Owner-Directed Facility Seismic Evaluation

Ashland City Hall
20 E. Main Street, Ashland, Oregon

Prepared for
The City of Ashland

STRUCTURAL
REGISTERED PROFESSIONAL
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51043
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TABLE OF CONTENTS

ASHLAND CITY HALL EXECUTIVE SUMMARY

Overview

The City of Ashland has hired Miller Consulting Engineers (MCE) to update our previous seismic evaluation report for the Ashland City Hall building dated August 5, 1994, considering the current state building code, which is the *2014 Oregon Structural Specialty Code* (*OSSC*).

The purpose and goal of this facility seismic evaluation is to provide a schematic design for the upgrade of the building that meets the current code, as well as an engineer's preliminary estimate of probable construction costs to upgrade the building based on our review. **This evaluation develops a plan to upgrade the facility to allow the occupants to safely exit the structure after a major earthquake, which may not be the case in its current state.**

The evaluation identified the following items that need to be addressed as part of the proposed seismic upgrade.

- Strengthening the roof and floor diaphragms to transfer lateral loads to the walls
- The connections between the walls and the diaphragm for in-plane and out-of-plane loads
- Addressing the connections at the roof diaphragm to account for the plan irregularities
- Providing shear walls in the east-west direction to support the diaphragm loading
- Providing bonded shear walls to reinforce the unreinforced masonry walls for in-plane loads
- Providing special concentric braced frames on the north and west walls on the main level to resist the required lateral loading of the building

Based on the above items, we anticipate that the proposed upgrade work will take approximately seven months to complete and will require at a minimum the second floor to be vacated during construction. The following is our preliminary estimate of probable construction cost to upgrade the building:

Performance Objective	Construction Cost	Contingency Cost	Total Cost	Cost per sq. foot
OSSC Code Upgrade	$1,136,464	$227,293	$1,363,757	$176

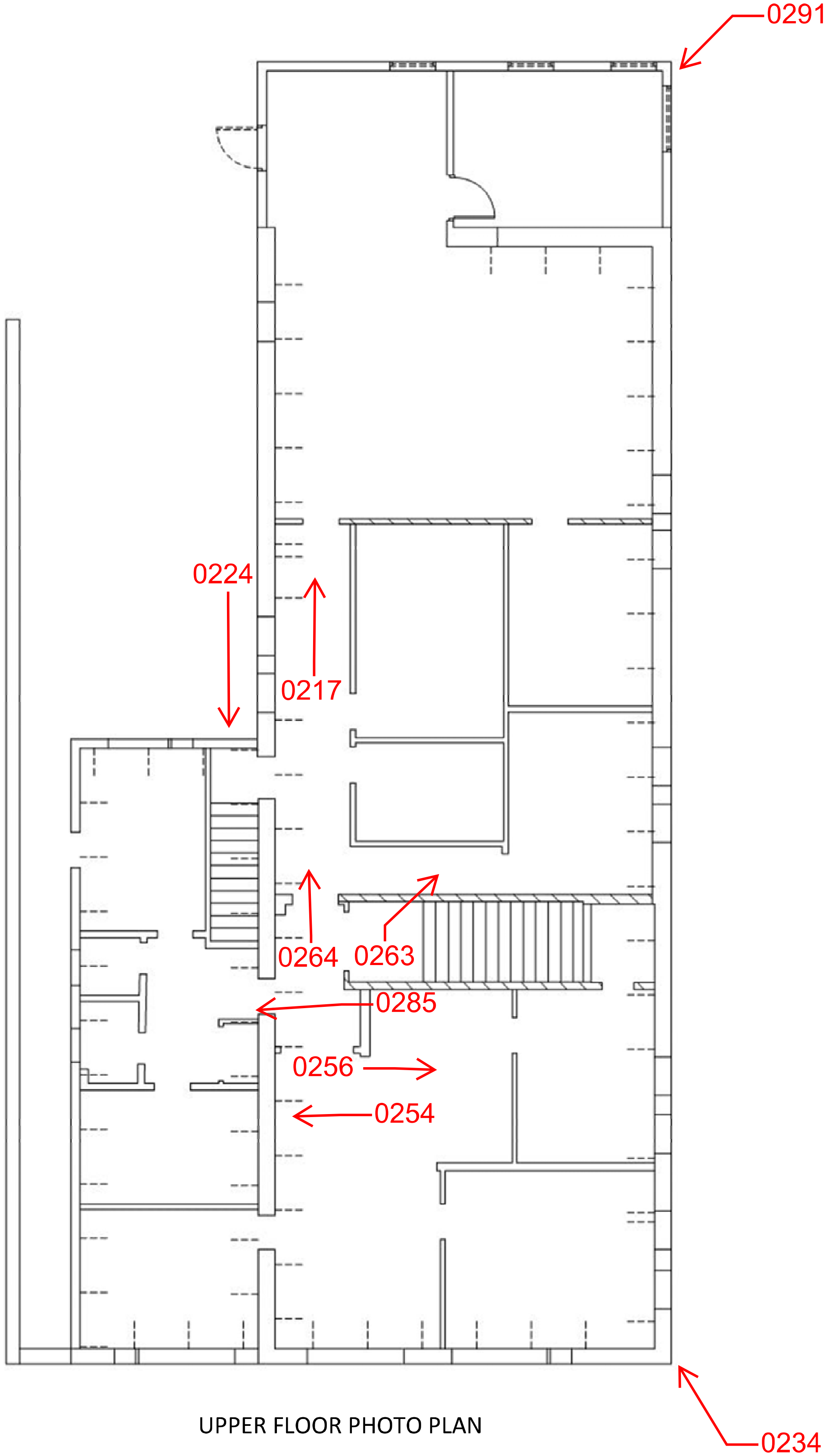
The above costs only consider the structural upgrades and corresponding work as required by the structural upgrades; the other concerns with the building such HVAC, plumbing, ADA access or other non-structural costs such as bracing the suspended ceilings are not included in this cost.

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 to account for some differences in the field during construction. The structural cost is less than constructing a new facility, but when you add in the other costs, the upgrade cost may be more than the replacement cost.

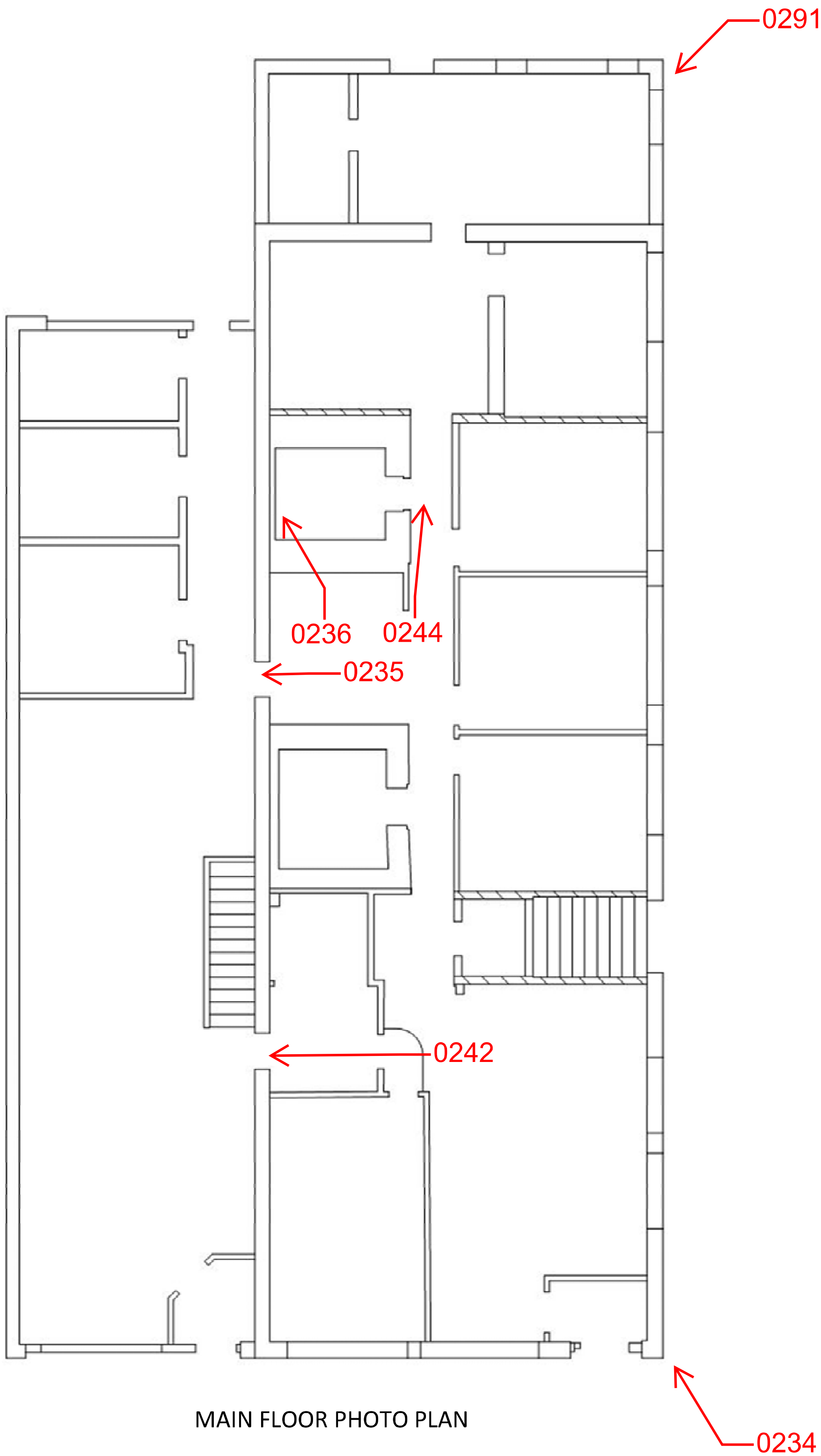
Background

The Ashland City Hall building is located at 20 East Main Street in Ashland, Oregon. The building was originally constructed around 1891 with two additions added since that time. One addition was added around 1913 that expanded the building to the north on both stories and to the south on the first story only. Another addition was added in 1995 that extended the second floor on the south to be in line with the south wall of the 1913 addition. The current size of the building is approximately 7,745 square feet. Original construction documents were not available for our review, but documents from 1982 (Savikko), 1991 (Marquess and Associates) and 1996 (Afseth) were used in our evaluation.

The building is comprised of two primary areas. One area is the main building area that is a two-story structure that has unreinforced masonry exterior walls on the east and west sides of the area, as well as an unreinforced masonry wall toward the south side of the building that was originally an exterior wall of the 1891 building. The other exterior walls of the main building area are cast-in-place concrete walls that were placed during the 1913 addition. The other area is the infill area that is between the main building area and the 1880 Pioneer Building to the east. The infill area is supported by the east wall of the main building area on the west and on the east by post-and-beam supports that are adjacent to the Pioneer Building. The infill area is approximately 18' wide by 75' long on the main floor. The upper floor is 14' wide by 45' long that has cast-in-place concrete exterior walls that are supported by the wood-framed diaphragm on the main level. The infill area does not have an apparent lateral force-resisting element on the main floor east side. The building houses several city offices including the mayor's office and support staff, as well as the water department.



UPPER FLOOR PHOTO PLAN



MAIN FLOOR PHOTO PLAN

PHOTO 0234 – North elevation of the building looking toward the northwest corner

PHOTO 0291 – West elevation of the building looking toward the southwest corner

PHOTO 0224 – Exposed east upper wall of the main building area looking toward the northeast

PHOTO 0217 – Main building area roof looking toward the south

PHOTO 0256 – Main building area roof framing looking west

PHOTO 0264 – Main building area roof framing looking south

PHOTO 0254 – Joint at the east wall between the 1891 building to the south and the 1913 addition

PHOTO 0261 – HVAC access in the upper floor ceiling cavity through the east wall

PHOTO 0263 – Upper floor ceiling cavity looking toward the southwest

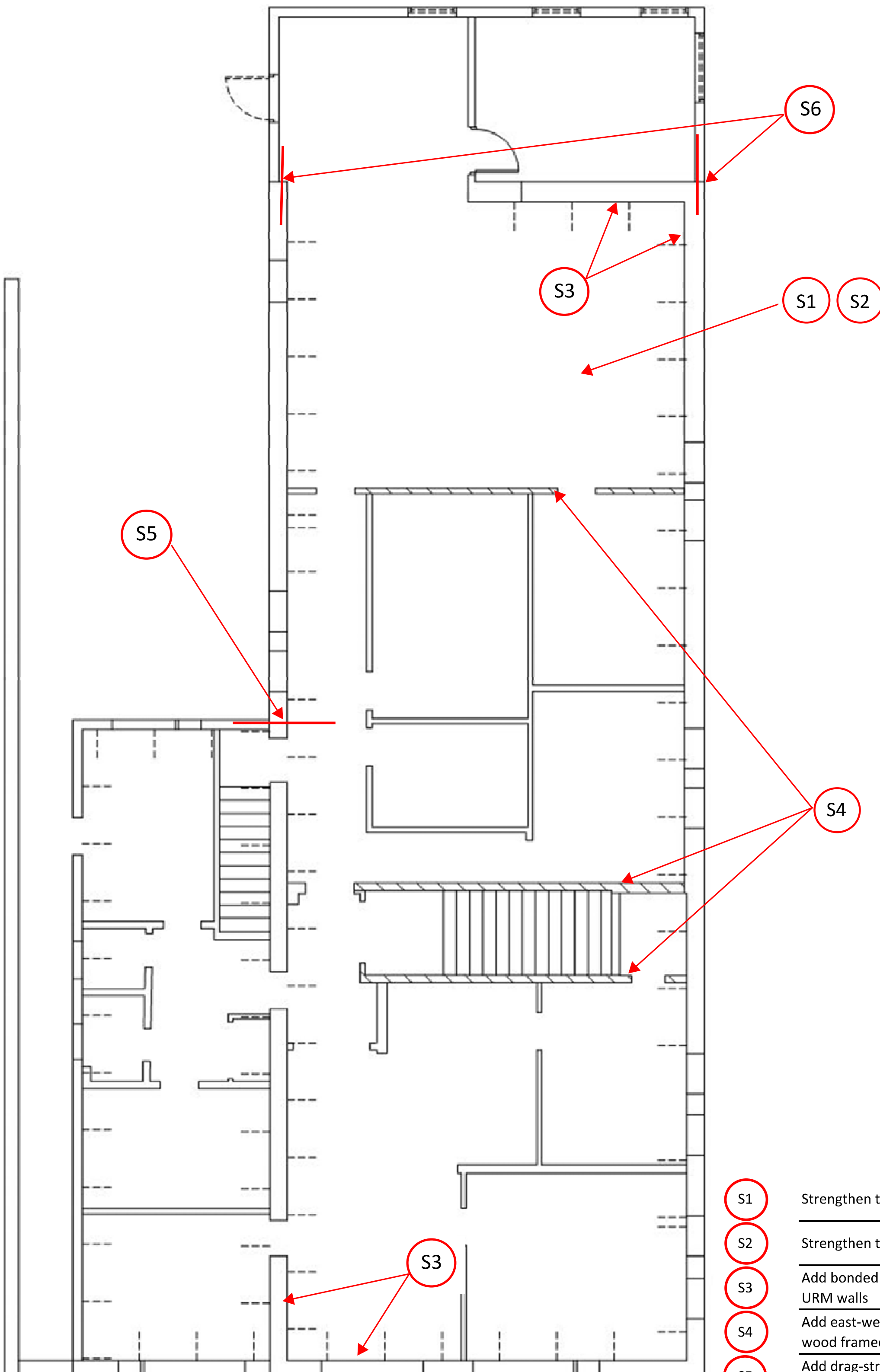
PHOTO 0285 – Exposed unreinforced masonry wall in the mayor's office vestibule

PHOTO 0236 – Upper floor framing visible in the main floor ceiling cavity adjacent to a vault

PHOTO 0242 – Upper floor connection at the east URM wall of the main building area

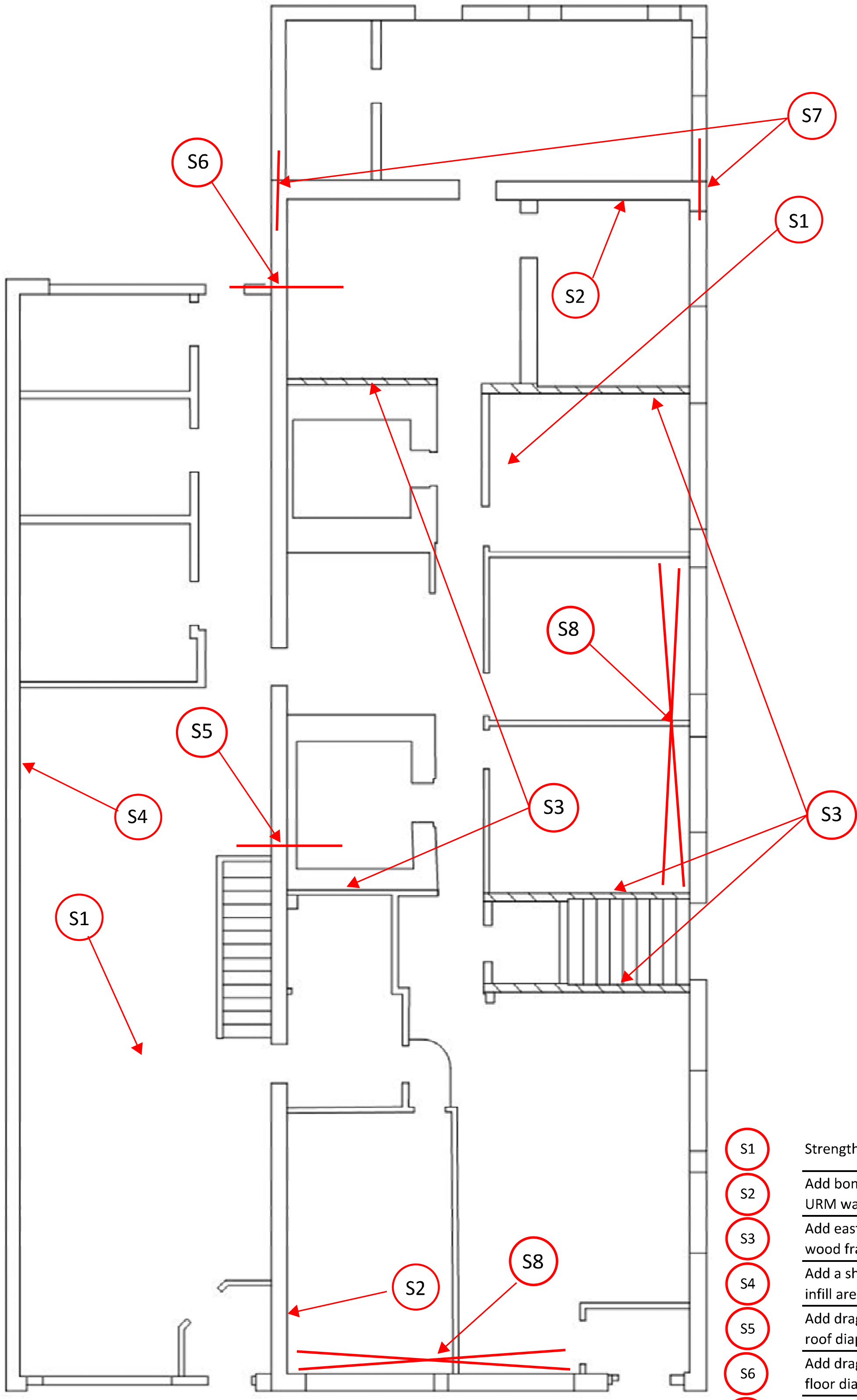
PHOTO 0244 – Main floor ceiling cavity in the main building area

PHOTO 0235 – Access opening through the east URM wall in the main building area



S1	Strengthen the roof diaphragm
S2	Strengthen the upper floor ceiling diaphragm
S3	Add bonded shear walls to strengthen the URM walls
S4	Add east-west shear walls to support the wood framed diaphragm
S5	Add drag-struts at the end of the infill roof area as required
S6	Add drag-struts to connect the 1995 addition to the main building area

SECOND STRUCTURAL PLAN



MAIN STRUCTURAL PLAN

S1	Strengthen the upper floor diaphragm
S2	Add bonded shear walls to strengthen the URM walls
S3	Add east-west shear walls to support the wood framed diaphragm
S4	Add a shear wall along the east edge of the infill area as required
S5	Add drag-struts at the end of the infill area roof diaphragm as required
S6	Add drag-struts at the end of the infill area floor diaphragm as required
S7	Add drag-struts to connect the 1995 addition to the main building area
S8	Add Special Concentric Braced Frame (SCBF) to the main building area

Background

The Ashland City Hall building is comprised of two primary building areas described below. The building was originally constructed around 1891 as an unreinforced masonry building that has had two major additions added since that time, along with several interior remodels over the life of the building. One addition was added around 1913, which expanded the building to the north on both stories and to the south on the first story only. Another addition was added in 1995 that extended the second floor to the south so to be in line with the south wall of the 1913 main floor addition. The building houses several city offices including the mayor's office and support staff, as well as the water department.

One area is the main building area that is a two-story structure that has unreinforced masonry walls on the east and west sides of the area, as well as an unreinforced masonry wall from the original building toward the south side of the building. The other exterior walls of the main building area are cast-in-place concrete walls that were placed during the 1913 addition.

The other area of the building is the infill area that is between the main building area and the 1880 Pioneer Building to the East. The infill area is supported by the main building area on the west and on the east by post-and-beam supports that are adjacent to the 1880 Pioneer Building. The infill area is approximately 18' wide by 75' long on the main floor that does not have an apparent lateral force-resisting element on the east side. The upper floor is 14' wide by 45' long that has cast-in-place concrete exterior walls that are supported by the wood-framed diaphragm of the main level.

Main Building Area

The main building section is a two-story area that is constructed of three different wall materials including unreinforced masonry walls, cast-in-place concrete walls and light wood-framed walls along the exterior of the 1995 addition. The roof is built with straight sheathing that spans in the north and south direction and is perpendicular to the roof framing. In addition, there are several interior partition walls on the main floor that may be seismic hazards, including the unreinforced masonry wall at the south office area and the cell block area.

The main building area can be evaluated in three different conditions. One condition is as the building exists today, where it appears that none of the previous recommendations have been considered. Another condition would be if the building had been seismically upgraded in 1994 when the report was originally generated; the last condition is if the work that is being proposed today is considered. Currently, the existing main building area roof diaphragm has a demand-to-capacity ratio (DCR) of 29 and the main floor diaphragm has a DCR of 19. The unreinforced masonry walls on the upper floor have a DCR of up to 5.4 and on the main floor the unreinforced masonry walls have a DCR of 4.8 in the north-south direction and a DCR of 21 in the east-west direction. Typically, when reporting DCR numbers that are greater than 10, the number is capped at 10 to demonstrate that the element has no capacity, but, in this case, we have not done that in order to demonstrate the deficiencies of the building.

If the upgrade work was completed following the 1994 report, there would be some concerns today based on the changes to the OSSC, which in this case the seismic base loading has increased by 52% (the seismic coefficient increased from 0.17 g to 0.26 g). There have also been some changes in the way designers address seismic loads and how materials are used in resisting the seismic loading. In addition, the roof diaphragm would also be a concern based on the fact that straight sheathing is not recommended for use in masonry buildings based on the lack of stiffness of the diaphragm. The unreinforced masonry walls on the upper floor would have a DCR of up to 2.5 and on the main floor the unreinforced masonry walls would have a DCR of 4.8 in the north-south direction. The plywood shear walls on the upper floor would have a DCR of up to 1.2 and on the main floor the plywood shear walls would have a DCR of 2.5 in the east-west direction. The fact that the 1994 renovation is deficient is a result of.

As part of the third condition, there are a number of structural concerns that will need to be addressed including the following: the lack of parapet bracing for the parapet walls that are taller than 12" above the roof diaphragm; the need to strengthen the roof, upper floor ceiling and floor diaphragms for the imposed lateral loads; the need to provide lateral force resisting elements to support the in-plane unreinforced masonry wall loading; the lack of anchorage between the diaphragms and the unreinforced masonry walls; and the lack of drag struts at the end of the infill area on the upper and main floors. It is assumed that the exterior unreinforced masonry walls have adequate strength to resist the out-of-plane loading, but this assumption will need to be verified in the field prior to commencing the seismic strengthening of the building.

Main Building Area Recommendations

For the lack of parapet bracing, use custom brackets to connect the parapet walls back to the roof diaphragm. For the inadequate diaphragms, add plywood sheathing at the diaphragm level, as well as additional in-plane shear walls on the upper and main floors to reduce the loading on the diaphragm as well as the exterior shear walls lines. In addition, bonded shear walls may be used in the attic area to transfer loads between the roof diaphragm and the ceiling diaphragm of the upper level. 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. For the lack of drag strut connections between the infill area and the main building area, use custom brackets to connect the areas together.

Infill Area

It appears that the infill area existed before the 1913 addition was built, but it was most likely significantly remodeled during the 1913 addition. Consequently, we considered the infill portion of the building to be part of the 1913 addition. The infill area is supported on the west by the east unreinforced masonry wall of the main building area and on the east by post-and-beam supports that are adjacent to the Pioneer Building.

The infill area is approximately 18’ wide by 75’ long on the main floor, which does not have an apparent lateral force resisting element on the east side adjacent to the Pioneer Building. The upper floor is 14’ wide by 45’ long that has cast-in-place concrete exterior walls that are supported by the wood-framed diaphragm of the main level. The concrete walls that are supported by the wood-framed floor diaphragm are of some concern, because the infill area does not have an apparent lateral force resisting element on the main floor east side. There is also some concern with the north wall of the infill area and how the diaphragms are connected to the exterior walls. Lastly, there is not an isolation joist between the north wall or the floor diaphragm of the infill area and the adjacent Pioneer Building. During a seismic event pounding of these two buildings may occur that could cause additional damage to the City Hall building, as well as to the Pioneer Building. In order to address pounding, the buildings can either be separated by a seismic isolation joint or tied together so that the two buildings work together.

Infill Area Recommendations

Most of the repairs at the infill area are similar to the repairs as required at the main building area, including the addition of connections between the walls and the diaphragm for in-plane and out-of-plane loads. For the potential pounding concern, a seismic isolation joist should be added between the infill area and the adjacent Pioneer Building. Due to the existing concrete vaults, the bonded shear walls that are being added to strengthen the east unreinforced masonry wall of the main building area will need to be built on the east side of the unreinforced masonry wall in the infill area.

Limitations

The information contained in this report is for the exclusive use of the City of Ashland. 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; and 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.

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.

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.

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.

APPENDIX – A

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

PROJECT:	Owner Directed Facility Seismic Evaluations	JOB NO.:	150799
LOCATION:	Ashland City Hall		
CLIENT:	City of Ashland	DATE:	7-Dec-15
TAKE-OFF BY:	ERW	CHECKED BY:	RGV

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Mobilization	Job Trailer	7	Month	$500	$3,500	
	Supervisor	30	Week	$4,125	$123,750	
Construction Aids						
Lift 500# capacity	Provide scissor lifts	7	Month	$4,300	$30,100	
Temporary Storage	Provide (2) 40' container for storage as required	7	Month	$1,000	$7,000	
Ventalation	Negative Air Equipment per (4) zones	30	Week	$2,000	$60,000	
						$224,350
Strengthen Roof Diaphragm	Remove (E) TPO roofing	2880	S.F.	$2	$5,760	
	Add Plywood	2520	S.F.	$4	$10,080	
	Add parapet bracing @ 4' o.c. for walls > 16" tall	120	L.F.	$400	$48,000	
	Add Blocking between rafters@ anchors	228	L.F.	$25	$5,700	
	Build bonded shear walls in attic for IP and OOP (using 3.5' tall attic walls)	798	S.F.	$15	$11,970	
New Roofing	New TPO Roofing with rigid insulation	2880	S.F.	$20	$57,600	
Strengthen Upper Floor Ceiling Diaphragm	Remove all contents on upper floor	2880	S.F.	$15	$43,200	
	Temporary work area	2880	S.F.	$2	$4,320	
	Add Plywood	2520	S.F.	$8	$20,160	
	Out of plane ties @ 4' o.c. (excluding bonded walls)	24	Ea.	$400	$9,527	
	Add Blocking between rafters@ anchors	228	L.F.	$25	$5,700	
	Add perimeter angle and anchors in masonry	228	L.F.	$50	$11,400	

ESTIMATE OF PROBABLE CONSTRUCTION COST

STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Drag-strut and continuity ties	Custom drag-strut at Gridline 3	1	Ea.	$2,000	$2,000	
	Install strap at Gridline 3	1	Ea.	$1,000	$1,000	
	Install Simpson HD at 7-A and 7-B	2	Ea.	$500	$1,000	
Strengthen the east exterior masonry wall	Repoint exterior masonry walls	600	S.F.	$20	$12,000	
Strengthen upper floor diaphragm	Add Plywood	2520	S.F.	$4	$10,080	
	Add ties @ 2' o.c. for the (E) parallel walls	41	Ea.	$400	$16,500	
	Out of plane ties @ 4' o.c. (excluding bonded walls)	36	Ea.	$400	$14,250	
	Add Blocking between rafters@ anchors	228	L.F.	$25	$5,700	
	Add perimeter angle and anchors in masonry	228	L.F.	$50	$11,400	
Drag-strut and continuity ties	Custom drag-strut at Gridline 3 and 6	2	Ea.	$2,000	$4,000	
	Install strap at Gridline 3	2	Ea.	$1,000	$2,000	
	Install Simpson HD at 7-A and 7-B	2	Ea.	$500	$1,000	
Strengthen main floor diaphragm	Out of plane ties @ 4' o.c. (excluding bonded walls)	36	Ea.	$400	$14,250	
	Add Blocking between rafters@ anchors	228	L.F.	$25	$5,700	
	Add perimeter angle and anchors in masonry	228	L.F.	$50	$11,400	
Second floor shear wall at Grid 1	Demo (E) wall finishes	336	S.F.	$4	$1,344	
	Build 5'-4" and 7'-6" bonded shear walls	336	S.F.	$25	$8,400	
	Holdown at each wall end	4	Ea.	$400	$1,600	
	(N) gypsum and finish	336	S.F.	$12	$4,032	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Second floor shear wall at Grid 2(N)	Demo (E) wall finishes	228	S.F.	$4	$912	
	Build 19'-0" shear wall	228	S.F.	$8	$1,824	
	Holdown at each wall end	2	Ea.	$400	$800	
	(N) gypsum and finish	228	S.F.	$12	$2,736	
Second floor shear wall at Grid 2(S)	Demo (E) wall finishes	279	S.F.	$4	$1,116	
	Build 23'-3" shear wall	279	S.F.	$8	$2,232	
	Holdown at each wall end	2	Ea.	$400	$800	
	(N) gypsum and finish	279	S.F.	$12	$3,348	
Second floor shear wall at Grid 4	Demo (E) wall finishes	243	S.F.	$4	$972	
	Build 6'-3" and 14'-0" shear walls	243	S.F.	$10	$2,430	
	Holdown at each wall end	4	Ea.	$400	$1,600	
	(N) gypsum and finish	243	S.F.	$12	$2,916	
Second floor shear wall at Grid 7	Demo (E) wall finishes	154	S.F.	$4	$616	
	Build 12'-10" bonded shear walls	154	S.F.	$25	$3,852	
	(N) gypsum and finish	154	S.F.	$12	$1,849	
Second floor shear wall at Grid 8	Demo (E) wall finishes	243	S.F.	$4	$972	
	Holdown at each wall end	6	Ea.	$400	$2,400	
	(N) gypsum and finish	243	S.F.	$12	$2,916	
Second floor shear wall at Grid A	Demo (E) wall finishes	568	S.F.	$4	$2,272	
	Build 13'-3", 15'-10" and 18'-3" bonded shear walls	568	S.F.	$25	$14,200	
	(N) gypsum and finish	568	S.F.	$12	$6,816	
Second floor shear wall at Grid B	Demo (E) wall finishes	581	S.F.	$4	$2,324	
	Build 13'-4", 14'-10" and 20'-3" bonded shear walls	581	S.F.	$25	$14,524	
	(N) gypsum and finish	581	S.F.	$12	$6,971	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Second floor shear wall at Grid C	Demo (E) wall finishes	408	S.F.	$4	$1,632	
	Build 6'-0", 7'-0" and 21'-0" bonded shear walls	408	S.F.	$25	$10,200	
	(N) gypsum and finish	408	S.F.	$12	$4,896	
Main floor braced frame at Grid 1	Demo (E) wall finishes	306	S.F.	$4	$1,224	
	Demo floor to build footing	96	S.F.	$10	$960	
	Excavate for new footing	24	L.F.	$60	$1,440	
	New concrete foundation	24	L.F.	$200	$4,800	
	Build 20' SCBF frame	1	Ea.	$30,000	$30,000	
	Connections at SCBF	1	Ea.	$5,000	$5,000	
	Rebuild floor with finish	96	S.F.	$20	$1,920	
	(N) gypsum and finish	306	S.F.	$12	$3,672	
Main floor shear wall at Grid 2(N)	Demo (E) wall finishes	183	S.F.	$4	$731	
	Demo floor to build footing	64	S.F.	$10	$640	
	Excavate for new footing	16	L.F.	$60	$960	
	Add concrete foundation	16	L.F.	$200	$3,200	
	Build 14'-4" shear wall	183	S.F.	$8	$1,462	
	Holdown at each wall end	2	Ea.	$1,000	$2,000	
	Rebuild floor with finish	64	S.F.	$20	$1,280	
	(N) gypsum and finish	183	S.F.	$12	$2,192	
Main floor shear wall at Grid 2(S)	Demo (E) wall finishes	314	S.F.	$4	$1,258	
	Demo floor to build footing	99	S.F.	$10	$986	
	Excavate for new footing	25	L.F.	$60	$1,480	
	Add concrete foundation	25	L.F.	$200	$4,932	
	Build 10'-4" and 14'-4" shear walls	314	S.F.	$8	$2,515	
	Holdown at each wall end	4	Ea.	$1,000	$4,000	
	Rebuild floor with finish	99	S.F.	$20	$1,973	
	(N) gypsum and finish	314	S.F.	$12	$3,773	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Main floor shear wall at Grid 5	Demo (E) wall finishes	314	S.F.	$4	$1,258	
	Demo floor to build footing	99	S.F.	$10	$986	
	Excavate for new footing	25	L.F.	$60	$1,480	
	Add concrete foundation	25	L.F.	$200	$4,932	
	Build 10'-4" and 14'-4" shear walls	314	S.F.	$8	$2,515	
	Holdown at each wall end	4	Ea.	$1,000	$4,000	
	Rebuild floor with finish	99	S.F.	$20	$1,973	
	(N) gypsum and finish	314	S.F.	$12	$3,773	
Main floor shear wall at Grid 7	Demo (E) wall finishes	351	S.F.	$4	$1,403	
	Build 13'-0" and 14'-6" bonded shear walls	351	S.F.	$25	$8,766	
	(N) gypsum and finish	351	S.F.	$12	$4,208	
Main floor shear wall at Grid 8	Demo (E) wall finishes	132	S.F.	$4	$527	
	Build 10'-4" bonded shear wall	132	S.F.	$25	$3,293	
	Holdown at each wall end	2	Ea.	$1,000	$2,000	
	(N) gypsum and finish	132	S.F.	$12	$1,580	
Main floor shear wall at Grid A	Demo (E) wall finishes	306	S.F.	$4	$1,224	
	Demo floor to build footing	96	S.F.	$10	$960	
	Excavate for new footing	24	L.F.	$60	$1,440	
	New concrete foundation	24	L.F.	$200	$4,800	
	Build 20' SCBF frame	1	Ea.	$30,000	$30,000	
	Connections at SCBF	1	Ea.	$5,000	$5,000	
	Rebuild floor with finish	96	S.F.	$20	$1,920	
	(N) gypsum and finish	306	S.F.	$12	$3,672	
Main floor shear wall at Grid B	Demo (E) wall finishes	861	S.F.	$4	$3,443	
	Build 11'-0", 12'-0", 20'-0" and 24'-6" bonded shear walls	861	S.F.	$25	$21,516	
	(N) gypsum and finish	861	S.F.	$12	$10,328	

ESTIMATE OF PROBABLE CONSTRUCTION COST
STRUCTURAL

SEISMIC DESCRIPTION	ACTIVITY	QUANTITY	UNIT	UNIT PRICE	TOTAL PRICE	SUBTOTALS
Main floor shear wall at Grid D	Demo (E) wall finishes	255	S.F.	$4	$1,020	
	Demo floor to build footing	80	S.F.	$10	$800	
	Excavate for new footing	20	L.F.	$60	$1,200	
	Add concrete foundation	20	L.F.	$200	$4,000	
	Build 20'-0" shear wall	255	S.F.	$10	$2,550	
	Holdown at each wall end	2	Ea.	$1,000	$2,000	
	Rebuild floor with finish	80	S.F.	$20	$1,600	
	(N) gypsum and finish	255	S.F.	$12	$3,060	
						$684,821
					Sub-total	**$909,171**
					Contractor Overhead and Profit (25%)	**$227,293**
					Construction Contingency (25%)	**$227,293**
					Total Seismic Construction Cost	**$1,363,757**

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	Wood Wall
	Load Type	Dead	Dead	Dead	Dead
	Value (PSF)	15	15	15	8
	Load Type	Snow	Floor Live	Corridor Live	Brick Wall
	Value (PSF)	25	40	100	Dead
	Deflection Criteria	L/240	L/360	L/360	120

Lateral Design Parameters:

Wind Design: ASCE 7-10 **Wind Speed (3 sec Gust):** 120 MPH

Exposure B

Importance Factors I_W = 1.00 (ice w/ wind) I_E = 1.00 (seismic) I_S = 1.00 (snow) I_i = 1.00 (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 schematic analysis of the existing URM building located in Ashland, Oregon considering current code loading with proposed shear wall upgrade design concepts. The braced frame analysis and design is not included in this scope of work.

MILLER CONSULTING ENGINEERS

9570 SW Barbur Blvd.
Suite One Hundred
Portland, OR 97219

(503)246-1250
FAX: 246-1395

Project Name: Ashland City Hall Seismic Evaluation Project #: 150899

Location: 20 E Main Street, Ashland, Oregon

Client: City of Ashland

BY: ERW Ck'd: Date: 12/08/15 Page 1

USGS Design Maps Summary Report

User–Specified Input

Report Title Ashland City Hall Evaluation
Wed October 14, 2015 00:21:15 UTC

Building Code Reference Document 2012 International Building Code
(which utilizes USGS hazard data available in 2008)

Site Coordinates 42.19673°N, 122.71453°W

Site Soil Classification Site Class D – "Stiff Soil"

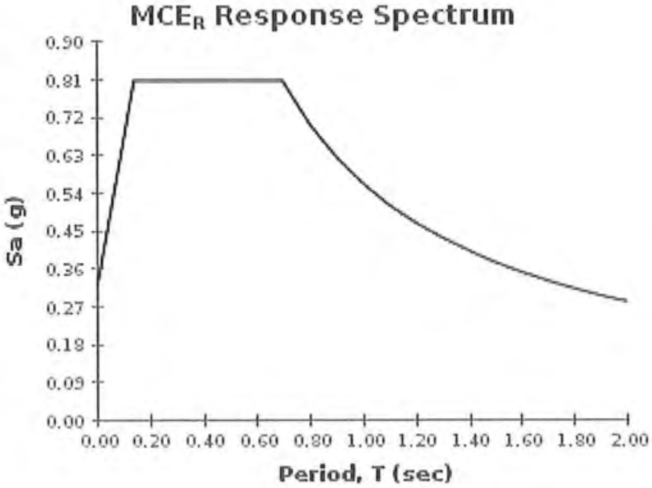
Risk Category I/II/III

USGS–Provided Output

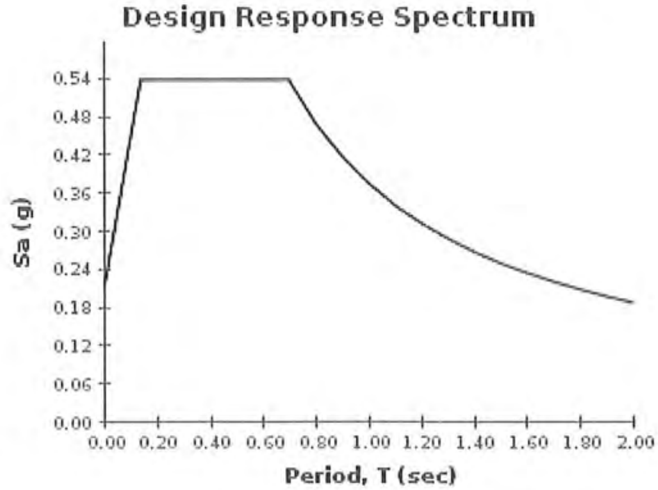
$S_S =$	0.619 g	$S_{MS} =$	0.808 g	$S_{DS} =$	0.538 g
$S_1 =$	0.318 g	$S_{M1} =$	0.561 g	$S_{D1} =$	0.374 g

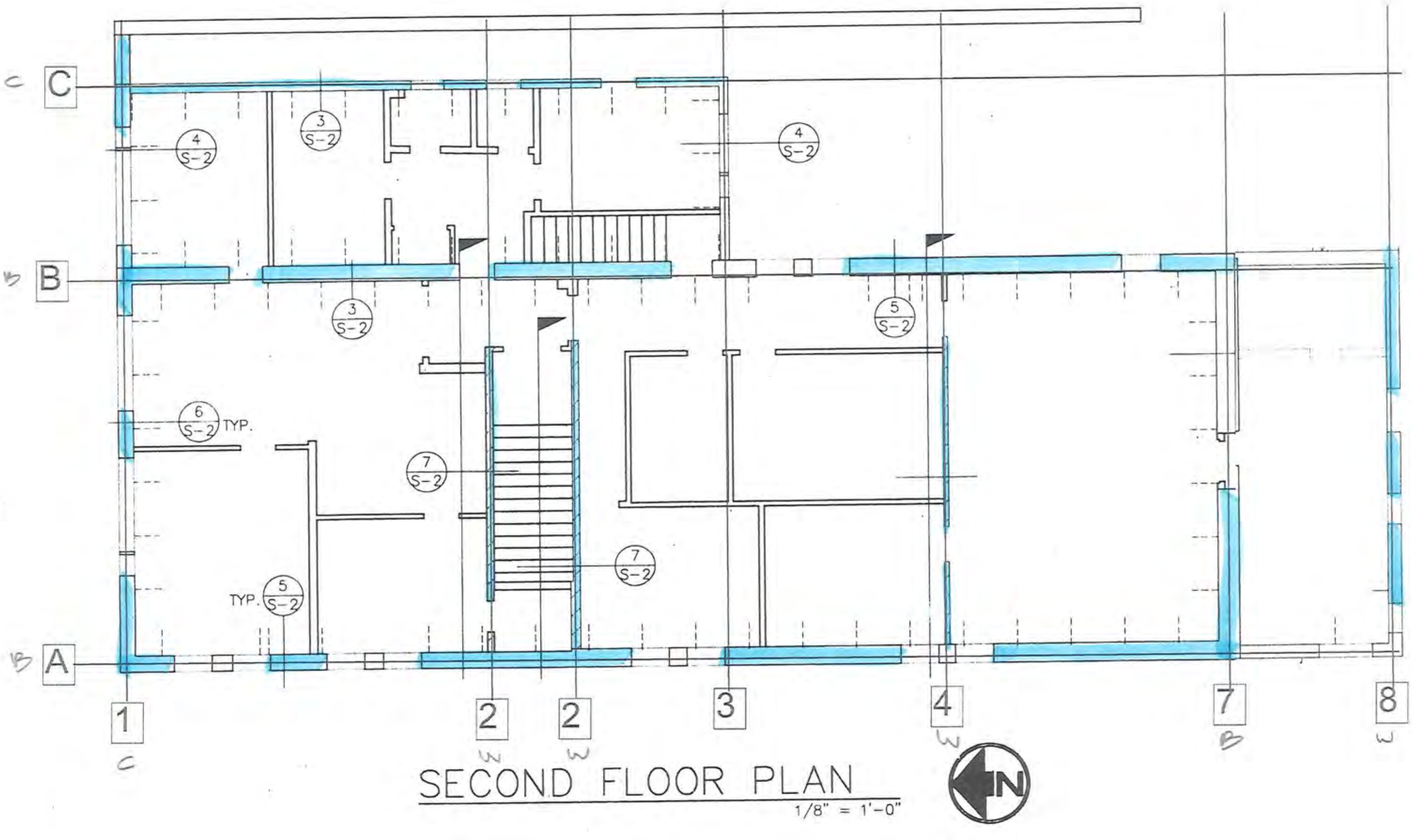
For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.

MCE_R Response Spectrum



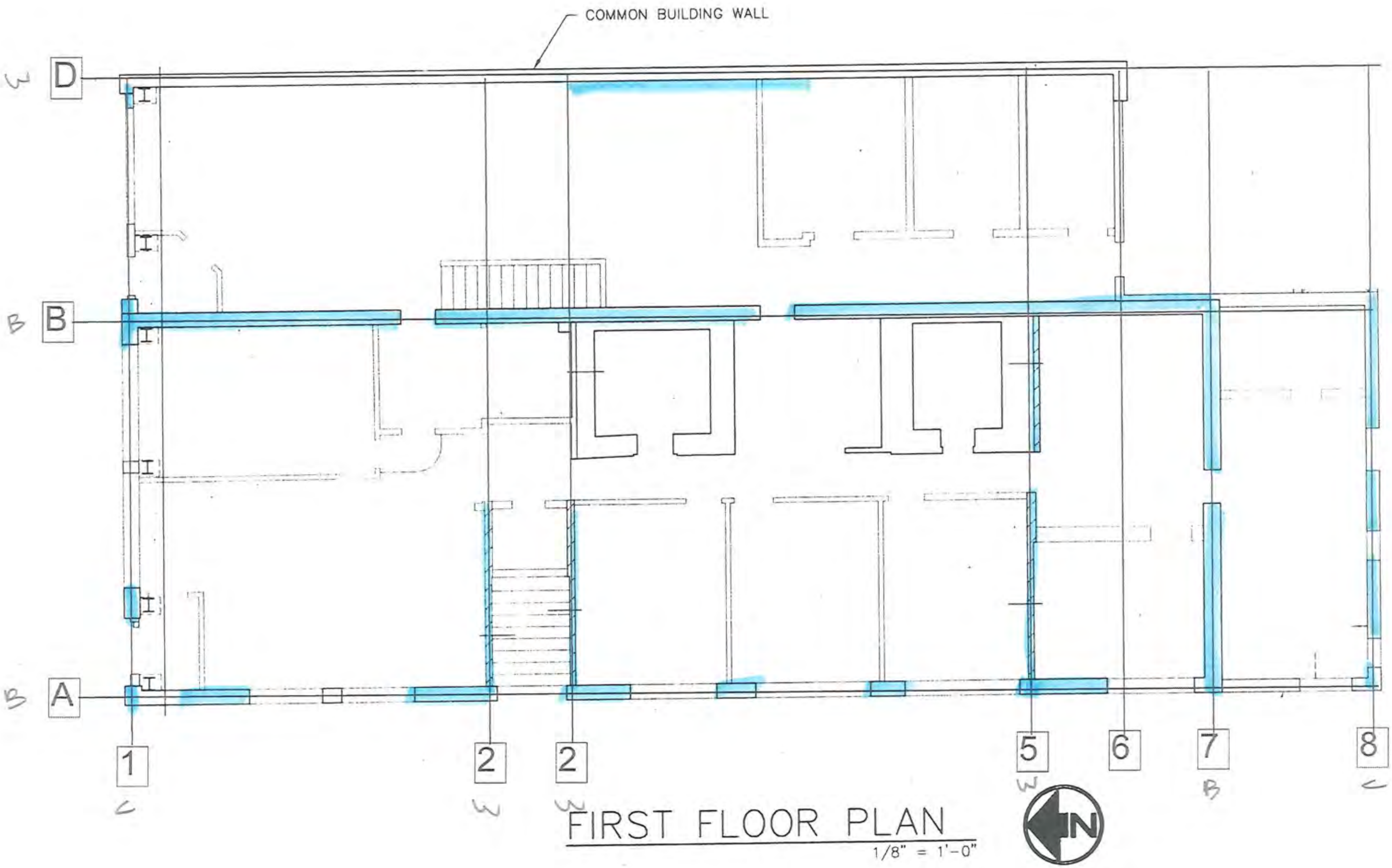
Design Response Spectrum





SECOND FLOOR PLAN

1/8" = 1'-0"



FIRST FLOOR PLAN

1/8" = 1'-0"

Building Geometries and Weights:

Roofs:

East =	11500	lb
West =	51800	lb
Sub Total =	63300	lb

Second Floor:

East =	24500	lb
West =	51800	lb
Sub Total =	76300	lb

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

GL 1 =	97200	lb
GL 2(N) =	0	lb
GL 2(S) =	0	lb
GL 3 =	7840	lb
GL 4 =	0	lb
GL 5 =	0	lb
GL 6 =	0	lb
GL 7 =	36000	lb
GL 8 =	0	lb
Sub Total =	141040	lb

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

GL A =	101000	lb
GL B =	101000	lb
GL C =	24750	lb
GL D =	0	lb
Sub Total =	226750	lb
Total =	**431090**	**lb**

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

GL 1 =	100400	lb
GL 2(N) =	0	lb
GL 2(S) =	0	lb
GL 3 =	7840	lb
GL 4 =	0	lb
GL 5 =	0	lb
GL 6 =	11480	lb
GL 7 =	44600	lb
GL 8 =	15300	lb
Sub Total =	179620	lb

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

GL A =	130850	lb
GL B =	130850	lb
GL C =	24750	lb
GL D =	0	lb
Sub Total =	286450	lb
Total =	**542370**	**lb**

East Upper Section

Width =	14.25	ft
Length =	45.00	ft
2nd Floor Height =	12.00	ft

East Main Section

Width =	78.00	ft
Length =	64.00	ft
1st Floor Height =	12.75	ft

West Section

Width =	30.00	ft
Length =	96.00	ft
2nd Floor Height =	12.00	ft
1st Floor Height =	12.75	ft

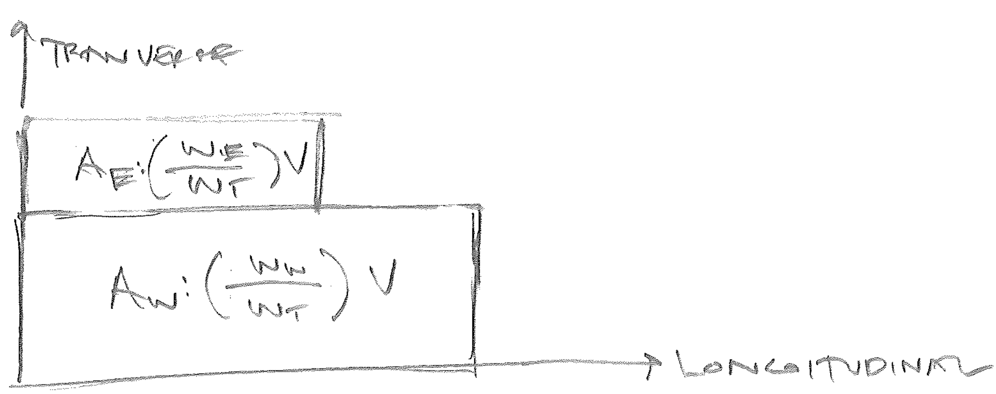
Roof Weight =	18	psf
8" Concrete Wall =	80	psf
13.5" Concrete Wall =	168.75	psf
2 Wythe Wall =	80	psf
3 Wythe Wall =	120	psf
Second floor =	18	psf

ASCII 7-10 Seismic Design (w / URM WALLS)

S_s =	61.90%	*Risk Targeted Maximum Considered Earthquake (Figure 22-1, 22-3, 22-5, and 22-6)(pages 158 through 165)*
S_1 =	31.80%	*Risk Targeted Maximum Considered Earthquake (Figure 22-2, 22-4, 22-5, and 22-6)(pages 158 through 165)*
Fa=	1.30	*Table 11.4-1, page 55*
Fv=	1.76	*Table 11.4-1, page 55*
$S_{MS} = F_a S_s$ =	0.81	*eqn. 11.4-1 page 55*
$S_{M1} = F_v S_1$ =	0.56	*eqn. 11.4-2 page 55*
S_{DS} = (2) S_{MS}/(3) =	0.54	*eqn. 11.4-3 page 55*
S_{D1} = (2) S_{M1}/(3) =	0.37	*eqn. 11.4-4 page 55*
Site Class	D	*Table 20.3-1, page 152*
Risk Category	II	*Table 1.5-1, page 2*
Seismic Force Resisting System	A. Bearing Wall System	
	10. Ordinary plain masonry shear walls	
Seismic Design Category (short per.)	D	*Table 11.6-1, page 56*
Seismic Design Category (1 sec)	D	*Table 11.6-2, page 56*
Seismic Design Category	D	(Controls)
R =	1.50	*Table 12.2-1, pages 60-62*
Ω =	2.50	*Table 12.2-1, pages 60-62*
Cd =	1.25	*Table 12.2-1, pages 60-62*
I_E =	1.00	*Importance factor, Table 1.5-2, page 4*
C_T =	0.02	*Table 12.8-2, page 72*
x =	0.75	*Table 12.8-2, page 72*
h_n =	24.75	*ft, defined in section 12.8.2.1, page 72*
T =	0.222	*Eqn. 12.8-7, page 72*
k =	1.00	*Section 12.8.3, page 73*
Cu =	1.4	*Table 12.8-1, page 72*
Ta_{max} =	0.311	*section 12.8.2, page 72*
C_s =	0.359	*Eqn. 12.8-2,pg 72*
T_L =	16	*Fig. 22-12 through 22-16, page 170-173*
C_s =	1.123	*need not exceed - Eq. 12.8-3 & 12.8-4, page 71-72*
C_s =	0.024	*shall not be less than -Eq. 12.8-5 & 12.8-6, page 72*
C_s =	0.251	*(control) * 0.7 (for allowable loads)*
W_{roof} =	431090	lb
W_{2nd} =	539170	lb
Redundancy Factor ρ:	1.00	*Section 12.3.4, pg 67*
Vroof = CsW = 0.251(W) =	108322	lb, *Eq. 12.8-1, pg 71*
V2nd = CsW = 0.251(W) =	135480	lb, *Eq. 12.8-1, pg 71*

Vertical Distribution of Seismic Forces: *Eqn. 12.8-11 & Eq. 12.8-12, pg 72-73*

Level	h_x (ft.)	w_x(lb)	$w_x{*}h_x^k$	Cv	Force (lbs)	with ρ:
Roof	24.75	431090	10669477.5	0.61	148271	**148271**
2nd	12.75	539170	6874417.5	0.39	95532	**95532**
			17543895	Total Base Shear	243803	



Shear Load from Vertical Distribution: (W/ URM WALLS)

Roof Load =	148271	lb
Main Load =	95532	lb
	243803	

Shear Wall Distribution:

	Roof-Long	Roof-Trans	Main-Long	Main-Trans
East =	19340	36250	43820	49250
West =	185000	253800	212100	313500
Total =	204340	290050	255920	362750

Shear Wall Loading/Diaphragm Loads:

	Seismic (lb)					
	Roof	Roof SW	Shear (PLF)	Second	2nd SW	Shear (PLF)
Gridline 1	27848	20	1370	46158	9	5130
GL 2(N) =	22299	19	1174	36489	14	2546
GL 2(S) =	28569	23	1229	54153	14	3779
GL 3 =	To GL2 & 4	N/A	N/A	N/A	N/A	
GL 4 =	36951	20	1825	N/A	N/A	
GL 5 =	N/A	N/A	N/A	53359	25	2164
GL 6 =	N/A	N/A	N/A	To GL5 &7	N/A	
GL 7 =	23313	13	1817	37759	28	1373
GL 8 =	8953	21	426	14185	21	675
	147933			242104		
GL A =	67119	56	1206	106706	31	3442
GL B =	74136	66	1131	123104	77	1597
GL C =	7017	37	190	N/A	N/A	
GL D =	N/A	N/A	N/A	13993	20	700
	148271			243803		

CAP OF URM @ 5 psi : 5(12(12)) = 720 plf

MAX TWO SIDED SW : 1540 plf

MAX ONE SIDED SW : 770 plf

∴ LOADS ARE TOO HIGH, ADD WOOD SHEAR WALLS AT THE (E) WALLS TO TAKE ALL THE IN-PLANE LOADING

CHECK DIAPHRAGMS:

$V_{RW} = 148271\left(\frac{253800}{290050}\right) = 130\ k$

$v_{RW} = \frac{1}{2}(130\ k)/30 = 2160\ plf$

$V_{SW} = 95532\left(\frac{313500}{362750}\right) = 83.6\ k$

$v_{SW} = \frac{1}{2}(83.6)/30 = 1393\ plf$

ASCE 7-10 Seismic Design (w/ wood shear walls)

S_s =	61.90%	*Risk Targeted Maximum Considered Earthquake (Figure 22-1, 22-3, 22-5, and 22-6)(pages 158 through 165)*
S_1 =	31.80%	*Risk Targeted Maximum Considered Earthquake (Figure 22-2, 22-4, 22-5, and 22-6)(pages 158 through 165)*
Fa=	1.30	*Table 11.4-1, page 55*
Fv=	1.76	*Table 11.4-1, page 55*
$S_{MS} = F_a S_s$ =	0.81	*eqn. 11.4-1 page 55*
$S_{M1} = F_v S_1$ =	0.56	*eqn. 11.4-2 page 55*
S_{DS} = (2) S_{MS}/(3) =	0.54	*eqn. 11.4-3 page 55*
S_{D1} = (2) S_{M1}/(3) =	0.37	*eqn. 11.4-4 page 55*
Site Class	D	*Table 20.3-1, page 152*
Risk Category	II	*Table 1.5-1, page 2*
Seismic Force Resisting System	A. Bearing Wall System	
	15. Light framed (wood) walls sheathed with wood structural panels rated for shear resistance	
Seismic Design Category (short per.)	D	*Table 11.6-1, page 56*
Seismic Design Category (1 sec)	D	*Table 11.6-2, page 56*
Seismic Design Category	D	(Controls)
R =	6.50	*Table 12.2-1, pages 60-62*
Ω =	3.00	*Table 12.2-1, pages 60-62*
Cd =	4.00	*Table 12.2-1, pages 60-62*
I_E =	1.00	*Importance factor, Table 1.5-2, page 4*
C_T =	0.02	*Table 12.8-2, page 72*
x =	0.75	*Table 12.8-2, page 72*
h_n =	24.75	*ft, defined in section 12.8.2.1, page 72*
T =	0.222	*Eqn. 12.8-7, page 72*
k =	1.00	*Section 12.8.3, page 73*
Cu =	1.4	*Table 12.8-1, page 72*
Ta_{max} =	0.311	*section 12.8.2, page 72*
C_s =	0.083	*Eqn. 12.8-2,pg 72*
T_L =	16	*Fig. 22-12 through 22-16, page 170-173*
C_s =	0.259	*need not exceed - Eq. 12.8-3 & 12.8-4, page 71-72*
C_s =	0.024	*shall not be less than -Eq. 12.8-5 & 12.8-6, page 72*
C_s =	0.058	*(control) * 0.7 (for allowable loads)*
W_{roof} =	431090	lb
W_{2nd} =	542370	lb
Redundancy Factor ρ:	1.00	*Section 12.3.4, pg 67*
Vroof = CsW = 0.058(W) =	24997	lb, *Eq. 12.8-1, pg 71*
V2nd = CsW = 0.058(W) =	31450	lb, *Eq. 12.8-1, pg 71*

Vertical Distribution of Seismic Forces: *Eqn. 12.8-11 & Eq. 12.8-12, pg 72-73*

Level	h_x (ft.)	w_x(lb)	w_x*h_x^k	Cv	Force (lbs)	with ρ:
Roof	24.75	431090	10669477.5	0.61	34250	**34250**
2nd	12.75	542370	6915217.5	0.39	22198	**22198**
			17584695	Total Base Shear	56448	

Shear Load from Vertical Distribution: (w/ WOOD SHEAR WALLS)

Roof Load =	34250	lb
Main Load =	22198	lb
	56448	

Shear Wall Distribution:

	Roof-Long	Roof-Trans	Main-Long	Main-Trans
East =	19340	36250	43820	49250
West =	185000	253800	212100	313500
Total =	204340	290050	255920	362750

Shear Wall Loading/Diaphragm Loads:

	Seismic (lb) Roof	Roof SW	Shear (PLF)	Second	2nd SW	Shear (PLF)
Gridline 1	6433	20	316	10687	4	3054
GL 2(N) =	5151	19	271	8448	14	590
GL 2(S) =	6599	23	284	12534	14	875
GL 3 =	To GL2 & 4	N/A	N/A	N/A	N/A	
GL 4 =	8535	20	422	N/A	N/A	
GL 5 =	N/A	N/A	N/A	12359	25	501
GL 6 =	N/A	N/A	N/A	To GL5 &7	N/A	
GL 7 =	5385	13	420	8742	28	318
GL 8 =	2068	21	98	3284	10	318
	34172			56054		
GL A =	15504	47	327	24703	23	1066
GL B =	17125	48	354	28502	77	370
GL C =	1621	37	44	N/A	N/A	
GL D =	N/A	N/A	N/A	3243	20	162
	34250			56448		

SEISMIC DESIGN FORCE, Section 13.3: (OUT OF PLANE AT PARAPET)

Elements of Structures, Nonstructural Components, and Equipment Supported by Structures

Site Class:	D	*ASCE 7-10, Sec. 20.3, Table 20.3-1, pg. 152*
Seismic Design Category:	D	*ASCE 7-10, Sec. 11.6, pg 56*
Risk Category:	II	*IBC 2012 Sec 1604.5, pg 336*
S_s =	61.90%	*ASCE 7-10, Figure 22-1 page 158*
Fa=	1.30	*ASCE 7-10 Table 11.4-1 pg 55 (Linear interpolation is used)*
S_{MS} =	0.81	*ASCE 7-10 eqn. 11.4-1 pg 55*
S_{DS} =	0.54	*ASCE 7-10 eqn. 11.4-3 pg 55*
I_E=	1.00	*ASCE 7-10 Sec 13.1.3, pg 87*
a_p=	1.0	*ASCE 7-10 Table 13.6-1, pg 93*
R_p=	2.50	*ASCE 7-10 Table 13.6-1, pg 93*
z (ft)=	26.75	*Component attachment elevation w/ respect to grade*
h (ft)=	26.75	*Structure roof elevation with respect to grade*
F_p=	0.258	* Wp *ASCE 7-10 Eq. 13.3-1, pg 88*
OR	0.862	* Wp *ASCE 7-10 Eq. 13.3-2, pg 89*
Not less than	0.162	* Wp *ASCE 7-10 Eq. 13.3-3, pg 89*

F_p=	**0.181**	*** Wp**	*x 0.7 (Allowable Loading)*
$0.2S_{DS}Wp$ =	**0.108**	*** Wp**	*(ASCE7-10 Sec. 13.3, pg 88)*

(12.11.1) $0.4 S_{DS}(0.7) = 0.151 \Rightarrow$ USE 13.3-1

TRY BRACING ONLY AT CEILING:

$$M_{PW} = \frac{0.181(80)(6)^2}{2} = 261'\#$$

$$M_R = \left[0.6(6(80))\right]\cdot\left[\tfrac{1}{2}(0.667)\right] = 96'\#$$

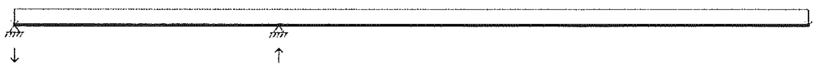
$$F_B = \frac{(261-96)}{(8/12)(12)} = 20.6\ psi >> 5\ psi$$

$\Rightarrow$ BRACE WALL AT ROOF AND CEILING

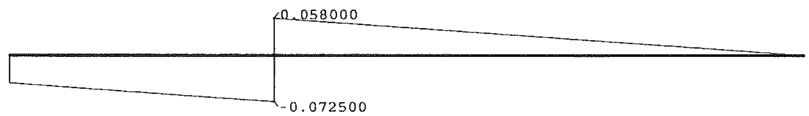
WinBeam

Project: PARAPET BRACING AT ROOF/CEILING				
By:	Date:	Checked:	Date:	Page:

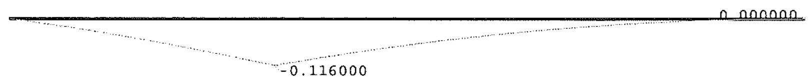
Reactions - kips, kip ft



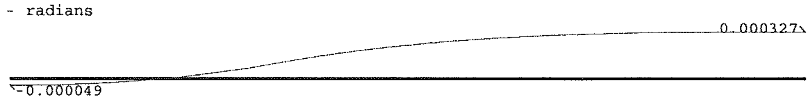
Shear - kips



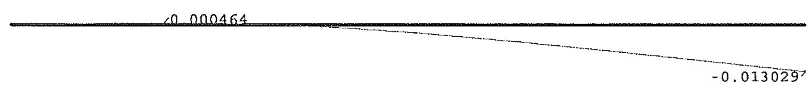
Moment - kip ft



Rotation - radians



Deflection - inches



LOAD AT MASONRY

$M_R = \frac{1}{2}(0.667)[0.6(6(80))]$

$\Rightarrow M_R = 96$ '#

$F_B = \frac{(116-96)}{(8/12)(12)}$

$F_B = 1.12$ psi

WinBeam

Project:				
By:	Date:	Checked:	Date:	Page:

Analysis Data:

Beam Length = 6. feet
Number of Nodes = 201
Number of Elements = 200
Number of Degrees of Freedom = 402

Reactions:

X feet	Vert kips	Rot kip ft
0	-0.043500	
2.000	0.130500	

Equilibrium:

	Force	Reaction	Diff
Vert	-0.087000	0.087000	-0.000000 kips
Rot	0.261000	-0.261000	0.000000 kip ft

Min & Max values:

Min Shear	= -0.072500	kips	at	2.000 feet
Max Shear	= 0.058000	kips	at	2.000 feet
Min Moment	= -0.116000	kip ft	at	2.000 feet
Max Moment	= 1.421e-014	kip ft	at	6.000 feet
Min Rotation	=-4.872e-005	radians	at	0 feet
Max Rotation	= 0.0003271	radians	at	6.000 feet
Min Deflection	= -0.013029	in	at	6.000 feet
Max Deflection	= 0.0004638	in	at	1.164 feet

SEISMIC DESIGN FORCE, Section 13.3: (OOP AT UPPER WALLS)

Elements of Structures, Nonstructural Components, and Equipment Supported by Structures

Site Class:	D	*ASCE 7-10, Sec. 20.3, Table 20.3-1, pg. 152*
Seismic Design Category:	D	*ASCE 7-10, Sec. 11.6, pg 56*
Risk Category:	II	*IBC 2012 Sec 1604.5, pg 336*
S_s =	61.90%	*ASCE 7-10, Figure 22-1 page 158*
Fa=	1.30	*ASCE 7-10 Table 11.4-1 pg 55 (Linear interpolation is used)*
S_{MS} =	0.81	*ASCE 7-10 eqn. 11.4-1 pg 55*
S_{DS} =	0.54	*ASCE 7-10 eqn. 11.4-3 pg 55*
I_E=	1.00	*ASCE 7-10 Sec 13.1.3, pg 87*
a_p=	1.0	*ASCE 7-10 Table 13.6-1, pg 93*
R_p=	2.50	*ASCE 7-10 Table 13.6-1, pg 93*
z (ft)=	18.75	*Component attachment elevation w/ respect to grade*
h (ft)=	26.75	*Structure roof elevation with respect to grade*
F_p=	0.207	* Wp *ASCE 7-10 Eq. 13.3-1, pg 88*
OR	0.862	* Wp *ASCE 7-10 Eq. 13.3-2, pg 89*
Not less than	0.162	* Wp *ASCE 7-10 Eq. 13.3-3, pg 89*

F_p=	**0.145**	*** Wp**	*x 0.7 (Allowable Loading)*
$0.2S_{DS}Wp$ =	**0.108**	*** Wp**	*(ASCE7-10 Sec. 13.3, pg 88)*

(12.11.1) $0.4S_{DS} \cdot 0.7 = 0.151$ ∴ USE 12.11.1

$$M = \frac{[0.151(120)](12)^2}{8} = 326\ '\#$$

$$M_p = 0.6 \cdot [6(80) + 6(120)]\ 1/2(1) = 360\ '\#$$

∴ WALL UNDER COMPRESSION, NO TENSION

∴ DUE TO OOP LOADING

SEISMIC DESIGN FORCE, Section 13.3: (OOP AT MAIN WALLS)

Elements of Structures, Nonstructural Components, and Equipment Supported by Structures

Site Class:	D	*ASCE 7-10, Sec. 20.3, Table 20.3-1, pg. 152*	
Seismic Design Category:	D	*ASCE 7-10, Sec. 11.6, pg 56*	
Risk Category:	II	*IBC 2012 Sec 1604.5, pg 336*	
S_s =	61.90%	*ASCE 7-10, Figure 22-1 page 158*	
Fa=	1.30	*ASCE 7-10 Table 11.4-1 pg 55 (Linear interpolation is used)*	
S_{MS} =	0.81	*ASCE 7-10 eqn. 11.4-1 pg 55*	
S_{DS} =	0.54	*ASCE 7-10 eqn. 11.4-3 pg 55*	
I_E=	1.00	*ASCE 7-10 Sec 13.1.3, pg 87*	
a_p=	1.0	*ASCE 7-10 Table 13.6-1, pg 93*	
R_p=	2.50	*ASCE 7-10 Table 13.6-1, pg 93*	
z (ft)=	6.375	*Component attachment elevation w/ respect to grade*	
h (ft)=	26.75	*Structure roof elevation with respect to grade*	
F_p=	0.127	* Wp	*ASCE 7-10 Eq. 13.3-1, pg 88*
OR	0.862	* Wp	*ASCE 7-10 Eq. 13.3-2, pg 89*
Not less than	0.162	* Wp	*ASCE 7-10 Eq. 13.3-3, pg 89*

F_p=	**0.113**	*** Wp**	*x 0.7 (Allowable Loading)*
$0.2S_{DS}Wp$ =	**0.108**	*** Wp**	*(ASCE7-10 Sec. 13.3, pg 88)*

(12.11.1) $0.4 S_{DS} \cdot 0.7 = 0.151$ ⇒ USE 12.11.1

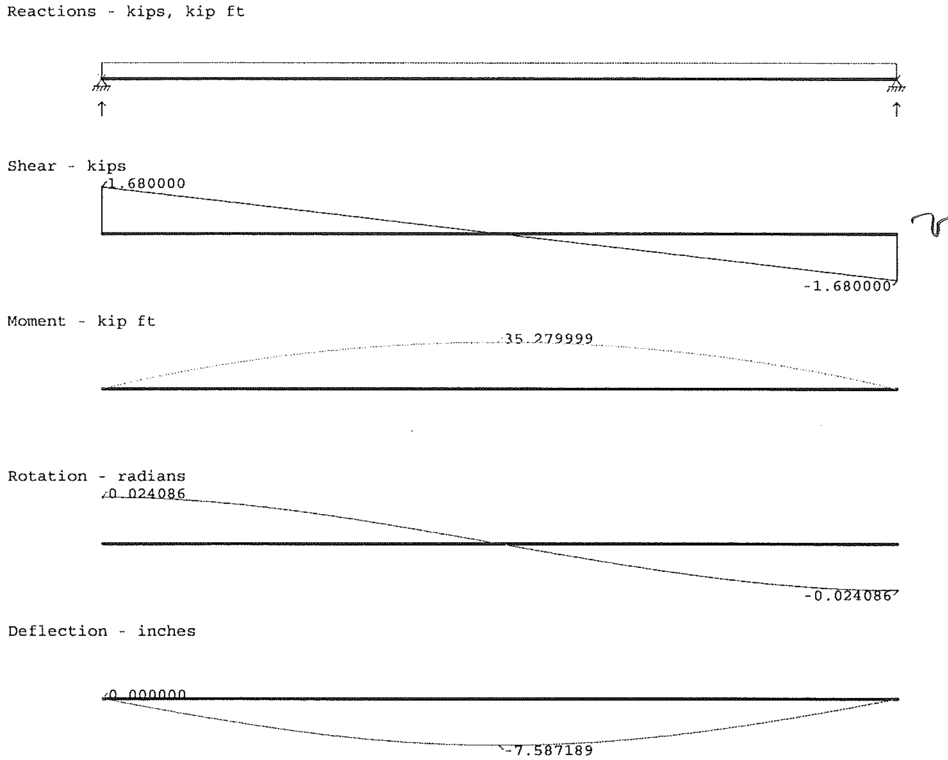
$$M_w = \frac{[0.151(120)](12.75)^2}{8} = 368 \text{ '\#}$$

$$M_P = 0.6[6(80) + (12+6.375)(120)][\tfrac{1}{2}(1)] = 806 \text{ '\#}$$

⇒ WALL UNDER COMPRESSION, NO TENSION IN MASONRY DUE TO OOP LOADING

WinBeam

Project: DIAPHRAGM - ROOF				
By:	Date:	Checked:	Date:	Page:



$V_D = \frac{1680}{30} = 56$ plf

WinBeam

Project:				
By:	Date:	Checked:	Date:	Page: FROM C&G DIAPHRAGM

$V_R = 357\left[\frac{9/10(51800) + 2(3(60(84)))}{290050}\right]$

$\Rightarrow V_R = 40$ plf

Analysis Data:

Beam Length = 84. feet
Number of Nodes = 201
Number of Elements = 200
Number of Degrees of Freedom = 402

Reactions:

X feet	Vert kips	Rot kip ft
0	1.680	
84.000	1.680	

Equilibrium:

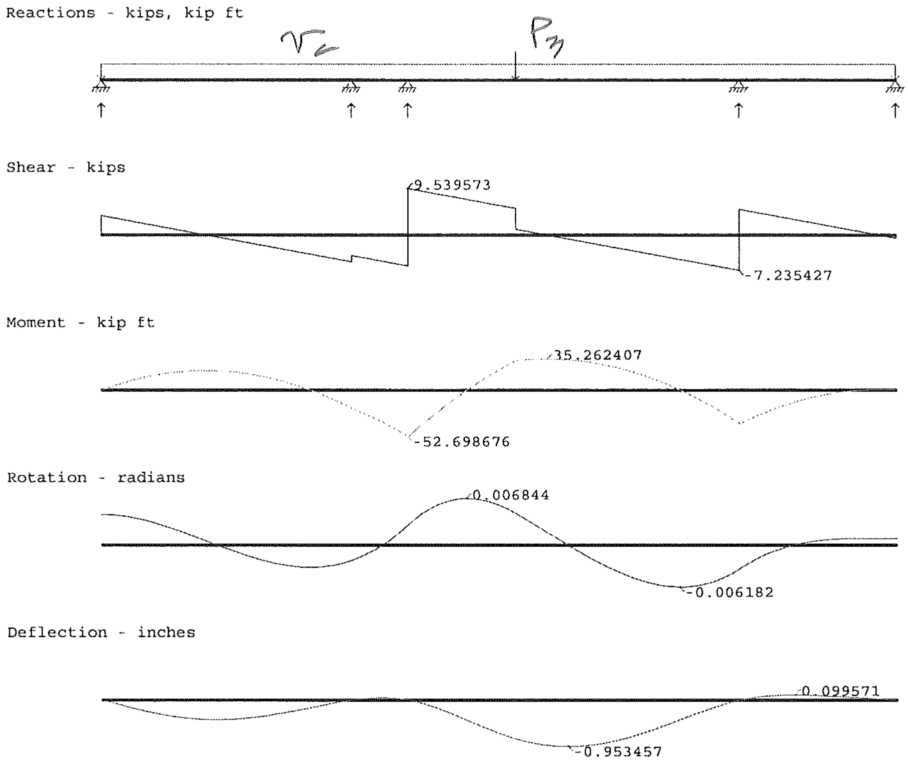
	Force	Reaction	Diff	
Vert	-3.360	3.360	-0.000	kips
Rot	141.120	-141.120	0.000	kip ft

Min & Max values:

Min Shear	=	-1.680 kips	at	84.000	feet
Max Shear	=	1.680 kips	at	0	feet
Min Moment	=	1.828e-013 kip ft	at	0	feet
Max Moment	=	35.280 kip ft	at	42.000	feet
Min Rotation	=	-0.024086 radians	at	84.000	feet
Max Rotation	=	0.024086 radians	at	0	feet
Min Deflection	=	-7.587 in	at	42.000	feet
Max Deflection	=	0 in	at	0	feet

WinBeam

Project: DIAPHRAGM - CEILING AT SECOND				
By:	**Date:**	**Checked:**	**Date:**	**Page:**



$$V_D = \frac{9540}{30} = 318 \text{ plf}$$

WinBeam 3.30 - Registered to Miller Consulting Engrs.

WinBeam

Project:				
By:	**Date:**	**Checked:**	**Date:**	**Page:**

Analysis Data:

Beam Length = 84. feet
Number of Nodes = 203
Number of Elements = 202
Number of Degrees of Freedom = 406

$$V_L = \frac{34280\left(\frac{253850}{290080}\right)}{84} = 357 \text{ plf}$$

$$P_3 = 34280\left(\frac{36250}{290080}\right) = 4280\#$$

Reactions:

X feet	Vert kips	Rot kip ft
0	3.938	
26.500	1.307	
32.500	15.896	
67.500	12.487	
84.000	0.638818	

Equilibrium:

	Force	Reaction	Diff
Vert	-34.268	34.268	0.000 kips
Rot	1447.816	-1447.816	-0.000 kip ft

Min & Max values:

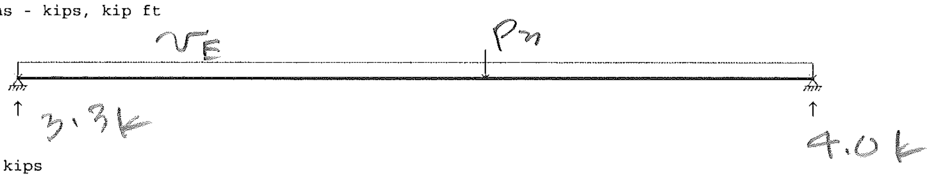
Min Shear	=	-7.235 kips	at	67.500	feet
Max Shear	=	9.540 kips	at	32.500	feet
Min Moment	=	-52.699 kip ft	at	32.500	feet
Max Moment	=	35.262 kip ft	at	47.357	feet
Min Rotation	=	-0.006182 radians	at	61.205	feet
Max Rotation	=	0.006844 radians	at	38.661	feet
Min Deflection	=	-0.953457 in	at	49.455	feet
Max Deflection	=	0.099571 in	at	73.275	feet

WinBeam 3.30 - Registered to Miller Consulting Engrs.

WinBeam

Project: DIAPHRAGM - SECOND FLOOR (EAST)				
By:	Date:	Checked:	Date:	Page:

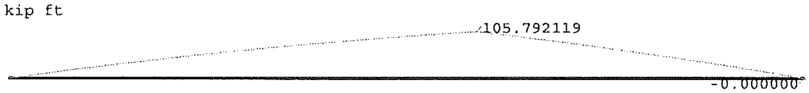
Reactions - kips, kip ft



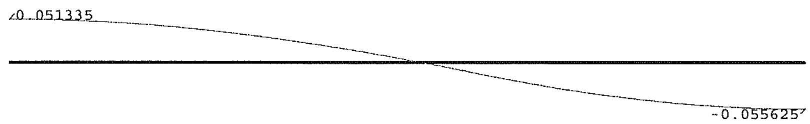
Shear - kips

$V = \frac{4033}{18} = 224$ plf

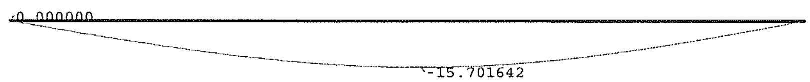
Moment - kip ft



Rotation - radians



Deflection - inches



WinBeam

Project:				
By:	Date:	Checked:	Date:	Page:

Analysis Data:

$V_E = \frac{2198\left(\frac{49790}{762750}\right)}{75.5} = 40$ plf

Beam Length = 75.5 feet
Number of Nodes = 201
Number of Elements = 200
Number of Degrees of Freedom = 402

Reactions:

X feet	Vert kips	Rot kip ft
0	3.267	
75.500	4.033	

Equilibrium:

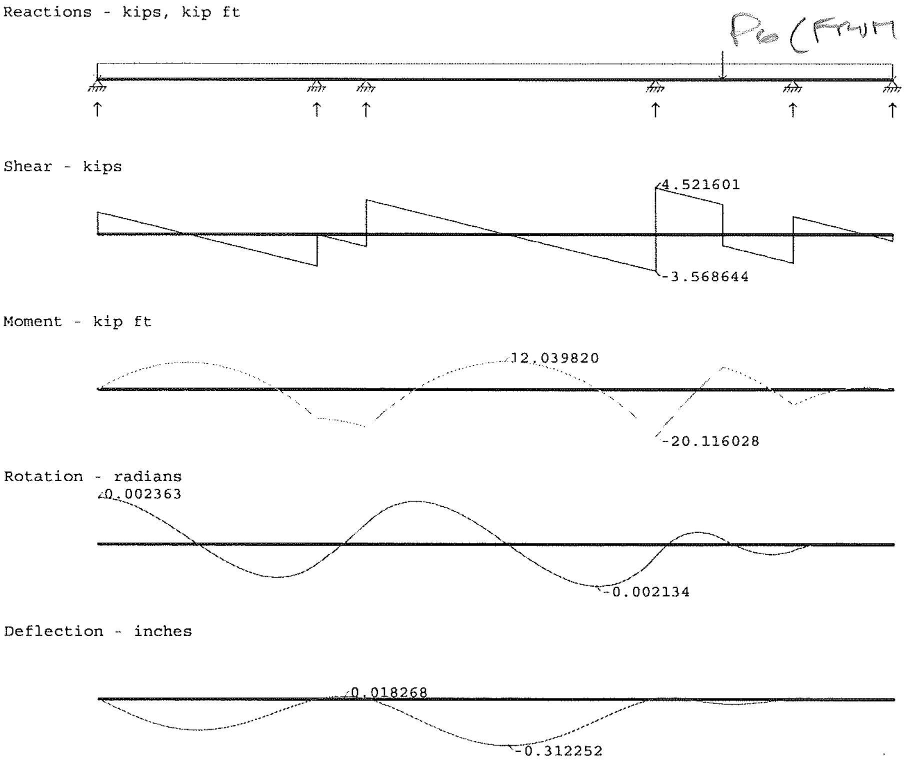
	Force	Reaction	Diff	
Vert	-7.300	7.300	0.000	kips
Rot	304.465	-304.465	-0.000	kip ft

Min & Max values:

Min Shear	=	-4.033 kips	at	75.500	feet
Max Shear	=	3.267 kips	at	0	feet
Min Moment	=	-8.577e-013 kip ft	at	75.500	feet
Max Moment	=	105.792 kip ft	at	44.500	feet
Min Rotation	=	-0.055625 radians	at	75.500	feet
Max Rotation	=	0.051335 radians	at	0	feet
Min Deflection	=	-15.702 in	at	39.220	feet
Max Deflection	=	0 in	at	0	feet

WinBeam

Project: DIAPHRAGM - SECOND FLOOR (WEST)				
By:	Date:	Checked:	Date:	Page:



$v_s = \frac{4520}{30} = 151$ plf

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WinBeam

Project:				
By:	Date:	Checked:	Date:	Page:

$v_W = \frac{22198\left(\frac{713800}{762750}\right)}{96} = 199$ plf

Analysis Data:

Beam Length = 96. feet
Number of Nodes = 202
Number of Elements = 201
Number of Degrees of Freedom = 404

Reactions:

X feet	Vert kips	Rot kip ft
0	2.140	
26.500	3.089	
32.500	4.568	
67.500	8.090	
84.000	4.523	
96.000	0.628220	

Equilibrium:

	Force	Reaction	Diff	
Vert	-23.038	23.038	0.000	kips
Rot	1216.649	-1216.649	-0.000	kip ft

Min & Max values:

Min Shear	=	-3.569	kips	at	67.500 feet
Max Shear	=	4.522	kips	at	67.500 feet
Min Moment	=	-20.116	kip ft	at	67.500 feet
Max Moment	=	12.040	kip ft	at	49.281 feet
Min Rotation	=	-0.002134	radians	at	60.308 feet
Max Rotation	=	0.002363	radians	at	0 feet
Min Deflection	=	-0.312252	in	at	49.760 feet
Max Deflection	=	0.018268	in	at	29.731 feet

WinBeam 3.30 - Registered to Miller Consulting Engrs.

Shearwall Design **# of Shearwall Levels:** 2

Load Combination:

Basic Load Combination: 0.6D+0.7E; 0.6D+W

Structure Information:

Wall Wt. (W_W) =	120.00	psf
Roof Wt. (R_W) =	18.00	psf
Roof Wall Height =	12.00	ft
Upper Floor Depth =	1.00	ft
Upper Floor Wt. (U_W) =	15.00	psf
Upper Floor Wall Height =	11.75	ft
Floor Depth =	N/A	ft
Wt. (M_W) =	N/A	psf
Wall Height =	N/A	ft
Wind Uplift (U) =	10.00	psf
0.2Sds =	0.11	(ultimate)

Roof Wall Line Marks	Wind (lb)	Min. Wind (lb)	Seismic (lb)	Stacks on:
R1			6433	S1
R2 (N)			5151	S2 (N)
R2 (S)			6599	S2 (S)
R4			8535	S4
R5			0	
R7			5385	S7
R8			2068	S8
RA			15504	SA
RB			17125	SB
RC			1621	

Upper Floor Wall Line Marks	Wind (lb)	Min. Wind (lb)	Seismic (lb)	
S1			4254	
S2 (N)			3297	
S2 (S)			5935	
S4			0	
S5			12359	
S7			3357	
S8			1216	
SA			9199	
SB			11377	
SD			3243	

Roof Level Holdown to: floor below

pper Floor Level Holdown to: concrete

Sheathing Thickness: 15/32 in | Typical Sheathing Nailing: 8d Nails (uno) | (IBC 2306.3)

Shearwalls in Use	Wall Capacity (plf)
Wall Type A: 15/32" APA Rated Sheathing w/8d At 6" o.c. edges, 12" o.c. field	260
Wall Type B: 15/32" APA Rated Sheathing w/8d At 4" o.c. edges, 12" o.c. field	380
Wall Type C: 15/32" APA Rated Sheathing w/8d At 3" o.c. edges, 12" o.c. field	490
Wall Type D: 15/32" APA Rated Sheathing w/8d At 2" o.c. edges, 12" o.c. field	640

Simpson Holdowns and Anchors in Use; (Verify Fdn. Config. w/ Loads Listed in Catalog for Anchor Bolt Types)		Seismic Capacity (lb)	Wind Capacity (lb)
#0	(No Holdown Required)	0	0
#1	Holdown Type MSTC28 w/ (6) 16d sinkers at each end	1155	1155
#2	Holdown Type MSTC40 w/ (14) 16d sinkers at each end	2695	2695
#4	Holdown Type MSTC66/78 w/ (38) 16d sinkers at each end	5860	5860
#6	Holdown Type HDU2-SDS2.5 w/ (6) 1/4" dia. x 2 1/2" SDS Screws and SSTB16 Anchor Bolt	2550	3075
#9	Holdown Type HDU8-SDS2.5 w/ (20) 1/4" dia. x 2 1/2" SDS Screws and SSTB28 Anchor Bolt	5980	5980
#12	Holdown Type HDU14-SDS2.5 w/ (36) 1/4" dia. x 2 1/2" SDS Screws and PAB8x30 Anchor Bolt	14375	14375

Sill Plate to Concrete Anchorage:	2x plate	3x min. plate
5/8 " dia. anchor capacity (lb) =	860	1070
Wall Type A: 0.625" dia. anchors spaced at	48" o.c.	48" o.c.
Wall Type B: 0.625" dia. anchors spaced at	43" o.c.	48" o.c.
Wall Type C: 0.625" dia. anchors spaced at	34" o.c.	42" o.c.
Wall Type D: 0.625" dia. anchors spaced at	3x req.	32" o.c.

(anchor spacing based from values from NDS using hemfir and a 1.6 duration increase)

(PL 1/4x3x0'-3" required)

Roof Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
R1	0	0	6433	Wind Uplift (U) =	10	psf

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L, ft:
Overall Seg. (Lo), ft:	3.50	4.00	5.33	7.50		20.33
R1 Wall Height (H1), ft:	12.00	12.00	12.00	12.00		
Trib. 1, ft:	2.00	2.00	2.00			
Rtn. Load (RL), lb:	200	200	200	200		
Trib. 1 Weight (Rw), psf:	18.00	18.00	18.00	18.00		
Wall Weight (Ww), psf:	120.00	120.00	120.00	120.00		
Seismic, plf:	316	316	316	316		*= 6433 lb / 20.33'*
w/ H/W, plf:	542	474	356			*(includes h/w ratio increases)*
M_{OT} (Seismic), ft-lb:	13290	15189	20239	28479		*= (6433 lb / 20.33' * Lo*H1)*
Mr (Seismic), ft-lb:	9741	12608	22032	42000		*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	2336	2142	1627	857		*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	2695	2695	2695	1155		
HD Type:	2	2	2	1		
						15/32" APA Rated Sheathing
Shear critical, plf:	**542**	**474**	**356**	**316**		**Seg. 1: 'D' = 8d At 2" o.c. edges, 12" o.c. field**
HD critical, lb:	**2336**	**2142**	**1627**	**857**		**Seg. 2: 'C' = 8d At 3" o.c. edges, 12" o.c. field**
40% increase for wind?:	No	No	No	No		**Seg. 3: 'B' = 8d At 4" o.c. edges, 12" o.c. field**
Sheathing Layers:	Single	Single	Single	Single		**Seg. 4: 'B' = 8d At 4" o.c. edges, 12" o.c. field**
Concrete Anch.?	No	No	No	No		
Wall Type:	**D**	**C**	**B**	**B**		**Holdown Types:**
Wall Capacity, plf:	640	490	380	380		**Seg. 1: '2' = MSTC40 with (14) 16d sinkers at each end**
Controlling HD Type:	**2**	**2**	**2**	**1**		**Seg. 2: '2' = MSTC40 with (14) 16d sinkers at each end**
Holdown Capacity, lb:	2695	2695	2695	1155		**Seg. 3: '2' = MSTC40 with (14) 16d sinkers at each end**
3x Members req.:	Yes	No	No	No		**Seg. 4: '1' = MSTC28 with (6) 16d sinkers at each end**

Roof Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
R2 (N)	0	0	5151	Wind Uplift (U) =	10	psf

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:
Overall Seg. (Lo), ft:	19.00					19.00
: (N) Wall Height (H1), ft:	12.00					
Trib. 1, ft:	2.00					
Rtn. Load (RL), lb:	200					
Trib. 1 Weight (Rw), psf:	18.00					
Wall Weight (Ww), psf:	8.00					
Seismic, plf:	271					*= 5151 lb / 19'*
M_{OT} (Seismic), ft-lb:	61812					*= (5151 lb / 19' * Lo*H1)*
Mr (Seismic), ft-lb:	27626					*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	2490					*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	2695					
HD Type:	2					
						15/32" APA Rated Sheathing
Shear critical, plf:	**271**					**Seg. 1: 'B' = 8d At 4" o.c. edges, 12" o.c. field**
HD critical, lb:	**2490**					
40% increase for wind?:	No					
Sheathing Layers:	Single					
Concrete Anch.?	No					
Wall Type:	**B**					**Holdown Types:**
Wall Capacity, plf:	380					**Seg. 1: '2' = MSTC40 with (14) 16d sinkers at each end**
Controlling HD Type:	**2**					
Holdown Capacity, lb:	2695					
3x Members req.:	No					

Roof Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
R2 (S)	0	0	6599	Wind Uplift (U) =		10	psf
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:	
Overall Seg. (Lo), ft:	23.25					23.25	
2 (S) Wall Height (H1), ft:	12.00						
Trib. 1, ft:	2.00						
Rtn. Load (RL), lb:	200						
Trib. 1 Weight (Rw), psf:	18.00						
Wall Weight (Ww), psf:	8.00						
Seismic, plf:	284					*= 6599 lb / 23.25'*	
M_{OT} (Seismic), ft-lb:	79188					*= (6599 lb / 23.25' * Lo*H1)*	
Mr (Seismic), ft-lb:	40327					*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	2495					*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:	2695						
HD Type:	2						
						15/32" APA Rated Sheathing	
Shear critical, plf:	**284**					Seg. 1: **'B' = 8d At 4" o.c. edges, 12" o.c. field**	
HD critical, lb:	**2495**						
40% increase for wind?:	No						
Sheathing Layers:	Single						
Concrete Anch.?	No						
Wall Type:	**B**					**Holdown Types:**	
Wall Capacity, plf:	380					Seg. 1: **'2' = MSTC40 with (14) 16d sinkers at each end**	
Controlling HD Type:	**2**						
Holdown Capacity, lb:	2695						
3x Members req.:	No						

Roof Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
R4	0	0	8535	Wind Uplift (U) =		10	psf
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:	
Overall Seg. (Lo), ft:	6.25	14.00				20.25	
R4 Wall Height (H1), ft:	12.00	12.00					
Trib. 1, ft:	4.00	4.00					
Rtn. Load (RL), lb:	200	200					
Trib. 1 Weight (Rw), psf:	18.00	18.00					
Wall Weight (Ww), psf:	8.00	8.00					
Seismic, plf:	421	421				*= 8535 lb / 20.25'*	
M_{OT} (Seismic), ft-lb:	31611	70809				*= (8535 lb / 20.25' * Lo*H1)*	
Mr (Seismic), ft-lb:	4531	19264				*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	4677	4335				*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:	5860	5860					
HD Type:	4	4					
						15/32" APA Rated Sheathing	
Shear critical, plf:	**421**	**421**				Seg. 1: **'C' = 8d At 3" o.c. edges, 12" o.c. field**	
HD critical, lb:	**4677**	**4335**				Seg. 2: **'C' = 8d At 3" o.c. edges, 12" o.c. field**	
40% increase for wind?:	No	No					
Sheathing Layers:	Single	Single					
Concrete Anch.?	No	No					
Wall Type:	**C**	**C**				**Holdown Types:**	
Wall Capacity, plf:	490	490				Seg. 1: **'4' = MSTC66/78 with (38) 16d sinkers at each end**	
Controlling HD Type:	**4**	**4**				Seg. 2: **'4' = MSTC66/78 with (38) 16d sinkers at each end**	
Holdown Capacity, lb:	5860	5860					
3x Members req.:	Yes	Yes					

Roof Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
R7	0	0	5385	Wind Uplift (U) =	10	psf

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:
Overall Seg. (Lo), ft:	12.83					12.83
R7 Wall Height (H1), ft:	12.00					
Trib. 1, ft:	4.00					
Rtn. Load (RL), lb:	200					
Trib. 1 Weight (Rw), psf:	18.00					
Wall Weight (Ww), psf:	120.00					
Seismic, plf:	420					*= 5385 lb / 12.83'*
M_{OT} (Seismic), ft-lb:	64635					*= (5385 lb / 12.83' * Lo*H1)*
Mr (Seismic), ft-lb:	127069					*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	-162					*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	0					
HD Type:	0					
						15/32" APA Rated Sheathing
Shear critical, plf:	**420**					Seg. 1: **'C' = 8d At 3" o.c. edges, 12" o.c. field**
HD critical, lb:	**0**					
40% increase for wind?:	No					
Sheathing Layers:	Single					
Concrete Anch.?	No					
Wall Type:	**C**					**Holdown Types:**
Wall Capacity, plf:	490					Seg. 1: **'0' no holdown required**
Controlling HD Type:	**0**					
Holdown Capacity, lb:	0					
3x Members req.:	Yes					

Roof Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
R8	0	0	2068	Wind Uplift (U) =	10	psf

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:
Overall Seg. (Lo), ft:	4.50	6.00	10.50			21.00
R8 Wall Height (H1), ft:	12.00	12.00	12.00			
Trib. 1, ft:	4.00	4.00	4.00			
Rtn. Load (RL), lb:	200	200	200			
Trib. 1 Weight (Rw), psf:	18.00	18.00	18.00			
Wall Weight (Ww), psf:	8.00	8.00	8.00			
Seismic, plf:	98	98	98			*= 2068 lb / 21'*
w/ H/W, plf:	131					*(includes h/w ratio increases)*
M_{OT} (Seismic), ft-lb:	5318	7090	12408			*= (2068 lb / 21' * Lo*H1)*
Mr (Seismic), ft-lb:	2601	4224	11361			*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	878	812	614			*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	1155	1155	1155			
HD Type:	1	1	1			
						15/32" APA Rated Sheathing
Shear critical, plf:	**131**	**98**	**98**			Seg. 1: **'A' = 8d At 6" o.c. edges, 12" o.c. field**
HD critical, lb:	**878**	**812**	**614**			Seg. 2: **'A' = 8d At 6" o.c. edges, 12" o.c. field**
40% increase for wind?:	No	No	No			Seg. 3: **'A' = 8d At 6" o.c. edges, 12" o.c. field**
Sheathing Layers:	Single	Single	Single			
Concrete Anch.?	No	No	No			
Wall Type:	**A**	**A**	**A**			**Holdown Types:**
Wall Capacity, plf:	260	260	260			Seg. 1: **'1' = MSTC28 with (6) 16d sinkers at each end**
Controlling HD Type:	**1**	**1**	**1**			Seg. 2: **'1' = MSTC28 with (6) 16d sinkers at each end**
Holdown Capacity, lb:	1155	1155	1155			Seg. 3: **'1' = MSTC28 with (6) 16d sinkers at each end**
3x Members req.:	No	No	No			

Roof Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
RA	0	0	15504	Wind Uplift (U) =	10	psf

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:
Overall Seg. (Lo), ft:	13.25	15.83	18.33			47.41
RA Wall Height (H1), ft:	12.00	12.00	12.00			
Trib. 1, ft:	4.00	4.00	4.00			
Rtn. Load (RL), lb:	200	200	200			
Trib. 1 Weight (Rw), psf:	18.00	18.00	18.00			
Wall Weight (Ww), psf:	120.00	120.00	120.00			
Seismic, plf:	327	327	327			*= 15504 lb / 47.41'*
M_{OT} (Seismic), ft-lb:	51996	62132	71931			*= (15504 lb / 47.41' * Lo*H1)*
Mr (Seismic), ft-lb:	135375	192684	257674			*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	-1440	-2465	-3456			*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	0	0	0			
HD Type:	0	0	0			
						15/32" APA Rated Sheathing
Shear critical, plf:	**327**	**327**	**327**			Seg. 1: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
HD critical, lb:	**0**	**0**	**0**			Seg. 2: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
40% increase for wind?:	No	No	No			Seg. 3: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
Sheathing Layers:	Single	Single	Single			
Concrete Anch.?	No	No	No			
Wall Type:	**B**	**B**	**B**			**Holdown Types:**
Wall Capacity, plf:	380	380	380			Seg. 1: **'0' no holdown required**
Controlling HD Type:	**0**	**0**	**0**			Seg. 2: **'0' no holdown required**
Holdown Capacity, lb:	0	0	0			Seg. 3: **'0' no holdown required**
3x Members req.:	No	No	No			

Roof Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
RB	0	0	17125	Wind Uplift (U) =	10	psf

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:
Overall Seg. (Lo), ft:	13.33	14.83	20.25			48.41
RB Wall Height (H1), ft:	12.00	12.00	12.00			
Trib. 1, ft:	4.00	4.00	4.00			
Rtn. Load (RL), lb:	200	200	200			
Trib. 1 Weight (Rw), psf:	18.00	18.00	18.00			
Wall Weight (Ww), psf:	120.00	120.00	120.00			
Seismic, plf:	354	354	354			*= 17125 lb / 48.41'*
M_{OT} (Seismic), ft-lb:	56586	62966	85961			*= (17125 lb / 48.41' * Lo*H1)*
Mr (Seismic), ft-lb:	136999	169300	314057			*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	-1151	-1747	-3897			*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	0	0	0			
HD Type:	0	0	0			
						15/32" APA Rated Sheathing
Shear critical, plf:	**354**	**354**	**354**			Seg. 1: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
HD critical, lb:	**0**	**0**	**0**			Seg. 2: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
40% increase for wind?:	No	No	No			Seg. 3: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
Sheathing Layers:	Single	Single	Single			
Concrete Anch.?	No	No	No			
Wall Type:	**B**	**B**	**B**			**Holdown Types:**
Wall Capacity, plf:	380	380	380			Seg. 1: **'0' no holdown required**
Controlling HD Type:	**0**	**0**	**0**			Seg. 2: **'0' no holdown required**
Holdown Capacity, lb:	0	0	0			Seg. 3: **'0' no holdown required**
3x Members req.:	Yes	Yes	Yes			

Roof Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
RC	0	0	1621	Wind Uplift (U) =	10	psf	
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total L:	
Overall Seg. (Lo), ft:	6.00	7.00	21.00			34.00	
RC Wall Height (H1), ft:	12.00	12.00	12.00				
Trib. 1, ft:	4.00	4.00	4.00				
Rtn. Load (RL), lb:	200	200	200				
Trib. 1 Weight (Rw), psf:	18.00	18.00	18.00				
Wall Weight (Ww), psf:	80.00	80.00	80.00				
Seismic, plf:	48	48	48			*= 1621 lb / 34'*	
M_{OT} (Seismic), ft-lb:	3433	4005	12014			*= (1621 lb / 34' * Lo*H1)*	
Mr (Seismic), ft-lb:	19776	26684	231756			*= (Trib. 1*Rw +H1*Ww)*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	-1158	-1429	-5222			*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:	0	0	0				
HD Type:	0	0	0				
						15/32" APA Rated Sheathing	
Shear critical, plf:	**48**	**48**	**48**			Seg. 1: **'A' = 8d At 6" o.c. edges, 12" o.c. field**	
HD critical, lb:	**0**	**0**	**0**			Seg. 2: **'A' = 8d At 6" o.c. edges, 12" o.c. field**	
40% increase for wind?:	No	No	No			Seg. 3: **'A' = 8d At 6" o.c. edges, 12" o.c. field**	
Sheathing Layers:	Single	Single	Single				
Concrete Anch.?	No	No	No				
Wall Type:	**A**	**A**	**A**			**Holdown Types:**	
Wall Capacity, plf:	260	260	260			Seg. 1: **'0' no holdown required**	
Controlling HD Type:	**0**	**0**	**0**			Seg. 2: **'0' no holdown required**	
Holdown Capacity, lb:	0	0	0			Seg. 3: **'0' no holdown required**	
3x Members req.:	No	No	No				

Upper Floor Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
S1	0	0	4254	Wind Uplift (U) =	10.00	psf	
Load from R1, lb:	0	0	6433	Upper Flr Depth (Fd) =	1.00	ft	
Total, lb:	0	0	10687				
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:	
Overall Seg. (Lo), ft:	3.50					3.50	
R1 Wall Height (H1), ft:	12.00						
S1 Wall Height (H2), ft:	11.75						
Trib. 1, ft:	2.00						
Trib. 2, ft:	4.00						
Rtn. Load (RL), lb:	200						
Trib. 1 Weight (Rw), psf:	18.00						
Trib. 2 Weight (Uw), psf:	15.00						
Wall Weight (Ww), psf:	120.00						
Seismic, plf:	3053					*= (4254 lb + 6433 lb) / 3.5'*	
w/ H/W, plf:	5125					*(includes h/w ratio increases)*	
M_{OT} (Seismic), ft-lb:	209201					*= (4254 lb / 3.5 *Lo*H2)+(6433 lb / 3.5' *Lo*(H1+H2+Fd))*	
Mr (Seismic), ft-lb:	18744					*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	56960					*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:							
HD Type:	NG						
						15/32" APA Rated Sheathing	
Shear critical, plf:	**5125**						
HD critical, lb:	**56960**						
40% increase for wind?:	No						
Sheathing Layers:	Single						
Concrete Anch.?	Yes						
Wall Type:	**Dbl. Req.**					**Holdown Types:**	
Wall Capacity, plf:						Seg. 1: **'NG' larger holdown required**	
Controlling HD Type:	**NG**						
Holdown Capacity, lb:							
3x Members req.:	Yes						

Upper Floor Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
S2 (N)	0	0	3297	Wind Uplift (U) =	10	psf	
Load from R2 (N), lb:	0	0	5151	Upper Flr Depth (Fd) =	1.00	ft	
Total, lb:	0	0	8448				
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:	
Overall Seg. (Lo), ft:	14.33					14.33	
? (N) Wall Height (H1), ft:	12.00						
? (N) Wall Height (H2), ft:	11.75						
Trib. 1, ft:	2.00						
Trib. 2, ft:	4						
Rtn. Load (RL), lb:	200						
Trib. 1 Weight (Rw), psf:	18.00						
Trib. 2 Weight (Uw), psf:	15.00						
Wall Weight (Ww), psf:	8.00						
Seismic, plf:	590					*= (3297 lb + 5151 lb) / 14.33'*	
M_{OT} (Seismic), ft-lb:	166227					*= (3297 lb / 14.33 *Lo*H2)+(5151 lb / 14.33' *Lo*(H1+H2+Fd))*	
Mr (Seismic), ft-lb:	32231					*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	10419					*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:	14375						
HD Type:	12						
						15/32" APA Rated Sheathing	
Shear critical, plf:	**590**					Seg. 1: **'D' = 8d At 2" o.c. edges, 12" o.c. field**	
HD critical, lb:	**10419**						
40% increase for wind?:	No						
Sheathing Layers:	Single						
Concrete Anch.?	Yes						
Wall Type:	**D**					**Holdown Types:**	
Wall Capacity, plf:	640					Seg. 1: **'12' = HDU14-SDS2.5 and PAB8x30 Anchor Bolt**	
Controlling HD Type:	**12**						
Holdown Capacity, lb:	14375						
3x Members req.:	Yes						

Upper Floor Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
S2 (S)	0	0	5935	Wind Uplift (U) =	10	psf
Load from R2 (S), lb:	0	0	6599	Upper Flr Depth (Fd) =	1.00	ft
Total, lb:	0	0	12534			

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:
Overall Seg. (Lo), ft:	10.33	14.33				24.66
2 (S) Wall Height (H1), ft:	12.00	12.00				
2 (S) Wall Height (H2), ft:	11.75	11.75				
Trib. 1, ft:	2.00	2.00				
Trib. 2, ft:	4	4				
Rtn. Load (RL), lb:	200	200				
Trib. 1 Weight (Rw), psf:	18.00	18.00				
Trib. 2 Weight (Uw), psf:	15.00	15.00				
Wall Weight (Ww), psf:	8.00	8.00				
Seismic, plf:	508	508				*= (5935 lb + 6599 lb) / 24.66'*
M_{OT} (Seismic), ft-lb:	97629	135433				*= (5935 lb / 24.66 *Lo*H2)+(6599 lb / 24.66' *Lo*(H1+H2+Fd))*
Mr (Seismic), ft-lb:	17325	32231				*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	8571	8270				*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	14375	8315				
HD Type:	12	11				
						15/32" APA Rated Sheathing
Shear critical, plf:	**508**	**508**				Seg. 1: **'D' = 8d At 2" o.c. edges, 12" o.c. field**
HD critical, lb:	**8571**	**8270**				Seg. 2: **'D' = 8d At 2" o.c. edges, 12" o.c. field**
40% increase for wind?:	No	No				
Sheathing Layers:	Single	Single				
Concrete Anch.?	Yes	Yes				
Wall Type:	**D**	**D**				**Holdown Types:**
Wall Capacity, plf:	640	640				Seg. 1: **'12' = HDU14-SDS2.5 and PAB8x30 Anchor Bolt**
Controlling HD Type:	**12**	**11**				Seg. 2: **'11' = HHDQ11-SDS2.5 and SB1x30 Anchor Bolt**
Holdown Capacity, lb:	14375	8315				
3x Members req.:	Yes	Yes				

Upper Floor Level	0.2Sds =	0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb			
S5	0	0	12359	Wind Uplift (U) =	10	psf
Load from R5, lb:	0	0	0	Upper Flr Depth (Fd) =	1.00	ft
Total, lb:	0	0	12359			

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:
Overall Seg. (Lo), ft:	10.33	14.33				24.66
R5 Wall Height (H1), ft:	12.00	12.00				
S5 Wall Height (H2), ft:	11.75	11.75				
Trib. 1, ft:	0.00	0.00				
Trib. 2, ft:	4.00	4.00				
Rtn. Load (RL), lb:	200	200				
Trib. 1 Weight (Rw), psf:	18.00	18.00				
Trib. 2 Weight (Uw), psf:	15.00	15.00				
Wall Weight (Ww), psf:	8.00	8.00				
Seismic, plf:	501	501				*= (12359 lb + 0 lb) / 24.66'*
M_{OT} (Seismic), ft-lb:	60831	84387				*= (12359 lb / 24.66 *Lo*H2)+(0 lb / 24.66' *Lo*(H1+H2+Fd))*
Mr (Seismic), ft-lb:	15405	28535				*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	5106	4843				*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	5980	5980				
HD Type:	9	9				
						15/32" APA Rated Sheathing
Shear critical, plf:	**501**	**501**				Seg. 1: **'D' = 8d At 2" o.c. edges, 12" o.c. field**
HD critical, lb:	**5106**	**4843**				Seg. 2: **'D' = 8d At 2" o.c. edges, 12" o.c. field**
40% increase for wind?:	No	No				
Sheathing Layers:	Single	Single				
Concrete Anch.?	Yes	Yes				
Wall Type:	**D**	**D**				**Holdown Types:**
Wall Capacity, plf:	640	640				Seg. 1: **'9' = HDU8-SDS2.5 and SSTB28 Anchor Bolt**
Controlling HD Type:	**9**	**9**				Seg. 2: **'9' = HDU8-SDS2.5 and SSTB28 Anchor Bolt**
Holdown Capacity, lb:	5980	5980				
3x Members req.:	Yes	Yes				

Upper Floor Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
S7	0	0	3357	Wind Uplift (U) =	10	psf	
Load from R7, lb:	0	0	5385	Upper Flr Depth (Fd) =	1.00	ft	
Total, lb:	0	0	8742				

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:
Overall Seg. (Lo), ft:	13.00	14.50				27.50
R7 Wall Height (H1), ft:	12.00	10.33				
S7 Wall Height (H2), ft:	11.75	11.75				
Trib. 1, ft:	4.00	4.00				
Trib. 2, ft:	4.00	4.00				
Rtn. Load (RL), lb:	200	200				
Trib. 1 Weight (Rw), psf:	18.00	18.00				
Trib. 2 Weight (Uw), psf:	15.00	15.00				
Wall Weight (Ww), psf:	120.00	120.00				
Seismic, plf:	318	318				*= (3357 lb + 5385 lb) / 27.5'*
M_{OT} (Seismic), ft-lb:	81651	86331				*= (3357 lb / 27.5 *Lo*H2)+(5385 lb / 27.5' *Lo*(H1+H2+Fd))*
Mr (Seismic), ft-lb:	254579	295316				*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	-4000	-4739				*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	0	0				
HD Type:	0	0				
						15/32" APA Rated Sheathing
Shear critical, plf:	**318**	**318**				Seg. 1: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
HD critical, lb:	**0**	**0**				Seg. 2: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
40% increase for wind?:	No	No				
Sheathing Layers:	Single	Single				
Concrete Anch.?	Yes	Yes				
Wall Type:	**B**	**B**				**Holdown Types:**
Wall Capacity, plf:	380	380				Seg. 1: **'0' no holdown required**
Controlling HD Type:	**0**	**0**				Seg. 2: **'0' no holdown required**
Holdown Capacity, lb:	0	0				
3x Members req.:	No	No				

Upper Floor Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
S8	0	0	1216	Wind Uplift (U) =	10	psf	
Load from R8, lb:	0	0	2068	Upper Flr Depth (Fd) =	1.00	ft	
Total, lb:	0	0	3284				

	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:
Overall Seg. (Lo), ft:	10.33					10.33
R8 Wall Height (H1), ft:	12.00					
S8 Wall Height (H2), ft:	11.75					
Trib. 1, ft:	4.00					
Trib. 2, ft:	4.00					
Rtn. Load (RL), lb:	200					
Trib. 1 Weight (Rw), psf:	18.00					
Trib. 2 Weight (Uw), psf:	15.00					
Wall Weight (Ww), psf:	80.00					
Seismic, plf:	318					*= (1216 lb + 2068 lb) / 10.33'*
M_{OT} (Seismic), ft-lb:	65471					*= (1216 lb / 10.33 *Lo*H2)+(2068 lb / 10.33' *Lo*(H1+H2+Fd))*
Mr (Seismic), ft-lb:	110482					*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*
HD (Seismic), lb:	723					*= (Mot - (0.6-0.075)*Mr) / Lo*
Holdown Capacity, lb:	2550					
HD Type:	6					
						15/32" APA Rated Sheathing
Shear critical, plf:	**318**					Seg. 1: **'B' = 8d At 4" o.c. edges, 12" o.c. field**
HD critical, lb:	**723**					
40% increase for wind?:	No					
Sheathing Layers:	Single					
Concrete Anch.?	Yes					
Wall Type:	**B**					**Holdown Types:**
Wall Capacity, plf:	380					Seg. 1: **'6' = HDU2-SDS2.5 and SSTB16 Anchor Bolt**
Controlling HD Type:	**6**					
Holdown Capacity, lb:	2550					
3x Members req.:	No					

Upper Floor Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
SA	0	0	9199	Wind Uplift (U) =		10	psf
Load from RA, lb:	0	0	15504	Upper Flr Depth (Fd) =		1.00	ft
Total, lb:	0	0	24703				
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:	
Overall Seg. (Lo), ft:	4.83	5.00	6.50	6.83		23.17	
RA Wall Height (H1), ft:	12.00	12.00	12.00	12.00			
SA Wall Height (H2), ft:	11.75	11.75	11.75	11.75			
Trib. 1, ft:	4.00	4.00	4.00	4.00			
Trib. 2, ft:	4.00	4.00	4.00	4.00			
Rtn. Load (RL), lb:	200	200	200	200			
Trib. 1 Weight (Rw), psf:	18.00	18.00	18.00	18.00			
Trib. 2 Weight (Uw), psf:	15.00	15.00	15.00	15.00			
Wall Weight (Ww), psf:	120.00	120.00	120.00	120.00			
Seismic, plf:	1066	1066	1066	1066		*= (9199 lb + 15504 lb) / 23.17'*	
w/ H/W, plf:	1296	1253				*(includes h/w ratio increases)*	
M_{OT} (Seismic), ft-lb:	102586	106131	137971	145039		*= (9199 lb / 23.17 *Lo*H2)+(15504 lb / 23.17' *Lo*(H1+H2+Fd))*	
Mr (Seismic), ft-lb:	35793	38275	64295	70981		*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	17338	17207	16033	15773		*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:							
HD Type:	NG	NG	NG	NG			
						15/32" APA Rated Sheathing	
Shear critical, plf:	**1296**	**1253**	**1066**	**1066**			
HD critical, lb:	**17338**	**17207**	**16033**	**15773**			
40% increase for wind?:	No	No	No	No			
Sheathing Layers:	Single	Single	Single	Single			
Concrete Anch.?	Yes	Yes	Yes	Yes			
Wall Type:	**Dbl. Req.**	**Dbl. Req.**	**Dbl. Req.**	**Dbl. Req.**		**Holdown Types:**	
Wall Capacity, plf:						Seg. 1: **'NG' larger holdown required**	
Controlling HD Type:	**NG**	**NG**	**NG**	**NG**		Seg. 2: **'NG' larger holdown required**	
Holdown Capacity, lb:						Seg. 3: **'NG' larger holdown required**	
3x Members req.:	Yes	Yes	Yes	Yes		Seg. 4: **'NG' larger holdown required**	

Upper Floor Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
SB	0	0	11377	Wind Uplift (U) =		10	psf
Load from RB, lb:	0	0	17125	Upper Flr Depth (Fd) =		1.00	ft
Total, lb:	0	0	28502				
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:	
Overall Seg. (Lo), ft:	11.00	12.00	20.00	24.50		67.50	
RB Wall Height (H1), ft:	12.00	12.00	12.00	12.00			
SB Wall Height (H2), ft:	11.75	11.75	11.75	11.75			
Trib. 1, ft:	4.00	4.00	4.00	4.00			
Trib. 2, ft:	4.00	4.00	4.00	4.00			
Rtn. Load (RL), lb:	200	200	200	200			
Trib. 1 Weight (Rw), psf:	18.00	18.00	18.00	18.00			
Trib. 2 Weight (Uw), psf:	15.00	15.00	15.00	15.00			
Wall Weight (Ww), psf:	120.00	120.00	120.00	120.00			
Seismic, plf:	422	422	422	422		*= (11377 lb + 17125 lb) / 67.5'*	
M_{OT} (Seismic), ft-lb:	90856	99115	165192	202360		*= (11377 lb / 67.5 *Lo*H2)+(17125 lb / 67.5' *Lo*(H1+H2+Fd))*	
Mr (Seismic), ft-lb:	182611	217104	600400	899873		*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	-456	-1239	-7501	-11023		*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:	0	0	0	0			
HD Type:	0	0	0	0			
						15/32" APA Rated Sheathing	
Shear critical, plf:	**422**	**422**	**422**	**422**		Seg. 1: **'C' = 8d At 3" o.c. edges, 12" o.c. field**	
HD critical, lb:	**0**	**0**	**0**	**0**		Seg. 2: **'C' = 8d At 3" o.c. edges, 12" o.c. field**	
40% increase for wind?:	No	No	No	No		Seg. 3: **'C' = 8d At 3" o.c. edges, 12" o.c. field**	
Sheathing Layers:	Single	Single	Single	Single		Seg. 4: **'C' = 8d At 3" o.c. edges, 12" o.c. field**	
Concrete Anch.?	Yes	Yes	Yes	Yes			
Wall Type:	**C**	**C**	**C**	**C**		**Holdown Types:**	
Wall Capacity, plf:	490	490	490	490		Seg. 1: **'0' no holdown required**	
Controlling HD Type:	**0**	**0**	**0**	**0**		Seg. 2: **'0' no holdown required**	
Holdown Capacity, lb:	0	0	0	0		Seg. 3: **'0' no holdown required**	
3x Members req.:	Yes	Yes	Yes	Yes		Seg. 4: **'0' no holdown required**	

Upper Floor Level	0.2Sds =	0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb				
SD	0	0	3243	Wind Uplift (U) =		10	psf
Load from RC, lb:	0	0	0	Upper Flr Depth (Fd) =		1.00	ft
Total, lb:	0	0	3243				
	Segment 1	Segment 2	Segment 3	Segment 4	Segment 5	Total Lo:	
Overall Seg. (Lo), ft:	20.00					20.00	
RC Wall Height (H1), ft:	12.00						
SD Wall Height (H2), ft:	11.75						
Trib. 1, ft:	4.00						
Trib. 2, ft:	4.00						
Rtn. Load (RL), lb:	200						
Trib. 1 Weight (Rw), psf:	18.00						
Trib. 2 Weight (Uw), psf:	15.00						
Wall Weight (Ww), psf:	8.00						
Seismic, plf:	162					*= (3243 lb + 0 lb) / 20'*	
M_{OT} (Seismic), ft-lb:	38105					*= (3243 lb / 20 *Lo*H2)+(0 lb / 20' *Lo*(H1+H2+Fd))*	
Mr (Seismic), ft-lb:	68400					*= (Rw *Trib. 1+ Uw *Trib. 2+ Ww *(H1+H2))*(Lo)^2/2+RL*Lo*	
HD (Seismic), lb:	110					*= (Mot - (0.6-0.075)*Mr) / Lo*	
Holdown Capacity, lb:	2550						
HD Type:	6						
						15/32" APA Rated Sheathing	
Shear critical, plf:	**162**					Seg. 1: **'A' = 8d At 6" o.c. edges, 12" o.c. field**	
HD critical, lb:	**110**						
40% increase for wind?:	No						
Sheathing Layers:	Single						
Concrete Anch.?	Yes						
Wall Type:	**A**					**Holdown Types:**	
Wall Capacity, plf:	260					Seg. 1: **'6' = HDU2-SDS2.5 and SSTB16 Anchor Bolt**	
Controlling HD Type:	**6**						
Holdown Capacity, lb:	2550						
3x Members req.:	No						

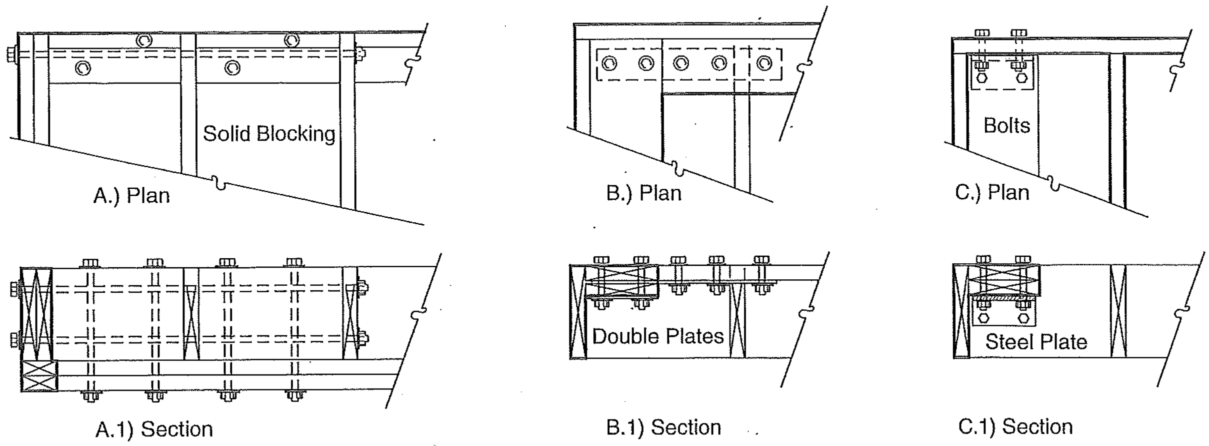


Figure 10.14 Typical corner details

TRANSVERSELY SHEATHED DIAPHRAGMS (Douglas Fir-Larch Lumber) Table **10.1**

Maximum Span-Width or Height-Width Ratio: Horizontal Diaphragms Restraining: Masonry or Concrete Walls	Horizontal Diaphragms Restraining: Wood or Similar Walls	Vertical Diaphragms	Nominal Width of Sheathing Boards (inches)	Number of 8d Common Nails per Board per Crossing of Stud, Joist, or Perimeter Member and at Butted Ends	Allowable Lateral Shear Load lbs. per lineal ft. for Stud or Joist Spacing in inches of: 12	16	24
Not recommended	Limited by acceptable deflection of wall	Limited by acceptable deflection	6	2	100	75	50
			8	2	114	86	57
			10	2	124	93	62

DIAGONALLY SHEATHED DIAPHRAGMS

Maximum Span-Width or Height-Width Ratio: Horizontal Diaphragms Restraining: Masonry or Concrete Walls	Horizontal Diaphragms Restraining: Wood or Similar Walls	Vertical Diaphragms	Nominal Width of Sheathing Boards (inches)	Number of 8d Common Nails per Board per Crossing at: Perimeter Members and Butted Ends of Boards	Stud or Joist	Allowable Lateral Shear Load lbs. per lineal ft.
3:1	4:1	2:1	6	2	2	345
			8	2	2	262
				3	2	393

continued

VITUS CONSTRUCTION INC.

January 27, 2016

Mr. Pieter Smeenk
City of Ashland
20 East Main Street
Ashland, OR 97520

SUBJECT: Preliminary Seismic Upgrade, Relocation and Temporary Facilities Construction Budget for Ashland City Hall

Dear Mr. Smeenk:

Vitus Construction, Inc. appreciates the opportunity to provide a preliminary seismic upgrade, relocation and temporary facilities budget for the existing Ashland City Hall. The budget is based on site meetings, Miller Consulting Engineers, Inc. Report (dated 12/07/15) and drawings from Marquess & Associates (dated May 1991) and Savikko Engineering (dated 4/14/85). The following preliminary scope and per square foot cost is being provided to the City of Ashland for budgeting purposes only.

Preliminary Budgeting Items

1. Additional per square foot costs for new roofing, non-structural bracing of suspended ceilings and anchorage of equipment, and piping anchorages to add to the Miller Consulting Engineer's estimate. Also please specifically include temporary relocation costs and soft costs not included in the engineer's estimate.
   a. Miller Consulting Engineers, Inc. published cost — $184.00 Sq Ft
   b. Flashing, bracing, weather proofing and sealants — $18.00 Sq Ft
   c. Temporary operational facilities — $20.00 Sq Ft
      i. Includes relocation, lease / rental of storage, offices, meeting rooms and restrooms occupied by city staff and the public during the construction period (9 months $156,834.00).
   d. Soft Costs — $42.00 Sq Ft
      i. Includes temporary signage, safety protection, parking and structural / architectural drawings for roof and bracing

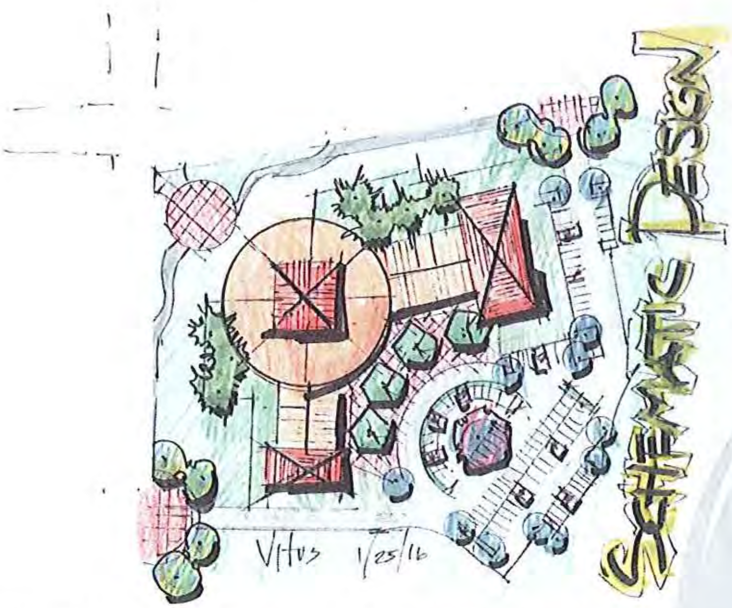
   e. Total — $264.00 Sq Ft

2. Additional per square foot mechanical, electrical, plumbing, egress, ADA, Fire Suppression, and Tenant Improvements costs in total.
   a. Mechanical, electrical and plumbing — $110.00 Sq Ft
   b. ADA upgrades and modifications — $28.00 Sq Ft

c. Fire suppression $15.00 Sq Ft
d. Tenant improvements $185.00 Sq Ft
e. **Total** **$338.00 Sq Ft**

3. Total per square foot cost to construct a new building on the current City Hall site, replacing everything except the existing north and west, historic facades. If the per square foot costs for space added as a new third or fourth floor, please indicate the additional per square foot space cost for that, but if it is not significantly different, please indicate that.
   a. **Rebuild building keeping exterior walls (10,595 Sq Ft) $405.00 Sq FT**

4. Total per square foot cost to construct a new building on a new site on city-owned property, including the cost of parking, site development, building costs and associated elements to relocate.
   a. Site Development $125.00 Sq Ft
   b. Construction of Building $325.00 Sq ft
   c. **Total** **$450.00 Sq Ft**



Drawing is for informational use only Vitus Construction, Inc. does not represent value associated with the site or building illustration.

The above budget proposal will be updated as the project develops. If you have any questions please do not hesitate to contact Vitus Construction, Inc.

Sincerely,

Corey E. Vitus, President
Vitus Construction, Inc.