ATTACHMENT B

ASCE 31-03
Tier 1 Evaluation Report - 2013

By CROSBY GROUP
ASCE 31-03
Tier 1 Evaluation Report

Goleta Valley Community Center

5679, 5681 and 5689 Hollister Ave
Goleta, CA 93117

April 24, 2013
The following report summarizes the results of a Seismic Assessment which has been prepared for the Goleta Valley Community Center located at 5679 through 5689 Hollister Avenue in Goleta, California. This report has been prepared at the request of the City of Goleta as part of the overall Seismic, ADA and Fire/Life Safety Need Assessment as referenced in the RFP/RFQ dated May 10th, 2012.

The scope of this evaluation includes the review of the three main structures built between 1927 and 1958. The first of these buildings is the Main Community Center which was originally constructed in 1927 and is labeled Building A. The second and third buildings were constructed in 1948 and 1958 respectively as classroom additions and are labeled Building B and Building C4. The current uses for Buildings A, B and C are as follows:

- Building A. Main Community Center 5679 Hollister Avenue
- Building B. Head Start Classrooms 5681 Hollister Avenue
- Building C4. Rainbow School, Extension 5689 Hollister Avenue

As part of this investigation, each of these buildings was evaluated to determine whether there were existing seismic deficiencies which would pose a risk to life-safety. To conduct this evaluation, our team completed onsite investigations, reviewed all available existing building plans, and completed a Structural Tier 1, limited Tier 2 life-safety evaluation in accordance with ASCE 31-03. A formal ASCE 31 Tier 1 nonstructural components evaluation was not conducted, however, we believe the current condition of the existing nonstructural components do not pose a significant life safety threat. Based upon this analysis and our own professional experience, we found most items to be compliant with the life-safety checks except for the items listed below. Since an ASCE 31 evaluation simply assesses whether a particular component is compliant, we have grouped each of our found deficiencies into two categories termed Priority 1 and Priority 2. Based upon our experience with similar buildings, we believe Priority 1 items pose the greatest risk to Life-Safety and based upon these particular building circumstances, are those deficiencies that can generally lead to local areas of collapse. Priority 2 items are those which we would expect to be associated with extensive structural and non-structural damage, but are not usually linked to local areas of collapse. Priority 1 and 2 items are as follows:

[To be continued]
Building A - Priority 1 Items

1. Existing wall anchorage-to-diaphragm connections at the Auditorium were found to be significantly overstressed. See Details 3 and 8 on drawing sheet S-2.

2. The long span diaphragm over the main auditorium does not have compliant sheathing and is therefore considered non-compliant per ASCE 31’s guidelines for Life Safety compliance. It is recommended that this area be sheathed with \( \frac{1}{2}'' \) plywood as indicated on plan sheet S-1.

3. Existing in-plane roof to concrete wall connections at the Auditorium are insufficient to transfer anticipated seismic loads and should be strengthened per Detail 7 on sheet S-2.

Building A - Priority 2 Items

1. Existing roof diaphragm to wall anchorage connections throughout the remainder of the Main building was found to be overstressed and non-compliant. Since these walls are partially restrained at the base and are a maximum of 11'-0" high, we believe that the risk of local collapse is less than that of the similar connections at the Auditorium. Nonetheless, conditions such as these often lead to significant structural and non-structural damage to a building and in rare cases, local collapse. These conditions are listed as Priority 2 items and are addressed in details 1, 2, 5, 6, 9, 15, and 18 on sheet S-2.

2. All existing roof sheathing, except for the area of the barrel vault over the exiting dining room, is composed of 1x straight sheathing which has been shown to have very low capacity to resist and transmit seismic forces. We recommend that these areas be sheathed with new \( \frac{1}{2}'' \) plywood throughout the structure as indicated on sheet S1.

3. Concrete shear walls were found to have insufficient reinforcement ratios per the ASCE 31 structural checklist; however an additional analysis shows that these walls are not overstressed against expected earthquake loads. Mitigation is not recommended at this time.

Building B – No deficiencies were found for this building

Building C4 – No deficiencies were found for this building
Application to Future Codes

The next edition of ASCE 31 will be combined with the related standard ASCE 41 to form ASCE 41-13. As the name suggests, this new code is expected to be published in the near future. ASCE 41-13 will combine the seismic evaluation procedures of ASCE 31-03 with the seismic retrofit procedures of ASCE 41-06 to form a unified standard and a new state of the practice in seismic evaluation and retrofit of existing buildings. Our team performed a parallel review of the three buildings in question with the upcoming standard’s proposed provisions. In particular, there are new seismic hazard demands and out-of-plane wall anchorage procedures. We have determined that these changes are nonetheless consistent with our current findings and mitigation strategies and that our recommendations in this report will be consistent and valid with the proposed provisions.
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A. Scope & Intent of Evaluation

The ASCE 31-03 (formerly FEMA 310) Tier 1 report is intended to allow a rapid evaluation of the seismic capabilities of the structural system in a building. The objective of this evaluation is to determine whether the building meets the 'Life Safety' performance level. Using the checklists provided in ASCE 31-03, building system and component deficiencies are identified and described in this report.

A.1 Method and Scope of Evaluation

As noted above, the appropriate standard to assess whether or not the building meets a level of "Life Safety" in regards to future seismic events is ASCE-31-03. This document identifies buildings of certain age and construction type as “Benchmark Building” or buildings that are generally perceived as compliant. A matrix indicating these parameters is presented in Table 3-1 of Appendix B. Where a particular building falls within the parameters of this classification, only limited evaluation is normally required. Where a building falls outside of these parameters, a full assessment is required to determine whether the said building does or does not meet the stated performance level. For the purposes of this evaluation, the Performance Level is "Life-Safety."

Since the buildings in question do fall outside of these parameters, a full ASCE 31 evaluation has been completed. This evaluation starts with a “Tier 1 – Screening Phase” that attempts to assess main components of the buildings' seismic force resisting system by the use of standard checklists and simplified structural calculations.

Where any potential deficiencies are identified during the initial Tier 1 analysis, ASCE-31 methodology requires that these potential deficiencies be corrected or that a refined evaluation be completed to better assess the potential risk of these items. The second order evaluation phases are termed Tier 2 and Tier 3 and generally require substantially greater levels of effort. These subsequent assessments can either consist of a complete evaluation, or in some instances, may consist of a limited Tier 2 evaluation focused on a certain “perceived” deficiency. This procedure is outlined in Figure 1 below.
For the purposes of this assessment, complete Tier 1 and limited Tier 2 evaluations were required to completely assess these buildings and included the following items:

1. Review of original tenant improvement construction documents dating from 1926 to 1958.
2. Site visit to establish general conditions of the building and to generally verify information shown on the drawings.
3. Preparation of rapid screening evaluation techniques and calculations for the structural system using ASCE 31-03 design guidelines for Life-Safety Performance Level Tier 1.
4. Additional limited analysis using ASCE 31-03 design guidelines for Life-Safety Performance Level Tier 2.

A.2 Limitations

The services performed for this project have been provided at a level that is consistent with the general level of skill and care ordinarily provided by engineers practicing Structural Engineering. Work provided is done under the constraints of time and budget. Conclusions and information presented in this report are dependent on information provided by others. No warranty is expressed or implied. It should also be noted that a number of factors make it difficult to fully and easily assess the current condition of the existing structural elements. These include both the limited documentation available and the presence of hard finishes in many areas.
General Provisions

Understand the Evaluation Process

1. Collect Data and Visit Site
2. Determine Level of Seismicity
3. Determine Level of Performance

Evaluation Requirements

Benchmark Building? OR
1. Complete the Structural Checklist(s)
2. Complete the Foundation Checklist
3. Complete the Nonstructural Checklist

Tier 1: Screening Phase

yes

Further Evaluation?

yes

FULL BUILDING or DEFICIENCY-ONLY EVALUATION
Evaluate building using one of the following procedures:
1. Linear Static Procedure
2. Linear Dynamic Procedure
3. Special Procedure

Tier 2: Evaluation Phase

no

Deficiencies?

yes

Further Evaluation?

yes

Comprehensive Investigation (Nonlinear Analysis)

Tier 3: Detailed Evaluation Phase

Building Complies

 yes

Deficiencies?

yes

Building Does NOT Comply

Final Evaluation and Report

Figure 1-1. Evaluation Process

Figure 1 – Evaluation Process from ASCE 31-03
B. Site Seismicity & Soil Profile

B.1 General

The successful performance of buildings in areas of high seismicity depends upon the combination of strength, structural component ductility, and the presence of a fully interconnected, balanced, and complete lateral force resisting system. As the level of seismicity is decreased, the demands on the structural system are decreased.

The Goleta Valley area is in a region of historically high seismic activity and high seismic potential. The nearest active fault to the site is the Santa Ynez fault, located approximately 7 miles from the site. A table indicating the potentially active faults and their respective magnitude potential is shown below.

<table>
<thead>
<tr>
<th>Fault Name (Seismic Source)</th>
<th>Maximum Moment Magnitude</th>
<th>Slip Rate (mm/yr.)</th>
<th>Distance From Site (miles)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Santa Ynez</td>
<td>7</td>
<td>2.0</td>
<td>7</td>
</tr>
<tr>
<td>San Andreas (1857 Rupture)</td>
<td>7.8</td>
<td>34.0</td>
<td>42</td>
</tr>
<tr>
<td>Red Mountain</td>
<td>6.8</td>
<td>2</td>
<td>9</td>
</tr>
<tr>
<td>Ventura Pitas-Point</td>
<td>6.8</td>
<td>1</td>
<td>16</td>
</tr>
</tbody>
</table>

TABLE 1 – Site Seismicity Based on Design Accelerations

B.2 Site Accelerations

In general, site “Seismicity” or the potential for strong ground motion is classified into regions of Low, Medium, and High. These regions are based upon mapped site accelerations $S_S$ and $S_1$, which are then modified by site coefficients $F_a$ and $F_v$ to produce Design Spectral Accelerations $S_{DS}$ (short period) and $S_{D1}$ (1 second period).

Design Spectral Accelerations computed for this site are as follows:

\[ S_{DS} = 1.14g \]
S\(_{D1}\) = 0.665g

As indicated below by the site accelerations, this standard places the subject property in a region of HIGH Seismicity.

<table>
<thead>
<tr>
<th>Region of Seismicity</th>
<th>S(_{DS})</th>
<th>S(_{D1})</th>
</tr>
</thead>
<tbody>
<tr>
<td>Low</td>
<td>&lt;.167g</td>
<td>&lt;.067g</td>
</tr>
<tr>
<td>Moderate</td>
<td>&lt;.500g</td>
<td>&lt;.200g</td>
</tr>
<tr>
<td>High</td>
<td>&gt;.500g</td>
<td>&gt;.200g</td>
</tr>
</tbody>
</table>

Site Class D is assumed per the recommendations of the Geotechnical report by Earth Systems Pacific, dated December 3, 2012. All pseudo-static lateral demands required for the Tier 1 evaluation are computed based on this site classification.

**B.3 Site Soils**

<table>
<thead>
<tr>
<th>Site Characteristics</th>
</tr>
</thead>
<tbody>
<tr>
<td>Building Pad Preparation</td>
</tr>
<tr>
<td>Fault Rupture Potential</td>
</tr>
<tr>
<td>Liquefaction Potential</td>
</tr>
<tr>
<td>Land Slide Potential</td>
</tr>
</tbody>
</table>

**TABLE 3 – Site Characteristics**

See attached geotechnical report located in Appendix D
C. Building Information

C.1 Building Descriptions

Building A

Building A was designed in 1926 and constructed in 1927. The single story building has an overall plan area of approximately 19,607 square feet. The building consists of a main auditorium in the middle of the building flanked by an open court and row of classrooms to the east side and an assembly room and row of classrooms to the west side. The assembly room to the immediate west side of the main auditorium was originally an open court as well but was modified to include a barrel vaulted roof above the space.

In the main auditorium, the gravity load carrying system generally consists of a wood diaphragm over 2x6 rafters and 42 foot long wood trusses. The trusses are spaced 12 feet on center and anchored to 10” concrete shear walls. The lateral load resisting system consists of the same wood diaphragm which distributes lateral loads in to the concrete shear walls on each side of the auditorium.

For the classroom sections of the building, the gravity system consists of a flat truss with 2x6 members and the truss is spaced at 32” oc. The roof truss is then anchored to the perimeter concrete shear walls which are either 8” or 10” in thickness.

The main auditorium is connected and tied to the side classroom wings by a primary corridor which consists of a queen post truss system spaced at 32” oc and composed of 2x6 rafters. The truss system bears on 8 and 10 inch concrete shear walls. The space immediately to the west side of the middle auditorium was originally an open court, similar to the space immediately to the east side of said auditorium. After a tenant improvement, a barrel-vaulted wood roof was installed which ties the former open court together (See Figure 4). The foundation system for this building consists of concrete wall footings. Due to the use of concrete shear walls with flexible diaphragms, this building is classified as Type C2A per ASCE 31 terminology.

Building B

Building B was designed in 1948-1949. The original construction was built in 1948 and consisted of two classrooms. Four classrooms were added to this building in 1950. The single
story building has an overall plan area of approximately 6,850 square feet. The gravity load carrying system generally consists of a diaphragm of 1” solid diagonal sheathing over double 2x4 rafters at 24” which are then supported by 4” steel I-beams spaced approximately 5'-1” apart. The I-beams are supported by 12”, 27 lb./ft. wide flange steel girders spaced 10'-10” apart. The roofing system is supported by 5” std. pipe columns within bearing walls that also include 2x6 studs at 16”. The lateral load resisting system consists of the same 1” solid diagonal sheathing diaphragm which distributes shear loads into the 1” solid diagonal sheathing shear walls. The foundation system consists of concrete strip footings. With wood diaphragms, steel beams and posts, and other intermediary wood members, this building is classified as a Building Type W2, per ASCE 31 terminology.

**Building C4**

Building C4 was designed and constructed in 1958. The single story building has an overall plan area of approximately 5,376 square feet. The gravity load carrying system generally consists of a diaphragm of 1/2” plywood over trussed 2x6 wood rafters which are then supported by 6x4 continuous top plates. The top plates are supported by stud walls which consist of 4x6 posts and 2x6 studs @ 16” on center. The walls are anchored to 2x6 sill plates which are then bolted to concrete foundations consisting of a 6” high curb and varied concrete footing dimensions. The lateral load resisting system consists of the same ½” plywood sheathing for the diaphragms which distributes lateral loads in to ½” plywood sheathed shear walls. This building is classified as a Building Type W2, per ASCE 31 terminology.

**C.2 Building Reference Documents**

**Building A**

Only one sheet (main floor plan) of the original drawings was retrieved. The drawing is dated 1926 and prepared by Louis N Crawford, Architect. Undated tenant improvement drawings prepared by “Arendt / Mosher / Grant / Pedersen / Phillips Architects” were recovered as well. Other building and site specific documentation was not available for review.

**Building B**

The available drawings are dated December 16, 1949, and were prepared by Winsor Soule F.A.I.A. and John Frederic Murphy A.I.A. Architects. Other building and site specific documentation was not available for review.
Building C4

The available drawings are dated May 9, 1958, and were prepared by Howell Arendt Mosher & Grant Architects & Planning Consultants. Other building and site specific documentation was not available for review.

C.3 Site Visit

Crosby Group conducted a site visit on September 20-21, 2012, to validate existing conditions. Another objective of the site visit is to identify potential deficiencies, unusual conditions and details. Additionally, the site visit is meant to compare the existing documents (made available to Crosby Group) with actual field conditions and to identify any discrepancies.

D. Tier 1 Evaluation Findings

Building A

The majority of the items from the ASCE 31-03 Tier 1 checklists were noted compliant except two items relating to the attachment of the interior and exterior concrete walls to the wood roof system and the construction of the wood roof system or more specifically the lack of roof sheathing (plywood or OSB board) and the proper nailing of this system to the roof joists and purlins. The mitigation strategies of these non-compliant items will be described in Section E. The damage caused by the absence of a positive horizontal connection between the roof system and heavy concrete walls has been well documented from past earthquakes and has been shown to be responsible for local and occasionally large areas of collapse. Undated tenant-improvement drawings indicate many areas of proposed work which could have addressed this deficiency; however it was noted during our recent site visit that this strengthening work does not appear to have been completed.

All areas of Building A, excluding the barrel vault over the dining area, are made from 1x6 straight continuous sheathing. In contrast to plywood, OSB or diagonally sheathed wood roof diaphragms, diaphragms composed of straight sheathing (1x4 or 1x6 wood members) are no longer used as they have limited capacity to transfer horizontal inertial loads. Where the span to width aspect ratio is also high, they are particularly vulnerable. The aspect ratio for the straight sheathed diaphragm at the auditorium is 2.25:1 which exceeds the maximum 2:1 ratio recommended by ASCE 31. Additionally, this diaphragm has a span longer than 40 feet which is
noncompliant. A majority of Building A has similar diaphragm spans and since they are intended to brace the heavy interior and exterior walls, these diaphragms should be strengthened.

Reinforcing steel ratios were also found to be below the minimum ratios (.0015 reinforcement area-to-gross concrete area) recommended in ASCE 31. A concrete shear wall with insufficient or low amounts of steel reinforcement can sustain excessive deflection and or flexural and shear cracking when subjected to large seismic events. This can lead extensive damage when demands are high and capacities are low. However, for Building A the average shear stress ratios (demand/capacities) are somewhat low due to the large number of walls present and are within the allowable threshold established within ASCE 31. As such, they can be deemed compliant. Similarly, calculations for out-of-plane loading and associated flexural stresses indicate that the existing walls are also adequate.

**Building B**

Building B was evaluated as compliant with all of the items in the ASCE 31-03 Tier 1 Screening Phase structural checklist. The shear stress quick check for the shear walls passed the allowable loads criteria identified by ASCE 31. Narrow shear walls, with aspect ratios greater than 2-to-1 are present in this building; however, these walls were not included in the calculations for checking the stress on the shear walls. Therefore, we conclude that this building passes the Tier 1 checklist item for narrow shear walls.

**Building C4**

Building C4 was evaluated to be mostly compliant with the checklist items in the ASCE 31 Tier 1 Screening Phase. The primary calculation in this phase is checking the average shear stress of the shear walls. For this building, the average shear stress expected in the shear walls was 707 plf which is lower than the listed allowable shear stress of 1000 plf for wood structural panels (see appended calculations). However, another checklist item requires that narrow wood shear walls not be used to resist lateral forces developed in the building in levels of moderate and high seismicity. In Building C4, there are many narrow wood shear walls present in the longitudinal direction of the building layout. These narrow shear walls were found to be necessary for the average shear stress of the walls to fall under the 1000 plf allowable maximum threshold. Therefore, this item was found to be non-compliant. A Tier 2 evaluation was performed where overturning and shear demands were calculated to determine whether the narrow shear walls
were compliant or non-compliant. The shear demand was found to be within the allowable loads. The expected overturning demands on the existing J-bolt hold-down anchors were also found to be within the capacity provided by the anchors. Therefore, compliance from this Tier 2 analysis replaces the Tier 1 check.

E. Conclusion & Mitigation Strategy

Building A

The seismic evaluation of Building A yielded two main deficiencies. They included the inadequate connection between the roof system and exterior concrete walls, and the absence of proper roof structural sheathing throughout the entire roof system excluding the barrel vault. It is our recommendation that these existing connections be retrofitted as shown on the attached plans and details sheets S61 and S62 attached to this report as Appendix E.

Since an ASCE 31 evaluation simply assesses whether or not a particular component is compliant, we have also grouped each of our found deficiencies into two categories termed Priority 1 and Priority 2. Based upon our experience with similar buildings, we believe Priority 1 items to pose the greatest risk to Life-Safety and based upon the particular building circumstances, are those that can generally lead to local areas of collapse under moderate to large earthquakes. We have chosen to group those upgrades around the auditorium area as Priority 1 due to the high concrete walls present as well as the occupancy of the space below. Priority 2 items include the remainder of the concrete to wall horizontal connections as well as additional plywood sheathing throughout the remainder of the roof system excluding the barrel vault. In our opinion, while noncompliant and requiring strengthening to reach a life safety level, we believe these areas pose a slightly lesser risk to life-safety than the Priority 1 items. We would encourage the owner to complete both Priority 1 and Priority 2 items at the same time and have grouped them so that appropriate cost-benefit-risk decisions can be made. Priority 1 and 2 items are as follows:

Building A - Priority 1 Items

1. Existing wall anchorage-to-diaphragm connections at the Auditorium were found to be significantly overstressed. See Details 3 and 8 on drawing sheet S-2.

2. The long span diaphragm over the main auditorium does not have compliant sheathing and is therefore considered non-compliant per ASCE 31’s guidelines for Life Safety
compliance. It is recommended that this area be sheathed with ½" plywood as indicated on plan sheet S-1

3. Existing in plane roof to concrete wall connections at the Auditorium are insufficient to transfer anticipated seismic loads and should be strengthened per Detail 7 on sheet S-2.

**Building A - Priority 2 Items**

1. The existing roof diaphragm to wall connections throughout the remainder of the main building was found to be overstressed and non-compliant. Since these walls are partially restrained at the base and are a maximum of 11'-0" high, we believe that the risk of local collapse is less than that of similar connections at the Auditorium. Nonetheless, conditions such as these often lead to significant structural and non-structural damage and in rare cases, local collapse. These details are listed as Priority 2 items and are addressed in details 1, 2, 5, 6, 9, 15, and 18 on sheet S2.

2. All existing roof sheathing, except for the area of the barrel vault over the exiting dining room, is composed of 1x straight sheathing which has been shown to have very low capacity to resist and transmit seismic forces. We recommend that these areas be sheathed with new ½" plywood throughout the structure as indicated on sheet S1.

3. Concrete shear walls were found to have insufficient reinforcement ratios per the ASCE 31 structural checklist; however an additional analysis shows that these walls are not overstressed against expected earthquake loads. Mitigation is not recommended at this time.

**Building B**

All structural checklist items were found to be compliant per ASCE 31-03’s Tier 1 Screening Phase. As a result, no mitigation strategy for structural-related items is necessary at this time for Building B.

**Building C4**

All structural checklist items were ultimately found to be compliant. The narrow shear walls check from the ASCE 31 Tier 1 checklist was originally noncompliant; however a further Tier 2 analysis shows that the narrow shear walls are expected to provide a performance level consistent with Life Safety standards. The narrow walls are not expected to uplift nor fail from applied shear loads. Therefore, no mitigation strategy is required for structural-related items in this building.
Appendix A – Site Photos

Figure 2 - Front Entrance of Building A

Figure 3 – Building A, Main Auditorium, Roof Truss
Figure 4 – Building A, Barrel Vaulted Roof Addition

Figure 5 – Building B
Figure 6 – Building B, Original building to the left, new addition to the right

Figure 7 – Building C4
Appendix B – Evaluation Method & Tier 1 Checklists
## Table 3-1. Benchmark Buildings

<table>
<thead>
<tr>
<th>Building Type 1,2</th>
<th>Model Building Seismic Design Provisions</th>
<th>FEMA 178 16</th>
<th>FEMA 310 16,18</th>
<th>CBC 16</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wood Frame, Wood Shear Panels (Type W1A)</td>
<td>* * 1997 2000</td>
<td>1977 *</td>
<td>1908 1973</td>
<td></td>
</tr>
<tr>
<td>Steel Moment-Resisting Frame (Type S1 &amp; S1A)</td>
<td>* * 1994 2000</td>
<td>* *</td>
<td>1908 1995</td>
<td></td>
</tr>
<tr>
<td>Light Metal Frame (Type S3)</td>
<td>* * 2000</td>
<td>*</td>
<td>1992 1998 1973</td>
<td></td>
</tr>
<tr>
<td>Steel Frame w/ Concrete Shear Walls (Type S4)</td>
<td>1993 1994 1976 2000</td>
<td>1985 *</td>
<td>1992 1908 1973</td>
<td></td>
</tr>
<tr>
<td>Steel Frame with URM Infill (Type S5, S5A)</td>
<td>* * 2000</td>
<td>*</td>
<td>1998 *</td>
<td></td>
</tr>
<tr>
<td>Concrete Frame with URM Infill (Type C3 &amp; C3A)</td>
<td>* * 2000</td>
<td>*</td>
<td>1998 *</td>
<td></td>
</tr>
<tr>
<td>Till-up Concrete (Type PC1 &amp; PC1A)</td>
<td>* * 1997 2000</td>
<td>*</td>
<td>1996 *</td>
<td></td>
</tr>
<tr>
<td>Precast Concrete Frame (Type PC2 &amp; PC2A)</td>
<td>* * 2000</td>
<td>*</td>
<td>1992 1998 1973</td>
<td></td>
</tr>
<tr>
<td>Reinforced Masonry (Type RM1)</td>
<td>* * 1997 2000</td>
<td>*</td>
<td>1998 *</td>
<td></td>
</tr>
<tr>
<td>Reinforced Masonry (Type RM2)</td>
<td>1993 1994 1976 2000</td>
<td>1985 *</td>
<td>1998 *</td>
<td></td>
</tr>
<tr>
<td>Unreinforced Masonry (Type URM) 16</td>
<td>* * 1991 2000</td>
<td>1992 *</td>
<td>*</td>
<td></td>
</tr>
<tr>
<td>Unreinforced Masonry (Type URMA)</td>
<td>* * 2000</td>
<td>*</td>
<td>1998</td>
<td></td>
</tr>
</tbody>
</table>

1 “Building Type” refers to one of the Common Building Types defined in Table 2-2.
2 Buildings are histile sites shall not be considered Benchmark Buildings.
3 Flat Slab Buildings shall not be considered Benchmark Buildings.
4 Steel Moment-Resisting Frames shall comply with the 1994 UBC Emergency Provisions, published September/October 1994, or subsequent requirements.
5 URM buildings evaluated using the ASI Methodology (ASI, 1994) may be considered benchmark buildings.
6 Refers to the GSREB or its predecessor, the Uniform Code of Building Conservation (UCBC).
7 Only buildings designed and constructed or evaluated in accordance with these documents and being evaluated to the Life Safety (LS) Performance Level may be considered Benchmark Buildings.
8 Buildings designed and constructed or evaluated in accordance with these documents and being evaluated to either the Life Safety or Immediate Occupancy (IO) Performance Level may be considered Benchmark Buildings.
9 No benchmark year; buildings shall be evaluated using this standard.
10 Local provisions shall be compared with the UBC.

FEMA 178 (See BSSC, 1992a)
FEMA 310 (See FEMA, 1998).
CBC = California Building Code, California Code of Regulations, Title 24 (CBSC, 1995).

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Figure 8 - Table of Benchmark Buildings (From ASCE 31-03)
### Building A

#### Basic Structural Checklist for Building Type C2A: Concrete Shear Walls with Flexible Diaphragms

<table>
<thead>
<tr>
<th>ITEM</th>
<th>C</th>
<th>NC</th>
<th>N/A</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>LOAD PATH:</strong> The structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1)</td>
<td>C</td>
<td>NC</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>ADJACENT BUILDINGS:</strong> The clear distance between the building being evaluated and any adjacent building shall be greater than 4 percent of the height of the shorter building for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.1.2)</td>
<td>N/A</td>
<td>NC</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>MEZZANINES:</strong> Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3)</td>
<td>N/A</td>
<td>NC</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>WEAK STORY:</strong> The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story, above or below, for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1)</td>
<td>N/A</td>
<td>NC</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>SOFT STORY:</strong> The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80 percent of the average lateral-force-resisting system stiffness of the three stories above or below for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2)</td>
<td>N/A</td>
<td>NC</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>GEOMETRY:</strong> There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3)</td>
<td>N/A</td>
<td>NC</td>
<td>C</td>
</tr>
<tr>
<td><strong>VERTICAL DISCONTINUITIES:</strong> All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4)</td>
<td>C</td>
<td>NC</td>
<td>N/A</td>
</tr>
<tr>
<td><strong>MASS:</strong> There shall be no change in effective mass more than 50 percent from one story to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses, and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5)</td>
<td>N/A</td>
<td>NC</td>
<td>C</td>
</tr>
<tr>
<td><strong>DETERIORATION OF WOOD:</strong> There shall be no signs of</td>
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</tbody>
</table>

Cirrus Group Comment:
decay, shrinkage, splitting, fire damage, or sagging in any of the wood members and none of the metal connection hardware shall be deteriorated, broken, or loose. (Tier 2: Sec. 4.3.3.1)

C DETERIORATION OF CONCRETE: There shall be no visible deterioration of concrete or reinforcing steel in any of the vertical- or lateral-force resisting elements. (Tier 2: Sec. 4.3.3.4)

N/A POST-TENSIONING ANCHORS: There shall be no evidence of corrosion or spalling in the vicinity of post-tensioning or end fittings. Coil anchors shall not have been used. (Tier 2: Sec. 4.3.3.5)

C CONCRETE WALL CRACKS: All existing diagonal cracks in wall elements shall be less than 1/8 inch for Life Safety and 1/16 inch for Immediate Occupancy, shall not be concentrated in one location, and shall not form an X pattern. (Tier 2: Sec. 4.3.3.9)

LATERAL-FORCE-RESISTING SYSTEM

C REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.1.1)

C SHEAR STRESS CHECK: The shear stress in the concrete shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than the greater of 100 psi or 2\(\sqrt{f_c}\) for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.1)

The average shear stresses in the concrete shear walls for all buildings were below the allowable values listed to the left.

NC REINFORCING STEEL: The ratio of reinforcing steel area to gross concrete area shall be not less than 0.0015 in the vertical direction and 0.0025 in the horizontal direction for Life Safety and Immediate Occupancy. The spacing of reinforcing steel shall be equal to or less than 18 inches for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.2)

The ratio of reinforcing steel area to gross concrete area in the concrete shear walls is below 0.0015.

CONNECTIONS

NC WALL ANCHORAGE: Exterior concrete or masonry walls that are dependent on the diaphragm for lateral support shall be anchored for out-of-plane forces at each diaphragm level with steel anchors, reinforcing dowels, or straps that are developed into the diaphragm. Connections shall have adequate strength to resist the connection force calculated in the Quick Check procedure of Section 3.5.3.7. (Tier 2: Sec. 4.6.1.1)

The existing anchorage detail is insufficient to transfer and develop the out-of-plane forces from the walls. A seismic retrofit concerning the wall anchorage is recommended.

C TRANSFER TO SHEAR WALLS: Diaphragms shall be connected for transfer of loads to the shear walls for Life Safety and the connections shall be able to develop the
less of the shear strength of the walls or diaphragms for Immediate Occupancy. (Tier 2 Sec. 4.6.2.1)

FOUNDATION DOWELS: Wall reinforcement shall be doweled into the foundation for Life Safety, and the dowels shall be able to develop the lesser of the strength of the walls or the uplift capacity of the foundation for Immediate Occupancy. (Tier 2: Sec. 4.6.3.5)
### Building A

**Supplemental Structural Checklist for Building Type C2A: Concrete Shear Walls with Flexible Diaphragms**

<table>
<thead>
<tr>
<th>C</th>
<th>NC</th>
<th>N/A</th>
<th>Crosby Group Comment:</th>
</tr>
</thead>
</table>

#### LATERAL FORCE RESISTING SYSTEM

**COUPLING BEAMS:** The stirrups in coupling beams over means of egress shall be spaced at or less than \( d/2 \) and shall be anchored into the confined core of the beam with hooks of 135° or more for Life Safety. All coupling beams shall comply with the requirements above and shall have the capacity in shear to develop the uplift capacity of the adjacent wall for Immediate Occupancy. (Tier 2: Sec. 4.4.2.2.3)

**OVERTURNING:** All shear walls shall have aspect ratios less than 4-to-1. Wall piers need not be considered. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.4)

**CONFINEMENT REINFORCING:** For shear walls with aspect ratios greater than 2-to-1, the boundary elements shall be confined with spirals or ties with spacing less than \( 8d_b \). This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.5)

**REINFORCING AT OPENINGS:** There shall be added trim reinforcement around all wall openings with a dimension greater than three times the thickness of the wall. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.6)

**WALL THICKNESS:** Thickness of bearing walls shall not be less than \( 1/25 \) the unsupported height or length, whichever is shorter, nor less than 4 inches. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.2.7)

#### DIAPHRAGMS

**DIAPHRAGM CONTINUITY:** The diaphragms shall not be composed of split-level floors and shall not have expansion joints. (Tier 2: Sec. 4.5.1.1)

**CROSS TIES:** There shall be continuous cross ties between diaphragm chords. (Tier 2: Sec. 4.5.1.2)

**OPENINGS AT SHEAR WALLS:** Diaphragm openings immediately adjacent to the shear walls shall be less than 25 percent of the wall length for Life Safety and 15 percent of the wall length for Immediate Occupancy. (Tier 2: Sec. 4.5.1.4)
PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7)

DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8)

STRAIGHT SHEATHING: All straight sheathed diaphragms shall have aspect ratios less than 2 to 1 for Life Safety and 1-to-1 for Immediate Occupancy in the direction being considered. (Tier 2: Sec. 4.5.2.1)

SPANS: All wood diaphragms with spans greater than 24 feet for Life Safety and 12 feet for Immediate Occupancy shall consist of wood structural panels or diagonal sheathing. (Tier 2: Sec. 4.5.2.2)

UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms shall have horizontal spans less than 40 feet for Life Safety and 30 feet for Immediate Occupancy and shall have aspect ratios less than or equal to 4-to-1 for Life Safety and 3-to-1 for Immediate Occupancy. (Tier 2: Sec. 4.5.2.3)

NON-CONCRETE FILLED DIAPHRAGMS: Untopped metal deck diaphragms or metal deck diaphragms with fill other than concrete shall consist of horizontal spans of less than 40 feet and shall have span/depth ratios less than 4-to-1. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.3.1)

OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Tier 2: Sec. 4.5.7.1)

UPLIFT AT PILE CAPS: Pile caps shall have top reinforcement and piles shall be anchored to the pile caps for Life Safety, and the pile cap reinforcement and pile anchorage shall be able to develop the tensile capacity of the piles for Immediate Occupancy. (Tier 2: Sec. 4.6.3.10)
## Building B

### Basic Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

<table>
<thead>
<tr>
<th>ITEM</th>
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</thead>
<tbody>
<tr>
<td><strong>LOAD PATH:</strong></td>
<td>The structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1)</td>
</tr>
<tr>
<td><strong>MEZZANINES:</strong></td>
<td>Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3)</td>
</tr>
<tr>
<td><strong>WEAK STORY:</strong></td>
<td>The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story, above or below, for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1)</td>
</tr>
<tr>
<td><strong>SOFT STORY:</strong></td>
<td>The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80 percent of the average lateral-force-resisting system stiffness of the three stories above or below for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2)</td>
</tr>
<tr>
<td><strong>GEOMETRY:</strong></td>
<td>There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3)</td>
</tr>
<tr>
<td><strong>VERTICAL DISCONTINUITIES:</strong></td>
<td>All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4)</td>
</tr>
<tr>
<td><strong>MASS:</strong></td>
<td>There shall be no change in effective mass more than 50 percent from one story to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses, and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5)</td>
</tr>
<tr>
<td><strong>DETERIORATION OF WOOD:</strong></td>
<td>There shall be no signs of decay, shrinkage, splitting, fire damage, or sagging in any of the wood members and none of the metal connection hardware shall be deteriorated, broken, or loose. (Tier 2: Sec. 4.3.3.1)</td>
</tr>
<tr>
<td><strong>WOOD STRUCTURAL PANEL SHEAR WALL</strong></td>
<td></td>
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</tbody>
</table>
**FASTENERS:** There shall be no more than 15 percent of inadequate fastening such as overdriven fasteners, omitted blocking, excessive fastening spacing, or inadequate edge distance. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.3.3.2)

**LATERAL FORCE-RESISTING SYSTEM**

<p>| C | REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.1.1) |
| N/A | STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings shall not rely on exterior stucco walls as the primary lateral-force-resisting system (Tier 2: Sec. 4.4.2.7.2) |
| N/A | GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard shall not be used as shear walls on buildings over one story in height with the exception of the uppermost level of a multi-story building. (Tier 2: Sec. 4.4.2.7.3) |
| C | SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than the following values for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.2.7.1): Structural panel sheathing: 1,000 plf Diagonal sheathing: 700 plf Straight sheathing: 100 plf All other conditions: 100 plf Shear stress of diagonal sheathed walls: 515 plf &lt; allowable 700 plf OK (See Appendix C) |
| N/A | NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2:1 for Life Safety and 1.5:1 for Immediate Occupancy shall not be used to resist lateral forces developed in the building in levels of moderate and high seismicity. Narrow wood shear walls with an aspect ratio greater than 2:1 for Immediate Occupancy shall not be used to resist lateral forces developed in the building in levels of low seismicity. (Tier 2: Sec. 4.4.2.7.4) Narrow wood shear walls (greater than 2:1 aspect ratio) exist in the building but were not used in calculations of the shear stress check. Therefore, this item is judged to be compliant. |
| N/A | WALLS CONNECTED THROUGH FLOORS: Shear walls shall have interconnection between stories to transfer overturning and shear forces through the floor. (Tier 2: Sec. 4.4.2.7.5) |
| N/A | HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story due to a sloping site, all shear walls on the downhill slope shall have an aspect ratio less than 1:1 for Life Safety and 1:2:1 for Immediate Occupancy. (Tier 2: Sec. 4.4.2.7.6) |</p>
<table>
<thead>
<tr>
<th>Code</th>
<th>Description</th>
</tr>
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<tbody>
<tr>
<td>N/A</td>
<td><strong>CRIPPLE WALLS</strong>: Cripple walls below first-floor-level shear walls shall be braced to the foundation with wood structural panels. (Tier 2: Sec. 4.4.2.7.7)</td>
</tr>
<tr>
<td>C</td>
<td><strong>OPENINGS</strong>: Walls with openings greater than 80 percent of the length shall be braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or shall be supported by adjacent construction through positive ties capable of transferring the lateral forces. (Tier 2: Sec. 4.4.2.7.8)</td>
</tr>
<tr>
<td>C</td>
<td><strong>CONNECTIONS</strong></td>
</tr>
<tr>
<td>C</td>
<td><strong>WOOD POSTS</strong>: There shall be a positive connection of wood posts to the foundation. (Tier 2: Sec. 4.6.3.3)</td>
</tr>
<tr>
<td>C</td>
<td><strong>WOOD SILLS</strong>: All wood sills shall be bolted to the foundation. (Tier 2: Sec. 4.6.3.4)</td>
</tr>
<tr>
<td>C</td>
<td><strong>GIRDER/COLUMN CONNECTION</strong>: There shall be a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Tier 2: Sec. 4.6.4.1)</td>
</tr>
</tbody>
</table>
## Building B

### Supplemental Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

<table>
<thead>
<tr>
<th>ITEM</th>
<th>C</th>
<th>NC</th>
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<th>Crosby Group Comment:</th>
</tr>
</thead>
<tbody>
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<td></td>
<td></td>
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<td></td>
</tr>
</tbody>
</table>

### LATERAL-FORCE-RESISTING SYSTEM

- **N/A** HOLD-DOWN ANCHORS: All shear walls shall have hold-down anchors constructed per acceptable construction practices, attached to the end studs. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.7.9)

### DIAPHRAGMS

- **C** DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors and shall not have expansion joints. (Tier 2: Sec. 4.5.1.1)

- **C** ROOF CHORD CONTINUITY: All chord elements shall be continuous, regardless of changes in roof elevation. (Tier 2: Sec. 4.5.1.3)

- **N/A** PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7)

- **N/A** DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8)

- **N/A** STRAIGHT SHEATHING: All straight sheathed diaphragms shall have aspect ratios less than 2-to-1 for Life Safety and 1-to-1 for Immediate Occupancy in the direction being considered. (Tier 2: Sec. 4.5.2.1)

- **C** SPANS: All wood diaphragms with spans greater than 24 feet for Life Safety and 12 feet for Immediate Occupancy shall consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Tier 2: Sec. 4.5.2.2)

- **C** UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms shall have horizontal spans less than 40 feet for Life Safety and 30 feet for Immediate Occupancy and shall have aspect ratios less than or equal to 4-to-1 for Life Safety and 3-to-1 for Immediate Occupancy. (Tier 2: Sec. 4.5.2.3)
OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Tier 2: Sec. 4.5.7.1)

WOOD SILL BOLTS: Sill bolts shall be spaced at 6 feet or less for Life Safety and 4 feet or less for Immediate Occupancy, with proper edge and end distance provided for wood and concrete. (Tier 2: Sec. 4.6.3.9)
# Building C4

## Basic Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

<table>
<thead>
<tr>
<th>ITEM</th>
<th>C</th>
<th>NC</th>
<th>N/A</th>
<th>Crosby Group Comment:</th>
</tr>
</thead>
</table>

### BUILDING SYSTEM

**LOAD PATH:** The structure shall contain a minimum of one complete load path for Life Safety and Immediate Occupancy for seismic force effects from any horizontal direction that serves to transfer the inertial forces from the mass to the foundation. (Tier 2: Sec. 4.3.1.1)

**MEZZANINES:** Interior mezzanine levels shall be braced independently from the main structure, or shall be anchored to the lateral-force-resisting elements of the main structure. (Tier 2: Sec. 4.3.1.3)

**WEAK STORY:** The strength of the lateral-force-resisting system in any story shall not be less than 80 percent of the strength in an adjacent story, above or below, for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.1)

**SOFT STORY:** The stiffness of the lateral-force-resisting system in any story shall not be less than 70 percent of the lateral-force-resisting system stiffness in an adjacent story above or below, or less than 80 percent of the average lateral-force-resisting system stiffness of the three stories above or below for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.3.2.2)

**GEOMETRY:** There shall be no changes in horizontal dimension of the lateral-force-resisting system of more than 30 percent in a story relative to adjacent stories for Life Safety and Immediate Occupancy, excluding one-story penthouses and mezzanines. (Tier 2: Sec. 4.3.2.3)

**VERTICAL DISCONTINUITIES:** All vertical elements in the lateral-force-resisting system shall be continuous to the foundation. (Tier 2: Sec. 4.3.2.4)

**MASS:** There shall be no change in effective mass more than 50 percent from one story to the next for Life Safety and Immediate Occupancy. Light roofs, penthouses, and mezzanines need not be considered. (Tier 2: Sec. 4.3.2.5)

**DETERIORATION OF WOOD:** There shall be no signs of decay, shrinkage, splitting, fire damage, or sagging in any...
of the wood members and none of the metal connection hardware shall be deteriorated, broken, or loose. (Tier 2: Sec. 4.3.3.1)

N/A WOOD STRUCTURAL PANEL SHEAR WALL FASTENERS: There shall be no more than 15 percent of inadequate fastening such as overdriven fasteners, omitted blocking, excessive fastening spacing, or inadequate edge distance. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.3.3.2)

LATERAL FORCE-RESISTING SYSTEM

C REDUNDANCY: The number of lines of shear walls in each principal direction shall be greater than or equal to 2 for Life Safety and Immediate Occupancy. (Tier 2: Sec. 4.4.2.1.1)

C SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 3.5.3.3, shall be less than the following values for Life Safety and Immediate Occupancy (Tier 2: Sec. 4.4.2.7.1):
  - Structural panel sheathing: 1,000 plf
  - Diagonal sheathing: 700 plf
  - Straight sheathing: 100 plf
  - All other conditions: 100 plf

The building has an average shear stress of 707 plf, which is under the allowable 1,000 plf for structural panel sheathing. However, narrow shear walls were required to produce a satisfactory shear stress. See “Narrow Shear Walls” below for more information.

N/A STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings shall not rely on exterior stucco walls as the primary lateral-force-resisting system (Tier 2: Sec. 4.4.2.7.2)

N/A GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard shall not be used as shear walls on buildings over one story in height with the exception of the uppermost level of a multi-story building. (Tier 2: Sec. 4.4.2.7.3)

NC NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 for Life Safety and 1.5-to-1 for Immediate Occupancy shall not be used to resist lateral forces developed in the building in levels of moderate and high seismicity. Narrow wood shear walls with an aspect ratio greater than 2-to-1 for Immediate Occupancy shall not be used to resist lateral forces developed in the building in levels of low seismicity. (Tier 2: Sec. 4.4.2.7.4)

The building possesses many narrow shear walls with an aspect ratio greater than 2:1. These narrow shear walls are likely to be “highly stressed and subject to severe deformations that will reduce the capacity of the walls” (ASCE 316-03 §C4.4.2.7.4). A Tier 2 Evaluation was performed and overturning and shear demands were calculated. The hold-down capacity is insufficient for the overturning forces. A seismic retrofit addressing this detail is recommended.

N/A WALLS CONNECTED THROUGH FLOORS: Shear walls
shall have interconnection between stories to transfer overturning and shear forces through the floor. (Tier 2: Sec. 4.4.2.7.5)

N/A HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story due to a sloping site, all shear walls on the downhill slope shall have an aspect ratio less than 1-to-1 for Life Safety and 1-to-2 for Immediate Occupancy. (Tier 2: Sec. 4.4.2.7.6)

N/A CRIPPLE WALLS: Cripple walls below first-floor-level shear walls shall be braced to the foundation with wood structural panels. (Tier 2: Sec. 4.4.2.7.7)

C OPENINGS: Walls with openings greater than 80 percent of the length shall be braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or shall be supported by adjacent construction through positive ties capable of transferring the lateral forces. (Tier 2: Sec. 4.4.2.7.8)

CONNECTIONS

C WOOD POSTS: There shall be a positive connection of wood posts to the foundation. (Tier 2: Sec. 4.6.3.3)

C WOOD SILLS: All wood sills shall be bolted to the foundation. (Tier 2: Sec. 4.6.3.4)

C GIRDER/COLUMN CONNECTION: There shall be a positive connection utilizing plates, connection hardware, or straps between the girder and the column support. (Tier 2: Sec. 4.6.4.1)

Building C4

Supplemental Structural Checklist for Building Type W2: Wood Frames, Commercial and Industrial

<table>
<thead>
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</tr>
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<tbody>
<tr>
<td>LATERAL-FORCE-RESISTING SYSTEM</td>
<td></td>
</tr>
<tr>
<td>N/A</td>
<td>HOLD-DOWN ANCHORS: All shear walls shall have hold-down anchors constructed per acceptable construction practices, attached to the end studs. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.4.2.7.9)</td>
</tr>
<tr>
<td>DIAPHRAGMS</td>
<td></td>
</tr>
<tr>
<td>C</td>
<td>DIAPHRAGM CONTINUITY: The diaphragms shall not be composed of split-level floors and shall not have expansion joints. (Tier 2: Sec. 4.5.1.1)</td>
</tr>
<tr>
<td>C</td>
<td>ROOF CHORD CONTINUITY: All chord elements shall be continuous, regardless of changes in roof elevation. (Tier</td>
</tr>
</tbody>
</table>
PLAN IRREGULARITIES: There shall be tensile capacity to develop the strength of the diaphragm at re-entrant corners or other locations of plan irregularities. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.7)

DIAPHRAGM REINFORCEMENT AT OPENINGS: There shall be reinforcing around all diaphragm openings larger than 50 percent of the building width in either major plan dimension. This statement shall apply to the Immediate Occupancy Performance Level only. (Tier 2: Sec. 4.5.1.8)

STRAIGHT SHEATHING: All straight sheathed diaphragms shall have aspect ratios less than 26 to 61 for Life Safety and 16 to 61 for Immediate Occupancy in the direction being considered. (Tier 2: Sec. 4.5.2.1)

SPANS: All wood diaphragms with spans greater than 24 feet for Life Safety and 12 feet for Immediate Occupancy shall consist of wood structural panels or diagonal sheathing. Wood commercial and industrial buildings may have rod-braced systems. (Tier 2: Sec. 4.5.2.2)

UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms shall have horizontal spans less than 40 feet for Life Safety and 30 feet for Immediate Occupancy and shall have aspect ratios less than or equal to 4 to 1 for Life Safety and 3 to 1 for Immediate Occupancy. (Tier 2: Sec. 4.5.2.3)

OTHER DIAPHRAGMS: The diaphragm shall not consist of a system other than wood, metal deck, concrete, or horizontal bracing. (Tier 2: Sec. 4.5.7.1)

WOOD SILL BOLTS: Sill bolts shall be spaced at 6 feet or less for Life Safety and 4 feet or less for Immediate Occupancy, with proper edge and end distance provided for wood and concrete. (Tier 2: Sec. 4.6.3.9)
Building A - Seismic Dead Loads

**Typical Roof/Ceiling**

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing</td>
<td>2.0</td>
</tr>
<tr>
<td>1&quot; Straight Sheath.</td>
<td>2.5</td>
</tr>
<tr>
<td>Rafters/framing</td>
<td>3.0</td>
</tr>
<tr>
<td>Lath and Plaster</td>
<td>8.0</td>
</tr>
<tr>
<td>MEP</td>
<td>1.0</td>
</tr>
<tr>
<td>Misc</td>
<td>1.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>18.0</strong></td>
</tr>
</tbody>
</table>

**Auditorium/Barrel Vault Roof**

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing</td>
<td>2.0</td>
</tr>
<tr>
<td>Sheathing</td>
<td>2.5</td>
</tr>
<tr>
<td>Framing/Trusses</td>
<td>5.5</td>
</tr>
<tr>
<td>MEP</td>
<td>1.0</td>
</tr>
<tr>
<td>Misc</td>
<td>1.0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>12.0</strong></td>
</tr>
</tbody>
</table>

**Floating Ground Floor**

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Carpet/finishing</td>
<td>3.0</td>
</tr>
<tr>
<td>Subfloor</td>
<td>2.5</td>
</tr>
<tr>
<td>Joists/framing</td>
<td>3.0</td>
</tr>
<tr>
<td>MEP</td>
<td>1.0</td>
</tr>
<tr>
<td>Misc</td>
<td>2.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>9.0</strong></td>
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</tbody>
</table>

**Walls:**

**10" Concrete Walls**

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5&quot; Stucco Finish</td>
<td>15</td>
</tr>
<tr>
<td>10&quot; Concrete</td>
<td>125</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>140.0</strong></td>
</tr>
</tbody>
</table>

**8" Concrete Walls**

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.5&quot; Stucco Finish</td>
<td>15</td>
</tr>
<tr>
<td>8&quot; Concrete</td>
<td>100</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>115.0</strong></td>
</tr>
</tbody>
</table>
**PSEUDO LATERAL FORCE %**

PER A2E-31-03 § 3:5:2.1

\[ V = C \times S_a \times W \]  \hspace{2cm} (EQ 3-1)

\[ C = 1.0 \]  \hspace{2cm} (TABLE 3-4)

\[ S_a = \frac{S_{D1}}{T} \]  \hspace{2cm} (§ 3:5:2.3.1)

\[ S_a \] SHALL NOT EXCEED 0.665

WHERE:

\[ S_{D1} = 0.665 \]

\[ S_D = 1.138 \]

\[ T = C_t \times h_n^k \]  \hspace{2cm} (EQ 3-8)

\[ C_t = 0.020 \]

\[ h_n = 15 \text{ ft} \]

\[ k = 0.75 \]

\[ T = (0.020)(15 \text{ ft})^{0.75} = 0.152 \text{ sec.} \]

\[ S_a = \frac{0.665}{0.152} = 4.375 \geq 1.138 \therefore S_a = 1.138 \]

\[ V = (1.0)(1.138)W \]

\[ V = 1.138W \]
**Building A - Seismic Base Shear**

\[ V_{BASE} = CS_a W = 1.138W \]

- Typ Roof DL = 18.0 psf
- Aud/B. Vault DL = 12.0 psf
- Floor DL = 9.0 psf

- Roof Area = 14937 ft\(^2\)
- Aud/ B. Vault Area = 7395 ft\(^2\)
- Floor Area = 14580 ft\(^2\)

- 8" Walls N/S Dirc = 349 kips
- 10" Walls N/S Dirc = 864 kips
- 8" Walls E/W Dirc = 530 kips
- 10" Walls E/W Dirc = 363 kips

- Seismic Wt, \(W_{N/S} = 1702\) kips
- Seismic Wt, \(W_{E/W} = 1382\) kips

\[ V_{N/S} = 1937 \text{ kips} \]

\[ V_{E/W} = 1572 \text{ kips} \]
PSEUDO LATERAL FORCE/BASE SHEAR

**Typical Roof**: 18 PSF
**Hedetone or Barrel Vault Roof**: 12 PSF
**Floating Ground Floor**: 9.0 PSF

\[ V = 1.12 \text{ kips} \]

**Identify Individual Demands**

---

**Plan View**

**Scale**: 1/4" = 1'-0"
**Notes**: 3/12 = 3'-0"

**Shear Stress in Shear Walls**: 
\[ V_y = \frac{1}{2} \left( \frac{V}{2A} \right) \]
**Left Deer Room 10-52**

**Room Area:**

\[ 141' \times 24' = 3384 \text{ ft}^2 \quad \text{(Classroom)} \]

\[ \frac{1}{2}(44' \times 13' + 44' \times 32') = 940 \text{ ft}^2 \quad \text{(Pedestrian)} \]

\[ 126' \times 11' = 1386 \text{ ft}^2 \quad \text{(Local Hallways)} \]

\[ \Sigma = 5670 \text{ ft}^2 \]

**Floor Area:**

\[ 3384 + 940 + 47 \times 11 = 4801 \text{ ft}^2 \]

**Walls:**

- **Lengthwise:**
  \[ 24' \times 4 = 96 \text{ ft} \quad \text{(Transverse, Dir)} \]
  \[ 96' \times 8' \text{ Treb HT} \times 140 \text{ PSF} = 108 \text{ kips} \]

- **Longitudinal:**
  \[ 141' \times 2 = 282 \text{ ft} \quad \text{(Longitudinal Dir)} \]
  \[ 282' \times 8' \text{ Treb HT} \times 140 \text{ PSF} = 316 \text{ kips} \]

**Total Seismic Weight:**

\[ (5670 \times 100 \text{ PSF} + 4801 \times 9.75 \text{ PSF}) / 1000 \]

\[ + 108k + 316k = 572k \]

\[ V = 1.15 \times 572k = 659k \]
Calculate Area of Walls

Length: Approx. 138 ft

\[ 138 \text{ ft} \times 12 \times 8'' = 13248 \text{ in}^2 \]

\[ V_{\text{anch}} = \frac{1}{4} \times \frac{651000 \text{ ft}}{13248 \text{ in}^2} = 12.3 \text{ ft}^3 \leq \left\{ \begin{array}{l} 100 \text{ ft}^3 \text{ max} \\ 20 \text{ ft}^3 \text{ recommended} \end{array} \right. \]

\( \therefore \text{ OK} \quad \checkmark \)
**Middle Diaphragm (10'5")**

**Floor Area:**

\[
40' \times (76' + 32') = 4320 \text{ ft}^2
\]
\[
(54' \times 32') \frac{1}{2} = 864 \text{ ft}^2
\]
\[
(44' \times 13') \frac{1}{2} = 286 \text{ ft}^2
\]
\[
(32' \times 32') \frac{1}{2} = 832 \text{ ft}^2
\]
\[
6' \times 75' = 450 \text{ ft}^2
\]

\[Z = 6752 \text{ ft}^2\]

**Floor Area:**

\[6752 \text{ ft}^2 - 286 \text{ ft}^2 = 6466 \text{ ft}^2\]

**Wall Lengths:**

\[(76' + 32') \times 2 + 40' \times 2 + 34' \frac{1}{2} \times 2 \times 2\]
\[+ 40' \frac{1}{2} \times 2 \times 2 + 40' = 5886 \text{ ft}\]

\[5886 \text{ ft} \times 11' \text{ TIE PLOTED WIDTH} \times 140 \text{ PSF} = 903K\]

**Total Seismic Weight:**

\[
\left(\frac{6752 \text{ ft}^2 \times 10 \text{ PSF} + 6466 \text{ ft}^2 \times 9 \text{ PSF}}{1000}
\right) + 903K = 1082K
\]

\[V = 1.138K = 1232 \text{ KIPS}\]

**Calculate Area of Walls**

Length of Wall's about 46' (limit to 10' walls)

\[40' \times 12' \times 8'' = 4416 \text{ in}^2\]

\[V_{\text{AVE}} = \frac{1}{4} \frac{1232000 \#}{4416 \text{ in}^2} = 70 \text{ PSI} < \sqrt[2]{100} \text{ PSI}\]

\[\checkmark \text{ OK}\]
Roof Area:

\[191' \times 24' = 3384 \text{ ft}^2\]
\[(32' \times 32') / 2 = 832 \text{ ft}^2\]
\[(73' \times 9') / 2 = 339 \text{ ft}^2\]
\[\sum = 4554 \text{ ft}^2\]

Floor Area:

\[4554 \text{ ft}^2 - 339 \text{ ft}^2 = 4216 \text{ ft}^2\]

Wall Lengths:

\[141' \times 2 + 24' \times 4 + 32' \times 2 / 2 = 430 \text{ ft}\]

Total Seismic Weight:

\[18 \text{ psf} \times 4554 \text{ ft}^2 + 9 \text{ psf} \times 4216 \text{ ft}^2\]
\[+430 \text{ ft} \times 101'3' / 12 \times 140 \text{ psf} = 707 \text{ k}\]
\[V = 1.138 \omega = 8013 \text{ k}\]

Calculate Area of Walls

Lengths: About 118' (Exclude Narrow Walls)

Area: 118' \times 12' \times 8'' = 11328 \text{ in}^2

\[V_{\text{avg}} = \frac{1}{4} \times \frac{8013 \text{ k}}{11328 \text{ in}^2} = 17.8 \text{ psf} < 5100 \text{ psf}\]

\[\checkmark\]
BARREL VAULTED ROOM

Roof Area: 9' Floor Area:

37' x 63 = 2331 ft²

Length of Walls:

Approx. 200' (including both directions)

Total Seismic Weight:

2331 ft² (12 psf + 9 psf) + 200' x 8' x 140 plf

= 273 k

V = 1.138 x 20 = 310.7 kips

Calculate Area of Walls:

Approx. 24'² (excluding narrow walls)

A = 24' x 8" x 12 = 2304 ft²

V_s, Anch. = \frac{1}{4} \cdot \frac{3107000 ft}{2304 ft²} = 337.7 psf ≤ \frac{100 psf}{2.2 H}

OK
Diaphragm Analysis/Shear Stress

Check in E-0 Direction

Left or Right Classroom Wall Diaphragms

Use 200 (Conservative) from Prev Call for Pseudo Lat. Force

Total Length of E-0 Shear Wall: Approx 95'
Area: 95' x 12' x 8" = 9120 sq ft

\[ V_{S, \text{ANC}} = \frac{1}{4} \frac{9120,000\#}{9120 \, \text{sq ft}} = 22.1 \, \text{psf} \leq \frac{2 \times 100 \, \text{psf}}{240} \]

\[ 0.8 \, \text{psf} \, \text{OK} \checkmark \]

Middle Diaphragm:

Use 1232# from Prev. Call for Pseudo Lat. Forces

Total Length of E-0 Shear Wall: Approx 85'
Area: 85' x 12' x 8" = 52800 sq ft

\[ V_{S, \text{ANC}} = \frac{1}{4} \frac{1222,000\#}{52800 \, \text{sq ft}} = 58.3 \, \text{psf} \leq \frac{100 \, \text{psf}}{240} \]

\[ 0.8 \, \text{psf} \, \text{OK} \checkmark \]
Reinforcement Steel Check

(2) 3/8" mats @ 18" O.C. Both Ways

\[ A_s = \pi \left(\frac{3/8}{2}\right)^2 = 0.111 \text{ in}^2 \quad (\text{Circular Bars}) \]

\[ A_s = \left(\frac{3}{8}\right)^2 = 0.114 \quad (\text{Square Bars}) \]

\[ \rho = \frac{A_s}{bd} = \frac{0.111 \times 2}{18 \times 8} = 0.0015 \]

\[ = \frac{0.114 \times 2}{18 \times 8} = 0.0014 \]

\[ 0.0025 \text{ min.} \]

For Horiz. Dir.

\[ 0.0015 \text{ per axe at checks.} \]

See ENF-1201 (Existing Reinforcement) for out-of-plane loads.
Concrete Slender Wall

Description: Goleta - Main Building, OOP reinforcement check

Code References
Calculations per ACI 318-05 Sec 14.8, IBC 2006, CBC 2007, ASCE 7-05
Load Combinations Used: ASCE 7-05

General Information

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>f_c: Concrete 28 day strength</td>
<td>2.50 ksi</td>
</tr>
<tr>
<td>f_y: Rebar Yield</td>
<td>40.0 ksi</td>
</tr>
<tr>
<td>E_c: Concrete Elastic Modulus</td>
<td>3,122.0 ksi</td>
</tr>
<tr>
<td>λ: Lt Wt Conc Factor</td>
<td>1.0</td>
</tr>
<tr>
<td>f_r: Rupture Modulus</td>
<td>250.0 psi</td>
</tr>
<tr>
<td>Max % of p balanced</td>
<td>0.01693</td>
</tr>
<tr>
<td>Max Pul/Ag = f_c*</td>
<td>0.060</td>
</tr>
<tr>
<td>Concrete Density</td>
<td>150.0 pcf</td>
</tr>
<tr>
<td>Width of Design Strip</td>
<td>12.0 in</td>
</tr>
<tr>
<td>Wall Thickness</td>
<td>8.0 in</td>
</tr>
<tr>
<td>Temp Diff across thickness</td>
<td>0.0 deg F</td>
</tr>
<tr>
<td>Min Allow Out-of-Plane Defl Ratio = L/150.0</td>
<td></td>
</tr>
<tr>
<td>Minimum Vertical Steel %</td>
<td>0.00150</td>
</tr>
<tr>
<td>Using Stiff. Reduction Factor per ACI R.10.12.3</td>
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</tbody>
</table>

One-Story Wall Dimensions

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Value</th>
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</thead>
<tbody>
<tr>
<td>A: Clear Height</td>
<td>14.0 ft</td>
</tr>
<tr>
<td>B: Parapet height</td>
<td>0.0 ft</td>
</tr>
</tbody>
</table>

Wall Support Condition: Top & Bottom Pinned

Vertical Loads

<table>
<thead>
<tr>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Ledger Load Eccentricity 4.0 in</td>
<td>0.2160</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0 klf</td>
</tr>
<tr>
<td>Concentric Load</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0</td>
<td>0.0 klf</td>
</tr>
</tbody>
</table>

Lateral Loads

<table>
<thead>
<tr>
<th>Full area WIND load</th>
<th>0.0 psf</th>
</tr>
</thead>
<tbody>
<tr>
<td>F_p = Wall Wt. * 0.6828</td>
<td>68.280 psf</td>
</tr>
</tbody>
</table>

Wall Weight Seismic Load Input Method: ASCE seismic factors entered

Results reported for "Strip Width" of 12.0 in

DESIGN SUMMARY

<table>
<thead>
<tr>
<th>Governing Load Combination</th>
<th>Actual Values</th>
<th>Allowable Values</th>
</tr>
</thead>
<tbody>
<tr>
<td>PASS Moment Capacity Check</td>
<td>Max Mu = 1.706 klf, Min. Defl. Ratio = 4.509.38</td>
<td>1.941 klf</td>
</tr>
<tr>
<td>PASS Service Deflection Check</td>
<td>Max. Deflection = 0.3645 in</td>
<td>1.120 in</td>
</tr>
<tr>
<td>PASS Axial Load Check +1.20D+0.50L+0.20S+E</td>
<td>Max Pul/Ag = 11,450 psi</td>
<td>150.0 psi</td>
</tr>
<tr>
<td>PASS Reinforcing Limit Check</td>
<td>Controlling As/ bd = 0.000905</td>
<td>0.01963</td>
</tr>
<tr>
<td>FAIL Minimum Moment Check</td>
<td>Max Cracking = 2.667 klf</td>
<td>1.709 klf</td>
</tr>
</tbody>
</table>

Maximum Reactions... for Load Combination...

Top Horizontal: E Only
Base Horizontal: E Only
Vertical Reaction: D Only

Licensee: CROSBY GROUP
Lic. #: KW-00023900

ENERCALC, INC. 1993-2012 Build6.12.11.1 Ver6.12.11.1
## Concrete Slender Wall

**Design Maximum Combinations - Moments**

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial Load Pu k</th>
<th>0.06Ftc'b't k</th>
<th>Mcr k/ft</th>
<th>Mu k/ft</th>
<th>Moment Values Phi k/ft</th>
<th>Phi Mn k/ft</th>
<th>As in²</th>
<th>As Ratio</th>
<th>0.6 * rho bal</th>
</tr>
</thead>
<tbody>
<tr>
<td>D Only at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.10</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 0.50L + 1.60L at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 1.80L + 0.50S at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 1.60L + 0.50L at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 0.50L + 1.60S at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 1.80S + 0.80W at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 0.50L + 0.50S at 13.53 to 14.</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 0.50S + 1.80W at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.20D + 0.50L + 0.80E at 7.00 to 7.47</td>
<td>0.999</td>
<td>14.400</td>
<td>2.67</td>
<td>1.72</td>
<td>0.90</td>
<td>2.02</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+0.90D + 1.60W at 13.53 to 14.00</td>
<td>0.000</td>
<td>14.400</td>
<td>2.67</td>
<td>0.09</td>
<td>0.90</td>
<td>1.71</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
<tr>
<td>+0.90D + 0.80E at 7.00 to 7.47</td>
<td>0.824</td>
<td>14.400</td>
<td>2.67</td>
<td>1.71</td>
<td>0.50</td>
<td>1.54</td>
<td>0.073</td>
<td>0.0009</td>
<td>0.0169</td>
</tr>
</tbody>
</table>

## Design Maximum Combinations - Deflections

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial Load Pu k</th>
<th>0.06Ftc'b't k</th>
<th>Mcr k/ft</th>
<th>Mactual k/ft</th>
<th>I gross in⁴</th>
<th>I cracked in⁴</th>
<th>I effective in⁴</th>
<th>Deflection Defl. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>D + L + Lr at 7.93 to 8.40</td>
<td>0.823</td>
<td>2.67</td>
<td>0.04</td>
<td>512.00</td>
<td>31.50</td>
<td>384.000</td>
<td>0.001</td>
<td>128666.4</td>
</tr>
<tr>
<td>D + L + W at 7.93 to 8.40</td>
<td>0.823</td>
<td>2.67</td>
<td>0.04</td>
<td>512.00</td>
<td>31.50</td>
<td>384.000</td>
<td>0.001</td>
<td>128666.4</td>
</tr>
<tr>
<td>D + L + W + 0.5L at 7.93 to 8.40</td>
<td>0.823</td>
<td>2.67</td>
<td>0.04</td>
<td>512.00</td>
<td>31.50</td>
<td>384.000</td>
<td>0.001</td>
<td>128666.4</td>
</tr>
<tr>
<td>D + L + S + W2 + E at 7.93 to 8.40</td>
<td>0.823</td>
<td>2.57</td>
<td>1.23</td>
<td>512.00</td>
<td>31.97</td>
<td>384.000</td>
<td>0.036</td>
<td>4,609.4</td>
</tr>
<tr>
<td>D + L + S + E/4 at 7.00 to 7.47</td>
<td>0.916</td>
<td>2.57</td>
<td>1.23</td>
<td>512.00</td>
<td>31.97</td>
<td>384.000</td>
<td>0.001</td>
<td>128666.4</td>
</tr>
<tr>
<td>D + 0.5(L−Lr) + 0.7W at 7.93 to 8.40</td>
<td>0.823</td>
<td>2.67</td>
<td>0.04</td>
<td>512.00</td>
<td>31.50</td>
<td>384.000</td>
<td>0.001</td>
<td>128666.4</td>
</tr>
<tr>
<td>D + 0.5(L−Lr) + 0.7E at 7.00 to 7.47</td>
<td>0.916</td>
<td>2.67</td>
<td>1.21</td>
<td>512.00</td>
<td>31.97</td>
<td>384.000</td>
<td>0.036</td>
<td>4,700.1</td>
</tr>
</tbody>
</table>

## Reactions - Vertical & Horizontal

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Base Horizontal</th>
<th>Top Horizontal</th>
<th>Vertical @ Wall Base</th>
</tr>
</thead>
<tbody>
<tr>
<td>D Only</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>1.516 k</td>
</tr>
<tr>
<td>S Only</td>
<td>0.0 k</td>
<td>0.00 k</td>
<td>0.000 k</td>
</tr>
<tr>
<td>W Only</td>
<td>0.0 k</td>
<td>0.00 k</td>
<td>0.000 k</td>
</tr>
<tr>
<td>E Only</td>
<td>0.5 k</td>
<td>0.48 k</td>
<td>0.000 k</td>
</tr>
<tr>
<td>D + L + Lr</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>1.516 k</td>
</tr>
<tr>
<td>D + L + S</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>1.516 k</td>
</tr>
<tr>
<td>D + L + W + S2</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>1.516 k</td>
</tr>
<tr>
<td>D + L + S + W2</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>1.516 k</td>
</tr>
<tr>
<td>D + L + S + E/1.4</td>
<td>0.3 k</td>
<td>0.34 k</td>
<td>1.516 k</td>
</tr>
</tbody>
</table>
Concrete Slender Wall

Lic. #: KW-60029000
Description: Goleta - Main Building, Out-of-plane reinforcement check, 10" wall

Code References
Calculations per ACI 318-05 Sec 14.8, IBC 2006, CBC 2007, ASCE 7-05
Load Combinations Used: ASCE 7-05

General Information
\[ f_c: \text{Concrete 28 day strength} = 2.50 \text{ ksi} \]
\[ f_y: \text{Rebar Yield} = 40.0 \text{ ksi} \]
\[ E_c: \text{Concrete Elastic Modulus} = 3,122.0 \text{ ksi} \]
\[ \lambda: \text{LT WI Conc Factor} = 1.0 \]
\[ F_r: \text{Rupture Modulus} = 250.0 \text{ psi} \]
\[ \text{Max % of } \rho_{\text{balanced}} = 0.01683 \]
\[ \text{Max Pu/Ag} = \frac{f_c^*}{f_y} = 0.060 \]
\[ \text{Concrete Density} = 150.0 \text{pcf} \]
\[ \text{Width of Design Strip} = 12.0 \text{ in} \]

Wall Thickness 10.0 in
Temp Diff across thickness = \deg F
Min. Allow Out-of-Plane Defl. Ratio = L / 150.0
Minimum Vertical Steel % = 0.00150
Using Stiff. Reduction Factor per ACI R.10.12.3

One-Story Wall Dimensions

A Clear Height = 20.0 ft
B Parapet height = 0.0 ft

Wall Support Condition: Top & Bottom Pinned

Vertical Loads
Vertical Uniform Loads ... (Applied per foot of Strip Width)

Ledger Load | Concentric Load
--- | ---
Eccentricity 5.0 in | 0.360 | 0.0

0.0 | 0.0 | 0.0 | 0.0

Wall Weight Seismic Load Input Method:
SDS Value per ASCE 12.11.1
S_D = 0.001270

Results reported for "Strip Width" of 12.0 in

DESIGN SUMMARY

D MEYER

Moment Capacity Check +0.90D+E

Min, Defl. Ratio 2,481.88
Max. Deflection 0.09670 in
Max. Pu / Ag 16.10 psi
Location 10.333 ft

Controlling Asfod 0.001270

Asfod = 0.0 rho bal 0.01693

Max Cracking 4.167 k-ft
Minimum Phi Mn 3.833 k-ft

Maximum Reactions for Load Combination...
Top Horizontal E Only 0.8535 k
Base Horizontal E Only 0.8535 k
Vertical Reaction D + L + S + E/1.4 2.860 k
## Concrete Slender Wall

**Description:** Goleta - Main Building, Out-of-plane reinforcement check, 10" wall

### Design Maximum Combinations - Moments

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial Load Pu k</th>
<th>0.06 ft c b</th>
<th>Mcr k-ft</th>
<th>Mu k-ft</th>
<th>Moment Values Phi k-ft</th>
<th>Phi Mn k-ft</th>
<th>As in²</th>
<th>As Ratio</th>
<th>0.8 * ( \rho ) ( \text{bar} )</th>
</tr>
</thead>
<tbody>
<tr>
<td>D Only at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.21</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.2D+0.50L+1.80L at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.18</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.2D+1.60L+0.50S at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.18</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.2D+1.50L+0.50L at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.18</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.2D+0.50L+1.60S at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.18</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.2D+1.50L+0.60W at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.18</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.2D+0.50L+0.60L at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.18</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+1.2D+0.50L+0.50S+0.20S+E at 10.00 to 10.67</td>
<td>1.932</td>
<td>18.000</td>
<td>4.17</td>
<td>4.41</td>
<td>0.90</td>
<td>4.49</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+0.90D+1.60W at 19.33 to 20.00</td>
<td>0.000</td>
<td>18.000</td>
<td>4.17</td>
<td>0.14</td>
<td>0.90</td>
<td>3.83</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
<tr>
<td>+0.90D+E at 10.00 to 10.67</td>
<td>1.443</td>
<td>18.000</td>
<td>4.17</td>
<td>4.37</td>
<td>0.90</td>
<td>4.33</td>
<td>0.133</td>
<td>0.0013</td>
<td>0.0169</td>
</tr>
</tbody>
</table>

### Design Maximum Combinations - Deflections

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Axial Load Pu k</th>
<th>Mcr k-ft</th>
<th>Mactual k-ft</th>
<th>I gross in⁴</th>
<th>Stiffness I cracked in⁴</th>
<th>I effective in⁴</th>
<th>Deflections Defl. in</th>
<th>Defl. Defl. Ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>D + L + Lr at 11.33 to 12.00</td>
<td>1.443</td>
<td>4.17</td>
<td>0.09</td>
<td>1,000.00</td>
<td>92.17</td>
<td>750,000</td>
<td>0.003</td>
<td>84,315.6</td>
</tr>
<tr>
<td>D + L + W at 11.33 to 12.00</td>
<td>1.443</td>
<td>4.17</td>
<td>0.09</td>
<td>1,000.00</td>
<td>92.17</td>
<td>750,000</td>
<td>0.003</td>
<td>84,315.6</td>
</tr>
<tr>
<td>D + L + W + S2 at 11.33 to 12.00</td>
<td>1.443</td>
<td>4.17</td>
<td>0.09</td>
<td>1,000.00</td>
<td>92.17</td>
<td>750,000</td>
<td>0.003</td>
<td>84,315.6</td>
</tr>
<tr>
<td>D + L + S + W2 at 11.33 to 12.00</td>
<td>1.443</td>
<td>4.17</td>
<td>0.09</td>
<td>1,000.00</td>
<td>92.17</td>
<td>750,000</td>
<td>0.003</td>
<td>84,315.6</td>
</tr>
<tr>
<td>D + L + S + E/14 at 10.00 to 10.67</td>
<td>1.610</td>
<td>4.17</td>
<td>3.13</td>
<td>1,000.00</td>
<td>93.47</td>
<td>750,000</td>
<td>0.097</td>
<td>2,481.9</td>
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<tr>
<td>D + 0.5(L+Lr)+0.7W at 11.33 to 12.00</td>
<td>1.443</td>
<td>4.17</td>
<td>0.09</td>
<td>1,000.00</td>
<td>92.17</td>
<td>750,000</td>
<td>0.003</td>
<td>84,315.6</td>
</tr>
<tr>
<td>D + 0.5(L+Lr)+0.7E at 10.00 to 10.67</td>
<td>1.610</td>
<td>4.17</td>
<td>3.07</td>
<td>1,000.00</td>
<td>93.47</td>
<td>750,000</td>
<td>0.095</td>
<td>2,531.0</td>
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</tbody>
</table>

### Reactions - Vertical & Horizontal

<table>
<thead>
<tr>
<th>Load Combination</th>
<th>Base Horizontal</th>
<th>Top Horizontal</th>
<th>Vertical @ Wall Base</th>
</tr>
</thead>
<tbody>
<tr>
<td>D Only</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>2.860 k</td>
</tr>
<tr>
<td>S Only</td>
<td>0.0 k</td>
<td>0.00 k</td>
<td>0.000 k</td>
</tr>
<tr>
<td>W Only</td>
<td>0.0 k</td>
<td>0.00 k</td>
<td>2.860 k</td>
</tr>
<tr>
<td>E Only</td>
<td>0.9 k</td>
<td>0.85 k</td>
<td>2.860 k</td>
</tr>
<tr>
<td>D + L + Lr</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>2.860 k</td>
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<tr>
<td>D + L + S</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>2.860 k</td>
</tr>
<tr>
<td>D + L + W + S2</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>2.860 k</td>
</tr>
<tr>
<td>D + L + S + W2</td>
<td>0.0 k</td>
<td>0.01 k</td>
<td>2.860 k</td>
</tr>
<tr>
<td>D + L + S + E/1.4</td>
<td>0.6 k</td>
<td>0.60 k</td>
<td>2.860 k</td>
</tr>
</tbody>
</table>
**Out of Plane Anchorages Forces**

\[ T_c = 0.95 S_{ds} L \eta A_p \]  

\[ S_{ds} = 1.138 \text{ k} \]  

\[ W_p = \text{Gross wt. of wall} \]  
\[ = 140 \text{ PSF (10')} \]  
\[ = 115 \text{ PSF (90')} \]  

**Anchorages Connections**

- Are spaced @ 12'-0" OC
- Ties wall wt = 9'-0"

\[ A_p = 9' \times 12' - \frac{15' \times 5'}{2} = 83 \text{ ft}^2 \]

\[ T_c = 0.9 \times (1.138)(140 \text{ PSF})(83 \text{ ft}^2) \]

\[ = 11.9 \text{ kips} \]
**Diaphragm Anchorage**

**Emplacement:** 14.5"
**Bolt Size:** 3/4"

\[ F_c = 11.9 \text{ kips demand} \]

**Steel Strength (Assume: A307 bolts, } F_u = 60 \text{ ksi} )**

\[ V_{sa} = n A_{sb} f_{tub} 	imes 0.6 \]

\[ = (0.339 \text{ in}^2)(60 \text{ ksi}) \times 0.6 = 12.02 \text{ kips} \]

\[ \phi = 0.65 \]

\[ \phi V_{sa} = 7.82 \text{ kips} \]

**Concrete Breakout**

\[ V_{cb} = \frac{A_{pc} f_{c} d_c}{A_{co}} \]

\[ C_{co} = 6'' \]

\[ A_{pc} = 9.316 \text{ in}^2 \]

\[ A_{co} = 162 \text{ in}^2 \]

\[ V_{cb} = (7 \text{ in} \cdot 9.316 \text{ in}^2) \times \frac{2500 \text{ psi}}{6''} \]

\[ V_{cb} = 76.73 \text{ kips} \]

\[ \phi = 0.70 \]

\[ V_{cb} = 6.75 \text{ kips} \]

\[ \psi_{cd} = 1.0 \]

\[ \psi_{c} = 1.0 \]

\[ \psi_{a} = 1.0 \]

\[ V_{cb} = 6.75 \text{ kips} \]

\[ \phi V_{cb} = 4.73 \text{kips} \]
\[ V_{cp} = K_{cp} N_{eb} \]

\[ K_{cp} = 2.0 \quad \text{FOR} \quad h_{et} > 2.15'' \]

\[ N_{eb} = \frac{A_{cc}}{A_{cc} \cdot R_{c} \cdot 0.70 \cdot V_{cp}} \]

\[ A_{cc} = 9(14'')^2 = 1764 \text{in}^2 \]

\[ A_{cc} = 11.5 \times 14'' \times 12'' \times 10'' = 920.50 \text{in}^2 \]

\[ R_{b} = K_{ed} \cdot W_{c} \cdot h_{et}^{1.5} \]

\[ = 24\sqrt{2500 \times 14''} \times 1.5 = 62.86 \text{K} \]

\[ N_{eb} = 920.50 \times 0.70 \times 0.70 \times 62.86 \text{K} = 11.37 \text{K} \]

\[ V_{cp} = 22.73 \text{K} \]

\[ \Delta V_{cp} = 15.92 \text{K} \]
VBC 1991 Anchorages Capacity

\[ P_u = N_b (N_s, F_u) \]

Assumed: A308 Bolts
\[ F_u = 60 \, \text{kips} \]

\[ N_s = 0.75A_b f'c \]
\[ = 0.75 \times (0.334 \times 10^{-5}) (60 \, \text{kips}) = 15.02 \, \text{kips} \quad \text{Steel}\]

\[ A_c = \phi \frac{2\pi d_e^2}{4} \frac{1}{f'c} \]
\[ = 0.65 \times \frac{2\pi (4/2)^2}{4} \sqrt{2500} = 3.27 \, \text{kips} \quad \text{Concrete}\]

\[ DCR = \frac{110 \, \text{kips}}{3.27 \, \text{kips}} = 3.49 \, \text{kips} \]

[Posttension Needed]
**Notes:**

- **ADP MORE ANCHORS, SPACED SUFFICIENTLY APART TO PREVENT CONC. BREAKOUT**

  \[ T_e = 11.9 \text{ KIPS DEMAND} \]

**Appendix D:**

\[ 11.9 = \phi V_{c0} V_a \]

\[ = \frac{A_{vc0}}{A_{ro}} \phi_o \phi_e \phi_i \phi_r \]

\[ c_{ai} = 6'' \]

\[ A_{vc0} = 9.5(6)^2 = 162.5 \text{ in}^2 \]

\[ A_{vc} = 1.5(6)(1506x2 + 45) = 162 + 360 \text{ in}^2 \]

**Diagram:**

- 10° WALL
- SIDE VIEW
- ADDITIONAL ANCHORS
- 4 X SPACING

\[ v_b = 6.75k \]

\[ 11.9 = 0.7 \left( \frac{72+245}{72} \right) 6.75 k \]

\[ s = 4.56'' \]

\[ S = 7.07'' \]

**See Hilti Preliminary**
NDS 8.5.2 - Minimum Bolt Edge Distance
Perpendicular to Gravel Loading,

Max. Edge Dist. = 4d

= \frac{9(3\frac{3}{4}\text{"})}{4} = 3\text{"} (2\frac{1}{4}\text{"} Anchor)

4(5\frac{1}{8}\text{"}) = 2.13\text{"} (5\frac{1}{8}\text{"}"")

4(1\frac{1}{2}\text{"}) = 2\text{"} (1\frac{1}{2}\text{"}"")

<table>
<thead>
<tr>
<th>Anchor Dia.</th>
<th>Min. Edge Dist.</th>
<th>Controlling Anchor Edge Dist. in Concrete</th>
</tr>
</thead>
<tbody>
<tr>
<td>1/2&quot;</td>
<td>2&quot;</td>
<td>8&quot;</td>
</tr>
<tr>
<td>5/8&quot;</td>
<td>2.5&quot;</td>
<td>7.5&quot;</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>3&quot;</td>
<td>7&quot;</td>
</tr>
<tr>
<td>7/8&quot;</td>
<td>3.5&quot;</td>
<td>6.5&quot;</td>
</tr>
</tbody>
</table>
2010 CBC 1908.1.9 Exemption does not require ductile failure for anchors designed to resist wall out-of-plane forces when ASCE 7-05 Eq 12.11-1 is used.
2 Load case/Resulting anchor forces

Load case: Design loads

<table>
<thead>
<tr>
<th>Anchor</th>
<th>Tension force</th>
<th>Shear force</th>
<th>Shear force x</th>
<th>Shear force y</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0</td>
<td>2380</td>
<td>0</td>
<td>-2380</td>
</tr>
<tr>
<td>2</td>
<td>0</td>
<td>2380</td>
<td>0</td>
<td>-2380</td>
</tr>
<tr>
<td>3</td>
<td>0</td>
<td>2380</td>
<td>0</td>
<td>-2380</td>
</tr>
<tr>
<td>4</td>
<td>0</td>
<td>2380</td>
<td>0</td>
<td>-2380</td>
</tr>
<tr>
<td>5</td>
<td>0</td>
<td>2380</td>
<td>0</td>
<td>-2380</td>
</tr>
</tbody>
</table>

max. concrete compressive strain: - [%]
max. concrete compressive stress: - [psi]
resulting tension force in (xy)=(0.000/0.000): 0 [lb]
resulting compression force in (xy)=(0.000/0.000): 0 [lb]

3 Tension load

<table>
<thead>
<tr>
<th>Steel Strength*</th>
<th>Load N_u [lb]</th>
<th>Capacity qN_0 [lb]</th>
<th>Utilization p_u = N_u/qN_0</th>
<th>Status</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bond Strength*</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
<tr>
<td>Concrete Breakout Strength**</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
<td>N/A</td>
</tr>
</tbody>
</table>

* anchor having the highest loading  ** anchor group (anchors in tension)
4 Shear load

| Steel Strength*                      | 2380 | 8730 | 28 | OK |
| Steel failure (with lever arm)*      | N/A  | N/A  | N/A | N/A |
| Pryout Strength (Concrete Breakout   | 11900| 33856| 36 | OK |
| Strength controls)**                 |     |     |    |    |
| Concrete edge failure in direction y**| 11900| 12763| 94 | OK |

* anchor having the highest loading  **anchor group (relevant anchors)

4.1 Steel Strength

\[ V_{as} = (n \cdot 0.6 \cdot A_{max} f_{u,k}) \]

\( \psi_{V_{as}} = \frac{V_{as}}{A} \)

Refer to ICC-ES ESR 2322

** ACI 318-08 Eq. (D-2)

Variables

\[
\begin{array}{ccc}
\text{n} & A_{max} \text{[in.]} & f_{u,k} \text{[ksi]} & (n \cdot 0.6 \cdot A_{max} f_{u,k}) \text{[lb]} \\
1 & 0.33 & 72500 & 14550
\end{array}
\]

Calculations

<table>
<thead>
<tr>
<th>( V_{as} ) [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>14550</td>
</tr>
</tbody>
</table>

Results

<table>
<thead>
<tr>
<th>( V_{as} ) [lb]</th>
<th>( \psi_{V_{as}} )</th>
<th>( V_{as} ) [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>14550</td>
<td>0.33</td>
<td>14550</td>
</tr>
</tbody>
</table>

4.2 Pryout Strength (Concrete Breakout Strength controls)

\[ V_{org} = k_o \left( \frac{A_{org}}{A_{org}} \right) V_{org} \]

ACI 318-08 Eq. (D-31)

\( \psi_{V_{org}} = \frac{V_{org}}{V_{as}} \)

ACI 318-08 Eq. (D-2)

\( A_{org} \) see ACI 318-08, Part D.5.2.1, Fig. RD.5.2.1(b)

\( A_{org} = 0.9 \cdot h_{org} \cdot 3 \cdot h_{org} \)

ACI 318-08 Eq. (D-6)

\( \psi_{V_{org}} = \left( \frac{1}{1 + \frac{2 \cdot f_{c} \cdot 1.5 \cdot h_{org}}{3 \cdot h_{org}}} \right) \)

ACI 318-08 Eq. (D-9)

\( V_{con,N} = 0.7 + 0.3 \left( \frac{c_{min}}{1.5 \cdot h_{org}} \right) \)

ACI 318-08 Eq. (D-11)

\( V_{con,N} = \max \left( \frac{c_{min}}{c_{a}, \lambda} \right) \)

ACI 318-08 Eq. (D-13)

\( N_{b} = k_o \cdot \lambda \cdot V_{con,N} \)

ACI 318-08 Eq. (D-7)

** Variables

\[
\begin{array}{cccccc}
\text{k_o} & h_{org} \text{[in.]} & c_{a} \text{[in.]} & e_{a} \text{[in.]} & c_{a} \text{[in.]} \\
2 & 8.000 & 0.000 & 0.000 & 7.000
\end{array}
\]

\[
\begin{array}{cccccc}
V_{max,N} & c_{a} \text{[in.]} & k_o & \lambda & f_{c} \text{[psi]} \\
1.000 & 9.000 & 17 & 1 & 2500
\end{array}
\]

Calculations

<table>
<thead>
<tr>
<th>( V_{org} ) [lb]</th>
<th>( A_{org} ) [in.²]</th>
<th>( \psi_{V_{org}} )</th>
<th>( V_{org} ) [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>672.00</td>
<td>324.00</td>
<td>1.000</td>
<td>1.000</td>
</tr>
<tr>
<td>0.933</td>
<td>1.000</td>
<td>12492</td>
<td></td>
</tr>
</tbody>
</table>

Results

<table>
<thead>
<tr>
<th>V_{org} [lb]</th>
<th>( \psi_{V_{org}} )</th>
<th>( V_{org} ) [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>485.66</td>
<td>0.700</td>
<td>33856</td>
</tr>
<tr>
<td>11900</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>
4.3 Concrete edge failure in direction y-

\[ V_{cqd} = \left( \frac{A_{sq}}{A_{vqc}} \right) V_{c,q,V} \frac{V_{c,q,V}}{V_{q,V}} \frac{V_{h,v}}{V_{q,V}} \frac{V_{p,v}}{V_{q,V}} V_6 \]

\[ \phi V_{cqd} \geq V_{u} \]

\[ A_{c} \text{ see ACI 318-08, Part D.6.2.1, Fig. RD.6.2.1(b)} \]

\[ A_{v,c} = 4.5 c_{t}^{2} \]

\[ V_{c,q,V} = \left( \frac{1}{1 + 2e_{c,q}} \right) \frac{c_{t}}{3C_{t}} \leq 1.0 \]

\[ V_{h,v} = \frac{1.5C_{t}}{h_{a}} \leq 1.0 \]

\[ V_{p,v} = 0.7 + 0.3 \left( \frac{c_{t}}{C_{t}} \right) \leq 1.0 \]

\[ V_{c} = \left( \frac{0.12}{c_{t}} \right) \frac{e_{b}}{C_{t}} \lambda \frac{V_{b}}{c_{t}} \]

Variables

<table>
<thead>
<tr>
<th>( c_{t} ) [in.]</th>
<th>( c_{a} ) [in.]</th>
<th>( e_{b} ) [in.]</th>
<th>( V_{c,q,V} )</th>
<th>( h_{a} ) [in.]</th>
</tr>
</thead>
<tbody>
<tr>
<td>7.000</td>
<td>0.000</td>
<td>1.000</td>
<td>120.000</td>
<td></td>
</tr>
<tr>
<td>6.000</td>
<td>1.000</td>
<td>0.750</td>
<td>2500</td>
<td>1.000</td>
</tr>
</tbody>
</table>

Calculations

<table>
<thead>
<tr>
<th>( A_{c} ) [in.]</th>
<th>( A_{v,c} ) [in.]</th>
<th>( V_{c,q,V} )</th>
<th>( V_{h,v} )</th>
<th>( V_{p,v} )</th>
<th>( V_{c} ) [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>472.50</td>
<td>220.50</td>
<td>1.000</td>
<td>1.000</td>
<td>1.000</td>
<td>8509</td>
</tr>
</tbody>
</table>

Results

<table>
<thead>
<tr>
<th>( V_{cqd} ) [lb]</th>
<th>( \phi V_{cqd} )</th>
<th>( \phi V_{cqd} ) [lb]</th>
<th>( V_{u} ) [lb]</th>
</tr>
</thead>
<tbody>
<tr>
<td>18233</td>
<td>0.700</td>
<td>12763</td>
<td>11500</td>
</tr>
</tbody>
</table>

5 Warnings

- To avoid failure of the anchor plate the required thickness can be calculated in PROFIS Anchor. Load re-distributions on the anchors due to elastic deformations of the anchor plate are not considered. The anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the loading!

- Condition A applies when supplementary reinforcement is used. The \( \Phi \) factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to ACI 318, Part D.4.4(c).

- Design Strengths of adhesive anchor systems are influenced by the cleaning method. Refer to the INSTRUCTIONS FOR USE given in the Evaluation Service Report for cleaning and installation instructions.

- The present version of the software does not account for adhesive anchor special design provisions corresponding to overhead applications. Refer to the ICC-ES Evaluation Service Report (e.g. section 4.1.1 of the ICC-ESR 2322) for details.

- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI318 or the relevant standard!

Fastening meets the design criteria!
6 Installation data

Anchor plate, steel:
Profile: no profile; 0.000 x 0.000 x 0.000 in.
Hole diameter in the fixture: d₁ = 0.813 in.
Plate thickness (input): 0.500 in.
Recommended plate thickness: not calculated
Cleaning: Premium cleaning of the drilled hole is required

Anchor type and diameter: HIT-RE 500-SD + HAS, 3/4
Installation torque: 1200,000 in.lb
Hole diameter in the base material: 0.875 in.
Hole depth in the base material: 6.000 in.
Minimum thickness of the base material: 7.750 in.

Coordinates Anchor in.

<table>
<thead>
<tr>
<th>Anchor</th>
<th>x</th>
<th>y</th>
<th>cₓ</th>
<th>cᵧ</th>
<th>cₓᵧ</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-12.000</td>
<td>0.000</td>
<td>-</td>
<td>-</td>
<td>7.000</td>
</tr>
<tr>
<td>2</td>
<td>-6.000</td>
<td>0.000</td>
<td>-</td>
<td>-</td>
<td>7.000</td>
</tr>
<tr>
<td>3</td>
<td>6.000</td>
<td>0.000</td>
<td>-</td>
<td>-</td>
<td>7.000</td>
</tr>
<tr>
<td>4</td>
<td>12.000</td>
<td>0.000</td>
<td>-</td>
<td>-</td>
<td>7.000</td>
</tr>
<tr>
<td>5</td>
<td>0.000</td>
<td>0.000</td>
<td>-</td>
<td>-</td>
<td>7.000</td>
</tr>
</tbody>
</table>
7 Remarks; Your Cooperation Duties

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AUDITORIUM

NINTH DEPARTMENT - SUB BASE

\[ A_d = \frac{1}{2}bh = \frac{1}{2} (6.8') (9.5'-8.0') = 51\text{ ft}^2 \]

4x10 RODS @ 7'-6" O.C.

\[ \tan \theta = \frac{9.5'}{20'} \]

\[ \theta = 25.9' \]

HORIZONTAL SPACING = 7.5' x 60 = 6'8"

SCALE: 2" = 1'-0"

ASCE 31-03 § 4.25.1 OUT-OF-PLACE WALL ANCHORAGE FOR TO FLEXIBLE DEPHRAMGAMS

MIDDLE FLOOR AREA = \( A_f = (176.14 + 51.1)/2 = 90.62 \text{ ft}^2 \)

\[ F_p = 0.85 S_d (A_p) = 11.6' \]

\[ W/75\% = 8.06' \]
**Classroom Side - Outside**

- Tropes @ 32" OC (2.67')
- 6" wall → 90 PSF
- 10" wall → 140 PSF
- FP = 0.15 50% UP

**Total Ht = 15.94'/2 = 7.97' → 8'-0"**

\[ FP = 0.06(1.135) \times (90 \text{ PSF} \times 3.75' \times 2.67' + 140 \text{ PSF} \times 1' \times 2.67') \]

\[ = 1303 \text{ lb} \]

\[ UF/75\% \text{ Performance} \]

\[ = 977 \text{ #} \]

\[ UF/6" \text{ Anchorage Space} \]

\[ 977 \times 2 = 1954 \text{ #} \]

Assume 72" window

**Total Ht = 15.94' of Wall**

**Total Ht = 24'-9"**

Floor Level
**Classroom Side - Inside**

Tress @ 32" OC (2.67')

8" Conc Wall ⇒ 115 psf
2x6 @ 32" ⇒ 0.7 psf

More Wall Not at Tress ⇒ 15 psf

Total Ht = 8'-0"

- $F_r = 0.8 \left( \frac{S_{ps \cdot LP}}{0.8} \right)
- = 0.8(1.138)(2.67' \times 15.7 psf
  \times 3.75' + 2.67' \times 115 psf \times (8' - 3.75'))

= 1331#

W/ 75% SEismic Level,

= 9918#

For Anchorage @ 64" OC, Demand = 1996#
**Cloister**

2x6 @ 24" OC

Wall HT = 12' - 2/4" - 2' = 10.19'

10" CONC = 140 PSF

Per 8.5'

Per 8.5'

(140 PSF x 2.5' x 5.5')

+ (140 PSF x 1.75' x 6') / 85

= 383 PLF

---

**Fp = 0.85 + Lf**

\[
F_p = 0.8 \times (1.128) = 0.9
\]

PLF = 383 PLF

\[
W/ 75\% \text{ PERFORMANCE} = 262 \text{ PLF}
\]

<table>
<thead>
<tr>
<th>Height</th>
<th>Shear Demand</th>
</tr>
</thead>
<tbody>
<tr>
<td>24&quot;</td>
<td>524#</td>
</tr>
<tr>
<td>48&quot;</td>
<td>1048#</td>
</tr>
<tr>
<td>72&quot;</td>
<td>1572#</td>
</tr>
</tbody>
</table>

* W/ 75% PERFORMANCE
OOP Anchorages for Office

10" concrete wall, 140 psf

Forts @ 32" O.C.

Tie HT = 14'/2 = 7'

Fp = 0.80 SP65

= 0.80 (1.133)(140 psf \times 32/12 \times 7')

= 2379.18 lbf

W = 75%

= \frac{1784 \text{ lbf}}{2379.18 \text{ lbf}}

For 64°, = 3569 lbf
Building B - Seismic Dead Loads

### Typical Roof/Ceiling

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing</td>
<td>2.0</td>
</tr>
<tr>
<td>1&quot; Diagonal Sheathing</td>
<td>2.5</td>
</tr>
<tr>
<td>2x4 Sub Purlins @ 24&quot;oc</td>
<td>1.0</td>
</tr>
<tr>
<td>Steel Purlins</td>
<td>1.5</td>
</tr>
<tr>
<td>Steel Girders</td>
<td>2.7</td>
</tr>
<tr>
<td>Acoustical Ceiling</td>
<td>3.0</td>
</tr>
<tr>
<td>Low Suspended Ceiling</td>
<td>2.0</td>
</tr>
<tr>
<td>MEP</td>
<td>1.0</td>
</tr>
<tr>
<td>Partitions</td>
<td>5.0</td>
</tr>
<tr>
<td>Misc</td>
<td>1.3</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>22.0</strong></td>
</tr>
</tbody>
</table>

### Overhang

<table>
<thead>
<tr>
<th>Item</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing</td>
<td>2.0</td>
</tr>
<tr>
<td>1&quot; Diagonal Sheathing</td>
<td>2.5</td>
</tr>
<tr>
<td>2x8 @ 24&quot; oc</td>
<td>1.5</td>
</tr>
<tr>
<td>Ceiling</td>
<td>1.0</td>
</tr>
<tr>
<td>Steel Posts</td>
<td>0.5</td>
</tr>
<tr>
<td>Misc</td>
<td>1.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>9.0</strong></td>
</tr>
</tbody>
</table>

### Walls:

Interior and Exterior walls are accounted for in partition load for roof seismic weight.
PSUEDO LATERAL FORCE:

PER ASCE 31-03 § 3.5.2.1

\[ V = C \cdot S_o \cdot W \]  \hspace{1cm} (EQ 3-1)

\[ C = 1.3 \]  \hspace{1cm} (TABLE 2-A)

\[ S_o = \frac{S_{D1}}{T} \] (BUT NOT EXCEED \( S_{D5} \)) \hspace{1cm} (§ 3.5.2.3.1)

WHERE:

\[ S_{D1} = 0.065 \]

\[ S_{D5} = 1.138 \]

\[ T = \frac{C \cdot h_n \cdot \beta}{h_n} \] \hspace{1cm} (EQ 2-8)

\[ C = 0.060 \]

\[ h_n = 14.25 \text{ ft} \]

\[ \beta = 0.75 \]

\[ T = \left( 0.060 \right) \left( 14.25 \text{ ft} \right) \cdot \left( 0.75 \right) = 0.440 \text{ sec.} \]

\[ S_o = \frac{0.665}{0.440} = 1.511 > 1.138 \Rightarrow S_o = 1.138 \]

\[ V = 1.3 \cdot S_o \cdot W = 1.3 \cdot (1.138) \cdot W \]

\[ V = 1.479 \text{ W} \]
### Building B - Seismic Base Shear

\[ V_{\text{BASE}} = C_S W = 1.479W \]

- Roof DL = 22.0 psf
- Overhang DL = 9.0 psf
- Roof Area = 8512 \( \text{ft}^2 \)
- Overhang Area = 2112 \( \text{ft}^2 \)
- Seismic Weight, \( W = 206 \) kips

\[ V_{\text{BASE}} = 305 \text{ kips} \]
**Shear Stress Check**

\[ V_{s,\text{ave}} = \frac{1}{q} \left( \frac{N}{A_o} \right) \]

\[ V_s = 305 \text{ kips} \]

\[ m = 4.0 \]

\[ A_o = \text{length of walls, each direction} \]

**In N-S Direction:**

Assume (5) footing has 3 N-S walls.

Each N-S wall is \( \approx 31.4' \)

\[ 3 \times 31.4' = 94.2' \]

**In E-W Direction:**

Note: Wall ht \( \approx 14' \)

Narrow wall ht limitation: \( H/B < 2 \)

Ignore walls narrower than \( 14/2 = 7' \)

**Estimate wall's length:**

Assume 30' of wall is \( 6 > \text{plunge} \)

About 14' total

**Check shear stress:**

\[ V_{s,\text{ave}} = \frac{1}{q} \left( \frac{205 \times 148}{148} \right) = 515 \text{ kips} \]

\( < 700 \text{ kips} \)

Shear walls OK
# Building C4 - Seismic Dead Loads

**Typical Roof/Ceiling**

<table>
<thead>
<tr>
<th>Material</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing</td>
<td>2.0</td>
</tr>
<tr>
<td>1/2&quot; Plywood</td>
<td>1.5</td>
</tr>
<tr>
<td>Wood Trusses @ 24&quot; oc</td>
<td>3.0</td>
</tr>
<tr>
<td>Insulation</td>
<td>0.5</td>
</tr>
<tr>
<td>Ceiling</td>
<td>2.0</td>
</tr>
<tr>
<td>MEP</td>
<td>1.0</td>
</tr>
<tr>
<td>Partitions</td>
<td>5.0</td>
</tr>
<tr>
<td>Misc</td>
<td>2.0</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>17.0</strong></td>
</tr>
</tbody>
</table>

**Overhang**

<table>
<thead>
<tr>
<th>Material</th>
<th>Load (psf)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Roofing</td>
<td>2.0</td>
</tr>
<tr>
<td>1/2&quot; Plywood</td>
<td>1.5</td>
</tr>
<tr>
<td>2x8 @ 24&quot; oc</td>
<td>1.5</td>
</tr>
<tr>
<td>Ceiling</td>
<td>1.0</td>
</tr>
<tr>
<td>Steel Posts</td>
<td>0.5</td>
</tr>
<tr>
<td>Misc</td>
<td>1.5</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>8.0</strong></td>
</tr>
</tbody>
</table>

**Walls:**

Interior and Exterior walls are accounted for in partition load for roof seismic weight.
PSUEDO LATERAL FORCE:

FER ASCE 31-03 § 3.5.2.1

\[ V = C \cdot s_a \cdot W \]  
\[ C = 1.3 \]  

\[ s_a = \frac{s_d_1}{T} \text{, but not exceed } s_{ds} \]  
\[ s_d_1 = 0.065 \]  
\[ s_{ds} = 1.138 \]

\[ T = C_t \cdot h_n^B \]  
\[ C_t = 0.60 \]  
\[ h_n = 9.5\text{ft} \]  
\[ B = 0.75 \]

\[ T = (0.060)(9.5\text{ft})^{0.75} = 0.325 \text{ sec.} \]

\[ s_a = \frac{0.665}{0.325} = 2.046 < 1.138 \Rightarrow s_a = 1.138 \]

\[ V = 1.3 \cdot (1.138)W \]

\[ V = 1.479W \]
Building C4 - Seismic Base Shear

\[ V_{\text{BASE}} = C_{S4}W = 1.479W \]

- Roof DL = 17.0 psf
- Overhang DL = 8.0 psf
- Roof Area = 5880 ft²
- Overhang Area = 1348 ft²

Seismic Weight, \( W = 111 \) kips

\[ V_{\text{BASE}} = 164 \text{ kips} \]
Shear Stress Check

Pseudo "Lat. Force" \( V = 164 \text{ kips} \)

\[
V_{S,\text{ANC1}} = \frac{1}{3} \frac{V_S}{A_w}
\]

\( A_w = \text{Length of Shear Walls} \)

(Small Walls with 2:1 Aspect Ratio or Loaders)

\( M = 4 \) (ASCE 31 Table 3-7)

\[
A_{S,} = \text{Longitudinal Dir.'s 22.5° (Ignoring 2:1 Aspect Ratio Loads)}
\]

\[
V_{S,\text{ANC1}} = \frac{1}{4} \frac{164,000}{22.5} = 1322 \text{ PLF} > 1000 \text{ PLF} \text{ NG}
\]

Includes Total Length of Shear Walls, Regardless of Aspect Ratio.

\( L = 58' \) of Wall

\[
V_{S,\text{ANC1}} = \frac{1}{4} \frac{164,000}{58} = 707 \text{ PLF} < 1000 \text{ PLF} \text{ OK}
\]
Narrow Shear Wall Tier 2 Check

- Overturning
- Shear Demands

M-Factor (Table 9.8)
M = 3.5 for 3.5 ≥ W/L > 2.0, (Life Safety)

Shear Demand Check
Per Prev Calc,

\[ V_{\text{demand}} = \frac{1}{3.5} \times \frac{164,000 \text{ lb}}{58'} = 808 \text{ lb/lf} < 1000 \text{ lb/lf} \quad \text{OK} \]

Overturning Check

h = 8.5'
b = 4'-0''
3-Bolts are 6'' from S.W. Edges

\[ V = 808 \text{ lb/lf} \times 4' = 3232 \text{ #} \]

Roof Loads:
17 PSF - 5 PSF (Partition Loads) = 12 PSF

\[ W_{\text{roof}} = 12 \text{ PSF} \times 16' = 192 \text{ plf} \]

\[ \frac{1}{2}' \text{ plywood} = 1.5 \text{ psf} \]
\[ 2 \times 6 @16'' = 1.4 \text{ psf} \]
\[ 2.9 \text{ psf} \times 8.5' \times 4' = 98.6'\]
\[ \text{total} = 99'\]
EM T-BOLT = 0
LOCATION

- (3232#) (8.5')(0.91)(1.5') + (0.91)(192 PLF x 4') (1.5')

\[-R(3') = 0\]

\[R = 8767 \# \quad \text{UPLIFT FORCE}\]

(From Pego Lateral Force Method)

**Equivalent Lateral Force Procedures (ELF)**

\[V = C_6 \cdot L\]
\[= \frac{1.1338}{6.5/1.0} (111 KIPS)\]
\[= 191.4 KIPS\]

191.4 KIPS / 38° = 335 PLF

**Lateral Force Per 4' Segment**

\[335 \text{ PLF} \times 4' = 1340 \#\]

EM T-BOLT = 0
LOCATION

- (1340#) (8.5') + (0.91)(192 PLF x 4') (1.5')

\[-R(13') = 0\]

\[R = -3407 \# \quad \text{UPLIFT FORC}\]

(From Lateral Force Method)
Tensile Capacity (Steel Strength)

\[ N_{sa} = \phi A \sigma_s \]

\[ A_{se} = 0.0334 \text{ in}^2 \quad (\text{Ref: PCA 318-08 Table 34-1}) \]

For steel strength, assume 60 ksi (ACI 318-08 Bours Assumption)

\[ N_{sa} = 20.0 \text{ kips} \]

\[ \phi = 0.75 \]

\[ \phi N_{sa} = 15.0 \text{ kips} \]

Tensile Capacity (Concrete Breakout)

\[ \phi = 0.70 \]

\[ N_{cb} = \frac{A_{co} \phi_d \mu_B \mu_c N_s}{A_{co}} \]

Approach: Smooth 6" high curb

\[ A_{co} = 9 \text{ in}^2 \]

\[ = 0.009 (240)^2 \]

\[ = 5184 \text{ in}^2 \]
**Project:**

**Golst A. Nabey**

**Description:**
Building CR

\[
N_b = 16 \times \frac{f_c}{F_{h,e}} \times \frac{1}{3}
\]

\[= 16 \times (1.0) \times 2000 \times (24.5 \times 3)\]

\[= 142,891 \text{ KIPS}\]

\[A_{nc} = (1.5 \times h_{ef} \times 19'') = 1304 \text{ in}^2\]

\[
\psi_d = 0.7 + 0.3 \left(\frac{h_{ef}}{1.5h_{ef}}\right) = 0.7 + 0.3 \left(\frac{3''}{1.5 \times 24''}\right) = 0.725
\]

\[
\psi_c = 1.0 \quad \text{(Cracking @ Service Level)}
\]

\[
\psi_{op} = 1.0 \quad \text{(CIP Anchors)}
\]

\[N_{ub} = 10,000 \text{ KIPS}\]

\[\phi N_{ub} = 7,000 \text{ KIPS}\]

\[\text{Ponsest Specification}\]

\[N_{po} = \phi N_{p}\]

\[
\psi_p = 1.0 \quad \text{(Cracking @ Service Level Loads)}
\]

\[N_p = 0.9 \times \frac{F_{c}}{\gamma_{nd}}\]

\[e_n = 6''\]

\[N_p = 0.9 \times \frac{20,000}{(6'' \times 3/4''/in)}\]

\[= 8,100 \text{ KIPS}\]

\[\phi N_p = 5,670 \text{ KIPS}\]
Architectural Capacity (WRC 1991)

\[ R_n = \text{men}(P_s, \varphi P_c) \]

\[ P_s = 0.9 \times 15' \]
\[ = 0.9 \times (0.339 \text{ in}^2) \times (60,000 \text{ psi}) = 18.0 \text{ ksi} \]

\[ \varphi P_c = \frac{1}{2} P_c (2.8 P_s + 4 A_T) \]

\[ A = 1.0 \]
\[ \varphi = 0.63 \]
\[ P_c = 2000 \text{ psi} \]

\[ A_s = \text{Suction Area} \]
\[ A_T = \text{Flat Area} \]

\[ \phi P_c = 0.65 \times \frac{1}{2} \times 2000 \times (4 \times 672 \text{ ksi}) \times \frac{3}{24} = 9.8 \text{ ksi} \]

Approach:

- Ignore 8° High Work

Eddie Story Emembement
Appendix D – Geotechnical Report
GEOTECHNICAL FEASIBILITY / GEOLOGIC HAZARDS STUDY
SEISMIC AND ADA NEEDS ASSESSMENT
GOLETA VALLEY COMMUNITY CENTER
5679 HOLLISTER AVENUE
GOLETA, CALIFORNIA

December 3, 2012

Prepared for

Mr. Colin Blaney, SE
Crosby Group

Prepared by

Earth Systems Pacific
4378 Old Santa Fe Road
San Luis Obispo, CA 93401

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December 3, 2012

FILE NO.: SL-16901-SA

Mr. Colin Blaney, SE  
Crosby Group  
2200 Bridge Parkway, Suite 104  
Redwood City, CA  94065

PROJECT: SEISMIC AND ADA NEEDS ASSESSMENT  
GOLETA VALLEY COMMUNITY CENTER  
5679 HOLLISTER AVENUE  
GOLETA, CALIFORNIA

SUBJECT: Geotechnical Feasibility / Geologic Hazards Study

CONTRACT

REFS.:  
1) Proposal to Provide a Geotechnical Feasibility / Geologic Hazards Study, Seismic and ADA Needs Assessment for the Goleta Valley Community Center, 5679 Hollister Avenue, Goleta, California, by Earth systems Pacific, Doc. No. 1205-102.PR.P.REV, revised November 6, 2012

2) Request for Qualifications and Proposal (RFQ/RFP) for Seismic and ADA Needs Assessment for the Goleta Valley Community Center, by the City of Goleta, dated May 10, 2012

Dear Mr. Blaney:

In accordance with your authorization of our revised proposal (Ref. No. 1), this geotechnical feasibility / geologic hazards study has been prepared to provide information for your use in providing the Seismic and ADA Needs Assessment for the Goleta Valley Community Center in Goleta, California. This report is based on the information contained in the project's RFQ/RFP (Ref. No. 2), and on our telephone conversations regarding the revised scope of services contained in our proposal.

We appreciate the opportunity to have provided services for this project. If there are any questions concerning this report, please feel free to contact the undersigned at your convenience.

Sincerely,

Earth Systems Pacific

Fred J. C. Post, G.E.

Richard T. Gorman, C.E.G.

Doc. No.: 1205-102.PR.P.REV

[Signature]

[Signature]
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APPENDIX

Geologic Map
Historical Earthquake / Fault Map
FEMA Flood Zone Map
Site Design Response Spectrum
1.0 INTRODUCTION
According to information provided by the City of Goleta, the Goleta Valley Community Center (GVCC) consists of a Main Building, which contains an auditorium, dining hall, kitchen, meeting rooms, dance studios, restroom and storage rooms. There are several other detached buildings serving as classrooms on the 9.85-acre site, and a large building at the rear of the site that houses the Goleta Valley Boys & Girls Club. The site also has lighted parking for 200 cars, basketball courts, a tennis court, athletic fields and picnic areas.

We understand that the Main Building, reportedly constructed in 1927, was utilized as an elementary school until 1976, when the School District elected to close the school rather than retrofit it to meet State of California earthquake standards for schools. The site was converted to the Community Center, and it is currently under a lease agreement assumed by the City of Goleta ("the City") with the School district; the City subleases the facility to the non-profit GVCC. In 2013, the City will acquire the property, and it is anticipating a larger maintenance responsibility for the site upon assuming ownership.

In preparation for the transition to ownership, the City has asked for an assessment of the physical condition of the GVCC property and all structures, and an identification of seismic and ADA modifications that would be needed to meet the current building code for existing and future uses. The RFQ/RFP (City of Goleta, 2012) provided the overall requirements for the assessment.

2.0 SCOPE OF SERVICES
The purpose of this study was to provide an assessment of the geotechnical and geologic characteristics of the site, including identification of issues that could constitute a constraint to the current use of the existing facilities, and possible future renovations or expansions. The study was based upon a review of available published geologic and geotechnical information; Earth Systems’ archival geologic, soils and seismic data for the general project area; geologic maps; aerial photographs; geologic literature; the Santa Barbara County Seismic Safety Element; FEMA maps; and soil survey information. A field reconnaissance was performed by a Certified Engineering Geologist; however, no geologic mapping, subsurface exploration or laboratory testing was requested or performed. In view of this, the assessment of geotechnical/geologic hazards is necessarily of a qualitative nature. Conclusions about specific conditions may be subject to modification once actual subsurface conditions are determined and/or laboratory tests are performed during possible future phases of the project or during planning for additional structures.
Pertinent information is presented regarding local and regional geology, geologic units, mapped landslides, liquefaction potential, and locations of faults. It includes general site characteristics and potential problem areas, and preliminary ground motion information.

This report is intended to be used primarily for informational purposes, with the objective of identifying potential constraints and to provide general information to be used in the assessment by the client of potential structural modifications of the existing facilities. It is intended to be in accordance with common geotechnical engineering and engineering geologic practices in this area under similar conditions at this time.

3.0 GEOLOGY

Geologic Setting
The site is located within Goleta Valley, which is a coastal plain in southern Santa Barbara County. The Goleta Valley is approximately 8 miles long from west to east, and ranges in width from approximately 0.25 mile in the west to almost 3 miles from north to south in the central area near Fairview Avenue. It comprises alluvial plains on the east, north, and west that slope gently south. These plains merge into the Goleta Slough in the south-central portion of the valley and drain to the ocean past Mescalitan Island. The Goleta Valley is bounded on the south and southeast by the nearly continuous terraces formed by More Mesa, Goleta Mesa, and Mescalitan Island.

Locally, the site lies in the central area of the broad, east-west trending Goleta Valley. This part of the valley is underlain by alluvium and colluvium deposits of Holocene and upper Pleistocene age. The geologic units are indicated on the Geologic Map in the Appendix, which is an extract from the Geologic Map of Goleta (Minor and others, 2007).

Faulting
The closest active fault to the site is the Santa Ynez fault, located approximately 7 miles north of the site. The closest mapped faults to the site, regardless of activity, are the Late Quaternary age More Ranch fault, located about 1 mile south-southeast of the site, and the Late Quaternary age Carneros fault, located approximately 1 mile north. Regional and local faults and locations of historic earthquake events are depicted on the Historical Earthquake/Fault Map in the Appendix.
Slope Stability
The site has no geologically significant slopes on or immediately adjacent to it, therefore the potential for slope stability to affect the site is considered to be nil.

Flooding
According to the Flood Insurance Rate Map Number 06083C1362F, dated September 30, 2005, published by the Federal Emergency Management Agency (FEMA), the majority of the site is located in Flood Zone X. Areas designated as Flood Zone X have a 0.20 percent chance of being flooded in any given year. The average depth of flooding is less than 1 foot. The north side and the northwest corner of the site are located within Flood Zone AE, which has 1.0 percent annual chance of flooding (100-year flood). The FEMA Flood Zone Map in the Appendix is an extract from Map No. 06083C1362F.

Earthquake History
The historic seismicity in the site’s region was researched using EQSEARCH (Blake, updated 2010) and the Boore and Joyner (1997) ground attenuation method. Based on archive information for this area, a Site Class “D” (stiff soil profile) per the 2010 California Building Code (CBC, 2010) was assumed. EQSEARCH is a computer program that performs automated searches of a custom catalog of historical California earthquakes. As the program searches the catalog, it computes and prints the epicentral distance from the selected site to each of the earthquakes within the specified search area. The epicentral distances should be considered estimated distances, particularly for earthquake data that dates prior to 1932, before instruments were used to record earthquake data. The parameters used for the search consisted of earthquake Richter magnitudes ranging from 5.0 to 9.0 that occurred in a 65-mile radius from the site from 1800 to 2010.

The results of the search indicated that within the search parameters, 48 earthquakes have occurred; some nearby earthquakes are shown on the appended Historical Earthquake / Fault Map. The highest peak horizontal ground acceleration (PGA) estimated to have occurred at the site from those historical earthquakes is 0.37g. This earthquake had a 5.7 magnitude and occurred in 1862; it was also the closest earthquake to the site, occurring approximately 1 mile southeast. The largest earthquake that the search revealed was a 7.9 magnitude earthquake, approximately 62 miles east of the site; this is known as the 1857 earthquake on the San Andreas Fault. Due to its distance, it only produced an estimated PGA of 0.10g at the site.
Ground Acceleration Parameters
The site is in a region of generally high seismicity and has the potential to experience strong ground shaking from earthquakes on regional or local causative faults. To characterize the seismicity at the site and to provide seismic design parameters, a General Procedure Ground Motion Analysis for deterministic ground motions was performed. The deterministic ground motions are based on the 2009 International Building Code General Procedure that was obtained from the United States Geological Survey Earthquake Hazards Program website (USGS, 2012) and the 2005 ASCE 7 Standard Analysis Method. The results of the seismic hazard analysis are presented in the following table and in the appended Site Design Response Spectrum.

### SUMMARY OF DESIGN RESPONSE ACCELERATION PARAMETERS

<table>
<thead>
<tr>
<th>Mapped Acceleration Values for Site Class B</th>
<th>2010 CBC Site Coefficients and General Procedure Adjusted MCE Spectral Response Acceleration Parameters For Site Class D $(\text{PGA} = S_{DS}/2.5 = 1.138 / 2.5 = 0.455g)$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Seismic Parameters</td>
<td>Values $(\text{g})$</td>
</tr>
<tr>
<td>$S_S$</td>
<td>1.71</td>
</tr>
<tr>
<td>$S_1$</td>
<td>0.665</td>
</tr>
</tbody>
</table>

Seismic Design Category
Section 1613A.5.6 of the 2010 CBC indicates that structures will be assigned to Category D unless $S_1$ is less than or equal to 0.75. The $S_1$ calculated for the site is 0.665g; therefore, the site is assigned to Seismic Design Category D.

Surface Ground Rupture
Surface ground rupture generally occurs at sites that are traversed by, or that lie very near, a causative fault. The site is not in a State Earthquake Fault Zone, and there are no mapped faults crossing or immediately adjacent to the site. Therefore, the potential for surface ground rupture to occur within the site is considered to be very low.
Liquefaction
Soil liquefaction is the loss of soil strength during a significant seismic event. It occurs primarily where the groundwater level is shallow and loose to medium-dense, fine to medium grained sands and sandy silts occur within a depth of about 50 feet. Liquefaction potential decreases as sand and gravel sizes increase, and as silt and clay contents increase. The site area is underlain by alluvium and colluviums deposits of Holocene and upper Pleistocene age, which are generally susceptible to liquefaction. The Santa Barbara County Seismic Safety & Seismic Element (2010) indicates that the site lies within a moderate liquefaction potential zone. The depth of groundwater in the site area is approximately 35 feet, based on groundwater elevations for June 2008 well measurements by Steven Bachman (2010). If liquefaction were to occur at depths, its effects at the surface would be at least partially attenuated by the relatively thick layer of non-liquefied soil. Therefore, in our opinion, the site has a low to moderate potential for liquefaction.

4.0 GEOTECHNICAL
Expansive Soils
Archival information indicates the site soils would be considered expansive, per 2010 CBC Section 1803.5.3, with actual expansion index values (ASTM D 4829-11) expected to be in the range of 35 to 70. Expansive soils tend to swell with seasonal increases in soil moisture and shrink during the dry season as soil moisture decreases. The volume changes that the soils undergo in this cyclical pattern can stress and damage slabs and foundations if precautionary measures are not incorporated into the design and construction.

To protect foundations from the damaging effects of expansive soils, they are generally deepened and more heavily reinforced. For expansion indices in this range, minimum foundation depths of 18 to 21 inches below lowest adjacent grade would generally be considered sufficient to mitigate the soil’s expansion potential. Slab protection generally could involve placement of nonexpansive material beneath the slab in conjunction with premoistening of subslab soils. For expansion indices in this range, it is probable that a layer of nonexpansive material 12 to 18 inches thick would generally be recommended if conventional slabs and foundations are used. Premoistening of the native soils to optimum moisture content or above, to depths of 12 to 18 inches below the nonexpansive layer would also be recommended. Minimum reinforcement for foundations of #4 or #5 rebar top and bottom, and #3 rebar at 18 to 24 inches on center each way would likely be recommended.
The use of deepened foundations, imported nonexpansive soils, premoistening of subslab soils and increased reinforcement are commonly used for structures similar to those on this site in this area at this time. There are a number of other options available, such as post-tensioned slab foundations, conventionally reinforced mat foundations, or 3 to 5-foot deep nonexpansive building pads. However, these solutions are typically not as cost-effective at this time as the measures presented above for structures similar to those on this site.

**Differential Settlement**

Differential settlement can occur when a foundation of a particular structure spans materials with different settlement characteristics, or variable *in situ* moisture and density conditions. These conditions can stress and possibly damage foundations, often resulting in severe cracks and displacement. To reduce the differential settlement potential, it is necessary for all foundations of an individual structure to bear in sufficiently uniform material. On this site, sufficiently uniform bearing for new structures would likely be achieved by overexcavating and recompacting the soils within a building area, to depths of 3 to 5 feet below existing grade.

It is our understanding that the existing structures are performing satisfactorily given their age, and there are no indications of excess differential settlement, therefore no additional recommendations for mitigation of differential settlement in the existing structures are considered necessary at this time.

**Foundation Bearing and Lateral Pressures**

Assuming proper preparation of bearing soils to mitigate differential settlement and sufficient depths to mitigate expansion potential, it is expected that allowable bearing capacities in the range of 1,800 to 2,500 psf can be utilized. The on-site soils are expected to exert moderate lateral pressures on retaining walls; active equivalent fluid pressure in the range of 40 to 50 pcf and at rest pressures of 55 to 65 pcf should be anticipated for level backfill. Clean sand or gravel backfill will apply the least equivalent fluid pressure, on the order of 35 pcf for active conditions and 50 pcf for at-rest conditions. Passive resistance values of 250 to 350 pcf (equivalent fluid pressure) for the soil would be expected, with friction coefficients in the range of 0.35 to 0.40.

**Surface and Subsurface Water**

As previously noted in the Geology Section of this report, *subsurface* water is expected to lie at such a depth that it would not be expected to affect the design or performance shallow conventional foundations. Appropriate *surface* drainage systems should be installed and maintained in any unimproved areas where a minimum slope of 2 percent for a minimum
distance of 10 feet from foundations cannot be provided. Proper drainage of retaining walls and thorough waterproofing of walls where transmission of moisture is undesirable should be provided.

**Erosion**

Drainage on the site is by sheet flow, and the majority of the expected surface water runoff on site appears to be controlled by existing hardscape and surface drains; the existing playfields and landscape areas also provide a limited amount of infiltration area for rainfall. No significant erosion features were observed during the site reconnaissance. Conventional erosion control measures such as vegetation, erosion matting, and maintenance of irrigation systems should be sufficient to control erosion. Rodent activity can disrupt drainage patterns; therefore an aggressive program of rodent control should be maintained.

**Asphalt Pavement**

The soil types anticipated at this site typically exhibit low to moderate resistance to the types of loads imposed by traffic. Moderately thick aggregate base sections with hot mix asphalt surface layers, based on the expected Traffic Index values, should be anticipated for any new or reconstructed parking areas.

5.0 CLOSURE

This report is valid for conditions as they exist at this time for the project described herein. Our intent was to perform this study in a manner consistent with the level of care and skill ordinarily exercised by members of the profession currently practicing in the locality of this project under similar conditions. No representation, warranty, or guarantee, is either expressed or implied.

If changes to the project become necessary, or if any of the assumptions used in the preparation of this report are not correct, this firm should be notified to provide modifications to this report as necessary. Any items not specifically addressed in this report are beyond our scope of services.

This document, the data, conclusions, and recommendations contained herein are the property of Earth Systems Pacific. This report shall be used in its entirety, with no individual sections reproduced or used out of context. Copies may be made only by Earth Systems Pacific, the client, and the client’s authorized agents for use exclusively on the subject project. Any other use is subject to federal copyright laws and the written approval of Earth Systems Pacific.

End of Text.
REFERENCES


California Building Standards Commission, *California Building Code* (CBC), 2010

City of Goleta, *Request for Qualification and Proposal (RFQ/RFP) for Seismic and ADA Needs Assessment for the Goleta Valley Community Center*, May 10, 2012


Jennings, C.W. and Bryant, B. *Preliminary Activity Fault Map of California*, California Division of Mines and Geology Open File Report 92-03, 2010


Santa Barbara County, *Seismic Safety and Seismic Element*, amended 2010

APPENDIX

Geologic Map

Historical Earthquake / Fault Map

FEMA Flood Zone Map

Site Design Response Spectrum
GEOLOGIC MAP
GOLETA VALLEY COMMUNITY CENTER
5679 Hollister Avenue
Goleta, California

EXPLANATION

Geologic Units

- Qa
  Active channel alluvium (Holocene)
- Qac
  Alluvium and colluvium (Holocene and upper Pleistocene)
- Qoq
  Intermediate alluvial deposits (Upper Pleistocene)
- Qoq
  Older alluvial deposits (Upper and middle Pleistocene)

Geologic Symbols

- Contact
  Dashed where approximately located or inferred
- High-angle fault
  Dashed where approximately located or inferred; dotted where concealed
- Thrust or reverse fault
  Dashed where approximately located or inferred; dotted where concealed.
  Striae on upper plate. Dip of fault plane between 30° and 80°
- Anticline
  Showing out on surface. Dashed where approximately located; dotted where concealed
- Syncline
  Showing out on surface. Dashed where approximately located; dotted where concealed
- Strike and dip of beds
  Horizontal / Inclined / Vertical

Approx. Scale: 1" = 1000'

Extract from: Geologic Map of Goleta, by Minor and others, 2007

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Appendix E – Structural Drawings
Appendix E – Structural Drawings