

Council Communication

February 1, 2016, Study Session

Continued discussion of planning for City Hall replacement

FROM:

Michael R. Faught, Director of Public Works, Public Works, Mike.Faught@ashland.or.us

SUMMARY

The Council at its June 15, 2015, study session requested that the City conduct a seismic evaluation of City Hall to determine the cost of bringing the building up to current seismic codes. Necessary seismic upgrades would involve removing the roof, the floor on the second floor and all of the drywall on the interior of the north and west walls, then building a system of ties and braces to secure the building. The estimated cost (including contingency but excluding soft costs and temporary staff relocation) is \$176/sq. ft., or \$1,363,757. The seismic upgrades will require the relocation of city staff for approximately nine months at a cost of just under \$157,000. Soft costs would be roughly \$322,000. If the City did necessary and long-overdue HVAC and plumbing replacement as well as fire suppression and ADA improvements, the total cost of the project exceeds the cost of simply demolishing and rebuilding City Hall or building a new City Hall at a different location on City-owned property.

BACKGROUND AND POLICY IMPLICATIONS:

At the Council's June 15, 2015, Study Session, City Administrator Dave Kanner proposed a comprehensive examination of options for replacing City Hall using the \$100,000 appropriated in the current budget for a long-term facilities master plan. To that end, Mr. Kanner provided background information on past plans to replace City Hall which included a detailed history of City Hall remodels, additions, spacing needs, and needed seismic mitigation based on a 1994 seismic evaluation (see attached June 15, 2015, council communication).

Based on the age of the existing seismic report, Council directed staff to complete a new seismic study on City Hall and the Parks and Recreation building. The City Hall seismic evaluation has been completed by Miller Consulting Engineers, Inc. (see attached report). The consultant's report outlines a plan with estimated costs to upgrade City Hall to allow occupants to safely exit the building after a major earthquake which, as he indicates in his executive summary, "may not be the case in its current state." The estimated cost to upgrade the existing facility to meet seismic requirements only (excluding soft and relocation costs) is estimated to be \$176 per square foot or a total cost of \$1,363,757. If the City Council were to choose this option, all employees in the existing building would have to be relocated for about nine months at an estimated cost of \$17,426 per month or just under \$157,000.

Given that a seismic renovation of the existing facility requires a major rebuild to the building, Michael Morrison, Public Works Superintendent, recommended evaluating costs associated with modernizing all of the existing (aging) plumbing, HVAC systems and electrical systems. To assist with development of additional planning level cost estimates, staff hired Matt Small, Kistler Small +



White Architects, and John Kennedy, chief estimator with Vitus Construction, to provide economy of scale cost estimates for the following four options (see attached estimate):

1. Include soft cost and Temporary relocations costs to the to the seismic estimate
2. Estimated square foot costs for mechanical, electrical and plumbing, egress, ADA, fire suppression and tenant improvements
3. Estimated square foot costs to rebuild on the current City Hall site, replacing everything except the existing north and west historic facades.
4. Total estimated square foot costs to construct a new building elsewhere on City-Owned property, including parking and other issues arising out of its location.

These costs are shown in table 1.

Table 1:

Seismic Upgrades plus soft costs and relocation costs	Additional cost for mechanical, electrical and plumbing, egress, ADA, Fire Suppression and Tenant Improvements	Total Remodel Costs	Rebuild on the current City hall site replacing everything except the existing north and west wall	New building on City Owned Property
\$236	\$338	\$576	\$405	\$450

*All costs are per-square foot

COUNCIL GOALS:

Organization

- 4 Evaluate real property and City assets to strategically support city mission and goals
- 4.3 Examine city hall replacement and other facility needs.

FISCAL IMPLICATIONS:

The current budget includes \$100,000 to fund a study for City Hall replacement. To date staff has expended \$12,000 for the seismic upgrade analysis and an additional \$1,000 for a building contractor to estimate cost for additional remodeling costs, rebuild in place costs and to build a new building in a different location on city owned property.

STAFF RECOMMENDATION AND REQUESTED ACTION:

Given the seismic report and the need for additional capital improvements at City Hall, staff recommends that the City move forward with a comprehensive examination of options for replacing or rebuilding City Hall. Staff seeks approval to issue an RFP to select a consultant team to examine options for replacing or rebuilding City Hall.

SUGGESTED MOTION:

N/A



ATTACHMENTS:

- March 3, 2014 Staff Report
- June 15, 2015 Staff Report
- Miller Consulting Engineers City Hall Seismic Upgrade Report
- Vitus Construction Inc Cost estimates



Council Communication

March 3, 2014, Study Session

Seismic Structural Improvements to Existing Buildings

FROM:

Michael Grubbs, Building Official, Community Development Department,
michael.grubbs@ashland.or.us

SUMMARY

Community Development staff is interested in developing an amendment to our local building code for City Council's consideration regarding the protection of one of the community's greatest resources, its buildings. Specifically, this would entail amendments to the Ashland Municipal Code, Buildings and Construction section, requiring seismic improvements when buildings undergo remodel or reconstruction. Many buildings within the City of Ashland are vulnerable to the destructive effects of an earthquake. Extreme damage or collapse will result, all due to their age and construction type. If an ordinance is in place, the survival rate of these buildings and the persons who occupy them will greatly be improved.

BACKGROUND AND POLICY IMPLICATIONS:

The major earthquake projected to take place in Oregon sometime in the not so distant future would have a devastating effect on older existing non-code-compliant buildings within the City of Ashland. To preserve Ashland's building stock and provide a safer environment for all, the Building Division proposes consideration of an ordinance which would require structural improvements to existing buildings when a planned alteration or remodel takes place. Such an ordinance reduces the likelihood of severe damage to Ashland's distinctive buildings. Currently, the cities of Medford and Portland have similar ordinances enacted and enforced.

Stakeholder outreach would target local building owners, construction specialists, and interested parties through our local media, Ashland web page, mailings, open public meetings, and email messaging - all designed to inform and allow public input into the crafting of this ordinance.

Facts:

- Scientific studies have identified the Cascadia Subduction zone off the coast as the source of a long history of earthquakes and tsunamis that have affected the State of Oregon.
- Building codes prior to 1976 did not contain adequate provisions for seismic structural safety for structures.
- Older structures in Ashland, including its downtown core are constructed mainly of unreinforced masonry or concrete, which makes them very vulnerable to damage or collapse during an earthquake. Please note: Newer buildings and those currently under construction have been designed with appropriate seismic safeguards in place.



- The current Oregon Structural Specialty Code does not require seismic improvements when a building is remodeled unless a more hazardous use is proposed; seismic upgrades are only voluntary.
- This type of ordinance is endorsed by the State of Oregon Building Codes Division, Ashland Fire Department, the City of Medford and the City of Portland

This new code could include the following triggers for seismic improvement requirements:

- When renovation costs exceed \$100,000 or \$15.00 per square foot.
- When re-roofing is planned, parapet walls need to be braced.
- Change of Occupancy, for example a change from a retail outlet (M-Mercantile) to a restaurant or lounge (A-Assembly).
- T-bar ceiling seismic upgrades required when any work is planned.
- Tilt-up concrete buildings constructed prior to 1998 require the roof system to be evaluated for connection to the walls.
- If interior ceiling materials are removed in single story buildings (thereby exposing the roof wood framing system), the roof system could be required to be evaluated for connection to the walls.
- If interior ceiling materials are removed in multi-story buildings, upper floor systems and roof diaphragm systems required to be evaluated for connection to the walls.
- Most single family homes would be exempt.

Affected buildings that have undergone seismic improvements would provide a higher degree of safety for occupants through added structural strength, thereby greatly improving personal survival. The cities of Portland and Medford have such ordinances in place. In Portland, it is estimated that 20% of all remodeled commercial building stock have been seismically improved.

FISCAL IMPLICATIONS:

There would be minimal fiscal impact to the City of Ashland. Building/business owners would realize a increase in the cost of construction, depending on size and scope of work or size of the building when an alteration or renovation is planned. City of Ashland incentives will be explored and possibly offered to permit applicants.

STAFF RECOMMENDATION AND REQUESTED ACTION:

Instruct staff to continue to research and draft an ordinance for Council consideration.

SUGGESTED MOTION:

N/A

ATTACHMENTS:

Link to the City of Medford Seismic Ordinance: www.ci.medford.or.us/Code.asp?CodeID=3643

Link to the City of Portland Seismic Ordinance: www.portlandonline.com/auditor/index.cfm?c=28673



Council Communication

June 15, 2015, Study Session

Discussion of planning for City Hall replacement

FROM:

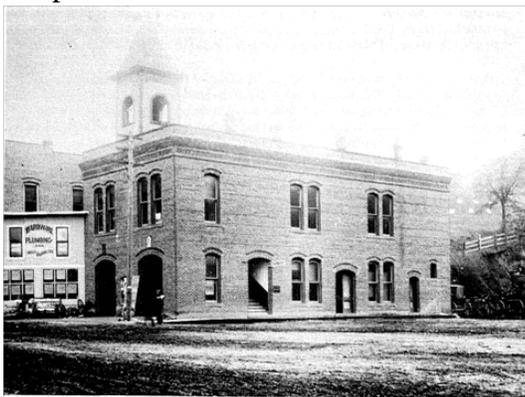
Dave Kanner, City Administrator, dave.kanner@ashland.or.us

SUMMARY

The City has been discussing the reconstruction or relocation of City Hall for more than 20 years. As part of its 2014 strategic planning initiative, the City Council identified “Examine City Hall replacement and other facility needs” as a priority goal. The proposed BN 2015-17 budget included a request for \$200,000 for a study of City Hall replacement, a long-term facilities master plan and a study of underutilized assets. However, the Budget Committee approved only \$100,000. Staff proposes to use this money for a comprehensive examination of options for replacing City Hall and will use this study session for a discussion with the Council on how to proceed with that effort.

BACKGROUND AND POLICY IMPLICATIONS:

Ashland’s City Hall building is more than 120 years old. It was originally built in 1891 as a fire station and expanded in 1913 to include the portion of the building that now houses Administration on the second floor and the Utility Billing lobby and City Recorder’s Office on the first floor. The current stucco veneer was added to the building at that time, however the unreinforced brick masonry walls of the original building continue to form the “skeleton” of City Hall. The photo below left shows the original building. The photo at right is City Hall after it was expanded out to Main Street and the cement stucco exterior was added. At that time, a one-story addition to the east was constructed in the space between City Hall and the adjacent building (now occupied by Utility Billing). A partial second story added to this expansion at an unknown date. Two smaller additions were constructed out of concrete and concrete block at the south end of the building. Also, the interior of the building has been remodeled several times. [Click here to view City Hall historical photos.](#) The newer rear section of the building that now houses the Finance Department was built in two phases, with the second floor completed in 1998.



The City has been discussing the reconstruction or relocation of City Hall for more than 20 years. The building is seismically vulnerable, lacks meeting space and has no room for growth. However, it is the seismic vulnerability of the building that presents the most pressing problem. The interior of City Hall has been reconfigured a number of times since 1913 but has never has any structural improvements related to seismic mitigation.

A history of recent discussions and planning efforts is as follows:

October 1993 – new building approved

In October 1993 the Planning Commission approved a site review for the construction of a 10,000 square foot City office building to be located to the rear of the City Council chambers. In November 1993, the Council sustained the Planning Commission's decision and approved a site review for the construction of the new City office building. Due to community opposition to moving city offices out of the downtown, the City Council withdrew the City's application in December of 1993 for a new building and agreed by resolution to the formation of an ad hoc committee to study the space needs of the City, and to recommend how and when the solutions should be implemented.

1993 – 1994 ad hoc Space Needs Study Committee

An ad hoc Space Needs Study Committee was formed by Resolution 93-41. In April 1994 the committee presented its report to the City Council. The Committee concluded that additional space is needed for city offices located downtown. They recommended that the funds earmarked for the construction of the proposed building at the rear of the City Council Chambers be earmarked for a fund to acquire, maximize and improve space for city offices in the downtown including: acquisition of the Hillah Temple as soon as possible, rehabilitation of the existing City Hall building as soon as possible to include at a minimum, compliance with all current applicable codes, ordinances and energy conservation standards. If acquisition of the Hillah Temple was delayed, the rehabilitation of the existing City Hall to include the addition of a third floor and expansion of the second floor to accommodate all current downtown city employees. The committee also recommended contracting for engineering and architectural services for the purpose of evaluating the existing City Hall building and making recommendations regarding cost and methods to accomplish the rehabilitation. [Click here to read the 1994 ad hoc Space Needs Study Committee report to the City Council.](#)

1994 Seismic Evaluation Report for City Hall

In 1994, the City contracted with Miller-Gardner, Inc., Consulting Engineers to conduct a seismic evaluation of City Hall. The summary reads: Based on the findings of our field investigation and our preliminary calculations, the scope of seismic upgrade of the Ashland City Hall will include repairs or modification to five building components. These consist of bracing the exterior brick wall above the roof (parapet), exterior and interior brick wall lateral ties at the roof, ceiling and floor levels, adding interior shear walls, roof diaphragm reinforcement, and the addition of a braced steel frame inside at the north wall. The estimated total cost of the upgrade including the second floor addition is \$330,000. [Click here to read the 1994 Seismic Report.](#) (Note: None of this recommended work has been done, although the parapet bracing and roof strapping are planned for this summer.)

1996-1997 Space Needs at City Hall

In 1996 the City was unable to reach a mutually acceptable price for the Hillah Temple. The City Council directed staff to pursue the addition of a third story to City Hall and to downsize the proposed building at the East Main site. [Click here to read the August 1997 report to City Council.](#)

In December of 1997 the City Council authorized a Sire Review for the construction of a 7800 square foot City office located at 1175 E. Main Street (behind the City Council Chambers).

1998 – 1999

In October of 1998 the City purchased the Hillah Temple for \$650,360. In September of 1999, the City Council held a public hearing to consider design alternatives for the Hillah Temple building. [Click here to read the Council minutes of September 1999.](#)

A second story was added over the earlier addition at the south end of City Hall. This is now part of Finance and the corner conference room. Use the link to City Hall historical photos to view this addition.

2002

The City contracted with Architectural Design Works to develop a conceptual study for the long term use of City Hall. The study identified the employee space needs, current floor plans and proposed future floor plans. [Click here to view the conceptual designs.](#)

2008

In 2008 the City Council formed the Facilities Master Plan Committee ([read purpose of committee here](#)) and contracted with OgdenRoemerWilkerson to develop a Facilities Master Plan with a long-term vision. [Click here to read the report.](#)

The committee recommended Option 2 which is a combination of existing remodel and new construction, including City Hall. The report did not address the seismic needs of the building. Option 3 identified possible locations to build a new City Hall in the downtown core.

The Council deferred action on the 2008 final report pending completion of a new Fire Station #2 and expansion of the Police Station.

In light of the 20+ year history of City Hall replacement discussions and the Council's priority goal, this communication takes it as a given that the City must move forward with addressing its current and future City Hall needs, both in terms of safety and space. To that end, staff proposes using the \$100,000 budgeted in BN 2015-17 to hire a consultant team for a planning and public involvement process that would – in broad terms – encompass the following steps:

1. Through a public engagement process, determine the key criteria the community would want us to consider when planning for a new City Hall. These criteria could include: civic identity; energy/sustainability; public space; accessibility; reinforce history; mixed uses, etc.
2. Identify locations where those criteria could be met.
3. Analyze the feasibility and pros/cons of each location.
4. Determine the costs and potential financing mechanisms for each option.
5. Council, with community input, selects a preferred alternative.

Staff seeks Council concurrence with this broad process outline and approval to move forward with an RFP to select a consultant team for this project.

COUNCIL GOALS SUPPORTED:

Organization

4. *Evaluate real property and facility assets to strategically support city mission and goals.*
 - 4.3 Examine city hall replacement and other facility needs.

FISCAL IMPLICATIONS:

The approved BN 2015-17 budget includes \$100,000 in one-time money that staff proposes to use for this City Hall study.

STAFF RECOMMENDATION AND REQUESTED ACTION:

Staff seeks approval to move forward with an RFP to select a consultant team for this project.

SUGGESTED MOTION:

N/A

ATTACHMENTS:

None



MILLER
CONSULTING
ENGINEERS

January 27, 2016

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

**Subject: Ashland City Hall Seismic Upgrade
20 East Main Street, Ashland, OR
MCE Project No. 150899**

Dear Mr. Smeenk:

Thank you for the opportunity to provide seismic evaluation services for the City of Ashland. The purpose of this update 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. Based on this review, the following is our preliminary estimate of probable construction cost to upgrade the building:

Performance Objective	Construction Cost	Engineering Fee	Total Cost	Cost per sq. foot
OSSC Code Upgrade	\$1,345,757	\$80,000	\$1,425,757	\$184

Miller Consulting Engineers, Inc. would also like to extend our knowledge, expertise, and passion of structural engineering to the City of Ashland for the work proposed within the seismic evaluation report.

Since our inception in 1978, Miller Consulting Engineers has been providing practical, diverse structural solutions with a history of providing inventive results that save time and money. MCE's team has extensive experience in providing seismic evaluations and code upgrades for a variety of buildings that include essential facilities and historic structures. Our past experience and current technology will be very beneficial in the performance of the seismic upgrade of City Hall. These benefits include:

- Skilled staff in seismic upgrades of historic and essential structures
- Construction cost and engineering cost estimating expertise
- Past experience working closely with the State of Oregon Building Codes
- Knowledge and experience utilizing the *2014 Oregon Structural Specialty Code*
- Experienced staff with the "Partnering/Teaming" process
- Economical and innovative solutions to field problems
- Historic preservation practice
- Outstanding narrative reporting
- AUTOCAD "Standardized Details" for common seismic evaluations/REVIT

If you have any questions about the contents enclosed, you may contact me. I am reachable by phone at 503-246-1250; facsimile at 503-246-1395; or by email at eric@millers-e.com.

Respectfully submitted,

Miller Consulting Engineers, Inc.

Eric Watson, S.E., P.E.
Principal

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Owner-Directed Facility Seismic Evaluation

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



Prepared for
The City of Ashland



EXPIRES: 06-30-2016

Prepared by

MILLER CONSULTING ENGINEERS, INC.
9570 SW Barbur Boulevard, Suite 100
Portland, Oregon 97219
Ph.: 503-246-1250
MCE Project Number: 150899
December 7, 2015

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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

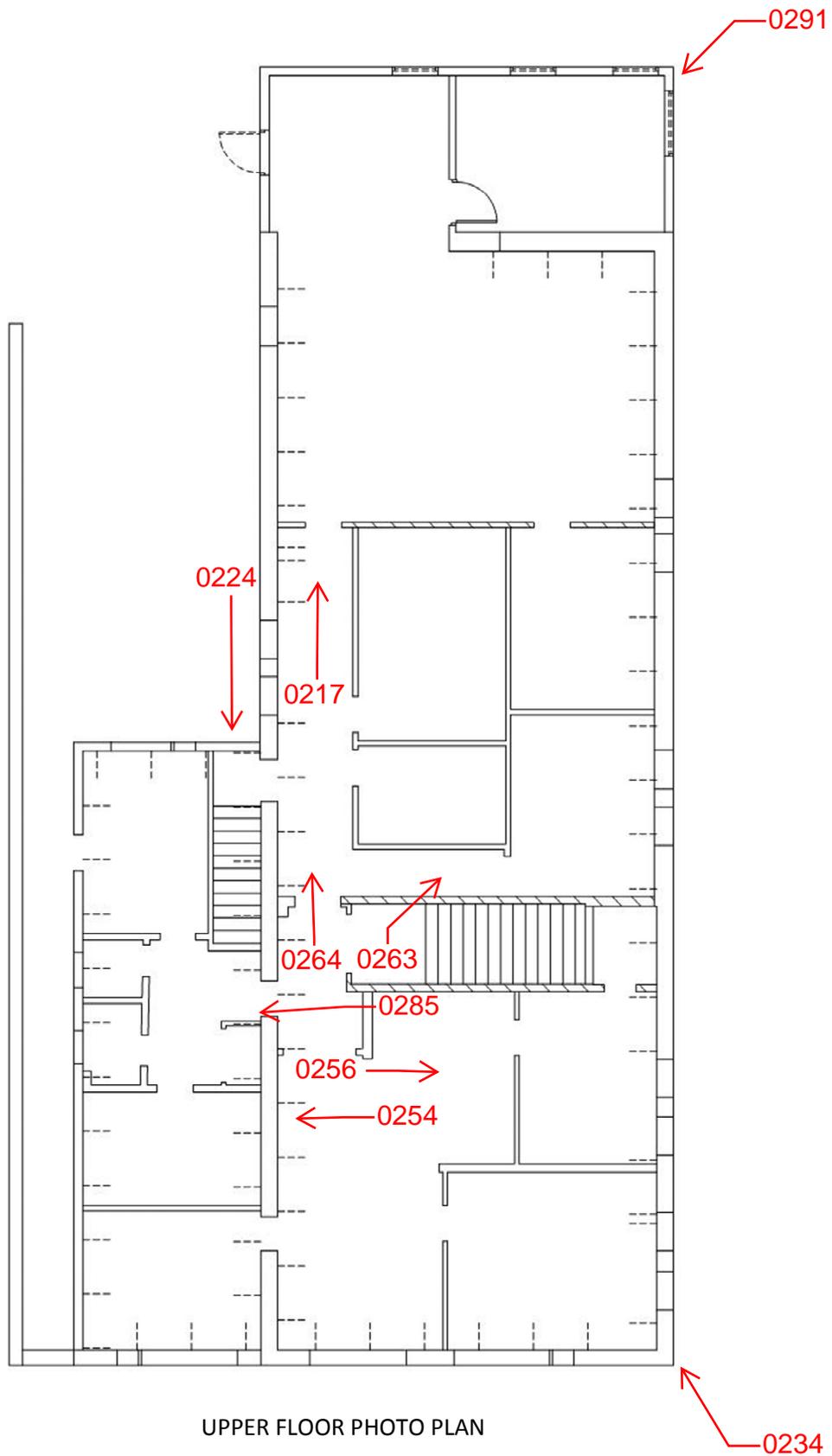
The above costs only consider the structural upgrades and corresponding work as required by the structural upgrades; the other concerns with the building such HVAC, plumbing, ADA access or other non-structural costs such as bracing the suspended ceilings are not included in this cost.

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.

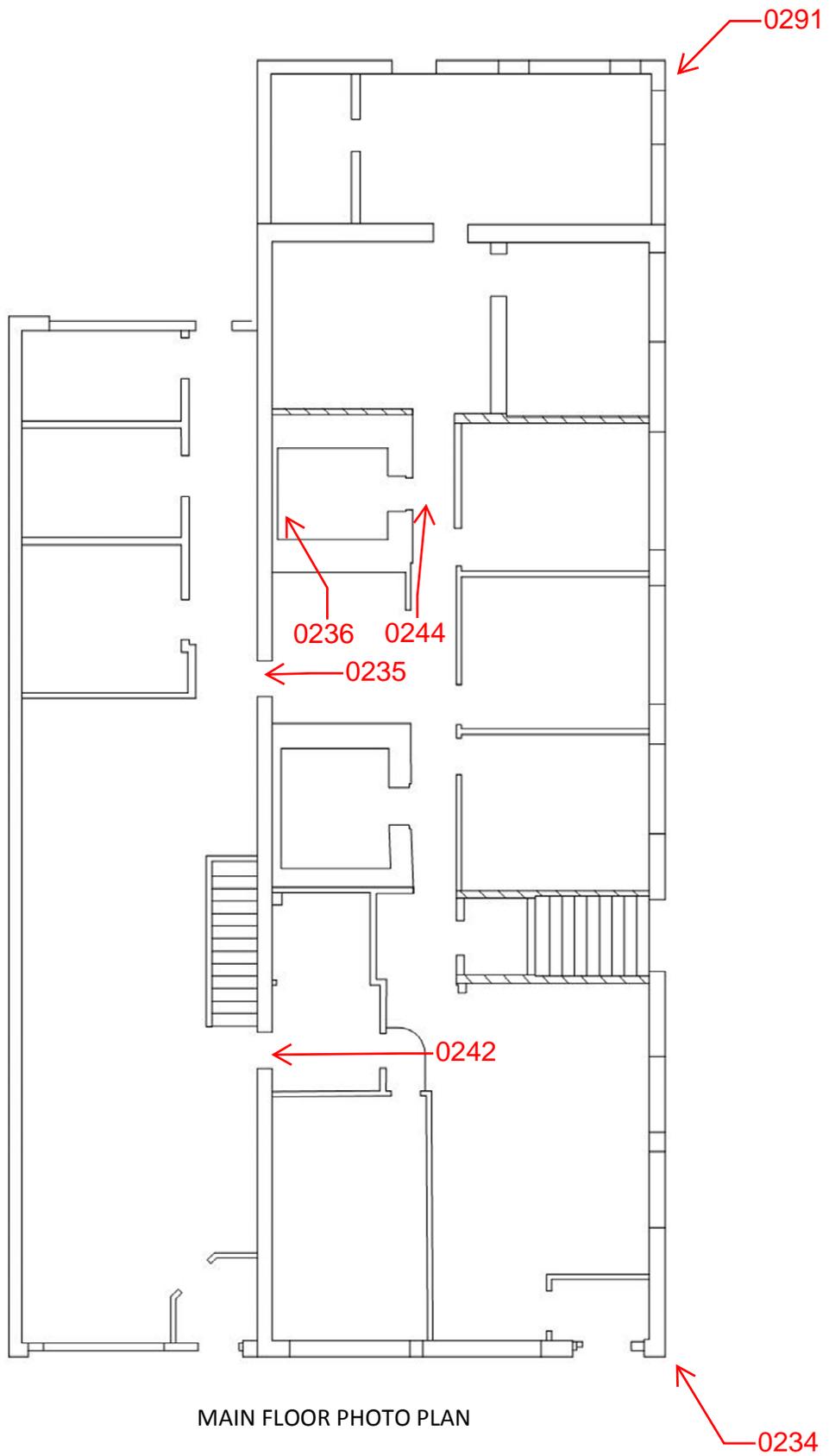
Background

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

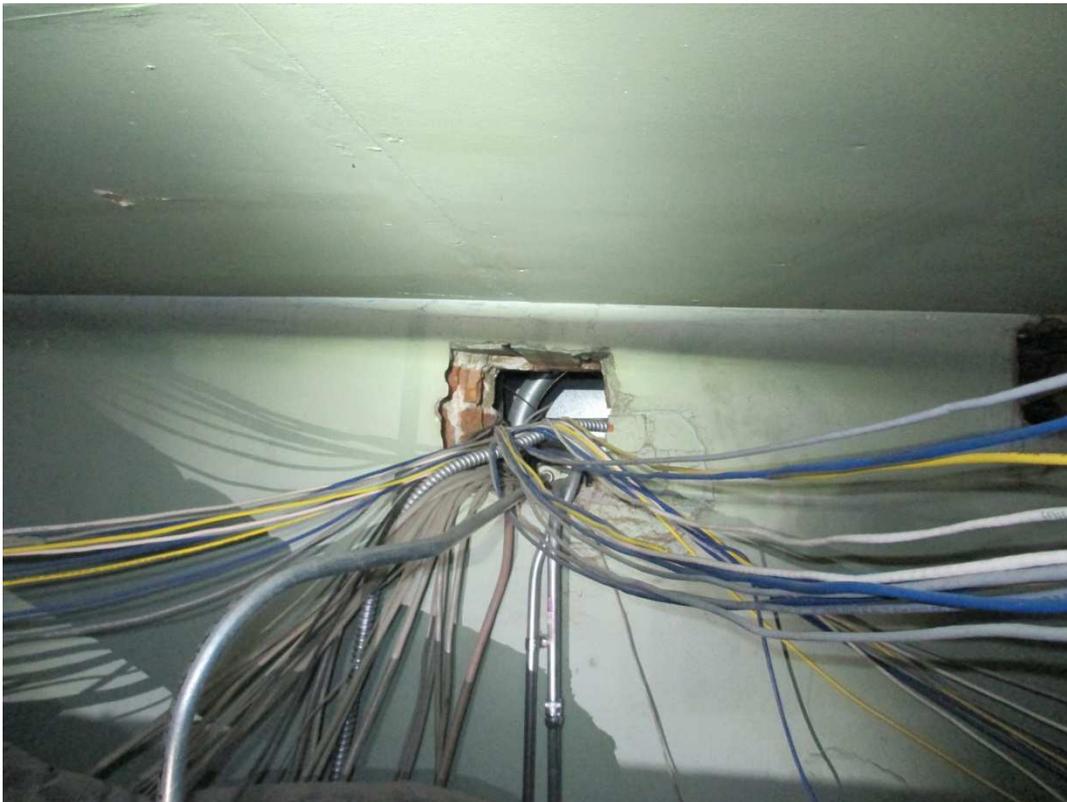
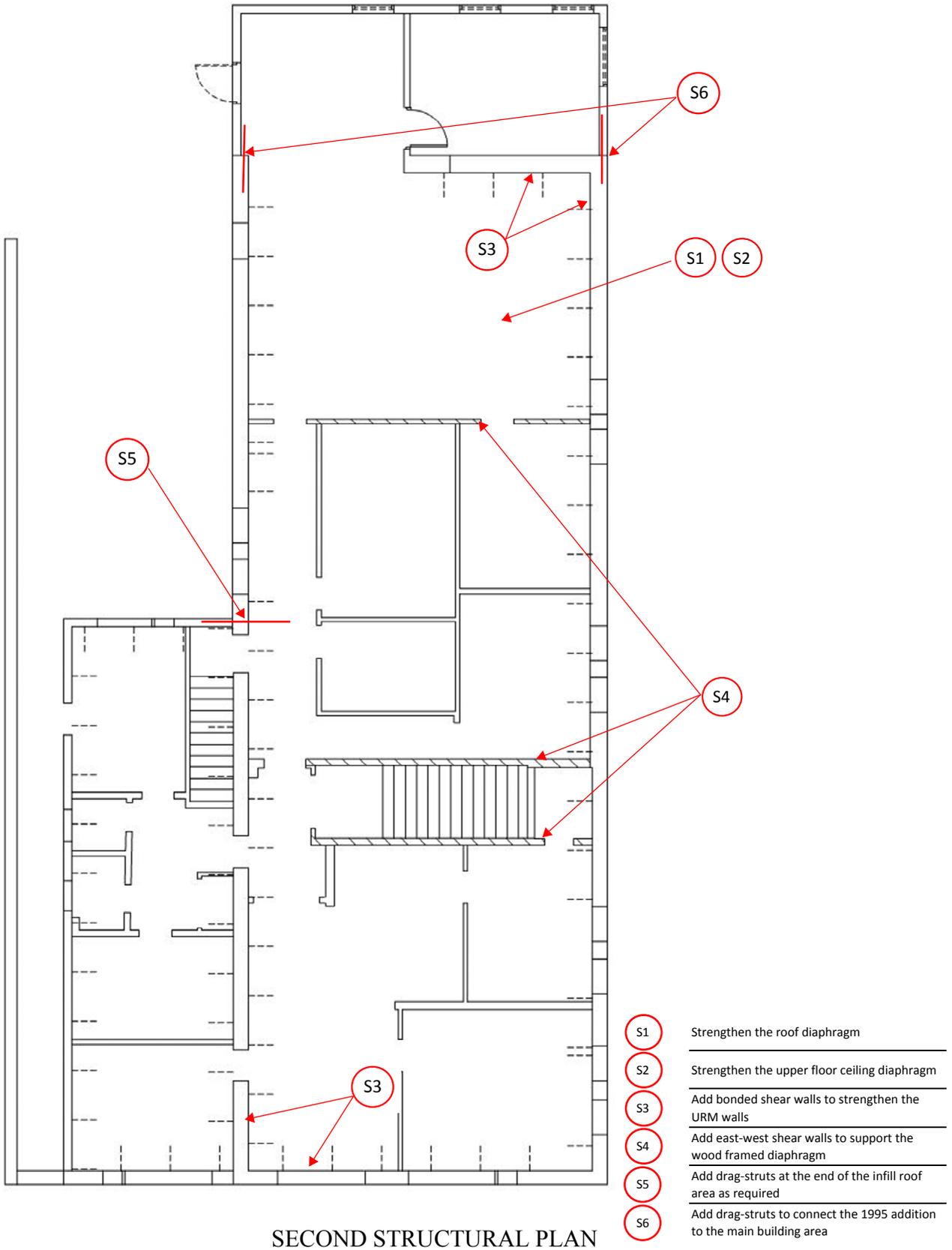
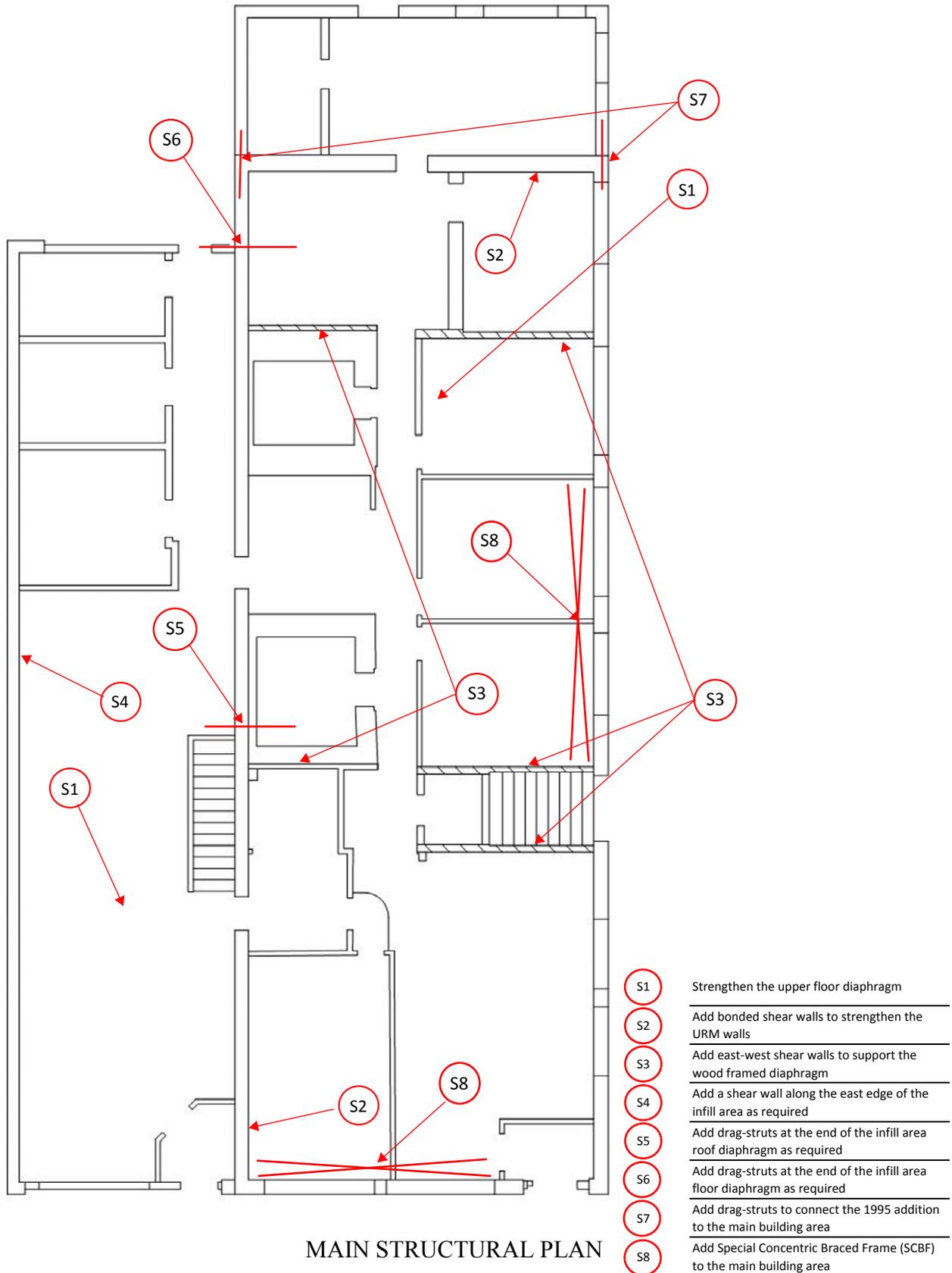


PHOTO 0235 – Access opening through the east URM wall in the main building area





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	
Ventilation	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 **Exposure:** B **Wind Speed (3 sec Gust):** 120 MPH

Importance Factors: $I_w = \frac{1.00}{(\text{ice w/ wind})}$ $I_E = \frac{1.00}{(\text{seismic})}$ $I_s = \frac{1.00}{(\text{snow})}$ $I_i = \frac{1.00}{(\text{ice})}$ **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 Location: 20 E Main Street, Ashland, Oregon Client: City of Ashland BY: ERW	Project #: 150899 Ck'd:  Date: 12/08/15 Page 1
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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”

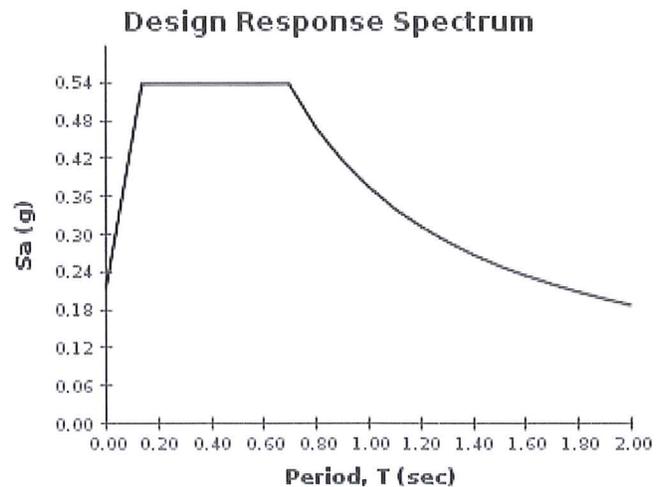
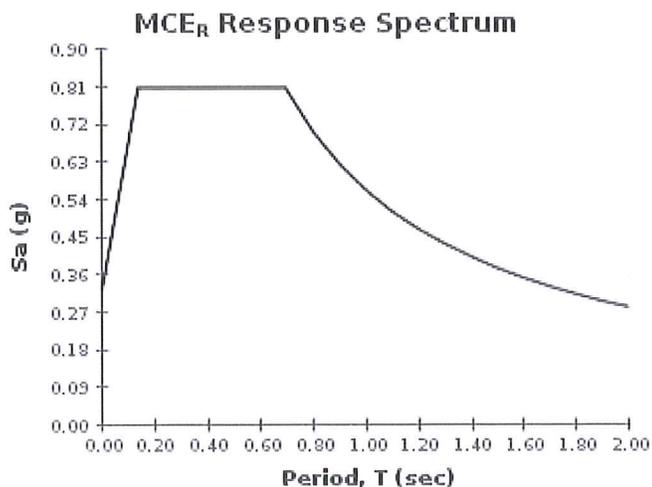
Risk Category I/II/III

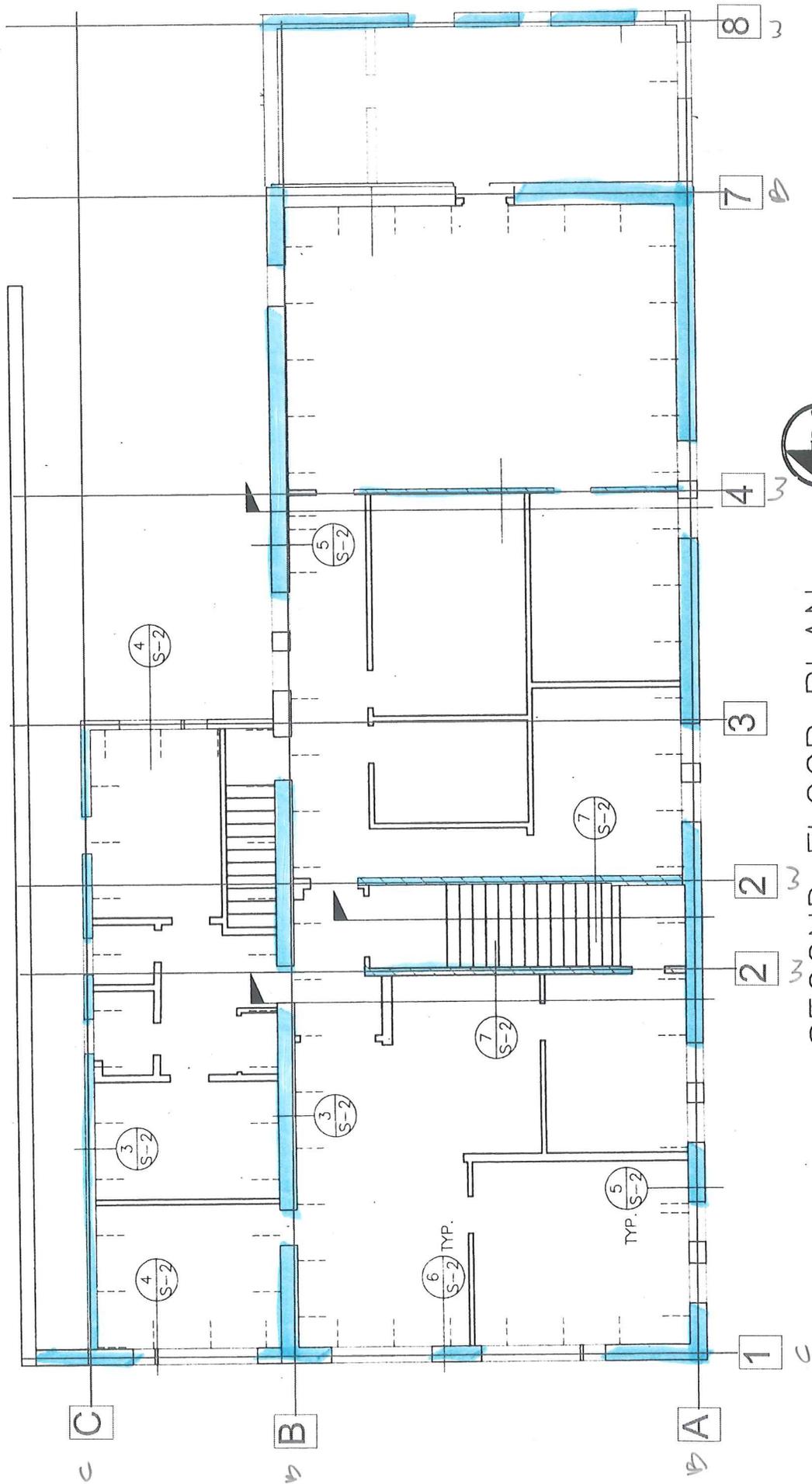


USGS-Provided Output

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

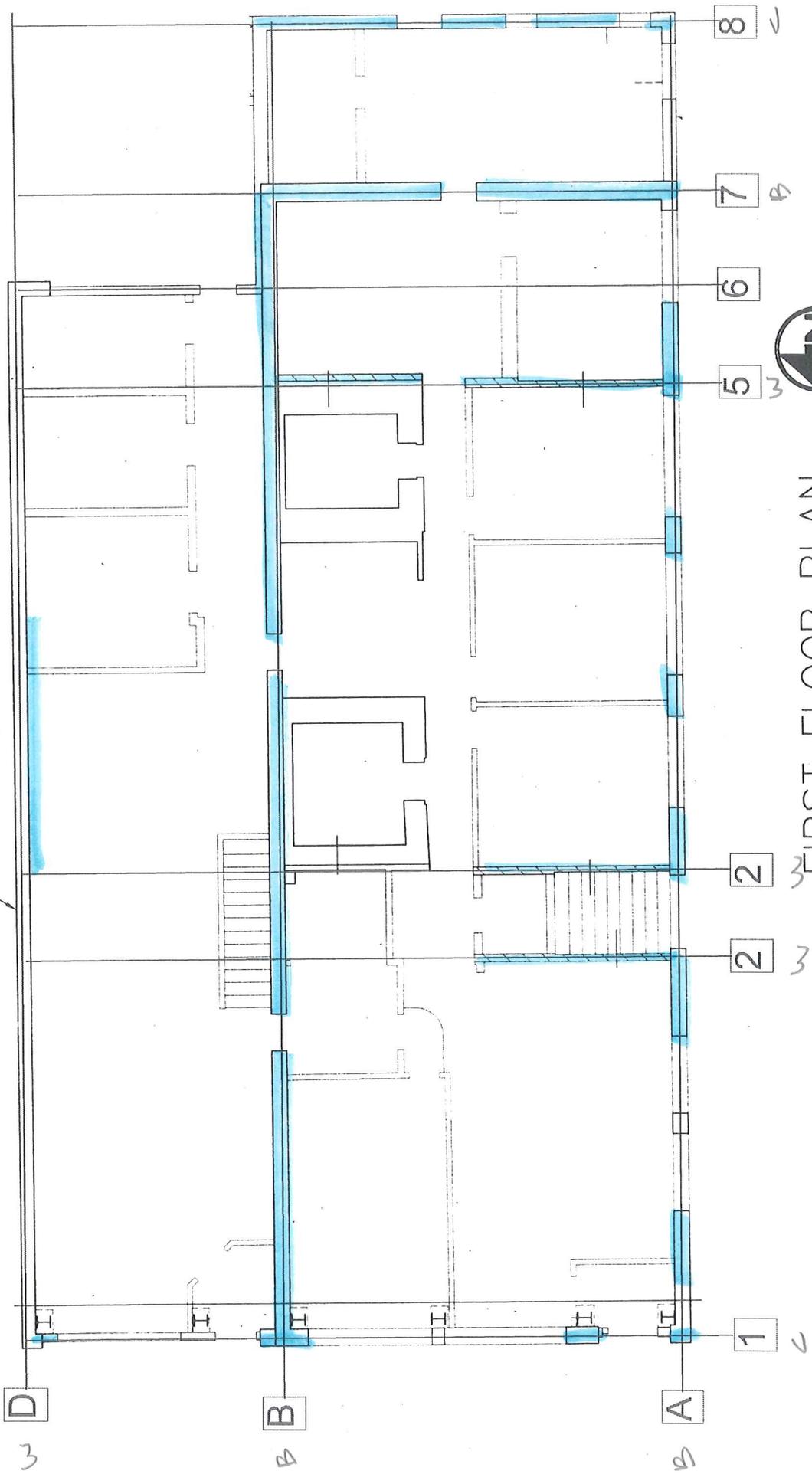
For information on how the S_s and S_1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the “2009 NEHRP” building code reference document.





SECOND FLOOR PLAN
 1/8" = 1'-0"

COMMON BUILDING WALL



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

(w / JRM WARS)

$S_s =$	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)
$F_a =$	1.30	Table 11.4-1, page 55
$F_v =$	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

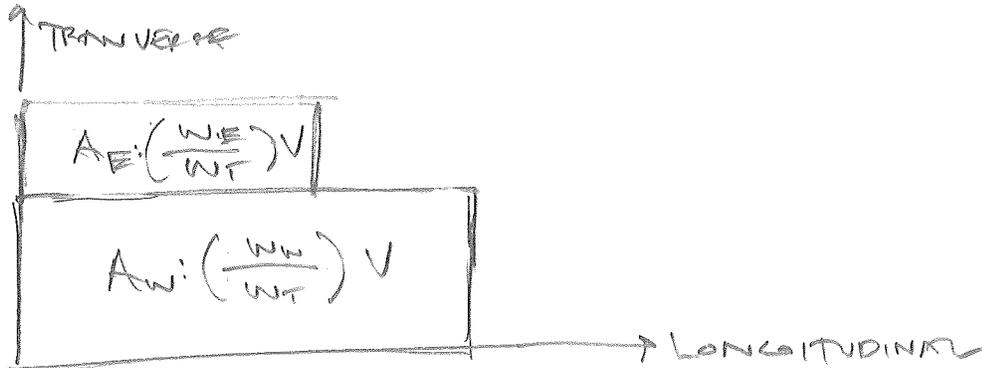
10. Ordinary plain masonry shear walls

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
$\rho =$	2.50	Table 12.2-1, pages 60-62
$C_d =$	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
$C_u =$	1.4	Table 12.8-1, page 72
$T_{a\max} =$	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 $\rho:$	1.00	Section 12.3.4, pg 67
$V_{roof} = C_s W = 0.251(W) =$	108322	lb, Eq. 12.8-1, pg 71
$V_{2nd} = C_s W = 0.251(W) =$	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)	$w_x \cdot h_x^k$	C_v	Force (lbs)	with $\rho:$
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 : $S(12(12)) = 720$ pif
 MAX TWO SIDED SW : 1940 pif
 MAX ONE SIDED SW : 770 pif

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

CHECK DIAPHRAGMS:

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

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

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

$$v_{SW} = \frac{1}{2} (83.6) / 21 = 1397 \text{ pif}$$

(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)
$F_a =$	1.30	Table 11.4-1, page 55
$F_v =$	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
$\Omega =$	3.00	Table 12.2-1, pages 60-62
$C_d =$	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
$C_u =$	1.4	Table 12.8-1, page 72
$T_{a_{max}} =$	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 $\rho:$	1.00	Section 12.3.4, pg 67
$V_{roof} = C_s W = 0.058(W) =$	24997	lb, Eq. 12.8-1, pg 71
$V_{2nd} = C_s W = 0.058(W) =$	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)	$w_x \cdot h_x^k$	C_v	Force (lbs)	with $\rho:$
Roof	24.75	431090	10689477.5	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
$F_a =$	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	* W_p ASCE 7-10 Eq. 13.3-1, pg 88
OR	0.862	* W_p ASCE 7-10 Eq. 13.3-2, pg 89
Not less than	0.162	* W_p ASCE 7-10 Eq. 13.3-3, pg 89

$F_p =$	0.181	* W_p	x 0.7 (Allowable Loading)
$0.2S_{DS}W_p =$	0.108	* W_p	(ASCE 7-10 Sec. 13.3, pg 88)

(12.11.1) $0.4S_{DS}(0.7) = 0.151 \neq \text{USE } 13.3-1$
 TRY BRACING ONLY AT CEILING:

$$M_{PW} = \frac{0.181(20)(6)^2}{2} = 261 \text{ '}\#$$

$$M_P = [0.5(6(20))] \cdot [1/2(0.667)] = 96 \text{ '}\#$$

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

> 1 BRACE W/ AT ROOF AND CEILING

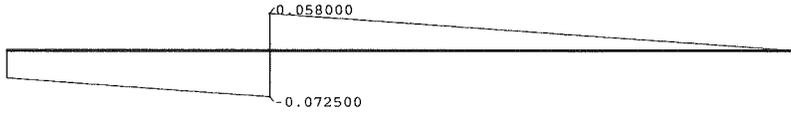
WinBeam

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

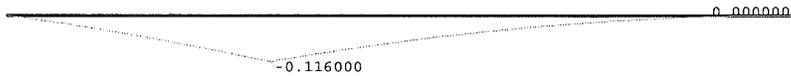
Reactions - kips, kip ft



Shear - kips



Moment - kip ft



Rotation - radians



Deflection - inches



LOAD AT MASONRY
 $M_R = \frac{1}{2}(0.667) [0.6(6(80))]$
 $\Rightarrow M_R = 96 \text{ 'ft}$
 $F_B = \frac{(116 - 96)}{(8/12)(12)}$
 $F_B = 1.12 \text{ psi}$

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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

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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
$F_a =$	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	* W_p ASCE 7-10 Eq. 13.3-1, pg 88
OR	0.862	* W_p ASCE 7-10 Eq. 13.3-2, pg 89
Not less than	0.162	* W_p ASCE 7-10 Eq. 13.3-3, pg 89

$F_p =$	0.145	* W_p	x 0.7 (Allowable Loading)
$0.2S_{DS}W_p =$	0.108	* W_p	(ASCE 7-10 Sec. 13.3, pg 88)

$(12 \cdot 11 \cdot 1) \cdot 0.4 S_{DS} \cdot 0.7 = 0.151 \Rightarrow$ USE 12.11.1

$$M = \frac{[0.151 (120)] (12)}{2} = 326 \text{ k-ft}$$

$$M_p = 0.6 \cdot [6(30) + 6(120)] \cdot \frac{1}{2} (1) = 360 \text{ k-ft}$$

\Rightarrow WALL UNDER COMPRESSION, NO TENSION
 DUE TO ODP LOADINGS

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
$F_a =$	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	* W_p ASCE 7-10 Eq. 13.3-1, pg 88
OR	0.862	* W_p ASCE 7-10 Eq. 13.3-2, pg 89
Not less than	0.162	* W_p ASCE 7-10 Eq. 13.3-3, pg 89

$F_p =$	0.113	* W_p	x 0.7 (Allowable Loading)
$0.2S_{DS}W_p =$	0.108	* W_p	(ASCE 7-10 Sec. 13.3, pg 88)

$(12 \cdot 11 \cdot 1) \cdot 0.4 S_{DS} \cdot 0.7 = 0.151 \Rightarrow$ USE 12-11-1

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

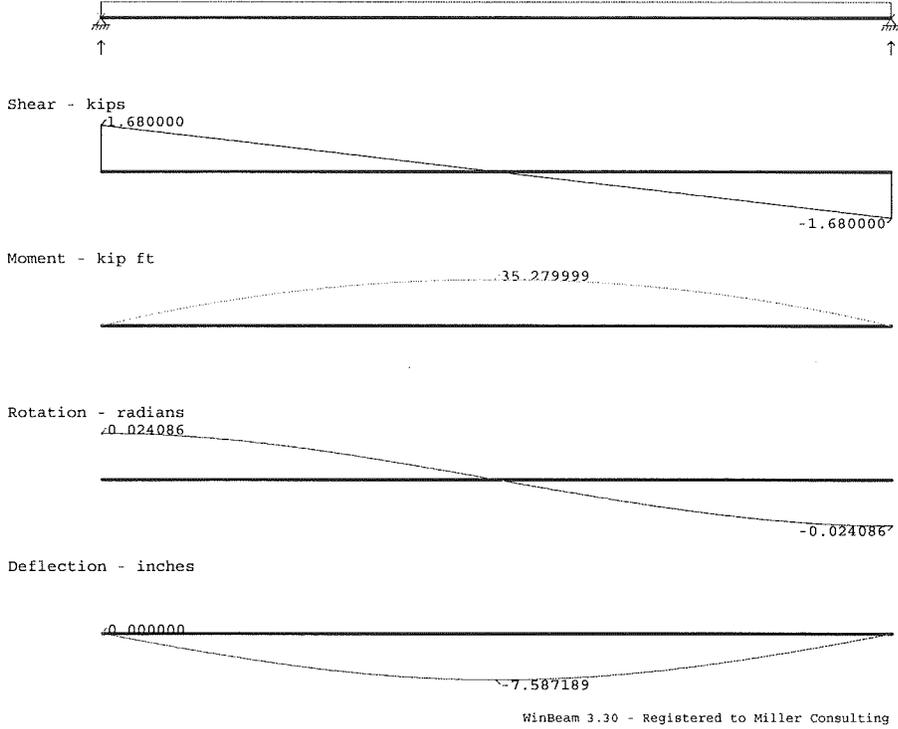
$$M_p = 0.6 [6(80) + (12 + 6.375)(120)] \left[\frac{1}{2}(1) \right] = 816 \text{ \#}$$

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

WinBeam

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

Reactions - kips, kip ft



$$V_R = \frac{1680}{30} = 56 \text{ plf}$$

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WinBeam

Project: _____
 By: _____ Date: _____ Checked: _____ Date: _____ Page: **FRONT CLG DIAPHRAGM**

Analysis Data:

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

$$V_R = 357 \left[\frac{9/18 (9180) + 2(3(60(24)))}{200090} \right]$$

= 40 plf

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

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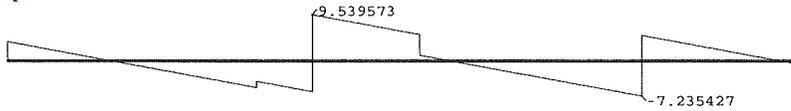
WinBeam

Project: **DAPPAW - CEILING AT BLOND**
 By: _____ Date: _____ Checked: _____ Date: _____ Page: _____

Reactions - kips, kip ft

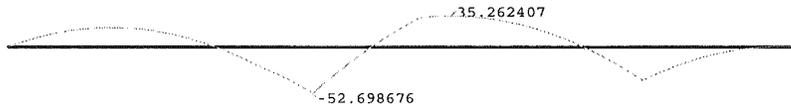


Shear - kips

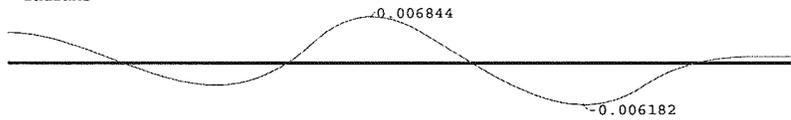


$$V_D = \frac{9540}{30} = 318 \text{ plf}$$

Moment - kip ft



Rotation - radians



Deflection - inches



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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

$$V_L = \frac{39280 \left(\frac{293800}{250000} \right)}{84} = 357 \text{ plf}$$

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	

$$P_2 = 39280 \left(\frac{76200}{250000} \right) = 4280 \#$$

Equilibrium:

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

Min & Max values:

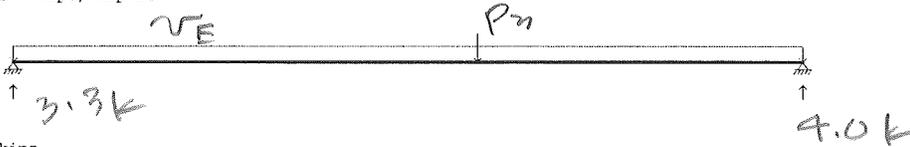
Min Shear	=	-7.235 kips	at	67.500 feet
Max Shear	=	9.540 kips	at	32.500 feet
Min Moment	=	-52.699 kip ft	at	32.500 feet
Max Moment	=	35.262 kip ft	at	47.357 feet
Min Rotation	=	-0.006182 radians	at	61.205 feet
Max Rotation	=	0.006844 radians	at	38.661 feet
Min Deflection	=	-0.953457 in	at	49.455 feet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

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WinBeam

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

Reactions - kips, kip ft



Shear - kips



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

Moment - kip ft



Rotation - radians



Deflection - inches



WinBeam 3.30 - Registered to Miller Consulting Engrs.

WinBeam

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

Analysis Data:

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

$$V_E = \frac{2498 \left(\frac{49790}{762790} \right)}{75.5} = 40 \text{ plf}$$

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:

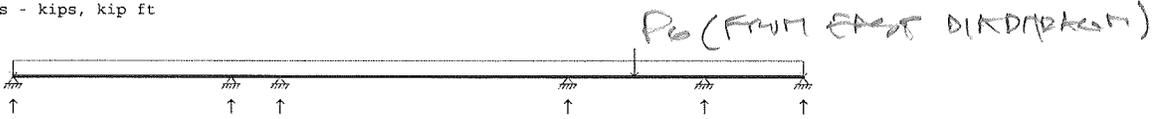
Min Shear	=	-4.033 kips	at	75.500 feet
Max Shear	=	3.267 kips	at	0 feet
Min Moment	=	-8.577e-013 kip ft	at	75.500 feet
Max Moment	=	105.792 kip ft	at	44.500 feet
Min Rotation	=	-0.055625 radians	at	75.500 feet
Max Rotation	=	0.051335 radians	at	0 feet
Min Deflection	=	-15.702 in	at	39.220 feet
Max Deflection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

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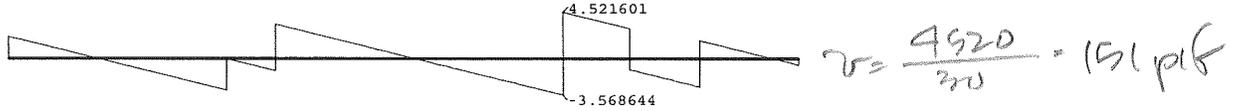
WinBeam

Project: **DIAMBRAM - SECOND FLOOR (WEST)**
 By: _____ Date: _____ Checked: _____ Date: _____ Page: _____

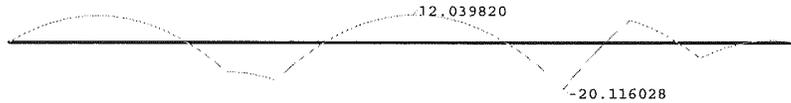
Reactions - kips, kip ft



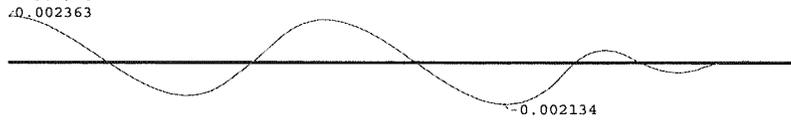
Shear - kips



Moment - kip ft



Rotation - radians



Deflection - inches



WinBeam 3.30 - Registered to Miller Consulting Engrs.

WinBeam

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

Analysis Data:

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

$$v_w = \frac{22198 \left(\frac{7138w}{762750} \right)}{96} = 199 \text{ plf}$$

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
Lower 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

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

Roof Wall Line Marks	Wind (lb)	Min. Wind (lb)	Seismic (lb)	Stacks on:	Seismic Capacity (lb)	Wind Capacity (lb)
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			
Simpson Holdowns and Anchors in Use; (Verify Fdn. Config. w/ Loads Listed in Catalog for Anchor Bolt Types)						
				#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 Holddown to: floor below
Upper Floor Level Holddown to: concrete

Roof Level	0.2Sds = 0.075		Wind (W), lb		Min. Wind, lb	Seismic (E), lb	Wind Uplift (U) = 10 psf	
Wall Line	R1		0		6433			
	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	
Holddown Capacity, lb:	2695	2695	2695	1155				
HD Type:	2	2	2	1				
Shear critical, plf:	542	474	356	316			15/32" APA Rated Sheathing	
HD critical, lb:	2336	2142	1627	857			Seg. 1: 'D' = 8d At 2" o.c. edges, 12" o.c. field	
40% increase for wind?:	No	No	No	No			Seg. 2: 'C' = 8d At 3" o.c. edges, 12" o.c. field	
Sheathing Layers:	Single	Single	Single	Single			Seg. 3: 'B' = 8d At 4" o.c. edges, 12" o.c. field	
Concrete Anch.?	No	No	No	No			Seg. 4: 'B' = 8d At 4" o.c. edges, 12" o.c. field	
Wall Type:	D	C	B	B			Holddown 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	
Holddown 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		Wind (W), lb		Min. Wind, lb	Seismic (E), lb	Wind Uplift (U) = 10 psf	
Wall Line	R2 (N)		0		5151			
	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	
Holddown Capacity, lb:	2695							
HD Type:	2							
Shear critical, plf:	271						15/32" APA Rated Sheathing	
HD critical, lb:	2490						Seg. 1: 'B' = 8d At 4" o.c. edges, 12" o.c. field	
40% increase for wind?:	No							
Sheathing Layers:	Single							
Concrete Anch.?	No							
Wall Type:	B						Holddown Types:	
Wall Capacity, plf:	380						Seg. 1: '2' = MSTC40 with (14) 16d sinkers at each end	
Controlling HD Type:	2							
Holddown 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
(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
Holddown Capacity, lb:	2695				
HD Type:	2				
Shear critical, plf:	284				15/32" APA Rated Sheathing
HD critical, lb:	2495				Seg. 1: 'B' = 8d At 4" o.c. edges, 12" o.c. field
40% increase for wind?:	No				
Sheathing Layers:	Single				
Concrete Anch.?:	No				
Wall Type:	B				Holddown Types:
Wall Capacity, plf:	380				Seg. 1: '2' = MSTC40 with (14) 16d sinkers at each end
Controlling HD Type:	2				
Holddown 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
Holddown Capacity, lb:	5860	5860			
HD Type:	4	4			
Shear critical, plf:	421	421			15/32" APA Rated Sheathing
HD critical, lb:	4677	4335			Seg. 1: 'C' = 8d At 3" o.c. edges, 12" o.c. field
40% increase for wind?:	No	No			Seg. 2: 'C' = 8d At 3" o.c. edges, 12" o.c. field
Sheathing Layers:	Single	Single			
Concrete Anch.?:	No	No			
Wall Type:	C	C			Holddown 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
Holddown 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	Wind Uplift (U) = 10 psf		
R7	0	0	5385			
	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
Holddown Capacity, lb:	0					
HD Type:	0					
Shear critical, plf:	420					15/32" APA Rated Sheathing
HD critical, lb:	0					Seg. 1: 'C' = 8d At 3" o.c. edges, 12" o.c. field
40% increase for wind?:	No					
Sheathing Layers:	Single					
Concrete Anch.?:	No					
Wall Type:	C					Holddown Types:
Wall Capacity, plf:	490					Seg. 1: '0' no holdown required
Controlling HD Type:	0					
Holddown Capacity, lb:	0					
3x Members req.:	Yes					

Roof Level	0.2Sds = 0.075					
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb	Wind Uplift (U) = 10 psf		
R8	0	0	2068			
	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
Holddown Capacity, lb:	1155	1155	1155			
HD Type:	1	1	1			
Shear critical, plf:	131	98	98			15/32" APA Rated Sheathing
HD critical, lb:	878	812	614			Seg. 1: 'A' = 8d At 6" o.c. edges, 12" o.c. field
40% increase for wind?:	No	No	No			Seg. 2: 'A' = 8d At 6" o.c. edges, 12" o.c. field
Sheathing Layers:	Single	Single	Single			Seg. 3: 'A' = 8d At 6" o.c. edges, 12" o.c. field
Concrete Anch.?:	No	No	No			
Wall Type:	A	A	A			Holddown 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
Holddown 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		Wind Uplift (U) = 10 psf		
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb		
RA	0	0	15504		
	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
Holddown 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		Holddown Types:
Wall Capacity, plf:	380	380	380		Seg. 1: '0' no holddown required
Controlling HD Type:	0	0	0		Seg. 2: '0' no holddown required
Holddown Capacity, lb:	0	0	0		Seg. 3: '0' no holddown required
3x Members req.:	No	No	No		

Roof Level	0.2Sds = 0.075		Wind Uplift (U) = 10 psf		
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb		
RB	0	0	17125		
	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
Holddown 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		Holddown Types:
Wall Capacity, plf:	380	380	380		Seg. 1: '0' no holddown required
Controlling HD Type:	0	0	0		Seg. 2: '0' no holddown required
Holddown Capacity, lb:	0	0	0		Seg. 3: '0' no holddown 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
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	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
Holddown 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			Holddown Types:
Wall Capacity, plf:	260	260	260			Seg. 1: '0' no holddown required
Controlling HD Type:	0	0	0			Seg. 2: '0' no holddown required
Holddown Capacity, lb:	0	0	0			Seg. 3: '0' no holddown 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	Wind Uplift (U) =	psf	
S1	0	0	4254	10.00		
Load from R1, lb:	0	0	6433	Upper Fir 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/v 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
Holddown 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.					Holddown Types:
Wall Capacity, plf:						Seg. 1: 'NG' larger holddown required
Controlling HD Type:	NG					
Holddown Capacity, lb:						
3x Members req.:	Yes					

Upper Floor Level		0.2Sds = 0.075				
Wall Line	Wind (W), lb	Min. Wind, lb	Seismic (E), lb	Wind Uplift (U) =	psf	
S2 (N)	0	0	3297	10		
Load from R2 (N), lb:	0	0	5151	Upper Fir 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
Holddown 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					Holddown Types:
Wall Capacity, plf:	640					Seg. 1: '12' = HDU14-SDS2.5 and PAB8x30 Anchor Bolt
Controlling HD Type:	12					
Holddown 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	Wind Uplift (U) =	psf	
S2 (S)	0	0	5935	10		
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
R2 (S) Wall Height (H1), ft:	12.00	12.00				
R2 (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
Holddown 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				Holddown 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
Holddown 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	Wind Uplift (U) =	psf	
S5	0	0	12359	10		
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
Holddown 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				Holddown 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
Holddown 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 Fir 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
Holddown 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				Holddown Types:
Wall Capacity, plf:	380	380				Seg. 1: '0' no holddown required
Controlling HD Type:	0	0				Seg. 2: '0' no holddown required
Holddown 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 Fir 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
Holddown 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					Holddown Types:
Wall Capacity, plf:	380					Seg. 1: '6' = HDU2-SDS2.5 and SSTB16 Anchor Bolt
Controlling HD Type:	6					
Holddown 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
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
Holddown 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.	Holddown Types:
Wall Capacity, plf:					Seg. 1: 'NG' larger holddown required
Controlling HD Type:	NG	NG	NG	NG	Seg. 2: 'NG' larger holddown required
Holddown Capacity, lb:					Seg. 3: 'NG' larger holddown required
3x Members req.:	Yes	Yes	Yes	Yes	Seg. 4: 'NG' larger holddown 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
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
Holddown 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	Holddown Types:
Wall Capacity, plf:	490	490	490	490	Seg. 1: '0' no holddown required
Controlling HD Type:	0	0	0	0	Seg. 2: '0' no holddown required
Holddown Capacity, lb:	0	0	0	0	Seg. 3: '0' no holddown required
3x Members req.:	Yes	Yes	Yes	Yes	Seg. 4: '0' no holddown 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
Overall Seg. (Lo), ft:	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				
Mr (Seismic), ft-lb:	68400				
HD (Seismic), lb:	110				
Holddown Capacity, lb:	2550				
HD Type:	6				
					15/32" APA Rated Sheathing
Shear critical, plf:	162				
HD critical, lb:	110				
40% increase for wind?:	No				
Sheathing Layers:	Single				
Concrete Anch.?	Yes				
Wall Type:	A				
Wall Capacity, plf:	260				
Controlling HD Type:	6				
Holddown Capacity, lb:	2550				
3x Members req.:	No				

15/32" APA Rated Sheathing
 Seg. 1: 'A' = 8d At 6" o.c. edges, 12" o.c. field

Holddown Types:
 Seg. 1: '6' = HDU2-SDS2.5 and SSTB16 Anchor Bolt

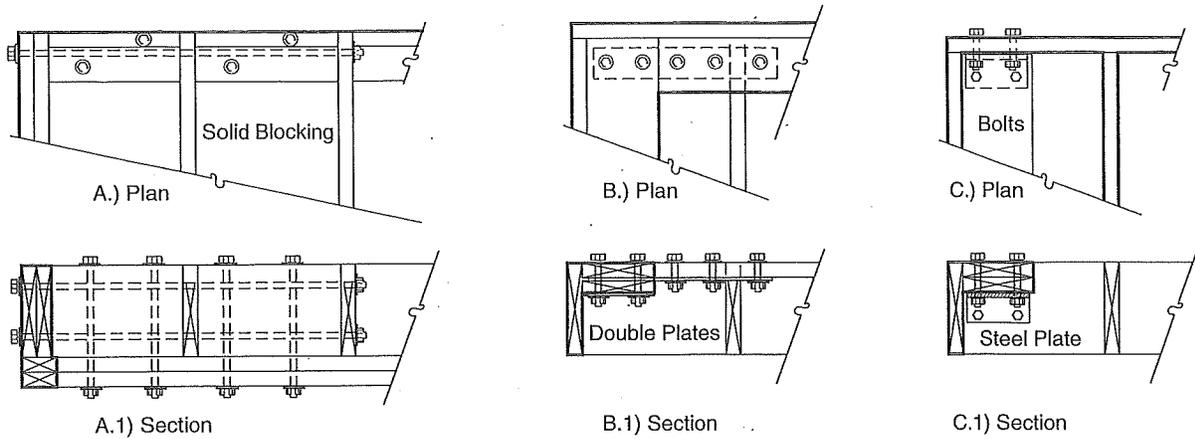


Figure 10.14 Typical corner details

TRANSVERSELY SHEATHED DIAPHRAGMS (Douglas Fir-Larch Lumber)							Table 10.1		
Maximum Span-Width or Height-Width Ratio			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				
Horizontal Diaphragms Restraining:		Vertical Diaphragms			12	16	24		
Masonry or Concrete Walls	Wood or Similar Walls								
Not recommended	Limited by acceptable deflection of wall	Limited by acceptable deflection	6 8 10	2 2 2	100 114 124	75 86 93	50 57 62		
DIAGONALLY SHEATHED DIAPHRAGMS									
Maximum Span-Width or Height-Width Ratio			Nominal Width of Sheathing Boards (inches)	Number of 8d Common Nails per Board per Crossing at:		Allowable Lateral Shear Load lbs. per lineal ft.			
Horizontal Diaphragms Restraining:		Vertical Diaphragms		Perimeter Members and Butted Ends of Boards	Stud or Joist				
Masonry or Concrete Walls	Wood or Similar Walls								
3:1	4:1	2:1	6 8	2 2 3	2 2 2	345 262 393			

continued



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

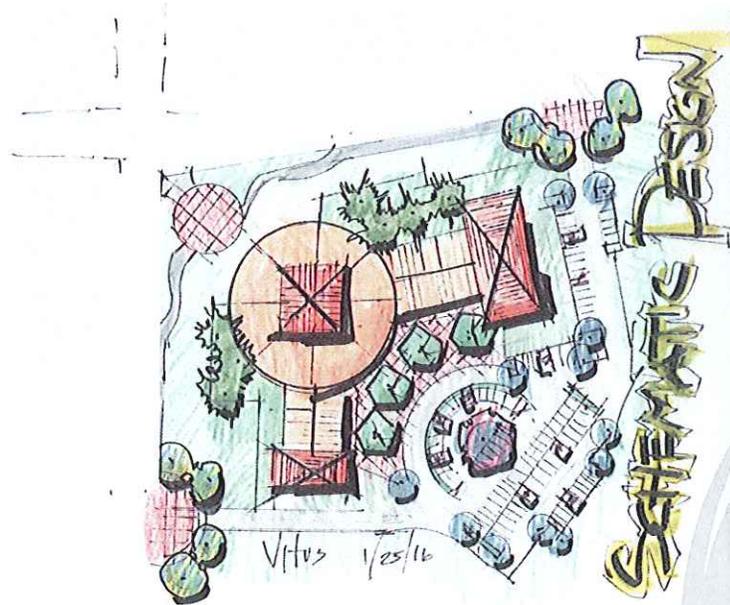
- c. Fire suppression \$15.00 Sq Ft
- d. Tenant improvements \$185.00 Sq Ft
- e. Total \$338.00 Sq Ft

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.