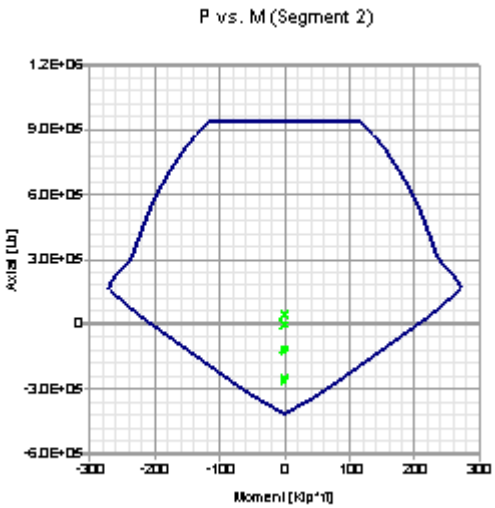
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	40352.30	0.00	222.66	0.00
2	DC1 (Top)	41953.42	0.00	223.36	0.00
3	DC1 (Top)	28734.89	0.00	85.01	0.00
4	DC1 (Top)	69682.96	0.00	154.83	0.00
5	DC1 (Top)	140633.10	0.00	262.71	0.00
6	DC1 (Top)	70783.40	0.00	119.72	0.00
7	DC1 (Top)	47834.69	0.00	225.82	0.00
8	DC1 (Top)	11924.59	0.00	209.99	0.00
9	DC1 (Top)	10349.53	0.00	62.26	0.00
10	DC1 (Top)	37198.55	0.00	151.74	0.00
11	DC1 (Top)	18656.28	0.00	79.53	0.00
12	DC1 (Top)	63590.52	0.00	151.46	0.00
13	DC1 (Top)	120110.60	0.00	251.86	0.00
14	DC1 (Top)	40085.11	0.00	103.87	0.00
15	DC1 (Top)	54736.92	0.00	111.51	0.00
16	DC1 (Top)	34872.82	0.00	108.68	0.00
17	DC1 (Top)	31447.36	0.00	90.22	0.00
18	DC1 (Top)	13692.58	0.00	22.07	0.00
19	DC1 (Top)	37270.77	0.00	51.77	0.00
20	DC1 (Top)	71844.96	0.00	91.68	0.00
21	DC1 (Top)	34596.49	0.00	50.88	0.00
22	DC1 (Top)	32870.36	0.00	50.31	0.00
23	DC1 (Top)	21811.83	0.00	105.08	0.00
24	DC1 (Top)	21397.50	0.00	42.32	0.00
25	DC1 (Top)	10939.64	0.00	16.77	0.00
26	DC1 (Top)	23600.88	0.00	87.98	0.00
27	DC1 (Top)	12943.31	0.00	21.84	0.00
28	DC1 (Top)	16214.36	0.00	44.82	0.00
29	DC1 (Top)	71995.56	0.00	91.73	0.00
30	DC1 (Top)	32590.30	0.00	50.22	0.00
31	DC1 (Top)	30636.79	0.00	49.58	0.00
32	DC1 (Top)	10306.96	0.00	45.67	0.00
33	DC1 (Top)	11438.90	0.00	37.75	0.00
34	DC1 (Top)	6222.39	0.00	18.98	0.00
35	DC1 (Top)	13489.18	0.00	38.19	0.00
36	DC1 (Top)	8226.65	0.00	19.41	0.00
37	DC1 (Top)	41230.54	0.00	63.99	0.00

38	DC1 (Top)	16352.62	0.00	35.29	0.00
39	DC1 (Top)	15209.80	0.00	35.01	0.00
40	DC1 (Top)	9543.35	0.00	45.51	0.00
41	DC1 (Top)	9587.47	0.00	37.36	0.00
42	DC1 (Top)	4820.90	0.00	18.69	0.00
43	DC1 (Top)	9616.38	0.00	37.37	0.00
44	DC1 (Top)	4821.57	0.00	18.69	0.00
45	DC1 (Top)	13981.52	0.00	34.71	0.00
46	DC1 (Top)	29181.69	0.00	60.55	0.00
47	DC1 (Top)	14357.55	0.00	34.80	0.00
48	DC1 (Top)	14316.38	0.00	34.79	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	51723.20	934511.80	0.06
2	D1 (Bottom)	43576.42	934511.80	0.05
3	D1 (Top)	28734.89	470623.90	0.06
4	DC1 (Bottom)	77566.62	1421472.00	0.05
5	D2 (Bottom)	561685.60	2847824.00	0.20
6	D2 (Bottom)	716437.50	1425079.00	0.50
7	DC1 (Top)	47834.69	934511.80	0.05
8	D1 (Bottom)	41953.15	934511.80	0.04
9	DC1 (Top)	10349.53	472056.00	0.02
10	D1 (Top)	37198.55	942149.50	0.04
11	DC1 (Bottom)	28732.38	470623.90	0.06
12	D1 (Bottom)	69682.95	1421472.00	0.05
13	D2 (Bottom)	291149.90	2847824.00	0.10
14	D2 (Top)	223971.40	1425079.00	0.16
15	D2 (Bottom)	645884.30	1425079.00	0.45
16	DC1 (Bottom)	41473.77	620568.00	0.07
17	DC1 (Top)	31447.36	623909.50	0.05
18	DC1 (Bottom)	14558.05	315906.20	0.05
19	D1 (Bottom)	48240.86	949787.30	0.05

20	D2 (Bottom)	174361.80	1901219.00	0.09
21	D2 (Bottom)	224274.50	949787.30	0.24
22	D2 (Bottom)	470551.40	949787.30	0.50
23	DC1 (Bottom)	34872.48	620568.00	0.06
24	D1 (Top)	21397.50	631812.50	0.03
25	D1 (Top)	10939.64	316595.80	0.03
26	DC1 (Bottom)	31444.05	623909.50	0.05
27	D2 (Top)	59690.48	315906.20	0.19
28	D1 (Bottom)	37270.76	949787.30	0.04
29	D2 (Bottom)	96511.26	1901219.00	0.05
30	D2 (Bottom)	183490.50	949787.30	0.19
31	D2 (Bottom)	280826.40	949787.30	0.30
32	DC1 (Bottom)	13432.60	500803.70	0.03
33	D1 (Bottom)	13700.37	502819.20	0.03
34	D2 (Top)	14879.76	251409.60	0.06
35	D2 (Bottom)	23025.23	502819.20	0.05
36	D2 (Bottom)	68137.88	251409.60	0.27
37	D2 (Top)	164071.30	1519331.00	0.11
38	D2 (Bottom)	120965.70	758843.30	0.16
39	D2 (Bottom)	185105.70	758843.30	0.24
40	D2 (Top)	75017.74	500803.70	0.15
41	D2 (Top)	23499.19	502819.20	0.05
42	D2 (Bottom)	14879.84	251409.60	0.06
43	D1 (Bottom)	13489.18	502819.20	0.03
44	D1 (Bottom)	8226.52	251409.60	0.03
45	DC1 (Top)	13981.52	758843.30	0.02
46	D2 (Bottom)	164068.60	1519331.00	0.11
47	D2 (Bottom)	25856.02	758843.30	0.03
48	D2 (Bottom)	25357.61	758843.30	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	433828.10	411480.00	1.05
2	D3 (Top)	260339.70	411480.00	0.63
3	D3 (Bottom)	178121.50	137160.00	1.30
4	D3 (Bottom)	209423.00	216000.00	0.97
5	DC1 (Top)	0.00	332640.00	0.00
6	DC1 (Top)	0.00	142560.00	0.00
7	D3 (Top)	280216.60	411480.00	0.68
8	D3 (Bottom)	260337.50	411480.00	0.63
9	D3 (Bottom)	93876.43	108000.00	0.87
10	D3 (Top)	216097.50	255960.00	0.84
11	D3 (Bottom)	52517.43	137160.00	0.38
12	D3 (Bottom)	110013.00	216000.00	0.51
13	DC1 (Top)	0.00	332640.00	0.00
14	DC1 (Top)	0.00	142560.00	0.00
15	DC1 (Top)	0.00	142560.00	0.00
16	D3 (Bottom)	292642.50	324000.00	0.90
17	D3 (Bottom)	222877.60	255960.00	0.87
18	D3 (Bottom)	44861.11	47520.00	0.94
19	D3 (Bottom)	53652.71	100440.00	0.53
20	DC1 (Top)	0.00	167400.00	0.00
21	DC1 (Top)	0.00	100440.00	0.00
22	DC1 (Top)	0.00	100440.00	0.00
23	D3 (Bottom)	254822.90	324000.00	0.79
24	D3 (Top)	84800.91	95040.00	0.89

25	D3 (Top)	15008.93	33480.00	0.45
26	D3 (Bottom)	27845.22	255960.00	0.11
27	D3 (Bottom)	16372.79	47520.00	0.34
28	D3 (Bottom)	36487.71	100440.00	0.36
29	D3 (Top)	16905.70	167400.00	0.10
30	DC1 (Top)	0.00	100440.00	0.00
31	DC1 (Top)	0.00	100440.00	0.00
32	D3 (Bottom)	151030.80	170640.00	0.89
33	D3 (Bottom)	88144.50	129600.00	0.68
34	D3 (Bottom)	18374.42	64800.00	0.28
35	DC1 (Top)	0.00	129600.00	0.00
36	D3 (Top)	52867.32	64800.00	0.82
37	D3 (Bottom)	35364.83	167400.00	0.21
38	DC1 (Top)	0.00	100440.00	0.00
39	DC1 (Top)	0.00	100440.00	0.00
40	D3 (Bottom)	91195.33	170640.00	0.53
41	DC1 (Top)	0.00	129600.00	0.00
42	DC1 (Top)	0.00	64800.00	0.00
43	DC1 (Top)	0.00	129600.00	0.00
44	D3 (Bottom)	52866.23	64800.00	0.82
45	DC1 (Top)	0.00	100440.00	0.00
46	DC1 (Top)	0.00	167400.00	0.00
47	DC1 (Top)	0.00	100440.00	0.00
48	DC1 (Top)	0.00	100440.00	0.00

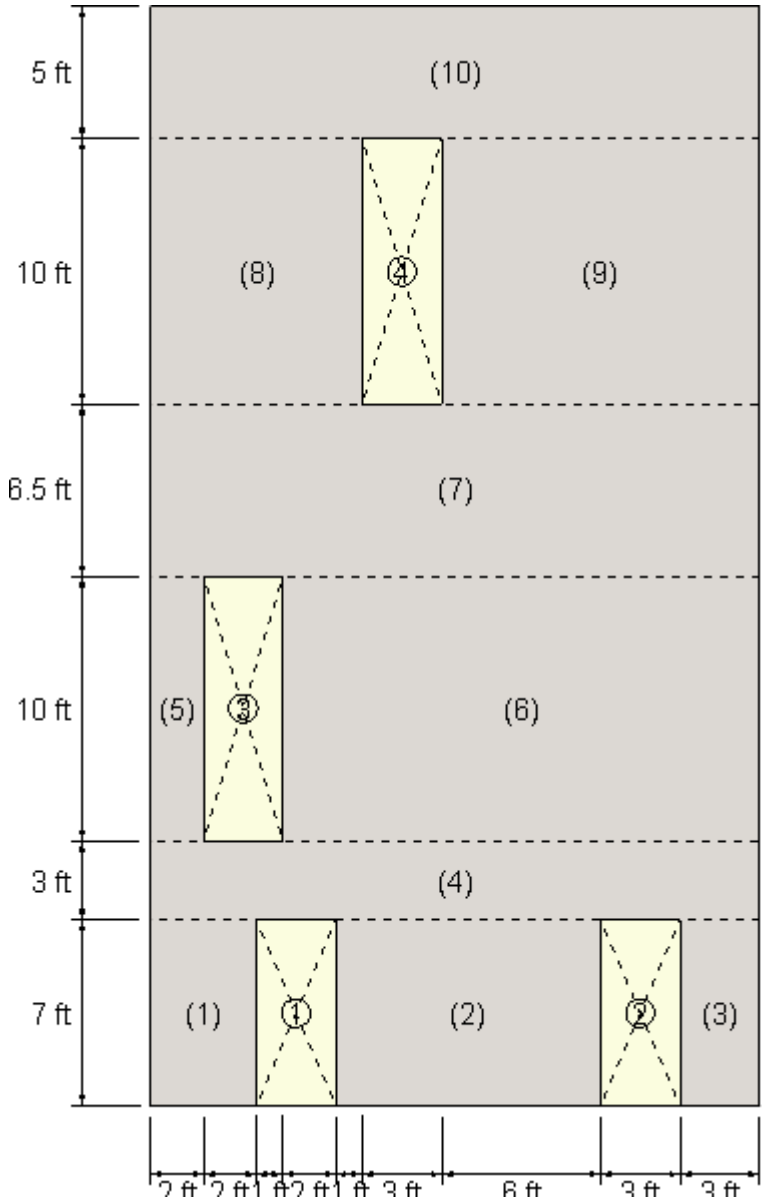
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	35874.670	0.00
2	DC1 (Top)	0.000	35874.670	0.00
3	DC1 (Top)	0.000	18111.619	0.00
4	DC1 (Top)	0.000	54631.838	0.00
5	DC1 (Top)	0.000	110844.780	0.00
6	DC1 (Top)	0.000	55422.392	0.00
7	DC1 (Top)	0.000	35874.670	0.00
8	DC1 (Top)	0.000	35874.670	0.00
9	DC1 (Top)	0.000	18036.329	0.00
10	DC1 (Top)	0.000	36251.124	0.00
11	DC1 (Top)	0.000	17937.335	0.00
12	DC1 (Top)	0.000	54108.986	0.00
13	DC1 (Top)	0.000	109799.080	0.00
14	DC1 (Top)	0.000	54899.539	0.00
15	DC1 (Top)	0.000	54899.539	0.00
16	DC1 (Top)	0.000	22478.486	0.00
17	DC1 (Top)	0.000	22656.953	0.00
18	DC1 (Top)	0.000	11502.761	0.00
19	DC1 (Top)	0.000	34769.708	0.00
20	DC1 (Top)	0.000	69539.416	0.00
21	DC1 (Top)	0.000	34769.708	0.00
22	DC1 (Top)	0.000	34769.708	0.00
23	DC1 (Top)	0.000	22478.486	0.00
24	DC1 (Top)	0.000	23005.521	0.00
25	DC1 (Top)	0.000	11589.903	0.00
26	DC1 (Top)	0.000	22656.953	0.00
27	DC1 (Top)	0.000	11502.761	0.00
28	DC1 (Top)	0.000	34769.708	0.00
29	DC1 (Top)	0.000	69539.416	0.00

30	DC1 (Top)	0.000	34769.708	0.00	
31	DC1 (Top)	0.000	34769.708	0.00	
32	DC1 (Top)	0.000	17079.857	0.00	
33	DC1 (Top)	0.000	17254.141	0.00	
34	DC1 (Top)	0.000	8627.070	0.00	
35	DC1 (Top)	0.000	17254.141	0.00	
36	DC1 (Top)	0.000	8627.070	0.00	
37	DC1 (Top)	0.000	52808.128	0.00	
38	DC1 (Top)	0.000	26404.064	0.00	
39	DC1 (Top)	0.000	26404.064	0.00	
40	DC1 (Top)	0.000	17079.857	0.00	
41	DC1 (Top)	0.000	17254.141	0.00	
42	DC1 (Top)	0.000	8627.070	0.00	
43	DC1 (Top)	0.000	17254.141	0.00	
44	DC1 (Top)	0.000	8627.070	0.00	
45	DC1 (Top)	0.000	26404.064	0.00	
46	DC1 (Top)	0.000	52808.128	0.00	
47	DC1 (Top)	0.000	26404.064	0.00	
48	DC1 (Top)	0.000	26404.064	0.00	

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	4.00	7.00
2	7.00	0.00	10.00	7.00
3	20.00	0.00	3.00	7.00
4	0.00	7.00	23.00	3.00
5	0.00	10.00	2.00	10.00
6	5.00	10.00	18.00	10.00
7	0.00	20.00	23.00	6.50
8	0.00	26.50	8.00	10.00
9	11.00	26.50	12.00	10.00
10	0.00	36.50	23.00	5.00

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	3-#10	10.00	49.19	6-#6	14.00	30.21
	3-#10	10.00	49.19	6-#6	14.00	30.21
2	1-#10	10.00	49.19	6-#5	16.00	25.17
	2-#9	18.00	49.19	6-#5	16.00	25.17
	7-#6	10.00	49.19	6-#5	16.00	25.17
3	3-#6	12.00	23.24	5-#5	18.00	25.17
4	3-#10	10.00	49.19	2-#6	18.00	30.21
	3-#10	10.00	49.19	2-#6	18.00	30.21
	1-#9	18.00	49.19	2-#6	18.00	30.21
	3-#8	10.00	49.19	2-#6	18.00	30.21
	1-#10	10.00	49.19	2-#6	18.00	30.21
	2-#9	18.00	49.19	2-#6	18.00	30.21
	7-#6	10.00	49.19	2-#6	18.00	30.21
	3-#6	12.00	49.19	2-#6	18.00	30.21
	3-#6	12.00	49.19	2-#6	18.00	30.21
5	3-#9	10.00	43.69	7-#5	18.00	25.17
6	3-#8	10.00	38.73	7-#5	18.00	25.17
	1-#6	18.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
	5-#5	14.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
7	3-#9	10.00	43.69	5-#5	18.00	25.17
	2-#6	18.00	43.69	5-#5	18.00	25.17
	1-#5	14.00	43.69	5-#5	18.00	25.17
	3-#8	10.00	43.69	5-#5	18.00	25.17
	1-#6	18.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
	5-#5	14.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
8	2-#8	18.00	38.73	7-#5	18.00	25.17
	2-#7	18.00	38.73	7-#5	18.00	25.17
	1-#7	18.00	38.73	7-#5	18.00	25.17
	2-#7	18.00	38.73	7-#5	18.00	25.17
	1-#7	18.00	38.73	7-#5	18.00	25.17
9	5-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
10	2-#8	18.00	38.73	4-#5	18.00	25.17
	2-#7	18.00	38.73	4-#5	18.00	25.17
	1-#7	18.00	38.73	4-#5	18.00	25.17
	2-#7	18.00	38.73	4-#5	18.00	25.17
	1-#7	18.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17
	5-#5	14.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17

Intermediate results for axial-bending

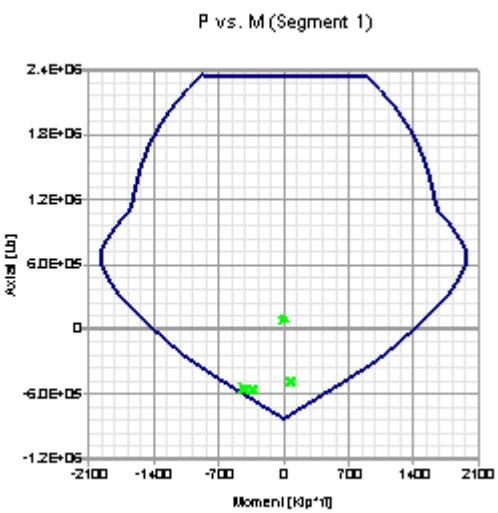
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	2.05	38.40
2	D3 (Bottom)	11.74	96.00
3	D2 (Bottom)	19.10	28.80
4	D3 (Bottom)	31.60	220.80
5	D3 (Bottom)	0.61	19.20

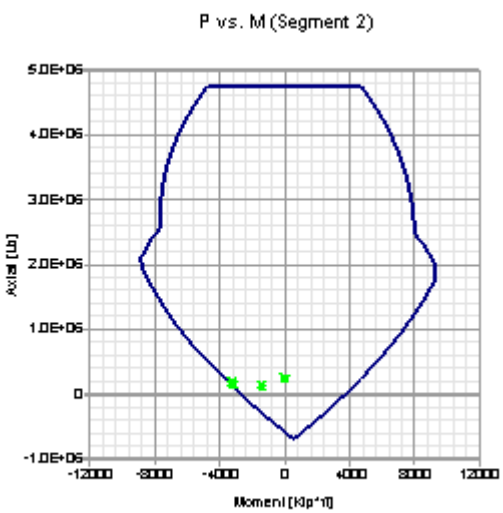
6	D3 (Max)	26.84	172.80
7	D3 (Bottom)	29.49	220.80
8	D3 (Bottom)	7.88	76.80
9	D3 (Bottom)	20.50	115.20
10	D3 (Bottom)	30.82	220.80

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	-555613.60	-431.44	508.01	0.85
2	D3 (Bottom)	140215.50	-3201.56	3250.97	0.98
3	D2 (Bottom)	716407.70	-232.70	729.57	0.32
4	D3 (Bottom)	181808.20	-13263.75	16182.40	0.82
5	D3 (Bottom)	-292589.40	-20.36	29.81	0.68
6	D3 (Max)	457995.20	-6960.61	8276.19	0.84
7	D3 (Bottom)	169543.70	-7604.02	10453.93	0.73
8	D3 (Bottom)	-171763.10	-839.59	1302.88	0.64
9	D3 (Bottom)	262165.50	-1423.27	3457.22	0.41
10	D3 (Bottom)	80427.17	-1663.74	9039.71	0.18

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	95304.88	2344512.00	0.04
2	DC1 (Bottom)	264716.50	4739920.00	0.06
3	D2 (Max)	728146.90	1425079.00	0.51
4	DC1 (Bottom)	422433.10	1.087331E07	0.04
5	DC1 (Bottom)	41478.04	807768.00	0.05
6	D2 (Bottom)	567412.50	5690396.00	0.10
7	DC1 (Bottom)	267793.90	7259373.00	0.04
8	DC1 (Bottom)	60218.63	2332493.00	0.03
9	D2 (Bottom)	281142.40	3037017.00	0.09

10	D2 (Top)	184128.70	5805122.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Max)	567332.50	822960.00	0.69
2	DC1 (Top)	0.00	685800.00	0.00
3	DC1 (Top)	0.00	142560.00	0.00
4	DC1 (Top)	0.00	2157840.00	0.00
5	D3 (Bottom)	292589.40	324000.00	0.90
6	DC1 (Top)	0.00	772200.00	0.00
7	DC1 (Top)	0.00	1224720.00	0.00
8	D3 (Bottom)	171763.10	559440.00	0.31
9	DC1 (Top)	0.00	368280.00	0.00
10	DC1 (Top)	0.00	1028160.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Bottom)	100596.220	108617.140	0.93
2	D3 (Max)	328685.950	465338.110	0.71
3	D3 (Bottom)	62412.208	217169.650	0.29
4	D3 (Max)	511999.200	1039453.600	0.49
5	D3 (Top)	7027.482	29760.000	0.24
6	D3 (Top)	414691.140	666346.850	0.62
7	D3 (Bottom)	419051.550	790973.140	0.53
8	D3 (Bottom)	91971.404	177899.640	0.52
9	D3 (Top)	240182.340	385216.190	0.62
10	D3 (Max)	338275.380	693428.130	0.49

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 9:10 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL C.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	26.50 [ft]
Total length	:	22.50 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	6.33	10.00	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
2	EQ	Horizontal	336820.00	0.00	0.00
1	EQ	Horizontal	116474.00	0.00	0.00

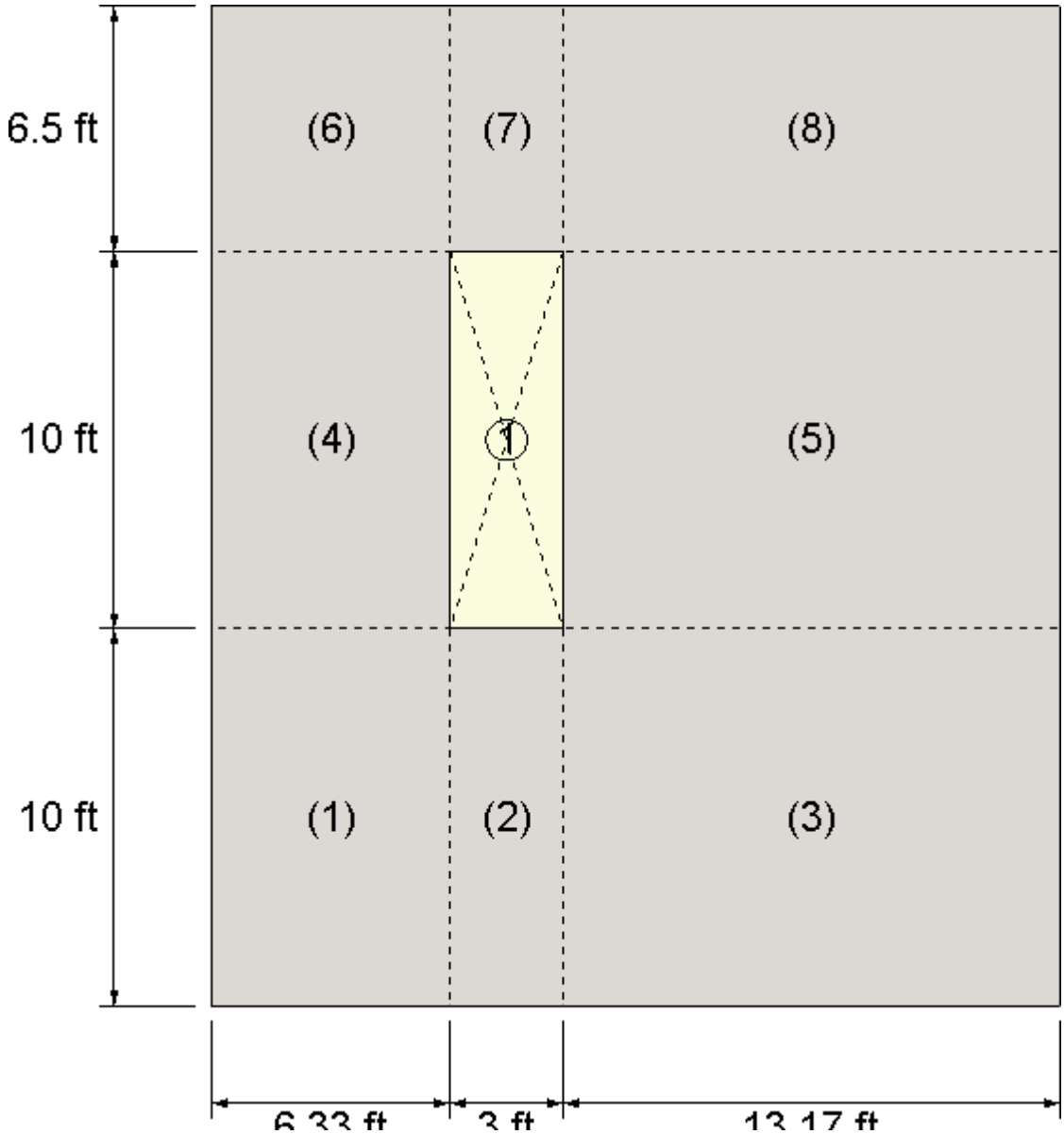
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
2	DL	Vertical	5850.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	6.33	10.00
2	6.33	0.00	3.00	10.00
3	9.33	0.00	13.17	10.00
4	0.00	10.00	6.33	10.00
5	9.33	10.00	13.17	10.00
6	0.00	20.00	6.33	6.50
7	6.33	20.00	3.00	6.50
8	9.33	20.00	13.17	6.50

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#7	12.00	33.89
2	3-#5	14.00	19.36
3	12-#5	14.00	19.36
4	8-#6	10.00	23.24
5	9-#5	18.00	19.36
6	8-#6	10.00	23.24
7	2-#5	18.00	19.36
8	9-#5	18.00	19.36

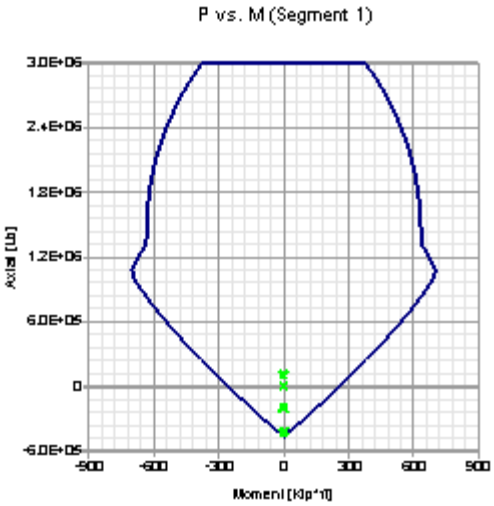
Intermediate results for axial-bending

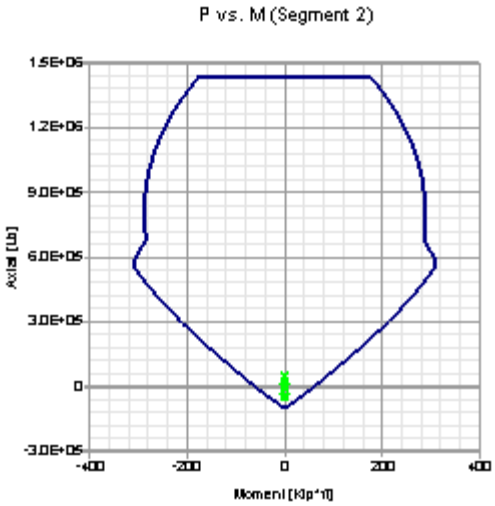
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.54	13.19
2	DC1 (Top)	0.92	13.31
3	DC1 (Top)	1.04	13.31
4	DC1 (Top)	1.34	8.25
5	DC1 (Top)	0.80	8.31
6	DC1 (Top)	1.31	8.25
7	DC1 (Top)	0.74	8.31
8	DC1 (Top)	0.75	8.31

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	107954.70	0.00	307.13	0.00
2	DC1 (Top)	19347.17	0.00	70.67	0.00
3	DC1 (Top)	213666.10	0.00	358.19	0.00
4	DC1 (Top)	71207.09	0.00	158.49	0.00
5	DC1 (Top)	133647.80	0.00	170.15	0.00
6	DC1 (Top)	52668.48	0.00	152.90	0.00
7	DC1 (Top)	24661.37	0.00	35.97	0.00
8	DC1 (Top)	109283.80	0.00	161.05	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	115725.00	2999412.00	0.04
2	D1 (Bottom)	47536.10	1427147.00	0.03
3	D2 (Bottom)	780934.20	6267100.00	0.12
4	D1 (Bottom)	81814.68	1995789.00	0.04
5	D2 (Bottom)	381941.90	4176423.00	0.09
6	D2 (Top)	137598.80	1995789.00	0.07
7	D1 (Top)	24661.37	951431.50	0.03
8	D2 (Bottom)	308265.00	4176423.00	0.07

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	432253.80	453600.00	0.95
2	D3 (Bottom)	47768.95	100440.00	0.48
3	DC1 (Top)	0.00	401760.00	0.00
4	D3 (Bottom)	206955.20	380160.00	0.54
5	DC1 (Top)	0.00	301320.00	0.00
6	D3 (Bottom)	166480.90	380160.00	0.44
7	DC1 (Top)	0.00	66960.00	0.00
8	DC1 (Top)	0.00	301320.00	0.00

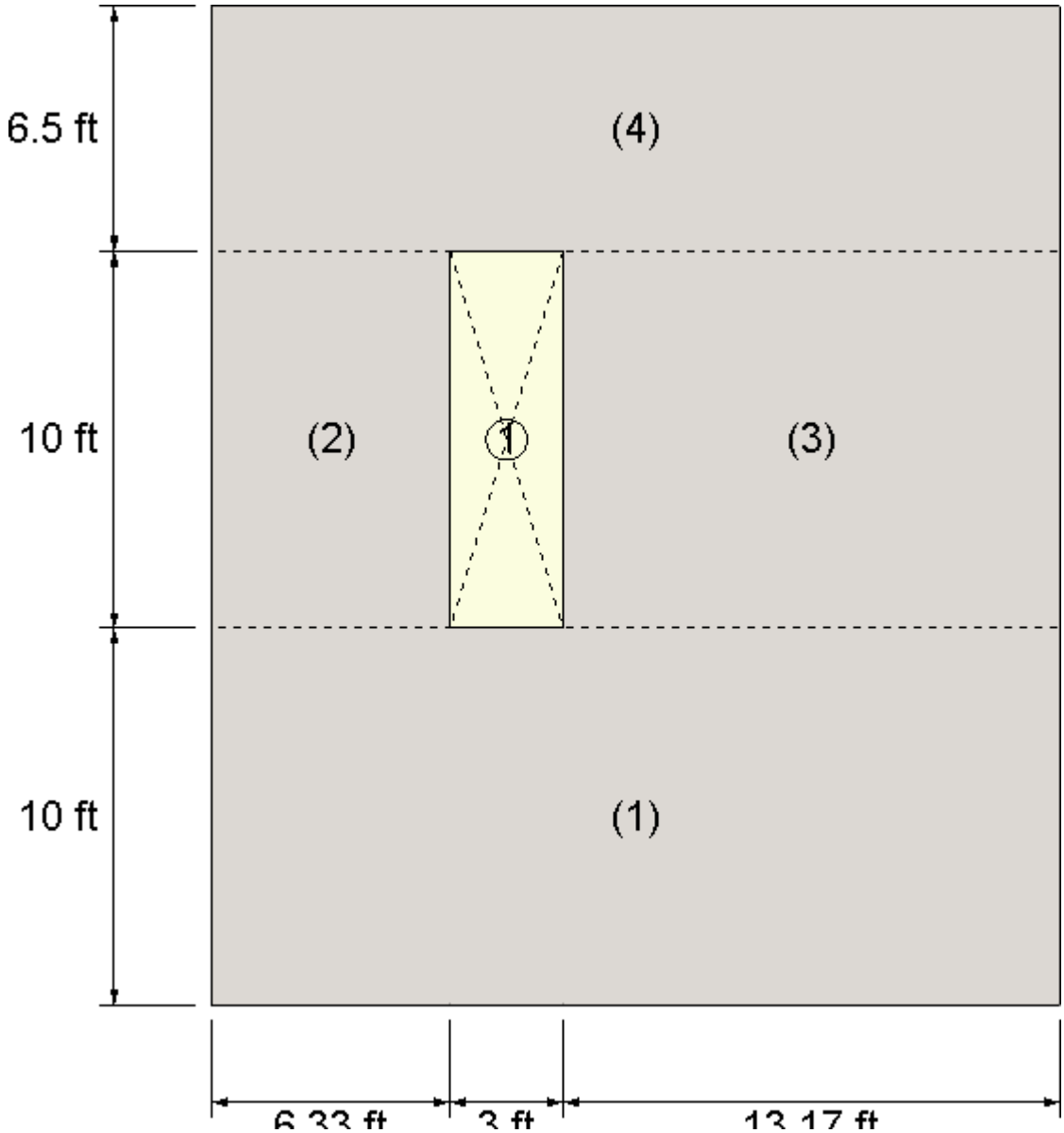
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	116389.640	0.00
2	DC1 (Top)	0.000	55683.818	0.00
3	DC1 (Top)	0.000	244451.960	0.00
4	DC1 (Top)	0.000	72812.474	0.00
5	DC1 (Top)	0.000	152639.020	0.00

6	DC1 (Top)	0.000	72812.474	0.00
7	DC1 (Top)	0.000	34769.708	0.00
8	DC1 (Top)	0.000	152639.020	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	22.50	10.00
2	0.00	10.00	6.33	10.00
3	9.33	10.00	13.17	10.00
4	0.00	20.00	22.50	6.50

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	7-#7	12.00	33.89	8-#5	16.00	25.17
	3-#5	14.00	33.89	8-#5	16.00	25.17
	12-#5	14.00	33.89	8-#5	16.00	25.17
2	8-#6	10.00	23.24	7-#5	18.00	25.17
3	9-#5	18.00	19.36	7-#5	18.00	25.17
4	8-#6	10.00	23.24	5-#5	18.00	25.17
	2-#5	18.00	23.24	5-#5	18.00	25.17
	9-#5	18.00	23.24	5-#5	18.00	25.17

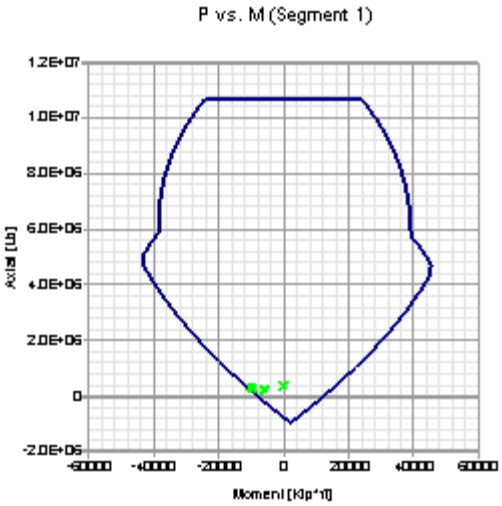
Intermediate results for axial-bending

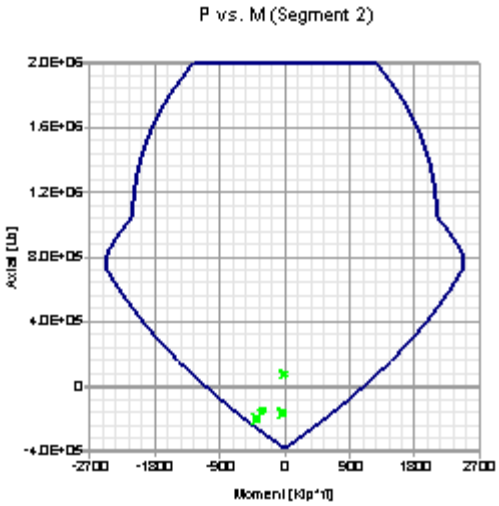
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	20.17	216.00
2	D3 (Bottom)	3.77	60.77
3	D3 (Bottom)	17.32	126.43
4	D3 (Bottom)	21.46	216.00

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	251792.80	-9836.55	10579.03	0.93
2	D3 (Bottom)	-206741.10	-393.16	527.75	0.74
3	D3 (Bottom)	348007.50	-2016.99	3903.63	0.52
4	D3 (Bottom)	135741.30	-2235.20	7360.60	0.30

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	392564.30	1.069366E07	0.04
2	D1 (Bottom)	81820.70	1995789.00	0.04
3	D2 (Bottom)	381833.70	4176423.00	0.09
4	D2 (Top)	254149.70	7123643.00	0.04

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	955800.00	0.00
2	D3 (Bottom)	206741.10	380160.00	0.54
3	DC1 (Top)	0.00	301320.00	0.00
4	DC1 (Top)	0.00	748440.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	457614.000	1035137.500	0.44
2	D3 (Bottom)	44487.234	136931.340	0.32
3	D3 (Top)	294490.860	482044.520	0.61
4	D3 (Max)	346384.460	769069.510	0.45

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

APPENDIX - C

SUMMARY DATA SHEET

BUILDING DATA

Building Name: Benton County Courthouse Date: January 29, 2016

Building Address: 120 NW 4th Street, Corvallis, Oregon

Latitude: 44.56528 N Longitude: 123.26237 W By:

Year Built: 1887 Year(s) Remodeled: 1977 Original Design Code: N/A

Area (sf): 33,052 Length (ft): Varies Width (ft): Varies

No. of Stories: Four Story Height: Varies Total Height: 59'-6"(Tower:115'-5")

USE ☐ Industrial ☒ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☒ Other: Assembly

CONSTRUCTION DATA

Gravity Load Structural System: Unreinforced masonry bearing walls with interior steel supports

Exterior Transverse Walls: Unreinforced masonry shear walls Openings? Punched openings

Exterior Longitudinal Walls: Unreinforced masonry shear walls Openings? Punched openings

Roof Materials/Framing: Timber trusses and 2x rafters with T&G straight sheathing

Intermediate Floors/Framing: Timber 3 x 16 floor joists with T&G straight sheathing

Ground Floor: Concrete slab on grade

Columns: Wood and steel columns Foundation: Unknown foundation

General Condition of Structure: Well maintained structure in good condition

Levels Below Grade? N/A

Special Features and Comments: The building houses the Benton County Courts

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Masonry shear wall	Masonry shear wall
Vertical Elements:	Bearing wall	Bearing wall
Diaphragms:	Flexible wood sheathing	Flexible wood sheathing
Connections:	Unknown	Unknown

EVALUATION DATA

BSE-1N Spectral Response Accelerations: S_{Ds} = 0.712 S_{D1} = 0.480

Soil Factors: Class = D F_a = 1.12 F_v = 1.53

BSE-1E Spectral Response Accelerations: S_{XS} = 0.36 S_{X1} = 0.22

Level of Seismicity: High Performance Level: Immediate Occupancy

Building Period: T = 0.43

Spectral Acceleration: S_a = 0.36

Modification Factor: $C_mC_1C_2$ = 1.0 Building Weight: W = 7,344,016 lb

Pseudo Lateral Force: V= 0.36

$C_mC_1C_2S_aW$= 2,608,595 lb

BUILDING CLASSIFICATION:

REQUIRED TIER 1 CHECKLISTS	**Yes**	**No**
Basic Configuration Checklist	☒	☐
Building Type URM **Structural Checklist**	☒	☐
Nonstructural Component Checklist	☒	☐

FURTHER EVALUATION REQUIREMENT:

Table 2-1. Basic Performance Objective for Existing Buildings (BPOE)

	Tier 1[a]	Tier 2[a]	Tier 3	
Risk Category	**BSE-1E**	**BSE-1E**	**BSE-1E**	**BSE-2E**
I & II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Nonstructural Performance Not Considered (5-D)
III	See footnote *b* for Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Nonstructural Performance Not Considered (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Nonstructural Performance Not Considered (3-D)

[a]For Tier 1 and 2 assessments, seismic performance for the BSE-2E is not explicitly evaluated.
[b]For Risk Category III, the Tier 1 screening checklists shall be based on the Life Safety Performance Level (S-3), except that checklist statements using the Quick Check procedures of Section 4.5.3 shall be based on MS-factors and other limits that are an average of the values for Life Safety and Immediate Occupancy.

Per USGS Sds = 0.712, so the courthouse site is considered in a region with high level of seismicity.

Table 2-5. Level of Seismicity Definitions

Level of Seismicity[a]	S_{DS}	S_{D1}
Very low	<0.167 g	<0.067 g
Low	≥0.167 g <0.33 g	≥0.067 g <0.133 g
Moderate	≥0.33 g <0.50 g	≥0.133 g <0.20 g
High	≥ 0.50 g	≥0.20 g

[a]The higher level of seismicity defined by S_{DS} or S_{D1} shall govern.

C2.5 LEVEL OF SEISMICITY

The Levels of Seismicity in this standard have been adjusted to match the Seismic Design Categories in ASCE 7 as follows:

SDC A: Very Low
SDC B: Low
SDC C: Moderate
SDC D–F: High

Therefore, the parameters S_{DS} and S_{D1} correspond to the parameters at the BSE-1N level.

The courthouse does not meet the Benchmark Building Criteria

Table 4-6. Benchmark Buildings

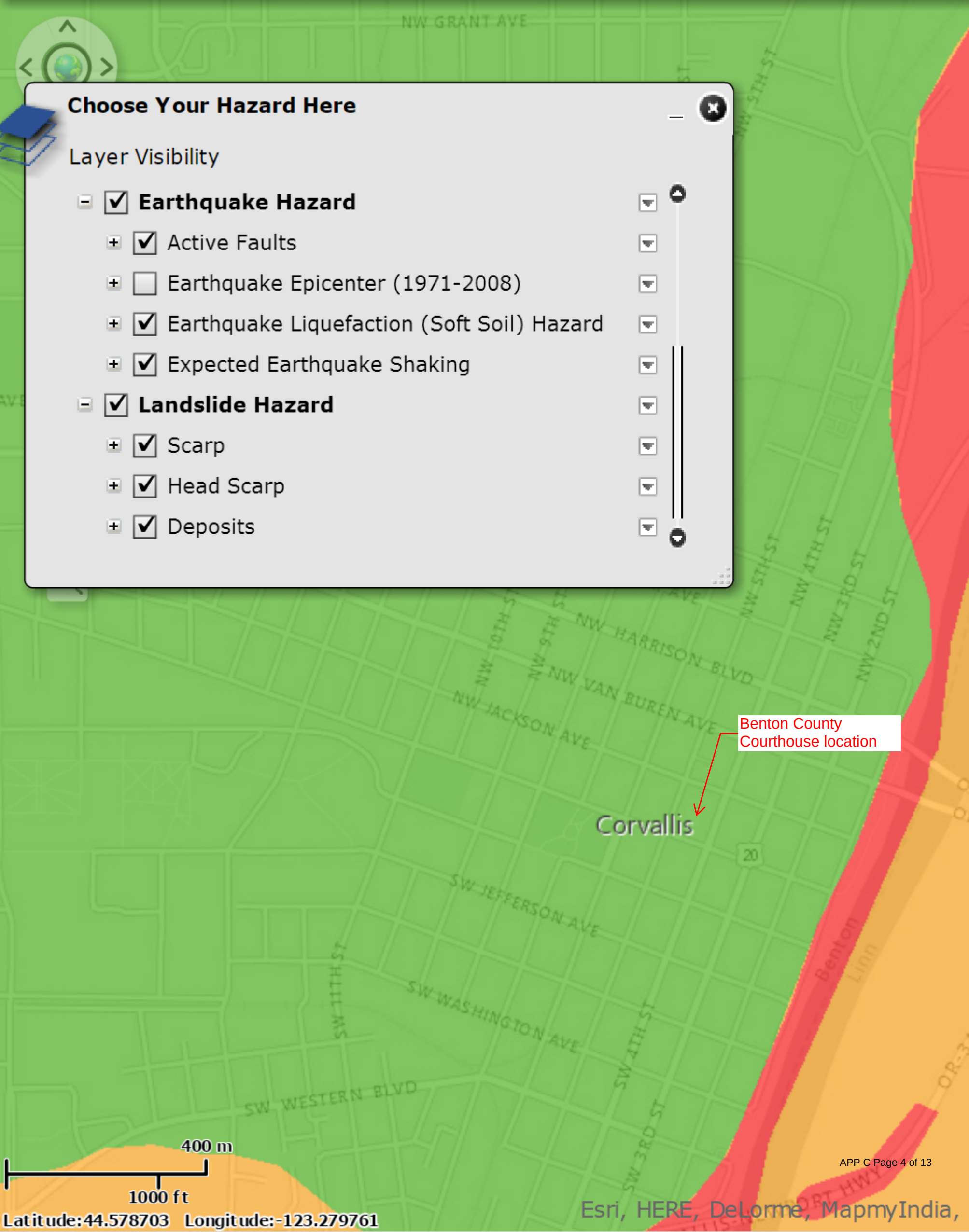
Building Type[a,b]	Building Seismic Design Provisions				Seismic Evaluation or Retrofit Provisions		
	NBC[LS] SBC[LS]	UBC[LS]	IBC[LS]	NEHRP[LS]	FEMA 178[LS]	FEMA 310 (1998e)/ ASCE 31[LS,IO]	FEMA 356 (2000)/ ASCE 41[LS,IO][c,d]
Wood frame, wood shear panels (Types W1 & W2)	1993	1976	2000	1985	[e]	1998	2000
Wood frame, wood shear panels (Type W1a)	[e]	1997	2000	1997	[e]	1998	2000
Steel moment-resisting frame (Types S1 & S1a)	[e]	1994[f]	2000	1997	[e]	1998	2000
Steel concentrically braced frame (Types S2 & S2a)	[e]	1997	2000	[e]	[e]	1998	2000
Steel eccentrically braced frame (Types S2 & S2a)	[e]	1988[f]	2000	1997	[e]	[e]	2000
Buckling-restrained braced frame (Types S2 & S2a)	[e]	[e]	2006	[e]	[e]	[e]	2000
Light metal frame (Type S3)	[e]	[e]	2000	[e]	1992	1998	2000
Steel frame w/ concrete shear walls (Type S4)	1993	1994?	2000	1985	[e]	1998	2000
Steel frame with URM infill (Types S5 & S5a)	[e]	[e]	2000	[e]	[e]	1998	2000
Steel plate shear wall (Type S6)	[e]	[e]	2006	[e]	[e]	[e]	2000
Reinforced concrete moment-resisting frame (Type C1)[g]	1993	1994	2000	1997	[e]	1998	2000
Reinforced concrete shear walls (Types C2 & C2a)	1993	1994	2000	1985	[e]	1998	2000
Concrete frame with URM infill (Types C3 & C3a)	[e]	[e]	2000	[e]	[e]	1998	2000
Tilt-up concrete (Types PC1 & PC1a)	[e]	1997	2000	[e]	[e]	1998	2000
Precast concrete frame (Types PC2 & PC2a)	[e]	[e]	2000	[e]	1992	1998	2000
Reinforced masonry (Type RM1)	[e]	1997	2000	[e]	[e]	1998	2000
Reinforced masonry (Type RM2)	1993	1994?	2000	1985	[e]	1998	2000
Unreinforced masonry (Type URM)[h]	[e]	1991[i]	2000	[e]	1992	1998	2000
Unreinforced masonry (Type URMa)	[e]	[e]	2000	[e]	[e]	1998	2000
Seismic isolation or passive dissipation	[e]	1991	2000	[e]	[e]	[e]	2000

[a]Building type refers to one of the common building types defined in Table 3-1.
[b]Buildings on hillside sites shall not be considered Benchmark Buildings.
[c]LS: S-3 Structural Performance Level for the BSE-1.
[d]IO: S-1 Structural Performance Level for the BSE-1.
[e]No benchmark year; buildings shall be evaluated using this standard.
[f]Steel moment-resisting frames and eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
[g]Flat slab concrete moment frames shall not be considered Benchmark Buildings.
[h]URM buildings evaluated or retrofitted and shown to be acceptable using Special Procedure (the ABK Methodology, 1984) may be considered benchmark buildings subject to the limitation of Section 15.2.
[i]Refers to the GSREB or its predecessor, the Uniform Code of Building Conservation (UCBC), or its successor, IEBC Appendix Chapter A1.
[LS]Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life Safety Performance Level may be considered Benchmark Buildings.
[IO]Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Immediate Occupancy Performance Level may be considered Benchmark Buildings.

Table 4-7. Checklists Required for a Tier 1 Screening

Level of Seismicity[b]	Level of Building Performance[c]	Required Checklists[a]					
		Very Low Seismicity Checklist (Sec 16.1.1)	Basic Configuration Checklist (Sec. 16.1.2)	Life Safety Checklist (Sec. 16.2LS through 16.15LS)	Immediate Occupancy Checklist (Sec. 16.2IO through 16.15IO)	Life Safety Nonstructural Checklist (Sec. 16.17)	Position Retention Nonstructural Checklist (Sec. 16.17)
Very low	LS	X					
Very low	IO		X		X		X
Low	LS		X	X		X	
Low	IO		X		X		X
Moderate	LS		X	X		X	
Moderate	IO		X		X		X
High	LS		X	X		X	
High	IO		X		X		X

[a]An X designates the checklist that must be completed for a Tier 1 screening as a function of the level of seismicity and level of performance.
[b]Defined in Section 2.5.
[c]LS = Life Safety Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).


Choose Your Hazard Here
Layer Visibility
Earthquake Hazard
Active Faults
Earthquake Epicenter (1971-2008)
Earthquake Liquefaction (Soft Soil) Hazard
Expected Earthquake Shaking
Landslide Hazard
Scarp
Head Scarp
Deposits
Benton County Courthouse location
Corvallis
NW GRANT AVE
NW HARRISON BLVD
NW VAN BUREN AVE
NW JACKSON AVE
SW JEFFERSON AVE
SW WASHINGTON AVE
SW WESTERN BLVD
SW 11TH ST
SW 4TH ST
SW 3RD ST
NW 5TH ST
NW 2ND
Benton
Linn
20
400 m
1000 ft
Latitude:44.578703 Longitude:-123.279761
Esri, HERE, DeLorme, MapmyIndia,

Project: Benton County Courthouse Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers Date: January 29, 2016

16.1.2IO IMMEDIATE OCCUPANCY BASIC CONFIGURATION CHECKLIST

Very Low Seismicity

Building System

General

C NC N/A U LOAD PATH: The structure shall contain a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

C NC N/A U ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement need not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

C NC N/A U MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

Building Configuration

C NC N/A U WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction shall not be less than 80% of the strength in the adjacent story above. (Commentary: Sec. A.2.2.2. Tier 2: Sec. 5.4.2.1)

C NC N/A U SOFT STORY: The stiffness of the seismic-force-resisting system in any story shall not be less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffnessof the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)

C NC N/A U VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)

C NC N/A U GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)

C NC N/A U MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

C NC N/A U TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Low Seismicity: Complete the Following Items in Additionto the Items for Very Low Seismicity.

Geologic Site Hazards Based on information from the DOGAMI Statewide Geohazard Online Map

C NC N/A U LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)

C NC N/A U SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

C NC N/A U SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

Moderate and High Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Foundation Configuration

C NC N/A U OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

C NC N/A U TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

Project: Benton County Courthouse Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers Date: January 29, 2016

16.16IO IMMEDIATE OCCUPANCY STRUCTURAL CHECKLIST FOR BUILDING TYPES URM: UNREINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS AND URMA: UNREINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

Very Low Seismicity

The following shear walls are NC: 3rd floor gridline 6; 2nd FL GL 3, 6, B & C; 1st FL GL 2, 3, 6, 7, & B - E; and basement GL 3, 6, B & C

Seismic-Force-Resisting System

C NC N/A U REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)

C NC N/A U SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 30 lb/in.^{2} for clay units and 70 lb/in.^{2} for concrete units. (Commentary: Sec. A.3.2.5.1. Tier 2: Sec. 5.5.3.1.1)

Connections

The masonry walls have anchors at 8' o.c. at the ceiling of the upper level; no anchors were observed at the other floors

C NC N/A U WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

C NC N/A U WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)

C NC N/A U TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)

C NC N/A U GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)

Foundation System

C NC N/A U DEEP FOUNDATIONS: Piles and piers are capable of transferring the lateral forces between the structure and the soil. (Commentary: Sec. A.6.2.3.)

C NC N/A U SLOPING SITES: The difference in foundation embedment depth from one side of the building to another shall not exceed one story high. (Commentary: Sec. A.6.2.4)

Low, Moderate, and High Seismicity: Complete the Following Items in Addition to the Items for Very Low Seismicity.

Seismic-Force-Resisting System

The bell tower lounge walls are NC

C NC N/A U PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following (Commentary: Sec. A.3.2.5.2. Tier 2: Sec. 5.5.3.1.2):

Top story of multi-story building	9
First story of multi-story building	15
All other conditions	13

C NC N/A U MASONRY LAYUP: Filled collar joints of multi-wythe masonry walls have negligible voids. (Commentary: Sec. A.3.2.5.3. Tier 2: Sec. 5.5.3.4.1)

Diaphragms (Stiff or Flexible)

Openings are > 4 feet at: 4th GL6; 3rd GL A, E & 6; 2nd GL A, 3 & 6; and 1st GL 3 & 6.

C NC N/A U OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 15% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)

C NC N/A U OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are be greater than 4 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)

C NC N/A U PLAN IRREGULARITIES: There is tensile capacity to develop the strength of the diaphragm at reentrant corners or other locations of plan irregularities. (Commentary: Sec. A.4.1.7. Tier 2: Sec. 5.6.1.4)

There is not a connection from the core to the west or east wings at Gridlines 2 and 7 of the building

C NC N/A U DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension. (Commentary: Sec. A.4.1.8. Tier 2: Sec. 5.6.1.5)

Flexible Diaphragms

The west roof diaphragm has a 2.75 : 1 ratio in the E-W direction

The west roof diaphragm has straight sheathing with 71.5 span in the E-W direction

C NC N/A U CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) Cross ties were not observed

C NC N/A U STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 1-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

C NC N/A U SPANS: All wood diaphragms with spans greater than 12 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

C NC N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 30 ft and aspect ratios less than or equal to 3-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

C NC N/A U NONCONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete shall consist of horizontal spans of less than 40 ft and have aspect ratios less than 4-to-1. (Commentary: Sec. A.4.3.1. and Tier 2: Sec. 5.6.3)

C NC N/A U OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

The core floor diaphragm span is 51.5' x 65.5' with a 1.27 : 1 ratio in the N-S direction

Connections

The (E) anchors are installed tight, but the conn. from the joist to the diaphragm is unknown

C NC N/A U STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)

C NC N/A U BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads. (Commentary: Sec. A.5.4.5. Tier 2: Sec. 5.7.4.4)

The trusses do not have secondary supports and there is not a Tier 2 path to evaluate the load path

Project: Benton County Courthouse Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers Date: January 29, 2016

16.17 NONSTRUCTURAL CHECKLIST

Life Safety Systems

One transverse brace was observed in Storage Room #10, but does not meet NFPA-13, Sect. 9.3.5.1 bracing requirements

C NC N/A U LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)

C NC N/A U LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)

C NC N/A U LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4) Observed 1" clearance per NFPA-13, Sect. 9.3.4.2

C NC N/A U LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)

Hazardous Materials

The natural gas line does not have seismic bracing or flexible couplings

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)

C NC N/A U LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)

Partitions

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)

Ceilings

The Category C suspended gypsum ceilings are not braced at 144 ft2 and do not have a sufficient gap to allow for relative displacement in accordance with 13.6.4.3

C NC N/A U LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 ft^2 and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)

Light Fixtures

The Courtroom #3 lights are not independently supported

C NC N/A U LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)

C NC N/A U LS-not required; PR-H. PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft and, if rigidly supported, are free to move with the structure to which they are attached without damaging adjoining components. (Commentary: A.7.3.3. Tier 2: Sec. 13.7.9)

C NC N/A U LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)

The lens covers do not appear to be attached with safety devices

Cladding and Glazing

C NC N/A U LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. CLADDING ISOLATION: For steel or concrete moment frame buildings, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02. (Commentary: Sec. A.7.4.3. Tier 2: Section 13.6.1)

The glazing for the original and renovated skylights are laminated wire glass annealed material

C NC N/A U LS-MH; PR-MH. MULTI-STORY PANELS: For multi-story panels attached at more than one floor level, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicty, 0.02. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 ft^2 in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)

Masonry Veneer

C NC N/A U LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. STUD TRACKS: For veneer with metal stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)

C NC N/A U LS-not required; PR-MH. OPENINGS: For veneer with metal stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)

Parapets, Cornices, Ornamentation, and Appendages

C NC N/A U LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)

C NC N/A U LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)

C NC N/A U LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)

C NC N/A U LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)

The statue and sloped cornices at the east wall elevation are anchored back to the structure less than 6' o.c.; it is unknown whether they are reinforced.

Masonry Chimneys

The west chimney h/w = 2.1; the URM chimneys are built integrally with the exterior walls

C NC N/A U LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)

C NC N/A U LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)

Stairs

Each of the exterior walls at the stairway openings is non-compliant

C NC N/A U LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)

C NC N/A U LS-LMH; PR-LMH. STAIR DETAILS: In moment frame structures, the connection between the stairs and the structure does not rely on shallow anchors in concrete. Alternatively, the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.5.3.1 without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: 13.6.8)

Contents and Furnishings

Non-compliant contents are located on the 1st and 3rd Floors with aspect ratios as high as 8.4 : 1

C NC N/A U LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)

C NC N/A U LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)

C NC N/A U LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)

C NC N/A U LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)

C NC N/A U LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)

C NC N/A U LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)

Mechanical and Electrical Equipment

The suspended HVAC equipment in the boiler room, storage room 10, storage room 02 and vault room 105 are not seismically

C NC N/A U LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)

C NC N/A U LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)

C NC N/A U LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)

C NC N/A U LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)

C NC N/A U LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)

The boiler in the boiler room is not anchored to the concrete slab on grade

Piping

Piping was not observed, other than the fire suppression or gas piping

C NC N/A U LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)

Ducts

C NC N/A U LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)

Elevators

C NC N/A U LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)

C NC N/A U LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)

C	NC	N/A	U	LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)

APPENDIX - D

Building Code: 2014 Oregon Structural Specialty Code

Soils Report: No **Soils Report by:** N/A **Dated:** N/A

Soil Bearing: 1500 PSF **Retaining Walls:** No

Equivalent Fluid Pressure (active): N/A PCF **Passive bearing:** N/A PCF **Friction:** N/A

Structural System: Building Structure

Vertical System: Wood framed roof / masonry walls **Lateral Sys:** Flexible Diaphragm / Concrete or Masonry Shearwalls

Basic Design Loads:

Element	Roof	Floor	Corridor	
Load Type	Dead	Dead	Dead	
Value (PSF)	20	20	20	
Load Type	Snow	Floor Live	Corridor Live	
Value (PSF)	25	50	100	
Deflection Criteria	L/240	L/360	L/360	

Lateral Design Parameters:

Wind Design: Other: ASCE 41-13 **Wind Speed (3 sec Gust):** 120 MPH

Exposure B

Importance Factors I_W = (ice) I_E = (seismic) I_S = (snow) I_i = (ice) **Risk Cat:** II

Seismic Design

Seismic design parameters are based on published values from the USGS web site.

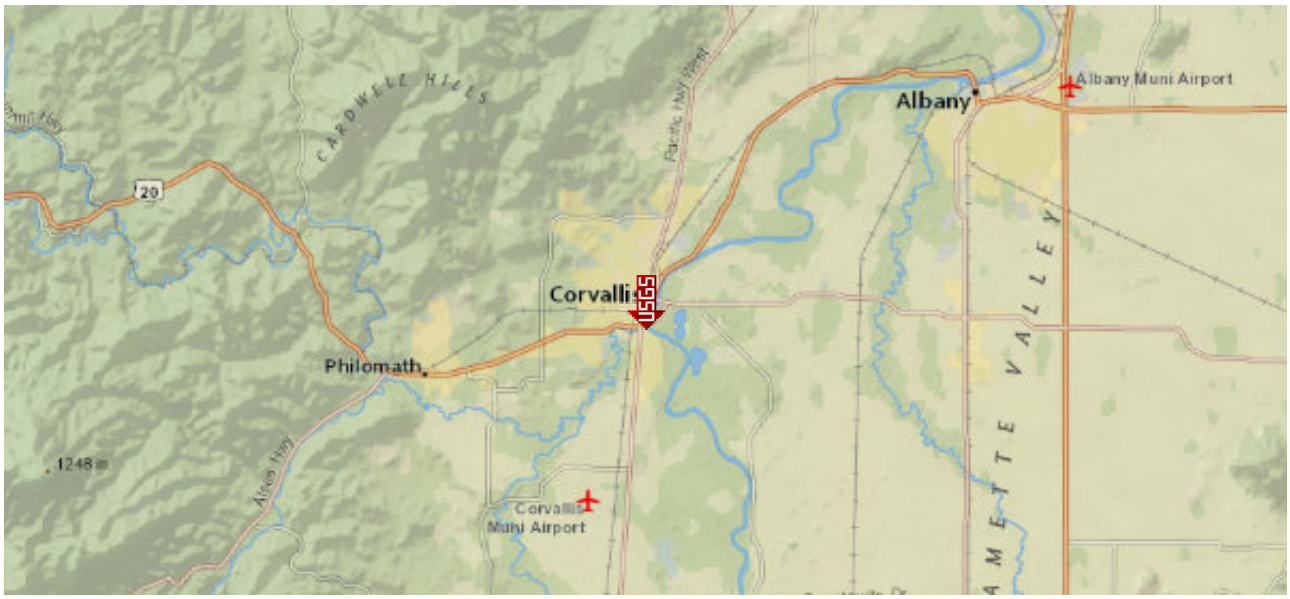
Design Summary:

The following calculations are the Tier I and II analysis of the courthouse building based on the ASCE 41-13 standard for evaluating an existing building considering seismic loading.

USGS Design Maps Summary Report

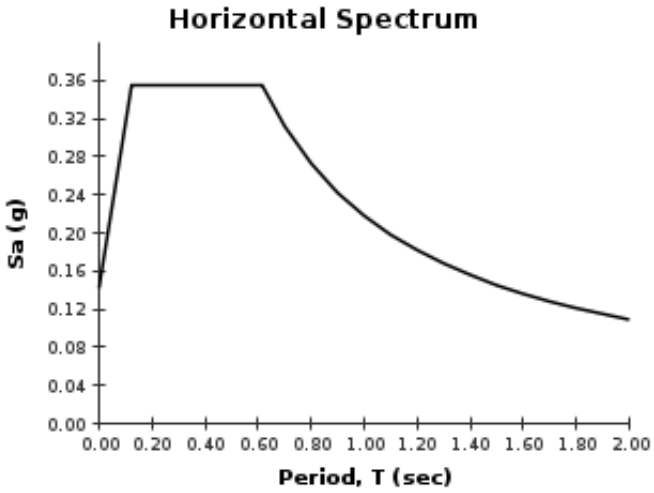
User–Specified Input

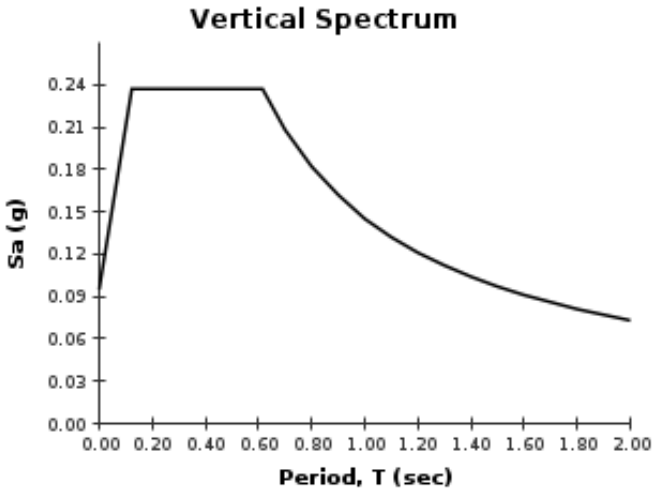
Report Title	Benton County Courthouse Thu December 10, 2015 17:17:07 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-1E (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

$S_{S,20/50}$	0.222 g	$S_{XS,BSE-1E}$	0.355 g
$S_{1,20/50}$	0.091 g	$S_{X1,BSE-1E}$	0.218 g





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USGS Design Maps Summary Report

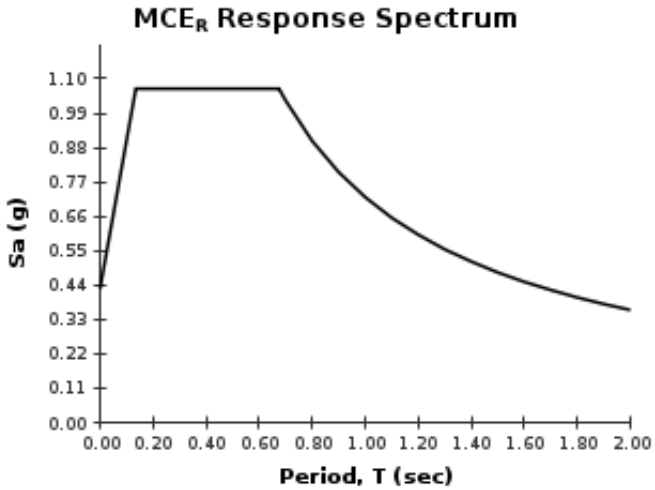
User–Specified Input

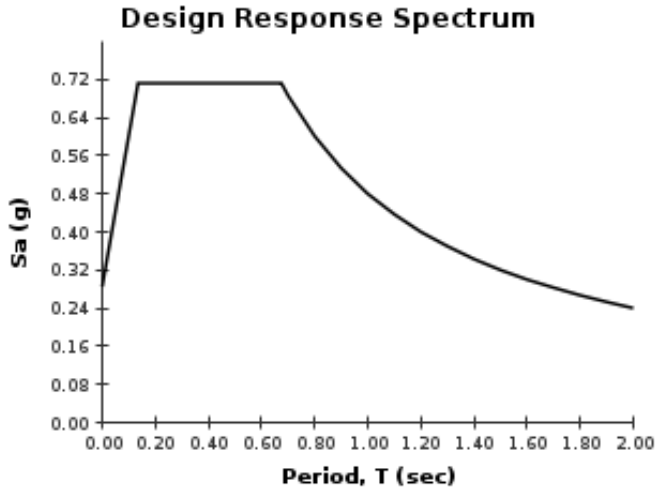
Report Title	Benton County Couthouse Wed November 18, 2015 23:04:50 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.956 g	S_{MS} =	1.068 g	S_{DS} =	0.712 g
S_1 =	0.472 g	S_{M1} =	0.721 g	S_{D1} =	0.480 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





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Benton County Courthouse EPOE Calculations:

Building Weights:

Roof Weights =	20	psf
Wood Floor =	20	psf
Light Framed =	8	psf
8" CMU =	80	psf
12" brick =	120	psf
18" brick =	180	psf
20" brick =	200	psf
30" stone =	350	psf

Building Geometries:

West =	1752	ft^2
Center =	3373	ft^2
East =	1752	ft^2
Total sq footage =	6877	ft^2
Total width =	114.50	ft
Total length =	71.50	ft
Total height =	59.50	ft

Roofs:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

Partial Floors:

Upper Foyer=	14513	lb
Speaker Floor=	9138	lb
Bell Floor=	4102	lb
Top Floor=	995	lb
Sub Total =	28747	lb

Floors:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

East Section

Width =	24.50	ft
Length =	71.50	ft
Bell Tower Lounge Height =	13.67	ft
3rd Floor Height =	14.25	ft
2nd Floor Height =	15.00	ft
1st Floor Height =	16.50	ft
Basement Floor Height =	10.50	ft

Center Section

Width =	65.50	ft
Length =	51.50	ft
4th Floor Height =	8.50	ft
3rd Floor Height =	9.00	ft

West Section

Width =	24.50	ft
Length =	71.50	ft

4th East-West Walls: (starting at north wall)

Gridline 8 =	11760	lb
GL 7 =	2180	lb
GL 6 =	42445	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	42445	lb
GL 2 =	2180	lb
GL 1 =	11760	lb
Sub Total =	112770	lb

4th North-South Walls: (starting at west wall)

Gridline A =	17160	lb
GL B =	17160	lb
GL C =	38451	lb
GL D =	12000	lb
GL E =	26451	lb
Sub Total =	111223	lb
Total =	**375763**	**lb**

3rd East-West Walls: (starting at north wall)

Gridline 8 =	41895	lb
GL 7 =	35370	lb
GL 6 =	86692	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	86692	lb
GL 2 =	35370	lb
GL 1 =	41895	lb
Sub Total =	327913	lb

3rd North-South Walls: (starting at west wall)

Gridline A =	61133	lb
GL B =	61133	lb
GL C =	96775	lb
GL D =	42750	lb
GL E =	54025	lb
Sub Total =	315815	lb
Total =	**795776**	**lb**

2nd East-West Walls: (starting at north wall)

Gridline 8 =	128993	lb
GL 7 =	172429	lb
GL 6 =	90821	lb

2nd North-South Walls: (starting at west wall)

Gridline A =	188224	lb
GL B =	188224	lb
GL C =	188224	lb

GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	90821	lb
GL 2 =	172429	lb
GL 1 =	128993	lb
Sub Total =	784485	lb

GL D =	131625	lb
GL E =	56599	lb
Sub Total =	752895	lb
Total =	**1674915**	**lb**

1st East-West Walls: (starting at north wall)

Gridline 8 =	154350	lb
GL 7 =	206325	lb
GL 6 =	108675	lb
GL 5 =	108075	lb
GL 4 =	108075	lb
GL 3 =	108675	lb
GL 2 =	206325	lb
GL 1 =	154350	lb
Sub Total =	1154850	lb

1st North-South Walls: (starting at west wall)

Gridline A =	225225	lb
GL B =	225225	lb
GL C =	225225	lb
GL D =	157500	lb
GL E =	67725	lb
Sub Total =	900900	lb
Total =	**2193285**	**lb**

Basement East-West Walls: (starting at north wall)

Gridline 8 =	162803	lb
GL 7 =	217624	lb
GL 6 =	114626	lb
GL 5 =	159165	lb
GL 4 =	159165	lb
GL 3 =	114626	lb
GL 2 =	217624	lb
GL 1 =	162803	lb
Sub Total =	1308435	lb

Basement North-South Walls: (starting at west wall)

Gridline A =	237559	lb
GL B =	191595	lb
GL C =	191595	lb
GL D =	166125	lb
GL E =	71434	lb
Sub Total =	858308	lb
Total =	**2304278**	**lb**

4.5.2.1 - Pseudo Seismic Force - for use with quick check proceedures

S_s=	22.20%	
S_1 =	9.10%	
F_a =	1.60	Table 2-3, page 46
F_v =	2.40	Table 2-4, page 46
S_{XS} =	0.36	eqn. 2-1, pg. 46
S_{X1} =	0.22	eqn. 2-2, pg. 46
C_t =	0.02	eqn. 4-5, pg. 69
h_n =	59.50	ft (roof height)
β =	0.75	eqn. 4-5, pg. 69
T =	0.428	eqn. 4-5, pg. 69
Sa =	0.36	eqn. 4-4, pg. 68

Evaluation: Immediate Occupancy

Immediate Occupancy Responses:	
S_s=	22.20%
S_1 =	9.10%
Life Safety:	
S_s=	22.20%
S_1 =	9.10%
Collapse Prevention:	
S_s=	68.50%
S_1 =	33.40%

Total Diaphragm Weight

C =	1	table 4-8, pg. 68
W_1 =	2304278	lb (lowest floor)
W_2 =	2193285	lb
W_3 =	1674915	lb
W_4 =	795776	lb
W_5 =	375763	lb

Heights of floors to base:

h_1 =	10.50	ft
h_2 =	27.00	ft
h_3 =	42.00	ft
h_4 =	51.00	ft
h_5 =	59.50	ft

W_{total} =	7344016	lb
V_{total} =	2608595	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

k =	1.00		
F_1 =	291251	lb	0.36
F_2 =	712855	lb	0.92
F_3 =	846808	lb	1.42
F_4 =	488544	lb	1.73
F_5 =	269137	lb	2.02
Σ =	2608595	lb	

7.4.1.3.1 - Pseudo Seismic Force for LSP (linear static proceedure)

Sa =	0.4	
C_1C_2 =	1	from Table 7-3
C_m =	1	from Table 7-4
V =	0.36	*W

Diaphragm Weights East-West Walls:

W_1 =	995843	lb (lowest floor)
W_2 =	1038435	lb
W_3 =	890430	lb
W_4 =	467863	lb
W_5 =	262992	lb
W_{total} =	3655563	lb
V_{total} =	1298456	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	117651	lb
F_2 =	315471	lb
F_3 =	420791	lb
F_4 =	268476	lb
F_5 =	176067	lb
Σ =	1298456	lb

Diaphragm Weights North-South Walls:

W_1 =	1445970	lb (lowest floor)
W_2 =	1292385	lb
W_3 =	922020	lb
W_4 =	479961	lb
W_5 =	264540	lb
W_{total} =	4404875	lb
V_{total} =	1564612	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	184119	lb
F_2 =	423160	lb
F_3 =	469612	lb
F_4 =	296842	lb
F_5 =	190879	lb
Σ =	1564612	lb

URM Shear Wall Loads/Diaphragm Loads for Immediate Occupancy Evaluation:

Ms =	1.0	URM Wall, from Table 4-9, pg. 70
M_1 =	1.0	Straight Diaphragm (chorder), from Table 12-3
M_2 =	1.0	Straight Diaphragm (unchorder), from Table 12-4
M_3 =	1.5	Wood Diaphragm (chorder), from Table 12-3
M_4 =	1.3	Wood Diaphragm (unchorded), from Table 12-3

4th Floor / Roof	Shear (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	16489	49	337	2	30	OK	251	120	NG
GL 7 =	31555	66	482	3	30	OK	470	120	NG
GL 6 =	60017	20	3078	14	30	OK	300	450	OK
GL 5 =	0	0							
GL 4 =	0	0							
GL 3 =	60017	20	3078	14	30	OK	300	450	OK
GL 2 =	31555	66	482	3	30	OK	470	120	NG
GL 1 =	16489	49	337	2	30	OK	251	120	NG
Gridline A =	24876	72	348	2	30	OK	263	120	NG
GL B =	75088	72	1050	7	30	OK	975	120	NG
GL C =	82650	72	1156	6	30	OK	661	120	NG

GL D =	30709	50	614	4	30	OK	263	120	NG
GL E =	17061	15	1177	5	30	OK	285	300	OK
	216123	230385	268	279					

3rd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	33656	50145	38	1320	9	30	OK	307	300	NG
GL 7 =	59500	91055	66	1390	10	30	OK	478	450	NG
GL 6 =	99320	159337	24	6639	31	30	NG	368	450	OK
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	99320	159337	32	4928	23	30	OK	368	450	OK
GL 2 =	59500	91055	66	1390	10	30	OK	478	450	NG
GL 1 =	33656	50145	38	1320	9	30	OK	307	300	NG
Gridline A =	50922	75798	58	1307	9	30	OK	272	450	OK
GL B =	129007	204094	67	3031	21	30	OK	1011	450	NG
GL C =	141667	224317	62	3647	17	30	OK	856	450	NG
GL D =	56314	87023	39	2260	16	30	OK	272	450	OK
GL E =	31111	48172	16	3011	14	30	OK	370	450	OK
	384951	409020	263	241						

2nd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	75244	125389	38	3300	15	30	OK	480	300	NG
GL 7 =	134811	225866	47	4789	22	30	OK	749	450	NG
GL 6 =	168651	327988	33	10039	46	30	NG	577	450	NG
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	168651	327988	33	10039	46	30	NG	577	450	NG
GL 2 =	134811	225866	47	4789	22	30	OK	749	450	NG
GL 1 =	75244	125389	38	3300	15	30	OK	480	300	NG
Gridline A =	113064	188862	55	3434	16	30	OK	431	450	OK
GL B =	236596	440690	63	7032	33	30	NG	1599	450	NG
GL C =	236596	460913	59	7845	36	30	NG	1355	450	NG
GL D =	111820	198843	39	5099	24	30	OK	431	450	OK
GL E =	38964	87136	16	5446	25	30	OK	585	450	NG
	757414	737040	236	231						

1st Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	76886	202275	36	5580	23	30	OK	360	300	NG
GL 7 =	128439	354305	47	7538	31	30	NG	411	450	OK
GL 6 =	84929	412917	29	14078	59	30	NG	309	450	OK
GL 5 =	72583	72583	80	909	4	30	OK	253	450	OK
GL 4 =	72583	72583	81	892	4	30	OK	253	450	OK
GL 3 =	84929	412917	33	12639	53	30	NG	309	450	OK
GL 2 =	128439	354305	47	7538	31	30	NG	411	450	OK
GL 1 =	76886	202275	36	5580	23	30	OK	360	300	NG
Gridline A =	121636	310498	50	6210	29	30	OK	388	450	OK
GL B =	232949	673640	62	10808	50	30	NG	1441	450	NG
GL C =	232949	693863	62	11191	52	30	NG	1221	450	NG
GL D =	114575	313418	36	8627	40	30	NG	388	450	OK
GL E =	41050	128187	16	7850	36	30	NG	527	450	NG
	725674	743160	390	227						

Basement Floor	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	66055	268330	36	7402	21	30	OK	134	300	OK
GL 7 =	97868	452174	59	7642	21	30	OK	209	450	OK
GL 6 =	57993	470910	24	19355	54	30	NG	334	450	OK
GL 5 =	69288	141871	78	1811	5	30	OK	94	450	OK
GL 4 =	69288	141871	69	2049	6	30	OK	94	450	OK
GL 3 =	57993	470910	29	16425	46	30	NG	334	450	OK
GL 2 =	97868	452174	62	7323	20	30	OK	209	450	OK
GL 1 =	66055	268330	36	7402	21	30	OK	134	300	OK
Gridline A =	102497	412995	60	6902	19	30	OK	169	450	OK
GL B =	134603	808243	63	12829	36	30	NG	627	450	NG
GL C =	134603	828466	67	12334	34	30	NG	531	450	NG
GL D =	84518	397936	39	10118	28	30	OK	169	450	OK
GL E =	32768	160954	16	10060	28	30	OK	229	450	OK
	582407	488990	394	245						

Foundation Design	M_{OT} (ft*lb)	M_R (ft*lb)	Uplift (lb)	250k MP
Gridline 8 =	4246573.8	3589560	25269.76136	0.1
GL 7 =	14843486	21075390	-239688.6043	-1.0
GL 6 =	19537828	8106480	439667.2159	1.8
GL 5 =	2616319.5	10076625	-286934.827	-1.1
GL 4 =	2616319.5	10076625	-286934.827	-1.1
GL 3 =	19537828	8106480	439667.2159	1.8
GL 2 =	14843486	21075390	-239688.6043	-1.0
GL 1 =	4246573.8	3589560	25269.76136	0.1
				0.0
Gridline A =	12979732	27146047.5	-544858.2894	-2.2
GL B =	14141469	4163040	356372.4571	1.4
GL C =	14684241	4163040	375757.192	1.5
GL D =	6588812.2	4163040	86634.72319	0.3
GL E =	5952716.2	4228380	66320.62431	0.3

Section 16.1.2 Basic Configuration - Immediate Occupancy (Tier 1 evaluations)

Immediate Occupancy

Adjacent Building, Sect. 5.4.1.1

Taller Building =	59.50	ft
Shorter Building =	10	ft
Clear Distance =	4.80	inch

Weak Story, Sect. 5.4.2.1

Basement SW (NS) =	245.33	ft	108% of story above
1st Floor SW (NS) =	226.99	ft	98% of story above
2nd Floor SW (NS) =	231.42	ft	96% of story above
3rd Floor SW (NS) =	241.33	ft	86% of story above
4th Floor SW (NS) =	279.00	ft	
Basement SW (EW) =	394.00	ft	101% of story above
1st Floor SW (EW) =	389.66	ft	165% of story above
2nd Floor SW (EW) =	235.67	ft	89% of story above
3rd Floor SW (EW) =	263.33	ft	98% of story above
4th Floor SW (EW) =	268.00	ft	

Change in Building Mass, Sect. 5.4.2.5

Basement Mass =	2304278	lbs	105% of story above
1st Floor Mass =	2193285	lbs	131% of story above
2nd Floor Mass =	1674915	lbs	210% of story above
3rd Floor Mass =	795776	lbs	212% of story above
4th Floor Mass =	375763	lbs	

Foundation - Overturning, Sect. 5.4.3.3

Basement SW (NS) =	71.5	ft	
b/h Ratio (NS) =	1.20		
Basement SW (EW) =	113.5	ft	
b/h Ratio (EW) =	1.91		
0.6 x Sa =	0.21		**< MIN b/h ratio of1.2 OK**

Section 16.16 URM Bearing Walls - Immediate Occupancy Evaluation

Wall Shear Stress Check - Shear Stress in Shear Walls, Sect. 4.5.3.3

Tier 1 Quick Check - Shear Wall loading see page

Tier 2 - Shear Wall Capacities - Sect. 7.5.1.5

m =	1	from Table 11-3 for Immediate Occupancy
K =	0.75	from 11.2.4
F_m =	600	PSI, from Table 11-2(a)
F_t =	45	PSI, from Table 11-2(a)
F_v =	78.38367	PSI per shear area, 80% of TMS 402, Section 3.2.4
E_m =	420000	PSI, per TMS 402, Section 1.8.2.2
G_m =	168000	PSI, per TMS 402, Section 1.8.2.2

Check 3rd Floor Gridline 6:

Total Wall Load=	159337	lb
Wall height =	14.25	ft
Pier height =	8.00	ft
Pier length =	24	ft
Pier thickness =	1.33	ft
L / h_{eff} =	1.68	for (11-12)
Trib DL P_D =	48969.6	lb
Pier SW P_W =	61560	lb
Wall Rigidity =	569140	
Load to Pier =	159336.6	6639.026
Shear Stress=	35	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1
LB Bed-Joint V_{mL} =	43	psi, 11.2.3.6.3 (11-6)
In-plane V_{bjs1} =	196853	lb, 11.3.2.2.2 (11-9)
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)
Rocking V_r =	120884	lb, 11.3.2.2.1 (11-8)
Lower Bound V_{tc} =	126625	lb, 11.3.2.2.3 (11-11)
Lower Bound V_{dt} =	214008	lb, 11.3.2.2.4 (11-12)
Lower Bound P_{CL} =	1875364	lb, 11.3.2.2.5 (11-13)
Controlling Q_{CE} =	120884	lb
Allowable Stress=	20	psi **NG**
DCR =	1.76	2861.414

Check 2nd Floor Gridline 3:

Total Wall Load=	327988	lb				
Wall height =	15.00	ft	15.00	ft		
Pier height =	8.00	ft	8.00	ft		
Pier length =	11.5	ft	21.25	ft		
Pier thickness =	1.5	ft	1.5	ft		
L / h_{eff} =	0.77	for (11-12)	1.42			
Trib DL P_D =	55262.1	lb	102114.8	lb		
Pier SW P_W =	31050	lb	57375	lb		
Wall Rigidity =	204166		545641			
Load to Pier =	89308.3	7765.939	238679.6	11231.98	327987.9	
Shear Stress=	36	psi	52	psi		
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0			
LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)	49	psi		
In-plane V_{bjs1} =	120781	lb, 11.3.2.2.2 (11-9)	223182	lb		
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)	N/A	lb		
Rocking V_r =	48843	lb, 11.3.2.2.1 (11-8)	166773	lb		
Lower Bound V_{tc} =	49780	lb, 11.3.2.2.3 (11-11)	169973	lb		
Lower Bound V_{dt} =	137595	lb, 11.3.2.2.4 (11-12)	292242	lb		
Lower Bound P_{CL} =	1013472	lb, 11.3.2.2.5 (11-13)	1872720	lb		
Controlling Q_{CE} =	48843	lb	166773	lb		
Allowable Stress=	15	psi **NG**	27	psi	**NG**	

DCR =	2.44	4580.52	DCR =	1.91	5345.88

Check 2nd Floor Gridline 6:

Total Wall Load=	327988	lb				
Wall height =	15.00	ft	15.00	ft		
Pier height =	8.00	ft	8.00	ft		
Pier length =	9.5	ft	23.33	ft		
Pier thickness =	1.5	ft	1.5	ft		
L / h_{eff} =	0.63	for (11-12)	1.56			
Trib DL P_D =	45651.3	lb	112110	lb		
Pier SW P_W =	25650	lb	62991	lb		
Wall Rigidity =	140188		618528			
Load to Pier =	60602.51	6379.212	267385.4	11461.01	327987.9	
Shear Stress=	30	psi	53	psi		
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0			
LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)	49	psi		
In-plane V_{bjs1} =	99776	lb, 11.3.2.2.2 (11-9)	245028	lb		
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)	N/A	lb		
Rocking V_r =	33331	lb, 11.3.2.2.1 (11-8)	201019	lb		
Lower Bound V_{tc} =	33971	lb, 11.3.2.2.3 (11-11)	204876	lb		
Lower Bound V_{dt} =	113665	lb, 11.3.2.2.4 (11-12)	320848	lb		
Lower Bound P_{CL} =	837216	lb, 11.3.2.2.5 (11-13)	2056026	lb		
Controlling Q_{CE} =	33331	lb	201019	lb		
Allowable Stress=	12	psi **NG**	30	psi **NG**		
DCR =	2.42	3747.779	DCR = 1.77	4998.764		

Check 2nd Floor Gridline B:

Total Wall Load=	440690	lb						
Wall height =	15.00	ft	15.00	ft		15.00	ft	
Pier height =	8.00	ft	8.00	ft		8.00	ft	
Pier length =	17.833	ft	18.75	ft		24	ft	
Pier thickness =	1.33	ft	1.33	ft		1.33	ft	
L / h_{eff} =	1.19	for (11-12)	1.25			1.60		
Trib DL P_D =	30672.76	lb	32250	lb		41280	lb	
Pier SW P_W =	48149.1	lb	50625	lb		64800	lb	
Wall Rigidity =	376760		405560			569140		
Load to Pier =	122855.8	6889.237	132246.8	7053.165		185587.8	7732.824	440690.4
Shear Stress=	36	psi	37	psi		40	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0			1.0		
LB Bed-Joint V_{mL} =	42	psi, 11.2.3.6.3 (11-6)	42	psi		42	psi	
In-plane V_{bjs1} =	143413	lb, 11.3.2.2.2 (11-9)	150788	lb		193008	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)	N/A	lb		N/A	lb	
Rocking V_r =	58579	lb, 11.3.2.2.1 (11-8)	64758	lb		106099	lb	
Lower Bound V_{tc} =	61511	lb, 11.3.2.2.3 (11-11)	67999	lb		111410	lb	
Lower Bound V_{dt} =	132109	lb, 11.3.2.2.4 (11-12)	170812	lb		240263	lb	
Lower Bound P_{CL} =	1393473	lb, 11.3.2.2.5 (11-13)	1465128	lb		1875364	lb	
Controlling Q_{CE} =	58579	lb	64758	lb		106099	lb	
Allowable Stress=	13	psi **NG**	14	psi **NG**		17	psi **NG**	
DCR =	2.80	4425.609	DCR = 2.72	4462.852	DCR =	2.33	4417.224	

Check 2nd Floor Gridline C:

Total Wall Load=	460913	lb						
Wall height =	15.00	ft	15.00	ft		15.00	ft	
Pier height =	8.00	ft	8.00	ft		8.00	ft	
Pier length =	16.33	ft	19	ft		19	ft	
Pier thickness =	1.5	ft	1.33	ft		1.33	ft	
L / h_{eff} =	1.09	for (11-12)	1.27			1.27		
Trib DL P_D =	28087.6	lb	32680	lb		32680	lb	
Pier SW P_W =	44091	lb	51300	lb		51300	lb	
Wall Rigidity =	371674		413405			413405		
Load to Pier =	142938.4	8753.117	158987.5	8367.763		158987.5	8367.763	460913.4
Shear Stress=	41	psi	44	psi		44	psi	

Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0					
LB Bed-Joint V_{mL} =	41	psi, 11.2.3.6.3 (11-6)		42	psi		42	psi				
In-plane V_{bjs1} =	146317	lb, 11.3.2.2.2 (11-9)		152798	lb		152798	lb				
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb				
Rocking V_r =	49120	lb, 11.3.2.2.1 (11-8)		66496	lb		66496	lb				
Lower Bound V_{tc} =	51919	lb, 11.3.2.2.3 (11-11)		69825	lb		69825	lb				
Lower Bound V_{dt} =	132312	lb, 11.3.2.2.4 (11-12)		173090	lb		190208	lb				
Lower Bound P_{CL} =	1439130	lb, 11.3.2.2.5 (11-13)		1484663	lb		1484663	lb				
Controlling Q_{CE} =	49120	lb		66496	lb		66496	lb				
Allowable Stress=	10	psi	**NG**	14	psi	**NG**	14	psi	**NG**			
DCR =	3.88	6497.127	DCR =	3.19	5742.913	DCR =	3.19	5742.913				

Check 1st Floor Gridline 2 (& 7):

Total Wall Load=	354305	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier length =	7.167	ft	(3) at	7.5	ft		8.5	ft		8.83	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.86	ft	
L / h_{eff} =	0.43	for (11-12)		0.45			0.52			0.54		
Trib DL P_D =	32394.84	lb		33900	lb		38420	lb		39911.6	lb	
Pier SW P_W =	21285.99	lb		22275	lb		25245	lb		26225.1	lb	
Wall Rigidity =	53547		(3) at	59977	lb for (3) piers		81295			90403		
Load to Pier =	46824.17	6533.302		157340.9	6992.931		71087.6	8363.247		79052.68	8952.738	
Shear Stress=	25	psi		27	psi / pier		32	psi		33	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	87022	lb, 11.3.2.2.2 (11-9)		91065	lb		103207	lb		108644	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb		N/A	lb	
Rocking V_r =	16825	lb, 11.3.2.2.1 (11-8)		18424	lb		23665	lb		25538	lb	
Lower Bound V_{tc} =	17429	lb, 11.3.2.2.3 (11-11)		19086	lb		24515	lb		26487	lb	
Lower Bound V_{dt} =	96268	lb, 11.3.2.2.4 (11-12)		115795	lb		131234	lb		137800	lb	
Lower Bound P_{CL} =	770568	lb, 11.3.2.2.5 (11-13)		806371	lb		913887	lb		964931	lb	
Controlling Q_{CE} =	16825	lb		18424	lb		23665	lb		25538	lb	
Allowable Stress=	7	psi	**NG**	7	psi	**NG**	8	psi	**NG**	8	psi	
DCR =	3.71	4772.663	DCR =	3.80	5150.488	DCR =	4.01	6275.145	DCR =	4.13	6783.568	
											NG	

Check 1st Floor Gridline 3:

Total Wall Load=	412917	lb			
Wall height =	16.50	ft	16.50	ft	
Pier height =	10.00	ft	10.00	ft	
Pier length =	11.5	ft	21.25	ft	
Pier thickness =	1.83	ft	1.83	ft	
L / h_{eff} =	0.70	for (11-12)	1.29		
Trib DL P_D =	88612.1	lb	163739.8	lb	
Pier SW P_W =	34155	lb	63112.5	lb	
Wall Rigidity =	159992		482388		
Load to Pier =	102841.9	8942.776	310075.3	14591.78	412917.2
Shear Stress=	34	psi	55	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)	52	psi	
In-plane V_{bjs1} =	157949	lb, 11.3.2.2.2 (11-9)	291862	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)	N/A	lb	
Rocking V_r =	66296	lb, 11.3.2.2.1 (11-8)	226366	lb	
Lower Bound V_{tc} =	66557	lb, 11.3.2.2.3 (11-11)	227258	lb	
Lower Bound V_{dt} =	183194	lb, 11.3.2.2.4 (11-12)	389093	lb	
Lower Bound P_{CL} =	1236436	lb, 11.3.2.2.5 (11-13)	2284718	lb	
Controlling Q_{CE} =	66296	lb	226366	lb	
Allowable Stress=	16	psi	**NG**	30	psi	**NG**
DCR =	2.07	4619.111	DCR =	1.83	6602.396

Check 1st Floor Gridline 6:

Total Wall Load=	412917	lb
Wall height =	16.50	ft
Pier height =	10.00	ft
Pier length =	29.25	ft
Pier thickness =	1.83	ft
L / h_{eff} =	1.77	for (11-12)
Trib DL P_D =	225383	lb
Pier SW P_W =	86872.5	lb
Wall Rigidity =	757585	
Load to Pier =	412917.2	14116.83
Shear Stress=	54	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)
In-plane V_{bjs1} =	401740	lb, 11.3.2.2.2 (11-9)
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)
Rocking V_r =	428889	lb, 11.3.2.2.1 (11-8)
Lower Bound V_{tc} =	430579	lb, 11.3.2.2.3 (11-11)
Lower Bound V_{dt} =	535575	lb, 11.3.2.2.4 (11-12)
Lower Bound P_{CL} =	3144848	lb, 11.3.2.2.5 (11-13)
Controlling Q_{CE} =	401740	lb
Allowable Stress=	39	psi **NG**
DCR =	1.37	3815.803

Check 1st Floor Gridline B:

Total Wall Load=	673640	lb									
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft
Pier height =	8.00	ft		9.00	ft		8.00	ft		9.00	ft
Pier length =	11.5	ft		14.25	ft		16.83	ft		19.75	ft
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.70	for (11-12)		0.86			1.02			1.20	
Trib DL P_D =	53130	lb		65835	lb		77754.6	lb		91245	lb
Pier SW P_W =	34155	lb		42322.5	lb		49985.1	lb		58657.5	lb
Wall Rigidity =	249083			297138			475040			506405	
Load to Pier =	109835.5	9550.912		131026.1	9194.817		209473.9	12446.46		223304.3	11306.55
Shear Stress=	36	psi		35	psi		47	psi		43	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi
In-plane V_{bjs1} =	140208	lb, 11.3.2.2.2 (11-9)		173736	lb		205191	lb		240792	lb
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb		N/A	lb
Rocking V_r =	44039	lb, 11.3.2.2.1 (11-8)		67620	lb		94322	lb		129891	lb
Lower Bound V_{tc} =	45577	lb, 11.3.2.2.3 (11-11)		69981	lb		97615	lb		134426	lb
Lower Bound V_{dt} =	132161	lb, 11.3.2.2.4 (11-12)		201386	lb		261371	lb		306718	lb
Lower Bound P_{CL} =	1236436	lb, 11.3.2.2.5 (11-13)		1532105	lb		1809497	lb		2123444	lb
Controlling Q_{CE} =	44039	lb		67620	lb		94322	lb		129891	lb
Allowable Stress=	11	psi **NG**		14	psi **NG**		16	psi **NG**		19	psi
DCR =	3.33	6678.787	DCR =	2.58	5635.879	DCR =	2.96	8243.168	DCR =	2.29	6373.985
											NG

Check 1st Floor Gridline C:

Total Wall Load=	693863	lb									
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft
Pier height =	8.00	ft		9.00	ft		8.00	ft		9.00	ft
Pier length =	14.25	ft		15.25	ft		15.5	ft		17	ft
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.86	for (11-12)		0.92			0.94			1.03	
Trib DL P_D =	65835	lb		70455	lb		71610	lb		78540	lb
Pier SW P_W =	42322.5	lb		45292.5	lb		46035	lb		50490	lb
Wall Rigidity =	364047			334521			417652			400927	
Load to Pier =	166495.7	11683.91		152992.4	10032.29		191012	12323.35		183362.8	10786.05
Shear Stress=	44	psi		38	psi		47	psi		41	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	

LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	173736	lb, 11.3.2.2.2 (11-9)		185928	lb		188976	lb		207264	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb		N/A	lb	
Rocking V_r =	67620	lb, 11.3.2.2.1 (11-8)		77443	lb		80003	lb		96237	lb	
Lower Bound V_{tc} =	69981	lb, 11.3.2.2.3 (11-11)		80147	lb		82797	lb		99597	lb	
Lower Bound V_{dt} =	201386	lb, 11.3.2.2.4 (11-12)		232096	lb		240716	lb		264011	lb	
Lower Bound P_{CL} =	1532105	lb, 11.3.2.2.5 (11-13)		1639621	lb		1666500	lb		1827775	lb	
Controlling Q_{CE} =	67620	lb		77443	lb		80003	lb		96237	lb	
Allowable Stress=	14	psi	**NG**	14	psi	**NG**	15	psi	**NG**	16	psi	
DCR =	3.28	8124.97	DCR =	2.63	6223.6	DCR =	3.18	8452.229	DCR =	2.54	6540.299	

Check 1st Floor Gridline D: **NG**

Total Wall Load=	313418	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier length =	9	ft		9	ft		9.25	ft		9.25	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft	
L / h_{eff} =	0.55	for (11-12)		0.55			0.56			0.56		
Trib DL P_D =	41580	lb		41580	lb		42735	lb		42735	lb	
Pier SW P_W =	26730	lb		26730	lb		27472.5	lb		27472.5	lb	
Wall Rigidity =	92997			92997			99089			99089		
Load to Pier =	75869.72	8429.969		75869.72	8429.969		80839.04	8739.356		80839.04	8739.356	
Shear Stress=	32	psi		32	psi		33	psi		33	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	109728	lb, 11.3.2.2.2 (11-9)		109728	lb		112776	lb		112776	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		21368	lb		21368	lb	
Rocking V_r =	26973	lb, 11.3.2.2.1 (11-8)		26973	lb		28492	lb		28492	lb	
Lower Bound V_{tc} =	27915	lb, 11.3.2.2.3 (11-11)		27915	lb		29487	lb		29487	lb	
Lower Bound V_{dt} =	127191	lb, 11.3.2.2.4 (11-12)		136975	lb		143653	lb		143653	lb	
Lower Bound P_{CL} =	967645	lb, 11.3.2.2.5 (11-13)		967645	lb		994524	lb		994524	lb	
Controlling Q_{CE} =	26973	lb		26973	lb		21368	lb		21368	lb	
Allowable Stress=	9	psi	**NG**	9	psi	**NG**	7	psi	**NG**	7	psi	
DCR =	3.75	6182.219	DCR =	3.75	6182.219	DCR =	5.04	7006.856	DCR =	5.04	7006.856	

Check 1st Floor Gridline E: **NG**

Total Wall Load=	128187	lb				
Wall height =	16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft	
Pier length =	8.25	ft		8.25	ft	
Pier thickness =	1.83	ft		1.83	ft	
L / h_{eff} =	0.50	for (11-12)		0.50		
Trib DL P_D =	63569.55	lb		63569.55	lb	
Pier SW P_W =	24502.5	lb		24502.5	lb	
Wall Rigidity =	75695			75695		
Load to Pier =	64093.29	7768.884		64093.29	7768.884	128186.6
Shear Stress=	29	psi		29	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)		52	psi	
In-plane V_{bjs1} =	113311	lb, 11.3.2.2.2 (11-9)		113311	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb	
Rocking V_r =	34119	lb, 11.3.2.2.1 (11-8)		34119	lb	
Lower Bound V_{tc} =	34254	lb, 11.3.2.2.3 (11-11)		34254	lb	
Lower Bound V_{dt} =	131422	lb, 11.3.2.2.4 (11-12)		151060	lb	
Lower Bound P_{CL} =	887008	lb, 11.3.2.2.5 (11-13)		887008	lb	
Controlling Q_{CE} =	34119	lb		34119	lb	
Allowable Stress=	12	psi	**NG**	12	psi	**NG**
DCR =	2.50	4667.124	DCR =	2.50	4667.124	

Check Basement Gridline 3:

Total Wall Load=	470910	lb

Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		5.00	ft		8.00	ft		8.00	ft
Pier length =	3.75	ft		6.75	ft		8.33	ft		11.25	ft
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.38	for (11-12)		0.68			0.83			1.13	
Trib DL P_D =	40782.75	lb		73408.95	lb		90592.08	lb		122348.3	lb
Pier SW P_W =	13125	lb		23625	lb		14994	lb		20250	lb
Wall Rigidity =	81936			301931			129304			238981	
Load to Pier =	51299.06	13679.75		189033.8	28005.01		80954.82	9718.466		149622.2	13299.75
Shear Stress=	38	psi		78	psi		37	psi		50	psi
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5	
LB Bed-Joint V_{mL} =	53	psi, 11.2.3.6.3 (11-6)		53	psi		58	psi		58	psi
In-plane V_{bjs1} =	71016	lb, 11.3.2.2.2 (11-9)		127829	lb		127613	lb		172347	lb
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb		N/A	lb
Rocking V_r =	9097	lb, 11.3.2.2.1 (11-8)		29474	lb		39579	lb		72190	lb
Lower Bound V_{tc} =	9147	lb, 11.3.2.2.3 (11-11)		29635	lb		38940	lb		71025	lb
Lower Bound V_{dt} =	69700	lb, 11.3.2.2.4 (11-12)		154282	lb		172509	lb		232980	lb
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		895610	lb		1209557	lb
Controlling Q_{CE} =	9097	lb		29474	lb		38940	lb		71025	lb
Allowable Stress=	5	psi	**NG**	9	psi	**NG**	13	psi	**NG**	18	**NG**
DCR =	7.52	11860.36	DCR =	8.55	24730.12	DCR =	2.77	6212.446	DCR =	2.81	8564.729

Check Basement Gridline 6:

Total Wall Load=	470910	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		6.50	ft		8.00	ft		8.00	ft
Pier length =	3.75	ft		6.75	ft		5.5	ft		9.33	ft
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.38	for (11-12)		0.68			0.55			0.93	
Trib DL P_D =	40782.75	lb		73408.95	lb		59814.7	lb		101467.5	lb
Pier SW P_W =	13125	lb		23625	lb		9900	lb		16794	lb
Wall Rigidity =	81936			175608			48200			164749	
Load to Pier =	117314.2	31283.78		175763.3	26039.01		69012.01	12547.64		235883	25282.21
Shear Stress=	87	psi		72	psi		48	psi		96	psi
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5	
LB Bed-Joint V_{mL} =	53	psi, 11.2.3.6.3 (11-6)		53	psi		58	psi		58	psi
In-plane V_{bjs1} =	71016	lb, 11.3.2.2.2 (11-9)		127829	lb		84258	lb		142933	lb
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb		N/A	lb
Rocking V_r =	9097	lb, 11.3.2.2.1 (11-8)		29474	lb		17254	lb		49652	lb
Lower Bound V_{tc} =	9147	lb, 11.3.2.2.3 (11-11)		29635	lb		16976	lb		48851	lb
Lower Bound V_{dt} =	69700	lb, 11.3.2.2.4 (11-12)		154282	lb		113901	lb		193218	lb
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		591339	lb		1003126	lb
Controlling Q_{CE} =	9097	lb		29474	lb		16976	lb		48851	lb
Allowable Stress=	5	psi	**NG**	9	psi	**NG**	9	psi	**NG**	15	psi
DCR =	17.19	29464.39	DCR =	7.95	22764.12	DCR =	5.42	10232.74	DCR =	6.44	21355.3
											NG

Check Basement Gridline B:

Total Wall Load=	808243	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	10.00	ft		10.00	ft		8.00	ft		8.00	ft
Pier length =	12.33	ft		12.33	ft		17.75	ft		21.75	ft
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft
L / h_{eff} =	1.23	for (11-12)		1.23			1.78			2.18	
Trib DL P_D =	96050.7	lb		96050.7	lb		138272.5	lb		169432.5	lb
Pier SW P_W =	22194	lb		22194	lb		31950	lb		39150	lb
Wall Rigidity =	252315			252315			514811			687118	
Load to Pier =	119499	9691.73		119499	9691.73		243819.6	13736.32		325425.7	14962.1
Shear Stress=	27	psi		27	psi		52	psi		57	psi
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5	
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		52	psi		52	psi

In-plane V_{bjs1} =	214480	lb, 11.3.2.2.2 (11-9)		214480	lb		244542	lb		299650	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb		N/A	lb	
Rocking V_r =	65608	lb, 11.3.2.2.1 (11-8)		65608	lb		135965	lb		204150	lb	
Lower Bound V_{tc} =	68274	lb, 11.3.2.2.3 (11-11)		68274	lb		137982	lb		207179	lb	
Lower Bound V_{dt} =	197682	lb, 11.3.2.2.4 (11-12)		243096	lb		318476	lb		390246	lb	
Lower Bound P_{CL} =	1811030	lb, 11.3.2.2.5 (11-13)		1811030	lb		1908412	lb		2338476	lb	
Controlling Q_{CE} =	65608	lb		65608	lb		135965	lb		204150	lb	
Allowable Stress=	11	psi	**NG**	11	psi	**NG**	22	psi	**NG**	27	psi	
DCR =	2.43	5700.971	DCR =	2.43	5700.971	DCR =	2.39	7991.308	DCR =	2.13	7922.441	
											NG	

Check Basement Gridline C:

Total Wall Load=	828466	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	10.00	ft		10.00	ft		8.00	ft		8.00	ft	
Pier length =	12.33	ft		12.33	ft		21.75	ft		22	ft	
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft	
L / h_{eff} =	1.23	for (11-12)		1.23			2.18			2.20		
Trib DL P_D =	96050.7	lb		96050.7	lb		169432.5	lb		171380	lb	
Pier SW P_W =	22194	lb		22194	lb		39150	lb		39600	lb	
Wall Rigidity =	252315			252315			687118			697822		
Load to Pier =	110625.6	8972.067		110625.6	8972.067		301261.1	13851.08		305954.1	13907.01	
Shear Stress=	25	psi		25	psi		53	psi		53	psi	
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		52	psi		52	psi	
In-plane V_{bjs1} =	214480	lb, 11.3.2.2.2 (11-9)		214480	lb		299650	lb		303094	lb	
In-plane V_{bjs2} =	N/A	lb, 11.3.2.2.2 (11-10)		N/A	lb		N/A	lb		N/A	lb	
Rocking V_r =	65608	lb, 11.3.2.2.1 (11-8)		65608	lb		204150	lb		208870	lb	
Lower Bound V_{tc} =	68274	lb, 11.3.2.2.3 (11-11)		68274	lb		207179	lb		211969	lb	
Lower Bound V_{dt} =	243096	lb, 11.3.2.2.4 (11-12)		261796	lb		390246	lb		394731	lb	
Lower Bound P_{CL} =	1811030	lb, 11.3.2.2.5 (11-13)		1811030	lb		2338476	lb		2365356	lb	
Controlling Q_{CE} =	65608	lb		65608	lb		204150	lb		208870	lb	
Allowable Stress=	11	psi	**NG**	11	psi	**NG**	27	psi	**NG**	27	psi	
DCR =	2.25	4981.308	DCR =	2.25	4981.308	DCR =	1.97	6811.424	DCR =	1.95	6786.431	
											NG	

Wall Anchorage - Check shear capacity of anchorage, Sect. 5.7.1.1

Tier 1 - Quick Check, Sect. 4.5.3.7

Wall thickness =	1.5	ft	
Wall W =	180	psf	
X =	1.8	from Sect 4.5.3.7	
S_x =	0.36	from 4.5.2.1 (2-1)	
A_p =	54.68	sf, area to anchor	
T_c =	6292.84	lb	**Tc > Ta NG**
m =	3	from Table 12-3 for Immediate Occupancy	
Anchor Capacity =	5845.5	lb, per TMS 3.1.6.3.1.2	

Tier 2 - Sect. 5.7.1.1 and 7.2.11.1

Height =	13.67	ft	
Wall thickness =	1.5	ft	
Ratio =	9.11		
X =	2	from Table 7-2	
K_a =	1.515	Eq 7-11	
K_h =	1	Eq 7-12	
F_p =	38.75	psf, from 7.2.11.1 (7-9)	
F_{pMIN} =	109.08	psf, from 7.2.11.1 (7-10)	
T_c =	5964.49	lb	**Tc > Ta NG**
DCR =	1.02		

Proportions - Check Height to Thickness Rations, Sect. 5.5.3.1.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

Top story height =	9.00	ft			
Wall thickness =	1	ft	Ratio:	9.0	**OK**
Bell Lounge h =	13.67	ft			
Wall thickness =	1.5	ft	Ratio:	9.1	**NG**
First story height =	10.00	ft			
Wall thickness =	1.5	ft	Ratio:	6.7	**OK**
Other story height =	16.5	ft			
Wall thickness =	1.5	ft	Ratio:	11.0	**OK**

Tier 2 - Bell Tower Lounge - Sect. 11.3.3.2

Height =	13.67	ft	
Wall thickness =	1.5	ft	
Ratio =	9.11		
X =	2	from Table 7-2	
F_p =	127.87	psf, from 7.2.11.2 (7-13)	
OOP Moment =	2986.912	ft-lbs	
F_t =	55.31	psi	**> Ft = 45 psi NG**
DCR =	1.23		

For Collapse Prevention - Bell Tower Lounge - Sect. 11.3.3.2

S_{X1} =	0.22	from 4.5.2.1	
Allowable h/t =	14	**per Table 11-5, OK**	

Diaphragms - Openings at Shear Walls, Sect. 5.6.1.3

Bell Tower Lounge Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	8.50	ft	
Diaphragm V =	300	plf, from page	
Revised Dia. V =	397.55	plf	**OK**
Dia. Capacity =	450	plf, from page	
DCR =	0.88		

Wall OOP, Sect. 11.3.3.2

Height =	27.92	ft	
Wall thickness =	1.5	ft	
Ratio =	18.61		
X =	2	from Table 7-2	
F_p =	127.87	psf, from 7.2.11.2 (7-13)	
OOP Moment =	12459.95	ft-lbs	
F_t =	230.74	psi	**> Ft = 45 psi NG**
DCR =	5.13		

3rd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	16.75	ft	
Diaphragm V =	368	plf, from page	
Revised Dia. V =	716.09	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.59		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	2	from Table 7-2	
F_p =	127.87	psf, from 7.2.11.2 (7-13)	
OOP Moment =	7561.431	ft-lbs	
F_t =	140.03	psi	**> Ft = 45 psi NG**
DCR =	3.11		

Diaphragm strength, Sect. 12.5.2.1

GL E Opening =	9.75	ft	
Diaphragm V =	370	plf, from page	
Revised Dia. V =	721.28	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.60		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	2	from Table 7-2	
F_p =	127.87	psf, from 7.2.11.2 (7-13)	
OOP Moment =	7561.431	ft-lbs	
F_t =	140.03	psi	**> Ft = 45 psi NG**
DCR =	3.11		

3rd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft	
Diaphragm V =	272	plf, from page	
Revised Dia. V =	309.07	plf	**OK**

Dia. Capacity =	450	plf, from page
DCR =	0.69	

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft
Wall thickness =	1.5	ft
Ratio =	14.50	
X =	2	from Table 7-2
F_p =	127.87	psf, from 7.2.11.2 (7-13)
OOP Moment =	7561.431	ft-lbs
F_t =	140.03	psi **> Ft = 45 psi NG**
DCR =	3.11	

2nd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 & 6 Opening =	14.25	ft
Diaphragm V =	577	plf, from page
Revised Dia. V =	983.79	plf **NG**
Dia. Capacity =	450	plf, from page
DCR =	2.19	

Wall OOP, Sect. 11.3.3.2

Height =	31.50	ft
Wall thickness =	1.5	ft
Ratio =	21.00	
X =	2	from Table 7-2
F_p =	127.87	psf, from 7.2.11.2 (7-13)
OOP Moment =	15860.12	ft-lbs
F_t =	293.71	psi **> Ft = 45 psi NG**
DCR =	6.53	

2nd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft
Diaphragm V =	431	plf, from page
Revised Dia. V =	488.96	plf **NG**
Dia. Capacity =	450	plf, from page
DCR =	1.09	

Wall OOP, Sect. 11.3.3.2

Height =	16.50	ft
Wall thickness =	1.5	ft
Ratio =	11.00	
X =	2	from Table 7-2
F_p =	127.87	psf, from 7.2.11.2 (7-13)
OOP Moment =	4351.644	ft-lbs
F_t =	80.59	psi **> Ft = 45 psi NG**
DCR =	1.79	

1st Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	9.00	ft
Diaphragm V =	309	plf, from page
Revised Dia. V =	417.86	plf **OK**
Dia. Capacity =	450	plf, from page
DCR =	0.93	

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft
Wall thickness =	1.5	ft
Ratio =	17.00	
X =	2	from Table 7-2
F_p =	127.87	psf, from 7.2.11.2 (7-13)
OOP Moment =	10393.6	ft-lbs
F_t =	192.47	psi **> Ft = 45 psi NG**
DCR =	4.28	

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	13.33	ft
Diaphragm V =	309	plf, from page
Revised Dia. V =	503.33	plf **NG**
Dia. Capacity =	450	plf, from page
DCR =	1.12	

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft
Wall thickness =	1.5	ft
Ratio =	17.00	
X =	2	from Table 7-2
F_p =	127.87	psf, from 7.2.11.2 (7-13)
OOP Moment =	10393.6	ft-lbs
F_t =	192.47	psi **> Ft = 45 psi NG**
DCR =	4.28	

Stair Enclosures - Check Height to Thickness Rations, Sect. A.7.10.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

3rd Floor Opng =	21.75	ft			
Wall thickness =	1	ft	Ratio:	21.8	**> 12, NG**
2nd Floor Opng =	31.50	ft			
Wall thickness =	1.5	ft	Ratio:	21.0	**> 12, NG**
First story height =	25.50	ft			
Wall thickness =	1.5	ft	Ratio:	17.0	**> 12, NG**

Tier 2 - 3rd Floor Opening - Sect. Sect. 11.3.3.2				**2rd Floor Opening**			**1st Floor Opening**		
Height =	21.75	ft		31.50	ft		25.50	ft	
Wall thickness =	1	ft		1.5	ft		1.5	ft	
Ratio =	21.75			21.00			17.00		
X =	2	from Table 7-2		2	from Table 7-2		2	from Table 7-2	
F_p =	85.25	psf, from 7.2.11.2 (7-13)		127.87	psf, from 7.2.11.2 (7-13)		127.87	psf, from 7.2.11.2 (7-13)	
OOP Moment =	5040.954	ft-lbs		15860.12	ft-lbs		10393.6	ft-lbs	
F_t =	210.04	psi	**> Ft = 45 psi NG**	293.71	psi	**> Ft = 45 psi NG**	192.47	psi	**> Ft = 45 psi NG**
DCR =	4.67			6.53			4.28		

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS AT ROOF **Shape Capacity = 0.49 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	30.21	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.49	
Tr/Tc =		
	0.49	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS LOWER **Shape Capacity = 0.68 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	41.34	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.68	
Tr/Tc =		
	0.68	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Concrete Shear Wall Design per ACI 318-05

Wall Mark 2FL GL 3

Wall Properties

t_{wall} =	8	in
h_{wall} =	15	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	7505	plf, dead laod		
w_{wall} =	1500	plf, wall weight		
w_s =	100	plf, snow load		
w_l =	320	plf, live load		

Load Factors

DL =	0.9	w_{du} =	6755	plf
LL =	0.0	w_{wallu} =	1350	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	327998	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	10.19318	0.78	2.35	51%	Controls
1	8	9.931818	0.81	2.27	49%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		20.125	ft	4.62	100%	

P_{du} =	69161	lb/pier
P_{wallu} =	13822	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	82983	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	97.85455	in	(ACI 11.10.4)	
V_u =	233893	lb/pier		
$\phi V_{n,max}$ =	415162	lb > 233893 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	69194	lb < 233893 NG	(ACI 21.7.2.1)	
ϕV_n =	354363	lb < 233893 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	138387	lb < 233893 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	196442	lb		
$\phi V_c/2$ =	73666	lb < 233893 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0343 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	24.46364	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0025	(EQ. 11-32)	$l_w/3$ =	40.77273	in
R'qd A_v =	0.163	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.163 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	1890963	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	82983	lb
$M_{u(max)}$ =	1890963	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	2124681	lb > 82983 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	97.85455	in	$0.10f'_cA_g$ =	489272.7	lb > 82983 OK
R_n =	329	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0057		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.47	in^2	$A_{s,min}$ =	2.77	in^2
Try	8	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0076	
A_s =	4.8	in^2	ρ_{act} =	0.0061	> 0.0035 OK
a =	8.47	in			
c =	9.97	in			
ε_t =	0.0265	> 0.004 OK (Tension Controls)			
Mn =	2246862	ft-lb			
ϕM_n =	2022176	ft-lb > 1890963 OK			

<u>Boundary Elements</u>

A_g =	979	in^2	
S_g =	19949	in^3	
P_u =	82983	lb	
M_u =	1890963	ft-lb	
σ =	1222	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1222 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0061	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 2FL GL 6

Wall Properties

t_{wall} =	8	in
h_{wall} =	15	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	7505	plf, dead laod	DL =	0.9	w_{du} =	6755	plf
w_{wall} =	1500	plf, wall weight	LL =	0.0	w_{wallu} =	1350	plf
w_s =	100	plf, snow load	SL =	0.0	w_{su} =	0	plf
w_l =	320	plf, live load	W =	1.6	w_{lu} =	0	plf
			E =	1.4			

Load Factors

Lateral Loading

C_s =	0.36		
V_{seis} =	327988	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	12.02273	0.67	2.91	63%	Controls
1	8	8.102273	0.99	1.70	37%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		20.125	ft	4.61	100%	

P_{du} =	85838	lb/pier
P_{wallu} =	17155	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	102994	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	115.4182	in	(ACI 11.10.4)	
V_u =	290286	lb/pier		
$\phi V_{n,max}$ =	489678	lb > 290286 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	81613	lb < 290286 NG	(ACI 21.7.2.1)	
ϕV_n =	417966	lb < 290286 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	163226	lb < 290286 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	231701	lb		
$\phi V_c/2$ =	86888	lb < 290286 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0370 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 16"o.c.

A =	0.31	in^2
s_2 =	16	in
A_v =	0.23	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	28.85455	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0024	< 0.0025 NG		**16" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0025	(EQ. 11-32)	$l_w/3$ =	48.09091	in
R'qd A_v =	0.160	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 16 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	16	in		**16" < 18" OK**	
A_v =	0.2325	> 0.16 OK			
$\rho_{l,actual}$ =	0.0024	< 0.0025 NG, need more steel			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	2345527	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	102994	lb
$M_{u(max)}$ =	2345527	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	2482979	lb > 102994 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	115.4182	in	$0.10f'_cA_g$ =	577090.9	lb > 102994 OK
R_n =	293	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0051		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.68	in^2	$A_{s,min}$ =	3.26	in^2
Try	8	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0068	
A_s =	4.8	in^2	ρ_{act} =	0.0052	> 0.0035 OK
a =	8.47	in			
c =	9.97	in			
ε_t =	0.0317	> 0.004 OK (Tension Controls)			
Mn =	2668389	ft-lb			
ϕM_n =	2401550	ft-lb > 2345527 OK			

<u>Boundary Elements</u>

A_g =	1154	in^2	
S_g =	27753	in^3	
P_u =	102994	lb	
M_u =	2345527	ft-lb	
σ =	1103	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1103 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0052	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL 3

Wall Properties

t_{wall} =	8	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod	DL =	0.9	w_{du} =	9350	plf
w_{wall} =	1650	plf, wall weight	LL =	0.0	w_{wallu} =	1485	plf
w_s =	100	plf, snow load	SL =	0.0	w_{su} =	0	plf
w_l =	320	plf, live load	W =	1.6	w_{lu} =	0	plf
			E =	1.4			

Load Factors

Lateral Loading

C_s =	0.36		
V_{seis} =	412917	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	424	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	17.5	0.46	4.54	100%	Controls
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		17.5	ft	4.54	100%	

P_{du} =	163633	lb/pier
P_{wallu} =	25988	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	189620	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	168	in	(ACI 11.10.4)	
V_u =	578678	lb/pier		
$\phi V_{n,max}$ =	712764	lb > 578678 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	118794	lb < 578678 NG	(ACI 21.7.2.1)	
ϕV_n =	608382	lb < 578678 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	237588	lb < 578678 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	190070	lb		
$\phi V_c/2$ =	71276	lb < 578678 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0577 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

			Max spacing		
A =	0.31	in^2	$l_w/5$ =	42	in
s_2 =	15	in	$3t_{wall}$ =	24	in
A_v =	0.25	in^2/ft	18 =	18	in
A_g =	96	in^2	s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	70	in
R'qd A_v =	0.164	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.164 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	4666250	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	189620	lb
$M_{u(max)}$ =	4666250	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3635928	lb > 189620 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	168	in	$0.10f'_cA_g$ =	840000	lb > 189620 OK
R_n =	276	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0048		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	6.39	in^2	$A_{s,min}$ =	4.75	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0063	
A_s =	7.8	in^2	ρ_{act} =	0.0058	> 0.0035 OK
a =	13.76	in			
c =	16.19	in			
ε_t =	0.0281	> 0.004 OK (Tension Controls)			
Mn =	6283588	ft-lb			
ϕM_n =	5655229	ft-lb > 4666250 OK			

Boundary Elements

A_g =	1680	in^2	
S_g =	58800	in^3	
P_u =	189620	lb	
M_u =	4666250	ft-lb	
σ =	1065	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1065 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0058	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL 6

Wall Properties

t_{wall} =	8	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1650	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1485	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	412917	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	424	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	18.33	0.44	4.79	100%	Controls
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		18.33	ft	4.79	100%	

P_{du} =	171394	lb/pier
P_{wallu} =	27220	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	198614	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	175.968	in	(ACI 11.10.4)	
V_u =	578678	lb/pier		
$\phi V_{n,max}$ =	746569	lb > 578678 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	124428	lb < 578678 NG	(ACI 21.7.2.1)	
ϕV_n =	637236	lb < 578678 NG	(ACI EQ. 21-7)	
$2A_{cv}$sqrt(f_c') =	248856	lb < 578678 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	199085	lb		
$\phi V_c/2$ =	74657	lb < 578678 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0542 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	43.992	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	73.32	in
R'qd A_v =	0.164	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.164 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	4668222	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	198614	lb
$M_{u(max)}$ =	4668222	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3798475	lb > 198614 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	175.968	in	$0.10f'_cA_g$ =	879840	lb > 198614 OK
R_n =	251	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0043		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	6.08	in^2	$A_{s,min}$ =	4.98	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0058	
A_s =	7.8	in^2	ρ_{act} =	0.0055	> 0.0035 OK
a =	13.76	in			
c =	16.19	in			
ε_t =	0.0296	> 0.004 OK (Tension Controls)			
Mn =	6594340	ft-lb			
ϕM_n =	5934906	ft-lb > 4668222 OK			

<u>Boundary Elements</u>

A_g =	1760	in^2	
S_g =	64510	in^3	
P_u =	198614	lb	
M_u =	4668222	ft-lb	
σ =	981	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 981 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0055	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL B

Wall Properties

t_{wall} =	10	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod	DL =	0.9	w_{du} =	9350	plf
w_{wall} =	2062.5	plf, wall weight	LL =	0.0	w_{wallu} =	1856	plf
w_s =	100	plf, snow load	SL =	0.0	w_{su} =	0	plf
w_l =	320	plf, live load	W =	1.6	w_{lu} =	0	plf
			E =	1.4			

Load Factors

Lateral Loading

C_s =	0.36		
V_{seis} =	673640	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	530	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	16.25	0.49	5.22	32%	Controls
1	8	15.33	0.52	4.88	30%	
1	8	10.67	0.75	3.12	19%	
1	8	10.25	0.78	2.96	18%	
total wall length =		52.5	ft	16.18	100%	

P_{du} =	158401	lb/pier
P_{wallu} =	31446	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	189847	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	156	in	(ACI 11.10.4)	
V_u =	304554	lb/pier		
$\phi V_{n,max}$ =	827315	lb > 304554 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	137886	lb < 304554 NG	(ACI 21.7.2.1)	
ϕV_n =	706157	lb < 304554 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	275772	lb < 304554 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	394407	lb		
$\phi V_c/2$ =	147903	lb < 304554 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0198 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 12"o.c.

			Max spacing		
A =	0.31	in^2	$l_w/5$ =	39	in
s_2 =	12	in	$3t_{wall}$ =	30	in
A_v =	0.31	in^2/ft	18 =	18	in
A_g =	120	in^2	s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**12" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	65	in
R'qd A_v =	0.205	in^2/ft	$3t_{wall}$ =	30	in
Try #5 at 12 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	12	in		**12" < 18" OK**	
A_v =	0.31	> 0.205 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	30	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	120	in^2		**16" < 18" OK**	
ρ_t =	0.0019	< 0.002 NG, more steel required			

Vertical Steel - (ACI 14.3.2)

Try #5 at 15 o.c.		
A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	120	in^2
ρ_t =	0.0021	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2482780	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	189847	lb
$M_{u(max)}$ =	2482780	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	4122504	lb > 189847 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	156	in	$0.10f'_cA_g$ =	975000	lb > 189847 OK
R_n =	136	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0023		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	3.60	in^2	$A_{s,min}$ =	5.52	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0031	
A_s =	5.4	in^2	ρ_{act} =	0.0035	> 0.0031 OK
a =	7.62	in			
c =	8.97	in			
ε_t =	0.0492	> 0.004 OK (Tension Controls)			
Mn =	4109082	ft-lb			
ϕM_n =	3698174	ft-lb > 2482780 OK			

Boundary Elements

A_g =	1950	in^2	
S_g =	63375	in^3	
P_u =	189847	lb	
M_u =	2482780	ft-lb	
σ =	567	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 567 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0035	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL C

Wall Properties

t_{wall} =	10	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	2062.5	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1856	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	673640	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	530	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	13.75	0.58	4.29	37%	Controls
1	8	12.67	0.63	3.88	34%	
1	8	7.67	1.04	1.95	17%	
1	8	6.33	1.26	1.43	12%	
total wall length =		40.42	ft	11.56	100%	

P_{du} =	140238	lb/pier
P_{wallu} =	27840	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	168079	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	132	in	(ACI 11.10.4)	
V_u =	350216	lb/pier		
$\phi V_{n,max}$ =	700036	lb > 350216 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	116673	lb < 350216 NG	(ACI 21.7.2.1)	
ϕV_n =	597518	lb < 350216 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	233345	lb < 350216 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	333729	lb		
$\phi V_c/2$ =	125149	lb < 350216 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0354 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 12"o.c.

A =	0.31	in^2
s_2 =	12	in
A_v =	0.31	in^2/ft
A_g =	120	in^2

Max spacing

$l_w/5$ =	33	in
$3t_{wall}$ =	30	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**12" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	55	in
R'qd A_v =	0.204	in^2/ft	$3t_{wall}$ =	30	in
Try #5 at 12 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	12	in		**12" < 18" OK**	
A_v =	0.31	> 0.204 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	30	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	120	in^2		**16" < 18" OK**	
ρ_t =	0.0019	< 0.002 NG, more steel required			

Vertical Steel - (ACI 14.3.2)

Try #5 at 15 o.c.		
A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	120	in^2
ρ_t =	0.0021	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2840364	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	168079	lb
$M_{u(max)}$ =	2840364	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3510504	lb > 168079 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	132	in	$0.10f'_cA_g$ =	825000	lb > 168079 OK
R_n =	217	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0037		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.91	in^2	$A_{s,min}$ =	4.67	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0050	
A_s =	5.4	in^2	ρ_{act} =	0.0041	> 0.0035 OK
a =	7.62	in			
c =	8.97	in			
ε_t =	0.0412	> 0.004 OK (Tension Controls)			
Mn =	3461082	ft-lb			
ϕM_n =	3114974	ft-lb > 2840364 OK			

Boundary Elements

A_g =	1650	in^2	
S_g =	45375	in^3	
P_u =	168079	lb	
M_u =	2840364	ft-lb	
σ =	853	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 853 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0041	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL 3

Wall Properties

t_{wall} =	12	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1500	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1350	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	470910	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	9.5	0.84	3.20	43%	Controls
1	8	8.25	0.97	2.62	35%	
1	8	6	1.33	1.57	21%	
0	0	0	0.00	0.00	0%	
total wall length =		23.75	ft	7.39	100%	

P_{du} =	96228	lb/pier
P_{wallu} =	13893	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	110121	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	91.2	in	(ACI 11.10.4)	
V_u =	285908	lb/pier		
$\phi V_{n,max}$ =	580393	lb > 285908 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	96732	lb < 285908 NG	(ACI 21.7.2.1)	
ϕV_n =	495397	lb < 285908 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	193464	lb < 285908 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	273139	lb		
$\phi V_c/2$ =	102427	lb < 285908 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0414 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	144	in^2

Max spacing

$l_w/5$ =	22.8	in
$3t_{wall}$ =	36	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0017	< 0.0025 NG		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0019	(EQ. 11-32)	$l_w/3$ =	38	in
R'qd A_v =	0.240	in^2/ft	$3t_{wall}$ =	36	in
Try #5 at 9 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	9	in		**9" < 18"**	**OK**
A_v =	0.413333	> 0.24 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 9 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	36	in
s_2 =	9	in	18 =	18	in
A_v =	0.41	in^2/ft	s_{max} =	18	in
A_g =	144	in^2		**9" < 18"**	**OK**
ρ_t =	0.0029	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	144	in^2
ρ_t =	0.0014	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2305909	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	110121	lb
$M_{u(max)}$ =	2305909	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3031560	lb > 110121 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	91.2	in	$0.10f'_cA_g$ =	684000	lb > 110121 OK
R_n =	308	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0053		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	5.84	in^2	$A_{s,min}$ =	3.87	in^2
Try	15	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0071	
A_s =	9	in^2	ρ_{act} =	0.0082	> 0.0035 OK
a =	10.59	in			
c =	12.46	in			
ε_t =	0.0190	> 0.004 OK (Tension Controls)			
Mn =	3865765	ft-lb			
ϕM_n =	3479188	ft-lb > 2305909 OK			

Boundary Elements

A_g =	1368	in^2	
S_g =	25992	in^3	
P_u =	110121	lb	
M_u =	2305909	ft-lb	
σ =	1145	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1145 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0082	> 0.0067 Provide Boundary Ties	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL 6

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod		
w_{wall} =	1875	plf, wall weight		
w_s =	100	plf, snow load		
w_l =	320	plf, live load		

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	470910	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	9	0.89	3.71	53%	**Controls**
1	8	6.75	1.19	2.39	34%	
1	8	4	2.00	0.89	13%	
0	0	0	0.00	0.00	0%	
total wall length =		19.75	ft	7.00	100%	

P_{du} =	97917	lb/pier
P_{wallu} =	17671	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	115588	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	86.4	in	(ACI 11.10.4)	
V_u =	349919	lb/pier		
$\phi V_{n,max}$ =	687308	lb > 349919 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	114551	lb < 349919 NG	(ACI 21.7.2.1)	
ϕV_n =	586654	lb < 349919 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	229103	lb < 349919 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	319246	lb		
$\phi V_c/2$ =	119717	lb < 349919 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0546 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	21.6	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0014	< 0.0025 NG		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0017	(EQ. 11-32)	$l_w/3$ =	36	in
R'qd A_v =	0.300	in^2/ft	$3t_{wall}$ =	45	in
Try #5 at 8 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	8	in		**8" < 18"**	**OK**
A_v =	0.465	> 0.3 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 9 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	45	in
s_2 =	9	in	18 =	18	in
A_v =	0.41	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**9" < 18"**	**OK**
ρ_t =	0.0023	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 9 o.c.		
A =	0.31	in^2
s_2 =	9	in
A_v =	0.41	in^2/ft
A_g =	180	in^2
ρ_t =	0.0023	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2820793	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	115588	lb
$M_{u(max)}$ =	2820793	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3513528	lb > 115588 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	86.4	in	$0.10f'_cA_g$ =	810000	lb > 115588 OK
R_n =	336	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0058		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	7.57	in^2	$A_{s,min}$ =	4.58	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0078	
A_s =	7.8	in^2	ρ_{act} =	0.0060	> 0.0035 OK
a =	7.34	in			
c =	8.64	in			
ε_t =	0.0270	> 0.004 OK (Tension Controls)			
Mn =	3226447	ft-lb			
ϕM_n =	2903802	ft-lb > 2820793 OK			

Boundary Elements

A_g =	1620	in^2	
S_g =	29160	in^3	
P_u =	115588	lb	
M_u =	2820793	ft-lb	
σ =	1232	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1232 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0060	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL B

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod		
w_{wall} =	1875	plf, wall weight		
w_s =	100	plf, snow load		
w_l =	320	plf, live load		

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	808243	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	28	0.29	14.20	58%	Controls
1	8	13.5	0.59	6.29	26%	
1	8	9.75	0.82	4.15	17%	
0	0	0	0.00	0.00	0%	
total wall length =		51.25	ft	24.64	100%	

P_{du} =	276123	lb/pier
P_{wallu} =	49833	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	325955	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	268.8	in	(ACI 11.10.4)	
V_u =	652387	lb/pier		
$\phi V_{n,max}$ =	2138291	lb > 652387 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	356382	lb < 652387 NG	(ACI 21.7.2.1)	
ϕV_n =	1825145	lb > 652387 OK	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	712764	lb > 652387 OK	(ACI 21.7.2.2)	**Min. One Curtain of ReStl**
V_c =	570211	lb		
$\phi V_c/2$ =	213829	lb < 652387 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0186 $in^2/in/ft$ (EQ. 11-31)

Try #6 at 10"o.c.

A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	67.2	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0029	> 0.0025 OK		**10" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0030	(EQ. 11-32)	$l_w/3$ =	112	in
R'qd A_v =	0.356	in^2/ft	$3t_{wall}$ =	45	in
Try #6 at 10 o.c.			18 =	18	in
A =	0.44	in^2	s_{max} =	18	in
s =	10	in		**10" < 18" OK**	
A_v =	0.528	> 0.356 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #6 at 10 o.c.			**Max spacing**		
A =	0.44	in^2	$3t_{wall}$ =	45	in
s_2 =	10	in	18 =	18	in
A_v =	0.53	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**10" < 18" OK**	
ρ_t =	0.0029	> 0.0025 OK			

Vertical Steel - (ACI 14.3.2)

Try #6 at 12 o.c.		
A =	0.44	in^2
s_2 =	12	in
A_v =	0.44	in^2/ft
A_g =	180	in^2
ρ_t =	0.0024	> 0.0015 OK

BENDING - ACI Chapter 10

M_u =	5291587	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	325955	lb
$M_{u(max)}$ =	5291587	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	10426104	lb > 325955 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	268.8	in	$0.10f'_cA_g$ =	2520000	lb > 325955 OK
R_n =	65	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0011		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.41	in^2	$A_{s,min}$ =	14.26	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0015	
A_s =	5.4	in^2	ρ_{act} =	0.0013	< 0.0015 NG, more steel required
a =	5.08	in			
c =	5.98	in			
ε_t =	0.1319	> 0.004 OK (Tension Controls)			
Mn =	7188988	ft-lb			
ϕM_n =	6470089	ft-lb > 5291587 OK			

Boundary Elements

A_g =	5040	in^2	
S_g =	282240	in^3	
P_u =	325955	lb	
M_u =	5291587	ft-lb	
σ =	290	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 290 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0013	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL C

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod		
w_{wall} =	1875	plf, wall weight		
w_s =	100	plf, snow load		
w_l =	320	plf, live load		

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	808243	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	22.5	0.36	11.24	50%	Controls
1	8	22.25	0.36	11.11	50%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		44.75	ft	22.35	100%	

P_{du} =	210480	lb/pier
P_{wallu} =	37986	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	248466	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	216	in	(ACI 11.10.4)	
V_u =	569529	lb/pier		
$\phi V_{n,max}$ =	1718269	lb > 569529 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	286378	lb < 569529 NG	(ACI 21.7.2.1)	
ϕV_n =	1466635	lb > 569529 OK	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	572756	lb > 569529 OK	(ACI 21.7.2.2)	**Min. One Curtain of ReStl**
V_c =	458205	lb		
$\phi V_c/2$ =	171827	lb < 569529 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0232 in^2/in/ft (EQ. 11-31)

Try #6 at 10"o.c.

A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	54	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0029	> 0.0025 OK		**10" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0029	(EQ. 11-32)	$l_w/3$ =	90	in
R'qd A_v =	0.353	in^2/ft	$3t_{wall}$ =	45	in
Try #6 at 10 o.c.			18 =	18	in
A =	0.44	in^2	s_{max} =	18	in
s =	10	in		**10" < 18" OK**	
A_v =	0.528	> 0.353 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #6 at 10 o.c.			**Max spacing**		
A =	0.44	in^2	$3t_{wall}$ =	45	in
s_2 =	10	in	18 =	18	in
A_v =	0.53	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**10" < 18" OK**	
ρ_t =	0.0029	> 0.0025 OK			

Vertical Steel - (ACI 14.3.2)

Try #6 at 10 o.c.		
A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2
ρ_t =	0.0029	> 0.0015 OK

BENDING - ACI Chapter 10

M_u =	4614264	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	248466	lb
$M_{u(max)}$ =	4614264	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	8406504	lb > 248466 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	216	in	$0.10f'_cA_g$ =	2025000	lb > 248466 OK
R_n =	88	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0015		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.80	in^2	$A_{s,min}$ =	11.46	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0020	
A_s =	5.4	in^2	ρ_{act} =	0.0017	< 0.002 NG, more steel required
a =	5.08	in			
c =	5.98	in			
ε_t =	0.1054	> 0.004 OK (Tension Controls)			
Mn =	5763388	ft-lb			
ϕM_n =	5187049	ft-lb > 4614264 OK			

Boundary Elements

A_g =	4050	in^2	
S_g =	182250	in^3	
P_u =	248466	lb	
M_u =	4614264	ft-lb	
σ =	365	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 365 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0017	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Current Date: 2/25/2016 1:17 AM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 3.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	41.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 3

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00
3	15.00	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	9.50	0.00	3.00	7.00
Lower left	17.50	10.00	3.00	10.00
Lower left	10.00	26.50	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
3	EQ	Horizontal	327998.00	0.00	0.00
2	EQ	Horizontal	84930.00	0.00	0.00
1	EQ	Horizontal	57990.00	0.00	0.00

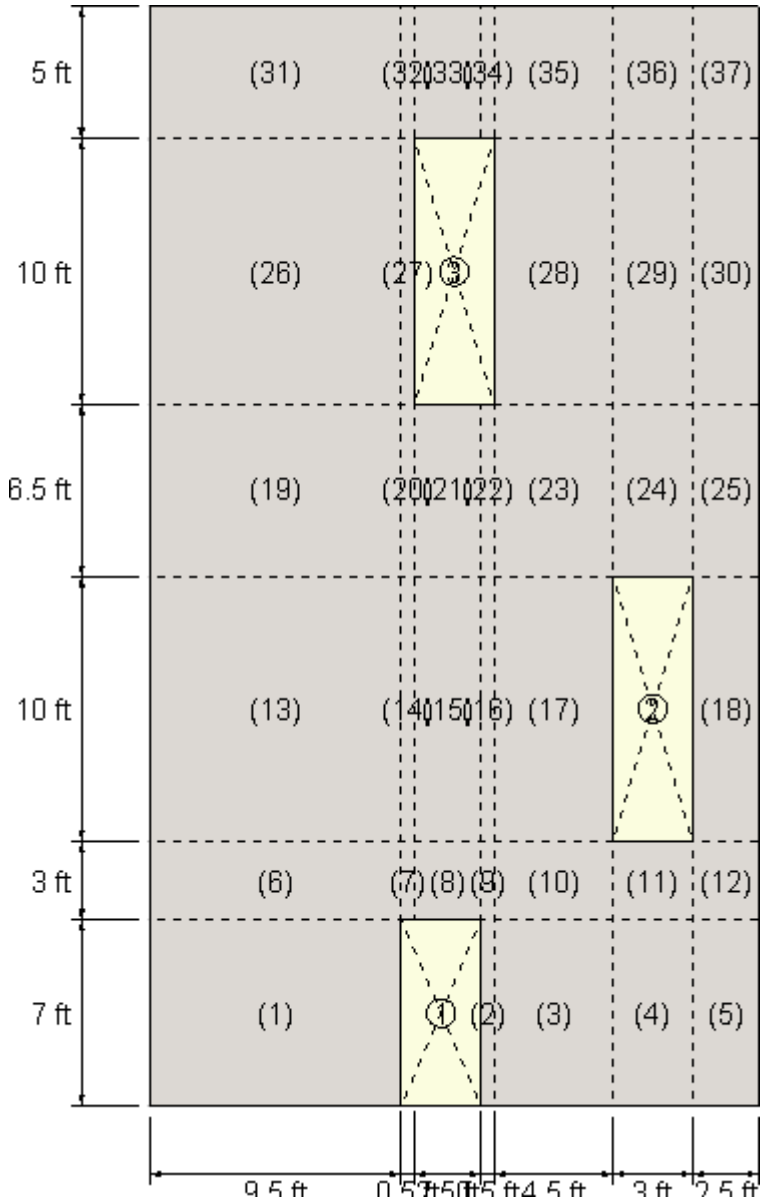
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
3	DL	Vertical	3360.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	0.50	7.00
3	13.00	0.00	4.50	7.00
4	17.50	0.00	3.00	7.00
5	20.50	0.00	2.50	7.00
6	0.00	7.00	9.50	3.00
7	9.50	7.00	0.50	3.00
8	10.00	7.00	2.50	3.00
9	12.50	7.00	0.50	3.00
10	13.00	7.00	4.50	3.00
11	17.50	7.00	3.00	3.00
12	20.50	7.00	2.50	3.00
13	0.00	10.00	9.50	10.00
14	9.50	10.00	0.50	10.00
15	10.00	10.00	2.50	10.00
16	12.50	10.00	0.50	10.00

17	13.00	10.00	4.50	10.00
18	20.50	10.00	2.50	10.00
19	0.00	20.00	9.50	6.50
20	9.50	20.00	0.50	6.50
21	10.00	20.00	2.50	6.50
22	12.50	20.00	0.50	6.50
23	13.00	20.00	4.50	6.50
24	17.50	20.00	3.00	6.50
25	20.50	20.00	2.50	6.50
26	0.00	26.50	9.50	10.00
27	9.50	26.50	0.50	10.00
28	13.00	26.50	4.50	10.00
29	17.50	26.50	3.00	10.00
30	20.50	26.50	2.50	10.00
31	0.00	36.50	9.50	5.00
32	9.50	36.50	0.50	5.00
33	10.00	36.50	2.50	5.00
34	12.50	36.50	0.50	5.00
35	13.00	36.50	4.50	5.00
36	17.50	36.50	3.00	5.00
37	20.50	36.50	2.50	5.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	12-#9	10.00	43.69
2	1-#6	14.00	23.24
3	4-#6	14.00	23.24
4	3-#6	14.00	23.24
5	2-#6	14.00	23.24
6	12-#9	10.00	43.69
7	1-#6	14.00	23.24
8	2-#6	14.00	23.24
9	1-#6	14.00	23.24
10	4-#6	14.00	23.24
11	3-#6	14.00	23.24
12	2-#6	14.00	23.24
13	7-#9	18.00	43.69
14	1-#8	16.00	38.73
15	2-#8	16.00	38.73
16	1-#8	16.00	38.73
17	4-#8	16.00	38.73
18	2-#5	18.00	19.36
19	7-#9	18.00	43.69
20	1-#8	16.00	38.73
21	2-#8	16.00	38.73
22	1-#8	16.00	38.73
23	4-#8	16.00	38.73
24	2-#5	18.00	19.36
25	2-#5	18.00	19.36
26	8-#6	14.00	23.24
27	1-#6	14.00	23.24
28	4-#5	14.00	19.36
29	3-#5	14.00	19.36
30	2-#5	14.00	19.36
31	8-#6	14.00	23.24
32	1-#6	14.00	23.24
33	2-#5	14.00	19.36

34	1-#5	14.00	19.36
35	4-#5	14.00	19.36
36	3-#5	14.00	19.36
37	2-#5	14.00	19.36

Intermediate results for axial-bending

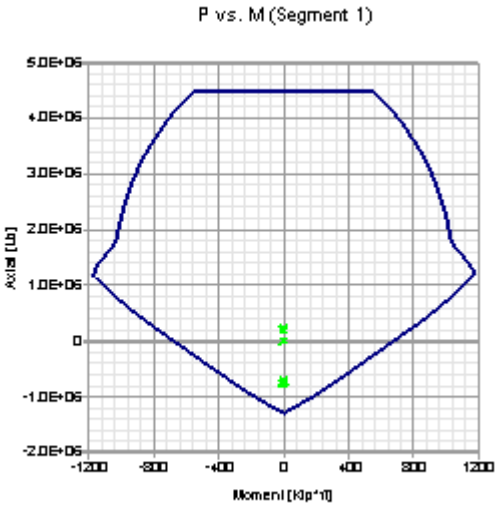
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	2.19	13.06
2	DC1 (Top)	1.82	13.25
3	DC1 (Top)	1.29	13.25
4	DC1 (Top)	1.27	13.25
5	DC1 (Top)	1.21	13.25
6	DC1 (Top)	2.17	12.94
7	DC1 (Top)	1.71	13.13
8	DC1 (Top)	1.13	13.13
9	DC1 (Top)	1.73	13.13
10	DC1 (Top)	1.34	13.13
11	DC1 (Top)	1.18	13.13
12	DC1 (Top)	1.31	13.13
13	DC1 (Top)	1.58	8.06
14	DC1 (Top)	2.34	8.13
15	DC1 (Top)	1.46	8.13
16	DC1 (Top)	2.36	8.13
17	DC1 (Top)	1.56	8.13
18	DC1 (Top)	1.01	8.31
19	DC1 (Top)	1.58	8.06
20	DC1 (Top)	2.35	8.13
21	DC1 (Top)	1.38	8.13
22	DC1 (Top)	2.28	8.13
23	DC1 (Top)	1.52	8.13
24	DC1 (Top)	0.79	8.31
25	DC1 (Top)	0.90	8.31
26	DC1 (Top)	1.05	6.25
27	DC1 (Top)	1.55	6.25
28	DC1 (Top)	0.88	6.31
29	DC1 (Top)	0.93	6.31
30	DC1 (Top)	0.77	6.31
31	DC1 (Top)	1.03	6.25
32	DC1 (Top)	1.52	6.25
33	DC1 (Top)	0.76	6.31
34	DC1 (Top)	1.29	6.31
35	DC1 (Top)	0.84	6.31
36	DC1 (Top)	0.91	6.31
37	DC1 (Top)	0.76	6.31

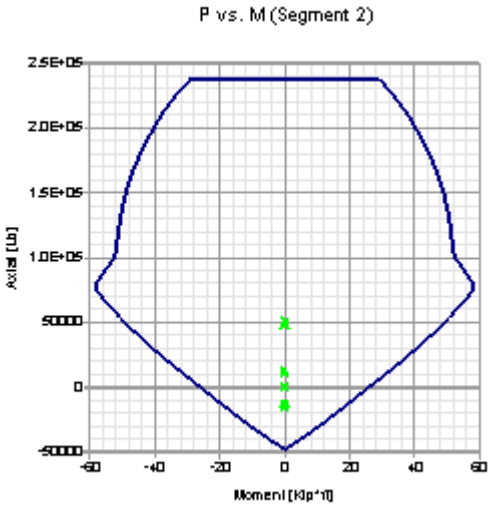
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	196336.50	0.00	771.10	0.00
2	DC1 (Top)	13511.45	0.00	32.58	0.00
3	DC1 (Top)	96222.32	0.00	162.32	0.00
4	DC1 (Top)	47900.73	0.00	107.81	0.00
5	DC1 (Top)	49754.27	0.00	82.74	0.00
6	DC1 (Top)	173847.30	0.00	754.12	0.00
7	DC1 (Top)	7411.47	0.00	29.35	0.00
8	DC1 (Top)	33625.52	0.00	74.27	0.00

9	DC1 (Top)	8628.80	0.00	29.95	0.00
10	DC1 (Top)	97167.51	0.00	163.05	0.00
11	DC1 (Top)	23454.13	0.00	95.06	0.00
12	DC1 (Top)	58513.21	0.00	87.55	0.00
13	DC1 (Top)	109443.00	0.00	290.00	0.00
14	DC1 (Top)	5701.29	0.00	27.72	0.00
15	DC1 (Top)	29099.90	0.00	68.65	0.00
16	DC1 (Top)	6122.53	0.00	27.83	0.00
17	DC1 (Top)	65914.50	0.00	137.06	0.00
18	DC1 (Top)	43014.94	0.00	41.64	0.00
19	DC1 (Top)	107326.60	0.00	289.39	0.00
20	DC1 (Top)	5980.71	0.00	27.80	0.00
21	DC1 (Top)	11632.51	0.00	63.55	0.00
22	DC1 (Top)	4394.59	0.00	27.35	0.00
23	DC1 (Top)	53636.53	0.00	133.52	0.00
24	DC1 (Top)	31272.20	0.00	38.44	0.00
25	DC1 (Top)	27831.40	0.00	36.58	0.00
26	DC1 (Top)	56444.18	0.00	128.32	0.00
27	DC1 (Top)	3538.71	0.00	12.68	0.00
28	DC1 (Top)	29902.82	0.00	50.09	0.00
29	DC1 (Top)	16872.29	0.00	35.42	0.00
30	DC1 (Top)	12762.66	0.00	24.61	0.00
31	DC1 (Top)	45639.82	0.00	125.63	0.00
32	DC1 (Top)	2374.27	0.00	12.44	0.00
33	DC1 (Top)	11859.14	0.00	24.35	0.00
34	DC1 (Top)	2379.63	0.00	9.58	0.00
35	DC1 (Top)	21727.12	0.00	47.75	0.00
36	DC1 (Top)	14372.06	0.00	34.80	0.00
37	DC1 (Top)	11926.93	0.00	24.37	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	216777.00	4471272.00	0.05
2	D2 (Top)	50682.07	236346.20	0.21
3	D2 (Top)	401981.50	2138785.00	0.19
4	D2 (Bottom)	371917.10	1425079.00	0.26
5	D2 (Bottom)	501030.40	1188732.00	0.42
6	D1 (Bottom)	196334.90	4471272.00	0.04
7	DC1 (Bottom)	7543.46	236346.20	0.03
8	D2 (Top)	51427.67	1188732.00	0.04
9	D2 (Bottom)	50681.58	236346.20	0.21
10	D2 (Top)	478757.80	2138785.00	0.22
11	D2 (Bottom)	275152.30	1425079.00	0.19
12	D2 (Top)	374595.00	1188732.00	0.32
13	D1 (Bottom)	128535.00	2986152.00	0.04
14	D2 (Top)	6169.06	154929.80	0.04
15	D2 (Top)	45936.00	787219.70	0.06
16	D2 (Bottom)	25335.92	154929.80	0.16
17	D2 (Bottom)	474710.40	1415319.00	0.34
18	D2 (Bottom)	373166.80	792311.50	0.47
19	DC1 (Bottom)	109443.00	2986152.00	0.04
20	D2 (Top)	42576.12	154929.80	0.27
21	D2 (Top)	52188.64	787219.70	0.07
22	D2 (Bottom)	11918.36	154929.80	0.08
23	D2 (Bottom)	151741.30	1415319.00	0.11
24	D2 (Top)	128289.80	951431.50	0.13
25	D2 (Bottom)	338107.70	792311.50	0.43
26	DC1 (Bottom)	69366.26	2399954.00	0.03
27	D2 (Bottom)	55943.86	124962.20	0.45
28	D2 (Top)	142396.30	1139087.00	0.13
29	D2 (Bottom)	117760.70	758843.30	0.16
30	D2 (Bottom)	187223.80	633191.50	0.30
31	D2 (Top)	127969.20	2399954.00	0.05
32	DC1 (Bottom)	3515.11	124962.20	0.03
33	DC1 (Top)	11859.14	633191.50	0.02
34	D2 (Bottom)	42286.45	125651.80	0.34
35	D2 (Bottom)	142295.00	1139087.00	0.12
36	D2 (Bottom)	25685.95	758843.30	0.03
37	D2 (Bottom)	13870.79	633191.50	0.02

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	771236.50	1296000.00	0.60
2	D3 (Bottom)	14811.33	47520.00	0.31
3	DC1 (Top)	0.00	190080.00	0.00
4	DC1 (Top)	0.00	142560.00	0.00
5	DC1 (Top)	0.00	95040.00	0.00
6	D3 (Bottom)	754700.80	1296000.00	0.58
7	D3 (Bottom)	21837.08	47520.00	0.46
8	DC1 (Bottom)	1753.25	95040.00	0.02
9	DC1 (Top)	0.00	47520.00	0.00
10	DC1 (Top)	0.00	190080.00	0.00
11	DC1 (Top)	0.00	142560.00	0.00
12	DC1 (Top)	0.00	95040.00	0.00
13	D3 (Bottom)	720928.20	756000.00	0.95

14	D3 (Bottom)	7456.77	85320.00	0.09
15	DC1 (Top)	0.00	170640.00	0.00
16	DC1 (Top)	0.00	85320.00	0.00
17	DC1 (Top)	0.00	341280.00	0.00
18	DC1 (Top)	0.00	66960.00	0.00
19	D3 (Bottom)	380051.20	756000.00	0.50
20	DC1 (Top)	0.00	85320.00	0.00
21	DC1 (Top)	0.00	170640.00	0.00
22	D3 (Top)	33526.12	85320.00	0.39
23	D3 (Top)	38289.84	341280.00	0.11
24	DC1 (Top)	0.00	66960.00	0.00
25	DC1 (Top)	0.00	66960.00	0.00
26	D3 (Bottom)	214493.40	380160.00	0.56
27	D3 (Top)	31360.71	47520.00	0.66
28	D3 (Bottom)	57351.15	133920.00	0.43
29	DC1 (Top)	0.00	100440.00	0.00
30	DC1 (Top)	0.00	66960.00	0.00
31	D3 (Bottom)	68969.52	380160.00	0.18
32	D3 (Bottom)	31136.23	47520.00	0.66
33	D2 (Bottom)	35111.83	66960.00	0.52
34	DC1 (Top)	0.00	33480.00	0.00
35	DC1 (Top)	0.00	133920.00	0.00
36	DC1 (Top)	0.00	100440.00	0.00
37	DC1 (Top)	0.00	66960.00	0.00

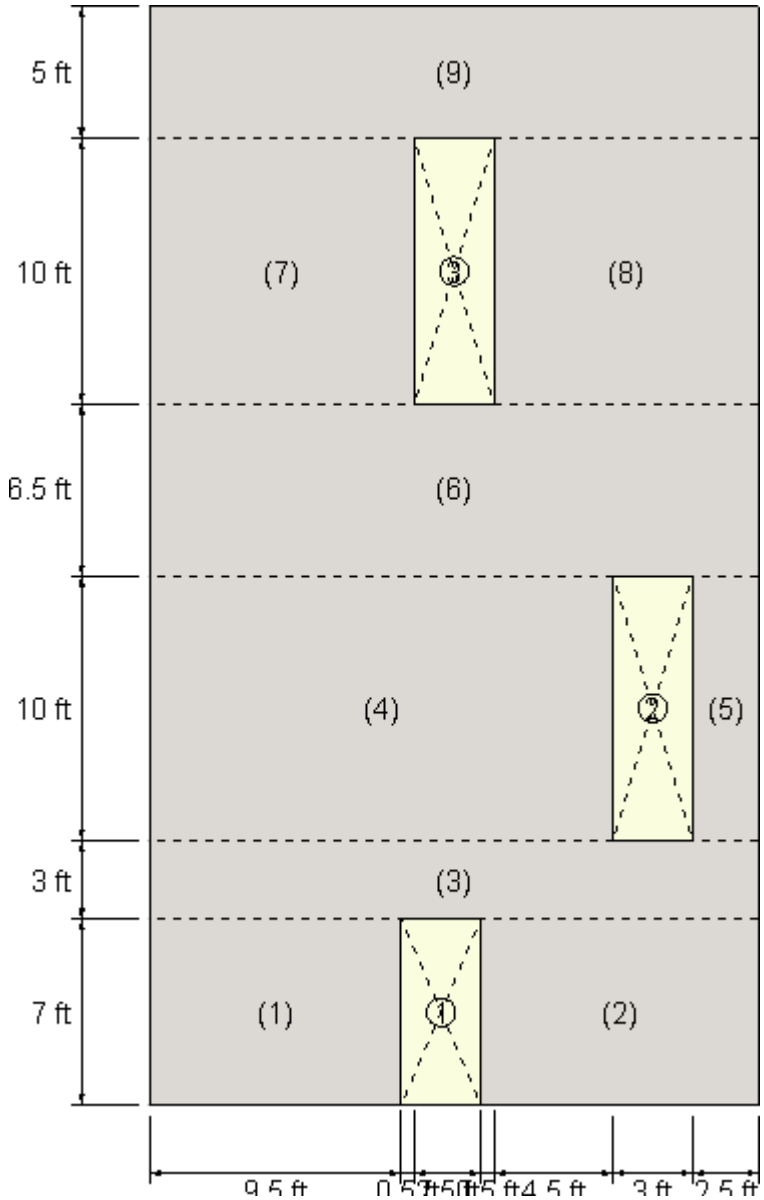
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	173000.820	0.00
2	DC1 (Top)	0.000	9237.065	0.00
3	DC1 (Top)	0.000	83133.588	0.00
4	DC1 (Top)	0.000	55422.392	0.00
5	DC1 (Top)	0.000	46185.326	0.00
6	DC1 (Top)	0.000	171345.120	0.00
7	DC1 (Top)	0.000	9149.923	0.00
8	DC1 (Top)	0.000	45749.616	0.00
9	DC1 (Top)	0.000	9149.923	0.00
10	DC1 (Top)	0.000	82349.308	0.00
11	DC1 (Top)	0.000	54899.539	0.00
12	DC1 (Top)	0.000	45749.616	0.00
13	DC1 (Top)	0.000	106772.810	0.00
14	DC1 (Top)	0.000	5664.238	0.00
15	DC1 (Top)	0.000	28321.191	0.00
16	DC1 (Top)	0.000	5664.238	0.00
17	DC1 (Top)	0.000	50978.143	0.00
18	DC1 (Top)	0.000	28974.757	0.00
19	DC1 (Top)	0.000	106772.810	0.00
20	DC1 (Top)	0.000	5664.238	0.00
21	DC1 (Top)	0.000	28321.191	0.00
22	DC1 (Top)	0.000	5664.238	0.00
23	DC1 (Top)	0.000	50978.143	0.00
24	DC1 (Top)	0.000	34769.708	0.00
25	DC1 (Top)	0.000	28974.757	0.00
26	DC1 (Top)	0.000	82785.019	0.00
27	DC1 (Top)	0.000	4357.106	0.00
28	DC1 (Top)	0.000	39606.096	0.00
29	DC1 (Top)	0.000	26404.064	0.00

30	DC1 (Top)	0.000	22003.387	0.00
31	DC1 (Top)	0.000	82785.019	0.00
32	DC1 (Top)	0.000	4357.106	0.00
33	DC1 (Top)	0.000	22003.387	0.00
34	DC1 (Top)	0.000	4400.677	0.00
35	DC1 (Top)	0.000	39606.096	0.00
36	DC1 (Top)	0.000	26404.064	0.00
37	DC1 (Top)	0.000	22003.387	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	10.50	7.00
3	0.00	7.00	23.00	3.00
4	0.00	10.00	17.50	10.00
5	20.50	10.00	2.50	10.00
6	0.00	20.00	23.00	6.50
7	0.00	26.50	10.00	10.00
8	13.00	26.50	10.00	10.00
9	0.00	36.50	23.00	5.00

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	12-#9	10.00	43.69	6-#5	16.00	25.17
2	1-#6	14.00	23.24	6-#5	16.00	25.17
	4-#6	14.00	23.24	6-#5	16.00	25.17
	3-#6	14.00	23.24	6-#5	16.00	25.17
	2-#6	14.00	23.24	6-#5	16.00	25.17
3	12-#9	10.00	43.69	2-#6	18.00	30.21
	1-#6	14.00	43.69	2-#6	18.00	30.21
	2-#6	14.00	43.69	2-#6	18.00	30.21
	1-#6	14.00	43.69	2-#6	18.00	30.21
	4-#6	14.00	43.69	2-#6	18.00	30.21
	3-#6	14.00	43.69	2-#6	18.00	30.21
	2-#6	14.00	43.69	2-#6	18.00	30.21
4	7-#9	18.00	43.69	7-#5	18.00	25.17
	1-#8	16.00	43.69	7-#5	18.00	25.17
	2-#8	16.00	43.69	7-#5	18.00	25.17
	1-#8	16.00	43.69	7-#5	18.00	25.17
	4-#8	16.00	43.69	7-#5	18.00	25.17
5	2-#5	18.00	19.36	7-#5	18.00	25.17
6	7-#9	18.00	43.69	5-#5	18.00	25.17
	1-#8	16.00	43.69	5-#5	18.00	25.17
	2-#8	16.00	43.69	5-#5	18.00	25.17
	1-#8	16.00	43.69	5-#5	18.00	25.17
	4-#8	16.00	43.69	5-#5	18.00	25.17
	2-#5	18.00	43.69	5-#5	18.00	25.17
	2-#5	18.00	43.69	5-#5	18.00	25.17
7	8-#6	14.00	23.24	7-#5	18.00	25.17
	1-#6	14.00	23.24	7-#5	18.00	25.17
8	4-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
	2-#5	14.00	19.36	7-#5	18.00	25.17
9	8-#6	14.00	23.24	4-#5	18.00	25.17
	1-#6	14.00	23.24	4-#5	18.00	25.17
	2-#5	14.00	23.24	4-#5	18.00	25.17
	1-#5	14.00	23.24	4-#5	18.00	25.17
	4-#5	14.00	23.24	4-#5	18.00	25.17
	3-#5	14.00	23.24	4-#5	18.00	25.17
	2-#5	14.00	23.24	4-#5	18.00	25.17

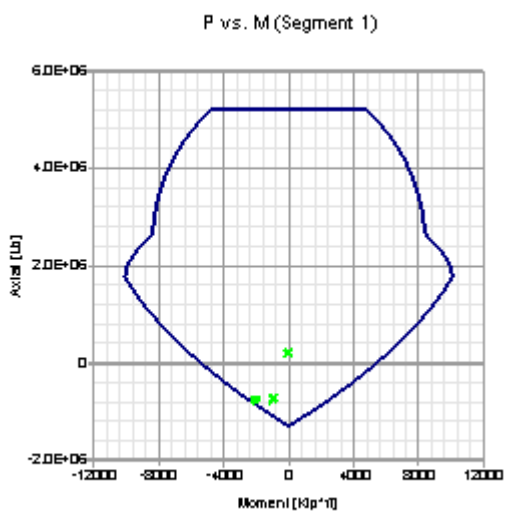
Intermediate results for axial-bending

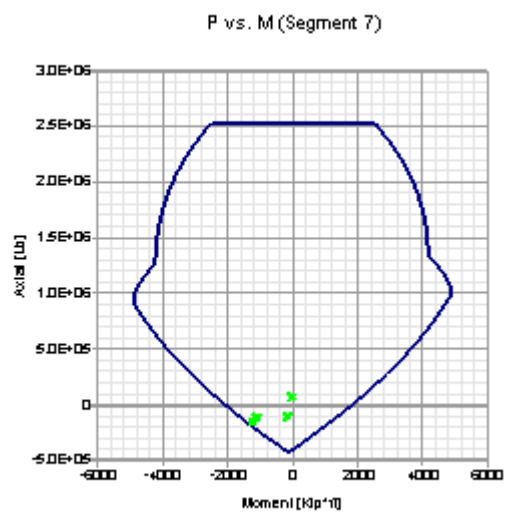
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	6.22	91.20
2	D3 (Bottom)	26.86	100.80
3	D3 (Bottom)	32.83	220.80
4	D3 (Bottom)	24.95	168.00
5	D2 (Bottom)	14.17	24.00
6	D3 (Bottom)	36.06	220.80
7	D3 (Bottom)	8.08	96.00
8	D3 (Bottom)	17.48	96.00
9	D3 (Bottom)	28.13	220.80

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	-771276.30	-2130.29	2403.00	0.89
2	D3 (Bottom)	1059704.00	-2188.94	6568.01	0.33
3	D3 (Bottom)	286510.00	-13162.87	16566.86	0.79
4	D3 (Bottom)	-209026.70	-7147.00	9757.08	0.73
5	D2 (Bottom)	373173.90	-44.07	354.78	0.12
6	D3 (Bottom)	177526.40	-7641.68	15236.73	0.50
7	D3 (Bottom)	-159506.10	-1241.97	1398.19	0.89
8	D3 (Bottom)	239555.20	-1167.44	2374.35	0.49
9	D3 (Bottom)	74015.34	-1604.91	8828.22	0.18

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	216786.30	5220072.00	0.04
2	D2 (Max)	1117333.00	4988942.00	0.22
3	D1 (Max)	415625.30	1.088529E07	0.04
4	D1 (Bottom)	245479.60	6329719.00	0.04
5	D2 (Max)	375149.00	792311.50	0.47
6	D1 (Bottom)	266300.50	8150838.00	0.03
7	DC1 (Bottom)	74787.95	2524916.00	0.03
8	D2 (Bottom)	255451.20	2531122.00	0.10
9	D2 (Top)	185997.80	5814881.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Max)	781391.10	1296000.00	0.60
2	DC1 (Top)	0.00	475200.00	0.00
3	DC1 (Top)	0.00	1913760.00	0.00
4	D3 (Bottom)	209026.70	1438560.00	0.15
5	DC1 (Top)	0.00	66960.00	0.00
6	DC1 (Top)	0.00	1572480.00	0.00
7	D3 (Bottom)	159506.10	427680.00	0.37
8	DC1 (Top)	0.00	301320.00	0.00
9	DC1 (Top)	0.00	829440.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	197782.340	216867.990	0.91
2	D2 (Top)	284603.910	627333.760	0.45
3	D3 (Bottom)	487438.430	1009002.600	0.48
4	D3 (Bottom)	400797.630	505706.360	0.79
5	D3 (Bottom)	9417.372	89752.509	0.10
6	D3 (Max)	418831.650	788173.350	0.53
7	D3 (Bottom)	132984.030	259985.580	0.51
8	D3 (Top)	200482.740	323318.950	0.62
9	D3 (Bottom)	362154.100	691983.570	0.52

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 1:11 AM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 6.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : N. G.

Design code	:	ACI 318-05

Geometry:

Total height	:	41.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 3

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00
3	15.00	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	4.00	0.00	3.00	7.00
Lower left	17.00	0.00	3.00	7.00
Lower left	2.00	10.00	3.00	10.00
Lower left	8.00	26.50	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ

D3	Yes	0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
3	EQ	Horizontal	327998.00	0.00	0.00
2	EQ	Horizontal	84930.00	0.00	0.00
1	EQ	Horizontal	57990.00	0.00	0.00

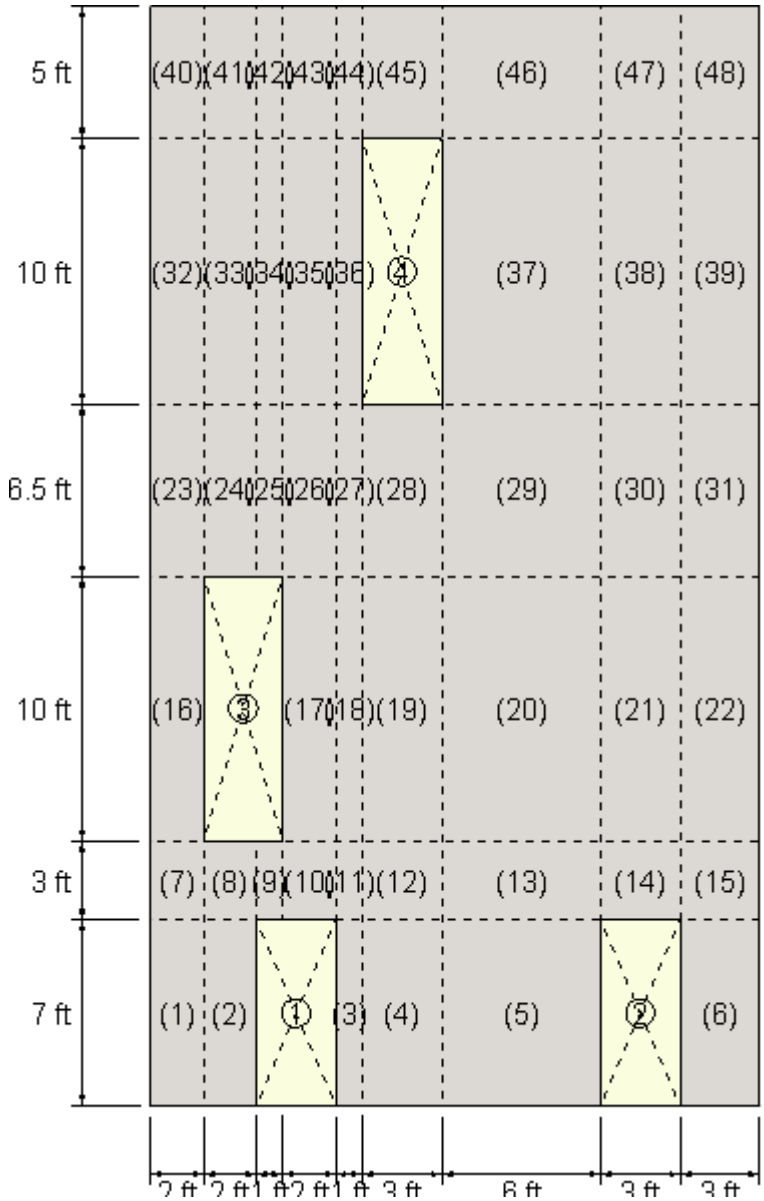
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
3	DL	Vertical	3360.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : N. G.
- Insufficient axial tension strength (Segment 1)



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.00	7.00
2	2.00	0.00	2.00	7.00
3	7.00	0.00	1.00	7.00
4	8.00	0.00	3.00	7.00
5	11.00	0.00	6.00	7.00
6	20.00	0.00	3.00	7.00
7	0.00	7.00	2.00	3.00
8	2.00	7.00	2.00	3.00
9	4.00	7.00	1.00	3.00
10	5.00	7.00	2.00	3.00
11	7.00	7.00	1.00	3.00
12	8.00	7.00	3.00	3.00
13	11.00	7.00	6.00	3.00
14	17.00	7.00	3.00	3.00
15	20.00	7.00	3.00	3.00
16	0.00	10.00	2.00	10.00

17	5.00	10.00	2.00	10.00
18	7.00	10.00	1.00	10.00
19	8.00	10.00	3.00	10.00
20	11.00	10.00	6.00	10.00
21	17.00	10.00	3.00	10.00
22	20.00	10.00	3.00	10.00
23	0.00	20.00	2.00	6.50
24	2.00	20.00	2.00	6.50
25	4.00	20.00	1.00	6.50
26	5.00	20.00	2.00	6.50
27	7.00	20.00	1.00	6.50
28	8.00	20.00	3.00	6.50
29	11.00	20.00	6.00	6.50
30	17.00	20.00	3.00	6.50
31	20.00	20.00	3.00	6.50
32	0.00	26.50	2.00	10.00
33	2.00	26.50	2.00	10.00
34	4.00	26.50	1.00	10.00
35	5.00	26.50	2.00	10.00
36	7.00	26.50	1.00	10.00
37	11.00	26.50	6.00	10.00
38	17.00	26.50	3.00	10.00
39	20.00	26.50	3.00	10.00
40	0.00	36.50	2.00	5.00
41	2.00	36.50	2.00	5.00
42	4.00	36.50	1.00	5.00
43	5.00	36.50	2.00	5.00
44	7.00	36.50	1.00	5.00
45	8.00	36.50	3.00	5.00
46	11.00	36.50	6.00	5.00
47	17.00	36.50	3.00	5.00
48	20.00	36.50	3.00	5.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	3-#10	10.00	49.19
2	3-#10	10.00	49.19
3	1-#10	10.00	49.19
4	2-#9	18.00	43.69
5	7-#6	10.00	23.24
6	3-#6	12.00	23.24
7	3-#10	10.00	49.19
8	3-#10	10.00	49.19
9	1-#9	18.00	43.69
10	3-#8	10.00	38.73
11	1-#10	10.00	49.19
12	2-#9	18.00	43.69
13	7-#6	10.00	23.24
14	3-#6	12.00	23.24
15	3-#6	12.00	23.24
16	3-#9	10.00	43.69
17	3-#8	10.00	38.73
18	1-#6	18.00	23.24
19	3-#5	14.00	19.36
20	5-#5	14.00	19.36
21	3-#5	14.00	19.36
22	3-#5	14.00	19.36

23	3-#9	10.00	43.69
24	2-#6	18.00	23.24
25	1-#5	14.00	19.36
26	3-#8	10.00	38.73
27	1-#6	18.00	23.24
28	3-#5	14.00	19.36
29	5-#5	14.00	19.36
30	3-#5	14.00	19.36
31	3-#5	14.00	19.36
32	2-#8	18.00	38.73
33	2-#7	18.00	33.89
34	1-#7	18.00	33.89
35	2-#7	18.00	33.89
36	1-#7	18.00	33.89
37	5-#5	14.00	19.36
38	3-#5	14.00	19.36
39	3-#5	14.00	19.36
40	2-#8	18.00	38.73
41	2-#7	18.00	33.89
42	1-#7	18.00	33.89
43	2-#7	18.00	33.89
44	1-#7	18.00	33.89
45	3-#5	14.00	19.36
46	5-#5	14.00	19.36
47	3-#5	14.00	19.36
48	3-#5	14.00	19.36

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	3.15	12.86
2	DC1 (Top)	3.18	12.86
3	DC1 (Top)	2.38	12.99
4	DC1 (Top)	1.67	13.06
5	DC1 (Top)	1.46	13.25
6	DC1 (Top)	1.38	13.25
7	DC1 (Top)	3.28	12.86
8	DC1 (Top)	2.63	12.86
9	DC1 (Top)	1.82	12.94
10	DC1 (Top)	2.10	13.00
11	DC1 (Top)	2.19	12.86
12	DC1 (Top)	1.69	12.94
13	DC1 (Top)	1.46	13.13
14	DC1 (Top)	1.27	13.13
15	DC1 (Top)	1.35	13.13
16	DC1 (Top)	2.43	8.06
17	DC1 (Top)	1.97	8.13
18	DC1 (Top)	1.25	8.25
19	DC1 (Top)	1.03	8.31
20	DC1 (Top)	0.93	8.31
21	DC1 (Top)	1.02	8.31
22	DC1 (Top)	1.01	8.31
23	DC1 (Top)	2.22	8.06
24	DC1 (Top)	1.21	8.25
25	DC1 (Top)	1.01	8.31
26	DC1 (Top)	1.91	8.13
27	DC1 (Top)	1.24	8.25
28	DC1 (Top)	0.91	8.31
29	DC1 (Top)	0.93	8.31
30	DC1 (Top)	1.00	8.31
31	DC1 (Top)	0.99	8.31

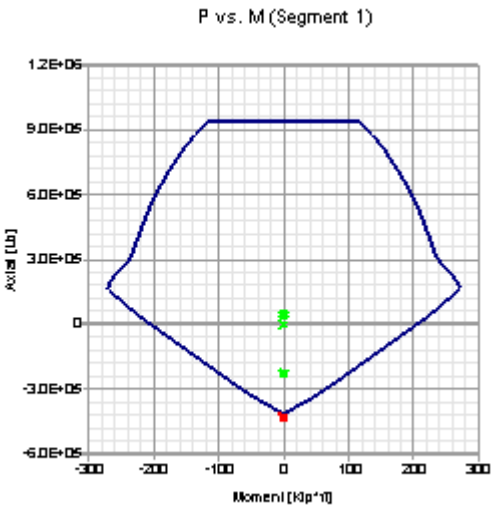
32	DC1 (Top)	1.52	6.13
33	DC1 (Top)	1.33	6.19
34	DC1 (Top)	1.34	6.19
35	DC1 (Top)	1.34	6.19
36	DC1 (Top)	1.36	6.19
37	DC1 (Top)	0.84	6.31
38	DC1 (Top)	0.93	6.31
39	DC1 (Top)	0.92	6.31
40	DC1 (Top)	1.51	6.13
41	DC1 (Top)	1.32	6.19
42	DC1 (Top)	1.32	6.19
43	DC1 (Top)	1.32	6.19
44	DC1 (Top)	1.32	6.19
45	DC1 (Top)	0.91	6.31
46	DC1 (Top)	0.79	6.31
47	DC1 (Top)	0.91	6.31
48	DC1 (Top)	0.91	6.31

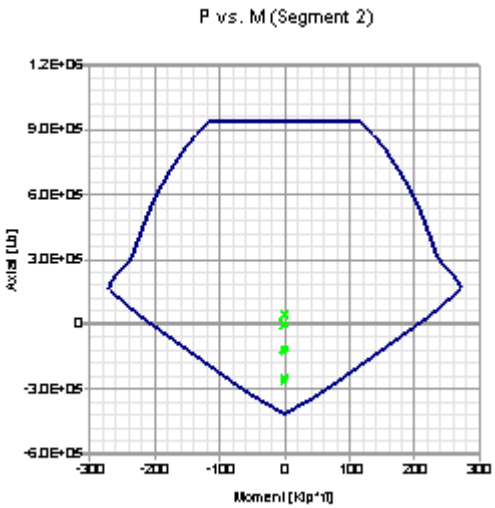
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	40352.30	0.00	222.66	0.00
2	DC1 (Top)	41953.42	0.00	223.36	0.00
3	DC1 (Top)	28734.89	0.00	85.01	0.00
4	DC1 (Top)	69682.96	0.00	154.83	0.00
5	DC1 (Top)	140633.10	0.00	262.71	0.00
6	DC1 (Top)	70783.40	0.00	119.72	0.00
7	DC1 (Top)	47834.69	0.00	225.82	0.00
8	DC1 (Top)	11924.59	0.00	209.99	0.00
9	DC1 (Top)	10349.53	0.00	62.26	0.00
10	DC1 (Top)	37198.55	0.00	151.74	0.00
11	DC1 (Top)	18656.28	0.00	79.53	0.00
12	DC1 (Top)	63590.52	0.00	151.46	0.00
13	DC1 (Top)	120110.60	0.00	251.86	0.00
14	DC1 (Top)	40085.11	0.00	103.87	0.00
15	DC1 (Top)	54736.92	0.00	111.51	0.00
16	DC1 (Top)	34872.82	0.00	108.68	0.00
17	DC1 (Top)	31447.36	0.00	90.22	0.00
18	DC1 (Top)	13692.58	0.00	22.07	0.00
19	DC1 (Top)	37270.77	0.00	51.77	0.00
20	DC1 (Top)	71844.96	0.00	91.68	0.00
21	DC1 (Top)	34596.49	0.00	50.88	0.00
22	DC1 (Top)	32870.36	0.00	50.31	0.00
23	DC1 (Top)	21811.83	0.00	105.08	0.00
24	DC1 (Top)	21397.50	0.00	42.32	0.00
25	DC1 (Top)	10939.64	0.00	16.77	0.00
26	DC1 (Top)	23600.88	0.00	87.98	0.00
27	DC1 (Top)	12943.31	0.00	21.84	0.00
28	DC1 (Top)	16214.36	0.00	44.82	0.00
29	DC1 (Top)	71995.56	0.00	91.73	0.00
30	DC1 (Top)	32590.30	0.00	50.22	0.00
31	DC1 (Top)	30636.79	0.00	49.58	0.00
32	DC1 (Top)	10306.96	0.00	45.67	0.00
33	DC1 (Top)	11438.90	0.00	37.75	0.00
34	DC1 (Top)	6222.39	0.00	18.98	0.00
35	DC1 (Top)	13489.18	0.00	38.19	0.00
36	DC1 (Top)	8226.65	0.00	19.41	0.00
37	DC1 (Top)	41230.54	0.00	63.99	0.00

38	DC1 (Top)	16352.62	0.00	35.29	0.00
39	DC1 (Top)	15209.80	0.00	35.01	0.00
40	DC1 (Top)	9543.35	0.00	45.51	0.00
41	DC1 (Top)	9587.47	0.00	37.36	0.00
42	DC1 (Top)	4820.90	0.00	18.69	0.00
43	DC1 (Top)	9616.38	0.00	37.37	0.00
44	DC1 (Top)	4821.57	0.00	18.69	0.00
45	DC1 (Top)	13981.52	0.00	34.71	0.00
46	DC1 (Top)	29181.69	0.00	60.55	0.00
47	DC1 (Top)	14357.55	0.00	34.80	0.00
48	DC1 (Top)	14316.38	0.00	34.79	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	51723.20	934511.80	0.06
2	D1 (Bottom)	43576.42	934511.80	0.05
3	D1 (Top)	28734.89	470623.90	0.06
4	DC1 (Bottom)	77566.62	1421472.00	0.05
5	D2 (Bottom)	561685.60	2847824.00	0.20
6	D2 (Bottom)	716437.50	1425079.00	0.50
7	DC1 (Top)	47834.69	934511.80	0.05
8	D1 (Bottom)	41953.15	934511.80	0.04
9	DC1 (Top)	10349.53	472056.00	0.02
10	D1 (Top)	37198.55	942149.50	0.04
11	DC1 (Bottom)	28732.38	470623.90	0.06
12	D1 (Bottom)	69682.95	1421472.00	0.05
13	D2 (Bottom)	291149.90	2847824.00	0.10
14	D2 (Top)	223971.40	1425079.00	0.16
15	D2 (Bottom)	645884.30	1425079.00	0.45
16	DC1 (Bottom)	41473.77	620568.00	0.07
17	DC1 (Top)	31447.36	623909.50	0.05
18	DC1 (Bottom)	14558.05	315906.20	0.05
19	D1 (Bottom)	48240.86	949787.30	0.05

20	D2 (Bottom)	174361.80	1901219.00	0.09
21	D2 (Bottom)	224274.50	949787.30	0.24
22	D2 (Bottom)	470551.40	949787.30	0.50
23	DC1 (Bottom)	34872.48	620568.00	0.06
24	D1 (Top)	21397.50	631812.50	0.03
25	D1 (Top)	10939.64	316595.80	0.03
26	DC1 (Bottom)	31444.05	623909.50	0.05
27	D2 (Top)	59690.48	315906.20	0.19
28	D1 (Bottom)	37270.76	949787.30	0.04
29	D2 (Bottom)	96511.26	1901219.00	0.05
30	D2 (Bottom)	183490.50	949787.30	0.19
31	D2 (Bottom)	280826.40	949787.30	0.30
32	DC1 (Bottom)	13432.60	500803.70	0.03
33	D1 (Bottom)	13700.37	502819.20	0.03
34	D2 (Top)	14879.76	251409.60	0.06
35	D2 (Bottom)	23025.23	502819.20	0.05
36	D2 (Bottom)	68137.88	251409.60	0.27
37	D2 (Top)	164071.30	1519331.00	0.11
38	D2 (Bottom)	120965.70	758843.30	0.16
39	D2 (Bottom)	185105.70	758843.30	0.24
40	D2 (Top)	75017.74	500803.70	0.15
41	D2 (Top)	23499.19	502819.20	0.05
42	D2 (Bottom)	14879.84	251409.60	0.06
43	D1 (Bottom)	13489.18	502819.20	0.03
44	D1 (Bottom)	8226.52	251409.60	0.03
45	DC1 (Top)	13981.52	758843.30	0.02
46	D2 (Bottom)	164068.60	1519331.00	0.11
47	D2 (Bottom)	25856.02	758843.30	0.03
48	D2 (Bottom)	25357.61	758843.30	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	433828.10	411480.00	1.05
2	D3 (Top)	260339.70	411480.00	0.63
3	D3 (Bottom)	178121.50	137160.00	1.30
4	D3 (Bottom)	209423.00	216000.00	0.97
5	DC1 (Top)	0.00	332640.00	0.00
6	DC1 (Top)	0.00	142560.00	0.00
7	D3 (Top)	280216.60	411480.00	0.68
8	D3 (Bottom)	260337.50	411480.00	0.63
9	D3 (Bottom)	93876.43	108000.00	0.87
10	D3 (Top)	216097.50	255960.00	0.84
11	D3 (Bottom)	52517.43	137160.00	0.38
12	D3 (Bottom)	110013.00	216000.00	0.51
13	DC1 (Top)	0.00	332640.00	0.00
14	DC1 (Top)	0.00	142560.00	0.00
15	DC1 (Top)	0.00	142560.00	0.00
16	D3 (Bottom)	292642.50	324000.00	0.90
17	D3 (Bottom)	222877.60	255960.00	0.87
18	D3 (Bottom)	44861.11	47520.00	0.94
19	D3 (Bottom)	53652.71	100440.00	0.53
20	DC1 (Top)	0.00	167400.00	0.00
21	DC1 (Top)	0.00	100440.00	0.00
22	DC1 (Top)	0.00	100440.00	0.00
23	D3 (Bottom)	254822.90	324000.00	0.79
24	D3 (Top)	84800.91	95040.00	0.89

25	D3 (Top)	15008.93	33480.00	0.45
26	D3 (Bottom)	27845.22	255960.00	0.11
27	D3 (Bottom)	16372.79	47520.00	0.34
28	D3 (Bottom)	36487.71	100440.00	0.36
29	D3 (Top)	16905.70	167400.00	0.10
30	DC1 (Top)	0.00	100440.00	0.00
31	DC1 (Top)	0.00	100440.00	0.00
32	D3 (Bottom)	151030.80	170640.00	0.89
33	D3 (Bottom)	88144.50	129600.00	0.68
34	D3 (Bottom)	18374.42	64800.00	0.28
35	DC1 (Top)	0.00	129600.00	0.00
36	D3 (Top)	52867.32	64800.00	0.82
37	D3 (Bottom)	35364.83	167400.00	0.21
38	DC1 (Top)	0.00	100440.00	0.00
39	DC1 (Top)	0.00	100440.00	0.00
40	D3 (Bottom)	91195.33	170640.00	0.53
41	DC1 (Top)	0.00	129600.00	0.00
42	DC1 (Top)	0.00	64800.00	0.00
43	DC1 (Top)	0.00	129600.00	0.00
44	D3 (Bottom)	52866.23	64800.00	0.82
45	DC1 (Top)	0.00	100440.00	0.00
46	DC1 (Top)	0.00	167400.00	0.00
47	DC1 (Top)	0.00	100440.00	0.00
48	DC1 (Top)	0.00	100440.00	0.00

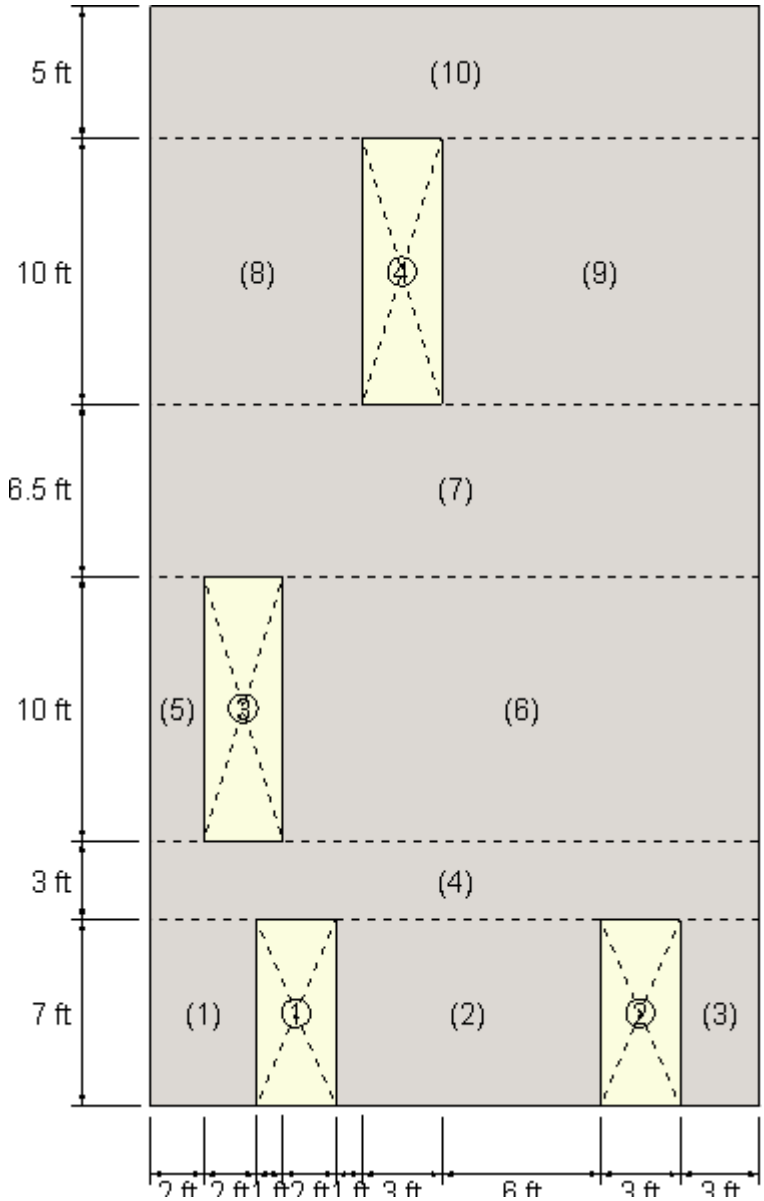
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	35874.670	0.00
2	DC1 (Top)	0.000	35874.670	0.00
3	DC1 (Top)	0.000	18111.619	0.00
4	DC1 (Top)	0.000	54631.838	0.00
5	DC1 (Top)	0.000	110844.780	0.00
6	DC1 (Top)	0.000	55422.392	0.00
7	DC1 (Top)	0.000	35874.670	0.00
8	DC1 (Top)	0.000	35874.670	0.00
9	DC1 (Top)	0.000	18036.329	0.00
10	DC1 (Top)	0.000	36251.124	0.00
11	DC1 (Top)	0.000	17937.335	0.00
12	DC1 (Top)	0.000	54108.986	0.00
13	DC1 (Top)	0.000	109799.080	0.00
14	DC1 (Top)	0.000	54899.539	0.00
15	DC1 (Top)	0.000	54899.539	0.00
16	DC1 (Top)	0.000	22478.486	0.00
17	DC1 (Top)	0.000	22656.953	0.00
18	DC1 (Top)	0.000	11502.761	0.00
19	DC1 (Top)	0.000	34769.708	0.00
20	DC1 (Top)	0.000	69539.416	0.00
21	DC1 (Top)	0.000	34769.708	0.00
22	DC1 (Top)	0.000	34769.708	0.00
23	DC1 (Top)	0.000	22478.486	0.00
24	DC1 (Top)	0.000	23005.521	0.00
25	DC1 (Top)	0.000	11589.903	0.00
26	DC1 (Top)	0.000	22656.953	0.00
27	DC1 (Top)	0.000	11502.761	0.00
28	DC1 (Top)	0.000	34769.708	0.00
29	DC1 (Top)	0.000	69539.416	0.00

30	DC1 (Top)	0.000	34769.708	0.00	
31	DC1 (Top)	0.000	34769.708	0.00	
32	DC1 (Top)	0.000	17079.857	0.00	
33	DC1 (Top)	0.000	17254.141	0.00	
34	DC1 (Top)	0.000	8627.070	0.00	
35	DC1 (Top)	0.000	17254.141	0.00	
36	DC1 (Top)	0.000	8627.070	0.00	
37	DC1 (Top)	0.000	52808.128	0.00	
38	DC1 (Top)	0.000	26404.064	0.00	
39	DC1 (Top)	0.000	26404.064	0.00	
40	DC1 (Top)	0.000	17079.857	0.00	
41	DC1 (Top)	0.000	17254.141	0.00	
42	DC1 (Top)	0.000	8627.070	0.00	
43	DC1 (Top)	0.000	17254.141	0.00	
44	DC1 (Top)	0.000	8627.070	0.00	
45	DC1 (Top)	0.000	26404.064	0.00	
46	DC1 (Top)	0.000	52808.128	0.00	
47	DC1 (Top)	0.000	26404.064	0.00	
48	DC1 (Top)	0.000	26404.064	0.00	

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	4.00	7.00
2	7.00	0.00	10.00	7.00
3	20.00	0.00	3.00	7.00
4	0.00	7.00	23.00	3.00
5	0.00	10.00	2.00	10.00
6	5.00	10.00	18.00	10.00
7	0.00	20.00	23.00	6.50
8	0.00	26.50	8.00	10.00
9	11.00	26.50	12.00	10.00
10	0.00	36.50	23.00	5.00

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	3-#10	10.00	49.19	6-#6	14.00	30.21
	3-#10	10.00	49.19	6-#6	14.00	30.21
2	1-#10	10.00	49.19	6-#5	16.00	25.17
	2-#9	18.00	49.19	6-#5	16.00	25.17
	7-#6	10.00	49.19	6-#5	16.00	25.17
3	3-#6	12.00	23.24	5-#5	18.00	25.17
4	3-#10	10.00	49.19	2-#6	18.00	30.21
	3-#10	10.00	49.19	2-#6	18.00	30.21
	1-#9	18.00	49.19	2-#6	18.00	30.21
	3-#8	10.00	49.19	2-#6	18.00	30.21
	1-#10	10.00	49.19	2-#6	18.00	30.21
	2-#9	18.00	49.19	2-#6	18.00	30.21
	7-#6	10.00	49.19	2-#6	18.00	30.21
	3-#6	12.00	49.19	2-#6	18.00	30.21
	3-#6	12.00	49.19	2-#6	18.00	30.21
5	3-#9	10.00	43.69	7-#5	18.00	25.17
6	3-#8	10.00	38.73	7-#5	18.00	25.17
	1-#6	18.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
	5-#5	14.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
7	3-#9	10.00	43.69	5-#5	18.00	25.17
	2-#6	18.00	43.69	5-#5	18.00	25.17
	1-#5	14.00	43.69	5-#5	18.00	25.17
	3-#8	10.00	43.69	5-#5	18.00	25.17
	1-#6	18.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
	5-#5	14.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
8	2-#8	18.00	38.73	7-#5	18.00	25.17
	2-#7	18.00	38.73	7-#5	18.00	25.17
	1-#7	18.00	38.73	7-#5	18.00	25.17
	2-#7	18.00	38.73	7-#5	18.00	25.17
	1-#7	18.00	38.73	7-#5	18.00	25.17
9	5-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
10	2-#8	18.00	38.73	4-#5	18.00	25.17
	2-#7	18.00	38.73	4-#5	18.00	25.17
	1-#7	18.00	38.73	4-#5	18.00	25.17
	2-#7	18.00	38.73	4-#5	18.00	25.17
	1-#7	18.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17
	5-#5	14.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17

Intermediate results for axial-bending

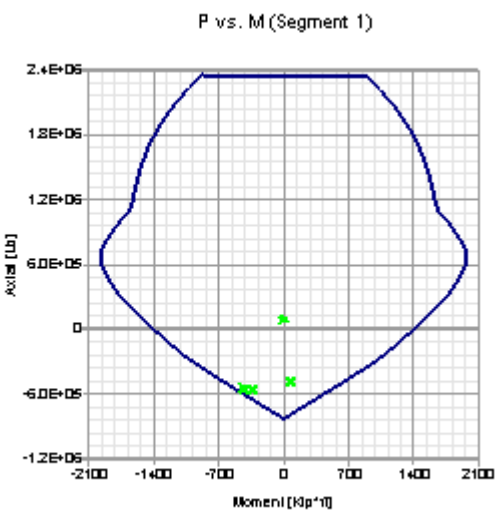
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	2.05	38.40
2	D3 (Bottom)	11.74	96.00
3	D2 (Bottom)	19.10	28.80
4	D3 (Bottom)	31.60	220.80
5	D3 (Bottom)	0.61	19.20

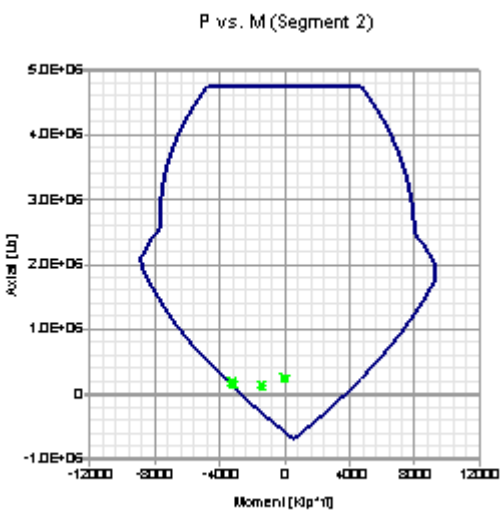
6	D3 (Max)	26.84	172.80
7	D3 (Bottom)	29.49	220.80
8	D3 (Bottom)	7.88	76.80
9	D3 (Bottom)	20.50	115.20
10	D3 (Bottom)	30.82	220.80

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	-555613.60	-431.44	508.01	0.85
2	D3 (Bottom)	140215.50	-3201.56	3250.97	0.98
3	D2 (Bottom)	716407.70	-232.70	729.57	0.32
4	D3 (Bottom)	181808.20	-13263.75	16182.40	0.82
5	D3 (Bottom)	-292589.40	-20.36	29.81	0.68
6	D3 (Max)	457995.20	-6960.61	8276.19	0.84
7	D3 (Bottom)	169543.70	-7604.02	10453.93	0.73
8	D3 (Bottom)	-171763.10	-839.59	1302.88	0.64
9	D3 (Bottom)	262165.50	-1423.27	3457.22	0.41
10	D3 (Bottom)	80427.17	-1663.74	9039.71	0.18

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	95304.88	2344512.00	0.04
2	DC1 (Bottom)	264716.50	4739920.00	0.06
3	D2 (Max)	728146.90	1425079.00	0.51
4	DC1 (Bottom)	422433.10	1.087331E07	0.04
5	DC1 (Bottom)	41478.04	807768.00	0.05
6	D2 (Bottom)	567412.50	5690396.00	0.10
7	DC1 (Bottom)	267793.90	7259373.00	0.04
8	DC1 (Bottom)	60218.63	2332493.00	0.03
9	D2 (Bottom)	281142.40	3037017.00	0.09

10	D2 (Top)	184128.70	5805122.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Max)	567332.50	822960.00	0.69
2	DC1 (Top)	0.00	685800.00	0.00
3	DC1 (Top)	0.00	142560.00	0.00
4	DC1 (Top)	0.00	2157840.00	0.00
5	D3 (Bottom)	292589.40	324000.00	0.90
6	DC1 (Top)	0.00	772200.00	0.00
7	DC1 (Top)	0.00	1224720.00	0.00
8	D3 (Bottom)	171763.10	559440.00	0.31
9	DC1 (Top)	0.00	368280.00	0.00
10	DC1 (Top)	0.00	1028160.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Bottom)	100596.220	108617.140	0.93
2	D3 (Max)	328685.950	465338.110	0.71
3	D3 (Bottom)	62412.208	217169.650	0.29
4	D3 (Max)	511999.200	1039453.600	0.49
5	D3 (Top)	7027.482	29760.000	0.24
6	D3 (Top)	414691.140	666346.850	0.62
7	D3 (Bottom)	419051.550	790973.140	0.53
8	D3 (Bottom)	91971.404	177899.640	0.52
9	D3 (Top)	240182.340	385216.190	0.62
10	D3 (Max)	338275.380	693428.130	0.49

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 10:47 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 7.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	26.50 [ft]
Total length	:	64.75 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	7.50	13.00	4.00	8.00
Lower left	19.00	13.00	4.00	8.00
Lower left	30.25	13.00	4.00	8.00
Lower left	41.75	13.00	4.00	8.00
Lower left	53.25	13.00	4.00	8.00
Lower left	30.25	3.50	4.00	5.00
Lower left	41.75	3.50	4.00	5.00
Lower left	53.25	3.50	4.00	5.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
2	EQ	Horizontal	452174.00	0.00	0.00
1	EQ	Horizontal	97868.00	0.00	0.00

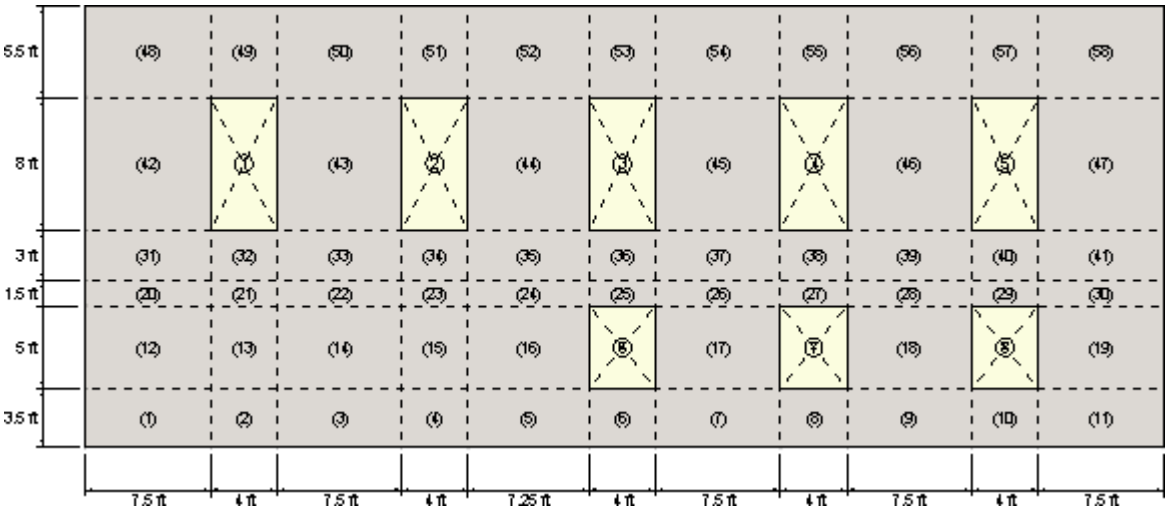
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
2	DL	Vertical	5850.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	7.50	3.50
2	7.50	0.00	4.00	3.50
3	11.50	0.00	7.50	3.50
4	19.00	0.00	4.00	3.50
5	23.00	0.00	7.25	3.50
6	30.25	0.00	4.00	3.50
7	34.25	0.00	7.50	3.50
8	41.75	0.00	4.00	3.50
9	45.75	0.00	7.50	3.50
10	53.25	0.00	4.00	3.50
11	57.25	0.00	7.50	3.50
12	0.00	3.50	7.50	5.00
13	7.50	3.50	4.00	5.00
14	11.50	3.50	7.50	5.00
15	19.00	3.50	4.00	5.00
16	23.00	3.50	7.25	5.00
17	34.25	3.50	7.50	5.00
18	45.75	3.50	7.50	5.00
19	57.25	3.50	7.50	5.00
20	0.00	8.50	7.50	1.50
21	7.50	8.50	4.00	1.50
22	11.50	8.50	7.50	1.50
23	19.00	8.50	4.00	1.50
24	23.00	8.50	7.25	1.50
25	30.25	8.50	4.00	1.50
26	34.25	8.50	7.50	1.50
27	41.75	8.50	4.00	1.50
28	45.75	8.50	7.50	1.50
29	53.25	8.50	4.00	1.50
30	57.25	8.50	7.50	1.50
31	0.00	10.00	7.50	3.00
32	7.50	10.00	4.00	3.00
33	11.50	10.00	7.50	3.00
34	19.00	10.00	4.00	3.00
35	23.00	10.00	7.25	3.00
36	30.25	10.00	4.00	3.00
37	34.25	10.00	7.50	3.00
38	41.75	10.00	4.00	3.00
39	45.75	10.00	7.50	3.00
40	53.25	10.00	4.00	3.00
41	57.25	10.00	7.50	3.00
42	0.00	13.00	7.50	8.00
43	11.50	13.00	7.50	8.00
44	23.00	13.00	7.25	8.00
45	34.25	13.00	7.50	8.00
46	45.75	13.00	7.50	8.00
47	57.25	13.00	7.50	8.00
48	0.00	21.00	7.50	5.50
49	7.50	21.00	4.00	5.50
50	11.50	21.00	7.50	5.50
51	19.00	21.00	4.00	5.50
52	23.00	21.00	7.25	5.50
53	30.25	21.00	4.00	5.50
54	34.25	21.00	7.50	5.50
55	41.75	21.00	4.00	5.50
56	45.75	21.00	7.50	5.50
57	53.25	21.00	4.00	5.50
58	57.25	21.00	7.50	5.50

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#5	14.00	19.36
2	3-#5	18.00	19.36
3	5-#5	18.00	19.36
4	3-#5	18.00	19.36
5	5-#5	18.00	19.36
6	3-#5	18.00	19.36
7	5-#5	18.00	19.36
8	3-#5	18.00	19.36
9	5-#5	18.00	19.36
10	3-#5	18.00	19.36
11	5-#5	18.00	19.36
12	7-#5	14.00	19.36
13	3-#5	18.00	19.36
14	5-#5	18.00	19.36
15	3-#5	18.00	19.36
16	5-#5	18.00	19.36
17	5-#5	18.00	19.36
18	5-#5	18.00	19.36
19	5-#5	18.00	19.36
20	7-#5	14.00	19.36
21	3-#5	18.00	19.36
22	5-#5	18.00	19.36
23	3-#5	18.00	19.36
24	5-#5	18.00	19.36
25	3-#5	18.00	19.36
26	5-#5	18.00	19.36
27	3-#5	18.00	19.36
28	5-#5	18.00	19.36
29	3-#5	18.00	19.36
30	5-#5	18.00	19.36
31	8-#5	12.00	19.36
32	3-#5	18.00	19.36
33	6-#5	16.00	19.36
34	3-#5	18.00	19.36
35	5-#5	18.00	19.36
36	3-#5	18.00	19.36
37	5-#5	18.00	19.36
38	3-#5	18.00	19.36
39	5-#5	18.00	19.36
40	3-#5	18.00	19.36
41	5-#5	18.00	19.36
42	8-#5	12.00	19.36
43	6-#5	16.00	19.36
44	5-#5	18.00	19.36
45	5-#5	18.00	19.36
46	5-#5	18.00	19.36
47	5-#5	18.00	19.36
48	8-#5	12.00	19.36
49	3-#5	18.00	19.36
50	6-#5	16.00	19.36
51	3-#5	18.00	19.36
52	5-#5	18.00	19.36
53	3-#5	18.00	19.36
54	5-#5	18.00	19.36
55	3-#5	18.00	19.36
56	5-#5	18.00	19.36

57	3-#5	18.00	19.36
58	5-#5	18.00	19.36

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.08	13.31
2	DC1 (Top)	0.95	13.31
3	DC1 (Top)	0.91	13.31
4	DC1 (Top)	0.96	13.31
5	DC1 (Top)	0.97	13.31
6	DC1 (Top)	0.82	13.31
7	DC1 (Top)	0.99	13.31
8	DC1 (Top)	0.80	13.31
9	DC1 (Top)	1.00	13.31
10	DC1 (Top)	0.73	13.31
11	DC1 (Top)	0.99	13.31
12	DC1 (Top)	1.05	13.31
13	DC1 (Top)	0.91	13.31
14	DC1 (Top)	0.90	13.31
15	DC1 (Top)	0.92	13.31
16	DC1 (Top)	0.97	13.31
17	DC1 (Top)	0.99	13.31
18	DC1 (Top)	0.98	13.31
19	DC1 (Top)	0.96	13.31
20	DC1 (Top)	1.05	13.31
21	DC1 (Top)	0.91	13.31
22	DC1 (Top)	0.90	13.31
23	DC1 (Top)	0.92	13.31
24	DC1 (Top)	0.91	13.31
25	DC1 (Top)	0.78	13.31
26	DC1 (Top)	0.96	13.31
27	DC1 (Top)	0.70	13.31
28	DC1 (Top)	0.96	13.31
29	DC1 (Top)	0.79	13.31
30	DC1 (Top)	0.93	13.31
31	DC1 (Top)	1.06	8.31
32	DC1 (Top)	0.73	8.31
33	DC1 (Top)	0.95	8.31
34	DC1 (Top)	0.75	8.31
35	DC1 (Top)	0.88	8.31
36	DC1 (Top)	0.67	8.31
37	DC1 (Top)	0.88	8.31
38	DC1 (Top)	0.71	8.31
39	DC1 (Top)	0.87	8.31
40	DC1 (Top)	0.70	8.31
41	DC1 (Top)	0.84	8.31
42	DC1 (Top)	1.04	8.31
43	DC1 (Top)	0.95	8.31
44	DC1 (Top)	0.84	8.31
45	DC1 (Top)	0.81	8.31
46	DC1 (Top)	0.83	8.31
47	DC1 (Top)	0.82	8.31
48	DC1 (Top)	1.00	8.31
49	DC1 (Top)	0.80	8.31
50	DC1 (Top)	0.86	8.31
51	DC1 (Top)	0.79	8.31
52	DC1 (Top)	0.77	8.31
53	DC1 (Top)	0.80	8.31
54	DC1 (Top)	0.75	8.31
55	DC1 (Top)	0.80	8.31

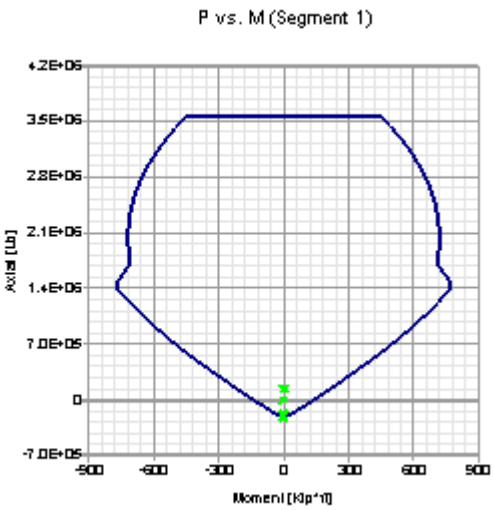
56	DC1 (Top)	0.75	8.31
57	DC1 (Top)	0.80	8.31
58	DC1 (Top)	0.75	8.31

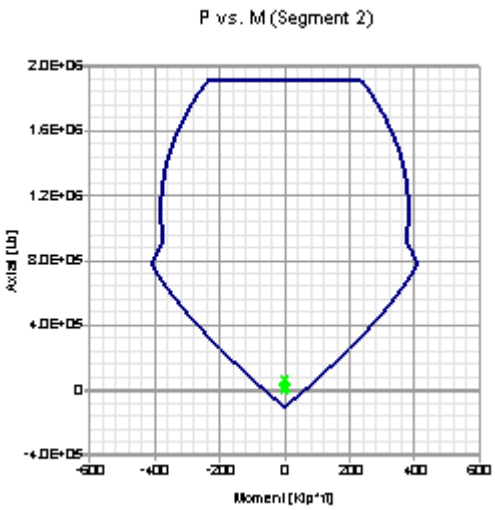
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	133977.40	0.00	213.48	0.00
2	DC1 (Top)	63791.68	0.00	96.75	0.00
3	DC1 (Top)	121149.50	0.00	171.03	0.00
4	DC1 (Top)	66769.26	0.00	98.35	0.00
5	DC1 (Top)	138737.50	0.00	179.97	0.00
6	DC1 (Top)	34448.25	0.00	80.81	0.00
7	DC1 (Top)	154621.30	0.00	189.07	0.00
8	DC1 (Top)	31094.30	0.00	78.84	0.00
9	DC1 (Top)	158886.50	0.00	191.37	0.00
10	DC1 (Top)	20767.41	0.00	72.76	0.00
11	DC1 (Top)	157062.80	0.00	190.38	0.00
12	DC1 (Top)	120457.90	0.00	206.26	0.00
13	DC1 (Top)	55163.40	0.00	92.11	0.00
14	DC1 (Top)	118516.60	0.00	169.62	0.00
15	DC1 (Top)	57722.12	0.00	93.49	0.00
16	DC1 (Top)	138027.70	0.00	179.58	0.00
17	DC1 (Top)	156668.70	0.00	190.17	0.00
18	DC1 (Top)	151275.20	0.00	187.27	0.00
19	DC1 (Top)	142625.20	0.00	182.61	0.00
20	DC1 (Top)	120142.10	0.00	206.10	0.00
21	DC1 (Top)	54636.84	0.00	91.83	0.00
22	DC1 (Top)	118321.60	0.00	169.51	0.00
23	DC1 (Top)	57230.47	0.00	93.22	0.00
24	DC1 (Top)	113761.60	0.00	166.52	0.00
25	DC1 (Top)	28953.27	0.00	77.58	0.00
26	DC1 (Top)	143081.00	0.00	182.85	0.00
27	DC1 (Top)	16033.48	0.00	69.98	0.00
28	DC1 (Top)	144720.50	0.00	183.73	0.00
29	DC1 (Top)	30649.63	0.00	78.58	0.00
30	DC1 (Top)	132385.20	0.00	177.09	0.00
31	DC1 (Top)	89381.71	0.00	133.58	0.00
32	DC1 (Top)	20767.09	0.00	47.61	0.00
33	DC1 (Top)	105511.50	0.00	117.07	0.00
34	DC1 (Top)	22847.96	0.00	48.38	0.00
35	DC1 (Top)	98871.05	0.00	103.22	0.00
36	DC1 (Top)	10011.76	0.00	43.59	0.00
37	DC1 (Top)	106130.00	0.00	106.14	0.00
38	DC1 (Top)	17646.97	0.00	46.44	0.00
39	DC1 (Top)	103958.70	0.00	105.41	0.00
40	DC1 (Top)	15531.68	0.00	45.65	0.00
41	DC1 (Top)	93908.05	0.00	101.98	0.00
42	DC1 (Top)	83838.01	0.00	131.76	0.00
43	DC1 (Top)	104227.90	0.00	116.64	0.00
44	DC1 (Top)	83928.14	0.00	98.10	0.00
45	DC1 (Top)	84884.14	0.00	98.61	0.00
46	DC1 (Top)	88227.70	0.00	99.85	0.00
47	DC1 (Top)	85226.88	0.00	98.73	0.00
48	DC1 (Top)	64672.87	0.00	125.46	0.00
49	DC1 (Top)	32324.23	0.00	51.92	0.00
50	DC1 (Top)	67422.16	0.00	104.37	0.00

51	DC1 (Top)	30061.14	0.00	51.08	0.00
52	DC1 (Top)	63765.22	0.00	90.57	0.00
53	DC1 (Top)	32229.18	0.00	51.89	0.00
54	DC1 (Top)	65875.70	0.00	91.51	0.00
55	DC1 (Top)	31245.44	0.00	51.52	0.00
56	DC1 (Top)	66511.73	0.00	91.75	0.00
57	DC1 (Top)	32244.99	0.00	51.89	0.00
58	DC1 (Top)	64856.66	0.00	91.13	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	141293.90	3568690.00	0.04
2	D1 (Bottom)	66562.84	1904507.00	0.03
3	D2 (Top)	137524.90	3571979.00	0.04
4	D2 (Bottom)	97791.29	1904507.00	0.05
5	D2 (Top)	220249.10	3452639.00	0.06
6	D2 (Top)	47963.63	1904507.00	0.03
7	D2 (Top)	164660.60	3571979.00	0.05
8	D2 (Bottom)	59018.54	1904507.00	0.03
9	D2 (Top)	164631.10	3571979.00	0.05
10	D2 (Bottom)	54163.25	1904507.00	0.03
11	D2 (Bottom)	217336.40	3571979.00	0.06
12	DC1 (Bottom)	133977.40	3568690.00	0.04
13	D1 (Bottom)	63791.68	1904507.00	0.03
14	D2 (Top)	151941.90	3571979.00	0.04
15	D2 (Bottom)	96896.10	1904507.00	0.05
16	D2 (Bottom)	220249.20	3452639.00	0.06
17	D2 (Top)	167947.60	3571979.00	0.05
18	D2 (Bottom)	164631.60	3571979.00	0.05
19	D2 (Bottom)	206535.50	3571979.00	0.06
20	D1 (Bottom)	120457.90	3568690.00	0.03
21	D1 (Bottom)	55163.40	1904507.00	0.03
22	D2 (Top)	156184.80	3571979.00	0.04

23	D2 (Top)	90152.36	1904507.00	0.05
24	D2 (Bottom)	175060.00	3452639.00	0.05
25	D2 (Bottom)	71778.30	1904507.00	0.04
26	D2 (Bottom)	167946.60	3571979.00	0.05
27	D2 (Bottom)	79643.82	1904507.00	0.04
28	DC1 (Bottom)	151274.60	3571979.00	0.04
29	D2 (Top)	42833.69	1904507.00	0.02
30	D2 (Bottom)	173202.10	3571979.00	0.05
31	DC1 (Top)	89381.71	2373646.00	0.04
32	DC1 (Bottom)	29402.74	1268027.00	0.02
33	D2 (Bottom)	139820.30	2376935.00	0.06
34	D2 (Top)	61258.00	1268027.00	0.05
35	D1 (Top)	98871.05	2299019.00	0.04
36	D2 (Bottom)	29173.07	1268027.00	0.02
37	D2 (Top)	107751.20	2378579.00	0.05
38	D2 (Bottom)	31020.72	1268027.00	0.02
39	DC1 (Top)	103958.70	2378579.00	0.04
40	D2 (Top)	21839.27	1268027.00	0.02
41	D2 (Top)	124817.50	2378579.00	0.05
42	D1 (Bottom)	89381.83	2373646.00	0.04
43	D2 (Top)	166548.10	2376935.00	0.07
44	D1 (Bottom)	98871.58	2299019.00	0.04
45	D2 (Bottom)	107751.90	2378579.00	0.05
46	DC1 (Bottom)	103959.00	2378579.00	0.04
47	D2 (Bottom)	124817.40	2378579.00	0.05
48	D2 (Top)	180347.20	2373646.00	0.08
49	D2 (Top)	61430.72	1268027.00	0.05
50	D2 (Bottom)	97671.88	2376935.00	0.04
51	D2 (Top)	50642.55	1268027.00	0.04
52	D1 (Bottom)	83927.16	2299019.00	0.04
53	DC1 (Top)	32229.18	1268027.00	0.03
54	DC1 (Bottom)	84883.18	2378579.00	0.04
55	DC1 (Top)	31245.44	1268027.00	0.02
56	D1 (Bottom)	88226.71	2378579.00	0.04
57	DC1 (Top)	32244.99	1268027.00	0.03
58	D1 (Bottom)	77556.71	2378579.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	219277.60	234360.00	0.94
2	DC1 (Top)	0.00	100440.00	0.00
3	DC1 (Top)	0.00	167400.00	0.00
4	DC1 (Top)	0.00	100440.00	0.00
5	DC1 (Top)	0.00	167400.00	0.00
6	DC1 (Top)	0.00	100440.00	0.00
7	DC1 (Top)	0.00	167400.00	0.00
8	DC1 (Top)	0.00	100440.00	0.00
9	DC1 (Top)	0.00	167400.00	0.00
10	DC1 (Top)	0.00	100440.00	0.00
11	DC1 (Top)	0.00	167400.00	0.00
12	D3 (Bottom)	206050.80	234360.00	0.88
13	DC1 (Top)	0.00	100440.00	0.00
14	DC1 (Top)	0.00	167400.00	0.00
15	DC1 (Top)	0.00	100440.00	0.00
16	DC1 (Top)	0.00	167400.00	0.00
17	DC1 (Top)	0.00	167400.00	0.00

18	DC1 (Top)	0.00	167400.00	0.00
19	DC1 (Top)	0.00	167400.00	0.00
20	D3 (Top)	162917.00	234360.00	0.70
21	DC1 (Top)	0.00	100440.00	0.00
22	DC1 (Top)	0.00	167400.00	0.00
23	DC1 (Top)	0.00	100440.00	0.00
24	DC1 (Top)	0.00	167400.00	0.00
25	DC1 (Top)	0.00	100440.00	0.00
26	DC1 (Top)	0.00	167400.00	0.00
27	DC1 (Top)	0.00	100440.00	0.00
28	DC1 (Top)	0.00	167400.00	0.00
29	DC1 (Top)	0.00	100440.00	0.00
30	DC1 (Top)	0.00	167400.00	0.00
31	D3 (Bottom)	151825.40	267840.00	0.57
32	DC1 (Top)	0.00	100440.00	0.00
33	DC1 (Top)	0.00	200880.00	0.00
34	DC1 (Top)	0.00	100440.00	0.00
35	DC1 (Top)	0.00	167400.00	0.00
36	DC1 (Top)	0.00	100440.00	0.00
37	DC1 (Top)	0.00	167400.00	0.00
38	DC1 (Top)	0.00	100440.00	0.00
39	DC1 (Top)	0.00	167400.00	0.00
40	DC1 (Top)	0.00	100440.00	0.00
41	DC1 (Top)	0.00	167400.00	0.00
42	D3 (Bottom)	127861.10	267840.00	0.48
43	DC1 (Top)	0.00	200880.00	0.00
44	DC1 (Top)	0.00	167400.00	0.00
45	DC1 (Top)	0.00	167400.00	0.00
46	DC1 (Top)	0.00	167400.00	0.00
47	DC1 (Top)	0.00	167400.00	0.00
48	DC1 (Top)	0.00	267840.00	0.00
49	DC1 (Top)	0.00	100440.00	0.00
50	DC1 (Top)	0.00	200880.00	0.00
51	DC1 (Top)	0.00	100440.00	0.00
52	DC1 (Top)	0.00	167400.00	0.00
53	DC1 (Top)	0.00	100440.00	0.00
54	DC1 (Top)	0.00	167400.00	0.00
55	DC1 (Top)	0.00	100440.00	0.00
56	DC1 (Top)	0.00	167400.00	0.00
57	DC1 (Top)	0.00	100440.00	0.00
58	DC1 (Top)	0.00	167400.00	0.00

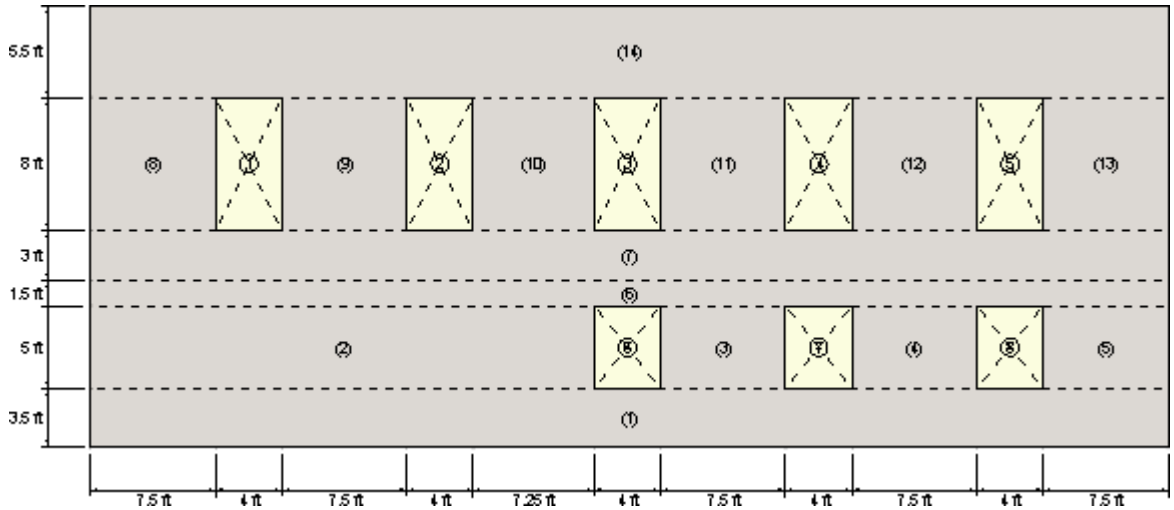
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	139209.550	0.00
2	DC1 (Top)	0.000	74245.091	0.00
3	DC1 (Top)	0.000	139209.550	0.00
4	DC1 (Top)	0.000	74245.091	0.00
5	DC1 (Top)	0.000	134569.230	0.00
6	DC1 (Top)	0.000	74245.091	0.00
7	DC1 (Top)	0.000	139209.550	0.00
8	DC1 (Top)	0.000	74245.091	0.00
9	DC1 (Top)	0.000	139209.550	0.00
10	DC1 (Top)	0.000	74245.091	0.00
11	DC1 (Top)	0.000	139209.550	0.00
12	DC1 (Top)	0.000	139209.550	0.00

13	DC1 (Top)	0.000	74245.091	0.00
14	DC1 (Top)	0.000	139209.550	0.00
15	DC1 (Top)	0.000	74245.091	0.00
16	DC1 (Top)	0.000	134569.230	0.00
17	DC1 (Top)	0.000	139209.550	0.00
18	DC1 (Top)	0.000	139209.550	0.00
19	DC1 (Top)	0.000	139209.550	0.00
20	DC1 (Top)	0.000	139209.550	0.00
21	DC1 (Top)	0.000	74245.091	0.00
22	DC1 (Top)	0.000	139209.550	0.00
23	DC1 (Top)	0.000	74245.091	0.00
24	DC1 (Top)	0.000	134569.230	0.00
25	DC1 (Top)	0.000	74245.091	0.00
26	DC1 (Top)	0.000	139209.550	0.00
27	DC1 (Top)	0.000	74245.091	0.00
28	DC1 (Top)	0.000	139209.550	0.00
29	DC1 (Top)	0.000	74245.091	0.00
30	DC1 (Top)	0.000	139209.550	0.00
31	DC1 (Top)	0.000	86924.270	0.00
32	DC1 (Top)	0.000	46359.611	0.00
33	DC1 (Top)	0.000	86924.270	0.00
34	DC1 (Top)	0.000	46359.611	0.00
35	DC1 (Top)	0.000	84026.794	0.00
36	DC1 (Top)	0.000	46359.611	0.00
37	DC1 (Top)	0.000	86924.270	0.00
38	DC1 (Top)	0.000	46359.611	0.00
39	DC1 (Top)	0.000	86924.270	0.00
40	DC1 (Top)	0.000	46359.611	0.00
41	DC1 (Top)	0.000	86924.270	0.00
42	DC1 (Top)	0.000	86924.270	0.00
43	DC1 (Top)	0.000	86924.270	0.00
44	DC1 (Top)	0.000	84026.794	0.00
45	DC1 (Top)	0.000	86924.270	0.00
46	DC1 (Top)	0.000	86924.270	0.00
47	DC1 (Top)	0.000	86924.270	0.00
48	DC1 (Top)	0.000	86924.270	0.00
49	DC1 (Top)	0.000	46359.611	0.00
50	DC1 (Top)	0.000	86924.270	0.00
51	DC1 (Top)	0.000	46359.611	0.00
52	DC1 (Top)	0.000	84026.794	0.00
53	DC1 (Top)	0.000	46359.611	0.00
54	DC1 (Top)	0.000	86924.270	0.00
55	DC1 (Top)	0.000	46359.611	0.00
56	DC1 (Top)	0.000	86924.270	0.00
57	DC1 (Top)	0.000	46359.611	0.00
58	DC1 (Top)	0.000	86924.270	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	64.75	3.50
2	0.00	3.50	30.25	5.00
3	34.25	3.50	7.50	5.00
4	45.75	3.50	7.50	5.00
5	57.25	3.50	7.50	5.00
6	0.00	8.50	64.75	1.50
7	0.00	10.00	64.75	3.00
8	0.00	13.00	7.50	8.00
9	11.50	13.00	7.50	8.00
10	23.00	13.00	7.25	8.00
11	34.25	13.00	7.50	8.00
12	45.75	13.00	7.50	8.00
13	57.25	13.00	7.50	8.00
14	0.00	21.00	64.75	5.50

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	7-#5	14.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
2	7-#5	14.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17

3	5-#5	18.00	19.36	4-#5	18.00	25.17
4	5-#5	18.00	19.36	4-#5	18.00	25.17
5	5-#5	18.00	19.36	4-#5	18.00	25.17
6	7-#5	14.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
7	8-#5	12.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	6-#5	16.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
8	8-#5	12.00	19.36	6-#5	18.00	25.17
9	6-#5	16.00	19.36	6-#5	18.00	25.17
10	5-#5	18.00	19.36	6-#5	18.00	25.17
11	5-#5	18.00	19.36	6-#5	18.00	25.17
12	5-#5	18.00	19.36	6-#5	18.00	25.17
13	5-#5	18.00	19.36	6-#5	18.00	25.17
14	8-#5	12.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	6-#5	16.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17

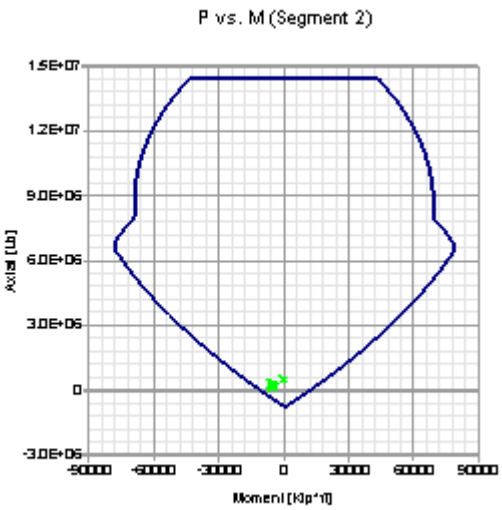
Intermediate results for axial-bending

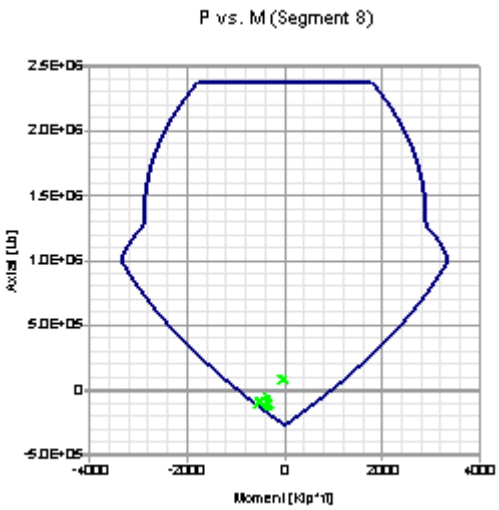
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	41.75	621.60
2	D3 (Bottom)	17.31	290.40
3	D3 (Max)	5.91	72.00
4	D3 (Bottom)	5.19	72.00
5	D3 (Bottom)	6.00	72.00
6	D3 (Top)	39.21	621.60
7	D3 (Bottom)	54.07	621.60
8	D3 (Max)	3.97	72.00
9	D3 (Top)	9.11	72.00
10	D3 (Max)	7.00	69.60
11	D3 (Max)	6.55	72.00
12	D3 (Max)	6.30	72.00
13	D2 (Max)	6.60	72.00
14	D3 (Top)	59.02	621.60

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	720391.50	-12134.34	68982.33	0.18
2	D3 (Bottom)	186904.80	-5445.74	13171.36	0.41
3	D3 (Max)	168176.80	-124.46	1196.97	0.10
4	D3 (Bottom)	131072.00	-116.08	1072.14	0.11
5	D3 (Bottom)	172878.50	-126.05	1212.53	0.10
6	D3 (Top)	581627.90	-8347.63	64780.87	0.13
7	D3 (Bottom)	423279.10	-7144.26	59385.40	0.12
8	D3 (Max)	-100430.60	-517.42	605.10	0.86
9	D3 (Top)	144249.10	416.70	1199.79	0.35
10	D3 (Max)	104050.20	248.96	925.83	0.27
11	D3 (Max)	88656.92	159.56	908.58	0.18
12	D3 (Max)	80062.16	137.79	880.14	0.16
13	D2 (Max)	90515.60	113.93	914.73	0.12
14	D3 (Top)	526511.40	4999.99	69180.97	0.07

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	1098205.00	3.083178E07	0.04
2	DC1 (Bottom)	524435.40	1.440232E07	0.04
3	D2 (Max)	205589.20	3571979.00	0.06
4	D2 (Max)	188137.10	3571979.00	0.05
5	D2 (Max)	210672.10	3571979.00	0.06
6	DC1 (Bottom)	1027537.00	3.083178E07	0.03
7	D1 (Top)	684478.00	2.052547E07	0.03
8	D1 (Max)	89792.25	2373646.00	0.04
9	D2 (Max)	179050.80	2376935.00	0.08
10	D2 (Max)	126691.50	2299019.00	0.06
11	D2 (Max)	111061.30	2378579.00	0.05

12	DC1 (Max)	106057.80	2378579.00	0.04
13	D2 (Bottom)	124714.20	2378579.00	0.05
14	D2 (Top)	644626.60	2.052547E07	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	1573560.00	0.00
2	DC1 (Top)	0.00	770040.00	0.00
3	DC1 (Top)	0.00	167400.00	0.00
4	DC1 (Top)	0.00	167400.00	0.00
5	DC1 (Top)	0.00	167400.00	0.00
6	DC1 (Top)	0.00	1573560.00	0.00
7	DC1 (Top)	0.00	1640520.00	0.00
8	D3 (Bottom)	127095.80	267840.00	0.47
9	DC1 (Top)	0.00	200880.00	0.00
10	DC1 (Top)	0.00	167400.00	0.00
11	DC1 (Top)	0.00	167400.00	0.00
12	DC1 (Top)	0.00	167400.00	0.00
13	DC1 (Top)	0.00	167400.00	0.00
14	DC1 (Top)	0.00	1640520.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	553128.030	2858435.400	0.19
2	D3 (Top)	362052.520	1314814.700	0.28
3	D2 (Max)	92113.112	349488.080	0.26
4	D2 (Max)	79069.085	346870.250	0.23
5	D2 (Top)	51699.419	344637.340	0.15
6	D3 (Max)	610988.270	2856002.800	0.21
7	D3 (Max)	470904.870	2219068.100	0.21
8	D3 (Max)	37778.003	167034.570	0.23
9	D3 (Bottom)	174960.890	260663.230	0.67
10	D3 (Max)	118123.970	256919.560	0.46
11	D3 (Max)	84303.169	262931.670	0.32
12	D3 (Max)	53566.807	261642.450	0.20
13	D2 (Max)	30360.186	263210.470	0.12
14	D3 (Max)	507963.740	2233756.900	0.23

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 9:10 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL C.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	26.50 [ft]
Total length	:	22.50 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	6.33	10.00	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
2	EQ	Horizontal	336820.00	0.00	0.00
1	EQ	Horizontal	116474.00	0.00	0.00

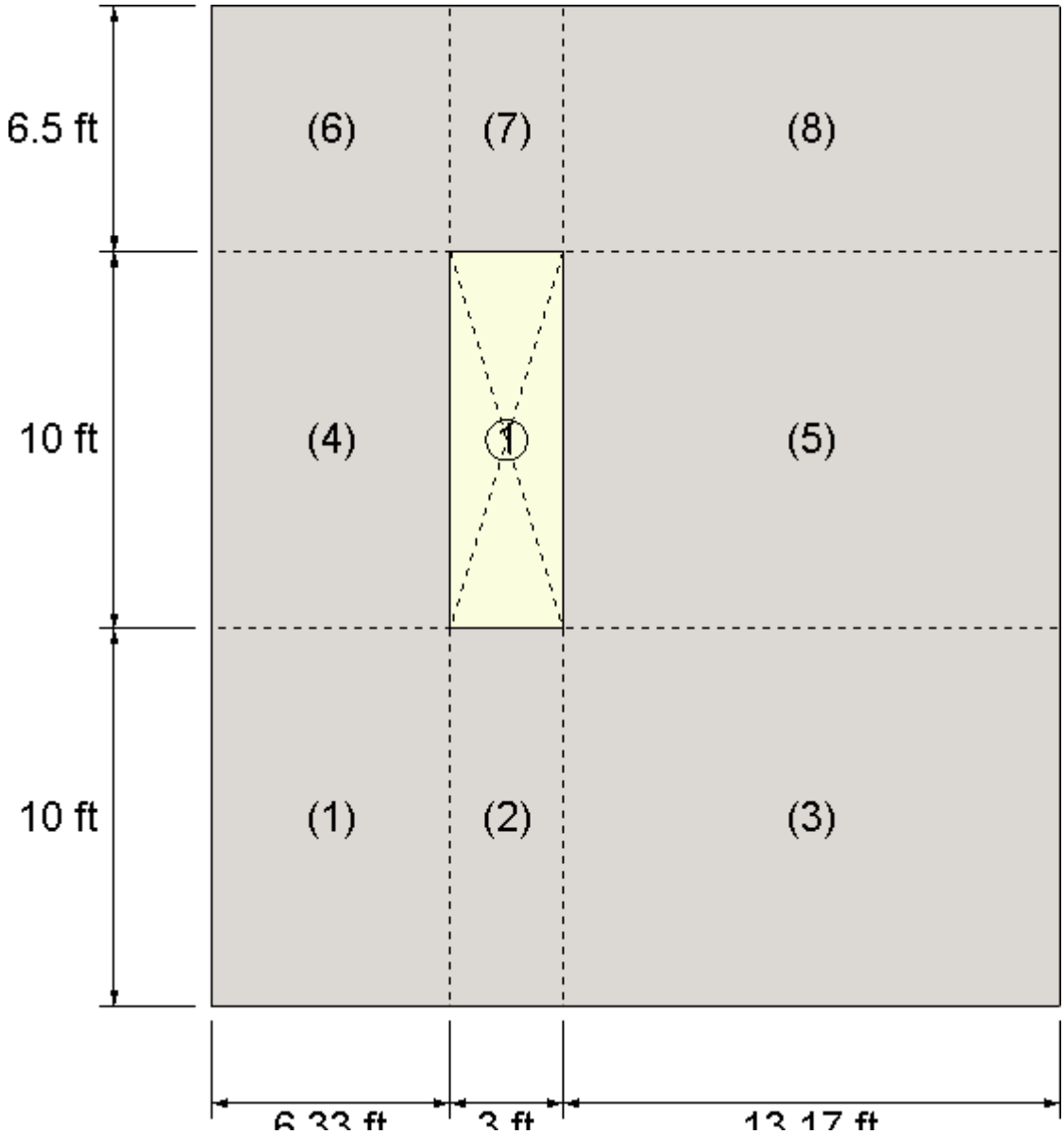
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
2	DL	Vertical	5850.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	6.33	10.00
2	6.33	0.00	3.00	10.00
3	9.33	0.00	13.17	10.00
4	0.00	10.00	6.33	10.00
5	9.33	10.00	13.17	10.00
6	0.00	20.00	6.33	6.50
7	6.33	20.00	3.00	6.50
8	9.33	20.00	13.17	6.50

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#7	12.00	33.89
2	3-#5	14.00	19.36
3	12-#5	14.00	19.36
4	8-#6	10.00	23.24
5	9-#5	18.00	19.36
6	8-#6	10.00	23.24
7	2-#5	18.00	19.36
8	9-#5	18.00	19.36

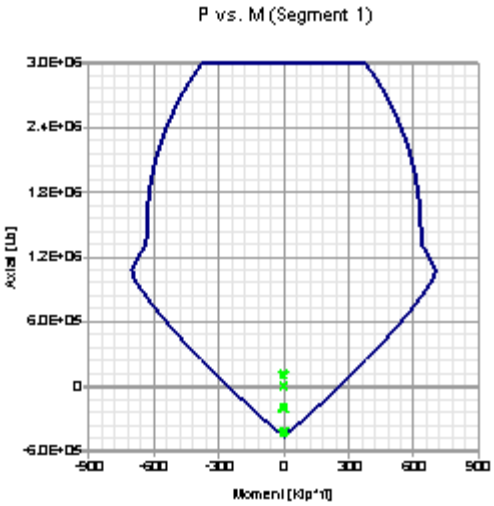
Intermediate results for axial-bending

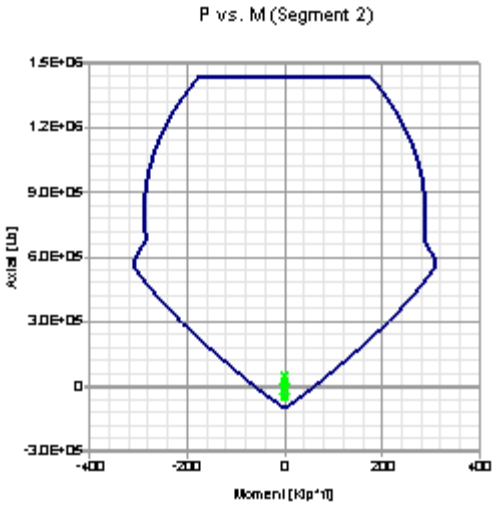
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.54	13.19
2	DC1 (Top)	0.92	13.31
3	DC1 (Top)	1.04	13.31
4	DC1 (Top)	1.34	8.25
5	DC1 (Top)	0.80	8.31
6	DC1 (Top)	1.31	8.25
7	DC1 (Top)	0.74	8.31
8	DC1 (Top)	0.75	8.31

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	107954.70	0.00	307.13	0.00
2	DC1 (Top)	19347.17	0.00	70.67	0.00
3	DC1 (Top)	213666.10	0.00	358.19	0.00
4	DC1 (Top)	71207.09	0.00	158.49	0.00
5	DC1 (Top)	133647.80	0.00	170.15	0.00
6	DC1 (Top)	52668.48	0.00	152.90	0.00
7	DC1 (Top)	24661.37	0.00	35.97	0.00
8	DC1 (Top)	109283.80	0.00	161.05	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	115725.00	2999412.00	0.04
2	D1 (Bottom)	47536.10	1427147.00	0.03
3	D2 (Bottom)	780934.20	6267100.00	0.12
4	D1 (Bottom)	81814.68	1995789.00	0.04
5	D2 (Bottom)	381941.90	4176423.00	0.09
6	D2 (Top)	137598.80	1995789.00	0.07
7	D1 (Top)	24661.37	951431.50	0.03
8	D2 (Bottom)	308265.00	4176423.00	0.07

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	432253.80	453600.00	0.95
2	D3 (Bottom)	47768.95	100440.00	0.48
3	DC1 (Top)	0.00	401760.00	0.00
4	D3 (Bottom)	206955.20	380160.00	0.54
5	DC1 (Top)	0.00	301320.00	0.00
6	D3 (Bottom)	166480.90	380160.00	0.44
7	DC1 (Top)	0.00	66960.00	0.00
8	DC1 (Top)	0.00	301320.00	0.00

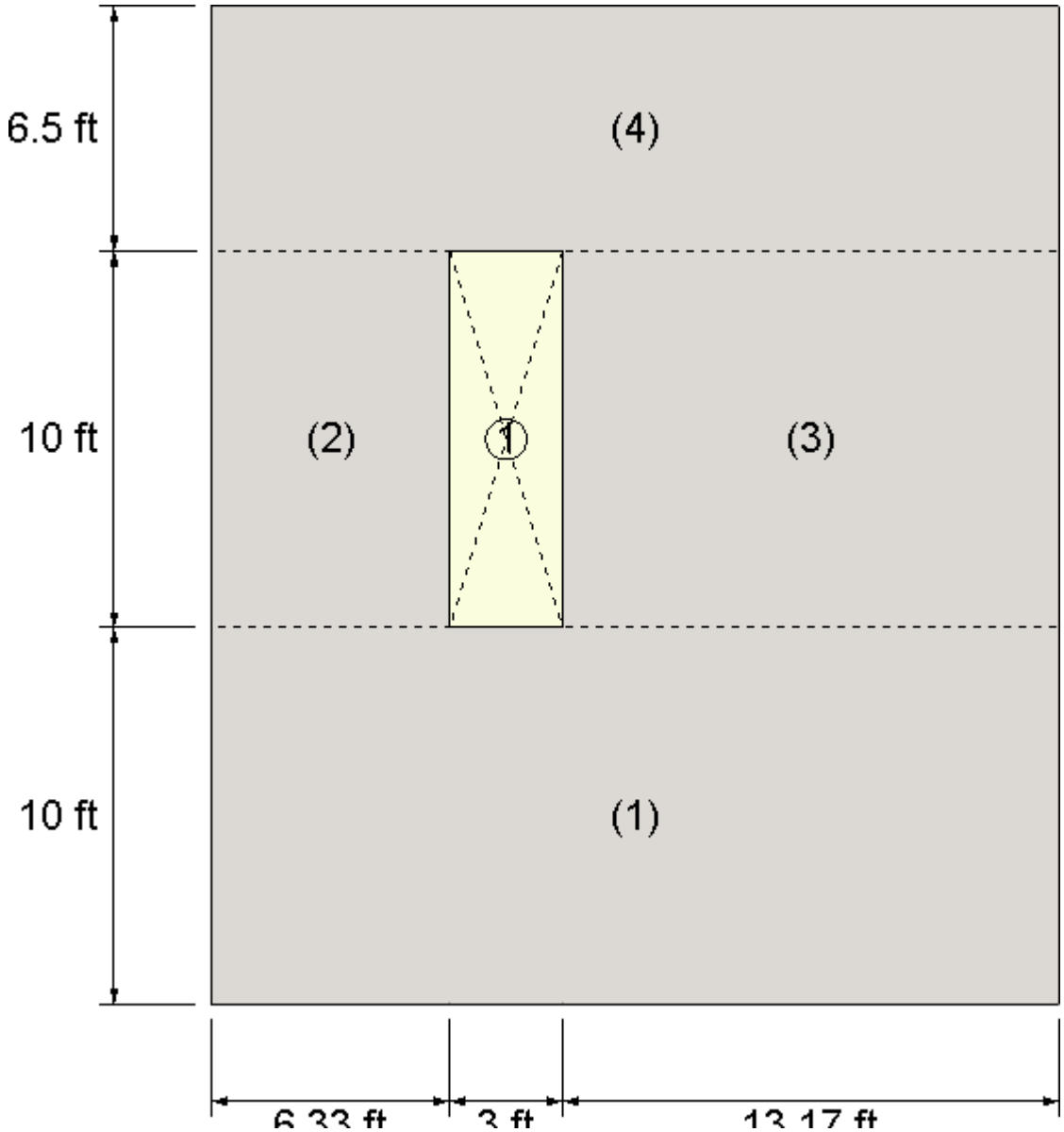
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	116389.640	0.00
2	DC1 (Top)	0.000	55683.818	0.00
3	DC1 (Top)	0.000	244451.960	0.00
4	DC1 (Top)	0.000	72812.474	0.00
5	DC1 (Top)	0.000	152639.020	0.00

6	DC1 (Top)	0.000	72812.474	0.00
7	DC1 (Top)	0.000	34769.708	0.00
8	DC1 (Top)	0.000	152639.020	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	22.50	10.00
2	0.00	10.00	6.33	10.00
3	9.33	10.00	13.17	10.00
4	0.00	20.00	22.50	6.50

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	7-#7	12.00	33.89	8-#5	16.00	25.17
	3-#5	14.00	33.89	8-#5	16.00	25.17
	12-#5	14.00	33.89	8-#5	16.00	25.17
2	8-#6	10.00	23.24	7-#5	18.00	25.17
3	9-#5	18.00	19.36	7-#5	18.00	25.17
4	8-#6	10.00	23.24	5-#5	18.00	25.17
	2-#5	18.00	23.24	5-#5	18.00	25.17
	9-#5	18.00	23.24	5-#5	18.00	25.17

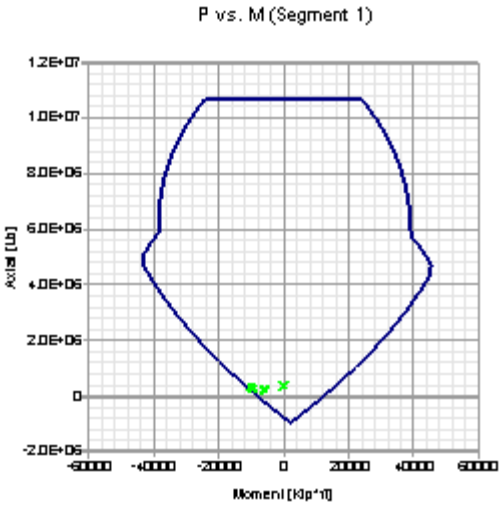
Intermediate results for axial-bending

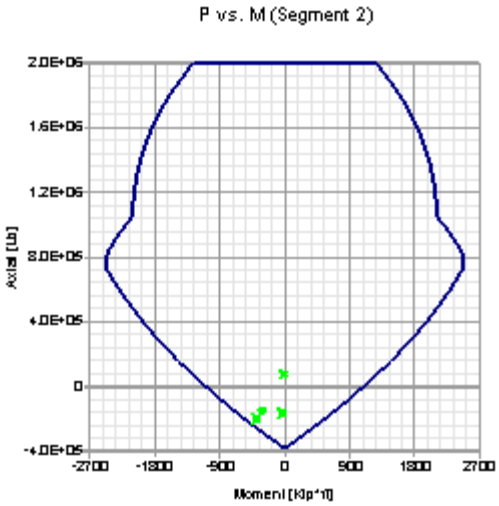
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	20.17	216.00
2	D3 (Bottom)	3.77	60.77
3	D3 (Bottom)	17.32	126.43
4	D3 (Bottom)	21.46	216.00

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	251792.80	-9836.55	10579.03	0.93
2	D3 (Bottom)	-206741.10	-393.16	527.75	0.74
3	D3 (Bottom)	348007.50	-2016.99	3903.63	0.52
4	D3 (Bottom)	135741.30	-2235.20	7360.60	0.30

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	392564.30	1.069366E07	0.04
2	D1 (Bottom)	81820.70	1995789.00	0.04
3	D2 (Bottom)	381833.70	4176423.00	0.09
4	D2 (Top)	254149.70	7123643.00	0.04

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	955800.00	0.00
2	D3 (Bottom)	206741.10	380160.00	0.54
3	DC1 (Top)	0.00	301320.00	0.00
4	DC1 (Top)	0.00	748440.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	457614.000	1035137.500	0.44
2	D3 (Bottom)	44487.234	136931.340	0.32
3	D3 (Top)	294490.860	482044.520	0.61
4	D3 (Max)	346384.460	769069.510	0.45

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

APPENDIX - E

SUMMARY DATA SHEET

BUILDING DATA

Building Name: Benton County Courthouse Date: January 29, 2016

Building Address: 120 NW 4th Street, Corvallis, Oregon

Latitude: 44.56528 N Longitude: 123.26237 W By:

Year Built: 1887 Year(s) Remodeled: 1977 Original Design Code: N/A

Area (sf): 33,052 Length (ft): Varies Width (ft): Varies

No. of Stories: Four Story Height: Varies Total Height: 59'-6"(Tower:115'-5")

USE ☐ Industrial ☒ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☒ Other: Assembly

CONSTRUCTION DATA

Gravity Load Structural System: Unreinforced masonry bearing walls with interior steel supports

Exterior Transverse Walls: Unreinforced masonry shear walls Openings? Punched openings

Exterior Longitudinal Walls: Unreinforced masonry shear walls Openings? Punched openings

Roof Materials/Framing: Timber trusses and 2x rafters with T&G straight sheathing

Intermediate Floors/Framing: Timber 3 x 16 floor joists with T&G straight sheathing

Ground Floor: Concrete slab on grade

Columns: Wood and steel columns Foundation: Unknown foundation

General Condition of Structure: Well maintained structure in good condition

Levels Below Grade? N/A

Special Features and Comments: The building houses the Benton County Courts

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Masonry shear wall	Masonry shear wall
Vertical Elements:	Bearing wall	Bearing wall
Diaphragms:	Flexible wood sheathing	Flexible wood sheathing
Connections:	Unknown	Unknown

EVALUATION DATA

BSE-1N Spectral Response Accelerations: S_{Ds} = 0.712 S_{D1} = 0.480

Soil Factors: Class = D F_a = 1.12 F_v = 1.53

BSE-1E Spectral Response Accelerations: S_{XS} = 0.36 S_{X1} = 0.58

Level of Seismicity: High Performance Level: Collapse Prevention

Building Period: T = 0.43

Spectral Acceleration: S_a = 0.36

Modification Factor: $C_mC_1C_2$ = 1.0 Building Weight: W = 7,344,016 lb

Pseudo Lateral Force: V= 0.36

$C_mC_1C_2S_aW$= 2,608,595 lb

BUILDING CLASSIFICATION:

REQUIRED TIER 1 CHECKLISTS

	Yes	No
Basic Configuration Checklist	☐	☒
Building Type URM **Structural Checklist**	☐	☒
Nonstructural Component Checklist	☐	☒

FURTHER EVALUATION REQUIREMENT:

Table 2-1. Basic Performance Objective for Existing Buildings (BPOE)

	Tier 1[a]	Tier 2[a]	Tier 3	
Risk Category	**BSE-1E**	**BSE-1E**	**BSE-1E**	**BSE-2E**
I & II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Nonstructural Performance Not Considered (5-D)
III	See footnote *b* for Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Nonstructural Performance Not Considered (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Nonstructural Performance Not Considered (3-D)

[a]For Tier 1 and 2 assessments, seismic performance for the BSE-2E is not explicitly evaluated.
[b]For Risk Category III, the Tier 1 screening checklists shall be based on the Life Safety Performance Level (S-3), except that checklist statements using the Quick Check procedures of Section 4.5.3 shall be based on MS-factors and other limits that are an average of the values for Life Safety and Immediate Occupancy.

Per USGS Sds = 0.712, so the courthouse site is considered in a region with high level of seismicity.

Table 2-5. Level of Seismicity Definitions

Level of Seismicity[a]	S_{DS}	S_{D1}
Very low	<0.167 g	<0.067 g
Low	≥0.167 g <0.33 g	≥0.067 g <0.133 g
Moderate	≥0.33 g <0.50 g	≥0.133 g <0.20 g
High	≥ 0.50 g	≥0.20 g

[a]The higher level of seismicity defined by S_{DS} or S_{D1} shall govern.

C2.5 LEVEL OF SEISMICITY

The Levels of Seismicity in this standard have been adjusted to match the Seismic Design Categories in ASCE 7 as follows:

SDC A: Very Low
SDC B: Low
SDC C: Moderate
SDC D–F: High

Therefore, the parameters S_{DS} and S_{D1} correspond to the parameters at the BSE-1N level.

The courthouse does not meet the Benchmark Building Criteria

Table 4-6. Benchmark Buildings

Building Type[a,b]	Building Seismic Design Provisions				Seismic Evaluation or Retrofit Provisions		
	NBC[LS] SBC[LS]	UBC[LS]	IBC[LS]	NEHRP[LS]	FEMA 178[LS]	FEMA 310 (1998e)/ ASCE 31[LS,IO]	FEMA 356 (2000)/ ASCE 41[LS,IO][c,d]
Wood frame, wood shear panels (Types W1 & W2)	1993	1976	2000	1985	[e]	1998	2000
Wood frame, wood shear panels (Type W1a)	[e]	1997	2000	1997	[e]	1998	2000
Steel moment-resisting frame (Types S1 & S1a)	[e]	1994[f]	2000	1997	[e]	1998	2000
Steel concentrically braced frame (Types S2 & S2a)	[e]	1997	2000	[e]	[e]	1998	2000
Steel eccentrically braced frame (Types S2 & S2a)	[e]	1988[f]	2000	1997	[e]	[e]	2000
Buckling-restrained braced frame (Types S2 & S2a)	[e]	[e]	2006	[e]	[e]	[e]	2000
Light metal frame (Type S3)	[e]	[e]	2000	[e]	1992	1998	2000
Steel frame w/ concrete shear walls (Type S4)	1993	1994?	2000	1985	[e]	1998	2000
Steel frame with URM infill (Types S5 & S5a)	[e]	[e]	2000	[e]	[e]	1998	2000
Steel plate shear wall (Type S6)	[e]	[e]	2006	[e]	[e]	[e]	2000
Reinforced concrete moment-resisting frame (Type C1)[g]	1993	1994	2000	1997	[e]	1998	2000
Reinforced concrete shear walls (Types C2 & C2a)	1993	1994	2000	1985	[e]	1998	2000
Concrete frame with URM infill (Types C3 & C3a)	[e]	[e]	2000	[e]	[e]	1998	2000
Tilt-up concrete (Types PC1 & PC1a)	[e]	1997	2000	[e]	[e]	1998	2000
Precast concrete frame (Types PC2 & PC2a)	[e]	[e]	2000	[e]	1992	1998	2000
Reinforced masonry (Type RM1)	[e]	1997	2000	[e]	[e]	1998	2000
Reinforced masonry (Type RM2)	1993	1994?	2000	1985	[e]	1998	2000
Unreinforced masonry (Type URM)[h]	[e]	1991[i]	2000	[e]	1992	1998	2000
Unreinforced masonry (Type URMa)	[e]	[e]	2000	[e]	[e]	1998	2000
Seismic isolation or passive dissipation	[e]	1991	2000	[e]	[e]	[e]	2000

[a]Building type refers to one of the common building types defined in Table 3-1.
[b]Buildings on hillside sites shall not be considered Benchmark Buildings.
[c]LS: S-3 Structural Performance Level for the BSE-1.
[d]IO: S-1 Structural Performance Level for the BSE-1.
[e]No benchmark year; buildings shall be evaluated using this standard.
[f]Steel moment-resisting frames and eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
[g]Flat slab concrete moment frames shall not be considered Benchmark Buildings.
[h]URM buildings evaluated or retrofitted and shown to be acceptable using Special Procedure (the ABK Methodology, 1984) may be considered benchmark buildings subject to the limitation of Section 15.2.
[i]Refers to the GSREB or its predecessor, the Uniform Code of Building Conservation (UCBC), or its successor, IEBC Appendix Chapter A1.
[LS]Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life Safety Performance Level may be considered Benchmark Buildings.
[IO]Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Immediate Occupancy Performance Level may be considered Benchmark Buildings.

Table 4-7. Checklists Required for a Tier 1 Screening

Level of Seismicity[b]	Level of Building Performance[c]	Required Checklists[a]					
		Very Low Seismicity Checklist (Sec 16.1.1)	Basic Configuration Checklist (Sec. 16.1.2)	Life Safety Checklist (Sec. 16.2LS through 16.15LS)	Immediate Occupancy Checklist (Sec. 16.2IO through 16.15IO)	Life Safety Nonstructural Checklist (Sec. 16.17)	Position Retention Nonstructural Checklist (Sec. 16.17)
Very low	LS	X					
Very low	IO		X		X		X
Low	LS		X	X		X	
Low	IO		X		X		X
Moderate	LS		X	X		X	
Moderate	IO		X		X		X
High	LS		X	X		X	
High	IO		X		X		X

[a]An X designates the checklist that must be completed for a Tier 1 screening as a function of the level of seismicity and level of performance.
[b]Defined in Section 2.5.
[c]LS = Life Safety Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).

APPENDIX - F

Building Code: 2014 Oregon Structural Specialty Code

Soils Report: No **Soils Report by:** N/A **Dated:** N/A

Soil Bearing: 1500 PSF **Retaining Walls:** No

Equivalent Fluid Pressure (active): N/A PCF **Passive bearing:** N/A PCF **Friction:** N/A

Structural System: Building Structure

Vertical System: Wood framed roof / masonry walls **Lateral Sys:** Flexible Diaphragm / Concrete or Masonry Shearwalls

Basic Design Loads:

Element	Roof	Floor	Corridor	
Load Type	Dead	Dead	Dead	
Value (PSF)	20	20	20	
Load Type	Snow	Floor Live	Corridor Live	
Value (PSF)	25	50	100	
Deflection Criteria	L/240	L/360	L/360	

Lateral Design Parameters:

Wind Design: Other: ASCE 41-13 **Wind Speed (3 sec Gust):** 120 MPH

Exposure B

Importance Factors I_W = (ice) I_E = (seismic) I_S = (snow) I_i = (ice) **Risk Cat:** II

Seismic Design

Seismic design parameters are based on published values from the USGS web site.

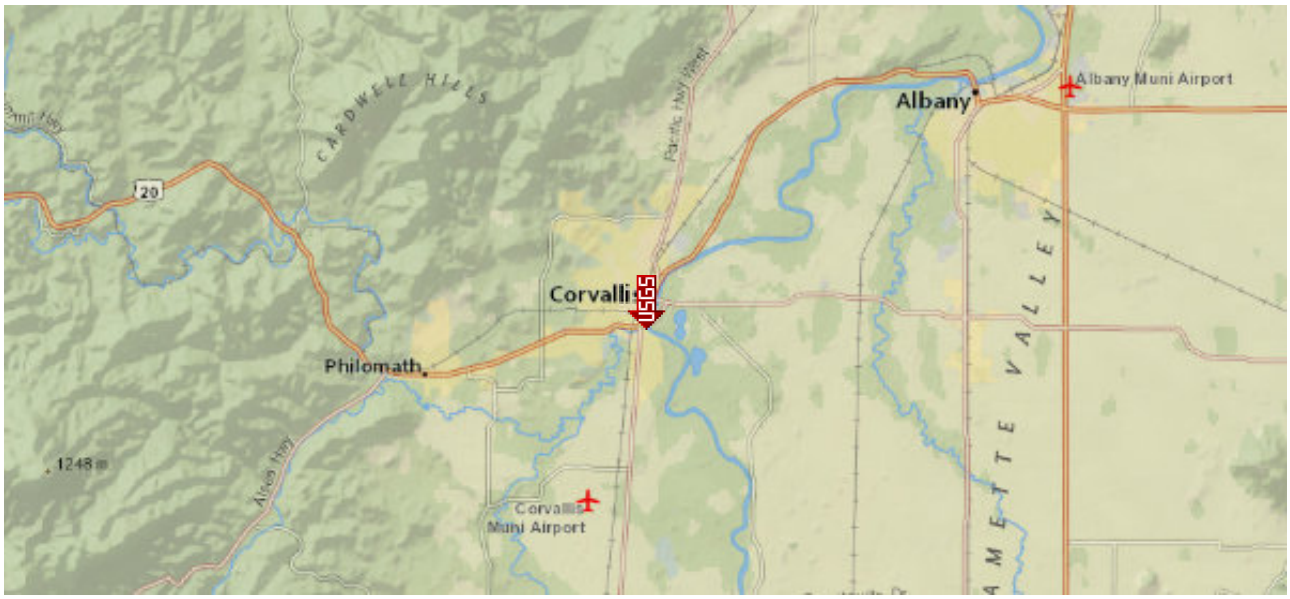
Design Summary:

The following calculations are the Tier I and II analysis of the courthouse building based on the ASCE 41-13 standard for evaluating an existing building considering seismic loading.

USGS Design Maps Summary Report

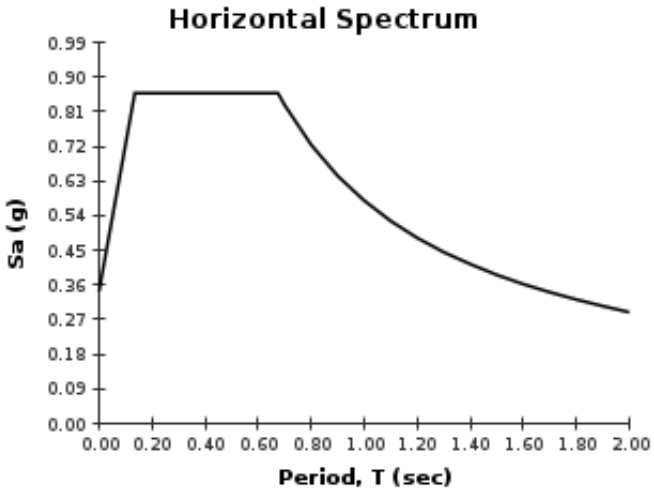
User–Specified Input

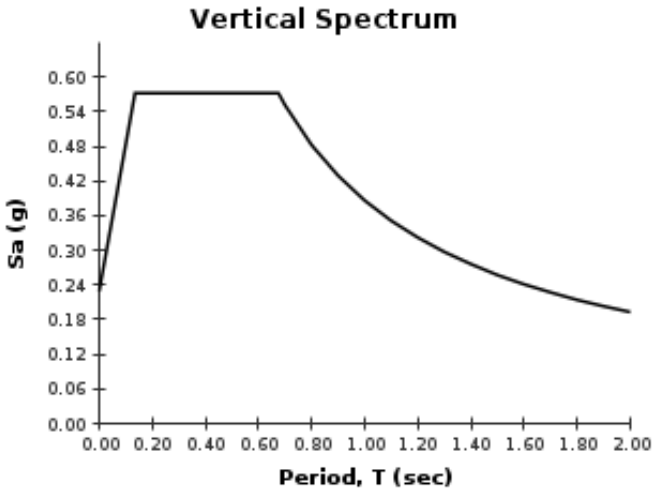
Report Title	Benton County Courthouse Thu December 10, 2015 17:15:56 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-2E (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

$S_{S,5/50}$	0.685 g	$S_{XS,BSE-2E}$	0.858 g
$S_{1,5/50}$	0.334 g	$S_{X1,BSE-2E}$	0.579 g





Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Summary Report

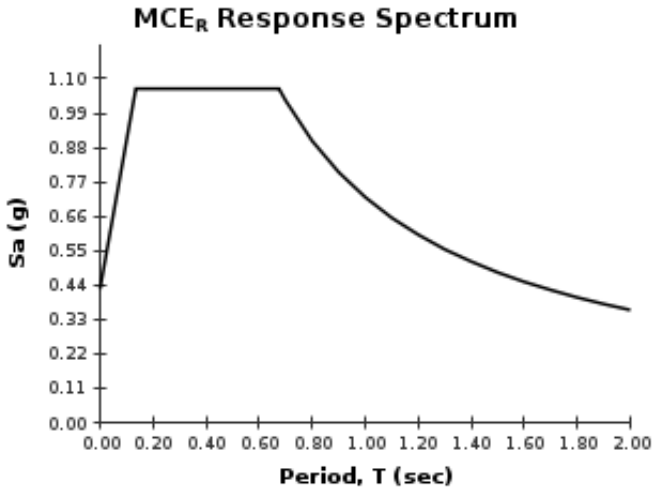
User–Specified Input

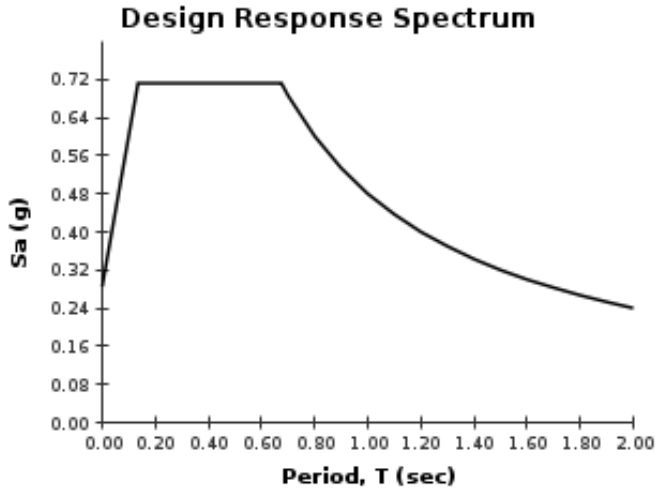
Report Title	Benton County Couthouse Wed November 18, 2015 23:04:50 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.956 g	S_{MS} =	1.068 g	S_{DS} =	0.712 g
S_1 =	0.472 g	S_{M1} =	0.721 g	S_{D1} =	0.480 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the

Benton County Courthouse LPOE Calculations:

Building Weights:

Roof Weights =	20	psf
Wood Floor =	20	psf
Light Framed =	8	psf
8" CMU =	80	psf
12" brick =	120	psf
18" brick =	180	psf
20" brick =	200	psf
30" stone =	350	psf

Building Geometries:

West =	1752	ft^2
Center =	3373	ft^2
East =	1752	ft^2
Total sq footage =	6877	ft^2
Total width =	114.50	ft
Total length =	71.50	ft
Total height =	59.50	ft

Roofs:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

Partial Floors:

Upper Foyer=	14513	lb
Speaker Floor=	9138	lb
Bell Floor=	4102	lb
Top Floor=	995	lb
Sub Total =	28747	lb

Floors:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

East Section

Width =	24.50	ft
Length =	71.50	ft
Bell Tower Lounge Height =	13.67	ft
3rd Floor Height =	14.25	ft
2nd Floor Height =	15.00	ft
1st Floor Height =	16.50	ft
Basement Floor Height =	10.50	ft

Center Section

Width =	65.50	ft
Length =	51.50	ft
4th Floor Height =	8.50	ft
3rd Floor Height =	9.00	ft

West Section

Width =	24.50	ft
Length =	71.50	ft

4th East-West Walls: (starting at north wall)

Gridline 8 =	11760	lb
GL 7 =	2180	lb
GL 6 =	42445	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	42445	lb
GL 2 =	2180	lb
GL 1 =	11760	lb
Sub Total =	112770	lb

4th North-South Walls: (starting at west wall)

Gridline A =	17160	lb
GL B =	17160	lb
GL C =	38451	lb
GL D =	12000	lb
GL E =	26451	lb
Sub Total =	111223	lb
Total =	**375763**	**lb**

3rd East-West Walls: (starting at north wall)

Gridline 8 =	41895	lb
GL 7 =	35370	lb
GL 6 =	86692	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	86692	lb
GL 2 =	35370	lb
GL 1 =	41895	lb
Sub Total =	327913	lb

3rd North-South Walls: (starting at west wall)

Gridline A =	61133	lb
GL B =	61133	lb
GL C =	96775	lb
GL D =	42750	lb
GL E =	54025	lb
Sub Total =	315815	lb
Total =	**795776**	**lb**

2nd East-West Walls: (starting at north wall)

Gridline 8 =	128993	lb
GL 7 =	172429	lb
GL 6 =	90821	lb

2nd North-South Walls: (starting at west wall)

Gridline A =	188224	lb
GL B =	188224	lb
GL C =	188224	lb

GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	90821	lb
GL 2 =	172429	lb
GL 1 =	128993	lb
Sub Total =	784485	lb

GL D =	131625	lb
GL E =	56599	lb
Sub Total =	752895	lb
Total =	**1674915**	**lb**

1st East-West Walls: (starting at north wall)

Gridline 8 =	154350	lb
GL 7 =	206325	lb
GL 6 =	108675	lb
GL 5 =	108075	lb
GL 4 =	108075	lb
GL 3 =	108675	lb
GL 2 =	206325	lb
GL 1 =	154350	lb
Sub Total =	1154850	lb

1st North-South Walls: (starting at west wall)

Gridline A =	225225	lb
GL B =	225225	lb
GL C =	225225	lb
GL D =	157500	lb
GL E =	67725	lb
Sub Total =	900900	lb
Total =	**2193285**	**lb**

Basement East-West Walls: (starting at north wall)

Gridline 8 =	162803	lb
GL 7 =	217624	lb
GL 6 =	114626	lb
GL 5 =	159165	lb
GL 4 =	159165	lb
GL 3 =	114626	lb
GL 2 =	217624	lb
GL 1 =	162803	lb
Sub Total =	1308435	lb

Basement North-South Walls: (starting at west wall)

Gridline A =	237559	lb
GL B =	191595	lb
GL C =	191595	lb
GL D =	166125	lb
GL E =	71434	lb
Sub Total =	858308	lb
Total =	**2304278**	**lb**

4.5.2.1 - Pseudo Seismic Force - for use with quick check proceedures

S_s=	68.50%	
S_1 =	33.40%	
F_a =	1.25	Table 2-3, page 46
F_v =	1.73	Table 2-4, page 46
S_{XS} =	0.86	eqn. 2-1, pg. 46
S_{X1} =	0.58	eqn. 2-2, pg. 46
C_t =	0.02	eqn. 4-5, pg. 69
h_n =	59.50	ft (roof height)
β =	0.75	eqn. 4-5, pg. 69
T =	0.428	eqn. 4-5, pg. 69
Sa =	0.86	eqn. 4-4, pg. 68

Evaluation: Immediate Occupancy

Immediate Occupancy Responses:	
S_s=	22.20%
S_1 =	9.10%
Life Safety:	
S_s=	22.20%
S_1 =	9.10%
Collapse Prevention:	
S_s=	68.50%
S_1 =	33.40%

Total Diaphragm Weight

C =	1	table 4-8, pg. 68
W_1 =	2304278	lb (lowest floor)
W_2 =	2193285	lb
W_3 =	1674915	lb
W_4 =	795776	lb
W_5 =	375763	lb

Heights of floors to base:

h_1 =	10.50	ft
h_2 =	27.00	ft
h_3 =	42.00	ft
h_4 =	51.00	ft
h_5 =	59.50	ft

W_{total} =	7344016	lb
V_{total} =	6298375	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

k =	1.00		
F_1 =	703216	lb	0.36
F_2 =	1721169	lb	0.92
F_3 =	2044592	lb	1.42
F_4 =	1179575	lb	1.73
F_5 =	649823	lb	2.02
Σ =	6298375	lb	

7.4.1.3.1 - Pseudo Seismic Force for LSP (linear static proceedure)

Sa =	0.9	
C_1C_2 =	1	from Table 7-3
C_m =	1	from Table 7-4
V =	0.86	*W

Diaphragm Weights East-West Walls:

W_1 =	995843	lb (lowest floor)
W_2 =	1038435	lb
W_3 =	890430	lb
W_4 =	467863	lb
W_5 =	262992	lb
W_{total} =	3655563	lb
V_{total} =	3135084	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	284066	lb
F_2 =	761696	lb
F_3 =	1015986	lb
F_4 =	648228	lb
F_5 =	425108	lb
Σ =	3135084	lb

Diaphragm Weights North-South Walls:

W_1 =	1445970	lb (lowest floor)
W_2 =	1292385	lb
W_3 =	922020	lb
W_4 =	479961	lb
W_5 =	264540	lb
W_{total} =	4404875	lb
V_{total} =	3777709	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	444549	lb
F_2 =	1021709	lb
F_3 =	1133864	lb
F_4 =	716716	lb
F_5 =	460871	lb
Σ =	3777709	lb

URM Shear Wall Loads/Diaphragm Loads for Collapse Prevention Evaluation:

Ms =	1.5	URM Wall, from Table 4-9, pg. 70
M_1 =	2.5	Straight Diaphragm (chorder), from Table 12-3
M_2 =	2.0	Straight Diaphragm (unchorder), from Table 12-4
M_3 =	3.0	Wood Diaphragm (chorder), from Table 12-3
M_4 =	2.5	Wood Diaphragm (unchorded), from Table 12-3

4th Floor / Roof	Shear (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	39813	49	542	4	30	OK	243	120	NG
GL 7 =	76189	66	775	5	30	OK	454	120	NG
GL 6 =	144908	20	4954	23	30	OK	362	450	OK
GL 5 =	0	0							
GL 4 =	0	0							
GL 3 =	144908	20	4954	23	30	OK	362	450	OK
GL 2 =	76189	66	775	5	30	OK	454	120	NG
GL 1 =	39813	49	542	4	30	OK	243	120	NG
Gridline A =	60063	72	560	4	30	OK	317	120	NG
GL B =	181297	72	1690	12	30	OK	1177	120	NG
GL C =	199557	72	1861	10	30	OK	798	120	NG

GL D =	74147	50	989	7	30	OK	317	120	NG
GL E =	41194	15	1894	9	30	OK	344	300	NG
	521822	556258	268	279					

3rd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	81261	121074	38	2124	15	30	OK	370	300	NG
GL 7 =	143661	219850	66	2238	16	30	OK	577	450	NG
GL 6 =	239805	384714	24	10686	49	30	NG	445	450	OK
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	239805	384714	32	7933	37	30	NG	445	450	OK
GL 2 =	143661	219850	66	2238	16	30	OK	577	450	NG
GL 1 =	81261	121074	38	2124	15	30	OK	370	300	NG
Gridline A =	122949	183012	58	2104	15	30	OK	329	450	OK
GL B =	311483	492779	67	4879	34	30	NG	1220	450	NG
GL C =	342050	541607	62	5871	31	30	NG	1034	450	NG
GL D =	135967	210114	39	3638	25	30	OK	329	450	OK
GL E =	75117	116311	16	4846	22	30	OK	446	450	OK
	929453	987566	263	241						

2nd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	181675	302749	38	5311	25	30	OK	580	300	NG
GL 7 =	325498	545348	47	7708	36	30	NG	904	450	NG
GL 6 =	407204	791917	33	16160	75	30	NG	697	450	NG
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	407204	791917	33	16160	75	30	NG	697	450	NG
GL 2 =	325498	545348	47	7708	36	30	NG	904	450	NG
GL 1 =	181675	302749	38	5311	25	30	OK	580	300	NG
Gridline A =	272989	456002	55	5527	26	30	OK	520	450	NG
GL B =	571255	1064034	63	11319	52	30	NG	1931	450	NG
GL C =	571255	1112862	59	12628	58	30	NG	1636	450	NG
GL D =	269986	480100	39	8207	38	30	NG	520	450	NG
GL E =	94077	210388	16	8766	41	30	NG	706	450	NG
	1828753	1779562	236	231						

1st Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	185639	488388	36	8982	37	30	NG	435	300	NG
GL 7 =	310112	855460	47	12134	51	30	NG	496	450	NG
GL 6 =	205059	996977	29	22661	94	30	NG	373	450	OK
GL 5 =	175249	175249	80	1463	6	30	OK	306	450	OK
GL 4 =	175249	175249	81	1437	6	30	OK	306	450	OK
GL 3 =	205059	996977	33	20344	85	30	NG	373	450	OK
GL 2 =	310112	855460	47	12134	51	30	NG	496	450	NG
GL 1 =	185639	488388	36	8982	37	30	NG	435	300	NG
Gridline A =	293687	749689	50	9996	46	30	NG	469	450	NG
GL B =	562450	1626484	62	17396	81	30	NG	1740	450	NG
GL C =	562450	1675312	62	18014	83	30	NG	1474	450	NG
GL D =	276637	756737	36	13886	64	30	NG	469	450	NG
GL E =	99115	309503	16	12635	58	30	NG	636	450	NG
	1752119	1794338	390	227						

Basement Floor	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	159487	647875	36	11915	33	30	NG	162	300	OK
GL 7 =	236300	1091760	59	12301	34	30	NG	253	450	OK
GL 6 =	140022	1136998	24	31155	87	30	NG	403	450	OK
GL 5 =	167293	342542	78	2915	8	30	OK	114	450	OK
GL 4 =	167293	342542	69	3298	9	30	OK	114	450	OK
GL 3 =	140022	1136998	29	26439	73	30	NG	403	450	OK
GL 2 =	236300	1091760	62	11787	33	30	NG	253	450	OK
GL 1 =	159487	647875	36	11915	33	30	NG	162	300	OK
Gridline A =	247476	997165	60	11111	31	30	NG	204	450	OK
GL B =	324996	1951480	63	20651	57	30	NG	757	450	NG
GL C =	324996	2000308	67	19854	55	30	NG	641	450	NG
GL D =	204066	960804	39	16286	45	30	NG	204	450	OK
GL E =	79116	388619	16	16192	45	30	NG	277	450	OK
	1406206	1180651	394	245						

Foundation Design	M_{OT} (ft*lb)	M_R (ft*lb)	Uplift (lb)	250k MP
Gridline 8 =	10253228	3589560	256294.9266	1.0
GL 7 =	35839163	21075390	567837.4245	2.3
GL 6 =	47173513	8106480	1502578.182	6.0
GL 5 =	6317026.8	10076625	-144599.9298	-0.6
GL 4 =	6317026.8	10076625	-144599.9298	-0.6
GL 3 =	47173513	8106480	1502578.182	6.0
GL 2 =	35839163	21075390	567837.4245	2.3
GL 1 =	10253228	3589560	256294.9266	1.0
				0.0
Gridline A =	31339183	27146047.5	161274.4354	0.6
GL B =	34144162	4163040	1070754.37	4.3
GL C =	35454671	4163040	1117558.245	4.5
GL D =	15908494	4163040	419480.5093	1.7
GL E =	14372659	4228380	390164.5789	1.6

Section 16.1.2 Basic Configuration - Life Safety (Tier 1 evaluations)

Life Safety

Adjacent Building, Sect. 5.4.1.1

Taller Building =	59.50	ft
Shorter Building =	10	ft
Clear Distance =	4.80	inch

Weak Story, Sect. 5.4.2.1

Basement SW (NS) =	245.33	ft	108% of story above
1st Floor SW (NS) =	226.99	ft	98% of story above
2nd Floor SW (NS) =	231.42	ft	96% of story above
3rd Floor SW (NS) =	241.33	ft	86% of story above
4th Floor SW (NS) =	279.00	ft	
Basement SW (EW) =	394.00	ft	101% of story above
1st Floor SW (EW) =	389.66	ft	165% of story above
2nd Floor SW (EW) =	235.67	ft	89% of story above
3rd Floor SW (EW) =	263.33	ft	98% of story above
4th Floor SW (EW) =	268.00	ft	

Change in Building Mass, Sect. 5.4.2.5

Basement Mass =	2304278	lbs	105% of story above
1st Floor Mass =	2193285	lbs	131% of story above
2nd Floor Mass =	1674915	lbs	210% of story above
3rd Floor Mass =	795776	lbs	212% of story above
4th Floor Mass =	375763	lbs	

Foundation - Overturning, Sect. 5.4.3.3

Basement SW (NS) =	71.5	ft	
b/h Ratio (NS) =	1.20		
Basement SW (EW) =	113.5	ft	
b/h Ratio (EW) =	1.91		
0.6 x Sa =	0.51		**< MIN b/h ratio of 1.2 OK**

Section 16.16 URM Bearing Walls - Collapse Prevention Evaluation

Wall Shear Stress Check - Shear Stress in Shear Walls, Sect. 4.5.3.3

Tier 1 Quick Check - Shear Wall loading see page

Tier 2 - Shear Wall Capacities - Sect. 7.5.1.5

m =	4	from Table 11-3 for Collapse Prevention
K =	0.75	from 11.2.4
F_m =	600	PSI, from Table 11-2(a)
F_t =	45	PSI, from Table 11-2(a)
F_v =	78.38367	PSI per shear area, 80% of TMS 402, Section 3.2.4
E_m =	420000	PSI, per TMS 402, Section 1.8.2.2
G_m =	168000	PSI, per TMS 402, Section 1.8.2.2

Check 3rd Floor Gridline 3:

Total Wall Load=	384714	lb				
Wall height =	14.25	ft		14.25	ft	
Pier height =	8.00	ft		8.00	ft	
Pier length =	11.5	ft		21.25	ft	
Pier thickness =	1.33	ft		1.33	ft	
L / h_{eff} =	0.81	for (11-12)		1.49		
Trib DL P_D =	23464.6	lb		43358.5	lb	
Pier SW P_W =	29497.5	lb		54506.25	lb	
Wall Rigidity =	181027			483802		
Load to Pier =	104754.2	9109.064		279959.4	13174.56	384713.6
Shear Stress=	48	psi		69	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0		
LB Bed-Joint V_{mL} =	43	psi, 11.2.3.6.3 (11-6)		43	psi	
In-plane V_{bjs1} =	94325	lb, 11.3.2.2.2 (11-9)		174297	lb	
In-plane V_{bjs2} =	11732	lb, 11.3.2.2.2 (11-10)		21679	lb	
Rocking V_r =	27755	lb, 11.3.2.2.1 (11-8)		94768	lb	
Lower Bound V_{tc} =	29073	lb, 11.3.2.2.3 (11-11)		99269	lb	
Lower Bound V_{dt} =	102545	lb, 11.3.2.2.4 (11-12)		217800	lb	
Lower Bound P_{CL} =	898612	lb, 11.3.2.2.5 (11-13)		1660478	lb	
Controlling Q_{CE} =	11732	lb		21679	lb	
Allowable Stress=	16	psi **NG**		16	psi **NG**	
DCR =	2.98	6048.464	DCR =	4.30	10113.96	

Check 3rd Floor Gridline 6:

Total Wall Load=	384714	lb
Wall height =	14.25	ft
Pier height =	8.00	ft
Pier length =	24	ft
Pier thickness =	1.33	ft
L / h_{eff} =	1.68	for (11-12)
Trib DL P_D =	48969.6	lb
Pier SW P_W =	61560	lb
Wall Rigidity =	569140	
Load to Pier =	384713.6	16029.73
Shear Stress=	84	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1
LB Bed-Joint V_{mL} =	43	psi, 11.2.3.6.3 (11-6)
In-plane V_{bjs1} =	196853	lb, 11.3.2.2.2 (11-9)
In-plane V_{bjs2} =	24485	lb, 11.3.2.2.2 (11-10)
Rocking V_r =	120884	lb, 11.3.2.2.1 (11-8)
Lower Bound V_{tc} =	126625	lb, 11.3.2.2.3 (11-11)
Lower Bound V_{dt} =	214008	lb, 11.3.2.2.4 (11-12)
Lower Bound P_{CL} =	1875364	lb, 11.3.2.2.5 (11-13)
Controlling Q_{CE} =	24485	lb
Allowable Stress=	16	psi **NG**

DCR =	5.24	12969.13									

Check 3rd Floor Gridline B:

Total Wall Load=	492779	lb									
Wall height =	15.00	ft		15.00	ft						
Pier height =	8.00	ft		8.00	ft						
Pier length =	33.67	ft		33.67	ft						
Pier thickness =	1	ft		1	ft						
L / h_{eff} =	2.24	for (11-12)		2.24							
Trib DL P_D =	14814.8	lb		14814.8	lb						
Pier SW P_W =	90909	lb		90909	lb						
Wall Rigidity =	648494			648494							
Load to Pier =	246389.6	7317.778		246389.6	7317.778						
Shear Stress=	51	psi		51	psi						
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0							
LB Bed-Joint V_{mL} =	39	psi, 11.2.3.6.3 (11-6)		39	psi						
In-plane V_{bjs1} =	189225	lb, 11.3.2.2.2 (11-9)		189225	lb						
In-plane V_{bjs2} =	7407	lb, 11.3.2.2.2 (11-10)		7407	lb						
Rocking V_r =	121756	lb, 11.3.2.2.1 (11-8)		121756	lb						
Lower Bound V_{tc} =	128261	lb, 11.3.2.2.3 (11-11)		128261	lb						
Lower Bound V_{dt} =	174822	lb, 11.3.2.2.4 (11-12)		214983	lb						
Lower Bound P_{CL} =	1978180	lb, 11.3.2.2.5 (11-13)		1978180	lb						
Controlling Q_{CE} =	7407	lb		7407	lb						
Allowable Stress=	5	psi	**NG**	5	psi	**NG**					
DCR =	11.09	6657.778	DCR =	11.09	6657.778						

Check 3rd Floor Gridline C:

Total Wall Load=	541607	lb									
Wall height =	15.00	ft		15.00	ft		15.00	ft		16.50	ft
Pier height =	8.00	ft		8.00	ft		8.00	ft		8.00	ft
Pier length =	10.75	ft		10.75	ft		20	ft		20	ft
Pier thickness =	1.33	ft		1.33	ft		1.33	ft		1.33	ft
L / h_{eff} =	0.72	for (11-12)		0.72			1.33			1.21	
Trib DL P_D =	4730	lb		4730	lb		8800	lb		8800	lb
Pier SW P_W =	29025	lb		29025	lb		54000	lb		59400	lb
Wall Rigidity =	159190			159190			444745			444745	
Load to Pier =	71380.37	6640.034		71380.37	6640.034		199423.1	9971.157		199423.1	9971.157
Shear Stress=	35	psi		35	psi		52	psi		52	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	39	psi, 11.2.3.6.3 (11-6)		39	psi		39	psi		39	psi
In-plane V_{bjs1} =	79572	lb, 11.3.2.2.2 (11-9)		79572	lb		148040	lb		148040	lb
In-plane V_{bjs2} =	2365	lb, 11.3.2.2.2 (11-10)		2365	lb		4400	lb		4400	lb
Rocking V_r =	12411	lb, 11.3.2.2.1 (11-8)		12411	lb		42960	lb		42000	lb
Lower Bound V_{tc} =	13252	lb, 11.3.2.2.3 (11-11)		13252	lb		45870	lb		44688	lb
Lower Bound V_{dt} =	70271	lb, 11.3.2.2.4 (11-12)		86414	lb		176671	lb		178919	lb
Lower Bound P_{CL} =	840007	lb, 11.3.2.2.5 (11-13)		840007	lb		1562803	lb		1562803	lb
Controlling Q_{CE} =	2365	lb		2365	lb		4400	lb		4400	lb
Allowable Stress=	3	psi	**NG**	3	psi	**NG**	3	psi	**NG**	3	**NG**
DCR =	10.06	5980.034	DCR =	10.06	5980.034	DCR =	15.11	9311.157	DCR =	15.11	9311.157

Check 2nd Floor Gridline 2 (& 7):

Total Wall Load=	545348	lb									
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier length =	7.167	ft	(3) at	7.5	ft		8.5	ft		8.83	ft
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.86	ft
L / h_{eff} =	0.43	for (11-12)		0.45			0.52			0.54	
Trib DL P_D =	11252.19	lb		11775	lb		13345	lb		13863.1	lb
Pier SW P_W =	21285.99	lb		22275	lb		25245	lb		26225.1	lb
Wall Rigidity =	53547		(3) at	59977	lb for (3) piers		81295			90403	
Load to Pier =	72071.9	10056.08		242179.6	10763.54		109418.2	12872.73		121678.1	13780.08
Shear Stress=	38	psi		41	psi / pier		49	psi		51	psi

Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	40	psi, 11.2.3.6.3 (11-6)		40	psi		40	psi		40	psi
In-plane V_{bjs1} =	76450	lb, 11.3.2.2.2 (11-9)		80003	lb		90670	lb		95620	lb
In-plane V_{bjs2} =	5626	lb, 11.3.2.2.2 (11-10)		5888	lb		6673	lb		6932	lb
Rocking V_r =	8559	lb, 11.3.2.2.1 (11-8)		9373	lb		12039	lb		12992	lb
Lower Bound V_{tc} =	9120	lb, 11.3.2.2.3 (11-11)		9988	lb		12828	lb		13853	lb
Lower Bound V_{dt} =	79414	lb, 11.3.2.2.4 (11-12)		95522	lb		108258	lb		113914	lb
Lower Bound P_{CL} =	770568	lb, 11.3.2.2.5 (11-13)		806371	lb		913887	lb		964931	lb
Controlling Q_{CE} =	5626	lb		5888	lb		6673	lb		6932	lb
Allowable Stress=	9	psi	**NG**	9	psi	**NG**	9	psi	**NG**	9	**NG**
DCR =	4.27	7701.077	DCR =	4.57	8408.539	DCR =	5.47	10517.73	DCR =	5.85	11425.08

Check 2nd Floor Gridline 3:

Total Wall Load=	791917	lb			
Wall height =	15.00	ft	15.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	11.5	ft	21.25	ft	
Pier thickness =	1.5	ft	1.5	ft	
L / h_{eff} =	0.77	for (11-12)	1.42		
Trib DL P_D =	55262.1	lb	102114.8	lb	
Pier SW P_W =	31050	lb	57375	lb	
Wall Rigidity =	204166		545641		
Load to Pier =	215632.3	18750.63	576284.9	27119.29	791917.2
Shear Stress=	87	psi	126	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)	49	psi	
In-plane V_{bjs1} =	120781	lb, 11.3.2.2.2 (11-9)	223182	lb	
In-plane V_{bjs2} =	27631	lb, 11.3.2.2.2 (11-10)	51057	lb	
Rocking V_r =	48843	lb, 11.3.2.2.1 (11-8)	166773	lb	
Lower Bound V_{tc} =	49780	lb, 11.3.2.2.3 (11-11)	169973	lb	
Lower Bound V_{dt} =	137595	lb, 11.3.2.2.4 (11-12)	292242	lb	
Lower Bound P_{CL} =	1013472	lb, 11.3.2.2.5 (11-13)	1872720	lb	
Controlling Q_{CE} =	27631	lb	51057	lb	
Allowable Stress=	33	psi **NG**	33	psi **NG**	
DCR =	2.60	11542.53	DCR = 3.76	19911.19	

Check 2nd Floor Gridline 6:

Total Wall Load=	791917	lb			
Wall height =	15.00	ft	15.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	9.5	ft	23.33	ft	
Pier thickness =	1.5	ft	1.5	ft	
L / h_{eff} =	0.63	for (11-12)	1.56		
Trib DL P_D =	45651.3	lb	112110	lb	
Pier SW P_W =	25650	lb	62991	lb	
Wall Rigidity =	140188		618528		
Load to Pier =	146323	15402.42	645594.2	27672.27	791917.2
Shear Stress=	71	psi	128	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)	49	psi	
In-plane V_{bjs1} =	99776	lb, 11.3.2.2.2 (11-9)	245028	lb	
In-plane V_{bjs2} =	22826	lb, 11.3.2.2.2 (11-10)	56055	lb	
Rocking V_r =	33331	lb, 11.3.2.2.1 (11-8)	201019	lb	
Lower Bound V_{tc} =	33971	lb, 11.3.2.2.3 (11-11)	204876	lb	
Lower Bound V_{dt} =	113665	lb, 11.3.2.2.4 (11-12)	320848	lb	
Lower Bound P_{CL} =	837216	lb, 11.3.2.2.5 (11-13)	2056026	lb	
Controlling Q_{CE} =	22826	lb	56055	lb	
Allowable Stress=	33	psi **NG**	33	psi **NG**	
DCR =	2.14	8194.321	DCR = 3.84	20464.17	

Check 2nd Floor Gridline B:

Total Wall Load=	1064034	lb							
Wall height =	15.00	ft		15.00	ft		15.00	ft	
Pier height =	8.00	ft		8.00	ft		8.00	ft	
Pier length =	17.833	ft		18.75	ft		24	ft	
Pier thickness =	1.33	ft		1.33	ft		1.33	ft	
L / h_{eff} =	1.19	for (11-12)		1.25			1.60		
Trib DL P_D =	30672.76	lb		32250	lb		41280	lb	
Pier SW P_W =	48149.1	lb		50625	lb		64800	lb	
Wall Rigidity =	376760			405560			569140		
Load to Pier =	296631.7	16633.86		319306.1	17029.66		448096.2	18670.68	1064034
Shear Stress=	87	psi		89	psi		97	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0		
LB Bed-Joint V_{mL} =	42	psi, 11.2.3.6.3 (11-6)		42	psi		42	psi	
In-plane V_{bjs1} =	143413	lb, 11.3.2.2.2 (11-9)		150788	lb		193008	lb	
In-plane V_{bjs2} =	15336	lb, 11.3.2.2.2 (11-10)		16125	lb		20640	lb	
Rocking V_r =	58579	lb, 11.3.2.2.1 (11-8)		64758	lb		106099	lb	
Lower Bound V_{tc} =	61511	lb, 11.3.2.2.3 (11-11)		67999	lb		111410	lb	
Lower Bound V_{dt} =	132109	lb, 11.3.2.2.4 (11-12)		170812	lb		240263	lb	
Lower Bound P_{CL} =	1393473	lb, 11.3.2.2.5 (11-13)		1465128	lb		1875364	lb	
Controlling Q_{CE} =	15336	lb		16125	lb		20640	lb	
Allowable Stress=	13	psi	**NG**	13	psi	**NG**	13	psi	**NG**
DCR =	6.45	14053.86	DCR =	6.60	14449.66	DCR =	7.24	16090.68	

Check 2nd Floor Gridline C:

Total Wall Load=	1112862	lb							
Wall height =	15.00	ft		15.00	ft		15.00	ft	
Pier height =	8.00	ft		8.00	ft		8.00	ft	
Pier length =	16.33	ft		19	ft		19	ft	
Pier thickness =	1.5	ft		1.33	ft		1.33	ft	
L / h_{eff} =	1.09	for (11-12)		1.27			1.27		
Trib DL P_D =	28087.6	lb		32680	lb		32680	lb	
Pier SW P_W =	44091	lb		51300	lb		51300	lb	
Wall Rigidity =	371674			413405			413405		
Load to Pier =	345120.6	21134.14		383870.6	20203.72		383870.6	20203.72	1112862
Shear Stress=	98	psi		105	psi		105	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0		
LB Bed-Joint V_{mL} =	41	psi, 11.2.3.6.3 (11-6)		42	psi		42	psi	
In-plane V_{bjs1} =	146317	lb, 11.3.2.2.2 (11-9)		152798	lb		152798	lb	
In-plane V_{bjs2} =	14044	lb, 11.3.2.2.2 (11-10)		16340	lb		16340	lb	
Rocking V_r =	49120	lb, 11.3.2.2.1 (11-8)		66496	lb		66496	lb	
Lower Bound V_{tc} =	51919	lb, 11.3.2.2.3 (11-11)		69825	lb		69825	lb	
Lower Bound V_{dt} =	132312	lb, 11.3.2.2.4 (11-12)		173090	lb		190208	lb	
Lower Bound P_{CL} =	1439130	lb, 11.3.2.2.5 (11-13)		1484663	lb		1484663	lb	
Controlling Q_{CE} =	14044	lb		16340	lb		16340	lb	
Allowable Stress=	12	psi	**NG**	13	psi	**NG**	13	psi	**NG**
DCR =	8.19	18554.14	DCR =	7.83	17623.72	DCR =	7.83	17623.72	

Check 2nd Floor Gridline D:

Total Wall Load=	480100	lb									
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier length =	9	ft		9	ft		9.25	ft		9.25	ft
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.55	for (11-12)		0.55			0.56			0.56	
Trib DL P_D =	15480	lb		15480	lb		15910	lb		15910	lb
Pier SW P_W =	26730	lb		26730	lb		27472.5	lb		27472.5	lb
Wall Rigidity =	92997			92997			99089			99089	
Load to Pier =	116219	12913.22		116219	12913.22		123831.1	13387.15		123831.1	13387.15
Shear Stress=	49	psi		49	psi		51	psi		51	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	

LB Bed-Joint V_{mL} =	41	psi, 11.2.3.6.3 (11-6)		41	psi		41	psi		41	psi	
In-plane V_{bjs1} =	96678	lb, 11.3.2.2.2 (11-9)		96678	lb		99364	lb		99364	lb	
In-plane V_{bjs2} =	7740	lb, 11.3.2.2.2 (11-10)		7740	lb		7955	lb		7955	lb	
Rocking V_r =	14160	lb, 11.3.2.2.1 (11-8)		14160	lb		14958	lb		14958	lb	
Lower Bound V_{tc} =	15067	lb, 11.3.2.2.3 (11-11)		15067	lb		15916	lb		15916	lb	
Lower Bound V_{dt} =	105448	lb, 11.3.2.2.4 (11-12)		113559	lb		119096	lb		119096	lb	
Lower Bound P_{CL} =	967645	lb, 11.3.2.2.5 (11-13)		967645	lb		994524	lb		994524	lb	
Controlling Q_{CE} =	7740	lb		7740	lb		7955	lb		7955	lb	
Allowable Stress=	10	psi	**NG**	10	psi	**NG**	10	psi	**NG**	10	**NG**	
DCR =	5.01	10333.22	DCR =	5.01	10333.22	DCR =	5.19	10807.15	DCR =	5.19	10807.15	

Check 2nd Floor Gridline E:

Total Wall Load=	210388	lb			
Wall height =	16.50	ft	16.50	ft	
Pier height =	10.00	ft	10.00	ft	
Pier length =	8.25	ft	8.25	ft	
Pier thickness =	1.83	ft	1.83	ft	
L / h_{eff} =	0.50	for (11-12)	0.50		
Trib DL P_D =	39644.55	lb	39644.55	lb	
Pier SW P_W =	24502.5	lb	24502.5	lb	
Wall Rigidity =	75695		75695		
Load to Pier =	105194	12750.79	105194	12750.79	210388
Shear Stress=	48	psi	48	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	47	psi, 11.2.3.6.3 (11-6)	47	psi	
In-plane V_{bjs1} =	101349	lb, 11.3.2.2.2 (11-9)	101349	lb	
In-plane V_{bjs2} =	19822	lb, 11.3.2.2.2 (11-10)	19822	lb	
Rocking V_r =	23353	lb, 11.3.2.2.1 (11-8)	23353	lb	
Lower Bound V_{tc} =	24125	lb, 11.3.2.2.3 (11-11)	24125	lb	
Lower Bound V_{dt} =	112674	lb, 11.3.2.2.4 (11-12)	129510	lb	
Lower Bound P_{CL} =	887008	lb, 11.3.2.2.5 (11-13)	887008	lb	
Controlling Q_{CE} =	19822	lb	19822	lb	
Allowable Stress=	27	psi **NG**	27	psi **NG**	
DCR =	1.77	5542.686	DCR = 1.77	5542.686	

Check 1st Floor Gridline 1 (& 8):

Total Wall Load=	488388	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier length =	4.25	ft	(3) at	7.5	ft		4.25	ft		7.5	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.86	ft	
L / h_{eff} =	0.26	for (11-12)		0.45			0.26			0.45		
Trib DL P_D =	19210	lb		33900	lb		19210	lb		33900	lb	
Pier SW P_W =	12622.5	lb		22275	lb		12622.5	lb		22275	lb	
Wall Rigidity =	13254		(3) at	59977	lb for (3) piers		13254			60961		
Load to Pier =	24207.81	5695.955		328632.2	14605.88		24207.81	5695.955		111339.9	14845.32	
Shear Stress=	22	psi		55	psi / pier		22	psi		55	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	51604	lb, 11.3.2.2.2 (11-9)		91065	lb		51604	lb		92280	lb	
In-plane V_{bjs2} =	9605	lb, 11.3.2.2.2 (11-10)		16950	lb		9605	lb		16950	lb	
Rocking V_r =	5916	lb, 11.3.2.2.1 (11-8)		18424	lb		5916	lb		18424	lb	
Lower Bound V_{tc} =	6129	lb, 11.3.2.2.3 (11-11)		19086	lb		6129	lb		19109	lb	
Lower Bound V_{dt} =	57087	lb, 11.3.2.2.4 (11-12)		115795	lb		65617	lb		117045	lb	
Lower Bound P_{CL} =	456944	lb, 11.3.2.2.5 (11-13)		806371	lb		456944	lb		819590	lb	
Controlling Q_{CE} =	5916	lb		16950	lb		5916	lb		16950	lb	
Allowable Stress=	16	psi	**NG**	26	psi	**NG**	16	psi	**NG**	25	**NG**	
DCR =	1.36	1519.751	DCR =	2.15	7825.875	DCR =	1.36	1519.751	DCR =	2.19	8065.316	

Check 1st Floor Gridline 2 (& 7):

Total Wall Load=	855460	lb

Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft		
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft		
Pier length =	7.167	ft	(3) at	7.5	ft		8.5	ft		8.83	ft		
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.86	ft		
L / h_{eff} =	0.43	for (11-12)		0.45			0.52			0.54			
Trib DL P_D =	32394.84	lb		33900	lb		38420	lb		39911.6	lb		
Pier SW P_W =	21285.99	lb		22275	lb		25245	lb		26225.1	lb		
Wall Rigidity =	53547		(3) at	59977			81295			90403			
Load to Pier =	113055.6	15774.47		379895.1	16884.23		171638.9	20192.82		190870.4	21616.12		
Shear Stress=	60	psi		64	psi / pier		77	psi		81	psi		
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0			
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi		
In-plane V_{bjs1} =	87022	lb, 11.3.2.2.2 (11-9)		91065	lb		103207	lb		108644	lb		
In-plane V_{bjs2} =	16197	lb, 11.3.2.2.2 (11-10)		16950	lb		19210	lb		19956	lb		
Rocking V_r =	16825	lb, 11.3.2.2.1 (11-8)		18424	lb		23665	lb		25538	lb		
Lower Bound V_{tc} =	17429	lb, 11.3.2.2.3 (11-11)		19086	lb		24515	lb		26487	lb		
Lower Bound V_{dt} =	96268	lb, 11.3.2.2.4 (11-12)		115795	lb		131234	lb		137800	lb		
Lower Bound P_{CL} =	770568	lb, 11.3.2.2.5 (11-13)		806371	lb		913887	lb		964931	lb		
Controlling Q_{CE} =	16197	lb		16950	lb		19210	lb		19956	lb		
Allowable Stress=	26	psi	**NG**	26	psi	**NG**	26	psi	**NG**	25	**NG**		
DCR =	2.33	8994.466		DCR = 2.49	10104.23		DCR = 2.98	13412.82		DCR = 3.19	14836.12		

Check 1st Floor Gridline 3:

Total Wall Load=	996977	lb			
Wall height =	16.50	ft	16.50	ft	
Pier height =	10.00	ft	10.00	ft	
Pier length =	11.5	ft	21.25	ft	
Pier thickness =	1.83	ft	1.83	ft	
L / h_{eff} =	0.70	for (11-12)	1.29		
Trib DL P_D =	88612.1	lb	163739.8	lb	
Pier SW P_W =	34155	lb	63112.5	lb	
Wall Rigidity =	159992		482388		
Load to Pier =	248308.8	21592.07	748667.7	35231.42	996976.5
Shear Stress=	82	psi	134	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)	52	psi	
In-plane V_{bjs1} =	157949	lb, 11.3.2.2.2 (11-9)	291862	lb	
In-plane V_{bjs2} =	44306	lb, 11.3.2.2.2 (11-10)	81870	lb	
Rocking V_r =	66296	lb, 11.3.2.2.1 (11-8)	226366	lb	
Lower Bound V_{tc} =	66557	lb, 11.3.2.2.3 (11-11)	227258	lb	
Lower Bound V_{dt} =	183194	lb, 11.3.2.2.4 (11-12)	389093	lb	
Lower Bound P_{CL} =	1236436	lb, 11.3.2.2.5 (11-13)	2284718	lb	
Controlling Q_{CE} =	44306	lb	81870	lb	
Allowable Stress=	44	psi **NG**	44	psi **NG**	
DCR =	1.87	10033.97	DCR = 3.05	23673.32	

Check 1st Floor Gridline 6:

Total Wall Load=	996977	lb
Wall height =	16.50	ft
Pier height =	10.00	ft
Pier length =	29.25	ft
Pier thickness =	1.83	ft
L / h_{eff} =	1.77	for (11-12)
Trib DL P_D =	225383	lb
Pier SW P_W =	86872.5	lb
Wall Rigidity =	757585	
Load to Pier =	996976.5	34084.67
Shear Stress=	129	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)

In-plane V_{bjs1} =	401740	lb, 11.3.2.2.2 (11-9)
In-plane V_{bjs2} =	112691	lb, 11.3.2.2.2 (11-10)
Rocking V_r =	428889	lb, 11.3.2.2.1 (11-8)
Lower Bound V_{tc} =	430579	lb, 11.3.2.2.3 (11-11)
Lower Bound V_{dt} =	535575	lb, 11.3.2.2.4 (11-12)
Lower Bound P_{CL} =	3144848	lb, 11.3.2.2.5 (11-13)
Controlling Q_{CE} =	112691	lb
Allowable Stress=	44	psi **NG**
DCR =	2.95	22526.57

Check 1st Floor Gridline A:

Total Wall Load=	749689	lb								
Wall height =	16.50	ft	16.50	ft		16.50	ft		16.50	ft
Pier height =	10.00	ft	10.00	ft		10.00	ft		10.00	ft
Pier length =	9.25	ft	10	ft		10	ft	(2) at	12.25	ft
Pier thickness =	1.83	ft	1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.56	for (11-12)	0.61			0.61			0.74	
Trib DL P_D =	42735	lb	46200	lb		46200	lb		56595	lb
Pier SW P_W =	27472.5	lb	29700	lb		29700	lb		36382.5	lb
Wall Rigidity =	99089		118246			118246		(2) at	182272	
Load to Pier =	106103.4	11470.64	126617.2	12661.72		126617.2	12661.72		390350.8	31865.37
Shear Stress=	44	psi	48	psi		48	psi		60	psi / pier
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)	46	psi		46	psi		46	psi
In-plane V_{bjs1} =	112776	lb, 11.3.2.2.2 (11-9)	121920	lb		121920	lb		149352	lb
In-plane V_{bjs2} =	21368	lb, 11.3.2.2.2 (11-10)	23100	lb		23100	lb		28298	lb
Rocking V_r =	28492	lb, 11.3.2.2.1 (11-8)	33300	lb		33300	lb		49971	lb
Lower Bound V_{tc} =	29487	lb, 11.3.2.2.3 (11-11)	34463	lb		34463	lb		51716	lb
Lower Bound V_{dt} =	106303	lb, 11.3.2.2.4 (11-12)	141323	lb		155300	lb		190243	lb
Lower Bound P_{CL} =	994524	lb, 11.3.2.2.5 (11-13)	1075162	lb		1075162	lb		1317073	lb
Controlling Q_{CE} =	21368	lb	23100	lb		23100	lb		28298	lb
Allowable Stress=	26	psi **NG**	26	psi	**NG**	26	psi	**NG**	26	**NG**
DCR =	1.66	4540.637	DCR = 1.83	5731.721		DCR = 1.83	5731.721		DCR = 2.30	18005.37

Check 1st Floor Gridline B:

Total Wall Load=	1626484	lb								
Wall height =	16.50	ft	16.50	ft		16.50	ft		16.50	ft
Pier height =	8.00	ft	9.00	ft		8.00	ft		9.00	ft
Pier length =	11.5	ft	14.25	ft		16.83	ft		19.75	ft
Pier thickness =	1.83	ft	1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.70	for (11-12)	0.86			1.02			1.20	
Trib DL P_D =	53130	lb	65835	lb		77754.6	lb		91245	lb
Pier SW P_W =	34155	lb	42322.5	lb		49985.1	lb		58657.5	lb
Wall Rigidity =	249083		297138			475040			506405	
Load to Pier =	265194.6	23060.4	316358.8	22200.62		505768.7	30051.61		539161.7	27299.33
Shear Stress=	88	psi	84	psi		114	psi		104	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)	46	psi		46	psi		46	psi
In-plane V_{bjs1} =	140208	lb, 11.3.2.2.2 (11-9)	173736	lb		205191	lb		240792	lb
In-plane V_{bjs2} =	26565	lb, 11.3.2.2.2 (11-10)	32918	lb		38877	lb		45623	lb
Rocking V_r =	44039	lb, 11.3.2.2.1 (11-8)	67620	lb		94322	lb		129891	lb
Lower Bound V_{tc} =	45577	lb, 11.3.2.2.3 (11-11)	69981	lb		97615	lb		134426	lb
Lower Bound V_{dt} =	132161	lb, 11.3.2.2.4 (11-12)	201386	lb		261371	lb		306718	lb
Lower Bound P_{CL} =	1236436	lb, 11.3.2.2.5 (11-13)	1532105	lb		1809497	lb		2123444	lb
Controlling Q_{CE} =	26565	lb	32918	lb		38877	lb		45623	lb
Allowable Stress=	26	psi **NG**	26	psi	**NG**	26	psi	**NG**	26	**NG**
DCR =	3.33	16130.4	DCR = 3.20	15270.62		DCR = 4.34	23121.61		DCR = 3.94	20369.33

Check 1st Floor Gridline C:

Total Wall Load=	1675312	lb								
Wall height =	16.50	ft	16.50	ft		16.50	ft		16.50	ft

Pier height =	8.00	ft		9.00	ft		8.00	ft		9.00	ft	
Pier length =	14.25	ft		15.25	ft		15.5	ft		17	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft	
L / h_{eff} =	0.86	for (11-12)		0.92			0.94			1.03		
Trib DL P_D =	65835	lb		70455	lb		71610	lb		78540	lb	
Pier SW P_W =	42322.5	lb		45292.5	lb		46035	lb		50490	lb	
Wall Rigidity =	364047			334521			417652			400927		
Load to Pier =	401998.9	28210.45		369395.6	24222.66		461192.9	29754.38		442724.2	26042.6	
Shear Stress=	107	psi		92	psi		113	psi		99	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	173736	lb, 11.3.2.2.2 (11-9)		185928	lb		188976	lb		207264	lb	
In-plane V_{bjs2} =	32918	lb, 11.3.2.2.2 (11-10)		35228	lb		35805	lb		39270	lb	
Rocking V_r =	67620	lb, 11.3.2.2.1 (11-8)		77443	lb		80003	lb		96237	lb	
Lower Bound V_{tc} =	69981	lb, 11.3.2.2.3 (11-11)		80147	lb		82797	lb		99597	lb	
Lower Bound V_{dt} =	201386	lb, 11.3.2.2.4 (11-12)		232096	lb		240716	lb		264011	lb	
Lower Bound P_{CL} =	1532105	lb, 11.3.2.2.5 (11-13)		1639621	lb		1666500	lb		1827775	lb	
Controlling Q_{CE} =	32918	lb		35228	lb		35805	lb		39270	lb	
Allowable Stress=	26	psi	**NG**	26	psi	**NG**	26	psi	**NG**	26	**NG**	
DCR =	4.07	21280.45	DCR =	3.50	17292.66	DCR =	4.29	22824.38	DCR =	3.76	19112.6	

Check 1st Floor Gridline D:

Total Wall Load=	756737	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier length =	9	ft		9	ft		9.25	ft		9.25	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft	
L / h_{eff} =	0.55	for (11-12)		0.55			0.56			0.56		
Trib DL P_D =	41580	lb		41580	lb		42735	lb		42735	lb	
Pier SW P_W =	26730	lb		26730	lb		27472.5	lb		27472.5	lb	
Wall Rigidity =	92997			92997			99089			99089		
Load to Pier =	183185.2	20353.91		183185.2	20353.91		195183.5	21100.92		195183.5	21100.92	
Shear Stress=	77	psi		77	psi		80	psi		80	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	109728	lb, 11.3.2.2.2 (11-9)		109728	lb		112776	lb		112776	lb	
In-plane V_{bjs2} =	20790	lb, 11.3.2.2.2 (11-10)		20790	lb		21368	lb		21368	lb	
Rocking V_r =	26973	lb, 11.3.2.2.1 (11-8)		26973	lb		28492	lb		28492	lb	
Lower Bound V_{tc} =	27915	lb, 11.3.2.2.3 (11-11)		27915	lb		29487	lb		29487	lb	
Lower Bound V_{dt} =	127191	lb, 11.3.2.2.4 (11-12)		136975	lb		143653	lb		143653	lb	
Lower Bound P_{CL} =	967645	lb, 11.3.2.2.5 (11-13)		967645	lb		994524	lb		994524	lb	
Controlling Q_{CE} =	20790	lb		20790	lb		21368	lb		21368	lb	
Allowable Stress=	26	psi	**NG**	26	psi	**NG**	26	psi	**NG**	26	**NG**	
DCR =	2.94	13423.91	DCR =	2.94	13423.91	DCR =	3.04	14170.92	DCR =	3.04	14170.92	

Check 1st Floor Gridline E:

Total Wall Load=	309503	lb				
Wall height =	16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft	
Pier length =	8.25	ft		8.25	ft	
Pier thickness =	1.83	ft		1.83	ft	
L / h_{eff} =	0.50	for (11-12)		0.50		
Trib DL P_D =	63569.55	lb		63569.55	lb	
Pier SW P_W =	24502.5	lb		24502.5	lb	
Wall Rigidity =	75695			75695		
Load to Pier =	154751.4	18757.74		154751.4	18757.74	309502.8
Shear Stress=	71	psi		71	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)		52	psi	
In-plane V_{bjs1} =	113311	lb, 11.3.2.2.2 (11-9)		113311	lb	

In-plane V_{bjs2} =	31785	lb, 11.3.2.2.2 (11-10)		31785	lb							
Rocking V_r =	34119	lb, 11.3.2.2.1 (11-8)		34119	lb							
Lower Bound V_{tc} =	34254	lb, 11.3.2.2.3 (11-11)		34254	lb							
Lower Bound V_{dt} =	131422	lb, 11.3.2.2.4 (11-12)		151060	lb							
Lower Bound P_{CL} =	887008	lb, 11.3.2.2.5 (11-13)		887008	lb							
Controlling Q_{CE} =	31785	lb		31785	lb							
Allowable Stress=	44	psi **NG**		44	psi **NG**							
DCR =	1.62	7199.643	DCR =	1.62	7199.643							

Check Basement Gridline 1 (& 8):

Total Wall Load=	647875	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft	
Pier length =	4.25	ft	(3) at	7.5	ft		4.25	ft		7.5	ft	
Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft	
L / h_{eff} =	0.43	for (11-12)		0.75			0.43			0.75		
Trib DL P_D =	32895	lb		58050	lb		32895	lb		58050	lb	
Pier SW P_W =	7650	lb		13500	lb		7650	lb		13500	lb	
Wall Rigidity =	111058		(3) at	368182	lb for (3) piers		111058			368182		
Load to Pier =	42453.36	9989.025		422226.3	18765.61		42453.36	9989.025		140742.1	18765.61	
Shear Stress=	28	psi		52	psi / pier		28	psi		52	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi	
In-plane V_{bjs1} =	73823	lb, 11.3.2.2.2 (11-9)		130275	lb		73823	lb		130275	lb	
In-plane V_{bjs2} =	16448	lb, 11.3.2.2.2 (11-10)		29025	lb		16448	lb		29025	lb	
Rocking V_r =	14045	lb, 11.3.2.2.1 (11-8)		43740	lb		14045	lb		43740	lb	
Lower Bound V_{tc} =	14621	lb, 11.3.2.2.3 (11-11)		45534	lb		14621	lb		45534	lb	
Lower Bound V_{dt} =	79940	lb, 11.3.2.2.4 (11-12)		162150	lb		91885	lb		162150	lb	
Lower Bound P_{CL} =	624240	lb, 11.3.2.2.5 (11-13)		1101600	lb		624240	lb		1101600	lb	
Controlling Q_{CE} =	14045	lb		29025	lb		14045	lb		29025	lb	
Allowable Stress=	28	psi **NG**		32	psi **NG**		28	psi **NG**		32	**NG**	
DCR =	1.01	74.62483	DCR =	1.62	7155.613	DCR =	1.01	74.62483	DCR =	1.62	7155.613	

Check Basement Gridline 2 (& 7):

Total Wall Load=	1091760	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft	
Pier length =	7.167	ft	(3) at	7.5	ft		8.5	ft		8.83	ft	
Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft	
L / h_{eff} =	0.72	for (11-12)		0.75			0.85			0.88		
Trib DL P_D =	55472.58	lb		58050	lb		65790	lb		68344.2	lb	
Pier SW P_W =	12900.6	lb		13500	lb		15300	lb		15894	lb	
Wall Rigidity =	338460		(3) at	368182			459568			490223		
Load to Pier =	154429	21547.23		503970.5	22398.69		209686.9	24669.04		223673.9	25331.14	
Shear Stress=	60	psi		62	psi / pier		69	psi		70	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi	
In-plane V_{bjs1} =	124491	lb, 11.3.2.2.2 (11-9)		130275	lb		147645	lb		153377	lb	
In-plane V_{bjs2} =	27736	lb, 11.3.2.2.2 (11-10)		29025	lb		32895	lb		34172	lb	
Rocking V_r =	39942	lb, 11.3.2.2.1 (11-8)		43740	lb		56182	lb		60629	lb	
Lower Bound V_{tc} =	41580	lb, 11.3.2.2.3 (11-11)		45534	lb		58485	lb		63115	lb	
Lower Bound V_{dt} =	134807	lb, 11.3.2.2.4 (11-12)		162150	lb		183770	lb		190905	lb	
Lower Bound P_{CL} =	1052689	lb, 11.3.2.2.5 (11-13)		1101600	lb		1248480	lb		1296950	lb	
Controlling Q_{CE} =	27736	lb		29025	lb		32895	lb		34172	lb	
Allowable Stress=	32	psi **NG**		32	psi **NG**		32	psi **NG**		32	**NG**	
DCR =	1.86	9937.234	DCR =	1.93	10788.69	DCR =	2.12	13059.04	DCR =	2.18	13721.14	

Check Basement Gridline 3:

Total Wall Load=	1136998	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	5.00	ft		5.00	ft		8.00	ft		8.00	ft	

Pier length =	3.75	ft		6.75	ft		8.33	ft		11.25	ft
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.38	for (11-12)		0.68			0.83			1.13	
Trib DL P_D =	40782.75	lb		73408.95	lb		90592.08	lb		122348.3	lb
Pier SW P_W =	13125	lb		23625	lb		14994	lb		20250	lb
Wall Rigidity =	81936			301931			129304			238981	
Load to Pier =	123860.1	33029.35		456416.7	67617.29		195463	23464.95		361258.4	32111.86
Shear Stress=	92	psi		188	psi		89	psi		122	psi
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5	
LB Bed-Joint V_{mL} =	53	psi, 11.2.3.6.3 (11-6)		53	psi		58	psi		58	psi
In-plane V_{bjs1} =	71016	lb, 11.3.2.2.2 (11-9)		127829	lb		127613	lb		172347	lb
In-plane V_{bjs2} =	20391	lb, 11.3.2.2.2 (11-10)		36704	lb		45296	lb		61174	lb
Rocking V_r =	9097	lb, 11.3.2.2.1 (11-8)		29474	lb		39579	lb		72190	lb
Lower Bound V_{tc} =	9147	lb, 11.3.2.2.3 (11-11)		29635	lb		38940	lb		71025	lb
Lower Bound V_{dt} =	69700	lb, 11.3.2.2.4 (11-12)		154282	lb		172509	lb		232980	lb
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		895610	lb		1209557	lb
Controlling Q_{CE} =	9097	lb		29474	lb		38940	lb		61174	lb
Allowable Stress=	20	psi	**NG**	36	psi	**NG**	53	psi	**NG**	62	**NG**
DCR =	4.54	25751.81	DCR =	5.16	54517.7	DCR =	1.67	9440.87	DCR =	1.97	15798.76

Check Basement Gridline 6:

Total Wall Load=	1136998	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		6.50	ft		8.00	ft		8.00	ft
Pier length =	3.75	ft		6.75	ft		5.5	ft		9.33	ft
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.38	for (11-12)		0.68			0.55			0.93	
Trib DL P_D =	40782.75	lb		73408.95	lb		59814.7	lb		101467.5	lb
Pier SW P_W =	13125	lb		23625	lb		9900	lb		16794	lb
Wall Rigidity =	81936			175608			48200			164749	
Load to Pier =	283251.6	75533.77		424375.4	62870.43		166627.5	30295.91		569532.7	61043.16
Shear Stress=	210	psi		175	psi		115	psi		232	psi
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5	
LB Bed-Joint V_{mL} =	53	psi, 11.2.3.6.3 (11-6)		53	psi		58	psi		58	psi
In-plane V_{bjs1} =	71016	lb, 11.3.2.2.2 (11-9)		127829	lb		84258	lb		142933	lb
In-plane V_{bjs2} =	20391	lb, 11.3.2.2.2 (11-10)		36704	lb		29907	lb		50734	lb
Rocking V_r =	9097	lb, 11.3.2.2.1 (11-8)		29474	lb		17254	lb		49652	lb
Lower Bound V_{tc} =	9147	lb, 11.3.2.2.3 (11-11)		29635	lb		16976	lb		48851	lb
Lower Bound V_{dt} =	69700	lb, 11.3.2.2.4 (11-12)		154282	lb		113901	lb		193218	lb
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		591339	lb		1003126	lb
Controlling Q_{CE} =	9097	lb		29474	lb		16976	lb		48851	lb
Allowable Stress=	20	psi	**NG**	36	psi	**NG**	35	psi	**NG**	60	**NG**
DCR =	10.38	68256.22	DCR =	4.80	49770.85	DCR =	3.27	21036.31	DCR =	3.89	45335.52

Check Basement Gridline A:

Total Wall Load=	997165	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft
Pier length =	10	ft		10	ft		20	ft		20	ft
Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft
L / h_{eff} =	1.00	for (11-12)		1.00			2.00			2.00	
Trib DL P_D =	77900	lb		77900	lb		155800	lb		155800	lb
Pier SW P_W =	18000	lb		18000	lb		36000	lb		36000	lb
Wall Rigidity =	600000			600000			1527273			1527273	
Load to Pier =	140625.8	14062.58		140625.8	14062.58		357956.5	17897.83		357956.5	17897.83
Shear Stress=	39	psi		39	psi		50	psi		50	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi
In-plane V_{bjs1} =	173950	lb, 11.3.2.2.2 (11-9)		173950	lb		347900	lb		347900	lb
In-plane V_{bjs2} =	38950	lb, 11.3.2.2.2 (11-10)		38950	lb		77900	lb		77900	lb

Rocking V_r =	78210	lb, 11.3.2.2.1 (11-8)		78210	lb		312840	lb		312840	lb	
Lower Bound V_{tc} =	81388	lb, 11.3.2.2.3 (11-11)		81388	lb		325553	lb		325553	lb	
Lower Bound V_{dt} =	160326	lb, 11.3.2.2.4 (11-12)		197158	lb		433315	lb		433315	lb	
Lower Bound P_{CL} =	1468800	lb, 11.3.2.2.5 (11-13)		1468800	lb		2937600	lb		2937600	lb	
Controlling Q_{CE} =	38950	lb		38950	lb		77900	lb		77900	lb	
Allowable Stress=	32	psi	**NG**	32	psi	**NG**	32	psi	**NG**	32	**NG**	
DCR =	1.20	2377.578	DCR =	1.20	2377.578	DCR =	1.53	6212.826	DCR =	1.53	6212.826	

Check Basement Gridline B:

Total Wall Load=	1951480	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	10.00	ft		10.00	ft		8.00	ft		8.00	ft	
Pier length =	12.33	ft		12.33	ft		17.75	ft		21.75	ft	
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft	
L / h_{eff} =	1.23	for (11-12)		1.23			1.78			2.18		
Trib DL P_D =	96050.7	lb		96050.7	lb		138272.5	lb		169432.5	lb	
Pier SW P_W =	22194	lb		22194	lb		31950	lb		39150	lb	
Wall Rigidity =	252315			252315			514811			687118		
Load to Pier =	288526.9	23400.4		288526.9	23400.4		588695.4	33165.94		785730.8	36125.55	
Shear Stress=	65	psi		65	psi		126	psi		137	psi	
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		52	psi		52	psi	
In-plane V_{bjs1} =	214480	lb, 11.3.2.2.2 (11-9)		214480	lb		244542	lb		299650	lb	
In-plane V_{bjs2} =	48025	lb, 11.3.2.2.2 (11-10)		48025	lb		69136	lb		84716	lb	
Rocking V_r =	65608	lb, 11.3.2.2.1 (11-8)		65608	lb		135965	lb		204150	lb	
Lower Bound V_{tc} =	68274	lb, 11.3.2.2.3 (11-11)		68274	lb		137982	lb		207179	lb	
Lower Bound V_{dt} =	197682	lb, 11.3.2.2.4 (11-12)		243096	lb		318476	lb		390246	lb	
Lower Bound P_{CL} =	1811030	lb, 11.3.2.2.5 (11-13)		1811030	lb		1908412	lb		2338476	lb	
Controlling Q_{CE} =	48025	lb		48025	lb		69136	lb		84716	lb	
Allowable Stress=	32	psi	**NG**	32	psi	**NG**	44	psi	**NG**	44	**NG**	
DCR =	2.00	11715.4	DCR =	2.00	11715.4	DCR =	2.84	21480.94	DCR =	3.09	24440.55	

Check Basement Gridline C:

Total Wall Load=	2000308	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft	
Pier length =	12.33	ft		12.33	ft		21.75	ft		22	ft	
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft	
L / h_{eff} =	1.23	for (11-12)		1.23			2.18			2.20		
Trib DL P_D =	96050.7	lb		96050.7	lb		169432.5	lb		171380	lb	
Pier SW P_W =	22194	lb		22194	lb		39150	lb		39600	lb	
Wall Rigidity =	819977			819977			1233099			1249474		
Load to Pier =	397864.5	32268.01		397864.5	32268.01		598316.7	27508.82		606262.1	27557.37	
Shear Stress=	90	psi		90	psi		104	psi		105	psi	
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		52	psi		52	psi	
In-plane V_{bjs1} =	214480	lb, 11.3.2.2.2 (11-9)		214480	lb		299650	lb		303094	lb	
In-plane V_{bjs2} =	48025	lb, 11.3.2.2.2 (11-10)		48025	lb		84716	lb		85690	lb	
Rocking V_r =	65608	lb, 11.3.2.2.1 (11-8)		65608	lb		204150	lb		208870	lb	
Lower Bound V_{tc} =	68274	lb, 11.3.2.2.3 (11-11)		68274	lb		207179	lb		211969	lb	
Lower Bound V_{dt} =	243096	lb, 11.3.2.2.4 (11-12)		261796	lb		390246	lb		394731	lb	
Lower Bound P_{CL} =	1811030	lb, 11.3.2.2.5 (11-13)		1811030	lb		2338476	lb		2365356	lb	
Controlling Q_{CE} =	48025	lb		48025	lb		84716	lb		85690	lb	
Allowable Stress=	32	psi	**NG**	32	psi	**NG**	44	psi	**NG**	44	**NG**	
DCR =	2.76	20583.01	DCR =	2.76	20583.01	DCR =	2.35	15823.82	DCR =	2.36	15872.37	

Check Basement Gridline D:

Total Wall Load=	756737	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft	
Pier length =	9	ft		9	ft		9.25	ft		9.25	ft	

Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft	
L / h_{eff} =	0.55	for (11-12)		0.55			0.56			0.56		
Trib DL P_D =	70110	lb		70110	lb		72057.5	lb		72057.5	lb	
Pier SW P_W =	26730	lb		26730	lb		27472.5	lb		27472.5	lb	
Wall Rigidity =	506083			506083			529474			529474		
Load to Pier =	184911.1	20545.67		184911.1	20545.67		193457.7	20914.34		193457.7	20914.34	
Shear Stress=	57	psi		57	psi		58	psi		58	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi	
In-plane V_{bjs1} =	156555	lb, 11.3.2.2.2 (11-9)		156555	lb		160904	lb		160904	lb	
In-plane V_{bjs2} =	35055	lb, 11.3.2.2.2 (11-10)		35055	lb		36029	lb		36029	lb	
Rocking V_r =	40979	lb, 11.3.2.2.1 (11-8)		40979	lb		43287	lb		43287	lb	
Lower Bound V_{tc} =	42292	lb, 11.3.2.2.3 (11-11)		42292	lb		44674	lb		44674	lb	
Lower Bound V_{dt} =	181248	lb, 11.3.2.2.4 (11-12)		195190	lb		204707	lb		204707	lb	
Lower Bound P_{CL} =	1321920	lb, 11.3.2.2.5 (11-13)		1321920	lb		1358640	lb		1358640	lb	
Controlling Q_{CE} =	35055	lb		35055	lb		36029	lb		36029	lb	
Allowable Stress=	32	psi	**NG**	32	psi	**NG**	32	psi	**NG**	32	**NG**	
DCR =	1.76	8860.672	DCR =	1.76	8860.672	DCR =	1.79	9229.343	DCR =	1.79	9229.343	

Check Basement Gridline E:

Total Wall Load=	309503	lb			
Wall height =	10.00	ft	10.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	8.25	ft	8.25	ft	
Pier thickness =	2.5	ft	2.5	ft	
L / h_{eff} =	0.83	for (11-12)	0.83		
Trib DL P_D =	87494.55	lb	87494.55	lb	
Pier SW P_W =	14850	lb	14850	lb	
Wall Rigidity =	172939		172939		
Load to Pier =	154751.4	18757.74	154751.4	18757.74	309502.8
Shear Stress=	52	psi	52	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)	52	psi	
In-plane V_{bjs1} =	155122	lb, 11.3.2.2.2 (11-9)	155122	lb	
In-plane V_{bjs2} =	43747	lb, 11.3.2.2.2 (11-10)	43747	lb	
Rocking V_r =	70478	lb, 11.3.2.2.1 (11-8)	70478	lb	
Lower Bound V_{tc} =	71884	lb, 11.3.2.2.3 (11-11)	71884	lb	
Lower Bound V_{dt} =	173867	lb, 11.3.2.2.4 (11-12)	199847	lb	
Lower Bound P_{CL} =	1211760	lb, 11.3.2.2.5 (11-13)	1211760	lb	
Controlling Q_{CE} =	43747	lb	43747	lb	
Allowable Stress=	44	psi **NG**	44	psi **NG**	
DCR =	1.18	2849.643	DCR = 1.18	2849.643	

Wall Anchorage - Check shear capacity of anchorage, Sect. 5.7.1.1

Tier 1 - Quick Check, Sect. 4.5.3.7

Wall thickness =	1.5	ft
Wall W =	180	psf
X =	1.2	from Sect 4.5.3.7
S_x =	0.86	from 4.5.2.1 (2-1)
A_p =	54.68	sf, area to anchor
T_c =	10129.25	lb **Tc > Ta NG**
m =	3	from Table 12-3 for Immediate Occupancy
Anchor Capacity =	5845.5	lb, per TMS 3.1.6.3.1.2

Tier 2 - Sect. 5.7.1.1 and 7.2.11.1

Height =	13.67	ft
Wall thickness =	1.5	ft
Ratio =	9.11	
X =	1	from Table 7-2

K_a =	1.515	Eq 7-11
K_h =	1	Eq 7-12
F_p =	93.55	psf, from 7.2.11.1 (7-9)
F_{pMIN} =	54.54	psf, from 7.2.11.1 (7-10)
T_c =	5115.27	lb **Tc < Ta OK**
DCR =	0.88	

Proportions - Check Height to Thickness Rations, Sect. 5.5.3.1.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

Top story height =	9.00	ft			
Wall thickness =	1	ft	Ratio:	9.0	**OK**
Bell Lounge h =	13.67	ft			
Wall thickness =	1.33	ft	Ratio:	10.3	**NG**
First story height =	10.00	ft			
Wall thickness =	1.5	ft	Ratio:	6.7	**OK**
Other story height =	16.5	ft			
Wall thickness =	1.5	ft	Ratio:	11.0	**OK**

Tier 2 - Bell Tower Lounge - Sect. 11.3.3.2

Height =	13.67	ft	
Wall thickness =	1.33	ft	
Ratio =	10.28		
X =	1	from Table 7-2	
F_p =	136.88	psf, from 7.2.11.2 (7-13)	
OOP Moment =	3197.237	ft-lbs	
F_t =	75.31	psi	**> Ft = 45 psi NG**
DCR =	1.67		

Tier 2 - Third Floor - Sect. 11.3.3.2

Height =	9.00	ft	
Wall thickness =	1	ft	
Ratio =	9.00		
X =	1	from Table 7-2	
F_p =	102.91	psf, from 7.2.11.2 (7-13)	
OOP Moment =	1042.008	ft-lbs	
F_t =	43.42	psi	**< Ft = 45 psi OK**
DCR =	0.96		

For Collapse Prevention - Bell Tower Lounge - Sect. 11.3.3.2

S_{X1} =	0.58	from 4.5.2.1
TOP Allowable h/t =	9	**per Table 11-5, NG**
1st Allowable h/t =	13	**per Table 11-5, OK**
Other h/t =	15	**per Table 11-5, OK**

Diaphragms - Openings at Shear Walls, Sect. 5.6.1.3

Bell Tower Lounge Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	8.50	ft	
Diaphragm V =	362	plf, from page	
Revised Dia. V =	479.93	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.07		

Wall OOP, Sect. 11.3.3.2

Height =	27.92	ft	
Wall thickness =	1.5	ft	
Ratio =	18.61		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	15042.09	ft-lbs	
F_t =	278.56	psi	**> Ft = 45 psi NG**
DCR =	6.19		

3rd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	16.75	ft	
Diaphragm V =	445	plf, from page	
Revised Dia. V =	864.49	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.92		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft
Wall thickness =	1.5	ft

Diaphragm strength, Sect. 12.5.2.1

GL E Opening =	9.75	ft	
Diaphragm V =	446	plf, from page	
Revised Dia. V =	870.75	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.94		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft
Wall thickness =	1.5	ft

Ratio =	14.50		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	9128.427	ft-lbs	
F_t =	169.04	psi	**> Ft = 45 psi NG**
DCR =	3.76		

Ratio =	14.50		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	9128.427	ft-lbs	
F_t =	169.04	psi	**> Ft = 45 psi NG**
DCR =	3.76		

3rd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft	
Diaphragm V =	329	plf, from page	
Revised Dia. V =	373.12	plf	**OK**
Dia. Capacity =	450	plf, from page	
DCR =	0.83		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	9128.427	ft-lbs	
F_t =	169.04	psi	**> Ft = 45 psi NG**
DCR =	3.76		

2nd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 & 6 Opening =	14.25	ft	
Diaphragm V =	697	plf, from page	
Revised Dia. V =	1187.67	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	2.64		

Wall OOP, Sect. 11.3.3.2

Height =	31.50	ft	
Wall thickness =	1.5	ft	
Ratio =	21.00		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	19146.9	ft-lbs	
F_t =	354.57	psi	**> Ft = 45 psi NG**
DCR =	7.88		

2nd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft	
Diaphragm V =	520	plf, from page	
Revised Dia. V =	590.29	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.31		

Wall OOP, Sect. 11.3.3.2

Height =	16.50	ft	
Wall thickness =	1.5	ft	
Ratio =	11.00		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	5253.459	ft-lbs	
F_t =	97.29	psi	**> Ft = 45 psi NG**
DCR =	2.16		

1st Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	9.00	ft	
Diaphragm V =	373	plf, from page	
Revised Dia. V =	504.46	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.12		

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	13.33	ft	
Diaphragm V =	373	plf, from page	
Revised Dia. V =	607.63	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.35		

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft	
Wall thickness =	1.5	ft	
Ratio =	17.00		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	12547.52	ft-lbs	
F_t =	232.36	psi	**> Ft = 45 psi NG**
DCR =	5.16		

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft	
Wall thickness =	1.5	ft	
Ratio =	17.00		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	12547.52	ft-lbs	
F_t =	232.36	psi	**> Ft = 45 psi NG**
DCR =	5.16		

Stair Enclosures - Check Height to Thickness Rations, Sect. A.7.10.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

3rd Floor Opng =	21.75	ft			
Wall thickness =	1	ft	Ratio:	21.8	**> 12, NG**
2nd Floor Opng =	31.50	ft			
Wall thickness =	1.5	ft	Ratio:	21.0	**> 12, NG**
First story height =	25.50	ft			
Wall thickness =	1.5	ft	Ratio:	17.0	**> 12, NG**

Tier 2 - 3rd Floor Opening - Sect. Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1	ft	
Ratio =	21.75		
X =	1	from Table 7-2	
F_p =	102.91	psf, from 7.2.11.2 (7-13)	
OOP Moment =	6085.618	ft-lbs	
F_t =	253.57	psi	**> Ft = 45 psi NG**
DCR =	5.63		

2rd Floor Opening

31.50	ft	
1.5	ft	
21.00		
1	from Table 7-2	
154.37	psf, from 7.2.11.2 (7-13)	
19146.9	ft-lbs	
354.57	psi	**> Ft = 45 psi NG**
7.88		

1st Floor Opening

25.50	ft	
1.5	ft	
17.00		
1	from Table 7-2	
154.37	psf, from 7.2.11.2 (7-13)	
12547.52	ft-lbs	
232.36	psi	**> Ft = 45 psi NG**
5.16		

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS AT ROOF **Shape Capacity = 0.49 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	30.21	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.49	
Tr/Tc =		
	0.49	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS LOWER **Shape Capacity = 0.68 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	41.34	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.68	
Tr/Tc =		
	0.68	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Concrete Shear Wall Design per ACI 318-05

Wall Mark 2FL GL 3

Wall Properties

t_{wall} =	8	in
h_{wall} =	15	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	7505	plf, dead laod	DL =	0.9	w_{du} =	6755	plf
w_{wall} =	1500	plf, wall weight	LL =	0.0	w_{wallu} =	1350	plf
w_s =	100	plf, snow load	SL =	0.0	w_{su} =	0	plf
w_l =	320	plf, live load	W =	1.6	w_{lu} =	0	plf
			E =	1.4			

Load Factors

Lateral Loading

C_s =	0.36		
V_{seis} =	327998	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	10.19318	0.78	2.35	51%	Controls
1	8	9.931818	0.81	2.27	49%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		20.125	ft	4.62	100%	

P_{du} =	69161	lb/pier
P_{wallu} =	13822	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	82983	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	97.85455	in	(ACI 11.10.4)	
V_u =	233893	lb/pier		
$\phi V_{n,max}$ =	415162	lb > 233893 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	69194	lb < 233893 NG	(ACI 21.7.2.1)	
ϕV_n =	354363	lb < 233893 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	138387	lb < 233893 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	196442	lb		
$\phi V_c/2$ =	73666	lb < 233893 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0343 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2	**Max spacing**		
s_2 =	15	in	$l_w/5$ =	24.46364	in
A_v =	0.25	in^2/ft	$3t_{wall}$ =	24	in
A_g =	96	in^2	18 =	18	in
			s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0025	(EQ. 11-32)	$l_w/3$ =	40.77273	in
R'qd A_v =	0.163	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.163 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	1890963	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	82983	lb
$M_{u(max)}$ =	1890963	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	2124681	lb > 82983 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	97.85455	in	$0.10f'_cA_g$ =	489272.7	lb > 82983 OK
R_n =	329	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0057		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.47	in^2	$A_{s,min}$ =	2.77	in^2
Try	8	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0076	
A_s =	4.8	in^2	ρ_{act} =	0.0061	> 0.0035 OK
a =	8.47	in			
c =	9.97	in			
ε_t =	0.0265	> 0.004 OK (Tension Controls)			
Mn =	2246862	ft-lb			
ϕM_n =	2022176	ft-lb > 1890963 OK			

<u>Boundary Elements</u>

A_g =	979	in^2	
S_g =	19949	in^3	
P_u =	82983	lb	
M_u =	1890963	ft-lb	
σ =	1222	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1222 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0061	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 2FL GL 6

Wall Properties

t_{wall} =	8	in
h_{wall} =	15	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	7505	plf, dead laod	DL =	0.9	w_{du} =	6755	plf
w_{wall} =	1500	plf, wall weight	LL =	0.0	w_{wallu} =	1350	plf
w_s =	100	plf, snow load	SL =	0.0	w_{su} =	0	plf
w_l =	320	plf, live load	W =	1.6	w_{lu} =	0	plf
			E =	1.4			

Load Factors

Lateral Loading

C_s =	0.36		
V_{seis} =	327988	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	12.02273	0.67	2.91	63%	Controls
1	8	8.102273	0.99	1.70	37%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		20.125	ft	4.61	100%	

P_{du} =	85838	lb/pier
P_{wallu} =	17155	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	102994	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	115.4182	in	(ACI 11.10.4)	
V_u =	290286	lb/pier		
$\phi V_{n,max}$ =	489678	lb > 290286 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	81613	lb < 290286 NG	(ACI 21.7.2.1)	
ϕV_n =	417966	lb < 290286 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	163226	lb < 290286 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	231701	lb		
$\phi V_c/2$ =	86888	lb < 290286 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0370 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 16"o.c.

A =	0.31	in^2
s_2 =	16	in
A_v =	0.23	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	28.85455	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0024	< 0.0025 NG		**16" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0025	(EQ. 11-32)	$l_w/3$ =	48.09091	in
R'qd A_v =	0.160	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 16 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	16	in		**16" < 18" OK**	
A_v =	0.2325	> 0.16 OK			
$\rho_{l,actual}$ =	0.0024	< 0.0025 NG, need more steel			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

<u>BENDING</u> - ACI Chapter 10

M_u =	2345527	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	102994	lb
$M_{u(max)}$ =	2345527	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	2482979	lb > 102994 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	115.4182	in	$0.10f'_cA_g$ =	577090.9	lb > 102994 OK
R_n =	293	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0051		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.68	in^2	$A_{s,min}$ =	3.26	in^2
Try	8	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0068	
A_s =	4.8	in^2	ρ_{act} =	0.0052	> 0.0035 OK
a =	8.47	in			
c =	9.97	in			
ε_t =	0.0317	> 0.004 OK (Tension Controls)			
Mn =	2668389	ft-lb			
ϕM_n =	2401550	ft-lb > 2345527 OK			

<u>Boundary Elements</u>

A_g =	1154	in^2	
S_g =	27753	in^3	
P_u =	102994	lb	
M_u =	2345527	ft-lb	
σ =	1103	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1103 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0052	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL 3

Wall Properties

t_{wall} =	8	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod		
w_{wall} =	1650	plf, wall weight		
w_s =	100	plf, snow load		
w_l =	320	plf, live load		

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1485	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	412917	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	424	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	17.5	0.46	4.54	100%	Controls
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		17.5	ft	4.54	100%	

P_{du} =	163633	lb/pier
P_{wallu} =	25988	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	189620	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	168	in	(ACI 11.10.4)	
V_u =	578678	lb/pier		
$\phi V_{n,max}$ =	712764	lb > 578678 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	118794	lb < 578678 NG	(ACI 21.7.2.1)	
ϕV_n =	608382	lb < 578678 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	237588	lb < 578678 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	190070	lb		
$\phi V_c/2$ =	71276	lb < 578678 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0577 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	42	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	70	in
R'qd A_v =	0.164	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.164 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	4666250	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	189620	lb
$M_{u(max)}$ =	4666250	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3635928	lb > 189620 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	168	in	$0.10f'_cA_g$ =	840000	lb > 189620 OK
R_n =	276	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0048		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	6.39	in^2	$A_{s,min}$ =	4.75	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0063	
A_s =	7.8	in^2	ρ_{act} =	0.0058	> 0.0035 OK
a =	13.76	in			
c =	16.19	in			
ε_t =	0.0281	> 0.004 OK (Tension Controls)			
Mn =	6283588	ft-lb			
ϕM_n =	5655229	ft-lb > 4666250 OK			

Boundary Elements

A_g =	1680	in^2	
S_g =	58800	in^3	
P_u =	189620	lb	
M_u =	4666250	ft-lb	
σ =	1065	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1065 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0058	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL 6

Wall Properties

t_{wall} =	8	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1650	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1485	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	412917	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	424	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	18.33	0.44	4.79	100%	Controls
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		18.33	ft	4.79	100%	

P_{du} =	171394	lb/pier
P_{wallu} =	27220	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	198614	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	175.968	in	(ACI 11.10.4)	
V_u =	578678	lb/pier		
$\phi V_{n,max}$ =	746569	lb > 578678 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	124428	lb < 578678 NG	(ACI 21.7.2.1)	
ϕV_n =	637236	lb < 578678 NG	(ACI EQ. 21-7)	
$2A_{cv}$sqrt(f_c') =	248856	lb < 578678 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	199085	lb		
$\phi V_c/2$ =	74657	lb < 578678 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0542 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	96	in^2

Max spacing

$l_w/5$ =	43.992	in
$3t_{wall}$ =	24	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	73.32	in
R'qd A_v =	0.164	in^2/ft	$3t_{wall}$ =	24	in
Try #5 at 15 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	15	in		**15" < 18" OK**	
A_v =	0.248	> 0.164 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	24	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	96	in^2		**16" < 18" OK**	
ρ_t =	0.0024	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	96	in^2
ρ_t =	0.0022	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	4668222	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	198614	lb
$M_{u(max)}$ =	4668222	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3798475	lb > 198614 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	175.968	in	$0.10f'_cA_g$ =	879840	lb > 198614 OK
R_n =	251	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0043		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	6.08	in^2	$A_{s,min}$ =	4.98	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0058	
A_s =	7.8	in^2	ρ_{act} =	0.0055	> 0.0035 OK
a =	13.76	in			
c =	16.19	in			
ε_t =	0.0296	> 0.004 OK (Tension Controls)			
Mn =	6594340	ft-lb			
ϕM_n =	5934906	ft-lb > 4668222 OK			

Boundary Elements

A_g =	1760	in^2	
S_g =	64510	in^3	
P_u =	198614	lb	
M_u =	4668222	ft-lb	
σ =	981	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 981 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0055	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL B

Wall Properties

t_{wall} =	10	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	2062.5	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1856	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	673640	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	530	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	16.25	0.49	5.22	32%	Controls
1	8	15.33	0.52	4.88	30%	
1	8	10.67	0.75	3.12	19%	
1	8	10.25	0.78	2.96	18%	
total wall length =		52.5	ft	16.18	100%	

P_{du} =	158401	lb/pier
P_{wallu} =	31446	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	189847	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	156	in	(ACI 11.10.4)	
V_u =	304554	lb/pier		
$\phi V_{n,max}$ =	827315	lb > 304554 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	137886	lb < 304554 NG	(ACI 21.7.2.1)	
ϕV_n =	706157	lb < 304554 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	275772	lb < 304554 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	394407	lb		
$\phi V_c/2$ =	147903	lb < 304554 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0198 in^2/in/ft (EQ. 11-31)

Try #5 at 12"o.c.

A =	0.31	in^2
s_2 =	12	in
A_v =	0.31	in^2/ft
A_g =	120	in^2

Max spacing

$l_w/5$ =	39	in
$3t_{wall}$ =	30	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**12" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	65	in
R'qd A_v =	0.205	in^2/ft	$3t_{wall}$ =	30	in
Try #5 at 12 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	12	in		**12" < 18" OK**	
A_v =	0.31	> 0.205 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	30	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	120	in^2		**16" < 18" OK**	
ρ_t =	0.0019	< 0.002 NG, more steel required			

Vertical Steel - (ACI 14.3.2)

Try #5 at 15 o.c.		
A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	120	in^2
ρ_t =	0.0021	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2482780	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	189847	lb
$M_{u(max)}$ =	2482780	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	4122504	lb > 189847 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	156	in	$0.10f'_cA_g$ =	975000	lb > 189847 OK
R_n =	136	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0023		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	3.60	in^2	$A_{s,min}$ =	5.52	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0031	
A_s =	5.4	in^2	ρ_{act} =	0.0035	> 0.0031 OK
a =	7.62	in			
c =	8.97	in			
ε_t =	0.0492	> 0.004 OK (Tension Controls)			
Mn =	4109082	ft-lb			
ϕM_n =	3698174	ft-lb > 2482780 OK			

Boundary Elements

A_g =	1950	in^2	
S_g =	63375	in^3	
P_u =	189847	lb	
M_u =	2482780	ft-lb	
σ =	567	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 567 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0035	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark 1FL GL C

Wall Properties

t_{wall} =	10	in
h_{wall} =	16.5	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	2062.5	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1856	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	673640	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	530	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	13.75	0.58	4.29	37%	Controls
1	8	12.67	0.63	3.88	34%	
1	8	7.67	1.04	1.95	17%	
1	8	6.33	1.26	1.43	12%	
total wall length =		40.42	ft	11.56	100%	

P_{du} =	140238	lb/pier
P_{wallu} =	27840	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	168079	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	132	in	(ACI 11.10.4)	
V_u =	350216	lb/pier		
$\phi V_{n,max}$ =	700036	lb > 350216 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	116673	lb < 350216 NG	(ACI 21.7.2.1)	
ϕV_n =	597518	lb < 350216 NG	(ACI EQ. 21-7)	
$2A_{cv}$sqrt(f_c') =	233345	lb < 350216 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	333729	lb		
$\phi V_c/2$ =	125149	lb < 350216 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0354 $in^2/in/ft$ (EQ. 11-31)

Try #5 at 12"o.c.

A =	0.31	in^2
s_2 =	12	in
A_v =	0.31	in^2/ft
A_g =	120	in^2

Max spacing

$l_w/5$ =	33	in
$3t_{wall}$ =	30	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0026	> 0.0025 OK		**12" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0026	(EQ. 11-32)	$l_w/3$ =	55	in
R'qd A_v =	0.204	in^2/ft	$3t_{wall}$ =	30	in
Try #5 at 12 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	12	in		**12" < 18" OK**	
A_v =	0.31	> 0.204 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 16 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	30	in
s_2 =	16	in	18 =	18	in
A_v =	0.23	in^2/ft	s_{max} =	18	in
A_g =	120	in^2		**16" < 18" OK**	
ρ_t =	0.0019	< 0.002 NG, more steel required			

Vertical Steel - (ACI 14.3.2)

Try #5 at 15 o.c.		
A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	120	in^2
ρ_t =	0.0021	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2840364	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	168079	lb
$M_{u(max)}$ =	2840364	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3510504	lb > 168079 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	132	in	$0.10f'_cA_g$ =	825000	lb > 168079 OK
R_n =	217	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0037		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.91	in^2	$A_{s,min}$ =	4.67	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0050	
A_s =	5.4	in^2	ρ_{act} =	0.0041	> 0.0035 OK
a =	7.62	in			
c =	8.97	in			
ε_t =	0.0412	> 0.004 OK (Tension Controls)			
Mn =	3461082	ft-lb			
ϕM_n =	3114974	ft-lb > 2840364 OK			

Boundary Elements

A_g =	1650	in^2	
S_g =	45375	in^3	
P_u =	168079	lb	
M_u =	2840364	ft-lb	
σ =	853	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 853 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0041	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL 3

Wall Properties

t_{wall} =	12	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1500	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1350	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	470910	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	386	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	9.5	0.84	3.20	43%	Controls
1	8	8.25	0.97	2.62	35%	
1	8	6	1.33	1.57	21%	
0	0	0	0.00	0.00	0%	
total wall length =		23.75	ft	7.39	100%	

P_{du} =	96228	lb/pier
P_{wallu} =	13893	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	110121	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	91.2	in	(ACI 11.10.4)	
V_u =	285908	lb/pier		
$\phi V_{n,max}$ =	580393	lb > 285908 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	96732	lb < 285908 NG	(ACI 21.7.2.1)	
ϕV_n =	495397	lb < 285908 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	193464	lb < 285908 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	273139	lb		
$\phi V_c/2$ =	102427	lb < 285908 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0414 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	144	in^2

Max spacing

$l_w/5$ =	22.8	in
$3t_{wall}$ =	36	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0017	< 0.0025 NG		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0019	(EQ. 11-32)	$l_w/3$ =	38	in
R'qd A_v =	0.240	in^2/ft	$3t_{wall}$ =	36	in
Try #5 at 9 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	9	in		**9" < 18"**	**OK**
A_v =	0.413333	> 0.24 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 9 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	36	in
s_2 =	9	in	18 =	18	in
A_v =	0.41	in^2/ft	s_{max} =	18	in
A_g =	144	in^2		**9" < 18"**	**OK**
ρ_t =	0.0029	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 18 o.c.		
A =	0.31	in^2
s_2 =	18	in
A_v =	0.21	in^2/ft
A_g =	144	in^2
ρ_t =	0.0014	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2305909	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	110121	lb
$M_{u(max)}$ =	2305909	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3031560	lb > 110121 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	91.2	in	$0.10f'_cA_g$ =	684000	lb > 110121 OK
R_n =	308	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0053		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	5.84	in^2	$A_{s,min}$ =	3.87	in^2
Try	15	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0071	
A_s =	9	in^2	ρ_{act} =	0.0082	> 0.0035 OK
a =	10.59	in			
c =	12.46	in			
ε_t =	0.0190	> 0.004 OK (Tension Controls)			
Mn =	3865765	ft-lb			
ϕM_n =	3479188	ft-lb > 2305909 OK			

Boundary Elements

A_g =	1368	in^2	
S_g =	25992	in^3	
P_u =	110121	lb	
M_u =	2305909	ft-lb	
σ =	1145	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1145 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0082	> 0.0067 Provide Boundary Ties	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL 6

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf,	dead laod
w_{wall} =	1875	plf,	wall weight
w_s =	100	plf,	snow load
w_l =	320	plf,	live load

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	470910	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	9	0.89	3.71	53%	Controls
1	8	6.75	1.19	2.39	34%	
1	8	4	2.00	0.89	13%	
0	0	0	0.00	0.00	0%	
total wall length =		19.75	ft	7.00	100%	

P_{du} =	97917	lb/pier
P_{wallu} =	17671	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	115588	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	86.4	in	(ACI 11.10.4)	
V_u =	349919	lb/pier		
$\phi V_{n,max}$ =	687308	lb > 349919 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	114551	lb < 349919 NG	(ACI 21.7.2.1)	
ϕV_n =	586654	lb < 349919 NG	(ACI EQ. 21-7)	
2A_{cv}sqrt(f_c') =	229103	lb < 349919 NG	(ACI 21.7.2.2)	**Two curtains of steel req'd**
V_c =	319246	lb		
$\phi V_c/2$ =	119717	lb < 349919 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0546 in^2/in/ft (EQ. 11-31)

Try #5 at 15"o.c.

A =	0.31	in^2
s_2 =	15	in
A_v =	0.25	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	21.6	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0014	< 0.0025 NG		**15" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0017	(EQ. 11-32)	$l_w/3$ =	36	in
R'qd A_v =	0.300	in^2/ft	$3t_{wall}$ =	45	in
Try #5 at 8 o.c.			18 =	18	in
A =	0.31	in^2	s_{max} =	18	in
s =	8	in		**8" < 18"**	**OK**
A_v =	0.465	> 0.3 OK			
$\rho_{l,actual}$ =	0.0026	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #5 at 9 o.c.			**Max spacing**		
A =	0.31	in^2	$3t_{wall}$ =	45	in
s_2 =	9	in	18 =	18	in
A_v =	0.41	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**9" < 18"**	**OK**
ρ_t =	0.0023	> 0.002 OK			

Vertical Steel - (ACI 14.3.2)

Try #5 at 9 o.c.		
A =	0.31	in^2
s_2 =	9	in
A_v =	0.41	in^2/ft
A_g =	180	in^2
ρ_t =	0.0023	> 0.0012 OK

BENDING - ACI Chapter 10

M_u =	2820793	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	115588	lb
$M_{u(max)}$ =	2820793	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	3513528	lb > 115588 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	86.4	in	$0.10f'_cA_g$ =	810000	lb > 115588 OK
R_n =	336	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0058		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	7.57	in^2	$A_{s,min}$ =	4.58	in^2
Try	13	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0078	
A_s =	7.8	in^2	ρ_{act} =	0.0060	> 0.0035 OK
a =	7.34	in			
c =	8.64	in			
ε_t =	0.0270	> 0.004 OK (Tension Controls)			
Mn =	3226447	ft-lb			
ϕM_n =	2903802	ft-lb > 2820793 OK			

Boundary Elements

A_g =	1620	in^2	
S_g =	29160	in^3	
P_u =	115588	lb	
M_u =	2820793	ft-lb	
σ =	1232	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi < 1232 NG - Provide Boundary Steel	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0060	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL B

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod		
w_{wall} =	1875	plf, wall weight		
w_s =	100	plf, snow load		
w_l =	320	plf, live load		

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	808243	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	28	0.29	14.20	58%	Controls
1	8	13.5	0.59	6.29	26%	
1	8	9.75	0.82	4.15	17%	
0	0	0	0.00	0.00	0%	
total wall length =		51.25	ft	24.64	100%	

P_{du} =	276123	lb/pier
P_{wallu} =	49833	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	325955	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	268.8	in	(ACI 11.10.4)	
V_u =	652387	lb/pier		
$\phi V_{n,max}$ =	2138291	lb > 652387 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	356382	lb < 652387 NG	(ACI 21.7.2.1)	
ϕV_n =	1825145	lb > 652387 OK	(ACI EQ. 21-7)	
$2A_{cv}$sqrt(f_c') =	712764	lb > 652387 OK	(ACI 21.7.2.2)	**Min. One Curtain of ReStl**
V_c =	570211	lb		
$\phi V_c/2$ =	213829	lb < 652387 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0186 $in^2/in/ft$ (EQ. 11-31)

Try #6 at 10"o.c.

A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	67.2	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0029	> 0.0025 OK		**10" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0030	(EQ. 11-32)	$l_w/3$ =	112	in
R'qd A_v =	0.356	in^2/ft	$3t_{wall}$ =	45	in
Try #6 at 10 o.c.			18 =	18	in
A =	0.44	in^2	s_{max} =	18	in
s =	10	in		**10" < 18" OK**	
A_v =	0.528	> 0.356 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #6 at 10 o.c.			**Max spacing**		
A =	0.44	in^2	$3t_{wall}$ =	45	in
s_2 =	10	in	18 =	18	in
A_v =	0.53	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**10" < 18" OK**	
ρ_t =	0.0029	> 0.0025 OK			

Vertical Steel - (ACI 14.3.2)

Try #6 at 12 o.c.		
A =	0.44	in^2
s_2 =	12	in
A_v =	0.44	in^2/ft
A_g =	180	in^2
ρ_t =	0.0024	> 0.0015 OK

BENDING - ACI Chapter 10

M_u =	5291587	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	325955	lb
$M_{u(max)}$ =	5291587	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	10426104	lb > 325955 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	268.8	in	$0.10f'_cA_g$ =	2520000	lb > 325955 OK
R_n =	65	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0011		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.41	in^2	$A_{s,min}$ =	14.26	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0015	
A_s =	5.4	in^2	ρ_{act} =	0.0013	< 0.0015 NG, more steel required
a =	5.08	in			
c =	5.98	in			
ε_t =	0.1319	> 0.004 OK (Tension Controls)			
Mn =	7188988	ft-lb			
ϕM_n =	6470089	ft-lb > 5291587 OK			

Boundary Elements

A_g =	5040	in^2	
S_g =	282240	in^3	
P_u =	325955	lb	
M_u =	5291587	ft-lb	
σ =	290	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 290 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0013	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Concrete Shear Wall Design per ACI 318-05

Wall Mark Bsmt GL C

Wall Properties

t_{wall} =	15	in
h_{wall} =	10	ft
f'_c =	5000	psi
fy =	60000	psi

Vertical Loading

w_d =	10389	plf, dead laod		
w_{wall} =	1875	plf, wall weight		
w_s =	100	plf, snow load		
w_l =	320	plf, live load		

Load Factors

DL =	0.9	w_{du} =	9350	plf
LL =	0.0	w_{wallu} =	1688	plf
SL =	0.0	w_{su} =	0	plf
W =	1.6	w_{lu} =	0	plf
E =	1.4			

Lateral Loading

C_s =	0.36		
V_{seis} =	808243	lb	Seismic Wall Line Shear, (ASD)
V_{wall} =	482	plf	Wall Line Dead Load Shear
V_{wind} =	0	lb	Wind Wall Line Shear

Piers

No. of walls	Height h (ft)	Length l_w (ft)	h/l Ratio	Relative Rigidity	% of Load to Each Pier	
1	8	22.5	0.36	11.24	50%	**Controls**
1	8	22.25	0.36	11.11	50%	
0	0	0	0.00	0.00	0%	
0	0	0	0.00	0.00	0%	
total wall length =		44.75	ft	22.35	100%	

P_{du} =	210480	lb/pier
P_{wallu} =	37986	lb/pier
P_{su} =	0	lb/pier
P_u =	0	lb/pier
Total =	248466	lb/pier

SHEAR DESIGN - Chapter 11

α_c =	3.0		(ACI 21.7.4)	
ϕ =	0.75		(ACI 9.3.2.3)	
d =	216	in	(ACI 11.10.4)	
V_u =	569529	lb/pier		
$\phi V_{n,max}$ =	1718269	lb > 569529 OK	(ACI 11.10.3 & 21.7.4.4)	
A_{cv}sqrt(f_c') =	286378	lb < 569529 NG	(ACI 21.7.2.1)	
ϕV_n =	1466635	lb > 569529 OK	(ACI EQ. 21-7)	
$2A_{cv}$sqrt(f_c') =	572756	lb > 569529 OK	(ACI 21.7.2.2)	**Min. One Curtain of ReStl**
V_c =	458205	lb		
$\phi V_c/2$ =	171827	lb < 569529 NG	(ACI 11.10.8)	**Shear Steel Req'd**

Horizontal Shear Steel - (ACI 11.10.9)

$Av/s_2 = (V_u - \phi V_c)/\phi f_y d$ = 0.0232 in^2/in/ft (EQ. 11-31)

Try #6 at 10"o.c.

A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2

Max spacing

$l_w/5$ =	54	in
$3t_{wall}$ =	45	in
18 =	18	in
s_{max} =	18	in

ρ_t =	0.0029	> 0.0025 OK		**10" < 18" OK**	

Vertical Shear Steel - (ACI 11-32)

ρ_l =	0.0025	**OR**	**Max spacing**		
ρ_l =	0.0029	(EQ. 11-32)	$l_w/3$ =	90	in
R'qd A_v =	0.353	in^2/ft	$3t_{wall}$ =	45	in
Try #6 at 10 o.c.			18 =	18	in
A =	0.44	in^2	s_{max} =	18	in
s =	10	in		**10" < 18" OK**	
A_v =	0.528	> 0.353 OK			
$\rho_{l,actual}$ =	0.0029	> 0.0025 OK			

Horizontal steel - (ACI 14.3.3)

Try #6 at 10 o.c.			**Max spacing**		
A =	0.44	in^2	$3t_{wall}$ =	45	in
s_2 =	10	in	18 =	18	in
A_v =	0.53	in^2/ft	s_{max} =	18	in
A_g =	180	in^2		**10" < 18" OK**	
ρ_t =	0.0029	> 0.0025 OK			

Vertical Steel - (ACI 14.3.2)

Try #6 at 10 o.c.		
A =	0.44	in^2
s_2 =	10	in
A_v =	0.53	in^2/ft
A_g =	180	in^2
ρ_t =	0.0029	> 0.0015 OK

BENDING - ACI Chapter 10

M_u =	4614264	ft-lb, seismic			
M_u =	0	ft-lb, wind	P_u =	248466	lb
$M_{u(max)}$ =	4614264	ft-lb	Check axial strength (ACI 10.3.6.2):		
Assume tension controlled			$\phi P_{n,max}$ =	8406504	lb > 248466 OK
ϕ =	0.9		Check strain limit (ACI 10.3.5):		
d =	216	in	$0.10f'_cA_g$ =	2025000	lb > 248466 OK
R_n =	88	psi	$\varepsilon_{t(actual)}$ =	0.004	
ρ_{req} =	0.0015		Check minimum reinforcement (ACI 10.5):		
R'qd A_s =	4.80	in^2	$A_{s,min}$ =	11.46	in^2
Try	9	#7 Bars	ρ_{min} =	0.0035	
A =	0.60	in^2	1/3 greater ρ_{req} =	0.0020	
A_s =	5.4	in^2	ρ_{act} =	0.0017	< 0.002 NG, more steel required
a =	5.08	in			
c =	5.98	in			
ε_t =	0.1054	> 0.004 OK (Tension Controls)			
Mn =	5763388	ft-lb			
ϕM_n =	5187049	ft-lb > 4614264 OK			

Boundary Elements

A_g =	4050	in^2	
S_g =	182250	in^3	
P_u =	248466	lb	
M_u =	4614264	ft-lb	
σ =	365	psi = P/A + M/S	
$0.2f'_c$ =	1000	psi > 365 OK - No Boundary Steel Req'd	(ACI 21.7.6.3)
400/fy =	0.0067		
$\rho_{long.}$ =	0.0017	< 0.0067 OK - No Boundary Ties Req'd	(ACI 21.7.6.5)

Current Date: 2/25/2016 1:17 AM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 3.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	41.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 3

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00
3	15.00	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	9.50	0.00	3.00	7.00
Lower left	17.50	10.00	3.00	10.00
Lower left	10.00	26.50	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
3	EQ	Horizontal	327998.00	0.00	0.00
2	EQ	Horizontal	84930.00	0.00	0.00
1	EQ	Horizontal	57990.00	0.00	0.00

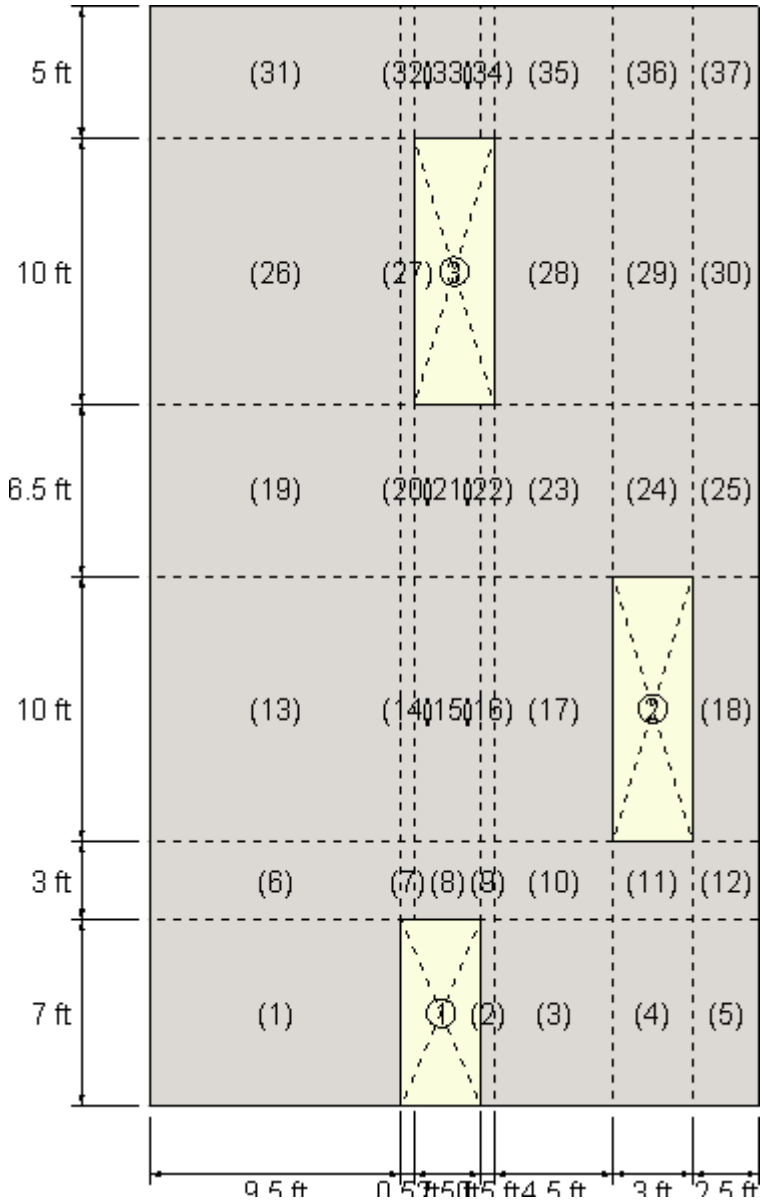
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
3	DL	Vertical	3360.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	0.50	7.00
3	13.00	0.00	4.50	7.00
4	17.50	0.00	3.00	7.00
5	20.50	0.00	2.50	7.00
6	0.00	7.00	9.50	3.00
7	9.50	7.00	0.50	3.00
8	10.00	7.00	2.50	3.00
9	12.50	7.00	0.50	3.00
10	13.00	7.00	4.50	3.00
11	17.50	7.00	3.00	3.00
12	20.50	7.00	2.50	3.00
13	0.00	10.00	9.50	10.00
14	9.50	10.00	0.50	10.00
15	10.00	10.00	2.50	10.00
16	12.50	10.00	0.50	10.00

17	13.00	10.00	4.50	10.00
18	20.50	10.00	2.50	10.00
19	0.00	20.00	9.50	6.50
20	9.50	20.00	0.50	6.50
21	10.00	20.00	2.50	6.50
22	12.50	20.00	0.50	6.50
23	13.00	20.00	4.50	6.50
24	17.50	20.00	3.00	6.50
25	20.50	20.00	2.50	6.50
26	0.00	26.50	9.50	10.00
27	9.50	26.50	0.50	10.00
28	13.00	26.50	4.50	10.00
29	17.50	26.50	3.00	10.00
30	20.50	26.50	2.50	10.00
31	0.00	36.50	9.50	5.00
32	9.50	36.50	0.50	5.00
33	10.00	36.50	2.50	5.00
34	12.50	36.50	0.50	5.00
35	13.00	36.50	4.50	5.00
36	17.50	36.50	3.00	5.00
37	20.50	36.50	2.50	5.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	12-#9	10.00	43.69
2	1-#6	14.00	23.24
3	4-#6	14.00	23.24
4	3-#6	14.00	23.24
5	2-#6	14.00	23.24
6	12-#9	10.00	43.69
7	1-#6	14.00	23.24
8	2-#6	14.00	23.24
9	1-#6	14.00	23.24
10	4-#6	14.00	23.24
11	3-#6	14.00	23.24
12	2-#6	14.00	23.24
13	7-#9	18.00	43.69
14	1-#8	16.00	38.73
15	2-#8	16.00	38.73
16	1-#8	16.00	38.73
17	4-#8	16.00	38.73
18	2-#5	18.00	19.36
19	7-#9	18.00	43.69
20	1-#8	16.00	38.73
21	2-#8	16.00	38.73
22	1-#8	16.00	38.73
23	4-#8	16.00	38.73
24	2-#5	18.00	19.36
25	2-#5	18.00	19.36
26	8-#6	14.00	23.24
27	1-#6	14.00	23.24
28	4-#5	14.00	19.36
29	3-#5	14.00	19.36
30	2-#5	14.00	19.36
31	8-#6	14.00	23.24
32	1-#6	14.00	23.24
33	2-#5	14.00	19.36

34	1-#5	14.00	19.36
35	4-#5	14.00	19.36
36	3-#5	14.00	19.36
37	2-#5	14.00	19.36

Intermediate results for axial-bending

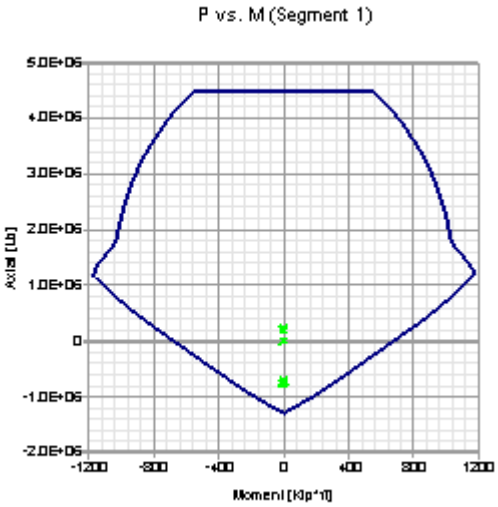
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	2.19	13.06
2	DC1 (Top)	1.82	13.25
3	DC1 (Top)	1.29	13.25
4	DC1 (Top)	1.27	13.25
5	DC1 (Top)	1.21	13.25
6	DC1 (Top)	2.17	12.94
7	DC1 (Top)	1.71	13.13
8	DC1 (Top)	1.13	13.13
9	DC1 (Top)	1.73	13.13
10	DC1 (Top)	1.34	13.13
11	DC1 (Top)	1.18	13.13
12	DC1 (Top)	1.31	13.13
13	DC1 (Top)	1.58	8.06
14	DC1 (Top)	2.34	8.13
15	DC1 (Top)	1.46	8.13
16	DC1 (Top)	2.36	8.13
17	DC1 (Top)	1.56	8.13
18	DC1 (Top)	1.01	8.31
19	DC1 (Top)	1.58	8.06
20	DC1 (Top)	2.35	8.13
21	DC1 (Top)	1.38	8.13
22	DC1 (Top)	2.28	8.13
23	DC1 (Top)	1.52	8.13
24	DC1 (Top)	0.79	8.31
25	DC1 (Top)	0.90	8.31
26	DC1 (Top)	1.05	6.25
27	DC1 (Top)	1.55	6.25
28	DC1 (Top)	0.88	6.31
29	DC1 (Top)	0.93	6.31
30	DC1 (Top)	0.77	6.31
31	DC1 (Top)	1.03	6.25
32	DC1 (Top)	1.52	6.25
33	DC1 (Top)	0.76	6.31
34	DC1 (Top)	1.29	6.31
35	DC1 (Top)	0.84	6.31
36	DC1 (Top)	0.91	6.31
37	DC1 (Top)	0.76	6.31

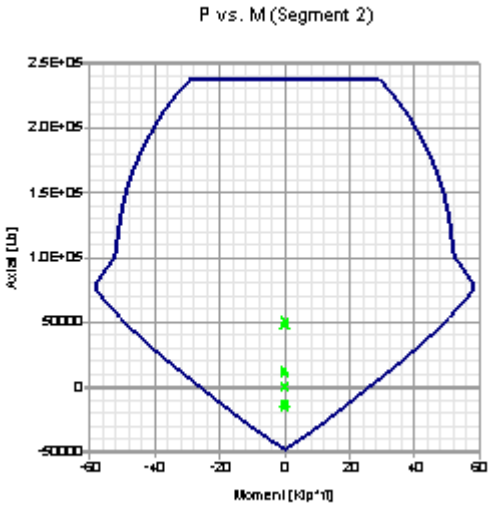
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	196336.50	0.00	771.10	0.00
2	DC1 (Top)	13511.45	0.00	32.58	0.00
3	DC1 (Top)	96222.32	0.00	162.32	0.00
4	DC1 (Top)	47900.73	0.00	107.81	0.00
5	DC1 (Top)	49754.27	0.00	82.74	0.00
6	DC1 (Top)	173847.30	0.00	754.12	0.00
7	DC1 (Top)	7411.47	0.00	29.35	0.00
8	DC1 (Top)	33625.52	0.00	74.27	0.00

9	DC1 (Top)	8628.80	0.00	29.95	0.00
10	DC1 (Top)	97167.51	0.00	163.05	0.00
11	DC1 (Top)	23454.13	0.00	95.06	0.00
12	DC1 (Top)	58513.21	0.00	87.55	0.00
13	DC1 (Top)	109443.00	0.00	290.00	0.00
14	DC1 (Top)	5701.29	0.00	27.72	0.00
15	DC1 (Top)	29099.90	0.00	68.65	0.00
16	DC1 (Top)	6122.53	0.00	27.83	0.00
17	DC1 (Top)	65914.50	0.00	137.06	0.00
18	DC1 (Top)	43014.94	0.00	41.64	0.00
19	DC1 (Top)	107326.60	0.00	289.39	0.00
20	DC1 (Top)	5980.71	0.00	27.80	0.00
21	DC1 (Top)	11632.51	0.00	63.55	0.00
22	DC1 (Top)	4394.59	0.00	27.35	0.00
23	DC1 (Top)	53636.53	0.00	133.52	0.00
24	DC1 (Top)	31272.20	0.00	38.44	0.00
25	DC1 (Top)	27831.40	0.00	36.58	0.00
26	DC1 (Top)	56444.18	0.00	128.32	0.00
27	DC1 (Top)	3538.71	0.00	12.68	0.00
28	DC1 (Top)	29902.82	0.00	50.09	0.00
29	DC1 (Top)	16872.29	0.00	35.42	0.00
30	DC1 (Top)	12762.66	0.00	24.61	0.00
31	DC1 (Top)	45639.82	0.00	125.63	0.00
32	DC1 (Top)	2374.27	0.00	12.44	0.00
33	DC1 (Top)	11859.14	0.00	24.35	0.00
34	DC1 (Top)	2379.63	0.00	9.58	0.00
35	DC1 (Top)	21727.12	0.00	47.75	0.00
36	DC1 (Top)	14372.06	0.00	34.80	0.00
37	DC1 (Top)	11926.93	0.00	24.37	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	216777.00	4471272.00	0.05
2	D2 (Top)	50682.07	236346.20	0.21
3	D2 (Top)	401981.50	2138785.00	0.19
4	D2 (Bottom)	371917.10	1425079.00	0.26
5	D2 (Bottom)	501030.40	1188732.00	0.42
6	D1 (Bottom)	196334.90	4471272.00	0.04
7	DC1 (Bottom)	7543.46	236346.20	0.03
8	D2 (Top)	51427.67	1188732.00	0.04
9	D2 (Bottom)	50681.58	236346.20	0.21
10	D2 (Top)	478757.80	2138785.00	0.22
11	D2 (Bottom)	275152.30	1425079.00	0.19
12	D2 (Top)	374595.00	1188732.00	0.32
13	D1 (Bottom)	128535.00	2986152.00	0.04
14	D2 (Top)	6169.06	154929.80	0.04
15	D2 (Top)	45936.00	787219.70	0.06
16	D2 (Bottom)	25335.92	154929.80	0.16
17	D2 (Bottom)	474710.40	1415319.00	0.34
18	D2 (Bottom)	373166.80	792311.50	0.47
19	DC1 (Bottom)	109443.00	2986152.00	0.04
20	D2 (Top)	42576.12	154929.80	0.27
21	D2 (Top)	52188.64	787219.70	0.07
22	D2 (Bottom)	11918.36	154929.80	0.08
23	D2 (Bottom)	151741.30	1415319.00	0.11
24	D2 (Top)	128289.80	951431.50	0.13
25	D2 (Bottom)	338107.70	792311.50	0.43
26	DC1 (Bottom)	69366.26	2399954.00	0.03
27	D2 (Bottom)	55943.86	124962.20	0.45
28	D2 (Top)	142396.30	1139087.00	0.13
29	D2 (Bottom)	117760.70	758843.30	0.16
30	D2 (Bottom)	187223.80	633191.50	0.30
31	D2 (Top)	127969.20	2399954.00	0.05
32	DC1 (Bottom)	3515.11	124962.20	0.03
33	DC1 (Top)	11859.14	633191.50	0.02
34	D2 (Bottom)	42286.45	125651.80	0.34
35	D2 (Bottom)	142295.00	1139087.00	0.12
36	D2 (Bottom)	25685.95	758843.30	0.03
37	D2 (Bottom)	13870.79	633191.50	0.02

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	771236.50	1296000.00	0.60
2	D3 (Bottom)	14811.33	47520.00	0.31
3	DC1 (Top)	0.00	190080.00	0.00
4	DC1 (Top)	0.00	142560.00	0.00
5	DC1 (Top)	0.00	95040.00	0.00
6	D3 (Bottom)	754700.80	1296000.00	0.58
7	D3 (Bottom)	21837.08	47520.00	0.46
8	DC1 (Bottom)	1753.25	95040.00	0.02
9	DC1 (Top)	0.00	47520.00	0.00
10	DC1 (Top)	0.00	190080.00	0.00
11	DC1 (Top)	0.00	142560.00	0.00
12	DC1 (Top)	0.00	95040.00	0.00
13	D3 (Bottom)	720928.20	756000.00	0.95

14	D3 (Bottom)	7456.77	85320.00	0.09
15	DC1 (Top)	0.00	170640.00	0.00
16	DC1 (Top)	0.00	85320.00	0.00
17	DC1 (Top)	0.00	341280.00	0.00
18	DC1 (Top)	0.00	66960.00	0.00
19	D3 (Bottom)	380051.20	756000.00	0.50
20	DC1 (Top)	0.00	85320.00	0.00
21	DC1 (Top)	0.00	170640.00	0.00
22	D3 (Top)	33526.12	85320.00	0.39
23	D3 (Top)	38289.84	341280.00	0.11
24	DC1 (Top)	0.00	66960.00	0.00
25	DC1 (Top)	0.00	66960.00	0.00
26	D3 (Bottom)	214493.40	380160.00	0.56
27	D3 (Top)	31360.71	47520.00	0.66
28	D3 (Bottom)	57351.15	133920.00	0.43
29	DC1 (Top)	0.00	100440.00	0.00
30	DC1 (Top)	0.00	66960.00	0.00
31	D3 (Bottom)	68969.52	380160.00	0.18
32	D3 (Bottom)	31136.23	47520.00	0.66
33	D2 (Bottom)	35111.83	66960.00	0.52
34	DC1 (Top)	0.00	33480.00	0.00
35	DC1 (Top)	0.00	133920.00	0.00
36	DC1 (Top)	0.00	100440.00	0.00
37	DC1 (Top)	0.00	66960.00	0.00

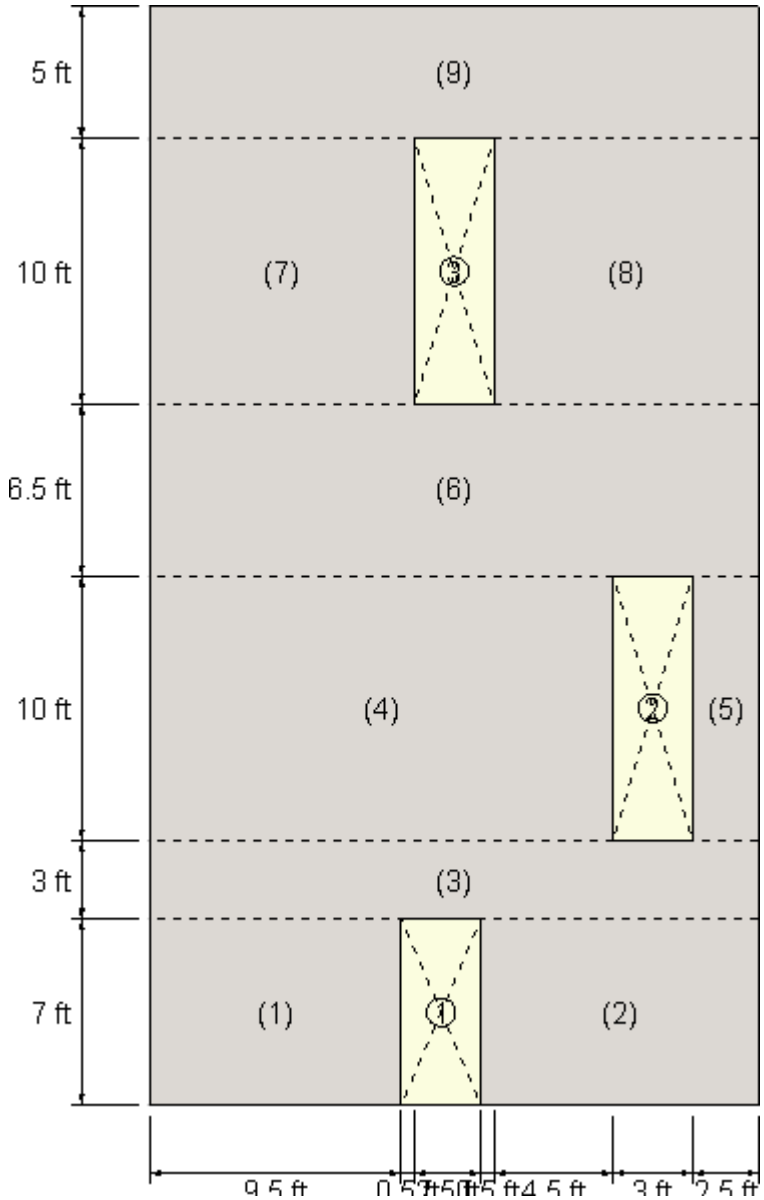
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	173000.820	0.00
2	DC1 (Top)	0.000	9237.065	0.00
3	DC1 (Top)	0.000	83133.588	0.00
4	DC1 (Top)	0.000	55422.392	0.00
5	DC1 (Top)	0.000	46185.326	0.00
6	DC1 (Top)	0.000	171345.120	0.00
7	DC1 (Top)	0.000	9149.923	0.00
8	DC1 (Top)	0.000	45749.616	0.00
9	DC1 (Top)	0.000	9149.923	0.00
10	DC1 (Top)	0.000	82349.308	0.00
11	DC1 (Top)	0.000	54899.539	0.00
12	DC1 (Top)	0.000	45749.616	0.00
13	DC1 (Top)	0.000	106772.810	0.00
14	DC1 (Top)	0.000	5664.238	0.00
15	DC1 (Top)	0.000	28321.191	0.00
16	DC1 (Top)	0.000	5664.238	0.00
17	DC1 (Top)	0.000	50978.143	0.00
18	DC1 (Top)	0.000	28974.757	0.00
19	DC1 (Top)	0.000	106772.810	0.00
20	DC1 (Top)	0.000	5664.238	0.00
21	DC1 (Top)	0.000	28321.191	0.00
22	DC1 (Top)	0.000	5664.238	0.00
23	DC1 (Top)	0.000	50978.143	0.00
24	DC1 (Top)	0.000	34769.708	0.00
25	DC1 (Top)	0.000	28974.757	0.00
26	DC1 (Top)	0.000	82785.019	0.00
27	DC1 (Top)	0.000	4357.106	0.00
28	DC1 (Top)	0.000	39606.096	0.00
29	DC1 (Top)	0.000	26404.064	0.00

30	DC1 (Top)	0.000	22003.387	0.00
31	DC1 (Top)	0.000	82785.019	0.00
32	DC1 (Top)	0.000	4357.106	0.00
33	DC1 (Top)	0.000	22003.387	0.00
34	DC1 (Top)	0.000	4400.677	0.00
35	DC1 (Top)	0.000	39606.096	0.00
36	DC1 (Top)	0.000	26404.064	0.00
37	DC1 (Top)	0.000	22003.387	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	10.50	7.00
3	0.00	7.00	23.00	3.00
4	0.00	10.00	17.50	10.00
5	20.50	10.00	2.50	10.00
6	0.00	20.00	23.00	6.50
7	0.00	26.50	10.00	10.00
8	13.00	26.50	10.00	10.00
9	0.00	36.50	23.00	5.00

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	**Bars**	**Spacing** [in]	**Ld** [in]	**Bars**	**Spacing** [in]	**Ld** [in]
1	12-#9	10.00	43.69	6-#5	16.00	25.17
2	1-#6	14.00	23.24	6-#5	16.00	25.17
	4-#6	14.00	23.24	6-#5	16.00	25.17
	3-#6	14.00	23.24	6-#5	16.00	25.17
	2-#6	14.00	23.24	6-#5	16.00	25.17
3	12-#9	10.00	43.69	2-#6	18.00	30.21
	1-#6	14.00	43.69	2-#6	18.00	30.21
	2-#6	14.00	43.69	2-#6	18.00	30.21
	1-#6	14.00	43.69	2-#6	18.00	30.21
	4-#6	14.00	43.69	2-#6	18.00	30.21
	3-#6	14.00	43.69	2-#6	18.00	30.21
	2-#6	14.00	43.69	2-#6	18.00	30.21
4	7-#9	18.00	43.69	7-#5	18.00	25.17
	1-#8	16.00	43.69	7-#5	18.00	25.17
	2-#8	16.00	43.69	7-#5	18.00	25.17
	1-#8	16.00	43.69	7-#5	18.00	25.17
	4-#8	16.00	43.69	7-#5	18.00	25.17
5	2-#5	18.00	19.36	7-#5	18.00	25.17
6	7-#9	18.00	43.69	5-#5	18.00	25.17
	1-#8	16.00	43.69	5-#5	18.00	25.17
	2-#8	16.00	43.69	5-#5	18.00	25.17
	1-#8	16.00	43.69	5-#5	18.00	25.17
	4-#8	16.00	43.69	5-#5	18.00	25.17
	2-#5	18.00	43.69	5-#5	18.00	25.17
	2-#5	18.00	43.69	5-#5	18.00	25.17
7	8-#6	14.00	23.24	7-#5	18.00	25.17
	1-#6	14.00	23.24	7-#5	18.00	25.17
8	4-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
	2-#5	14.00	19.36	7-#5	18.00	25.17
9	8-#6	14.00	23.24	4-#5	18.00	25.17
	1-#6	14.00	23.24	4-#5	18.00	25.17
	2-#5	14.00	23.24	4-#5	18.00	25.17
	1-#5	14.00	23.24	4-#5	18.00	25.17
	4-#5	14.00	23.24	4-#5	18.00	25.17
	3-#5	14.00	23.24	4-#5	18.00	25.17
	2-#5	14.00	23.24	4-#5	18.00	25.17

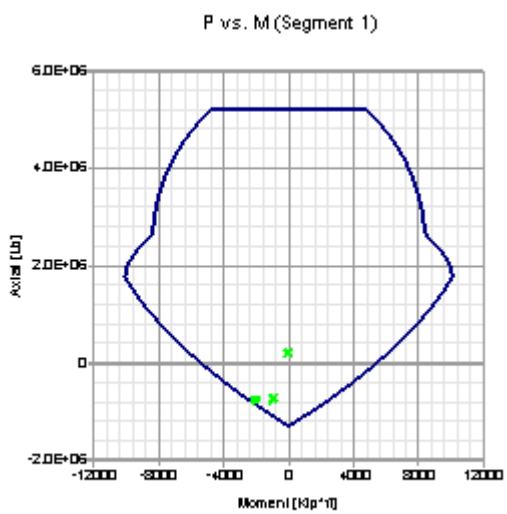
Intermediate results for axial-bending

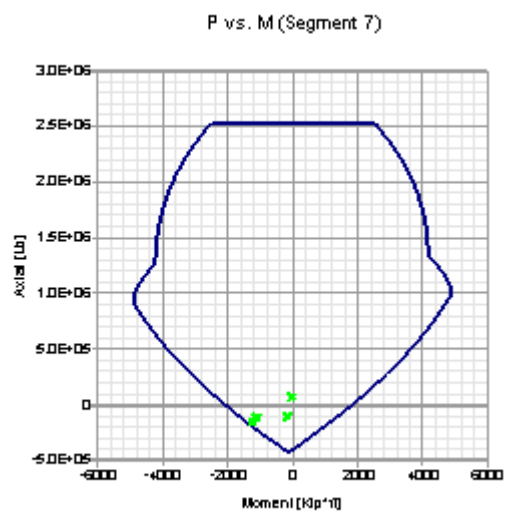
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	6.22	91.20
2	D3 (Bottom)	26.86	100.80
3	D3 (Bottom)	32.83	220.80
4	D3 (Bottom)	24.95	168.00
5	D2 (Bottom)	14.17	24.00
6	D3 (Bottom)	36.06	220.80
7	D3 (Bottom)	8.08	96.00
8	D3 (Bottom)	17.48	96.00
9	D3 (Bottom)	28.13	220.80

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	-771276.30	-2130.29	2403.00	0.89
2	D3 (Bottom)	1059704.00	-2188.94	6568.01	0.33
3	D3 (Bottom)	286510.00	-13162.87	16566.86	0.79
4	D3 (Bottom)	-209026.70	-7147.00	9757.08	0.73
5	D2 (Bottom)	373173.90	-44.07	354.78	0.12
6	D3 (Bottom)	177526.40	-7641.68	15236.73	0.50
7	D3 (Bottom)	-159506.10	-1241.97	1398.19	0.89
8	D3 (Bottom)	239555.20	-1167.44	2374.35	0.49
9	D3 (Bottom)	74015.34	-1604.91	8828.22	0.18

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	216786.30	5220072.00	0.04
2	D2 (Max)	1117333.00	4988942.00	0.22
3	D1 (Max)	415625.30	1.088529E07	0.04
4	D1 (Bottom)	245479.60	6329719.00	0.04
5	D2 (Max)	375149.00	792311.50	0.47
6	D1 (Bottom)	266300.50	8150838.00	0.03
7	DC1 (Bottom)	74787.95	2524916.00	0.03
8	D2 (Bottom)	255451.20	2531122.00	0.10
9	D2 (Top)	185997.80	5814881.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Max)	781391.10	1296000.00	0.60
2	DC1 (Top)	0.00	475200.00	0.00
3	DC1 (Top)	0.00	1913760.00	0.00
4	D3 (Bottom)	209026.70	1438560.00	0.15
5	DC1 (Top)	0.00	66960.00	0.00
6	DC1 (Top)	0.00	1572480.00	0.00
7	D3 (Bottom)	159506.10	427680.00	0.37
8	DC1 (Top)	0.00	301320.00	0.00
9	DC1 (Top)	0.00	829440.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	197782.340	216867.990	0.91
2	D2 (Top)	284603.910	627333.760	0.45
3	D3 (Bottom)	487438.430	1009002.600	0.48
4	D3 (Bottom)	400797.630	505706.360	0.79
5	D3 (Bottom)	9417.372	89752.509	0.10
6	D3 (Max)	418831.650	788173.350	0.53
7	D3 (Bottom)	132984.030	259985.580	0.51
8	D3 (Top)	200482.740	323318.950	0.62
9	D3 (Bottom)	362154.100	691983.570	0.52

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 1:11 AM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 6.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : N. G.

Design code	:	ACI 318-05

Geometry:

Total height	:	41.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 3

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00
3	15.00	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	4.00	0.00	3.00	7.00
Lower left	17.00	0.00	3.00	7.00
Lower left	2.00	10.00	3.00	10.00
Lower left	8.00	26.50	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ

D3	Yes	0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
3	EQ	Horizontal	327998.00	0.00	0.00
2	EQ	Horizontal	84930.00	0.00	0.00
1	EQ	Horizontal	57990.00	0.00	0.00

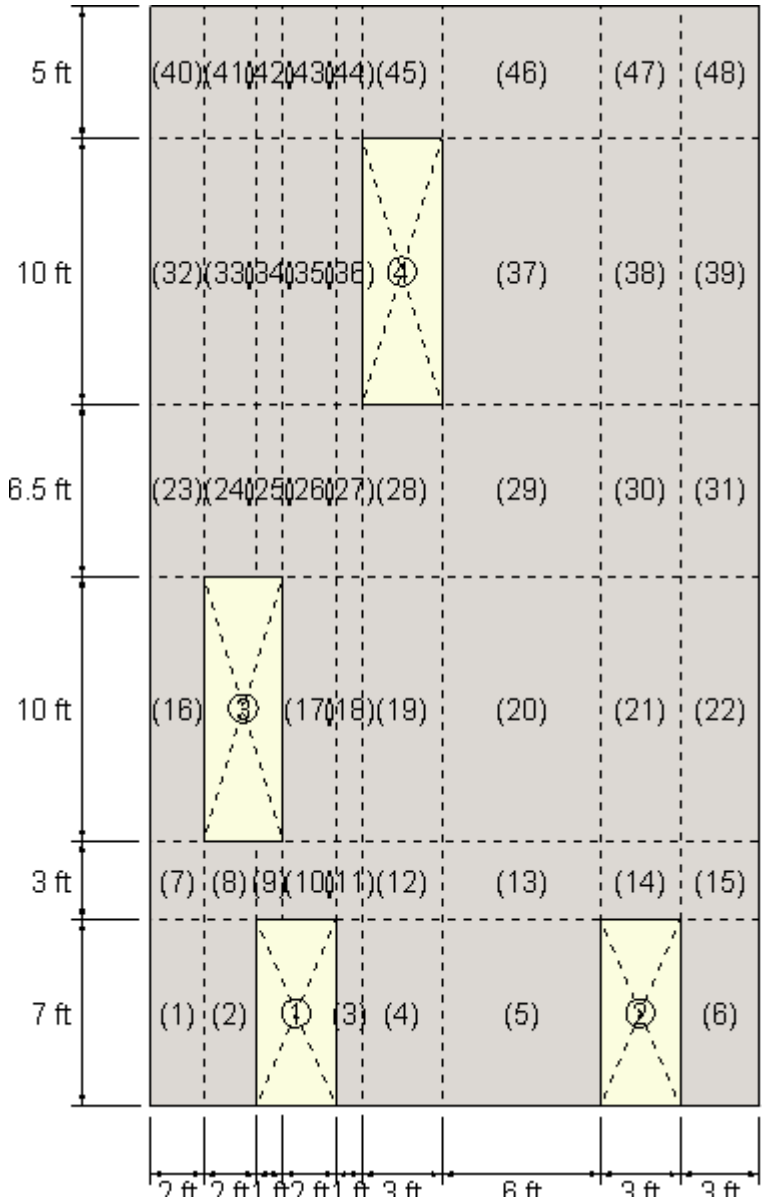
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
3	DL	Vertical	3360.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : N. G.
- Insufficient axial tension strength (Segment 1)



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.00	7.00
2	2.00	0.00	2.00	7.00
3	7.00	0.00	1.00	7.00
4	8.00	0.00	3.00	7.00
5	11.00	0.00	6.00	7.00
6	20.00	0.00	3.00	7.00
7	0.00	7.00	2.00	3.00
8	2.00	7.00	2.00	3.00
9	4.00	7.00	1.00	3.00
10	5.00	7.00	2.00	3.00
11	7.00	7.00	1.00	3.00
12	8.00	7.00	3.00	3.00
13	11.00	7.00	6.00	3.00
14	17.00	7.00	3.00	3.00
15	20.00	7.00	3.00	3.00
16	0.00	10.00	2.00	10.00

17	5.00	10.00	2.00	10.00
18	7.00	10.00	1.00	10.00
19	8.00	10.00	3.00	10.00
20	11.00	10.00	6.00	10.00
21	17.00	10.00	3.00	10.00
22	20.00	10.00	3.00	10.00
23	0.00	20.00	2.00	6.50
24	2.00	20.00	2.00	6.50
25	4.00	20.00	1.00	6.50
26	5.00	20.00	2.00	6.50
27	7.00	20.00	1.00	6.50
28	8.00	20.00	3.00	6.50
29	11.00	20.00	6.00	6.50
30	17.00	20.00	3.00	6.50
31	20.00	20.00	3.00	6.50
32	0.00	26.50	2.00	10.00
33	2.00	26.50	2.00	10.00
34	4.00	26.50	1.00	10.00
35	5.00	26.50	2.00	10.00
36	7.00	26.50	1.00	10.00
37	11.00	26.50	6.00	10.00
38	17.00	26.50	3.00	10.00
39	20.00	26.50	3.00	10.00
40	0.00	36.50	2.00	5.00
41	2.00	36.50	2.00	5.00
42	4.00	36.50	1.00	5.00
43	5.00	36.50	2.00	5.00
44	7.00	36.50	1.00	5.00
45	8.00	36.50	3.00	5.00
46	11.00	36.50	6.00	5.00
47	17.00	36.50	3.00	5.00
48	20.00	36.50	3.00	5.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	3-#10	10.00	49.19
2	3-#10	10.00	49.19
3	1-#10	10.00	49.19
4	2-#9	18.00	43.69
5	7-#6	10.00	23.24
6	3-#6	12.00	23.24
7	3-#10	10.00	49.19
8	3-#10	10.00	49.19
9	1-#9	18.00	43.69
10	3-#8	10.00	38.73
11	1-#10	10.00	49.19
12	2-#9	18.00	43.69
13	7-#6	10.00	23.24
14	3-#6	12.00	23.24
15	3-#6	12.00	23.24
16	3-#9	10.00	43.69
17	3-#8	10.00	38.73
18	1-#6	18.00	23.24
19	3-#5	14.00	19.36
20	5-#5	14.00	19.36
21	3-#5	14.00	19.36
22	3-#5	14.00	19.36

23	3-#9	10.00	43.69
24	2-#6	18.00	23.24
25	1-#5	14.00	19.36
26	3-#8	10.00	38.73
27	1-#6	18.00	23.24
28	3-#5	14.00	19.36
29	5-#5	14.00	19.36
30	3-#5	14.00	19.36
31	3-#5	14.00	19.36
32	2-#8	18.00	38.73
33	2-#7	18.00	33.89
34	1-#7	18.00	33.89
35	2-#7	18.00	33.89
36	1-#7	18.00	33.89
37	5-#5	14.00	19.36
38	3-#5	14.00	19.36
39	3-#5	14.00	19.36
40	2-#8	18.00	38.73
41	2-#7	18.00	33.89
42	1-#7	18.00	33.89
43	2-#7	18.00	33.89
44	1-#7	18.00	33.89
45	3-#5	14.00	19.36
46	5-#5	14.00	19.36
47	3-#5	14.00	19.36
48	3-#5	14.00	19.36

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	3.15	12.86
2	DC1 (Top)	3.18	12.86
3	DC1 (Top)	2.38	12.99
4	DC1 (Top)	1.67	13.06
5	DC1 (Top)	1.46	13.25
6	DC1 (Top)	1.38	13.25
7	DC1 (Top)	3.28	12.86
8	DC1 (Top)	2.63	12.86
9	DC1 (Top)	1.82	12.94
10	DC1 (Top)	2.10	13.00
11	DC1 (Top)	2.19	12.86
12	DC1 (Top)	1.69	12.94
13	DC1 (Top)	1.46	13.13
14	DC1 (Top)	1.27	13.13
15	DC1 (Top)	1.35	13.13
16	DC1 (Top)	2.43	8.06
17	DC1 (Top)	1.97	8.13
18	DC1 (Top)	1.25	8.25
19	DC1 (Top)	1.03	8.31
20	DC1 (Top)	0.93	8.31
21	DC1 (Top)	1.02	8.31
22	DC1 (Top)	1.01	8.31
23	DC1 (Top)	2.22	8.06
24	DC1 (Top)	1.21	8.25
25	DC1 (Top)	1.01	8.31
26	DC1 (Top)	1.91	8.13
27	DC1 (Top)	1.24	8.25
28	DC1 (Top)	0.91	8.31
29	DC1 (Top)	0.93	8.31
30	DC1 (Top)	1.00	8.31
31	DC1 (Top)	0.99	8.31

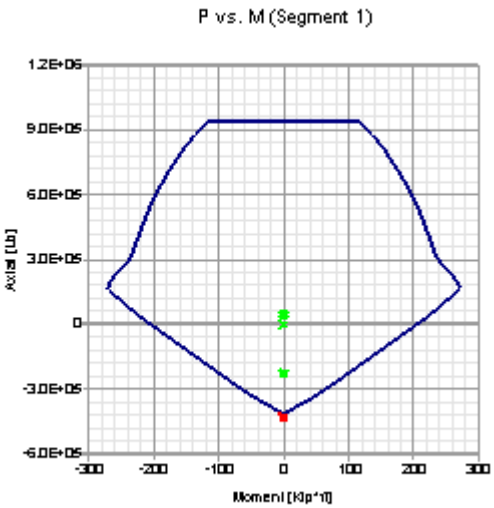
32	DC1 (Top)	1.52	6.13
33	DC1 (Top)	1.33	6.19
34	DC1 (Top)	1.34	6.19
35	DC1 (Top)	1.34	6.19
36	DC1 (Top)	1.36	6.19
37	DC1 (Top)	0.84	6.31
38	DC1 (Top)	0.93	6.31
39	DC1 (Top)	0.92	6.31
40	DC1 (Top)	1.51	6.13
41	DC1 (Top)	1.32	6.19
42	DC1 (Top)	1.32	6.19
43	DC1 (Top)	1.32	6.19
44	DC1 (Top)	1.32	6.19
45	DC1 (Top)	0.91	6.31
46	DC1 (Top)	0.79	6.31
47	DC1 (Top)	0.91	6.31
48	DC1 (Top)	0.91	6.31

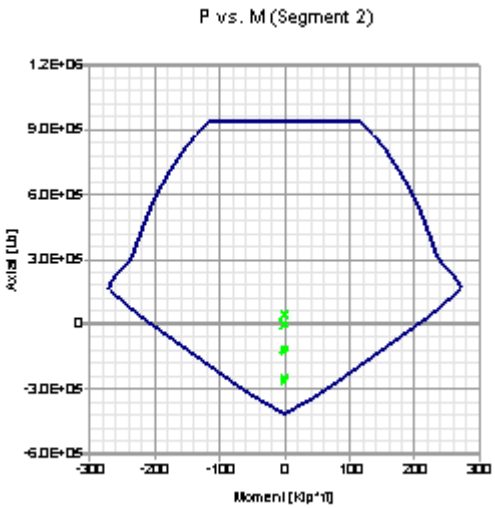
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	40352.30	0.00	222.66	0.00
2	DC1 (Top)	41953.42	0.00	223.36	0.00
3	DC1 (Top)	28734.89	0.00	85.01	0.00
4	DC1 (Top)	69682.96	0.00	154.83	0.00
5	DC1 (Top)	140633.10	0.00	262.71	0.00
6	DC1 (Top)	70783.40	0.00	119.72	0.00
7	DC1 (Top)	47834.69	0.00	225.82	0.00
8	DC1 (Top)	11924.59	0.00	209.99	0.00
9	DC1 (Top)	10349.53	0.00	62.26	0.00
10	DC1 (Top)	37198.55	0.00	151.74	0.00
11	DC1 (Top)	18656.28	0.00	79.53	0.00
12	DC1 (Top)	63590.52	0.00	151.46	0.00
13	DC1 (Top)	120110.60	0.00	251.86	0.00
14	DC1 (Top)	40085.11	0.00	103.87	0.00
15	DC1 (Top)	54736.92	0.00	111.51	0.00
16	DC1 (Top)	34872.82	0.00	108.68	0.00
17	DC1 (Top)	31447.36	0.00	90.22	0.00
18	DC1 (Top)	13692.58	0.00	22.07	0.00
19	DC1 (Top)	37270.77	0.00	51.77	0.00
20	DC1 (Top)	71844.96	0.00	91.68	0.00
21	DC1 (Top)	34596.49	0.00	50.88	0.00
22	DC1 (Top)	32870.36	0.00	50.31	0.00
23	DC1 (Top)	21811.83	0.00	105.08	0.00
24	DC1 (Top)	21397.50	0.00	42.32	0.00
25	DC1 (Top)	10939.64	0.00	16.77	0.00
26	DC1 (Top)	23600.88	0.00	87.98	0.00
27	DC1 (Top)	12943.31	0.00	21.84	0.00
28	DC1 (Top)	16214.36	0.00	44.82	0.00
29	DC1 (Top)	71995.56	0.00	91.73	0.00
30	DC1 (Top)	32590.30	0.00	50.22	0.00
31	DC1 (Top)	30636.79	0.00	49.58	0.00
32	DC1 (Top)	10306.96	0.00	45.67	0.00
33	DC1 (Top)	11438.90	0.00	37.75	0.00
34	DC1 (Top)	6222.39	0.00	18.98	0.00
35	DC1 (Top)	13489.18	0.00	38.19	0.00
36	DC1 (Top)	8226.65	0.00	19.41	0.00
37	DC1 (Top)	41230.54	0.00	63.99	0.00

38	DC1 (Top)	16352.62	0.00	35.29	0.00
39	DC1 (Top)	15209.80	0.00	35.01	0.00
40	DC1 (Top)	9543.35	0.00	45.51	0.00
41	DC1 (Top)	9587.47	0.00	37.36	0.00
42	DC1 (Top)	4820.90	0.00	18.69	0.00
43	DC1 (Top)	9616.38	0.00	37.37	0.00
44	DC1 (Top)	4821.57	0.00	18.69	0.00
45	DC1 (Top)	13981.52	0.00	34.71	0.00
46	DC1 (Top)	29181.69	0.00	60.55	0.00
47	DC1 (Top)	14357.55	0.00	34.80	0.00
48	DC1 (Top)	14316.38	0.00	34.79	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	51723.20	934511.80	0.06
2	D1 (Bottom)	43576.42	934511.80	0.05
3	D1 (Top)	28734.89	470623.90	0.06
4	DC1 (Bottom)	77566.62	1421472.00	0.05
5	D2 (Bottom)	561685.60	2847824.00	0.20
6	D2 (Bottom)	716437.50	1425079.00	0.50
7	DC1 (Top)	47834.69	934511.80	0.05
8	D1 (Bottom)	41953.15	934511.80	0.04
9	DC1 (Top)	10349.53	472056.00	0.02
10	D1 (Top)	37198.55	942149.50	0.04
11	DC1 (Bottom)	28732.38	470623.90	0.06
12	D1 (Bottom)	69682.95	1421472.00	0.05
13	D2 (Bottom)	291149.90	2847824.00	0.10
14	D2 (Top)	223971.40	1425079.00	0.16
15	D2 (Bottom)	645884.30	1425079.00	0.45
16	DC1 (Bottom)	41473.77	620568.00	0.07
17	DC1 (Top)	31447.36	623909.50	0.05
18	DC1 (Bottom)	14558.05	315906.20	0.05
19	D1 (Bottom)	48240.86	949787.30	0.05

20	D2 (Bottom)	174361.80	1901219.00	0.09
21	D2 (Bottom)	224274.50	949787.30	0.24
22	D2 (Bottom)	470551.40	949787.30	0.50
23	DC1 (Bottom)	34872.48	620568.00	0.06
24	D1 (Top)	21397.50	631812.50	0.03
25	D1 (Top)	10939.64	316595.80	0.03
26	DC1 (Bottom)	31444.05	623909.50	0.05
27	D2 (Top)	59690.48	315906.20	0.19
28	D1 (Bottom)	37270.76	949787.30	0.04
29	D2 (Bottom)	96511.26	1901219.00	0.05
30	D2 (Bottom)	183490.50	949787.30	0.19
31	D2 (Bottom)	280826.40	949787.30	0.30
32	DC1 (Bottom)	13432.60	500803.70	0.03
33	D1 (Bottom)	13700.37	502819.20	0.03
34	D2 (Top)	14879.76	251409.60	0.06
35	D2 (Bottom)	23025.23	502819.20	0.05
36	D2 (Bottom)	68137.88	251409.60	0.27
37	D2 (Top)	164071.30	1519331.00	0.11
38	D2 (Bottom)	120965.70	758843.30	0.16
39	D2 (Bottom)	185105.70	758843.30	0.24
40	D2 (Top)	75017.74	500803.70	0.15
41	D2 (Top)	23499.19	502819.20	0.05
42	D2 (Bottom)	14879.84	251409.60	0.06
43	D1 (Bottom)	13489.18	502819.20	0.03
44	D1 (Bottom)	8226.52	251409.60	0.03
45	DC1 (Top)	13981.52	758843.30	0.02
46	D2 (Bottom)	164068.60	1519331.00	0.11
47	D2 (Bottom)	25856.02	758843.30	0.03
48	D2 (Bottom)	25357.61	758843.30	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	433828.10	411480.00	1.05
2	D3 (Top)	260339.70	411480.00	0.63
3	D3 (Bottom)	178121.50	137160.00	1.30
4	D3 (Bottom)	209423.00	216000.00	0.97
5	DC1 (Top)	0.00	332640.00	0.00
6	DC1 (Top)	0.00	142560.00	0.00
7	D3 (Top)	280216.60	411480.00	0.68
8	D3 (Bottom)	260337.50	411480.00	0.63
9	D3 (Bottom)	93876.43	108000.00	0.87
10	D3 (Top)	216097.50	255960.00	0.84
11	D3 (Bottom)	52517.43	137160.00	0.38
12	D3 (Bottom)	110013.00	216000.00	0.51
13	DC1 (Top)	0.00	332640.00	0.00
14	DC1 (Top)	0.00	142560.00	0.00
15	DC1 (Top)	0.00	142560.00	0.00
16	D3 (Bottom)	292642.50	324000.00	0.90
17	D3 (Bottom)	222877.60	255960.00	0.87
18	D3 (Bottom)	44861.11	47520.00	0.94
19	D3 (Bottom)	53652.71	100440.00	0.53
20	DC1 (Top)	0.00	167400.00	0.00
21	DC1 (Top)	0.00	100440.00	0.00
22	DC1 (Top)	0.00	100440.00	0.00
23	D3 (Bottom)	254822.90	324000.00	0.79
24	D3 (Top)	84800.91	95040.00	0.89

25	D3 (Top)	15008.93	33480.00	0.45
26	D3 (Bottom)	27845.22	255960.00	0.11
27	D3 (Bottom)	16372.79	47520.00	0.34
28	D3 (Bottom)	36487.71	100440.00	0.36
29	D3 (Top)	16905.70	167400.00	0.10
30	DC1 (Top)	0.00	100440.00	0.00
31	DC1 (Top)	0.00	100440.00	0.00
32	D3 (Bottom)	151030.80	170640.00	0.89
33	D3 (Bottom)	88144.50	129600.00	0.68
34	D3 (Bottom)	18374.42	64800.00	0.28
35	DC1 (Top)	0.00	129600.00	0.00
36	D3 (Top)	52867.32	64800.00	0.82
37	D3 (Bottom)	35364.83	167400.00	0.21
38	DC1 (Top)	0.00	100440.00	0.00
39	DC1 (Top)	0.00	100440.00	0.00
40	D3 (Bottom)	91195.33	170640.00	0.53
41	DC1 (Top)	0.00	129600.00	0.00
42	DC1 (Top)	0.00	64800.00	0.00
43	DC1 (Top)	0.00	129600.00	0.00
44	D3 (Bottom)	52866.23	64800.00	0.82
45	DC1 (Top)	0.00	100440.00	0.00
46	DC1 (Top)	0.00	167400.00	0.00
47	DC1 (Top)	0.00	100440.00	0.00
48	DC1 (Top)	0.00	100440.00	0.00

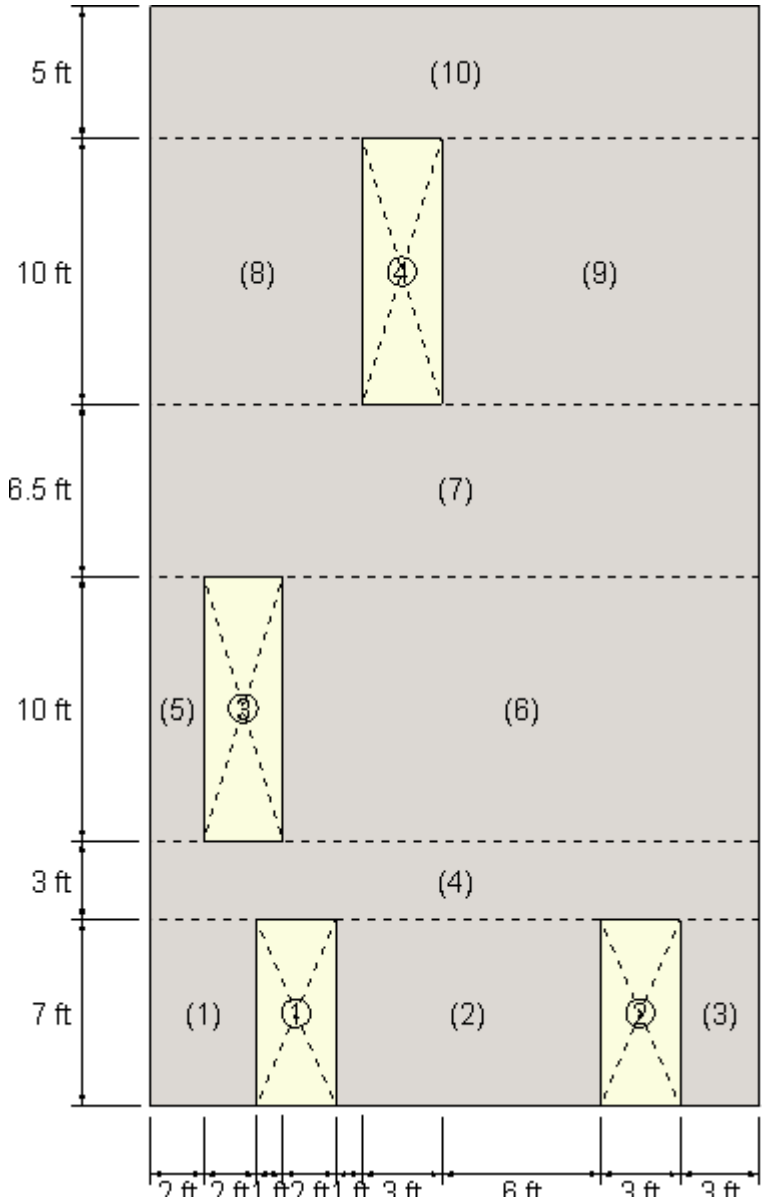
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	35874.670	0.00
2	DC1 (Top)	0.000	35874.670	0.00
3	DC1 (Top)	0.000	18111.619	0.00
4	DC1 (Top)	0.000	54631.838	0.00
5	DC1 (Top)	0.000	110844.780	0.00
6	DC1 (Top)	0.000	55422.392	0.00
7	DC1 (Top)	0.000	35874.670	0.00
8	DC1 (Top)	0.000	35874.670	0.00
9	DC1 (Top)	0.000	18036.329	0.00
10	DC1 (Top)	0.000	36251.124	0.00
11	DC1 (Top)	0.000	17937.335	0.00
12	DC1 (Top)	0.000	54108.986	0.00
13	DC1 (Top)	0.000	109799.080	0.00
14	DC1 (Top)	0.000	54899.539	0.00
15	DC1 (Top)	0.000	54899.539	0.00
16	DC1 (Top)	0.000	22478.486	0.00
17	DC1 (Top)	0.000	22656.953	0.00
18	DC1 (Top)	0.000	11502.761	0.00
19	DC1 (Top)	0.000	34769.708	0.00
20	DC1 (Top)	0.000	69539.416	0.00
21	DC1 (Top)	0.000	34769.708	0.00
22	DC1 (Top)	0.000	34769.708	0.00
23	DC1 (Top)	0.000	22478.486	0.00
24	DC1 (Top)	0.000	23005.521	0.00
25	DC1 (Top)	0.000	11589.903	0.00
26	DC1 (Top)	0.000	22656.953	0.00
27	DC1 (Top)	0.000	11502.761	0.00
28	DC1 (Top)	0.000	34769.708	0.00
29	DC1 (Top)	0.000	69539.416	0.00

30	DC1 (Top)	0.000	34769.708	0.00	
31	DC1 (Top)	0.000	34769.708	0.00	
32	DC1 (Top)	0.000	17079.857	0.00	
33	DC1 (Top)	0.000	17254.141	0.00	
34	DC1 (Top)	0.000	8627.070	0.00	
35	DC1 (Top)	0.000	17254.141	0.00	
36	DC1 (Top)	0.000	8627.070	0.00	
37	DC1 (Top)	0.000	52808.128	0.00	
38	DC1 (Top)	0.000	26404.064	0.00	
39	DC1 (Top)	0.000	26404.064	0.00	
40	DC1 (Top)	0.000	17079.857	0.00	
41	DC1 (Top)	0.000	17254.141	0.00	
42	DC1 (Top)	0.000	8627.070	0.00	
43	DC1 (Top)	0.000	17254.141	0.00	
44	DC1 (Top)	0.000	8627.070	0.00	
45	DC1 (Top)	0.000	26404.064	0.00	
46	DC1 (Top)	0.000	52808.128	0.00	
47	DC1 (Top)	0.000	26404.064	0.00	
48	DC1 (Top)	0.000	26404.064	0.00	

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	4.00	7.00
2	7.00	0.00	10.00	7.00
3	20.00	0.00	3.00	7.00
4	0.00	7.00	23.00	3.00
5	0.00	10.00	2.00	10.00
6	5.00	10.00	18.00	10.00
7	0.00	20.00	23.00	6.50
8	0.00	26.50	8.00	10.00
9	11.00	26.50	12.00	10.00
10	0.00	36.50	23.00	5.00

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	3-#10	10.00	49.19	6-#6	14.00	30.21
	3-#10	10.00	49.19	6-#6	14.00	30.21
2	1-#10	10.00	49.19	6-#5	16.00	25.17
	2-#9	18.00	49.19	6-#5	16.00	25.17
	7-#6	10.00	49.19	6-#5	16.00	25.17
3	3-#6	12.00	23.24	5-#5	18.00	25.17
4	3-#10	10.00	49.19	2-#6	18.00	30.21
	3-#10	10.00	49.19	2-#6	18.00	30.21
	1-#9	18.00	49.19	2-#6	18.00	30.21
	3-#8	10.00	49.19	2-#6	18.00	30.21
	1-#10	10.00	49.19	2-#6	18.00	30.21
	2-#9	18.00	49.19	2-#6	18.00	30.21
	7-#6	10.00	49.19	2-#6	18.00	30.21
	3-#6	12.00	49.19	2-#6	18.00	30.21
	3-#6	12.00	49.19	2-#6	18.00	30.21
5	3-#9	10.00	43.69	7-#5	18.00	25.17
6	3-#8	10.00	38.73	7-#5	18.00	25.17
	1-#6	18.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
	5-#5	14.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
	3-#5	14.00	38.73	7-#5	18.00	25.17
7	3-#9	10.00	43.69	5-#5	18.00	25.17
	2-#6	18.00	43.69	5-#5	18.00	25.17
	1-#5	14.00	43.69	5-#5	18.00	25.17
	3-#8	10.00	43.69	5-#5	18.00	25.17
	1-#6	18.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
	5-#5	14.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
	3-#5	14.00	43.69	5-#5	18.00	25.17
8	2-#8	18.00	38.73	7-#5	18.00	25.17
	2-#7	18.00	38.73	7-#5	18.00	25.17
	1-#7	18.00	38.73	7-#5	18.00	25.17
	2-#7	18.00	38.73	7-#5	18.00	25.17
	1-#7	18.00	38.73	7-#5	18.00	25.17
9	5-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
	3-#5	14.00	19.36	7-#5	18.00	25.17
10	2-#8	18.00	38.73	4-#5	18.00	25.17
	2-#7	18.00	38.73	4-#5	18.00	25.17
	1-#7	18.00	38.73	4-#5	18.00	25.17
	2-#7	18.00	38.73	4-#5	18.00	25.17
	1-#7	18.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17
	5-#5	14.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17
	3-#5	14.00	38.73	4-#5	18.00	25.17

Intermediate results for axial-bending

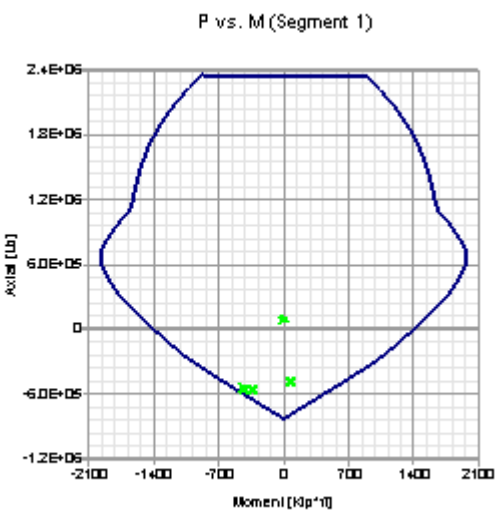
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	2.05	38.40
2	D3 (Bottom)	11.74	96.00
3	D2 (Bottom)	19.10	28.80
4	D3 (Bottom)	31.60	220.80
5	D3 (Bottom)	0.61	19.20

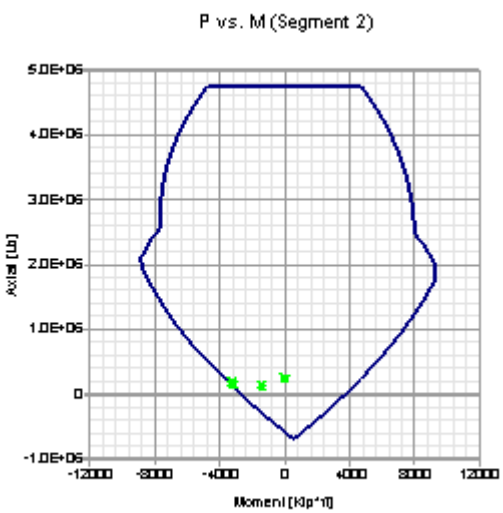
6	D3 (Max)	26.84	172.80
7	D3 (Bottom)	29.49	220.80
8	D3 (Bottom)	7.88	76.80
9	D3 (Bottom)	20.50	115.20
10	D3 (Bottom)	30.82	220.80

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	-555613.60	-431.44	508.01	0.85
2	D3 (Bottom)	140215.50	-3201.56	3250.97	0.98
3	D2 (Bottom)	716407.70	-232.70	729.57	0.32
4	D3 (Bottom)	181808.20	-13263.75	16182.40	0.82
5	D3 (Bottom)	-292589.40	-20.36	29.81	0.68
6	D3 (Max)	457995.20	-6960.61	8276.19	0.84
7	D3 (Bottom)	169543.70	-7604.02	10453.93	0.73
8	D3 (Bottom)	-171763.10	-839.59	1302.88	0.64
9	D3 (Bottom)	262165.50	-1423.27	3457.22	0.41
10	D3 (Bottom)	80427.17	-1663.74	9039.71	0.18

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	95304.88	2344512.00	0.04
2	DC1 (Bottom)	264716.50	4739920.00	0.06
3	D2 (Max)	728146.90	1425079.00	0.51
4	DC1 (Bottom)	422433.10	1.087331E07	0.04
5	DC1 (Bottom)	41478.04	807768.00	0.05
6	D2 (Bottom)	567412.50	5690396.00	0.10
7	DC1 (Bottom)	267793.90	7259373.00	0.04
8	DC1 (Bottom)	60218.63	2332493.00	0.03
9	D2 (Bottom)	281142.40	3037017.00	0.09

10	D2 (Top)	184128.70	5805122.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Max)	567332.50	822960.00	0.69
2	DC1 (Top)	0.00	685800.00	0.00
3	DC1 (Top)	0.00	142560.00	0.00
4	DC1 (Top)	0.00	2157840.00	0.00
5	D3 (Bottom)	292589.40	324000.00	0.90
6	DC1 (Top)	0.00	772200.00	0.00
7	DC1 (Top)	0.00	1224720.00	0.00
8	D3 (Bottom)	171763.10	559440.00	0.31
9	DC1 (Top)	0.00	368280.00	0.00
10	DC1 (Top)	0.00	1028160.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Bottom)	100596.220	108617.140	0.93
2	D3 (Max)	328685.950	465338.110	0.71
3	D3 (Bottom)	62412.208	217169.650	0.29
4	D3 (Max)	511999.200	1039453.600	0.49
5	D3 (Top)	7027.482	29760.000	0.24
6	D3 (Top)	414691.140	666346.850	0.62
7	D3 (Bottom)	419051.550	790973.140	0.53
8	D3 (Bottom)	91971.404	177899.640	0.52
9	D3 (Top)	240182.340	385216.190	0.62
10	D3 (Max)	338275.380	693428.130	0.49

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 10:47 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL 7.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	26.50 [ft]
Total length	:	64.75 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	7.50	13.00	4.00	8.00
Lower left	19.00	13.00	4.00	8.00
Lower left	30.25	13.00	4.00	8.00
Lower left	41.75	13.00	4.00	8.00
Lower left	53.25	13.00	4.00	8.00
Lower left	30.25	3.50	4.00	5.00
Lower left	41.75	3.50	4.00	5.00
Lower left	53.25	3.50	4.00	5.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
2	EQ	Horizontal	452174.00	0.00	0.00
1	EQ	Horizontal	97868.00	0.00	0.00

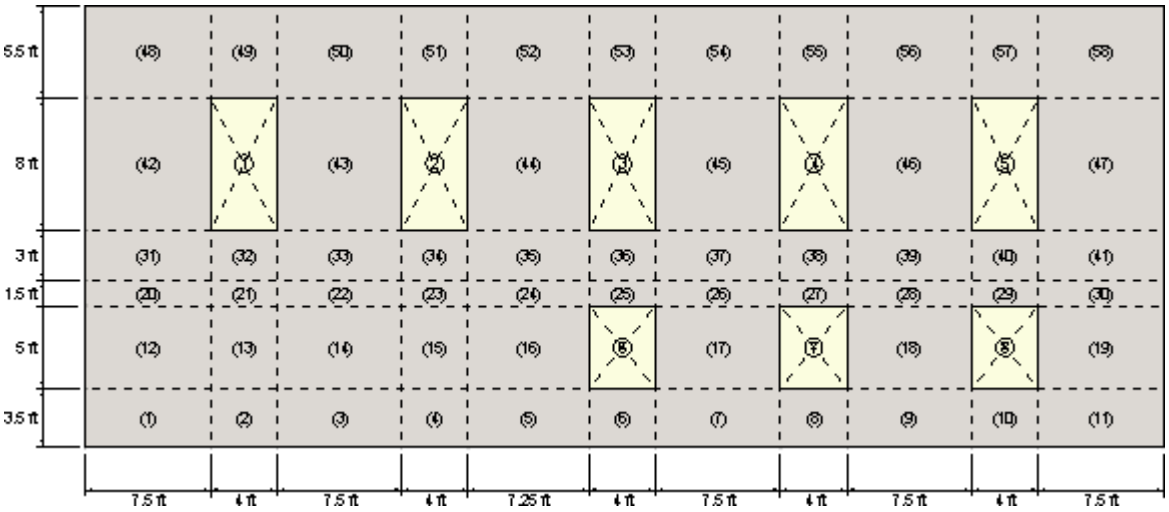
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
2	DL	Vertical	5850.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	7.50	3.50
2	7.50	0.00	4.00	3.50
3	11.50	0.00	7.50	3.50
4	19.00	0.00	4.00	3.50
5	23.00	0.00	7.25	3.50
6	30.25	0.00	4.00	3.50
7	34.25	0.00	7.50	3.50
8	41.75	0.00	4.00	3.50
9	45.75	0.00	7.50	3.50
10	53.25	0.00	4.00	3.50
11	57.25	0.00	7.50	3.50
12	0.00	3.50	7.50	5.00
13	7.50	3.50	4.00	5.00
14	11.50	3.50	7.50	5.00
15	19.00	3.50	4.00	5.00
16	23.00	3.50	7.25	5.00
17	34.25	3.50	7.50	5.00
18	45.75	3.50	7.50	5.00
19	57.25	3.50	7.50	5.00
20	0.00	8.50	7.50	1.50
21	7.50	8.50	4.00	1.50
22	11.50	8.50	7.50	1.50
23	19.00	8.50	4.00	1.50
24	23.00	8.50	7.25	1.50
25	30.25	8.50	4.00	1.50
26	34.25	8.50	7.50	1.50
27	41.75	8.50	4.00	1.50
28	45.75	8.50	7.50	1.50
29	53.25	8.50	4.00	1.50
30	57.25	8.50	7.50	1.50
31	0.00	10.00	7.50	3.00
32	7.50	10.00	4.00	3.00
33	11.50	10.00	7.50	3.00
34	19.00	10.00	4.00	3.00
35	23.00	10.00	7.25	3.00
36	30.25	10.00	4.00	3.00
37	34.25	10.00	7.50	3.00
38	41.75	10.00	4.00	3.00
39	45.75	10.00	7.50	3.00
40	53.25	10.00	4.00	3.00
41	57.25	10.00	7.50	3.00
42	0.00	13.00	7.50	8.00
43	11.50	13.00	7.50	8.00
44	23.00	13.00	7.25	8.00
45	34.25	13.00	7.50	8.00
46	45.75	13.00	7.50	8.00
47	57.25	13.00	7.50	8.00
48	0.00	21.00	7.50	5.50
49	7.50	21.00	4.00	5.50
50	11.50	21.00	7.50	5.50
51	19.00	21.00	4.00	5.50
52	23.00	21.00	7.25	5.50
53	30.25	21.00	4.00	5.50
54	34.25	21.00	7.50	5.50
55	41.75	21.00	4.00	5.50
56	45.75	21.00	7.50	5.50
57	53.25	21.00	4.00	5.50
58	57.25	21.00	7.50	5.50

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#5	14.00	19.36
2	3-#5	18.00	19.36
3	5-#5	18.00	19.36
4	3-#5	18.00	19.36
5	5-#5	18.00	19.36
6	3-#5	18.00	19.36
7	5-#5	18.00	19.36
8	3-#5	18.00	19.36
9	5-#5	18.00	19.36
10	3-#5	18.00	19.36
11	5-#5	18.00	19.36
12	7-#5	14.00	19.36
13	3-#5	18.00	19.36
14	5-#5	18.00	19.36
15	3-#5	18.00	19.36
16	5-#5	18.00	19.36
17	5-#5	18.00	19.36
18	5-#5	18.00	19.36
19	5-#5	18.00	19.36
20	7-#5	14.00	19.36
21	3-#5	18.00	19.36
22	5-#5	18.00	19.36
23	3-#5	18.00	19.36
24	5-#5	18.00	19.36
25	3-#5	18.00	19.36
26	5-#5	18.00	19.36
27	3-#5	18.00	19.36
28	5-#5	18.00	19.36
29	3-#5	18.00	19.36
30	5-#5	18.00	19.36
31	8-#5	12.00	19.36
32	3-#5	18.00	19.36
33	6-#5	16.00	19.36
34	3-#5	18.00	19.36
35	5-#5	18.00	19.36
36	3-#5	18.00	19.36
37	5-#5	18.00	19.36
38	3-#5	18.00	19.36
39	5-#5	18.00	19.36
40	3-#5	18.00	19.36
41	5-#5	18.00	19.36
42	8-#5	12.00	19.36
43	6-#5	16.00	19.36
44	5-#5	18.00	19.36
45	5-#5	18.00	19.36
46	5-#5	18.00	19.36
47	5-#5	18.00	19.36
48	8-#5	12.00	19.36
49	3-#5	18.00	19.36
50	6-#5	16.00	19.36
51	3-#5	18.00	19.36
52	5-#5	18.00	19.36
53	3-#5	18.00	19.36
54	5-#5	18.00	19.36
55	3-#5	18.00	19.36
56	5-#5	18.00	19.36

57	3-#5	18.00	19.36
58	5-#5	18.00	19.36

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.08	13.31
2	DC1 (Top)	0.95	13.31
3	DC1 (Top)	0.91	13.31
4	DC1 (Top)	0.96	13.31
5	DC1 (Top)	0.97	13.31
6	DC1 (Top)	0.82	13.31
7	DC1 (Top)	0.99	13.31
8	DC1 (Top)	0.80	13.31
9	DC1 (Top)	1.00	13.31
10	DC1 (Top)	0.73	13.31
11	DC1 (Top)	0.99	13.31
12	DC1 (Top)	1.05	13.31
13	DC1 (Top)	0.91	13.31
14	DC1 (Top)	0.90	13.31
15	DC1 (Top)	0.92	13.31
16	DC1 (Top)	0.97	13.31
17	DC1 (Top)	0.99	13.31
18	DC1 (Top)	0.98	13.31
19	DC1 (Top)	0.96	13.31
20	DC1 (Top)	1.05	13.31
21	DC1 (Top)	0.91	13.31
22	DC1 (Top)	0.90	13.31
23	DC1 (Top)	0.92	13.31
24	DC1 (Top)	0.91	13.31
25	DC1 (Top)	0.78	13.31
26	DC1 (Top)	0.96	13.31
27	DC1 (Top)	0.70	13.31
28	DC1 (Top)	0.96	13.31
29	DC1 (Top)	0.79	13.31
30	DC1 (Top)	0.93	13.31
31	DC1 (Top)	1.06	8.31
32	DC1 (Top)	0.73	8.31
33	DC1 (Top)	0.95	8.31
34	DC1 (Top)	0.75	8.31
35	DC1 (Top)	0.88	8.31
36	DC1 (Top)	0.67	8.31
37	DC1 (Top)	0.88	8.31
38	DC1 (Top)	0.71	8.31
39	DC1 (Top)	0.87	8.31
40	DC1 (Top)	0.70	8.31
41	DC1 (Top)	0.84	8.31
42	DC1 (Top)	1.04	8.31
43	DC1 (Top)	0.95	8.31
44	DC1 (Top)	0.84	8.31
45	DC1 (Top)	0.81	8.31
46	DC1 (Top)	0.83	8.31
47	DC1 (Top)	0.82	8.31
48	DC1 (Top)	1.00	8.31
49	DC1 (Top)	0.80	8.31
50	DC1 (Top)	0.86	8.31
51	DC1 (Top)	0.79	8.31
52	DC1 (Top)	0.77	8.31
53	DC1 (Top)	0.80	8.31
54	DC1 (Top)	0.75	8.31
55	DC1 (Top)	0.80	8.31

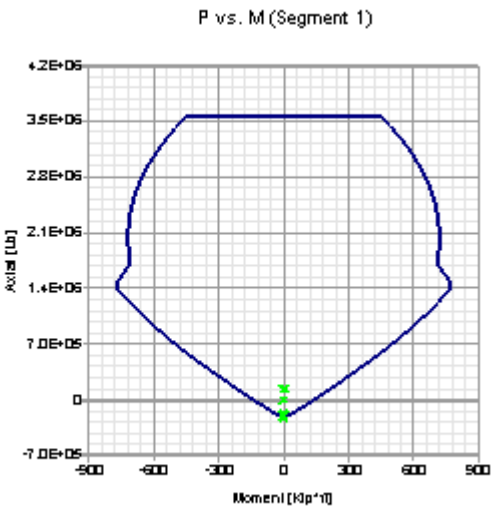
56	DC1 (Top)	0.75	8.31
57	DC1 (Top)	0.80	8.31
58	DC1 (Top)	0.75	8.31

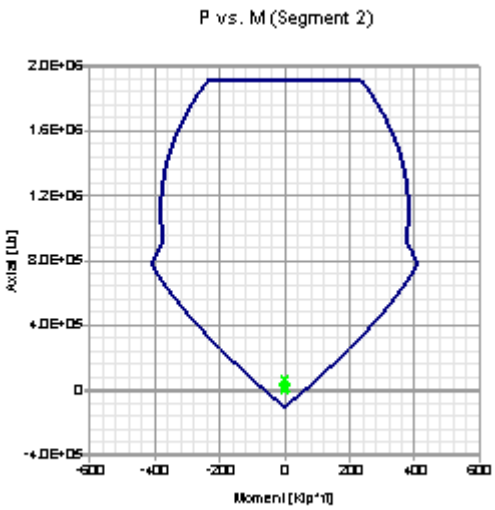
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	133977.40	0.00	213.48	0.00
2	DC1 (Top)	63791.68	0.00	96.75	0.00
3	DC1 (Top)	121149.50	0.00	171.03	0.00
4	DC1 (Top)	66769.26	0.00	98.35	0.00
5	DC1 (Top)	138737.50	0.00	179.97	0.00
6	DC1 (Top)	34448.25	0.00	80.81	0.00
7	DC1 (Top)	154621.30	0.00	189.07	0.00
8	DC1 (Top)	31094.30	0.00	78.84	0.00
9	DC1 (Top)	158886.50	0.00	191.37	0.00
10	DC1 (Top)	20767.41	0.00	72.76	0.00
11	DC1 (Top)	157062.80	0.00	190.38	0.00
12	DC1 (Top)	120457.90	0.00	206.26	0.00
13	DC1 (Top)	55163.40	0.00	92.11	0.00
14	DC1 (Top)	118516.60	0.00	169.62	0.00
15	DC1 (Top)	57722.12	0.00	93.49	0.00
16	DC1 (Top)	138027.70	0.00	179.58	0.00
17	DC1 (Top)	156668.70	0.00	190.17	0.00
18	DC1 (Top)	151275.20	0.00	187.27	0.00
19	DC1 (Top)	142625.20	0.00	182.61	0.00
20	DC1 (Top)	120142.10	0.00	206.10	0.00
21	DC1 (Top)	54636.84	0.00	91.83	0.00
22	DC1 (Top)	118321.60	0.00	169.51	0.00
23	DC1 (Top)	57230.47	0.00	93.22	0.00
24	DC1 (Top)	113761.60	0.00	166.52	0.00
25	DC1 (Top)	28953.27	0.00	77.58	0.00
26	DC1 (Top)	143081.00	0.00	182.85	0.00
27	DC1 (Top)	16033.48	0.00	69.98	0.00
28	DC1 (Top)	144720.50	0.00	183.73	0.00
29	DC1 (Top)	30649.63	0.00	78.58	0.00
30	DC1 (Top)	132385.20	0.00	177.09	0.00
31	DC1 (Top)	89381.71	0.00	133.58	0.00
32	DC1 (Top)	20767.09	0.00	47.61	0.00
33	DC1 (Top)	105511.50	0.00	117.07	0.00
34	DC1 (Top)	22847.96	0.00	48.38	0.00
35	DC1 (Top)	98871.05	0.00	103.22	0.00
36	DC1 (Top)	10011.76	0.00	43.59	0.00
37	DC1 (Top)	106130.00	0.00	106.14	0.00
38	DC1 (Top)	17646.97	0.00	46.44	0.00
39	DC1 (Top)	103958.70	0.00	105.41	0.00
40	DC1 (Top)	15531.68	0.00	45.65	0.00
41	DC1 (Top)	93908.05	0.00	101.98	0.00
42	DC1 (Top)	83838.01	0.00	131.76	0.00
43	DC1 (Top)	104227.90	0.00	116.64	0.00
44	DC1 (Top)	83928.14	0.00	98.10	0.00
45	DC1 (Top)	84884.14	0.00	98.61	0.00
46	DC1 (Top)	88227.70	0.00	99.85	0.00
47	DC1 (Top)	85226.88	0.00	98.73	0.00
48	DC1 (Top)	64672.87	0.00	125.46	0.00
49	DC1 (Top)	32324.23	0.00	51.92	0.00
50	DC1 (Top)	67422.16	0.00	104.37	0.00

51	DC1 (Top)	30061.14	0.00	51.08	0.00
52	DC1 (Top)	63765.22	0.00	90.57	0.00
53	DC1 (Top)	32229.18	0.00	51.89	0.00
54	DC1 (Top)	65875.70	0.00	91.51	0.00
55	DC1 (Top)	31245.44	0.00	51.52	0.00
56	DC1 (Top)	66511.73	0.00	91.75	0.00
57	DC1 (Top)	32244.99	0.00	51.89	0.00
58	DC1 (Top)	64856.66	0.00	91.13	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	141293.90	3568690.00	0.04
2	D1 (Bottom)	66562.84	1904507.00	0.03
3	D2 (Top)	137524.90	3571979.00	0.04
4	D2 (Bottom)	97791.29	1904507.00	0.05
5	D2 (Top)	220249.10	3452639.00	0.06
6	D2 (Top)	47963.63	1904507.00	0.03
7	D2 (Top)	164660.60	3571979.00	0.05
8	D2 (Bottom)	59018.54	1904507.00	0.03
9	D2 (Top)	164631.10	3571979.00	0.05
10	D2 (Bottom)	54163.25	1904507.00	0.03
11	D2 (Bottom)	217336.40	3571979.00	0.06
12	DC1 (Bottom)	133977.40	3568690.00	0.04
13	D1 (Bottom)	63791.68	1904507.00	0.03
14	D2 (Top)	151941.90	3571979.00	0.04
15	D2 (Bottom)	96896.10	1904507.00	0.05
16	D2 (Bottom)	220249.20	3452639.00	0.06
17	D2 (Top)	167947.60	3571979.00	0.05
18	D2 (Bottom)	164631.60	3571979.00	0.05
19	D2 (Bottom)	206535.50	3571979.00	0.06
20	D1 (Bottom)	120457.90	3568690.00	0.03
21	D1 (Bottom)	55163.40	1904507.00	0.03
22	D2 (Top)	156184.80	3571979.00	0.04

23	D2 (Top)	90152.36	1904507.00	0.05
24	D2 (Bottom)	175060.00	3452639.00	0.05
25	D2 (Bottom)	71778.30	1904507.00	0.04
26	D2 (Bottom)	167946.60	3571979.00	0.05
27	D2 (Bottom)	79643.82	1904507.00	0.04
28	DC1 (Bottom)	151274.60	3571979.00	0.04
29	D2 (Top)	42833.69	1904507.00	0.02
30	D2 (Bottom)	173202.10	3571979.00	0.05
31	DC1 (Top)	89381.71	2373646.00	0.04
32	DC1 (Bottom)	29402.74	1268027.00	0.02
33	D2 (Bottom)	139820.30	2376935.00	0.06
34	D2 (Top)	61258.00	1268027.00	0.05
35	D1 (Top)	98871.05	2299019.00	0.04
36	D2 (Bottom)	29173.07	1268027.00	0.02
37	D2 (Top)	107751.20	2378579.00	0.05
38	D2 (Bottom)	31020.72	1268027.00	0.02
39	DC1 (Top)	103958.70	2378579.00	0.04
40	D2 (Top)	21839.27	1268027.00	0.02
41	D2 (Top)	124817.50	2378579.00	0.05
42	D1 (Bottom)	89381.83	2373646.00	0.04
43	D2 (Top)	166548.10	2376935.00	0.07
44	D1 (Bottom)	98871.58	2299019.00	0.04
45	D2 (Bottom)	107751.90	2378579.00	0.05
46	DC1 (Bottom)	103959.00	2378579.00	0.04
47	D2 (Bottom)	124817.40	2378579.00	0.05
48	D2 (Top)	180347.20	2373646.00	0.08
49	D2 (Top)	61430.72	1268027.00	0.05
50	D2 (Bottom)	97671.88	2376935.00	0.04
51	D2 (Top)	50642.55	1268027.00	0.04
52	D1 (Bottom)	83927.16	2299019.00	0.04
53	DC1 (Top)	32229.18	1268027.00	0.03
54	DC1 (Bottom)	84883.18	2378579.00	0.04
55	DC1 (Top)	31245.44	1268027.00	0.02
56	D1 (Bottom)	88226.71	2378579.00	0.04
57	DC1 (Top)	32244.99	1268027.00	0.03
58	D1 (Bottom)	77556.71	2378579.00	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	219277.60	234360.00	0.94
2	DC1 (Top)	0.00	100440.00	0.00
3	DC1 (Top)	0.00	167400.00	0.00
4	DC1 (Top)	0.00	100440.00	0.00
5	DC1 (Top)	0.00	167400.00	0.00
6	DC1 (Top)	0.00	100440.00	0.00
7	DC1 (Top)	0.00	167400.00	0.00
8	DC1 (Top)	0.00	100440.00	0.00
9	DC1 (Top)	0.00	167400.00	0.00
10	DC1 (Top)	0.00	100440.00	0.00
11	DC1 (Top)	0.00	167400.00	0.00
12	D3 (Bottom)	206050.80	234360.00	0.88
13	DC1 (Top)	0.00	100440.00	0.00
14	DC1 (Top)	0.00	167400.00	0.00
15	DC1 (Top)	0.00	100440.00	0.00
16	DC1 (Top)	0.00	167400.00	0.00
17	DC1 (Top)	0.00	167400.00	0.00

18	DC1 (Top)	0.00	167400.00	0.00
19	DC1 (Top)	0.00	167400.00	0.00
20	D3 (Top)	162917.00	234360.00	0.70
21	DC1 (Top)	0.00	100440.00	0.00
22	DC1 (Top)	0.00	167400.00	0.00
23	DC1 (Top)	0.00	100440.00	0.00
24	DC1 (Top)	0.00	167400.00	0.00
25	DC1 (Top)	0.00	100440.00	0.00
26	DC1 (Top)	0.00	167400.00	0.00
27	DC1 (Top)	0.00	100440.00	0.00
28	DC1 (Top)	0.00	167400.00	0.00
29	DC1 (Top)	0.00	100440.00	0.00
30	DC1 (Top)	0.00	167400.00	0.00
31	D3 (Bottom)	151825.40	267840.00	0.57
32	DC1 (Top)	0.00	100440.00	0.00
33	DC1 (Top)	0.00	200880.00	0.00
34	DC1 (Top)	0.00	100440.00	0.00
35	DC1 (Top)	0.00	167400.00	0.00
36	DC1 (Top)	0.00	100440.00	0.00
37	DC1 (Top)	0.00	167400.00	0.00
38	DC1 (Top)	0.00	100440.00	0.00
39	DC1 (Top)	0.00	167400.00	0.00
40	DC1 (Top)	0.00	100440.00	0.00
41	DC1 (Top)	0.00	167400.00	0.00
42	D3 (Bottom)	127861.10	267840.00	0.48
43	DC1 (Top)	0.00	200880.00	0.00
44	DC1 (Top)	0.00	167400.00	0.00
45	DC1 (Top)	0.00	167400.00	0.00
46	DC1 (Top)	0.00	167400.00	0.00
47	DC1 (Top)	0.00	167400.00	0.00
48	DC1 (Top)	0.00	267840.00	0.00
49	DC1 (Top)	0.00	100440.00	0.00
50	DC1 (Top)	0.00	200880.00	0.00
51	DC1 (Top)	0.00	100440.00	0.00
52	DC1 (Top)	0.00	167400.00	0.00
53	DC1 (Top)	0.00	100440.00	0.00
54	DC1 (Top)	0.00	167400.00	0.00
55	DC1 (Top)	0.00	100440.00	0.00
56	DC1 (Top)	0.00	167400.00	0.00
57	DC1 (Top)	0.00	100440.00	0.00
58	DC1 (Top)	0.00	167400.00	0.00

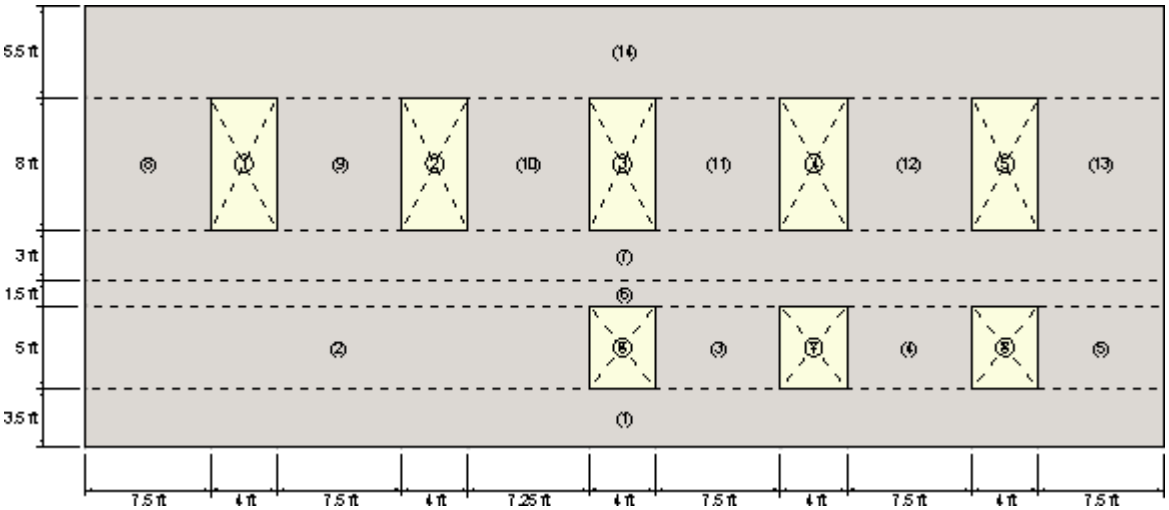
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	139209.550	0.00
2	DC1 (Top)	0.000	74245.091	0.00
3	DC1 (Top)	0.000	139209.550	0.00
4	DC1 (Top)	0.000	74245.091	0.00
5	DC1 (Top)	0.000	134569.230	0.00
6	DC1 (Top)	0.000	74245.091	0.00
7	DC1 (Top)	0.000	139209.550	0.00
8	DC1 (Top)	0.000	74245.091	0.00
9	DC1 (Top)	0.000	139209.550	0.00
10	DC1 (Top)	0.000	74245.091	0.00
11	DC1 (Top)	0.000	139209.550	0.00
12	DC1 (Top)	0.000	139209.550	0.00

13	DC1 (Top)	0.000	74245.091	0.00
14	DC1 (Top)	0.000	139209.550	0.00
15	DC1 (Top)	0.000	74245.091	0.00
16	DC1 (Top)	0.000	134569.230	0.00
17	DC1 (Top)	0.000	139209.550	0.00
18	DC1 (Top)	0.000	139209.550	0.00
19	DC1 (Top)	0.000	139209.550	0.00
20	DC1 (Top)	0.000	139209.550	0.00
21	DC1 (Top)	0.000	74245.091	0.00
22	DC1 (Top)	0.000	139209.550	0.00
23	DC1 (Top)	0.000	74245.091	0.00
24	DC1 (Top)	0.000	134569.230	0.00
25	DC1 (Top)	0.000	74245.091	0.00
26	DC1 (Top)	0.000	139209.550	0.00
27	DC1 (Top)	0.000	74245.091	0.00
28	DC1 (Top)	0.000	139209.550	0.00
29	DC1 (Top)	0.000	74245.091	0.00
30	DC1 (Top)	0.000	139209.550	0.00
31	DC1 (Top)	0.000	86924.270	0.00
32	DC1 (Top)	0.000	46359.611	0.00
33	DC1 (Top)	0.000	86924.270	0.00
34	DC1 (Top)	0.000	46359.611	0.00
35	DC1 (Top)	0.000	84026.794	0.00
36	DC1 (Top)	0.000	46359.611	0.00
37	DC1 (Top)	0.000	86924.270	0.00
38	DC1 (Top)	0.000	46359.611	0.00
39	DC1 (Top)	0.000	86924.270	0.00
40	DC1 (Top)	0.000	46359.611	0.00
41	DC1 (Top)	0.000	86924.270	0.00
42	DC1 (Top)	0.000	86924.270	0.00
43	DC1 (Top)	0.000	86924.270	0.00
44	DC1 (Top)	0.000	84026.794	0.00
45	DC1 (Top)	0.000	86924.270	0.00
46	DC1 (Top)	0.000	86924.270	0.00
47	DC1 (Top)	0.000	86924.270	0.00
48	DC1 (Top)	0.000	86924.270	0.00
49	DC1 (Top)	0.000	46359.611	0.00
50	DC1 (Top)	0.000	86924.270	0.00
51	DC1 (Top)	0.000	46359.611	0.00
52	DC1 (Top)	0.000	84026.794	0.00
53	DC1 (Top)	0.000	46359.611	0.00
54	DC1 (Top)	0.000	86924.270	0.00
55	DC1 (Top)	0.000	46359.611	0.00
56	DC1 (Top)	0.000	86924.270	0.00
57	DC1 (Top)	0.000	46359.611	0.00
58	DC1 (Top)	0.000	86924.270	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	64.75	3.50
2	0.00	3.50	30.25	5.00
3	34.25	3.50	7.50	5.00
4	45.75	3.50	7.50	5.00
5	57.25	3.50	7.50	5.00
6	0.00	8.50	64.75	1.50
7	0.00	10.00	64.75	3.00
8	0.00	13.00	7.50	8.00
9	11.50	13.00	7.50	8.00
10	23.00	13.00	7.25	8.00
11	34.25	13.00	7.50	8.00
12	45.75	13.00	7.50	8.00
13	57.25	13.00	7.50	8.00
14	0.00	21.00	64.75	5.50

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	7-#5	14.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
2	7-#5	14.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17

3	5-#5	18.00	19.36	4-#5	18.00	25.17
4	5-#5	18.00	19.36	4-#5	18.00	25.17
5	5-#5	18.00	19.36	4-#5	18.00	25.17
6	7-#5	14.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
	3-#5	18.00	19.36	1-#5	18.00	25.17
	5-#5	18.00	19.36	1-#5	18.00	25.17
7	8-#5	12.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	6-#5	16.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
8	8-#5	12.00	19.36	6-#5	18.00	25.17
9	6-#5	16.00	19.36	6-#5	18.00	25.17
10	5-#5	18.00	19.36	6-#5	18.00	25.17
11	5-#5	18.00	19.36	6-#5	18.00	25.17
12	5-#5	18.00	19.36	6-#5	18.00	25.17
13	5-#5	18.00	19.36	6-#5	18.00	25.17
14	8-#5	12.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	6-#5	16.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17

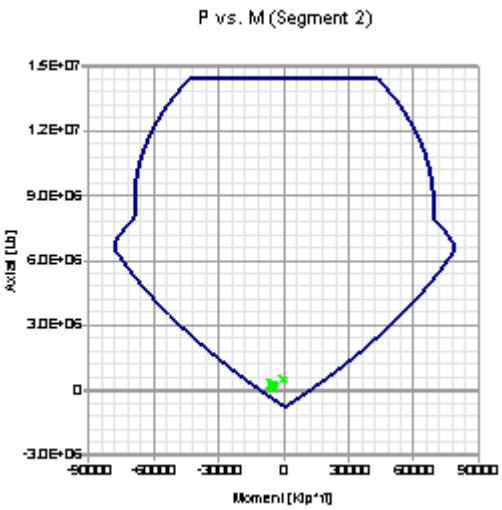
Intermediate results for axial-bending

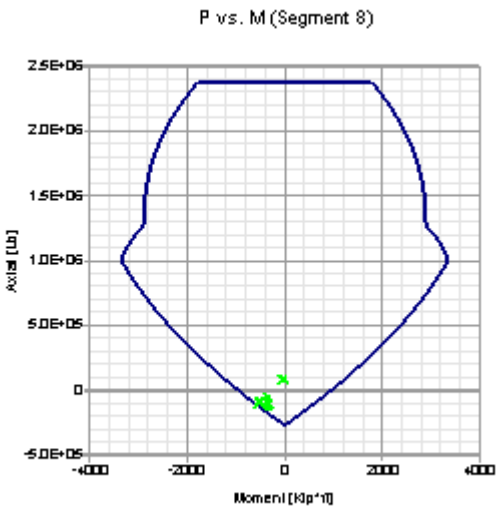
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	41.75	621.60
2	D3 (Bottom)	17.31	290.40
3	D3 (Max)	5.91	72.00
4	D3 (Bottom)	5.19	72.00
5	D3 (Bottom)	6.00	72.00
6	D3 (Top)	39.21	621.60
7	D3 (Bottom)	54.07	621.60
8	D3 (Max)	3.97	72.00
9	D3 (Top)	9.11	72.00
10	D3 (Max)	7.00	69.60
11	D3 (Max)	6.55	72.00
12	D3 (Max)	6.30	72.00
13	D2 (Max)	6.60	72.00
14	D3 (Top)	59.02	621.60

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	720391.50	-12134.34	68982.33	0.18
2	D3 (Bottom)	186904.80	-5445.74	13171.36	0.41
3	D3 (Max)	168176.80	-124.46	1196.97	0.10
4	D3 (Bottom)	131072.00	-116.08	1072.14	0.11
5	D3 (Bottom)	172878.50	-126.05	1212.53	0.10
6	D3 (Top)	581627.90	-8347.63	64780.87	0.13
7	D3 (Bottom)	423279.10	-7144.26	59385.40	0.12
8	D3 (Max)	-100430.60	-517.42	605.10	0.86
9	D3 (Top)	144249.10	416.70	1199.79	0.35
10	D3 (Max)	104050.20	248.96	925.83	0.27
11	D3 (Max)	88656.92	159.56	908.58	0.18
12	D3 (Max)	80062.16	137.79	880.14	0.16
13	D2 (Max)	90515.60	113.93	914.73	0.12
14	D3 (Top)	526511.40	4999.99	69180.97	0.07

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	1098205.00	3.083178E07	0.04
2	DC1 (Bottom)	524435.40	1.440232E07	0.04
3	D2 (Max)	205589.20	3571979.00	0.06
4	D2 (Max)	188137.10	3571979.00	0.05
5	D2 (Max)	210672.10	3571979.00	0.06
6	DC1 (Bottom)	1027537.00	3.083178E07	0.03
7	D1 (Top)	684478.00	2.052547E07	0.03
8	D1 (Max)	89792.25	2373646.00	0.04
9	D2 (Max)	179050.80	2376935.00	0.08
10	D2 (Max)	126691.50	2299019.00	0.06
11	D2 (Max)	111061.30	2378579.00	0.05

12	DC1 (Max)	106057.80	2378579.00	0.04
13	D2 (Bottom)	124714.20	2378579.00	0.05
14	D2 (Top)	644626.60	2.052547E07	0.03

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	1573560.00	0.00
2	DC1 (Top)	0.00	770040.00	0.00
3	DC1 (Top)	0.00	167400.00	0.00
4	DC1 (Top)	0.00	167400.00	0.00
5	DC1 (Top)	0.00	167400.00	0.00
6	DC1 (Top)	0.00	1573560.00	0.00
7	DC1 (Top)	0.00	1640520.00	0.00
8	D3 (Bottom)	127095.80	267840.00	0.47
9	DC1 (Top)	0.00	200880.00	0.00
10	DC1 (Top)	0.00	167400.00	0.00
11	DC1 (Top)	0.00	167400.00	0.00
12	DC1 (Top)	0.00	167400.00	0.00
13	DC1 (Top)	0.00	167400.00	0.00
14	DC1 (Top)	0.00	1640520.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	553128.030	2858435.400	0.19
2	D3 (Top)	362052.520	1314814.700	0.28
3	D2 (Max)	92113.112	349488.080	0.26
4	D2 (Max)	79069.085	346870.250	0.23
5	D2 (Top)	51699.419	344637.340	0.15
6	D3 (Max)	610988.270	2856002.800	0.21
7	D3 (Max)	470904.870	2219068.100	0.21
8	D3 (Max)	37778.003	167034.570	0.23
9	D3 (Bottom)	174960.890	260663.230	0.67
10	D3 (Max)	118123.970	256919.560	0.46
11	D3 (Max)	84303.169	262931.670	0.32
12	D3 (Max)	53566.807	261642.450	0.20
13	D2 (Max)	30360.186	263210.470	0.12
14	D3 (Max)	507963.740	2233756.900	0.23

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Current Date: 2/25/2016 9:10 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\BPOE GL C.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	26.50 [ft]
Total length	:	22.50 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	6.33	10.00	3.00	10.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
2	EQ	Horizontal	336820.00	0.00	0.00
1	EQ	Horizontal	116474.00	0.00	0.00

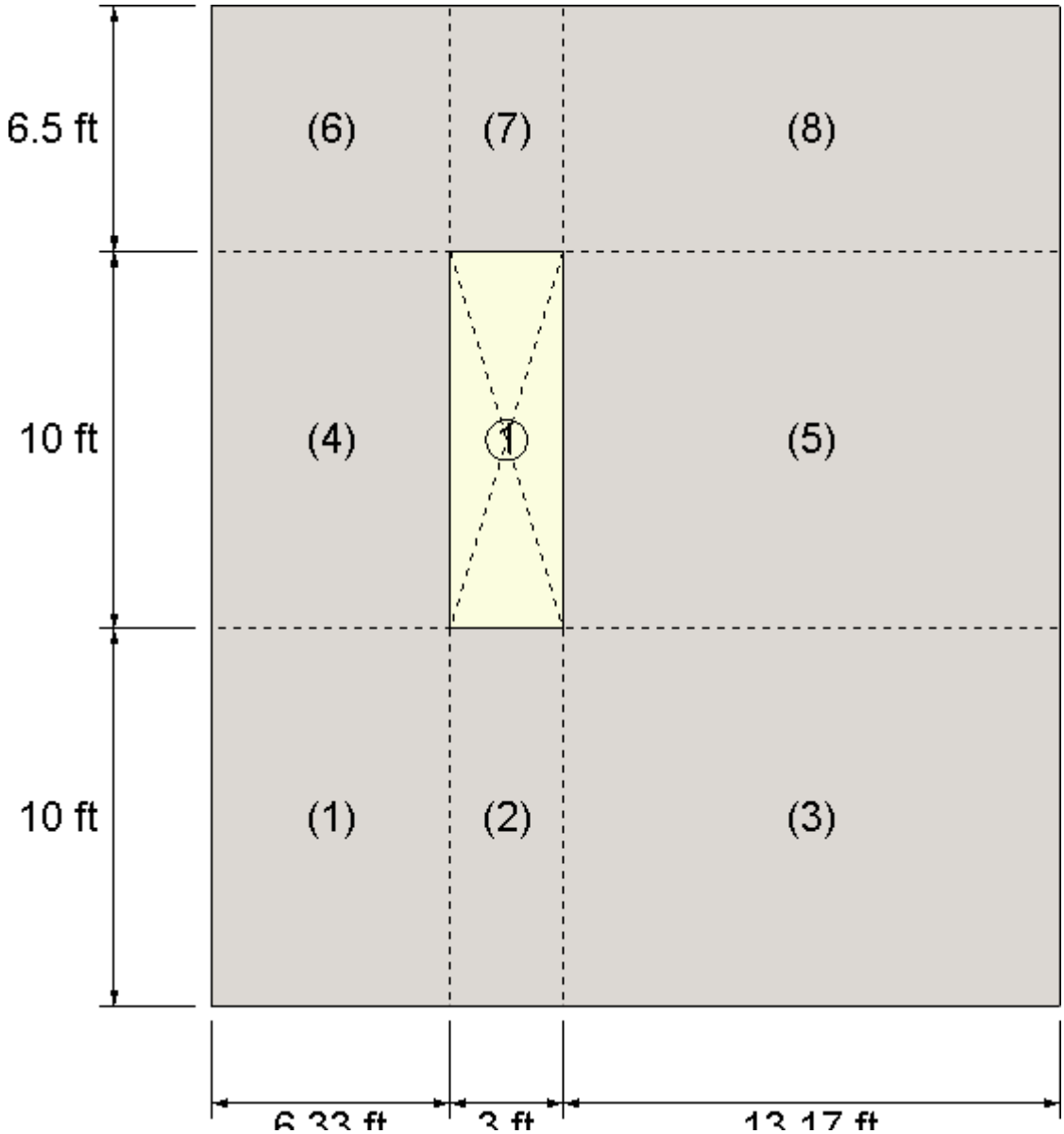
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
2	DL	Vertical	5850.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	6.33	10.00
2	6.33	0.00	3.00	10.00
3	9.33	0.00	13.17	10.00
4	0.00	10.00	6.33	10.00
5	9.33	10.00	13.17	10.00
6	0.00	20.00	6.33	6.50
7	6.33	20.00	3.00	6.50
8	9.33	20.00	13.17	6.50

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#7	12.00	33.89
2	3-#5	14.00	19.36
3	12-#5	14.00	19.36
4	8-#6	10.00	23.24
5	9-#5	18.00	19.36
6	8-#6	10.00	23.24
7	2-#5	18.00	19.36
8	9-#5	18.00	19.36

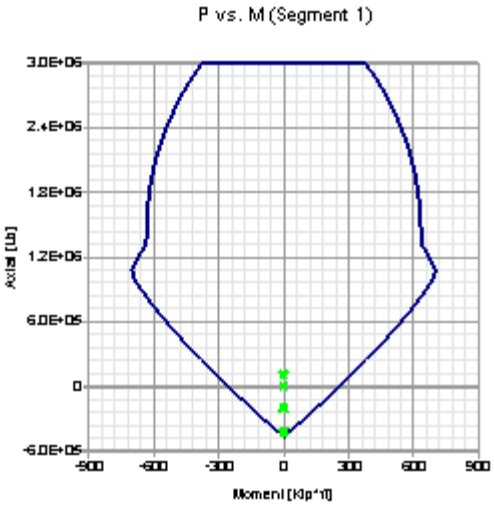
Intermediate results for axial-bending

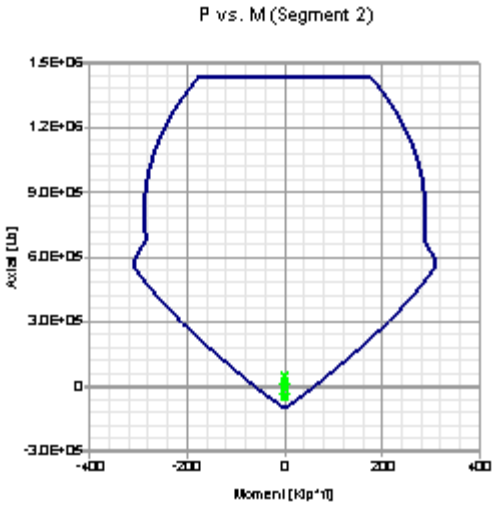
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.54	13.19
2	DC1 (Top)	0.92	13.31
3	DC1 (Top)	1.04	13.31
4	DC1 (Top)	1.34	8.25
5	DC1 (Top)	0.80	8.31
6	DC1 (Top)	1.31	8.25
7	DC1 (Top)	0.74	8.31
8	DC1 (Top)	0.75	8.31

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	107954.70	0.00	307.13	0.00
2	DC1 (Top)	19347.17	0.00	70.67	0.00
3	DC1 (Top)	213666.10	0.00	358.19	0.00
4	DC1 (Top)	71207.09	0.00	158.49	0.00
5	DC1 (Top)	133647.80	0.00	170.15	0.00
6	DC1 (Top)	52668.48	0.00	152.90	0.00
7	DC1 (Top)	24661.37	0.00	35.97	0.00
8	DC1 (Top)	109283.80	0.00	161.05	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	115725.00	2999412.00	0.04
2	D1 (Bottom)	47536.10	1427147.00	0.03
3	D2 (Bottom)	780934.20	6267100.00	0.12
4	D1 (Bottom)	81814.68	1995789.00	0.04
5	D2 (Bottom)	381941.90	4176423.00	0.09
6	D2 (Top)	137598.80	1995789.00	0.07
7	D1 (Top)	24661.37	951431.50	0.03
8	D2 (Bottom)	308265.00	4176423.00	0.07

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	432253.80	453600.00	0.95
2	D3 (Bottom)	47768.95	100440.00	0.48
3	DC1 (Top)	0.00	401760.00	0.00
4	D3 (Bottom)	206955.20	380160.00	0.54
5	DC1 (Top)	0.00	301320.00	0.00
6	D3 (Bottom)	166480.90	380160.00	0.44
7	DC1 (Top)	0.00	66960.00	0.00
8	DC1 (Top)	0.00	301320.00	0.00

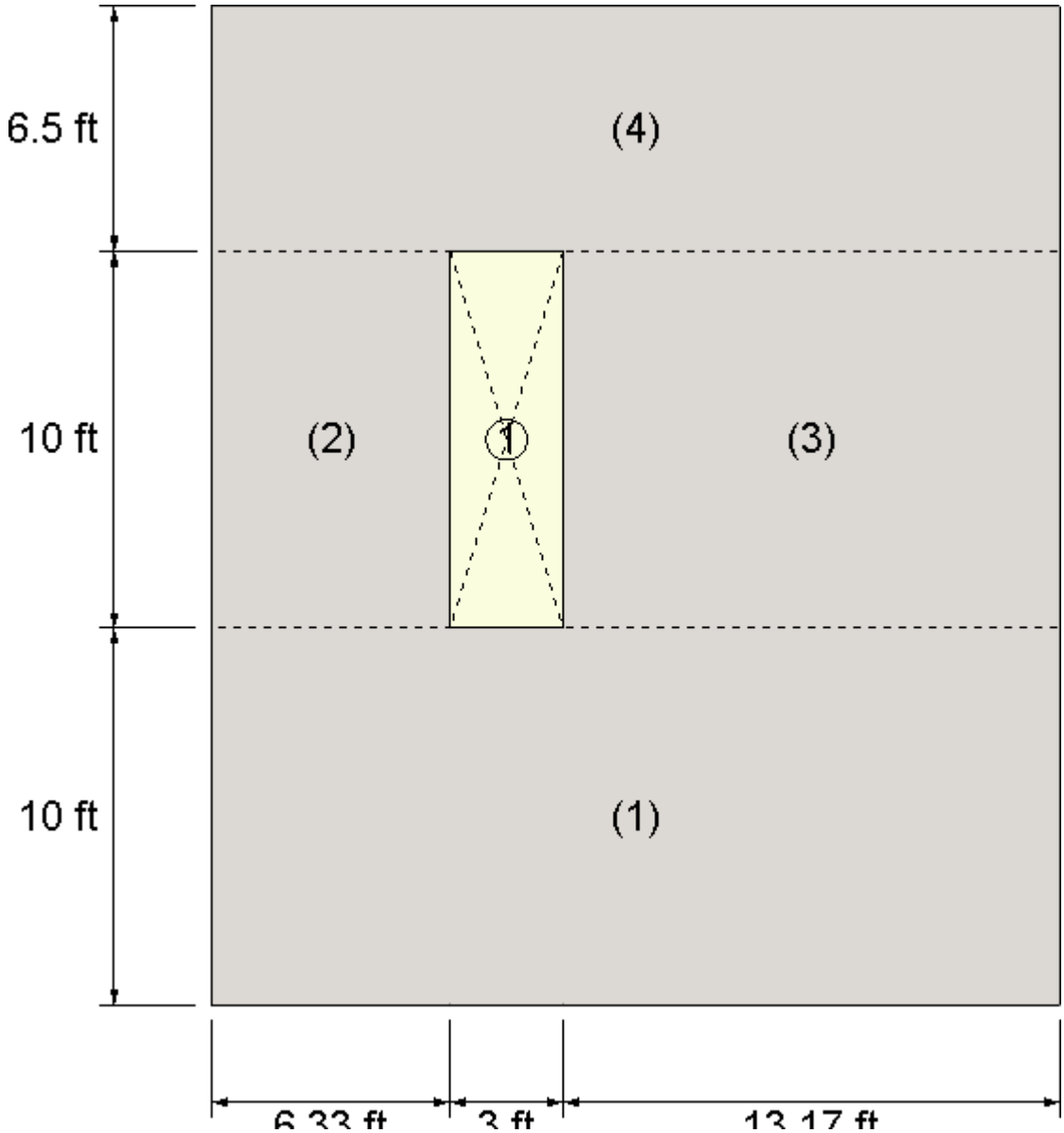
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	116389.640	0.00
2	DC1 (Top)	0.000	55683.818	0.00
3	DC1 (Top)	0.000	244451.960	0.00
4	DC1 (Top)	0.000	72812.474	0.00
5	DC1 (Top)	0.000	152639.020	0.00

6	DC1 (Top)	0.000	72812.474	0.00
7	DC1 (Top)	0.000	34769.708	0.00
8	DC1 (Top)	0.000	152639.020	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	22.50	10.00
2	0.00	10.00	6.33	10.00
3	9.33	10.00	13.17	10.00
4	0.00	20.00	22.50	6.50

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	7-#7	12.00	33.89	8-#5	16.00	25.17
	3-#5	14.00	33.89	8-#5	16.00	25.17
	12-#5	14.00	33.89	8-#5	16.00	25.17
2	8-#6	10.00	23.24	7-#5	18.00	25.17
3	9-#5	18.00	19.36	7-#5	18.00	25.17
4	8-#6	10.00	23.24	5-#5	18.00	25.17
	2-#5	18.00	23.24	5-#5	18.00	25.17
	9-#5	18.00	23.24	5-#5	18.00	25.17

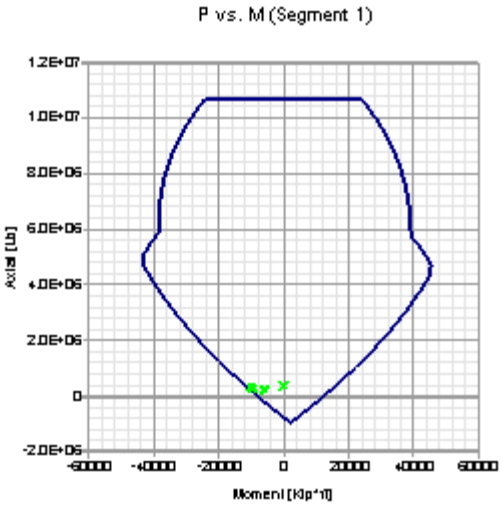
Intermediate results for axial-bending

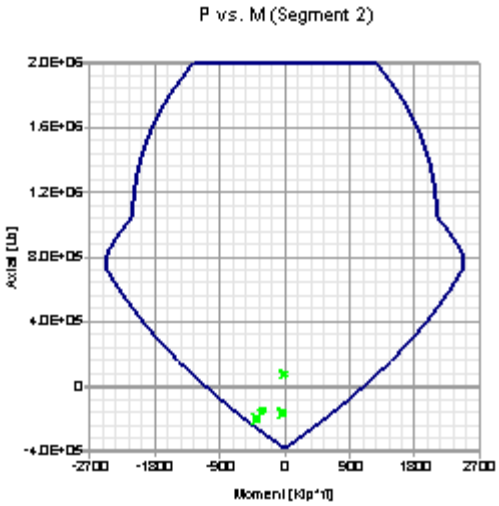
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	20.17	216.00
2	D3 (Bottom)	3.77	60.77
3	D3 (Bottom)	17.32	126.43
4	D3 (Bottom)	21.46	216.00

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	251792.80	-9836.55	10579.03	0.93
2	D3 (Bottom)	-206741.10	-393.16	527.75	0.74
3	D3 (Bottom)	348007.50	-2016.99	3903.63	0.52
4	D3 (Bottom)	135741.30	-2235.20	7360.60	0.30

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	392564.30	1.069366E07	0.04
2	D1 (Bottom)	81820.70	1995789.00	0.04
3	D2 (Bottom)	381833.70	4176423.00	0.09
4	D2 (Top)	254149.70	7123643.00	0.04

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	955800.00	0.00
2	D3 (Bottom)	206741.10	380160.00	0.54
3	DC1 (Top)	0.00	301320.00	0.00
4	DC1 (Top)	0.00	748440.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	457614.000	1035137.500	0.44
2	D3 (Bottom)	44487.234	136931.340	0.32
3	D3 (Top)	294490.860	482044.520	0.61
4	D3 (Max)	346384.460	769069.510	0.45

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

SUPPLEMENTAL REPORT

Supplemental Facility Seismic Evaluation

Benton County Courthouse
120 NW 4th Avenue, Corvallis, Oregon

Prepared for
Benton County

Prepared by

MILLER CONSULTING ENGINEERS, INC.
9570 SW Barbur Boulevard, Suite 100
Portland, Oregon 97219
Ph.: 503-246-1250
MCE Project Number: 151029
August 29, 2016

SUPPLEMENTAL TABLE OF CONTENTS

Supplemental Executive Summary and Scope of Work .....2 – 4

New Building Cost Estimate.....5

Structural and Non-structural Plans .....6 – 10

Performance Evaluation.....11 – 18

Demand Capacity Ratio .....19 – 21

Cost Benefit Table .....22 – 26

Cost Tables.....27 – 46

New Building Cost Table.....47 – 48

Appendix

SPOE Summary Data Sheet and Checklists .....APPENDIX – A

SPOE Supporting Calculations ..... APPENDIX – B

BENTON COUNTY COURTHOUSE EXECUTIVE SUMMARY

Overview

Benton County has contracted Miller Consulting Engineers, Inc. (MCE) to provide an additional seismic evaluation for the Benton County Courthouse using the *ASCE 41-13* standard to supplement our previous report dated January 29, 2016. For this additional seismic evaluation, we evaluated the building using the BPOE-2E seismic hazard, which is considered a severe earthquake, coupled with the Life Safety Building Performance Level objective. The goal of this objective is to allow the occupants to safely exit the structure after a severe earthquake; however, continued occupancy might not be likely before repair work is completed. This additional evaluation is a non-standard combination when reviewing the *ASCE 41-13* and will be referred to as the Supplemental Performance Objective (SPOE) henceforth and is a combination of the Basic Safety Objective for Life-Safety (3-C) with a BSE-2E seismic hazard level that is an earthquake that has the probability of exceedance of 5% in 50 years or a mean return period of 975 years between occurrences. In addition, Benton County also contracted MCE to provide an engineer's estimate of probable construction cost for a new criminal justice center that houses the courts and offices currently in the courthouse located on county property or new property. The new center is intended to be a generic multiple-story courthouse building at about 45,000 square feet (SF).

As previously noted, the ASCE 41-13 standard has been developed so that buildings meeting the Basic Performance Objective (BPOE) are expected to experience little damage from relatively frequent, moderate earthquakes but significantly more damage and potential economic loss from the most severe and infrequent earthquakes. To seismically evaluate the building, the *ASCE 41-13* uses site-specific information that is generated from the U.S. Geological Survey (USGS). The USGS information provides the spectral acceleration for different return periods in which the BPOE-1E seismic hazard, considered a moderate earthquake, has a 225-year return period and the BPOE-2E seismic hazard, considered a severe earthquake, has a 975-year return period. This information is then modified into a design acceleration considering the building's structural system along with other factors such as the soil conditions at the site.

To summarize the building performance objectives used within this report (see the table below for additional clarification), the Basic Performance Objective is to allow the occupants to safely

exit the structure after a moderate earthquake. However, continued occupancy might not be likely before repair work is completed. The Enhanced Performance Objective is to allow the building to be immediately operable after a moderate earthquake. It is expected that the building will develop cracks, but items that are damaged will remain in place. The Limited Performance Objective is to prevent the building from collapsing during a major earthquake. Therefore it is expected that the building will be severely damaged and should not continue to be occupied following the earthquake. Lastly, the Supplemental Performance Objective is similar to the Basic Performance Objective, but considers a severe earthquake as the seismic hazard in lieu of a moderate earthquake.

Performance Objective	Seismic Hazard	Occurrence	Performance Level
Basic	Moderate	225 Years	Life Safety standard
Enhanced	Moderate	225 Years	Immediate Occupancy standard
Limited	Severe	975 Years	Collapse Prevention standard
Supplemental	Severe	975 Years	Life Safety standard

This additional evaluation identified the following items that need to be addressed considering the performance objective as part of the proposed seismic upgrade:

- Strengthening the roof and floor diaphragms to transfer lateral loads to the walls.
- Providing connections between the walls and roof and floor diaphragms for in-plane and out-of-plane loads.
- Providing connections at the roof and floor diaphragm to account for plan irregularities.
- Providing concrete shear walls for many of the unreinforced masonry walls to support the in-plane loading.
- Providing bonded shear walls to reinforce the unreinforced masonry walls at the fourth floor bell lounge for out-of-plane loads.
- Providing structural steel horizontal framing members at the stairway openings to support the unreinforced masonry walls for out-of-plane loads.
- Providing structural steel support columns for the secondary support of the trusses and beams that are independent of the unreinforced masonry walls.
- Addressing the non-structural concerns identified in the evaluation including bracing the fire suppression lines, bracing the gas line, bracing the suspended gypsum ceilings, bracing the in-line mechanical equipment and anchoring the non-compliant shelves.

We anticipate that the proposed upgrade work for this supplemental evaluation will take approximately 24 months to complete and will require the building to be vacated during construction. It is intended that the above repairs will be completed in such a way that the new construction will be encapsulated within a wall cavity or masked in some manner to preserve the historical and architectural features where possible. The following is our preliminary estimate of probable construction cost to upgrade the building based on the additional seismic evaluation (listed as Supplemental), as well as the cost estimates from our previous report as well:

Performance Objective	Construction	Contingency	Total Cost	Cost per sq. foot
Basic	$4,312,271	$1,078,068	$6,737,923	$204
Enhanced	$5,824,570	$1,456,143	$9,100,891	$275
Limited	$6,966,813	$1,741,703	$10,885,645	$329
Supplemental	$7,876,117	$1,969,029	$12,306,433	$372

The above costs only consider the structural upgrades and corresponding work as required by the structural upgrades; the other concerns with the building such as HVAC, plumbing, ADA access or other tenant improvement costs are not included in this cost. The above cost estimate is based on the work being completed as a whole and not broken into phases to allow for the continued occupancy of the building during construction. Furthermore, these estimates are limited to the repairs of the detailed items and do not include relocation costs of staff or other soft costs. Within our opinion of probable construction cost estimate, we have included the contractor's overhead and profit, as well as a 25% construction contingency as further detailed in the cost portion of this report.

Scope of Work

For this additional evaluation, Miller Consulting Engineers, Inc. was contracted by Benton County to provide an ASCE 41-13 evaluation of the county courthouse located in Corvallis, Oregon, using criteria found in the *ASCE 41-13* document. In addition, Benton County also contracted MCE to provide an engineer's estimate of probable construction cost for a new criminal justice center that houses the courts and offices currently in the courthouse located on county property or new property. The new center is intended to be a generic multiple-story courthouse building at about 45,000 square foot.

For this additional evaluation, the non-structural review excludes building contents and furnishings. Furthermore, soils information was based on information obtained from the Oregon

Department of Geology and Mineral Information (DOGAMI) website, which provided the site classification, soil rupture information, and liquefaction information for this study.

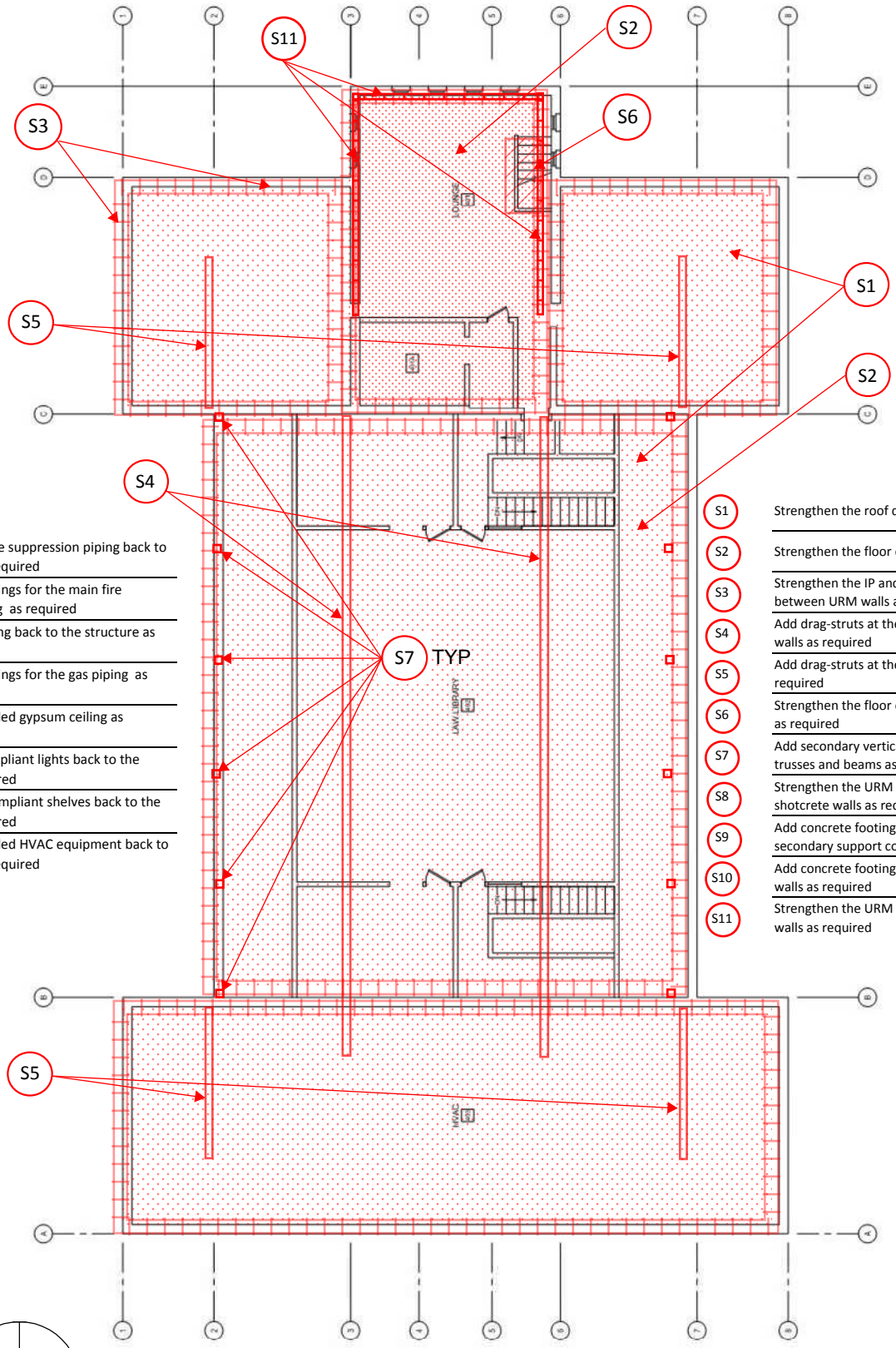
Miller Consulting Engineers, Inc. also provided an engineer's preliminary estimate of probable construction costs as detailed within this report to correct the deficiencies found based on this evaluation, as well as the cost estimate for the new criminal justice center. These estimates are limited to the repairs of the items within this report or for the new construction of the criminal justice center and do not include relocation costs of staff or other soft costs unless noted otherwise. Due to the current marketplace and the construction boom that is currently ongoing in the area, these cost estimates may not accurately depict the current construction costs due to rapid changes in construction material costs, as well as labor availability.

New Building Cost Estimate

With regard to the engineer's estimate of probable construction cost for a new criminal justice center, MCE developed a cost estimate considering a new 45,000 square foot, three-story, steel-framed courthouse building that has moderate interior and exterior finishes. The cost estimate does include some additional amenities that are included in the cost estimate table such as taller ceilings for two of the floors to enhance the courtrooms, closed circuit video production, and a secondary elevator for public and private access. It is estimated that the exterior facade would consist of glazing with limestone or masonry veneer. The following is our preliminary estimate of probable construction cost to build the above-proposed building:

Item	Construction	Contingency	Total Cost	Cost per sq. foot
Base Cost	$10,795,500	$2,698,875	$13,494,375	$300
Additive Cost	$1,620,864	$405,216	$2,026,080	$44
Total Cost	$12,416,364	$3,104,091	$15,520,455	$344

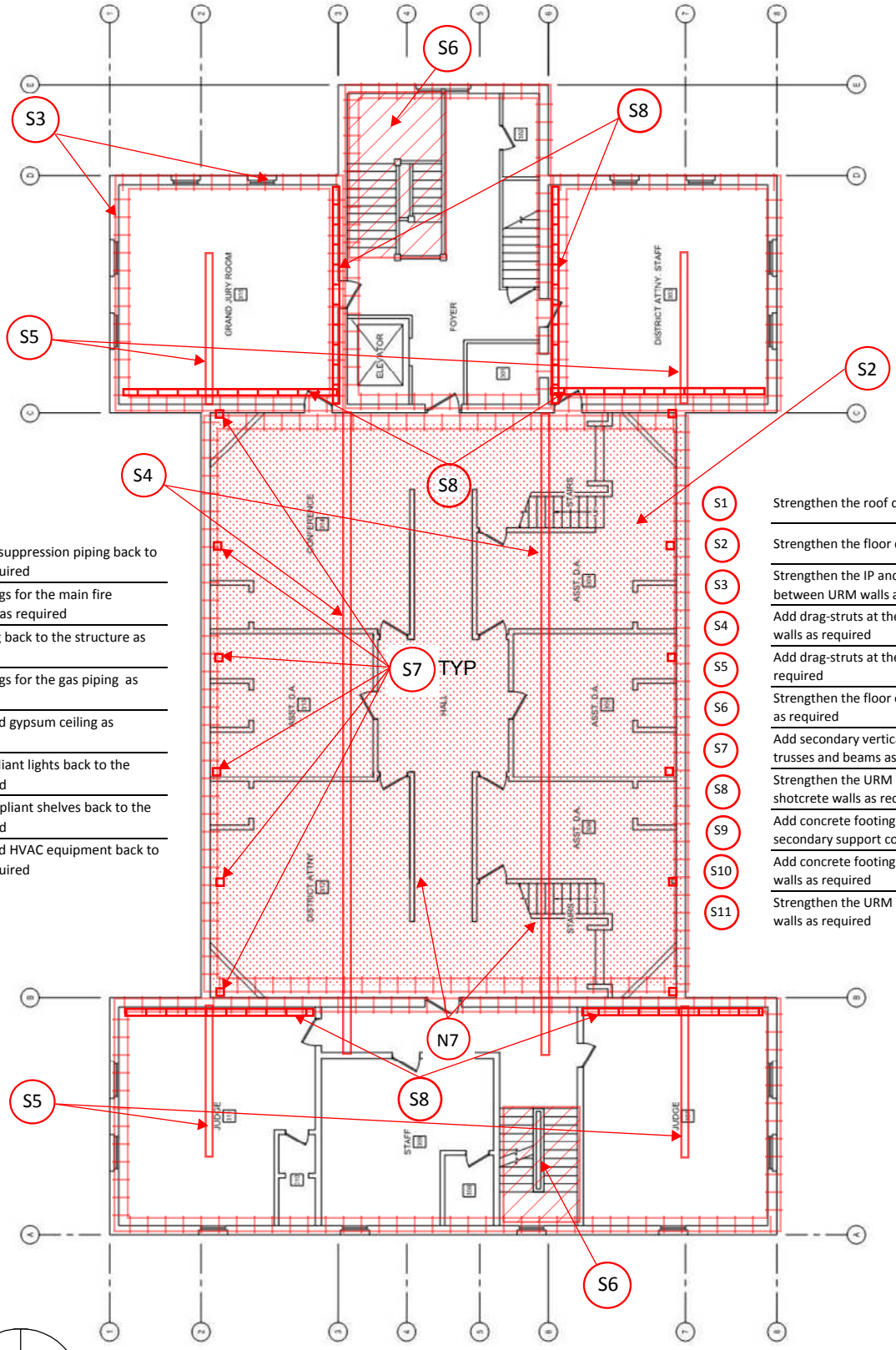
The above costs are based on using today's dollars and include soft costs such as permitting and architectural design fees, as well as the contractor's overhead and profit and a 25% construction contingency as further detailed in the cost portion of this report. However, only the structural permitting fees are included in the above and other permitting fees such as HVAC, plumbing, and electrical may apply. Lastly, the costs are for the building itself and do not include site development or site development upgrades that may be necessary to build the proposed building. For planning purposes using this cost estimate, an escalation factor should be used to account for when the construction can begin compared with today's dollars.



N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

FOURTH FLOOR PLAN



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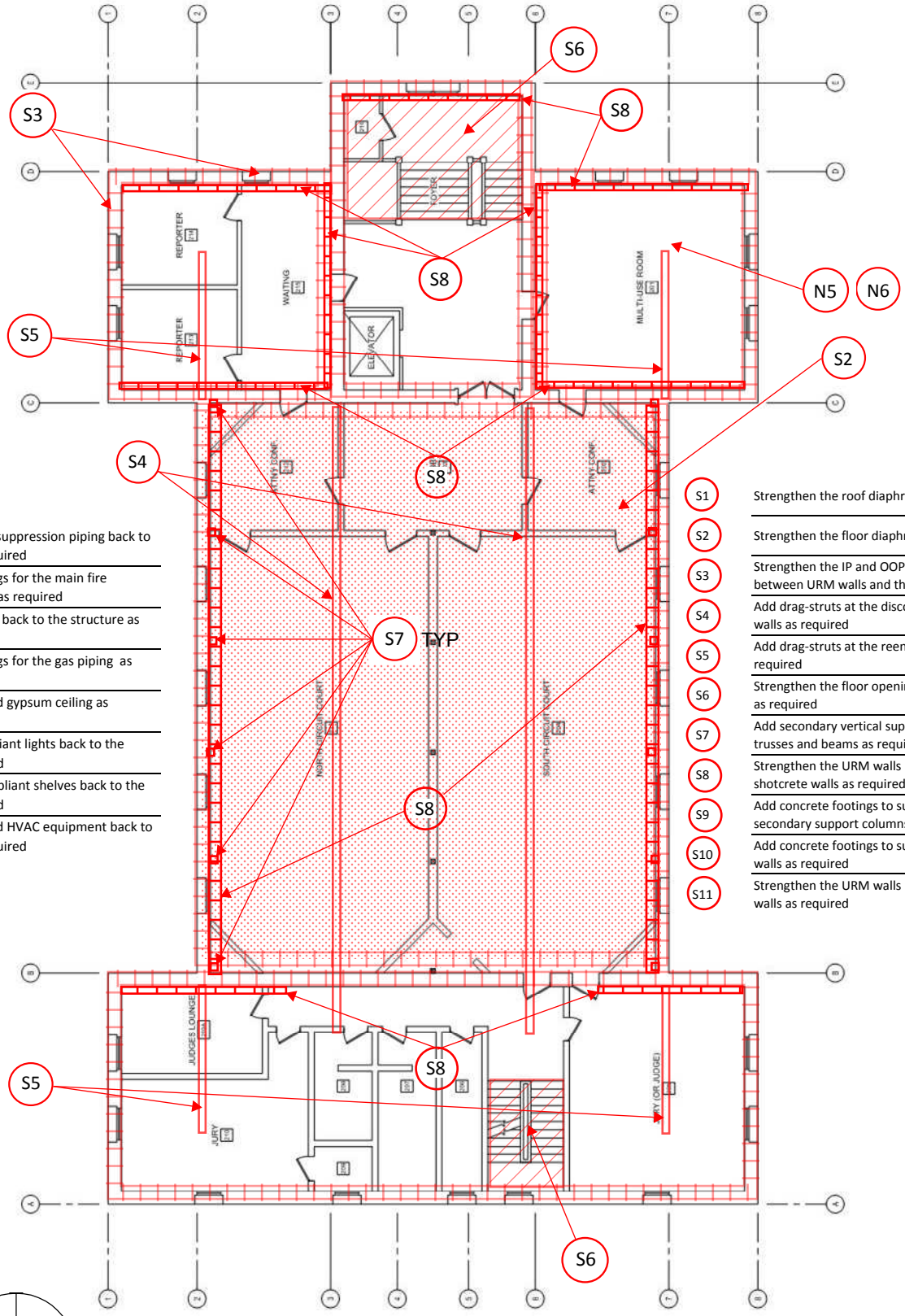
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

THIRD FLOOR PLAN

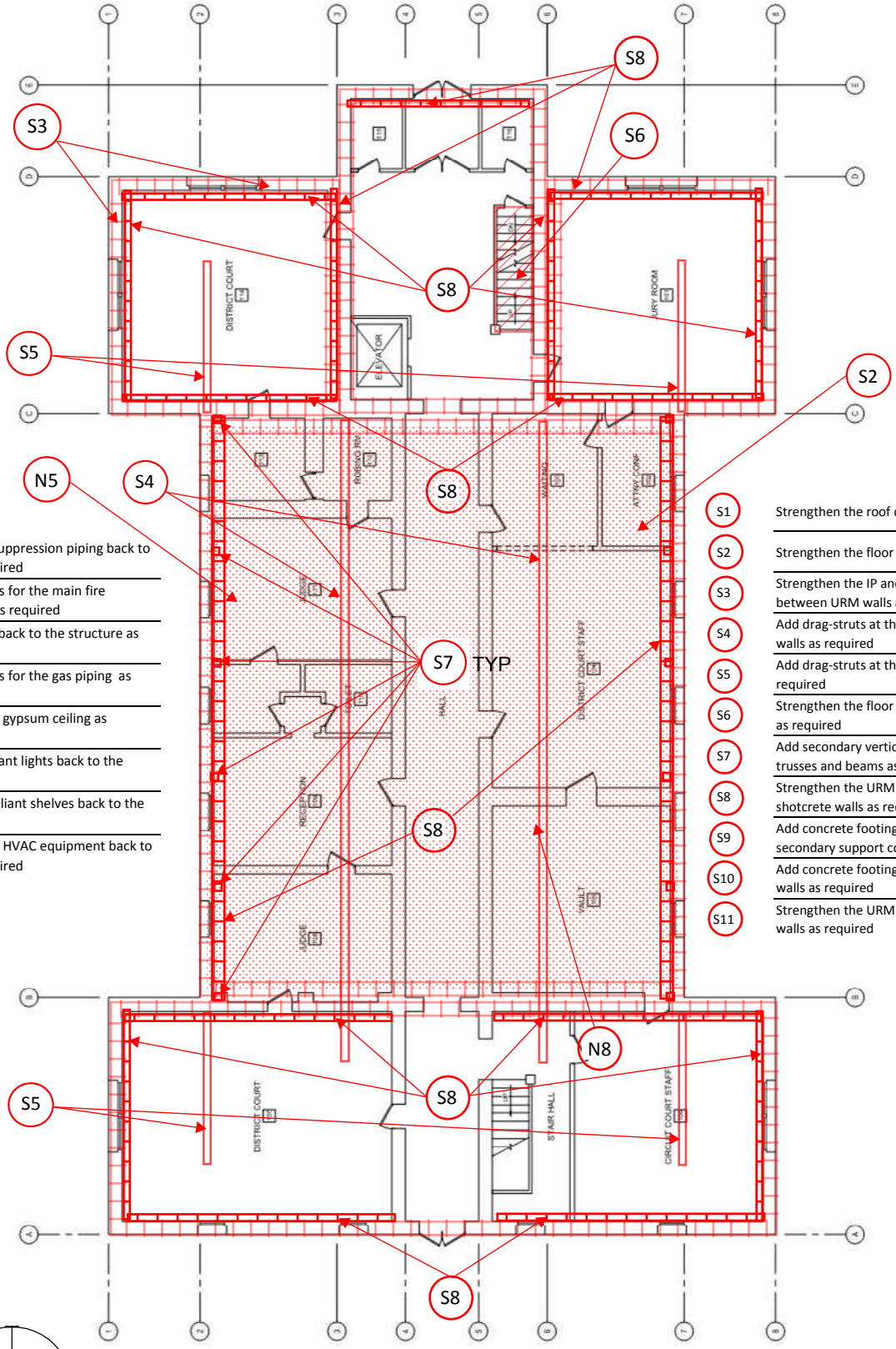


N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

SECOND FLOOR PLAN

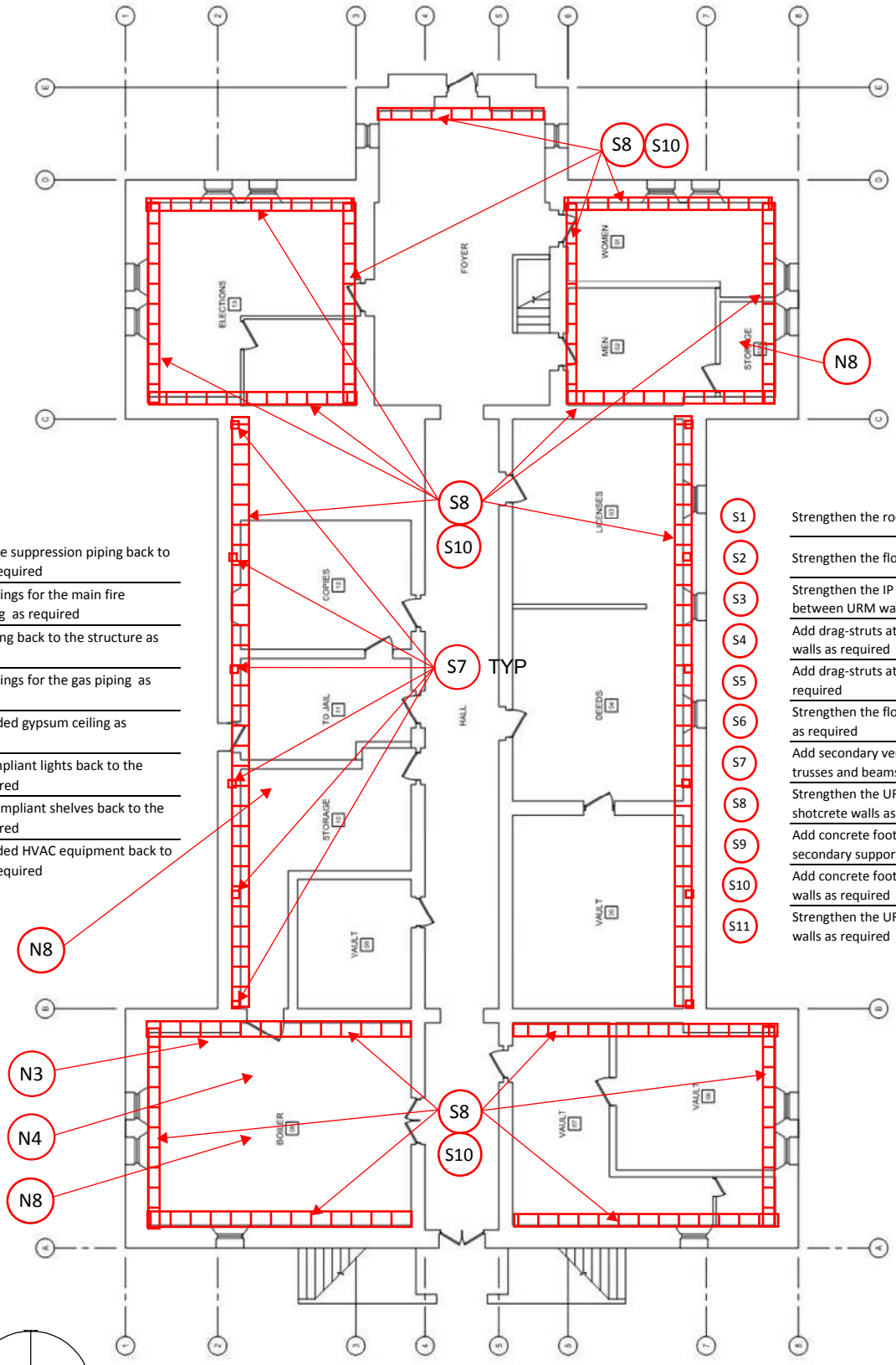


N1	Brace the main fire suppression piping back to the structure as required
N2	Add flexible couplings for the main fire suppression piping as required
N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

FIRST FLOOR PLAN



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N3	Brace the gas piping back to the structure as required
N4	Add flexible couplings for the gas piping as required
N5	Brace the suspended gypsum ceiling as required
N6	Brace the noncompliant lights back to the structure as required
N7	Anchor the noncompliant shelves back to the structure as required
N8	Brace the suspended HVAC equipment back to the structure as required

S1	Strengthen the roof diaphragm
S2	Strengthen the floor diaphragm
S3	Strengthen the IP and OOP connections between URM walls and the diaphragm
S4	Add drag-struts at the discontinuous URM walls as required
S5	Add drag-struts at the reentrant corners as required
S6	Strengthen the floor openings at the stairways as required
S7	Add secondary vertical support for the girder trusses and beams as required
S8	Strengthen the URM walls by adding concrete shotcrete walls as required
S9	Add concrete footings to support the secondary support columns as required
S10	Add concrete footings to support the shotcrete walls as required
S11	Strengthen the URM walls by adding bonded walls as required

N

BASEMENT FLOOR PLAN

Supplemental Performance Objective

The goal of this objective is to allow the occupants to safely exit the structure after an earthquake. Continued occupancy might not be likely before repair work is completed. The Supplemental Performance Objective (SPOE) uses the Life Safety standard and has checklists that provide a systematic approach to the evaluation of the building considering the system modification factors for the materials in the building. Furthermore, this evaluation uses the BSE-2E seismic hazard level, which is an earthquake that has the probability of exceedance of 5% in 50 years or a mean return period of 975 years between occurrences.

The courthouse building is a five-story structure comprised of three primary areas on each floor, which consist of a central core area and an east and west wing area to the east and west of the core area. Also, the clock tower is located at the center of the east wing and protrudes to the east 10 feet from the remainder of the east wing. There are several unknown structural conditions, which include the condition of the unreinforced masonry walls considering the tensile and shear strength of the masonry, if there is an isolation joint between the courthouse and the adjacent jail building and how the existing anchors at the ceiling of the fourth floor are attached to the ceiling diaphragm.

There is a concern with the unknown condition of the isolation joint; if there is no isolation joint between the jail and the courthouse there is a potential pounding concern between the buildings. There are a few options to address this concern one of which is to cut in a new isolation gap into the jail building and provide support for the jail building as required. The other option is to tie the courthouse to the jail building. We anticipate that the preliminary estimate of probable construction cost for the new isolation gap could be approximately $120,000 and could cause some disruption along the south wall of the jail. Tying the buildings together is a more complicated concern and more investigation and design will be required to provide the specifics and the probable construction cost estimate. However, this concern would be covered by the contingency we have used in our preliminary estimate of probable construction cost. The following is the structural evaluation of the courthouse building.

Roof Level Evaluation

The roof framing of the building consists of 2x roof rafters at 24" on center (o.c.) typically that bear on the exterior walls with straight tongue-and-groove lumber roof sheathing (the lumber is installed perpendicular to the framing). The rafters are notched and toenailed to a 4 x 12 wood bearing plate that was set in a bed of mortar on top of the brick walls that also has small diameter bolts (3/8" diameter) spaced at 10' o.c. that appear to be embedded into the top of the brick wall. The bolts have wood spacers for unknown reasons of varying length where the nut bears on top of the spacer. The core area roof is framed differently than the wings and has heavy timber trusses at 12' o.c. that bear on heavy steel plates on the exterior brick walls, of which the trusses support the roof and ceiling framing. However, the heavy steel plates at the truss bearing locations do not appear to be anchored or connected to the top of the brick wall in any way.

Considering the straight sheathing, the west wing exceeds the allowable aspect ratios, as allowed by the checklists, and based on our analysis has a demand-to-capacity ratio (DCR) of 2.53, which is to say that there is 2.53 times more demand (or load) than the capacity of the sheathing. At the core roof area the straight sheathing also spans greater than 24 feet, as allowed by the checklists, and based on our analysis has a DCR of 13.2.

As far as anchorage is concerned, the standard states that when anchors are nonexistent, mitigation with connections needed to anchor the walls to the diaphragm is necessary to meet the performance goals. This concern applies both to the connection of the roof diaphragm, as well as the connection at the heavy timber trusses. Furthermore, the roof lacks connections capable of transferring the loads as a result of the building irregularities (the offset walls on the north and south sides of the building) and offset shear walls (i.e., the shear walls at gridlines 3 and 6 are offset to the east).

Roof Level Recommendations

To address the noncompliant concerns at the roof level we recommend the following: For the straight sheathing, overlay the existing sheathing with new plywood sheathing and while addressing the sheathing add anchors and blocking for both the in-plane and out-of-plane loading concerns. For the missing connections, add custom structural steel drag struts from the offset shear walls and at the reentrant corners. Add connections from the heavy timber trusses to the heavy steel plates and then to the top of the unreinforced masonry walls. Lastly, provide

structural steel support columns for the secondary support of the trusses that are independent of the unreinforced masonry walls.

Fourth Floor Evaluation

The fourth floor is predominately located within the original attic space of the heavy timber trusses and was finished as part of the 1978 renovation. The fourth floor bell lounge room is an original finished space that is at the base of the clock tower and is slightly above the remaining fourth floor level; also, the unreinforced masonry walls in the Bell Tower Lounge are inadequate to support the out-of-plane loading. The renovated fourth floor space has light wood floors that span between the bottom chords of the heavy timber trusses, but the floor diaphragm does not extend out to the north and south walls of the core area. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 3.25. In addition, the floor opening at the stairway at the bell lounge room is more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 1.28 and the unreinforced masonry walls at the stairway opening has a DCR of 6.2.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Fourth Floor Recommendations

To address the noncompliant concerns on the fourth floor (most of the repairs at the fourth floor are similar to the repairs as required at the roof level), we recommend including the strengthening of the floor diaphragm and the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the unreinforced masonry walls in the Bell Tower Lounge, add bonded walls to support the out-of-plane loading. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls.

Third Floor Evaluation

The third floor layout covers the footprint of the building including the area within the core, as well as the east and west wings of the building. The third floor core area was added as part of the

1978 renovation when a floor was framed in the upper 10 feet of the second floor. The floor is supported by glulam beams that span in the north-south direction with floor joists spanning in between the glulam beams. The area in the east and west wing is part of the original finished space of the building and is level with the remaining third floor level. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 3.3. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 3, 6, B and C in the east wing are not adequate to resist the in-plane loading from the diaphragm and have a DCR as high as 20 (see the calculations for the individual DCR for each wall line). In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR of 2.3 and the unreinforced masonry walls at the stairway opening has a DCR of 3.8.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Third Floor Recommendations

To address the noncompliant concerns on the third floor (most of the repairs at the third floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. Also, continue to provide structural steel support columns to the secondary support of the trusses, as well as the glulam beams of the third floor that are independent of the unreinforced masonry walls.

Second Floor Evaluation

The second floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 5.2. Based on the Tier

I quick check, the unreinforced masonry walls at Gridlines 2, 3, 6, 7, B, C, D and E are not adequate to resist the in-plane loading from the diaphragm and have a DCR as high as 10.9 (see the calculations for the individual DCR for each wall line). In addition, the floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our analysis the diaphragm has a DCR as high as 3.2 and the unreinforced masonry walls at the stairway opening has a DCR as high as 7.9.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

Second Floor Recommendations

To address the noncompliant concerns on the second floor (most of the repairs at the second floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls.

First Floor Evaluation

The first floor layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. At the core area, the unblocked diaphragm has a span greater than 40 feet, as allowed by the checklists, and based on our analysis has a DCR of 1.92. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 1, 2, 3, 6, 7, 8, A, B, C, D and E are not adequate to resist the in-plane loading from the diaphragm and have a DCR as high as 4.1 (see the calculations for the individual DCR for each wall line). The floor opening at the stairways at the east and west wings are more than 8 feet, as allowed by the checklists, and based on our

analysis the diaphragm has a DCR of 1.6 and the unreinforced masonry walls at the stairway opening has a DCR of 5.2.

As far as anchorage is concerned, connections were not observed from the floor framing to the unreinforced masonry walls. This concern applies both to the connection of the floor diaphragm, as well as to the lack of connections capable of transferring the loads as a result of the building irregularities and offset shear walls.

First Floor Recommendations

To address the noncompliant concerns on the first floor (most of the repairs at the first floor are similar to the repairs as required at the roof level), we recommend including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the stairway openings, add structural steel horizontal framing members to support the unreinforced masonry walls for out-of-plane loads and drag struts to transfer the loads from the diaphragm to the shear walls. For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry walls. In addition, the proximity of the jail building creates a potential pounding concern, which, if it is determined to be too close, a seismic isolation joist should be added between the courthouse and the adjacent jail.

Basement Evaluation

The basement layout appears to be part of the original finished space of the building and covers the footprint of the building including the area within the core, as well as the east and west wings of the building. Based on the Tier I quick check, the unreinforced masonry walls at Gridlines 1, 2, 3, 6, 7, 8, A, B, C, D and E are not adequate to resist the in-plane loading from the diaphragm and have a DCR as high as 6.9 (see the calculations for the individual DCR for each wall line).

Basement Recommendations

To address the noncompliant concerns on the basement, we recommend the following: For the inadequate unreinforced masonry shear walls, add shotcrete concrete shear walls to support the in-plane loading. Continue to provide structural steel support columns as the secondary support of the trusses and third floor glulam beams that are independent of the unreinforced masonry

walls. The concrete shear walls and the independent support columns will both require new foundation supports, which will need to be placed without impacting the existing foundation system of the building. To address this concern, we used deep foundation elements such as grouted micropiles to provide vertical support to the heavily loaded foundation elements.

Nonstructural Evaluation

On all levels of the courthouse, a number of non-structural deficiencies were observed, including unbraced fire suppression lines, missing flexible pipe connections at the fire suppression lines, unbraced gas lines, missing flexible pipe connections at the gas lines, noncompliant suspended gypsum board ceiling bracing, and unbraced light fixtures that are supported by the suspended gypsum board ceiling. There are also some noncompliant tall slender contents (book shelves, vertical storage cabinets, etc.) located on the first and third floors of the building. There are a few unknown non-structural conditions, which include whether the appendages including the statue at the east wall are reinforced and whether the fall-prone equipment weighing more than 20 pounds are braced.

Limitations

The information contained in this report is for the exclusive use of Benton County. Miller Consulting Engineers, Inc. assumes no responsibility or liability for any use of this report by other parties. This report relates solely to the stated purpose of this investigation; no representations concerning other aspects (if any) of the circumstance, structure or site are included. The conclusions (if any) are based on the above stated visual structural observations, and no destructive testing or monitoring was performed. Specific construction details exceed the scope of this report. No guarantee or warranty, expressed or implied, is provided.

Opinions of Probable Construction Cost

In providing opinions of probable construction cost, the Client understands that the Consultant has no control over the cost or availability of labor, equipment or materials, or over market conditions of the Contractor's method of pricing, and that the Consultant's opinions of probable construction costs are made on the basis of the Consultant's professional judgment and experience. Within our opinion of probable construction cost estimate, we have included the contractor's overhead and profit, as well as a 25% construction contingency to account for some

differences in the field during construction. The Consultant makes no warranty, express or implied, that the bids or the negotiated cost of the Work will not vary from the Consultant's opinion of probable construction cost.

Opinions of probable construction cost only include the cost to perform the retrofit work, cost to provide temporary storage for displaced material, removal of contents, a contained work environment with ventilation, and protection of fragile finishes. Items excluded from the opinions of probable construction cost that should also be considered include, but are not limited to, staff relocation, bonding, insurance, and permits.

STRUCTURAL AND NON-STRUCTURAL DEMAND CAPACITY RATIOS

DEFICIENCIES - BASIC PERFORMANCE OBJECTIVES

DCR:	Structural Items	Repair Solutions:
20.1**	Overstressed URM shear walls at the third floor at gridlines 3, 6, B and C	Reinforce URM shear walls by adding concrete shotcrete walls.
10.4**	Overstressed URM shear walls at the second floor at GLs 2, 3, 6, 7 and B-E	Reinforce URM shear walls by adding concrete shotcrete walls.
5.8**	Overstressed URM shear walls at the first floor at gridlines 1-3, 6-8, and A-E	Reinforce URM shear walls by adding concrete shotcrete walls.
13.8**	Overstressed URM shear walls at the basement at gridlines 1-3, 6-8 and A-E	Reinforce URM shear walls by adding concrete shotcrete walls.
10*	Missing/overstressed out-of-plane anchorage at URM walls to floor/roof	Install out-of-plane anchorage from URM walls to wood diaphragms.
10*	Transfer to shear walls is inadequate	Add continuous angles from roofs and floors to shear walls.
10*	Missing connection between heavy timber trusses and the URM walls	Install anchors from URM walls to trusses
10*	Missing drag strut connections at gridlines 3 & 6 from roof to URM walls	Install anchors from URM walls to the wood roof diaphragm
10*	Missing drag strut connections at gridlines 3 & 6 from floor to URM walls	Install anchors from URM walls to the wood floor diaphragms
1.7	URM walls at the bell lounge do not have adequate out-of-plane capacity	Strengthen the URM walls for out-of-plane load capacity
3.2**	Openings at the URM walls do not have drag struts for in-plane loads	Strap wood diaphragm back to URM wall.
7.9**	URM walls at the openings do not have adequate out-of-plane capacity	Add horizontal girts at the openings to support the URM walls
10*	Missing diaphragm cross ties across the wood diaphragms	Install straps ties at the floor joists across the floor diaphragm
2.5**	Straight sheathed wood roof diaphragm aspect ratio too large	Add plywood overlay sheathing over the west wing roof area
13.1**	Straight sheathed wood roof diaphragm span > 24'	Add plywood overlay sheathing over the core roof area
5.2**	Unblocked wood structural panel diaphragm span > 40'	Remove floor finishes and add blocking and nailing to the sheathing as required
10*	Missing independent secondary support columns for the heavy timber trusses	Install secondary support columns for the heavy timber trusses

* DCR ranking was limited to 10 to items that have no capacity
** DCR varies from floor to floor or gridline to gridline

DCR: Non-structural Items		Repair Solutions:
10*	Main fire suppression piping is inadequately braced.	Brace main fire suppression piping back to the structure.
10*	Fire suppression piping does not have flexible couplings per the NFPA-13	Install flexible couplings as required by the NFPA-13
10*	Natural gas piping inadequately braced.	Brace gas piping back to the structure.
10*	No flexible connections at gas piping to	Install flexible connections to equipment to
10*	Suspended gypsum ceiling is not braced.	Brace ceiling with guy wires and posts every
10*	The lights in Courtroom #3 are not	Install connections for the lights
10*	Noncompliant shelves are located on the	Brace the shelves back to the structure.
10*	The suspended HVAC inline air handlers	Install connections to equipment back to

* DCR ranking was limited to 10 to items that have no capacity

** DCR varies from floor to floor or gridline to gridline

COST BENEFIT TABLES AND COST TABLES

COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - SPOE		
CLIENT:	Benton County	DATE:	11-Aug-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Structural Items:							
Roof Level							
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	8.1	6,600	8.72	18
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	7
Bell Tower Lower Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	2.2	828	12.19	20
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	11
Insufficient OOP URM capacity	4th Floor	Build bonded shear wall	$66,100	1.3	1,613	31.52	25
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	5.1	100	4.51	10
Fourth Floor Level							
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	2.3	3,392	8.73	19
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	6
Third Floor Level							
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	3.6	3,392	6.25	12
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	15
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	3.1	200	13.39	22
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$25,700	10	3,392	0.76	4
Insufficient IP shear wall capacity	3rd Floor	Build shotcrete walls	$352,542	20.1	14,840	1.18	5
Second Floor Level							

COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	3.2	3,392	7.97	17
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$553,000	10	7,420	7.45	16
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	23
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$33,100	10	6,784	0.49	2
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$649,288	10.9	22,260	2.68	8
First Floor Level							
Insufficient floor diaphragm	1st Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.7	3,392	15.00	24
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$485,000	10	7,420	6.54	14
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2	104	12.98	21
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$36,400	10	10,176	0.36	1
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$1,098,258	5.8	29,680	6.38	13
Basement Level							
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,400	10	13,568	0.51	3
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$2,026,098	13.8	37,100	3.96	9

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

RANKED COST BENEFIT TABLE

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - SPOE		
CLIENT:	Benton County	DATE:	11-Aug-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Structural Items:							
Missing secondary beam supports	1st Floor	Add independent HSS column supports	$36,400	10	10,176	0.36	1
Missing secondary beam supports	2nd Floor	Add independent HSS column supports	$33,100	10	6,784	0.49	2
Missing secondary beam supports	Basement	Add independent HSS column supports and footings	$69,400	10	13,568	0.51	3
Missing secondary truss supports	3rd Floor	Add independent HSS column supports	$25,700	10	3,392	0.76	4
Insufficient IP shear wall capacity	3rd Floor	Build shotcrete walls	$352,542	20.1	14,840	1.18	5
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$139,000	10	7,420	1.87	6
Insufficient IP and OOP anchorage	Roof	Install anchors from URM walls to the wood roof diaphragm	$172,000	10	7,420	2.32	7
Insufficient IP shear wall capacity	2nd Floor	Build shotcrete walls	$649,288	10.9	22,260	2.68	8
Insufficient IP shear wall capacity	Basement	Build shotcrete walls with concrete footings	$2,026,098	13.8	37,100	3.96	9
Floor Opening Strengthening	4th Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,300	5.1	100	4.51	10
Insufficient IP and OOP anchorage	4th Floor	Install anchors from URM walls to the wood roof diaphragm	$41,300	10	828	4.99	11
Insufficient floor diaphragm	3rd Floor	Add plywood overlay sheathing over the core roof area	$76,300	3.6	3,392	6.25	12
Insufficient IP shear wall capacity	1st Floor	Build shotcrete walls	$1,098,258	5.8	29,680	6.38	13
Insufficient IP and OOP anchorage	1st Floor	Install anchors from URM walls to the wood roof diaphragm	$485,000	10	7,420	6.54	14
Insufficient IP and OOP anchorage	3rd Floor	Install anchors from URM walls to the wood roof diaphragm	$500,300	10	7,420	6.74	15
Insufficient IP and OOP anchorage	2nd Floor	Install anchors from URM walls to the wood roof diaphragm	$553,000	10	7,420	7.45	16

RANKED COST BENEFIT TABLE

SEISMIC DESCRIPTION	LEVEL	ACTIVITY	COST	DCR RATIO	AREA	BENEFIT RATIO	RANKING OF 25
Insufficient floor diaphragm	2nd Floor	Add plywood overlay sheathing over the core roof area	$86,500	3.2	3,392	7.97	17
Insufficient roof diaphragm	Roof	Add plywood overlay sheathing over the core roof area	$466,000	8.1	6,600	8.72	18
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$68,100	2.3	3,392	8.73	19
Insufficient floor diaphragm	4th Floor	Add plywood overlay sheathing over the core roof area	$22,200	2.2	828	12.19	20
Floor Opening Strengthening	1st Floor	Install steel girt for OOP support of the URM wall at the stairway	$2,700	2	104	12.98	21
Floor Opening Strengthening	3rd Floor	Install steel girt for OOP support of the URM wall at the stairway	$8,300	3.1	200	13.39	22
Floor Opening Strengthening	2nd Floor	Install steel girt for OOP support of the URM wall at the stairway	$12,200	4.2	200	14.52	23
Insufficient floor diaphragm	1st Floor	Add plywood overlay sheathing over the core roof area	$86,500	1.7	3,392	15.00	24
Insufficient OOP URM capacity	4th Floor	Build bonded shear wall	$66,100	1.3	1,613	31.52	25

NOTES:

1	Benefit Ratio=cost/(floor area*DCR)
2	Lower ranking = better benefit vs. cost.
3	DCR= demand capacity ratio

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - SPOE		
CLIENT:	Benton County	DATE:	11-Aug-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
General						
Mobilization	Job Trailer	18	Month	$500	$9,000	
	Supervisor	78	Week	$4,125	$321,750	
Construction Aids						
Lift 500# capacity	Provide (2) boom lifts	18	Month	$8,600	$154,800	
Temporary Storage	Provide (2) 40' container for storage as required	18	Month	$300	$5,400	
Ventalation	Negative Air Equipment per (4) zones	78	Week	$2,000	$156,000	
						$646,950
Strengthen West, East & Core Roofs	Remove (E) tile roofing	6600	S.F.	$4	$26,400	
	Add framing angles from rafters to top PL and anchors to TOW @ 4' o.c.	470	L.F.	$50	$23,500	
	Add perimeter steel PL and anchors in masonry	470	L.F.	$223	$104,810	
	Add Blocking between rafters@ anchors	470	L.F.	$25	$11,750	
	Add Plywood	6600	S.F.	$6	$39,600	
Cross ties	Install steel PLs on each side of the rafters across the west/east wings (18 per wing)	36	Ea.	$1,000	$36,000	
	Install steel PLs on each side of the rafters splaices at the core area (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
Heavy timber truss connections	Install steel angles to the sides of the truss and welded to steel PL	12	Ea	$4,000	$48,000	
	Add (4) anchors from the steel PL to the URM wall	12	Ea	$2,000	$24,000	
Roof Drag- Strut	Custom steel strut (32k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Re-entrant corners	Custom drag-strut (7k load) at the corners of the core area	4	Ea.	$10,000	$40,000	
New Roofing	Add new tile roofing to match existing	6600	S.F.	$30	$198,000	
Strengthen Bell Tower	Temporary work area	828	S.F.	$2	$1,242	
	Out of plane ties @ 4' o.c.	30	Ea.	$400	$11,800	
	Install straps at E&W walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between rafters@ anchors	118	L.F.	$25	$2,950	
	Add perimeter angle and anchors in masonry	118	L.F.	$223	$26,314	
	Re-nail the plywood floor sheathing or add as req'd	828	S.F.	$6	$4,968	
	Replace the floor finishes	828	S.F.	$5	$4,140	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
Strengthen Bell Lounge Level URM Walls	Demo (E) wall finishes	1613	S.F.	$4	$6,452	
	Build 23' x 13.67 bonded shear wall	629	S.F.	$25	$15,721	
	Build 36' x 13.67 bonded shear wall	984	S.F.	$25	$24,606	
	(N) gypsum and finish	1613	S.F.	$8	$12,904	
						$763,453
Strengthen 4th Floor Core Area	Remove the contents from the floor	2368	S.F.	$10	$23,680	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes (along E&W walls)	2368	S.F.	$3	$7,104	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at N&S walls for subdiaphragm	32	Ea.	$1,000	$32,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Extend plywood sheathing to the N&S	3392	S.F.	$6	$20,352	
	Replace the floor finishes	2368	S.F.	$5	$11,840	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard truss)	26	Ea.	$1,000	$26,000	
						$207,496
Strengthen 3rd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing	3392	S.F.	$3	$10,176	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (13 per 2nd inboard beam)	26	Ea.	$1,000	$26,000	
Re-entrant corners	Custom drag-strut (11k load) at the corners of the core area	4	Ea.	$15,000	$60,000	
Add independent HSS column supports	Demo (E) 3FL wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Install HSS 4 x 4 column from truss to 3FL beam	12	Ea.	$1,500	$18,000	
	(N) gypsum and finish	480	S.F.	$8	$3,840	
GL 2 URM Strengthening	Demo (E) wall finishes	567	S.F.	$4	$2,268	
	Demo adjoining (E) walls and finishes	567	S.F.	$6	$3,402	
	Install shear anchors at URM wall @ 3 SF o.c.	189	Ea.	$100	$18,900	
	Build 63' x 9' x 8" shotcrete wall	1	Ea.	$21,000	$21,000	
	Re-build adjoining walls	567	S.F.	$6	$3,402	
	(N) gypsum and finish	1134	S.F.	$8	$9,072	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 7 URM Strengthening	Demo (E) wall finishes	567	S.F.	$4	$2,268	
	Demo adjoining (E) walls and finishes	567	S.F.	$6	$3,402	
	Install shear anchors at URM wall @ 3 SF o.c.	189	Ea.	$100	$18,900	
	Build 63' x 9' x 8" shotcrete wall	1	Ea.	$21,000	$21,000	
	Re-build adjoining walls	567	S.F.	$6	$3,402	
	(N) gypsum and finish	1134	S.F.	$8	$9,072	
Strengthen 3rd Floor East Wing Area	Remove contents from the URM walls (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Drag- Strut	Custom steel strut (49k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$45,000	$90,000	
Floor Opening Strengthening	Custom steel girt (10' span) for OOP support of the E URM wall at the stairway	1	Ea.	$1,440	$1,440	
	Custom steel girt (18' span) for OOP support of the N URM wall at the stairway	1	Ea.	$2,592	$2,592	
	Install straps at ea. end of the stairway opening	4	Ea.	$500	$2,000	
GL 3 URM Strengthening	Demo (E) wall finishes	328	S.F.	$4	$1,311	
	Install shear anchors at URM wall @ 3 SF o.c.	109	Ea.	$100	$10,925	
	Build 23' x 14.25' x 10" shotcrete wall	1	Ea.	$22,760	$22,760	
	(N) gypsum and finish	328	S.F.	$8	$2,622	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 6 URM Strengthening	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Demo (E) wall finishes	328	S.F.	$4	$1,311	
	Install shear anchors at URM wall @ 3 SF o.c.	109	Ea.	$100	$10,925	
	Build 23' x 14.25' x 10" shotcrete wall	1	Ea.	$22,760	$22,760	
	(N) gypsum and finish	328	S.F.	$8	$2,622	
GL C URM Strengthening	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Demo (E) wall finishes	670	S.F.	$4	$2,679	
	Install shear anchors at URM wall @ 3 SF o.c.	223	Ea.	$100	$22,325	
	Build 23' x 14.25' x 10" shotcrete wall	2	Ea.	$23,255	$46,510	
	(N) gypsum and finish	670	S.F.	$8	$5,358	
Strengthen 3rd Floor West Wing Area	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	
	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	792	S.F.	$5	$3,960	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
GL B URM Strengthening	Demo (E) wall finishes	656	S.F.	$4	$2,622	
	Install shear anchors at URM wall @ 3 SF o.c.	219	Ea.	$100	$21,850	
	Build 23' x 14.25' x 10" shotcrete wall	2	Ea.	$22,760	$45,521	
	(N) gypsum and finish	656	S.F.	$8	$5,244	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	
						$961,230
Strengthen 2nd Floor Core Area	Remove the contents from the floor	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing or add as req'd	3392	S.F.	$6	$20,352	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	
Re-entrant corners	Custom drag-strut (17k load) at the corners of the core area	4	Ea.	$25,000	$100,000	
Add independent HSS column supports	Demo (E) 2FL wall finishes at (12) locations	720	S.F.	$4	$2,880	
	Install HSS 4 x 4 column from 3FL to 2FL beam	12	Ea.	$1,800	$21,600	
	(N) gypsum and finish	720	S.F.	$12	$8,640	
GL 2 URM Strengthening	Demo (E) wall finishes	945	S.F.	$4	$3,780	
	Demo adjoining (E) walls and finishes	945	S.F.	$6	$5,670	
	Install shear anchors at URM wall @ 3 SF o.c.	315	Ea.	$100	$31,500	
	Build 63' x 15' x 8" shotcrete wall	1	Ea.	$35,000	$35,000	
	Re-build adjoining walls	945	S.F.	$6	$5,670	
	(N) gypsum and finish	1890	S.F.	$8	$15,120	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
GL 7 URM Strengthening	Demo (E) wall finishes	945	S.F.	$4	$3,780	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Demo adjoining (E) walls and finishes	945	S.F.	$6	$5,670	
	Install shear anchors at URM wall @ 3 SF o.c.	315	Ea.	$100	$31,500	
	Build 63' x 15' x 8" shotcrete wall	1	Ea.	$35,000	$35,000	
	Re-build adjoining walls	945	S.F.	$6	$5,670	
	(N) gypsum and finish	1890	S.F.	$8	$15,120	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
Strengthen 2nd Floor East Wing Area	Remove contents around the perimeter (4' wide)	1000	S.F.	$10	$10,000	
	Temporary work area	1000	S.F.	$2	$1,500	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Floor Opening Strengthening	Custom steel girt (14' span) for OOP support of the N/S URM wall at the stairs	2	Ea.	$2,016	$4,032	
	Custom steel girt (20' span) for OOP support of the E URM wall at the	1	Ea.	$2,880	$2,880	
	Install straps at ea. End of the stairway opening	6	Ea.	$500	$3,000	
Drag- Strut	Custom steel strut (77k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$75,000	$150,000	
GL 3 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 12" shotcrete wall	1	Ea.	$38,333	$38,333	
	(N) gypsum and finish	345	S.F.	$8	$2,760	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 6 URM Strengthening	Demo (E) wall finishes	345	S.F.	$4	$1,380	
	Install shear anchors at URM wall @ 3 SF o.c.	115	Ea.	$100	$11,500	
	Build 23' x 15' x 12" shotcrete wall	1	Ea.	$38,333	$38,333	
	(N) gypsum and finish	345	S.F.	$12	$4,140	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL C URM Strengthening	Demo (E) wall finishes	690	S.F.	$4	$2,760	
	Install shear anchors at URM wall @ 3 SF o.c.	230	Ea.	$100	$23,000	
	Build 23' x 15' x 12" shotcrete wall	2	Ea.	$38,333	$76,667	
	(N) gypsum and finish	690	S.F.	$8	$5,520	
	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	
GL D URM Strengthening	Demo (E) wall finishes	705	S.F.	$4	$2,820	
	Install shear anchors at URM wall @ 3 SF o.c.	235	Ea.	$100	$23,500	
	Build 23.5' x 15' x 8" shotcrete wall	2	Ea.	$13,056	$26,111	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	785	S.F.	$8	$6,280	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
GL E URM Strengthening	Demo (E) wall finishes	300	S.F.	$4	$1,200	
	Install shear anchors at URM wall @ 3 SF o.c.	100	Ea.	$100	$10,000	
	Build 20' x 15' x 8" shotcrete wall	1	Ea.	$11,111	$11,111	
	(N) gypsum and finish	300	S.F.	$8	$2,400	
	Re-build 3' tall wainscotting	60	S.F.	$22	$1,320	
Strengthen 2nd Floor West Wing Area	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	
	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$223	$44,154	
	Replace the floor finishes	198	S.F.	$5	$990	
Floor Opening Strengthening	Custom steel girt (9' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,296	$1,296	
	Install straps at ea. End of the stairway opening	2	Ea.	$500	$1,000	
GL B URM Strengthening	Demo (E) wall finishes	690	S.F.	$4	$2,760	
	Install shear anchors at URM wall @ 3 SF o.c.	230	Ea.	$100	$23,000	
	Build 23' x 15' x 12" shotcrete wall	2	Ea.	$38,333	$76,667	
	(N) gypsum and finish	690	S.F.	$8	$5,520	
	Re-build 3' tall wainscotting	138	S.F.	$22	$3,036	
						$1,368,422
Strengthen 1st Floor Core Area	Remove contents around the perimeter	3392	S.F.	$10	$33,920	
	Temporary work area	3392	S.F.	$2	$5,088	
	Remove (E) floor finishes	3392	S.F.	$3	$10,176	
	Out of plane ties @ 4' o.c.	59	Ea.	$400	$23,400	
	Install straps at E&W walls for subdiaphragm	28	Ea.	$1,000	$28,000	
	Add Blocking between rafters@ anchors	234	L.F.	$25	$5,850	
	Add perimeter angle and anchors in masonry	234	L.F.	$223	$52,182	
	Re-nail the plywood floor sheathing or add as req'd	3392	S.F.	$6	$20,352	
	Replace the floor finishes	3392	S.F.	$5	$16,960	
Cross ties	Install steel PLs on each side of the joists splices (1 lap per joist at midspan)	16	Ea.	$1,000	$16,000	
Re-entrant corners	Custom drag-strut (12k load) at the corners of the core area	4	Ea.	$18,000	$72,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Add independent HSS column supports	Demo (E) 1FL wall finishes at (12) locations	792	S.F.	$4	$3,168	
	Install HSS 4 x 4 column from 2FL to 1FL beam	12	Ea.	$1,980	$23,760	
	(N) gypsum and finish	792	S.F.	$8	$6,336	
GL 1 URM Strengthening	Demo (E) wall finishes	660	S.F.	$4	$2,640	
	Install shear anchors at URM wall @ 3 SF o.c.	220	Ea.	$100	$22,000	
	Build 20' x 16.5' x 8" shotcrete wall	2	Ea.	$12,222	$24,444	
	(N) gypsum and finish	660	S.F.	$8	$5,280	
	Re-build 3' tall wainscotting	120	S.F.	$22	$2,640	
GL 2 URM Strengthening	Demo (E) wall finishes	1040	S.F.	$4	$4,158	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Install shear anchors at URM wall @ 3 SF o.c.	347	Ea.	$100	$34,650	
	Build 63' x 16.5' x 10" shotcrete wall	1	Ea.	$72,188	$72,188	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	2079	S.F.	$8	$16,632	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
GL 7 URM Strengthening	Demo (E) wall finishes	1040	S.F.	$4	$4,158	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Install shear anchors at URM wall @ 3 SF o.c.	347	Ea.	$100	$34,650	
	Build 63' x 16.5' x 10" shotcrete wall	1	Ea.	$72,188	$72,188	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	2079	S.F.	$12	$24,948	
	Re-build 3' tall wainscotting	189	S.F.	$22	$4,158	
GL 8 URM Strengthening	Demo (E) wall finishes	660	S.F.	$4	$2,640	
	Install shear anchors at URM wall @ 3 SF o.c.	220	Ea.	$100	$22,000	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Strengthen 1st Floor East Wing Area	Build 20' x 16.5' x 8" shotcrete wall	2	Ea.	$12,222	$24,444	
	(N) gypsum and finish	660	S.F.	$8	$5,280	
	Re-build 3' tall wainscotting	120	S.F.	$22	$2,640	
	Remove contents around the perimeter (4' wide)	1898	S.F.	$10	$18,980	
	Temporary work area	1898	S.F.	$2	$2,847	
	Remove (E) floor finishes	1000	S.F.	$3	$3,000	
	Out of plane ties @ 4' o.c.	63	Ea.	$400	$25,000	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	250	L.F.	$25	$6,250	
	Add perimeter angle and anchors in masonry	250	L.F.	$223	$55,750	
	Replace the floor finishes	1000	S.F.	$5	$5,000	
Floor Opening Strengthening	Custom steel girt (12' span) for OOP support of the URM wall at the stairway	1	Ea.	$1,728	$1,728	
	Install straps at ea. end of the stairway opening	2	Ea.	$500	$1,000	
Drag- Strut	Custom steel strut (33k load) at GL 3 & 6 64' to west wing with URM wall anchor at east wing	2	Ea.	$30,000	$60,000	
	Remove contents from drag strut path (4' wide)	256	S.F.	$10	$2,560	
	Temporary work area	256	S.F.	$2	$384	
	Remove (E) floor finishes	256	S.F.	$3	$768	
	Replace the floor finishes	256	S.F.	$5	$1,280	
GL 3 URM Strengthening	Demo (E) wall finishes	380	S.F.	$4	$1,518	
	Install shear anchors at URM wall @ 3 SF o.c.	127	Ea.	$100	$12,650	
	Build 23' x 16.5' x 12" shotcrete wall	1	Ea.	$52,708	$52,708	
	(N) gypsum and finish	380	S.F.	$8	$3,036	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
GL 6 URM Strengthening	Demo (E) wall finishes	380	S.F.	$4	$1,518	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL C URM Strengthening	Install shear anchors at URM wall @ 3 SF o.c.	127	Ea.	$100	$12,650	
	Build 23' x 16.5' x 12" shotcrete wall	1	Ea.	$52,708	$52,708	
	(N) gypsum and finish	380	S.F.	$12	$4,554	
	Re-build 3' tall wainscotting	69	S.F.	$22	$1,518	
	Demo (E) wall finishes	776	S.F.	$4	$3,102	
GL D URM Strengthening	Install shear anchors at URM wall @ 3 SF o.c.	259	Ea.	$100	$25,850	
	Build 23.5' x 16.5' x 12" shotcrete wall	2	Ea.	$53,854	$107,708	
	Custom drag-strut (42k load) tieing new walls	1	Ea.	$45,000	$45,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	856	S.F.	$8	$6,844	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
	Demo (E) wall finishes	776	S.F.	$4	$3,102	
GL E URM Strengthening	Install shear anchors at URM wall @ 3 SF o.c.	259	Ea.	$100	$25,850	
	Build 23.5' x 16.5' x 8" shotcrete wall	2	Ea.	$21,542	$43,083	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	856	S.F.	$8	$6,844	
	Re-build 3' tall wainscotting	141	S.F.	$22	$3,102	
	Demo (E) wall finishes	330	S.F.	$4	$1,320	
Strengthen 1st Floor West Wing Area	Install shear anchors at URM wall @ 3 SF o.c.	110	Ea.	$100	$11,000	
	Build 20' x 16.5' x 8" shotcrete wall	1	Ea.	$18,333	$18,333	
	(N) gypsum and finish	330	S.F.	$8	$2,640	
	Re-build 3' tall wainscotting	60	S.F.	$22	$1,320	
	Remove contents around the perimeter (4' wide)	792	S.F.	$10	$7,920	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL B URM Strengthening	Temporary work area	792	S.F.	$2	$1,188	
	Remove (E) floor finishes	792	S.F.	$3	$2,376	
	Out of plane ties @ 4' o.c.	50	Ea.	$400	$19,800	
	Install straps at N&S walls for subdiaphragm	12	Ea.	$1,000	$12,000	
	Add Blocking between joist @ anchors	198	L.F.	$25	$4,950	
	Add perimeter angle and anchors in masonry	198	L.F.	$50	$9,900	
	Replace the floor finishes	198	S.F.	$5	$990	
	Demo (E) wall finishes	974	S.F.	$4	$3,894	
	Install shear anchors at URM wall @ 3 SF o.c.	325	Ea.	$100	$32,450	
	Build 29.5' x 16.5' x 12" shotcrete wall	2	Ea.	$67,604	$135,208	
	Custom drag-strut (17k load) tieing new walls	1	Ea.	$25,000	$25,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	1054	S.F.	$8	$8,428	
	Re-build 3' tall wainscotting	177	S.F.	$22	$3,894	
						$1,671,121
Strengthen Basement Area Add independent HSS column supports	Temporary work area for HSS columns	960	S.F.	$2	$1,440	
	Demo (E) basement wall finishes at (12) locations	480	S.F.	$4	$1,920	
	Demo (E) slab for new footings (12) locations	480	S.F.	$10	$4,800	
	Excavate for new footings (12) locations	192	S.F.	$30	$5,760	
	Add concrete foundation (12) locations	192	S.F.	$100	$19,200	
	Install HSS 4 x 4 column from 1FL to footing	12	Ea.	$2,700	$32,400	
	(N) gypsum and finish	480	S.F.	$8	$3,840	
GL 1 URM Strengthening	Temporary work area	240	S.F.	$2	$360	
	Demo (E) wall finishes	400	S.F.	$4	$1,600	
	Demo (E) slab for new footings	160	S.F.	$10	$1,600	
	Excavate for new footings	160	S.F.	$40	$6,400	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 2 URM Strengthening	Install (1) micropiles @ ea. end of the (N) SW	2	Ea.	$18,000	$36,000	
	Add concrete foundation	160	S.F.	$100	$16,000	
	Install shear anchors at URM wall @ 3 SF o.c.	133	Ea.	$100	$13,333	
	Build 20' x 10' x 10" shotcrete wall	1	Ea.	$13,889	$13,889	
	(N) gypsum and finish	400	S.F.	$8	$3,200	
	Temporary work area	378	S.F.	$2	$567	
	Demo (E) wall finishes	630	S.F.	$4	$2,520	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Demo (E) slab for new footings	252	S.F.	$10	$2,520	
	Excavate for new footings	252	S.F.	$40	$10,080	
GL 3 URM Strengthening	Install (3) micropiles @ ea. end of the (N) SW	1	Ea.	$48,000	$48,000	
	Add concrete foundation	252	S.F.	$100	$25,200	
	Install shear anchors at URM wall @ 3 SF o.c.	210	Ea.	$100	$21,000	
	Build 63' x 10' x 15" shotcrete wall	1	Ea.	$87,500	$87,500	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	1670	S.F.	$8	$13,356	
	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	
	Excavate for new footings	92	S.F.	$40	$3,680	
GL 6 URM Strengthening	Install (3) 500k micropiles @ ea. end of the (N) SW	1	Ea.	$72,000	$72,000	
	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 18" shotcrete wall	1	Ea.	$57,500	$57,500	
	(N) gypsum and finish	230	S.F.	$8	$1,840	
	Temporary work area	138	S.F.	$2	$207	
	Demo (E) wall finishes	230	S.F.	$4	$920	
	Demo (E) slab for new footings	92	S.F.	$10	$920	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL 7 URM Strengthening	Excavate for new footings	92	S.F.	$40	$3,680	
	Install (3) 500k micropiles @ ea. end of the (N) SW	1	Ea.	$72,000	$72,000	
	Add concrete foundation	92	S.F.	$100	$9,200	
	Install shear anchors at URM wall @ 3 SF o.c.	77	Ea.	$100	$7,667	
	Build 23' x 10' x 18" shotcrete wall	1	Ea.	$57,500	$57,500	
	(N) gypsum and finish	230	S.F.	$8	$1,840	
	Temporary work area	378	S.F.	$2	$567	
	Demo adjoining (E) walls and finishes	1040	S.F.	$6	$6,237	
	Demo (E) wall finishes	630	S.F.	$4	$2,520	
	Demo (E) slab for new footings	252	S.F.	$10	$2,520	
GL 8 URM Strengthening	Excavate for new footings	252	S.F.	$40	$10,080	
	Install (3) micropiles @ ea. end of the (N) SW	1	Ea.	$48,000	$48,000	
	Add concrete foundation	252	S.F.	$100	$25,200	
	Install shear anchors at URM wall @ 3 SF o.c.	210	Ea.	$100	$21,000	
	Build 63' x 10' x 15" shotcrete wall	1	Ea.	$87,500	$87,500	
	Re-build adjoining walls	1040	S.F.	$6	$6,237	
	(N) gypsum and finish	1670	S.F.	$8	$13,356	
	Temporary work area	240	S.F.	$2	$360	
	Demo (E) wall finishes	400	S.F.	$4	$1,600	
	Demo (E) slab for new footings	160	S.F.	$10	$1,600	
GL A URM Strengthening	Excavate for new footings	160	S.F.	$40	$6,400	
	Install (1) micropiles @ ea. end of the (N) SW	2	Ea.	$18,000	$36,000	
	Add concrete foundation	160	S.F.	$100	$16,000	
	Install shear anchors at URM wall @ 3 SF o.c.	133	Ea.	$100	$13,333	
	Build 20' x 10' x 10" shotcrete wall	1	Ea.	$13,889	$13,889	
	(N) gypsum and finish	400	S.F.	$8	$3,200	
	Temporary work area	354	S.F.	$2	$531	
	Demo (E) wall finishes	590	S.F.	$4	$2,360	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL B URM Strengthening	Demo (E) slab for new footings	236	S.F.	$10	$2,360	
	Excavate for new footings	236	S.F.	$40	$9,440	
	Install (1) micropiles @ ea. end of the (N) SW	2	Ea.	$18,000	$36,000	
	Add concrete foundation	236	S.F.	$100	$23,600	
	Install shear anchors at URM wall @ 3 SF o.c.	197	Ea.	$100	$19,667	
	Build 29.5' x 10' x 10" shotcrete wall	2	Ea.	$20,486	$40,972	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	32	S.F.	$4	$128	
	(N) gypsum and finish	622	S.F.	$8	$4,976	
	Temporary work area	354	S.F.	$2	$531	
GL C URM Strengthening	Demo (E) wall finishes	590	S.F.	$4	$2,360	
	Demo (E) slab for new footings	236	S.F.	$10	$2,360	
	Excavate for new footings	236	S.F.	$40	$9,440	
	Install (3) 500k micropiles @ ea. end of the (N) SW	2	Ea.	$60,000	$120,000	
	Add concrete foundation	236	S.F.	$100	$23,600	
	Install shear anchors at URM wall @ 3 SF o.c.	197	Ea.	$100	$19,667	
	Build 29.5' x 10' x 18" (avg thick) shotcrete wall	2	Ea.	$73,750	$147,500	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	32	S.F.	$4	$128	
	(N) gypsum and finish	622	S.F.	$8	$4,976	
	Temporary work area	282	S.F.	$2	$423	
	Demo (E) wall finishes	470	S.F.	$4	$1,880	
	Demo (E) slab for new footings	188	S.F.	$10	$1,880	
	Excavate for new footings	188	S.F.	$40	$7,520	
	Install (2) 500k micropiles @ ea. end of the (N) SW	2	Ea.	$60,000	$120,000	
	Add concrete foundation	188	S.F.	$100	$18,800	
	Install shear anchors at URM wall @ 3 SF o.c.	157	Ea.	$100	$15,667	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
GL D URM Strengthening	Build 23.5' x 10' x 18" (avg thick) shotcrete wall	2	Ea.	$58,750	$117,500	
	Custom drag-strut (18k load) tieing new walls	1	Ea.	$26,000	$26,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	(N) gypsum and finish	550	S.F.	$8	$4,400	
	Temporary work area	276	S.F.	$2	$414	
	Demo adjoining (E) walls and finishes	920	S.F.	$6	$5,520	
	Demo (E) wall finishes	460	S.F.	$4	$1,840	
	Demo (E) slab for new footings	184	S.F.	$10	$1,840	
	Excavate for new footings	184	S.F.	$40	$7,360	
	Install (2) micropiles @ ea. end of the (N) SW	2	Ea.	$36,000	$72,000	
	Add concrete foundation	184	S.F.	$100	$18,400	
	Install shear anchors at URM wall @ 3 SF o.c.	153	Ea.	$100	$15,333	
GL E URM Strengthening	Build 23.5' x 10' x 12" (avg thick) shotcrete wall	2	Ea.	$25,556	$51,111	
	Custom drag-strut (7k load) tieing new walls	1	Ea.	$10,000	$10,000	
	Demo (E) wall finishes at drag strut location	80	S.F.	$4	$320	
	Re-build adjoining walls	460	S.F.	$6	$2,760	
	(N) gypsum and finish	1460	S.F.	$8	$11,680	
	Temporary work area	120	S.F.	$2	$180	
	Demo adjoining (E) walls and finishes	200	S.F.	$6	$1,200	
	Demo (E) wall finishes	200	S.F.	$4	$800	
	Demo (E) slab for new footings	80	S.F.	$10	$800	
	Excavate for new footings	80	S.F.	$40	$3,200	
	Install (2) micropiles @ ea. end of the (N) SW	1	Ea.	$36,000	$36,000	
	Add concrete foundation	80	S.F.	$100	$8,000	
	Install shear anchors at URM wall @ 3 SF o.c.	67	Ea.	$100	$6,667	
	Build 20' x 10' x 12" (avg thick) shotcrete wall	1	Ea.	$22,222	$22,222	
	Re-build adjoining walls	200	S.F.	$6	$1,200	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
	(N) gypsum and finish	400	S.F.	$8	$3,200	
						$2,095,458
					Sub-total	**$7,714,131**
					Construction Contingency (25%)	**$1,928,533**
					Contractor Overhead and Profit (25%)	**$2,410,666**
					Total Seismic Construction Cost	**$12,053,329**

ESTIMATE OF PROBABLE CONSTRUCTION COST
NON-STRUCTURAL

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - SPOE		
CLIENT:	Benton County	DATE:	11-Aug-16
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Mobilization	Job Trailer	2	Month	$500	$1,000	
	Supervisor	9	Week	$4,125	$37,125	
Construction Aids						
Lift 500# capacity	(1) Scissor lift	2	Month	$4,300	$8,600	
Temporary Storage	Provide (1) 40' container for storage as required	2	Month	$500	$1,000	
Ventalation	Negative Air Equipment per each zone	9	Week	$500	$4,500	
						$52,225
Fire suppresion piping	Brace to NFPA standards	120	Each	$300	$36,000	
	Add Flex Connection	30	Each	$430	$12,900	
Gas piping	Brace to FM standards	6	Each	$300	$1,800	
	Add Flex Connection	2	Each	$430	$860	
Brace Courtroom #3 suspended ceiling	Remove contents	540.5	S.F.	$10	$5,405	
	Demo (E) ceiling	540.5	S.F.	$6	$3,243	
	Temporary work area	540.5	S.F.	$2	$811	
	Temporary flooring	540.5	S.F.	$2	$1,081	
	Add seismic bracing	540.5	S.F.	$16	$8,648	
	(N) metal suspended ceiling	540.5	S.F.	$5	$2,703	
	(N) gypsum and finish	540.5	S.F.	$12	$6,486	
Brace Room #111 suspended ceiling	Remove contents	310	S.F.	$10	$3,100	
	Demo (E) ceiling	310	S.F.	$6	$1,860	
	Temporary work area	310	S.F.	$2	$465	
	Temporary flooring	310	S.F.	$2	$620	
	Add seismic bracing	310	S.F.	$16	$4,960	
	(N) metal suspended ceiling	310	S.F.	$5	$1,550	
	(N) gypsum and finish	310	S.F.	$12	$3,720	
Brace Courtroom #3 suspended lights	Add independent braces/safety chains	5	Each	$300	$1,500	
Secure lens covers for Courtroom #3 suspended lights	Add clips to secure the lens covers of the lights	5	Each	$150	$750	

ESTIMATE OF PROBABLE CONSTRUCTION COST
NON-STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Brace NC tall contents on 1FL and 3FL	Add independent braces/safety chains	29	Each	$200	$5,800	
Brace suspended inline HVAC equip.	Add independent braces/safety chains	5	Each	$1,000	$5,000	
Anchor the gas boiler in the boiler room	Add concrete anchors at each corner	1	Each	$500	$500	
						$109,761
					Sub-total	**$161,986**
					Construction Contingency (25%)	**$40,497**
					Contractor Overhead and Profit (25%)	**$50,621**
					Total Seismic Construction Cost	**$253,104**

NEW BUILDING COST TABLE

ESTIMATE OF PROBABLE CONSTRUCTION COST
NEW COURTHOUSE BUILDING

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	151029
LOCATION:	Benton County Courthouse - SPOE		
CLIENT:	Benton County	DATE:	11-Aug-16
TAKE-OFF BY:	RGV	CHECKED BY:	ERW

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
General	Building Permit Fee	1	Each	$140,000	$140,000	
						$140,000
Base Building	Steel frame building w/ stone or brick veneer	45000	S.F.	$240	$10,795,500	
						$10,795,500
Common Additives						
Floor height addition	Increase 12' to 16' ceilings (2 floors)	8	L.F.	$148,500	$1,188,000	
Benches Hardwood	2 Courtrooms	96	L.F.	$234	$22,464	
Closed circuit Surveillance	1 Station	1	Each	$20,600	$20,600	
Directory boards	2 Locations	2	Each	$1,075	$2,150	
Elevators	1 Additional elevators	1	Each	$158,520	$158,520	
Emergency lighting	With nickel cadmium batteries	24	Each	$685	$16,440	
Flagpole	70 Foot flagpole	1	Each	$12,000	$12,000	
Intercom	12 Station and master	1	Each	$800	$800	
Safe	60" x 36" X 18" double door safe	1	Each	$9,350	$9,350	
Smoke detectors	Duct type smoke detectors	36	Each	$565	$20,340	
Clock system	20 Room clock system	1	Each	$20,600	$20,600	
						$1,471,264
					Sub-total	**$12,406,764**
					Construction Contingency (25%)	**$3,101,691**
					Contractor Overhead and Profit (25%) (included in the sub-total above)	**$3,101,691**
					Total Seismic Construction Cost	**$15,508,455**

APPENDIX A
ASCE 41 CHECKLIST

SUMMARY DATA SHEET

BUILDING DATA

Building Name: Benton County Courthouse Date: August 29, 2016

Building Address: 120 NW 4th Street, Corvallis, Oregon

Latitude: 44.56528 N Longitude: 123.26237 W By:

Year Built: 1887 Year(s) Remodeled: 1977 Original Design Code: N/A

Area (sf): 33,052 Length (ft): Varies Width (ft): Varies

No. of Stories: Four Story Height: Varies Total Height: 59'-6"(Tower:115'-5")

USE ☐ Industrial ☒ Office ☐ Warehouse ☐ Hospital ☐ Residential ☐ Educational ☒ Other: Assembly

CONSTRUCTION DATA

Gravity Load Structural System: Unreinforced masonry bearing walls with interior steel supports

Exterior Transverse Walls: Unreinforced masonry shear walls Openings? Punched openings

Exterior Longitudinal Walls: Unreinforced masonry shear walls Openings? Punched openings

Roof Materials/Framing: Timber trusses and 2x rafters with T&G straight sheathing

Intermediate Floors/Framing: Timber 3 x 16 floor joists with T&G straight sheathing

Ground Floor: Concrete slab on grade

Columns: Wood and steel columns Foundation: Unknown foundation

General Condition of Structure: Well maintained structure in good condition

Levels Below Grade? N/A

Special Features and Comments: The building houses the Benton County Courts

LATERAL-FORCE-RESISTING SYSTEM

	Longitudinal	Transverse
System:	Masonry shear wall	Masonry shear wall
Vertical Elements:	Bearing wall	Bearing wall
Diaphragms:	Flexible wood sheathing	Flexible wood sheathing
Connections:	Unknown	Unknown

EVALUATION DATA

BSE-1N Spectral Response Accelerations: S_{Ds} = 0.712 S_{D1} = 0.480

Soil Factors: Class = D F_a = 1.12 F_v = 1.53

BSE-1E Spectral Response Accelerations: S_{XS} = 0.86 S_{X1} = 0.58

Level of Seismicity: High Performance Level: Life Safety w/ BSE-2E

Building Period: T = 0.43

Spectral Acceleration: S_a = 0.86

Modification Factor: $C_mC_1C_2$ = 1.0 Building Weight: W = 7,344,016 lb

Pseudo Lateral Force: V= 0.86

$C_mC_1C_2S_aW$= 6,298,375 lb

BUILDING CLASSIFICATION: Unreinforced Masonry Building with flexible diaphragms

REQUIRED TIER 1 CHECKLISTS	**Yes**	**No**
Basic Configuration Checklist	☒	☐
Building Type URM **Structural Checklist**	☒	☐
Nonstructural Component Checklist	☒	☐

FURTHER EVALUATION REQUIREMENT:

Table 2-1. Basic Performance Objective for Existing Buildings (BPOE)

	Tier 1[a]	Tier 2[a]	Tier 3	
Risk Category	**BSE-1E**	**BSE-1E**	**BSE-1E**	**BSE-2E**
I & II	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Life Safety Structural Performance Life Safety Nonstructural Performance (3-C)	Collapse Prevention Structural Performance Nonstructural Performance Not Considered (5-D)
III	See footnote *b* for Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Damage Control Structural Performance Position Retention Nonstructural Performance (2-B)	Limited Safety Structural Performance Nonstructural Performance Not Considered (4-D)
IV	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Immediate Occupancy Structural Performance Position Retention Nonstructural Performance (1-B)	Life Safety Structural Performance Nonstructural Performance Not Considered (3-D)

[a]For Tier 1 and 2 assessments, seismic performance for the BSE-2E is not explicitly evaluated.
[b]For Risk Category III, the Tier 1 screening checklists shall be based on the Life Safety Performance Level (S-3), except that checklist statements using the Quick Check procedures of Section 4.5.3 shall be based on MS-factors and other limits that are an average of the values for Life Safety and Immediate Occupancy.

Per USGS Sds = 0.712, so the courthouse site is considered in a region with high level of seismicity.

Table 2-5. Level of Seismicity Definitions

Level of Seismicity[a]	S_{DS}	S_{D1}
Very low	<0.167 g	<0.067 g
Low	≥0.167 g <0.33 g	≥0.067 g <0.133 g
Moderate	≥0.33 g <0.50 g	≥0.133 g <0.20 g
High	≥ 0.50 g	≥0.20 g

[a]The higher level of seismicity defined by S_{DS} or S_{D1} shall govern.

C2.5 LEVEL OF SEISMICITY

The Levels of Seismicity in this standard have been adjusted to match the Seismic Design Categories in ASCE 7 as follows:

SDC A: Very Low
SDC B: Low
SDC C: Moderate
SDC D–F: High

Therefore, the parameters S_{DS} and S_{D1} correspond to the parameters at the BSE-1N level.

The courthouse does not meet the Benchmark Building Criteria

Table 4-6. Benchmark Buildings

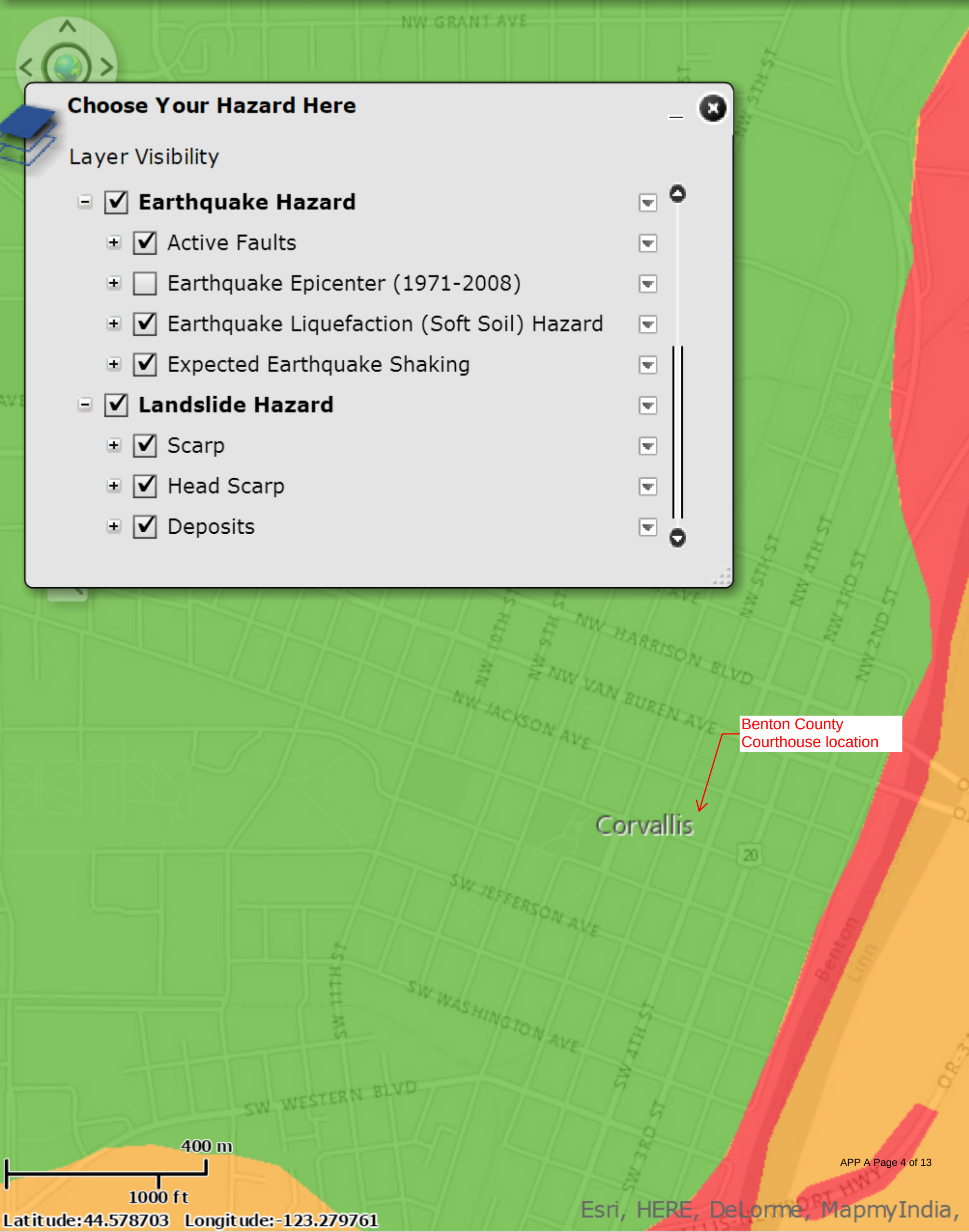
Building Type[a,b]	Building Seismic Design Provisions				Seismic Evaluation or Retrofit Provisions		
	NBC[LS] SBC[LS]	UBC[LS]	IBC[LS]	NEHRP[LS]	FEMA 178[LS]	FEMA 310 (1998e)/ ASCE 31[LS,IO]	FEMA 356 (2000)/ ASCE 41[LS,IO][c,d]
Wood frame, wood shear panels (Types W1 & W2)	1993	1976	2000	1985	[e]	1998	2000
Wood frame, wood shear panels (Type W1a)	[e]	1997	2000	1997	[e]	1998	2000
Steel moment-resisting frame (Types S1 & S1a)	[e]	1994[f]	2000	1997	[e]	1998	2000
Steel concentrically braced frame (Types S2 & S2a)	[e]	1997	2000	[e]	[e]	1998	2000
Steel eccentrically braced frame (Types S2 & S2a)	[e]	1988[f]	2000	1997	[e]	[e]	2000
Buckling-restrained braced frame (Types S2 & S2a)	[e]	[e]	2006	[e]	[e]	[e]	2000
Light metal frame (Type S3)	[e]	[e]	2000	[e]	1992	1998	2000
Steel frame w/ concrete shear walls (Type S4)	1993	1994[?]	2000	1985	[e]	1998	2000
Steel frame with URM infill (Types S5 & S5a)	[e]	[e]	2000	[e]	[e]	1998	2000
Steel plate shear wall (Type S6)	[e]	[e]	2006	[e]	[e]	[e]	2000
Reinforced concrete moment-resisting frame (Type C1)[g]	1993	1994	2000	1997	[e]	1998	2000
Reinforced concrete shear walls (Types C2 & C2a)	1993	1994	2000	1985	[e]	1998	2000
Concrete frame with URM infill (Types C3 & C3a)	[e]	[e]	2000	[e]	[e]	1998	2000
Tilt-up concrete (Types PC1 & PC1a)	[e]	1997	2000	[e]	[e]	1998	2000
Precast concrete frame (Types PC2 & PC2a)	[e]	[e]	2000	[e]	1992	1998	2000
Reinforced masonry (Type RM1)	[e]	1997	2000	[e]	[e]	1998	2000
Reinforced masonry (Type RM2)	1993	1994[?]	2000	1985	[e]	1998	2000
Unreinforced masonry (Type URM)[h]	[e]	1991[i]	2000	[e]	1992	1998	2000
Unreinforced masonry (Type URMa)	[e]	[e]	2000	[e]	[e]	1998	2000
Seismic isolation or passive dissipation	[e]	1991	2000	[e]	[e]	[e]	2000

[a]Building type refers to one of the common building types defined in Table 3-1.
[b]Buildings on hillside sites shall not be considered Benchmark Buildings.
[c]LS: S-3 Structural Performance Level for the BSE-1.
[d]IO: S-1 Structural Performance Level for the BSE-1.
[e]No benchmark year; buildings shall be evaluated using this standard.
[f]Steel moment-resisting frames and eccentrically braced frames with links adjacent to columns shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
[g]Flat slab concrete moment frames shall not be considered Benchmark Buildings.
[h]URM buildings evaluated or retrofitted and shown to be acceptable using Special Procedure (the ABK Methodology, 1984) may be considered benchmark buildings subject to the limitation of Section 15.2.
[i]Refers to the GSREB or its predecessor, the Uniform Code of Building Conservation (UCBC), or its successor, IEBC Appendix Chapter A1.
[LS]Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life Safety Performance Level may be considered Benchmark Buildings.
[IO]Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Immediate Occupancy Performance Level may be considered Benchmark Buildings.

Table 4-7. Checklists Required for a Tier 1 Screening

Level of Seismicity[b]	Level of Building Performance[c]	Required Checklists[a]					
		Very Low Seismicity Checklist (Sec 16.1.1)	Basic Configuration Checklist (Sec. 16.1.2)	Life Safety Checklist (Sec. 16.2LS through 16.15LS)	Immediate Occupancy Checklist (Sec. 16.2IO through 16.15IO)	Life Safety Nonstructural Checklist (Sec. 16.17)	Position Retention Nonstructural Checklist (Sec. 16.17)
Very low	LS	X					
Very low	IO		X		X		X
Low	LS		X	X		X	
Low	IO		X		X		X
Moderate	LS		X	X		X	
Moderate	IO		X		X		X
High	LS		X	X		X	
High	IO		X		X		X

[a]An X designates the checklist that must be completed for a Tier 1 screening as a function of the level of seismicity and level of performance.
[b]Defined in Section 2.5.
[c]LS = Life Safety Performance Level, and IO = Immediate Occupancy Performance Level (defined in Section 2.3.3).


Choose Your Hazard Here
Layer Visibility
Earthquake Hazard
Active Faults
Earthquake Epicenter (1971-2008)
Earthquake Liquefaction (Soft Soil) Hazard
Expected Earthquake Shaking
Landslide Hazard
Scarp
Head Scarp
Deposits
NW GRANT AVE
NW HARRISON BLVD
NW VAN BUREN AVE
NW JACKSON AVE
Corvallis
Benton County Courthouse location
SW JEFFERSON AVE
SW WASHINGTON AVE
SW WESTERN BLVD
SW 11TH ST
SW 4TH ST
SW 3RD ST
NW 2ND
Benton
Linn
20
400 m
1000 ft
Latitude:44.578703 Longitude:-123.279761
Esri, HERE, DeLorme, MapmyIndia,

Project: Benton County Courthouse

Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers

Date: August 29, 2016

16.1.2LS LIFE SAFETY BASIC CONFIGURATION CHECKLIST

Low Seismicity

Building System

General

C NC N/A U LOAD PATH: The structure shall contain a complete, well defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation. (Commentary: Sec. A.2.1.1. Tier 2: Sec. 5.4.1.1)

C NC N/A U ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 4% of the height of the shorter building. This statement shall not apply for the following building types: W1, W1a, and W2. (Commentary: Sec. A.2.1.2. Tier 2: Sec. 5.4.1.2)

C NC N/A U MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure. (Commentary: Sec. A.2.1.3. Tier 2: Sec. 5.4.1.3)

Building Configuration

C NC N/A U WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above. (Commentary: Sec. A2.2.2. Tier 2: Sec. 5.4.2.1)

C NC N/A U SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above. (Commentary: Sec. A.2.2.3. Tier 2: Sec. 5.4.2.2)

C NC N/A U VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation. (Commentary: Sec. A.2.2.4. Tier 2: Sec. 5.4.2.3)

C NC N/A U GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines. (Commentary: Sec. A.2.2.5. Tier 2: Sec. 5.4.2.4)

C NC N/A U MASS: There is no change in effective mass more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered. (Commentary: Sec. A.2.2.6. Tier 2: Sec. 5.4.2.5)

C NC N/A U TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension. (Commentary: Sec. A.2.2.7. Tier 2: Sec. 5.4.2.6)

Moderate Seismicity: Complete the Following Items in Addition to the Items for Low Seismicity.

Geologic Site Hazards

Based on information from the DOGAMI Statewide Geohazard Online Map

C NC N/A U LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance shall not exist in the foundation soils at depths within 50 ft under the building. (Commentary: Sec. A.6.1.1. Tier 2: 5.4.3.1)

C NC N/A U SLOPE FAILURE: The building site is sufficiently remote from potential earthquake-induced slope failures or rockfalls to be unaffected by such failures or is capable of accommodating any predicted movements without failure. (Commentary: Sec. A.6.1.2. Tier 2: 5.4.3.1)

C NC N/A U SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated. (Commentary: Sec. A.6.1.3. Tier 2: 5.4.3.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Foundation Configuration

C NC N/A U OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$. (Commentary: Sec. A.6.2.1. Tier 2: Sec. 5.4.3.3)

C NC N/A U TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C. (Commentary: Sec. A.6.2.2. Tier 2: Sec. 5.4.3.4)

Project: Benton County Courthouse Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers Date: August 29, 2016

16.16LS LIFE SAFETY STRUCTURAL CHECKLIST FOR BUILDING TYPES URM: UNREINFORCED MASONRY BEARING WALLS WITH FLEXIBLE DIAPHRAGMS AND URMA: UNREINFORCED MASONRY BEARING WALLS WITH STIFF DIAPHRAGMS

Low and Moderate Seismicity

Most of the existing shear walls from the basement up to the the 3rd floor are NC (see the calculations for locations)

Seismic-Force-Resisting System

C NC N/A U REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2. (Commentary: Sec. A.3.2.1.1. Tier 2: Sec. 5.5.1.1)

C NC N/A U SHEAR STRESS CHECK: The shear stress in the unreinforced masonry shear walls, calculated using the Quick Check procedure of Section 4.5.3.3, is less than 30 lb/in.^{2} for clay units and 70 lb/in.^{2} for concrete units. (Commentary: Sec. A.3.2.5.1. Tier 2: Sec. 5.5.3.1.1)

Connections

The masonry walls have anchors at 8' o.c. at the ceiling of the upper level; no anchors were observed at the other floors

C NC N/A U WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support are anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 4.5.3.7. (Commentary: Sec. A.5.1.1. Tier 2: Sec. 5.7.1.1)

C NC N/A U WOOD LEDGERS: The connection between the wall panels and the diaphragm does not induce cross-grain bending or tension in the wood ledgers. (Commentary: Sec. A.5.1.2. Tier 2: Sec. 5.7.1.3)

C NC N/A U TRANSFER TO SHEAR WALLS: Diaphragms are connected for transfer of seismic forces to the shear walls. (Commentary: Sec. A.5.2.1. Tier 2: Sec. 5.7.2)

C NC N/A U GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support. (Commentary: Sec. A.5.4.1. Tier 2: Sec. 5.7.4.1)

High Seismicity: Complete the Following Items in Addition to the Items for Low and Moderate Seismicity.

Seismic-Force-Resisting System

C NC N/A U PROPORTIONS: The height-to-thickness ratio of the shear walls at each story is less than the following (Commentary: Sec. A.3.2.5.2. Tier 2: Sec. 5.5.3.1.2):

Top story of multi-story building	9
First story of multi-story building	15
All other conditions	13

C NC N/A U MASONRY LAYUP: Filled collar joints of multi-wythe masonry walls have negligible voids. (Commentary: Sec. A.3.2.5.3. Tier 2: Sec. 5.5.3.4.1)

The bell tower lounge walls are NC

Diaphragms (Stiff or Flexible)

C NC N/A U OPENINGS AT SHEAR WALLS: Diaphragm openings immediately adjacent to the shear walls are less than 25% of the wall length. (Commentary: Sec. A.4.1.4. Tier 2: Sec. 5.6.1.3)

C NC N/A U OPENINGS AT EXTERIOR MASONRY SHEAR WALLS: Diaphragm openings immediately adjacent to exterior masonry shear walls are not greater than 8 ft long. (Commentary: Sec. A.4.1.6. Tier 2: Sec. 5.6.1.3)

Openings are > 8 feet at: 4th GL6; 3rd GL A, E & 6; 2nd GL A, 3 & 6; and 1st GL 3 & 6.

Flexible Diaphragms

The west roof diaphragm has a 2.75 : 1 ratio in the E-W direction

C NC N/A U CROSS TIES: There are continuous cross ties between diaphragm chords. (Commentary: Sec. A.4.1.2. Tier 2: Sec. 5.6.1.2) Cross ties were not observed

C NC N/A U STRAIGHT SHEATHING: All straight sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered. (Commentary: Sec. A.4.2.1. Tier 2: Sec. 5.6.2)

C NC N/A U SPANS: All wood diaphragms with spans greater than 24 ft consist of wood structural panels or diagonal sheathing. (Commentary: Sec. A.4.2.2. Tier 2: Sec. 5.6.2)

The west roof diaphragm has straight sheathing with 71.5 span in the E-W direction

The core floor diaphragm span is 51.5' x 65.5' with a 1.27 : 1 ratio in the N-S direction (Vact > Vall)

C NC N/A U DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft and aspect ratios less than or equal to 4-to-1. (Commentary: Sec. A.4.2.3. Tier 2: Sec. 5.6.2)

C NC N/A U OTHER DIAPHRAGMS: The diaphragm does not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Commentary: Sec. A.4.7.1. Tier 2: Sec. 5.6.5)

Connections

The (E) anchors are installed tight, but the connection from the joist to the diaphragm is unknown

C NC N/A U STIFFNESS OF WALL ANCHORS: Anchors of concrete or masonry walls to wood structural elements are installed taut and are stiff enough to limit the relative movement between the wall and the diaphragm to no greater than 1/8 in. before engagement of the anchors. (Commentary: Sec. A.5.1.4. Tier 2: Sec. 5.7.1.2)

C NC N/A U BEAM, GIRDER, AND TRUSS SUPPORTS: Beams, girders, and trusses supported by unreinforced masonry walls or pilasters have independent secondary columns for support of vertical loads. (Commentary: Sec. A.5.4.5. Tier 2: Sec. 5.7.4.4)

The trusses do not have secondary supports and there is not a Tier 2 path to evaluate the load path

Project: Benton County Courthouse Location: 120 NW 4th Street, Corvallis, OR

Completed by: Miller Consulting Engineers Date: August 29, 2016

16.17 NONSTRUCTURAL CHECKLIST

Life Safety Systems

One transverse brace was observed in Storage Room #10, but does not meet NFPA-13, Sect. 9.3.5.1 bracing requirements

C NC N/A U LS-LMH; PR-LMH. FIRE SUPPRESSION PIPING: Fire suppression piping is anchored and braced in accordance with NFPA-13. (Commentary: Sec. A.7.13.1. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Fire suppression piping has flexible couplings in accordance with NFPA-13. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.4)

C NC N/A U LS-LMH; PR-LMH. EMERGENCY POWER: Equipment used to power or control life safety systems is anchored or braced. (Commentary: Sec. A.7.12.1. Tier 2: Sec. 13.7.7)

C NC N/A U LS-LMH; PR-LMH. STAIR AND SMOKE DUCTS: Stair pressurization and smoke control ducts are braced and have flexible connections at seismic joints. (Commentary: Sec. A.7.14.1. Tier 2: Sec. 13.7.6)

C NC N/A U LS-MH; PR-MH. SPRINKLER CEILING CLEARANCE: Penetrations through panelized ceilings for fire suppression devices provide clearances in accordance with NFPA-13. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.4) Observed 1" clearance per NFPA-13, Sect. 9.3.4.2

C NC N/A U LS-not required; PR-LMH. EMERGENCY LIGHTING: Emergency and egress lighting equipment is anchored or braced. (Commentary: Sec. A.7.3.1. Tier 2: Sec. 13.7.9)

Hazardous Materials

The natural gas line does not have seismic bracing or flexible couplings

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL EQUIPMENT: Equipment mounted on vibration isolators and containing hazardous material is equipped with restraints or snubbers. (Commentary: Sec. A.7.12.2. Tier 2: 13.7.1)

C NC N/A U LS-LMH; PR-LMH. HAZARDOUS MATERIAL STORAGE: Breakable containers that hold hazardous material, including gas cylinders, are restrained by latched doors, shelf lips, wires, or other methods. (Commentary: Sec. A.7.15.1. Tier 2: Sec. 13.8.4)

C NC N/A U LS-MH; PR-MH. HAZARDOUS MATERIAL DISTRIBUTION: Piping or ductwork conveying hazardous materials is braced or otherwise protected from damage that would allow hazardous material release. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. SHUT-OFF VALVES: Piping containing hazardous material, including natural gas, has shut-off valves or other devices to limit spills or leaks. (Commentary: Sec. A.7.13.3. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-LMH; PR-LMH. FLEXIBLE COUPLINGS: Hazardous material ductwork and piping, including natural gas piping, has flexible couplings. (Commentary: Sec. A.7.15.4, Tier 2: Sec.13.7.3 and 13.7.5)

C NC N/A U LS-MH; PR-MH. PIPING OR DUCTS CROSSING SEISMIC JOINTS: Piping or ductwork carrying hazardous material that either crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.13.6. Tier 2: Sec.13.7.3, 13.7.5, and 13.7.6)

Partitions

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY: Unreinforced masonry or hollow-clay tile partitions are braced at a spacing of at most 10 ft in Low or Moderate Seismicity, or at most 6 ft in High Seismicity. (Commentary: Sec. A.7.1.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-LMH; PR-LMH. HEAVY PARTITIONS SUPPORTED BY CEILINGS: The tops of masonry or hollow-clay tile partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-MH; PR-MH. DRIFT: Rigid cementitious partitions are detailed to accommodate the following drift ratios: in steel moment frame, concrete moment frame, and wood frame buildings, 0.02; in other buildings, 0.005. (Commentary A.7.1.2 Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. LIGHT PARTITIONS SUPPORTED BY CEILINGS: The tops of gypsum board partitions are not laterally supported by an integrated ceiling system. (Commentary: Sec. A.7.2.1. Tier 2: Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. STRUCTURAL SEPARATIONS: Partitions that cross structural separations have seismic or control joints. (Commentary: Sec. A.7.1.3. Tier 2. Sec. 13.6.2)

C NC N/A U LS-not required; PR-MH. TOPS: The tops of ceiling-high framed or panelized partitions have lateral bracing to the structure at a spacing equal to or less than 6 ft. (Commentary: Sec. A.7.1.4. Tier 2. Sec. 13.6.2)

Ceilings

The Category C suspended gypsum ceilings are not braced at 144 ft2 and do not have a sufficient gap to allow for relative displacement in accordance with 13.6.4.3

C NC N/A U LS-MH; PR-LMH. SUSPENDED LATH AND PLASTER: Suspended lath and plaster ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-MH; PR-LMH. SUSPENDED GYPSUM BOARD: Suspended gypsum board ceilings have attachments that resist seismic forces for every 12 ft^2 of area. (Commentary: Sec. A.7.2.3. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. INTEGRATED CEILINGS: Integrated suspended ceilings with continuous areas greater than 144 ft^2, and ceilings of smaller areas that are not surrounded by restraining partitions, are laterally restrained at a spacing no greater than 12 ft with members attached to the structure above. Each restraint location has a minimum of four diagonal wires and compression struts, or diagonal members capable of resisting compression. (Commentary: Sec. A.7.2.2. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. EDGE CLEARANCE: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 have clearances from the enclosing wall or partition of at least the following: in Moderate Seismicity, 1/2 in.; in High Seismicity, 3/4 in. (Commentary: Sec. A.7.2.4. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-MH. CONTINUITY ACROSS STRUCTURE JOINTS: The ceiling system does not cross any seismic joint and is not attached to multiple independent structures. (Commentary: Sec. A.7.2.5. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. EDGE SUPPORT: The free edges of integrated suspended ceilings with continuous areas greater than 144 ft^2 are supported by closure angles or channels not less than 2 in. wide. (Commentary: Sec. A.7.2.6. Tier 2: Sec. 13.6.4)

C NC N/A U LS-not required; PR-H. SEISMIC JOINTS: Acoustical tile or lay-in panel ceilings have seismic separation joints such that each continuous portion of the ceiling is no more than 2500 ft^2 and has a ratio of long-to-short dimension no more than 4-to-1. (Commentary: Sec. A.7.2.7. Tier 2: 13.6.4)

Light Fixtures

C NC N/A U LS-MH; PR-MH. INDEPENDENT SUPPORT: Light fixtures that weigh more per square foot than the ceiling they penetrate are supported independent of the grid ceiling suspension system by a minimum of two wires at diagonally opposite corners of each fixture. (Commentary: Sec. A.7.3.2. Tier 2: Sec. 13.6.4 and 13.7.9)

C NC N/A U LS-not required; PR-H. PENDANT SUPPORTS: Light fixtures on pendant supports are attached at a spacing equal to or less than 6 ft and, if rigidly supported, are free to move with the structure to which they are attached without damaging adjoining components. (Commentary: A.7.3.3. Tier 2: Sec. 13.7.9)

C NC N/A U LS-not required; PR-H. LENS COVERS: Lens covers on light fixtures are attached with safety devices. (Commentary: Sec. A.7.3.4. Tier 2: Sec. 13.7.9)

Cladding and Glazing

C NC N/A U LS-MH; PR-MH. CLADDING ANCHORS: Cladding components weighing more than 10 lb/ft^2 are mechanically anchored to the structure at a spacing equal to or less than the following: for Life Safety in Moderate Seismicity, 6 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 ft. (Commentary: Sec. A.7.4.1. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. CLADDING ISOLATION: For steel or concrete moment frame buildings, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicity, 0.02. (Commentary: Sec. A.7.4.3. Tier 2: Section 13.6.1)

The glazing for the original and renovated skylights are laminated wire glass annealed material

C NC N/A U LS-MH; PR-MH. MULTI-STORY PANELS: For multi-story panels attached at more than one floor level, panel connections are detailed to accommodate a story drift ratio of at least the following: for Life Safety in Moderate Seismicity, 0.01; for Life Safety in High Seismicity and for Position Retention in any seismicty, 0.02. (Commentary: Sec. A.7.4.4. Tier 2: Sec. 13.6.1)

C NC N/A U LS-MH; PR-MH. PANEL CONNECTIONS: Cladding panels are anchored out-of-plane with a minimum number of connections for each wall panel, as follows: for Life Safety in Moderate Seismicity, 2 connections; for Life Safety in High Seismicity and for Position Retention in any seismicity, 4 connections. (Commentary: Sec. A.7.4.5. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. BEARING CONNECTIONS: Where bearing connections are used, there is a minimum of two bearing connections for each cladding panel. (Commentary: Sec. A.7.4.6. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. INSERTS: Where concrete cladding components use inserts, the inserts have positive anchorage or are anchored to reinforcing steel. (Commentary: Sec. A.7.4.7. Tier 2: Sec. 13.6.1.4)

C NC N/A U LS-MH; PR-MH. OVERHEAD GLAZING: Glazing panes of any size in curtain walls and individual interior or exterior panes over 16 ft^2 in area are laminated annealed or laminated heat-strengthened glass and are detailed to remain in the frame when cracked. (Commentary: Sec. A.7.4.8: Tier 2: Sec. 13.6.1.5)

Masonry Veneer

C NC N/A U LS-LMH; PR-LMH. TIES: Masonry veneer is connected to the backup with corrosion-resistant ties. There is a minimum of one tie for every 2-2/3 ft^2, and the ties have spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 36 in.; for Life Safety in High Seismicity and for Position Retention in any seismicity, 24 in. (Commentary: Sec. A.7.5.1. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. SHELF ANGLES: Masonry veneer is supported by shelf angles or other elements at each floor above the ground floor. (Commentary: Sec. A.7.5.2. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. WEAKENED PLANES: Masonry veneer is anchored to the backup adjacent to weakened planes, such as at the locations of flashing. (Commentary: Sec. A.7.5.3. Tier 2: Sec. 13.6.1.2)

C NC N/A U LS-LMH; PR-LMH. UNREINFORCED MASONRY BACKUP: There is no unreinforced masonry backup. (Commentary: Sec. A.7.7.2. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. STUD TRACKS: For veneer with metal stud backup, stud tracks are fastened to the structure at a spacing equal to or less than 24 in. on center. (Commentary: Sec. A.7.6.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-MH; PR-MH. ANCHORAGE: For veneer with concrete block or masonry backup, the backup is positively anchored to the structure at a horizontal spacing equal to or less than 4 ft along the floors and roof. (Commentary: Sec. A.7.7.1. Tier 2: Section 13.6.1.1 and 13.6.1.2)

C NC N/A U LS-not required; PR-MH. WEEP HOLES: In veneer anchored to stud walls, the veneer has functioning weep holes and base flashing. (Commentary: Sec. A.7.5.6. Tier 2: Section 13.6.1.2)

C NC N/A U LS-not required; PR-MH. OPENINGS: For veneer with metal stud backup, steel studs frame window and door openings. (Commentary: Sec. A.7.6.2. Tier 2: Sec. 13.6.1.1 and 13.6.1.2)

Parapets, Cornices, Ornamentation, and Appendages

C NC N/A U LS-LMH; PR-LMH. URM PARAPETS OR CORNICES: Laterally unsupported unreinforced masonry parapets or cornices have height-to-thickness ratios no greater than the following: for Life Safety in Low or Moderate Seismicity, 2.5; for Life Safety in High Seismicity and for Position Retention in any seismicity, 1.5. (Commentary: Sec. A.7.8.1. Tier 2: Sec. 13.6.5)

C NC N/A U LS-LMH; PR-LMH. CANOPIES: Canopies at building exits are anchored to the structure at a spacing no greater than the following: for Life Safety in Low or Moderate Seismicity, 10 ft; for Life Safety in High Seismicity and for Position Retention in any seismicity, 6 ft. (Commentary: Sec. A.7.8.2. Tier 2: Sec. 13.6.6)

C NC N/A U LS-MH; PR-LMH. CONCRETE PARAPETS: Concrete parapets with height-to-thickness ratios greater than 2.5 have vertical reinforcement. (Commentary: Sec. A.7.8.3. Tier 2: Sec. 13.6.5)

C NC N/A U LS-MH; PR-LMH. APPENDAGES: Cornices, parapets, signs, and other ornamentation or appendages that extend above the highest point of anchorage to the structure or cantilever from components are reinforced and anchored to the structural system at a spacing equal to or less than 6 ft. This checklist item does not apply to parapets or cornices covered by other checklist items. (Commentary: Sec. A.7.8.4. Tier 2: Sec. 13.6.6)

The statue and sloped cornices at the east wall elevation are anchored back to the structure less than 6' o.c.; it is unknown whether they are reinforced.

Masonry Chimneys

The west chimney h/w = 2.1; the URM chimneys are built integrally with the exterior walls

C NC N/A U LS-LMH; PR-LMH. URM CHIMNEYS: Unreinforced masonry chimneys extend above the roof surface no more than the following: for Life Safety in Low or Moderate Seismicity, 3 times the least dimension of the chimney; for Life Safety in High Seismicity and for Position Retention in any seismicity, 2 times the least dimension of the chimney. (Commentary: Sec. A.7.9.1. Tier 2: 13.6.7)

C NC N/A U LS-LMH; PR-LMH. ANCHORAGE: Masonry chimneys are anchored at each floor level, at the topmost ceiling level, and at the roof. (Commentary: Sec. A.7.9.2. Tier 2: 13.6.7)

Stairs

Each of the exterior walls at the stairway openings is non-compliant

C NC N/A U LS-LMH; PR-LMH. STAIR ENCLOSURES: Hollow-clay tile or unreinforced masonry walls around stair enclosures are restrained out-of-plane and have height-to-thickness ratios not greater than the following: for Life Safety in Low or Moderate Seismicity, 15-to-1; for Life Safety in High Seismicity and for Position Retention in any seismicity, 12-to-1. (Commentary: Sec. A.7.10.1. Tier 2: Sec. 13.6.2 and 13.6.8)

C NC N/A U LS-LMH; PR-LMH. STAIR DETAILS: In moment frame structures, the connection between the stairs and the structure does not rely on shallow anchors in concrete. Alternatively, the stair details are capable of accommodating the drift calculated using the Quick Check procedure of Section 4.5.3.1 without including any lateral stiffness contribution from the stairs. (Commentary: Sec. A.7.10.2. Tier 2: 13.6.8)

Contents and Furnishings

Non-compliant contents are located on the 1st and 3rd Floors with aspect ratios as high as 8.4 : 1

C NC N/A U LS-MH; PR-MH. INDUSTRIAL STORAGE RACKS: Industrial storage racks or pallet racks more than 12 ft high meet the requirements of ANSI/MH 16.1 as modified by ASCE 7 Chapter 15. (Commentary: Sec. A.7.11.1. Tier 2: Sec. 13.8.1)

C NC N/A U LS-H; PR-MH. TALL NARROW CONTENTS: Contents more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 are anchored to the structure or to each other. (Commentary: Sec. A.7.11.2. Tier 2: Sec. 13.8.2)

C NC N/A U LS-H; PR-H. FALL-PRONE CONTENTS: Equipment, stored items, or other contents weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level are braced or otherwise restrained. (Commentary: Sec. A.7.11.3. Tier 2: Sec. 13.8.2)

C NC N/A U LS-not required; PR-MH. ACCESS FLOORS: Access floors more than 9 in. high are braced. (Commentary: Sec. A.7.11.4. Tier 2: Sec. 13.8.3)

C NC N/A U LS-not required; PR-MH. EQUIPMENT ON ACCESS FLOORS: Equipment and other contents supported by access floor systems are anchored or braced to the structure independent of the access floor. (Commentary: Sec. A.7.11.5. Tier 2: Sec. 13.7.7 and 13.8.3)

C NC N/A U LS-not required; PR-H. SUSPENDED CONTENTS: Items suspended without lateral bracing are free to swing from or move with the structure from which they are suspended without damaging themselves or adjoining components. (Commentary. A.7.11.6. Tier 2: Sec. 13.8.2)

Mechanical and Electrical Equipment

The suspended HVAC equipment in the boiler room, storage room 10, storage room 02 and vault room 105 are not seismically anchored

C NC N/A U LS-H; PR-H. FALL-PRONE EQUIPMENT: Equipment weighing more than 20 lb whose center of mass is more than 4 ft above the adjacent floor level, and which is not in-line equipment, is braced. (Commentary: A.7.12.4. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-H; PR-H. IN-LINE EQUIPMENT: Equipment installed in-line with a duct or piping system, with an operating weight more than 75 lb, is supported and laterally braced independent of the duct or piping system. (Commentary: Sec. A.7.12.5. Tier 2: Sec. 13.7.1)

C NC N/A U LS-H; PR-MH. TALL NARROW EQUIPMENT: Equipment more than 6 ft high with a height-to-depth or height-to-width ratio greater than 3-to-1 is anchored to the floor slab or adjacent structural walls. (Commentary: Sec. A.7.12.6. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-MH. MECHANICAL DOORS: Mechanically operated doors are detailed to operate at a story drift ratio of 0.01. (Commentary: Sec. A.7.12.7. Tier 2: Sec. 13.6.9)

C NC N/A U LS-not required; PR-H. SUSPENDED EQUIPMENT: Equipment suspended without lateral bracing is free to swing from or move with the structure from which it is suspended without damaging itself or adjoining components. (Commentary: Sec. A.7.12.8. Tier 2: Sec. 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. VIBRATION ISOLATORS: Equipment mounted on vibration isolators is equipped with horizontal restraints or snubbers and with vertical restraints to resist overturning. (Commentary: Sec. A.7.12.9. Tier 2: Sec. 13.7.1)

C NC N/A U LS-not required; PR-H. HEAVY EQUIPMENT: Floor-supported or platform-supported equipment weighing more than 400 lb is anchored to the structure. (Commentary: Sec. A.7.12.10. Tier 2: 13.7.1 and 13.7.7)

C NC N/A U LS-not required; PR-H. ELECTRICAL EQUIPMENT: Electrical equipment is laterally braced to the structure. (Commentary: Sec. A.7.12.11. Tier 2: 13.7.7)

C NC N/A U LS-not required; PR-H. CONDUIT COUPLINGS: Conduit greater than 2.5 in. trade size that is attached to panels, cabinets, or other equipment and is subject to relative seismic displacement has flexible couplings or connections. (Commentary: Sec. A.7.12.12. Tier 2: 13.7.8)

Piping

C NC N/A U LS-not required; PR-H. FLEXIBLE COUPLINGS: Fluid and gas piping has flexible couplings. (Commentary: Sec. A.7.13.2. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. FLUID AND GAS PIPING: Fluid and gas piping is anchored and braced to the structure to limit spills or leaks. (Commentary: Sec. A.7.13.4. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. C-CLAMPS: One-sided C-clamps that support piping larger than 2.5 in. in diameter are restrained. (Commentary: Sec. A.7.13.5. Tier 2: Sec. 13.7.3 and 13.7.5)

C NC N/A U LS-not required; PR-H. PIPING CROSSING SEISMIC JOINTS: Piping that crosses seismic joints or isolation planes or is connected to independent structures has couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A7.13.6. Tier 2: Sec.13.7.3 and Sec. 13.7.5)

Ducts

C NC N/A U LS-not required; PR-H. DUCT BRACING: Rectangular ductwork larger than 6 ft^2 in cross-sectional area and round ducts larger than 28 in. in diameter are braced. The maximum spacing of transverse bracing does not exceed 30 ft. The maximum spacing of longitudinal bracing does not exceed 60 ft. (Commentary: Sec. A.7.14.2. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCT SUPPORT: Ducts are not supported by piping or electrical conduit. (Commentary: Sec. A.7.14.3. Tier 2: Sec. 13.7.6)

C NC N/A U LS-not required; PR-H. DUCTS CROSSING SEISMIC JOINTS: Ducts that cross seismic joints or isolation planes or are connected to independent structures have couplings or other details to accommodate the relative seismic displacements. (Commentary: Sec. A.7.14.5. Tier 2: Sec. 13.7.6)

Elevators

C NC N/A U LS-H; PR-H. RETAINER GUARDS: Sheaves and drums have cable retainer guards. (Commentary: Sec. A.7.16.1. Tier 2: 13.8.6)

C NC N/A U LS-H; PR-H. RETAINER PLATE: A retainer plate is present at the top and bottom of both car and counterweight. (Commentary: Sec. A.7.16.2. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. ELEVATOR EQUIPMENT: Equipment, piping, and other components that are part of the elevator system are anchored. (Commentary: Sec. A.7.16.3. Tier 2: 13.8.6)

C NC N/A U LS-not required; PR-H. SEISMIC SWITCH: Elevators capable of operating at speeds of 150 ft/min or faster are equipped with seismic switches that meet the requirements of ASME A17.1 or have trigger levels set to 20% of the acceleration of gravity at the base of the structure and 50% of the acceleration of gravity in other locations. (Commentary: Sec. A.7.16.4. Tier 2: 13.8.6)

C	NC	N/A	U	LS-not required; PR-H. SHAFT WALLS: Elevator shaft walls are anchored and reinforced to prevent toppling into the shaft during strong shaking. (Commentary: Sec. A.7.16.5. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. COUNTERWEIGHT RAILS: All counterweight rails and divider beams are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.6. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. BRACKETS: The brackets that tie the car rails and the counterweight rail to the structure are sized in accordance with ASME A17.1. (Commentary: Sec. A.7.16.7. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. SPREADER BRACKET: Spreader brackets are not used to resist seismic forces. (Commentary: Sec. A.7.16.8. Tier 2: 13.8.6)
C	NC	N/A	U	LS-not required; PR-H. GO-SLOW ELEVATORS: The building has a go-slow elevator system. (Commentary: Sec. A.7.16.9. Tier 2: 13.8.6)

APPENDIX B
CALCULATIONS

Building Code: 2014 Oregon Structural Specialty Code and the ASCE 41-13 Standard

Soils Report: No **Soils Report by:** N/A **Dated:** N/A

Soil Bearing: 1500 PSF **Retaining Walls:** No

Equivalent Fluid Pressure (active): N/A PCF **Passive bearing:** N/A PCF **Friction:** N/A

Structural System: Building Structure

Vertical System: Wood framed roof / masonry walls **Lateral Sys:** Flexible Diaphragm / Concrete or Masonry Shearwalls

Basic Design Loads:

Element	Roof	Floor	Corridor	
Load Type	Dead	Dead	Dead	
Value (PSF)	20	20	20	
Load Type	Snow	Floor Live	Corridor Live	
Value (PSF)	25	50	100	
Deflection Criteria	L/240	L/360	L/360	

Lateral Design Parameters:

Wind Design: Other: ASCE 41-13 **Wind Speed (3 sec Gust):** 120 MPH

Exposure B

Importance Factors I_W = (ice) I_E = (seismic) I_S = (snow) I_i = (ice) **Risk Cat:** II

Seismic Design

Seismic design parameters are based on published values from the USGS web site.

Design Summary:

The following calculations are for the Tier I and II analysis of the courthouse building based on the ASCE 41-13 standard for evaluating an existing building considering BSE-2E seismic loading with the Life Safety Performance Objective. In these calculations this combination will be referred to as the Supplemental Performance Objective (SPOE).

Calculation index:

- USGS Maps: 2-3
- Loading: 4-8
- Basic Checklist: 9
- Tier I and II: 10-25
- Sample Column: 26-27
- Sample Shear Wall: 28-113

MILLER CONSULTING ENGINEERS

9570 SW Barbur Blvd.
Suite One Hundred
Portland, OR 97219

(503)246-1250
FAX: 246-1395

Project Name: Benton County Courthouse Seismic Review Project #: 151029

Location: 120 NW 4th Street, Corvallis, Oregon

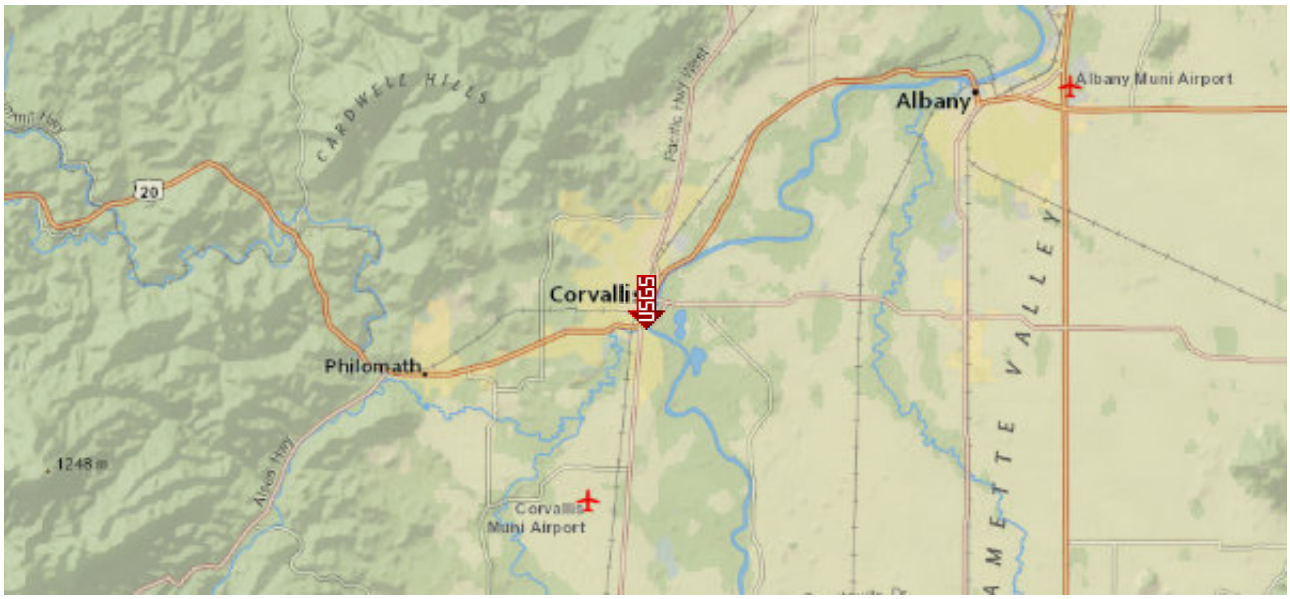
Client: Benton County Public Works

BY: ERW Ck'd: Date: 08/29/16 Page

USGS Design Maps Summary Report

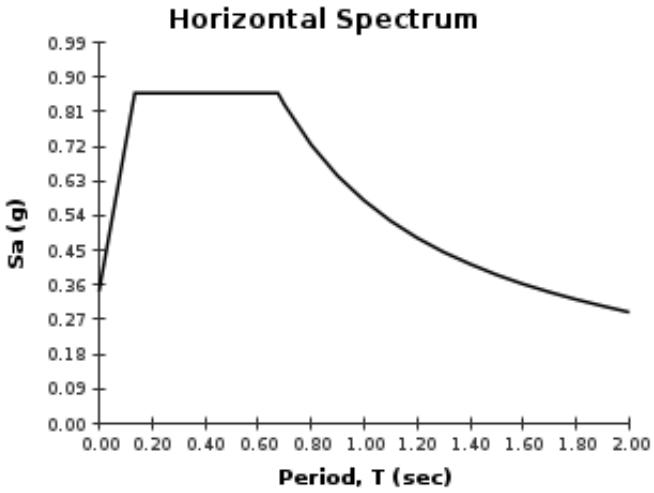
User–Specified Input

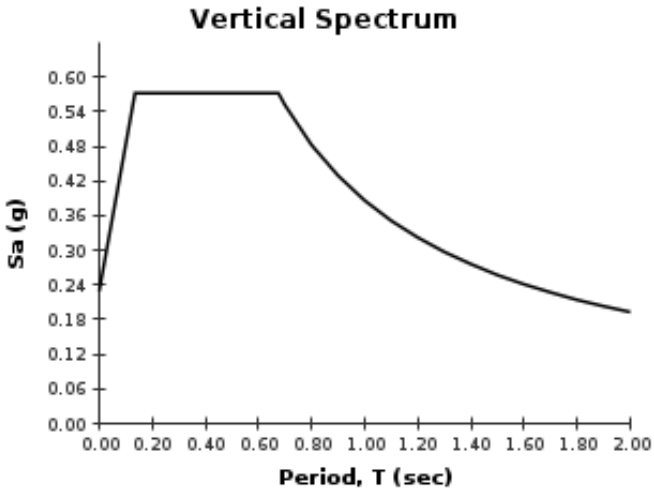
Report Title	Benton County Courthouse Thu December 10, 2015 17:15:56 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-2E (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"



USGS–Provided Output

$S_{S,5/50}$	0.685 g	$S_{XS,BSE-2E}$	0.858 g
$S_{1,5/50}$	0.334 g	$S_{X1,BSE-2E}$	0.579 g





Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Summary Report

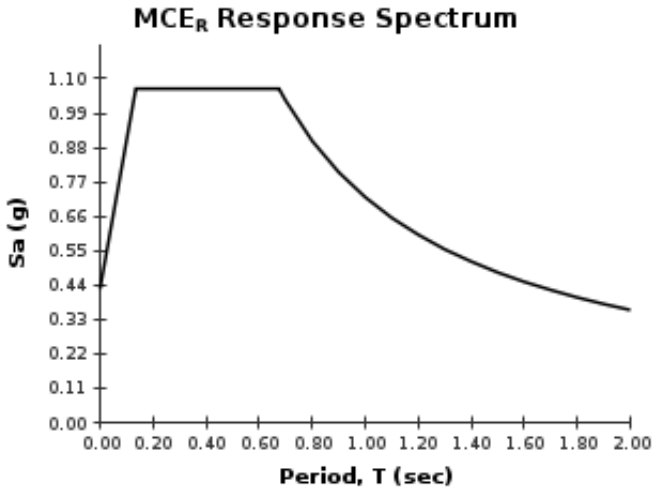
User–Specified Input

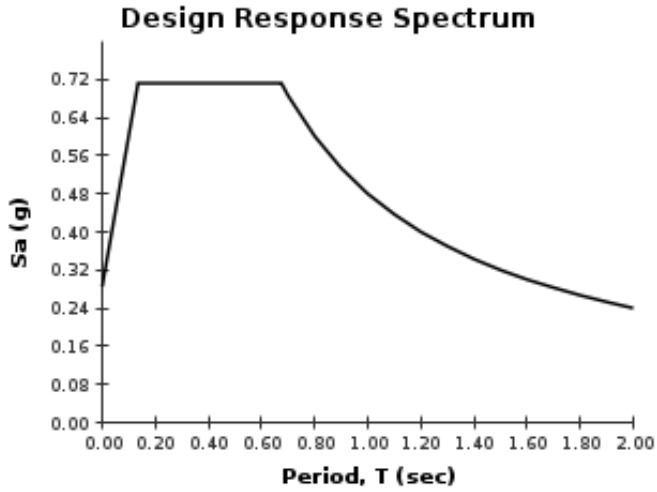
Report Title	Benton County Couthouse Wed November 18, 2015 23:04:50 UTC
Building Code Reference Document	2012 International Building Code (which utilizes USGS hazard data available in 2008)
Site Coordinates	44.56528°N, 123.26237°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S =	0.956 g	S_{MS} =	1.068 g	S_{DS} =	0.712 g
S_1 =	0.472 g	S_{M1} =	0.721 g	S_{D1} =	0.480 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





Benton County Courthouse SPOE Calculations:

Building Weights:

Roof Weights =	20	psf
Wood Floor =	20	psf
Light Framed =	8	psf
8" CMU =	80	psf
12" brick =	120	psf
18" brick =	180	psf
20" brick =	200	psf
30" stone =	350	psf

Building Geometries:

West =	1752	ft^2
Center =	3373	ft^2
East =	1752	ft^2
Total sq footage =	6877	ft^2
Total width =	114.50	ft
Total length =	71.50	ft
Total height =	59.50	ft

Roofs:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

Partial Floors:

Upper Foyer=	14513	lb
Speaker Floor=	9138	lb
Bell Floor=	4102	lb
Top Floor=	995	lb
Sub Total =	28747	lb

Floors:

West =	35035	lb
Center =	67465	lb
East =	35035	lb
Sub Total =	137535	lb

East Section

Width =	24.50	ft
Length =	71.50	ft
Bell Tower Lounge Height =	13.67	ft
3rd Floor Height =	14.25	ft
2nd Floor Height =	15.00	ft
1st Floor Height =	16.50	ft
Basement Floor Height =	10.50	ft

Center Section

Width =	65.50	ft
Length =	51.50	ft
4th Floor Height =	8.50	ft
3rd Floor Height =	9.00	ft

West Section

Width =	24.50	ft
Length =	71.50	ft

4th East-West Walls: (starting at north wall)

Gridline 8 =	11760	lb
GL 7 =	2180	lb
GL 6 =	42445	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	42445	lb
GL 2 =	2180	lb
GL 1 =	11760	lb
Sub Total =	112770	lb

4th North-South Walls: (starting at west wall)

Gridline A =	17160	lb
GL B =	17160	lb
GL C =	38451	lb
GL D =	12000	lb
GL E =	26451	lb
Sub Total =	111223	lb
Total =	**375763**	**lb**

3rd East-West Walls: (starting at north wall)

Gridline 8 =	41895	lb
GL 7 =	35370	lb
GL 6 =	86692	lb
GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	86692	lb
GL 2 =	35370	lb
GL 1 =	41895	lb
Sub Total =	327913	lb

3rd North-South Walls: (starting at west wall)

Gridline A =	61133	lb
GL B =	61133	lb
GL C =	96775	lb
GL D =	42750	lb
GL E =	54025	lb
Sub Total =	315815	lb
Total =	**795776**	**lb**

2nd East-West Walls: (starting at north wall)

Gridline 8 =	128993	lb
GL 7 =	172429	lb
GL 6 =	90821	lb

2nd North-South Walls: (starting at west wall)

Gridline A =	188224	lb
GL B =	188224	lb
GL C =	188224	lb

GL 5 =	0	lb
GL 4 =	0	lb
GL 3 =	90821	lb
GL 2 =	172429	lb
GL 1 =	128993	lb
Sub Total =	784485	lb

GL D =	131625	lb
GL E =	56599	lb
Sub Total =	752895	lb
Total =	**1674915**	**lb**

1st East-West Walls: (starting at north wall)

Gridline 8 =	154350	lb
GL 7 =	206325	lb
GL 6 =	108675	lb
GL 5 =	108075	lb
GL 4 =	108075	lb
GL 3 =	108675	lb
GL 2 =	206325	lb
GL 1 =	154350	lb
Sub Total =	1154850	lb

1st North-South Walls: (starting at west wall)

Gridline A =	225225	lb
GL B =	225225	lb
GL C =	225225	lb
GL D =	157500	lb
GL E =	67725	lb
Sub Total =	900900	lb
Total =	**2193285**	**lb**

Basement East-West Walls: (starting at north wall)

Gridline 8 =	162803	lb
GL 7 =	217624	lb
GL 6 =	114626	lb
GL 5 =	159165	lb
GL 4 =	159165	lb
GL 3 =	114626	lb
GL 2 =	217624	lb
GL 1 =	162803	lb
Sub Total =	1308435	lb

Basement North-South Walls: (starting at west wall)

Gridline A =	237559	lb
GL B =	191595	lb
GL C =	191595	lb
GL D =	166125	lb
GL E =	71434	lb
Sub Total =	858308	lb
Total =	**2304278**	**lb**

4.5.2.1 - Pseudo Seismic Force - for use with quick check proceedures

S_s=	68.50%	
S_1 =	33.40%	
F_a =	1.25	Table 2-3, page 46
F_v =	1.73	Table 2-4, page 46
S_{XS} =	0.86	eqn. 2-1, pg. 46
S_{X1} =	0.58	eqn. 2-2, pg. 46
C_t =	0.02	eqn. 4-5, pg. 69
h_n =	59.50	ft (roof height)
β =	0.75	eqn. 4-5, pg. 69
T =	0.428	eqn. 4-5, pg. 69
Sa =	0.86	eqn. 4-4, pg. 68

Evaluation: Life Safety

Immediate Occupancy Responses:	
S_s=	22.20%
S_1 =	9.10%
Life Safety:	
S_s=	68.50%
S_1 =	33.40%
Collapse Prevention:	
S_s=	68.50%
S_1 =	33.40%

Total Diaphragm Weight

C =	1	table 4-8, pg. 68
W_1 =	2304278	lb (lowest floor)
W_2 =	2193285	lb
W_3 =	1674915	lb
W_4 =	795776	lb
W_5 =	375763	lb

Heights of floors to base:

h_1 =	10.50	ft
h_2 =	27.00	ft
h_3 =	42.00	ft
h_4 =	51.00	ft
h_5 =	59.50	ft

W_{total} =	7344016	lb
V_{total} =	6298375	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

k =	1.00		
F_1 =	703216	lb	0.36
F_2 =	1721169	lb	0.92
F_3 =	2044592	lb	1.42
F_4 =	1179575	lb	1.73
F_5 =	649823	lb	2.02
Σ =	6298375	lb	

7.4.1.3.1 - Pseudo Seismic Force for LSP (linear static proceedure)

Sa =	0.9	
C_1C_2 =	1	from Table 7-3
C_m =	1	from Table 7-4
V =	0.86	*W

Diaphragm Weights East-West Walls:

W_1 =	995843	lb (lowest floor)
W_2 =	1038435	lb
W_3 =	890430	lb
W_4 =	467863	lb
W_5 =	262992	lb
W_{total} =	3655563	lb
V_{total} =	3135084	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	284066	lb
F_2 =	761696	lb
F_3 =	1015986	lb
F_4 =	648228	lb
F_5 =	425108	lb
Σ =	3135084	lb

Diaphragm Weights North-South Walls:

W_1 =	1445970	lb (lowest floor)
W_2 =	1292385	lb
W_3 =	922020	lb
W_4 =	479961	lb
W_5 =	264540	lb
W_{total} =	4404875	lb
V_{total} =	3777709	lb

4.5.2.2 - Story Shear Forces - eqn. 4-3a, pg. 68

F_1 =	444549	lb
F_2 =	1021709	lb
F_3 =	1133864	lb
F_4 =	716716	lb
F_5 =	460871	lb
Σ =	3777709	lb

URM Shear Wall Loads/Diaphragm Loads for Life Safety Evaluation:

Ms =	1.5	URM Wall, from Table 4-9, pg. 70
M_1 =	2.0	Straight Diaphragm (chorder), from Table 12-3
M_2 =	1.5	Straight Diaphragm (unchorder), from Table 12-4
M_3 =	2.5	Wood Diaphragm (chorder), from Table 12-3
M_4 =	2.0	Wood Diaphragm (unchorded), from Table 12-3

4th Floor / Roof	Shear (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	39813	49	542	4	30	OK	303	120	NG
GL 7 =	76189	66	775	5	30	OK	567	120	NG
GL 6 =	144908	20	4954	23	30	OK	434	450	OK
GL 5 =	0	0							
GL 4 =	0	0							
GL 3 =	144908	20	4954	23	30	OK	434	450	OK
GL 2 =	76189	66	775	5	30	OK	567	120	NG
GL 1 =	39813	49	542	4	30	OK	303	120	NG
Gridline A =	60063	72	560	4	30	OK	423	120	NG
GL B =	181297	72	1690	12	30	OK	1569	120	NG
GL C =	199557	72	1861	10	30	OK	1330	120	NG

GL D =	74147	50	989	7	30	OK	423	120	NG
GL E =	41194	15	1894	9	30	OK	430	300	NG
	521822	556258	268	279					

3rd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	81261	121074	38	2124	15	30	OK	463	300	NG
GL 7 =	143661	219850	66	2238	16	30	OK	692	450	NG
GL 6 =	239805	384714	24	10686	49	30	NG	534	450	NG
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	239805	384714	32	7933	37	30	NG	534	450	NG
GL 2 =	143661	219850	66	2238	16	30	OK	692	450	NG
GL 1 =	81261	121074	38	2124	15	30	OK	463	300	NG
Gridline A =	122949	183012	58	2104	15	30	OK	395	450	OK
GL B =	311483	492779	67	4879	34	30	NG	1464	450	NG
GL C =	342050	541607	62	5871	31	30	NG	1241	450	NG
GL D =	135967	210114	39	3638	25	30	OK	395	450	OK
GL E =	75117	116311	16	4846	22	30	OK	536	450	NG
	929453	987566	263	241						

2nd Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	181675	302749	38	5311	25	30	OK	725	300	NG
GL 7 =	325498	545348	47	7708	36	30	NG	1085	450	NG
GL 6 =	407204	791917	33	16160	75	30	NG	837	450	NG
GL 5 =	0	0								
GL 4 =	0	0								
GL 3 =	407204	791917	33	16160	75	30	NG	837	450	NG
GL 2 =	325498	545348	47	7708	36	30	NG	1085	450	NG
GL 1 =	181675	302749	38	5311	25	30	OK	725	300	NG
Gridline A =	272989	456002	55	5527	26	30	OK	624	450	NG
GL B =	571255	1064034	63	11319	52	30	NG	2317	450	NG
GL C =	571255	1112862	59	12628	58	30	NG	1963	450	NG
GL D =	269986	480100	39	8207	38	30	NG	624	450	NG
GL E =	94077	210388	16	8766	41	30	NG	847	450	NG
	1828753	1779562	236	231						

1st Floor

	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	185639	488388	36	8982	37	30	NG	544	300	NG
GL 7 =	310112	855460	47	12134	51	30	NG	595	450	NG
GL 6 =	205059	996977	29	22661	94	30	NG	447	450	OK
GL 5 =	175249	175249	80	1463	6	30	OK	367	450	OK
GL 4 =	175249	175249	81	1437	6	30	OK	367	450	OK
GL 3 =	205059	996977	33	20344	85	30	NG	447	450	OK
GL 2 =	310112	855460	47	12134	51	30	NG	595	450	NG
GL 1 =	185639	488388	36	8982	37	30	NG	544	300	NG
Gridline A =	293687	749689	50	9996	46	30	NG	562	450	NG
GL B =	562450	1626484	62	17396	81	30	NG	2087	450	NG
GL C =	562450	1675312	62	18014	83	30	NG	1769	450	NG
GL D =	276637	756737	36	13886	64	30	NG	562	450	NG
GL E =	99115	309503	16	12635	58	30	NG	763	450	NG
	1752119	1794338	390	227						

Basement Floor	Shear (lb)	Total (lb)	Length (ft)	v_{Total} (plf)	v_{Total} (psi)	v_{cap} (psi)		v_{dia} (plf)	v_{cap} (plf)	
Gridline 8 =	159487	647875	36	11915	33	30	NG	203	300	OK
GL 7 =	236300	1091760	59	12301	34	30	NG	303	450	OK
GL 6 =	140022	1136998	24	31155	87	30	NG	484	450	NG
GL 5 =	167293	342542	78	2915	8	30	OK	137	450	OK
GL 4 =	167293	342542	69	3298	9	30	OK	137	450	OK
GL 3 =	140022	1136998	29	26439	73	30	NG	484	450	NG
GL 2 =	236300	1091760	62	11787	33	30	NG	303	450	OK
GL 1 =	159487	647875	36	11915	33	30	NG	203	300	OK
Gridline A =	247476	997165	60	11111	31	30	NG	245	450	OK
GL B =	324996	1951480	63	20651	57	30	NG	908	450	NG
GL C =	324996	2000308	67	19854	55	30	NG	770	450	NG
GL D =	204066	960804	39	16286	45	30	NG	245	450	OK
GL E =	79116	388619	16	16192	45	30	NG	332	450	OK
	1406206	1180651	394	245						

Foundation Design	M_{OT} (ft*lb)	M_R (ft*lb)	Uplift (lb)	250k MP
Gridline 8 =	10253228	3589560	256294.9266	1.0
GL 7 =	35839163	21075390	567837.4245	2.3
GL 6 =	47173513	8106480	1502578.182	6.0
GL 5 =	6317026.8	10076625	-144599.9298	-0.6
GL 4 =	6317026.8	10076625	-144599.9298	-0.6
GL 3 =	47173513	8106480	1502578.182	6.0
GL 2 =	35839163	21075390	567837.4245	2.3
GL 1 =	10253228	3589560	256294.9266	1.0
				0.0
Gridline A =	31339183	27146047.5	161274.4354	0.6
GL B =	34144162	4163040	1070754.37	4.3
GL C =	35454671	4163040	1117558.245	4.5
GL D =	15908494	4163040	419480.5093	1.7
GL E =	14372659	4228380	390164.5789	1.6

Section 16.1.2 Basic Configuration - Life Safety (Tier 1 evaluations)

Life Safety

Adjacent Building, Sect. 5.4.1.1

Taller Building =	59.50	ft
Shorter Building =	10	ft
Clear Distance =	4.80	inch

Weak Story, Sect. 5.4.2.1

Basement SW (NS) =	245.33	ft	108% of story above
1st Floor SW (NS) =	226.99	ft	98% of story above
2nd Floor SW (NS) =	231.42	ft	96% of story above
3rd Floor SW (NS) =	241.33	ft	86% of story above
4th Floor SW (NS) =	279.00	ft	
Basement SW (EW) =	394.00	ft	101% of story above
1st Floor SW (EW) =	389.66	ft	165% of story above
2nd Floor SW (EW) =	235.67	ft	89% of story above
3rd Floor SW (EW) =	263.33	ft	98% of story above
4th Floor SW (EW) =	268.00	ft	

Change in Building Mass, Sect. 5.4.2.5

Basement Mass =	2304278	lbs	105% of story above
1st Floor Mass =	2193285	lbs	131% of story above
2nd Floor Mass =	1674915	lbs	210% of story above
3rd Floor Mass =	795776	lbs	212% of story above
4th Floor Mass =	375763	lbs	

Foundation - Overturning, Sect. 5.4.3.3

Basement SW (NS) =	71.5	ft	
b/h Ratio (NS) =	1.20		
Basement SW (EW) =	113.5	ft	
b/h Ratio (EW) =	1.91		
0.6 x Sa =	0.51		**< MIN b/h ratio of 1.2 OK**

Section 16.16 URM Bearing Walls - Life Safety Evaluation

Wall Shear Stress Check - Shear Stress in Shear Walls, Sect. 4.5.3.3

Tier 1 Quick Check - see previous pages for shear wall loading

Tier 2 - Shear Wall Capacities - Sect. 7.5.1.5

m =	3.0	from Table 11-3 for Life Safety
K =	0.75	from 11.2.4
F_m =	600	PSI, from Table 11-2(a)
F_t =	45	PSI, from Table 11-2(a)
F_v =	78.38367	PSI per shear area, 80% of TMS 402, Section 3.2.4
E_m =	420000	PSI, per TMS 402, Section 1.8.2.2
G_m =	168000	PSI, per TMS 402, Section 1.8.2.2

Check 3rd Floor Gridline 3:

Total Wall Load=	384714	lb				
Wall height =	14.25	ft		14.25	ft	
Pier height =	8.00	ft		8.00	ft	
Pier length =	11.5	ft		21.25	ft	
Pier thickness =	1.33	ft		1.33	ft	
L / h_{eff} =	0.81	for (11-12)		1.49		
Trib DL P_D =	23464.6	lb		43358.5	lb	
Pier SW P_W =	29497.5	lb		54506.25	lb	
Wall Rigidity =	181027			483802		
Load to Pier =	104754.2	9109.064		279959.4	13174.56	384713.6
Shear Stress=	48	psi		69	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0		
LB Bed-Joint V_{mL} =	43	psi, 11.2.3.6.3 (11-6)		43	psi	
In-plane V_{bjs1} =	94325	lb, 11.3.2.2.2 (11-9)		174297	lb	
In-plane V_{bjs2} =	11732	lb, 11.3.2.2.2 (11-10)		21679	lb	
Rocking V_r =	27755	lb, 11.3.2.2.1 (11-8)		94768	lb	
Lower Bound V_{tc} =	29073	lb, 11.3.2.2.3 (11-11)		99269	lb	
Lower Bound V_{dt} =	102545	lb, 11.3.2.2.4 (11-12)		217800	lb	
Lower Bound P_{CL} =	898612	lb, 11.3.2.2.5 (11-13)		1660478	lb	
Controlling Q_{CE} =	11732	lb		21679	lb	
Allowable Stress=	12	psi **NG**		12	psi **NG**	
DCR =	3.97	6813.614	DCR =	5.74	10879.11	

Check 3rd Floor Gridline 6:

Total Wall Load=	384714	lb
Wall height =	14.25	ft
Pier height =	8.00	ft
Pier length =	24	ft
Pier thickness =	1.33	ft
L / h_{eff} =	1.68	for (11-12)
Trib DL P_D =	48969.6	lb
Pier SW P_W =	61560	lb
Wall Rigidity =	569140	
Load to Pier =	384713.6	16029.73
Shear Stress=	84	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1
LB Bed-Joint V_{mL} =	43	psi, 11.2.3.6.3 (11-6)
In-plane V_{bjs1} =	196853	lb, 11.3.2.2.2 (11-9)
In-plane V_{bjs2} =	24485	lb, 11.3.2.2.2 (11-10)
Rocking V_r =	120884	lb, 11.3.2.2.1 (11-8)
Lower Bound V_{tc} =	126625	lb, 11.3.2.2.3 (11-11)

Lower Bound V_{dt} =	214008	lb, 11.3.2.2.4 (11-12)									
Lower Bound P_{CL} =	1875364	lb, 11.3.2.2.5 (11-13)									
Controlling Q_{CE} =	24485	lb									
Allowable Stress=	12	psi	**NG**								
DCR =	6.98	13734.28									

Check 3rd Floor Gridline B:

Total Wall Load=	492779	lb									
Wall height =	15.00	ft			15.00	ft					
Pier height =	8.00	ft			8.00	ft					
Pier length =	33.67	ft			33.67	ft					
Pier thickness =	1	ft			1	ft					
L / h_{eff} =	2.24	for (11-12)			2.24						
Trib DL P_D =	14814.8	lb			14814.8	lb					
Pier SW P_W =	90909	lb			90909	lb					
Wall Rigidity =	648494				648494						
Load to Pier =	246389.6	7317.778			246389.6	7317.778					
Shear Stress=	51	psi			51	psi					
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1			1.0						
LB Bed-Joint V_{mL} =	39	psi, 11.2.3.6.3 (11-6)			39	psi					
In-plane V_{bjs1} =	189225	lb, 11.3.2.2.2 (11-9)			189225	lb					
In-plane V_{bjs2} =	7407	lb, 11.3.2.2.2 (11-10)			7407	lb					
Rocking V_r =	121756	lb, 11.3.2.2.1 (11-8)			121756	lb					
Lower Bound V_{tc} =	128261	lb, 11.3.2.2.3 (11-11)			128261	lb					
Lower Bound V_{dt} =	174822	lb, 11.3.2.2.4 (11-12)			214983	lb					
Lower Bound P_{CL} =	1978180	lb, 11.3.2.2.5 (11-13)			1978180	lb					
Controlling Q_{CE} =	7407	lb			7407	lb					
Allowable Stress=	3	psi	**NG**		3	psi	**NG**				
DCR =	14.78	6822.778		DCR =	14.78	6822.778					

Check 3rd Floor Gridline C:

Total Wall Load=	541607	lb													
Wall height =	15.00	ft			15.00	ft			15.00	ft			16.50	ft	
Pier height =	8.00	ft			8.00	ft			8.00	ft			8.00	ft	
Pier length =	10.75	ft			10.75	ft			20	ft			20	ft	
Pier thickness =	1.33	ft			1.33	ft			1.33	ft			1.33	ft	
L / h_{eff} =	0.72	for (11-12)			0.72				1.33				1.21		
Trib DL P_D =	4730	lb			4730	lb			8800	lb			8800	lb	
Pier SW P_W =	29025	lb			29025	lb			54000	lb			59400	lb	
Wall Rigidity =	159190				159190				444745				444745		
Load to Pier =	71380.37	6640.034			71380.37	6640.034			199423.1	9971.157			199423.1	9971.157	
Shear Stress=	35	psi			35	psi			52	psi			52	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1			1.0				1.0				1.0		
LB Bed-Joint V_{mL} =	39	psi, 11.2.3.6.3 (11-6)			39	psi			39	psi			39	psi	
In-plane V_{bjs1} =	79572	lb, 11.3.2.2.2 (11-9)			79572	lb			148040	lb			148040	lb	
In-plane V_{bjs2} =	2365	lb, 11.3.2.2.2 (11-10)			2365	lb			4400	lb			4400	lb	
Rocking V_r =	12411	lb, 11.3.2.2.1 (11-8)			12411	lb			42960	lb			42000	lb	
Lower Bound V_{tc} =	13252	lb, 11.3.2.2.3 (11-11)			13252	lb			45870	lb			44688	lb	
Lower Bound V_{dt} =	70271	lb, 11.3.2.2.4 (11-12)			86414	lb			176671	lb			178919	lb	
Lower Bound P_{CL} =	840007	lb, 11.3.2.2.5 (11-13)			840007	lb			1562803	lb			1562803	lb	
Controlling Q_{CE} =	2365	lb			2365	lb			4400	lb			4400	lb	
Allowable Stress=	3	psi	**NG**		3	psi	**NG**		3	psi	**NG**		3	**NG**	
DCR =	13.41	6145.034		DCR =	13.41	6145.034		DCR =	20.14	9476.157		DCR =	20.14	9476.157	

Check 2nd Floor Gridline 2 (& 7):

Total Wall Load=	545348	lb													
Wall height =	16.50	ft			16.50	ft			16.50	ft			16.50	ft	
Pier height =	10.00	ft			10.00	ft			10.00	ft			10.00	ft	

Pier length =	7.167	ft	(3) at	7.5	ft		8.5	ft		8.83	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.86	ft	
L / h_{eff} =	0.43	for (11-12)		0.45			0.52			0.54		
Trib DL P_D =	11252.19	lb		11775	lb		13345	lb		13863.1	lb	
Pier SW P_W =	21285.99	lb		22275	lb		25245	lb		26225.1	lb	
Wall Rigidity =	53547		(3) at	59977	lb for (3) piers		81295			90403		
Load to Pier =	72071.9	10056.08		242179.6	10763.54		109418.2	12872.73		121678.1	13780.08	
Shear Stress=	38	psi		41	psi / pier		49	psi		51	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	40	psi, 11.2.3.6.3 (11-6)		40	psi		40	psi		40	psi	
In-plane V_{bjs1} =	76450	lb, 11.3.2.2.2 (11-9)		80003	lb		90670	lb		95620	lb	
In-plane V_{bjs2} =	5626	lb, 11.3.2.2.2 (11-10)		5888	lb		6673	lb		6932	lb	
Rocking V_r =	8559	lb, 11.3.2.2.1 (11-8)		9373	lb		12039	lb		12992	lb	
Lower Bound V_{tc} =	9120	lb, 11.3.2.2.3 (11-11)		9988	lb		12828	lb		13853	lb	
Lower Bound V_{dt} =	79414	lb, 11.3.2.2.4 (11-12)		95522	lb		108258	lb		113914	lb	
Lower Bound P_{CL} =	770568	lb, 11.3.2.2.5 (11-13)		806371	lb		913887	lb		964931	lb	
Controlling Q_{CE} =	5626	lb		5888	lb		6673	lb		6932	lb	
Allowable Stress=	7	psi **NG**		7	psi **NG**		7	psi **NG**		7	**NG**	
DCR =	5.69	8289.827	DCR =	6.09	8997.289	DCR =	7.29	11106.48	DCR =	7.80	12013.83	

Check 2nd Floor Gridline 3:

Total Wall Load=	791917	lb			
Wall height =	15.00	ft	15.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	11.5	ft	21.25	ft	
Pier thickness =	1.5	ft	1.5	ft	
L / h_{eff} =	0.77	for (11-12)	1.42		
Trib DL P_D =	55262.1	lb	102114.8	lb	
Pier SW P_W =	31050	lb	57375	lb	
Wall Rigidity =	204166		545641		
Load to Pier =	215632.3	18750.63	576284.9	27119.29	791917.2
Shear Stress=	87	psi	126	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)	49	psi	
In-plane V_{bjs1} =	120781	lb, 11.3.2.2.2 (11-9)	223182	lb	
In-plane V_{bjs2} =	27631	lb, 11.3.2.2.2 (11-10)	51057	lb	
Rocking V_r =	48843	lb, 11.3.2.2.1 (11-8)	166773	lb	
Lower Bound V_{tc} =	49780	lb, 11.3.2.2.3 (11-11)	169973	lb	
Lower Bound V_{dt} =	137595	lb, 11.3.2.2.4 (11-12)	292242	lb	
Lower Bound P_{CL} =	1013472	lb, 11.3.2.2.5 (11-13)	1872720	lb	
Controlling Q_{CE} =	27631	lb	51057	lb	
Allowable Stress=	25	psi **NG**	25	psi **NG**	
DCR =	3.47	13344.56	DCR = 5.02	21713.21	

Check 2nd Floor Gridline 6:

Total Wall Load=	791917	lb			
Wall height =	15.00	ft	15.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	9.5	ft	23.33	ft	
Pier thickness =	1.5	ft	1.5	ft	
L / h_{eff} =	0.63	for (11-12)	1.56		
Trib DL P_D =	45651.3	lb	112110	lb	
Pier SW P_W =	25650	lb	62991	lb	
Wall Rigidity =	140188		618528		
Load to Pier =	146323	15402.42	645594.2	27672.27	791917.2
Shear Stress=	71	psi	128	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		

LB Bed-Joint V_{mL} =	49	psi, 11.2.3.6.3 (11-6)		49	psi				
In-plane V_{bjs1} =	99776	lb, 11.3.2.2.2 (11-9)		245028	lb				
In-plane V_{bjs2} =	22826	lb, 11.3.2.2.2 (11-10)		56055	lb				
Rocking V_r =	33331	lb, 11.3.2.2.1 (11-8)		201019	lb				
Lower Bound V_{tc} =	33971	lb, 11.3.2.2.3 (11-11)		204876	lb				
Lower Bound V_{dt} =	113665	lb, 11.3.2.2.4 (11-12)		320848	lb				
Lower Bound P_{CL} =	837216	lb, 11.3.2.2.5 (11-13)		2056026	lb				
Controlling Q_{CE} =	22826	lb		56055	lb				
Allowable Stress=	25	psi	**NG**	25	psi	**NG**			
DCR =	2.85	9996.346	DCR =	5.12	22266.2				

Check 2nd Floor Gridline B:

Total Wall Load=	1064034	lb							
Wall height =	15.00	ft		15.00	ft		15.00	ft	
Pier height =	8.00	ft		8.00	ft		8.00	ft	
Pier length =	17.833	ft		18.75	ft		24	ft	
Pier thickness =	1.33	ft		1.33	ft		1.33	ft	
L / h_{eff} =	1.19	for (11-12)		1.25			1.60		
Trib DL P_D =	30672.76	lb		32250	lb		41280	lb	
Pier SW P_W =	48149.1	lb		50625	lb		64800	lb	
Wall Rigidity =	376760			405560			569140		
Load to Pier =	296631.7	16633.86		319306.1	17029.66		448096.2	18670.68	1064034
Shear Stress=	87	psi		89	psi		97	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0		
LB Bed-Joint V_{mL} =	42	psi, 11.2.3.6.3 (11-6)		42	psi		42	psi	
In-plane V_{bjs1} =	143413	lb, 11.3.2.2.2 (11-9)		150788	lb		193008	lb	
In-plane V_{bjs2} =	15336	lb, 11.3.2.2.2 (11-10)		16125	lb		20640	lb	
Rocking V_r =	58579	lb, 11.3.2.2.1 (11-8)		64758	lb		106099	lb	
Lower Bound V_{tc} =	61511	lb, 11.3.2.2.3 (11-11)		67999	lb		111410	lb	
Lower Bound V_{dt} =	132109	lb, 11.3.2.2.4 (11-12)		170812	lb		240263	lb	
Lower Bound P_{CL} =	1393473	lb, 11.3.2.2.5 (11-13)		1465128	lb		1875364	lb	
Controlling Q_{CE} =	15336	lb		16125	lb		20640	lb	
Allowable Stress=	10	psi	**NG**	10	psi	**NG**	10	psi	**NG**
DCR =	8.60	14698.86	DCR =	8.80	15094.66	DCR =	9.65	16735.68	

Check 2nd Floor Gridline C:

Total Wall Load=	1112862	lb							
Wall height =	15.00	ft		15.00	ft		15.00	ft	
Pier height =	8.00	ft		8.00	ft		8.00	ft	
Pier length =	16.33	ft		19	ft		19	ft	
Pier thickness =	1.5	ft		1.33	ft		1.33	ft	
L / h_{eff} =	1.09	for (11-12)		1.27			1.27		
Trib DL P_D =	28087.6	lb		32680	lb		32680	lb	
Pier SW P_W =	44091	lb		51300	lb		51300	lb	
Wall Rigidity =	371674			413405			413405		
Load to Pier =	345120.6	21134.14		383870.6	20203.72		383870.6	20203.72	1112862
Shear Stress=	98	psi		105	psi		105	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0		
LB Bed-Joint V_{mL} =	41	psi, 11.2.3.6.3 (11-6)		42	psi		42	psi	
In-plane V_{bjs1} =	146317	lb, 11.3.2.2.2 (11-9)		152798	lb		152798	lb	
In-plane V_{bjs2} =	14044	lb, 11.3.2.2.2 (11-10)		16340	lb		16340	lb	
Rocking V_r =	49120	lb, 11.3.2.2.1 (11-8)		66496	lb		66496	lb	
Lower Bound V_{tc} =	51919	lb, 11.3.2.2.3 (11-11)		69825	lb		69825	lb	
Lower Bound V_{dt} =	132312	lb, 11.3.2.2.4 (11-12)		173090	lb		190208	lb	
Lower Bound P_{CL} =	1439130	lb, 11.3.2.2.5 (11-13)		1484663	lb		1484663	lb	
Controlling Q_{CE} =	14044	lb		16340	lb		16340	lb	

Allowable Stress=	9	psi	**NG**		10	psi	**NG**		10	psi	**NG**	
DCR =	10.92	19199.14		DCR =	10.44	18268.72		DCR =	10.44	18268.72		

Check 2nd Floor Gridline D:

Total Wall Load=	480100	lb										
Wall height =	16.50	ft			16.50	ft			16.50	ft		16.50	ft
Pier height =	10.00	ft			10.00	ft			10.00	ft		10.00	ft
Pier length =	9	ft			9	ft			9.25	ft		9.25	ft
Pier thickness =	1.83	ft			1.83	ft			1.83	ft		1.83	ft
L / h_{eff} =	0.55	for (11-12)			0.55				0.56			0.56	
Trib DL P_D =	15480	lb			15480	lb			15910	lb		15910	lb
Pier SW P_W =	26730	lb			26730	lb			27472.5	lb		27472.5	lb
Wall Rigidity =	92997				92997				99089			99089	
Load to Pier =	116219	12913.22			116219	12913.22			123831.1	13387.15		123831.1	13387.15
Shear Stress=	49	psi			49	psi			51	psi		51	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1			1.0				1.0			1.0	
LB Bed-Joint V_{mL} =	41	psi, 11.2.3.6.3 (11-6)			41	psi			41	psi		41	psi
In-plane V_{bjs1} =	96678	lb, 11.3.2.2.2 (11-9)			96678	lb			99364	lb		99364	lb
In-plane V_{bjs2} =	7740	lb, 11.3.2.2.2 (11-10)			7740	lb			7955	lb		7955	lb
Rocking V_r =	14160	lb, 11.3.2.2.1 (11-8)			14160	lb			14958	lb		14958	lb
Lower Bound V_{tc} =	15067	lb, 11.3.2.2.3 (11-11)			15067	lb			15916	lb		15916	lb
Lower Bound V_{dt} =	105448	lb, 11.3.2.2.4 (11-12)			113559	lb			119096	lb		119096	lb
Lower Bound P_{CL} =	967645	lb, 11.3.2.2.5 (11-13)			967645	lb			994524	lb		994524	lb
Controlling Q_{CE} =	7740	lb			7740	lb			7955	lb		7955	lb
Allowable Stress=	7	psi	**NG**		7	psi	**NG**		7	psi	**NG**	7	**NG**
DCR =	6.67	10978.22		DCR =	6.67	10978.22		DCR =	6.92	11452.15	DCR =	6.92	11452.15

Check 2nd Floor Gridline E:

Total Wall Load=	210388	lb				
Wall height =	16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft	
Pier length =	8.25	ft		8.25	ft	
Pier thickness =	1.83	ft		1.83	ft	
L / h_{eff} =	0.50	for (11-12)		0.50		
Trib DL P_D =	39644.55	lb		39644.55	lb	
Pier SW P_W =	24502.5	lb		24502.5	lb	
Wall Rigidity =	75695			75695		
Load to Pier =	105194	12750.79		105194	12750.79	210388
Shear Stress=	48	psi		48	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0		
LB Bed-Joint V_{mL} =	47	psi, 11.2.3.6.3 (11-6)		47	psi	
In-plane V_{bjs1} =	101349	lb, 11.3.2.2.2 (11-9)		101349	lb	
In-plane V_{bjs2} =	19822	lb, 11.3.2.2.2 (11-10)		19822	lb	
Rocking V_r =	23353	lb, 11.3.2.2.1 (11-8)		23353	lb	
Lower Bound V_{tc} =	24125	lb, 11.3.2.2.3 (11-11)		24125	lb	
Lower Bound V_{dt} =	112674	lb, 11.3.2.2.4 (11-12)		129510	lb	
Lower Bound P_{CL} =	887008	lb, 11.3.2.2.5 (11-13)		887008	lb	
Controlling Q_{CE} =	19822	lb		19822	lb	
Allowable Stress=	21	psi	**NG**	21	psi	**NG**
DCR =	2.36	7344.711	DCR =	2.36	7344.711	

Check 1st Floor Gridline 1 (& 8):

Total Wall Load=	488388	lb										
Wall height =	16.50	ft			16.50	ft			16.50	ft		16.50	ft
Pier height =	10.00	ft			10.00	ft			10.00	ft		10.00	ft
Pier length =	4.25	ft		(3) at	7.5	ft			4.25	ft		7.5	ft
Pier thickness =	1.83	ft			1.83	ft			1.83	ft		1.86	ft
L / h_{eff} =	0.26	for (11-12)			0.45				0.26			0.45	

Trib DL P_D =	19210	lb		33900	lb		19210	lb		33900	lb
Pier SW P_W =	12622.5	lb		22275	lb		12622.5	lb		22275	lb
Wall Rigidity =	13254		(3) at	59977	lb for (3) piers		13254			60961	
Load to Pier =	24207.81	5695.955		328632.2	14605.88		24207.81	5695.955		111339.9	14845.32
Shear Stress=	22	psi		55	psi / pier		22	psi		55	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi
In-plane V_{bjs1} =	51604	lb, 11.3.2.2.2 (11-9)		91065	lb		51604	lb		92280	lb
In-plane V_{bjs2} =	9605	lb, 11.3.2.2.2 (11-10)		16950	lb		9605	lb		16950	lb
Rocking V_r =	5916	lb, 11.3.2.2.1 (11-8)		18424	lb		5916	lb		18424	lb
Lower Bound V_{tc} =	6129	lb, 11.3.2.2.3 (11-11)		19086	lb		6129	lb		19109	lb
Lower Bound V_{dt} =	57087	lb, 11.3.2.2.4 (11-12)		115795	lb		65617	lb		117045	lb
Lower Bound P_{CL} =	456944	lb, 11.3.2.2.5 (11-13)		806371	lb		456944	lb		819590	lb
Controlling Q_{CE} =	5916	lb		16950	lb		5916	lb		16950	lb
Allowable Stress=	12	psi **NG**		19	psi **NG**		12	psi **NG**		19	**NG**
DCR =	1.82	2563.802	DCR =	2.87	9520.875	DCR =	1.82	2563.802	DCR =	2.92	9760.316

Check 1st Floor Gridline 2 (& 7):

Total Wall Load=	855460	lb									
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier length =	7.167	ft	(3) at	7.5	ft		8.5	ft		8.83	ft
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.86	ft
L / h_{eff} =	0.43	for (11-12)		0.45			0.52			0.54	
Trib DL P_D =	32394.84	lb		33900	lb		38420	lb		39911.6	lb
Pier SW P_W =	21285.99	lb		22275	lb		25245	lb		26225.1	lb
Wall Rigidity =	53547		(3) at	59977			81295			90403	
Load to Pier =	113055.6	15774.47		379895.1	16884.23		171638.9	20192.82		190870.4	21616.12
Shear Stress=	60	psi		64	psi / pier		77	psi		81	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi
In-plane V_{bjs1} =	87022	lb, 11.3.2.2.2 (11-9)		91065	lb		103207	lb		108644	lb
In-plane V_{bjs2} =	16197	lb, 11.3.2.2.2 (11-10)		16950	lb		19210	lb		19956	lb
Rocking V_r =	16825	lb, 11.3.2.2.1 (11-8)		18424	lb		23665	lb		25538	lb
Lower Bound V_{tc} =	17429	lb, 11.3.2.2.3 (11-11)		19086	lb		24515	lb		26487	lb
Lower Bound V_{dt} =	96268	lb, 11.3.2.2.4 (11-12)		115795	lb		131234	lb		137800	lb
Lower Bound P_{CL} =	770568	lb, 11.3.2.2.5 (11-13)		806371	lb		913887	lb		964931	lb
Controlling Q_{CE} =	16197	lb		16950	lb		19210	lb		19956	lb
Allowable Stress=	19	psi **NG**		19	psi **NG**		19	psi **NG**		19	**NG**
DCR =	3.10	10689.47	DCR =	3.32	11799.23	DCR =	3.97	15107.82	DCR =	4.25	16531.12

Check 1st Floor Gridline 3:

Total Wall Load=	996977	lb			
Wall height =	16.50	ft	16.50	ft	
Pier height =	10.00	ft	10.00	ft	
Pier length =	11.5	ft	21.25	ft	
Pier thickness =	1.83	ft	1.83	ft	
L / h_{eff} =	0.70	for (11-12)	1.29		
Trib DL P_D =	88612.1	lb	163739.8	lb	
Pier SW P_W =	34155	lb	63112.5	lb	
Wall Rigidity =	159992		482388		
Load to Pier =	248308.8	21592.07	748667.7	35231.42	996976.5
Shear Stress=	82	psi	134	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)	52	psi	
In-plane V_{bjs1} =	157949	lb, 11.3.2.2.2 (11-9)	291862	lb	
In-plane V_{bjs2} =	44306	lb, 11.3.2.2.2 (11-10)	81870	lb	

Rocking V_r =	66296	lb, 11.3.2.2.1 (11-8)		226366	lb	
Lower Bound V_{tc} =	66557	lb, 11.3.2.2.3 (11-11)		227258	lb	
Lower Bound V_{dt} =	183194	lb, 11.3.2.2.4 (11-12)		389093	lb	
Lower Bound P_{CL} =	1236436	lb, 11.3.2.2.5 (11-13)		2284718	lb	
Controlling Q_{CE} =	44306	lb		81870	lb	
Allowable Stress=	33	psi	**NG**	33	psi	**NG**
DCR =	2.49	12923.5	DCR =	4.06	26562.85	

Check 1st Floor Gridline 6:

Total Wall Load=	996977	lb
Wall height =	16.50	ft
Pier height =	10.00	ft
Pier length =	29.25	ft
Pier thickness =	1.83	ft
L / h_{eff} =	1.77	for (11-12)
Trib DL P_D =	225383	lb
Pier SW P_W =	86872.5	lb
Wall Rigidity =	757585	
Load to Pier =	996976.5	34084.67
Shear Stress=	129	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)
In-plane V_{bjs1} =	401740	lb, 11.3.2.2.2 (11-9)
In-plane V_{bjs2} =	112691	lb, 11.3.2.2.2 (11-10)
Rocking V_r =	428889	lb, 11.3.2.2.1 (11-8)
Lower Bound V_{tc} =	430579	lb, 11.3.2.2.3 (11-11)
Lower Bound V_{dt} =	535575	lb, 11.3.2.2.4 (11-12)
Lower Bound P_{CL} =	3144848	lb, 11.3.2.2.5 (11-13)
Controlling Q_{CE} =	112691	lb
Allowable Stress=	33	psi **NG**
DCR =	3.93	25416.09

Check 1st Floor Gridline A:

Total Wall Load=	749689	lb								
Wall height =	16.50	ft	16.50	ft		16.50	ft		16.50	ft
Pier height =	10.00	ft	10.00	ft		10.00	ft		10.00	ft
Pier length =	9.25	ft	10	ft		10	ft	(2) at	12.25	ft
Pier thickness =	1.83	ft	1.83	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.56	for (11-12)	0.61			0.61			0.74	
Trib DL P_D =	42735	lb	46200	lb		46200	lb		56595	lb
Pier SW P_W =	27472.5	lb	29700	lb		29700	lb		36382.5	lb
Wall Rigidity =	99089		118246			118246		(2) at	182272	
Load to Pier =	106103.4	11470.64	126617.2	12661.72		126617.2	12661.72		390350.8	31865.37
Shear Stress=	44	psi	48	psi		48	psi		60	psi / pier
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)	46	psi		46	psi		46	psi
In-plane V_{bjs1} =	112776	lb, 11.3.2.2.2 (11-9)	121920	lb		121920	lb		149352	lb
In-plane V_{bjs2} =	21368	lb, 11.3.2.2.2 (11-10)	23100	lb		23100	lb		28298	lb
Rocking V_r =	28492	lb, 11.3.2.2.1 (11-8)	33300	lb		33300	lb		49971	lb
Lower Bound V_{tc} =	29487	lb, 11.3.2.2.3 (11-11)	34463	lb		34463	lb		51716	lb
Lower Bound V_{dt} =	106303	lb, 11.3.2.2.4 (11-12)	141323	lb		155300	lb		190243	lb
Lower Bound P_{CL} =	994524	lb, 11.3.2.2.5 (11-13)	1075162	lb		1075162	lb		1317073	lb
Controlling Q_{CE} =	21368	lb	23100	lb		23100	lb		28298	lb
Allowable Stress=	20	psi **NG**	20	psi	**NG**	20	psi	**NG**	20	**NG**
DCR =	2.21	6273.137	DCR = 2.44	7464.221	DCR =	2.44	7464.221	DCR =	3.07	21470.37

Check 1st Floor Gridline B:

Total Wall Load=	1626484	lb

Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	8.00	ft		9.00	ft		8.00	ft		9.00	ft	
Pier length =	11.5	ft		14.25	ft		16.83	ft		19.75	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft	
L / h_{eff} =	0.70	for (11-12)		0.86			1.02			1.20		
Trib DL P_D =	53130	lb		65835	lb		77754.6	lb		91245	lb	
Pier SW P_W =	34155	lb		42322.5	lb		49985.1	lb		58657.5	lb	
Wall Rigidity =	249083			297138			475040			506405		
Load to Pier =	265194.6	23060.4		316358.8	22200.62		505768.7	30051.61		539161.7	27299.33	
Shear Stress=	88	psi		84	psi		114	psi		104	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	140208	lb, 11.3.2.2.2 (11-9)		173736	lb		205191	lb		240792	lb	
In-plane V_{bjs2} =	26565	lb, 11.3.2.2.2 (11-10)		32918	lb		38877	lb		45623	lb	
Rocking V_r =	44039	lb, 11.3.2.2.1 (11-8)		67620	lb		94322	lb		129891	lb	
Lower Bound V_{tc} =	45577	lb, 11.3.2.2.3 (11-11)		69981	lb		97615	lb		134426	lb	
Lower Bound V_{dt} =	132161	lb, 11.3.2.2.4 (11-12)		201386	lb		261371	lb		306718	lb	
Lower Bound P_{CL} =	1236436	lb, 11.3.2.2.5 (11-13)		1532105	lb		1809497	lb		2123444	lb	
Controlling Q_{CE} =	26565	lb		32918	lb		38877	lb		45623	lb	
Allowable Stress=	20	psi	**NG**	20	psi	**NG**	20	psi	**NG**	20	**NG**	
DCR =	4.44	17862.9	DCR =	4.27	17003.12	DCR =	5.78	24854.11	DCR =	5.25	22101.83	

Check 1st Floor Gridline C:

Total Wall Load=	1675312	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	8.00	ft		9.00	ft		8.00	ft		9.00	ft	
Pier length =	14.25	ft		15.25	ft		15.5	ft		17	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft	
L / h_{eff} =	0.86	for (11-12)		0.92			0.94			1.03		
Trib DL P_D =	65835	lb		70455	lb		71610	lb		78540	lb	
Pier SW P_W =	42322.5	lb		45292.5	lb		46035	lb		50490	lb	
Wall Rigidity =	364047			334521			417652			400927		
Load to Pier =	401998.9	28210.45		369395.6	24222.66		461192.9	29754.38		442724.2	26042.6	
Shear Stress=	107	psi		92	psi		113	psi		99	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi	
In-plane V_{bjs1} =	173736	lb, 11.3.2.2.2 (11-9)		185928	lb		188976	lb		207264	lb	
In-plane V_{bjs2} =	32918	lb, 11.3.2.2.2 (11-10)		35228	lb		35805	lb		39270	lb	
Rocking V_r =	67620	lb, 11.3.2.2.1 (11-8)		77443	lb		80003	lb		96237	lb	
Lower Bound V_{tc} =	69981	lb, 11.3.2.2.3 (11-11)		80147	lb		82797	lb		99597	lb	
Lower Bound V_{dt} =	201386	lb, 11.3.2.2.4 (11-12)		232096	lb		240716	lb		264011	lb	
Lower Bound P_{CL} =	1532105	lb, 11.3.2.2.5 (11-13)		1639621	lb		1666500	lb		1827775	lb	
Controlling Q_{CE} =	32918	lb		35228	lb		35805	lb		39270	lb	
Allowable Stress=	20	psi	**NG**	20	psi	**NG**	20	psi	**NG**	20	**NG**	
DCR =	5.43	23012.95	DCR =	4.66	19025.16	DCR =	5.72	24556.88	DCR =	5.01	20845.1	

Check 1st Floor Gridline D:

Total Wall Load=	756737	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier length =	9	ft		9	ft		9.25	ft		9.25	ft	
Pier thickness =	1.83	ft		1.83	ft		1.83	ft		1.83	ft	
L / h_{eff} =	0.55	for (11-12)		0.55			0.56			0.56		
Trib DL P_D =	41580	lb		41580	lb		42735	lb		42735	lb	
Pier SW P_W =	26730	lb		26730	lb		27472.5	lb		27472.5	lb	
Wall Rigidity =	92997			92997			99089			99089		
Load to Pier =	183185.2	20353.91		183185.2	20353.91		195183.5	21100.92		195183.5	21100.92	

Shear Stress=	77	psi		77	psi		80	psi		80	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	46	psi, 11.2.3.6.3 (11-6)		46	psi		46	psi		46	psi
In-plane V_{bjs1} =	109728	lb, 11.3.2.2.2 (11-9)		109728	lb		112776	lb		112776	lb
In-plane V_{bjs2} =	20790	lb, 11.3.2.2.2 (11-10)		20790	lb		21368	lb		21368	lb
Rocking V_r =	26973	lb, 11.3.2.2.1 (11-8)		26973	lb		28492	lb		28492	lb
Lower Bound V_{tc} =	27915	lb, 11.3.2.2.3 (11-11)		27915	lb		29487	lb		29487	lb
Lower Bound V_{dt} =	127191	lb, 11.3.2.2.4 (11-12)		136975	lb		143653	lb		143653	lb
Lower Bound P_{CL} =	967645	lb, 11.3.2.2.5 (11-13)		967645	lb		994524	lb		994524	lb
Controlling Q_{CE} =	20790	lb		20790	lb		21368	lb		21368	lb
Allowable Stress=	20	psi	**NG**	20	psi	**NG**	20	psi	**NG**	20	**NG**
DCR =	3.92	15156.41	DCR =	3.92	15156.41	DCR =	4.06	15903.42	DCR =	4.06	15903.42

Check 1st Floor Gridline E:

Total Wall Load=	309503	lb				
Wall height =	16.50	ft		16.50	ft	
Pier height =	10.00	ft		10.00	ft	
Pier length =	8.25	ft		8.25	ft	
Pier thickness =	1.83	ft		1.83	ft	
L / h_{eff} =	0.50	for (11-12)		0.50		
Trib DL P_D =	63569.55	lb		63569.55	lb	
Pier SW P_W =	24502.5	lb		24502.5	lb	
Wall Rigidity =	75695			75695		
Load to Pier =	154751.4	18757.74		154751.4	18757.74	309502.8
Shear Stress=	71	psi		71	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)		52	psi	
In-plane V_{bjs1} =	113311	lb, 11.3.2.2.2 (11-9)		113311	lb	
In-plane V_{bjs2} =	31785	lb, 11.3.2.2.2 (11-10)		31785	lb	
Rocking V_r =	34119	lb, 11.3.2.2.1 (11-8)		34119	lb	
Lower Bound V_{tc} =	34254	lb, 11.3.2.2.3 (11-11)		34254	lb	
Lower Bound V_{dt} =	131422	lb, 11.3.2.2.4 (11-12)		151060	lb	
Lower Bound P_{CL} =	887008	lb, 11.3.2.2.5 (11-13)		887008	lb	
Controlling Q_{CE} =	31785	lb		31785	lb	
Allowable Stress=	33	psi	**NG**	33	psi	**NG**
DCR =	2.16	10089.17	DCR =	2.16	10089.17	

Check Basement Gridline 1 (& 8):

Total Wall Load=	647875	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft
Pier length =	4.25	ft	(3) at	7.5	ft		4.25	ft		7.5	ft
Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft
L / h_{eff} =	0.43	for (11-12)		0.75			0.43			0.75	
Trib DL P_D =	32895	lb		58050	lb		32895	lb		58050	lb
Pier SW P_W =	7650	lb		13500	lb		7650	lb		13500	lb
Wall Rigidity =	111058		(3) at	368182	lb for (3) piers		111058			368182	
Load to Pier =	42453.36	9989.025		422226.3	18765.61		42453.36	9989.025		140742.1	18765.61
Shear Stress=	28	psi		52	psi / pier		28	psi		52	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi
In-plane V_{bjs1} =	73823	lb, 11.3.2.2.2 (11-9)		130275	lb		73823	lb		130275	lb
In-plane V_{bjs2} =	16448	lb, 11.3.2.2.2 (11-10)		29025	lb		16448	lb		29025	lb
Rocking V_r =	14045	lb, 11.3.2.2.1 (11-8)		43740	lb		14045	lb		43740	lb
Lower Bound V_{tc} =	14621	lb, 11.3.2.2.3 (11-11)		45534	lb		14621	lb		45534	lb
Lower Bound V_{dt} =	79940	lb, 11.3.2.2.4 (11-12)		162150	lb		91885	lb		162150	lb

Lower Bound P_{CL} =	624240	lb, 11.3.2.2.5 (11-13)		1101600	lb		624240	lb		1101600	lb
Controlling Q_{CE} =	14045	lb		29025	lb		14045	lb		29025	lb
Allowable Stress=	21	psi	**NG**	24	psi	**NG**	21	psi	**NG**	24	**NG**
DCR =	1.34	2553.225	DCR =	2.16	10058.11	DCR =	1.34	2553.225	DCR =	2.16	10058.11

Check Basement Gridline 2 (& 7):

Total Wall Load=	1091760	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft
Pier length =	7.167	ft	(3) at	7.5	ft		8.5	ft		8.83	ft
Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft
L / h_{eff} =	0.72	for (11-12)		0.75			0.85			0.88	
Trib DL P_D =	55472.58	lb		58050	lb		65790	lb		68344.2	lb
Pier SW P_W =	12900.6	lb		13500	lb		15300	lb		15894	lb
Wall Rigidity =	338460		(3) at	368182			459568			490223	
Load to Pier =	154429	21547.23		503970.5	22398.69		209686.9	24669.04		223673.9	25331.14
Shear Stress=	60	psi		62	psi / pier		69	psi		70	psi
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0	
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi
In-plane V_{bjs1} =	124491	lb, 11.3.2.2.2 (11-9)		130275	lb		147645	lb		153377	lb
In-plane V_{bjs2} =	27736	lb, 11.3.2.2.2 (11-10)		29025	lb		32895	lb		34172	lb
Rocking V_r =	39942	lb, 11.3.2.2.1 (11-8)		43740	lb		56182	lb		60629	lb
Lower Bound V_{tc} =	41580	lb, 11.3.2.2.3 (11-11)		45534	lb		58485	lb		63115	lb
Lower Bound V_{dt} =	134807	lb, 11.3.2.2.4 (11-12)		162150	lb		183770	lb		190905	lb
Lower Bound P_{CL} =	1052689	lb, 11.3.2.2.5 (11-13)		1101600	lb		1248480	lb		1296950	lb
Controlling Q_{CE} =	27736	lb		29025	lb		32895	lb		34172	lb
Allowable Stress=	24	psi	**NG**	24	psi	**NG**	24	psi	**NG**	24	**NG**
DCR =	2.47	12839.73	DCR =	2.57	13691.19	DCR =	2.83	15961.54	DCR =	2.91	16623.64

Check Basement Gridline 3:

Total Wall Load=	1136998	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		5.00	ft		8.00	ft		8.00	ft
Pier length =	3.75	ft		6.75	ft		8.33	ft		11.25	ft
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft
L / h_{eff} =	0.38	for (11-12)		0.68			0.83			1.13	
Trib DL P_D =	40782.75	lb		73408.95	lb		90592.08	lb		122348.3	lb
Pier SW P_W =	13125	lb		23625	lb		14994	lb		20250	lb
Wall Rigidity =	81936			301931			129304			238981	
Load to Pier =	123860.1	33029.35		456416.7	67617.29		195463	23464.95		361258.4	32111.86
Shear Stress=	92	psi		188	psi		89	psi		122	psi
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5	
LB Bed-Joint V_{mL} =	53	psi, 11.2.3.6.3 (11-6)		53	psi		58	psi		58	psi
In-plane V_{bjs1} =	71016	lb, 11.3.2.2.2 (11-9)		127829	lb		127613	lb		172347	lb
In-plane V_{bjs2} =	20391	lb, 11.3.2.2.2 (11-10)		36704	lb		45296	lb		61174	lb
Rocking V_r =	9097	lb, 11.3.2.2.1 (11-8)		29474	lb		39579	lb		72190	lb
Lower Bound V_{tc} =	9147	lb, 11.3.2.2.3 (11-11)		29635	lb		38940	lb		71025	lb
Lower Bound V_{dt} =	69700	lb, 11.3.2.2.4 (11-12)		154282	lb		172509	lb		232980	lb
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		895610	lb		1209557	lb
Controlling Q_{CE} =	9097	lb		29474	lb		38940	lb		61174	lb
Allowable Stress=	15	psi	**NG**	27	psi	**NG**	40	psi	**NG**	46	**NG**
DCR =	6.05	27571.19	DCR =	6.88	57792.6	DCR =	2.23	12946.89	DCR =	2.62	19877.03

Check Basement Gridline 6:

Total Wall Load=	1136998	lb									
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft
Pier height =	5.00	ft		6.50	ft		8.00	ft		8.00	ft
Pier length =	3.75	ft		6.75	ft		5.5	ft		9.33	ft

Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft	
L / h_{eff} =	0.38	for (11-12)		0.68			0.55			0.93		
Trib DL P_D =	40782.75	lb		73408.95	lb		59814.7	lb		101467.5	lb	
Pier SW P_W =	13125	lb		23625	lb		9900	lb		16794	lb	
Wall Rigidity =	81936			175608			48200			164749		
Load to Pier =	283251.6	75533.77		424375.4	62870.43		166627.5	30295.91		569532.7	61043.16	
Shear Stress=	210	psi		175	psi		115	psi		232	psi	
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5		
LB Bed-Joint V_{mL} =	53	psi, 11.2.3.6.3 (11-6)		53	psi		58	psi		58	psi	
In-plane V_{bjs1} =	71016	lb, 11.3.2.2.2 (11-9)		127829	lb		84258	lb		142933	lb	
In-plane V_{bjs2} =	20391	lb, 11.3.2.2.2 (11-10)		36704	lb		29907	lb		50734	lb	
Rocking V_r =	9097	lb, 11.3.2.2.1 (11-8)		29474	lb		17254	lb		49652	lb	
Lower Bound V_{tc} =	9147	lb, 11.3.2.2.3 (11-11)		29635	lb		16976	lb		48851	lb	
Lower Bound V_{dt} =	69700	lb, 11.3.2.2.4 (11-12)		154282	lb		113901	lb		193218	lb	
Lower Bound P_{CL} =	550800	lb, 11.3.2.2.5 (11-13)		991440	lb		591339	lb		1003126	lb	
Controlling Q_{CE} =	9097	lb		29474	lb		16976	lb		48851	lb	
Allowable Stress=	15	psi	**NG**	27	psi	**NG**	26	psi	**NG**	45		**NG**
DCR =	13.84	70075.61	DCR =	6.40	53045.74	DCR =	4.36	23351.21	DCR =	5.18	49262.43	

Check Basement Gridline A:

Total Wall Load=	997165	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft	
Pier length =	10	ft		10	ft		20	ft		20	ft	
Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft	
L / h_{eff} =	1.00	for (11-12)		1.00			2.00			2.00		
Trib DL P_D =	77900	lb		77900	lb		155800	lb		155800	lb	
Pier SW P_W =	18000	lb		18000	lb		36000	lb		36000	lb	
Wall Rigidity =	600000			600000			1527273			1527273		
Load to Pier =	140625.8	14062.58		140625.8	14062.58		357956.5	17897.83		357956.5	17897.83	
Shear Stress=	39	psi		39	psi		50	psi		50	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi	
In-plane V_{bjs1} =	173950	lb, 11.3.2.2.2 (11-9)		173950	lb		347900	lb		347900	lb	
In-plane V_{bjs2} =	38950	lb, 11.3.2.2.2 (11-10)		38950	lb		77900	lb		77900	lb	
Rocking V_r =	78210	lb, 11.3.2.2.1 (11-8)		78210	lb		312840	lb		312840	lb	
Lower Bound V_{tc} =	81388	lb, 11.3.2.2.3 (11-11)		81388	lb		325553	lb		325553	lb	
Lower Bound V_{dt} =	160326	lb, 11.3.2.2.4 (11-12)		197158	lb		433315	lb		433315	lb	
Lower Bound P_{CL} =	1468800	lb, 11.3.2.2.5 (11-13)		1468800	lb		2937600	lb		2937600	lb	
Controlling Q_{CE} =	38950	lb		38950	lb		77900	lb		77900	lb	
Allowable Stress=	24	psi	**NG**	24	psi	**NG**	24	psi	**NG**	24		**NG**
DCR =	1.60	5298.828	DCR =	1.60	5298.828	DCR =	2.04	9134.076	DCR =	2.04	9134.076	

Check Basement Gridline B:

Total Wall Load=	1951480	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	10.00	ft		10.00	ft		8.00	ft		8.00	ft	
Pier length =	12.33	ft		12.33	ft		17.75	ft		21.75	ft	
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft	
L / h_{eff} =	1.23	for (11-12)		1.23			1.78			2.18		
Trib DL P_D =	96050.7	lb		96050.7	lb		138272.5	lb		169432.5	lb	
Pier SW P_W =	22194	lb		22194	lb		31950	lb		39150	lb	
Wall Rigidity =	252315			252315			514811			687118		
Load to Pier =	288526.9	23400.4		288526.9	23400.4		588695.4	33165.94		785730.8	36125.55	
Shear Stress=	65	psi		65	psi		126	psi		137	psi	
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		52	psi		52	psi	

In-plane V_{bjs1} =	214480	lb, 11.3.2.2.2 (11-9)		214480	lb		244542	lb		299650	lb	
In-plane V_{bjs2} =	48025	lb, 11.3.2.2.2 (11-10)		48025	lb		69136	lb		84716	lb	
Rocking V_r =	65608	lb, 11.3.2.2.1 (11-8)		65608	lb		135965	lb		204150	lb	
Lower Bound V_{tc} =	68274	lb, 11.3.2.2.3 (11-11)		68274	lb		137982	lb		207179	lb	
Lower Bound V_{dt} =	197682	lb, 11.3.2.2.4 (11-12)		243096	lb		318476	lb		390246	lb	
Lower Bound P_{CL} =	1811030	lb, 11.3.2.2.5 (11-13)		1811030	lb		1908412	lb		2338476	lb	
Controlling Q_{CE} =	48025	lb		48025	lb		69136	lb		84716	lb	
Allowable Stress=	24	psi	**NG**	24	psi	**NG**	33	psi	**NG**	33	**NG**	
DCR =	2.67	14636.65	DCR =	2.67	14636.65	DCR =	3.78	24402.19	DCR =	4.12	27361.8	

Check Basement Gridline C:

Total Wall Load=	2000308	lb										
Wall height =	10.00	ft		10.00	ft		10.00	ft		10.00	ft	
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft	
Pier length =	12.33	ft		12.33	ft		21.75	ft		22	ft	
Pier thickness =	2.5	ft		2.5	ft		1.83	ft		1.83	ft	
L / h_{eff} =	1.23	for (11-12)		1.23			2.18			2.20		
Trib DL P_D =	96050.7	lb		96050.7	lb		169432.5	lb		171380	lb	
Pier SW P_W =	22194	lb		22194	lb		39150	lb		39600	lb	
Wall Rigidity =	819977			819977			1233099			1249474		
Load to Pier =	397864.5	32268.01		397864.5	32268.01		598316.7	27508.82		606262.1	27557.37	
Shear Stress=	90	psi		90	psi		104	psi		105	psi	
Alpha =	0.5	for 11-8 & 11-11 per 11.3.2.2.1		0.5			0.5			0.5		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		52	psi		52	psi	
In-plane V_{bjs1} =	214480	lb, 11.3.2.2.2 (11-9)		214480	lb		299650	lb		303094	lb	
In-plane V_{bjs2} =	48025	lb, 11.3.2.2.2 (11-10)		48025	lb		84716	lb		85690	lb	
Rocking V_r =	65608	lb, 11.3.2.2.1 (11-8)		65608	lb		204150	lb		208870	lb	
Lower Bound V_{tc} =	68274	lb, 11.3.2.2.3 (11-11)		68274	lb		207179	lb		211969	lb	
Lower Bound V_{dt} =	243096	lb, 11.3.2.2.4 (11-12)		261796	lb		390246	lb		394731	lb	
Lower Bound P_{CL} =	1811030	lb, 11.3.2.2.5 (11-13)		1811030	lb		2338476	lb		2365356	lb	
Controlling Q_{CE} =	48025	lb		48025	lb		84716	lb		85690	lb	
Allowable Stress=	24	psi	**NG**	24	psi	**NG**	33	psi	**NG**	33	**NG**	
DCR =	3.68	23504.26	DCR =	3.68	23504.26	DCR =	3.14	18745.07	DCR =	3.14	18793.62	

Check Basement Gridline D:

Total Wall Load=	756737	lb										
Wall height =	16.50	ft		16.50	ft		16.50	ft		16.50	ft	
Pier height =	5.00	ft		5.00	ft		5.00	ft		5.00	ft	
Pier length =	9	ft		9	ft		9.25	ft		9.25	ft	
Pier thickness =	2.5	ft		2.5	ft		2.5	ft		2.5	ft	
L / h_{eff} =	0.55	for (11-12)		0.55			0.56			0.56		
Trib DL P_D =	70110	lb		70110	lb		72057.5	lb		72057.5	lb	
Pier SW P_W =	26730	lb		26730	lb		27472.5	lb		27472.5	lb	
Wall Rigidity =	506083			506083			529474			529474		
Load to Pier =	184911.1	20545.67		184911.1	20545.67		193457.7	20914.34		193457.7	20914.34	
Shear Stress=	57	psi		57	psi		58	psi		58	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1		1.0			1.0			1.0		
LB Bed-Joint V_{mL} =	48	psi, 11.2.3.6.3 (11-6)		48	psi		48	psi		48	psi	
In-plane V_{bjs1} =	156555	lb, 11.3.2.2.2 (11-9)		156555	lb		160904	lb		160904	lb	
In-plane V_{bjs2} =	35055	lb, 11.3.2.2.2 (11-10)		35055	lb		36029	lb		36029	lb	
Rocking V_r =	40979	lb, 11.3.2.2.1 (11-8)		40979	lb		43287	lb		43287	lb	
Lower Bound V_{tc} =	42292	lb, 11.3.2.2.3 (11-11)		42292	lb		44674	lb		44674	lb	
Lower Bound V_{dt} =	181248	lb, 11.3.2.2.4 (11-12)		195190	lb		204707	lb		204707	lb	
Lower Bound P_{CL} =	1321920	lb, 11.3.2.2.5 (11-13)		1321920	lb		1358640	lb		1358640	lb	
Controlling Q_{CE} =	35055	lb		35055	lb		36029	lb		36029	lb	
Allowable Stress=	24	psi	**NG**	24	psi	**NG**	24	psi	**NG**	24	**NG**	

DCR =	2.34	11781.92	DCR =	2.34	11781.92	DCR =	2.39	12150.59	DCR =	2.39	12150.59

Check Basement Gridline E:

Total Wall Load=	309503	lb			
Wall height =	10.00	ft	10.00	ft	
Pier height =	8.00	ft	8.00	ft	
Pier length =	8.25	ft	8.25	ft	
Pier thickness =	2.5	ft	2.5	ft	
L / h_{eff} =	0.83	for (11-12)	0.83		
Trib DL P_D =	87494.55	lb	87494.55	lb	
Pier SW P_W =	14850	lb	14850	lb	
Wall Rigidity =	172939		172939		
Load to Pier =	154751.4	18757.74	154751.4	18757.74	309502.8
Shear Stress=	52	psi	52	psi	
Alpha =	1	for 11-8 & 11-11 per 11.3.2.2.1	1.0		
LB Bed-Joint V_{mL} =	52	psi, 11.2.3.6.3 (11-6)	52	psi	
In-plane V_{bjs1} =	155122	lb, 11.3.2.2.2 (11-9)	155122	lb	
In-plane V_{bjs2} =	43747	lb, 11.3.2.2.2 (11-10)	43747	lb	
Rocking V_r =	70478	lb, 11.3.2.2.1 (11-8)	70478	lb	
Lower Bound V_{tc} =	71884	lb, 11.3.2.2.3 (11-11)	71884	lb	
Lower Bound V_{dt} =	173867	lb, 11.3.2.2.4 (11-12)	199847	lb	
Lower Bound P_{CL} =	1211760	lb, 11.3.2.2.5 (11-13)	1211760	lb	
Controlling Q_{CE} =	43747	lb	43747	lb	
Allowable Stress=	33	psi **NG**	33	psi **NG**	
DCR =	1.57	6826.668	DCR = 1.57	6826.668	

Wall Anchorage - Check shear capacity of anchorage, Sect. 5.7.1.1

Tier 1 - Quick Check, Sect. 4.5.3.7

Wall thickness =	1.5	ft
Wall W =	180	psf
X =	1.2	from Sect 4.5.3.7
S_x =	0.86	from 4.5.2.1 (2-1)
A_p =	54.68	sf, area to anchor
T_c =	10129.25	lb **Tc > Ta NG**
m =	3.0	from Table 12-3 for Life Safety
Anchor Capacity =	5845.5	lb, per TMS 3.1.6.3.1.2

Tier 2 - Sect. 5.7.1.1 and 7.2.11.1

Height =	13.67	ft
Wall thickness =	1.5	ft
Ratio =	9.11	
X =	1.3	from Table 7-2
K_a =	1.515	Eq 7-11
K_h =	1	Eq 7-12
F_p =	93.55	psf, from 7.2.11.1 (7-9)
F_{pMIN} =	70.90	psf, from 7.2.11.1 (7-10)
T_c =	5115.27	lb **Tc < Ta OK**
DCR =	0.88	

Proportions - Check Height to Thickness Rations, Sect. 5.5.3.1.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

Top story height =	9.00	ft			
Wall thickness =	1	ft	Ratio:	9.0	**OK**
Bell Lounge h =	13.67	ft			
Wall thickness =	1.33	ft	Ratio:	10.3	**NG**
First story height =	10.00	ft			

Wall thickness =	1.5	ft	Ratio:	6.7	**OK**
Other story height =	16.5	ft			
Wall thickness =	1.5	ft	Ratio:	11.0	**OK**

Tier 2 - Bell Tower Lounge - Sect. 11.3.3.2

Height =	13.67	ft	
Wall thickness =	1.33	ft	
Ratio =	10.28		
X =	1	from Table 7-2	
F_p =	136.88	psf, from 7.2.11.2 (7-13)	
OOP Moment =	3197.237	ft-lbs	
F_t =	75.31	psi	**> Ft = 45 psi NG**
DCR =	1.67		

Tier 2 - Third Floor - Sect. 11.3.3.2

Height =	9.00	ft	
Wall thickness =	1	ft	
Ratio =	9.00		
X =	1	from Table 7-2	
F_p =	102.91	psf, from 7.2.11.2 (7-13)	
OOP Moment =	1042.008	ft-lbs	
F_t =	43.42	psi	**< Ft = 45 psi OK**
DCR =	0.96		

For Collapse Prevention - Bell Tower Lounge - Sect. 11.3.3.2

S_{X1} =	0.58	from 4.5.2.1
TOP Allowable h/t =	9	**per Table 11-5, NG**
1st Allowable h/t =	13	**per Table 11-5, OK**
Other h/t =	15	**per Table 11-5, OK**

Diaphragms - Openings at Shear Walls, Sect. 5.6.1.3

Bell Tower Lounge Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	8.50	ft	
Diaphragm V =	434	plf, from page	
Revised Dia. V =	575.92	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	1.28		

Wall OOP, Sect. 11.3.3.2

Height =	27.92	ft	
Wall thickness =	1.5	ft	
Ratio =	18.61		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	15042.09	ft-lbs	
F_t =	278.56	psi	**> Ft = 45 psi NG**
DCR =	6.19		

3rd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	16.75	ft	
Diaphragm V =	534	plf, from page	
Revised Dia. V =	1037.39	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	2.31		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	9128.427	ft-lbs	
F_t =	169.04	psi	**> Ft = 45 psi NG**
DCR =	3.76		

Diaphragm strength, Sect. 12.5.2.1

GL E Opening =	9.75	ft	
Diaphragm V =	536	plf, from page	
Revised Dia. V =	1044.90	plf	**NG**
Dia. Capacity =	450	plf, from page	
DCR =	2.32		

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft	
Wall thickness =	1.5	ft	
Ratio =	14.50		
X =	1	from Table 7-2	
F_p =	154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	9128.427	ft-lbs	
F_t =	169.04	psi	**> Ft = 45 psi NG**
DCR =	3.76		

3rd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft	
Diaphragm V =	395	plf, from page	
Revised Dia. V =	447.75	plf	**OK**

Dia. Capacity =	450	plf, from page
DCR =	0.99	

Wall OOP, Sect. 11.3.3.2

Height =	21.75	ft
Wall thickness =	1.5	ft
Ratio =	14.50	
X =	1	from Table 7-2
F_p =	154.37	psf, from 7.2.11.2 (7-13)
OOP Moment =	9128.427	ft-lbs
F_t =	169.04	psi **> Ft = 45 psi NG**
DCR =	3.76	

2nd Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 & 6 Opening =	14.25	ft
Diaphragm V =	837	plf, from page
Revised Dia. V =	1425.20	plf **NG**
Dia. Capacity =	450	plf, from page
DCR =	3.17	

Wall OOP, Sect. 11.3.3.2

Height =	31.50	ft
Wall thickness =	1.5	ft
Ratio =	21.00	
X =	1	from Table 7-2
F_p =	154.37	psf, from 7.2.11.2 (7-13)
OOP Moment =	19146.9	ft-lbs
F_t =	354.57	psi **> Ft = 45 psi NG**
DCR =	7.88	

2nd Floor West Stairs Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL A Opening =	8.50	ft
Diaphragm V =	624	plf, from page
Revised Dia. V =	708.35	plf **NG**
Dia. Capacity =	450	plf, from page
DCR =	1.57	

Wall OOP, Sect. 11.3.3.2

Height =	16.50	ft
Wall thickness =	1.5	ft
Ratio =	11.00	
X =	1	from Table 7-2
F_p =	154.37	psf, from 7.2.11.2 (7-13)
OOP Moment =	5253.459	ft-lbs
F_t =	97.29	psi **> Ft = 45 psi NG**
DCR =	2.16	

1st Floor East Foyer Floor Opening

Diaphragm strength, Sect. 12.5.2.1

GL 3 Opening =	9.00	ft
Diaphragm V =	447	plf, from page
Revised Dia. V =	605.35	plf **NG**
Dia. Capacity =	450	plf, from page
DCR =	1.35	

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft
Wall thickness =	1.5	ft
Ratio =	17.00	
X =	1	from Table 7-2
F_p =	154.37	psf, from 7.2.11.2 (7-13)
OOP Moment =	12547.52	ft-lbs
F_t =	232.36	psi **> Ft = 45 psi NG**

Diaphragm strength, Sect. 12.5.2.1

GL 6 Opening =	13.33	ft
Diaphragm V =	447	plf, from page
Revised Dia. V =	729.16	plf **NG**
Dia. Capacity =	450	plf, from page
DCR =	1.62	

Wall OOP, Sect. 11.3.3.2

Height =	25.50	ft
Wall thickness =	1.5	ft
Ratio =	17.00	
X =	1	from Table 7-2
F_p =	154.37	psf, from 7.2.11.2 (7-13)
OOP Moment =	12547.52	ft-lbs
F_t =	232.36	psi **> Ft = 45 psi NG**

DCR =	5.16				DCR =	5.16	

Stair Enclosures - Check Height to Thickness Rations, Sect. A.7.10.2

Tier 1 - Quick Check, Sect. 5.5.3.1.2

3rd Floor Opng =	21.75	ft			
Wall thickness =	1	ft	Ratio:	21.8	**> 12, NG**
2nd Floor Opng =	31.50	ft			
Wall thickness =	1.5	ft	Ratio:	21.0	**> 12, NG**
First story height =	25.50	ft			
Wall thickness =	1.5	ft	Ratio:	17.0	**> 12, NG**

Tier 2 - 3rd Floor Opening - Sect. Sect. 11.3.3.2				**2rd Floor Opening**			**1st Floor Opening**		
Height =	21.75	ft		31.50	ft		25.50	ft	
Wall thickness =	1	ft		1.5	ft		1.5	ft	
Ratio =	21.75			21.00			17.00		
X =	1	from Table 7-2		1	from Table 7-2		1	from Table 7-2	
F_p =	102.91	psf, from 7.2.11.2 (7-13)		154.37	psf, from 7.2.11.2 (7-13)		154.37	psf, from 7.2.11.2 (7-13)	
OOP Moment =	6085.618	ft-lbs		19146.9	ft-lbs		12547.52	ft-lbs	
F_t =	253.57	psi	**> Ft = 45 psi NG**	354.57	psi	**> Ft = 45 psi NG**	232.36	psi	**> Ft = 45 psi NG**
DCR =	5.63			7.88			5.16		

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS AT ROOF **Shape Capacity = 0.49 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	30.21	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.49	
Tr/Tc =		
	0.49	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Steel Column/Beam Design - AISC 13th Addition

Shape: HSS LOWER **Shape Capacity = 0.68 < 1.0**

Size: 4X4X1/4

	ASD	
Pr =	41.34	k, axial compression load
Mr_x =	0.00	ft-k, strong axis moment
Mr_y =	0.00	ft-k, weak axis moment
Vr_x =	0.00	k, strong axis shear
Vr_y =	0.00	k, weak axis shear
Tr =	0.00	ft-k, torsional load
K_x =	1.00	*(Table C-C2.2, pg 16.1-240)*
K_y =	1.00	*(Table C-C2.2, pg 16.1-240)*
L_x =	10.00	ft
L_y =	10.00	ft
Lv =	N/A	ft, (for round shapes)
KL/r x =	78.95	
KL/r y =	78.95	
E =	29000	ksi
Fy =	46	ksi
Ag =	3.37	in^2
t =	0.233	in
Z_x =	4.69	in^3
Z_y =	4.69	in^3
S_x =	3.9	in^3
S_y =	3.9	in^3
I_x =	7.8	in^4
I_y =	7.8	in^4
r_x =	1.52	in
r_y =	1.52	in
Section is	Compact	in the strong axis (x)
Section is	Compact	in the weak axis (y)

Axial Capacity, Chapter E		
Aeff =	3.37	in^2
Qa =	1	*(Section E7, pg 16.1-42)*
Fe_x =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fe_y =	45.92	ksi, *(Section E3 pg 16.1-33)*
Fcr_x =	30.25	ksi, *(Section E3 pg 16.1-33)*
Fcr_y =	30.25	ksi, *(Section E3 pg 16.1-33)*
Pn_x =	102	k, *(Section E3 pg 16.1-33)*
Pn_y =	102	k, *(Section E3 pg 16.1-33)*
Moment Capacity, Chapter F		
$Seff_x$ =	3.9	in^3
$Seff_y$ =	3.90	in^3
Mn_x =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Mn_y =	17.98	*ft-k, (Section F7 pgs 16.1-55,56)*
Shear Capacity: Chapter G		
kv =	5	*(Section G5, pg 16.1-68)*
Cv_x =	1.00	*(Section G2, pg 16.1-65)*
Cv_y =	1.00	*(Section G2, pg 16.1-65)*
Aw_x =	1.54	in^2, *(Section G5, pg 16.1-68)*
Aw_y =	1.54	in^2, *(Section G5, pg 16.1-68)*
Fcr =	N/A	ksi, *(Section G6, pg 16.1-68)*
Vn_x =	42.6	*k, (Section G2, pg 16.1-65)*
Vn_y =	42.6	*k, (Section G2, pg 16.1-65)*
Torsional Capacity: Chapter H		
Fcr =	27.60	*ksi, (Section H3 pg 16.1-75)*
C =	6.56	torsional shear constant
Tn =	15.1	ft-k, *(Section H3 pg 16.1-74)*

Allowable Capacities: Rn / Ω (ASD); Rn * Φ (LRFD)

(ASD)	Pc, k	Mc, ft-k	Vc, k	Tc, ft-k
x-axis	**61.0**	**10.8**	**25.5**	**9.0**
y-axis		**10.8**	**25.5**	

Interaction Equations:

Pr/Pc =	0.68	
Tr/Tc =		
	0.68	< 1.0 OK

Equation H1-1a, AISC 13 ed., pg 16.1-70

Use HSS 4X4X1/4

Schematic Shear Wall Design for Gridline 1

Current Date: 8/8/2016 5:17 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\SPOE GL 1.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	26.50 [ft]
Total length	:	21.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 2

Story	Story height [ft]	Wall thickness [in]
1	10.00	12.00
2	16.50	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	6.50	13.00	8.00	8.00
Lower left	6.50	3.50	8.00	5.00

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	14400.00	0.00	0.00
1	DL	Vertical	14400.00	0.00	21.00
2	DL	Vertical	13700.00	0.00	0.00
2	DL	Vertical	13700.00	0.00	21.00
2	EQ	Horizontal	244194.00	0.00	0.00
1	EQ	Horizontal	79744.00	0.00	0.00

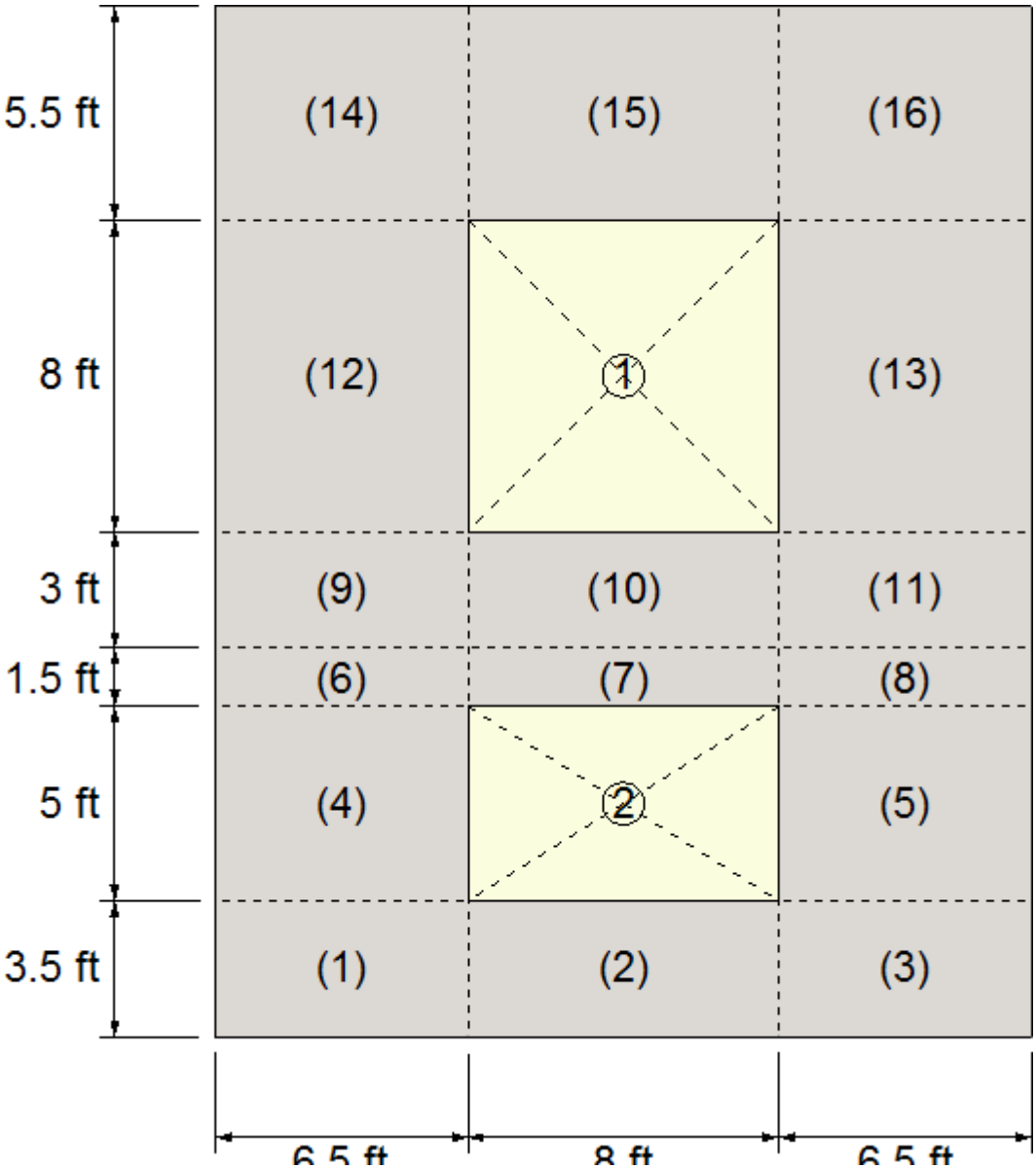
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
2	DL	Vertical	5460.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	6.50	3.50
2	6.50	0.00	8.00	3.50
3	14.50	0.00	6.50	3.50
4	0.00	3.50	6.50	5.00
5	14.50	3.50	6.50	5.00
6	0.00	8.50	6.50	1.50
7	6.50	8.50	8.00	1.50
8	14.50	8.50	6.50	1.50
9	0.00	10.00	6.50	3.00
10	6.50	10.00	8.00	3.00
11	14.50	10.00	6.50	3.00
12	0.00	13.00	6.50	8.00
13	14.50	13.00	6.50	8.00
14	0.00	21.00	6.50	5.50
15	6.50	21.00	8.00	5.50
16	14.50	21.00	6.50	5.50

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	6-#8	14.00	38.73
2	6-#5	18.00	19.36
3	5-#5	18.00	19.36
4	6-#8	14.00	38.73
5	5-#5	18.00	19.36
6	6-#8	14.00	38.73
7	6-#5	18.00	19.36
8	5-#5	18.00	19.36
9	6-#6	14.00	23.24
10	6-#5	18.00	19.36
11	5-#5	18.00	19.36
12	6-#6	14.00	23.24
13	5-#5	18.00	19.36
14	6-#6	14.00	23.24
15	6-#5	18.00	19.36
16	5-#5	18.00	19.36

Intermediate results for axial-bending

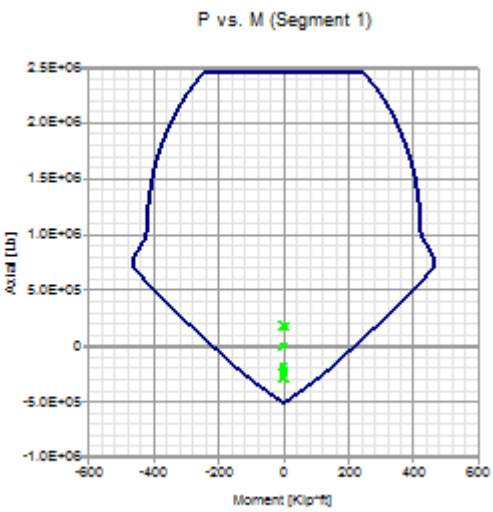
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.75	10.13
2	DC1 (Top)	0.67	10.31
3	DC1 (Top)	1.20	10.31
4	DC1 (Top)	1.73	10.13
5	DC1 (Top)	1.18	10.31
6	DC1 (Top)	1.71	10.13
7	DC1 (Top)	0.70	10.31
8	DC1 (Top)	1.15	10.31
9	DC1 (Top)	1.26	6.25
10	DC1 (Top)	0.62	6.31
11	DC1 (Top)	1.01	6.31
12	DC1 (Top)	1.23	6.25
13	DC1 (Top)	0.97	6.31
14	DC1 (Top)	1.16	6.25
15	DC1 (Top)	0.79	6.31
16	DC1 (Top)	0.85	6.31

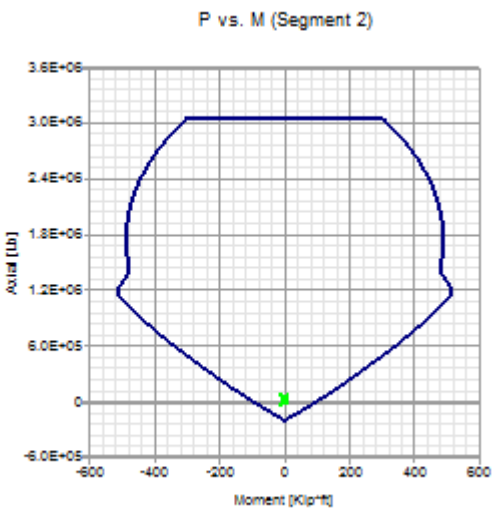
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	184613.50	0.00	287.02	0.00
2	DC1 (Top)	20518.34	0.00	105.98	0.00
3	DC1 (Top)	184599.20	0.00	159.40	0.00
4	DC1 (Top)	175520.70	0.00	283.64	0.00
5	DC1 (Top)	175313.90	0.00	155.68	0.00
6	DC1 (Top)	164501.30	0.00	279.55	0.00
7	DC1 (Top)	30136.45	0.00	110.21	0.00

8	DC1 (Top)	164529.60	0.00	151.35	0.00
9	DC1 (Top)	114858.70	0.00	110.98	0.00
10	DC1 (Top)	3460.10	0.00	64.09	0.00
11	DC1 (Top)	114772.30	0.00	83.79	0.00
12	DC1 (Top)	99824.79	0.00	107.61	0.00
13	DC1 (Top)	99688.89	0.00	80.02	0.00
14	DC1 (Top)	60993.07	0.00	98.92	0.00
15	DC1 (Top)	61577.74	0.00	80.83	0.00
16	DC1 (Top)	60958.81	0.00	69.99	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Top)	184613.50	2457131.00	0.08
2	D2 (Bottom)	49206.52	3045239.00	0.02
3	D2 (Bottom)	545965.70	2474051.00	0.22
4	D1 (Bottom)	184615.20	2457131.00	0.08
5	D2 (Bottom)	495087.40	2474051.00	0.20
6	DC1 (Bottom)	175518.90	2457131.00	0.07
7	DC1 (Top)	30136.45	3045239.00	0.01
8	D2 (Bottom)	393334.20	2474051.00	0.16
9	D1 (Bottom)	121164.30	1640845.00	0.07
10	DC1 (Top)	3460.10	2026871.00	0.00
11	D2 (Bottom)	342123.70	1646627.00	0.21
12	D1 (Bottom)	114858.80	1640845.00	0.07
13	D2 (Bottom)	287005.30	1646627.00	0.17
14	D2 (Top)	115293.10	1640845.00	0.07
15	DC1 (Top)	61577.74	2026871.00	0.03
16	D2 (Bottom)	176701.00	1646627.00	0.11

Axial tension

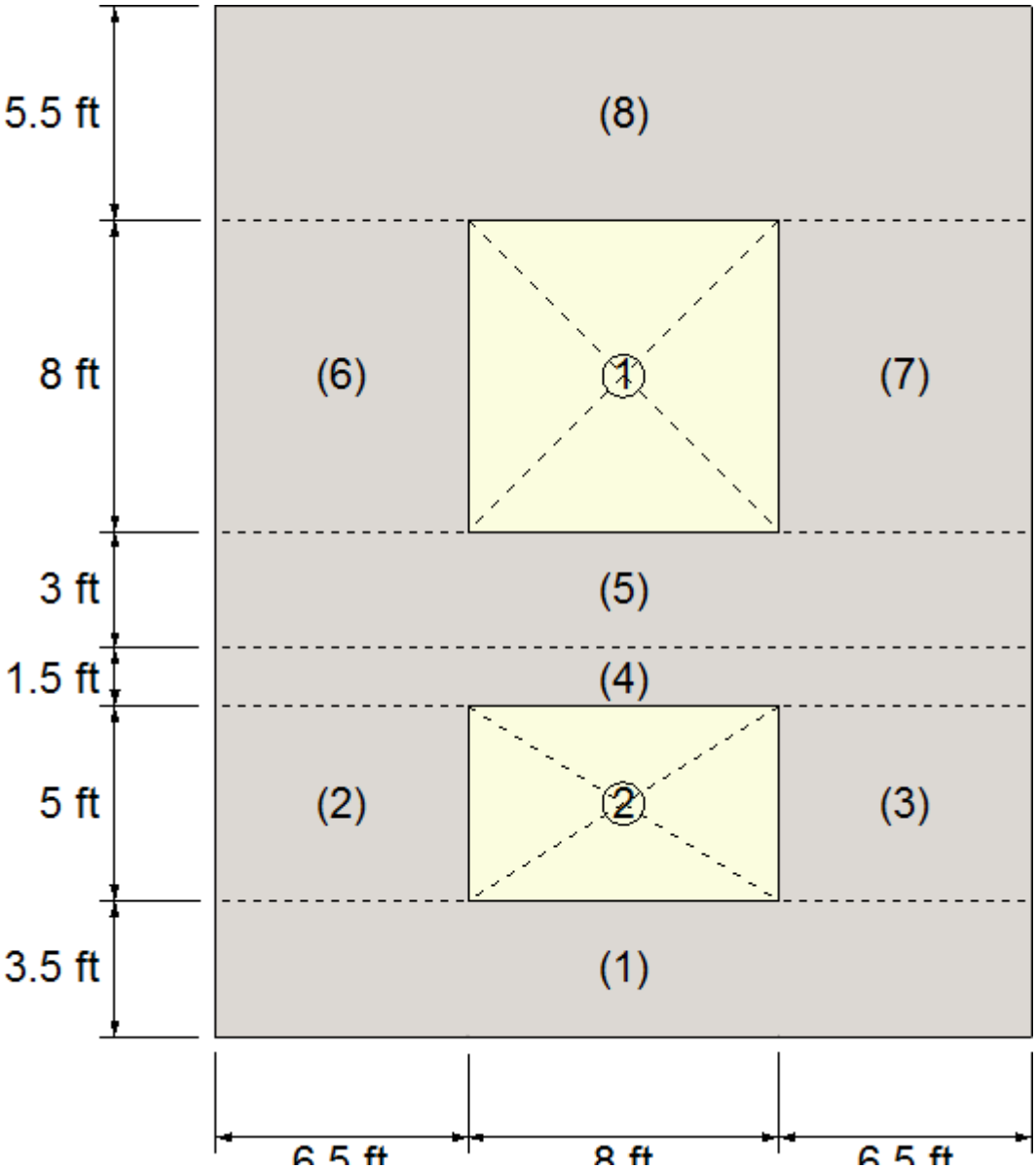
Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	289862.10	511920.00	0.57
2	DC1 (Top)	0.00	200880.00	0.00
3	DC1 (Top)	0.00	167400.00	0.00
4	D3 (Bottom)	220081.80	511920.00	0.43
5	DC1 (Top)	0.00	167400.00	0.00
6	D3 (Top)	142603.40	511920.00	0.28
7	DC1 (Top)	0.00	200880.00	0.00
8	DC1 (Top)	0.00	167400.00	0.00
9	D3 (Bottom)	172260.80	285120.00	0.60
10	D3 (Bottom)	1481.03	200880.00	0.01
11	DC1 (Top)	0.00	167400.00	0.00
12	D3 (Bottom)	113957.10	285120.00	0.40
13	DC1 (Top)	0.00	167400.00	0.00
14	D3 (Bottom)	33341.92	285120.00	0.12
15	DC1 (Top)	0.00	200880.00	0.00
16	DC1 (Top)	0.00	167400.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	91760.658	0.00
2	DC1 (Top)	0.000	115027.610	0.00
3	DC1 (Top)	0.000	93459.929	0.00
4	DC1 (Top)	0.000	91760.658	0.00
5	DC1 (Top)	0.000	93459.929	0.00
6	DC1 (Top)	0.000	91760.658	0.00
7	DC1 (Top)	0.000	115027.610	0.00
8	DC1 (Top)	0.000	93459.929	0.00
9	DC1 (Top)	0.000	56642.381	0.00
10	DC1 (Top)	0.000	70410.837	0.00
11	DC1 (Top)	0.000	57208.805	0.00
12	DC1 (Top)	0.000	56642.381	0.00
13	DC1 (Top)	0.000	57208.805	0.00
14	DC1 (Top)	0.000	56642.381	0.00
15	DC1 (Top)	0.000	70410.837	0.00
16	DC1 (Top)	0.000	57208.805	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	21.00	3.50
2	0.00	3.50	6.50	5.00
3	14.50	3.50	6.50	5.00
4	0.00	8.50	21.00	1.50
5	0.00	10.00	21.00	3.00
6	0.00	13.00	6.50	8.00
7	14.50	13.00	6.50	8.00
8	0.00	21.00	21.00	5.50

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	6-#8	14.00	38.73	3-#5	18.00	25.17
	6-#5	18.00	38.73	3-#5	18.00	25.17
	5-#5	18.00	38.73	3-#5	18.00	25.17
2	6-#8	14.00	38.73	4-#5	18.00	25.17
3	5-#5	18.00	19.36	4-#5	18.00	25.17
4	6-#8	14.00	38.73	1-#5	18.00	25.17
	6-#5	18.00	38.73	1-#5	18.00	25.17
	5-#5	18.00	38.73	1-#5	18.00	25.17
5	6-#6	14.00	23.24	2-#5	18.00	25.17
	6-#5	18.00	23.24	2-#5	18.00	25.17
	5-#5	18.00	23.24	2-#5	18.00	25.17
6	6-#6	14.00	23.24	6-#5	18.00	25.17
7	5-#5	18.00	19.36	6-#5	18.00	25.17
8	6-#6	14.00	23.24	4-#5	18.00	25.17
	6-#5	18.00	23.24	4-#5	18.00	25.17
	5-#5	18.00	23.24	4-#5	18.00	25.17

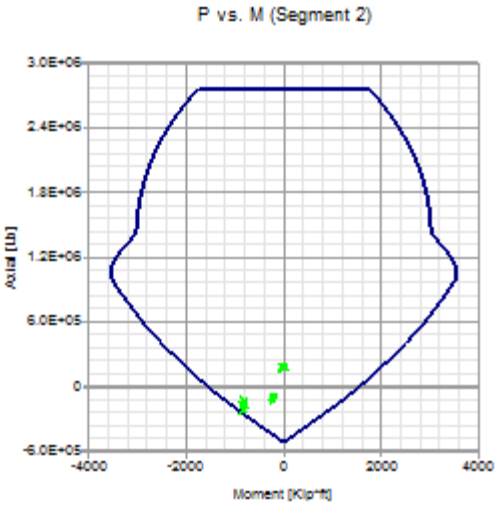
Intermediate results for axial-bending

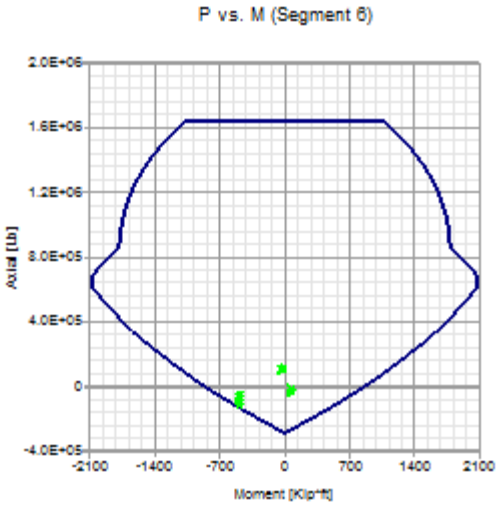
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	22.56	201.60
2	D3 (Bottom)	3.65	62.40
3	D3 (Bottom)	13.97	62.40
4	D3 (Bottom)	22.34	201.60
5	D3 (Bottom)	24.53	201.60
6	D3 (Bottom)	4.64	62.40
7	D3 (Top)	10.73	62.40
8	D3 (Bottom)	24.58	201.60

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	257305.90	-7070.76	8542.22	0.83
2	D3 (Bottom)	-215314.10	-822.36	929.14	0.89
3	D3 (Bottom)	452265.10	-803.77	1737.60	0.46
4	D3 (Bottom)	246898.20	-4409.52	8447.52	0.52
5	D3 (Bottom)	142455.00	-4252.57	6841.63	0.62
6	D3 (Bottom)	-113718.80	-485.71	534.37	0.91
7	D3 (Top)	159380.10	433.54	958.57	0.45
8	D3 (Bottom)	144021.20	-1364.13	6855.92	0.20

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	398671.60	7976420.00	0.05
2	D1 (Max)	190480.80	2752907.00	0.07
3	D2 (Bottom)	492170.30	2474051.00	0.20
4	DC1 (Bottom)	373436.70	7976420.00	0.05
5	DC1 (Bottom)	244191.10	5314343.00	0.05
6	DC1 (Bottom)	114870.10	1640845.00	0.07
7	D2 (Bottom)	286815.60	1646627.00	0.17
8	D2 (Top)	219752.70	5314343.00	0.04

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	880200.00	0.00
2	D3 (Bottom)	215314.10	511920.00	0.42
3	DC1 (Top)	0.00	167400.00	0.00
4	DC1 (Top)	0.00	880200.00	0.00
5	DC1 (Top)	0.00	653400.00	0.00
6	D3 (Bottom)	113718.80	285120.00	0.40
7	DC1 (Top)	0.00	167400.00	0.00
8	DC1 (Top)	0.00	653400.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Bottom)	325665.720	783691.960	0.42
2	D3 (Max)	185856.200	214856.290	0.87
3	D3 (Top)	161752.570	294183.880	0.55
4	D3 (Max)	367501.030	813460.230	0.45
5	D3 (Max)	261903.980	643530.050	0.41

6	D3 (Bottom)	104007.740	175365.140	0.59
7	D3 (Top)	150130.910	216329.980	0.69
8	D3 (Max)	250452.380	639959.860	0.39

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Schematic Shear Wall Design for Gridline 2

Current Date: 8/4/2016 3:49 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\SPOE GL 2 v2.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	50.67 [ft]
Total length	:	64.75 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 4

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	10.00
3	14.17	8.00
4	10.00	8.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	7.50	13.00	4.00	8.00
Lower left	19.00	13.00	4.00	8.00
Lower left	30.25	13.00	4.00	8.00
Lower left	41.75	13.00	4.00	8.00
Lower left	53.25	13.00	4.00	8.00
Lower left	30.25	3.50	4.00	5.00
Lower left	41.75	3.50	4.00	5.00
Lower left	53.25	3.50	4.00	5.00
Lower left	7.50	30.00	4.00	11.75
Lower left	19.00	30.00	4.00	11.75
Lower left	30.25	30.00	4.00	11.75
Lower left	41.75	30.00	4.00	11.75
Lower left	53.25	30.00	4.00	11.75

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	14400.00	0.00	0.00
1	DL	Vertical	14400.00	0.00	64.75
2	DL	Vertical	13700.00	0.00	0.00
2	DL	Vertical	13700.00	0.00	64.75
3	DL	Vertical	13100.00	0.00	0.00
3	DL	Vertical	13100.00	0.00	64.75
4	EQ	Horizontal	219850.00	0.00	0.00
3	EQ	Horizontal	325498.00	0.00	0.00
2	EQ	Horizontal	310112.00	0.00	0.00
1	EQ	Horizontal	236300.00	0.00	0.00

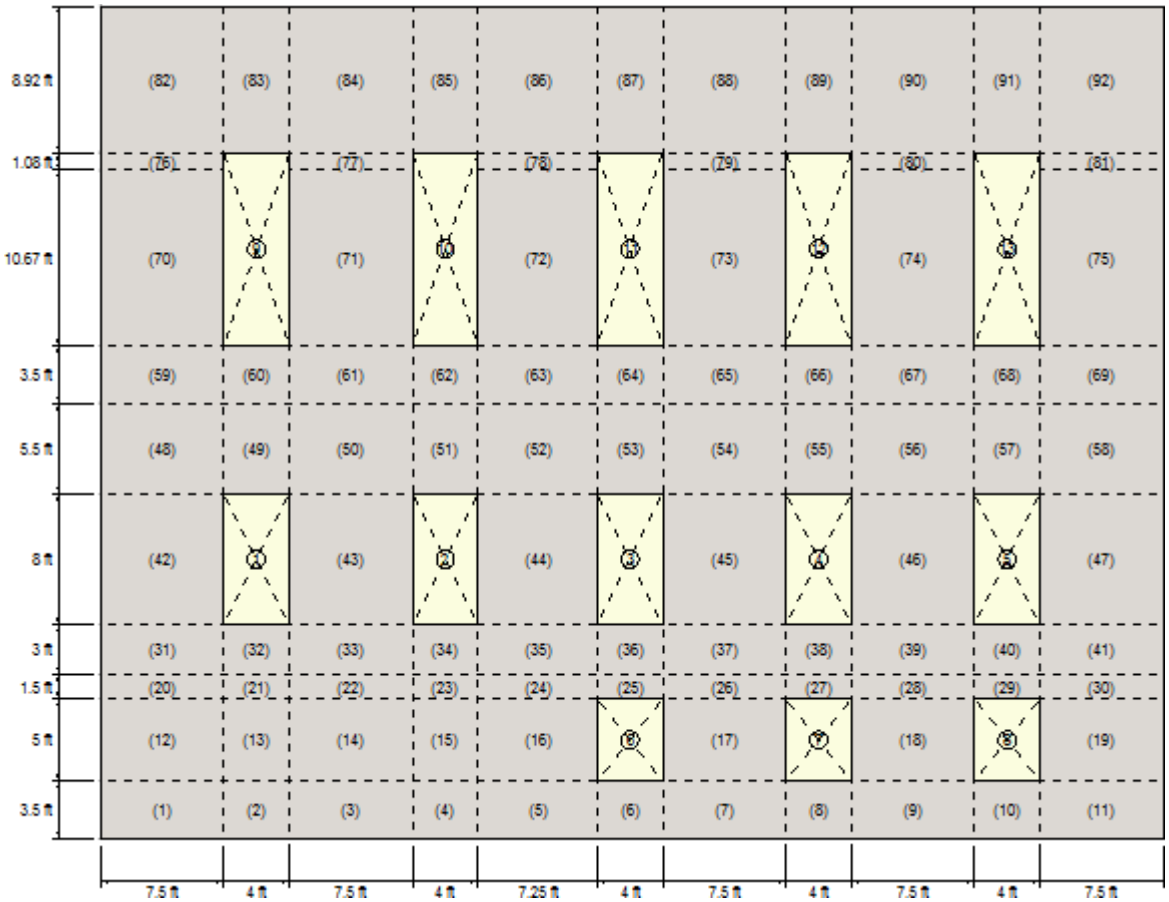
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
4	DL	Vertical	1050.00	0.00
3	DL	Vertical	1710.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	7.50	3.50
2	7.50	0.00	4.00	3.50
3	11.50	0.00	7.50	3.50
4	19.00	0.00	4.00	3.50
5	23.00	0.00	7.25	3.50
6	30.25	0.00	4.00	3.50
7	34.25	0.00	7.50	3.50
8	41.75	0.00	4.00	3.50
9	45.75	0.00	7.50	3.50
10	53.25	0.00	4.00	3.50
11	57.25	0.00	7.50	3.50
12	0.00	3.50	7.50	5.00
13	7.50	3.50	4.00	5.00
14	11.50	3.50	7.50	5.00
15	19.00	3.50	4.00	5.00
16	23.00	3.50	7.25	5.00
17	34.25	3.50	7.50	5.00
18	45.75	3.50	7.50	5.00
19	57.25	3.50	7.50	5.00
20	0.00	8.50	7.50	1.50
21	7.50	8.50	4.00	1.50
22	11.50	8.50	7.50	1.50
23	19.00	8.50	4.00	1.50
24	23.00	8.50	7.25	1.50
25	30.25	8.50	4.00	1.50
26	34.25	8.50	7.50	1.50

27	41.75	8.50	4.00	1.50
28	45.75	8.50	7.50	1.50
29	53.25	8.50	4.00	1.50
30	57.25	8.50	7.50	1.50
31	0.00	10.00	7.50	3.00
32	7.50	10.00	4.00	3.00
33	11.50	10.00	7.50	3.00
34	19.00	10.00	4.00	3.00
35	23.00	10.00	7.25	3.00
36	30.25	10.00	4.00	3.00
37	34.25	10.00	7.50	3.00
38	41.75	10.00	4.00	3.00
39	45.75	10.00	7.50	3.00
40	53.25	10.00	4.00	3.00
41	57.25	10.00	7.50	3.00
42	0.00	13.00	7.50	8.00
43	11.50	13.00	7.50	8.00
44	23.00	13.00	7.25	8.00
45	34.25	13.00	7.50	8.00
46	45.75	13.00	7.50	8.00
47	57.25	13.00	7.50	8.00
48	0.00	21.00	7.50	5.50
49	7.50	21.00	4.00	5.50
50	11.50	21.00	7.50	5.50
51	19.00	21.00	4.00	5.50
52	23.00	21.00	7.25	5.50
53	30.25	21.00	4.00	5.50
54	34.25	21.00	7.50	5.50
55	41.75	21.00	4.00	5.50
56	45.75	21.00	7.50	5.50
57	53.25	21.00	4.00	5.50
58	57.25	21.00	7.50	5.50
59	0.00	26.50	7.50	3.50
60	7.50	26.50	4.00	3.50
61	11.50	26.50	7.50	3.50
62	19.00	26.50	4.00	3.50
63	23.00	26.50	7.25	3.50
64	30.25	26.50	4.00	3.50
65	34.25	26.50	7.50	3.50
66	41.75	26.50	4.00	3.50
67	45.75	26.50	7.50	3.50
68	53.25	26.50	4.00	3.50
69	57.25	26.50	7.50	3.50
70	0.00	30.00	7.50	10.67
71	11.50	30.00	7.50	10.67
72	23.00	30.00	7.25	10.67
73	34.25	30.00	7.50	10.67
74	45.75	30.00	7.50	10.67
75	57.25	30.00	7.50	10.67
76	0.00	40.67	7.50	1.08
77	11.50	40.67	7.50	1.08
78	23.00	40.67	7.25	1.08
79	34.25	40.67	7.50	1.08
80	45.75	40.67	7.50	1.08
81	57.25	40.67	7.50	1.08
82	0.00	41.75	7.50	8.92
83	7.50	41.75	4.00	8.92
84	11.50	41.75	7.50	8.92
85	19.00	41.75	4.00	8.92
86	23.00	41.75	7.25	8.92
87	30.25	41.75	4.00	8.92
88	34.25	41.75	7.50	8.92
89	41.75	41.75	4.00	8.92
90	45.75	41.75	7.50	8.92

91	53.25	41.75	4.00	8.92
92	57.25	41.75	7.50	8.92

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	7-#7	14.00	33.89
2	3-#5	16.00	19.36
3	6-#5	16.00	19.36
4	3-#5	16.00	19.36
5	6-#5	16.00	19.36
6	3-#5	16.00	19.36
7	6-#5	16.00	19.36
8	3-#5	16.00	19.36
9	6-#5	16.00	19.36
10	3-#5	16.00	19.36
11	6-#5	16.00	19.36
12	7-#7	14.00	33.89
13	3-#5	16.00	19.36
14	6-#5	16.00	19.36
15	3-#5	16.00	19.36
16	6-#5	16.00	19.36
17	6-#5	16.00	19.36
18	6-#5	16.00	19.36
19	6-#5	16.00	19.36
20	7-#7	14.00	33.89
21	3-#5	16.00	19.36
22	6-#5	16.00	19.36
23	3-#5	16.00	19.36
24	6-#5	16.00	19.36
25	3-#5	16.00	19.36
26	6-#5	16.00	19.36
27	3-#5	16.00	19.36
28	6-#5	16.00	19.36
29	3-#5	16.00	19.36
30	6-#5	16.00	19.36
31	9-#5	10.00	19.36
32	3-#5	18.00	19.36
33	6-#5	16.00	19.36
34	3-#5	18.00	19.36
35	5-#5	18.00	19.36
36	3-#5	18.00	19.36
37	6-#5	16.00	19.36
38	3-#5	18.00	19.36
39	5-#5	18.00	19.36
40	3-#5	18.00	19.36
41	5-#5	18.00	19.36
42	9-#5	10.00	19.36
43	6-#5	16.00	19.36
44	5-#5	18.00	19.36
45	6-#5	16.00	19.36
46	5-#5	18.00	19.36
47	5-#5	18.00	19.36
48	9-#5	10.00	19.36
49	3-#5	18.00	19.36
50	6-#5	16.00	19.36
51	3-#5	18.00	19.36
52	5-#5	18.00	19.36

53	3-#5	18.00	19.36
54	6-#5	16.00	19.36
55	3-#5	18.00	19.36
56	5-#5	18.00	19.36
57	3-#5	18.00	19.36
58	5-#5	18.00	19.36
59	5-#5	18.00	19.36
60	3-#5	18.00	19.36
61	5-#5	18.00	19.36
62	3-#5	18.00	19.36
63	5-#5	18.00	19.36
64	3-#5	18.00	19.36
65	5-#5	18.00	19.36
66	3-#5	18.00	19.36
67	5-#5	18.00	19.36
68	3-#5	18.00	19.36
69	5-#5	18.00	19.36
70	5-#5	18.00	19.36
71	5-#5	18.00	19.36
72	5-#5	18.00	19.36
73	5-#5	18.00	19.36
74	5-#5	18.00	19.36
75	5-#5	18.00	19.36
76	6-#8	16.00	38.73
77	8-#5	12.00	19.36
78	8-#5	12.00	19.36
79	8-#5	12.00	19.36
80	5-#5	18.00	19.36
81	5-#5	18.00	19.36
82	6-#8	16.00	38.73
83	3-#5	18.00	19.36
84	8-#5	12.00	19.36
85	3-#5	18.00	19.36
86	8-#5	12.00	19.36
87	3-#5	18.00	19.36
88	8-#5	12.00	19.36
89	3-#5	18.00	19.36
90	5-#5	18.00	19.36
91	3-#5	18.00	19.36
92	5-#5	18.00	19.36

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	1.54	13.19
2	DC1 (Top)	1.02	13.31
3	DC1 (Top)	1.03	13.31
4	DC1 (Top)	1.00	13.31
5	DC1 (Top)	1.08	13.31
6	DC1 (Top)	0.83	13.31
7	DC1 (Top)	1.12	13.31
8	DC1 (Top)	0.84	13.31
9	DC1 (Top)	1.13	13.31
10	DC1 (Top)	0.77	13.31
11	DC1 (Top)	1.20	13.31
12	DC1 (Top)	1.52	13.19
13	DC1 (Top)	0.97	13.31
14	DC1 (Top)	1.02	13.31
15	DC1 (Top)	0.96	13.31
16	DC1 (Top)	1.08	13.31
17	DC1 (Top)	1.12	13.31

18	DC1 (Top)	1.10	13.31
19	DC1 (Top)	1.16	13.31
20	DC1 (Top)	1.56	13.06
21	DC1 (Top)	0.98	13.19
22	DC1 (Top)	1.03	13.19
23	DC1 (Top)	0.96	13.19
24	DC1 (Top)	1.03	13.19
25	DC1 (Top)	0.82	13.19
26	DC1 (Top)	1.09	13.19
27	DC1 (Top)	0.71	13.19
28	DC1 (Top)	1.11	13.19
29	DC1 (Top)	0.86	13.19
30	DC1 (Top)	1.15	13.19
31	DC1 (Top)	1.19	8.31
32	DC1 (Top)	0.78	8.31
33	DC1 (Top)	1.01	8.31
34	DC1 (Top)	0.78	8.31
35	DC1 (Top)	0.92	8.31
36	DC1 (Top)	0.68	8.31
37	DC1 (Top)	1.00	8.31
38	DC1 (Top)	0.74	8.31
39	DC1 (Top)	0.93	8.31
40	DC1 (Top)	0.74	8.31
41	DC1 (Top)	0.94	8.31
42	DC1 (Top)	1.19	8.31
43	DC1 (Top)	1.01	8.31
44	DC1 (Top)	0.89	8.31
45	DC1 (Top)	0.94	8.31
46	DC1 (Top)	0.88	8.31
47	DC1 (Top)	0.93	8.31
48	DC1 (Top)	1.15	8.31
49	DC1 (Top)	0.83	8.31
50	DC1 (Top)	0.91	8.31
51	DC1 (Top)	0.82	8.31
52	DC1 (Top)	0.83	8.31
53	DC1 (Top)	0.82	8.31
54	DC1 (Top)	0.90	8.31
55	DC1 (Top)	0.82	8.31
56	DC1 (Top)	0.81	8.31
57	DC1 (Top)	0.83	8.31
58	DC1 (Top)	0.87	8.31
59	DC1 (Top)	0.73	6.31
60	DC1 (Top)	0.69	6.31
61	DC1 (Top)	0.74	6.31
62	DC1 (Top)	0.69	6.31
63	DC1 (Top)	0.75	6.31
64	DC1 (Top)	0.65	6.31
65	DC1 (Top)	0.72	6.31
66	DC1 (Top)	0.69	6.31
67	DC1 (Top)	0.73	6.31
68	DC1 (Top)	0.69	6.31
69	DC1 (Top)	0.74	6.31
70	DC1 (Top)	0.71	6.31
71	DC1 (Top)	0.72	6.31
72	DC1 (Top)	0.73	6.31
73	DC1 (Top)	0.71	6.31
74	DC1 (Top)	0.72	6.31
75	DC1 (Top)	0.71	6.31
76	DC1 (Top)	1.40	6.00
77	DC1 (Top)	0.93	6.31
78	DC1 (Top)	0.95	6.31
79	DC1 (Top)	0.92	6.31
80	DC1 (Top)	0.62	6.31
81	DC1 (Top)	0.59	6.31

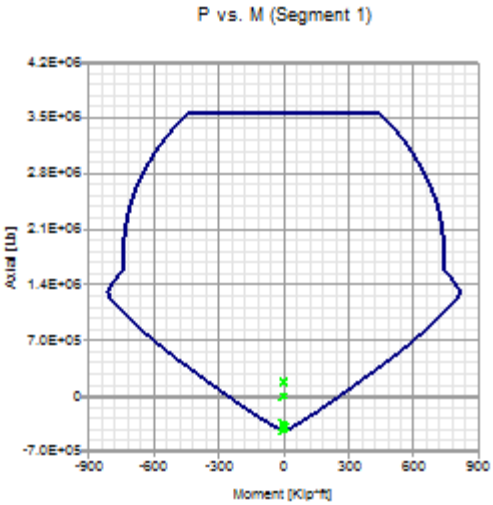
82	DC1 (Top)	1.34	6.13
83	DC1 (Top)	0.65	6.31
84	DC1 (Top)	0.90	6.31
85	DC1 (Top)	0.65	6.31
86	DC1 (Top)	0.92	6.31
87	DC1 (Top)	0.65	6.31
88	DC1 (Top)	0.90	6.31
89	DC1 (Top)	0.65	6.31
90	DC1 (Top)	0.58	6.31
91	DC1 (Top)	0.65	6.31
92	DC1 (Top)	0.58	6.31

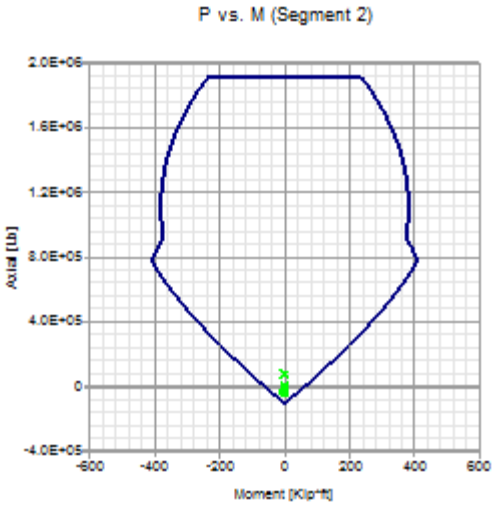
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	182387.20	0.00	349.98	0.00
2	DC1 (Top)	78757.71	0.00	104.78	0.00
3	DC1 (Top)	141287.00	0.00	199.71	0.00
4	DC1 (Top)	75877.26	0.00	103.24	0.00
5	DC1 (Top)	154833.90	0.00	206.38	0.00
6	DC1 (Top)	37163.75	0.00	82.41	0.00
7	DC1 (Top)	182815.00	0.00	221.97	0.00
8	DC1 (Top)	38747.76	0.00	83.29	0.00
9	DC1 (Top)	187196.50	0.00	224.32	0.00
10	DC1 (Top)	26741.58	0.00	76.28	0.00
11	DC1 (Top)	214227.70	0.00	238.82	0.00
12	DC1 (Top)	169847.00	0.00	343.67	0.00
13	DC1 (Top)	69004.83	0.00	99.55	0.00
14	DC1 (Top)	138565.00	0.00	198.25	0.00
15	DC1 (Top)	65929.69	0.00	97.89	0.00
16	DC1 (Top)	154934.20	0.00	206.43	0.00
17	DC1 (Top)	181501.10	0.00	221.27	0.00
18	DC1 (Top)	173966.70	0.00	217.23	0.00
19	DC1 (Top)	198054.90	0.00	230.15	0.00
20	DC1 (Top)	169032.90	0.00	342.59	0.00
21	DC1 (Top)	68241.23	0.00	99.16	0.00
22	DC1 (Top)	138251.10	0.00	198.15	0.00
23	DC1 (Top)	65167.95	0.00	97.50	0.00
24	DC1 (Top)	127429.90	0.00	191.77	0.00
25	DC1 (Top)	35609.66	0.00	81.51	0.00
26	DC1 (Top)	161890.60	0.00	210.87	0.00
27	DC1 (Top)	16561.17	0.00	70.30	0.00
28	DC1 (Top)	168300.70	0.00	214.32	0.00
29	DC1 (Top)	42035.46	0.00	85.04	0.00
30	DC1 (Top)	186108.40	0.00	223.89	0.00
31	DC1 (Top)	126500.60	0.00	156.20	0.00
32	DC1 (Top)	28552.35	0.00	50.51	0.00
33	DC1 (Top)	131325.20	0.00	125.68	0.00
34	DC1 (Top)	27990.62	0.00	50.30	0.00
35	DC1 (Top)	116045.40	0.00	108.99	0.00
36	DC1 (Top)	11947.81	0.00	44.31	0.00
37	DC1 (Top)	123872.40	0.00	123.19	0.00
38	DC1 (Top)	21713.72	0.00	47.96	0.00
39	DC1 (Top)	127958.50	0.00	113.48	0.00
40	DC1 (Top)	22392.95	0.00	48.21	0.00
41	DC1 (Top)	131944.00	0.00	114.82	0.00
42	DC1 (Top)	125853.10	0.00	156.00	0.00
43	DC1 (Top)	130721.70	0.00	125.48	0.00

44	DC1 (Top)	101976.00	0.00	104.27	0.00
45	DC1 (Top)	101924.90	0.00	115.88	0.00
46	DC1 (Top)	108959.60	0.00	107.09	0.00
47	DC1 (Top)	129559.80	0.00	114.02	0.00
48	DC1 (Top)	103977.10	0.00	149.10	0.00
49	DC1 (Top)	36492.59	0.00	53.48	0.00
50	DC1 (Top)	87056.55	0.00	110.92	0.00
51	DC1 (Top)	34637.67	0.00	52.79	0.00
52	DC1 (Top)	80070.15	0.00	96.66	0.00
53	DC1 (Top)	35143.76	0.00	52.98	0.00
54	DC1 (Top)	81727.15	0.00	109.14	0.00
55	DC1 (Top)	34739.34	0.00	52.82	0.00
56	DC1 (Top)	83700.77	0.00	98.16	0.00
57	DC1 (Top)	36643.22	0.00	53.54	0.00
58	DC1 (Top)	106115.90	0.00	106.14	0.00
59	DC1 (Top)	59823.98	0.00	70.41	0.00
60	DC1 (Top)	13206.30	0.00	35.40	0.00
61	DC1 (Top)	62150.36	0.00	71.07	0.00
62	DC1 (Top)	13004.83	0.00	35.35	0.00
63	DC1 (Top)	56136.49	0.00	69.17	0.00
64	DC1 (Top)	6681.82	0.00	33.53	0.00
65	DC1 (Top)	56688.92	0.00	69.52	0.00
66	DC1 (Top)	13472.32	0.00	35.48	0.00
67	DC1 (Top)	60294.37	0.00	70.55	0.00
68	DC1 (Top)	13143.29	0.00	35.38	0.00
69	DC1 (Top)	60813.90	0.00	70.69	0.00
70	DC1 (Top)	52565.93	0.00	68.34	0.00
71	DC1 (Top)	56049.15	0.00	69.33	0.00
72	DC1 (Top)	52287.18	0.00	68.07	0.00
73	DC1 (Top)	52947.23	0.00	68.45	0.00
74	DC1 (Top)	55002.14	0.00	69.04	0.00
75	DC1 (Top)	53692.48	0.00	68.66	0.00
76	DC1 (Top)	12703.30	0.00	140.54	0.00
77	DC1 (Top)	27781.89	0.00	89.09	0.00
78	DC1 (Top)	24834.33	0.00	87.80	0.00
79	DC1 (Top)	21904.34	0.00	87.66	0.00
80	DC1 (Top)	25392.85	0.00	60.46	0.00
81	DC1 (Top)	15504.89	0.00	57.49	0.00
82	DC1 (Top)	12010.14	0.00	140.29	0.00
83	DC1 (Top)	6573.22	0.00	33.50	0.00
84	DC1 (Top)	12682.52	0.00	85.41	0.00
85	DC1 (Top)	6307.44	0.00	33.42	0.00
86	DC1 (Top)	11949.85	0.00	84.66	0.00
87	DC1 (Top)	6533.89	0.00	33.49	0.00
88	DC1 (Top)	12186.53	0.00	85.29	0.00
89	DC1 (Top)	6338.54	0.00	33.43	0.00
90	DC1 (Top)	12076.97	0.00	56.46	0.00
91	DC1 (Top)	6643.30	0.00	33.52	0.00
92	DC1 (Top)	12206.85	0.00	56.50	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	188395.30	3557923.00	0.05
2	D1 (Bottom)	82285.04	1904507.00	0.04
3	DC1 (Bottom)	145080.50	3570335.00	0.04
4	DC1 (Bottom)	80692.89	1904507.00	0.04
5	D2 (Bottom)	245468.80	3450995.00	0.07
6	D1 (Bottom)	47488.31	1904507.00	0.02
7	D2 (Top)	232982.30	3570335.00	0.07
8	D2 (Bottom)	101706.10	1904507.00	0.05
9	D2 (Top)	284674.00	3570335.00	0.08
10	D2 (Bottom)	110101.30	1904507.00	0.06
11	D2 (Bottom)	541280.40	3570335.00	0.15
12	D1 (Bottom)	182387.20	3557923.00	0.05
13	D1 (Bottom)	78757.69	1904507.00	0.04
14	DC1 (Bottom)	141287.00	3570335.00	0.04
15	DC1 (Bottom)	75877.27	1904507.00	0.04
16	D2 (Bottom)	229977.10	3450995.00	0.07
17	D2 (Bottom)	232989.40	3570335.00	0.07
18	D2 (Bottom)	284674.50	3570335.00	0.08
19	D2 (Bottom)	526335.50	3570335.00	0.15
20	D1 (Bottom)	169847.00	3557923.00	0.05
21	D1 (Bottom)	69004.82	1904507.00	0.04
22	DC1 (Bottom)	138564.90	3570335.00	0.04
23	DC1 (Bottom)	65929.68	1904507.00	0.03
24	D2 (Bottom)	162685.10	3450995.00	0.05
25	D2 (Bottom)	107031.50	1904507.00	0.06
26	D2 (Bottom)	191322.70	3570335.00	0.05
27	D2 (Bottom)	118928.80	1904507.00	0.06
28	D2 (Top)	236921.40	3570335.00	0.07
29	D2 (Top)	100266.60	1904507.00	0.05
30	D2 (Bottom)	456404.70	3570335.00	0.13
31	DC1 (Top)	126500.60	2372002.00	0.05
32	DC1 (Bottom)	38892.80	1268027.00	0.03
33	D1 (Top)	131325.20	2376935.00	0.06
34	D1 (Bottom)	39881.11	1268027.00	0.03

35	D1 (Top)	116045.40	2299019.00	0.05
36	D2 (Top)	22875.65	1268027.00	0.02
37	D2 (Top)	164855.00	2376935.00	0.07
38	D2 (Bottom)	54420.56	1268027.00	0.04
39	D2 (Top)	219985.00	2378579.00	0.09
40	D2 (Top)	57469.85	1268027.00	0.05
41	D2 (Top)	378307.00	2378579.00	0.16
42	D1 (Bottom)	126500.80	2372002.00	0.05
43	D1 (Bottom)	131325.60	2376935.00	0.06
44	D1 (Bottom)	116046.00	2299019.00	0.05
45	D2 (Bottom)	164856.50	2376935.00	0.07
46	D2 (Bottom)	219985.60	2378579.00	0.09
47	D2 (Bottom)	378306.80	2378579.00	0.16
48	D1 (Bottom)	118667.00	2372002.00	0.05
49	DC1 (Top)	36492.59	1268027.00	0.03
50	D1 (Bottom)	113910.90	2376935.00	0.05
51	D2 (Top)	48834.60	1268027.00	0.04
52	DC1 (Bottom)	101974.90	2299019.00	0.04
53	D2 (Top)	41128.26	1268027.00	0.03
54	D2 (Bottom)	109176.50	2376935.00	0.05
55	D2 (Top)	34813.15	1268027.00	0.03
56	D2 (Bottom)	165714.20	2378579.00	0.07
57	D2 (Top)	58438.65	1268027.00	0.05
58	D2 (Bottom)	266080.00	2378579.00	0.11
59	D1 (Bottom)	63282.47	1901219.00	0.03
60	D2 (Bottom)	19954.86	1013435.00	0.02
61	D1 (Top)	62150.36	1901219.00	0.03
62	D1 (Bottom)	18667.97	1013435.00	0.02
63	D1 (Top)	56136.49	1837571.00	0.03
64	D2 (Bottom)	27656.31	1013435.00	0.03
65	D2 (Bottom)	61646.78	1901219.00	0.03
66	D2 (Bottom)	55664.10	1013435.00	0.05
67	D2 (Top)	103498.20	1901219.00	0.05
68	D2 (Bottom)	42360.84	1013435.00	0.04
69	D2 (Top)	164986.30	1901219.00	0.09
70	DC1 (Bottom)	59824.35	1901219.00	0.03
71	D1 (Bottom)	62150.60	1901219.00	0.03
72	DC1 (Bottom)	56136.85	1837571.00	0.03
73	D2 (Top)	67237.90	1901219.00	0.04
74	D2 (Bottom)	103498.30	1901219.00	0.05
75	D2 (Bottom)	164985.20	1901219.00	0.09
76	D1 (Bottom)	35664.46	1884299.00	0.02
77	D2 (Bottom)	138427.70	1896286.00	0.07
78	D2 (Bottom)	118884.60	1832638.00	0.06
79	D2 (Bottom)	117566.00	1896286.00	0.06
80	D2 (Bottom)	115621.20	1901219.00	0.06
81	D2 (Bottom)	104941.10	1901219.00	0.06
82	D2 (Top)	77189.60	1884299.00	0.04
83	D2 (Bottom)	53602.62	1013435.00	0.05
84	D1 (Bottom)	27781.26	1896286.00	0.01
85	D2 (Bottom)	35983.90	1013435.00	0.04
86	DC1 (Bottom)	24833.96	1832638.00	0.01
87	D2 (Bottom)	14496.70	1013435.00	0.01
88	D1 (Bottom)	21904.12	1896286.00	0.01
89	DC1 (Top)	6338.54	1013435.00	0.01
90	D1 (Bottom)	25392.32	1901219.00	0.01
91	D1 (Top)	6643.30	1013435.00	0.01
92	D2 (Bottom)	32480.60	1901219.00	0.02

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	428434.30	453600.00	0.94
2	D3 (Top)	39874.38	100440.00	0.40
3	DC1 (Top)	0.00	200880.00	0.00
4	DC1 (Top)	0.00	100440.00	0.00
5	DC1 (Top)	0.00	200880.00	0.00
6	DC1 (Top)	0.00	100440.00	0.00
7	DC1 (Top)	0.00	200880.00	0.00
8	DC1 (Top)	0.00	100440.00	0.00
9	DC1 (Top)	0.00	200880.00	0.00
10	DC1 (Top)	0.00	100440.00	0.00
11	DC1 (Top)	0.00	200880.00	0.00
12	D3 (Bottom)	381648.40	453600.00	0.84
13	D3 (Top)	47794.58	100440.00	0.48
14	DC1 (Top)	0.00	200880.00	0.00
15	DC1 (Top)	0.00	100440.00	0.00
16	DC1 (Top)	0.00	200880.00	0.00
17	DC1 (Top)	0.00	200880.00	0.00
18	DC1 (Top)	0.00	200880.00	0.00
19	DC1 (Top)	0.00	200880.00	0.00
20	D3 (Top)	304455.40	453600.00	0.67
21	D3 (Bottom)	47794.55	100440.00	0.48
22	DC1 (Top)	0.00	200880.00	0.00
23	DC1 (Top)	0.00	100440.00	0.00
24	DC1 (Top)	0.00	200880.00	0.00
25	DC1 (Top)	0.00	100440.00	0.00
26	DC1 (Top)	0.00	200880.00	0.00
27	D3 (Top)	11583.21	100440.00	0.12
28	DC1 (Top)	0.00	200880.00	0.00
29	DC1 (Top)	0.00	100440.00	0.00
30	DC1 (Top)	0.00	200880.00	0.00
31	D3 (Bottom)	285819.70	301320.00	0.95
32	D3 (Bottom)	33341.52	100440.00	0.33
33	D3 (Top)	48120.96	200880.00	0.24
34	DC1 (Top)	0.00	100440.00	0.00
35	DC1 (Top)	0.00	167400.00	0.00
36	DC1 (Top)	0.00	100440.00	0.00
37	DC1 (Top)	0.00	200880.00	0.00
38	DC1 (Top)	0.00	100440.00	0.00
39	DC1 (Top)	0.00	167400.00	0.00
40	DC1 (Top)	0.00	100440.00	0.00
41	DC1 (Top)	0.00	167400.00	0.00
42	D3 (Bottom)	216414.20	301320.00	0.72
43	D3 (Bottom)	48120.60	200880.00	0.24
44	DC1 (Top)	0.00	167400.00	0.00
45	DC1 (Top)	0.00	200880.00	0.00
46	DC1 (Top)	0.00	167400.00	0.00
47	DC1 (Top)	0.00	167400.00	0.00
48	D3 (Top)	152985.10	301320.00	0.51
49	DC1 (Top)	0.00	100440.00	0.00
50	D3 (Top)	5852.32	200880.00	0.03
51	DC1 (Top)	0.00	100440.00	0.00
52	DC1 (Top)	0.00	167400.00	0.00
53	DC1 (Top)	0.00	100440.00	0.00
54	DC1 (Top)	0.00	200880.00	0.00
55	DC1 (Top)	0.00	100440.00	0.00

56	DC1 (Top)	0.00	167400.00	0.00
57	DC1 (Top)	0.00	100440.00	0.00
58	DC1 (Top)	0.00	167400.00	0.00
59	D3 (Bottom)	122791.10	167400.00	0.73
60	D3 (Top)	8638.16	100440.00	0.09
61	D3 (Top)	76522.60	167400.00	0.46
62	D3 (Bottom)	12611.46	100440.00	0.13
63	DC1 (Top)	0.00	167400.00	0.00
64	DC1 (Top)	0.00	100440.00	0.00
65	DC1 (Top)	0.00	167400.00	0.00
66	DC1 (Top)	0.00	100440.00	0.00
67	DC1 (Top)	0.00	167400.00	0.00
68	DC1 (Top)	0.00	100440.00	0.00
69	DC1 (Top)	0.00	167400.00	0.00
70	D3 (Top)	58070.73	167400.00	0.35
71	D3 (Bottom)	76522.08	167400.00	0.46
72	DC1 (Top)	0.00	167400.00	0.00
73	DC1 (Top)	0.00	167400.00	0.00
74	DC1 (Top)	0.00	167400.00	0.00
75	DC1 (Top)	0.00	167400.00	0.00
76	D3 (Bottom)	250896.40	511920.00	0.49
77	D3 (Top)	134943.40	267840.00	0.50
78	D3 (Top)	91779.74	267840.00	0.34
79	D3 (Top)	36149.13	267840.00	0.13
80	D3 (Top)	52667.02	167400.00	0.31
81	DC1 (Top)	0.00	167400.00	0.00
82	DC1 (Top)	0.00	511920.00	0.00
83	DC1 (Top)	0.00	100440.00	0.00
84	D3 (Bottom)	134940.30	267840.00	0.50
85	DC1 (Top)	0.00	100440.00	0.00
86	D3 (Bottom)	91777.81	267840.00	0.34
87	DC1 (Top)	0.00	100440.00	0.00
88	D3 (Bottom)	36149.02	267840.00	0.13
89	D3 (Bottom)	5512.83	100440.00	0.05
90	D3 (Bottom)	52665.10	167400.00	0.31
91	D3 (Bottom)	13768.78	100440.00	0.14
92	DC1 (Top)	0.00	167400.00	0.00

Shear

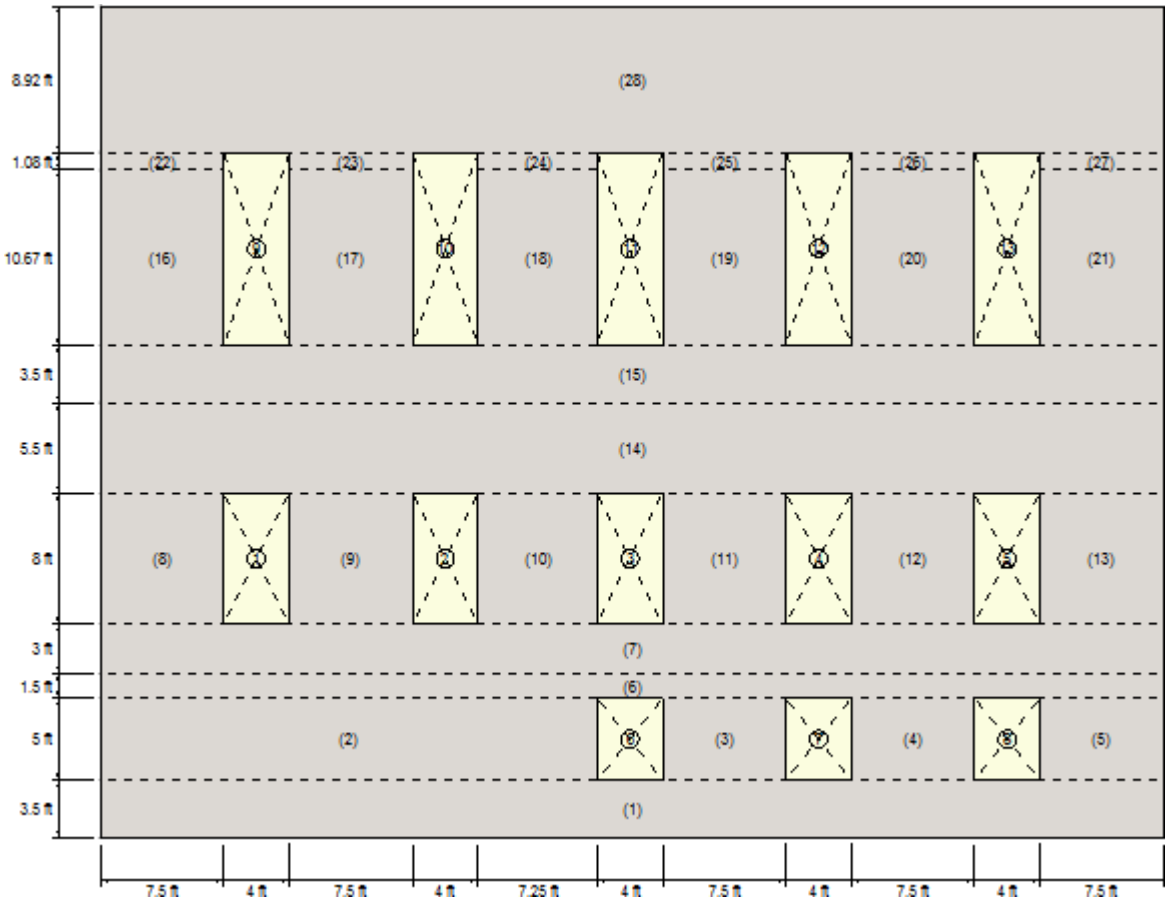
Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	137902.410	0.00
2	DC1 (Top)	0.000	74245.091	0.00
3	DC1 (Top)	0.000	139209.550	0.00
4	DC1 (Top)	0.000	74245.091	0.00
5	DC1 (Top)	0.000	134569.230	0.00
6	DC1 (Top)	0.000	74245.091	0.00
7	DC1 (Top)	0.000	139209.550	0.00
8	DC1 (Top)	0.000	74245.091	0.00
9	DC1 (Top)	0.000	139209.550	0.00
10	DC1 (Top)	0.000	74245.091	0.00
11	DC1 (Top)	0.000	139209.550	0.00
12	DC1 (Top)	0.000	137902.410	0.00
13	DC1 (Top)	0.000	74245.091	0.00
14	DC1 (Top)	0.000	139209.550	0.00
15	DC1 (Top)	0.000	74245.091	0.00
16	DC1 (Top)	0.000	134569.230	0.00

17	DC1 (Top)	0.000	139209.550	0.00
18	DC1 (Top)	0.000	139209.550	0.00
19	DC1 (Top)	0.000	139209.550	0.00
20	DC1 (Top)	0.000	136595.280	0.00
21	DC1 (Top)	0.000	73547.954	0.00
22	DC1 (Top)	0.000	137902.410	0.00
23	DC1 (Top)	0.000	73547.954	0.00
24	DC1 (Top)	0.000	133305.670	0.00
25	DC1 (Top)	0.000	73547.954	0.00
26	DC1 (Top)	0.000	137902.410	0.00
27	DC1 (Top)	0.000	73547.954	0.00
28	DC1 (Top)	0.000	137902.410	0.00
29	DC1 (Top)	0.000	73547.954	0.00
30	DC1 (Top)	0.000	137902.410	0.00
31	DC1 (Top)	0.000	86924.270	0.00
32	DC1 (Top)	0.000	46359.611	0.00
33	DC1 (Top)	0.000	86924.270	0.00
34	DC1 (Top)	0.000	46359.611	0.00
35	DC1 (Top)	0.000	84026.794	0.00
36	DC1 (Top)	0.000	46359.611	0.00
37	DC1 (Top)	0.000	86924.270	0.00
38	DC1 (Top)	0.000	46359.611	0.00
39	DC1 (Top)	0.000	86924.270	0.00
40	DC1 (Top)	0.000	46359.611	0.00
41	DC1 (Top)	0.000	86924.270	0.00
42	DC1 (Top)	0.000	86924.270	0.00
43	DC1 (Top)	0.000	86924.270	0.00
44	DC1 (Top)	0.000	84026.794	0.00
45	DC1 (Top)	0.000	86924.270	0.00
46	DC1 (Top)	0.000	86924.270	0.00
47	DC1 (Top)	0.000	86924.270	0.00
48	DC1 (Top)	0.000	86924.270	0.00
49	DC1 (Top)	0.000	46359.611	0.00
50	DC1 (Top)	0.000	86924.270	0.00
51	DC1 (Top)	0.000	46359.611	0.00
52	DC1 (Top)	0.000	84026.794	0.00
53	DC1 (Top)	0.000	46359.611	0.00
54	DC1 (Top)	0.000	86924.270	0.00
55	DC1 (Top)	0.000	46359.611	0.00
56	DC1 (Top)	0.000	86924.270	0.00
57	DC1 (Top)	0.000	46359.611	0.00
58	DC1 (Top)	0.000	86924.270	0.00
59	DC1 (Top)	0.000	66010.160	0.00
60	DC1 (Top)	0.000	35205.419	0.00
61	DC1 (Top)	0.000	66010.160	0.00
62	DC1 (Top)	0.000	35205.419	0.00
63	DC1 (Top)	0.000	63809.821	0.00
64	DC1 (Top)	0.000	35205.419	0.00
65	DC1 (Top)	0.000	66010.160	0.00
66	DC1 (Top)	0.000	35205.419	0.00
67	DC1 (Top)	0.000	66010.160	0.00
68	DC1 (Top)	0.000	35205.419	0.00
69	DC1 (Top)	0.000	66010.160	0.00
70	DC1 (Top)	0.000	66010.160	0.00
71	DC1 (Top)	0.000	66010.160	0.00
72	DC1 (Top)	0.000	63809.821	0.00
73	DC1 (Top)	0.000	66010.160	0.00
74	DC1 (Top)	0.000	66010.160	0.00
75	DC1 (Top)	0.000	66010.160	0.00
76	DC1 (Top)	0.000	62742.330	0.00

77	DC1 (Top)	0.000	66010.160	0.00
78	DC1 (Top)	0.000	63809.821	0.00
79	DC1 (Top)	0.000	66010.160	0.00
80	DC1 (Top)	0.000	66010.160	0.00
81	DC1 (Top)	0.000	66010.160	0.00
82	DC1 (Top)	0.000	64049.462	0.00
83	DC1 (Top)	0.000	35205.419	0.00
84	DC1 (Top)	0.000	66010.160	0.00
85	DC1 (Top)	0.000	35205.419	0.00
86	DC1 (Top)	0.000	63809.821	0.00
87	DC1 (Top)	0.000	35205.419	0.00
88	DC1 (Top)	0.000	66010.160	0.00
89	DC1 (Top)	0.000	35205.419	0.00
90	DC1 (Top)	0.000	66010.160	0.00
91	DC1 (Top)	0.000	35205.419	0.00
92	DC1 (Top)	0.000	66010.160	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	64.75	3.50
2	0.00	3.50	30.25	5.00
3	34.25	3.50	7.50	5.00
4	45.75	3.50	7.50	5.00
5	57.25	3.50	7.50	5.00
6	0.00	8.50	64.75	1.50
7	0.00	10.00	64.75	3.00
8	0.00	13.00	7.50	8.00
9	11.50	13.00	7.50	8.00
10	23.00	13.00	7.25	8.00
11	34.25	13.00	7.50	8.00
12	45.75	13.00	7.50	8.00
13	57.25	13.00	7.50	8.00
14	0.00	21.00	64.75	5.50
15	0.00	26.50	64.75	3.50
16	0.00	30.00	7.50	10.67
17	11.50	30.00	7.50	10.67
18	23.00	30.00	7.25	10.67
19	34.25	30.00	7.50	10.67
20	45.75	30.00	7.50	10.67
21	57.25	30.00	7.50	10.67
22	0.00	40.67	7.50	1.08
23	11.50	40.67	7.50	1.08
24	23.00	40.67	7.25	1.08
25	34.25	40.67	7.50	1.08
26	45.75	40.67	7.50	1.08
27	57.25	40.67	7.50	1.08
28	0.00	41.75	64.75	8.92

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	Bars	Spacing [in]	Ld [in]	Bars	Spacing [in]	Ld [in]
1	7-#7	14.00	33.89	3-#5	18.00	25.17
	3-#5	16.00	33.89	3-#5	18.00	25.17
	6-#5	16.00	33.89	3-#5	18.00	25.17
	3-#5	16.00	33.89	3-#5	18.00	25.17
	6-#5	16.00	33.89	3-#5	18.00	25.17
	3-#5	16.00	33.89	3-#5	18.00	25.17
	6-#5	16.00	33.89	3-#5	18.00	25.17
	3-#5	16.00	33.89	3-#5	18.00	25.17
	6-#5	16.00	33.89	3-#5	18.00	25.17
	3-#5	16.00	33.89	3-#5	18.00	25.17
	6-#5	16.00	33.89	3-#5	18.00	25.17
2	7-#7	14.00	33.89	4-#5	18.00	25.17
	3-#5	16.00	33.89	4-#5	18.00	25.17
	6-#5	16.00	33.89	4-#5	18.00	25.17
	3-#5	16.00	33.89	4-#5	18.00	25.17
	6-#5	16.00	33.89	4-#5	18.00	25.17
3	6-#5	16.00	19.36	4-#5	18.00	25.17
4	6-#5	16.00	19.36	4-#5	18.00	25.17
5	6-#5	16.00	19.36	4-#5	18.00	25.17
6	7-#7	14.00	33.89	1-#6	18.00	30.21
	3-#5	16.00	33.89	1-#6	18.00	30.21
	6-#5	16.00	33.89	1-#6	18.00	30.21
	3-#5	16.00	33.89	1-#6	18.00	30.21

	6-#5	16.00	33.89	1-#6	18.00	30.21
	3-#5	16.00	33.89	1-#6	18.00	30.21
	6-#5	16.00	33.89	1-#6	18.00	30.21
	3-#5	16.00	33.89	1-#6	18.00	30.21
	6-#5	16.00	33.89	1-#6	18.00	30.21
	3-#5	16.00	33.89	1-#6	18.00	30.21
	6-#5	16.00	33.89	1-#6	18.00	30.21
7	9-#5	10.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	6-#5	16.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	6-#5	16.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
	3-#5	18.00	19.36	2-#5	18.00	25.17
	5-#5	18.00	19.36	2-#5	18.00	25.17
8	9-#5	10.00	19.36	6-#5	18.00	25.17
9	6-#5	16.00	19.36	6-#5	18.00	25.17
10	5-#5	18.00	19.36	6-#5	18.00	25.17
11	6-#5	16.00	19.36	6-#5	18.00	25.17
12	5-#5	18.00	19.36	6-#5	18.00	25.17
13	5-#5	18.00	19.36	6-#5	18.00	25.17
14	9-#5	10.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	6-#5	16.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	6-#5	16.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
	3-#5	18.00	19.36	4-#5	18.00	25.17
	5-#5	18.00	19.36	4-#5	18.00	25.17
15	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
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	3-#5	18.00	19.36	3-#5	18.00	25.17
	5-#5	18.00	19.36	3-#5	18.00	25.17
16	5-#5	18.00	19.36	7-#5	18.00	25.17
17	5-#5	18.00	19.36	7-#5	18.00	25.17
18	5-#5	18.00	19.36	7-#5	18.00	25.17
19	5-#5	18.00	19.36	7-#5	18.00	25.17
20	5-#5	18.00	19.36	7-#5	18.00	25.17
21	5-#5	18.00	19.36	7-#5	18.00	25.17
22	6-#8	16.00	38.73	1-#6	14.00	30.21
23	8-#5	12.00	19.36	1-#5	18.00	25.17
24	8-#5	12.00	19.36	1-#5	18.00	25.17
25	8-#5	12.00	19.36	1-#5	18.00	25.17
26	5-#5	18.00	19.36	1-#5	18.00	25.17
27	5-#5	18.00	19.36	1-#5	18.00	25.17
28	6-#8	16.00	38.73	6-#5	18.00	25.17
	3-#5	18.00	38.73	6-#5	18.00	25.17
	8-#5	12.00	38.73	6-#5	18.00	25.17
	3-#5	18.00	38.73	6-#5	18.00	25.17
	8-#5	12.00	38.73	6-#5	18.00	25.17
	3-#5	18.00	38.73	6-#5	18.00	25.17

8-#5	12.00	38.73	6-#5	18.00	25.17
3-#5	18.00	38.73	6-#5	18.00	25.17
5-#5	18.00	38.73	6-#5	18.00	25.17
3-#5	18.00	38.73	6-#5	18.00	25.17
5-#5	18.00	38.73	6-#5	18.00	25.17

Intermediate results for axial-bending

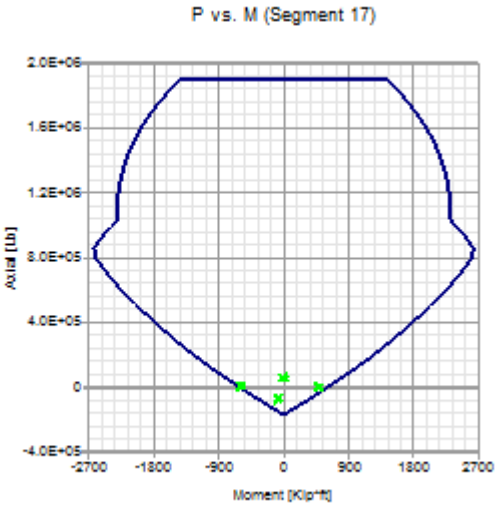
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	49.09	621.60
2	D3 (Bottom)	15.61	290.40
3	D3 (Max)	6.95	72.00
4	D3 (Max)	8.87	72.00
5	D3 (Bottom)	12.43	72.00
6	D3 (Top)	45.23	621.60
7	D3 (Bottom)	56.99	621.60
8	D3 (Max)	2.45	72.00
9	D3 (Bottom)	3.56	72.00
10	D3 (Max)	6.39	69.60
11	D3 (Max)	8.65	72.00
12	D3 (Max)	9.96	72.00
13	D3 (Max)	12.04	72.00
14	D3 (Max)	55.37	621.60
15	D3 (Bottom)	57.68	621.60
16	D3 (Max)	5.00	72.00
17	D3 (Max)	5.06	72.00
18	D3 (Max)	5.78	69.60
19	D3 (Top)	7.00	72.00
20	D3 (Max)	7.74	72.00
21	D3 (Max)	7.85	72.00
22	D3 (Max)	4.28	72.00
23	D3 (Top)	3.72	72.00
24	D3 (Top)	5.28	69.60
25	D3 (Top)	7.18	72.00
26	D3 (Top)	3.06	72.00
27	D2 (Max)	8.69	72.00
28	D3 (Max)	64.69	621.60

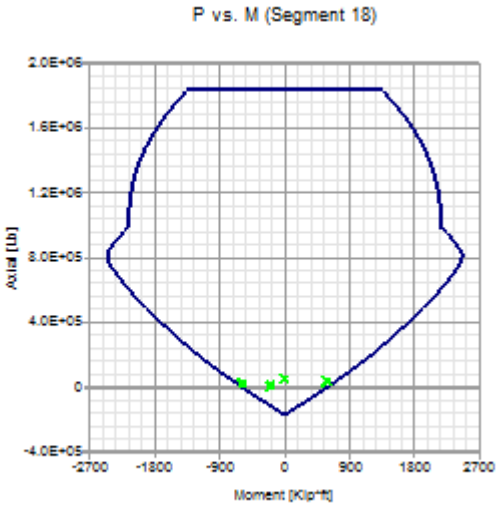
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	889406.30	-33271.24	80158.43	0.42
2	D3 (Bottom)	-137603.80	-7922.32	10534.86	0.75
3	D3 (Max)	188671.80	-378.47	1375.56	0.28
4	D3 (Max)	288047.00	-330.13	1702.78	0.19
5	D3 (Bottom)	480500.00	-367.07	2287.81	0.16
6	D3 (Top)	668337.40	-26461.90	73745.02	0.36
7	D3 (Bottom)	494602.40	-23917.05	62547.40	0.38
8	D3 (Max)	-186425.00	-303.89	420.28	0.72
9	D3 (Bottom)	-47789.99	-354.50	554.70	0.64
10	D3 (Max)	83190.94	-416.06	859.12	0.48
11	D3 (Max)	127547.40	356.66	1148.04	0.31
12	D3 (Max)	206232.60	-348.92	1288.78	0.27
13	D3 (Max)	277863.90	217.93	1510.64	0.14
14	D3 (Max)	432366.00	-12640.27	60791.42	0.21
15	D3 (Bottom)	217700.50	-11017.48	52286.10	0.21
16	D3 (Max)	967.99	412.93	606.15	0.68

17	D3 (Max)	2626.72	-577.73	611.99	0.94
18	D3 (Max)	22481.02	-584.89	655.50	0.89
19	D3 (Top)	55891.22	601.77	790.05	0.76
20	D3 (Max)	76404.05	-531.91	857.93	0.62
21	D3 (Max)	79425.65	-170.23	867.93	0.20
22	D3 (Max)	-243422.80	771.54	973.27	0.79
23	D3 (Top)	-134856.50	415.52	481.62	0.86
24	D3 (Top)	-91701.76	475.30	611.44	0.78
25	D3 (Top)	-36063.03	229.72	815.73	0.28
26	D3 (Top)	-52581.04	331.73	417.70	0.79
27	D2 (Max)	102587.40	314.92	944.57	0.33
28	D3 (Max)	75646.73	-2651.33	55175.43	0.05

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Top)	1317794.00	3.081279E07	0.04
2	DC1 (Max)	633673.70	1.438827E07	0.04
3	D2 (Bottom)	232905.40	3570335.00	0.07
4	D2 (Max)	332079.60	3570335.00	0.09
5	D2 (Max)	536895.50	3570335.00	0.15
6	DC1 (Bottom)	1246162.00	3.081279E07	0.04
7	DC1 (Top)	870145.40	2.052218E07	0.04
8	DC1 (Max)	130535.50	2372002.00	0.06
9	DC1 (Max)	137636.30	2376935.00	0.06
10	DC1 (Max)	128943.50	2299019.00	0.06
11	D2 (Bottom)	166238.80	2376935.00	0.07
12	D2 (Max)	236019.10	2378579.00	0.10
13	D2 (Bottom)	378134.20	2378579.00	0.16
14	D1 (Bottom)	790784.30	2.052218E07	0.04
15	DC1 (Bottom)	425402.70	1.641084E07	0.03
16	DC1 (Max)	66577.31	1901219.00	0.04
17	DC1 (Bottom)	62373.29	1901219.00	0.03
18	D1 (Max)	57214.86	1837571.00	0.03

19	D2 (Max)	79621.21	1901219.00	0.04
20	D2 (Bottom)	103600.30	1901219.00	0.05
21	D2 (Bottom)	164232.80	1901219.00	0.09
22	DC1 (Bottom)	35638.72	2180075.00	0.02
23	D2 (Bottom)	138355.30	1896286.00	0.07
24	D2 (Bottom)	118803.30	1832638.00	0.06
25	D2 (Bottom)	117488.10	1896286.00	0.06
26	D2 (Bottom)	115555.20	1901219.00	0.06
27	D2 (Bottom)	104885.90	1901219.00	0.06
28	DC1 (Max)	188584.40	1.637912E07	0.01

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	1960200.00	0.00
2	D3 (Bottom)	137603.80	1056240.00	0.13
3	DC1 (Top)	0.00	200880.00	0.00
4	DC1 (Top)	0.00	200880.00	0.00
5	DC1 (Top)	0.00	200880.00	0.00
6	DC1 (Top)	0.00	1960200.00	0.00
7	DC1 (Top)	0.00	1707480.00	0.00
8	D3 (Bottom)	216041.90	301320.00	0.72
9	D3 (Bottom)	47789.99	200880.00	0.24
10	DC1 (Top)	0.00	167400.00	0.00
11	DC1 (Top)	0.00	200880.00	0.00
12	DC1 (Top)	0.00	167400.00	0.00
13	DC1 (Top)	0.00	167400.00	0.00
14	DC1 (Top)	0.00	1707480.00	0.00
15	DC1 (Top)	0.00	1506600.00	0.00
16	D3 (Top)	57922.67	167400.00	0.35
17	D3 (Bottom)	75999.34	167400.00	0.45
18	DC1 (Top)	0.00	167400.00	0.00
19	DC1 (Top)	0.00	167400.00	0.00
20	DC1 (Top)	0.00	167400.00	0.00
21	DC1 (Top)	0.00	167400.00	0.00
22	D3 (Bottom)	250720.10	511920.00	0.49
23	D3 (Top)	134856.50	267840.00	0.50
24	D3 (Top)	91701.76	267840.00	0.34
25	D3 (Top)	36063.03	267840.00	0.13
26	D3 (Top)	52581.04	167400.00	0.31
27	DC1 (Top)	0.00	167400.00	0.00
28	D3 (Bottom)	163288.60	2152440.00	0.08

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	1095764.600	2881994.300	0.38
2	D3 (Top)	686260.590	1266837.800	0.54
3	D3 (Bottom)	166513.060	347706.240	0.48
4	D3 (Bottom)	165721.370	355436.630	0.47
5	D3 (Top)	108892.750	380756.420	0.29
6	D3 (Max)	1214400.100	3268409.200	0.37
7	D3 (Max)	880788.570	2229816.500	0.40
8	D2 (Max)	89680.504	226037.020	0.40
9	D3 (Bottom)	210709.890	242464.630	0.87

10	D3 (Max)	212940.790	253790.660	0.84
11	D3 (Max)	196563.130	268765.230	0.73
12	D3 (Max)	148280.160	280568.010	0.53
13	D3 (Max)	75599.348	291312.710	0.26
14	D3 (Max)	920547.760	2220020.900	0.41
15	D3 (Max)	579024.700	1950808.800	0.30
16	D2 (Max)	123887.750	224174.780	0.55
17	D3 (Bottom)	99231.186	210626.600	0.47
18	D3 (Max)	115270.290	217997.770	0.53
19	D3 (Top)	114940.850	230410.180	0.50
20	D3 (Top)	88483.144	233759.150	0.38
21	D3 (Max)	48088.750	233940.350	0.21
22	D3 (Bottom)	265758.950	276475.630	0.96
23	D2 (Max)	111751.550	205707.730	0.54
24	D3 (Max)	122438.440	212561.840	0.58
25	D3 (Max)	126186.450	235799.570	0.54
26	D3 (Max)	96612.219	216345.510	0.45
27	D3 (Max)	49553.466	236408.300	0.21
28	D3 (Max)	228997.600	1928175.800	0.12

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Schematic Shear Wall Design for Gridline 3

Current Date: 8/4/2016 3:33 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\SPOE GL 3.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : N. G.

Design code	:	ACI 318-05

Geometry:

Total height	:	53.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 4

Story	Story height [ft]	Wall thickness [in]
1	10.00	18.00
2	16.50	12.00
3	15.00	12.00
4	12.00	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	9.50	0.00	3.00	7.00
Lower left	17.50	10.00	3.00	10.00
Lower left	10.00	26.50	3.00	10.00
Lower left	10.00	41.50	3.00	10.00

Columns:

Distance [ft]	Width X [in]	Width Z [in]	Position Z
1.25	30.00	18.00	Centered
21.75	30.00	18.00	Centered

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	14400.00	0.00	0.00
1	DL	Vertical	14400.00	0.00	23.00
2	DL	Vertical	13700.00	0.00	0.00
2	DL	Vertical	13700.00	0.00	23.00
3	DL	Vertical	13100.00	0.00	0.00
3	DL	Vertical	13100.00	0.00	23.00
4	EQ	Horizontal	384714.00	0.00	0.00
3	EQ	Horizontal	407204.00	0.00	0.00
2	EQ	Horizontal	205059.00	0.00	0.00
1	EQ	Horizontal	140022.00	0.00	0.00

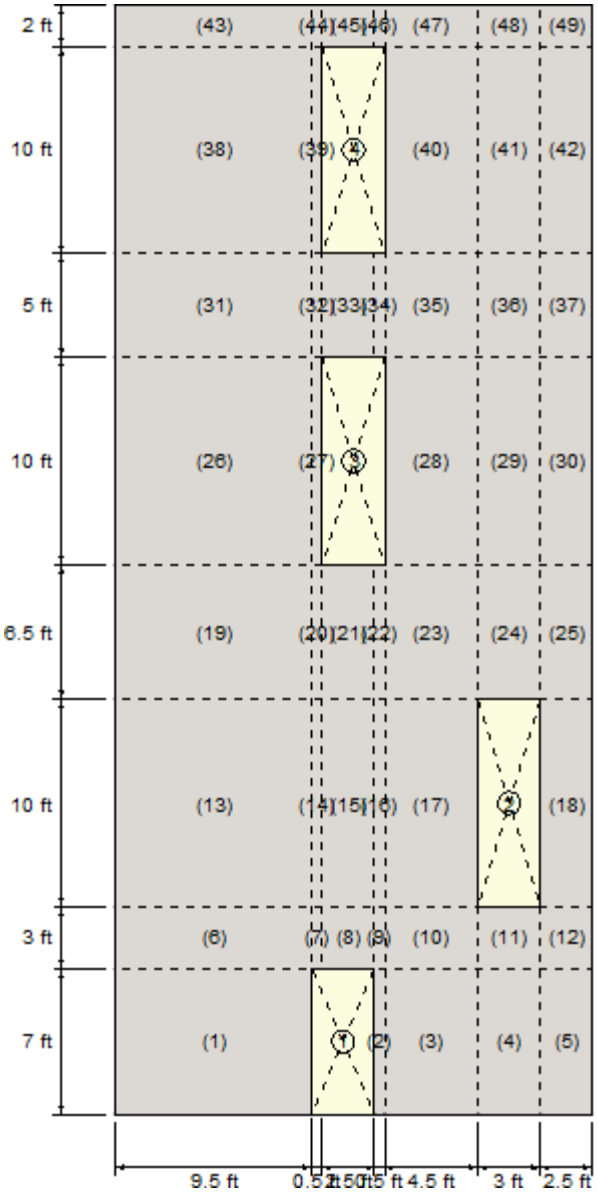
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
4	DL	Vertical	1640.00	0.00
3	DL	Vertical	1710.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	0.50	7.00
3	13.00	0.00	4.50	7.00
4	17.50	0.00	3.00	7.00
5	20.50	0.00	2.50	7.00
6	0.00	7.00	9.50	3.00
7	9.50	7.00	0.50	3.00
8	10.00	7.00	2.50	3.00
9	12.50	7.00	0.50	3.00
10	13.00	7.00	4.50	3.00
11	17.50	7.00	3.00	3.00
12	20.50	7.00	2.50	3.00
13	0.00	10.00	9.50	10.00
14	9.50	10.00	0.50	10.00
15	10.00	10.00	2.50	10.00
16	12.50	10.00	0.50	10.00

17	13.00	10.00	4.50	10.00
18	20.50	10.00	2.50	10.00
19	0.00	20.00	9.50	6.50
20	9.50	20.00	0.50	6.50
21	10.00	20.00	2.50	6.50
22	12.50	20.00	0.50	6.50
23	13.00	20.00	4.50	6.50
24	17.50	20.00	3.00	6.50
25	20.50	20.00	2.50	6.50
26	0.00	26.50	9.50	10.00
27	9.50	26.50	0.50	10.00
28	13.00	26.50	4.50	10.00
29	17.50	26.50	3.00	10.00
30	20.50	26.50	2.50	10.00
31	0.00	36.50	9.50	5.00
32	9.50	36.50	0.50	5.00
33	10.00	36.50	2.50	5.00
34	12.50	36.50	0.50	5.00
35	13.00	36.50	4.50	5.00
36	17.50	36.50	3.00	5.00
37	20.50	36.50	2.50	5.00
38	0.00	41.50	9.50	10.00
39	9.50	41.50	0.50	10.00
40	13.00	41.50	4.50	10.00
41	17.50	41.50	3.00	10.00
42	20.50	41.50	2.50	10.00
43	0.00	51.50	9.50	2.00
44	9.50	51.50	0.50	2.00
45	10.00	51.50	2.50	2.00
46	12.50	51.50	0.50	2.00
47	13.00	51.50	4.50	2.00
48	17.50	51.50	3.00	2.00
49	20.50	51.50	2.50	2.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	12-#10	10.00	49.19
2	1-#8	10.00	38.73
3	6-#8	10.00	38.73
4	4-#8	10.00	38.73
5	2-#6	18.00	23.24
6	12-#10	10.00	49.19
7	1-#8	10.00	38.73
8	3-#8	10.00	38.73
9	1-#8	10.00	38.73
10	6-#8	10.00	38.73
11	4-#8	10.00	38.73
12	2-#6	18.00	23.24
13	12-#9	10.00	43.69
14	1-#9	12.00	43.69
15	3-#9	12.00	43.69
16	1-#9	12.00	43.69
17	5-#9	12.00	43.69
18	3-#5	12.00	19.36
19	12-#9	10.00	43.69
20	1-#9	12.00	43.69
21	3-#9	12.00	43.69

22	1-#9	12.00	43.69
23	5-#9	12.00	43.69
24	3-#7	14.00	33.89
25	3-#5	12.00	19.36
26	10-#7	12.00	33.89
27	1-#9	18.00	43.69
28	3-#7	18.00	33.89
29	2-#7	18.00	33.89
30	2-#5	18.00	19.36
31	10-#7	12.00	33.89
32	1-#9	18.00	43.69
33	2-#6	18.00	23.24
34	1-#8	18.00	38.73
35	3-#7	18.00	33.89
36	2-#7	18.00	33.89
37	2-#5	18.00	19.36
38	7-#5	18.00	19.36
39	1-#6	18.00	23.24
40	5-#5	12.00	19.36
41	3-#5	12.00	19.36
42	2-#5	18.00	19.36
43	7-#5	18.00	19.36
44	1-#6	18.00	23.24
45	2-#5	18.00	19.36
46	1-#5	18.00	19.36
47	5-#5	12.00	19.36
48	3-#5	12.00	19.36
49	2-#5	18.00	19.36

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	2.76	15.62
2	DC1 (Top)	3.06	16.00
3	DC1 (Top)	2.10	16.00
4	DC1 (Top)	2.02	16.00
5	DC1 (Top)	1.38	16.25
6	DC1 (Top)	2.70	15.74
7	DC1 (Top)	2.49	15.88
8	DC1 (Top)	1.92	15.88
9	DC1 (Top)	2.58	15.88
10	DC1 (Top)	2.11	15.88
11	DC1 (Top)	1.89	15.88
12	DC1 (Top)	1.43	16.13
13	DC1 (Top)	2.08	9.94
14	DC1 (Top)	3.20	9.94
15	DC1 (Top)	2.04	9.94
16	DC1 (Top)	3.21	9.94
17	DC1 (Top)	2.02	9.94
18	DC1 (Top)	1.23	10.31
19	DC1 (Top)	2.11	9.81
20	DC1 (Top)	3.22	9.81
21	DC1 (Top)	1.97	9.81
22	DC1 (Top)	2.99	9.81
23	DC1 (Top)	2.02	9.81
24	DC1 (Top)	1.53	9.94
25	DC1 (Top)	1.20	10.31
26	DC1 (Top)	1.40	10.19
27	DC1 (Top)	3.11	10.06
28	DC1 (Top)	1.17	10.06
29	DC1 (Top)	1.16	10.06

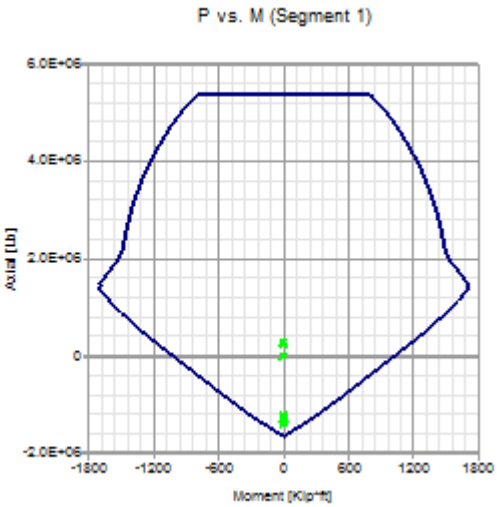
30	DC1 (Top)	0.84	10.31
31	DC1 (Top)	1.39	10.19
32	DC1 (Top)	2.98	10.06
33	DC1 (Top)	0.98	10.25
34	DC1 (Top)	2.17	10.13
35	DC1 (Top)	1.13	10.19
36	DC1 (Top)	1.13	10.19
37	DC1 (Top)	0.85	10.31
38	DC1 (Top)	0.65	8.31
39	DC1 (Top)	1.52	8.25
40	DC1 (Top)	0.94	8.31
41	DC1 (Top)	0.86	8.31
42	DC1 (Top)	0.68	8.31
43	DC1 (Top)	0.65	8.31
44	DC1 (Top)	1.50	8.25
45	DC1 (Top)	0.70	8.31
46	DC1 (Top)	1.28	8.31
47	DC1 (Top)	0.92	8.31
48	DC1 (Top)	0.86	8.31
49	DC1 (Top)	0.68	8.31

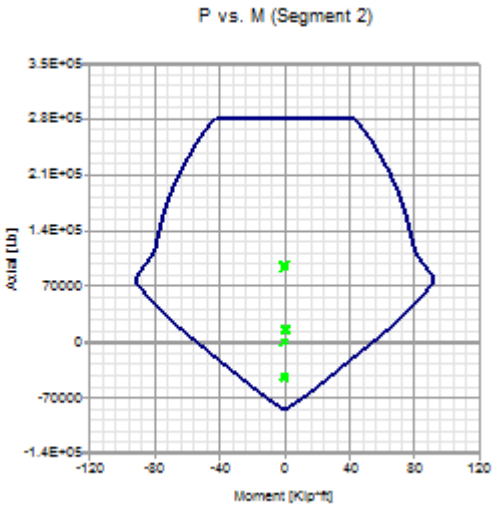
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	260548.50	0.00	1172.79	0.00
2	DC1 (Top)	16923.93	0.00	64.08	0.00
3	DC1 (Top)	116145.10	0.00	405.46	0.00
4	DC1 (Top)	60015.73	0.00	259.69	0.00
5	DC1 (Top)	73309.55	0.00	116.08	0.00
6	DC1 (Top)	232612.00	0.00	1164.81	0.00
7	DC1 (Top)	8755.71	0.00	58.94	0.00
8	DC1 (Top)	42787.28	0.00	194.11	0.00
9	DC1 (Top)	10623.43	0.00	60.05	0.00
10	DC1 (Top)	116662.50	0.00	403.42	0.00
11	DC1 (Top)	21443.36	0.00	235.10	0.00
12	DC1 (Top)	78227.43	0.00	119.11	0.00
13	DC1 (Top)	145141.20	0.00	559.85	0.00
14	DC1 (Top)	7471.25	0.00	42.62	0.00
15	DC1 (Top)	37528.17	0.00	141.28	0.00
16	DC1 (Top)	7799.75	0.00	42.72	0.00
17	DC1 (Top)	84258.79	0.00	245.46	0.00
18	DC1 (Top)	48079.33	0.00	66.26	0.00
19	DC1 (Top)	146351.00	0.00	554.55	0.00
20	DC1 (Top)	7895.83	0.00	42.24	0.00
21	DC1 (Top)	13141.83	0.00	131.56	0.00
22	DC1 (Top)	6067.94	0.00	41.70	0.00
23	DC1 (Top)	69069.70	0.00	238.06	0.00
24	DC1 (Top)	39760.80	0.00	100.19	0.00
25	DC1 (Top)	41404.03	0.00	63.63	0.00
26	DC1 (Top)	81338.36	0.00	313.23	0.00
27	DC1 (Top)	5657.18	0.00	42.57	0.00
28	DC1 (Top)	39242.60	0.00	106.04	0.00
29	DC1 (Top)	25108.03	0.00	70.28	0.00
30	DC1 (Top)	21415.16	0.00	41.55	0.00
31	DC1 (Top)	72216.32	0.00	309.75	0.00
32	DC1 (Top)	2891.20	0.00	41.74	0.00
33	DC1 (Top)	6537.92	0.00	47.57	0.00
34	DC1 (Top)	2042.20	0.00	33.99	0.00
35	DC1 (Top)	31035.86	0.00	102.86	0.00

36	DC1 (Top)	21053.18	0.00	68.71	0.00
37	DC1 (Top)	22735.57	0.00	42.13	0.00
38	DC1 (Top)	22560.98	0.00	101.51	0.00
39	DC1 (Top)	2126.49	0.00	16.52	0.00
40	DC1 (Top)	15138.01	0.00	69.68	0.00
41	DC1 (Top)	7073.63	0.00	41.80	0.00
42	DC1 (Top)	3577.27	0.00	27.81	0.00
43	DC1 (Top)	20729.06	0.00	100.83	0.00
44	DC1 (Top)	1327.86	0.00	16.29	0.00
45	DC1 (Top)	5687.15	0.00	28.59	0.00
46	DC1 (Top)	1331.90	0.00	12.25	0.00
47	DC1 (Top)	11154.14	0.00	68.37	0.00
48	DC1 (Top)	7368.77	0.00	41.90	0.00
49	DC1 (Top)	3642.05	0.00	27.83	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	284811.10	5361071.00	0.05
2	D2 (Top)	97250.59	282225.80	0.34
3	D2 (Top)	691491.70	2552603.00	0.27
4	D2 (Bottom)	640930.70	1701735.00	0.38
5	D2 (Bottom)	851800.80	1427412.00	0.60
6	DC1 (Bottom)	261003.60	5361071.00	0.05
7	DC1 (Bottom)	11287.48	282225.80	0.04
8	D2 (Top)	82821.37	1419510.00	0.06
9	D2 (Bottom)	97249.70	282225.80	0.34
10	D2 (Top)	824170.60	2552603.00	0.32
11	D2 (Bottom)	469796.20	1701735.00	0.28
12	D2 (Bottom)	632274.20	1427412.00	0.44
13	D1 (Bottom)	170224.40	3564288.00	0.05
14	D2 (Top)	7888.62	185640.00	0.04
15	D2 (Bottom)	62552.25	938808.00	0.07
16	D2 (Bottom)	43359.49	185640.00	0.23

17	D2 (Bottom)	826920.30	1691976.00	0.49
18	D2 (Bottom)	530666.20	949787.30	0.56
19	DC1 (Top)	146351.00	3564288.00	0.04
20	D2 (Top)	73977.00	185640.00	0.40
21	D2 (Top)	94463.16	938808.00	0.10
22	D2 (Bottom)	13985.04	185640.00	0.08
23	D2 (Bottom)	127768.20	1691976.00	0.08
24	D2 (Top)	198711.60	1136117.00	0.17
25	D2 (Bottom)	506228.10	949787.30	0.53
26	D1 (Bottom)	97685.62	3596112.00	0.03
27	D2 (Bottom)	97244.89	185640.00	0.52
28	D2 (Top)	279085.00	1708949.00	0.16
29	D2 (Bottom)	186084.30	1139299.00	0.16
30	D2 (Bottom)	325271.00	951431.50	0.34
31	DC1 (Bottom)	81338.35	3596112.00	0.02
32	D2 (Top)	67817.54	185640.00	0.37
33	D2 (Top)	112662.10	950052.50	0.12
34	D2 (Bottom)	108658.00	186753.80	0.58
35	D2 (Bottom)	279075.50	1708949.00	0.16
36	D2 (Bottom)	96631.69	1139299.00	0.08
37	D2 (Bottom)	120960.80	951431.50	0.13
38	D1 (Bottom)	35707.53	3011770.00	0.01
39	D2 (Bottom)	101621.40	156786.20	0.65
40	D2 (Top)	57151.39	1423859.00	0.04
41	D2 (Bottom)	77046.47	949787.30	0.08
42	D2 (Bottom)	77322.54	792311.50	0.10
43	D2 (Top)	134460.50	3011770.00	0.04
44	D1 (Bottom)	2123.57	156786.20	0.01
45	DC1 (Top)	5687.15	792311.50	0.01
46	D2 (Bottom)	50699.92	157475.80	0.32
47	D2 (Bottom)	57122.15	1423859.00	0.04
48	D1 (Top)	7368.77	949787.30	0.01
49	D2 (Bottom)	6690.05	792311.50	0.01

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Top)	1398847.00	1645920.00	0.85
2	D3 (Bottom)	45669.07	85320.00	0.54
3	DC1 (Top)	0.00	511920.00	0.00
4	DC1 (Top)	0.00	341280.00	0.00
5	DC1 (Top)	0.00	95040.00	0.00
6	D3 (Bottom)	1388692.00	1645920.00	0.84
7	D3 (Bottom)	65291.29	85320.00	0.77
8	DC1 (Top)	0.00	255960.00	0.00
9	DC1 (Top)	0.00	85320.00	0.00
10	DC1 (Top)	0.00	511920.00	0.00
11	DC1 (Top)	0.00	341280.00	0.00
12	DC1 (Top)	0.00	95040.00	0.00
13	D3 (Bottom)	1198945.00	1296000.00	0.93
14	D3 (Bottom)	15694.37	108000.00	0.15
15	DC1 (Top)	0.00	324000.00	0.00
16	DC1 (Top)	0.00	108000.00	0.00
17	DC1 (Top)	0.00	540000.00	0.00
18	DC1 (Top)	0.00	100440.00	0.00
19	D3 (Bottom)	596583.10	1296000.00	0.46
20	DC1 (Top)	0.00	108000.00	0.00

21	DC1 (Top)	0.00	324000.00	0.00
22	D3 (Top)	76003.78	108000.00	0.70
23	D3 (Top)	156544.60	540000.00	0.29
24	DC1 (Top)	0.00	194400.00	0.00
25	DC1 (Top)	0.00	100440.00	0.00
26	D3 (Bottom)	329212.00	648000.00	0.51
27	D3 (Top)	89084.59	108000.00	0.82
28	D3 (Bottom)	183041.60	194400.00	0.94
29	DC1 (Top)	0.00	129600.00	0.00
30	DC1 (Top)	0.00	66960.00	0.00
31	D3 (Bottom)	260301.30	648000.00	0.40
32	D3 (Bottom)	89077.07	108000.00	0.82
33	D3 (Bottom)	85572.20	95040.00	0.90
34	D3 (Top)	71153.38	85320.00	0.83
35	D3 (Top)	72021.66	194400.00	0.37
36	DC1 (Top)	0.00	129600.00	0.00
37	DC1 (Top)	0.00	66960.00	0.00
38	D3 (Bottom)	38416.96	234360.00	0.16
39	D3 (Top)	40190.58	47520.00	0.85
40	D3 (Bottom)	115054.50	167400.00	0.69
41	DC1 (Top)	0.00	100440.00	0.00
42	DC1 (Top)	0.00	66960.00	0.00
43	DC1 (Top)	0.00	234360.00	0.00
44	D3 (Bottom)	40094.73	47520.00	0.84
45	D3 (Bottom)	34644.66	66960.00	0.52
46	DC1 (Top)	0.00	33480.00	0.00
47	D3 (Top)	5042.40	167400.00	0.03
48	DC1 (Top)	0.00	100440.00	0.00
49	DC1 (Top)	0.00	66960.00	0.00

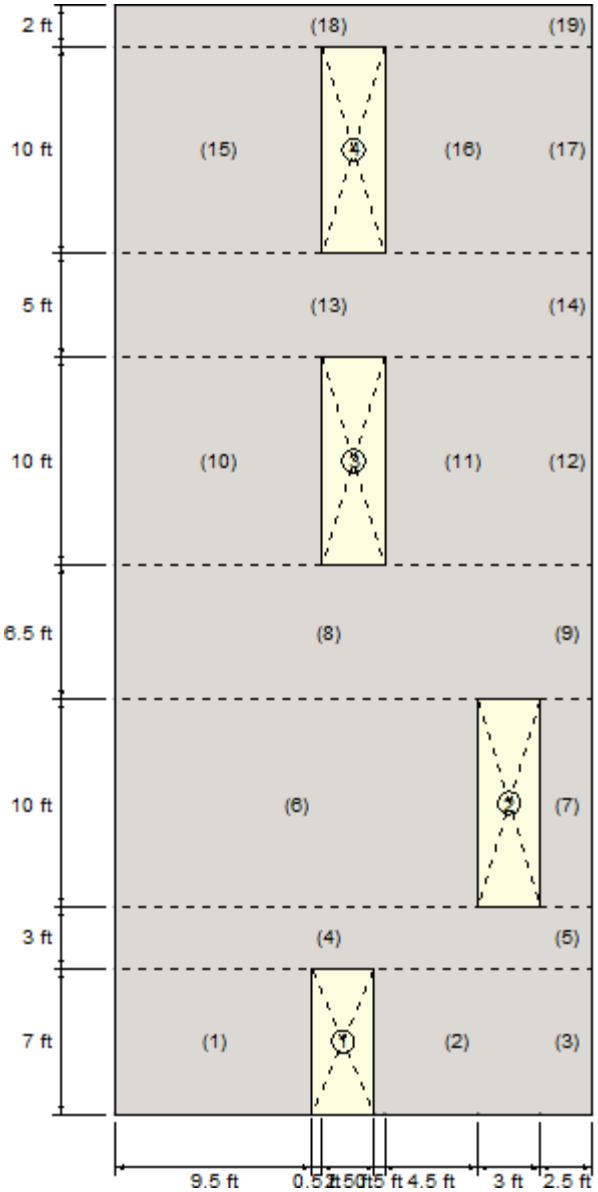
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	206830.090	0.00
2	DC1 (Top)	0.000	11154.192	0.00
3	DC1 (Top)	0.000	100387.730	0.00
4	DC1 (Top)	0.000	66925.152	0.00
5	DC1 (Top)	0.000	56642.381	0.00
6	DC1 (Top)	0.000	208485.790	0.00
7	DC1 (Top)	0.000	11067.050	0.00
8	DC1 (Top)	0.000	55335.250	0.00
9	DC1 (Top)	0.000	11067.050	0.00
10	DC1 (Top)	0.000	99603.449	0.00
11	DC1 (Top)	0.000	66402.299	0.00
12	DC1 (Top)	0.000	56206.671	0.00
13	DC1 (Top)	0.000	131608.310	0.00
14	DC1 (Top)	0.000	6926.753	0.00
15	DC1 (Top)	0.000	34633.766	0.00
16	DC1 (Top)	0.000	6926.753	0.00
17	DC1 (Top)	0.000	62340.779	0.00
18	DC1 (Top)	0.000	35946.127	0.00
19	DC1 (Top)	0.000	129952.610	0.00
20	DC1 (Top)	0.000	6839.611	0.00
21	DC1 (Top)	0.000	34198.056	0.00
22	DC1 (Top)	0.000	6839.611	0.00
23	DC1 (Top)	0.000	61556.500	0.00
24	DC1 (Top)	0.000	41566.794	0.00

25	DC1 (Top)	0.000	35946.127	0.00
26	DC1 (Top)	0.000	134939.580	0.00
27	DC1 (Top)	0.000	7013.895	0.00
28	DC1 (Top)	0.000	63134.470	0.00
29	DC1 (Top)	0.000	42089.647	0.00
30	DC1 (Top)	0.000	35946.127	0.00
31	DC1 (Top)	0.000	134939.580	0.00
32	DC1 (Top)	0.000	7013.895	0.00
33	DC1 (Top)	0.000	35728.271	0.00
34	DC1 (Top)	0.000	7058.512	0.00
35	DC1 (Top)	0.000	63918.749	0.00
36	DC1 (Top)	0.000	42612.499	0.00
37	DC1 (Top)	0.000	35946.127	0.00
38	DC1 (Top)	0.000	110104.080	0.00
39	DC1 (Top)	0.000	5751.380	0.00
40	DC1 (Top)	0.000	52154.562	0.00
41	DC1 (Top)	0.000	34769.708	0.00
42	DC1 (Top)	0.000	28974.757	0.00
43	DC1 (Top)	0.000	110104.080	0.00
44	DC1 (Top)	0.000	5751.380	0.00
45	DC1 (Top)	0.000	28974.757	0.00
46	DC1 (Top)	0.000	5794.951	0.00
47	DC1 (Top)	0.000	52154.562	0.00
48	DC1 (Top)	0.000	34769.708	0.00
49	DC1 (Top)	0.000	28974.757	0.00

SHEAR WALL DESIGN:

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 1)
- Insufficient shear strength (Segment 10)



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	9.50	7.00
2	12.50	0.00	8.00	7.00
3	20.50	0.00	2.50	7.00
4	0.00	7.00	20.50	3.00
5	20.50	7.00	2.50	3.00
6	0.00	10.00	17.50	10.00
7	20.50	10.00	2.50	10.00
8	0.00	20.00	20.50	6.50
9	20.50	20.00	2.50	6.50
10	0.00	26.50	10.00	10.00
11	13.00	26.50	7.50	10.00
12	20.50	26.50	2.50	10.00
13	0.00	36.50	20.50	5.00
14	20.50	36.50	2.50	5.00
15	0.00	41.50	10.00	10.00
16	13.00	41.50	7.50	10.00

17	20.50	41.50	2.50	10.00
18	0.00	51.50	20.50	2.00
19	20.50	51.50	2.50	2.00

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	**Bars**	**Spacing** [in]	**Ld** [in]	**Bars**	**Spacing** [in]	**Ld** [in]
1	12-#10	10.00	49.19	7-#8	12.00	50.35
2	1-#8	10.00	38.73	6-#6	16.00	30.21
	6-#8	10.00	38.73	6-#6	16.00	30.21
	4-#8	10.00	38.73	6-#6	16.00	30.21
3	2-#6	18.00	23.24	7-#5	12.00	25.17
4	12-#10	10.00	49.19	2-#7	18.00	44.06
	1-#8	10.00	49.19	2-#7	18.00	44.06
	3-#8	10.00	49.19	2-#7	18.00	44.06
	1-#8	10.00	49.19	2-#7	18.00	44.06
	6-#8	10.00	49.19	2-#7	18.00	44.06
	4-#8	10.00	49.19	2-#7	18.00	44.06
5	2-#6	18.00	23.24	2-#6	18.00	30.21
6	12-#9	10.00	43.69	12-#6	10.00	30.21
	1-#9	12.00	43.69	12-#6	10.00	30.21
	3-#9	12.00	43.69	12-#6	10.00	30.21
	1-#9	12.00	43.69	12-#6	10.00	30.21
	5-#9	12.00	43.69	12-#6	10.00	30.21
7	3-#5	12.00	19.36	7-#5	18.00	25.17
8	12-#9	10.00	43.69	5-#7	16.00	44.06
	1-#9	12.00	43.69	5-#7	16.00	44.06
	3-#9	12.00	43.69	5-#7	16.00	44.06
	1-#9	12.00	43.69	5-#7	16.00	44.06
	5-#9	12.00	43.69	5-#7	16.00	44.06
	3-#7	14.00	43.69	5-#7	16.00	44.06
9	3-#5	12.00	19.36	5-#5	18.00	25.17
10	10-#7	12.00	43.69	8-#5	16.00	25.17
	1-#9	18.00	43.69	8-#5	16.00	25.17
11	3-#7	18.00	33.89	10-#6	12.00	30.21
	2-#7	18.00	33.89	10-#6	12.00	30.21
12	2-#5	18.00	19.36	7-#5	18.00	25.17
13	10-#7	12.00	43.69	4-#5	16.00	25.17
	1-#9	18.00	43.69	4-#5	16.00	25.17
	2-#6	18.00	43.69	4-#5	16.00	25.17
	1-#8	18.00	43.69	4-#5	16.00	25.17
	3-#7	18.00	43.69	4-#5	16.00	25.17
	2-#7	18.00	43.69	4-#5	16.00	25.17
14	2-#5	18.00	19.36	4-#5	18.00	25.17
15	7-#5	18.00	23.24	7-#5	18.00	25.17
	1-#6	18.00	23.24	7-#5	18.00	25.17
16	5-#5	12.00	19.36	7-#5	18.00	25.17
	3-#5	12.00	19.36	7-#5	18.00	25.17
17	2-#5	18.00	19.36	7-#5	18.00	25.17
18	7-#5	18.00	23.24	2-#5	18.00	25.17
	1-#6	18.00	23.24	2-#5	18.00	25.17
	2-#5	18.00	23.24	2-#5	18.00	25.17
	1-#5	18.00	23.24	2-#5	18.00	25.17
	5-#5	12.00	23.24	2-#5	18.00	25.17
	3-#5	12.00	23.24	2-#5	18.00	25.17
19	2-#5	18.00	19.36	2-#5	18.00	25.17

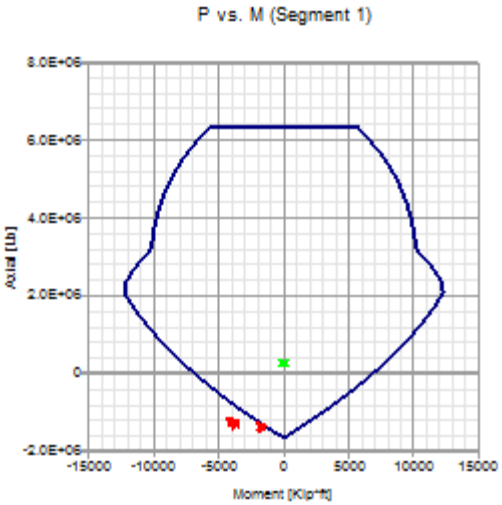
Intermediate results for axial-bending

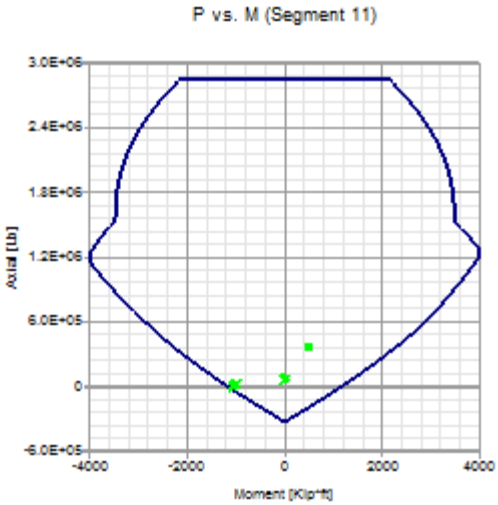
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	2.47	91.20
2	D3 (Bottom)	21.96	76.80
3	D2 (Bottom)	19.62	24.00
4	D3 (Bottom)	29.53	196.80
5	D2 (Top)	10.27	24.00
6	D3 (Bottom)	32.02	168.00
7	D3 (Bottom)	18.16	24.00
8	D3 (Bottom)	33.50	196.80
9	D2 (Bottom)	17.70	24.00
10	D3 (Bottom)	10.51	96.00
11	D3 (Bottom)	6.33	72.00
12	D3 (Max)	7.81	24.00
13	D3 (Bottom)	26.35	196.80
14	DC1 (Top)	1.55	24.00
15	D3 (Max)	10.04	96.00
16	D3 (Max)	4.92	72.00
17	DC1 (Bottom)	1.43	24.00
18	D3 (Top)	20.18	196.80
19	D2 (Top)	1.37	24.00

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	-1282484.00	-4021.46	1689.87	2.38
2	D3 (Bottom)	911917.80	-1737.16	5936.78	0.29
3	D2 (Bottom)	851795.20	-124.72	582.92	0.21
4	D3 (Bottom)	-213627.60	-17009.73	21814.26	0.78
5	D2 (Top)	584622.50	-117.00	641.03	0.18
6	D3 (Bottom)	-311840.60	-11960.10	15317.15	0.78
7	D3 (Bottom)	519064.00	-68.12	391.51	0.17
8	D3 (Bottom)	-399172.50	-5442.01	17432.18	0.31
9	D2 (Bottom)	504983.70	58.69	392.36	0.15
10	D3 (Bottom)	-233414.30	-1826.76	2759.89	0.66
11	D3 (Bottom)	-3183.32	-1059.12	1142.46	0.93
12	D3 (Max)	286182.00	-27.62	359.62	0.08
13	D3 (Bottom)	34082.24	-3754.41	10835.18	0.35
14	DC1 (Top)	22757.29	-7.28	106.54	0.07
15	D3 (Max)	91241.66	-1024.16	1912.30	0.54
16	D3 (Max)	-68001.73	-568.44	694.11	0.82
17	DC1 (Bottom)	4730.59	6.49	85.15	0.08
18	D3 (Top)	115340.20	848.14	6765.51	0.13
19	D2 (Top)	1234.85	-1.45	81.07	0.02

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	284817.80	6312047.00	0.05
2	D2 (Top)	1257185.00	5078820.00	0.25
3	D2 (Bottom)	851795.20	1427412.00	0.60
4	DC1 (Bottom)	468671.00	1.328979E07	0.04
5	D2 (Bottom)	632875.30	1427412.00	0.44
6	DC1 (Bottom)	319601.80	7939152.00	0.04
7	D2 (Bottom)	530657.30	949787.30	0.56
8	D1 (Max)	291138.60	9187589.00	0.03
9	D2 (Bottom)	504983.70	949787.30	0.53
10	D1 (Max)	105773.20	3781752.00	0.03
11	D2 (Top)	376002.80	2848248.00	0.13
12	D2 (Bottom)	325089.10	951431.50	0.34
13	D1 (Bottom)	155126.90	7766806.00	0.02
14	D2 (Bottom)	120871.00	951431.50	0.13
15	D2 (Max)	99234.94	3168557.00	0.03
16	D2 (Top)	61219.60	2373646.00	0.03
17	D2 (Bottom)	77268.07	792311.50	0.10
18	D2 (Top)	125540.20	6491990.00	0.02
19	D2 (Bottom)	6648.93	792311.50	0.01

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Top)	1399985.00	1645920.00	0.85
2	DC1 (Top)	0.00	938520.00	0.00
3	DC1 (Top)	0.00	95040.00	0.00
4	D3 (Top)	235680.30	2925720.00	0.08
5	DC1 (Top)	0.00	95040.00	0.00
6	D3 (Top)	415863.90	2376000.00	0.18
7	DC1 (Top)	0.00	100440.00	0.00
8	D3 (Bottom)	399172.50	2570400.00	0.16

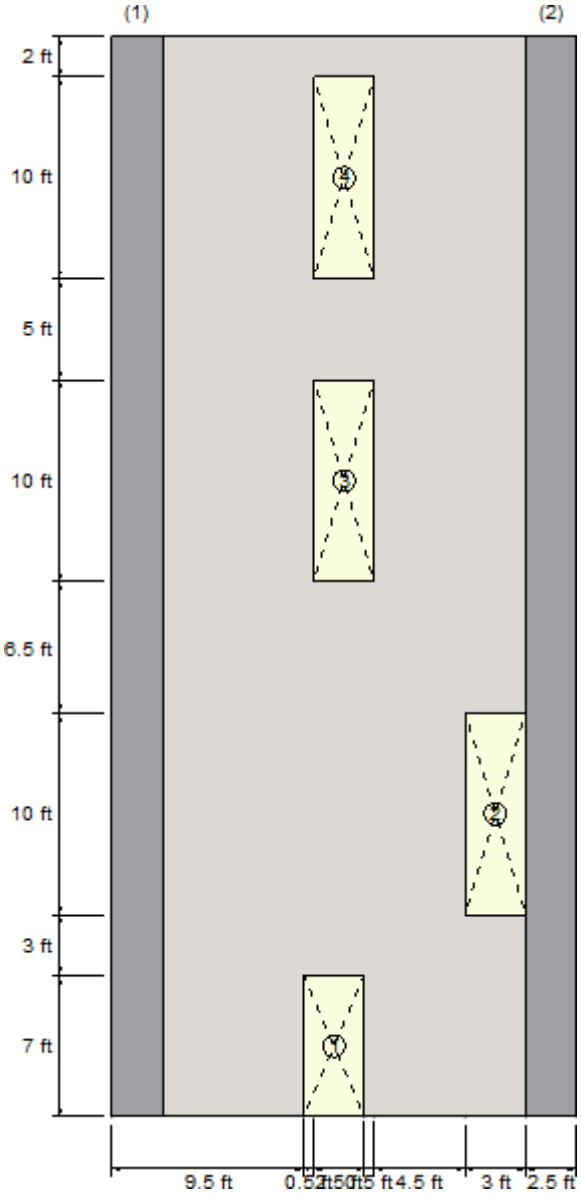
9	DC1 (Top)	0.00	100440.00	0.00
10	D3 (Top)	349683.40	756000.00	0.46
11	D3 (Bottom)	3183.32	324000.00	0.01
12	DC1 (Top)	0.00	66960.00	0.00
13	DC1 (Top)	0.00	1260360.00	0.00
14	DC1 (Top)	0.00	66960.00	0.00
15	D3 (Top)	65382.03	281880.00	0.23
16	D3 (Max)	68001.73	267840.00	0.25
17	DC1 (Top)	0.00	66960.00	0.00
18	DC1 (Top)	0.00	650160.00	0.00
19	DC1 (Top)	0.00	66960.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	504127.930	540360.000	0.93
2	D3 (Top)	635761.070	637467.790	1.00
3	D2 (Bottom)	138912.750	250969.320	0.55
4	D3 (Max)	1199322.400	1247917.500	0.96
5	D2 (Max)	33174.943	186556.730	0.18
6	D3 (Top)	970568.140	989393.180	0.98
7	D2 (Top)	49053.144	167312.880	0.29
8	D3 (Bottom)	1021687.500	1057072.800	0.97
9	D3 (Top)	58056.823	142451.860	0.41
10	D3 (Bottom)	355908.040	353240.860	1.01
11	D3 (Bottom)	386656.530	402762.250	0.96
12	D3 (Max)	73069.916	135340.540	0.54
13	D3 (Max)	781515.310	801259.660	0.98
14	D3 (Bottom)	28806.128	109855.490	0.26
15	D3 (Bottom)	230635.170	342287.250	0.67
16	D2 (Bottom)	201654.930	244542.660	0.82
17	D3 (Max)	29229.797	91740.926	0.32
18	D3 (Bottom)	333888.080	685848.340	0.49
19	D2 (Max)	11308.695	83729.518	0.14

COLUMN DESIGN:

Status : Warnings
- Every longitudinal bar shall have lateral support provided by the corner of a tie, Section 7.10.5.3 (Column 7)



Geometry:

Column	Distance [ft]	Position Z	Width X [in]	Width Z [in]	Height [ft]
1	1.25	Centered	30.00	18.00	53.50
2	21.75	Centered	30.00	18.00	53.50

Reinforcement:

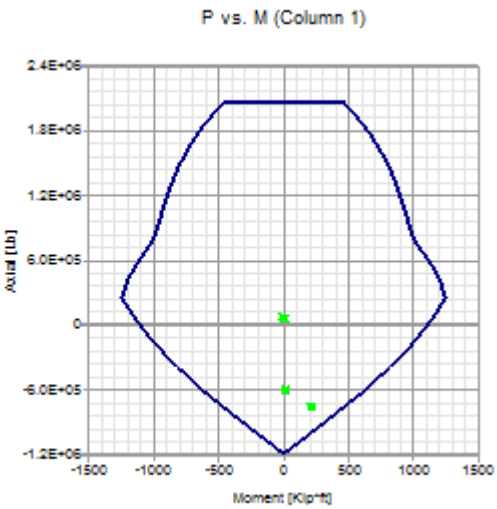
	Longitudinal reinforcement			Transverse reinforcement	
Column	Bars	As [in2]	Ld [in]	Bars	Spacing [in]
1	14-#11	21.84	54.61	#3	3.00
2	14-#11	21.84	54.61	#3	3.00
3	10-#11	15.60	54.61	#3	6.00
4	10-#11	15.60	54.61	#3	6.00
5	8-#11	12.48	54.61	#3	6.00
6	8-#11	12.48	54.61	#3	6.00
7	8-#11	12.48	54.61	#3	6.00

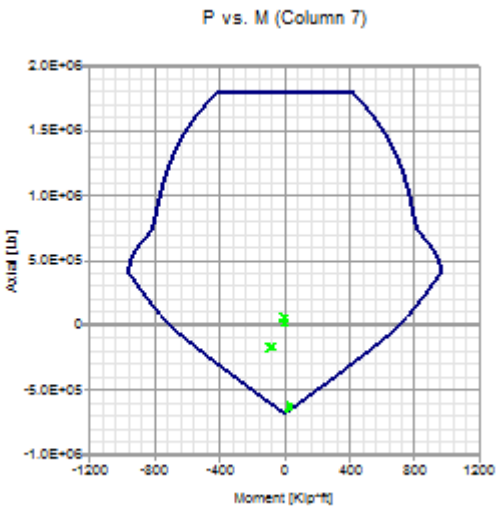
8	6-#11	9.36	54.61	#3	6.00

Combined biaxial flexure

Column	Condition	Pu [Lb]	Mc [Kip*ft]	ϕMn [Kip*ft]	Mc/(ϕ*Mn)
1	D3	756912.00	213.45	458.22	0.47
2	D2	-780071.30	211.94	1008.93	0.21
3	D3	551101.20	37.63	321.73	0.12
4	D2	-754743.30	95.40	610.66	0.16
5	D3	573364.40	39.72	114.74	0.35
6	D2	-756238.30	90.45	552.18	0.16
7	D3	631879.10	31.47	50.55	0.62
8	D2	-751682.70	71.41	495.26	0.14

Interaction diagrams, P vs. M:





Axial tension

Column	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/(ϕ*Pn)
1	D3	756912.00	1179360.00	0.64
2	SC1	0.00	1179360.00	0.00
3	D3	645929.80	842400.00	0.77
4	SC1	0.00	842400.00	0.00
5	D3	573364.40	673920.00	0.85
6	SC1	0.00	673920.00	0.00
7	D3	631879.10	673920.00	0.94
8	SC1	0.00	505440.00	0.00

Axial compression

Column	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/(ϕ*Pn)
1	DC1	-75467.38	-2055568.00	0.04
2	D2	-780071.30	-2055568.00	0.38
3	DC1	-65003.31	-1877429.00	0.03
4	D2	-754743.30	-1877429.00	0.40
5	DC1	-59425.70	-1788359.00	0.03
6	D2	-756238.30	-1788359.00	0.42
7	DC1	-62332.29	-1788359.00	0.03
8	D2	-751682.70	-1699289.00	0.44

Shear along X direction

Column	Condition	Pos	Vu [Lb]	Vc [Lb]	Vs [Lb]	ϕ*Vn [Lb]	Vu/(ϕ*Vn)
1	D3	At bot...	72213.238	0.000	180972.000	135729.000	0.53
2	D2	At bot...	81943.213	131689.580	180972.000	234496.190	0.35
3	D2	At bottom	6442.217	0.000	60324.000	45243.000	0.14
4	D3	At top	18890.013	128758.930	60324.000	141812.200	0.13
5	D2	At bot...	15338.773	0.000	60324.000	45243.000	0.34
6	D3	At top	19628.335	128846.370	60324.000	141877.780	0.14
7	D3	At bot...	41215.485	28882.740	60324.000	66905.055	0.62
8	D2	At top	21399.343	129679.720	60324.000	142502.790	0.15

Shear along Z direction

Column	Condition	Pos	Vu [Lb]	Vc [Lb]	Vs [Lb]	ϕ*Vn [Lb]	Vu/(ϕ*Vn)
1	SC1	At top	0.000	71665.684	203544.000	206407.260	0.00
2	SC1	At top	0.000	71665.684	203544.000	206407.260	0.00
3	SC1	At top	0.000	71665.684	84810.000	117356.760	0.00
4	SC1	At top	0.000	71665.684	84810.000	117356.760	0.00
5	SC1	At top	0.000	71665.684	67848.000	104635.260	0.00
6	SC1	At top	0.000	71665.684	67848.000	104635.260	0.00
7	SC1	At top	0.000	71665.684	50886.000	91913.763	0.00
8	SC1	At top	0.000	71665.684	50886.000	91913.763	0.00

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Schematic Shear Wall Design for Gridline 6

Current Date: 8/4/2016 3:31 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\SPOE GL 6.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : N. G.

Design code	:	ACI 318-05

Geometry:

Total height	:	53.50 [ft]
Total length	:	23.00 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 4

Story	Story height [ft]	Wall thickness [in]
1	10.00	18.00
2	16.50	12.00
3	15.00	12.00
4	12.00	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	4.00	0.00	3.00	7.00
Lower left	17.00	0.00	3.00	7.00
Lower left	2.00	10.00	3.00	10.00
Lower left	8.00	26.50	3.00	10.00
Lower left	2.25	41.50	2.75	10.00
Lower left	7.25	41.50	3.00	10.00

Columns:

Distance [ft]	Width X [in]	Width Z [in]	Position Z
1.00	24.00	18.00	Centered
21.75	30.00	18.00	Centered

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	14400.00	0.00	0.00
1	DL	Vertical	14400.00	0.00	23.00
2	DL	Vertical	13700.00	0.00	0.00
2	DL	Vertical	13700.00	0.00	23.00
3	DL	Vertical	13100.00	0.00	0.00
3	DL	Vertical	13100.00	0.00	23.00
4	EQ	Horizontal	384714.00	0.00	0.00
3	EQ	Horizontal	407204.00	0.00	0.00
2	EQ	Horizontal	205059.00	0.00	0.00
1	EQ	Horizontal	140022.00	0.00	0.00

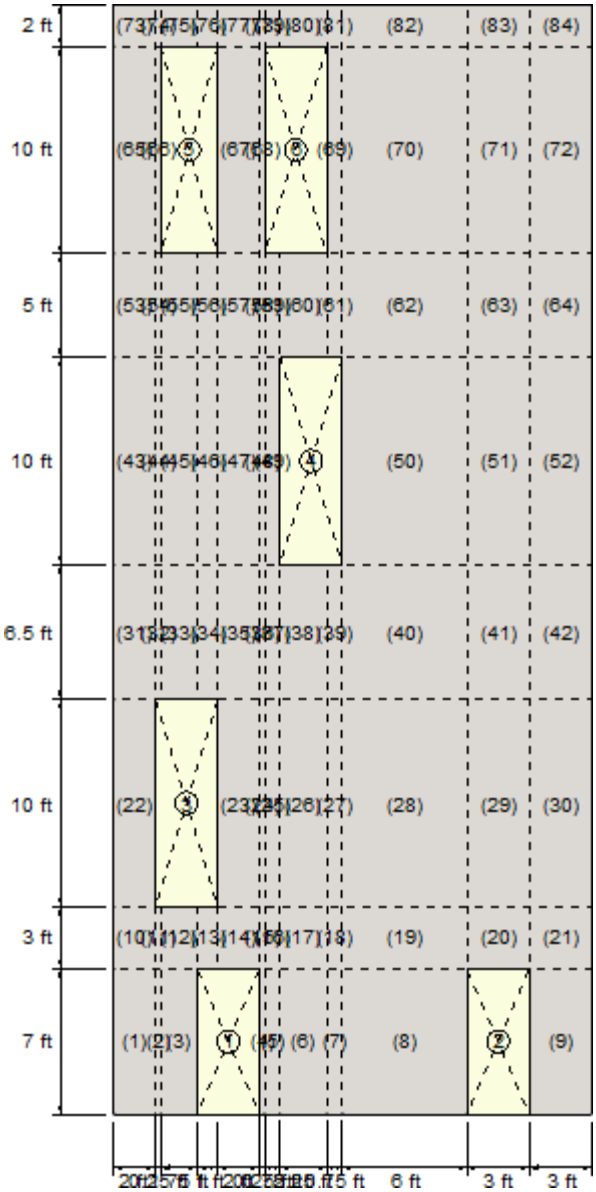
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
4	DL	Vertical	1640.00	0.00
3	DL	Vertical	1710.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : N. G.
- Insufficient axial tension strength (Segment 1)



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.00	7.00
2	2.00	0.00	0.25	7.00
3	2.25	0.00	1.75	7.00
4	7.00	0.00	0.25	7.00
5	7.25	0.00	0.75	7.00
6	8.00	0.00	2.25	7.00
7	10.25	0.00	0.75	7.00
8	11.00	0.00	6.00	7.00
9	20.00	0.00	3.00	7.00
10	0.00	7.00	2.00	3.00
11	2.00	7.00	0.25	3.00
12	2.25	7.00	1.75	3.00
13	4.00	7.00	1.00	3.00
14	5.00	7.00	2.00	3.00
15	7.00	7.00	0.25	3.00
16	7.25	7.00	0.75	3.00

17	8.00	7.00	2.25	3.00
18	10.25	7.00	0.75	3.00
19	11.00	7.00	6.00	3.00
20	17.00	7.00	3.00	3.00
21	20.00	7.00	3.00	3.00
22	0.00	10.00	2.00	10.00
23	5.00	10.00	2.00	10.00
24	7.00	10.00	0.25	10.00
25	7.25	10.00	0.75	10.00
26	8.00	10.00	2.25	10.00
27	10.25	10.00	0.75	10.00
28	11.00	10.00	6.00	10.00
29	17.00	10.00	3.00	10.00
30	20.00	10.00	3.00	10.00
31	0.00	20.00	2.00	6.50
32	2.00	20.00	0.25	6.50
33	2.25	20.00	1.75	6.50
34	4.00	20.00	1.00	6.50
35	5.00	20.00	2.00	6.50
36	7.00	20.00	0.25	6.50
37	7.25	20.00	0.75	6.50
38	8.00	20.00	2.25	6.50
39	10.25	20.00	0.75	6.50
40	11.00	20.00	6.00	6.50
41	17.00	20.00	3.00	6.50
42	20.00	20.00	3.00	6.50
43	0.00	26.50	2.00	10.00
44	2.00	26.50	0.25	10.00
45	2.25	26.50	1.75	10.00
46	4.00	26.50	1.00	10.00
47	5.00	26.50	2.00	10.00
48	7.00	26.50	0.25	10.00
49	7.25	26.50	0.75	10.00
50	11.00	26.50	6.00	10.00
51	17.00	26.50	3.00	10.00
52	20.00	26.50	3.00	10.00
53	0.00	36.50	2.00	5.00
54	2.00	36.50	0.25	5.00
55	2.25	36.50	1.75	5.00
56	4.00	36.50	1.00	5.00
57	5.00	36.50	2.00	5.00
58	7.00	36.50	0.25	5.00
59	7.25	36.50	0.75	5.00
60	8.00	36.50	2.25	5.00
61	10.25	36.50	0.75	5.00
62	11.00	36.50	6.00	5.00
63	17.00	36.50	3.00	5.00
64	20.00	36.50	3.00	5.00
65	0.00	41.50	2.00	10.00
66	2.00	41.50	0.25	10.00
67	5.00	41.50	2.00	10.00
68	7.00	41.50	0.25	10.00
69	10.25	41.50	0.75	10.00
70	11.00	41.50	6.00	10.00
71	17.00	41.50	3.00	10.00
72	20.00	41.50	3.00	10.00
73	0.00	51.50	2.00	2.00
74	2.00	51.50	0.25	2.00
75	2.25	51.50	1.75	2.00
76	4.00	51.50	1.00	2.00
77	5.00	51.50	2.00	2.00
78	7.00	51.50	0.25	2.00
79	7.25	51.50	0.75	2.00
80	8.00	51.50	2.25	2.00

81	10.25	51.50	0.75	2.00
82	11.00	51.50	6.00	2.00
83	17.00	51.50	3.00	2.00
84	20.00	51.50	3.00	2.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	Bars	Spacing [in]	Ld [in]
1	3-#10	10.00	49.19
2	1-#10	10.00	49.19
3	2-#10	10.00	49.19
4	1-#10	12.00	49.19
5	1-#10	12.00	49.19
6	3-#10	12.00	49.19
7	1-#10	12.00	49.19
8	6-#10	12.00	49.19
9	4-#8	10.00	38.73
10	3-#10	10.00	49.19
11	1-#10	10.00	49.19
12	2-#10	10.00	49.19
13	1-#10	10.00	49.19
14	3-#10	10.00	49.19
15	1-#10	12.00	49.19
16	1-#10	12.00	49.19
17	3-#10	12.00	49.19
18	1-#10	12.00	49.19
19	6-#10	12.00	49.19
20	4-#8	10.00	38.73
21	4-#8	10.00	38.73
22	3-#10	10.00	49.19
23	3-#10	10.00	49.19
24	1-#6	12.00	23.24
25	1-#7	18.00	33.89
26	3-#6	12.00	23.24
27	1-#9	18.00	43.69
28	6-#6	12.00	23.24
29	3-#6	12.00	23.24
30	3-#6	12.00	23.24
31	3-#10	10.00	49.19
32	1-#6	18.00	23.24
33	2-#7	18.00	33.89
34	1-#5	10.00	19.36
35	3-#10	10.00	49.19
36	1-#6	12.00	23.24
37	1-#7	18.00	33.89
38	3-#6	12.00	23.24
39	1-#9	18.00	43.69
40	6-#6	12.00	23.24
41	3-#6	12.00	23.24
42	3-#6	12.00	23.24
43	3-#8	10.00	38.73
44	1-#8	14.00	38.73
45	2-#8	14.00	38.73
46	1-#8	14.00	38.73
47	2-#8	14.00	38.73
48	1-#8	14.00	38.73
49	1-#10	10.00	49.19
50	4-#6	18.00	23.24

51	3-#5	14.00	19.36
52	3-#5	14.00	19.36
53	3-#8	10.00	38.73
54	1-#8	14.00	38.73
55	2-#8	14.00	38.73
56	1-#8	14.00	38.73
57	2-#8	14.00	38.73
58	1-#8	14.00	38.73
59	1-#10	10.00	49.19
60	3-#6	12.00	23.24
61	1-#9	18.00	43.69
62	4-#6	18.00	23.24
63	3-#5	14.00	19.36
64	3-#5	14.00	19.36
65	2-#7	18.00	33.89
66	1-#7	18.00	33.89
67	2-#9	18.00	43.69
68	1-#9	18.00	43.69
69	1-#10	10.00	49.19
70	5-#6	16.00	23.24
71	3-#6	16.00	23.24
72	3-#6	16.00	23.24
73	2-#7	18.00	33.89
74	1-#7	18.00	33.89
75	2-#6	16.00	23.24
76	1-#6	16.00	23.24
77	2-#9	18.00	43.69
78	1-#9	18.00	43.69
79	1-#8	18.00	38.73
80	2-#6	16.00	23.24
81	1-#10	10.00	49.19
82	5-#6	16.00	23.24
83	3-#6	16.00	23.24
84	3-#6	16.00	23.24

Intermediate results for axial-bending

Segment	Condition	c [in]	d [in]
1	DC1 (Top)	3.24	15.49
2	DC1 (Top)	7.85	15.49
3	DC1 (Top)	2.60	15.49
4	DC1 (Top)	8.13	15.74
5	DC1 (Top)	3.07	15.74
6	DC1 (Top)	2.95	15.74
7	DC1 (Top)	2.94	15.74
8	DC1 (Top)	2.43	15.74
9	DC1 (Top)	2.18	16.13
10	DC1 (Top)	3.53	15.86
11	DC1 (Top)	7.59	15.86
12	DC1 (Top)	2.14	15.86
13	DC1 (Top)	2.08	15.86
14	DC1 (Top)	3.15	15.86
15	DC1 (Top)	7.71	15.86
16	DC1 (Top)	2.77	15.86
17	DC1 (Top)	2.85	15.86
18	DC1 (Top)	2.87	15.86
19	DC1 (Top)	2.27	15.86
20	DC1 (Top)	1.99	16.00
21	DC1 (Top)	2.10	16.00
22	DC1 (Top)	3.18	9.99
23	DC1 (Top)	3.15	9.74

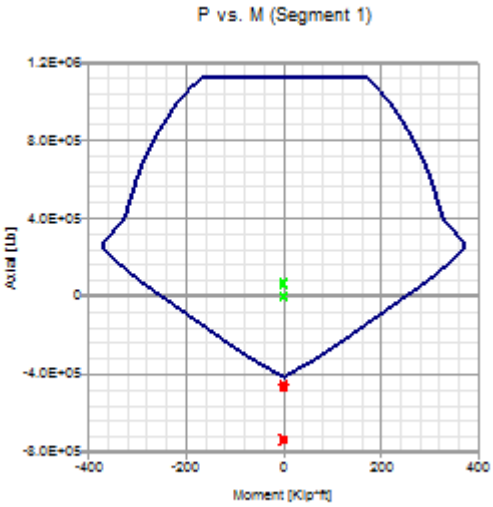
24	DC1 (Top)	2.92	10.00
25	DC1 (Top)	1.74	9.94
26	DC1 (Top)	1.52	10.00
27	DC1 (Top)	2.19	9.81
28	DC1 (Top)	1.35	10.00
29	DC1 (Top)	1.34	10.00
30	DC1 (Top)	1.33	10.00
31	DC1 (Top)	3.12	9.99
32	DC1 (Top)	2.83	10.25
33	DC1 (Top)	1.51	10.19
34	DC1 (Top)	1.08	10.31
35	DC1 (Top)	3.07	9.99
36	DC1 (Top)	2.91	10.25
37	DC1 (Top)	1.64	10.19
38	DC1 (Top)	1.31	10.25
39	DC1 (Top)	2.04	10.06
40	DC1 (Top)	1.28	10.25
41	DC1 (Top)	1.27	10.25
42	DC1 (Top)	1.28	10.25
43	DC1 (Top)	1.93	10.00
44	DC1 (Top)	4.67	10.00
45	DC1 (Top)	1.70	10.00
46	DC1 (Top)	1.61	10.00
47	DC1 (Top)	1.61	10.00
48	DC1 (Top)	4.68	10.00
49	DC1 (Top)	2.47	9.86
50	DC1 (Top)	0.99	10.25
51	DC1 (Top)	0.97	10.31
52	DC1 (Top)	0.97	10.31
53	DC1 (Top)	1.89	10.13
54	DC1 (Top)	4.58	10.13
55	DC1 (Top)	1.57	10.13
56	DC1 (Top)	1.53	10.13
57	DC1 (Top)	1.56	10.13
58	DC1 (Top)	4.66	10.13
59	DC1 (Top)	2.32	9.99
60	DC1 (Top)	1.28	10.25
61	DC1 (Top)	2.00	10.06
62	DC1 (Top)	0.94	10.25
63	DC1 (Top)	0.95	10.31
64	DC1 (Top)	0.98	10.31
65	DC1 (Top)	1.30	8.19
66	DC1 (Top)	3.48	8.19
67	DC1 (Top)	1.70	8.06
68	DC1 (Top)	5.08	8.06
69	DC1 (Top)	2.32	7.99
70	DC1 (Top)	0.98	8.25
71	DC1 (Top)	1.09	8.25
72	DC1 (Top)	1.08	8.25
73	DC1 (Top)	1.29	8.19
74	DC1 (Top)	3.45	8.19
75	DC1 (Top)	1.18	8.25
76	DC1 (Top)	1.08	8.25
77	DC1 (Top)	1.68	8.06
78	DC1 (Top)	5.04	8.06
79	DC1 (Top)	1.68	8.13
80	DC1 (Top)	1.01	8.25
81	DC1 (Top)	2.27	7.99
82	DC1 (Top)	0.97	8.25
83	DC1 (Top)	1.09	8.25
84	DC1 (Top)	1.08	8.25

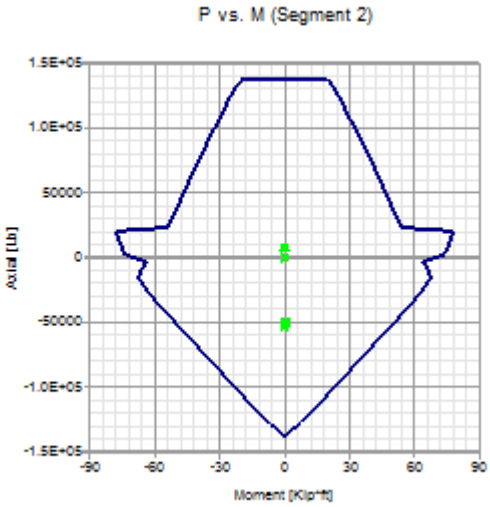
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	62277.51	0.00	284.96	0.00
2	DC1 (Top)	6671.15	0.00	74.84	0.00
3	DC1 (Top)	43128.81	0.00	195.56	0.00
4	DC1 (Top)	9589.81	0.00	76.88	0.00
5	DC1 (Top)	23745.09	0.00	99.08	0.00
6	DC1 (Top)	63432.86	0.00	292.90	0.00
7	DC1 (Top)	20905.08	0.00	97.50	0.00
8	DC1 (Top)	178246.00	0.00	629.37	0.00
9	DC1 (Top)	96843.64	0.00	283.85	0.00
10	DC1 (Top)	68161.01	0.00	293.94	0.00
11	DC1 (Top)	3998.15	0.00	76.08	0.00
12	DC1 (Top)	11146.82	0.00	180.42	0.00
13	DC1 (Top)	10692.40	0.00	94.21	0.00
14	DC1 (Top)	44007.62	0.00	281.03	0.00
15	DC1 (Top)	5215.26	0.00	76.41	0.00
16	DC1 (Top)	17255.18	0.00	96.16	0.00
17	DC1 (Top)	57629.80	0.00	291.82	0.00
18	DC1 (Top)	19539.92	0.00	97.46	0.00
19	DC1 (Top)	150938.80	0.00	617.04	0.00
20	DC1 (Top)	53432.04	0.00	255.68	0.00
21	DC1 (Top)	78728.03	0.00	271.10	0.00
22	DC1 (Top)	39685.27	0.00	167.86	0.00
23	DC1 (Top)	39229.82	0.00	163.90	0.00
24	DC1 (Top)	4352.28	0.00	19.56	0.00
25	DC1 (Top)	12721.90	0.00	31.77	0.00
26	DC1 (Top)	35891.26	0.00	76.24	0.00
27	DC1 (Top)	11793.56	0.00	45.76	0.00
28	DC1 (Top)	94625.20	0.00	168.02	0.00
29	DC1 (Top)	46494.04	0.00	83.69	0.00
30	DC1 (Top)	45039.51	0.00	83.12	0.00
31	DC1 (Top)	34684.67	0.00	166.30	0.00
32	DC1 (Top)	3451.40	0.00	19.71	0.00
33	DC1 (Top)	23840.39	0.00	64.81	0.00
34	DC1 (Top)	13869.22	0.00	21.77	0.00
35	DC1 (Top)	30408.38	0.00	164.97	0.00
36	DC1 (Top)	4287.68	0.00	19.99	0.00
37	DC1 (Top)	12593.88	0.00	32.04	0.00
38	DC1 (Top)	12980.92	0.00	67.77	0.00
39	DC1 (Top)	6800.27	0.00	45.00	0.00
40	DC1 (Top)	94441.51	0.00	167.81	0.00
41	DC1 (Top)	43290.54	0.00	82.38	0.00
42	DC1 (Top)	46582.86	0.00	83.66	0.00
43	DC1 (Top)	17515.37	0.00	108.03	0.00
44	DC1 (Top)	2016.41	0.00	29.12	0.00
45	DC1 (Top)	13630.89	0.00	75.30	0.00
46	DC1 (Top)	7654.90	0.00	38.68	0.00
47	DC1 (Top)	16353.26	0.00	77.74	0.00
48	DC1 (Top)	2020.36	0.00	29.12	0.00
49	DC1 (Top)	7976.11	0.00	54.45	0.00
50	DC1 (Top)	56617.53	0.00	115.48	0.00
51	DC1 (Top)	24655.79	0.00	58.16	0.00
52	DC1 (Top)	24121.87	0.00	57.93	0.00
53	DC1 (Top)	21674.91	0.00	110.54	0.00
54	DC1 (Top)	978.38	0.00	27.90	0.00
55	DC1 (Top)	2064.53	0.00	71.45	0.00

56	DC1 (Top)	5481.49	0.00	38.02	0.00
57	DC1 (Top)	17113.24	0.00	78.32	0.00
58	DC1 (Top)	1815.39	0.00	28.79	0.00
59	DC1 (Top)	3311.35	0.00	53.50	0.00
60	DC1 (Top)	6536.95	0.00	65.30	0.00
61	DC1 (Top)	4987.64	0.00	44.34	0.00
62	DC1 (Top)	44291.26	0.00	109.98	0.00
63	DC1 (Top)	20834.25	0.00	56.49	0.00
64	DC1 (Top)	24972.07	0.00	58.30	0.00
65	DC1 (Top)	3481.79	0.00	47.14	0.00
66	DC1 (Top)	802.39	0.00	18.51	0.00
67	DC1 (Top)	10387.28	0.00	72.97	0.00
68	DC1 (Top)	1975.36	0.00	19.98	0.00
69	DC1 (Top)	3327.58	0.00	41.83	0.00
70	DC1 (Top)	16735.67	0.00	96.58	0.00
71	DC1 (Top)	7377.09	0.00	55.68	0.00
72	DC1 (Top)	4550.04	0.00	54.75	0.00
73	DC1 (Top)	2343.29	0.00	46.80	0.00
74	DC1 (Top)	528.69	0.00	18.37	0.00
75	DC1 (Top)	4116.85	0.00	36.14	0.00
76	DC1 (Top)	1483.40	0.00	18.24	0.00
77	DC1 (Top)	5558.82	0.00	71.62	0.00
78	DC1 (Top)	661.48	0.00	18.94	0.00
79	DC1 (Top)	1862.17	0.00	28.09	0.00
80	DC1 (Top)	5399.76	0.00	37.87	0.00
81	DC1 (Top)	1843.54	0.00	41.42	0.00
82	DC1 (Top)	14529.76	0.00	95.85	0.00
83	DC1 (Top)	7286.44	0.00	55.65	0.00
84	DC1 (Top)	4990.11	0.00	54.90	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	68609.59	1125456.00	0.06
2	D1 (Bottom)	7937.38	136471.90	0.06
3	DC1 (Bottom)	53760.83	988983.80	0.05
4	D1 (Top)	9589.81	136471.90	0.07
5	DC1 (Bottom)	26056.88	422887.90	0.06
6	DC1 (Bottom)	71344.30	1268664.00	0.06
7	DC1 (Bottom)	23368.14	422887.90	0.06
8	D2 (Bottom)	920404.30	3396576.00	0.27
9	D2 (Top)	1075170.00	1701735.00	0.63
10	D1 (Top)	68161.01	1125456.00	0.06
11	D1 (Bottom)	6671.15	136471.90	0.05
12	D1 (Bottom)	43128.57	988983.80	0.04
13	DC1 (Bottom)	14386.78	566095.90	0.03
14	DC1 (Top)	44007.62	1125456.00	0.04
15	DC1 (Bottom)	9589.33	136471.90	0.07
16	DC1 (Bottom)	23745.06	422887.90	0.06
17	D1 (Bottom)	63432.85	1268664.00	0.05
18	DC1 (Bottom)	20905.08	422887.90	0.05
19	D2 (Bottom)	343013.60	3396576.00	0.10
20	D2 (Top)	335399.60	1701735.00	0.20
21	D2 (Bottom)	1064372.00	1701735.00	0.63
22	DC1 (Bottom)	43350.15	743567.80	0.06
23	D1 (Top)	39229.82	743567.80	0.05
24	D1 (Bottom)	4528.32	93138.24	0.05
25	DC1 (Bottom)	13917.84	283233.60	0.05
26	DC1 (Bottom)	45852.53	852246.70	0.05
27	D1 (Bottom)	15946.59	281112.00	0.06
28	D2 (Bottom)	203317.90	2277325.00	0.09
29	D2 (Bottom)	348154.60	1138663.00	0.31
30	D2 (Bottom)	700026.00	1138663.00	0.61
31	DC1 (Bottom)	39685.17	743567.80	0.05
32	D1 (Top)	3451.40	93138.24	0.04
33	DC1 (Top)	23840.39	661939.20	0.04
34	D2 (Bottom)	43902.71	380243.80	0.12
35	D2 (Top)	59591.82	743567.80	0.08
36	D2 (Top)	22220.31	93138.24	0.24
37	D2 (Top)	93839.09	283233.60	0.33
38	D2 (Top)	116532.40	852246.70	0.14
39	DC1 (Bottom)	11793.55	281112.00	0.04
40	D2 (Bottom)	97687.82	2277325.00	0.04
41	D2 (Bottom)	275335.20	1138663.00	0.24
42	D2 (Bottom)	444063.70	1138663.00	0.39
43	D1 (Top)	17515.37	751205.50	0.02
44	D1 (Bottom)	2520.56	91281.84	0.03
45	D1 (Bottom)	17439.78	659923.70	0.03
46	D1 (Bottom)	10166.87	377697.80	0.03
47	D2 (Bottom)	38647.77	755395.70	0.05
48	D2 (Bottom)	18951.63	91281.84	0.21
49	D2 (Bottom)	124732.60	279679.90	0.45
50	D2 (Top)	187804.20	2281993.00	0.08
51	D2 (Bottom)	210917.60	1140731.00	0.18
52	D2 (Bottom)	334895.80	1140731.00	0.29
53	D2 (Top)	39224.23	751205.50	0.05
54	D2 (Top)	10068.08	91281.84	0.11
55	D2 (Top)	93387.15	659923.70	0.14
56	D1 (Bottom)	7654.89	377697.80	0.02
57	DC1 (Top)	17113.24	755395.70	0.02

58	D2 (Top)	18017.07	91281.84	0.20
59	D2 (Top)	82243.12	279679.90	0.29
60	DC1 (Top)	6536.95	852246.70	0.01
61	D2 (Bottom)	102687.60	281112.00	0.37
62	D2 (Bottom)	187800.70	2281993.00	0.08
63	D2 (Bottom)	101170.10	1140731.00	0.09
64	D2 (Bottom)	175341.60	1140731.00	0.15
65	D2 (Bottom)	46966.72	630115.20	0.07
66	D2 (Bottom)	12069.24	76377.60	0.16
67	D2 (Top)	43749.91	625872.00	0.07
68	D2 (Bottom)	42138.66	74256.00	0.57
69	D2 (Top)	31327.85	231943.90	0.14
70	DC1 (Bottom)	28101.92	1897771.00	0.01
71	D2 (Bottom)	88326.27	947718.70	0.09
72	D2 (Bottom)	113077.10	947718.70	0.12
73	D2 (Top)	103446.60	630115.20	0.16
74	D2 (Top)	8419.44	76377.60	0.11
75	DC1 (Top)	4116.85	552252.50	0.01
76	D2 (Bottom)	81969.57	315906.20	0.26
77	D2 (Top)	26262.21	625872.00	0.04
78	D1 (Bottom)	1235.62	74256.00	0.02
79	D1 (Top)	1862.17	234489.80	0.01
80	D2 (Bottom)	118183.10	711372.50	0.17
81	D2 (Bottom)	31242.56	231943.90	0.13
82	DC1 (Bottom)	16735.67	1897771.00	0.01
83	D2 (Bottom)	9410.09	947718.70	0.01
84	D2 (Bottom)	9848.44	947718.70	0.01

Axial tension

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	741282.40	411480.00	1.80
2	D3 (Bottom)	52475.38	137160.00	0.38
3	D3 (Top)	391767.80	274320.00	1.43
4	D3 (Bottom)	113000.70	137160.00	0.82
5	D3 (Bottom)	263927.00	137160.00	1.92
6	D3 (Bottom)	401984.00	411480.00	0.98
7	D3 (Bottom)	56486.14	137160.00	0.41
8	DC1 (Top)	0.00	822960.00	0.00
9	DC1 (Top)	0.00	341280.00	0.00
10	D3 (Bottom)	476445.20	411480.00	1.16
11	D3 (Bottom)	50751.55	137160.00	0.37
12	D3 (Bottom)	391764.40	274320.00	1.43
13	D3 (Bottom)	244618.90	137160.00	1.78
14	D3 (Top)	409183.50	411480.00	0.99
15	D3 (Top)	13723.91	137160.00	0.10
16	D3 (Bottom)	63068.09	137160.00	0.46
17	D3 (Bottom)	187192.90	411480.00	0.45
18	D3 (Bottom)	31546.55	137160.00	0.23
19	DC1 (Top)	0.00	822960.00	0.00
20	DC1 (Top)	0.00	341280.00	0.00
21	DC1 (Top)	0.00	341280.00	0.00
22	D3 (Top)	411438.40	411480.00	1.00
23	D3 (Bottom)	424731.30	411480.00	1.03
24	D3 (Bottom)	23638.32	47520.00	0.50
25	D3 (Bottom)	53564.13	64800.00	0.83
26	D3 (Bottom)	86446.13	142560.00	0.61

27	D3 (Top)	14518.90	108000.00	0.13
28	DC1 (Top)	0.00	285120.00	0.00
29	DC1 (Top)	0.00	142560.00	0.00
30	DC1 (Top)	0.00	142560.00	0.00
31	D3 (Bottom)	411436.90	411480.00	1.00
32	D3 (Bottom)	46251.05	47520.00	0.97
33	D3 (Top)	117295.00	129600.00	0.91
34	D3 (Top)	23218.89	33480.00	0.69
35	DC1 (Top)	0.00	411480.00	0.00
36	D3 (Bottom)	2727.26	47520.00	0.06
37	D3 (Bottom)	9765.39	64800.00	0.15
38	D3 (Bottom)	39404.28	142560.00	0.28
39	D3 (Top)	95839.67	108000.00	0.89
40	D3 (Top)	135712.70	285120.00	0.48
41	DC1 (Top)	0.00	142560.00	0.00
42	DC1 (Top)	0.00	142560.00	0.00
43	D3 (Bottom)	222719.30	255960.00	0.87
44	D3 (Bottom)	25396.11	85320.00	0.30
45	D3 (Bottom)	118583.50	170640.00	0.69
46	D3 (Bottom)	29192.14	85320.00	0.34
47	D3 (Top)	97012.41	170640.00	0.57
48	D3 (Top)	20432.88	85320.00	0.24
49	D3 (Top)	160638.90	137160.00	1.17
50	D3 (Bottom)	168753.80	190080.00	0.89
51	DC1 (Top)	0.00	100440.00	0.00
52	DC1 (Top)	0.00	100440.00	0.00
53	D3 (Bottom)	75045.96	255960.00	0.29
54	D3 (Bottom)	4914.92	85320.00	0.06
55	D3 (Bottom)	21454.44	170640.00	0.13
56	D3 (Top)	48706.37	85320.00	0.57
57	D3 (Bottom)	97012.51	170640.00	0.57
58	D3 (Bottom)	20432.73	85320.00	0.24
59	D3 (Bottom)	160620.60	137160.00	1.17
60	D3 (Bottom)	130039.50	142560.00	0.91
61	D3 (Top)	96069.13	108000.00	0.89
62	D3 (Top)	23176.96	190080.00	0.12
63	DC1 (Top)	0.00	100440.00	0.00
64	DC1 (Top)	0.00	100440.00	0.00
65	D3 (Top)	56653.27	129600.00	0.44
66	D3 (Top)	2356.26	64800.00	0.04
67	D3 (Bottom)	77390.71	216000.00	0.36
68	D3 (Top)	54269.46	108000.00	0.50
69	D3 (Bottom)	141006.60	137160.00	1.03
70	D3 (Top)	36632.00	237600.00	0.15
71	DC1 (Top)	0.00	142560.00	0.00
72	DC1 (Top)	0.00	142560.00	0.00
73	D3 (Bottom)	56652.94	129600.00	0.44
74	D3 (Bottom)	2356.17	64800.00	0.04
75	D3 (Bottom)	82284.85	95040.00	0.87
76	DC1 (Top)	0.00	47520.00	0.00
77	D3 (Bottom)	8189.16	216000.00	0.04
78	D3 (Bottom)	27782.71	108000.00	0.26
79	D3 (Bottom)	71926.16	85320.00	0.84
80	D3 (Top)	13479.47	95040.00	0.14
81	DC1 (Top)	0.00	137160.00	0.00
82	D3 (Bottom)	36631.96	237600.00	0.15
83	DC1 (Top)	0.00	142560.00	0.00
84	DC1 (Top)	0.00	142560.00	0.00

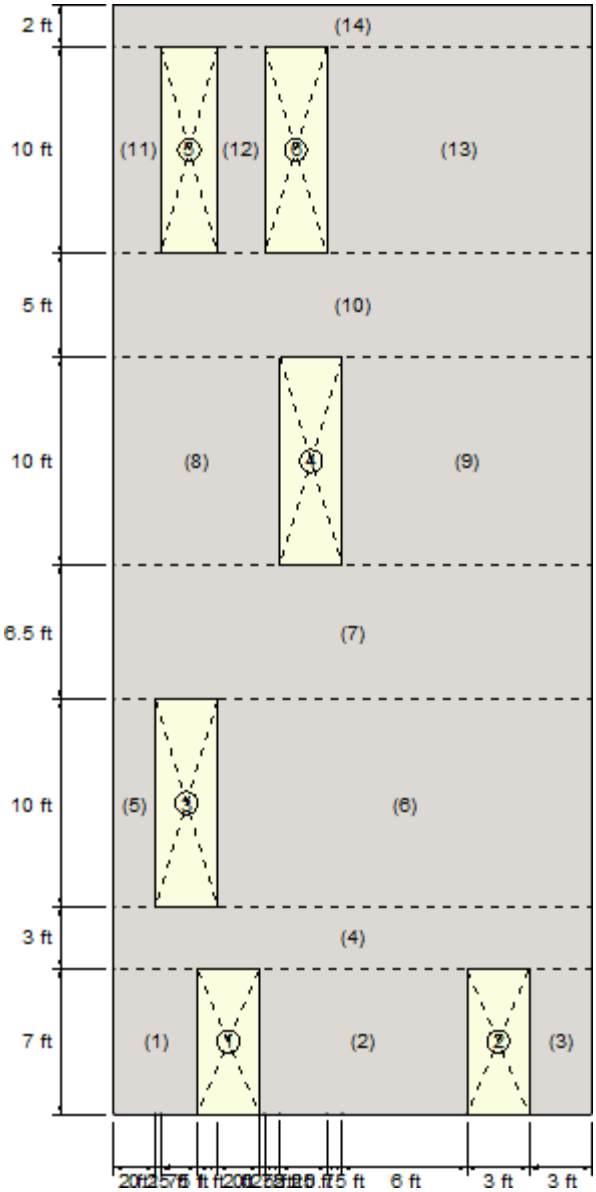
Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	43186.243	0.00
2	DC1 (Top)	0.000	5398.280	0.00
3	DC1 (Top)	0.000	37787.963	0.00
4	DC1 (Top)	0.000	5486.468	0.00
5	DC1 (Top)	0.000	16459.405	0.00
6	DC1 (Top)	0.000	49378.214	0.00
7	DC1 (Top)	0.000	16459.405	0.00
8	DC1 (Top)	0.000	131675.240	0.00
9	DC1 (Top)	0.000	67448.005	0.00
10	DC1 (Top)	0.000	44240.314	0.00
11	DC1 (Top)	0.000	5530.039	0.00
12	DC1 (Top)	0.000	38710.275	0.00
13	DC1 (Top)	0.000	22120.157	0.00
14	DC1 (Top)	0.000	44240.314	0.00
15	DC1 (Top)	0.000	5530.039	0.00
16	DC1 (Top)	0.000	16590.118	0.00
17	DC1 (Top)	0.000	49770.353	0.00
18	DC1 (Top)	0.000	16590.118	0.00
19	DC1 (Top)	0.000	132720.940	0.00
20	DC1 (Top)	0.000	66925.152	0.00
21	DC1 (Top)	0.000	66925.152	0.00
22	DC1 (Top)	0.000	27857.595	0.00
23	DC1 (Top)	0.000	27160.458	0.00
24	DC1 (Top)	0.000	3485.685	0.00
25	DC1 (Top)	0.000	10391.698	0.00
26	DC1 (Top)	0.000	31371.165	0.00
27	DC1 (Top)	0.000	10259.417	0.00
28	DC1 (Top)	0.000	83656.440	0.00
29	DC1 (Top)	0.000	41828.220	0.00
30	DC1 (Top)	0.000	41828.220	0.00
31	DC1 (Top)	0.000	27857.595	0.00
32	DC1 (Top)	0.000	3572.827	0.00
33	DC1 (Top)	0.000	24857.291	0.00
34	DC1 (Top)	0.000	14378.451	0.00
35	DC1 (Top)	0.000	27857.595	0.00
36	DC1 (Top)	0.000	3572.827	0.00
37	DC1 (Top)	0.000	10653.125	0.00
38	DC1 (Top)	0.000	32155.444	0.00
39	DC1 (Top)	0.000	10520.843	0.00
40	DC1 (Top)	0.000	85747.851	0.00
41	DC1 (Top)	0.000	42873.926	0.00
42	DC1 (Top)	0.000	42873.926	0.00
43	DC1 (Top)	0.000	27885.480	0.00
44	DC1 (Top)	0.000	3485.685	0.00
45	DC1 (Top)	0.000	24399.795	0.00
46	DC1 (Top)	0.000	13942.740	0.00
47	DC1 (Top)	0.000	27885.480	0.00
48	DC1 (Top)	0.000	3485.685	0.00
49	DC1 (Top)	0.000	10315.885	0.00
50	DC1 (Top)	0.000	85747.851	0.00
51	DC1 (Top)	0.000	43135.352	0.00
52	DC1 (Top)	0.000	43135.352	0.00
53	DC1 (Top)	0.000	28234.049	0.00
54	DC1 (Top)	0.000	3529.256	0.00
55	DC1 (Top)	0.000	24704.793	0.00

56	DC1 (Top)	0.000	14117.024	0.00
57	DC1 (Top)	0.000	28234.049	0.00
58	DC1 (Top)	0.000	3529.256	0.00
59	DC1 (Top)	0.000	10446.598	0.00
60	DC1 (Top)	0.000	32155.444	0.00
61	DC1 (Top)	0.000	10520.843	0.00
62	DC1 (Top)	0.000	85747.851	0.00
63	DC1 (Top)	0.000	43135.352	0.00
64	DC1 (Top)	0.000	43135.352	0.00
65	DC1 (Top)	0.000	22831.237	0.00
66	DC1 (Top)	0.000	2853.905	0.00
67	DC1 (Top)	0.000	22478.486	0.00
68	DC1 (Top)	0.000	2809.811	0.00
69	DC1 (Top)	0.000	8355.187	0.00
70	DC1 (Top)	0.000	69016.563	0.00
71	DC1 (Top)	0.000	34508.282	0.00
72	DC1 (Top)	0.000	34508.282	0.00
73	DC1 (Top)	0.000	22831.237	0.00
74	DC1 (Top)	0.000	2853.905	0.00
75	DC1 (Top)	0.000	20129.831	0.00
76	DC1 (Top)	0.000	11502.761	0.00
77	DC1 (Top)	0.000	22478.486	0.00
78	DC1 (Top)	0.000	2809.811	0.00
79	DC1 (Top)	0.000	8496.357	0.00
80	DC1 (Top)	0.000	25881.211	0.00
81	DC1 (Top)	0.000	8355.187	0.00
82	DC1 (Top)	0.000	69016.563	0.00
83	DC1 (Top)	0.000	34508.282	0.00
84	DC1 (Top)	0.000	34508.282	0.00

SHEAR WALL DESIGN:

Status : N. G.
- Insufficient combined axial-flexural resistance, ØMn < Mu (Segment 1)
- Insufficient axial tension strength (Segment 1)



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	4.00	7.00
2	7.00	0.00	10.00	7.00
3	20.00	0.00	3.00	7.00
4	0.00	7.00	23.00	3.00
5	0.00	10.00	2.00	10.00
6	5.00	10.00	18.00	10.00
7	0.00	20.00	23.00	6.50
8	0.00	26.50	8.00	10.00
9	11.00	26.50	12.00	10.00
10	0.00	36.50	23.00	5.00
11	0.00	41.50	2.25	10.00
12	5.00	41.50	2.25	10.00
13	10.25	41.50	12.75	10.00
14	0.00	51.50	23.00	2.00

Reinforcement:

Reinforcement layers : 2

	Vertical reinforcement			Horizontal reinforcement		
Segment	**Bars**	**Spacing** [in]	**Ld** [in]	**Bars**	**Spacing** [in]	**Ld** [in]
1	3-#10	10.00	49.19	7-#9	12.00	56.79
	1-#10	10.00	49.19	7-#9	12.00	56.79
	2-#10	10.00	49.19	7-#9	12.00	56.79
2	1-#10	12.00	49.19	7-#7	12.00	44.06
	1-#10	12.00	49.19	7-#7	12.00	44.06
	3-#10	12.00	49.19	7-#7	12.00	44.06
	1-#10	12.00	49.19	7-#7	12.00	44.06
	6-#10	12.00	49.19	7-#7	12.00	44.06
3	4-#8	10.00	38.73	7-#5	12.00	25.17
4	3-#10	10.00	49.19	2-#6	18.00	30.21
	1-#10	10.00	49.19	2-#6	18.00	30.21
	2-#10	10.00	49.19	2-#6	18.00	30.21
	1-#10	10.00	49.19	2-#6	18.00	30.21
	3-#10	10.00	49.19	2-#6	18.00	30.21
	1-#10	12.00	49.19	2-#6	18.00	30.21
	1-#10	12.00	49.19	2-#6	18.00	30.21
	3-#10	12.00	49.19	2-#6	18.00	30.21
	1-#10	12.00	49.19	2-#6	18.00	30.21
	6-#10	12.00	49.19	2-#6	18.00	30.21
	4-#8	10.00	49.19	2-#6	18.00	30.21
	4-#8	10.00	49.19	2-#6	18.00	30.21
5	3-#10	10.00	49.19	8-#5	16.00	25.17
6	3-#10	10.00	49.19	7-#7	18.00	44.06
	1-#6	12.00	49.19	7-#7	18.00	44.06
	1-#7	18.00	49.19	7-#7	18.00	44.06
	3-#6	12.00	49.19	7-#7	18.00	44.06
	1-#9	18.00	49.19	7-#7	18.00	44.06
	6-#6	12.00	49.19	7-#7	18.00	44.06
	3-#6	12.00	49.19	7-#7	18.00	44.06
	3-#6	12.00	49.19	7-#7	18.00	44.06
7	3-#10	10.00	49.19	7-#5	12.00	25.17
	1-#6	18.00	49.19	7-#5	12.00	25.17
	2-#7	18.00	49.19	7-#5	12.00	25.17
	1-#5	10.00	49.19	7-#5	12.00	25.17
	3-#10	10.00	49.19	7-#5	12.00	25.17
	1-#6	12.00	49.19	7-#5	12.00	25.17
	1-#7	18.00	49.19	7-#5	12.00	25.17
	3-#6	12.00	49.19	7-#5	12.00	25.17
	1-#9	18.00	49.19	7-#5	12.00	25.17
	6-#6	12.00	49.19	7-#5	12.00	25.17
	3-#6	12.00	49.19	7-#5	12.00	25.17
	3-#6	12.00	49.19	7-#5	12.00	25.17
8	3-#8	10.00	49.19	10-#6	12.00	30.21
	1-#8	14.00	49.19	10-#6	12.00	30.21
	2-#8	14.00	49.19	10-#6	12.00	30.21
	1-#8	14.00	49.19	10-#6	12.00	30.21
	2-#8	14.00	49.19	10-#6	12.00	30.21
	1-#8	14.00	49.19	10-#6	12.00	30.21
	1-#10	10.00	49.19	10-#6	12.00	30.21
9	4-#6	18.00	23.24	7-#5	18.00	25.17
	3-#5	14.00	23.24	7-#5	18.00	25.17
	3-#5	14.00	23.24	7-#5	18.00	25.17
10	3-#8	10.00	49.19	4-#5	18.00	25.17
	1-#8	14.00	49.19	4-#5	18.00	25.17
	2-#8	14.00	49.19	4-#5	18.00	25.17
	1-#8	14.00	49.19	4-#5	18.00	25.17

	2-#8	14.00	49.19	4-#5	18.00	25.17
	1-#8	14.00	49.19	4-#5	18.00	25.17
	1-#10	10.00	49.19	4-#5	18.00	25.17
	3-#6	12.00	49.19	4-#5	18.00	25.17
	1-#9	18.00	49.19	4-#5	18.00	25.17
	4-#6	18.00	49.19	4-#5	18.00	25.17
	3-#5	14.00	49.19	4-#5	18.00	25.17
	3-#5	14.00	49.19	4-#5	18.00	25.17
11	2-#7	18.00	33.89	7-#5	18.00	25.17
	1-#7	18.00	33.89	7-#5	18.00	25.17
12	2-#9	18.00	43.69	7-#5	18.00	25.17
	1-#9	18.00	43.69	7-#5	18.00	25.17
13	1-#10	10.00	49.19	7-#5	18.00	25.17
	5-#6	16.00	49.19	7-#5	18.00	25.17
	3-#6	16.00	49.19	7-#5	18.00	25.17
	3-#6	16.00	49.19	7-#5	18.00	25.17
14	2-#7	18.00	49.19	2-#5	18.00	25.17
	1-#7	18.00	49.19	2-#5	18.00	25.17
	2-#6	16.00	49.19	2-#5	18.00	25.17
	1-#6	16.00	49.19	2-#5	18.00	25.17
	2-#9	18.00	49.19	2-#5	18.00	25.17
	1-#9	18.00	49.19	2-#5	18.00	25.17
	1-#8	18.00	49.19	2-#5	18.00	25.17
	2-#6	16.00	49.19	2-#5	18.00	25.17
	1-#10	10.00	49.19	2-#5	18.00	25.17
	5-#6	16.00	49.19	2-#5	18.00	25.17
	3-#6	16.00	49.19	2-#5	18.00	25.17
	3-#6	16.00	49.19	2-#5	18.00	25.17

Intermediate results for axial-bending

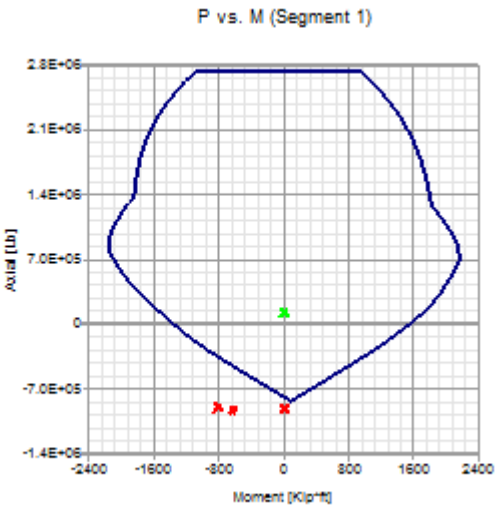
Segment	Condition	c [in]	d [in]
1	D2 (Top)	48.00	38.40
2	D3 (Bottom)	15.65	96.00
3	D2 (Bottom)	25.01	28.80
4	D3 (Bottom)	38.81	220.80
5	D3 (Top)	24.00	19.20
6	D3 (Max)	26.20	172.80
7	D3 (Bottom)	32.67	220.80
8	D3 (Bottom)	12.52	76.80
9	D3 (Bottom)	16.84	115.20
10	D3 (Bottom)	29.50	220.80
11	D3 (Top)	1.51	21.60
12	D3 (Top)	3.03	21.60
13	D3 (Max)	13.21	122.40
14	D3 (Bottom)	31.60	220.80

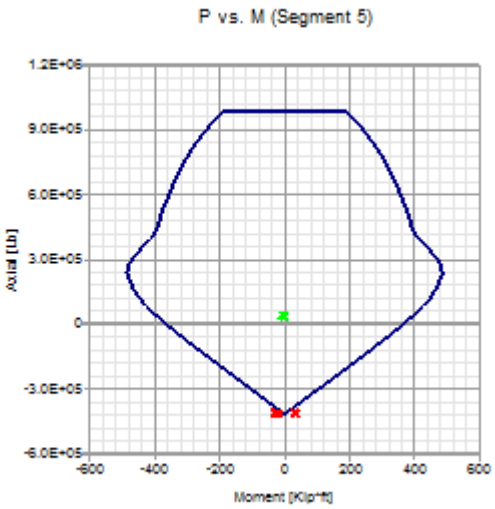
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D2 (Top)	-891665.40	5.36	0.00	NPC
2	D3 (Bottom)	43405.83	-6190.26	6545.87	0.95
3	D2 (Bottom)	1073930.00	-421.09	842.55	0.50
4	D3 (Bottom)	43914.33	-23713.56	32083.27	0.74
5	D3 (Top)	-412064.70	30.97	0.00	NPC
6	D3 (Max)	530048.10	-11197.42	11677.99	0.96
7	D3 (Bottom)	280269.30	-11516.79	17142.59	0.67

8	D3 (Bottom)	-221277.40	-1404.48	2763.38	0.51
9	D3 (Bottom)	364849.20	-2983.08	4156.05	0.72
10	D3 (Bottom)	29252.09	-5163.58	11518.46	0.45
11	D3 (Top)	-59114.34	-69.88	188.37	0.37
12	D3 (Top)	-12764.60	179.17	222.80	0.80
13	D3 (Max)	-26736.70	-1880.21	3267.43	0.58
14	D3 (Bottom)	-38880.69	-1068.34	12625.36	0.08

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	130306.10	2726400.00	0.05
2	D1 (Bottom)	324362.10	6598463.00	0.05
3	D2 (Max)	1082751.00	1701735.00	0.64
4	DC1 (Bottom)	547553.10	1.513124E07	0.04
5	D1 (Bottom)	43337.45	981311.80	0.04
6	D2 (Top)	803628.30	7584829.00	0.11
7	D2 (Max)	367225.40	9823142.00	0.04
8	D1 (Max)	84769.28	3578674.00	0.02
9	D2 (Top)	464638.00	4563456.00	0.10
10	DC1 (Bottom)	175963.00	8703281.00	0.02
11	D2 (Bottom)	58719.27	818812.80	0.07
12	DC1 (Max)	16506.39	887328.00	0.02
13	D2 (Top)	56260.11	4025153.00	0.01
14	D2 (Top)	135682.90	8113154.00	0.02

Axial tension

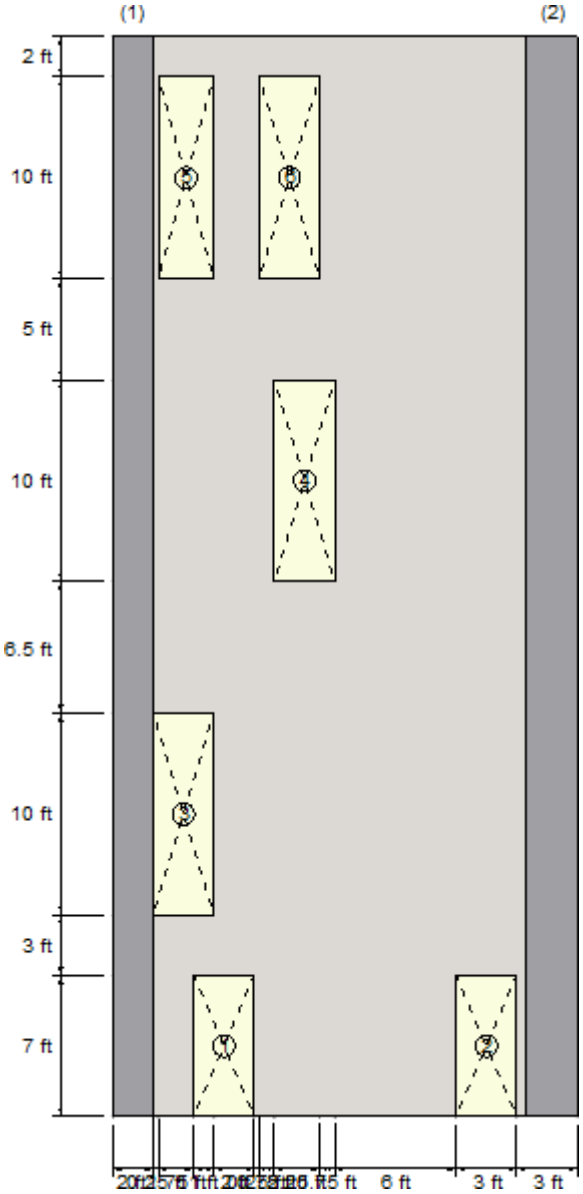
Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Max)	936498.90	822960.00	1.14
2	DC1 (Top)	0.00	1645920.00	0.00
3	DC1 (Top)	0.00	341280.00	0.00
4	DC1 (Top)	0.00	3700080.00	0.00
5	D3 (Max)	416815.70	411480.00	1.01
6	DC1 (Top)	0.00	1344600.00	0.00
7	DC1 (Top)	0.00	1966680.00	0.00
8	D3 (Top)	395683.50	990360.00	0.40
9	DC1 (Top)	0.00	390960.00	0.00
10	DC1 (Top)	0.00	1631880.00	0.00
11	D3 (Top)	59114.34	194400.00	0.30
12	D3 (Bottom)	35436.88	324000.00	0.11
13	D3 (Max)	26736.70	659880.00	0.04
14	D3 (Max)	46630.33	1501200.00	0.03

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Top)	258870.160	288000.000	0.90
2	D3 (Max)	746225.730	770358.730	0.97
3	D2 (Top)	164925.030	301163.190	0.55
4	D3 (Max)	1241294.600	1257350.000	0.99
5	D3 (Top)	31996.023	33480.000	0.96
6	D3 (Max)	978157.920	995442.610	0.98
7	D3 (Bottom)	1034596.500	1063362.300	0.97
8	D3 (Top)	361848.480	370769.880	0.98
9	D3 (Max)	450698.340	494155.060	0.91
10	D3 (Max)	808084.830	854937.870	0.95
11	D3 (Top)	10111.923	44350.119	0.23
12	D3 (Bottom)	42602.299	50379.289	0.85
13	D3 (Top)	373912.960	431757.800	0.87
14	D3 (Bottom)	419794.390	759709.480	0.55

COLUMN DESIGN:

Status : N. G.
- Mu > phi*Mn (Column 1)
- Stirrups: adopted spacing < calculated spacing (Column 7)



Geometry:

Column	Distance [ft]	Position Z	Width X [in]	Width Z [in]	Height [ft]
1	1.00	Centered	24.00	18.00	53.50
2	21.75	Centered	30.00	18.00	53.50

Reinforcement:

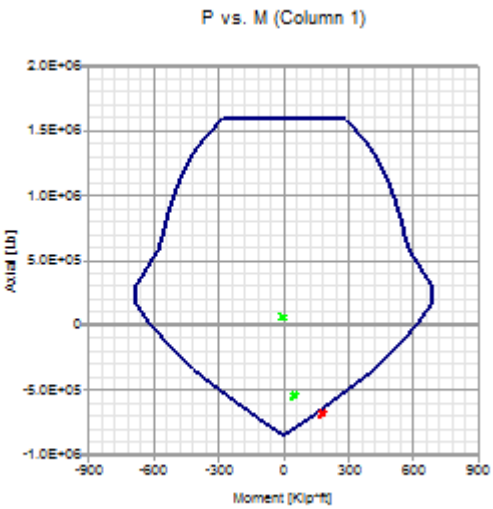
	Longitudinal reinforcement			Transverse reinforcement	
Column	Bars	As [in2]	Ld [in]	Bars	Spacing [in]
1	10-#11	15.60	54.61	#3	3.00
2	6-#8	4.74	38.73	#3	3.00
3	8-#11	12.48	54.61	#3	5.00
4	6-#8	4.74	38.73	#3	5.00
5	8-#11	12.48	54.61	#3	6.00
6	6-#8	4.74	38.73	#3	6.00
7	8-#11	12.48	54.61	#3	8.00

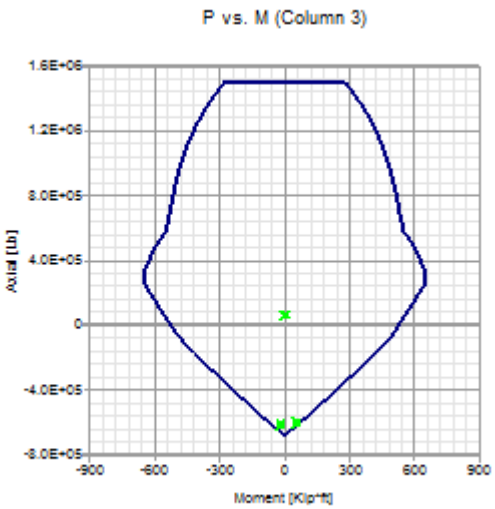
8	6-#8	4.74	38.73	#3	8.00

Combined biaxial flexure

Column	Condition	Pu [Lb]	Mc [Kip*ft]	ϕMn [Kip*ft]	Mc/(ϕ*Mn)
1	D3	680375.00	179.40	144.58	1.24
2	D2	-963945.30	294.32	650.55	0.45
3	D3	605225.40	51.27	63.86	0.80
4	D2	-826196.40	107.80	412.00	0.26
5	D3	616747.10	3.92	53.54	0.07
6	D2	-694643.00	81.36	418.95	0.19
7	D3	609739.10	28.05	59.82	0.47
8	D2	-801824.10	76.17	412.96	0.18

Interaction diagrams, P vs. M:





Axial tension

Column	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/(ϕ*Pn)
1	D3	680375.00	842400.00	0.81
2	SC1	0.00	255960.00	0.00
3	D3	617420.20	673920.00	0.92
4	SC1	0.00	255960.00	0.00
5	D3	616747.10	673920.00	0.92
6	SC1	0.00	255960.00	0.00
7	D3	609739.10	673920.00	0.90
8	SC1	0.00	255960.00	0.00

Axial compression

Column	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/(ϕ*Pn)
1	DC1	-66574.30	-1591013.00	0.04
2	D2	-963945.30	-1567398.00	0.61
3	DC1	-67487.23	-1501943.00	0.04
4	D2	-826196.40	-1567398.00	0.53
5	DC1	-63061.11	-1501943.00	0.04
6	D2	-694643.00	-1567398.00	0.44
7	DC1	-65578.05	-1501943.00	0.04
8	D2	-801824.10	-1567398.00	0.51

Shear along X direction

Column	Condition	Pos	Vu [Lb]	Vc [Lb]	Vs [Lb]	ϕ*Vn [Lb]	Vu/(ϕ*Vn)
1	D3	At bot...	61544.090	0.000	94248.000	70686.000	0.87
2	D2	At bot...	64807.420	145789.390	121550.000	200504.550	0.32
3	D3	At bot...	12936.392	0.000	56548.800	42411.600	0.31
4	D3	At top	7408.679	123304.340	72930.000	147175.760	0.05
5	D2	At bot...	12520.433	0.000	47124.000	35343.000	0.35
6	D3	At top	8820.298	125687.230	60775.000	139846.670	0.06
7	D3	At top	9093.273	0.000	35343.000	26507.250	0.34
8	D3	At top	9887.398	133265.540	45581.250	134135.090	0.07

Shear along Z direction

Column	Condition	Pos	Vu [Lb]	Vc [Lb]	Vs [Lb]	ϕ*Vn [Lb]	Vu/(ϕ*Vn)
1	SC1	At top	0.000	57332.547	169620.000	170214.410	0.00
2	SC1	At top	0.000	72618.438	103125.000	131807.580	0.00
3	SC1	At top	0.000	57332.547	40708.800	73531.010	0.00
4	SC1	At top	0.000	72618.438	61875.000	100870.080	0.00
5	SC1	At top	0.000	57332.547	33924.000	68442.410	0.00
6	SC1	At top	0.000	72618.438	51562.500	93135.703	0.00
7	SC1	At top	0.000	57332.547	25443.000	62081.660	0.00
8	SC1	At top	0.000	72618.438	38671.875	83467.735	0.00

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement

Schematic Shear Wall Design for Gridline C

Current Date: 8/4/2016 4:07 PM
Units system: English
File name: C:\ProgramData\Bentley\Engineering\RAM Elements.en\Data\SPOE GL C.cwd\

Design Results

Concrete Wall

GENERAL INFORMATION:

Global status : OK

Design code	:	ACI 318-05

Geometry:

Total height	:	53.50 [ft]
Total length	:	22.50 [ft]
Base support type	:	Continuous
Wall bottom restraint	:	Pinned
Column bottom restraint	:	Fixed
Rigidity elements	:	Columns

Materials:

Material	:	C 6-60
Steel tension strength (Fy)	:	60 [Kip/in2]
Concrete compressive strength (fc)	:	6 [Kip/in2]
Steel elasticity modulus (Es)	:	29000 [Kip/in2]
Concrete modulus of elasticity (E)	:	4415.2 [Kip/in2]
Concrete unit weight	:	0.149818 [Kip/ft3]

Number of stories: 4

Story	Story height [ft]	Wall thickness [in]
1	10.00	15.00
2	16.50	12.00
3	15.00	12.00
4	12.00	10.00

Openings:

Reference	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
Lower left	6.33	10.00	3.00	10.00
Lower left	2.75	26.50	3.00	10.00
Lower left	2.75	41.50	3.00	10.00

Columns:

Distance [ft]	Width X [in]	Width Z [in]	Position Z
1.25	30.00	15.00	Centered
21.25	30.00	15.00	Centered

Load conditions:

ID	Comb.	Category	Description
DL	No	DL	Dead Load
EQ	No	EQ	Seismic
SC1	Yes		DL
DC1	Yes		1.4DL
D1	Yes		1.4DL
D2	Yes		1.2DL+EQ
D3	Yes		0.9DL+EQ

Concentrated loads:

Story	Condition	Direction	Magnitude [Lb]	Eccentricity [in]	Distance [ft]
1	DL	Vertical	14400.00	0.00	0.00
1	DL	Vertical	14400.00	0.00	22.50
2	DL	Vertical	13700.00	0.00	0.00
2	DL	Vertical	13700.00	0.00	22.50
3	DL	Vertical	13100.00	0.00	0.00
3	DL	Vertical	13100.00	0.00	22.50
4	EQ	Horizontal	155742.00	0.00	0.00
3	EQ	Horizontal	285628.00	0.00	0.00
2	EQ	Horizontal	281225.00	0.00	0.00
1	EQ	Horizontal	162498.00	0.00	0.00

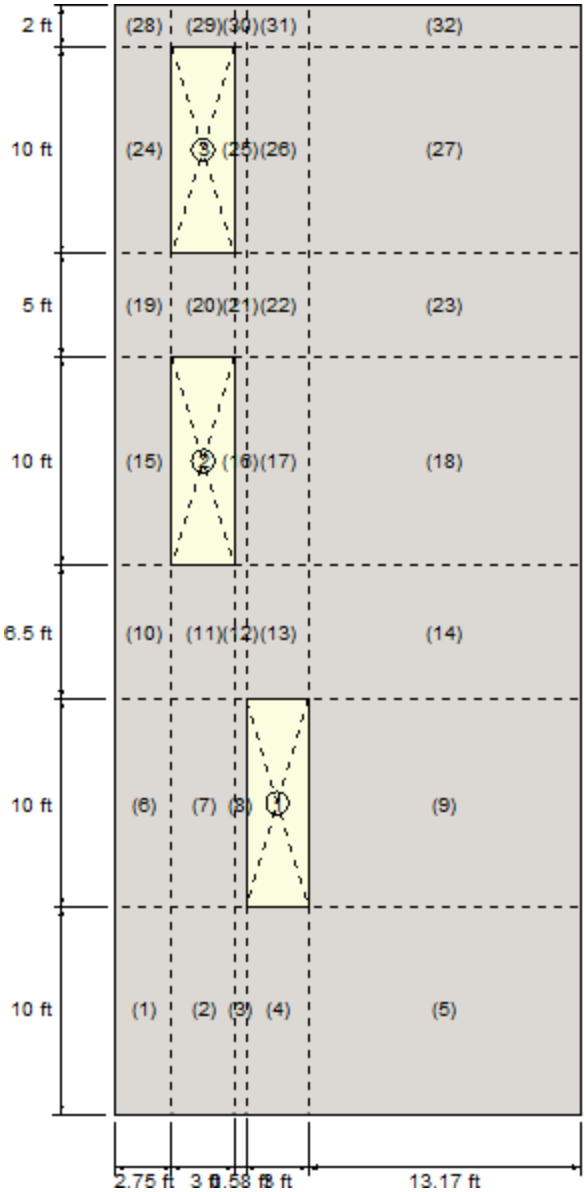
Distributed loads:

Consider self weight : DL

Story	Condition	Direction	Magnitude [Lb/ft]	Eccentricity [ft]
4	DL	Vertical	1640.00	0.00
3	DL	Vertical	1710.00	0.00
2	DL	Vertical	2700.00	0.00
1	DL	Vertical	2970.00	0.00

BEARING WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	2.75	10.00
2	2.75	0.00	3.00	10.00
3	5.75	0.00	0.58	10.00
4	6.33	0.00	3.00	10.00
5	9.33	0.00	13.17	10.00
6	0.00	10.00	2.75	10.00
7	2.75	10.00	3.00	10.00
8	5.75	10.00	0.58	10.00
9	9.33	10.00	13.17	10.00
10	0.00	20.00	2.75	6.50
11	2.75	20.00	3.00	6.50
12	5.75	20.00	0.58	6.50
13	6.33	20.00	3.00	6.50
14	9.33	20.00	13.17	6.50
15	0.00	26.50	2.75	10.00
16	5.75	26.50	0.58	10.00

17	6.33	26.50	3.00	10.00
18	9.33	26.50	13.17	10.00
19	0.00	36.50	2.75	5.00
20	2.75	36.50	3.00	5.00
21	5.75	36.50	0.58	5.00
22	6.33	36.50	3.00	5.00
23	9.33	36.50	13.17	5.00
24	0.00	41.50	2.75	10.00
25	5.75	41.50	0.58	10.00
26	6.33	41.50	3.00	10.00
27	9.33	41.50	13.17	10.00
28	0.00	51.50	2.75	2.00
29	2.75	51.50	3.00	2.00
30	5.75	51.50	0.58	2.00
31	6.33	51.50	3.00	2.00
32	9.33	51.50	13.17	2.00

Vertical reinforcement:

Reinforcement layers : 2

Segment	**Bars**	**Spacing** [in]	**Ld** [in]
1	4-#10	10.00	49.19
2	3-#9	16.00	43.69
3	1-#8	16.00	38.73
4	3-#8	16.00	38.73
5	10-#8	16.00	38.73
6	4-#8	10.00	38.73
7	4-#8	10.00	38.73
8	1-#9	18.00	43.69
9	12-#5	14.00	19.36
10	4-#8	10.00	38.73
11	4-#8	10.00	38.73
12	1-#9	18.00	43.69
13	3-#6	16.00	23.24
14	12-#5	14.00	19.36
15	3-#6	14.00	23.24
16	1-#9	18.00	43.69
17	4-#5	10.00	19.36
18	9-#5	18.00	19.36
19	3-#6	14.00	23.24
20	2-#5	18.00	19.36
21	1-#9	18.00	43.69
22	4-#5	10.00	19.36
23	9-#5	18.00	19.36
24	2-#5	18.00	19.36
25	1-#5	18.00	19.36
26	2-#5	18.00	19.36
27	9-#5	18.00	19.36
28	2-#5	18.00	19.36
29	2-#5	18.00	19.36
30	1-#5	18.00	19.36
31	2-#5	18.00	19.36
32	9-#5	18.00	19.36

Intermediate results for axial-bending

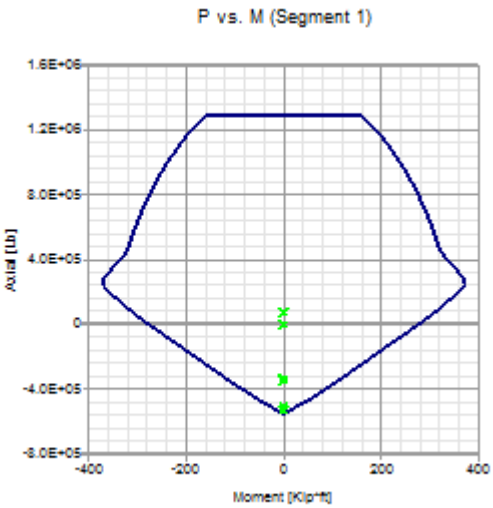
Segment	Condition	c [in]	d [in]
1	DC1 (Top)	3.26	12.99
2	DC1 (Top)	2.00	13.06
3	DC1 (Top)	2.44	13.13
4	DC1 (Top)	1.57	13.13
5	DC1 (Top)	1.61	13.13
6	DC1 (Top)	2.01	10.00
7	DC1 (Top)	1.93	10.00
8	DC1 (Top)	2.85	9.94
9	DC1 (Top)	1.09	10.31
10	DC1 (Top)	1.99	10.13
11	DC1 (Top)	1.72	10.13
12	DC1 (Top)	2.71	10.06
13	DC1 (Top)	1.26	10.25
14	DC1 (Top)	1.06	10.31
15	DC1 (Top)	1.26	10.25
16	DC1 (Top)	2.53	10.06
17	DC1 (Top)	1.14	10.31
18	DC1 (Top)	0.75	10.31
19	DC1 (Top)	1.25	10.25
20	DC1 (Top)	0.60	10.31
21	DC1 (Top)	2.41	10.06
22	DC1 (Top)	1.13	10.31
23	DC1 (Top)	0.73	10.31
24	DC1 (Top)	0.65	8.31
25	DC1 (Top)	1.21	8.31
26	DC1 (Top)	0.62	8.31
27	DC1 (Top)	0.61	8.31
28	DC1 (Top)	0.63	8.31
29	DC1 (Top)	0.60	8.31
30	DC1 (Top)	1.19	8.31
31	DC1 (Top)	0.60	8.31
32	DC1 (Top)	0.61	8.31

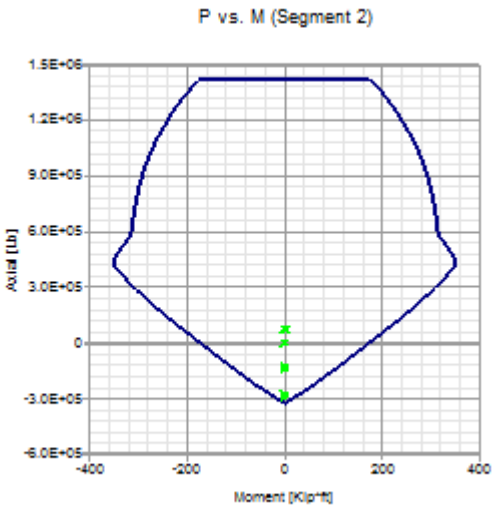
Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	DC1 (Top)	72322.90	0.00	308.45	0.00
2	DC1 (Top)	76457.18	0.00	209.93	0.00
3	DC1 (Top)	14859.32	0.00	51.60	0.00
4	DC1 (Top)	23929.53	0.00	151.79	0.00
5	DC1 (Top)	326526.90	0.00	644.68	0.00
6	DC1 (Top)	46349.71	0.00	152.82	0.00
7	DC1 (Top)	50719.01	0.00	155.99	0.00
8	DC1 (Top)	10129.61	0.00	44.41	0.00
9	DC1 (Top)	231670.20	0.00	290.72	0.00
10	DC1 (Top)	50432.94	0.00	155.80	0.00
11	DC1 (Top)	17350.82	0.00	145.03	0.00
12	DC1 (Top)	8888.88	0.00	44.53	0.00
13	DC1 (Top)	41306.17	0.00	81.61	0.00
14	DC1 (Top)	197382.30	0.00	277.05	0.00
15	DC1 (Top)	28575.35	0.00	75.73	0.00
16	DC1 (Top)	6614.26	0.00	43.79	0.00
17	DC1 (Top)	25898.13	0.00	72.19	0.00
18	DC1 (Top)	108734.60	0.00	194.55	0.00
19	DC1 (Top)	25265.89	0.00	74.45	0.00
20	DC1 (Top)	7954.37	0.00	35.97	0.00
21	DC1 (Top)	3806.96	0.00	42.80	0.00

22	DC1 (Top)	22017.58	0.00	70.67	0.00
23	DC1 (Top)	98055.88	0.00	189.85	0.00
24	DC1 (Top)	6490.07	0.00	29.04	0.00
25	DC1 (Top)	2556.30	0.00	12.92	0.00
26	DC1 (Top)	9460.40	0.00	30.30	0.00
27	DC1 (Top)	30865.26	0.00	131.77	0.00
28	DC1 (Top)	4432.46	0.00	28.27	0.00
29	DC1 (Top)	7759.58	0.00	29.66	0.00
30	DC1 (Top)	1357.60	0.00	12.56	0.00
31	DC1 (Top)	7472.68	0.00	29.55	0.00
32	DC1 (Top)	29611.33	0.00	131.30	0.00

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D1 (Bottom)	78019.47	1285796.00	0.06
2	DC1 (Top)	76457.18	1416168.00	0.05
3	D2 (Top)	45717.73	272678.60	0.17
4	D1 (Bottom)	66632.92	1419510.00	0.05
5	D2 (Bottom)	1374426.00	6244930.00	0.22
6	D1 (Top)	46349.71	1033431.00	0.04
7	DC1 (Bottom)	63030.18	1128903.00	0.06
8	D2 (Bottom)	56436.29	216191.00	0.26
9	D2 (Top)	823522.30	5009734.00	0.16
10	D1 (Top)	50432.94	1033431.00	0.05
11	D1 (Bottom)	50718.88	1128903.00	0.04
12	D1 (Bottom)	10129.02	216191.00	0.05
13	D1 (Top)	41306.17	1138663.00	0.04
14	D2 (Bottom)	823497.20	5009734.00	0.16
15	DC1 (Bottom)	30959.39	1043191.00	0.03
16	D2 (Top)	50175.51	216191.00	0.23
17	D2 (Top)	30769.50	1139087.00	0.03
18	D2 (Bottom)	521197.70	5014667.00	0.10
19	D2 (Top)	31152.39	1043191.00	0.03

20	D2 (Bottom)	30422.98	1142376.00	0.03
21	D2 (Bottom)	50172.69	216191.00	0.23
22	D2 (Bottom)	30769.38	1139087.00	0.03
23	D2 (Bottom)	213364.80	5014667.00	0.04
24	D2 (Bottom)	23655.84	871871.50	0.03
25	D2 (Top)	16954.16	182935.00	0.09
26	D1 (Bottom)	14010.21	951431.50	0.01
27	D2 (Bottom)	135624.20	4176423.00	0.03
28	D2 (Top)	45855.46	871871.50	0.05
29	D2 (Bottom)	19069.74	951431.50	0.02
30	D2 (Bottom)	16947.02	182935.00	0.09
31	D1 (Bottom)	9460.39	951431.50	0.01
32	D2 (Bottom)	31122.67	4176423.00	0.01

Axial tension

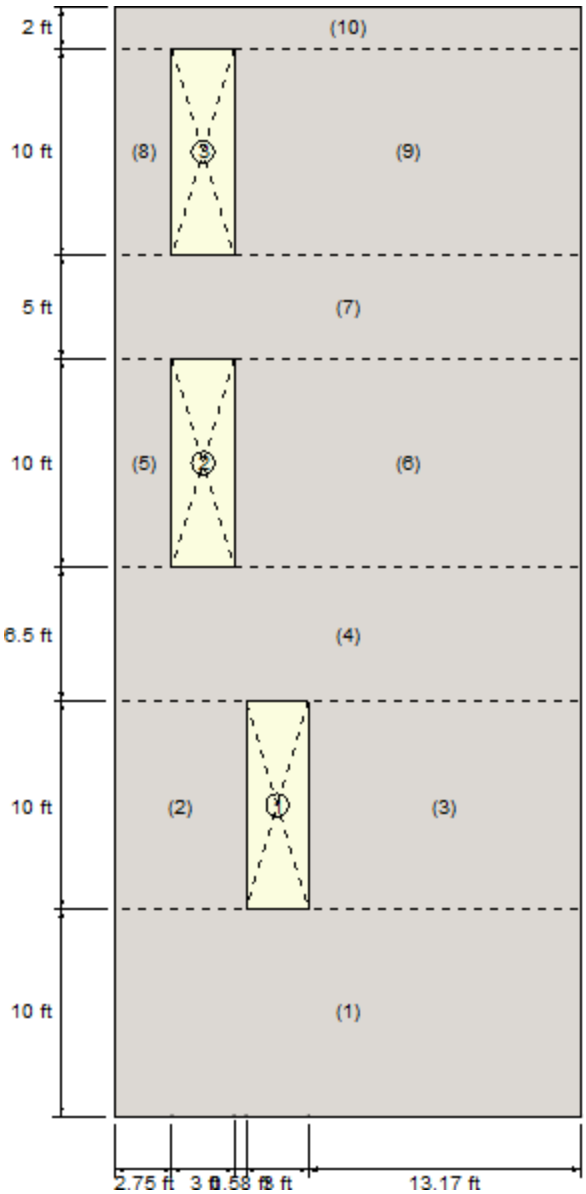
Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	D3 (Bottom)	525146.30	548640.00	0.96
2	D3 (Bottom)	291899.40	324000.00	0.90
3	D3 (Bottom)	35289.87	85320.00	0.41
4	D3 (Bottom)	111799.00	255960.00	0.44
5	DC1 (Top)	0.00	853200.00	0.00
6	D3 (Bottom)	334036.90	341280.00	0.98
7	D3 (Top)	270261.60	341280.00	0.79
8	D3 (Top)	95033.16	108000.00	0.88
9	DC1 (Top)	0.00	401760.00	0.00
10	D3 (Bottom)	176859.00	341280.00	0.52
11	D3 (Bottom)	270260.70	341280.00	0.79
12	D3 (Bottom)	95027.25	108000.00	0.88
13	D3 (Top)	140352.20	142560.00	0.98
14	DC1 (Top)	0.00	401760.00	0.00
15	D3 (Top)	121511.80	142560.00	0.85
16	D3 (Bottom)	93752.45	108000.00	0.87
17	D3 (Bottom)	133139.90	133920.00	0.99
18	DC1 (Top)	0.00	301320.00	0.00
19	D3 (Bottom)	121511.10	142560.00	0.85
20	DC1 (Top)	0.00	66960.00	0.00
21	D3 (Top)	19911.96	108000.00	0.18
22	D3 (Top)	26187.20	133920.00	0.20
23	DC1 (Top)	0.00	301320.00	0.00
24	D3 (Top)	40598.62	66960.00	0.61
25	D3 (Bottom)	31361.62	33480.00	0.94
26	D3 (Bottom)	24732.38	66960.00	0.37
27	DC1 (Top)	0.00	301320.00	0.00
28	D3 (Bottom)	43934.11	66960.00	0.66
29	D3 (Top)	863.96	66960.00	0.01
30	DC1 (Top)	0.00	33480.00	0.00
31	D3 (Bottom)	5138.35	66960.00	0.08
32	DC1 (Top)	0.00	301320.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	DC1 (Top)	0.000	49806.953	0.00
2	DC1 (Top)	0.000	54631.838	0.00
3	DC1 (Top)	0.000	10613.911	0.00
4	DC1 (Top)	0.000	54899.539	0.00
5	DC1 (Top)	0.000	241008.980	0.00
6	DC1 (Top)	0.000	38342.535	0.00
7	DC1 (Top)	0.000	41828.220	0.00
8	DC1 (Top)	0.000	8035.034	0.00
9	DC1 (Top)	0.000	189364.200	0.00
10	DC1 (Top)	0.000	38821.817	0.00
11	DC1 (Top)	0.000	42351.073	0.00
12	DC1 (Top)	0.000	8136.119	0.00
13	DC1 (Top)	0.000	42873.926	0.00
14	DC1 (Top)	0.000	189364.200	0.00
15	DC1 (Top)	0.000	39301.099	0.00
16	DC1 (Top)	0.000	8136.119	0.00
17	DC1 (Top)	0.000	43135.352	0.00
18	DC1 (Top)	0.000	189364.200	0.00
19	DC1 (Top)	0.000	39301.099	0.00
20	DC1 (Top)	0.000	43135.352	0.00
21	DC1 (Top)	0.000	8136.119	0.00
22	DC1 (Top)	0.000	43135.352	0.00
23	DC1 (Top)	0.000	189364.200	0.00
24	DC1 (Top)	0.000	31872.232	0.00
25	DC1 (Top)	0.000	6722.144	0.00
26	DC1 (Top)	0.000	34769.708	0.00
27	DC1 (Top)	0.000	152639.020	0.00
28	DC1 (Top)	0.000	31872.232	0.00
29	DC1 (Top)	0.000	34769.708	0.00
30	DC1 (Top)	0.000	6722.144	0.00
31	DC1 (Top)	0.000	34769.708	0.00
32	DC1 (Top)	0.000	152639.020	0.00

SHEAR WALL DESIGN:

Status : OK



Geometry:

Segment	X Coordinate [ft]	Y Coordinate [ft]	Width [ft]	Height [ft]
1	0.00	0.00	22.50	10.00
2	0.00	10.00	6.33	10.00
3	9.33	10.00	13.17	10.00
4	0.00	20.00	22.50	6.50
5	0.00	26.50	2.75	10.00
6	5.75	26.50	16.75	10.00
7	0.00	36.50	22.50	5.00
8	0.00	41.50	2.75	10.00
9	5.75	41.50	16.75	10.00
10	0.00	51.50	22.50	2.00

Reinforcement:

Reinforcement layers : 2

Segment	Vertical reinforcement Bars	Spacing [in]	Ld [in]	Horizontal reinforcement Bars	Spacing [in]	Ld [in]
1	4-#10	10.00	49.19	8-#5	16.00	25.17
	3-#9	16.00	49.19	8-#5	16.00	25.17
	1-#8	16.00	49.19	8-#5	16.00	25.17
	3-#8	16.00	49.19	8-#5	16.00	25.17
	10-#8	16.00	49.19	8-#5	16.00	25.17
2	4-#8	10.00	43.69	8-#6	16.00	30.21
	4-#8	10.00	43.69	8-#6	16.00	30.21
	1-#9	18.00	43.69	8-#6	16.00	30.21
3	12-#5	14.00	19.36	7-#5	18.00	25.17
4	4-#8	10.00	43.69	5-#5	18.00	25.17
	4-#8	10.00	43.69	5-#5	18.00	25.17
	1-#9	18.00	43.69	5-#5	18.00	25.17
	3-#6	16.00	43.69	5-#5	18.00	25.17
	12-#5	14.00	43.69	5-#5	18.00	25.17
5	3-#6	14.00	23.24	7-#5	18.00	25.17
6	1-#9	18.00	43.69	7-#5	18.00	25.17
	4-#5	10.00	43.69	7-#5	18.00	25.17
	9-#5	18.00	43.69	7-#5	18.00	25.17
7	3-#6	14.00	43.69	4-#5	18.00	25.17
	2-#5	18.00	43.69	4-#5	18.00	25.17
	1-#9	18.00	43.69	4-#5	18.00	25.17
	4-#5	10.00	43.69	4-#5	18.00	25.17
	9-#5	18.00	43.69	4-#5	18.00	25.17
8	2-#5	18.00	19.36	7-#5	18.00	25.17
9	1-#5	18.00	19.36	7-#5	18.00	25.17
	2-#5	18.00	19.36	7-#5	18.00	25.17
	9-#5	18.00	19.36	7-#5	18.00	25.17
10	2-#5	18.00	19.36	2-#5	18.00	25.17
	2-#5	18.00	19.36	2-#5	18.00	25.17
	1-#5	18.00	19.36	2-#5	18.00	25.17
	2-#5	18.00	19.36	2-#5	18.00	25.17
	9-#5	18.00	19.36	2-#5	18.00	25.17

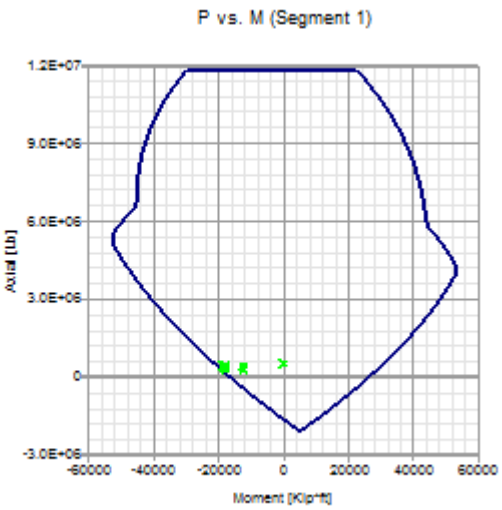
Intermediate results for axial-bending

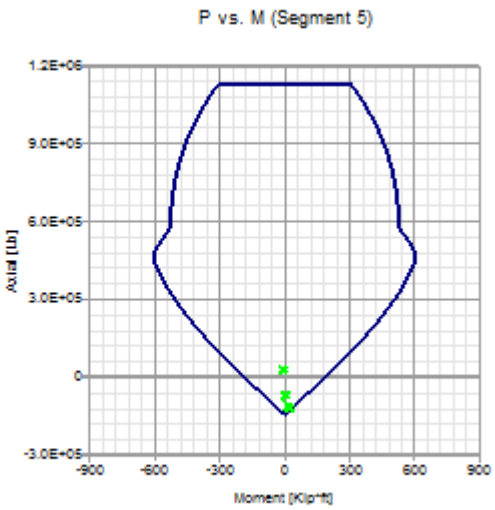
Segment	Condition	c [in]	d [in]
1	D3 (Bottom)	29.07	216.00
2	D3 (Bottom)	4.65	60.77
3	D3 (Bottom)	24.27	126.43
4	D3 (Bottom)	24.84	216.00
5	D3 (Top)	0.37	26.40
6	D3 (Bottom)	15.70	160.80
7	D3 (Bottom)	19.70	216.00
8	D3 (Top)	0.52	26.40
9	D3 (Bottom)	11.82	160.80
10	D3 (Bottom)	15.01	216.00

Combined axial flexure

Segment	Condition	Pu [Lb]	Mu [Kip*ft]	ϕ*Mn [Kip*ft]	Mu/ϕ*Mn
1	D3 (Bottom)	339041.80	-18436.09	19461.58	0.95
2	D3 (Bottom)	-426857.20	-705.27	1154.06	0.61
3	D3 (Bottom)	686676.70	-4385.82	6291.00	0.70
4	D3 (Bottom)	197225.20	-8558.96	10881.03	0.79
5	D3 (Top)	-121633.50	24.47	27.91	0.88
6	D3 (Bottom)	265412.90	-3357.70	5168.06	0.65
7	D3 (Bottom)	170354.10	-2108.06	8225.36	0.26
8	D3 (Top)	-40623.36	-14.68	35.06	0.42
9	D3 (Bottom)	68198.10	-803.93	3599.71	0.22
10	D3 (Bottom)	11781.59	-419.66	5767.70	0.07

Interaction diagrams, P vs. M:





Axial compression

Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Bottom)	563115.50	1.183342E07	0.05
2	D1 (Bottom)	125397.20	2835294.00	0.04
3	D2 (Top)	823723.10	5009734.00	0.16
4	D1 (Bottom)	346000.90	9298187.00	0.04
5	DC1 (Max)	31389.14	1125559.00	0.03
6	D2 (Bottom)	302289.20	6369945.00	0.05
7	D2 (Bottom)	207474.90	8555511.00	0.02
8	D2 (Bottom)	23713.79	871871.50	0.03
9	D2 (Bottom)	83119.73	5310789.00	0.02
10	D2 (Top)	77720.55	7134092.00	0.01

Axial tension

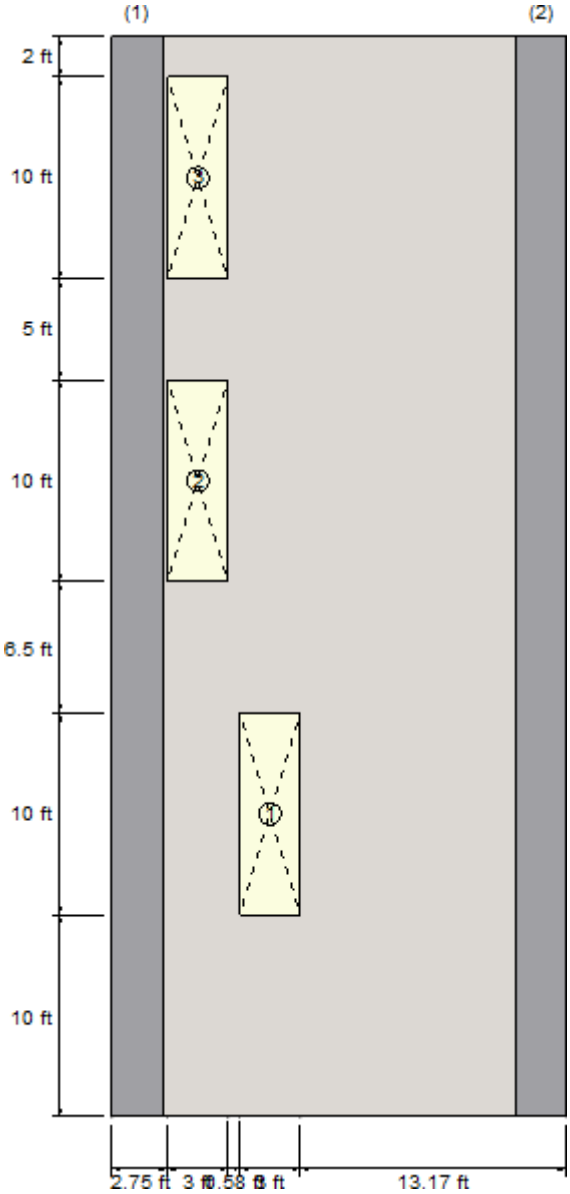
Segment	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/ϕ*Pn
1	DC1 (Top)	0.00	2067120.00	0.00
2	D3 (Top)	547943.20	790560.00	0.69
3	DC1 (Top)	0.00	401760.00	0.00
4	DC1 (Top)	0.00	1334880.00	0.00
5	D3 (Top)	121633.50	142560.00	0.85
6	DC1 (Top)	0.00	543240.00	0.00
7	DC1 (Top)	0.00	752760.00	0.00
8	D3 (Top)	40623.36	66960.00	0.61
9	DC1 (Top)	0.00	401760.00	0.00
10	DC1 (Top)	0.00	535680.00	0.00

Shear

Segment	Condition	Vu [Lb]	ϕ*Vn [Lb]	Vu/ϕ*Vn
1	D3 (Max)	895880.770	1047646.800	0.86
2	D3 (Bottom)	183283.760	191868.440	0.96
3	D3 (Max)	544926.060	584385.500	0.93
4	D3 (Max)	760473.730	865399.520	0.88
5	D3 (Top)	14151.324	83409.546	0.17
6	D3 (Max)	439574.060	654905.820	0.67
7	D3 (Max)	454120.590	854680.510	0.53
8	D2 (Max)	8861.921	92289.943	0.10
9	D3 (Top)	163547.650	563706.860	0.29
10	D3 (Bottom)	163689.930	750666.620	0.22

COLUMN DESIGN:

Status : OK



Geometry:

Column	Distance [ft]	Position Z	Width X [in]	Width Z [in]	Height [ft]
1	1.25	Centered	30.00	15.00	53.50
2	21.25	Centered	30.00	15.00	53.50

Reinforcement:

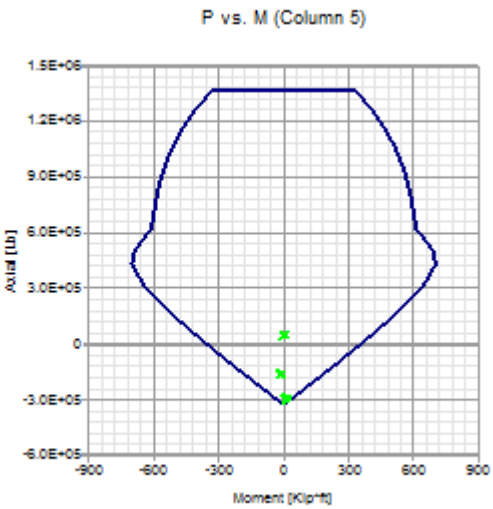
	Longitudinal reinforcement			Transverse reinforcement	
Column	Bars	As [in2]	Ld [in]	Bars	Spacing [in]
1	12-#11	18.72	54.61	#3	6.00
2	6-#8	4.74	38.73	#3	6.00
3	6-#11	9.36	54.61	#3	6.00
4	6-#8	4.74	38.73	#3	6.00
5	6-#9	6.00	43.69	#3	6.00
6	6-#8	4.74	38.73	#3	6.00
7	6-#10	7.62	49.19	#3	6.00

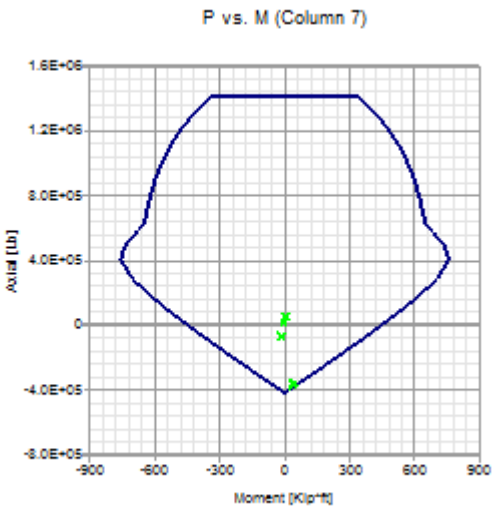
8	6-#8	4.74	38.73	#3	6.00

Combined biaxial flexure

Column	Condition	Pu [Lb]	Mc [Kip*ft]	ϕMn [Kip*ft]	Mc/(ϕ*Mn)
1	D3	450756.70	123.01	576.18	0.21
2	D2	-579805.10	129.95	606.75	0.21
3	D3	389548.50	27.26	130.29	0.21
4	D2	-475790.30	57.07	316.74	0.18
5	D3	299835.80	13.23	29.81	0.44
6	D2	-398411.40	42.89	306.72	0.14
7	D3	365782.90	39.12	53.86	0.73
8	D2	-461616.70	40.39	317.54	0.13

Interaction diagrams, P vs. M:





Axial tension

Column	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/(ϕ*Pn)
1	D3	450756.70	1010880.00	0.45
2	SC1	0.00	255960.00	0.00
3	D3	389548.50	505440.00	0.77
4	SC1	0.00	255960.00	0.00
5	D3	299835.80	324000.00	0.93
6	SC1	0.00	255960.00	0.00
7	D3	365782.90	411480.00	0.89
8	SC1	0.00	255960.00	0.00

Axial compression

Column	Condition	Pu [Lb]	ϕ*Pn [Lb]	Pu/(ϕ*Pn)
1	DC1	-68935.27	-1727819.00	0.04
2	D2	-579805.10	-1328718.00	0.44
3	DC1	-58392.10	-1460609.00	0.04
4	D2	-475790.30	-1328718.00	0.36
5	DC1	-52119.34	-1364688.00	0.04
6	D2	-398411.40	-1328718.00	0.30
7	DC1	-55348.73	-1410936.00	0.04
8	D2	-461616.70	-1328718.00	0.35

Shear along X direction

Column	Condition	Pos	Vu [Lb]	Vc [Lb]	Vs [Lb]	ϕ*Vn [Lb]	Vu/(ϕ*Vn)
1	D3	At bot...	41639.642	0.000	60324.000	45243.000	0.92
2	D2	At bot...	45847.216	105550.710	60775.000	124744.280	0.37
3	D2	At top	12441.704	0.000	60324.000	45243.000	0.27
4	D3	At top	6733.372	88652.359	60775.000	112070.520	0.06
5	D2	At top	15708.790	0.000	60634.200	45475.650	0.35
6	D3	At top	6913.367	91837.866	60775.000	114459.650	0.06
7	D2	At top	4573.118	0.000	60478.000	45358.500	0.10
8	D3	At top	6266.016	96293.506	60775.000	117801.380	0.05

Shear along Z direction

Column	Condition	Pos	Vu [Lb]	Vc [Lb]	Vs [Lb]	ϕ*Vn [Lb]	Vu/(ϕ*Vn)
1	SC1	At top	0.000	57722.944	40986.000	74031.708	0.00
2	SC1	At top	0.000	58675.698	41662.500	75253.648	0.00
3	SC1	At top	0.000	57722.944	40986.000	74031.708	0.00
4	SC1	At top	0.000	58675.698	41662.500	75253.648	0.00
5	SC1	At top	0.000	58378.253	41451.300	74872.164	0.00
6	SC1	At top	0.000	58675.698	41662.500	75253.648	0.00
7	SC1	At top	0.000	58048.274	41217.000	74448.956	0.00
8	SC1	At top	0.000	58675.698	41662.500	75253.648	0.00

Notes:

* Pu = Axial load
* Pn = Nominal axial load
* Mu = Section moment
* Mn = Maximum nominal moment
* Vu = Design shear force
* Vn = Nominal shear force
* ld = Embedment length
* As = Effective cross sectional area of reinforcement