BEAUCHEMIN RESIDENCE

SENIOR PROJECT
ARCE 451

HANNAH ROGERS
FEBRUARY 19, 2018

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ABSTRACT

The following report details the structural engineering completed on the Beauchemin Residence, as well as the associated drawings, details, and special considerations. The Beauchemin Residence is an existing single story wood frame building on raised wood floor, located in the city of San Clemente. The scope of work includes calculations for a new roof, new walls (gravity & lateral), retrofit of the existing foundation, new foundation, and providing calculations. The process and progression of the structural design is documented, and correlated to the final product in the Appendix A & B.

PREFACE

On the last day of my undergraduate education, I absentmindedly listened to the lecture on wood sub-diaphragms, tension straps, and the importance of applying engineering ethics in the field. The more I listened, the more the burning question rang true: Do I really know what I am doing? That is the millennial question: 'Did I learn enough to make myself useful in my career path?' The ARCE student body collectively understands that this is a career of lifelong learning. Pride comes before the fall, and is especially relevant in structural engineering. This phrase still rings as true in practice, as it did when receiving my diploma. As an employee, the mantra is expressed in my daily decisions, amplified by the reality of the projects.

Interdisciplinary culmination and application of all theoretical and applied coursework defines the Architectural Engineering senior project. It exemplifies the skills earned at Cal Poly, and develops the communication within dissimilar majors. The Beauchemin Residence provided a real opportunity to apply my education, interact with other disciplines, and understand work in my trained career.

This report describes the scope, observations, process, problems, solutions, and production associated with a real world application of an anticipated Architectural Engineering undergrad degree, while addressing the comical comparisons between the student expectation of this career path, and the employee reality.

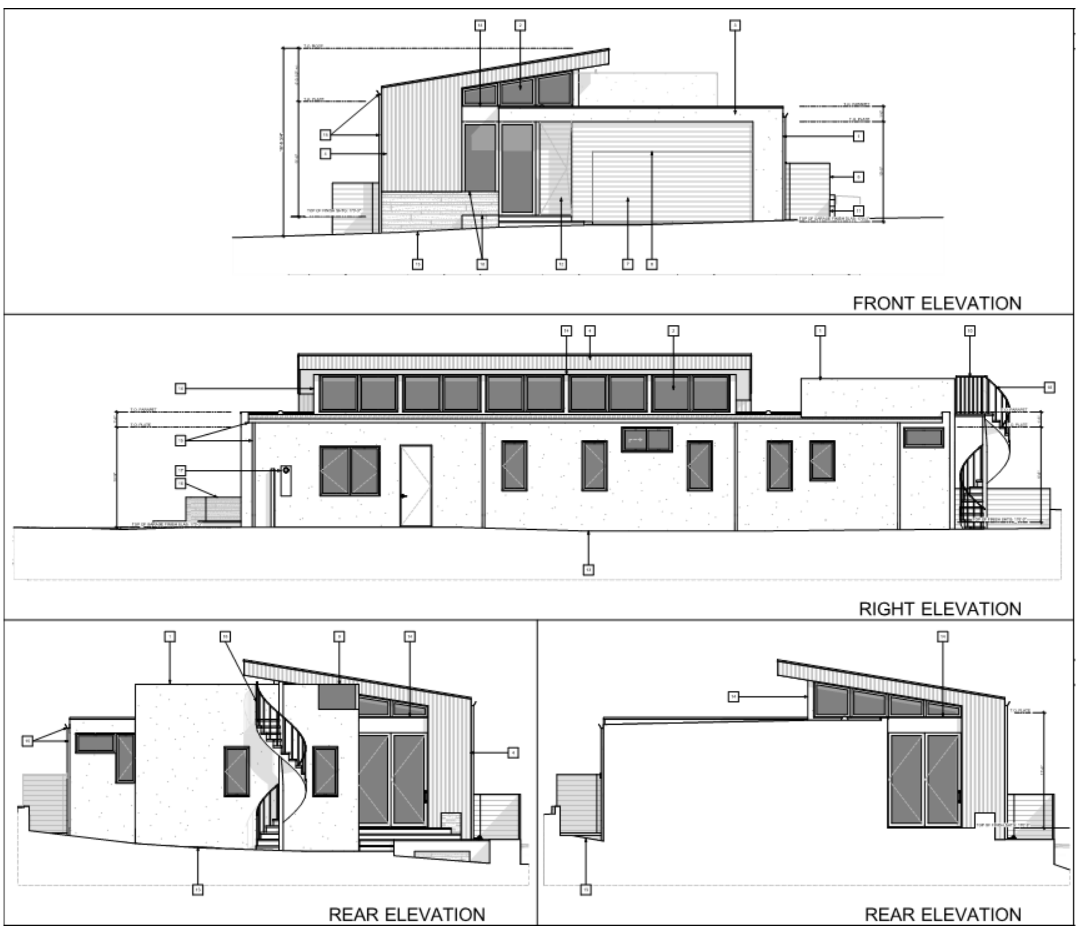
INTRODUCTION

The Beauchemin Residence is the current home of the Beauchemin family in San Clemente, CA. San Clemente building department grants separate permits according to the type of construction (new vs. remodel). A "new" building permit would cost much more than a "remodel" so the architect of this project desired to keep cost low by categorizing the home as a remodel. In San Clemente, for construction to be considered a remodel, the entire existing floor and foundation must remain. (If any is removed, the home then becomes new construction and will result in more costs and strenuous building permit process.) All else will be new construction. The owner hired James Glover Home as the architect and designer. The architect then is the owner's liaison with the other disciplines. The architect then hired the structural engineers on the project (Coastline Engineering Inc.) and the Geotechnical Engineer (LGC Geotechnical). As the structural engineer, I was provided with completed architectural elements, including plans, sections, and elevations. I also was provided with a full geotechnical report, with foundation recommendations by the geotechnical engineer of record.

Due to the nature of this project, and the layout of most structural engineering related projects, the text in this report will frequently reference Appendix A (Structural Calculations) and B (Structural Drawings).

SCOPE OF WORK

As the structural design engineer for the Beauchemin Residence, the purpose and objective of this project was to provide structural plans, calculations, and details to show how the building will resist gravity and lateral loading, and instructions for construction. I worked, and continue to work with the other disciplines on this project, including but not limited to the architect, geotechnical engineer, plan check department of San Clemente, and the contractor. The following figure shows the architectural elevations, and prospective construction.



Architectural Elevations (Fig.1)

INITIAL OBSERVATIONS

After reviewing the plans provided by the architect, I made note of several things that needed special consideration. First, the middle (East-West) line has clear-story windows, which are attached to separate roof diaphragms. As the engineer, I immediately recognized that the shear from the upper diaphragm would have to be transferred down through these openings into the lower diaphragm or shear walls, depending on my chosen lateral system. Later, this factored into determining the location of shear walls. Additionally, the architect selected various roofing materials, and I needed to be aware of the specific weights and location of each in my calculations and detailing. The last unique roof addition was a roof deck on new framing and foundation on the southwest corner of the building which affects type, slope, and size of framing; with an additional live load.

On the interior, the home will be vaulted ceilings, and therefore all beams and joists must be flush. I considered this while designing the depth of the gravity members. After the initial submission to the architect, the owner chose to include a dropped floor in the shower at the new bathroom. This required special attention and some framing modifications, with additional detailing. This is addressed in more detail in the "Special Considerations" portion of the report.

The floor framing as displayed by the architect is raised wood floor, but retains all the existing floor framing and structure. This is significant in transferring lateral load into the ground, since the existing framing would have to be retrofit and is inherently be weaker than new concrete, or framing members. I also noted that there is slab on grade at the garage, which is typical in residential construction. This requires special detailing in the transition between foundation and holdown types.

In addition to the observations drawn from the architectural foundation elements, the geotechnical report documented that the existing footing was not code compliant due to expanding soil over the life of the building. These observations can be located in the full soils report (Appendix C). After extracting these applicable statements and suggestions, I used both the suggested strength values for soil bearing and concrete requirements in the calculations. I also paid careful attention to detailing, and specific problems on foundations caused by expansive, sulfuric soils. These problems are further addressed in the “Special Considerations” section.

GRAVITY SYSTEM

PROCESS

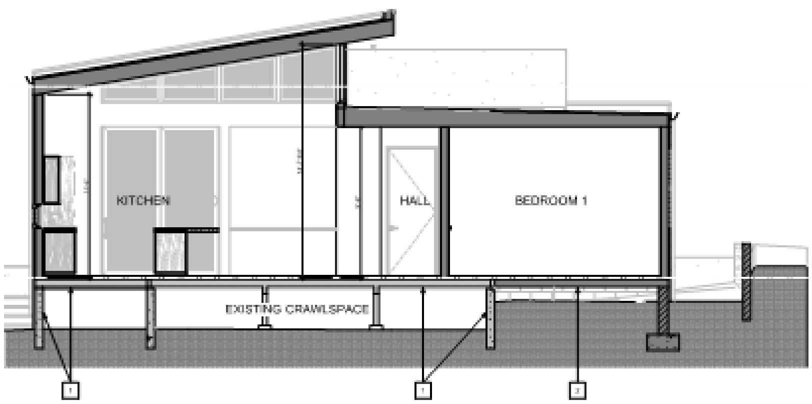
In analyzing the gravity system in this structure, I was especially aware of the specific material weights, as they vary significantly throughout the project, in addition to the available depth of members prescribed by the designer, and combining the new framing with the existing framing, specifically at the foundation and raised floor.

DESIGN LOAD DERIVATIONS

The architect, in consultation with the owner, selected a lightweight asphalt shingle for the roofing on the monoslope portion of the roof, with a layered bitumen and gravel roof on the remaining roof areas. A thin concrete composite called "Dex-O-Tex" which is applied much like concrete, but weighs significantly less, was selected for the roof deck. The live loads were prescribed per the ASCE 7-10 code (Section 4-1) and noted on the "Design Loads" sheet on page 2 of the calculation package (Appendix A). To determine the deck live load, the ASCE 7-10 states that "Balconies and Decks" are to be "1.5 times the live load for the area served" (ASCE 7-10 Table 4-1), and therefore, is sixty pounds per square foot. After determining my design loads (Pg. 2/Appendix A), I started initial layouts of the structural elements.

SLOPED UPPER ROOF FRAMING

As in many modern homes, the architect desired a thin roof eave that extended visually along the midline of the roof. The thin profile and slope draws the eye upward and is visually appealing for many homeowners. For the structural engineer, this limits the depth of the joists or beams and can be difficult to satisfy. Figure 2 below shows the profile for gravity framing, provided by the architect.



Architectural Section (Fig. 2)

After determining the roof-rafter layout, I supported the roof-rafters with cantilevered beams, where required. I assumed that all stud walls would be available for bearing, and that posts could be "hidden" within. This will be explained in greater depth later.

I visually located places that required beam support, and used tributary areas and joist spans to determine the loading of each member. Beams were labeled arbitrarily initially, and then consecutively, and correspond to the calculation sheets. If the members had similar span, tributary area, and loading, they would receive the same size but no nomenclature. I used Enercalc to determine the size required for each upper roof member, and noted the reactions (Appendix A).

Enercalc is a computer program that allows an engineer to model members and apply loads. The program checks unity after the user applies system variables including but not limited to weight, type of wood, factors (incising, temperature, etc.) and size of the beam. As shown in the calculation package, the cantilevered beams took small load, only supporting one tip of the roof, and therefore had an uplift load (Appendix A). I made note of the uplift load on the cantilevered beams, since this connection would need hardware sufficient for uplift. I also decided to approach posts after the full analysis of the roof framing, since some posts would be supporting multiple beams.

FLAT ROOF FRAMING

After resolving the gravity force at the monoslope roof, I addressed the lower roof, which had a much higher material load, in addition to a roof deck. As mentioned earlier, the gravity members at the deck were potential issues as the live load was triple the regular roof live load. I also included any gravity load from the upper roof translating down into the lower roof.

After reviewing the architectural “as-builts” sheets, I decided that the interior wall, along grid line 3 would be used as bearing, since it previously was an exterior bearing wall, with a existing footing below. I selected this line to reduce the number of new continuous footings for cost and ease of construction. If the roof rafters spanned North to South, similar to the upper sloped roof, then this bearing line would only need one continuous footing, rather than making several new continuous footings at the interior, which is costly for the contractor and owner. I also determined that all exterior walls (on both new and existing footings) would be assumed to be bearing.

At the lower roof, the architect allowed 14 inches of depth for the roof members. I wanted to use as few footings as possible, and since line 3 already required bearing wall from the upper roof loading, I chose to frame the lower roof rafters in the North-South direction. Because the span was so large (20 feet), the full depth was required, and the spacing was reduced to create an acceptable deflection per rafter. (Appendix A & B)

Beams and headers were provided over every opening along the bearing lines 1, 3, & 5. Following the same process as the upper roof, the tributary area and loading for each was determined based on the span of the rafters, and were calculated using Enercalc. The calculation and the reactions for each member are recorded in the calculation package (Appendix A). In the calculations, I attempted to use sawn lumber (DF-L) whenever possible, as it is cheaper, and most available. However, as the span and loading increased, I fortified the beam type to be manufactured lumber, where required to maintain the depth required by the architect.

Special attention to the type of connections was required because the roof type, span, and depths vary throughout this project. Sometimes, because the rafter was 14" depth, the beam was limited the same depth, to match the flush ceiling or to connect via Simpson 'U' hangers. This means the member selected is not efficient, and may require further cost, but the governing aspect is the visual depth of the rafters.

After completing the calculations and layout of the beams, I completed a similar analysis with the headers. (Appendix A) The roof beams are labeled with the prefix “RB-.” Similarly, roof headers have the prefix “RH-“ and floor beams have the prefix “FB-.” Finally posts were provided where required for gravity load. (Note: Because post calculations are simply the pressure over the area, the calculation is not included in the submitted set.) The sizes or locations were not finalized until after completing a lateral analysis since uplift, overturning, and shear loads could be applied at some locations.

CONCLUSIONS FOR VERTICAL SYSTEM ANALYSIS SYSTEM

Overall, this system is reasonably efficient and is within the architect’s desired depths and locations. The architect desired a thin profile, and also wanted to maximize the space while maintaining a single story profile. This home was initially very small (just over 1000 square feet), so the owner desired to double the space. The architect was hired to modernize and amplify the space. Since it is located near the beach the owner also wanted plenty of windows and vaulted ceilings. As the structural engineer I struggled to make sure each location complied with what the designer and owner wanted, especially due to the fact that I had no experience with construction flow at this point in my career. To this day, I am still catching mistakes and making edits to the gravity system, especially with hardware as I become more familiar with general practice, and contractor’s preferences.

LATERAL SYSTEM

LAYOUT

Timber frame shear resistance is governed by the panel thickness and the number and spacing of fasteners, and therefore is simple to construct, and can easily be adjusted both in the office and the field. It is also simple to trace diaphragm shear due to the fact that timber construction acts as a flexible diaphragm, and so torsion can be completely ignored in calculation. As a budding engineer, this project helped build basic understanding of lateral system construction.

This home is modern, so the space is very rectangular, which allowed for numerous options of shear wall length and location. Since everything but the floor and foundation is new construction, and it is a small single-story building; the governing factor in shear resistance is the existing foundation capacity.

This project is residential and may have more than one person occupying it over the home's lifespan. As an engineering precaution the exterior walls of the building should resist the all of the lateral force, in case a new owner wants to remove interior walls. However, because this home has two diaphragms that act separately, the upper and lower roof, both need to be laterally supported at the exterior of each diaphragm, respectively. Therefore, the center-line of the building running East to West was also used for lateral resistance since this is already being used for bearing and has an existing footing.

I assumed that the floor, although technically a diaphragm, did not act as one because it is attached directly to the foundation element, and the ground, and therefore will not affect the seismic distribution of forces, using the equivalent lateral force procedure. I also assumed that the diaphragm is flexible, and that seismic forces will act in only one direction at any given moment. This is reflected in the calculations. (Appendix A)

PROCESS

The full lateral analysis of this home is included in Appendix A, (pgs. 27-40). I started by measuring the area of the building in AutoCAD, making sure to account for differing material weights. Each area associated was multiplied by the dead load (PSF) and summed to get the building total weight (Appendix A, pg. 27). The floor load of the building was not included in the lateral calculations because it is attached to the foundation. The floor load that is shown on this page is the roof deck load.

After determining the weight of the building, I used the longitude and latitude and USGS.gov to determine the seismic parameters for this site. This is a residential, Type II risk-category, site class D (default) structure. (IBC 2015) I followed the Equivalent Lateral Force Procedure (ASCE 7-10) to determine the base shear, and the seismic distribution of forces (Appendix A, pg 27).

From previous engineering experience, I expected seismic loads to govern, especially on the Southern California coast. However, wind pressures are high, especially around open bodies of water on the Pacific Coast (Exposure C), and therefore needed to be accounted for. Wind pressures were determined via ASCE 7-10 and recorded in the calculations (Appendix A, pg. 28).

Assuming lateral loads only act in one direction at a time and are periodic, I selected walls on plan that could be used as shear in the North-South, and the East-West directions. From prior experience, I know that shear walls should not be less than 2:1 height to length ratio, so no reduction in capacity is required. (NDS 4.2.4). After laying out the available lengths, I determined the shear load applied to each grid line, and the available shear capacity. (Note: Coastline Engineering has a specific typical schedule of shear walls already determined, so my results reflect a limited set of options. However, these are typical in wood construction, and I also verified the type required in NDS correctly corresponded to the shear wall schedule on Sheet S5, Appendix B).

The shear load was determined by the multiplying the tributary area that the shear wall resists by the portion of base shear that is distributed to that diaphragm. Each lateral calculation sheet (Appendix A) displays both the wind load and seismic load per wall line. Using engineering judgment, I used the governing load to determine the capacity required per shear wall. Wood shear walls act communicatively along each line, since the roof acts as a flexible diaphragm. Shear load to each shear wall is the length of that specific shear wall divided by the sum of shear wall in that line. This is clearly recorded in the lateral sheets. (Note: the Simpson Wood Shear Wall will be mentioned in the "Special Considerations" section.)

After resisting the base shear in each direction by the compilation of shear walls, shown in calculation pages (Appendix A, 29-39). I used the applied lateral load and the dead load to determine the overturning at the base of the shear wall.

Using the geometry of the wall, and the dead loads applied to each wall, I determined the uplift and overturning loads at each wall end. Each end that had a positive uplift load required a holdown. I used the Simpson catalog to find hardware that would resist each load. For the selected hardware, I verified that the end member (post) was sufficient for the holdown. (Coastline has a set holdown schedule that has been modified throughout the experience of my superiors, in terms of cost, effectiveness, and ease of construction, so my selection was narrowed.) After selecting a holdown, I quickly hand checked that the tension in the vertical post member was sufficient, and that the bottom plate did not crush under overturning load. This is not shown in the calculations because it is a minor aspect of the project, and is not required for plan check. (Several things that are inherent calculations to an experienced engineer were not included in the final calculation package.)

It was assumed that the existing continuous footing concrete could handle any uplift that would cause cracking or break out. So the retrofit capacity was limited to the pull out capacity on the epoxied anchor bolts. More foundation analysis was done in the overall analysis. (Appendix A, pgs 41-46)

After determining the posts required at each holdown, I verified that the posts were sufficient to manage any gravity load. If the gravity load required an increased post area, I provided sufficient support and graphically adjusted this on the plans (Appendix B).

Finally, I checked the redundancy in the project, to make sure my rho factor was not required. I was confident that the rho factor would be 1.0, since several redundant shear walls were provided in each direction. The check confirmed the assumption. The base shear was sufficiently resisted.

CONCLUSION OF SYSTEM

This system is effective. The shear capacity is sufficiently larger than the applied shear in each direction. The holdowns are conservative, which helps to reduce visual cracking on the exterior cladding if an earthquake were to occur. It was interesting to learn lateral load path from a realistic perspective. Due to my lack of construction knowledge, I always had trouble tracing the shear load. Now, after this project, and guidance from my employer, I understand how to transfer this load in a safe and effective way.

FOUNDATION

After the gravity and lateral load was transferred to the anchor bolts, foundation elements were required to transfer this load to the ground. All exterior walls, bearing walls, and shear walls required continuous footings below. I graphically showed this on S2 (Appendix B). I checked the existing footings with the additional load from the new roof, and new exterior wall weight. After adding up the pounds per linear foot applied to this location, and comparing this to the bearing capacity of the soil under the area of the footing, the existing continuous footing was determined to be more than sufficient and would not need any underpinning or additional footing. The bearing capacity of the soil was determined by the geotechnical engineer of record, which can be found in Appendix C.

The same process was completed at the new continuous footings. The final foundation layout can be seen on S2, Appendix B.

Following the continuous footing analysis, I identified potential areas that high point loads were applied to the foundation to provide isolated pad footings. I used the reactions at each post from the reactions page (Appendix A, pg. 26) to determine the size of footing required by summing the load applied, divided by the available bearing resistance of soil. The footings required were then graphically shown on S2, and called out according to the schedule on S5. Typical minimum (A_s) rebar was assumed in all footings. Foundation detailing will be addressed further in the report.

DRAWINGS

PROCESS

Coastline does not bid on or accept projects without AutoCAD file and PDF set from the architect. This ensures that all sizes, layout of doors, windows, and roof slopes are exactly how the architect has determined, and all further communication is clear. After receiving this from the architect, I used the layout provided as a base layer and drew all structural elements graphically.

The general notes section includes the company standard for foundation, wood, steel, and masonry construction. I completed this page by using a template from other jobs. The site information, and any specific information is adjusted in response to the USGS report, and the geotechnical recommendations.

I began drawing roof framing (S3, Appendix B), which then determined the foundation requirements. Each structural member was drawn in bold and the notations call out size and nomenclature that corresponds to the calculations. Posts were drawn in section to the size required, where required on plan. As I drew each post, I used the Simpson catalog to also call out appropriate hardware, taking into account the load, type of beam, and available connection. For continuous posts, only member top hardware was called out on the roof framing plan. For the king posts, specifically at the clear-story windows, both top and bottom hardware was called out on the roof plan. Typical construction wood and fastener schedule is provided on S1, therefore all areas without notations can be assumed to be normal construction from the schedule.

For the lateral framing at the roof level, I applied a hatch to indicate each shear wall, and the solid side of the pentagon indicates the side that the shear panel should be applied. The shear wall types correlate to the shear wall schedule on S5. Holdowns on plan correlate to the holdown schedule (S5), and are called out with leaders and a boxed number at the locations required. The holdown schedule specifies the type of post required for the holdown, but if the gravity load required a larger member, it was noted on plan with a text-callout.

After completing the roof layout and lateral framing, I used the wall layout and shear walls from the roof plan to draw a foundation plan (S2, Appendix B). The architect provided some existing foundation outline, so I used the new walls and the old foundation to determine where new foundation would be added. I drew the new foundation to be a six-inch stem wall to match the existing construction (raised wood floor). The new footing was hatched to differentiate from the existing.

After completing this, I drew the floor joists, floor beams, and isolated footings required per my calculations. Using a similar process as the roof framing, I called out the size and nomenclature corresponding to the calculations. Posts were copied from the roof framing plan, and the required size of the footing was denoted by the alphabetical callout corresponding to the pad footing schedule (S4, Appendix B). Hardware at the base of each column, and at the beam connections were selected (via Simpson catalog) and then graphically noted on plan.

After completing the plans, I started detailing each location that required special construction instructions. This includes locations where shear is transferred, there is a change in the type of system (such as a beam or header), or gravity transfer that is unique, and not simply hardware (if the hardware is called out on plan, the construction is then self explanatory and therefore requires no detail).

Each detail was drawn and associated with a section graphic on both the roof and the foundation plans. The foundation retrofit detail (14/S6) is a response to the geotechnical report requiring either compacted fill or slurry mix on the existing foundation. This was reviewed by the geotechnical engineer and verified before submission to plan check. The shower floor transition requires specific detailing in floor type and connection since the diaphragm is discontinuous (17 & 18/S7, Appendix B). Finally, on S8, the details show the construction of the clear-story window framing, which was a unique challenge during this project.

In addition, S4 and S5 are typical details and schedules that Coastline uses regularly. I drew these from the template and reviewed them to make sure each applied to this residence. Attached to the end of these plans are the detail sheets pertaining to the Wood Strong Wall construction from Simpson. Because I used a StrongWall Wood Shear Wall in this project, I needed to attach these sheets to my drawings for approval by plan check.

CONCLUSION OF DRAWINGS

The goal of the plans is to be a completely self-explanatory set of instructions for the contractor. In reality, I expect some communication from the contractor, but my goal is to receive as few phone calls as possible. I believe these drawings include all information related to the residence, and clearly show the construction flow.

SPECIAL CONSIDERATIONS

Existing Foundation Retrofit

The geotechnical engineer of record observed that the soil was expansive (due to the high sulfur content in the soil) and that the existing footings had been pushed up past the required embedment depth per code (12" minimum embedment). The geotechnical report suggestions included the following:

"the onsite soils should be considered as having a designated sulfate exposure class "S2" per ACI 318-14, Table 19.3.1.1. As a result, (per ACI Table 19.3.2.1) the minimum compressive strength of structural concrete shall be 4,500 psi, the maximum water to cement ratio shall be 0.45 and the cementitious material type under ASTM C-150 shall be Type V."

Furthermore, "the existing stem wall footing for the residence must have less than 12" minimum embedment required by the current building code. To achieve this minimum embedment, we recommend placement of either 6 inches of compacted fill adjacent to the stem wall and extending at least 3 horizontal feet away from the stem wall. Alternatively, 6 inches of 2-sack slurry cement may be placed adjacent to the interior side of the stem wall and extending at least 3 horizontal feet away from the stem wall. Alternatively, the existing stem wall may be deepened to achieve the minimum required embedment. Recommendations for deepening should be provided by the project foundation engineer."

And finally: " an allowable soil bearing pressure of 1,500 psf may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface."

To retrofit the existing stem wall and continuous footing, I added a detail to place a two-sack slurry mix on the interior edge and extending 3 feet inward from the footing. (S6, Appendix B).

Bathroom Floor Modification

After the first submittal, the architect and owner decided to drop the shower floor in the master bathroom. As the engineer, I accounted for the lack of curb by creating a step in the framing. To achieve this, I chose a smaller floor joist (which would provide the required step), and spaced the joists closer together in that area.

The change in floor height caused a discontinuous diaphragm, and therefore I provided two new details to show the diaphragm connection. These can be found in the structural details (17 & 18/ S7, Appendix B).

Simpson Strong Wall

The last special problem in this home was the small length of wall provided at Grid F. Simpson Strong Walls are simple to use because the capacity is already tabulated in the catalog, and includes the connections required. Using the dimension of wall provided, I selected wall type from the catalog, and graphically noted this on plan. The challenge of using a Strong Wall was accounting for the shear load and making sure that the foundation element could handle such a concentrated shear, and uplift. Since the Strong Wall would be on slab on grade at the garage, the governing factor was simply the weight of the concrete resisting the overturning. To combat this, I provided a isolated pad below the Strong Wall. (S2, Appendix B).

INTERDISCIPLINARY INTERACTION

Residential construction is more of a conveyor belt process than commercial construction. Prior this opportunity, I worked for a commercial engineering firm, and learned how to interact with the architect, the owner, and the construction manager throughout the course of the project. Residential construction begins with an owner. They own land or an existing home, and want to build new, add to an existing home, or remodel an existing home. Owners have a specific idea, and they hire an architect or designer to make that a reality. The architect on this project interacted regularly with the owner and hired me as the engineer. The architect also is the plan check liaison, and determines what elements are required for a permit. I received the architect's initial design and communicated and coordinated my work with their plans. The contractor is often hired at the same time as the structural engineer, but we communicate with the contractor long after the project is approved by plan check.

The architect hired a geotechnical engineer because the homes built in San Clemente are on poor soil, so code minimums are too conservative for design. The geotechnical engineer has the least interaction in the process, and my only communication with the geotechnical engineer to date is the report provided via email. The recommendations for retrofitting the existing foundations, the allowable bearing pressure, and the existing conditions of expansive soil were all incorporated into my project in calculations, details, and analysis.

The architect has been in contact several times, mostly via phone and email. This architect specifically is well-known to provide complete and accurate drawings, which results in less verbal communication since the plans are effective. However, initial conversations included bidding, material choices, and preferred layout which included both the architect's and owner's preferences relayed by the architect. After this, the architect hired our company and emailed the specific materials desired for each portion of the building.

After these conversations, and the provided dimensions and architectural layout, I drew structural plans, and completed the analysis. Periodically, I provided updates to the architect to verify and determine the available heights of ceiling, or thickness of roof members. Each time, the architect would review and relay any more minor changes made by the owner. After my initial work was done, the architect relayed that the owner changed his mind, and wanted a dropped shower floor. I updated and revised the framing and provided the new details associated with the change.

A few weeks later, I received an email with an updated set of architectural drawings, indicating a change in some interior walls. The wall location had changed along Grid (3) but available length for shear did not change. Although the shear wall length did remain the same, the length and location of some posts and beams had changed as a result. After verifying that the initial beam depths were sufficient for the new load pattern, I revised my plans and calculations. I then sent this back to the architect, who has since submitted it to plan check of San Clemente.

Because this is an ongoing project, plan check comments or calls from the contractor are still pending. I do expect to hear from them soon, as the project begins to pick up momentum. After the construction is complete, expected summer 2019, I will be able to drive past this new home. I also have begun to build a relationship with the architect, and will look forward to working with him in the future.

CONCLUSION

As a new engineer, I struggled to learn rules of thumb associated with this company. The angle of the lines, the direction of text, and the minute detailing of certain multi-leader arrows was completely new to me. In addition to the new company standards, I was re-learning the AutoCAD program. In previous experience, I predominately used Revit, so this project challenged me to learn several new fields at once.

Additionally, because I am inexperienced in construction flow, detailing real connections was challenging. This project also introduced me to the vast options for wood connections. In a comical way, residential design is simply a giant jigsaw puzzle; determining which pieces can fit, are reasonably constructible, and are cost effective. I look forward to learning more techniques and knowledge of construction as my career advances.

REFLECTION

My first project in “the real world” of structural engineering was architecturally sleek, set in the beautiful, laidback, and ironically strict (in terms of building regulations) city of San Clemente, CA.

As time passes since graduation and more projects have passed across my desk, I have learned that each project is so uniquely problematic, that I will never be bored. A new challenge arises everyday, providing opportunity for my brain to either grow or explode.

I also learned that I will draw fewer moment diagrams the rest of my life, than all the moment diagrams from only one year in school. My expectation was that I would need to know all these fundamentals, but there are advanced computer programs that account for numerous factors in a single click of a button. But I understand that I am here to know what the computer is doing.

Even though it took me four years to learn what the computer does in one second, my education taught me to catch calculation mistakes. My new experience is teaching me to be aware of the subtle efficiency of a member and adjust a calculation to accommodate it.

My education taught me the components of a building and some construction process. My experience is teaching me that construction never goes as the engineer intends.

My education taught me that flipping the pages in the Simpson manual is the easiest part of the process. The contractor on the other end of the phone reminds me that my arbitrary choice of hardware is too costly, and that I need to be aware of the subtleties that can save hundreds of dollars.

My education taught me how to share workloads, teach via texting my classmates, and scour the internet for textbook PDFs . My experience is teaching me to professionally answer the phone and reply to emails with engineering confidence.

My education gave me the basics;

my new job is proving I really learned them.

APPENDIX A : CALCULATION PACKAGE

Coastline Engineering, Inc.
STRUCTURAL ENGINEERING SERVICES
www.CoastlineEngInc.com
(760) 436-1344

STRUCTURAL CALCULATIONS

BEAUCHEMIN RESIDENCE

148 W. AVENIDA CADIZ,
SAN CLEMENTE, CA 92672

PROJECT NUMBER: 17-046
8/2/2017
CALCULATED BY: H.R.
REVIEWED BY: M.I.

DESIGNER
JAMES GLOVER HOME, INC.
www.jamesglover.com
(949) 492-7618

OWNER
BRANDON & KYLIE BEAUCHMIN
148 W. AVENIDA CADIZ,
SAN CLEMENTE, CA 92672

APPENDIX A

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GOVERNING CODES

2016 CALIFORNIA RESIDENTIAL CODE (CRC)

WOOD	NDS-15
CONCRETE	ACI 318-14
MASONRY	TMS 402-13 / ACI 530-13 / ASCE 5-13
STEEL	AISC 360-10 / AISC 341-10
MINIMUM DESIGN LOADS	ASCE/SEI 7-10

APPENDIX A

DESIGN LOADS

ROOF LOADS

RL1		RL2	
ASPHALT SHINGLE	2	BUILT UP W/ GRAVEL	6
15/32" SHEATHING	1.5	15/32" SHEATHING	1.5
FRAMING	4	FRAMING	4
INSULATION	2	RIGID INSULATION	
DRYWALL	3	DRYWALL	3
OTHER	1.5	OTHER	1.5
DEAD LOAD	14 PSF		16 PSF
LIVE LOAD	20 PSF		20 PSF
TOTAL LOAD	34 PSF		36 PSF

FLOOR / DECK LOADS

FL1		FL2		FL3	
DEX-O-TEX (OR EQUIV.)	2.5	HARDWOOD	4	TILE	10
3/4" SHEATHING	2.5	3/4" SHEATHING	2.5	3/4" SHEATHING	2.5
FRAMING	4	FRAMING	4	FRAMING	4
DRYWALL	3	DRYWALL	3	DRYWALL	3
OTHER	1.5	OTHER	1.5	OTHER	1.5
DEAD LOAD	13.5 PSF		15 PSF		21 PSF
LIVE LOAD	60		40 PSF		40
TOTAL LOAD	73.5 PSF		55 PSF		61 PSF

EXTERIOR WALL LOADS

EW1		EW2	
STUCCO	3		
SHEATHING	1		
FRAMING	2		
INSULATION	1		
DRYWALL	3		
OTHER	1.5		
DEAD LOAD	11.5 PSF		

INTERIOR WALL LOADS

IW1		IW2	
DRYWALL x2	5.5		
FRAMING	2		
OTHER	1.5		
DEAD LOAD	9 PSF		

GUARDRAIL / PARAPET LOADS

GP1		GP2	
STUCCO	20		
OTHER	1.5		
DEAD LOAD	21.5 PSF		

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

ROOF LEVEL

RB-1 - GRID LINE C-E

	LOAD TYPE	RL1			TOTAL
W1	TRIB. LENGTH	8.5		D =	119 PLF
	D	119		Lr =	170 PLF
	Lr (L)	170		L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

RB-2 - GRID LINE 3

	LOAD TYPE	RL1			TOTAL
W1	TRIB. LENGTH	9.5		D =	133 PLF
	D	133		Lr =	190 PLF
	Lr (L)	190		L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

RB-3 - GRID LINE 2

	LOAD TYPE	RL2	GP1		TOTAL
W1	TRIB. LENGTH	3.5	3.5	D =	131 PLF
	D	56	75	Lr =	70 PLF
	Lr (L)	70		L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

Multiple Simple Beam

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Description :

Wood Beam Design : RB-1

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-1(

BEAM Size : **3.5x11.25, TimberStrand LSL, Fully Unbraced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

Wood Species : Trus Joist Wood Grade : TimberStrand LSL 1.55E

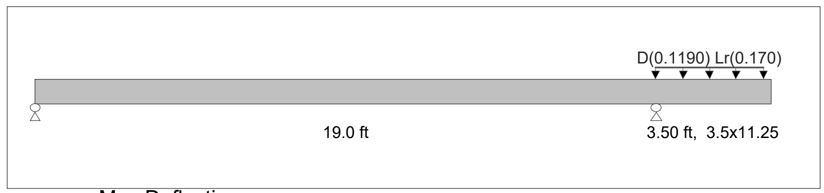
Fb - Tension	2,325.0 psi	Fc - Prll	2,170.0 psi	Fv	310.0 psi	Ebend- xx	1,550.0 ksi	Density	44.990 pcf
Fb - Compr	2,325.0 psi	Fc - Perp	900.0 psi	Ft	1,070.0 psi	Eminbend - xx	787.82 ksi		

Applied Loads

Unif Load: D = 0.1190, Lr = 0.170 k/ft, 19.0 to 22.250 ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.085** : 1
fb : Actual : 248.08 psi at 19.000 ft in Span # 1
Fb : Allowable : 2,906.25 psi
Load Comb : +D+Lr+H, LL Comb Run (*L)
Max fv/FvRatio = **0.066** : 1
fv : Actual : 25.57 psi at 19.000 ft in Span # 1
Fv : Allowable : 387.50 psi
Load Comb : +D+Lr+H, LL Comb Run (*L)



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	-0.03		-0.05				
Right Support	0.42		0.60				

Max Deflections			
Downward L+Lr+S	0.060 in	Downward Total	0.103 in
Upward L+Lr+S	-0.056 in	Upward Total	-0.096 in
Live Load Defl Ratio	1390 >240	Total Defl Ratio	816 >180

Wood Beam Design : RB-2

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-1(

BEAM Size : **4x12, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

Wood Species : Douglas Fir - Larch Wood Grade : No.1

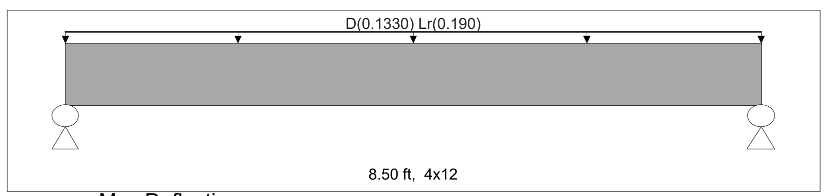
Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.20 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.1330, Lr = 0.190 k/ft, Trib= 1.0 ft

Design Summary

Max fb/Fb Ratio = **0.255** : 1
fb : Actual : 474.14 psi at 4.250 ft in Span # 1
Fb : Allowable : 1,856.25 psi
Load Comb : +D+Lr+H
Max fv/FvRatio = **0.192** : 1
fv : Actual : 40.79 psi at 7.565 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.57		0.81				
Right Support	0.57		0.81				

Max Deflections			
Downward L+Lr+S	0.034 in	Downward Total	0.057 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	3020 >240	Total Defl Ratio	1777 >180

Wood Beam Design : RB-3

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-1(

BEAM Size : **5.25x14.0, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

Wood Species : Trus Joist Wood Grade : Parallam PSL 2.0E

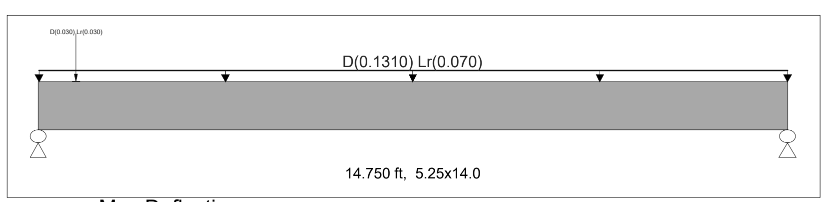
Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	45.050 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	625.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

Unif Load: D = 0.1310, Lr = 0.070 k/ft, Trib= 1.0 ft
Point: D = 0.030, Lr = 0.030 k @ 0.750 ft

Design Summary

Max fb/Fb Ratio = **0.108** : 1
fb : Actual : 384.05 psi at 7.375 ft in Span # 1
Fb : Allowable : 3,563.50 psi
Load Comb : +D+Lr+H
Max fv/FvRatio = **0.071** : 1
fv : Actual : 25.68 psi at 13.619 ft in Span # 1
Fv : Allowable : 362.50 psi
Load Comb : +D+Lr+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.99		0.54				
Right Support	0.97		0.52				

Max Deflections			
Downward L+Lr+S	0.031 in	Downward Total	0.090 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	5630 >240	Total Defl Ratio	1965 >180

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

ROOF LEVEL

RB-4 - GRID LINE 3

	LOAD TYPE	RL2			TOTAL
W1	TRIB. LENGTH	10.25		D =	164 PLF
	D	164		Lr =	205 PLF
	Lr (L)	205		L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

RB-5 - GRID LINE B

	LOAD TYPE	FL1	GP1		TOTAL
W1	TRIB. LENGTH	7.5	3.5	D =	177 PLF
	D	101	75	Lr =	
	Lr (L)	450		L =	-450 PLF
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

RB-6 - GRID LINE 4

	LOAD TYPE	GP1			TOTAL
W1	TRIB. LENGTH	3.75		D =	81 PLF
	D	81		Lr =	
	Lr (L)			L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

Multiple Simple Beam

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Description :

Wood Beam Design : RB-4

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **5.25x14.0, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

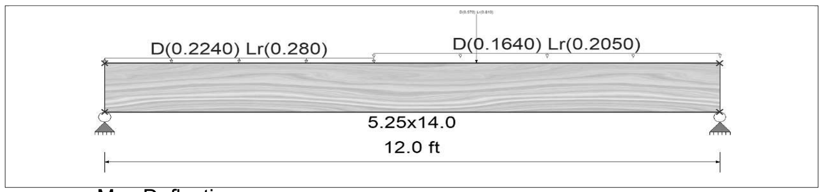
Wood Species : Trus Joist | Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	45.050 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	625.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

Unif Load: D = 0.2240, Lr = 0.280 k/ft, 0.0 ft to 5.250 ft, Trib= 1.0 ft
Unif Load: D = 0.1640, Lr = 0.2050 k/ft, 5.250 to 12.0 ft, Trib= 1.0 ft
Point: D = 0.570, Lr = 0.810 k @ 7.250 ft

Design Summary



Max fb/Fb Ratio = **0.217** : 1
fb : Actual : 773.67 psi at 7.080 ft in Span # 1
Fb : Allowable : 3,563.50 psi
Load Comb : +D+Lr+H
Max fv/FvRatio = **0.156** : 1
fv : Actual : 56.63 psi at 10.840 ft in Span # 1
Fv : Allowable : 362.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.46		1.86				
Right Support	1.40		1.81				

Max Deflections

Transient Downward	0.066 in	Total Downward	0.116 in	
Ratio	2190 >240	Ratio	1236	>180
	LC: Lr Only		LC: +D+Lr+H	
Transient Upward	0.000 in	Total Upward	0.000 in	
Ratio	9999	Ratio	9999	
	LC:		LC:	

Wood Beam Design : RB-5

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **5.25x14.0, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

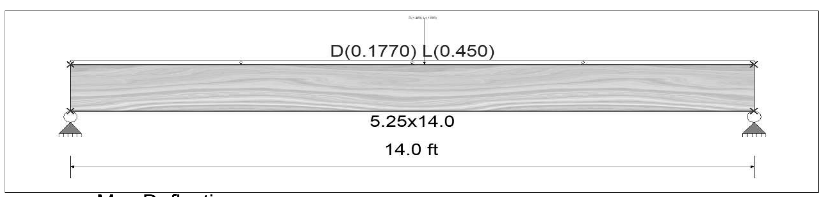
Wood Species : Trus Joist | Wood Grade : Parallam PSL 2.0E

Fb - Tension	2,900.0 psi	Fc - Prll	2,900.0 psi	Fv	290.0 psi	Ebend- xx	2,000.0 ksi	Density	45.050 pcf
Fb - Compr	2,900.0 psi	Fc - Perp	625.0 psi	Ft	2,025.0 psi	Eminbend - xx	1,016.54 ksi		

Applied Loads

Unif Load: D = 0.1770, L = 0.450 k/ft, 0.0 ft to 14.0 ft, Trib= 1.0 ft
Point: D = 1.460, Lr = 1.860 k @ 7.250 ft

Design Summary



Max fb/Fb Ratio = **0.502** : 1
fb : Actual : 1,429.94 psi at 7.233 ft in Span # 1
Fb : Allowable : 2,850.80 psi
Load Comb : +D+L+H
Max fv/FvRatio = **0.311** : 1
fv : Actual : 90.07 psi at 12.833 ft in Span # 1
Fv : Allowable : 290.00 psi
Load Comb : +D+L+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.94	3.15	0.90				
Right Support	2.00	3.15	0.96				

Max Deflections

Transient Downward	0.163 in	Total Downward	0.304 in	
Ratio	1031 >360	Ratio	552	>180
	LC: L Only		C: +D+0.750Lr+0.750L+H	
Transient Upward	0.000 in	Total Upward	0.000 in	
Ratio	9999	Ratio	9999	
	LC:		LC:	

Multiple Simple Beam

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Wood Beam Design : RB-6

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-1(

BEAM Size : **5.25x14.0, Parallam PSL, Fully Unbraced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

Wood Species : Wood Grade :

Fb - Tension	1,000.0 psi	Fc - Prll	1,000.0 psi	Fv	65.0 psi	Ebend- xx	1,300.0 ksi	Density	34.0 pcf
Fb - Compr	1,000.0 psi	Fc - Perp	1,000.0 psi	Ft	65.0 psi	Eminbend - xx	1,300.0 ksi		

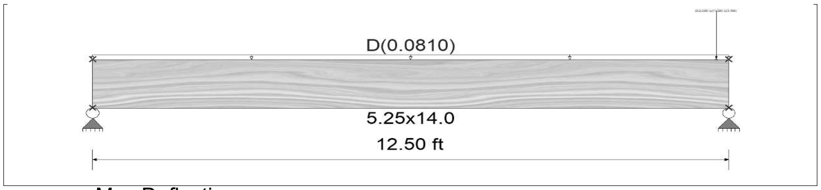
Applied Loads

Unif Load: D = 0.0810 k/ft, Trib= 1.0 ft
Point: D = 2.030, Lr = 1.020, L = 3.150 k @ 12.250 ft

Design Summary

Max fb/Fb Ratio = **0.164** : 1
fb : Actual : 160.64 psi at 7.542 ft in Span # 1
Fb : Allowable : 978.12 psi
Load Comb : +D+L+H

Max fv/FvRatio = **0.162** : 1
fv : Actual : 10.52 psi at 0.000 ft in Span # 1
Fv : Allowable : 65.00 psi
Load Comb : +D+L+H



Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.55	0.06	0.02				
Right Support	2.50	3.09	1.00				

Max Deflections

Transient Downward	0.009 in		Total Downward	0.043 in	
Ratio	9999	>360	Ratio	3502	>180
	LC: L Only			LC: +D+L+H	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

ROOF LEVEL

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RB-8 - GRID LINE B

	LOAD TYPE	FL1	GP1		TOTAL
W1	TRIB. LENGTH	7.5	3.5	D =	177 PLF
	D	101	75	Lr =	
	Lr (L)	450		L =	-450 PLF
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

RB-9 - GRID LINE 1

	LOAD TYPE	RL2		TOTAL
W1	TRIB. LENGTH	3.5	D =	56 PLF
	D	56	Lr =	70 PLF
	Lr (L)	70	L =	
P1	TRIB. AREA		D =	
	D		Lr =	
	Lr (L)		L =	

Multiple Simple Beam

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Description :

Wood Beam Design : RB-8

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

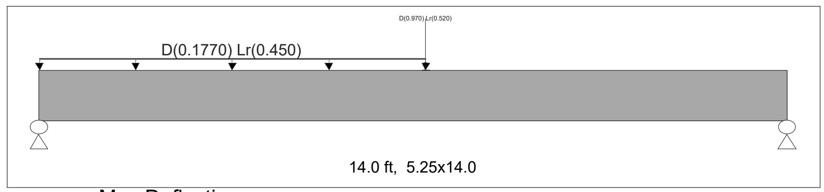
BEAM Size : **5.25x14.0, Parallam PSL, Fully Unbraced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch			Wood Grade : No.1					
Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.20 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.1770, Lr = 0.450 k/ft, 0.0 to 7.250 ft, Trib= 1.0 ft
Point: D = 0.970, Lr = 0.520 k @ 7.250 ft

Design Summary



Max fb/Fb Ratio = **0.562** : 1
fb : Actual : 932.08 psi at 6.533 ft in Span # 1
Fb : Allowable : 1,658.87 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.325** : 1
fv : Actual : 69.08 psi at 0.000 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.42		2.67				
Right Support	0.83		1.11				

Max Deflections

Downward L+Lr+S	0.135 in	Downward Total	0.227 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	1244 >360	Total Defl Ratio	738 >180

Wood Beam Design : RB-9

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

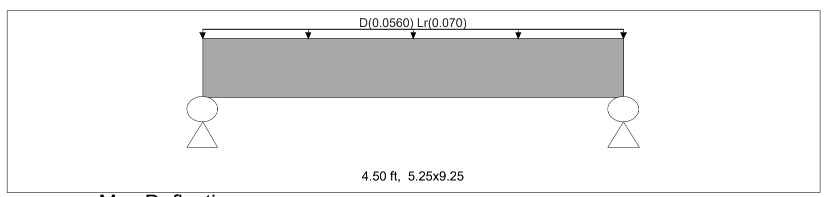
BEAM Size : **5.25x9.25, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

Wood Species :	Douglas Fir - Larch			Wood Grade : No.1					
Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.20 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.0560, Lr = 0.070 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.030** : 1
fb : Actual : 51.12 psi at 2.250 ft in Span # 1
Fb : Allowable : 1,687.50 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.027** : 1
fv : Actual : 5.78 psi at 0.000 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.13		0.16				
Right Support	0.13		0.16				

Max Deflections

Downward L+Lr+S	0.001 in	Downward Total	0.002 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	46076 >240	Total Defl Ratio	25597 >180

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

ROOF LEVEL

RB-10 - GRID LINE 1

	LOAD TYPE	RL2		TOTAL
W1	TRIB. LENGTH	10.25	D =	164 PLF
	D	164	Lr =	205 PLF
	Lr (L)	205	L =	
P1	TRIB. AREA		D =	
	D		Lr =	
	Lr (L)		L =	

RB-11 - GRID LINE 3

	LOAD TYPE	RL2		TOTAL
W1	TRIB. LENGTH	10.25	D =	164 PLF
	D	164	Lr =	205 PLF
	Lr (L)	205	L =	
P1	TRIB. AREA		D =	
	D		Lr =	
	Lr (L)		L =	

Multiple Simple Beam

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Description :

Wood Beam Design : RB-10

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **5.25x9.25, Parallam PSL, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

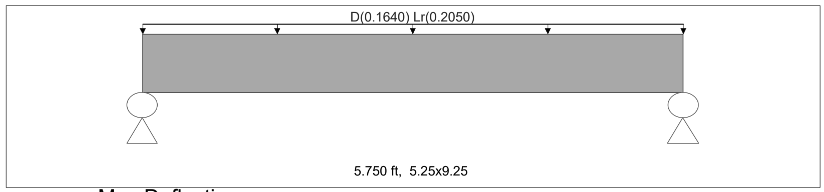
Wood Species : Trus Joist — Wood Grade : Parallam PSL 1.8E

Fb - Tension	2400 psi	Fc - Prll	2500 psi	Fv	190 psi	Ebend- xx	1800 ksi	Density	45.05 pcf
Fb - Compr	2400 psi	Fc - Perp	545 psi	Ft	1755 psi	Eminbend - xx	914.88 ksi		

Applied Loads

Unif Load: D = 0.1640, Lr = 0.2050 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.081** : 1
fb : Actual : 244.43 psi at 2.875 ft in Span # 1
Fb : Allowable : 3,000.00 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.101** : 1
fv : Actual : 24.03 psi at 0.000 ft in Span # 1
Fv : Allowable : 237.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.47		0.59				
Right Support	0.47		0.59				

Max Deflections

Downward L+Lr+S	0.008 in	Downward Total	0.015 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	8484 >240	Total Defl Ratio	4713 >180

Wood Beam Design : RB-11

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **5.25x14.0, Parallam PSL, Fully Unbraced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

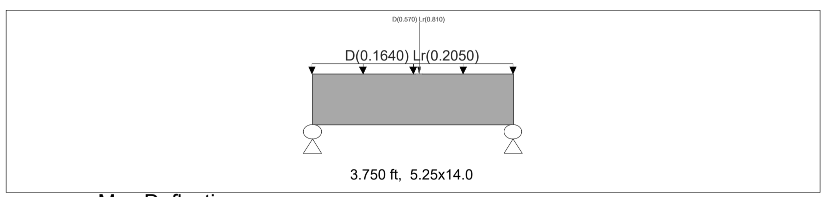
Wood Species : Douglas Fir - Larch — Wood Grade : No.1

Fb - Tension	1,350.0 psi	Fc - Prll	925.0 psi	Fv	170.0 psi	Ebend- xx	1,600.0 ksi	Density	31.20 pcf
Fb - Compr	1,350.0 psi	Fc - Perp	625.0 psi	Ft	675.0 psi	Eminbend - xx	580.0 ksi		

Applied Loads

Unif Load: D = 0.1640, Lr = 0.2050 k/ft, Trib= 1.0 ft
Point: D = 0.570, Lr = 0.810 k @ 2.0 ft
Point: D = 0.570, Lr = 0.810 k @ 2.0 ft

Design Summary



Max fb/Fb Ratio = **0.136** : 1
fb : Actual : 225.43 psi at 2.000 ft in Span # 1
Fb : Allowable : 1,658.87 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.167** : 1
fv : Actual : 35.41 psi at 2.588 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.84		1.14				
Right Support	0.92		1.25				

Max Deflections

Downward L+Lr+S	0.002 in	Downward Total	0.004 in
Upward L+Lr+S	0.000 in	Upward Total	0.000 in
Live Load Defl Ratio	21668 >240	Total Defl Ratio	12555 >180

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

ROOF LEVEL

RH-1 - GRID LINE 1

	LOAD TYPE	RL2	EW1		TOTAL
W1	TRIB. LENGTH	10.25	2.25	D =	190 PLF
	D	164	26	Lr =	205 PLF
	Lr (L)	205		L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

RH-2 - GRID LINE F

	LOAD TYPE	EW1		TOTAL
W1	TRIB. LENGTH	3.25	D =	37 PLF
	D	37	Lr =	
	Lr (L)		L =	
P1	TRIB. AREA		D =	
	D		Lr =	
	Lr (L)		L =	

RH-3 - GRID LINE 5

	LOAD TYPE	RL1	EW1		TOTAL
W1	TRIB. LENGTH	9.5	8	D =	225 PLF
	D	133	92	Lr =	190 PLF
	Lr (L)	190		L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

Multiple Simple Beam

File = Z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
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Description :

Wood Beam Design : RH-1

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-1(

BEAM Size : **6x6, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

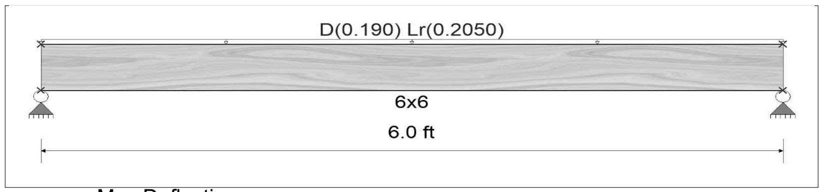
Wood Species : Douglas Fir - Larch   Wood Grade : No.2

Fb - Tension	875.0 psi	Fc - Prll	600.0 psi	Fv	170.0 psi	Ebend- xx	1,300.0 ksi	Density	31.20 pcf
Fb - Compr	875.0 psi	Fc - Perp	625.0 psi	Ft	425.0 psi	Eminbend - xx	470.0 ksi		

Applied Loads

Unif Load: D = 0.190, Lr = 0.2050 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.703** : 1
fb : Actual : 769.23 psi at 3.000 ft in Span # 1
Fb : Allowable : 1,093.75 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.236** : 1
fv : Actual : 50.14 psi at 5.560 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.57		0.62				
Right Support	0.57		0.62				

Max Deflections

Transient Downward	0.061 in		Total Downward	0.117	in
Ratio	1187	>240	Ratio	616	>180
	LC: Lr Only			LC: +D+Lr+H	
Transient Upward	0.000 in		Total Upward	0.000	in
Ratio	9999		Ratio	9999	
	LC:			LC:	

Wood Beam Design : RH-2

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-1(

BEAM Size : **6x6, Sawn, Fully Unbraced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

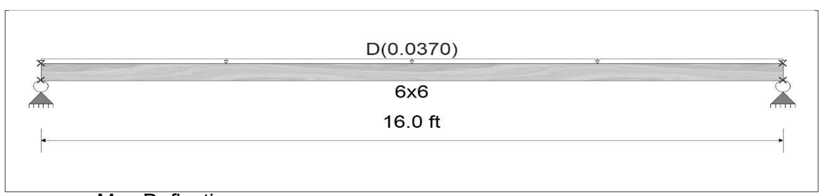
Wood Species : Douglas Fir - Larch   Wood Grade : No.2

Fb - Tension	875.0 psi	Fc - Prll	600.0 psi	Fv	170.0 psi	Ebend- xx	1,300.0 ksi	Density	31.20 pcf
Fb - Compr	875.0 psi	Fc - Perp	625.0 psi	Ft	425.0 psi	Eminbend - xx	470.0 ksi		

Applied Loads

Unif Load: D = 0.0370 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.651** : 1
fb : Actual : 512.38 psi at 8.000 ft in Span # 1
Fb : Allowable : 787.50 psi
Load Comb : +D+H

Max fv/FvRatio = **0.091** : 1
fv : Actual : 13.89 psi at 0.000 ft in Span # 1
Fv : Allowable : 153.00 psi
Load Comb : +D+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.30						
Right Support	0.30						

Max Deflections

Transient Downward	0.000 in		Total Downward	0.553	in
Ratio	9999	>240	Ratio	347	>180
	LC:			LC: +D+H	
Transient Upward	0.000 in		Total Upward	0.000	in
Ratio	9999		Ratio	9999	
	LC:			LC:	

Multiple Simple Beam

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Wood Beam Design : RH-3

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-1(

BEAM Size : **6x10, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

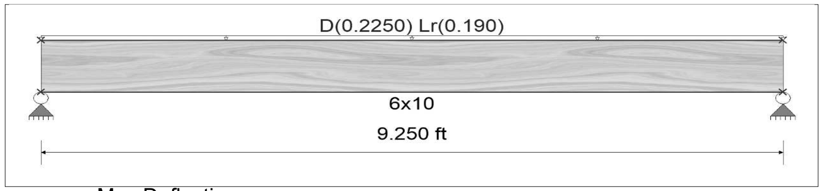
Wood Species : Douglas Fir - Larch Wood Grade : No.2

Fb - Tension	875.0 psi	Fc - Prll	600.0 psi	Fv	170.0 psi	Ebend- xx	1,300.0 ksi	Density	31.20 pcf
Fb - Compr	875.0 psi	Fc - Perp	625.0 psi	Ft	425.0 psi	Eminbend - xx	470.0 ksi		

Applied Loads

Unif Load: D = 0.2250, Lr = 0.190 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.589** : 1
fb : Actual : 643.82 psi at 4.625 ft in Span # 1
Fb : Allowable : 1,093.75 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.216** : 1
fv : Actual : 45.92 psi at 8.479 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	1.04		0.88				
Right Support	1.04		0.88				

Max Deflections

Transient Downward	0.062 in		Total Downward	0.135 in	
Ratio	1802	>240	Ratio	825	>180
	LC: Lr Only			LC: +D+Lr+H	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

ROOF LEVEL

RH-4 - GRID LINE 3

	LOAD TYPE	RL2	IW1		TOTAL
W1	TRIB. LENGTH	10.25	1.5	D =	178 PLF
	D	164	14	Lr =	205 PLF
	Lr (L)	205		L =	
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

Multiple Simple Beam

File = Z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
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Description :

Wood Beam Design : RH-4

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10

BEAM Size : **6x6, Sawn, Fully Braced**
Using Allowable Stress Design with IBC 2015 Load Combinations, Major Axis Bending

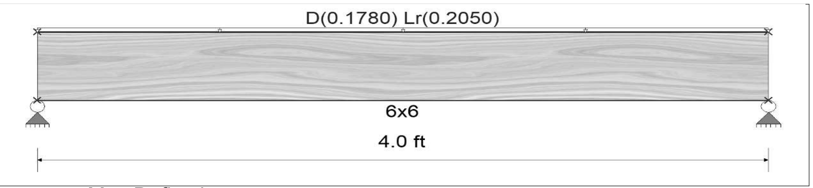
Wood Species : Douglas Fir - Larch   Wood Grade : No.2

Fb - Tension	875.0 psi	Fc - Prll	600.0 psi	Fv	170.0 psi	Ebend- xx	1,300.0 ksi	Density	31.20 pcf
Fb - Compr	875.0 psi	Fc - Perp	625.0 psi	Ft	425.0 psi	Eminbend - xx	470.0 ksi		

Applied Loads

Unif Load: D = 0.1780, Lr = 0.2050 k/ft, Trib= 1.0 ft

Design Summary



Max fb/Fb Ratio = **0.303** : 1
fb : Actual : 331.49 psi at 2.000 ft in Span # 1
Fb : Allowable : 1,093.75 psi
Load Comb : +D+Lr+H

Max fv/FvRatio = **0.138** : 1
fv : Actual : 29.37 psi at 0.000 ft in Span # 1
Fv : Allowable : 212.50 psi
Load Comb : +D+Lr+H

Max Reactions (k)	D	L	Lr	S	W	E	H
Left Support	0.36		0.41				
Right Support	0.36		0.41				

Max Deflections

Transient Downward	0.012 in		Total Downward	0.022 in	
Ratio	4008	>240	Ratio	2145	>180
	LC: Lr Only			LC: +D+Lr+H	
Transient Upward	0.000 in		Total Upward	0.000 in	
Ratio	9999		Ratio	9999	
	LC:			LC:	

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

1ST FLOOR LEVEL

FB-1 - GRID LINE 1.9

	LOAD TYPE	FL3	FL2	IW1		TOTAL
W1	TRIB. LENGTH	3.25	2.25	11.5	D =	206 PLF
	D	68	34	104	Lr =	
	Lr (L)	130	90		L =	-220 PLF
P1	TRIB. AREA				D =	
	D				Lr =	
	Lr (L)				L =	

FB-2 - GRID LINE 2.4

	LOAD TYPE	FL2	IW1		TOTAL
W1	TRIB. LENGTH	6.5	11.5	D =	201 PLF
	D	98	104	Lr =	
	Lr (L)	260		L =	-260 PLF
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

FB-3 - GRID LINE 2.9

	LOAD TYPE	FL2	IW1		TOTAL
W1	TRIB. LENGTH	8.75	11.5	D =	235 PLF
	D	131	104	Lr =	
	Lr (L)	350		L =	-350 PLF
P1	TRIB. AREA			D =	
	D			Lr =	
	Lr (L)			L =	

Wood Beam

File = z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
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Description : FB-1

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :IBC 2015

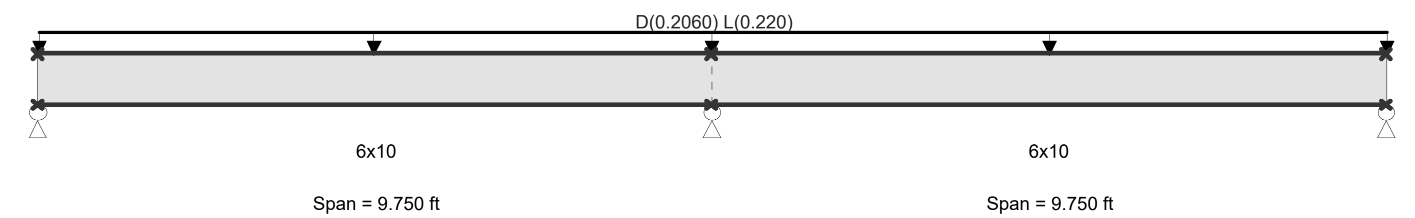
Wood Species : Douglas Fir - Larch
Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	1,350.0 psi	
Fb - Compr	1,350.0 psi	
Fc - Prll	925.0 psi	
Fc - Perp	625.0 psi	
Fv	170.0 psi	
Ft	675.0 psi	

E : Modulus of Elasticity
Ebend- xx 1,600.0 ksi
Eminbend - xx 580.0 ksi

Density 31.20 pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Loads on all spans...
Uniform Load on ALL spans : D = 0.2060, L = 0.220 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.680**: 1	Maximum Shear Stress Ratio	=	**0.479** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb : Actual	=	734.26 psi	fv : Actual	=	65.20 psi
FB : Allowable	=	1,080.00 psi	Fv : Allowable	=	136.00 psi
Load Combination		+D+L+H, LL Comb Run (LL)	Load Combination		+D+L+H, LL Comb Run (LL)
Location of maximum on span	=	9.750 ft	Location of maximum on span	=	8.987 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.053 in	Ratio =	2201 >=360
Max Upward Transient Deflection	-0.023 in	Ratio =	5000 >=360
Max Downward Total Deflection	0.082 in	Ratio =	1421 >=180
Max Upward Total Deflection	-0.009 in	Ratio =	13465 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 9.750 ft	1	0.365	0.258	0.90	1.000	0.80	1.00	1.00	1.00	1.00	2.45	355.07	972.00	1.10	31.53	122.40
Length = 9.750 ft	2	0.365	0.258	0.90	1.000	0.80	1.00	1.00	1.00	1.00	2.45	355.07	972.00	1.10	31.53	122.40
+D+L+H, LL Comb Run (*L)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.750 ft	1	0.504	0.451	1.00	1.000	0.80	1.00	1.00	1.00	1.00	3.75	544.66	1080.00	2.14	61.35	136.00
Length = 9.750 ft	2	0.504	0.451	1.00	1.000	0.80	1.00	1.00	1.00	1.00	3.75	544.66	1080.00	2.14	61.35	136.00
+D+L+H, LL Comb Run (L*)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.750 ft	1	0.504	0.451	1.00	1.000	0.80	1.00	1.00	1.00	1.00	3.75	544.66	1080.00	2.14	61.35	136.00
Length = 9.750 ft	2	0.504	0.451	1.00	1.000	0.80	1.00	1.00	1.00	1.00	3.75	544.66	1080.00	1.23	61.35	136.00
+D+L+H, LL Comb Run (LL)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.750 ft	1	0.680	0.479	1.00	1.000	0.80	1.00	1.00	1.00	1.00	5.06	734.26	1080.00	2.27	65.20	136.00

Wood Beam

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Description : FB-1

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 9.750 ft	2	0.288	0.248	1.60	1.000	0.80	1.00	1.00	1.00	1.00	3.43	497.26	1728.00	1.20	53.89	217.60
+D+0.750L+0.750S+0.5250E+H, LL					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.750 ft	1	0.370	0.261	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.41	639.46	1728.00	1.98	56.78	217.60
Length = 9.750 ft	2	0.370	0.261	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.41	639.46	1728.00	1.98	56.78	217.60
+0.60D+0.60W+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.750 ft	1	0.123	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.47	213.04	1728.00	0.66	18.92	217.60
Length = 9.750 ft	2	0.123	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.47	213.04	1728.00	0.66	18.92	217.60
+0.60D+0.70E+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.750 ft	1	0.123	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.47	213.04	1728.00	0.66	18.92	217.60
Length = 9.750 ft	2	0.123	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.47	213.04	1728.00	0.66	18.92	217.60

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H, LL Comb Run (L*)	1	0.0823	4.412		0.0000	0.000
+D+L+H, LL Comb Run (*L)	2	0.0818	5.392		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	1.692	5.192	1.692
Overall MINimum	-0.134	1.341	-0.134
+D+H	0.753	2.511	0.753
+D+L+H, LL Comb Run (*L)	0.619	3.851	1.692
+D+L+H, LL Comb Run (L*)	1.692	3.851	0.619
+D+L+H, LL Comb Run (LL)	1.558	5.192	1.558
+D+Lr+H, LL Comb Run (*L)	0.753	2.511	0.753
+D+Lr+H, LL Comb Run (L*)	0.753	2.511	0.753
+D+Lr+H, LL Comb Run (LL)	0.753	2.511	0.753
+D+S+H	0.753	2.511	0.753
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.653	3.516	1.457
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.457	3.516	0.653
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.356	4.522	1.356
+D+0.750L+0.750S+H, LL Comb Run (*L	0.653	3.516	1.457
+D+0.750L+0.750S+H, LL Comb Run (L*	1.457	3.516	0.653
+D+0.750L+0.750S+H, LL Comb Run (LL	1.356	4.522	1.356
+D+0.60W+H	0.753	2.511	0.753
+D+0.70E+H	0.753	2.511	0.753
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.653	3.516	1.457
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.457	3.516	0.653
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.356	4.522	1.356
+D+0.750L+0.750S+0.450W+H, LL Comb	0.653	3.516	1.457
+D+0.750L+0.750S+0.450W+H, LL Comb	1.457	3.516	0.653
+D+0.750L+0.750S+0.450W+H, LL Comb	1.356	4.522	1.356
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.653	3.516	1.457
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.457	3.516	0.653
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.356	4.522	1.356
+0.60D+0.60W+0.60H	0.452	1.506	0.452
+0.60D+0.70E+0.60H	0.452	1.506	0.452
D Only	0.753	2.511	0.753
Lr Only, LL Comb Run (*L)			
Lr Only, LL Comb Run (L*)			
Lr Only, LL Comb Run (LL)			
L Only, LL Comb Run (*L)	-0.134	1.341	0.938
L Only, LL Comb Run (L*)	0.938	1.341	-0.134
L Only, LL Comb Run (LL)	0.804	2.681	0.804
S Only			
W Only			
E Only			
H Only			

Wood Beam

File = z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

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Description : FB-2

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :IBC 2015

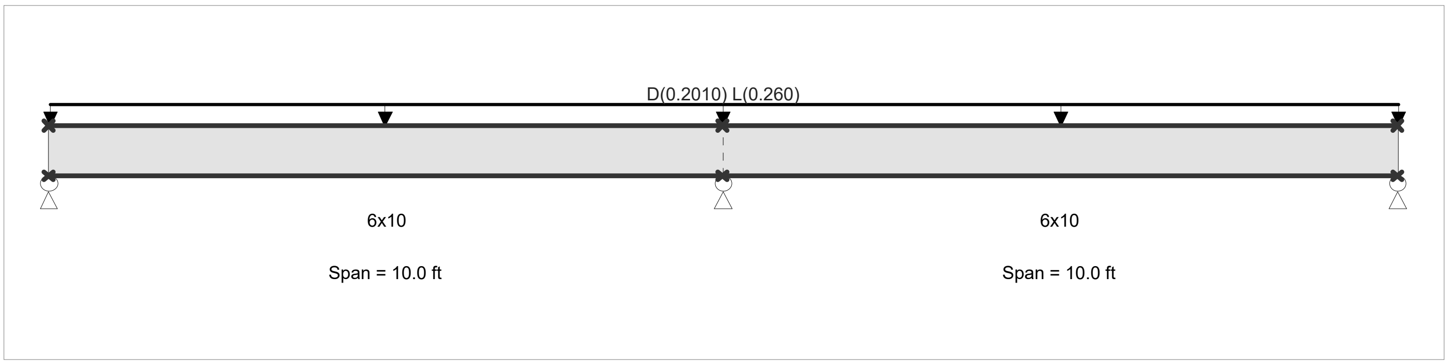
Wood Species : Douglas Fir - Larch
Wood Grade : No.1

Beam Bracing : Beam is Fully Braced against lateral-torsional buckling

Fb - Tension	1,350.0	psi
Fb - Compr	1,350.0	psi
Fc - Prll	925.0	psi
Fc - Perp	625.0	psi
Fv	170.0	psi
Ft	675.0	psi

E : Modulus of Elasticity

Ebend- xx	1,600.0	ksi
Eminbend - xx	580.0	ksi
Density	31.20	pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Loads on all spans...
Uniform Load on ALL spans : D = 0.2010, L = 0.260 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.774**: 1	Maximum Shear Stress Ratio	=	**0.532** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb : Actual	=	835.86 psi	fv : Actual	=	72.36 psi
FB : Allowable	=	1,080.00 psi	Fv : Allowable	=	136.00 psi
Load Combination		+D+L+H, LL Comb Run (LL)	Load Combination		+D+L+H, LL Comb Run (LL)
Location of maximum on span	=	10.000 ft	Location of maximum on span	=	9.218 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.069 in	Ratio =	1726 >=360
Max Upward Transient Deflection	-0.031 in	Ratio =	3921 >=360
Max Downward Total Deflection	0.101 in	Ratio =	1188 >=180
Max Upward Total Deflection	-0.013 in	Ratio =	9234 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.375	0.258	0.90	1.000	0.80	1.00	1.00	1.00	1.00	2.51	364.44	972.00	1.10	31.55	122.40
Length = 10.0 ft	2	0.375	0.258	0.90	1.000	0.80	1.00	1.00	1.00	1.00	2.51	364.44	972.00	1.10	31.55	122.40
+D+L+H, LL Comb Run (*L)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.556	0.498	1.00	1.000	0.80	1.00	1.00	1.00	1.00	4.14	600.15	1080.00	2.36	67.70	136.00
Length = 10.0 ft	2	0.556	0.498	1.00	1.000	0.80	1.00	1.00	1.00	1.00	4.14	600.15	1080.00	2.36	67.70	136.00
+D+L+H, LL Comb Run (L*)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.556	0.498	1.00	1.000	0.80	1.00	1.00	1.00	1.00	4.14	600.15	1080.00	2.36	67.70	136.00
Length = 10.0 ft	2	0.556	0.498	1.00	1.000	0.80	1.00	1.00	1.00	1.00	4.14	600.15	1080.00	1.26	67.70	136.00
+D+L+H, LL Comb Run (LL)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.774	0.532	1.00	1.000	0.80	1.00	1.00	1.00	1.00	5.76	835.86	1080.00	2.52	72.36	136.00

Wood Beam

File = z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

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Description : FB-2

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 10.0 ft	2	0.313	0.270	1.60	1.000	0.80	1.00	1.00	1.00	1.00	3.73	541.22	1728.00	1.22	58.66	217.60
+D+0.750L+0.750S+0.5250E+H, LL					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.416	0.286	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.95	718.01	1728.00	2.17	62.16	217.60
Length = 10.0 ft	2	0.416	0.286	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.95	718.01	1728.00	2.17	62.16	217.60
+0.60D+0.60W+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.127	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.51	218.67	1728.00	0.66	18.93	217.60
Length = 10.0 ft	2	0.127	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.51	218.67	1728.00	0.66	18.93	217.60
+0.60D+0.70E+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.127	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.51	218.67	1728.00	0.66	18.93	217.60
Length = 10.0 ft	2	0.127	0.087	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.51	218.67	1728.00	0.66	18.93	217.60

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H, LL Comb Run (L*)	1	0.1010	4.581		0.0000	0.000
+D+L+H, LL Comb Run (*L)	2	0.1004	5.475	L Only, LL Comb Run (L*)	-0.0009	0.056

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	1.891	5.762	1.891
Overall MINimum	-0.162	1.507	-0.162
+D+H	0.754	2.512	0.754
+D+L+H, LL Comb Run (*L)	0.591	4.137	1.891
+D+L+H, LL Comb Run (L*)	1.891	4.137	0.591
+D+L+H, LL Comb Run (LL)	1.729	5.762	1.729
+D+Lr+H, LL Comb Run (*L)	0.754	2.512	0.754
+D+Lr+H, LL Comb Run (L*)	0.754	2.512	0.754
+D+Lr+H, LL Comb Run (LL)	0.754	2.512	0.754
+D+S+H	0.754	2.512	0.754
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.632	3.731	1.607
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.607	3.731	0.632
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.485	4.950	1.485
+D+0.750L+0.750S+H, LL Comb Run (*L	0.632	3.731	1.607
+D+0.750L+0.750S+H, LL Comb Run (L*	1.607	3.731	0.632
+D+0.750L+0.750S+H, LL Comb Run (LL	1.485	4.950	1.485
+D+0.60W+H	0.754	2.512	0.754
+D+0.70E+H	0.754	2.512	0.754
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.632	3.731	1.607
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.607	3.731	0.632
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.485	4.950	1.485
+D+0.750L+0.750S+0.450W+H, LL Comb	0.632	3.731	1.607
+D+0.750L+0.750S+0.450W+H, LL Comb	1.607	3.731	0.632
+D+0.750L+0.750S+0.450W+H, LL Comb	1.485	4.950	1.485
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.632	3.731	1.607
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.607	3.731	0.632
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.485	4.950	1.485
+0.60D+0.60W+0.60H	0.452	1.507	0.452
+0.60D+0.70E+0.60H	0.452	1.507	0.452
D Only	0.754	2.512	0.754
Lr Only, LL Comb Run (*L)			
Lr Only, LL Comb Run (L*)			
Lr Only, LL Comb Run (LL)			
L Only, LL Comb Run (*L)	-0.162	1.625	1.138
L Only, LL Comb Run (L*)	1.138	1.625	-0.162
L Only, LL Comb Run (LL)	0.975	3.250	0.975
S Only			
W Only			
E Only			
H Only			

Wood Beam

File = z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

Lic. # : KW-06010381 **Licensee : Coastline Engineering, Inc**

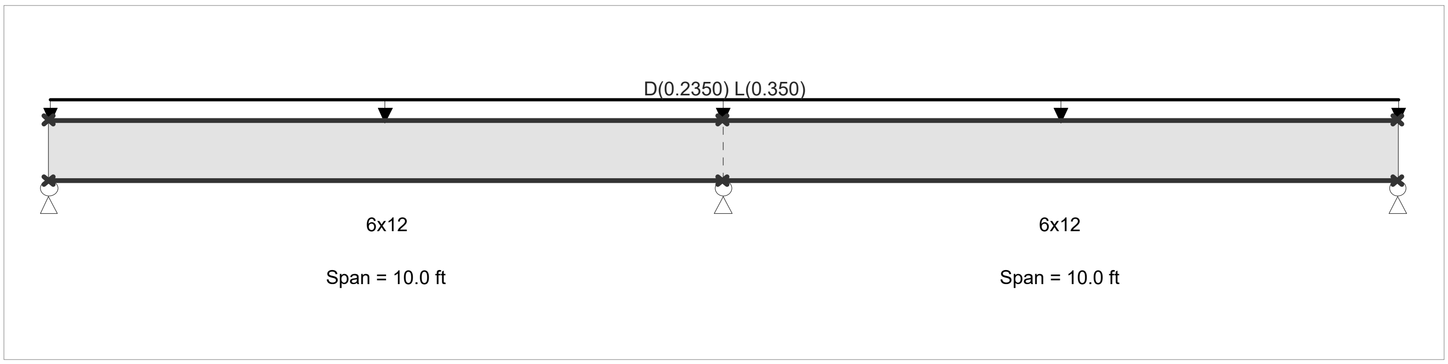
Description : FB-3

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2015	Fb - Compr	1,350.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.1	Fv	170.0 psi		
		Ft	675.0 psi	Density	31.20 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Loads on all spans...
Uniform Load on ALL spans : D = 0.2350, L = 0.350 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.670** : 1	Maximum Shear Stress Ratio	=	**0.541** : 1
Section used for this span		**6x12**	Section used for this span		**6x12**
fb : Actual	=	723.84 psi	fv : Actual	=	73.53 psi
FB : Allowable	=	1,080.00 psi	Fv : Allowable	=	136.00 psi
Load Combination	+D+L+H, LL Comb Run (LL)		Load Combination	+D+L+H, LL Comb Run (LL)	
Location of maximum on span	=	10.000 ft	Location of maximum on span	=	9.050 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.053 in	Ratio =	2275 >=360
Max Upward Transient Deflection	-0.023 in	Ratio =	5168 >=360
Max Downward Total Deflection	0.073 in	Ratio =	1632 >=180
Max Upward Total Deflection	-0.011 in	Ratio =	11090 >=180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.299	0.241	0.90	1.000	0.80	1.00	1.00	1.00	1.00	2.94	290.77	972.00	1.25	29.54	122.40
Length = 10.0 ft	2	0.299	0.241	0.90	1.000	0.80	1.00	1.00	1.00	1.00	2.94	290.77	972.00	1.25	29.54	122.40
+D+L+H, LL Comb Run (*L)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.470	0.503	1.00	1.000	0.80	1.00	1.00	1.00	1.00	5.12	507.30	1080.00	2.88	68.35	136.00
Length = 10.0 ft	2	0.470	0.503	1.00	1.000	0.80	1.00	1.00	1.00	1.00	5.12	507.30	1080.00	2.88	68.35	136.00
+D+L+H, LL Comb Run (L*)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.470	0.503	1.00	1.000	0.80	1.00	1.00	1.00	1.00	5.12	507.30	1080.00	2.88	68.35	136.00
Length = 10.0 ft	2	0.470	0.503	1.00	1.000	0.80	1.00	1.00	1.00	1.00	5.12	507.30	1080.00	1.46	68.35	136.00
+D+L+H, LL Comb Run (LL)					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.670	0.541	1.00	1.000	0.80	1.00	1.00	1.00	1.00	7.31	723.84	1080.00	3.10	73.53	136.00

Wood Beam

File = z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

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Description : FB-3

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 10.0 ft	2	0.262	0.270	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.58	453.17	1728.00	1.41	58.64	217.60
+D+0.750L+0.750S+0.5250E+H, LL					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.356	0.287	1.60	1.000	0.80	1.00	1.00	1.00	1.00	6.22	615.57	1728.00	2.64	62.53	217.60
Length = 10.0 ft	2	0.356	0.287	1.60	1.000	0.80	1.00	1.00	1.00	1.00	6.22	615.57	1728.00	2.64	62.53	217.60
+0.60D+0.60W+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.101	0.081	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.76	174.46	1728.00	0.75	17.72	217.60
Length = 10.0 ft	2	0.101	0.081	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.76	174.46	1728.00	0.75	17.72	217.60
+0.60D+0.70E+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.101	0.081	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.76	174.46	1728.00	0.75	17.72	217.60
Length = 10.0 ft	2	0.101	0.081	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.76	174.46	1728.00	0.75	17.72	217.60

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H, LL Comb Run (L*)	1	0.0735	4.581		0.0000	0.000
+D+L+H, LL Comb Run (*L)	2	0.0731	5.475		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2	Support 3
Overall MAXimum	2.413	7.312	2.413
Overall MINimum	-0.219	1.762	-0.219
+D+H	0.881	2.937	0.881
+D+L+H, LL Comb Run (*L)	0.663	5.125	2.413
+D+L+H, LL Comb Run (L*)	2.413	5.125	0.663
+D+L+H, LL Comb Run (LL)	2.194	7.312	2.194
+D+Lr+H, LL Comb Run (*L)	0.881	2.937	0.881
+D+Lr+H, LL Comb Run (L*)	0.881	2.937	0.881
+D+Lr+H, LL Comb Run (LL)	0.881	2.937	0.881
+D+S+H	0.881	2.937	0.881
+D+0.750Lr+0.750L+H, LL Comb Run (*	0.717	4.578	2.030
+D+0.750Lr+0.750L+H, LL Comb Run (L	2.030	4.578	0.717
+D+0.750Lr+0.750L+H, LL Comb Run (L	1.866	6.219	1.866
+D+0.750L+0.750S+H, LL Comb Run (*L	0.717	4.578	2.030
+D+0.750L+0.750S+H, LL Comb Run (L*	2.030	4.578	0.717
+D+0.750L+0.750S+H, LL Comb Run (LL	1.866	6.219	1.866
+D+0.60W+H	0.881	2.937	0.881
+D+0.70E+H	0.881	2.937	0.881
+D+0.750Lr+0.750L+0.450W+H, LL Comb	0.717	4.578	2.030
+D+0.750Lr+0.750L+0.450W+H, LL Comb	2.030	4.578	0.717
+D+0.750Lr+0.750L+0.450W+H, LL Comb	1.866	6.219	1.866
+D+0.750L+0.750S+0.450W+H, LL Comb	0.717	4.578	2.030
+D+0.750L+0.750S+0.450W+H, LL Comb	2.030	4.578	0.717
+D+0.750L+0.750S+0.450W+H, LL Comb	1.866	6.219	1.866
+D+0.750L+0.750S+0.5250E+H, LL Comb	0.717	4.578	2.030
+D+0.750L+0.750S+0.5250E+H, LL Comb	2.030	4.578	0.717
+D+0.750L+0.750S+0.5250E+H, LL Comb	1.866	6.219	1.866
+0.60D+0.60W+0.60H	0.529	1.762	0.529
+0.60D+0.70E+0.60H	0.529	1.762	0.529
D Only	0.881	2.937	0.881
Lr Only, LL Comb Run (*L)			
Lr Only, LL Comb Run (L*)			
Lr Only, LL Comb Run (LL)			
L Only, LL Comb Run (*L)	-0.219	2.187	1.531
L Only, LL Comb Run (L*)	1.531	2.187	-0.219
L Only, LL Comb Run (LL)	1.313	4.375	1.313
S Only			
W Only			
E Only			
H Only			

BEAM LOADS

Note: This sheet shows beam load calculations only. Loads from other calculated beams and discontinuous shear walls above are directly applied on the beam design sheet(s) that follow. A summary of the beam reactions is at the end of the beam calculations.

1ST FLOOR LEVEL

FB-4 - GRID LINE 1.9

	LOAD TYPE	FL3	FL2	IW1		TOTAL
W1	TRIB. LENGTH	3.25	2.25	11.5	D =	206 PLF
	D	68	34	104	Lr =	
	Lr (L)	130	90		L =	-220 PLF
P1	TRIB. AREA				D =	
	D				Lr =	
	Lr (L)				L =	

Wood Beam

File = z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

Lic. # : KW-06010381 **Licensee : Coastline Engineering, Inc**

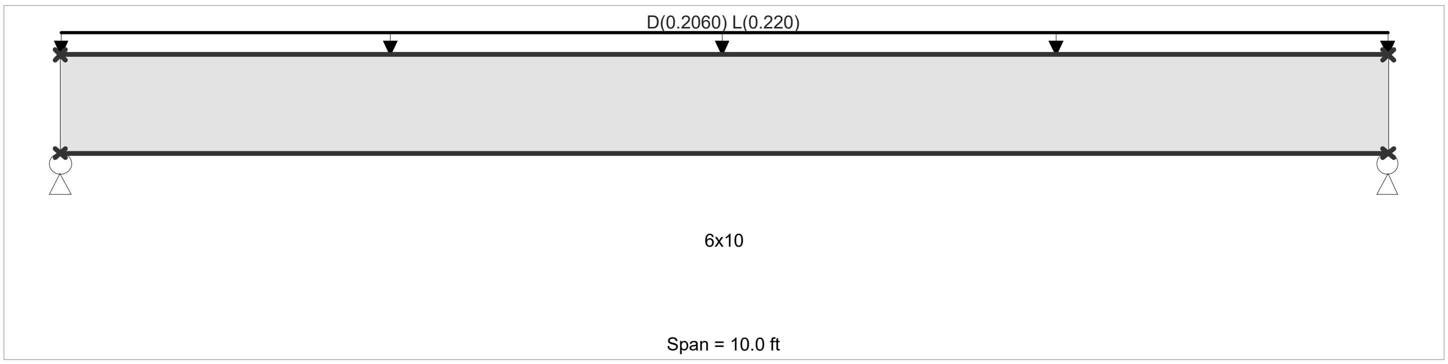
Description : FB-4

CODE REFERENCES

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combination Set : IBC 2015

Material Properties

Analysis Method :	Allowable Stress Design	Fb - Tension	1,350.0 psi	*E : Modulus of Elasticity*	
Load Combination :	IBC 2015	Fb - Compr	1,350.0 psi	Ebend- xx	1,600.0 ksi
		Fc - Prll	925.0 psi	Eminbend - xx	580.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: No.1	Fv	170.0 psi		
		Ft	675.0 psi	Density	31.20 pcf
Beam Bracing	: Beam is Fully Braced against lateral-torsional buckling				



Applied Loads

Service loads entered. Load Factors will be applied for calculations

Loads on all spans...
Uniform Load on ALL spans : D = 0.2060, L = 0.220 k/ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.715** : 1	Maximum Shear Stress Ratio	=	**0.381** : 1
Section used for this span		**6x10**	Section used for this span		**6x10**
fb : Actual	=	772.40 psi	fv : Actual	=	51.78 psi
FB : Allowable	=	1,080.00 psi	Fv : Allowable	=	136.00 psi
Load Combination		+D+L+H	Load Combination		+D+L+H
Location of maximum on span	=	5.000 ft	Location of maximum on span	=	0.000 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.083 in	Ratio =	1439 >=360
Max Upward Transient Deflection	0.000 in	Ratio =	0 <360
Max Downward Total Deflection	0.161 in	Ratio =	743 >=180
Max Upward Total Deflection	0.000 in	Ratio =	0 <180

Maximum Forces & Stresses for Load Combinations

Load Combination		Max Stress Ratios									Moment Values			Shear Values		
Segment Length	Span #	M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	M	fb	F'b	V	fv	F'v
+D+H													0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.384	0.205	0.90	1.000	0.80	1.00	1.00	1.00	1.00	2.58	373.51	972.00	0.87	25.04	122.40
+D+L+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.715	0.381	1.00	1.000	0.80	1.00	1.00	1.00	1.00	5.33	772.40	1080.00	1.80	51.78	136.00
+D+Lr+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.277	0.147	1.25	1.000	0.80	1.00	1.00	1.00	1.00	2.58	373.51	1350.00	0.87	25.04	170.00
+D+S+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.301	0.160	1.15	1.000	0.80	1.00	1.00	1.00	1.00	2.58	373.51	1242.00	0.87	25.04	156.40
+D+0.750Lr+0.750L+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.498	0.265	1.25	1.000	0.80	1.00	1.00	1.00	1.00	4.64	672.68	1350.00	1.57	45.09	170.00
+D+0.750L+0.750S+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

File = z:_VU9HD~H_Z5NTB~A\2017\1P99XK~C\17-046.ec6
ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

Lic. # : KW-06010381 **Licensee : Coastline Engineering, Inc**

Description : FB-4

Load Combination / Segment Length	Span #	Max Stress Ratios M	V	C_d	$C_{F/V}$	C_i	C_r	C_m	C_t	C_L	Moment Values M	fb	F'b	Shear Values V	fv	F'v
Length = 10.0 ft	1	0.542	0.288	1.15	1.000	0.80	1.00	1.00	1.00	1.00	4.64	672.68	1242.00	1.57	45.09	156.40
+D+0.60W+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.216	0.115	1.60	1.000	0.80	1.00	1.00	1.00	1.00	2.58	373.51	1728.00	0.87	25.04	217.60
+D+0.70E+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.216	0.115	1.60	1.000	0.80	1.00	1.00	1.00	1.00	2.58	373.51	1728.00	0.87	25.04	217.60
+D+0.750Lr+0.750L+0.450W+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.389	0.207	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.64	672.68	1728.00	1.57	45.09	217.60
+D+0.750L+0.750S+0.450W+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.389	0.207	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.64	672.68	1728.00	1.57	45.09	217.60
+D+0.750L+0.750S+0.5250E+H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.389	0.207	1.60	1.000	0.80	1.00	1.00	1.00	1.00	4.64	672.68	1728.00	1.57	45.09	217.60
+0.60D+0.60W+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.130	0.069	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.55	224.10	1728.00	0.52	15.02	217.60
+0.60D+0.70E+0.60H					1.000	0.80	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 10.0 ft	1	0.130	0.069	1.60	1.000	0.80	1.00	1.00	1.00	1.00	1.55	224.10	1728.00	0.52	15.02	217.60

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L+H	1	0.1614	5.036		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	2.130	2.130
Overall MINimum	0.618	0.618
+D+H	1.030	1.030
+D+L+H	2.130	2.130
+D+Lr+H	1.030	1.030
+D+S+H	1.030	1.030
+D+0.750Lr+0.750L+H	1.855	1.855
+D+0.750L+0.750S+H	1.855	1.855
+D+0.60W+H	1.030	1.030
+D+0.70E+H	1.030	1.030
+D+0.750Lr+0.750L+0.450W+H	1.855	1.855
+D+0.750L+0.750S+0.450W+H	1.855	1.855
+D+0.750L+0.750S+0.5250E+H	1.855	1.855
+0.60D+0.60W+0.60H	0.618	0.618
+0.60D+0.70E+0.60H	0.618	0.618
D Only	1.030	1.030
Lr Only		
L Only	1.100	1.100
S Only		
W Only		
E Only		
H Only		

BEAM REACTIONS

							FACTORED - ASD LOAD CASES				FACTORED - LRFD LOAD CASES			
							EXCLUDING W,E		INCLUDING W,E		EXCLUDING W,E		INCLUDING W,E	
		D	Lr	L	W	E	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.	MAX.	MIN.
RB-1	R1													
	R2	420	600				1,020	252			1,464	378		
RB-2	R1	570	810				1,380	342			1,980	513		
	R2	570	810				1,380	342			1,980	513		
RB-3	R1	990	540				1,530	594			2,052	891		
	R2	970	520				1,490	582			1,996	873		
RB-4	R1	1,460	1,860				3,320	876			4,728	1,314		
	R2	1,400	1,810				3,210	840			4,576	1,260		
RB-5	R1	1,940	900	3,150			5,090	1,164			7,818	1,746		
	R2	2,000	960	3,150			5,150	1,200			7,920	1,800		
RB-6	R1	550	60	20			610	330			770	495		
	R2	2,500	3,060	1,000			5,560	1,500			8,396	2,250		
RB-8	R1	1,420	2,670				4,090	852			5,976	1,278		
	R2	830	1,110				1,940	498			2,772	747		
RB-9	R1	130	160				290	78			412	117		
	R2	130	160				290	78			412	117		
RB-10	R1	470	590				1,060	282			1,508	423		
	R2	470	590				1,060	282			1,508	423		
RB-11	R1	840	1,140				1,980	504			2,832	756		
	R2	920	1,250				2,170	552			3,104	828		
RH-1	R1	570	620				1,190	342			1,676	513		
	R2	570	620				1,190	342			1,676	513		
RH-2	R1	300					300	180			420	270		
	R2	300					300	180			420	270		
RH-3	R1	1,040	880				1,920	624			2,656	936		
	R2	1,040	880				1,920	624			2,656	936		
RH-4	R1	330	380				710	198			1,004	297		
	R2	330	380				710	198			1,004	297		
-														
FB-1	R1	750		940			1,690	450			2,404	675		
	R2	2,510		2,680			5,190	1,506			7,300	2,259		
	R3	750		940			1,690	450			2,404	675		
FB-2	R1	780		1,230			2,010	468			2,904	702		
	R2	2,610		3,500			6,110	1,566			8,732	2,349		
	R3	780		1,230			2,010	468			2,904	702		
FB-3	R1	880		1,530			2,410	528			3,504	792		
	R2	2,940		4,370			7,310	1,764			10,520	2,646		
	R3	880		1,530			2,410	528			3,504	792		
FB-4	R1	1,030		1,100			2,130	618			2,996	927		
	R2	1,030		1,100			2,130	618			2,996	927		

TOTAL DEAD LOADS

ROOF LEVEL

LOAD TYPE	UNIT D.L.	AREA	HEIGHT 1	LENGTH 1	HEIGHT 2	LENGTH 2	TOTAL
RL1	14	1,025	-	-	-	-	14,350
RL2	16	1,340	-	-	-	-	21,440
FL1	13.5	345	-	-	-	-	4,658
FL2	15		-	-	-	-	
FL3	21		-	-	-	-	
EW1	11.5	-	11.5		10		
IW1	9	-	11.5		10		
GP1	21.5	-	3.5				
	Σ =	2,710 SQFT				Σ =	40,448 LBS

SEISMIC DESIGN FORCES (EQUIVALENT LATERAL FORCE PROCEDURE)

SEISMIC DESIGN CRITERIA

RISK CATEGORY	II
IMPORTANCE FACTOR (I_e)	1.00
SITE CLASS	D

MAIN SEISMIC FORCE-RESISTING SYSTEM

SYSTEM	WOOD SHEAR WALLS
R	6.5
Ω_o *	3
C_d	4
C_t	0.02
x	0.75
Δ_a	0.025h

* REDUCTION FOR FLEXIBLE DIAPHRAGMS, WHERE APPLICABLE, IS APPLIED ON LATERAL BRACING ANALYSIS SHEETS

SITE SPECIFIC PARAMETERS

S_s	1.269
S_1	0.481
S_{D1}	0.487
S_{DS}	0.846
SEISMIC DESIGN CATEGORY	D

PERIOD

h_n	14.00'
$T_a = C_t(h_n)^x$	0.145 SEC
k	1

SEISMIC BASE SHEAR

$C_s = S_{DS}I_e/R$	0.130	← GOVERNS
$C_{s,\ MAX.} = S_{D1}I_e/(TR)$	0.518	
$C_{s,\ MIN.} = 0.044S_{DS}I_e \geq 0.01$	0.037	
$C_{s,\ MIN.} = 0.5S_1I_e/R$ (IF $S_1 \geq 0.6$)	N/A	
$V = C_sW$	5,264 LBS	

VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx}V$

$C_{vx} = W_xh_x^{\kappa}/(\Sigma W_ih_i^{\kappa})$

					DISTRIBUTED OVER DIAPH.	
LEVEL	W_x	h_x	F_x (LRFD)	F_x (ASD)	F_x (LRFD)	F_x (ASD)
ROOF	40,448	14.00	5,264	3,685	1.94 PSF	1.36 PSF

VERTICAL DISTRUBUTION OF SEISMIC FORCES FOR DIAPHRAGM, CHORDS, & COLLECTORS

$F_{px} = (\Sigma F_i/\Sigma W_i)W_{px}$

$F_{px,\ MIN.} = 0.2S_{DS}I_eW_{px}$

$F_{px,\ MAX.} = 0.4S_{DS}I_eW_{px}$

			DISTRUBUTED OVER DIAPH.		COLLECTORS & THEIR CONN. (25% INCREASE) **	
LEVEL	F_{px} (LRFD)	F_{px} (ASD)	F_{px} (LRFD)	F_{px} (ASD)	F_{px} (LRFD)	F_{px} (ASD)
ROOF	6,844	4,791	2.53 PSF	1.77 PSF	3.16 PSF	2.21 PSF

** AT BUILDING PORTIONS NOT BRACED BY LIGHT FRAMED SHEAR WALLS, OVERSTRENGTH LOAD SHALL BE USED INSTEAD OF 25% INCREASE

ALLOWABLE DRIFT

$\delta_{xe,\ ALLOW.} = \Delta_aI_e/C_d$

LEVEL	$\delta_{xe,\ ALLOW.}$
ROOF	0.86''

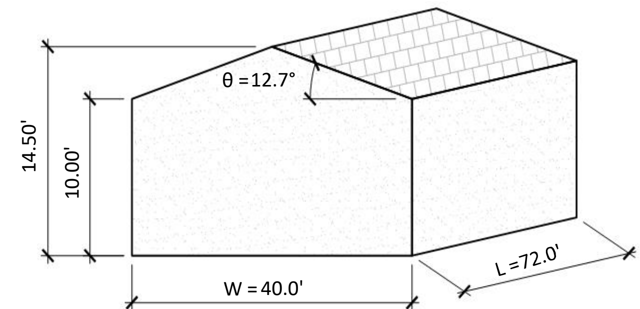
WIND DESIGN PRESSURES (ENVELOPE PROCEDURE, PART 1)

WIND PRESSURE 1

N-S GRID LINES	A	G
E-W GRID LINES	1	5
ENCLOSURE CLASSIFICATION	ENCLOSED	
ROOF TYPE	GABLE / HIP	
NUMBER OF STORIES	1	
DIRECTION OF RIDGE	EAST - WEST	
RISK CATEGORY	II	
BASIC WIND SPEED (V)	110	
EXPOSURE CATEGORY	C	
K_d	0.85	
K_{zt}	1.00	
K_h	0.85	
$q_h = 0.00256K_hK_{zt}K_dV^2$	22.35 PSF	
$p = q_h[(GC_{pf})-(GC_{pi})]$	SEE CHART BELOW	
GC_{pi}	+/- 0.18	

DIMENSIONS

LENGTH (L)	72.0'
WIDTH (W)	40.0'
ROOF HEIGHT	4.50'
1ST STORY HT.	10.00'



PRESSURE NORMAL TO SURFACE

SURFACE # *	N-S (LOAD CASE A) GC_{pf}	N-S (LOAD CASE A) p	E-W (LOAD CASE B) GC_{pf}	E-W (LOAD CASE B) p
1	0.47	6.40	-0.45	-14.08
2	-0.69	-19.45	-0.69	-19.45
3	-0.43	-13.55	-0.37	-12.29
4	-0.36	-12.11	-0.45	-14.08
5	-	-	0.40	4.92
6	-	-	-0.29	-10.50
1E	0.71	11.79	-0.48	-14.75
2E	-1.07	-27.94	-1.07	-27.94
3E	-0.61	-17.70	-0.53	-15.87
4E	-0.54	-16.04	-0.48	-14.75
5E	-	-	0.61	9.61
6E	-	-	-0.43	-13.63

* SEE ASCE 7-10 FIGURE 28.4-1 FOR SURFACE DESCRIPTIONS

TOTAL HORIZONTAL PRESSURE ON SURFACE

SURFACE # *	SURFACE AREA	N-S (LOAD CASE A) $P_{NORMAL\ TO\ SURFACE}$	N-S (LOAD CASE A) $P_{HORIZ.\ COMPONENT}$	E-W (LOAD CASE B) $P_{NORMAL\ TO\ SURFACE}$	E-W (LOAD CASE B) $P_{HORIZ.\ COMPONENT}$
1	640	4,099	4,099	-9,012	0
2	1,312	-25,323	-5,559	-25,512	0
3	1,312	-17,780	3,903	-16,128	0
4	640	-7,749	7,749	-9,012	0
5	448	0	0	2,204	2,204
6	448	0	0	-4,708	4,708
1E	80	943	943	-1,180	0
2E	164	-4,541	-997	-4,582	0
3E	164	-2,903	637	-2,603	0
4E	80	-1,283	1,283	-1,180	0
5E	42	0	0	402	402
6E	42	0	0	-570	570
		Σ =	12,058 LBS	Σ =	7,884 LBS
			MIN. = 14,112 LBS		MIN. = 7,840 LBS

DESIGN WIND PRESSURE

	N-S	E-W
$AREA_{VERT.\ PROJ.}$	1,044 SQFT	490 SQFT
p	13.52 PSF	16.09 PSF

ROOF OVERHANG UPLIFT PRESSURE

GC_p	0.7
GC_{pf}	1.07
p	39.56 PSF

VERTICAL DISTRUBUTION OF WIND PRESSURE

LEVEL	LRFD N-S	LRFD E-W	ASD N-S	ASD E-W
1ST STORY	135 PLF	125 PLF	81 PLF	75 PLF

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	NORTH-SOUTH
GRID LINE	A

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	580	0.5	290	394
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	394 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE		
LOAD FROM RIGHT	WORST CASE PRESSURE	20'	811
		LEVEL ABOVE	-
		Σ =	811 LBS

SHEAR WALL

SEGMENT LENGTH	8'	6'
SEGMENT HEIGHT	11.5'	11.5'
ASPECT RATIO	1.4 : 1	1.9 : 1
SHEAR CAPACITY FACTOR	1.00	1.00
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	0%	0%
SEISMIC LOAD TO SEGMENT	28 PLF	28 PLF
WIND LOAD TO SEGMENT	58 PLF	58 PLF
SHEAR WALL TYPE [REQUIRED : USED]	1 : 1	

HOLDOWN

SEGMENT LENGTH	8'		6'	
SEGMENT HEIGHT	11.5'		11.5'	
SEISMIC LOAD TO SEGMENT	225 LBS		169 LBS	
WIND LOAD TO SEGMENT	463 LBS		348 LBS	
UNIFORM DEAD LOAD	206 PLF		206 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]	990	8'		
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]				
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]				
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]				
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	-	-	28	28
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]				
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	179	-	313	313
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]				
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	-	-	12	12
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]				
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	385	-	587	587
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	1		1	1

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	1,157 LBS	1,157 LBS
SEISMIC LOAD	463 LBS	463 LBS
WIND LOAD	1,110 LBS	1,110 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	NORTH-SOUTH
GRID LINE	C NORTH

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	2,050	0.5	1,025	1,394
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	1,394 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	20'	811
LOAD FROM RIGHT	WORST CASE PRESSURE	30'	1,217
		LEVEL ABOVE	-
		Σ =	2,028 LBS

SHEAR WALL

SEGMENT LENGTH	9'	
SEGMENT HEIGHT	11.5'	
ASPECT RATIO	1.3 : 1	
SHEAR CAPACITY FACTOR	1.00	
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	0%	
SEISMIC LOAD TO SEGMENT	155 PLF	
WIND LOAD TO SEGMENT	225 PLF	
SHEAR WALL TYPE [REQUIRED : USED]	1	2

HOLDOWN

SEGMENT LENGTH	9'	
SEGMENT HEIGHT	11.5'	
SEISMIC LOAD TO SEGMENT	1,394 LBS	
WIND LOAD TO SEGMENT	2,028 LBS	
UNIFORM DEAD LOAD	90 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]		
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	1,647	1,647
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]		
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	2,438	2,438
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	2,335	2,335
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	4,106	4,106
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	3	3

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	6,361 LBS
SEISMIC LOAD	2,544 LBS
WIND LOAD	4,318 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	NORTH-SOUTH
GRID LINE	C SOUTH

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	2,050	0.5	1,025	1,394
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	1,394 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	20'	811
LOAD FROM RIGHT	WORST CASE PRESSURE	30'	1,217
		LEVEL ABOVE	-
		Σ =	2,028 LBS

SHEAR WALL

SEGMENT LENGTH	4'	
SEGMENT HEIGHT	11.5'	
ASPECT RATIO	2.9 : 1	
SHEAR CAPACITY FACTOR	0.70	
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	30%	
SEISMIC LOAD TO SEGMENT	348 PLF	
WIND LOAD TO SEGMENT	507 PLF	
SHEAR WALL TYPE [REQUIRED : USED]	4	4

HOLDOWN

SEGMENT LENGTH	4'	
SEGMENT HEIGHT	11.5'	
SEISMIC LOAD TO SEGMENT	1,394 LBS	
WIND LOAD TO SEGMENT	2,028 LBS	
UNIFORM DEAD LOAD	115 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]		
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	4,251	4,251
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]		
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	6,209	6,209
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	6,062	6,062
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	10,373	10,373
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	4	4

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	14,311 LBS
SEISMIC LOAD	5,725 LBS
WIND LOAD	9,716 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	NORTH-SOUTH
GRID LINE	D

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	1,120	0.5	560	761
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	761 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	30'	1,217
LOAD FROM RIGHT	WORST CASE PRESSURE	22'	892
		LEVEL ABOVE	-
		Σ =	2,109 LBS

SHEAR WALL

SEGMENT LENGTH	7'	
SEGMENT HEIGHT	11.5'	
ASPECT RATIO	1.6 : 1	
SHEAR CAPACITY FACTOR	1.00	
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	0%	
SEISMIC LOAD TO SEGMENT	109 PLF	
WIND LOAD TO SEGMENT	301 PLF	
SHEAR WALL TYPE [REQUIRED : USED]	2	2

HOLDOWN

SEGMENT LENGTH	7'	
SEGMENT HEIGHT	11.5'	
SEISMIC LOAD TO SEGMENT	761 LBS	
WIND LOAD TO SEGMENT	2,109 LBS	
UNIFORM DEAD LOAD	90 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]		
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	1,154	1,154
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]		
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	3,439	3,439
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	1,635	1,635
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	5,765	5,765
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	2	2

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	4,468 LBS
SEISMIC LOAD	1,787 LBS
WIND LOAD	5,774 LBS

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LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	NORTH-SOUTH
GRID LINE	E

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	1,025	0.5	513	697
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	697 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	44'	1,784
LOAD FROM RIGHT	WORST CASE PRESSURE	-	
		LEVEL ABOVE	-
		Σ =	1,784 LBS

SHEAR WALL

SEGMENT LENGTH	7'	
SEGMENT HEIGHT	11.5'	
ASPECT RATIO	1.6 : 1	
SHEAR CAPACITY FACTOR	1.00	
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	0%	
SEISMIC LOAD TO SEGMENT	100 PLF	
WIND LOAD TO SEGMENT	255 PLF	
SHEAR WALL TYPE [REQUIRED : USED]	1	2

HOLDOWN

SEGMENT LENGTH	7'	
SEGMENT HEIGHT	11.5'	
SEISMIC LOAD TO SEGMENT	697 LBS	
WIND LOAD TO SEGMENT	1,784 LBS	
UNIFORM DEAD LOAD	115 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]		
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	999	999
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]		
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	2,824	2,824
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	1,409	1,409
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	4,749	4,749
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	1	1

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	4,089 LBS
SEISMIC LOAD	1,636 LBS
WIND LOAD	4,886 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	NORTH-SOUTH
GRID LINE	F

TYPE OF LATERAL BRACE	PREFABRICATED SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	505	0.5	253	343
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	343 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	22'	892
LOAD FROM RIGHT	WORST CASE PRESSURE	-	
		LEVEL ABOVE	-
		Σ =	892 LBS

SHEAR WALL

SHEAR WALL TYPE	WSW24x12
ALLOWABLE SEISMIC SHEAR LOAD	2,920 LBS
DRIFT AT ALLOWABLE SEISMIC SHEAR LOAD	0.58''
ALLOWABLE WIND SHEAR LOAD	2,735 LBS
DRIFT AT ALLOWABLE WIND SHEAR LOAD	0.56''
SEISMIC LOAD TO WALL	343 LBS
WIND LOAD TO WALL	892 LBS
ALLOWABLE LOAD > LOAD TO WALL	O.K.

HOLDOWN

WALL LENGTH	2'	
WALL HEIGHT	11.5'	
SEISMIC LOAD TO WALL	343 LBS	
WIND LOAD TO WALL	892 LBS	
UNIFORM DEAD LOAD	115 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]		
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]		
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	2,193	2,193
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]		
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	5,784	5,784
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	3,127	3,127
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	9,653	9,653
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	PER PLAN	

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	7,051 LBS
SEISMIC LOAD	2,820 LBS
WIND LOAD	8,550 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	EAST-WEST
GRID LINE	1

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	1,640	0.5	820	1,115
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	1,115 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	27'	1,010
LOAD FROM RIGHT	WORST CASE PRESSURE	-	
		LEVEL ABOVE	-
		Σ =	1,010 LBS

SHEAR WALL

SEGMENT LENGTH	6'	5'	5'	6'
SEGMENT HEIGHT	11.5'	11.5'	11.5'	11.5'
ASPECT RATIO	1.9 : 1	2.3 : 1	2.3 : 1	1.9 : 1
SHEAR CAPACITY FACTOR	1.00	0.87	0.87	1.00
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	0%	13%	13%	0%
SEISMIC LOAD TO SEGMENT	54 PLF	47 PLF	47 PLF	54 PLF
WIND LOAD TO SEGMENT	49 PLF	42 PLF	42 PLF	49 PLF
SHEAR WALL TYPE [REQUIRED : USED]	1 : 1			

HOLDOWN

SEGMENT LENGTH	6'		5'		5'		6'	
SEGMENT HEIGHT	11.5'		11.5'		11.5'		11.5'	
SEISMIC LOAD TO SEGMENT	323 LBS		234 LBS		234 LBS		323 LBS	
WIND LOAD TO SEGMENT	293 LBS		212 LBS		212 LBS		293 LBS	
UNIFORM DEAD LOAD	171 PLF		300 PLF		280 PLF		280 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]	130	0'						
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]								
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]								
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]								
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	328	394	190	190	216	216	228	228
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]								
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	186	268	41	41	73	73	61	61
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]								
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	440	540	237	237	277	277	287	287
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]								
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	378	502	148	148	196	196	190	190
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	1	1	1	1	1	1	1	1

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	2,213 LBS	1,924 LBS	1,924 LBS	2,213 LBS
SEISMIC LOAD	885 LBS	770 LBS	770 LBS	885 LBS
WIND LOAD	935 LBS	813 LBS	813 LBS	935 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	EAST-WEST
GRID LINE	3

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	2,550	0.5	1,275	1,734
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	1,734 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	18.5'	692
LOAD FROM RIGHT	WORST CASE PRESSURE	21'	786
		LEVEL ABOVE	-
		Σ =	1,478 LBS

SHEAR WALL

SEGMENT LENGTH	3.75'		10'	15'
SEGMENT HEIGHT	11.5'		11.5'	11.5'
ASPECT RATIO	3.1 : 1		1.2 : 1	0.8 : 1
SHEAR CAPACITY FACTOR	0.65		1.00	1.00
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	35%		0%	0%
SEISMIC LOAD TO SEGMENT	41 PLF		63 PLF	63 PLF
WIND LOAD TO SEGMENT	35 PLF		54 PLF	54 PLF
SHEAR WALL TYPE [REQUIRED : USED]	1	2		

HOLDOWN

SEGMENT LENGTH	3.75'		10'		15'	
SEGMENT HEIGHT	11.5'		11.5'		11.5'	
SEISMIC LOAD TO SEGMENT	154 LBS		632 LBS		948 LBS	
WIND LOAD TO SEGMENT	132 LBS		538 LBS		808 LBS	
UNIFORM DEAD LOAD	164 PLF		164 PLF		170 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]	1,400	0'	840	0'		
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]	330	4'				
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]						
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]						
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	-	171	-	343	115	115
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]						
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	-	9	-	132	-	-
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]						
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	-	214	-	454	108	108
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]						
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	-	87	-	304	-	-
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]		1	1	7	7	1

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	1,692 LBS	2,595 LBS	2,595 LBS
SEISMIC LOAD	677 LBS	1,038 LBS	1,038 LBS
WIND LOAD	673 LBS	1,032 LBS	1,032 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	EAST-WEST
GRID LINE	4

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	565	0.5	283	384
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	384 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	-	
LOAD FROM RIGHT	WORST CASE PRESSURE	7.5'	281
		LEVEL ABOVE	-
		Σ =	281 LBS

SHEAR WALL

SEGMENT LENGTH	7'	
SEGMENT HEIGHT	11.5'	
ASPECT RATIO	1.6 : 1	
SHEAR CAPACITY FACTOR	1.00	
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	0%	
SEISMIC LOAD TO SEGMENT	55 PLF	
WIND LOAD TO SEGMENT	40 PLF	
SHEAR WALL TYPE [REQUIRED : USED]	1	1

HOLDOWN

SEGMENT LENGTH	7'	
SEGMENT HEIGHT	11.5'	
SEISMIC LOAD TO SEGMENT	384 LBS	
WIND LOAD TO SEGMENT	281 LBS	
UNIFORM DEAD LOAD	325 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]	20	7'
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]	2,500	0'
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]		
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	-	77
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]		
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	-	-
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	-	58
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]		
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	-	-
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]		1

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	2,254 LBS
SEISMIC LOAD	902 LBS
WIND LOAD	768 LBS

LATERAL BRACING ANALYSIS

LEVEL	1ST STORY
DIRECTION	EAST-WEST
GRID LINE	5

TYPE OF LATERAL BRACE	WOOD SHEAR WALL
R	6.5
Ω_O	2.5
C_d	4
DESIGN METHODOLOGY	ASD

CONTRIBUTING FORCES FROM DIAPHRAGM ON LEVEL BELOW

AREA	-
TRIBUTARY AREA	
DIST. TO NEXT BRACE	-
LOAD APPLICATION HT.	

CONTRIBUTING FORCES FROM LEVEL ABOVE

GRID LINE	-
OFFSET MULTIPLIER	-

SEISMIC LOAD

	ρ	ρF_x	AREA	TRIB. AREA MULTIPLIER	TRIB. AREA	LOAD
DIAPHRAGM	1.0	1.36	860	0.5	430	585
CANTILEVERED DIAPH.	1.0	1.36	-			
					LEVEL ABOVE	-
					Σ =	585 LBS

WIND LOAD

	SOURCE	DISTANCE TO NEXT BRACE	LOAD
LOAD FROM LEFT	WORST CASE PRESSURE	-	
LOAD FROM RIGHT	WORST CASE PRESSURE	19'	711
		LEVEL ABOVE	-
		Σ =	711 LBS

SHEAR WALL

SEGMENT LENGTH	7'	7'
SEGMENT HEIGHT	11.5'	11.5'
ASPECT RATIO	1.6 : 1	1.6 : 1
SHEAR CAPACITY FACTOR	1.00	1.00
REDUCTION IN SEISMIC SHEAR PANEL CAPACITY	0%	0%
SEISMIC LOAD TO SEGMENT	42 PLF	42 PLF
WIND LOAD TO SEGMENT	51 PLF	51 PLF
SHEAR WALL TYPE [REQUIRED : USED]	1 : 1	

HOLDOWN

SEGMENT LENGTH	7'		7'	
SEGMENT HEIGHT	11.5'		11.5'	
SEISMIC LOAD TO SEGMENT	292 LBS		292 LBS	
WIND LOAD TO SEGMENT	355 LBS		355 LBS	
UNIFORM DEAD LOAD	255 PLF		255 PLF	
CONCENTRATED DEAD LOAD 1 (LBS) [LOAD : LOCATION]		1.25'	1,350	7'
CONCENTRATED DEAD LOAD 2 (LBS) [LOAD : LOCATION]	1,350	7'		0'
CONCENTRATED DEAD LOAD 3 (LBS) [LOAD : LOCATION]				
ASD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]				
ASD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	53	-	53	-
ASD - WIND UPLIFT FROM ABOVE (PBS) [LEFT : RIGHT]				
ASD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	51	-	51	-
LRFD - SEISMIC UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]				
LRFD - TOTAL SEISMIC UPLIFT (LBS) [LEFT : RIGHT]	36	-	36	-
LRFD - WIND UPLIFT FROM ABOVE (LBS) [LEFT : RIGHT]				
LRFD - TOTAL WIND UPLIFT (LBS) [LEFT : RIGHT]	178	-	178	-
REQUIRED HOLDOWN TYPE [LEFT : RIGHT]	7		7	

VERTICAL LOAD AT ENDS OF SHEAR WALLS (LRFD, W/OUT DEAD LOAD: USED TO DESIGN BEAMS BELOW)

SEISMIC LOAD WITH OVERSTRENGTH FACTOR (Ω_o)	1,715 LBS	1,715 LBS
SEISMIC LOAD	686 LBS	686 LBS
WIND LOAD	973 LBS	973 LBS

REDUNDANCY CHECK

PERCENTAGE OF BASE SHEAR RESISTANCE

1ST STORY	100.0%

NORTH - SOUTH DIRECTION

1ST STORY

OVERALL STORY STRENGTH	24,463 LBS		
QUALIFYING ELEMENT WITH THE GREATEST STRENGTH	9' WOOD SHEAR WALL AT GRID LINE C NORTH		
STRENGTH OF THIS ELEMENT	4,474 LBS		
STORY STRENGTH REDUCTION WITH REMOVAL OF THIS ELEMENT	18.3%	<	33%

ρ	1.0

EAST - WEST DIRECTION

1ST STORY

OVERALL STORY STRENGTH	30,389 LBS		
QUALIFYING ELEMENT WITH THE GREATEST STRENGTH	10' WOOD SHEAR WALL AT GRID LINE 3		
STRENGTH OF THIS ELEMENT	4,971 LBS		
STORY STRENGTH REDUCTION WITH REMOVAL OF THIS ELEMENT	16.4%	<	33%

ρ	1.0

FOUNDATION ANALYSIS

SOIL PROPERTIES

BEARING PRESSURE	1,500 PSF
INCREASE FOR WIDTH	-
INCREASE FOR DEPTH	300 PSF
MAXIMUM BEARING PRESSURE	1,500 PSF

MINIMUM FOOTING DIMENSIONS

	NEW	EXISTING
CONTINUOUS FOOTING WIDTH	12''	12''
PAD FOOTING WIDTH	24''	12''
FOOTING DEPTH (BLW. LOWEST ADJ. GRADE)	12''	12''
FOOTING REINFORCEMENT PER PLAN		

CONTINUOUS FOOTING DESIGN

GRID LINE - 1

LOAD TYPE	FL3	EW1	RL2	TOTAL
TRIB. LENGTH	3	11.5	10	
D	63	132	160	355 PLF
Lr			200	200 PLF
L				

MAXIMUM FACTORED LOAD	555 PLF	(D+Lr)
REQUIRED FOOTING WIDTH	12''	
FOOTING DEPTH USED	12''	

GRID LINE - A

LOAD TYPE	FL1	EW1	RL1	TOTAL
TRIB. LENGTH	7.5	11.5	11.5	
D	101	132	161	395 PLF
Lr			230	230 PLF
L				

MAXIMUM FACTORED LOAD	625 PLF	(D+Lr)
REQUIRED FOOTING WIDTH	12''	
FOOTING DEPTH USED	12''	

GRID LINE - 3

LOAD TYPE	RL1	RL2	IW1	TOTAL
TRIB. LENGTH	11.5	10	11.5	
D	161	160	104	425 PLF
Lr	230	200		430 PLF
L				

MAXIMUM FACTORED LOAD	855 PLF	(D+Lr)
REQUIRED FOOTING WIDTH	12''	
FOOTING DEPTH USED	12''	

PAD FOOTING DESIGN

BEAM ID / LOAD TYPE	FB-1 : R2	TOTAL
TRIB. AREA		
D	2,510	2,510 LBS
Lr		
L	2,680	2,680 LBS
W		
E		

MAXIMUM FACTORED LOAD (EXCLUDING W,E)	5,190 LBS	(D+L)
MAXIMUM FACTORED LOAD (INCLUDING W,E)	-	
REQUIRED FOOTING DIMENSIONS	24'' SQUARE	
FOOTING DEPTH USED	12''	

FOUNDATION ANALYSIS

PAD FOOTING DESIGN

BEAM ID / LOAD TYPE	FB-1 : R3	FB-1 : R1	TOTAL
TRIB. AREA			
D	750	750	1,500 LBS
Lr			
L	940	940	1,880 LBS
W			
E			

MAXIMUM FACTORED LOAD (EXCLUDING W,E)	3,380 LBS	(D+L)
MAXIMUM FACTORED LOAD (INCLUDING W,E)	-	
REQUIRED FOOTING DIMENSIONS	24'' SQUARE	
FOOTING DEPTH USED	12''	

BEAM ID / LOAD TYPE	FB-1 : R3	FB-4 : R1	TOTAL
TRIB. AREA			
D	750	1,030	1,780 LBS
Lr			
L	940	1,100	2,040 LBS
W			
E			

MAXIMUM FACTORED LOAD (EXCLUDING W,E)	3,820 LBS	(D+L)
MAXIMUM FACTORED LOAD (INCLUDING W,E)	-	
REQUIRED FOOTING DIMENSIONS	24'' SQUARE	
FOOTING DEPTH USED	12''	

BEAM ID / LOAD TYPE	FB-2 : R2	TOTAL
TRIB. AREA		
D	2,610	2,610 LBS
Lr		
L	3,500	3,500 LBS
W		
E		

MAXIMUM FACTORED LOAD (EXCLUDING W,E)	6,110 LBS	(D+L)
MAXIMUM FACTORED LOAD (INCLUDING W,E)	-	
REQUIRED FOOTING DIMENSIONS	27'' SQUARE	
FOOTING DEPTH USED	12''	

BEAM ID / LOAD TYPE	FB-3 : R2	TOTAL
TRIB. AREA		
D	2,940	2,940 LBS
Lr		
L	4,370	4,370 LBS
W		
E		

MAXIMUM FACTORED LOAD (EXCLUDING W,E)	7,310 LBS	(D+L)
MAXIMUM FACTORED LOAD (INCLUDING W,E)	-	
REQUIRED FOOTING DIMENSIONS	27'' SQUARE	
FOOTING DEPTH USED	12''	

FOUNDATION ANALYSIS

PAD FOOTING DESIGN

BEAM ID / LOAD TYPE	RB-6 : R2	
TRIB. AREA		TOTAL
D	2,500	2,500 LBS
Lr	3,060	3,060 LBS
L	1,000	1,000 LBS
W		
E		

MAXIMUM FACTORED LOAD (EXCLUDING W,E)	5,560 LBS	(D+Lr)
MAXIMUM FACTORED LOAD (INCLUDING W,E)	-	
REQUIRED FOOTING DIMENSIONS	24'' SQUARE	
FOOTING DEPTH USED	12''	

SLAB-ON-GROUND FOUNDATION DESIGN PER WRI TF 700-R-03

GRADE BEAM SPACING AND CANTILEVER LENGTH

EFFECTIVE PLASTICITY INDEX		E.P.I.	25		
CLIMATE FACTOR		Cw	15		
CANTILEVER LENGTH		lc	4	FT	FIG. 15, FIG, 12
CANTILEVER ADJUSTMENT FACTOR		k	0.65		FIG. 13
ADJUSTED CANTILEVER LENGTH		lc adj.	2.6	FT	
BEAM SPACING		S	20	FT	FIG. 17
PERPENDICULAR PLAN DIMENSION OF SYSTEM		L'	20	FT	
REQ'D. NUMBER OF BEAMS		N	2		

LOAD

TOTAL WEIGHT OF BUILDING AND SLAB		w	200	PSF	
MOMENT		M	13520	LB-FT	
			13.52	K-FT	
FACTORED MOMENT		Mu	18.928	K-FT	
SHEAR		V	10400	LBS	
FACTORED SHEAR		Vu	10.4	K	
DEFLECTION		delta	0.003463	IN	

SYSTEM GEOMETRY AND SECTION PROPERTIES

BEAM LENGTH		L	21	FT	
SYSTEM DEPTH		h	20	IN	TOP OF SLAB TC
MIN. BEAM HT.(h) PER ACI T9.5b			12.6	IN	
BEAM DEPTH		d	16.75	IN	
BEAM WIDTH		b	12	IN	
TOTAL BEAM WIDTH		B	24	IN	
CRACKED MOMENT OF INERTIA (0.5Ig) OF BEAMS IGNORING FLANGES		Icr	8000	IN^4	
COMPRESSIVE STRENGTH OF CONCRETE		f'c	2500	PSI	
CREEP MODULUS OF CONCRETE		Ec	2850000	PSI	
YIELD STRENGTH		fy	60000	PSI	
REINFORCING PER BEAM		2	#4	BARS	
BAR DIAMETER			0.5	IN	
SINGLE BAR AREA			0.2	IN^2	
TOTAL BAR AREA		As	0.8	IN^2	

CAPACITY

NOMINAL MOMENT CAPACITY (SINGLY-REINFORCED)	a	Mn	781412	LB-IN	
	0.94		65	K-FT	
FACTORED MOMENT CAPACITY		phi Mn	59	K-FT	
NOMINAL SHEAR CAPACITY (PLAIN CONCRETE)		Vc	40200	LBS	
			40	K	
FACTORED SHEAR CAPACITY		phi Vc	30	K	
ALLOWABLE DEFLECTION RATIO		L/	10000		
ALLOWABLE DEFLECTION			0.0252	IN	

CHECK

	DEMAND		CAPACITY
MOMENT	19	<	59
SHEAR	10	<	30
DEFLECTION	0.0035	<	0.0252

SLAB REINFORCING

YIELD STRENGTH		60000	PSI	
REINFORCING	#4	@	18	" O.C.
BAR DIAMETER		0.5	IN	
SINGLE BAR AREA		0.2	IN^2	
BARS PER FT.		0.666666667		
STEEL RESISTANCE	Asfy	8000	LBS/FT	
RECOMMENDED STEEL RESISTANCE		5200	LBS/FT	

General Footing

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ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

Lic. # : KW-06010381 **Licensee : Coastline Engineering, Inc**

Description : WSW OVERTURNING CHECK

Code References

Calculations per ACI 318-14, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : IBC 2015

General Information

Material Properties

f'c : Concrete 28 day strength	=	2.50	ksi
fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.650	

Analysis Settings

Min Steel % Bending Reinf.	=		
Min Allow % Temp Reinf.	=	0.00090	
Min. Overturning Safety Factor	=	1.0	: 1
Min. Sliding Safety Factor	=	1.0	: 1
Add Ftg Wt for Soil Pressure	:	Yes	
Use ftg wt for stability, moments & shears	:	Yes	
Add Pedestal Wt for Soil Pressure	:	No	
Use Pedestal wt for stability, mom & shear	:	No	

Soil Design Values

Allowable Soil Bearing	=	2.0	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

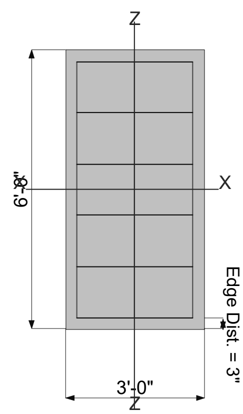
Footing base depth below soil surface	=	1.0	ft
Allow press. increase per foot of depth	=		ksf
when footing base is below	=		ft

Increases based on footing plan dimension

Allowable pressure increase per foot of depth	=		ksf
when max. length or width is greater than	=		ft

Dimensions

Width parallel to X-X Axis	=	3.0	ft
Length parallel to Z-Z Axis	=	6.0	ft
Footing Thickness	=	18.0	in

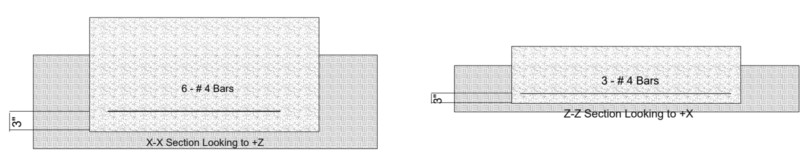


Pedestal dimensions...

px : parallel to X-X Axis	=		in
pz : parallel to Z-Z Axis	=		in
Height	=		in
Rebar Centerline to Edge of Concrete... at Bottom of footing	=	3.0	in

Reinforcing

Bars parallel to X-X Axis			
Number of Bars	=		6
Reinforcing Bar Size	=	#	4
Bars parallel to Z-Z Axis			
Number of Bars	=		3
Reinforcing Bar Size	=	#	4



Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation	ıg X-X Axis
# Bars required within zone	66.7 %
# Bars required on each side of zone	33.3 %

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	0.0							k
OB : Overburden	=								ksf
M-xx	=					8.920			k-ft
M-zz	=	0.0				0.0			k-ft
V-x	=					0.0			k
V-z	=					0.8920			k

General Footing

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ENERCALC, INC. 1983-2017, Build:6.17.3.29, Ver:6.17.3.29

Lic. # : KW-06010381 **Licensee : Coastline Engineering, Inc**

Description : WSW OVERTURNING CHECK

DESIGN SUMMARY

Design OK

	Min. Ratio	Item	Applied	Capacity	Governing Load Combination
PASS	0.6570	Soil Bearing	1.314 ksf	2.0 ksf	+0.60D+0.60W+0.60H about X-X axis
PASS	1.145	Overturning - X-X	6.155 k-ft	7.047 k-ft	+0.60D+0.60W+0.60H
PASS	n/a	Overturning - Z-Z	0.0 k-ft	0.0 k-ft	No Overturning
PASS	n/a	Sliding - X-X	0.0 k	0.0 k	No Sliding
PASS	2.017	Sliding - Z-Z	0.5352 k	1.080 k	+0.60D+0.60W+0.60H
PASS	n/a	Uplift	0.0 k	0.0 k	No Uplift
PASS	0.0	Z Flexure (+X)	0.0 k-ft	13.288 k-ft	+1.40D+1.60H
PASS	0.0	Z Flexure (-X)	0.0 k-ft	13.288 k-ft	+1.40D+1.60H
PASS	0.1865	X Flexure (+Z)	2.478 k-ft	13.288 k-ft	+0.90D+W+0.90H
PASS	0.08839	X Flexure (-Z)	1.175 k-ft	13.288 k-ft	+1.20D+0.50Lr+0.50L+W+1.60H
PASS	0.0	1-way Shear (+X)	0.0 psi	65.0 psi	+1.40D+1.60H
PASS	0.0	1-way Shear (-X)	0.0 psi	65.0 psi	+1.40D+1.60H
PASS	0.08428	1-way Shear (+Z)	5.478 psi	65.0 psi	+1.20D+0.50Lr+0.50L+W+1.60H
PASS	0.04015	1-way Shear (-Z)	2.610 psi	65.0 psi	+1.20D+0.50Lr+0.50L+W+1.60H
PASS	0.003196	2-way Punching	0.4155 psi	130.0 psi	+1.20D+0.50Lr+0.50L+W+1.60H

Detailed Results

Soil Bearing

Rotation Axis & Load Combination...	Gross Allowable	Xecc (in)	Zecc (in)	Actual Soil Bearing Stress @ Location: Bottom, -Z	Top, +Z	Left, -X	Right, +X	Actual / Allow Ratio
X-X, +D+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +D+L+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +D+Lr+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +D+S+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +D+0.750Lr+0.750L+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +D+0.750L+0.750S+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +D+0.60W+H	2.0	n/a	18.865	0.0	0.6022	n/a	n/a	0.301
X-X, +D+0.70E+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +D+0.750Lr+0.750L+0.450W+H	2.0	n/a	14.149	0.0	0.4734	n/a	n/a	0.237
X-X, +D+0.750L+0.750S+0.450W+H	2.0	n/a	14.149	0.0	0.4734	n/a	n/a	0.237
X-X, +D+0.750L+0.750S+0.5250E+H	2.0	n/a	0.0	0.2175	0.2175	n/a	n/a	0.109
X-X, +0.60D+0.60W+0.60H	2.0	n/a	31.442	0.0	1.314	n/a	n/a	0.657
X-X, +0.60D+0.70E+0.60H	2.0	n/a	0.0	0.1305	0.1305	n/a	n/a	0.065
Z-Z, +D+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+L+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+Lr+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+S+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+0.750Lr+0.750L+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+0.750L+0.750S+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+0.60W+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+0.70E+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+0.750Lr+0.750L+0.450W+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+0.750L+0.750S+0.450W+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +D+0.750L+0.750S+0.5250E+H	2.0	0.0	n/a	n/a	n/a	0.2175	0.2175	0.109
Z-Z, +0.60D+0.60W+0.60H	2.0	0.0	n/a	n/a	n/a	0.1305	0.1305	0.065
Z-Z, +0.60D+0.70E+0.60H	2.0	0.0	n/a	n/a	n/a	0.1305	0.1305	0.065

Overturning Stability

Rotation Axis & Load Combination...	Overturning Moment	Resisting Moment	Stability Ratio	Status
X-X, +D+H	None	0.0 k-ft	Infinity	OK
X-X, +D+L+H	None	0.0 k-ft	Infinity	OK
X-X, +D+Lr+H	None	0.0 k-ft	Infinity	OK
X-X, +D+S+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.750L+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750L+0.750S+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.60W+H	6.155 k-ft	11.745 k-ft	1.908	OK
X-X, +D+0.70E+H	None	0.0 k-ft	Infinity	OK
X-X, +D+0.750Lr+0.750L+0.450W+H	4.616 k-ft	11.745 k-ft	2.544	OK
X-X, +D+0.750L+0.750S+0.450W+H	4.616 k-ft	11.745 k-ft	2.544	OK

GUARDRAIL CALCULATION

INPUT	
GUARDRAIL HEIGHT, h (IN)	42
LOAD ON 12" STRIP, L (PLF)	50
DIST. BETWEEN LAG & EDGE OF PLATE, d (IN)	4
SPACING OF LAG SCREWS, S (FT)	1.33

ANALYSIS	
MOMENT = h * L * S (LB-IN)	2800
WITHDRAWEL VALUE$_{REQ'D.}$ = M/d (LBS)	700
LAG SCREW SIZE (IN)	1/4
W' (LBS / IN) (NDS-TABLE 11.2A)	225
PENETRATION$_{REQ'D.}$ = WITHDRAWEL$_{REQ'D.}$ / W' (IN)	3.11
USE 1/4" DIAM. LAG SCREW @ 16" O.C. W/ 4" PENETRATION	

GUARDRAIL CALCULATION

1/2" THICK TEMPERED GLASS PROPERTIES	
MODULUS OF RUPTURE, F_r (PSI)	24000
SAFETY FACTOR, S.F.	4
BENDING STRESS$_{ALL}$ = F_r/S.F. (PSI)	6000

DESIGN DIMENSIONS	
GUARDRAIL HEIGHT, h (IN)	42
MINIMUM THICKNESS, t (IN)	0.469
WIDTH BEING ANALIZED, w (IN)	12

DESIGN LOADS	
LOAD ON 12" STRIP, L (PLF)	50

ANALYSIS				
MOMENT = L*h (LB-IN)	2100			
Z = w*t^2/6 (IN^3)	0.440			
S = M/Z (PSI)	4774	>	6000	**O.K.**

BASE CONNECTION (LAG SCREW INTO WOOD BEAM BELOW)	
MOMENT (LB-IN) (SEE ABOVE)	2100
MIN. DISTANCE FROM SCREW TO EDGE OF SHOE (IN)	1
WITHDRAWEL VALUE$_{REQ'D.}$ = M/d (LBS)	2100
LAG SCREW SIZE (IN)	3/8
W' (LBS / IN) (NDS-TABLE 11.2A)	305
PENETRATION$_{REQ'D.}$ = WITHDRAWAL$_{REQ'D.}$ / W' (IN)	6.89
USE 3/8" DIAM. LAG SCREWS @ 12" O.C. W/ 7" EMBEDMENT	

APPENDIX B : STRUCTURAL DRAWINGS

GENERAL NOTES

1. THESE PLANS, SPECIFICATIONS, DRAWINGS, NOTES, DETAILS, AND ATTACHMENTS ARE THE SOLE PROPERTY OF COASTLINE ENGINEERING, INC., AND SHALL NOT BE REPRODUCED, OR USED IN CONJUNCTION WITH ANY OTHER PROJECT WITHOUT THE EXPRESS WRITTEN CONSENT OF THIS OFFICE.
2. ALL ENGINEERING AND CONSTRUCTION, INCLUDING MATERIALS AND WORKMANSHIP, SHALL CONFORM TO THE LATEST REQUIREMENTS OF THE 2016 CALIFORNIA BUILDING CODE WITH THE GOVERNING JURISDICTION AMENDMENTS.
3. ALL ASTM STANDARDS SHALL BE PER THE LATEST ISSUE OF THE AMERICAN SOCIETY FOR TESTING AND MATERIALS.
4. GENERAL CONTRACTOR SHALL VERIFY ALL DIMENSIONS, GRADES, ELEVATIONS, AND SITE CONDITIONS PRIOR TO STARTING CONSTRUCTION. THE ARCHITECT AND ENGINEER SHALL BE NOTIFIED IN WRITING OF ANY DISCREPANCIES, INCONSISTENCIES, AND/OR CONDITIONS NEEDING CLARIFICATIONS.
5. THE STRUCTURAL DRAWINGS AND SPECIFICATIONS REPRESENT THE FINISHED STRUCTURE AND DO NOT INDICATE THE METHOD OF CONSTRUCTION. GENERAL CONTRACTOR SHALL PROVIDE ALL MEASURES NECESSARY TO PROTECT THE STRUCTURE, WORKERS, AND OTHER PERSONS DURING CONSTRUCTION. SUCH MEASURES SHALL INCLUDE, BUT ARE NOT LIMITED TO, BRACING, SHORING FOR CONSTRUCTION EQUIPMENT, AND SHORING FOR THE STRUCTURE. OBSERVATION VISITS TO THE SITE BY THE ENGINEER SHALL NOT INCLUDE INSPECTIONS OF THE ABOVE ITEMS.
6. IN NO CASE SHALL DIMENSIONS BE SCALED FROM STRUCTURAL PLANS OR DETAILS.
7. ANY OMISSIONS OR DISCREPANCIES FOUND WITHIN THE STRUCTURAL DRAWINGS SHALL BE BROUGHT TO THE ATTENTION OF THE ARCHITECT AND ENGINEER PRIOR TO PROCEEDING.
8. VIBRATIONAL EFFECTS OF MECHANICAL AND/OR ANY OTHER EQUIPMENT HAVE NOT BEEN CONSIDERED IN THE STRUCTURAL DESIGN.
9. CONNECTIONS OF ALL ITEMS SUPPORTED BY THE STRUCTURE ARE THE RESPONSIBILITY OF THE DISCIPLINES WHO MAKE THESE ATTACHMENTS.
10. ALL MATERIALS, SPECIFICATIONS, CONNECTIONS, ETC. SHALL BE AS SHOWN IN THE STRUCTURAL DRAWINGS UNLESS ALTERNATES ARE APPROVED, IN WRITING, BY THE ARCHITECT, ENGINEER, AND OWNER.
11. WHERE A DETAIL, SECTION, OR NOTE IS SHOWN FOR ONE CONDITION, IT SHALL APPLY FOR ALL LIKE OR SIMILAR CONDITIONS UNLESS NOTED OTHERWISE.
12. SHOP DRAWINGS FOR ALL FABRICATED MATERIALS SHALL BE SUBMITTED TO AND APPROVED BY THE ENGINEER PRIOR TO FABRICATION.
13. IT SHALL BE THE RESPONSIBILITY OF THE CONTRACTOR TO LOCATE ALL EXISTING UTILITIES, WHETHER INDICATED ON THE CONSTRUCTION DOCUMENTS OR NOT, AND TO PROTECT THEM FROM DAMAGE.
14. SEE ARCHITECTURAL DRAWINGS FOR ALL WATER PROOFING REQUIREMENTS.
15. THE CONTRACTOR SHALL ASSUME SOLE AND COMPLETE RESPONSIBILITY FOR JOBSITE CONDITIONS DURING THE COURSE OF CONSTRUCTION, INCLUDING THE SAFETY OF ALL PERSONS OR PROPERTY. THIS REQUIREMENT SHALL APPLY CONTINUOUSLY AND NOT BE LIMITED TO NORMAL WORKING HOURS. THE CONTRACTOR SHALL DEFEND, INDEMNIFY, AND HOLD THE STRUCTURAL ENGINEER FREE AND HARMLESS FROM ALL CLAIMS, DEMANDS, AND LIABILITY, REAL OR ALLEGED, IN CONNECTION WITH THE PERFORMANCE OF WORK ON THIS PROJECT, EXCEPT FOR LIABILITY ARISING FROM THE SOLE NEGLIGENCE OF THE STRUCTURAL ENGINEER.

CONCRETE NOTES

1. CONCRETE MATERIALS, CONSTRUCTION, AND WORKMANSHIP SHALL CONFORM TO ACI 318-11.
2. THE MINIMUM COMPRESSIVE STRENGTH OF CONCRETE (f'_c) AT 28 DAYS SHALL BE AS FOLLOWS (U.N.O.):
   - SLAB-ON-GRADE 4,500 PSI
   - FOOTINGS / GRADE BEAMS 4,500 PSI
   - RETAINING WALLS 4,500 PSI
   - CONCRETE OVER METAL DECK 3,000 PSI
   - CAISSONS 4,500 PSI
3. WHERE CONCRETE COMPRESSION DESIGN STRENGTH IS 3,000 PSI OR GREATER, CYLINDER TESTS ARE REQUIRED.
4. THE CONCRETE SUPPLIER SHALL BEAR THE RESPONSIBILITY THAT THE MIX DESIGN WILL ATTAIN THE SPECIFIED STRENGTH. ACCEPTANCE OF MIX DESIGN SHALL BE BASED ONLY ON CONFORMANCE OF SPECIFIED COMPRESSION STRENGTH AND SLUMP.
5. CONCRETE SHALL HAVE A MAXIMUM SLUMP OF 4".
6. CEMENT SHALL CONFORM TO THE REQUIREMENTS OF ASTM C150, TYPE V. THE WATER/CEMENT RATIO SHALL BE A MAXIMUM OF 0.45.
7. AGGREGATES SHALL CONFORM TO THE REQUIREMENTS OF ASTM C33 FOR NORMAL WEIGHT CONCRETE AND ASTM C330 FOR LIGHTWEIGHT CONCRETE. THE MAXIMUM SIZE OF THE COARSE AGGREGATE SHALL NOT EXCEED ⅓ SLAB THICKNESS, ¾", OR THE MINIMUM CLEAR SPACING BETWEEN REINFORCING BARS.
8. ADMIXTURES SHALL NOT BE USED WITHOUT THE WRITTEN CONSENT OF THE ENGINEER OF RECORD. WHEN SUCH CONSENT IS PROVIDED, ADMIXTURES SHALL BE USED IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATIONS.
9. READY-MIX CONCRETE SHALL BE MIXED AND DELIVERED IN ACCORDANCE WITH THE REQUIREMENTS OF ASTM C94.
10. MINIMUM CONCRETE COVER FOR REINFORCING STEEL IN NON-PRESTRESSED CAST-IN-PLACE CONCRETE SHALL BE AS FOLLOWS (U.N.O.):
    - CAST AGAINST EARTH 3"
    - EXPOSED TO EARTH OR WEATHER:
      - #6 AND LARGER BARS 2"
      - #5 AND SMALLER BARS 1½"
    - NOT EXPOSED TO EARTH OR WEATHER:
      - SLABS AND WALLS ¾"
      - BEAMS AND COLUMNS (TIES, STIRRUPS, SPIRALS) 1½"
      - UNPROTECTED COLUMNS 2½"
11. ALL REINFORCING BARS, WIRE MESH, ANCHOR BOLTS, SLEEVES, AND OTHER CONCRETE INSERTS SHALL BE SECURED IN PLACE AND APPROVED BY THE BUILDING INSPECTOR PRIOR TO PLACING CONCRETE.
12. ALL CONCRETE SHALL BE CONSOLIDATED WITH MECHANICAL VIBRATORS.
13. PIPES, DUCTS, SLEEVES, CONDUITS, ETC. SHALL NOT BE PLACED THROUGH CONCRETE UNLESS SHOWN ON THE STRUCTURAL PLANS OR WITH WRITTEN APPROVAL FROM THE ENGINEER OF RECORD.

MASONRY NOTES

1. CONCRETE MASONRY UNITS MATERIALS, CONSTRUCTION, AND WORKMANSHIP SHALL CONFORM TO ASTM C90.
2. CONCRETE MASONRY UNITS SHALL BE MEDIUM WEIGHT WITH f'_m = 1,500 PSI. ALL UNITS SHALL BE SAMPLED AND TESTED TO VERIFY COMPLIANCE WITH ASTM C55 OR ASTM C90.
3. MASONRY SHALL BE SOLID GROUTED WITH REINFORCED CELLS.
4. MORTAR SHALL HAVE A COMPRESSIVE STRENGTH OF 1,900 PSI (MIN.) FOR TYPE 'S' MORTAR AND 2,150 PSI (MIN.) FOR TYPE 'N' MORTAR. MORTAR SHALL CONFORM TO ASTM C270 AND ARTICLES 20.1 AND 2.6A OF TMS 602/ACI 530.1/ASCE 6. TYPE 'S' MORTAR SHALL BE USED FOR ALL WALLS IN CONTACT WITH SOIL AND TYPE 'N' MORTAR FOR ALL OTHER LOCATIONS. ALL HEAD AND BED JOINT THICKNESS SHALL BE BETWEEN ⅜" AND ⅝". BED JOINT THICKNESS OF THE STARTING COURSE OVER THE CONCRETE FOUNDATION MAY BE BETWEEN ¼" AND ¾".
5. GROUT SHALL HAVE A COMPRESSIVE STRENGTH OF 2,000 PSI (MIN.), AS DETERMINED IN ACCORDANCE WITH ASTM C1019. GROUT SHALL CONFORM TO ASTM C476 AND ARTICLE 2.2 OF TMS 602/ACI 530.1/ASCE 6. 'FINE' AND 'COURSE' AGGREGATE SHALL COMPLY WITH ASTM C404.
6. THE FIRST COURSE OF BLOCK SHALL BE SET INTO CONCRETE UNLESS A MORTAR KEY IS USED.
7. HIGH LIFT GROUTING PROCEDURE MAY BE USED FOR HEIGHTS UP TO 24 FT. PER TABLE 7 OF TMS 602. FINE GROUT WITH ADMIXTURES SHALL BE USED FOR LIFTS OVER 12 FT. AND A CLEANOUT SHALL BE PROVIDED AT THE BOTTOM COURSE AT EVERY VERTICAL BAR TO HELP PREVENT VOIDS.

RETAINING WALL NOTES

1. ALL BACKFILL MATERIALS SHALL BE GRANULAR, NON COHESIVE SOIL. BACKFILL SHALL BE PLACED IN 12" MAX. HORIZONTAL LIFTS. ALL FILLS SHALL BE COMPACTED TO 90% (MIN.). IF EXPANSIVE SOIL IS ENCOUNTERED, THE ENGINEER OF RECORD SHALL BE NOTIFIED PRIOR TO WALL CONSTRUCTION.
2. DO NOT PLACE BACKFILL BEHIND WALLS UNTIL THEY HAVE ATTAINED THEIR SPECIFIED DESIGN STRENGTH. WALLS THAT ARE DESIGNED TO BE BRACED BY THE STRUCTURE, SHALL BE SHORED UNTIL THE SUPPORTING MEMBERS ARE IN PLACE.
3. CONTRACTOR IS RESPONSIBLE FOR ALL DESIGN AND CONSTRUCTION OF ALL UNDERPINNING, CRIBBING, BRACING, AND SHORING REQUIRED FOR THE RETAINING WALL.
4. WALL DRAINS, 4" Ø MIN., SHALL BE PLACED AT 6 FT. INTERVALS ALONG THE LENGTH OF THE WALL AND LOCATED JUST ABOVE THE LEVEL OF THE SOIL OR PAVING ON THE FRONT FACE OF THE WALL. BACKFILL BEHIND WALL DRAINS OR OPEN HEAD JOINTS MUST BE GRAVEL WITH A MINIMUM THICKNESS OF 12" AND EXTENDING FROM THE TOP OF THE WALL TO THE TOP OF THE FOOTING.

REINFORCING STEEL (REBAR) NOTES

1. REINFORCING STEEL SHALL CONFORM TO ASTM A615. ALL REINFORCING SHALL BEAR MILL STOCK IDENTIFICATION.
2. REINFORCING STEEL GRADE SHALL BE:
   - GRADE 60 - #4 BARS AND LARGER
   - GRADE 40 - #3 BARS AND SMALLER
   - ASTM A706 GRADE 60 - ALL WELDED REINFORCING STEEL
3. REINFORCING STEEL DETAILING, BENDING, AND PLACING SHALL BE IN ACCORDANCE WITH THE LATEST ADDITION OF THE "MANUAL OF STANDARD PRACTICE" BY THE CONCRETE REINFORCING STEEL INSTITUTE.
4. SEE CONCRETE NOTES FOR CLEAR COVER REQUIREMENTS.
5. EPOXY COATED REINFORCING STEEL SHALL CONFORM TO ASTM A934.
6. REINFORCING STEEL SHALL BE CLEAN OF RUST, OIL, GREASE, OR ANY OTHER COATING MATERIAL LIKELY TO IMPAIR BONDING.
7. SEE TYPICAL REBAR DETAIL ON SHEET S5 FOR LAP SPLICE, BEND, AND HOOK SPECIFICATIONS.
8. ALL REINFORCEMENT SHALL BE SECURELY TIED IN PLACE BEFORE PLACING CONCRETE OR GROUT.

WOOD NOTES

1. ALL LUMBER SHALL CONFORM TO THE GRADES AS SET BY AN INSPECTION AGENCY THAT HAS BEEN APPROVED BY AN ACCREDITATION BODY THAT COMPLIES WITH THE DOC PS 20 OR EQUIVALENT.
2. FRAMING LUMBER SHALL BE DOUGLAS FIR-LARCH WITH THE FOLLOWING GRADE (U.N.O.):
   - STUDS (8'-1" HT. AND LESS) STUD GRADE OR BETTER
   - STUDS (GREATER THAN 8'-1" HT.) #2 OR BETTER
   - SILLS/PLATES AND LEDGERS #2 OR BETTER
   - HEADERS #2 OR BETTER
   - POSTS AND BEAMS #1 OR BETTER
3. MOISTURE CONTENT OF SAWN LUMBER AT THE TIME OF PLACEMENT SHALL NOT EXCEED 19%.
4. WOOD BEARING ON CONCRETE OR MASONRY IN CONTACT WITH SOIL SHALL BE PRESSURE TREATED.
5. WOOD STUDS/POSTS LESS THAN 8" FROM GRADE SHALL BE PRESSURE TREATED.
6. FASTENERS, INCLUDING NUTS AND WASHERS, IN PRESSURE TREATED WOOD SHALL BE HOT DIPPED ZINC-COATED GALVANIZED STEEL PER ASTM A153. ANCHOR BOLTS MAY HAVE A MECHANICALLY DEPOSITED ZINC COATING WITH WEIGHTS PER ASTM B695, CLASS 55. IT IS ACCEPTABLE TO USE PLAIN CARBON STEEL FASTENERS IN ZINC BORATE TREATED WOOD IN AN INTERIOR, DRY ENVIRONMENT SUCH AS IN A WALL CAVITY.
7. ALL BOLT HEADS, NUTS, AND LAG SCREWS BEARING ON WOOD SHALL HAVE CUT WASHERS (U.N.O.).
8. BOLT HOLES IN WOOD SHALL BE DRILLED 1⁄16" LARGER THAN THE BOLT DIAMETER. BOLT HOLES SHALL BE ACCURATELY ALIGNED AND NOT FORCIBLY DRIVEN.
9. LEAD HOLES FOR LAG SCREWS IN WOOD SHALL BE BORED AS FOLLOWS:
   - FOR SHANK: SAME Ø AND LENGTH AS UNTHREADED SHANK
   - FOR THREADED PORTION: 60% - 75% OF SHANK DIAMETER AND LENGTH EQUAL TO THREADED PORTION
10. GLUED LAMINATED TIMBERS (GLULAM) SHALL BE FABRICATED IN ACCORDANCE WITH THE ANSI A190.1 AND ASTM D3737, USING DOUGLAS FIR INDUSTRIAL GRADE WOOD AND EXTERIOR GLUE WITH INTENDED DRY USE CONDITION . EACH GLULAM SHALL BE GRADE MARKED AND A CERTIFICATE OF CONFORMANCE MUST BE PROVIDED THAT INDICATES CONFORMANCE WITH ANSI/AITC A190.1.
11. SIMPLE SPAN GLULAM BEAMS SHALL BE TYPE 24F-V4 DF/DF AND CANTILEVERED / MULTI-SPAN GLULAM BEAMS SHALL BE 24F-V8 DF/DF.
12. MANUFACTURED LUMBER PRODUCTS SPECIFIED ON THE DRAWINGS SHALL BE MANUFACTURED BY 'WEYERHAEUSER' (ESR-1387, 1153) OR AN ENGINEER APPROVED EQUAL. THE MODULUS OF ELASTICITY FOR PARALLAM (PSL) BEAMS = 2.0E, PARALLAM (PSL) COLUMNS = 1.8E, MICROLLAM (LVL) = 2.0E, AND TIMBERSTRAND (LSL) = 1.55E. 'BOISE CASCADE' MANUFACTURED LUMBER CAN BE SUBSTITUTED AS SHOWN BELOW:
    - TJI 210 = BCI 6000 1.8 PSL (2.0E) = VERSA-LAM 2.0 3100
    - TJI 230 = BCI 6500 1.8 PSL (1.8E) = VERSA-LAM 2.0 3100
    - TJI 360 = BCI 60 2.0 LVL (2.0E) = VERSA-LAM 2.0 2800
    - TJI 560 = BCI 90 2.0 LSL (1.55E) = VERSA-LAM 1.7 2400 / 2650

WOOD FASTENING SCHEDULE

DESCRIPTION OF BUILDING ELEMENTS	NUMBER & TYPE OF FASTENERS	SPACING & LOCATION
ROOF		
1. BLOCKING BETWEEN CEILING JOISTS, RAFTERS, OR TRUSSES TO TOP PLATE OR OTHER FRAMING BELOW	3-8d	EACH END, TOENAIL
BLOCKING BETWEEN RAFTERS OR TRUSSES NOT AT THE WALL TOP PLATE, TO RAFTER OR TRUSS	2-8d & 2-16d	EACH END, TOENAIL END NAIL
FLAT BLOCKING TO TRUSS AND WEB FILLER	16d @ 6" O.C.	FACE NAIL
2. CEILING JOIST TO TOP PLATE	3-8d	EACH JOIST, TOENAIL
3. CEILING JOIST NOT ATTACHED TO PARALLEL RAFTER, LAPS OVER PARTITIONS (NO THRUST)	3-16d	FACE NAIL
4. CEILING JOIST ATTACHED TO PARALLEL RAFTER (HEAL JOINT)	3-16d MIN. (SEE PLANS)	FACE NAIL
5. COLLAR TIES TO RAFTERS	3-10d	FACE NAIL
6. RAFTER OR ROOF TRUSS TO TOP PLATE	3-10d	TOENAIL
7. ROOF RAFTER TO RIDGE VALLEY OR HIP RAFTERS; OR ROOF RAFTER TO 2" RIDGE BEAM	2-16d OR 3-10d	END NAIL TOENAIL
WALL		
8. STUD TO STUD (NOT AT BRACED WALL PANELS)	16d	24" O.C. FACE NAIL
9. STUD TO STUD AND ABUTTING STUDS AT INTERSECTING WALL CORNERS (AT BRACED WALL PANELS)	16d	16" O.C. FACE NAIL
10. BUILT-UP HEADER (2" TO 2" HEADER)	16d	16" O.C. EA. EDGE, FACE NAIL
11. CONTINUOUS HEADER TO STUD	4-8d	TOENAIL
12. TOP PLATE TO TOP PLATE	16d	16" O.C. FACENAIL
13. TOP PLATE TO TOP PLATE, AT END JOINTS	8-16d	EA. SIDE OF END JOINT, FACE NAIL
14. BOTTOM PLATE TO JOIST, RIM JOIST, BAND JOIST, OR BLOCKING (NOT AT BRACED WALL PANELS)	16d	16" O.C. FACE NAIL
15. BOTTOM PLATE TO JOIST, RIM JOIST, BAND JOIST, OR BLOCKING AT BRACED WALL PANELS	2-16d	16" O.C. FACE NAIL
16. STUD TO TOP OR BOTTOM PLATE	4-8d & 2-16d	TOENAIL END NAIL
17. TOP OR BOTTOM PLATE TO STUD	2-16d	END NAIL
18. TOP PLATES, LAPS AT CORNERS AND INTERSECTIONS	2-16d	FACE NAIL
19. 1" BRACE TO EACH STUD AND PLATE	2-8d	FACE NAIL
20. 1"x6" SHEATHING TO EACH BEARING	2-8d	FACE NAIL
21. 1"x8" AND WIDER SHEATHING TO EACH BEARING	3-8d	FACE NAIL
FLOOR		
22. JOIST TO SILL, TOP PLATE, OR GIRDER	3-8d	TOENAIL
23. RIM JOIST, BAND JOIST, OR BLOCKING TO TOP PLATE, SILL, OR OTHER FRAMING BELOW	8d	6" O.C., TOENAIL
24. 1"x6" SUBFLOOR OR LESS TO EACH JOIST	2-8d	FACE NAIL
25. 2" SUBFLOOR TO JOIST OR GIRDER	2-16d	FACE NAIL
26. 2" PLANKS (PLANK & BEAM - FLOOR & ROOF)	2-16d	EA. BRG., FACE NAIL
27. BUILT-UP GIRDERS AND BEAMS, 2" LUMBER LAYERS	20d & 2-20d	32" O.C., FACE NAIL AT TOP & BOTTOM STAGGERED ON OPPOSITE SIDES ENDS & AT EA. SPLICE, FACE NAIL
28. LEDGER STRIP SUPPORTING JOISTS OR RAFTERS	3-16d	EA. JOIST / RAFTER, FACE NAIL
29. JOIST TO BAND JOIST OR RIM JOIST	3-16d	END NAIL
30. BRIDGING OR BLOCKING TO JOIST, RAFTER, OR TRUSS	2-8d	EACH END, TOENAIL

WOOD STRUCTURAL PANELS (WSP), SUBFLOOR, ROOF, AND INTERIOR WALL SHEATHING TO FRAMING AND PARTICLEBOARD WALL SHEATHING TO FRAMING

		EDGES	INTERMEDIATE SUPPORTS
31. ⅜" - ½"	6d	6"	12"
32. 19⁄32" - ¾"	8d	6"	12"
33. ⅞" - 1¼"	10d	6"	12"
WOOD STRUCTURAL PANELS, COMBINATION SUBFLOOR UNDERLAYMENT TO FRAMING			
36. ¾" AND LESS	8d	6"	12"
37. ⅞" - 1"	8d	6"	12"
38. 1⅛" - 1¼"	10d	6"	12"
PANEL SIDING TO FRAMING			
39. ½" OR LESS	6d CORROSION-RESISTANT SIDING	6"	12"
40. ⅝"	8d CORROSION-RESISTANT SIDING	6"	12"
INTERIOR PANELING			
41. ¼"	4d CASING	6"	12"
42. ⅜"	6d CASING	6"	12"

FASTENING SCHEDULE NOTES:
A. ALL NAILS TO BE COMMONS (U.N.O.).
B. DETAILS AND NOTES SUPERCEDE THE FASTENING SCHEDULE WHERE THEY DIFFER.

WELDING NOTES

1. WELDING SHALL CONFORM TO THE LATEST ADDITION OF THE "STRUCTURAL WELDING CODE" ANSI / AWS D1.1.
2. WELDS SHALL USE E70 ELECTRODES (EXCEPT REINFORCING STEEL WELDS).
3. WELDS OF REINFORCING STEEL USING A706 GRADE 60 STEEL SHALL USE E80 ELECTRODES AND SHALL CONFORM TO AWS D1.4 & RGA3-77.
4. SHOP WELDING SHALL BE PERFORMED IN A SHOP THAT IS REGISTERED AND APPROVED BY THE GOVERNING JURISDICTION.
5. FIELD WELDING SHALL BE CONTINUOUSLY INSPECTED BY A REGISTERED INSPECTOR. ALL FIELD WELDING MUST BE INDICATED ON THE SHOP DRAWINGS.
6. ALL EXPOSED WELDED CONNECTIONS SHALL BE FILLED AND GROUND SMOOTH AND SUBJECT TO ARCHITECTURAL APPROVAL.
7. ALL WELDS NOT SPECIFIED SHALL BE CONTINUOUS FILLET WELDS. SIZE OF WELDS SHALL BE BASED ON AISC STANDARDS FOR THICKER MATERIAL CONNECTED.

STEEL NOTES

1. STEEL MATERIALS, CONSTRUCTION, AND WORKMANSHIP SHALL CONFORM TO THE 14TH ADDITION OF THE AISC MANUAL OF STEEL CONSTRUCTION AND SHALL BE DETAILED, FABRICATED, AND ERECTED IN CONFORMANCE WITH THE AISC SPECIFICATIONS.
2. STRUCTURAL STEEL MATERIAL SHALL BE AS FOLLOWS (U.N.O.):
   - W SHAPES: ASTM A992, F_y = 50 KSI
   - HSS SHAPES (RECTANGULAR): ASTM A500, GRADE B, F_y = 46 KSI
   - HSS SHAPES (ROUND): ASTM A500, GRADE B, F_y = 42 KSI
   - ALL OTHER SHAPES: ASTM A36, F_y = 36 KSI
   - UNHEADED BOLTS & WASHERS ASTM A36, F_y = 36 KSI
   - HEADED BOLTS & THREADED RODS ASTM A307, GRADE A
   - HIGH STRENGTH BOLTS ASTM A325N / ASTM A490 (SEE PLANS)
   - SHEAR STUDS: ASTM A108 & A.W.S. D1.1, F_y = 60 KSI
   - NUTS ASTM A563, GRADE A
   - ANCHOR BOLTS & HEAVY HEX HEAD BOLTS INSTALLED IN CONCRETE ASTM F1554, GRADE 36
3. STEEL FABRICATORS SHALL FURNISH SHOP DRAWINGS FOR REVIEW BY THE ENGINEER OF RECORD PRIOR TO FABRICATION.
4. STEEL FABRICATION SHALL BE PERFORMED IN A SHOP THAT IS APPROVED BY THE GOVERNING JURISDICTION.
5. EXPOSED STEEL SHALL BE PRIMED/PAINTED OR HOT DIPPED GALVANIZED.
6. HOLES SHALL NOT BE PLACED IN STEEL MEMBERS UNLESS SPECIFICALLY DETAILED ON DRAWINGS. HOLES SHALL BE 1⁄16" OVERSIZED FOR ORDINARY CONNECTIONS AND 3⁄16" OVERSIZED FOR ANCHOR BOLTS (U.N.O.).
7. GROUTING MATERIAL AT BASE PLATES SHALL BE NON-SHRINK GROUT / DRY PACK WITH A COMPRESSIVE STRENGTH OF f'_c = 6,000 PSI (MIN.). INSTALL GROUT AFTER COLUMN HAS BEEN PLUMBED AND PRIOR TO FRAMING ERECTION.
8. HIGH STRENGTH BOLTS SHALL BE PROVIDED WITH HARDENED WASHERS CONFORMING TO ASTM F436.
9. STRUCTURAL STEEL SHALL BE DELIVERED TO THE JOBSITE FREE OF RUST, MILL SCALE, GREASE, ETC.
10. STEEL BEAMS WITH SPECIFIED INDUCED CAMBER PER PLAN MAY BE COLD CAMBERED (U.N.O).

ABBREVIATIONS

A.B.	ANCHOR BOLT	GRD.	GRADE
ABV.	ABOVE	HDR.	HEADER
AISC	AMERICAN INSTITUTE OF STEEL CONSTRUCTION	HDR.	HANGER
ALT.	ALTERNATE	HORIZ.	HORIZONTAL
ARCH.	ARCHITECT	H.S.	HIGH STRENGTH
ASTM	AMERICAN SOCIETY OF TESTING MATERIALS	HSS	HOLLOW STRUCTURAL STEEL
BLK.	BLOCK	HT.	HEIGHT
BLK'G.	BLOCKING	IN.	INCHES
BLW.	BELOW	INT.	INTERIOR
BM.	BEAM	JST.	JOIST
B.N.	BOUNDARY NAIL	LBS.	POUNDS
BRG.	BEARING	MAX.	MAXIMUM
BTM.	BOTTOM	M.B.	MACHINE BOLT
CAMB.	CAMBER	MECH.	MECHANICAL
C/B	CEILING BEAM	MFGR.	MANUFACTURER
CBC	CALIFORNIA BUILDING CODE	MIN.	MINIMUM
C.J.	CONTROL JOINT	MISC.	MISCELLANEOUS
C/J	CEILING JOIST	(N)	NEW
C.L.	CENTERLINE	N/A	NOT APPLICABLE
CLR.	CLEARANCE	NO.	NUMBER
C.M.U.	CONCRETE MASONRY UNIT	N.T.S.	NOT TO SCALE
COL.	COLUMN	O.C.	ON CENTER
CONC.	CONCRETE	OPNG.	OPENING
CONN.	CONNECTION	PAR.	PARALLEL
CONST.	CONSTRUCTION	PERP.	PERPENDICULAR
CONT.	CONTINUOUS	PL.	PLATE
CVR.	COVER	PSF	POUNDS PER SQUARE FOOT
d	NAIL PENNY SIZE	PSI	POUNDS PER SQUARE INCH
D/B	DECK BEAM	P.T.	PRESSURE TREATED
DET.	DETAIL	RAD.	RADIUS
D.F.	DOUGLAS FIR LARCH	R/B	ROOF BEAM
DIA.	DIAMETER	REINF.	REINFORCING
D/J	DECK JOIST	REQ'D.	REQUIRED
(E)	EXISTING	RF.	ROOF
EA.	EACH	R/R	ROOF RAFTER
E.F.	EACH FACE	SCHED.	SCHDULE
EMBED.	EMBEDMENT	SHTG.	SHEATHING
E.N.	EDGE NAIL	SIM.	SIMILAR
E.O.R.	ENGINEER OF RECORD	SPEC.	SPECIFICATION
EQ.	EQUAL	SQ.	SQUARE
EQUIP.	EQUIPMENT	STD.	STANDARD
E.S.	EACH SIDE	STL.	STEEL
EXIST.	EXISTING	S.S.	SELECT STRUCTURAL
EXT.	EXTERIOR	STRUCT.	STRUCTURE / STRUCTURAL
F/B	FLOOR BEAM	T&B	TOP AND BOTTOM
F.G.	FINISH GRADE	T&G	TOUNGUE AND GROOVE
F/J	FLOOR JOIST	THK.	THICK
FLR.	FLOOR	TS	TUBE SHAPE
F.N.	FIELD NAIL	THRU	THROUGH
FNDT.	FOUNDATION	T.O.W.	TOP OF WALL
FRM'G.	FRAMING	U.N.O.	UNLESS NOTED OTHERWISE
FT.	FEET	VERT.	VERTICAL
FTG.	FOOTING	W	WIDE FLANGE
GA.	GAUGE	W/	WITH
GALV.	GALVANIZED	W/O	WITHOUT
GLB.	GLUED LAMINATED BEAM	WD.	WOOD

DESIGN CRITERIA

DESIGN LOADS

ROOF LOADS

ROOFING:	ASPHALT SHINGLE
TOTAL DEAD LOAD:	14.0 PSF
LIVE LOAD:	20.0 PSF
TOTAL LOAD:	34.0 PSF

ROOF LOADS

ROOFING:	BUILT UP W/GRAVEL
TOTAL DEAD LOAD:	16.0 PSF
LIVE LOAD:	20.0 PSF
TOTAL LOAD:	36.0 PSF

FLOOR LOADS

FLOORING:	HARDWOOD / TILE
TOTAL DEAD LOAD:	21.0 PSF
LIVE LOAD:	40.0 PSF
TOTAL LOAD:	61.0 PSF

DECK LOADS

DECKING:	DEX-OTEX
TOTAL DEAD LOAD:	13.5 PSF
LIVE LOAD:	60.0 PSF
TOTAL LOAD:	73.5 PSF

EXTERIOR WALL LOADS

MATERIAL:	STUCCO
TOTAL DEAD LOAD:	11.5 PSF

INTERIOR WALL LOADS

MATERIAL:	DRYWALL
TOTAL DEAD LOAD:	9.0 PSF

LATERAL LOAD DESIGN DATA

EARTHQUAKE DESIGN
EQUIV. LATERAL FORCE PROCEDURE

RISK CATEGORY:	II
IMPORTANCE FACTOR, I_e:	1.0
SITE CLASS	D
SEISMIC DESIGN CATEGORY:	D
S_{DS}:	0.846
S_{D1}:	0.487
R: (WOOD SHEAR WALLS)	6.5

WIND DESIGN
ENVELOPE PROCEDURE

RISK CATEGORY:	II
IMPORTANCE FACTOR, I_w:	1.0
EXPOSURE CATEGORY:	C
BASIC WIND SPEED, V (MPH):	110
TOPOGRAPHIC FACTOR, K_{zt}:	1.0
EXPOSURE COEFFICIENT, K_h:	0.85
DIRECTIONALITY FACTOR, K_d:	0.85

SOIL PROPERTIES

GEOTECHNICAL REPORT BY:	LGC GEOTECHNICAL, INC.
PROJECT NUMBER:	17032-01
DATED:	04/14/17
ALLOWABLE BEARING PRESSURE:	1500 PSF
PASSIVE PRESSURE:	250 PCF
SEISMIC SOIL PRESSURE:	⅓ INCREASE OF PASSIVE
COEFFICIENT OF FRICTION:	0.35

STATEMENT OF SPECIAL INSPECTION

DESCRIPTION & TYPE OF INSPECTION REQUIRED

1. NONE REQUIRED
2. RETROFIT ANCHOR BOLTS, HOLDOWNS, AND DOWELS USING SIMPSON 'SET-XP' EPOXY INTO CONCRETE (ESR-2508)
   - CONTINUOUS SPECIAL INSPECTION TO VERIFY ANCHOR TYPE, ADHESIVE IDENTIFICATION AND EXPIRATION DATE, ANCHOR DIMENSIONS, CONCRETE TYPE, CONCRETE COMPRESSIVE STRENGTH, HOLE DRILLING METHOD, HOLE DIMENSIONS, HOLE CLEANING PROCEDURES, ANCHOR SPACING, EDGE DISTANCES, CONCRETE THICKNESS, ANCHOR EMBEDMENT, TIGHTENING TORQUE, AND ADHERENCE TO THE MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS.
3. TITEN HD ANCHORS INTO CONCRETE (ESR-2713)
   - CONTINUOUS SPECIAL INSPECTION TO VERIFY THE FASTENER TYPE & DIMENSIONS, HOLE CLEANING PROCEDURE, EMBEDMENT DEPTH, CONCRETE TYPE, CONCRETE COMPRESSIVE STRENGTH, CONCRETE MEMBER THICKNESS, HOLE DIMENSIONS, ANCHOR SPACING, EDGE DISTANCE, INSTALLATION TORQUE, MAXIMUM IMPACT WRENCH TORQUE RATING, AND ADHERENCE TO THE MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS.
4. RETROFIT ANCHOR BOLTS, HOLDOWNS, AND DOWELS USING SIMPSON 'SET-XP' EPOXY INTO MASONRY (UES-265)
   - CONTINUOUS SPECIAL INSPECTION WHERE INSPECTOR SHALL PROVIDE TO THE BUILDING OFFICIAL AND THE ENGINEER OF RECORD AN INSPECTION REPORT THAT INCLUDES THE FOLLOWING:
     A. ANCHOR DESCRIPTION, INCLUDING THE ADHESIVE PRODUCT NAME AND EXPIRATION DATE, ANCHOR STEEL TYPE, NOMINAL DIAMETER, AND LENGTH.
     B. DRILLED HOLE DESCRIPTIONS, INCLUDING VERIFICATION OF DRILL BIT COMPLIANCE WITH ANSI B212.15-1994, HOLE DIAMETER, LOCATION, DEPTH, AND CLEANLINESS.
     C. INSTALLATION DESCRIPTION INCLUDING VERIFICATION OF MASONRY COMPRESSION STRENGTH, VERIFICATION OF ANCHOR INSTALLATION LOCATION (SPACING AND EDGE DISTANCE), INSTALLATION TEMPERATURE, AND GENERAL INSTALLATION REQUIREMENTS IN ACCORDANCE WITH THE MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS.
5. SOILS (REFER TO SPECIAL INSPECTION REQUIREMENTS SECTION)
6. WOOD SHEAR WALLS
   - PERIODIC SPECIAL INSPECTION TO VERIFY NAILING, BOLTING, ANCHORING, AND OTHER FASTENING WHERE NAIL SPACING OF THE SHEAR PANEL IS 4" O.C. OR LESS.
7. CAST-IN-PLACE DEEP FOUNDATIONS (REFER TO CONCRETE NOTES AND SPECIAL INSPECTION REQUIREMENTS SECTIONS)
8. CONCRETE (REFER TO CONCRETE NOTES AND SPECIAL INSPECTION REQUIREMENTS SECTIONS)
9. HIGH STRENGTH BOLTS (REFER TO SPECIAL INSPECTION REQUIREMENTS SECTION)
10. SIMPSON WOOD STRONG-WALLS (ESR-1267)
    - PERIODIC SPECIAL INSPECTION TO VERIFY THAT THE ANCHORAGE AND TOP-OF-WALL CONNECTION ADHERE TO THE MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS.

16. SIMPSON STRONG-WALL WOOD SHEARWALLS (ESR-2652)
    - PERIODIC SPECIAL INSPECTION TO VERIFY THAT THE ANCHORAGE AND TOP-OF-WALL CONNECTION ADHERE TO THE MANUFACTURER'S PUBLISHED INSTALLATION INSTRUCTIONS.

SPECIAL INSPECTION NOTES:
A. SPECIAL INSPECTIONS ARE IN ADDITION TO THOSE REQUIRED BY THE BUILDING DEPARTMENT. SPECIAL INSPECTION IS NOT A SUBSTITUTE FOR INSPECTIONS BY THE CITY INSPECTOR.
B. THE DUTIES OF THE SPECIAL INSPECTOR SHALL BE IN CONFORMANCE WITH THE REQUIREMENTS OF CHAPTER 17 OF THE CALIFORNIA BUILDING CODE.
C. THE SPECIAL INSPECTOR MUST BE CERTIFIED BY THE GOVERNING JURISDICTION TO PERFORM THE INSPECTION SPECIFIED, EXCEPT WHERE SPECIFICALLY STATED OTHERWISE.
D. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO OBTAIN THE SPECIAL INSPECTOR. FAILURE OF NOTIFICATION FOR INSPECTION MAY RESULT IN COMPLETE REMOVAL AND REPLACEMENT OF ALL WORK SPECIFIED AS NEEDING SPECIAL INSPECTION AT CONTRACTOR'S EXPENSE.
E. A 'CERTIFICATE OF COMPLIANCE' OF WORK REQUIRING SPECIAL INSPECTION MUST BE COMPLETED AND SUBMITTED TO THE INSPECTION SERVICES DEPARTMENT.
F. A PROPERTY OWNER'S FINAL REPORT FORM FOR WORK REQUIRED TO HAVE SPECIAL INSPECTIONS, TESTING, AND STRUCTURAL OBSERVATION MUST BE COMPLETED BY THE PROPERTY OWNER, PROPERTY OWNER'S AGENT OF RECORD, ARCHITECT OF RECORD, OR ENGINEER OF RECORD AND SUBMITTED TO THE INSPECTION SERVICES DIVISION.
G. NOTICE TO THE APPLICANT / OWNER / OWNER'S AGENT / ARCHITECT OR ENGINEER OF RECORD: BY USING THESE PERMITTED CONSTRUCTION DRAWINGS FOR CONSTRUCTION / INSTALLATION OF THE WORK SPECIFIED HEREIN, YOU AGREE TO COMPLY WITH THE REQUIREMENTS OF THE GOVERNING JURISDICTION FOR SPECIAL INSPECTIONS, STRUCTURAL OBSERVATIONS, CONSTRUCTION MATERIAL TESTING, AND OFF-SITE FABRICATION OF BUILDING COMPONENTS CONTAINED IN THE STATEMENT OF SPECIAL INSPECTIONS, AND AS REQUIRED BY THE CALIFORNIA CONSTRUCTION CODES.
H. NOTICE TO THE CONTRACTOR / BUILDER / INSTALLER / SUB-CONTRACTOR / OWNER-BUILDER: BY USING THESE PERMITTED CONSTRUCTION DRAWINGS FOR CONSTRUCTION / INSTALLATION OF THE WORK SPECIFIED HEREIN, YOU ACKNOWLEDGE AND ARE AWARE OF THE REQUIREMENTS OF THE GOVERNING JURISDICTION FOR SPECIAL INSPECTIONS, STRUCTURAL OBSERVATIONS, CONSTRUCTION MATERIAL TESTING, AND OFF-SITE FABRICATION OF BUILDING COMPONENTS CONTAINED IN THE STATEMENT OF SPECIAL INSPECTIONS, AND AS REQUIRED BY THE CALIFORNIA CONSTRUCTION CODES.
I. SHOP WELDING SHALL BE PERFORMED IN A SHOP THAT IS REGISTERED AND APPROVED BY THE GOVERNING JURISDICTION. FABRICATION DONE IN AN APPROVED SHOP NEED NOT HAVE SPECIAL INSPECTION. THE FABRICATOR SHALL SUBMIT AN 'APPLICATION TO PERFORM OFF-SITE FABRICATION' TO THE INSPECTION SERVICES DIVISION FOR APPROVAL PRIOR TO COMMENCEMENT OF FABRICATION. THE FABRICATOR SHALL SUBMIT A 'CERTIFICATE OF COMPLIANCE FOR OFF-SITE FABRICATION' TO THE INSPECTION SERVICES DIVISION PRIOR TO THE ERECTION OF FABRICATED ITEMS AND ASSEMBLIES.
J. FIELD SITE VISITS BY THE STRUCTURAL ENGINEER DO NOT CONSTITUTE AN INSPECTION.

SPECIAL INSPECTION REQUIREMENTS

REQUIRED VERIFICATION & INSPECTION OF SOILS (TABLE 1705.6)

VERIFICATION & INSPECTION	CONTINUOUS	PERIODIC
1. VERIFY MATERIALS BELOW SHALLOW FOUNDATIONS ARE ADEQUATE TO ACHIEVE THE DESIGN CAPACITY.		X
2. VERIFY EXCAVATIONS ARE EXTENDED TO PROPER DEPTH AND HAVE REACHED PROPER MATERIAL.		X
3. PERFORM CLASSIFICATION AND TESTING OF COMPACTED FILL MATERIALS.		X
4. VERIFY USE OF PROPER MATERIALS, DENSITIES, AND LIFT THICKNESSES DURING PLACEMENT AND COMPACTION OF COMPACTED FILL.	X	
5. PRIOR TO PLACEMENT OF COMPACTED FILL, OBSERVE SUBGRADE AND VERIFY THAT SITE HAS BEEN PREPARED PROPERLY.		X

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APPENDIX B

JAMES GLOVER HOME, INC.
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BEAUCHEMIN RESIDENCE
148 W. AVENIDA CADIZ SAN CLEMENTE CA 92672

REVISIONS

PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017
SCALE: N/A

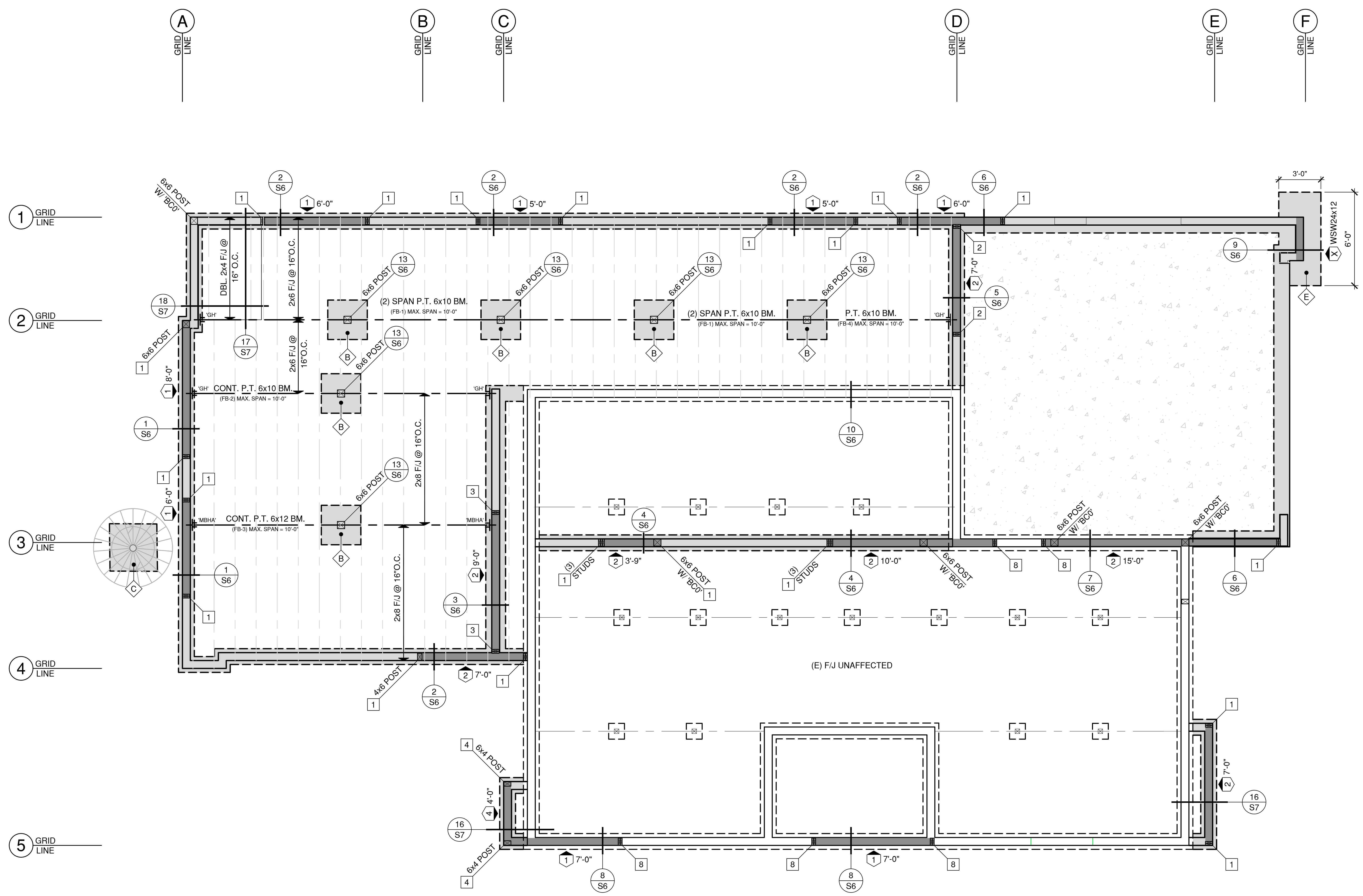
GENERAL NOTES

S1

FOR EXISTING STEM WALL FOOTINGS, EMBEDMENT REQ'D IS 12" MIN. IF THIS IS NOT MET, SEE DETAIL 14/S6

11/S6 TYP. FOR NEW CONSTRUCTION CONT. FOOTING TO EXISTING CONT. FOOTING

12/S6 TYP. FOR NEW CONSTRUCTION SLAB ON GRADE TO EXISTING CONT. FOOTING



FOUNDATION NOTES

1. REFER TO THE GENERAL STRUCTURAL NOTES SHEET (S1).
   - CONCRETE COMPRESSIVE STRENGTH AND CEMENT TYPE PER CONCRETE NOTES
   - ANCHOR BOLT STEEL TYPE PER STEEL NOTES
   - PRESSURE TREATED LUMBER REQUIREMENTS PER WOOD NOTES (WHERE REQ'D. & FINISH / COATING OF FASTENERS)
2. REFER TO THE TYPICAL STRUCTURAL DETAILS SHEET (S5).
3. CONTINUOUS FOOTINGS SHALL HAVE A MINIMUM WIDTH OF 12" AND BE EMBEDDED A MINIMUM DEPTH OF 12" BELOW LOWEST ADJACENT FINAL GRADE (U.N.O.). THERE SHALL BE A TOTAL OF (4) #4 CONTINUOUS REINFORCING BARS; (2) TOP AND (2) BOTTOM BARS AS SHOWN IN THE STRUCTURAL DETAILS (U.N.O.).
4. THE EDGE OF NEW CONTINUOUS FOOTINGS AT EXTERIOR STUD WALLS SHALL BE ALIGNED WITH THE EXTERIOR OF THE SHEATHING PER DETAILS.
5. SLABS ON GRADE SHALL BE A MINIMUM OF 4" THICK WITH #4 REBAR @ 18" O.C. EACH WAY IN THE CENTER OF THE SLAB. UNDERLAY WITH A 4" THICK LAYER OF CLEAN SAND (S.E. = 30 OR GREATER) WITH A 15 MIL. VAPOR RETARDER / BARRIER (STEGO WRAP OR EQUIVALENT) IN THE CENTER. THE MOISTURE BARRIER SHALL BE PROPERLY LAPPED AND SEALED AT JOINTS AND AROUND ANY BREAKS SUCH AS OPENINGS FOR UTILITY CONDUITS. REFER TO GEOTECHNICAL REPORT (IF APPLICABLE) FOR ADDITIONAL INFORMATION.
6. FLATWORK / HARDSCAPE SHALL BE INSTALLED IN ACCORDANCE WITH THE GEOTECHNICAL REPORT (IF APPLICABLE) OR JURISDICTIONAL STANDARDS.
7. BOTTOM OF ALL FOOTINGS SHALL MAINTAIN 7'-0" DISTANCE TO DAYLIGHT (UNLESS SPECIFIED DIFFERENTLY BY A GEOTECHNICAL REPORT). HORIZONTAL DISTANCE SHALL BE MEASURED FROM THE BOTTOM LEADING EDGE OF THE FOOTING.
8. ALL REINFORCING BARS, WIRE MESH, ANCHOR BOLTS, SLEEVES, AND OTHER CONCRETE INSERTS SHALL BE SECURED IN PLACE AND APPROVED BY THE BUILDING INSPECTOR PRIOR TO PLACING CONCRETE.
9. BEARING WALL SILL PLATES ON CONCRETE OR MASONRY SHALL HAVE ANCHOR BOLTS WITH THE FOLLOWING SPECIFICATIONS:
   - ½" Ø MIN. EMBEDDED 7" MIN. INTO CONCRETE OR MASONRY
   - PLACED 4" MIN. TO 12" MAX. FROM EACH SILL PLATE END (OR FROM NOTCH)
   - A MINIMUM OF 2 ANCHOR BOLTS PER SILL PLATE PIECE
   - MAXIMUM SPACING OF 72" O.C.
   - AT SHEAR WALL LOCATIONS, USE SPECIFICATIONS IN SHEAR WALL SCHEDULE
10. ALL NON-BEARING WALLS SHALL USE 2x P.T. SILL PLATES WITH HILTI X-CR CONCRETE FASTENERS (ESR-1663), OR EQUIVALENT, @ 32" O.C. AND 6" FROM PLATE ENDS.
11. THE STRUCTURE SHALL EITHER BE LOCATED ON COMPETENT (NATIVE) SOIL OR THE SOIL SHALL BE COMPACTED TO 90% AND BE TESTED BY A LICENSED GEOTECHNICAL ENGINEER WITH A COMPACTION REPORT SUBMITTED TO THE BUILDING OFFICIAL.
12. WHERE FILL IS REQUIRED, ALL FILL MATERIALS TO BE GRANULAR, NON-COHESIVE SOIL. ALL FILL OVER 12" IN DEPTH SHALL BE COMPACTED TO 90% AND BE TESTED BY A LICENSED GEOTECHNICAL ENGINEER WITH A COMPACTION REPORT SUBMITTED TO THE BUILDING OFFICIAL.
13. PRIOR TO THE CONTRACTOR REQUESTING A BUILDING DEPARTMENT FOUNDATION INSPECTION, THE CONTRACTOR / GEOTECHNICAL ENGINEER SHALL ADVISE THE BUILDING OFFICIAL IN WRITING THAT:
   - THE BUILDING PAD WAS PREPARED IN ACCORDANCE WITH THE GEOTECHNICAL REPORT (IF APPLICABLE) OR JURISDICTIONAL STANDARDS
   - THE UTILITY TRENCHES HAVE BEEN PROPERLY BACKFILLED AND COMPACTED
   - THE FOUNDATION EXCAVATIONS COMPLY WITH THE INTENT OF THE GEOTECHNICAL REPORT (IF APPLICABLE) OR JURISDICTIONAL STANDARDS
14. ANY STRUCTURAL ELEMENTS LABELED AS EXISTING SHALL BE FIELD-VERIFIED. NOTIFY THE ENGINEER OF RECORD OF ANY DISCREPANCIES PRIOR TO CONSTRUCTION.
15. THE ENGINEER OF RECORD IS NOT RESPONSIBLE FOR THE EXISTING CONDITIONS OR INTEGRITY OF THE EXISTING FOUNDATIONS. IT IS THE RESPONSIBILITY OF THE CONTRACTOR TO VERIFY THE SIZE OF THE EXISTING FOOTINGS AND TO NOTIFY THE ENGINEER OF RECORD OF ANY DISCREPANCIES OR PROBLEM AREAS PRIOR TO CONSTRUCTION.

GEOTECHNICAL ENGINEER OF RECORD:	LGC GEOTECHNICAL. INC. - BENJAMIN GRENIS
PROJECT NUMBER:	17032-01
DATED:	04/14/17

SYMBOLS

- # SHEAR WALL PER SCHEDULE
- # HOLDOWN PER SCHEDULE
- # PAD FOOTING PER SCHEDULE

SEE SHEET S4 FOR SCHEDULES & CORRESPONDING NOTES

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APPENDIX B

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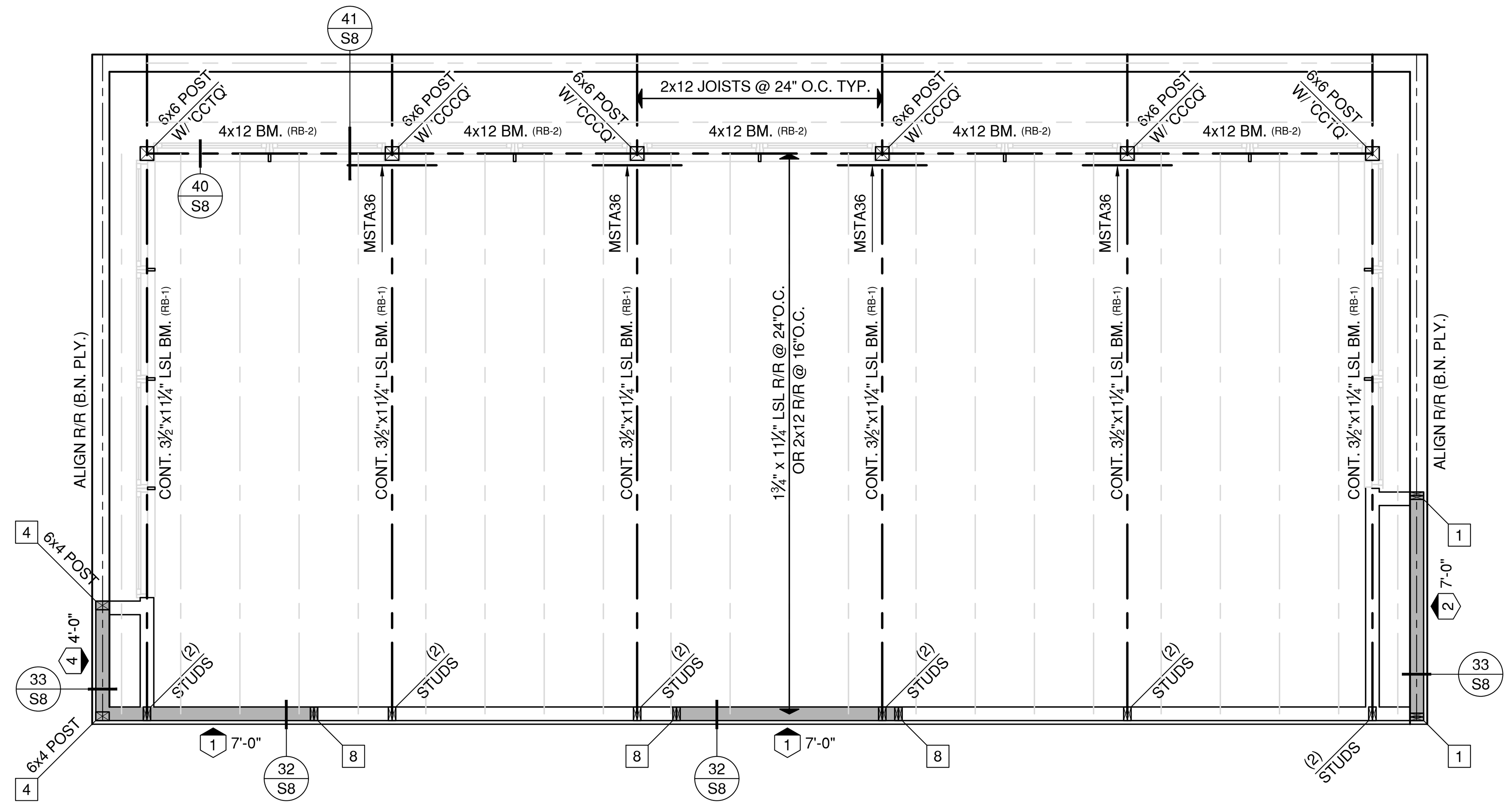
REVISIONS

PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017

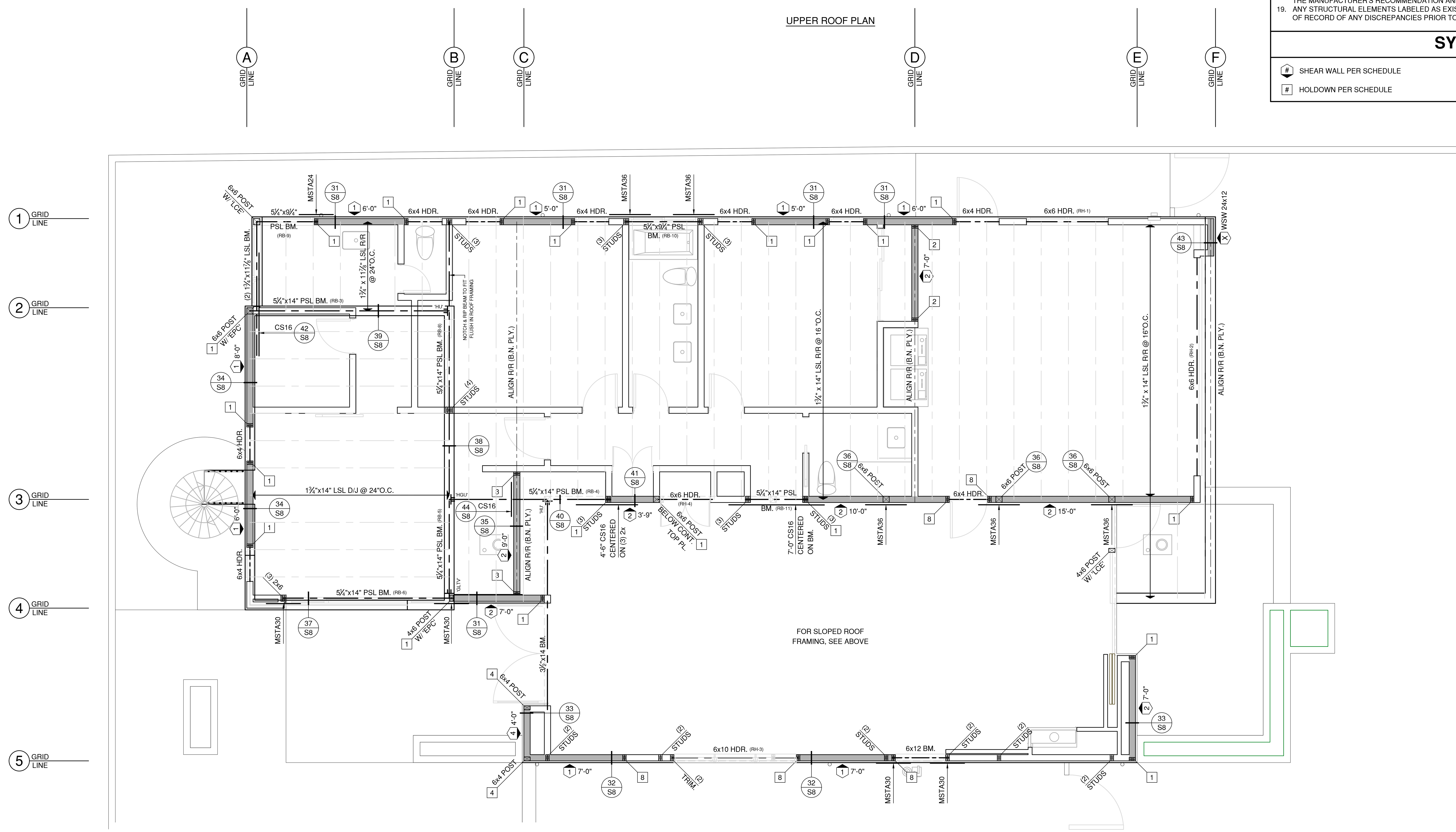
SCALE: 1/4" = 1'-0"

FOUNDATION PLAN

S2



UPPER ROOF PLAN



FRAMING NOTES

1. REFER TO THE GENERAL STRUCTURAL NOTES SHEET (S1) FOR WOOD NOTES AND FASTENING SCHEDULE.
2. REFER TO THE TYPICAL STRUCTURAL DETAILS SHEET (S5).
3. REFER TO THE STRUCTURAL SCHEDULES SHEET (S4) FOR ROOF AND FLOOR SHEATHING TYPE AND ATTACHMENT NOTES.
4. REFER TO ARCHITECTURAL, ELECTRICAL, & MECHANICAL PLANS FOR COORDINATION PRIOR TO LAYING OUT JOISTS / RAFTERS.
5. BEARING WALL STUD HEIGHTS SHALL NOT EXCEED THE FOLLOWING LIMITS:
   - 2x4 @ 16" O.C. 10'-0" MAX. PLATE HEIGHT
   - 2x4 @ 12" O.C. 12'-0" MAX. PLATE HEIGHT
   - 2x6 @ 16" O.C. 16'-0" MAX. PLATE HEIGHT
   - 2x6 @ 12" O.C. 20'-0" MAX. PLATE HEIGHT
6. NON-BEARING WALL STUD HEIGHTS SHALL NOT EXCEED THE FOLLOWING LIMITS:
   - 2x4 @ 16" O.C. 14'-0" MAX. PLATE HEIGHT
   - 2x6 @ 16" O.C. 20'-0" MAX. PLATE HEIGHT
7. ALL EXTERIOR AND/OR BEARING RAKE (SLOPING) WALLS SHALL HAVE CONTINUOUS STUDS BETWEEN FLOOR/FOUNDATION AND ROOF FRAMING.
8. ALL BEAMS SHALL BEAR ON DOUBLE TOP PLATES WITH 'A34' CONNECTORS ON EACH SIDE UNLESS A POST CAP IS SPECIFIED. WHERE NO DOUBLE TOP PLATES OCCUR, THE CAP SHALL BE 'PC' (U.N.O.).
9. ALL POST TO BOTTOM/SILL PLATE AND POST TO DOUBLE TOP PLATES SHALL HAVE 'A34' CONNECTORS ON EACH SIDE (U.N.O.). WHERE A POST BELOW IS NOT SPECIFIED, MATCH POST SIZE ABOVE.
10. PROVIDE BUILT-UP STUDS TO SUPPORT ALL BEAMS WHERE POSTS ARE NOT SPECIFIED. BUILT-UP STUDS TO MATCH WIDTH OF BEAM. SISTER TOGETHER WITH 16d @ 16" O.C.
11. PROVIDE DOUBLE JOISTS/RAFTERS AT SIDES AND ENDS OF ALL OPENINGS IN FLOOR/ROOF (U.N.O.).
12. PROVIDE DOUBLE JOISTS BELOW ALL PARALLEL INTERIOR / PARTITION WALLS 8'-0" OR GREATER IN LENGTH, WITH BLOCKING AT ONE-THIRD OF THE SPAN. PROVIDE 2x BLOCKING BELOW ALL PERPENDICULAR INTERIOR / PARTITION WALLS.
13. ALL DOUBLE JOISTS/RAFTERS SHALL BE SISTERED TOGETHER WITH 16d @ 12" O.C., STAGGERED.
14. WHERE DOUBLE TRIMMERS ARE SPECIFIED, SISTER TOGETHER WITH 10d @ 12" O.C.
15. EACH TRUSS SHALL BE LEGIBLY BRANDED, MARKED, OR OTHERWISE HAVE PERMANENTLY AFFIXED THERETO THE FOLLOWING INFORMATION LOCATED ON THE FACE OF THE BOTTOM CHORD:
    - IDENTITY OF THE TRUSS MANUFACTURER
    - DESIGN LOADS
    - SPACING OF THE TRUSS
16. PROVIDE 'ST6224' STRAP ACROSS ALL DISCONTINUOUS DOUBLE TOP PLATES (U.N.O.).
17. DO NOT CUT, NOTCH, DRILL, BORE, SHAVE, TAPER, OR MODIFY ANY WOOD OR MANUFACTURED LUMBER PRODUCTS UNLESS SUCH MODIFICATIONS ARE PER PLAN OR WITHIN THE PARAMETERS SET FORTH BY THE MANUFACTURER OF THAT PRODUCT. IN ADDITION, THE MANUFACTURER'S ENGINEER CAN PROVIDE A STAMPED LETTER ALLOWING THE MODIFICATIONS IF AUTHORIZED BY THE PROJECT ENGINEER OF RECORD AND APPROVED BY THE GOVERNING JURISDICTION.
18. FRAMING CONNECTIONS SPECIFIED ON DRAWINGS SHALL BE MANUFACTURED BY 'SIMPSON STRONG-TIE' OR AN ENGINEER APPROVED EQUIVALENT. ALL CONNECTIONS SHALL BE APPLIED IN ACCORDANCE WITH THE MANUFACTURER'S RECOMMENDATION AND SPECIFICATION TO DEVELOP THE MAXIMUM CAPACITY.
19. ANY STRUCTURAL ELEMENTS LABELED AS EXISTING SHALL BE FIELD-VERIFIED. NOTIFY THE ENGINEER OF RECORD OF ANY DISCREPANCIES PRIOR TO CONSTRUCTION.

SYMBOLS

\# SHEAR WALL PER SCHEDULE

\# HOLDOWN PER SCHEDULE

SEE SHEET S4 FOR SCHEDULES & CORRESPONDING NOTES

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REVISIONS

PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017

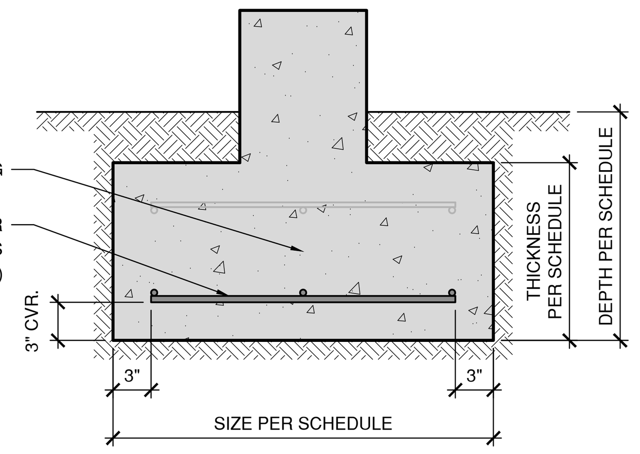
SCALE: 1/4" = 1'-0"

ROOF FRAMING PLAN

S3

SYMBOL	SIZE	DEPTH	THICKNESS (WHERE APPLICABLE)	REINFORCEMENT
A	24" SQUARE	12"	8"	#4 BOTTOM BARS @ 12" O.C. EACH WAY
B	30" SQUARE	12"	8"	#4 BOTTOM BARS @ 12" O.C. EACH WAY
C	36" SQUARE	12"	8"	#4 BOTTOM BARS @ 12" O.C. EACH WAY
D	42" SQUARE	18"	12"	#4 TOP & BOTTOM BARS @ 12" O.C. EACH WAY
E	PER PLAN	18"	12"	#4 TOP & BOTTOM BARS @ 12" O.C. EACH WAY

1. DEPTH OF FOOTING SHALL BE MEASURED BELOW LOWEST ADJACENT FINAL GRADE. WHERE SPECIFIED, USE DEPTH CALLED OUT ON THE PLAN.
2. REINFORCEMENT SHALL MAINTAIN 3" CLEAR DISTANCE FROM SOIL.
3. THERE SHALL BE A BAR 3" FROM EACH EDGE OF THE PAD FOOTING WITH SPACING PER SCHEDULE IN BETWEEN.
4. CENTER PAD FOOTING ON COLUMN / POST ABOVE (WHERE APPLICABLE).

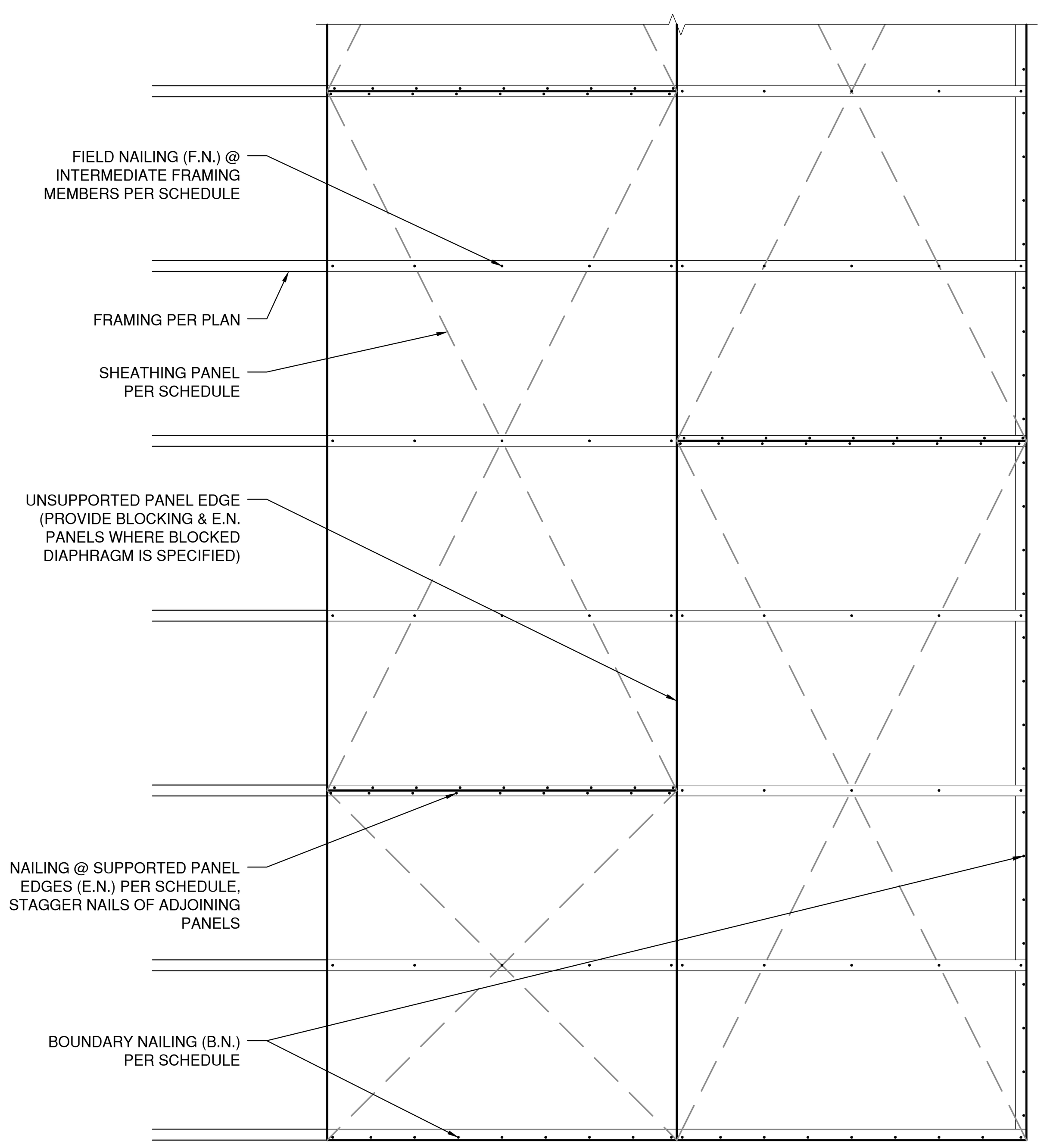


TYPICAL DIMENSIONS & REBAR LAYOUT

PAD FOOTING SCHEDULE & TYPICAL DETAILS

LOCATION	THICKNESS	SPAN RATING	MAX. RAFTER / JOIST SPACING	NAIL TYPE	NAIL SPACING @ DIAPHRAGM BOUNDARIES (B.N.) & SUPPORTED PANEL EDGES (E.N.)
ROOF	15⁄32"	32/16	32" O.C.	8d	6" O.C.
FLOOR	3⁄4"	24" O.C.	24" O.C.	10d	6" O.C.

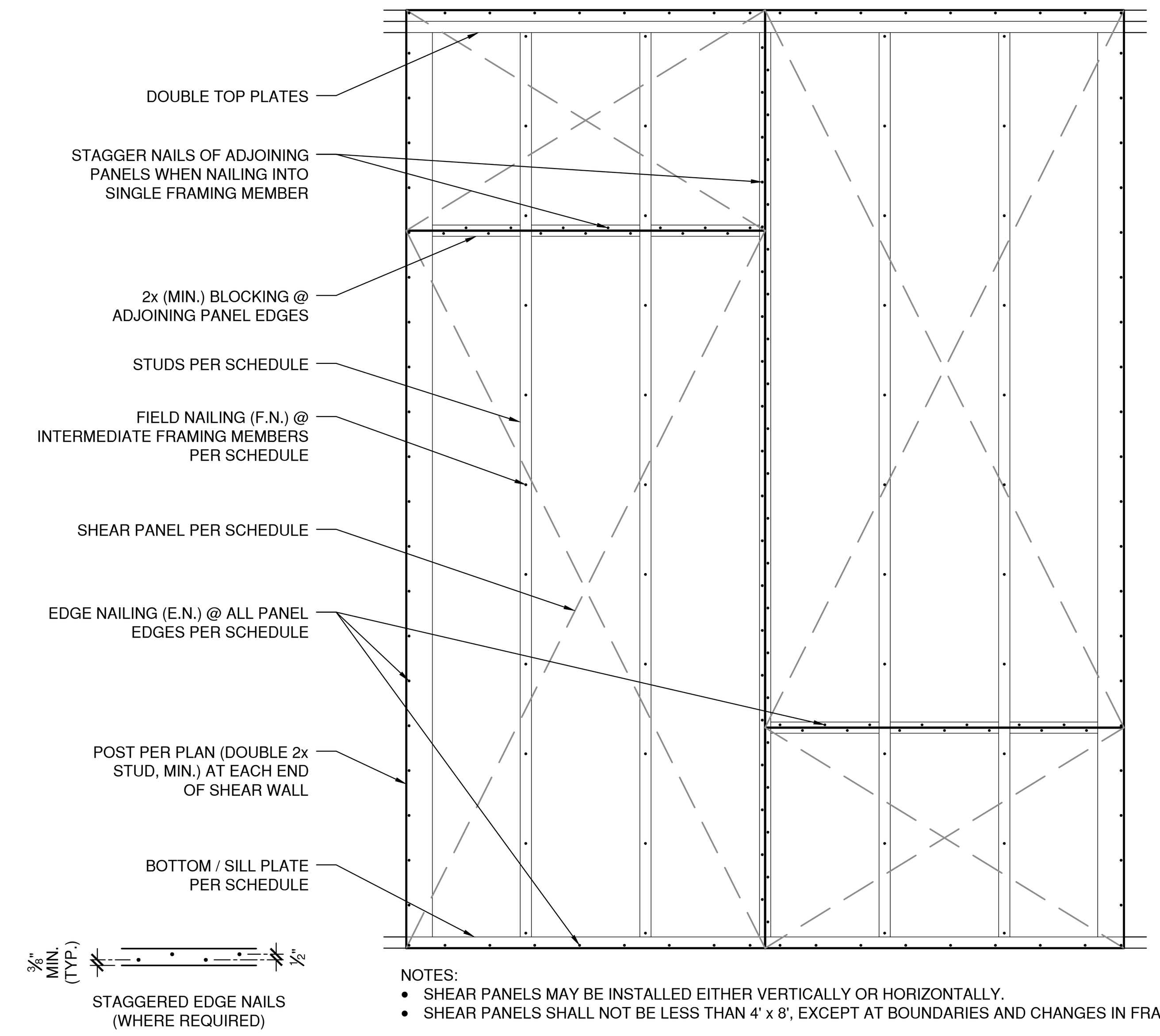
1. ALL SHEATHING SHALL BE APA RATED, EXPOSURE 1.
2. FLOOR SHEATHING SHALL BE T&G STURD-I-FLOOR AND SHALL BE GLUED AND NAILED.
3. PLYWOOD OR OSB CAN BE USED.
4. THICKER SHEATHING THAN INDICATED SHALL NOT BE USED WITHOUT WRITTEN CONSENT FROM THE ENGINEER OF RECORD AS NAILS SIZE MAY NEED TO BE ALTERED.
5. NAILS AT INTERMEDIATE FRAMING MEMBERS (F.N.) SHALL BE THE SAME SIZE AS INDICATED IN CHART AND BE SPACED @ 12" O.C.
6. ONLY COMMON NAILS SHALL BE USED. NAILS SHALL BE DRIVEN WITH THE HEAD OF THE NAIL FLUSH WITH THE SURFACE OF THE SHEATHING.
7. NAILS SHALL BE LOCATED AT LEAST 3⁄8" FROM THE EDGES OF PANELS.



NOTES:
- PANELS SHALL BE INSTALLED WITH THE LONG DIMENSION PERPENDICULAR TO FRAMING.
- PANEL JOINTS PARALLEL TO FRAMING MEMBERS SHALL BE STAGGERED.
- PANELS SHALL NOT BE LESS THAN 4' x 8' EXCEPT AT BOUNDARIES AND CHANGES IN FRAMING WHERE MINIMUM PANEL DIMENSIONS SHALL BE 24".
- BOUNDARY NAILING (B.N.) SHALL BE PROVIDED @ ALL BEARING WALLS & FLUSH BEAMS / DRAG MEMBERS.
- WHERE A BLOCKED DIAPHRAGM IS SPECIFIED ON PLANS, USE 2x4 FLAT BLOCKING AND E.N. PANELS. USE SIMPSON 'Z' CLIPS TO ATTACH BLOCKING TO FRAMING MEMBERS.

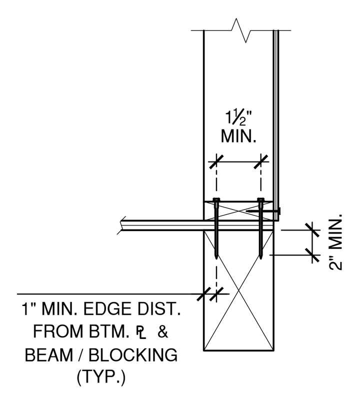
SHEATHING LAYOUT & NAILING

ROOF / FLOOR SHEATHING SCHEDULE & TYPICAL DETAILS

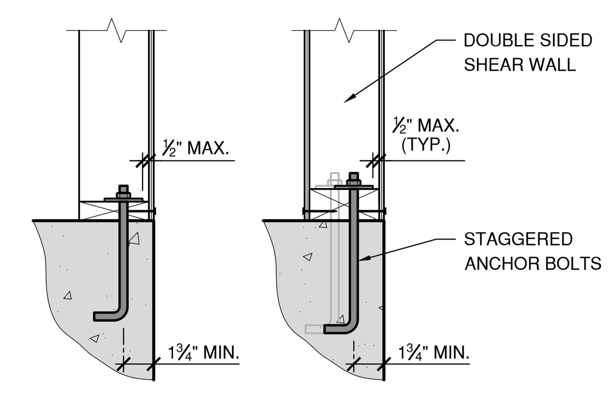


NOTES:
- SHEAR PANELS MAY BE INSTALLED EITHER VERTICALLY OR HORIZONTALLY.
- SHEAR PANELS SHALL NOT BE LESS THAN 4' x 8', EXCEPT AT BOUNDARIES AND CHANGES IN FRAMING.

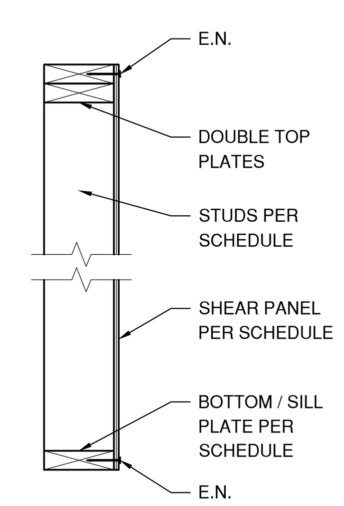
SHEAR PANEL LAYOUT & NAILING



2 ROWS OF SDWS SCREWS (WHERE REQUIRED)



ANCHOR BOLT PLACEMENT (TYPICAL OF NEW & RETROFIT BOLTS)



TYPICAL FRAMING @ SHEAR WALL

SYMBOL	CAPACITY (PLF)	SHEAR PANEL & EDGE NAILING (SEE NOTE 1-7)	SILL PLATE & ANCHOR BOLTS (SEE NOTE 12, 15-17)	BOTTOM PLATE & NAILS / SCREWS (SEE NOTE 11-14)	SHEAR TRANSFER HARDWARE TO TOP PLATES (SEE NOTE 10)
1	260 - SEIS. 261 - WIND	3⁄8" CDX PLYWOOD W/ 8d @ 6" O.C.	2x P.T. PLATE W/ ½" Ø A.B. (OR 5⁄8" Ø A.B.) @ 36" O.C.	2x PLATE W/ 16d @ 8" O.C. (OR SDWS SCREWS @ 12" O.C.)	A35 OR LTP4 @ 24" O.C.
2	348 - SEIS. 348 - WIND	3⁄8" CDX PLYWOOD W/ 8d @ 4" O.C.	2x P.T. PLATE W/ ½" Ø A.B. (OR 5⁄8" Ø A.B.) @ 24" O.C.	2x PLATE W/ 16d @ 6" O.C. (OR SDWS SCREWS @ 12" O.C.)	A35 OR LTP4 @ 16" O.C.
3	490 - SEIS. 522 - WIND	3⁄8" CDX PLYWOOD W/ 8d @ 3" O.C. (STAGGERED) (SEE NOTE 8)	2x P.T. PLATE W/ 5⁄8" Ø A.B. @ 20" O.C.	2x PLATE W/ 16d @ 4" O.C. (OR SDWS SCREWS @ 10" O.C.)	A35 OR LTP4 @ 12" O.C.
4	640 - SEIS. 660 - WIND	3⁄8" CDX PLYWOOD W/ 8d @ 2" O.C. (STAGGERED) (SEE NOTE 8)	2x P.T. PLATE W/ 5⁄8" Ø A.B. @ 16" O.C.	2x PLATE W/ 16d @ 3" O.C. (OR SDWS SCREWS @ 8" O.C.)	A35 OR LTP4 @ 10" O.C.
5	730 - SEIS. 756 - WIND	3⁄8" STRUCTURAL 1 PLYWOOD W/ 8d @ 2" O.C. (STAGGERED) (SEE NOTE 8)	2x P.T. PLATE W/ 5⁄8" Ø A.B. @ 14" O.C.	2x PLATE W/ SDWS SCREWS @ 6" O.C.	A35 OR LTP4 @ 9" O.C.
6	870 - SEIS. 880 - WIND	15⁄32" STRUCTURAL 1 PLYWOOD W/ 10d @ 2" O.C. (STAGGERED) (SEE NOTE 8)	2x P.T. PLATE W/ 5⁄8" Ø A.B. @ 12" O.C.	2x PLATE W/ SDWS SCREWS @ 6" O.C.	A35 OR LTP4 @ 8" O.C.
7	980 - SEIS. 1025 - WIND	3⁄8" CDX PLYWOOD EACH SIDE W/ 8d @ 3" O.C. (STAGGERED) (SEE NOTE 9)	3x6 (MIN.) P.T. PLATE W/ 5⁄8" Ø A.B. @ 10" O.C.	3x6 (MIN.) PLATE W/ SDWS SCREWS @ 5" O.C. (STAGGERED - 2 ROWS @ 10" O.C.) INTO 4x (MIN.) BEAM / BLOCKING	A35 OR LTP4 @ 6" O.C.
8	1100 - SEIS. 1142 - WIND	3⁄8" STRUCTURAL 1 PLYWOOD EACH SIDE W/ 8d @ 3" O.C. (STAGGERED) (SEE NOTE 9)	3x6 (MIN.) P.T. PLATE W/ 5⁄8" Ø A.B. @ 10" O.C. (TITEN-HD NOT ACCEPTABLE)	3x6 (MIN.) PLATE W/ SDWS SCREWS @ 4" O.C. (STAGGERED - 2 ROWS @ 8" O.C.) INTO 4x (MIN.) BEAM / BLOCKING	A35 OR LTP4 @ 6" O.C.
X	N/A	PREFABRICATED SHEAR WALL (SEE PLANS FOR SHEAR WALL TYPE AND ATTACHMENT SPECIFICATIONS)			

* WHEN AN ASTERISK (*) ACCOMPANIES THE SHEAR WALL SYMBOL, THE SHEAR PANEL IS TO BE CONTINUOUS THROUGH ADJACENT WALL FRAMING.

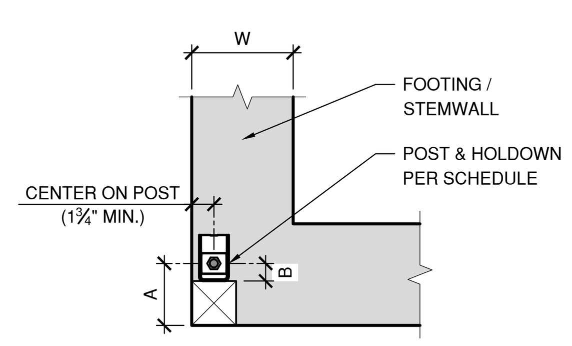
1. SHEAR PANELS SHALL BE APPLIED DIRECTLY TO STUD FRAMING.
2. ALL EDGES OF ALL SHEAR PANELS SHALL BE SUPPORTED BY AND FASTENED TO FRAMING MEMBERS OR BLOCKING. PANELS MORE THAN ONE VERTICAL PANEL IN HEIGHT SHALL HAVE STAGGERED, SPLICED JOINTS. ALL SPLICED JOINTS SHALL BE BLOCKED WITH 2x BLOCKING (MIN.).
3. OSB OF THE SAME TYPE AS SPECIFIED CAN BE USED IN LIEU OF PLYWOOD.
4. ONLY COMMON OR GALVANIZED BOX NAILS SHALL BE USED FOR SHEAR PANEL ATTACHMENT. GALVANIZED NAILS SHALL BE HOP DIPPED OR TUMBLED. NAILS SHALL BE DRIVEN WITH THE HEAD OF THE NAIL FLUSH WITH THE SURFACE OF THE SHEATHING.
5. ALL SHEAR PANEL NAILS AT INTERMEDIATE FRAMING MEMBERS (F.N.) SHALL BE THE SAME SIZE AS NAILS SPECIFIED FOR EDGE NAILING AND SPACED AT 12" O.C.
6. PROVIDE 3⁄8" MIN. EDGE DISTANCE FOR ALL SHEAR PANEL EDGE NAILING.
7. STUDS SHALL BE 2x (MIN.) AND SPACED AT 16" O.C. (MAX.).
8. FRAMING AT ADJOINING SHEAR PANEL EDGES SHALL BE 3x (MIN.) AND NAILS SHALL BE STAGGERED. DOUBLE 2x MEMBERS ARE ALLOWED FOR SHEAR WALL TYPES 3 & 4 AND SHALL BE SISTERED W/ 10d @ 4" O.C.
9. WHERE SHEAR PANELS ARE APPLIED TO BOTH FACES OF A WALL, PANEL JOINTS SHALL BE OFFSET TO FALL ON DIFFERENT FRAMING MEMBERS OR FRAMING MEMBERS SHALL BE 3x (MIN.) AT ADJOINING PANEL EDGES AND NAILS SHALL BE STAGGERED.
10. WHERE LTP4 IS INSTALLED OVER SHEATHING (½" THICK, MAX.), 8d COMMONS SHALL BE USED.
11. WHERE NAILS ARE SPECIFIED FOR BOTTOM PLATE ATTACHMENT, ONLY COMMON NAILS SHALL BE USED.
12. SEE WOOD NOTES (SHEET S1) FOR ATTACHMENT INTO PRESSURE TREATED LUMBER.
13. WHERE BOTTOM PLATE NAILING GOES THROUGH FLOOR SHEATHING THICKER THAN 3⁄4", USE SDWS SCREWS ONLY.
14. SDWS SCREWS SHALL PENETRATE 2" MINIMUM INTO RIM / BLOCKING. WHERE STAGGERED SDWS SCREWS ARE CALLED OUT, THEY SHALL BE IN 2 ROWS WITH 1½" MINIMUM SPACING BETWEEN ROWS AND 1" FROM EACH EDGE OF RIM / BLOCKING.
15. ANCHOR BOLTS SHALL BE EMBEDDED 7" (MIN.) INTO NEW POURED IN-PLACE CONCRETE. AT EXISTING CONCRETE, RETROFIT ANCHOR BOLTS SHALL BE 5⁄8" Ø THREADED RODS WITH 5" (MIN.) EMBEDMENT INSTALLED USING SIMPSON'S 'SET-XP' EPOXY (ESR-2508) WITH SPECIAL INSPECTION. ALTERNATIVELY, RETROFIT ANCHOR BOLTS CAN BE 5⁄8" Ø TITEN-HD ANCHORS (ESR-2713) WITH 4½" MINIMUM EMBEDMENT; SPECIAL INSPECTION REQUIRED.
16. FOUNDATION ANCHOR BOLTS (NEW AND RETROFIT) SHALL HAVE A STEEL PLATE WASHER UNDER EACH NUT NOT LESS THAN 0.229"x3"x3" IN SIZE. THE HOLE IN THE PLATE WASHER IS PERMITTED TO BE DIAGONALLY SLOTTED WITH A WIDTH OF UP TO 3⁄16" LARGER THAN THE BOLT DIAMETER AND A SLOT LENGTH NOT TO EXCEED 1¾", PROVIDED A STANDARD CUT WASHER IS PLACED BETWEEN THE PLATE WASHER AND THE NUT. THE PLATE WASHER SHALL EXTEND TO WITHIN ½" OF THE EDGE OF THE SILL PLATE ON THE SIDE(S) WITH THE SHEAR PANEL.
17. WELDED STEEL STUDS SHALL BE USED WHERE THE SHEAR WALL STACKS OVER A STEEL BEAM. THE STEEL STUD DIAMETER AND SPACING SHALL MATCH THAT OF THE ANCHOR BOLTS SHOWN IN THE SCHEDULE.

SHEAR WALL SCHEDULE & TYPICAL DETAILS

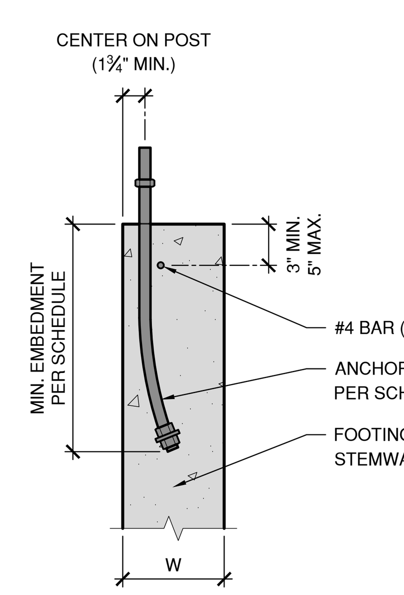
MINIMUM DIMENSIONS

ANCHOR BOLT	A	W
SB5⁄8x24	4¼"	6"
SB7⁄8x24	4¼"	8"
SB1x30	5"	8"
SSTB16	5"	6"
SSTB20	5"	6"
SSTB24	5"	6"
SSTB28	5"	8"

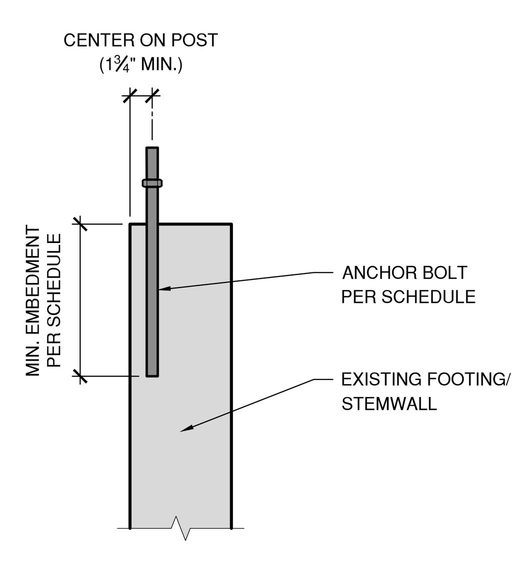
HOLDOWN	B
HDU2	15⁄16"
HDU4	15⁄16"
HDU5	15⁄16"
HDU8	13⁄8"
HDQ8	2½"
HHDQ11	3½"



PLAN VIEW



CAST-IN-PLACE ANCHOR BOLT



RETROFIT ANCHOR BOLT

HOLDOWN SCHEDULE NOTES:
1. CONCRETE AT ANCHOR BOLT SHALL BE A SINGLE POUR (NO COLD JOINT), UNLESS SPECIFICALLY DETAILED OTHERWISE.
2. BOTTOM OF ANCHOR BOLT SHALL HAVE 3" MINIMUM CONCRETE COVER TO SOIL. WHERE FOOTING NEEDS TO BE DEEPENED TO ACCOMMODATE THIS, DEEPENED FOOTING SHALL EXTEND THE LENGTH OF THE SHEAR WALL AND 12" MINIMUM BEYOND THE ANCHOR BOLTS AT EACH END.
3. SSTBL SHALL BE SUBSTITUTED FOR SSTB AT 3x SILL PLATES.
4. RETROFIT ANCHOR BOLTS INTO CONCRETE SHALL USE SIMPSON'S 'SET-XP' EPOXY (ESR-2508) WITH SPECIAL INSPECTION. RETROFIT ANCHOR BOLTS INTO MASONRY SHALL USE SIMPSON'S 'SET' EPOXY (ESR-1772) WITH SPECIAL INSPECTION.
5. AT RAISED WOOD FLOOR FOUNDATION, USE A 'CNW' COUPLER NUT TO ATTACH ANCHOR BOLT TO AN F1554, GRADE 36 THREADED ROD TO EXTEND AND ATTACH TO HOLDOWN.
6. DOUBLE 2x POST SHALL BE ATTACHED WITH 10d @ 6" O.C., STAGGERED (U.N.O.).
7. WHERE SPECIFIED, USE POST SIZE AS CALLED OUT ON PLANS.
8. WHERE EQUAL STRAP LENGTH ON POST AND BEAM IS NOT AVAILABLE DUE TO BEAM DEPTH, STRAP SHALL BE INSTALLED WITH THE BOTTOM OF THE STRAP FLUSH WITH THE BOTTOM OF THE BEAM.

SYMBOL	HOLDOWN	ANCHOR BOLT (SEE NOTE 1-5)			POST SIZE (SEE NOTE 6,7)			
					2x4 WALL			2x6 (MIN.) WALL
		TYPE		EMBEDMENT (MIN.)	8'-0" ℄ (MAX.)	10'-0" ℄ (MAX.)	12'-0" ℄ (MAX.)	12'-0" ℄ (MAX.)
1	HDU2	OR	SB5⁄8x24 / SSTB16	18" / 125⁄8"	DBL. 2x	DBL. 2x	DBL. 2x	DBL. 2x
2	HDU4	OR	SB5⁄8x24 / SSTB20	18" / 165⁄8"	DBL. 2x	DBL. 2x	6x	DBL. 2x
3	HDU5	OR	SB5⁄8x24 / SSTB24	18" / 205⁄8"	4x	4x	6x	DBL. 2x
4	HDU8	OR	SB7⁄8x24 / SSTB28	18" / 247⁄8"	6x	6x	8x	4x
5	HDQ8	OR	SB7⁄8x24 / SSTB28	18" / 247⁄8"	6x	8x	8x PSL	6x
6	HHDQ11	SB1x30		24"	8x	6x PSL	8x PSL	6x
7	HDU2	5⁄8" Ø THREADED ROD		10"	DBL. 2x	DBL. 2x	DBL. 2x	DBL. 2x
8	HDU4	5⁄8" Ø THREADED ROD		12"	DBL. 2x	DBL. 2x	4x6	DBL. 2x
9	HDU8	7⁄8" Ø THREADED ROD		15"	6x	6x	6x	4x

SYMBOL	HOLDOWN	POST SIZE (SEE NOTE 6,7)				FRAMING BELOW (SIZE PER PLAN)	NOTES
		2x4 WALL			2x6 (MIN.) WALL		
		8'-0" ℄ (MAX.)	10'-0" ℄ (MAX.)	12'-0" ℄ (MAX.)	12'-0" ℄ (MAX.)		
10	MSTC40	DBL. 2x	DBL. 2x	DBL. 2x	DBL. 2x	POST	• STRAP LENGTH SHALL BE EQUAL ON EACH POST
11	MSTC52	DBL. 2x	DBL. 2x	4x	DBL. 2x	POST	• STRAP LENGTH SHALL BE EQUAL ON EACH POST
12	MSTC66	4x	4x	6x	DBL. 2x	POST	• STRAP LENGTH SHALL BE EQUAL ON EACH POST
13	CMST14	4x	6x	8x	4x	POST	• STRAP LENGTH ON EACH POST SHALL BE 30" MIN. • EACH POST SHALL HAVE (28) 16d OR (33) 10d
14	MSTC28	DBL. 2x	DBL. 2x	DBL. 2x	DBL. 2x	FLUSH BEAM	• STRAP LENGTH SHALL BE EQUAL ON POST & BEAM (WHERE EQUAL LENGTH IS NOT AVAILABLE DUE TO BEAM DEPTH, SEE NOTE 8)
15	MSTC66B3	DBL. 2x	DBL. 2x	4x	DBL. 2x	FLUSH BEAM	• FOR 10" DEEP BEAM, USE MSTC48B3
16	(2) ST6224	6x	6x	6x	6x	FLUSH BEAM	• STRAP LENGTH SHALL BE EQUAL ON POST & BEAM (WHERE EQUAL LENGTH IS NOT AVAILABLE DUE TO BEAM DEPTH, SEE NOTE 8)
17	HST3	6x	6x	8x	6x	FLUSH BEAM	• BEAM MUST BE PSL (OR EQUIV.) • STRAP LENGTH SHALL BE EQUAL ON POST & BEAM (WHERE EQUAL LENGTH IS NOT AVAILABLE DUE TO BEAM DEPTH, SEE NOTE 8)
18	MSTC40	DBL. 2x	DBL. 2x	DBL. 2x	DBL. 2x	HEADER / DROPPED BEAM	• HEADER MUST BE DIRECTLY BELOW TOP PLATES • STRAP LENGTH SHALL BE EQUAL ON POST & BEAM (WHERE EQUAL LENGTH IS NOT AVAILABLE DUE TO BEAM DEPTH, SEE NOTE 8)
19	MSTC66B3	DBL. 2x	DBL. 2x	4x	DBL. 2x	HEADER / DROPPED BEAM	• HEADER MUST BE DIRECTLY BELOW TOP PLATES • FOR 10" DEEP HEADER/BEAM, USE MSTC48B3

HOLDOWN SCHEDULE & TYPICAL DETAILS

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APPENDIX B

JAMES GLOVER HOME, INC.
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BEAUCHEMIN RESIDENCE

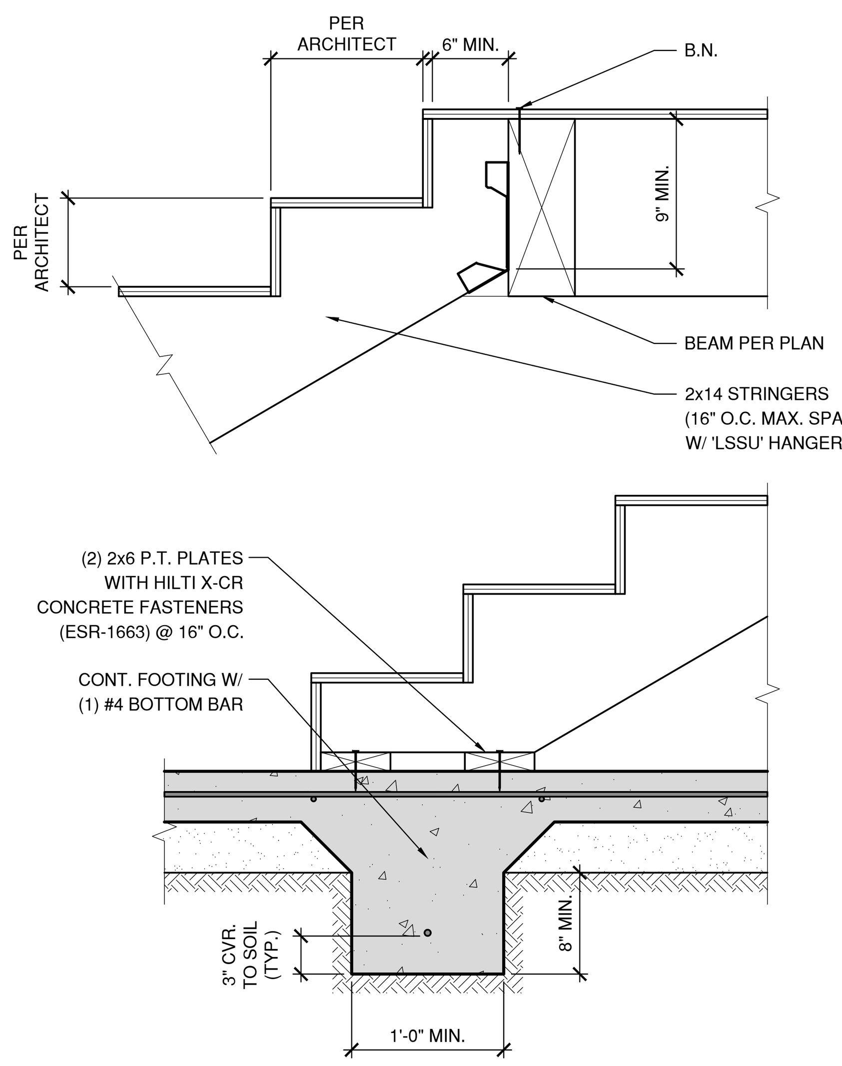
148 W. AVENIDA CADIZ SAN CLEMENTE CA 92672

REVISIONS

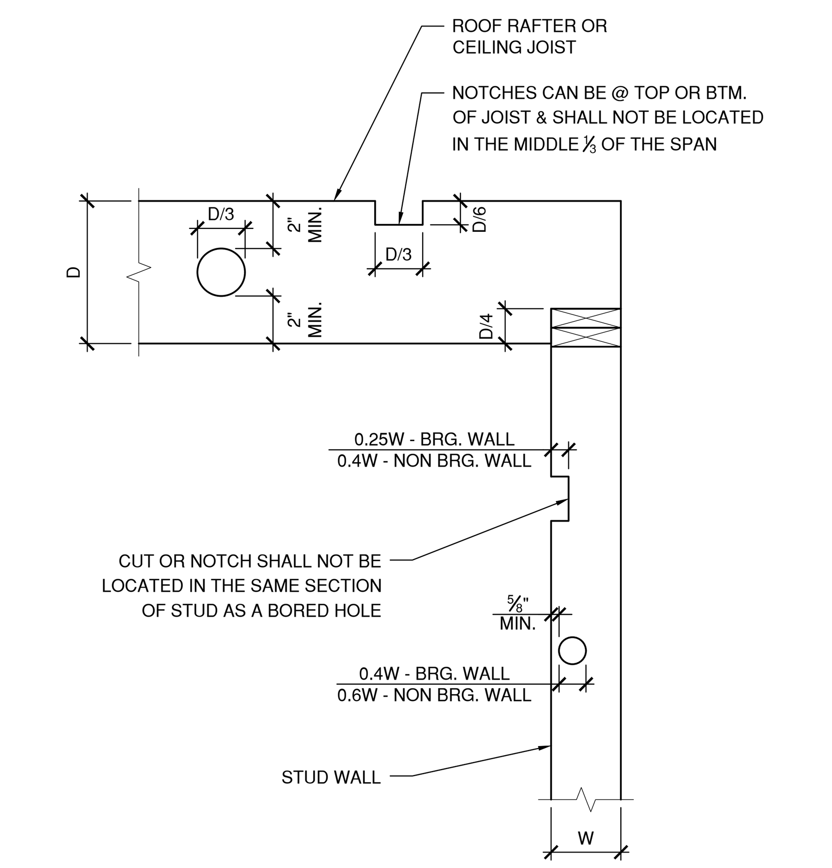
PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017
SCALE: N.T.S.

STRUCTURAL SCHEDULES

S4

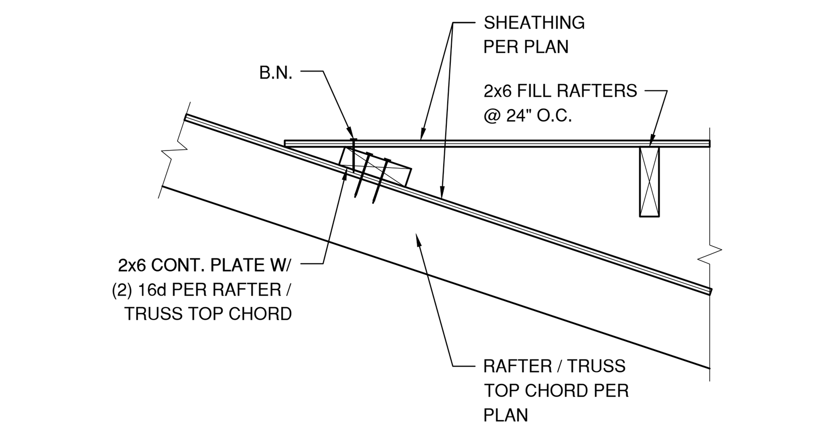


STAIR FRAMING 13



NOTES:
- ALLOWABLE NOTCHES AND BORED HOLES SHOWN ARE FOR SAWN LUMBER ONLY.
- FOR MANUFACTURED LUMBER PRODUCTS, SEE MANUFACTURER'S SPECIFICATIONS.

NOTCHES & BORED HOLES 10



NOTES:
- PROVIDE INTERMEDIATE SUPPORT FOR 2x6 FILL RAFTERS @ 6'-0" O.C., USE 2x4 VERTICAL BRACING TO 2x4 PLATE(S).
- WHERE FILL FRAMING IS PLACED OVER EXISTING ROOF, REMOVE THE EXISTING ROOFING MATERIAL PRIOR TO CONSTRUCTING NEW FILL ROOF.

'CALIFORNIA FILL' FRAMING 7

MINIMUM BEND DIAMETERS

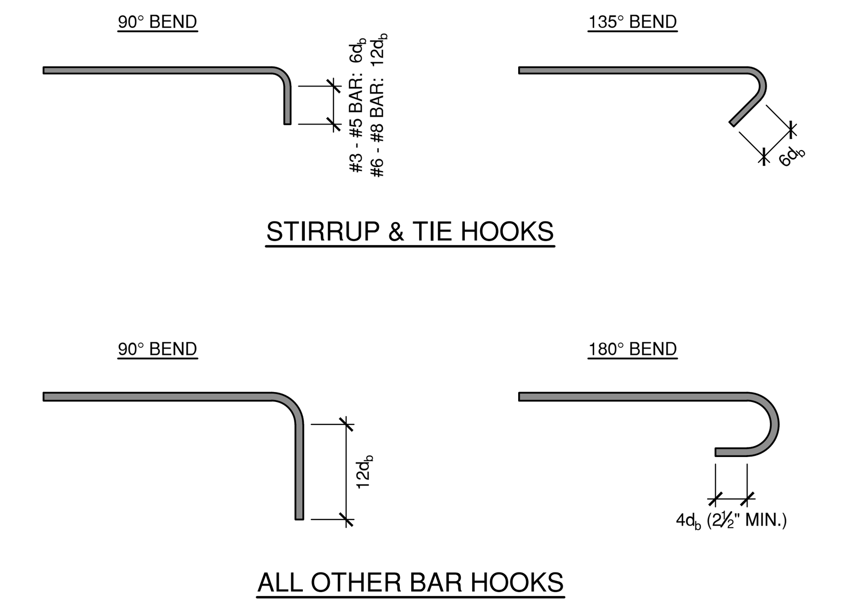
STIRRUPS & TIES	#3 - #5 BARS: $4d_b$
	#6 - #8 BARS: $6d_b$
ALL OTHER BARS	#3 - #8 BARS: $6d_b$
	#9 - #11 BARS: $8d_b$

BEND DIAMETER IS MEASURED FROM THE INSIDE OF THE BAR

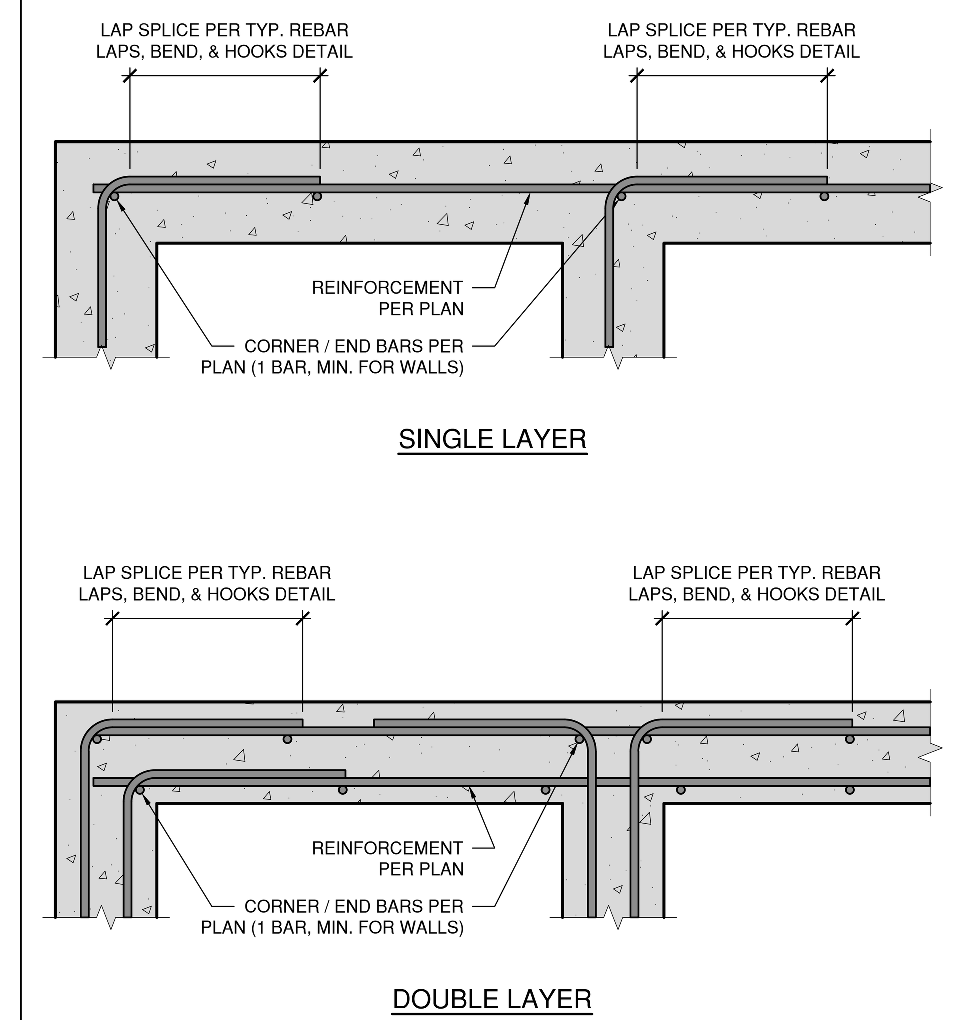
MINIMUM LAP SPLICE

BAR	CONCRETE			MASONRY
	2,500 PSI	3,000 PSI	4,500 PSI	
#3	12"	12"	12"	30"
#4	24"	22"	18"	36"
#5	30"	28"	23"	42"
#6	36"	33"	27"	48"
#7	53"	48"	40"	54"
#8	60"	55"	45"	60"

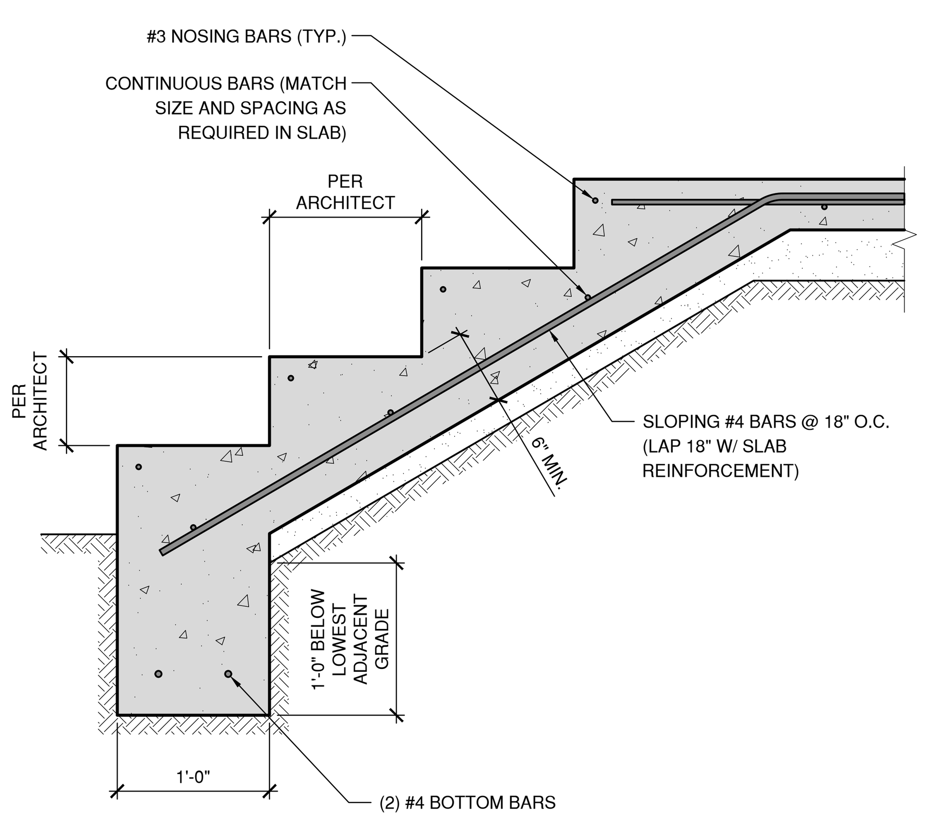
FOR EPOXY COATED BARS, INCREASE BY 20% IN CONCRETE & 50% IN MASONRY



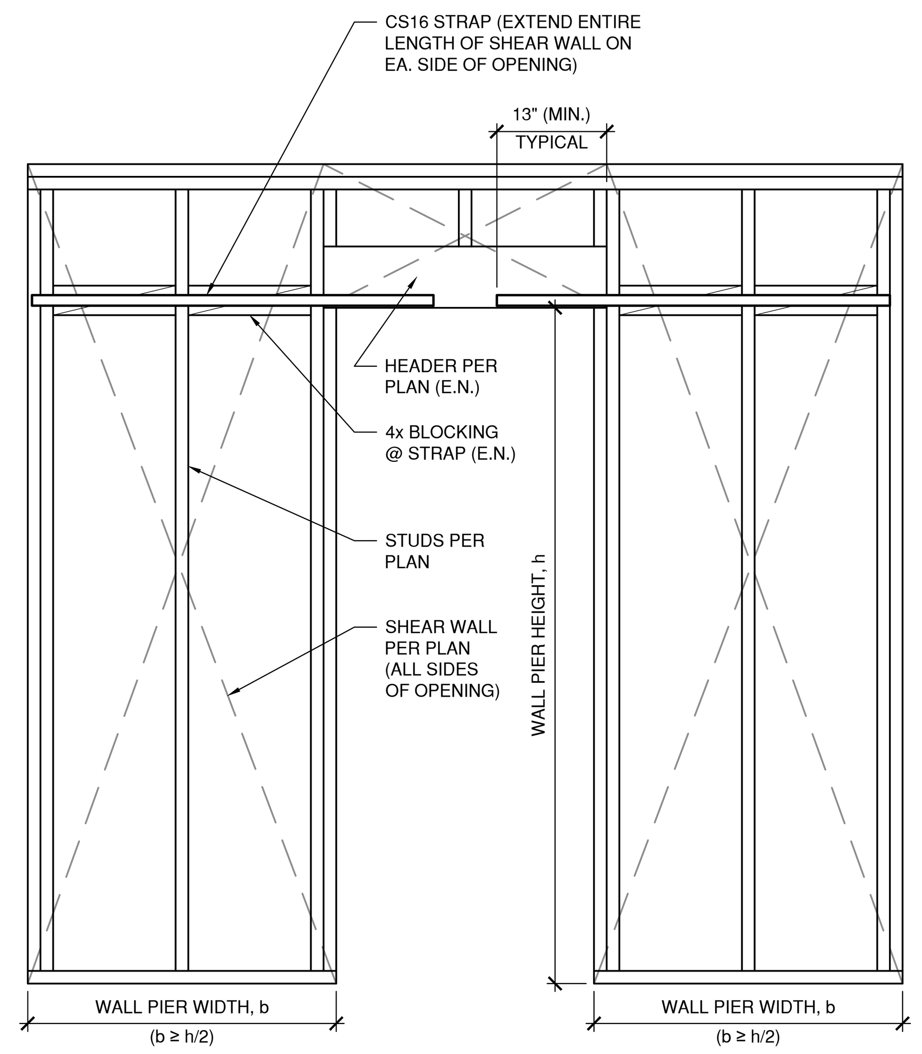
TYP. REBAR LAPS, BENDS, & HOOKS 4



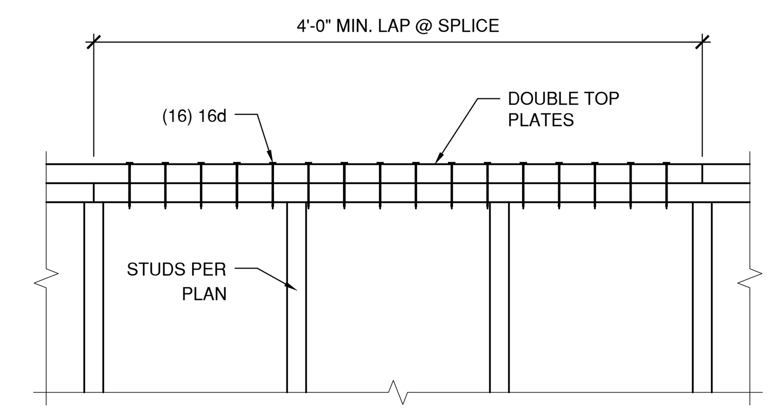
TYP. FTG. / WALL INTERSECTION 1



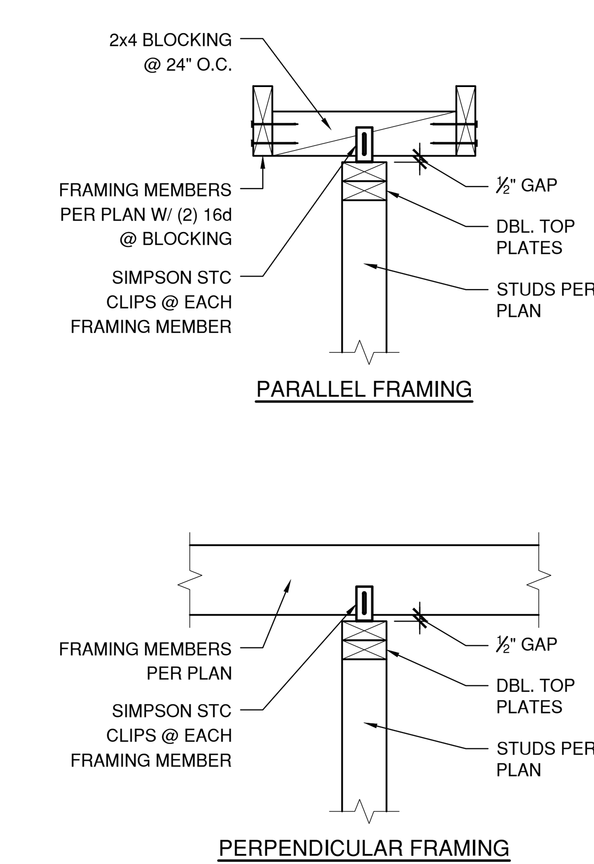
STAIRS ON GRADE 14



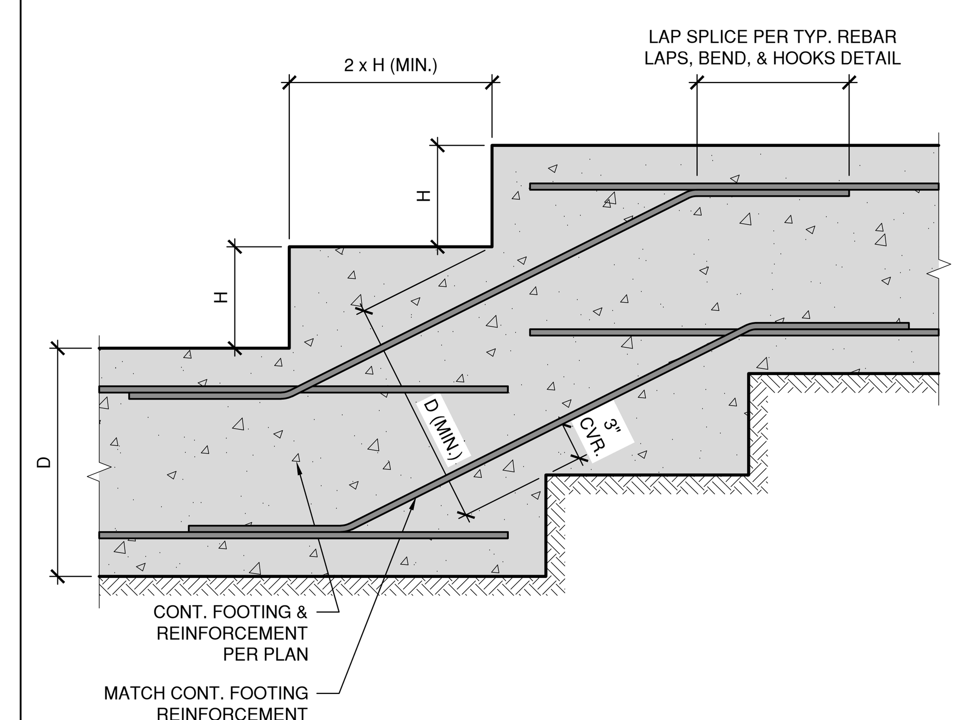
DOOR OPENING



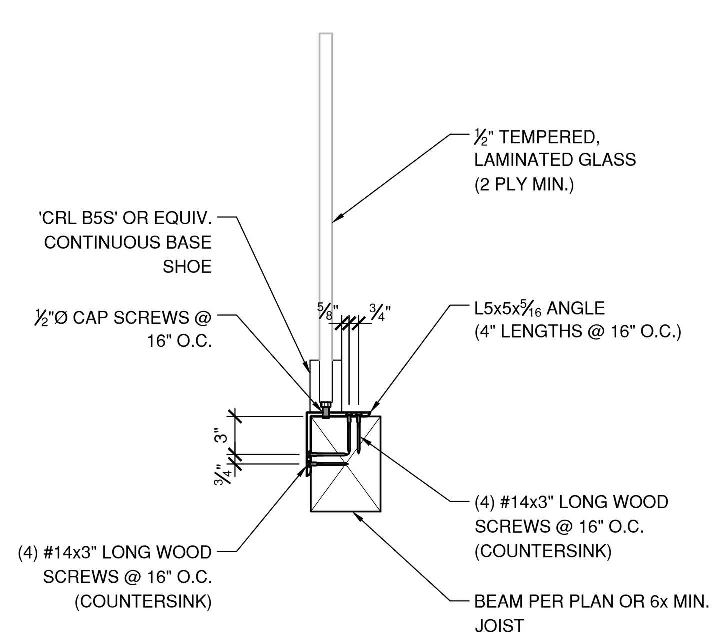
TOP PLATE SPLICE 8



NON-BEARING WALL 5

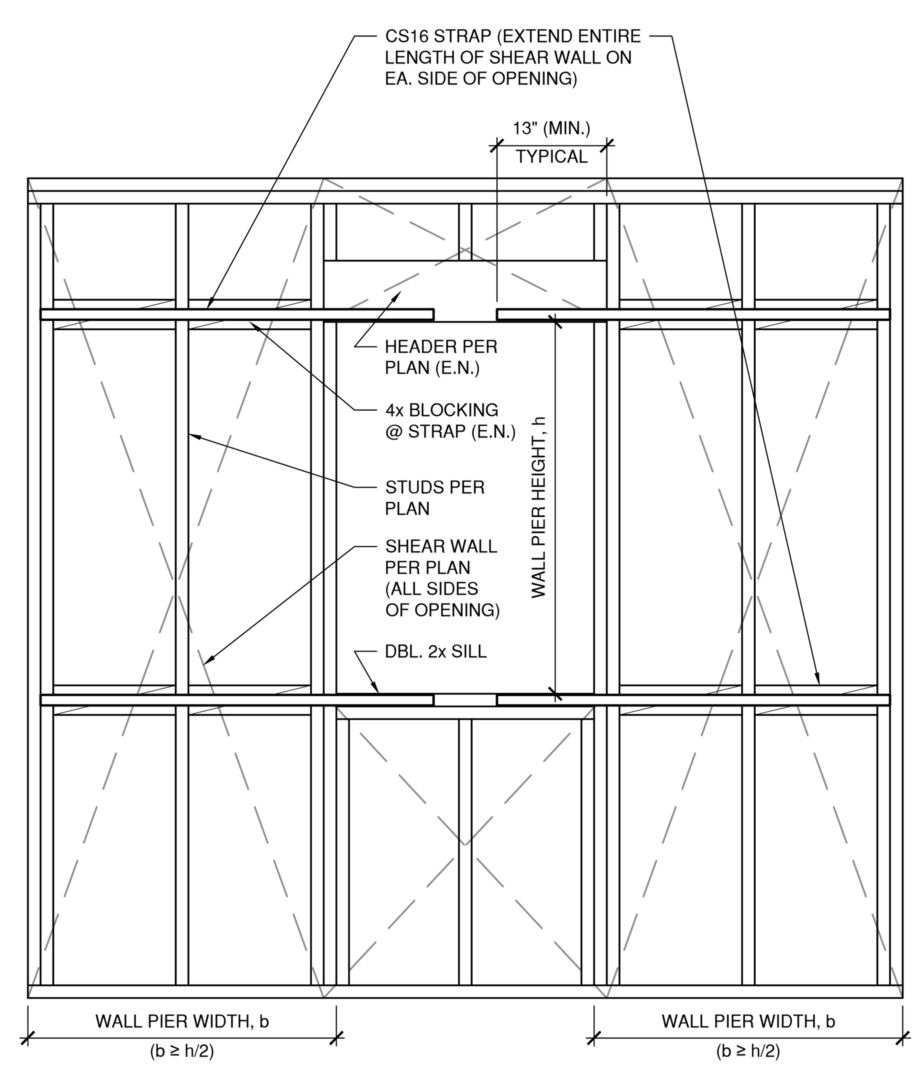


STEPPED FOOTING 2



ABOVE CONNECTION PER "FIGURE 4" OF CR LAURENCE ICC-ES REPORT (ESR-3269)

GLASS GUARDRAIL 15

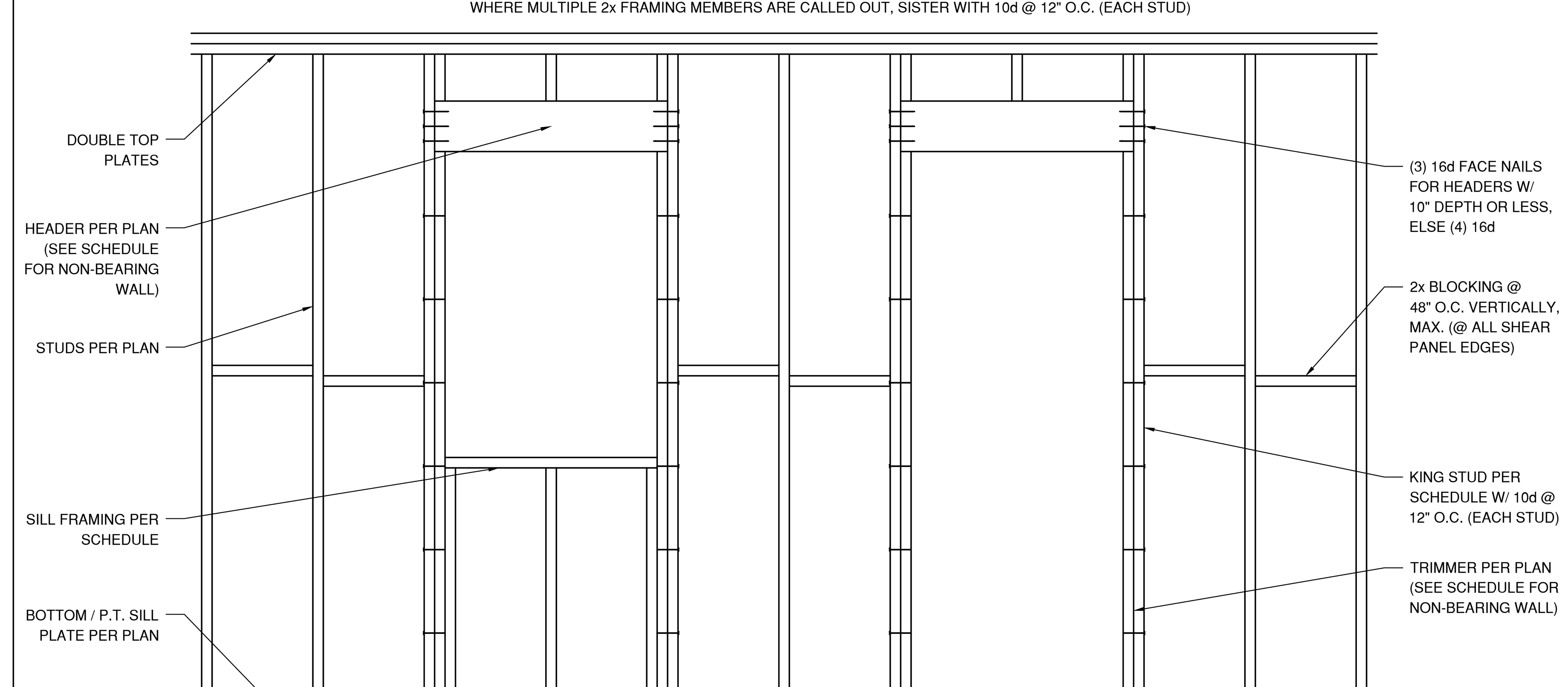


WINDOW OPENING

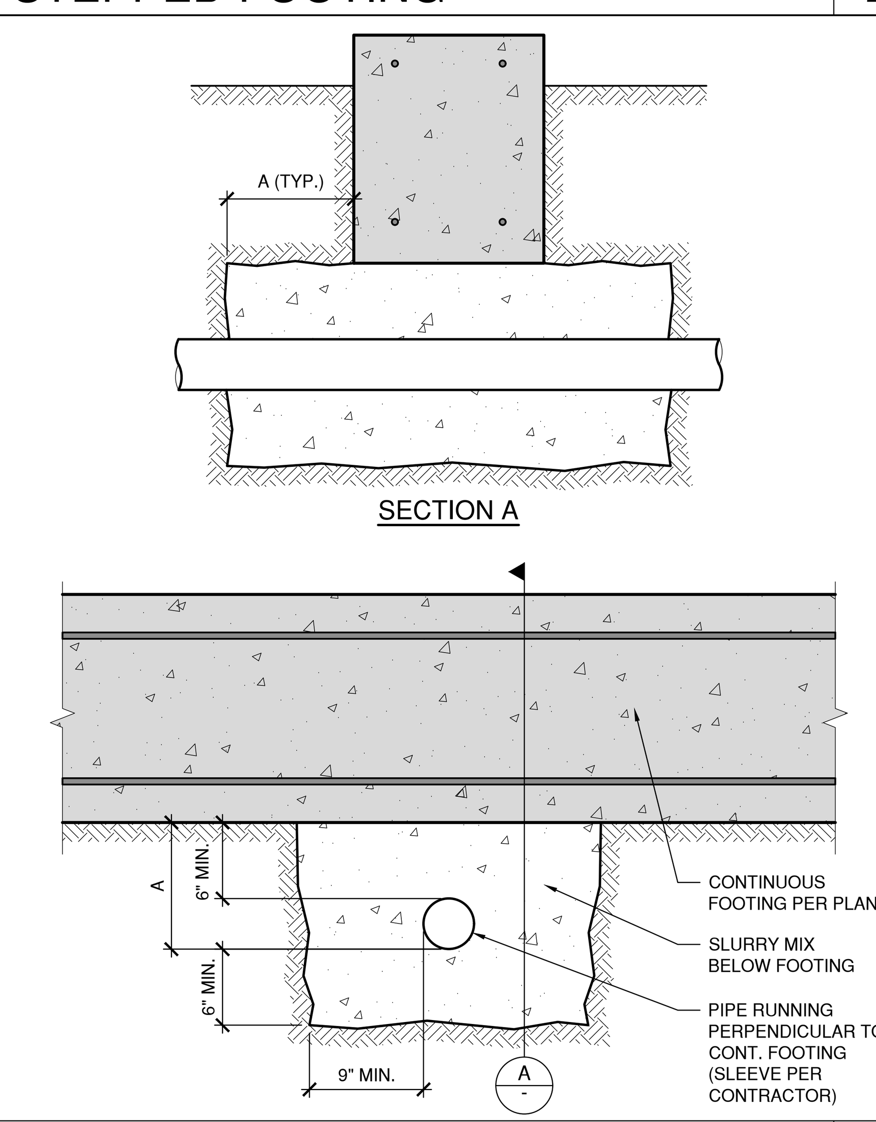
SHEAR WALL WITH OPENINGS 12

HEADER SPAN	NON-BEARING WALL HEADER SIZE	SILL FRAMING	TRIMMER	KING STUD	
				EXT. WALL	INT. WALL
4'-0"	4x4 (6x4)	2x	2x	2x	2x
8'-0"	4x6 (6x6)	2x	2x	2x	2x
12'-0"	4x8 (6x8)	2-2x	2-2x	2-2x	2-2x
16'-0"	4x10 (6x10)	2-2x	2-2x	3-2x	2-2x

WHERE MULTIPLE 2x FRAMING MEMBERS ARE CALLED OUT, SISTER WITH 10d @ 12" O.C. (EACH STUD)



TYPICAL WALL FRAMING 6



PIPES UNDER CONT. FOOTING 3

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APPENDIX B

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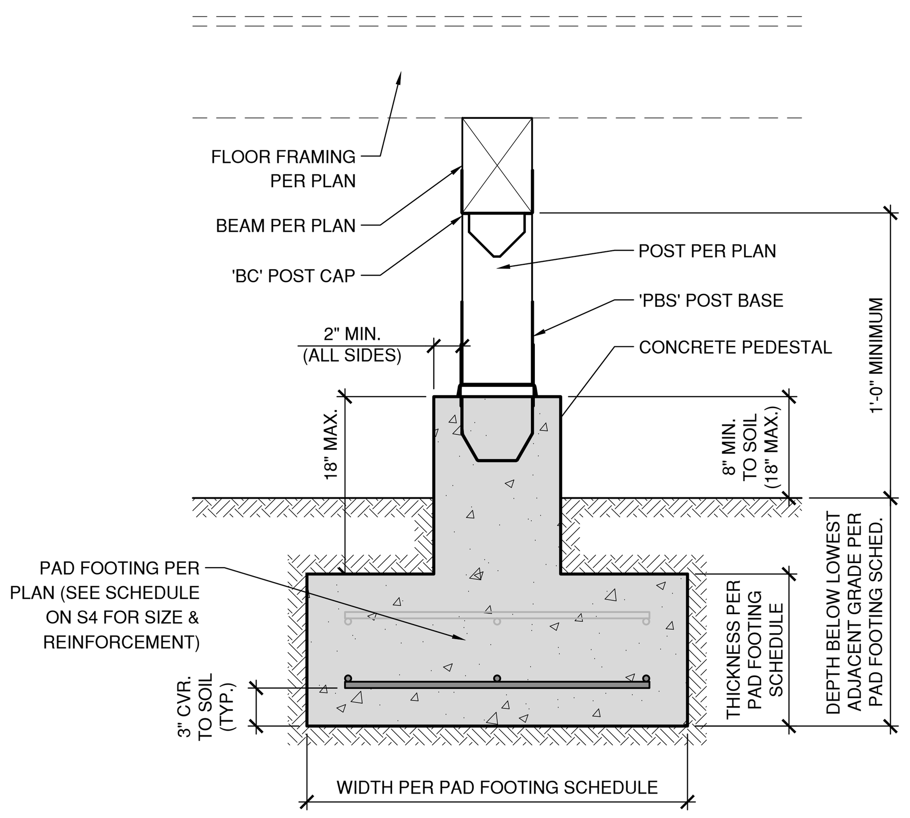
REVISIONS

PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017
SCALE: N.T.S.

TYPICAL STRUCTURAL DETAILS

S5

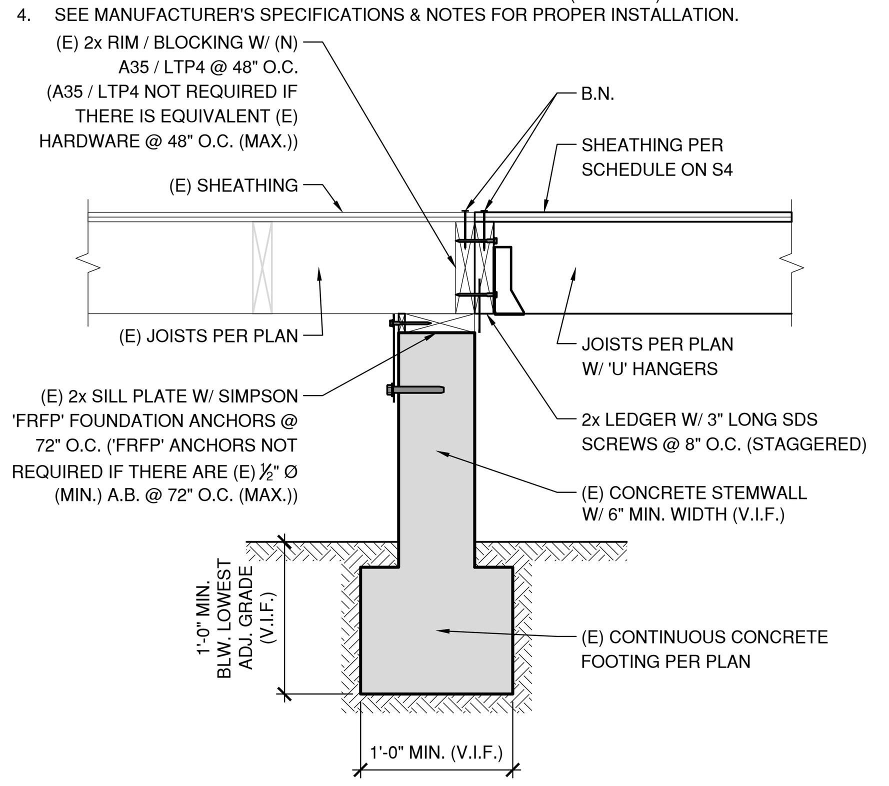


NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.

PAD FOOTING 13

'FRFP' FOUNDATION ANCHOR NOTES

1. ADD A SHIM BETWEEN SILL PLATE & 'FRFP' WHEN SPACE IS BETWEEN ³⁄₁₆" & 1½" (1½" MAX. SPACE).
2. ATTACH TO (E) SILL PLATE W/ (3) SDS SCREWS W/ LENGTH OF 2½" + SHIM THICKNESS.
3. ATTACH TO (E) CONCRETE STEMWALL W/ (2) ½" Ø x 4" LONG TITEN-HD ANCHORS (ESR-2713) OR SIMPSON RFB#4x6 RETROFIT BOLTS USING 'SET-XP' EPOXY (ESR-2508).
4. SEE MANUFACTURER'S SPECIFICATIONS & NOTES FOR PROPER INSTALLATION.

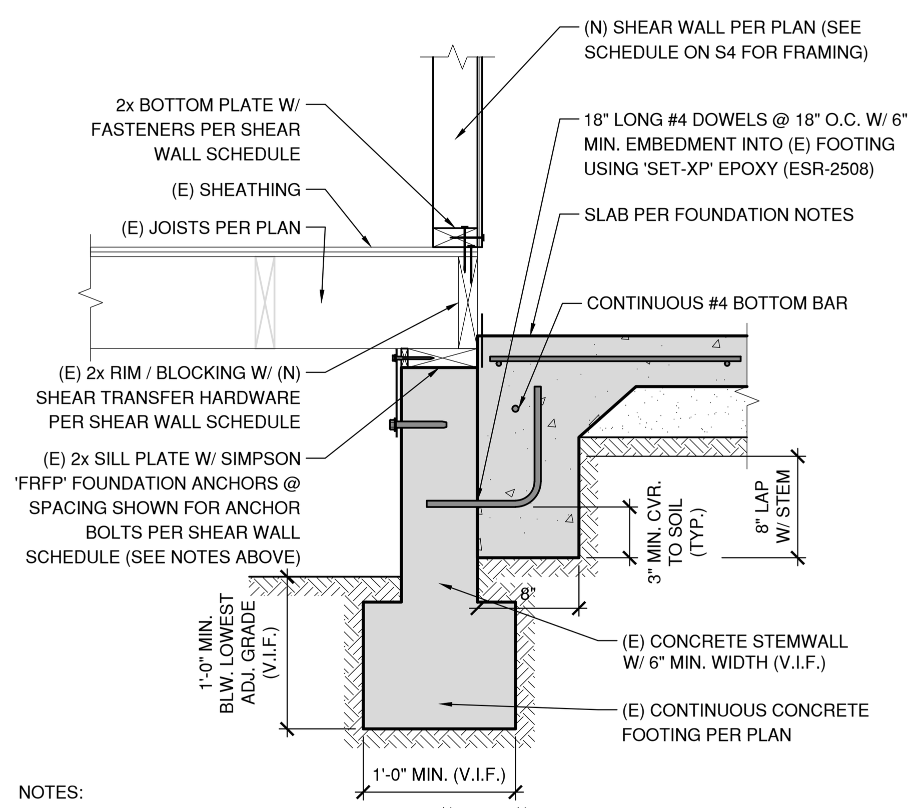


NOTES:
- WHERE STEMWALL THICKNESS IS BETWEEN 1½" AND 2½" THICKER THAN THE WIDTH OF THE SILL PLATE, SUBSTITUTE 'URFP' FOUNDATION PLATES FOR 'FRFP' FOUNDATION ANCHORS. INSTALL PER MANUFACTURER'S RECOMMENDATIONS (NO SHIM REQUIRED).
- SEE STATEMENT OF SPECIAL INSPECTION (SHEET S1) FOR RETROFIT ANCHOR BOLT INSTALLATION.

CONTINUOUS FOOTING 10

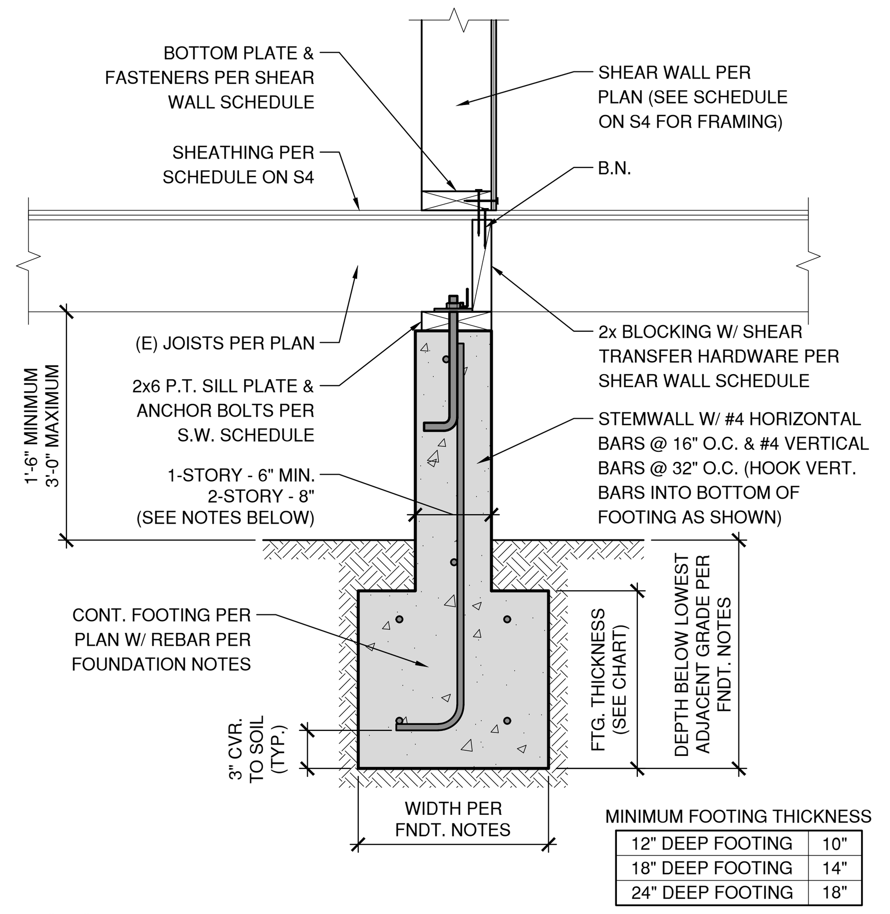
'FRFP' FOUNDATION ANCHOR NOTES

1. ADD A SHIM BETWEEN SILL PLATE & 'FRFP' WHEN SPACE IS BETWEEN ³⁄₁₆" & 1½" (1½" MAX. SPACE).
2. ATTACH TO (E) SILL PLATE W/ (3) SDS SCREWS W/ LENGTH OF 2½" + SHIM THICKNESS.
3. ATTACH TO (E) CONCRETE STEMWALL W/ (2) ½" Ø x 4" LONG TITEN-HD ANCHORS (ESR-2713) OR SIMPSON RFB#4x6 RETROFIT BOLTS USING 'SET-XP' EPOXY (ESR-2508).
4. SEE MANUFACTURER'S SPECIFICATIONS & NOTES FOR PROPER INSTALLATION.



NOTES:
- WHERE STEMWALL THICKNESS IS BETWEEN 1½" AND 2½" THICKER THAN THE WIDTH OF THE SILL PLATE, SUBSTITUTE 'URFP' FOUNDATION PLATES FOR 'FRFP' FOUNDATION ANCHORS. INSTALL PER MANUFACTURER'S RECOMMENDATIONS (NO SHIM REQUIRED).
- WATERPROOFING BETWEEN SLAB AND EXISTING WOOD FRAMING PER ARCHITECT.
- SEE STATEMENT OF SPECIAL INSPECTION (SHEET S1) FOR RETROFIT ANCHOR BOLT AND DOWEL INSTALLATION.

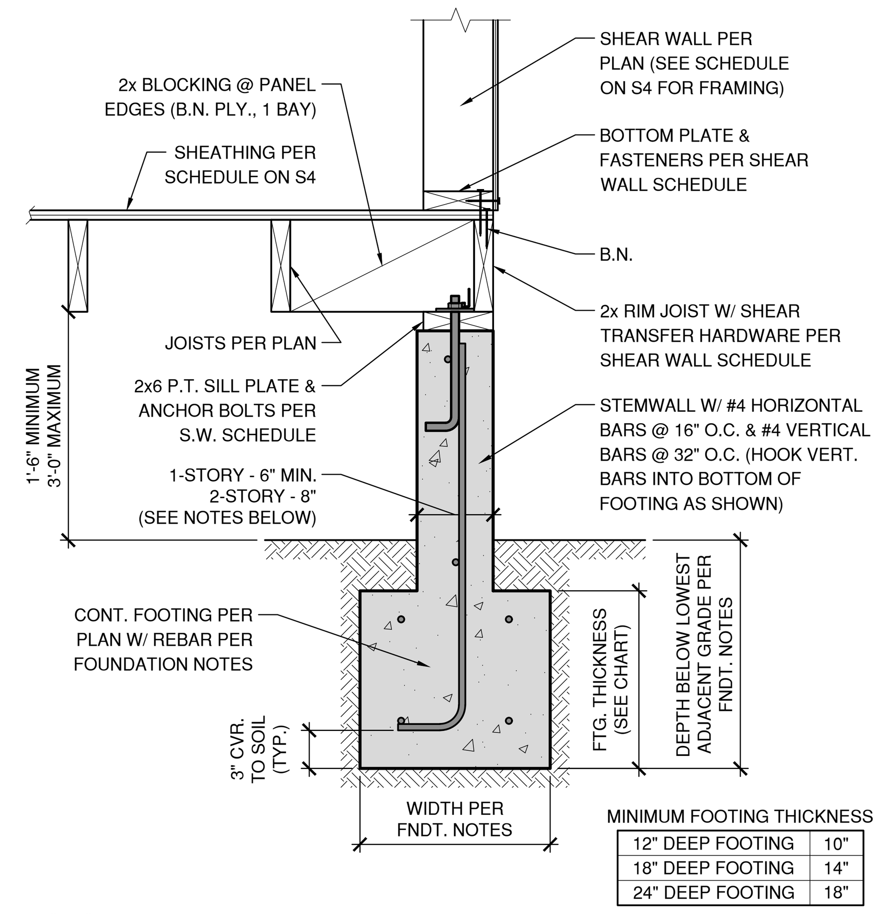
CONTINUOUS FOOTING 7



MINIMUM FOOTING THICKNESS	
12" DEEP FOOTING	10"
18" DEEP FOOTING	14"
24" DEEP FOOTING	18"

NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

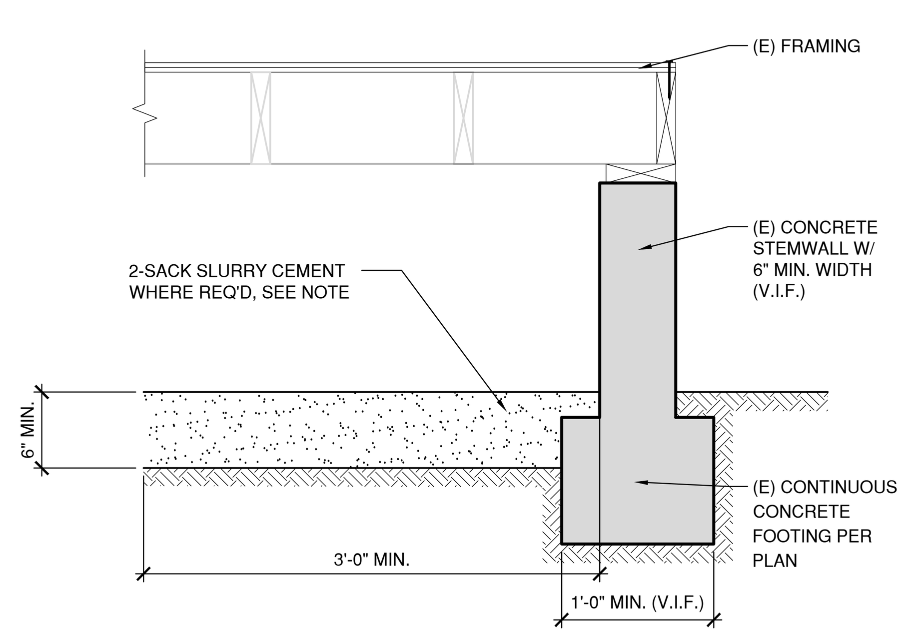
CONTINUOUS FOOTING 4



MINIMUM FOOTING THICKNESS	
12" DEEP FOOTING	10"
18" DEEP FOOTING	14"
24" DEEP FOOTING	18"

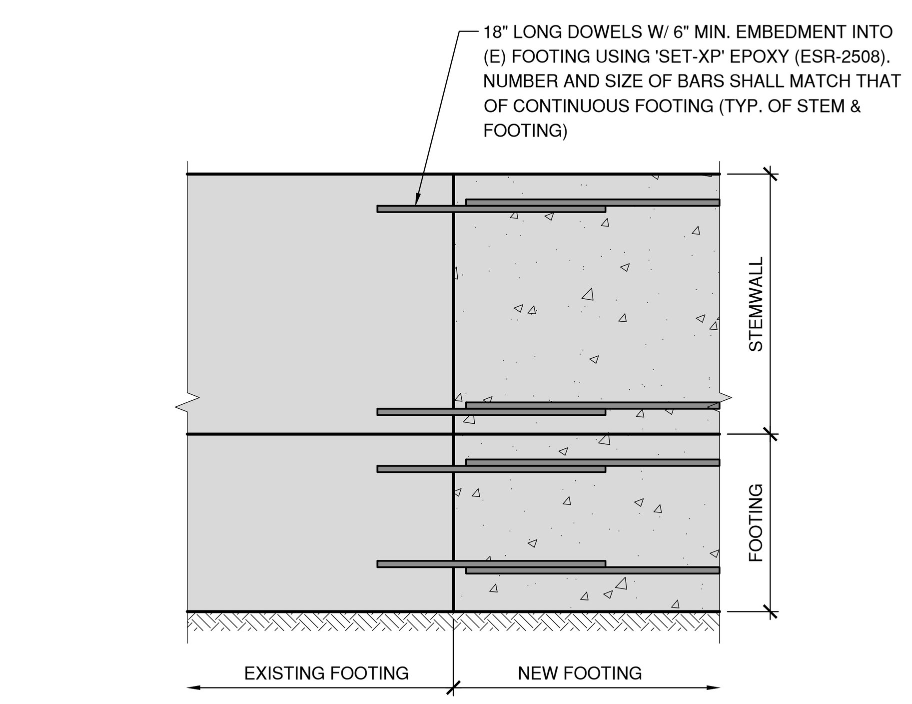
NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

CONTINUOUS FOOTING 1



NOTES:
- WHERE EXISTING FOOTING EMBEDEMENT IS LESS THAN 12" PLACE 6" OF 2-SACK SLURRY CEMENT ADJACENT TO THE INTERIOR SIDE OF THE STEM WALL AND EXTEND FOR AT LEAST 3 HORIZONTAL FEET AWAY FROM THE STEM WALL
- ALTERNATIVELY, MAY PLACE 6" OF COMPACTED FILL ADJACENT TO THE INTERIOR SIDE OF THE STEM WALL AND EXTEND FOR AT LEAST 3 HORIZONTAL FEET AWAY FROM THE STEM WALL.

(E) STEM WALL MODIFICATION 14

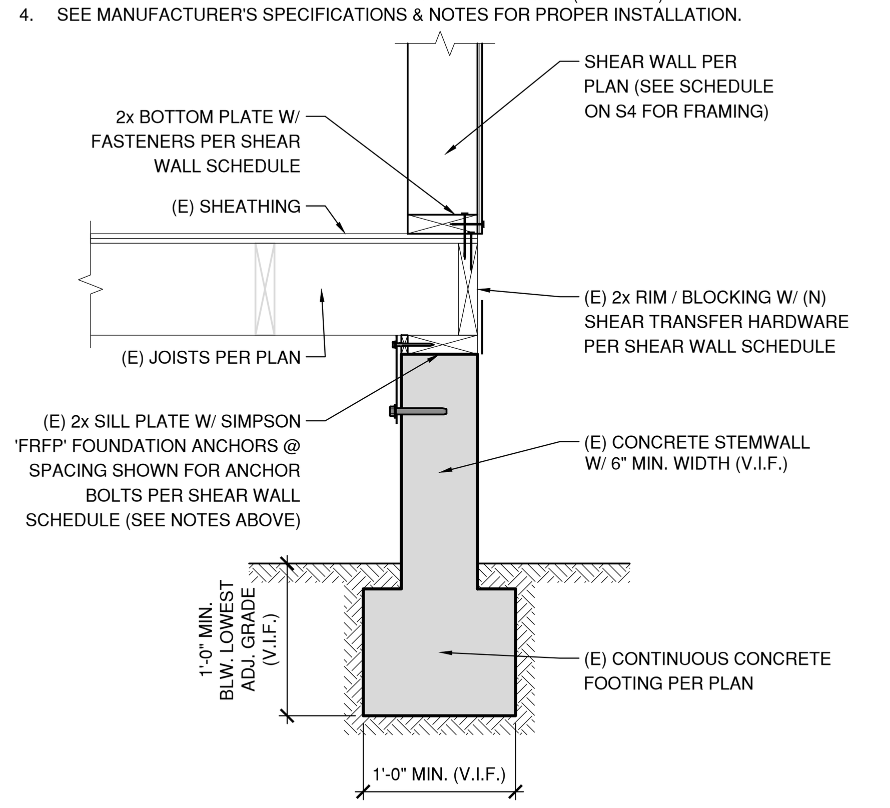


NOTES:
- SEE STATEMENT OF SPECIAL INSPECTION (SHEET S1) FOR RETROFIT DOWEL INSTALLATION.

(N) FOOTING TO (E) FOOTING 11

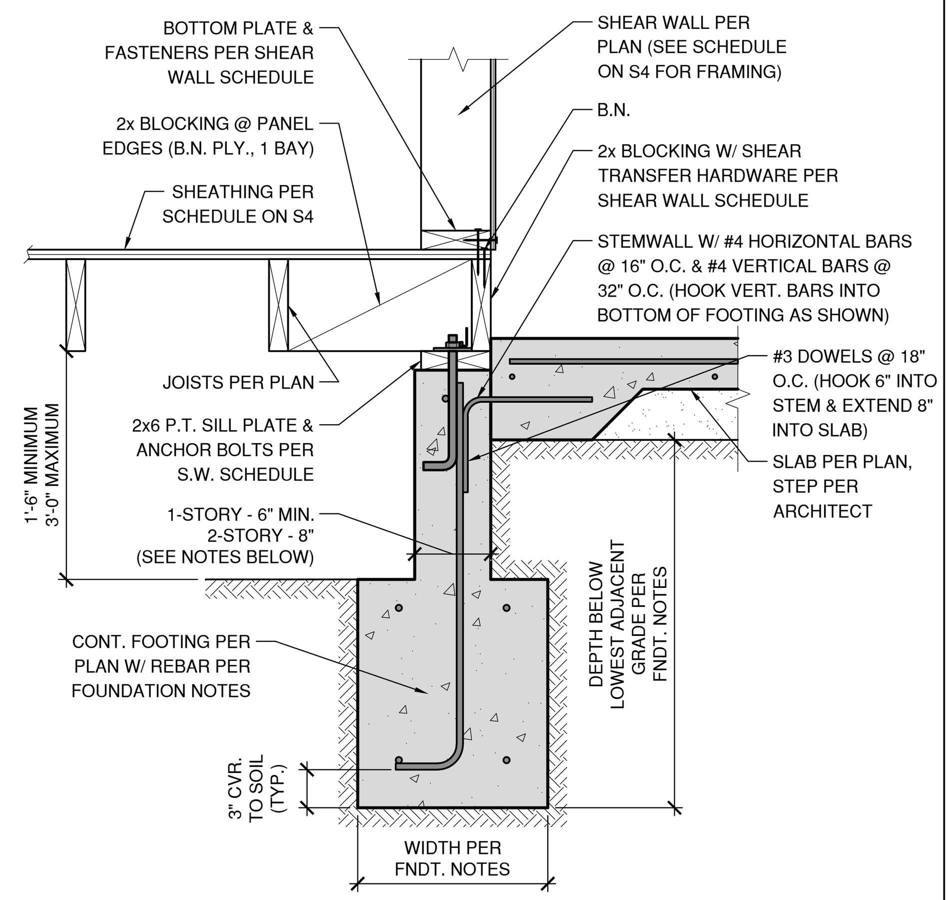
'FRFP' FOUNDATION ANCHOR NOTES

1. ADD A SHIM BETWEEN SILL PLATE & 'FRFP' WHEN SPACE IS BETWEEN ³⁄₁₆" & 1½" (1½" MAX. SPACE).
2. ATTACH TO (E) SILL PLATE W/ (3) SDS SCREWS W/ LENGTH OF 2½" + SHIM THICKNESS.
3. ATTACH TO (E) CONCRETE STEMWALL W/ (2) ½" Ø x 4" LONG TITEN-HD ANCHORS (ESR-2713) OR SIMPSON RFB#4x6 RETROFIT BOLTS USING 'SET-XP' EPOXY (ESR-2508).
4. SEE MANUFACTURER'S SPECIFICATIONS & NOTES FOR PROPER INSTALLATION.



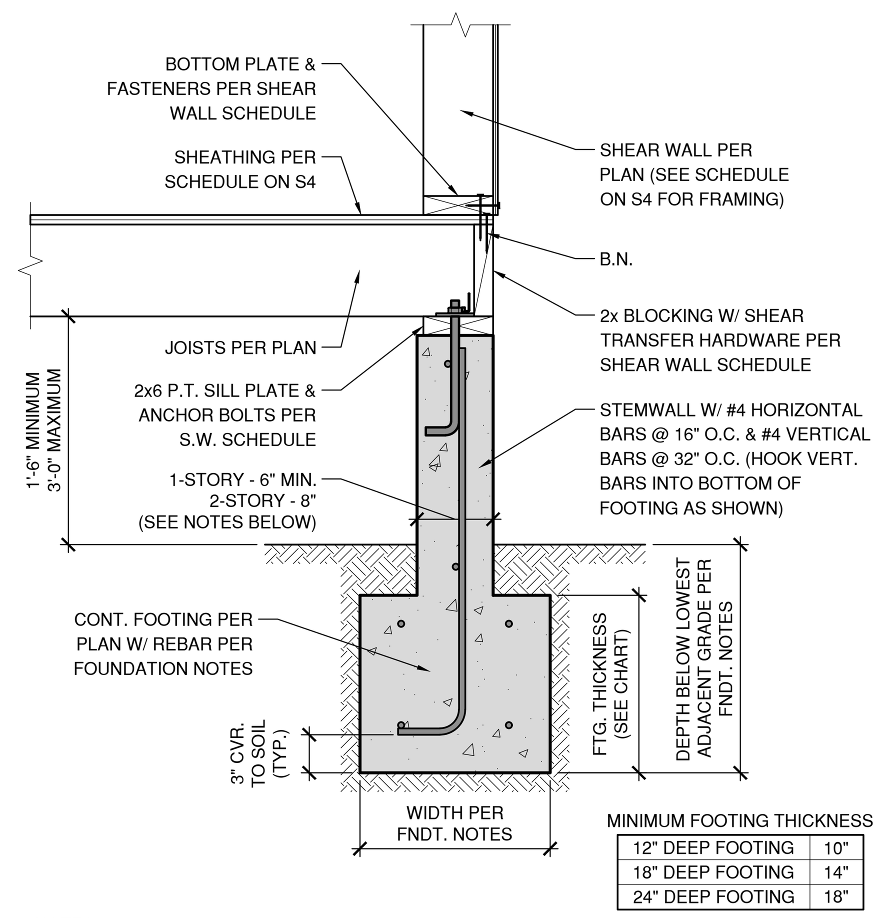
NOTES:
- WHERE STEMWALL THICKNESS IS BETWEEN 1½" AND 2½" THICKER THAN THE WIDTH OF THE SILL PLATE, SUBSTITUTE 'URFP' FOUNDATION PLATES FOR 'FRFP' FOUNDATION ANCHORS. INSTALL PER MANUFACTURER'S RECOMMENDATIONS (NO SHIM REQUIRED).
- SEE STATEMENT OF SPECIAL INSPECTION (SHEET S1) FOR RETROFIT ANCHOR BOLT INSTALLATION.

CONTINUOUS FOOTING 8



NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

CONTINUOUS FOOTING 5

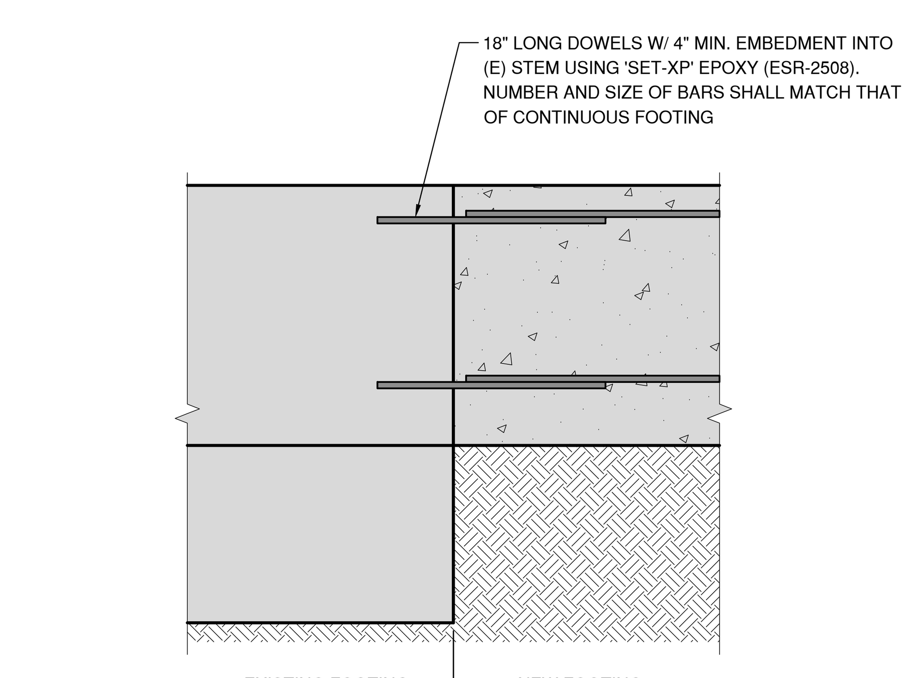


MINIMUM FOOTING THICKNESS	
12" DEEP FOOTING	10"
18" DEEP FOOTING	14"
24" DEEP FOOTING	18"

NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

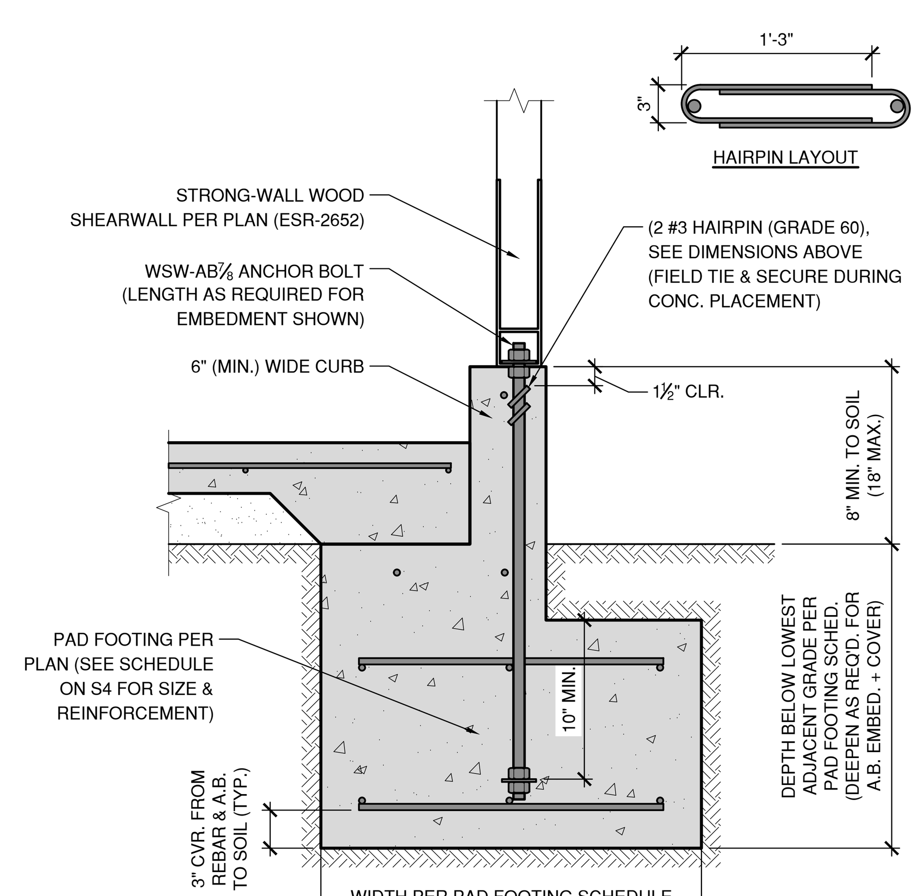
CONTINUOUS FOOTING 2

BLANK



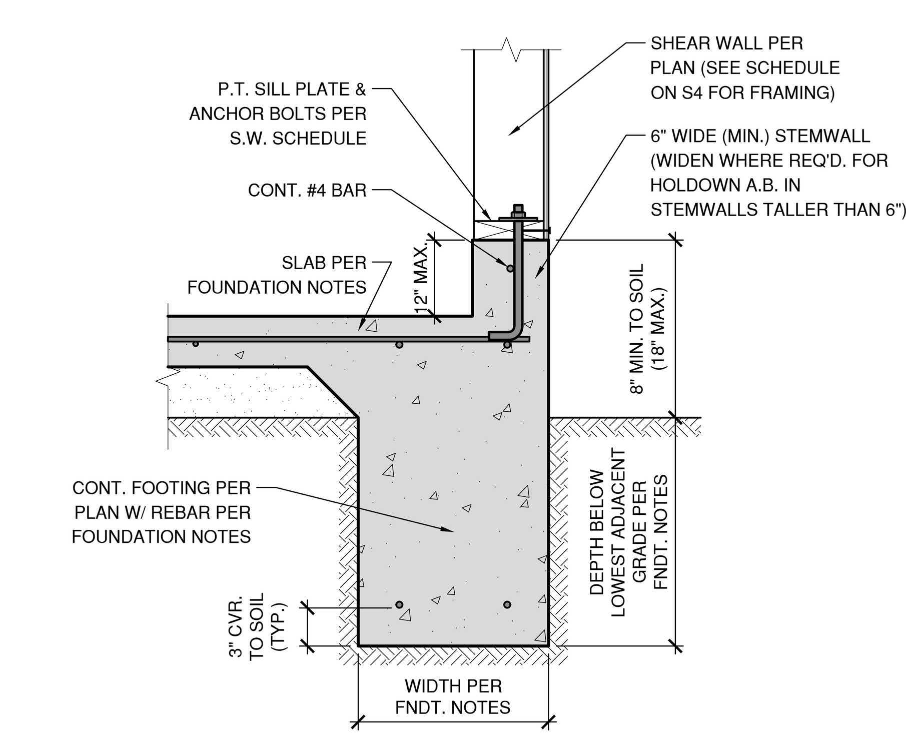
NOTES:
- SEE STATEMENT OF SPECIAL INSPECTION (SHEET S1) FOR RETROFIT DOWEL INSTALLATION.

(N) FOOTING TO (E) FOOTING 12

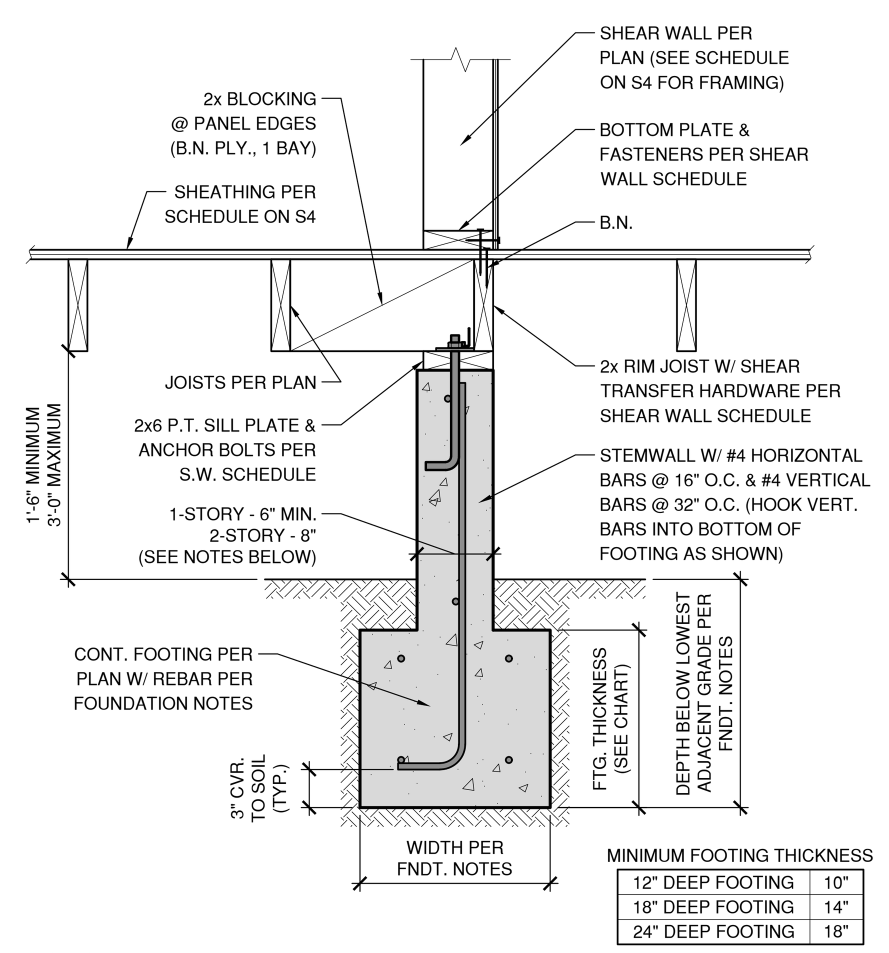


NOTES:
- SEE STATEMENT OF SPECIAL INSPECTION (SHEET S1) FOR STRONG-WALL WOOD SHEARWALL INSTALLATION.
- SEE SHEETS WSW-X FOR TYPICAL SIMPSON DETAILS.

PAD FOOTING 9



CONTINUOUS FOOTING 6



MINIMUM FOOTING THICKNESS	
12" DEEP FOOTING	10"
18" DEEP FOOTING	14"
24" DEEP FOOTING	18"

NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

CONTINUOUS FOOTING 3

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APPENDIX B

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BEAUCHEMIN RESIDENCE

148 W. AVENIDA CADIZ SAN CLEMENTE CA 92672

REVISIONS

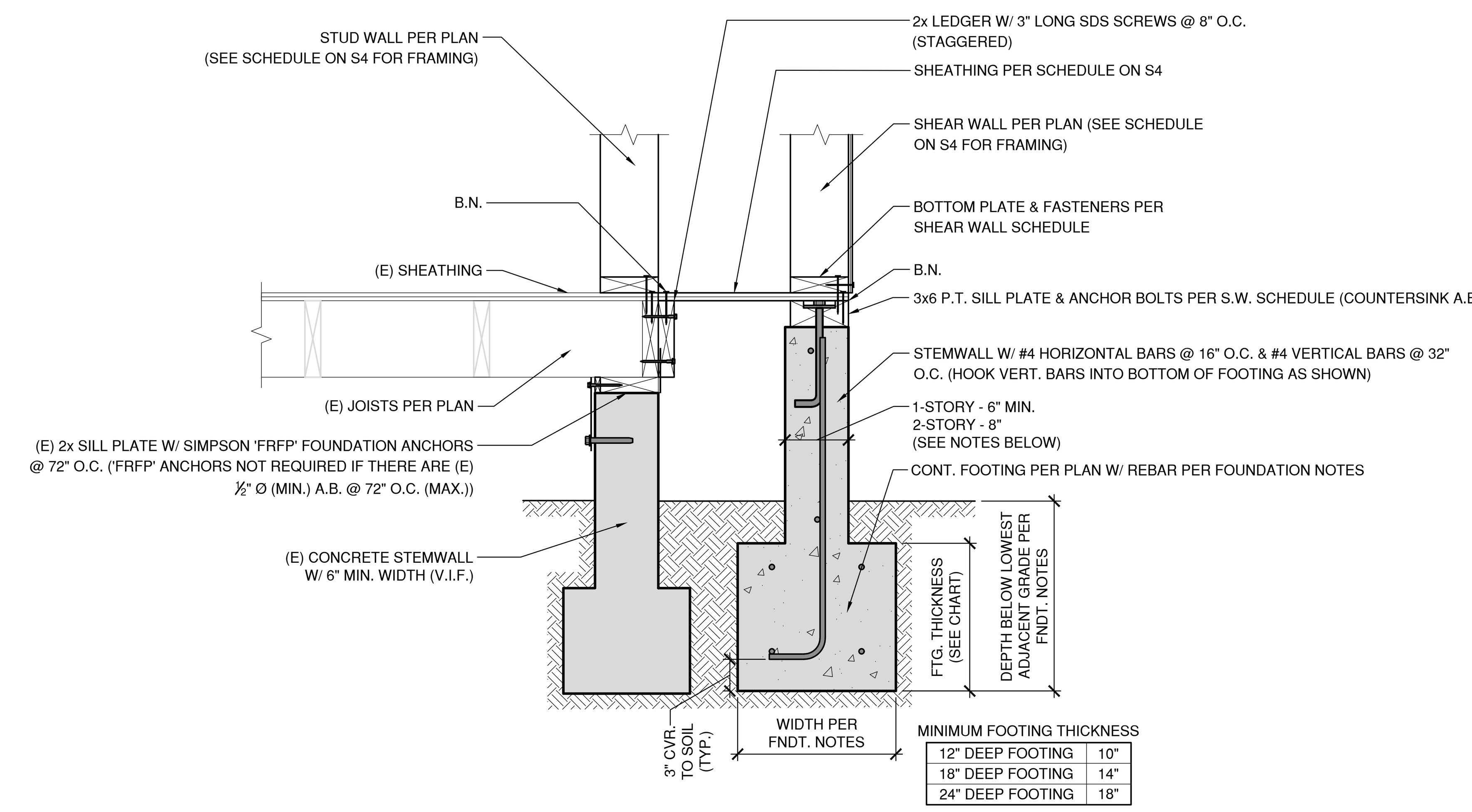
PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017

SCALE: N.T.S.

STRUCTURAL DETAILS

S6

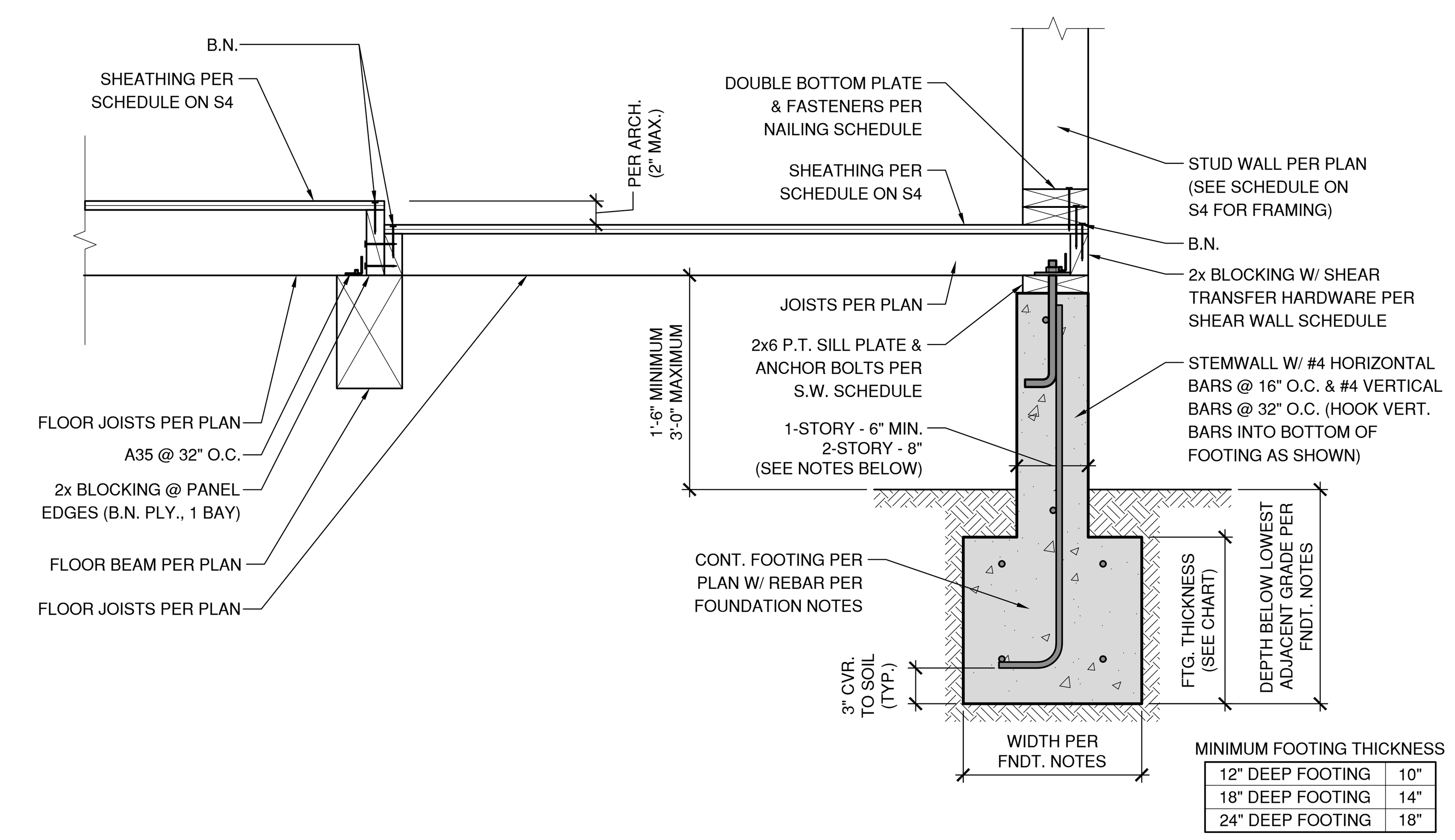
BLANK | BLANK | BLANK



NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

(N) CONTINUOUS FOOTING 16

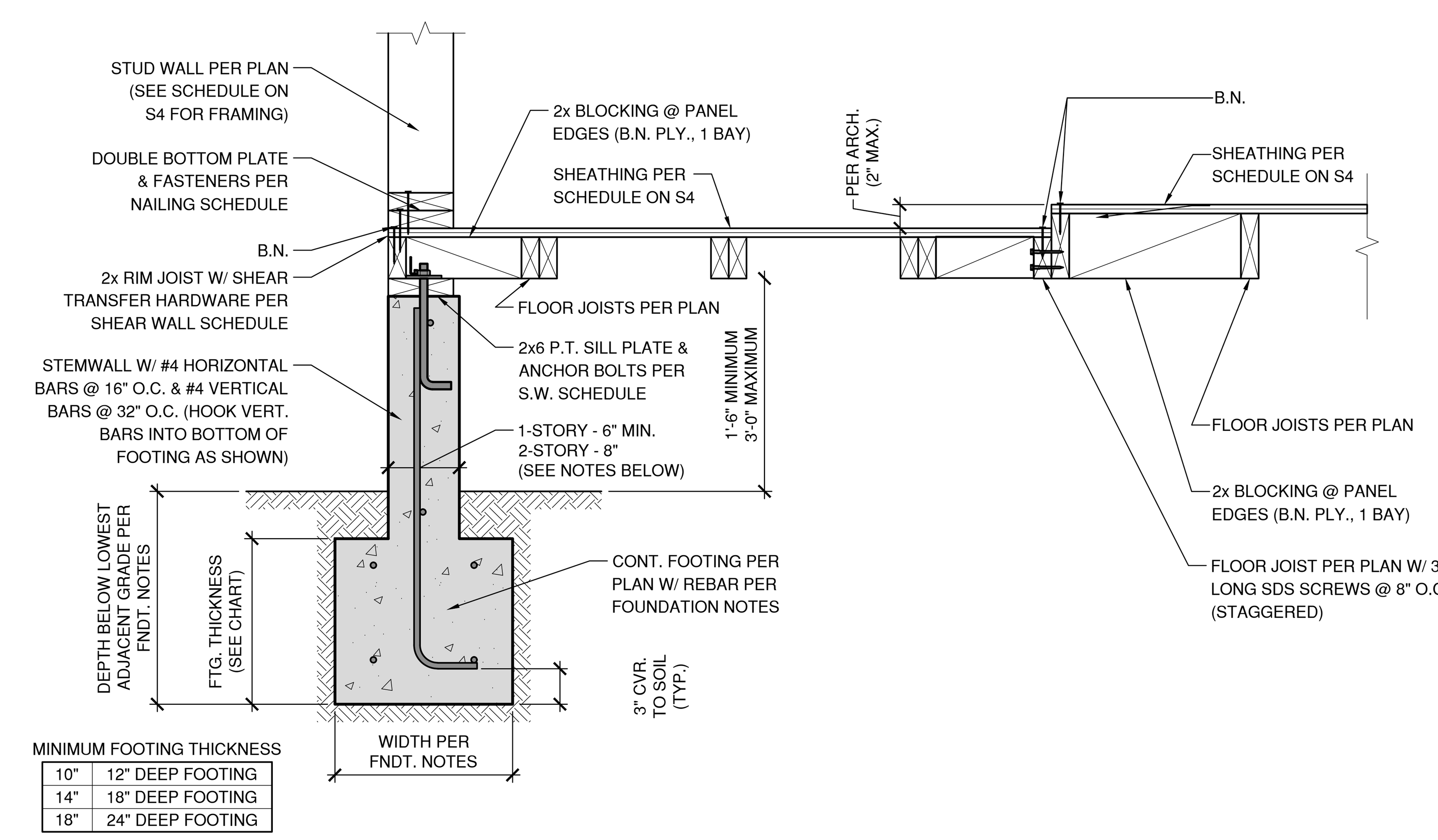
BLANK | BLANK | BLANK



NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

CONTINUOUS FOOTING 17

BLANK | BLANK | BLANK



NOTES:
- MAINTAIN 1'-6" MINIMUM CLEAR DISTANCE FROM SOIL TO FLOOR JOISTS THROUGHOUT.
- 8" THICK STEMWALL MAY BE REQUIRED @ HOLDOWN ANCHOR BOLT LOCATIONS, SEE SCHEDULE ON S4.

CONTINUOUS FOOTING 18

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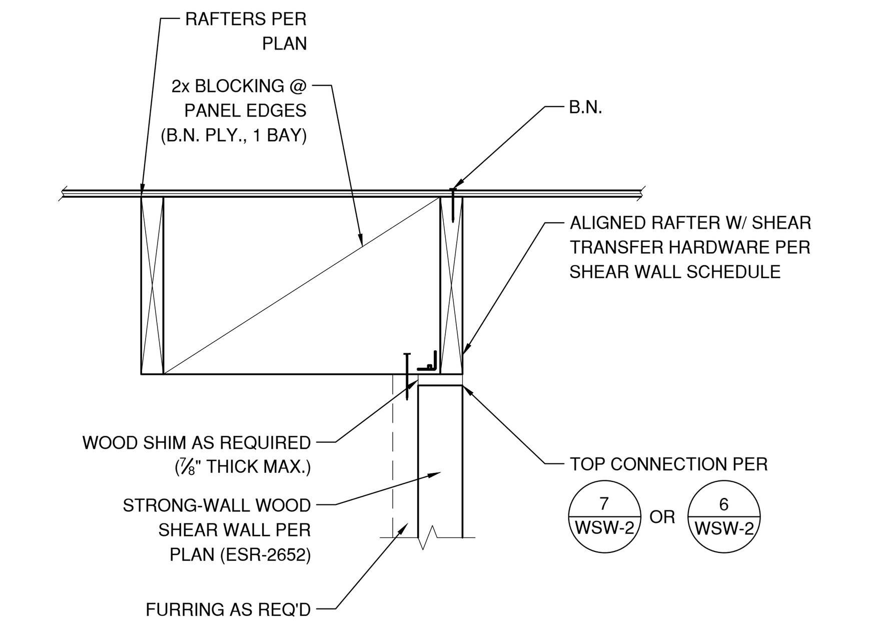
REVISIONS

PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017
SCALE: N.T.S.

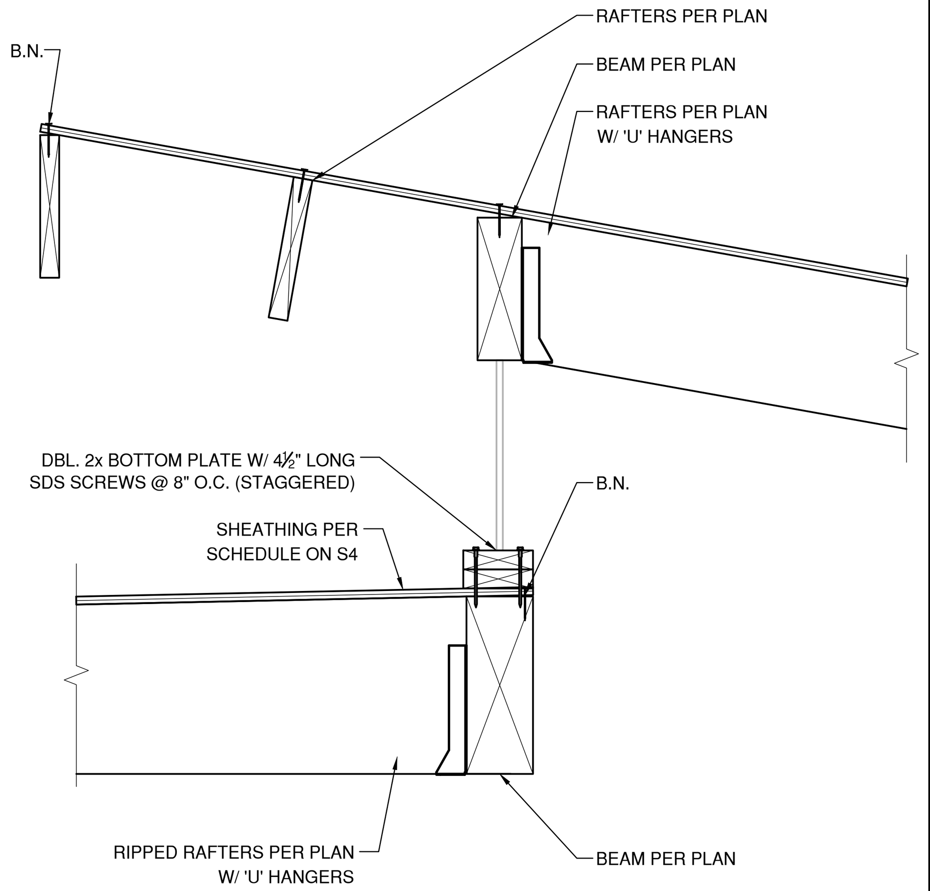
STRUCTURAL DETAILS

S7



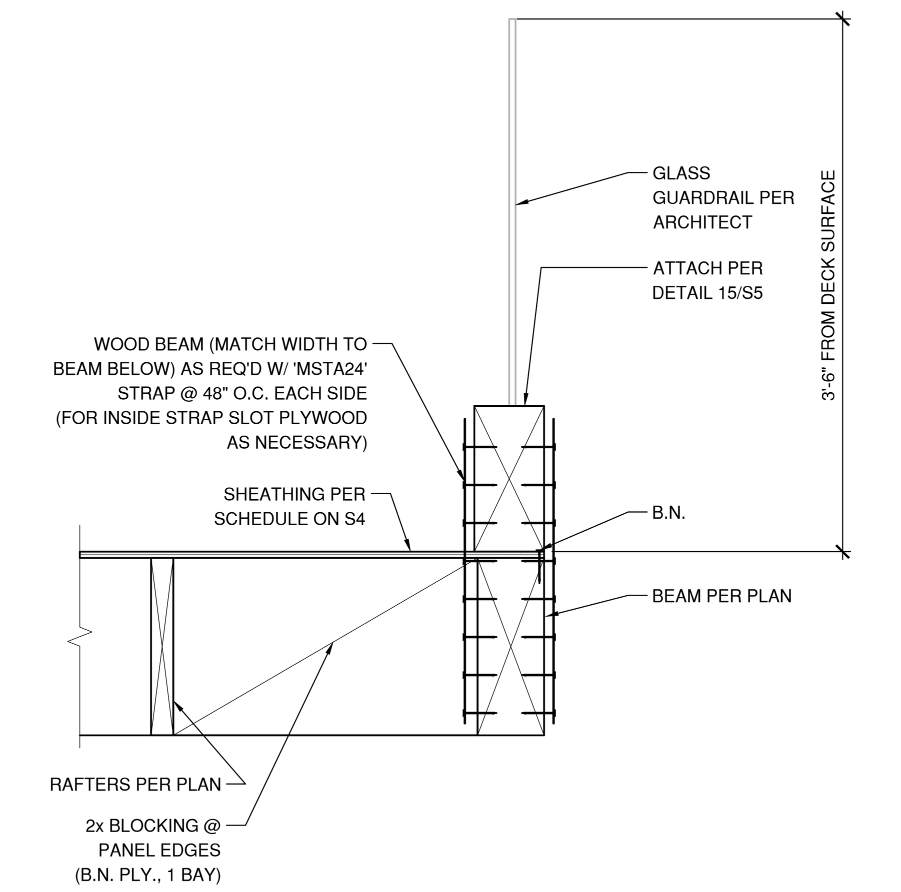
NOTES:
- SEE STATEMENT OF SPECIAL INSPECTION (SHEET S1) FOR STRONG-WALL WOOD SHEAR WALL INSTALLATION.
- SEE SHEETS WSW-X FOR TYPICAL SIMPSON DETAILS.

WOOD ROOF W/STRONG WALL 43



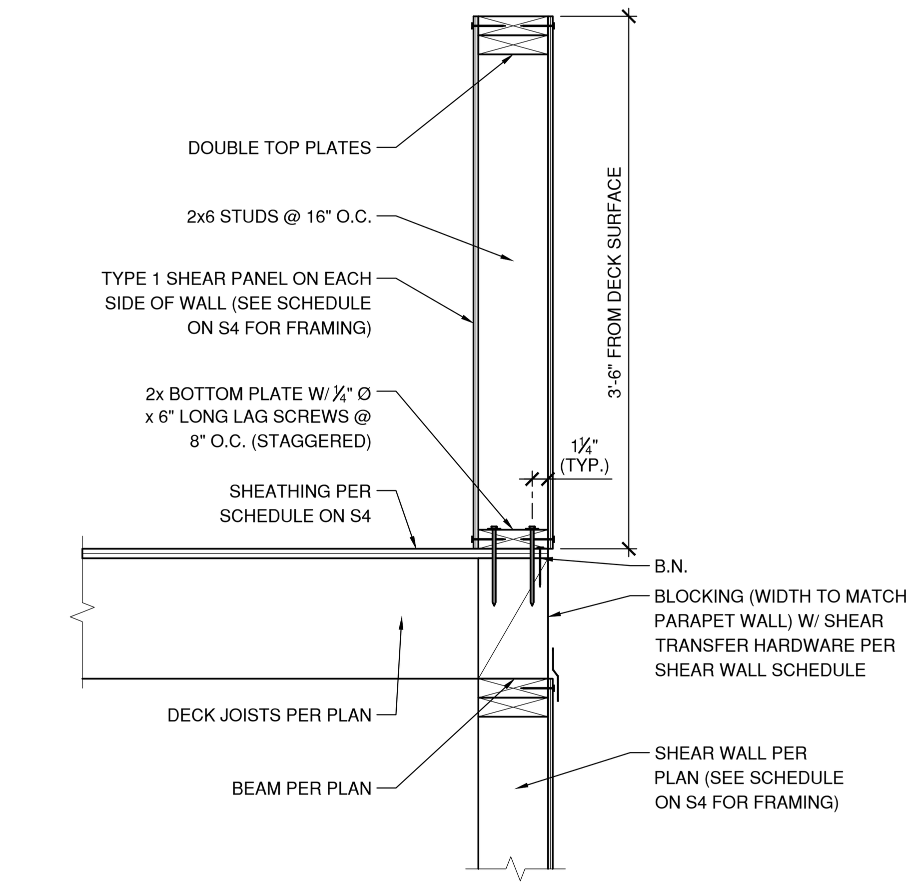
NOTES:
- TOP OF RAFTERS SHALL BE RIPPED PER ARCHITECT (¼" PER 1'-0" REQUIRED).

WOOD ROOF 40



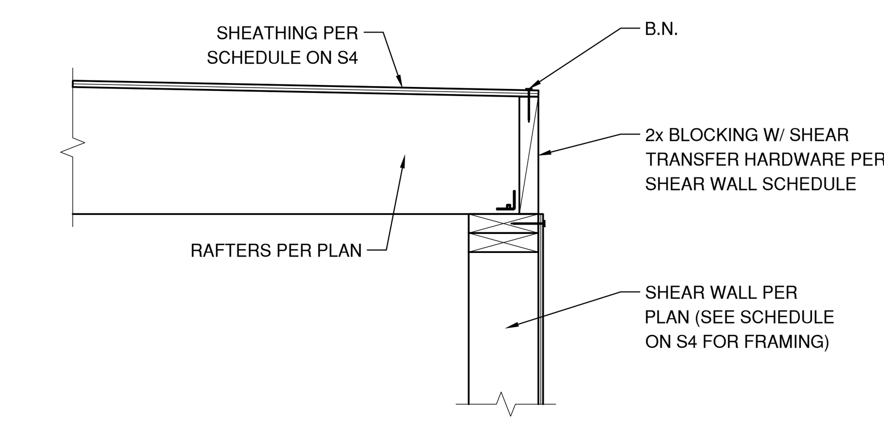
NOTES:
- RAFTERS SHALL BE BUILT FLAT WITH A BUILT-UP SLOPE ABOVE THE SHEATHING PER ARCHITECT (¼" PER 1'-0" REQUIRED).
- MAXIMUM PARAPET HEIGHT SHALL BE 18".

WOOD ROOF DECK 37



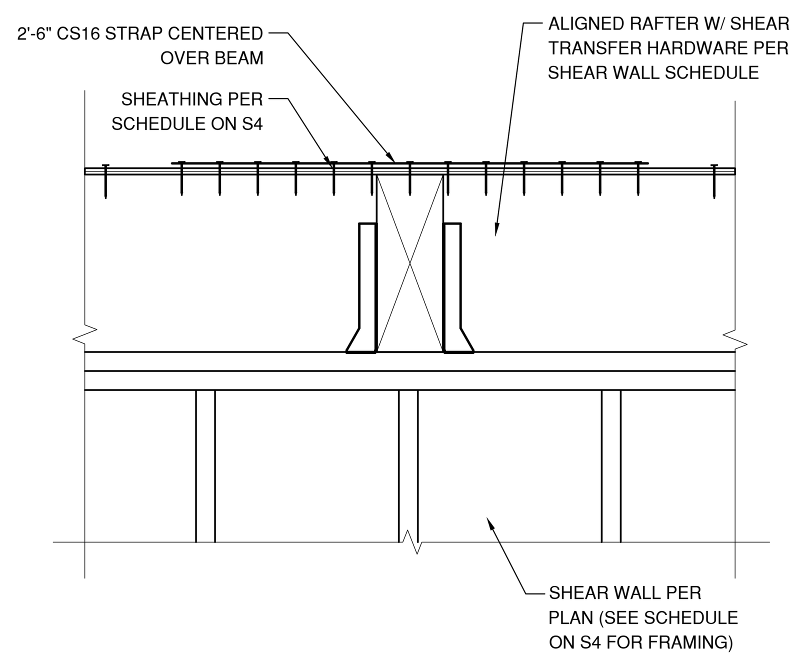
NOTES:
- DECK JOISTS SHALL BE BUILT FLAT WITH A BUILT-UP SLOPE ABOVE THE SHEATHING PER ARCHITECT (¼" PER 1'-0" REQUIRED).

WOOD ROOF DECK 34

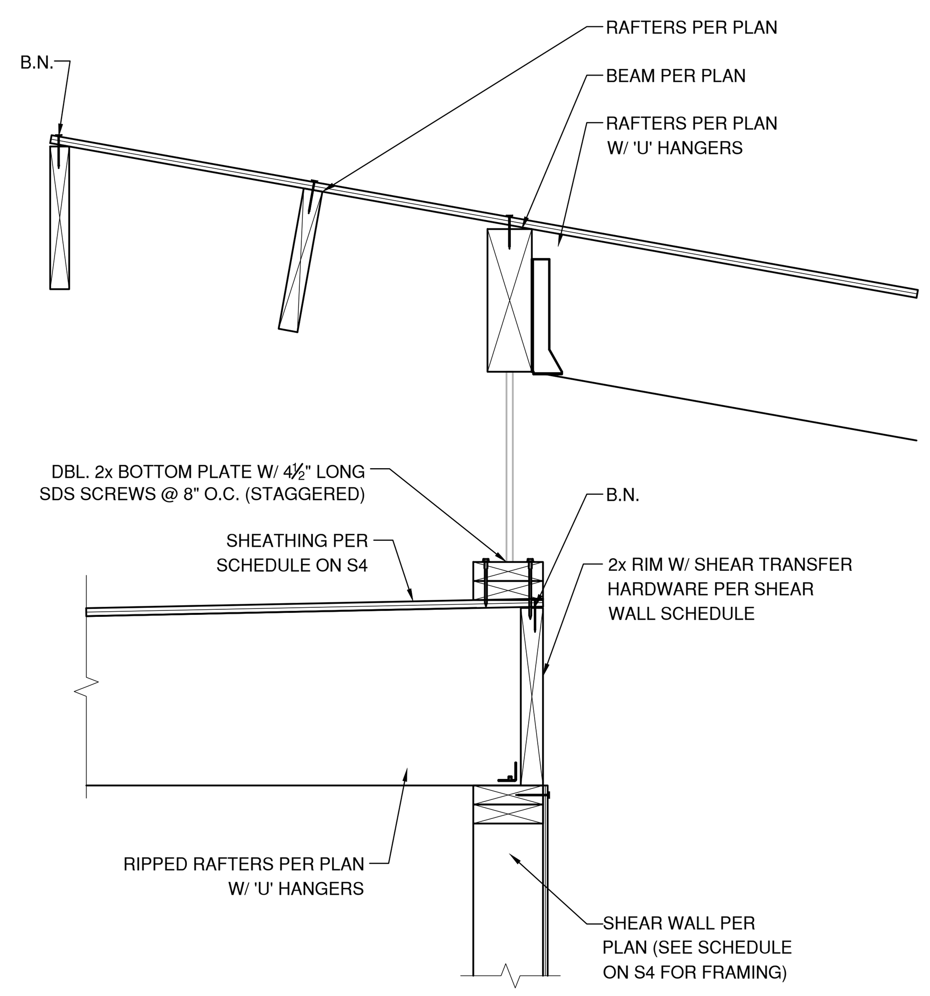


NOTES:
- TOP OF RAFTERS SHALL BE RIPPED PER ARCHITECT (¼" PER 1'-0" REQUIRED).

WOOD ROOF 31

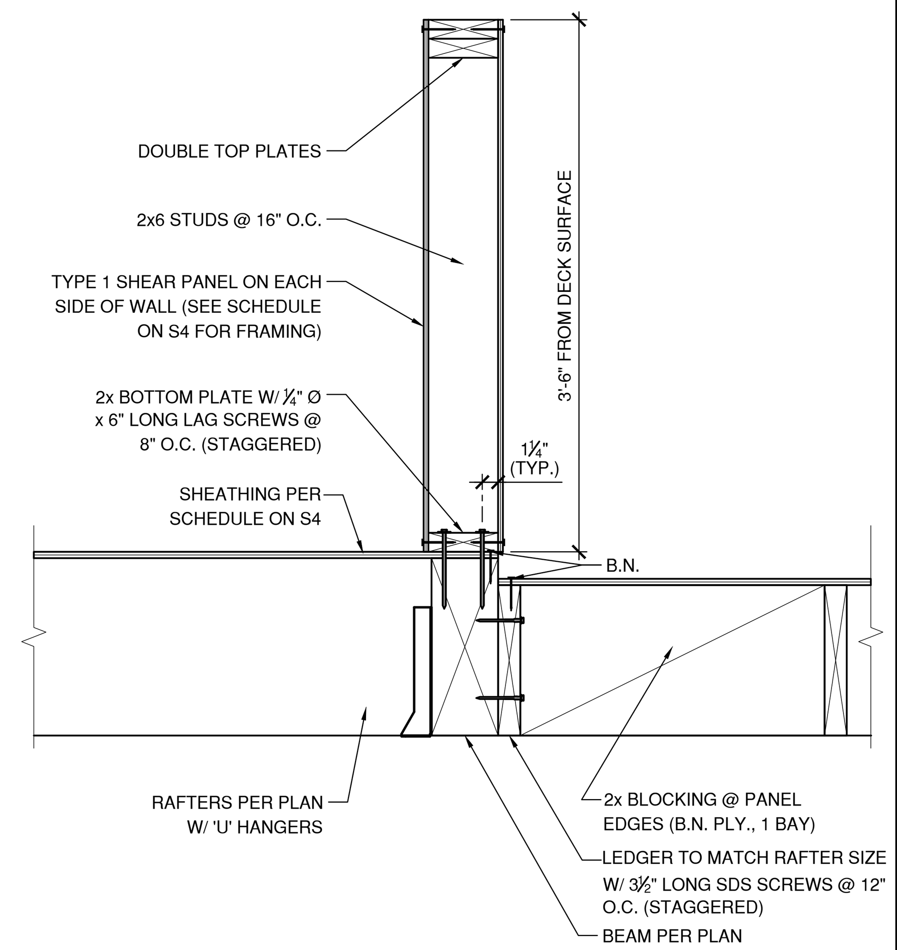


WOOD ROOF 44



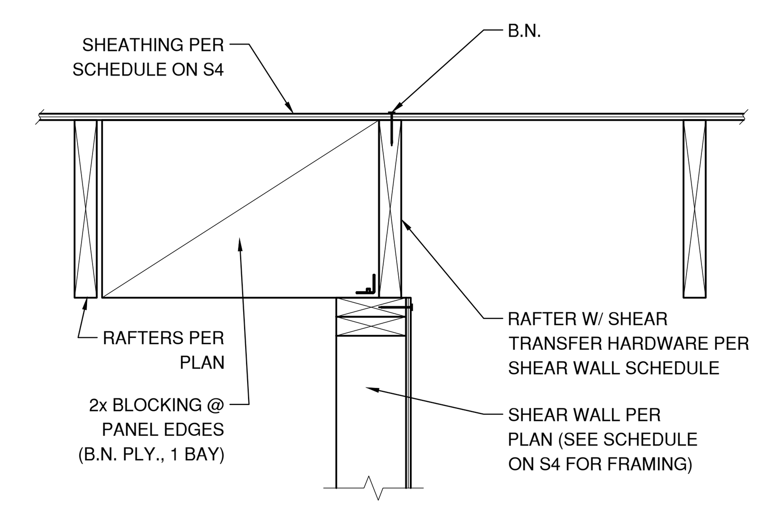
NOTES:
- RAFTERS SHALL BE BUILT FLAT WITH A BUILT-UP SLOPE ABOVE THE SHEATHING PER ARCHITECT (¼" PER 1'-0" REQUIRED).

WOOD ROOF 41

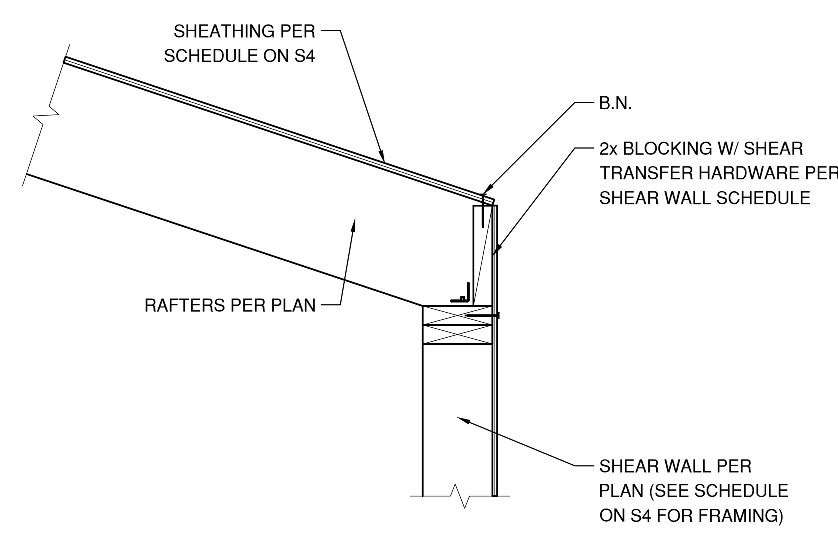


NOTES:
- RAFTERS SHALL BE BUILT FLAT WITH A BUILT-UP SLOPE ABOVE THE SHEATHING PER ARCHITECT (¼" PER 1'-0" REQUIRED).
- MAXIMUM PARAPET HEIGHT SHALL BE 18".

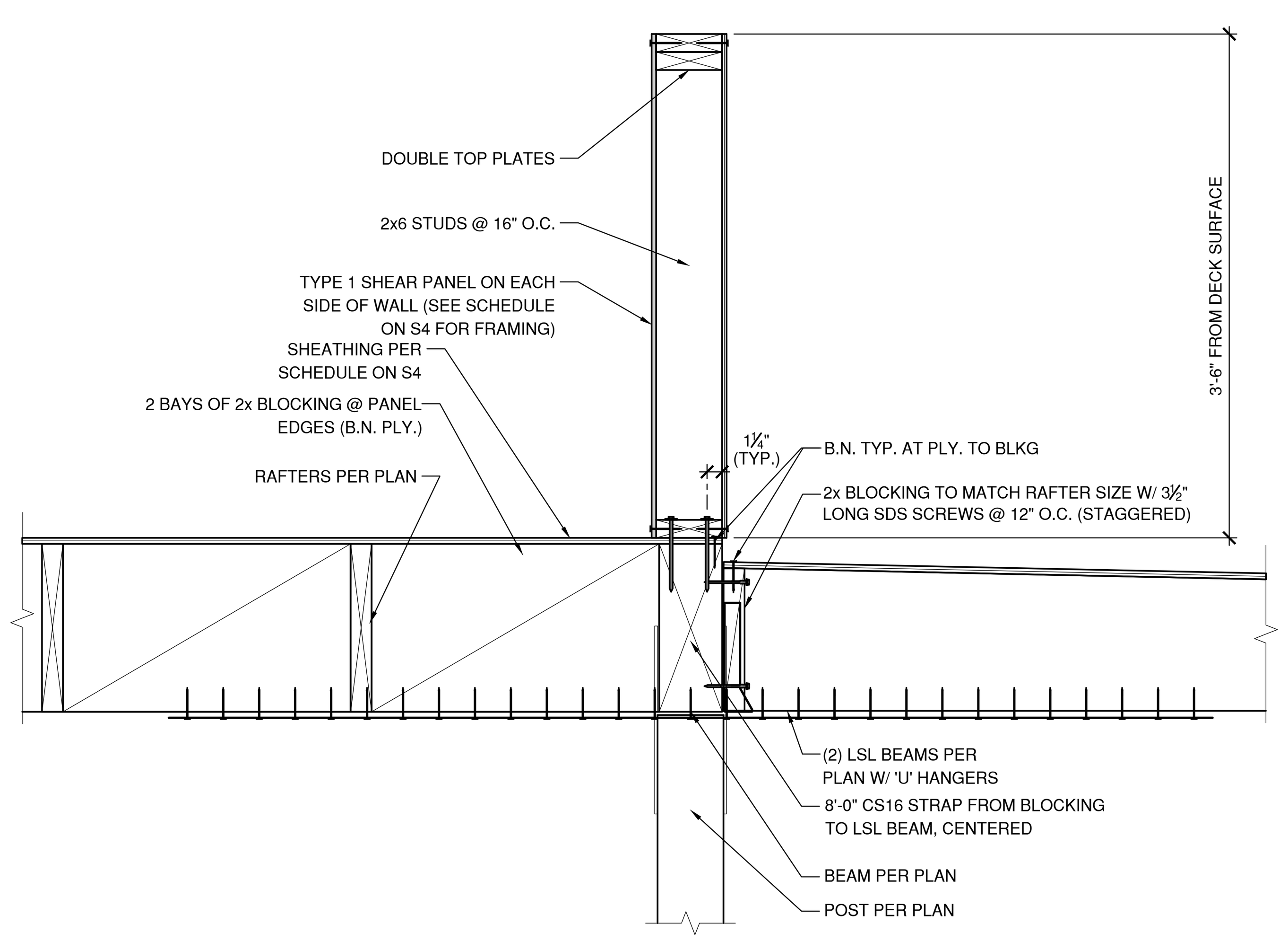
WOOD ROOF DECK 38



WOOD ROOF 35

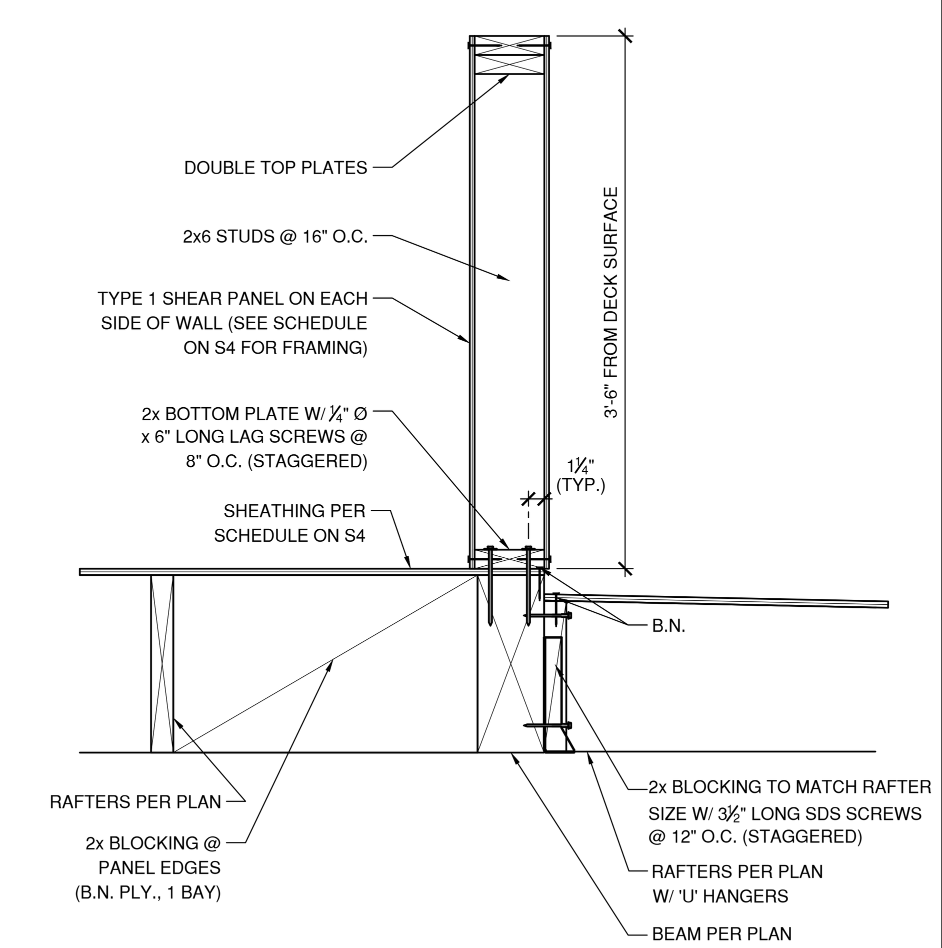


WOOD ROOF 32



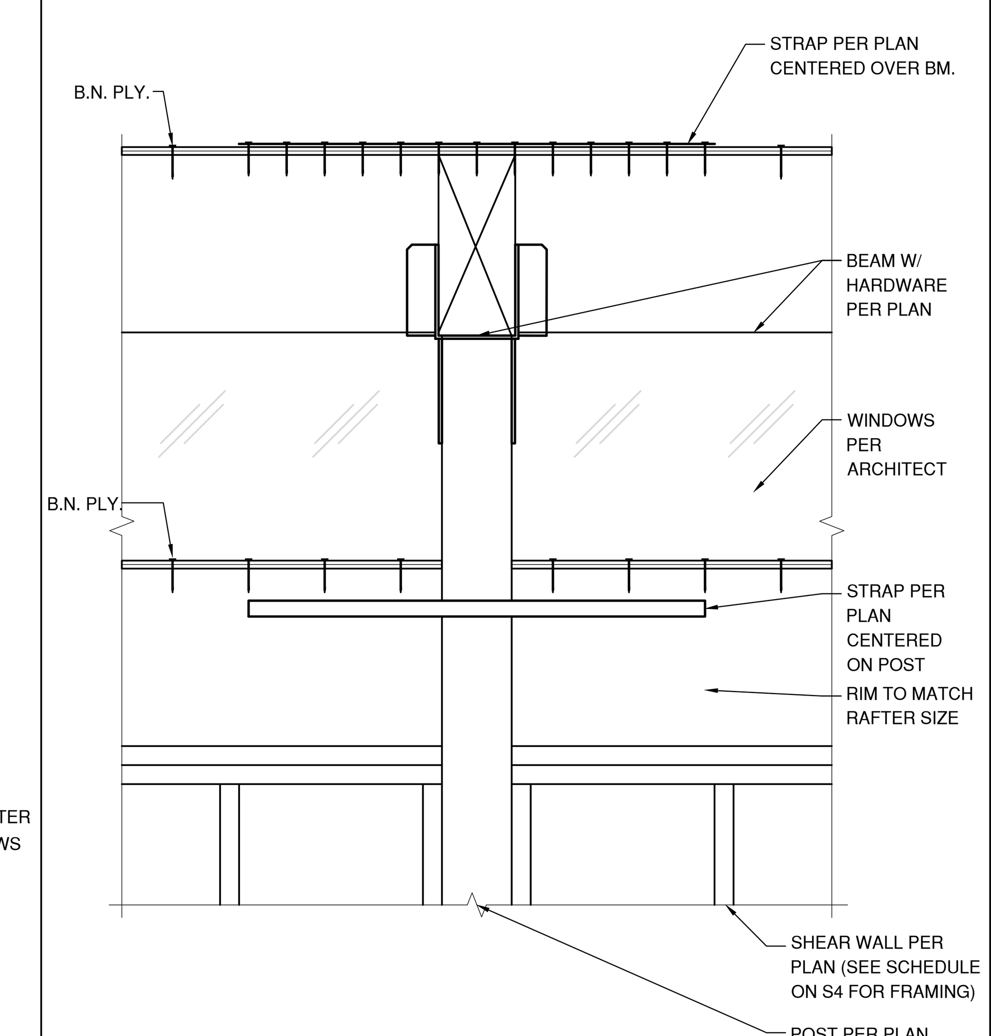
NOTES:
- RAFTERS SHALL EITHER BE BUILT FLAT WITH A BUILT-UP SLOPE ABOVE THE SHEATHING OR THE TOP OF RAFTERS SHALL BE RIPPED PER ARCHITECT (¼" PER 1'-0" REQUIRED).

DRAG STRAP 42

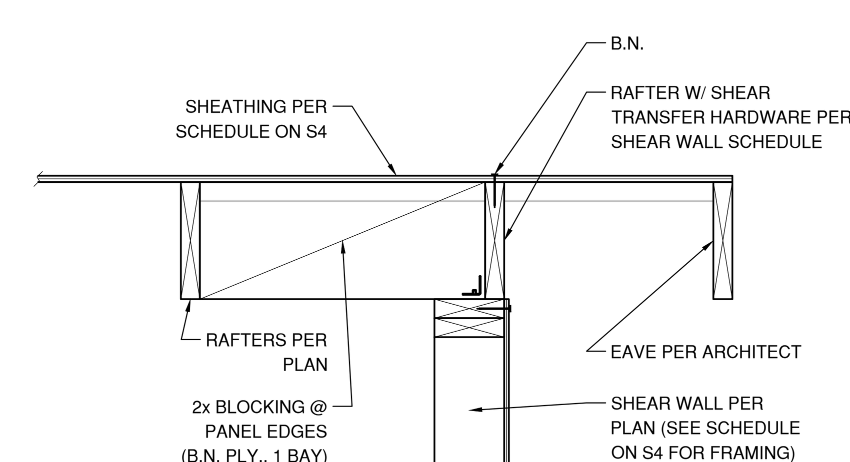


NOTES:
- RAFTERS SHALL EITHER BE BUILT FLAT WITH A BUILT-UP SLOPE ABOVE THE SHEATHING OR THE TOP OF RAFTERS SHALL BE RIPPED PER ARCHITECT (¼" PER 1'-0" REQUIRED).
- MAXIMUM PARAPET HEIGHT SHALL BE 18".

WOOD ROOF DECK 39



WOOD ROOF 36



WOOD ROOF 33

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APPENDIX B

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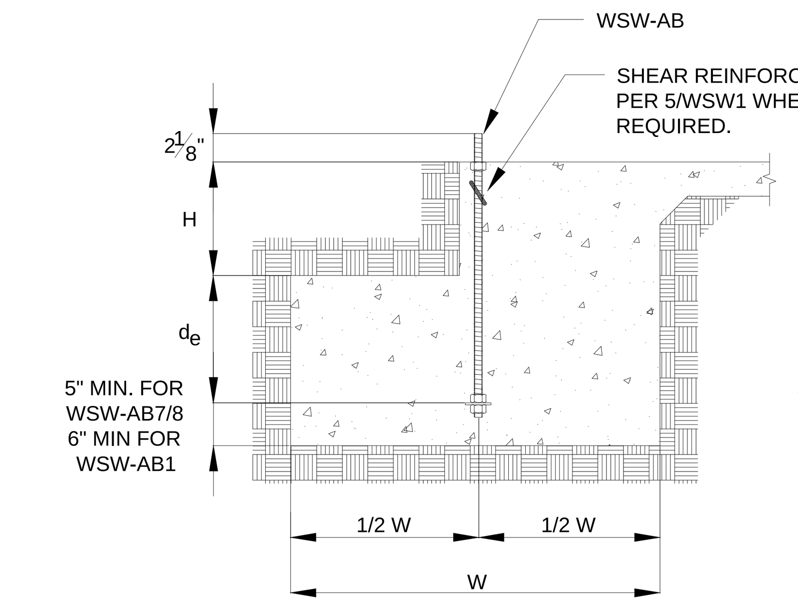
REVISIONS

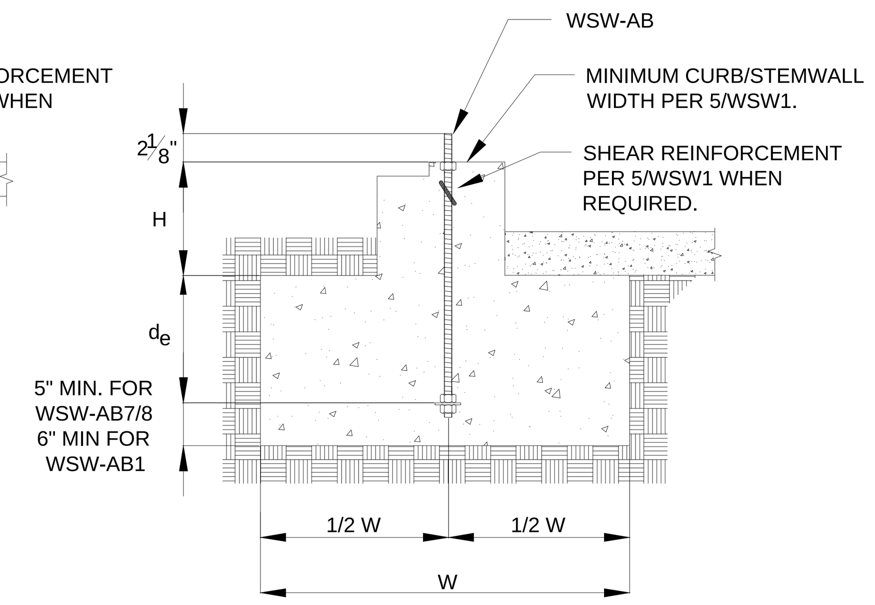
PROJECT #: 17-046
ENGINEER: H.R.
DATE: 08/07/2017
SCALE: N.T.S.

STRUCTURAL DETAILS

S8



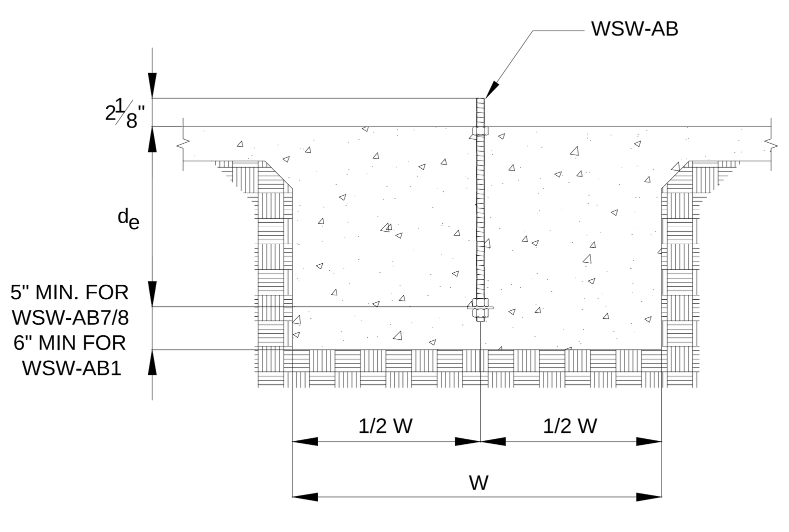
SLAB ON GRADE FOUNDATION



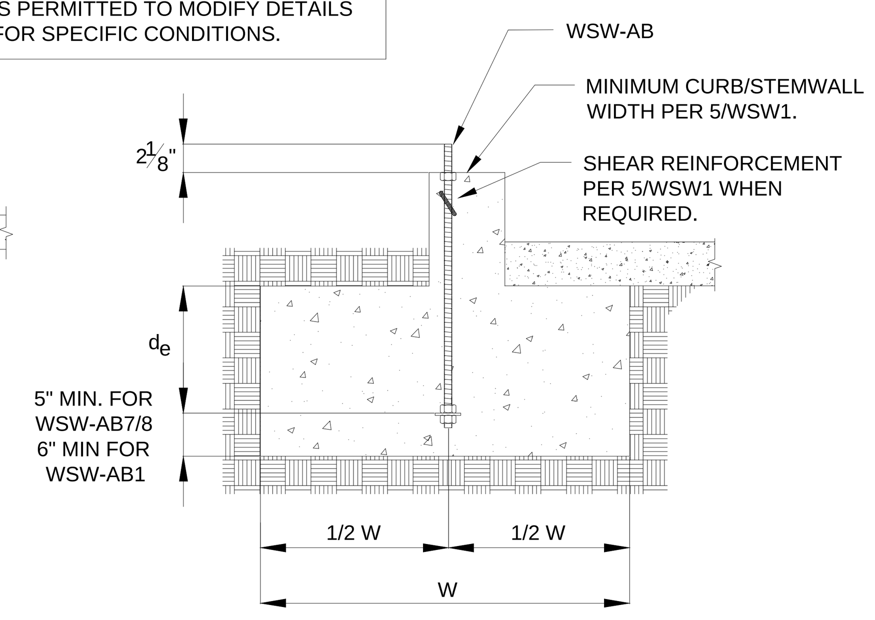
BRICK LEDGE FOUNDATION

NOTES:
1. SEE 2/WSW1 FOR DIMENSIONS AND ADDITIONAL NOTES.
2. SEE 5/WSW1 FOR SHEAR REINFORCEMENT WHEN REQUIRED.
3. MAXIMUM H = l_e - d_e. SEE 3/WSW1 AND 4/WSW1 FOR l_e.

REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.

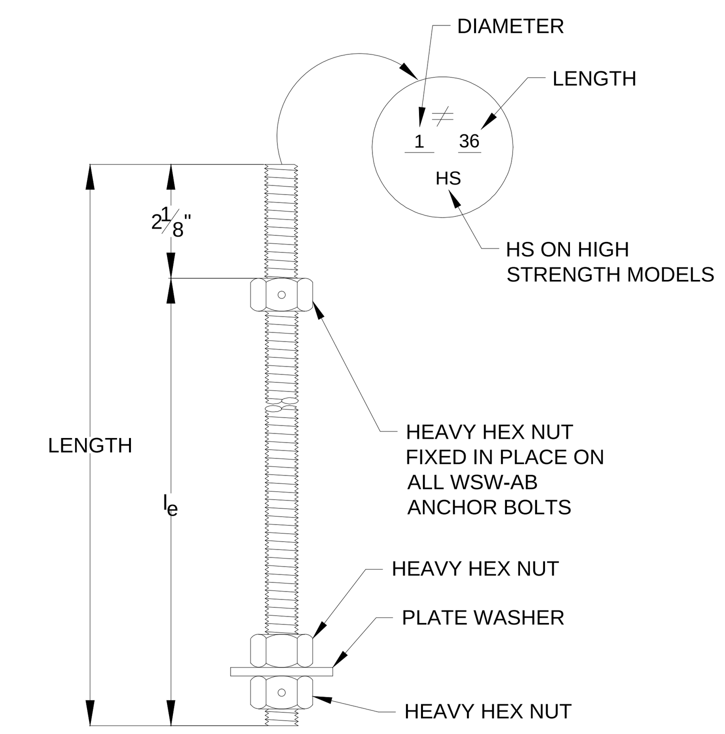


INTERIOR FOUNDATION



CURB OR STEMWALL FOUNDATION

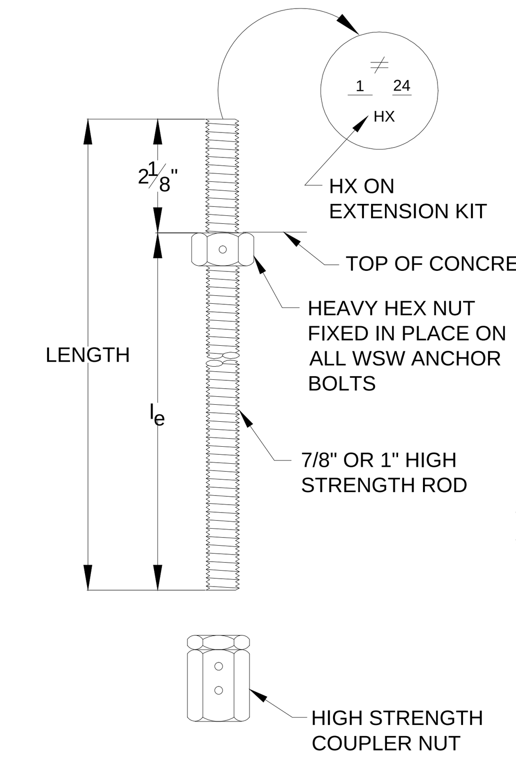
STRONG-WALL® WSW ANCHORAGE - TYPICAL SECTIONS | 1



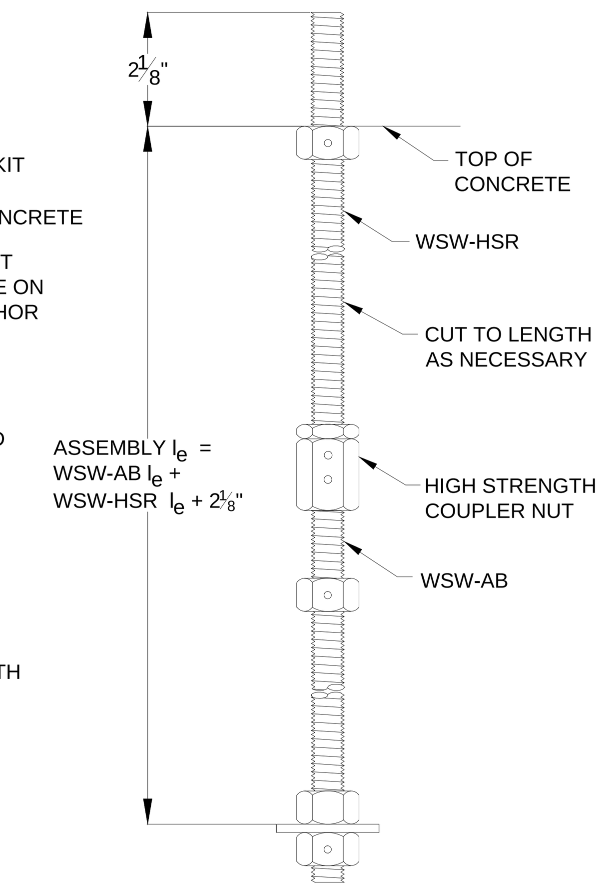
WSW PANEL MODEL	MODEL NO.	DIAMETER	LENGTH	l_e
WSW12 AND WSW18	WSW-AB7/8x24	7/8"	24"	20"
	WSW-AB7/8x24HS	7/8"	24"	20"
	WSW-AB7/8x30	7/8"	30"	26"
	WSW-AB7/8x30HS	7/8"	30"	26"
	WSW-AB7/8x36HS	7/8"	36"	32"
WSW24	WSW-AB1x24	1"	24"	20"
	WSW-AB1x24HS	1"	24"	20"
	WSW-AB1x30	1"	30"	26"
	WSW-AB1x30HS	1"	30"	26"
	WSW-AB1x36HS	1"	36"	32"

WSW ANCHOR BOLTS | 3

WSW-HSR_KT

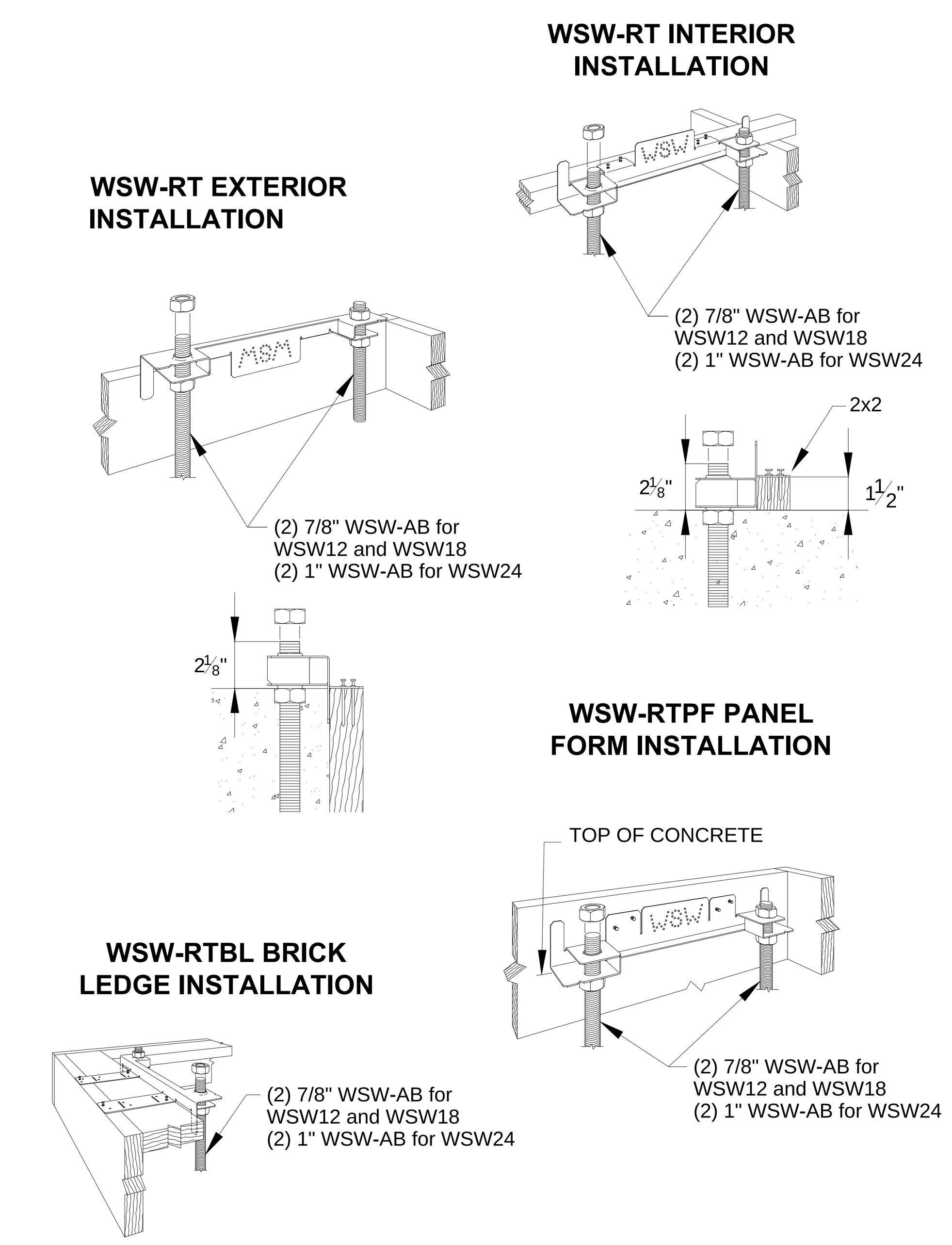


WSW-HSR AND WSW-AB ASSEMBLY

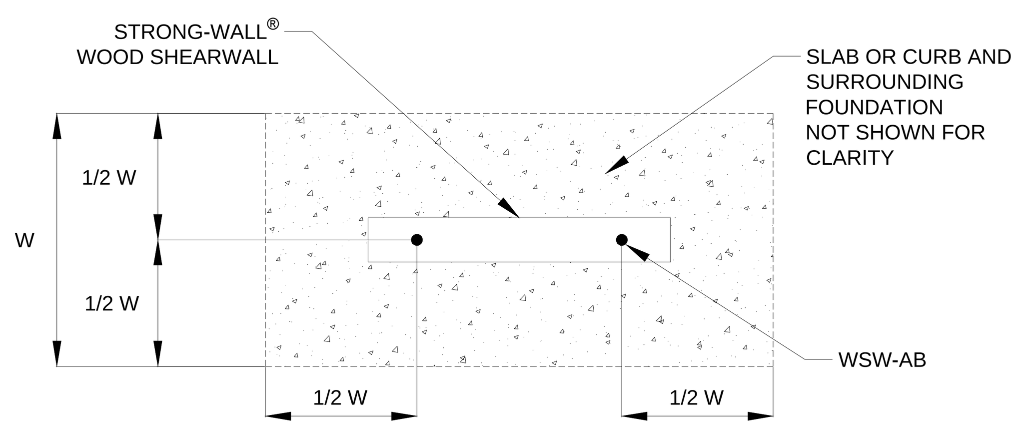


WSW PANEL MODEL	MODEL NO.	DIAMETER	LENGTH	l_e
WSW12 AND WSW18	WSW-HSR7/8x24KT	7/8"	24"	22"
	WSW-HSR7/8x36KT	7/8"	36"	34"
WSW24	WSW-HSR1x24KT	1"	24"	22"
	WSW-HSR1x36KT	1"	36"	34"

WSW ANCHOR BOLT EXTENSION | 4



WSW ANCHOR BOLT TEMPLATES | 6



FOUNDATION PLAN VIEW

WSW ANCHORAGE SOLUTIONS FOR 2500 PSI CONCRETE

DESIGN CRITERIA	CONCRETE CONDITION	ANCHOR STRENGTH	WSW-AB7/8 ANCHOR BOLT: ASD ALLOWABLE TENSION (lb.)	WSW-AB7/8: W (in.)	WSW-AB7/8: d_e (in.)	WSW-AB1 ANCHOR BOLT: ASD ALLOWABLE TENSION (lb.)	WSW-AB1: W (in.)	WSW-AB1: d_e (in.)
SEISMIC	CRACKED	STANDARD	11,900	27	9	16,100	33	11
			13,100	29	10	17,100	35	12
		HIGH STRENGTH	24,900	43	15	33,000	51	17
			27,100	46	16	35,300	54	18
	UNCRACKED	STANDARD	12,500	24	8	15,700	28	10
			13,100	25	9	17,100	30	10
		HIGH STRENGTH	25,300	38	13	32,300	44	15
			27,100	40	14	35,300	47	16
WIND	CRACKED	STANDARD	5,100	14	6	6,200	16	6
			8,700	20	7	11,400	24	8
			13,100	27	9	17,100	32	11
		HIGH STRENGTH	15,900	30	10	21,100	36	12
			18,400	33	11	27,300	42	14
			23,100	38	13	31,800	46	16
			27,100	42	14	35,300	50	17
	UNCRACKED	STANDARD	5,000	12	6	6,400	14	6
			9,300	18	6	12,500	22	8
			13,100	23	8	17,100	28	10
		HIGH STRENGTH	15,200	25	9	21,900	32	11
			19,900	30	10	26,400	36	12
			24,000	34	12	31,500	40	14
			27,100	37	13	35,300	43	15

NOTES:
1. ANCHORAGE DESIGNS CONFORM TO ACI 318-11 APPENDIX D AND ACI 318-14 WITH NO SUPPLEMENTARY REINFORCEMENT FOR CRACKED OR UNCRACKED CONCRETE AS NOTED.
2. ANCHOR STRENGTH INDICATES REQUIRED GRADE OF WSW-AB ANCHOR BOLT. STANDARD (ASTM F1554 GRADE 36) OR HIGH STRENGTH (HS) (ASTM A449).
3. SEISMIC INDICATES SEISMIC DESIGN CATEGORY C - F. DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C MAY USE WIND ANCHORAGE SOLUTIONS. SEISMIC ANCHORAGE DESIGNS CONFORM TO ACI 318-11 SECTION D.3.3.4.3 AND ACI 318-14 SECTION 17.2.3.4.3.
4. WIND INCLUDES SEISMIC DESIGN CATEGORY A AND B AND DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C.
5. FOUNDATION DIMENSIONS ARE FOR ANCHORAGE ONLY. FOUNDATION DESIGN (SIZE AND REINFORCEMENT) BY OTHERS. THE REGISTERED DESIGN PROFESSIONAL MAY SPECIFY ALTERNATE EMBEDMENT, FOOTING SIZE OR ANCHOR BOLT.
6. REFER TO 1/WSW1 FOR d_e.

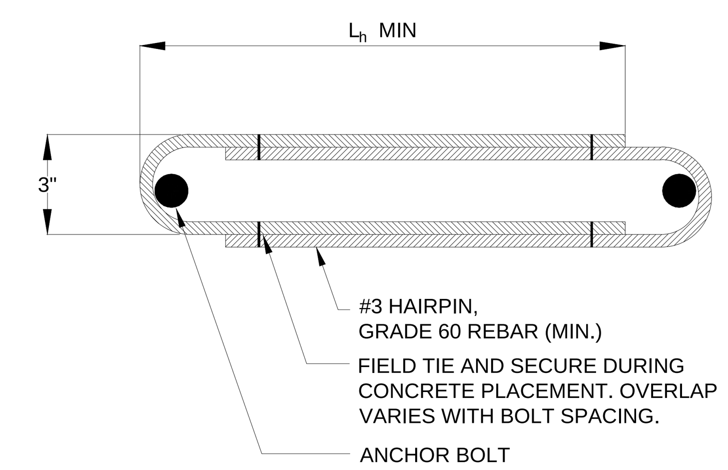
WSW ANCHORAGE SOLUTIONS FOR 3000 PSI CONCRETE

DESIGN CRITERIA	CONCRETE CONDITION	ANCHOR STRENGTH	WSW-AB7/8 ANCHOR BOLT: ASD ALLOWABLE TENSION (lb.)	WSW-AB7/8: W (in.)	WSW-AB7/8: d_e (in.)	WSW-AB1 ANCHOR BOLT: ASD ALLOWABLE TENSION (lb.)	WSW-AB1: W (in.)	WSW-AB1: d_e (in.)
SEISMIC	CRACKED	STANDARD	12,300	26	9	16,000	31	11
			13,100	28	10	17,100	33	11
		HIGH STRENGTH	25,200	41	14	32,700	48	16
			27,100	43	15	35,300	51	17
	UNCRACKED	STANDARD	12,000	22	8	16,300	27	9
			13,100	24	8	17,100	28	10
		HIGH STRENGTH	25,300	36	12	32,700	42	14
			27,100	38	13	35,300	44	15
WIND	CRACKED	STANDARD	5,000	13	6	5,600	14	6
			8,800	19	7	10,200	21	7
			13,100	25	9	17,100	30	10
		HIGH STRENGTH	15,700	28	10	20,100	33	11
			19,200	32	11	25,300	38	13
			23,200	36	12	32,300	44	15
			27,100	40	14	35,300	47	16
	UNCRACKED	STANDARD	5,500	12	6	6,200	13	6
			8,500	16	6	12,800	21	7
			13,100	22	8	17,100	26	9
		HIGH STRENGTH	16,600	25	9	21,800	30	10
			19,700	28	10	25,200	33	11
			24,000	32	11	31,700	38	13
			27,100	35	12	35,300	41	14

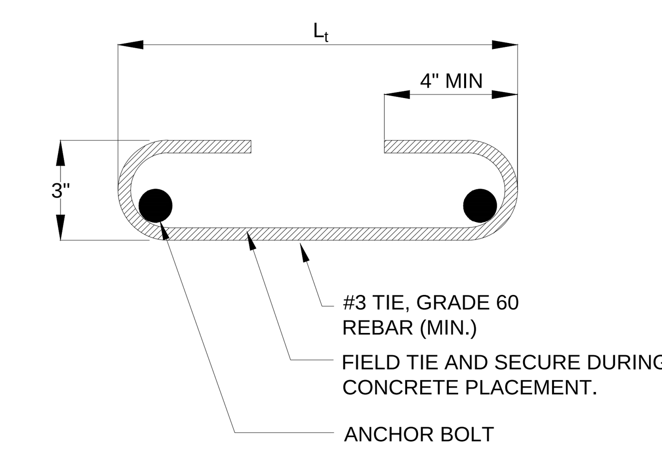
WSW ANCHORAGE SOLUTIONS FOR 4500 PSI CONCRETE

DESIGN CRITERIA	CONCRETE CONDITION	ANCHOR STRENGTH	WSW-AB7/8 ANCHOR BOLT: ASD ALLOWABLE TENSION (lb.)	WSW-AB7/8: W (in.)	WSW-AB7/8: d_e (in.)	WSW-AB1 ANCHOR BOLT: ASD ALLOWABLE TENSION (lb.)	WSW-AB1: W (in.)	WSW-AB1: d_e (in.)
SEISMIC	CRACKED	STANDARD	12,600	23	8	16,000	27	9
			13,100	24	8	17,100	29	10
		HIGH STRENGTH	24,800	36	12	32,100	42	14
			27,100	38	13	35,300	45	15
	UNCRACKED	STANDARD	12,700	20	7	15,700	23	8
			13,100	21	7	17,100	25	9
		HIGH STRENGTH	24,600	31	11	32,500	37	13
			27,100	34	12	35,300	39	13
WIND	CRACKED	STANDARD	5,400	12	6	6,800	14	6
			8,300	16	6	11,600	20	7
			13,100	22	8	17,100	26	9
		HIGH STRENGTH	15,300	24	8	21,400	30	10
			19,300	28	10	25,800	34	12
			23,600	32	11	31,000	38	13
			27,100	36	12	35,300	42	14
	UNCRACKED	STANDARD	6,800	12	6	6,800	12	6
			9,400	15	6	12,400	18	6
			13,100	19	7	17,100	23	8
		HIGH STRENGTH	16,800	22	8	21,600	26	9
			20,300	25	9	26,700	30	10
			24,100	28	10	32,200	34	12
			27,100	31	11	35,300	36	12

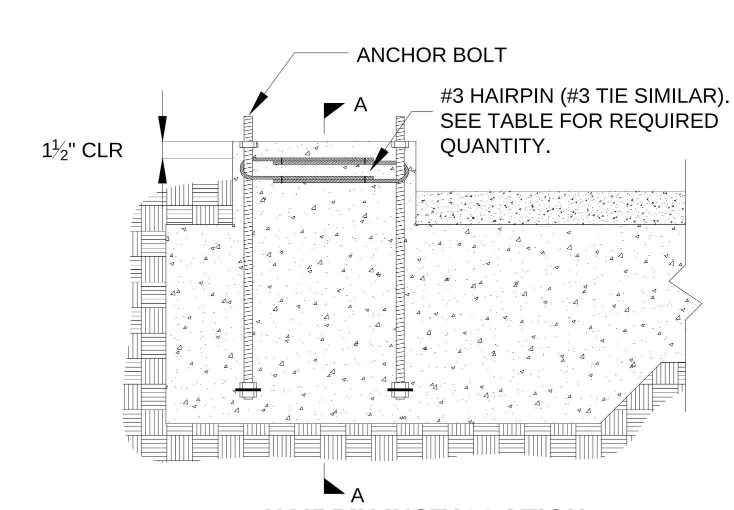
STRONG-WALL® WOOD SHEARWALL TENSION ANCHORAGE SCHEDULE 2,500, 3,000 AND 4,500 PSI | 2



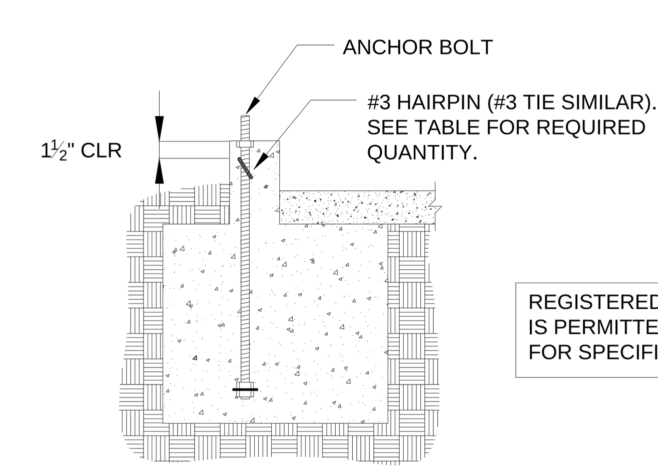
HAIRPIN SHEAR REINFORCEMENT



TIE SHEAR REINFORCEMENT



HAIRPIN INSTALLATION
(GARAGE CURB SHOWN. OTHER FOOTING TYPES SIMILAR.)



SECTION A-A

REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.

STRONG-WALL® WOOD SHEARWALL SHEAR ANCHORAGE

MODEL	L_t OR L_h (in.)	SEISMIC [3]: SHEAR REINFORCEMENT	SEISMIC [3]: MINIMUM CURB/ STEMWALL WIDTH (in.)	WIND [4]: SHEAR REINFORCEMENT	WIND [4]: MINIMUM CURB/ STEMWALL WIDTH (in.)	WIND [4]: ASD ALLOWABLE SHEAR LOAD, V (lb.) [6]: UNCRACKED	WIND [4]: ASD ALLOWABLE SHEAR LOAD, V (lb.) [6]: CRACKED
WSW12	$10\frac{1}{4}$	(1) #3 HAIRPIN	8[5]	SEE NOTE 6	6	1,035	740
WSW18	15	(1) #3 HAIRPIN	8[5]	(1) #3 HAIRPIN	6	HAIRPIN REINFORCEMENT ACHIEVES MAXIMUM ALLOWABLE SHEAR LOAD OF THE WSW	
WSW24	19	(2) #3 HAIRPINS	8[5]	(1) #3 HAIRPIN	6		

NOTES:
1. SHEAR ANCHORAGE DESIGNS CONFORM TO ACI 318-11 AND ACI 318-14 AND ASSUME MINIMUM 2,500 PSI CONCRETE.
2. SHEAR REINFORCEMENT IS NOT REQUIRED FOR INTERIOR FOUNDATION APPLICATIONS (PANEL INSTALLED AWAY FROM EDGE OF CONCRETE), OR BRACED WALL PANEL APPLICATIONS.
3. SEISMIC INDICATES SEISMIC DESIGN CATEGORY C THROUGH F. DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C MAY USE WIND ANCHORAGE SOLUTIONS.
4. WIND INCLUDES SEISMIC DESIGN CATEGORY A AND B AND DETACHED 1 AND 2 FAMILY DWELLINGS IN SDC C.
5. WHERE NOTED, MINIMUM CURB/STEMWALL WIDTH IS 6 INCHES WHEN STANDARD STRENGTH ANCHOR BOLT IS USED.
6. USE (1) #3 TIE FOR WSW12 WHEN PANEL DESIGN SHEAR FORCE EXCEEDS TABULATED ANCHORAGE ALLOWABLE SHEAR LOAD.
7. #4 GRADE 40 SHEAR REINFORCEMENT MAY BE SUBSTITUTED FOR WSW SHEAR ANCHORAGE SOLUTIONS.

STRONG-WALL® WSW SHEAR ANCHORAGE SCHEDULE AND DETAILS | 5

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0	07/01/2016	FIRST RELEASE - 2015 IBC

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5956 W. LAS POSITAS BLVD.
PLEASANTON, CA 94588
TEL: (800) 999-5099

SIMPSON Strong-Tie
THERE IS NO EQUAL

STRONG-WALL® WSW
ANCHORAGE DETAILS
ENGINEERED DESIGNS

SIMPSON Strong-Tie
THERE IS NO EQUAL

NAME
DATE 07-01-2016
SCALE N.T.S.
CHECKED
SHEET
WSW1
OF SHEETS
JOB NO.

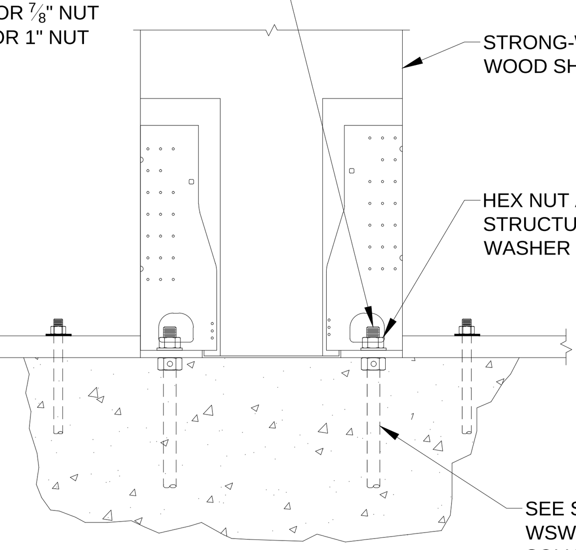
STRONG-WALL® WOOD SHEARWALL MODELS

MODEL NO.	W (in.)	H (in.)	ANCHOR BOLTS		TOTAL WALL WEIGHT (lb.)
			QUANTITY	DIA. (in.)	
WSW12x7	12	78	2	7/8	100
WSW18x7	18	78	2	7/8	145
WSW12x7.5	12	85 1/2	2	7/8	110
WSW18x7.5	18	85 1/2	2	7/8	155
WSW12x8	12	93 1/4	2	7/8	115
WSW18x8	18	93 1/4	2	7/8	165
WSW24x8	24	93 1/4	2	1	225
WSW12x9	12	105 1/4	2	7/8	130
WSW18x9	18	105 1/4	2	7/8	185
WSW24x9	24	105 1/4	2	1	245
WSW12x10	12	117 1/4	2	7/8	140
WSW18x10	18	117 1/4	2	7/8	205
WSW24x10	24	117 1/4	2	1	270
WSW12x11	12	129 1/4	2	7/8	150
WSW18x11	18	129 1/4	2	7/8	220
WSW24x11	24	129 1/4	2	1	295
WSW12x12	12	141 1/4	2	7/8	165
WSW18x12	18	141 1/4	2	7/8	240
WSW24x12	24	141 1/4	2	1	320
WSW18x13	18	153 1/4	2	7/8	255
WSW24x13	24	153 1/4	2	1	345
WSW24x14	24	168	2	1	375
WSW24x16	24	192	2	1	425
WSW18x20	18	240	2	7/8	385
WSW24x20	24	240	2	1	520

NOTES:
1. FOR HEIGHTS NOT LISTED, ORDER THE NEXT TALLEST PANEL AND TRIM TO FIT. MINIMUM TRIMMED HEIGHT FOR ALL PANELS IS 74½".
2. ALL PANELS COME WITH TWO PRE-ATTACHED HOLDOWNS, TWO STANDARD HEX NUTS, TWO STRUCTURAL WASHERS, TWO WSW-TOW PLATES AND INSTALLATION INSTRUCTIONS.
3. ALL PANELS ARE 3½" THICK.

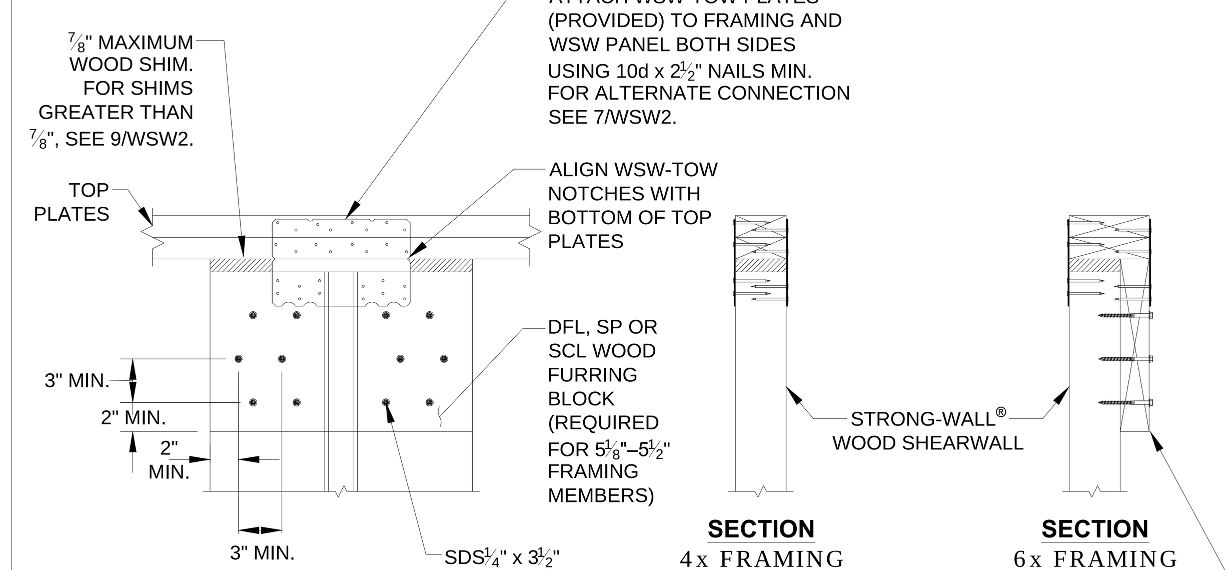
STRONG-WALL® WSW MODELS 1

REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.



STANDARD INSTALLATION BASE CONNECTION 4

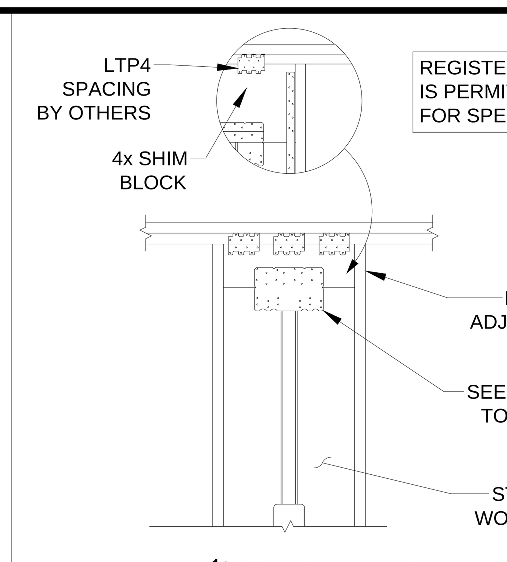
REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.

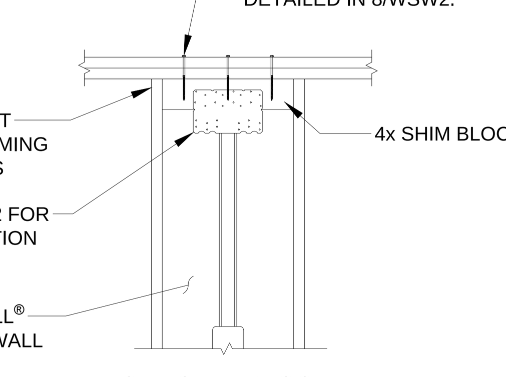


FOR 5 3/8"–5 1/2" FRAMING, ATTACH DFL, SP OR SCL WOOD FURRING BLOCK BETWEEN WSW PANEL AND WSW-TOW WITH SDS 1/4" x 3 1/2" SCREWS (MIN.) OR 16d COMMON NAILS (8 TOTAL FASTENERS FOR WSW12, 10 FOR WSW18 AND 14 FOR WSW24). MINIMUM BLOCK SIZE IS 1 3/4" x 11 7/8" x WSW PANEL WIDTH.

STANDARD TOP CONNECTION 6

REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.



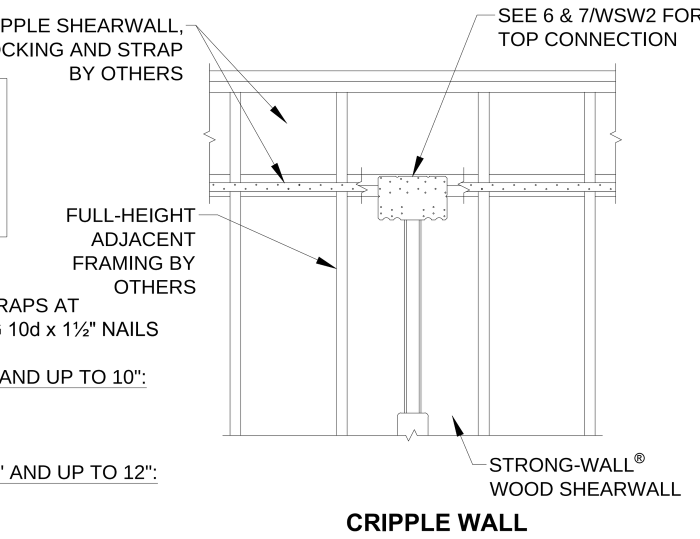


REGISTERED DESIGN PROFESSIONAL SHALL DESIGN AND DETAIL FOR :
1. SHEAR TRANSFER
2. OUT-OF-PLANE LOADING EFFECT
3. INCREASED OVERTURNING AND DRIFT DUE TO ADDITIONAL HEIGHT

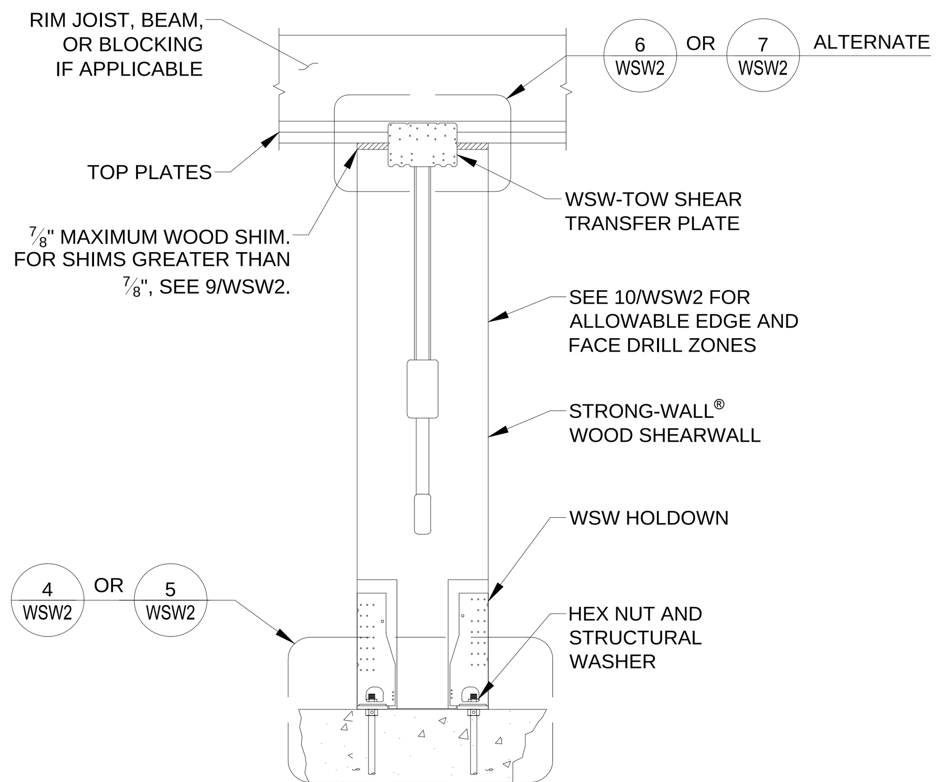
FOR 8" TO 12" BLOCK DEPTHS: ATTACH SIMPSON STRONG-TIE® CS16 STRAPS AT EDGE OF WSW PANEL (EACH SIDE) USING 10d x 1½" NAILS

SHIM BLOCK HEIGHTS GREATER THAN 8" AND UP TO 10":
- 8 NAILS INTO BLOCK
- 8 NAILS INTO WSW PANEL

SHIM BLOCK HEIGHTS GREATER THAN 10" AND UP TO 12":
- 10 NAILS INTO BLOCK
- 10 NAILS INTO WSW PANEL



TOP OF WALL HEIGHT ADJUSTMENTS 9

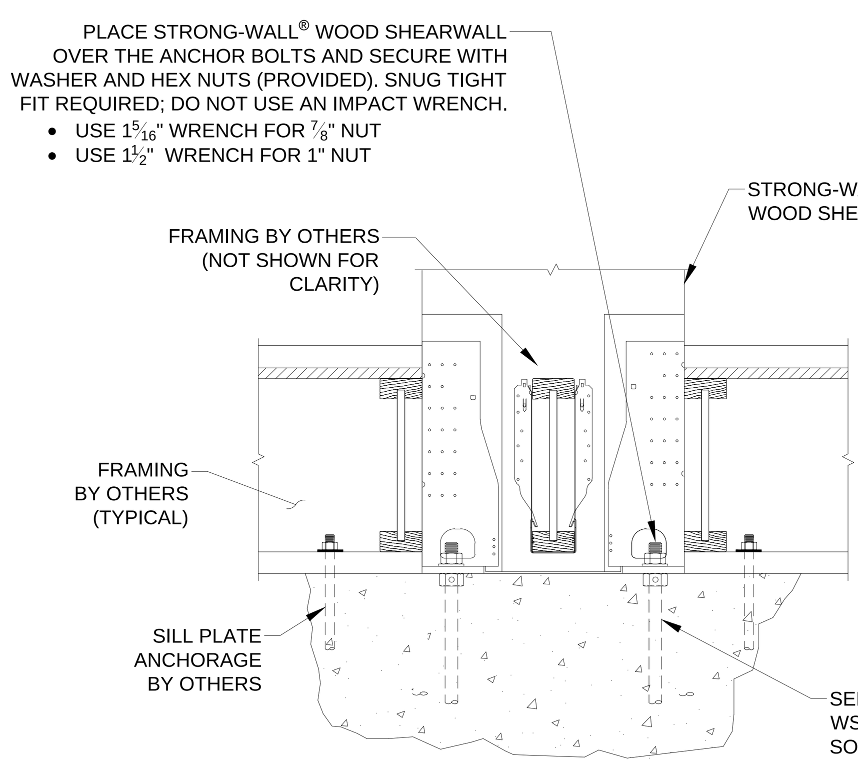


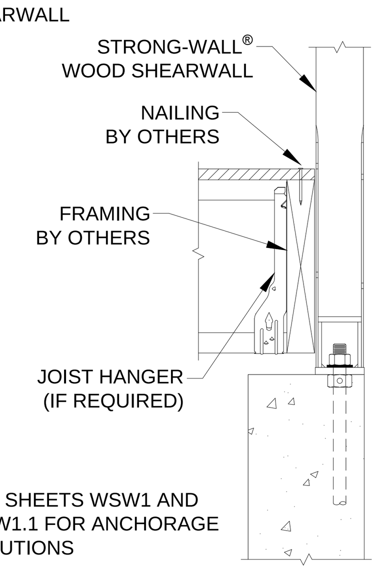
REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.

WSW DESIGNED TO PROVIDE 1/8" GAP BETWEEN LSL AT BASE OF WSW AND CONCRETE. ENSURE CONCRETE IS LEVEL AND SMOOTH BENEATH PANEL. GRIND OR FILL AS NECESSARY.

SINGLE STORY WSW ON CONCRETE 2

REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.



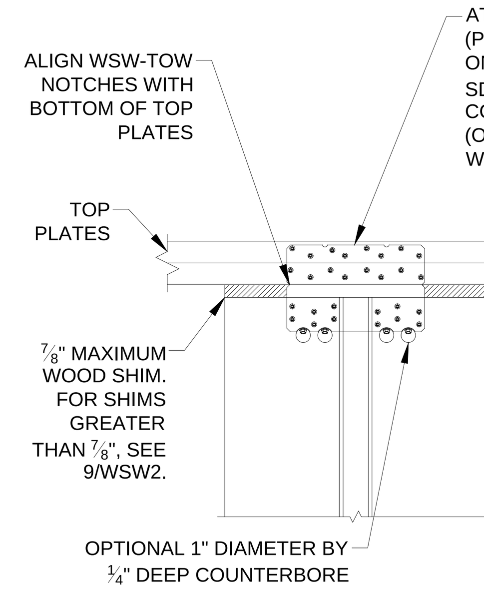


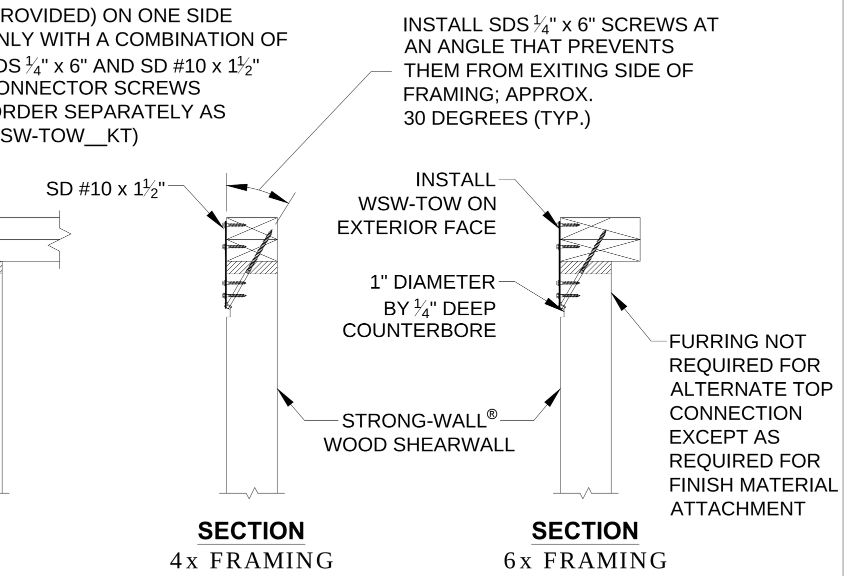
STRONG-WALL® WOOD SHEARWALL HEIGHT TO INCLUDE THE DEPTH OF THE FLOOR SYSTEM AND SHALL BE INSTALLED DIRECTLY ON THE FOUNDATION. SPECIFY PANEL HEIGHT FROM TOP OF FOUNDATION TO UNDERSIDE OF TOP PLATES OR BEAM.

WOOD FLOOR SYSTEM BASE CONNECTION 5

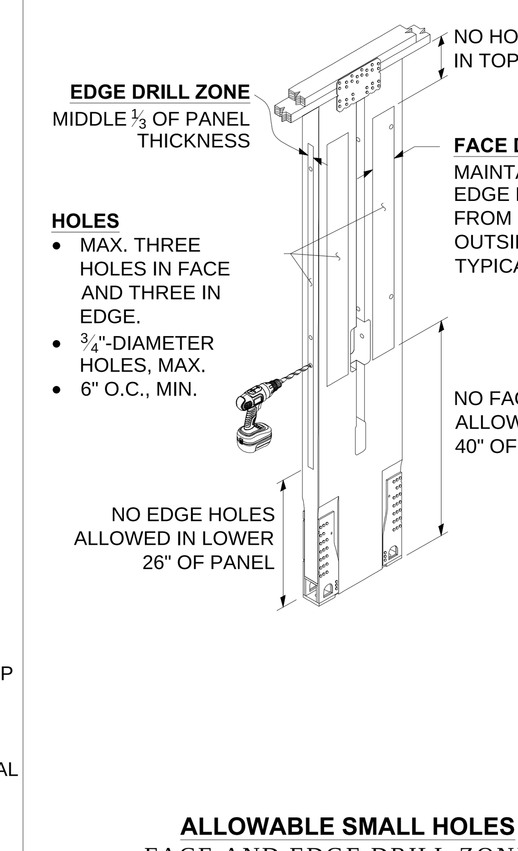
WSW-TOW ALTERNATE CONNECTION KIT		
MODEL NO.	FASTENER QUANTITY	
	SD #10 x 1½"	SDS ¼" x 6"
WSW-TOW12KT	20	2
WSW-TOW18KT	28	4
WSW-TOW24KT	40	8

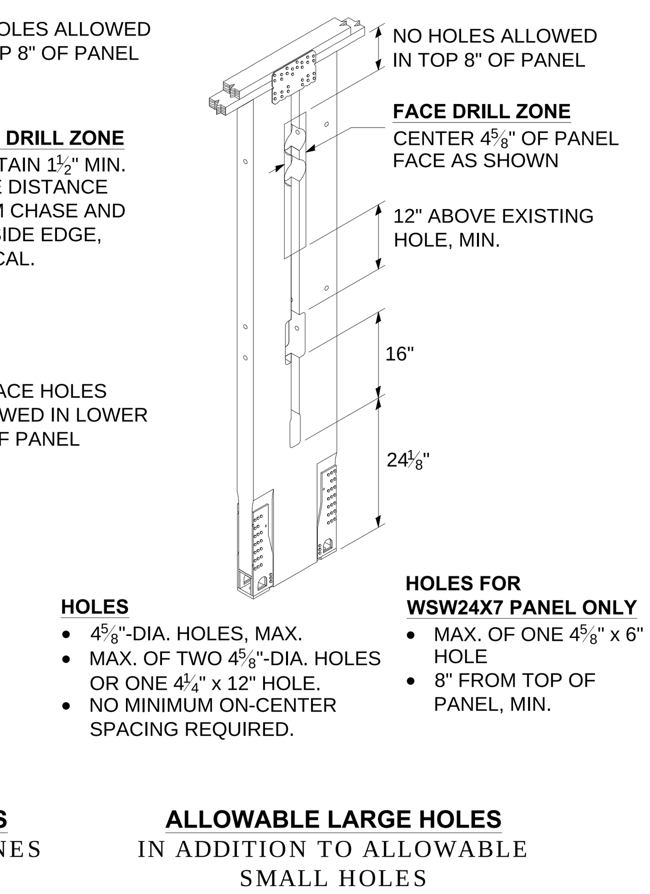
REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.





ALTERNATE TOP CONNECTION 7

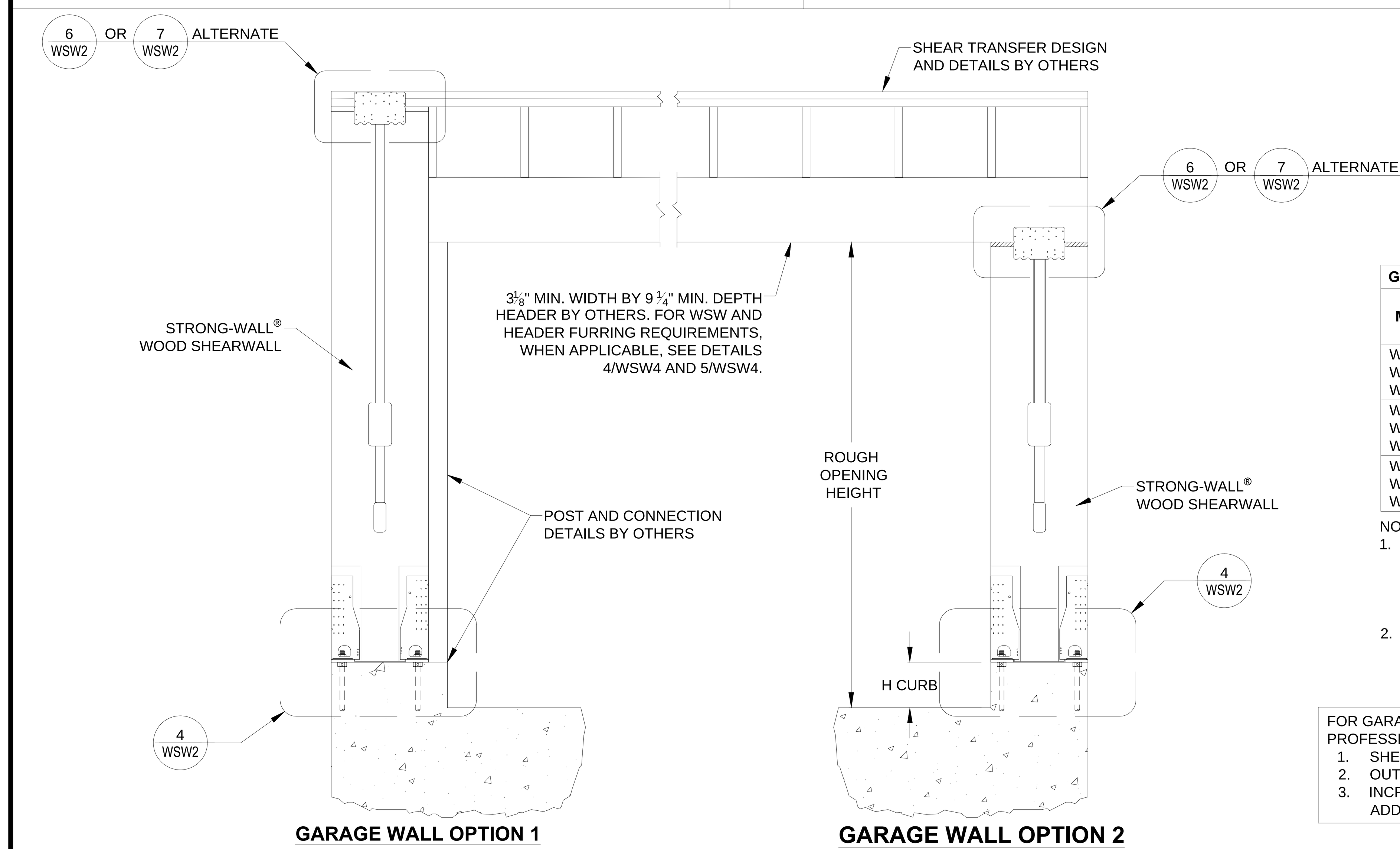




TRIM ZONE AND ALLOWABLE HOLES 10

REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.

WHEN WSW-PS STRAPS OMITTED, ALLOWABLE SHEAR VALUES FOR STANDARD PANEL APPLY.



GARAGE HEADER ROUGH OPENING HEIGHT		
MODEL NO.	H CURB	ROUGH OPENING HEIGHT
WSW12x7 WSW18x7 WSW24x7	5½"	6'-11½"
	6"	7'-0"
WSW12x7.5 WSW18x7.5 WSW24x7.5	0"	7'-1½"
WSW12x8 WSW18x8 WSW24x8	5½"	8'-2¾"
	6"	8'-3¼"

NOTES:
1. IF REQUIRED ROUGH OPENING HEIGHT EXCEEDS TABLE VALUE, SPECIFY NEXT TALLER PANEL AND TRIM AS NECESSARY. THE STRONG-WALL® WOOD SHEARWALL MAY BE TRIMMED TO A MINIMUM HEIGHT OF 74½".
2. FURRING DOWN GARAGE HEADER MAY BE REQUIRED FOR CORRECT ROUGH OPENING HEIGHT.

FOR GARAGE WALL OPTION 2, REGISTERED DESIGN PROFESSIONAL SHALL DESIGN AND DETAIL FOR :
1. SHEAR TRANSFER
2. OUT-OF-PLANE LOADING EFFECT
3. INCREASED OVERTURNING AND DRIFT DUE TO ADDITIONAL HEIGHT

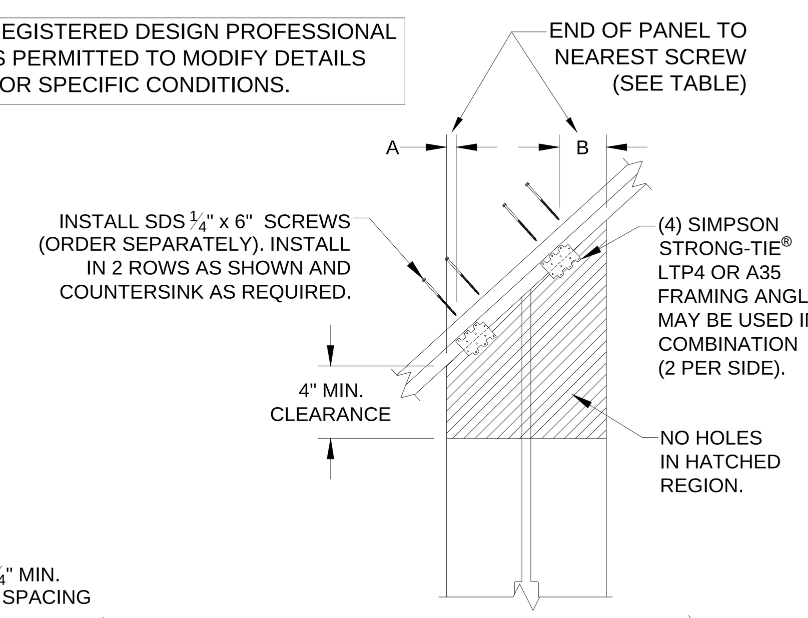
ALTERNATE WSW GARAGE FRONT OPTIONS 3

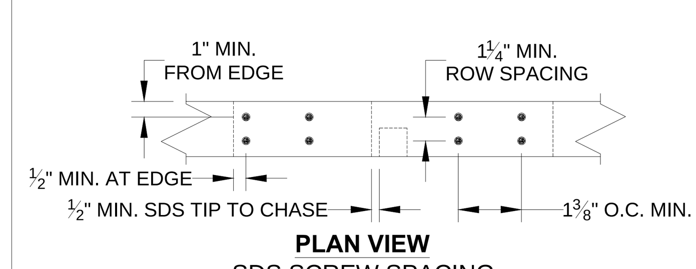
QTY. OF SDS¼" x 6" SCREWS REQ'D.	
WSW12	4
WSW18	8
WSW24	12

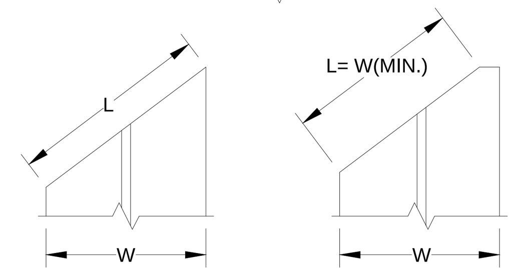
EDGE DISTANCE FOR SCREWS		
SLOPE	A (in.)	B (in.)
0:12-4:12	2	3
5:12-8:12	1 1/2	4 1/2
9:12-12:12	1/2	5 1/2

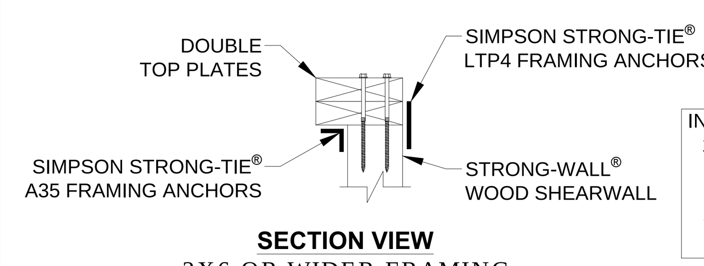
NOTES:
1. MAINTAIN END DISTANCES TO PREVENT SCREWS FROM PENETRATING THROUGH THE OUTER EDGES.
2. INSTALL SCREWS PERPENDICULAR TO THE TOP PLATE.
3. EDGE DISTANCES ASSUME DOUBLE TOP PLATE.

REGISTERED DESIGN PROFESSIONAL IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.









INSTALLATION NOTES :
1. ACTUAL CUT LENGTH (L) MUST BE GREATER THAN OR EQUAL TO PANEL WIDTH (W).
2. THIS DETAIL APPLICABLE FOR SLOPES UP TO 12:12.
3. PANELS TALLER THAN 12' MUST BE DESIGNED FOR THE APPLICATION.

RAKE WALL 8

1. STRONG-WALL WOOD SHEARWALL IS MANUFACTURED AND TRADEMARKED BY "SIMPSON STRONG-TIE COMPANY INC."HOME OFFICE: 5956 W. LAS POSITAS BLVD. , PLEASANTON, CA 94588 TEL: (800) 999-5099, FAX: (925) 847-1597. "SIMPSON STRONG-TIE COMPANY INC." IS AN ISO 9001-2008 REGISTERED COMPANY.
2. USE OF THIS PRODUCT IS SUBJECT TO THE APPROVAL OF THE LOCAL BUILDING DEPARTMENT.
3. THIS PRODUCT IS PART OF THE OVERALL LATERAL FORCE RESISTING SYSTEM OF THE STRUCTURE. DESIGN OF THE BUILDING'S LATERAL FORCE RESISTING SYSTEM, INCLUDING THE LOAD PATH TO TRANSFER LATERAL FORCES FROM THE STRUCTURE TO THE GROUND, IS THE RESPONSIBILITY OF THE DESIGNER.
4. ENGINEER OF RECORD IS PERMITTED TO MODIFY DETAILS FOR SPECIFIC CONDITIONS.
5. THE CONTRACTOR SHALL VERIFY ALL DIMENSIONS, CONDITIONS, ELEVATIONS, ETC. PRIOR TO INSTALLATION OF ANY COMPONENTS FOR THE STRONG-WALL SB SYSTEM. IF ANY DISCREPANCIES ARE FOUND, THEY SHALL BE BROUGHT TO THE ATTENTION OF THE DESIGNER FOR CLARIFICATION PRIOR TO CONSTRUCTION.
6. INSTALLATION OF PRODUCT SHALL BE DONE IN CONFORMANCE TO THESE DRAWINGS. THE PERFORMANCE OF MODIFIED PRODUCTS OR ALTERED INSTALLATION PROCEDURES ARE THE SOLE RESPONSIBILITY OF THE DESIGNER.
7. SIMPSON STRONG-TIE COMPANY INC. RESERVES THE RIGHT TO CHANGE SPECIFICATIONS, DESIGNS, AND MODELS WITHOUT NOTICE OR LIABILITY FOR SUCH CHANGES.
8. ALL HARDWARE CALLED OUT IS SIMPSON STRONG-TIE.
9. SEE ICC-ES ESR-2652 OR CITY OF LOS ANGELES RR25730 AS APPLICABLE FOR ADDITIONAL INFORMATION.

NOTES 11

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PLEASANTON, CA 94588
TEL: (800) 999-5099

STRONG-WALL WSW
FRAMING DETAILS
ENGINEERED DESIGNS

NAME
DATE 07-01-2016
SCALE N.T.S.
CHECKED
SHEET **WSW2**
OF SHEETS
JOB NO.

APPENDIX C : GEOTECHNICAL REPORT

LGC Geotechnical, Inc.

April 14, 2017

Project No. 17032-01

Mr. & Mrs. Brandon & Kylie Beauchemin
148 West Avenida Cadiz
San Clemente, California 92672

Subject: ***Geotechnical Evaluation, Proposed Building Addition and Remodel, 148 West Avenida Cadiz, San Clemente, California***

Introduction

In accordance with your request, LGC Geotechnical, Inc. (LGC Geotechnical) has performed a geotechnical evaluation for the proposed building addition and remodel of the residential property located at 148 West Avenida Cadiz in the city of San Clemente, California. The purpose of our study was to evaluate the site geotechnical conditions in the area of the proposed addition and remodel and to provide appropriate geotechnical design parameters and recommendations for the project. This report presents the results of our evaluation and geotechnical analyses, and provides a summary of our conclusions and recommendations relative to the proposed site improvements.

Should you have any questions regarding this report, please do not hesitate to contact our office. We appreciate this opportunity to be of service.

Respectfully,

LGC Geotechnical, Inc.

REGISTERED PROFESSIONAL ENGINEER
BENJAMIN R. GRENIS
No. C-83971
CIVIL
STATE OF CALIFORNIA

Benjamin R. Grenis, RCE 83971
Senior Staff Engineer

PROFESSIONAL GEOLOGIST
KEVIN B. COLSON
No. 2210
CERTIFIED ENGINEERING GEOLOGIST
EXP. 7/31/18
STATE OF CALIFORNIA

Kevin B. Colson, CEG 2210
Vice President

KBC/BRG/aca

Distribution: (5) Addressee (wet-signed copies)

APPENDIX C

131 Calle Iglesia, Suite 200, San Clemente, CA 92672 (949) 369-6141 www.lgcgeotechnical.com

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APPENDIX C

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Figure 1 – Site Location Map (Rear of Text)
Figure 2 – Typical French Drain Detail (Rear of Text)

Appendices

Appendix A – References
Appendix B – Laboratory Testing Procedures and Test Results
Appendix C – General Earthwork and Grading Specifications for Rough Grading

1.0 INTRODUCTION

LGC Geotechnical has performed a geotechnical evaluation for the proposed residential building addition and remodel of the property located at 148 West Avenida Cadiz in San Clemente, California (Figure 1). This report summarizes our findings, conclusions, and geotechnical design recommendations relative to the proposed improvements.

1.1 Project Description and Background

The site consists of a rectangular-shaped lot occupied by a single-story residence and associated improvements. The site was originally developed in 1955 as Lot 26 of Tract 822. Details regarding grading and construction of the structure were not available. The structure is in relatively good condition with no reports or obvious indication of geotechnical distress. Topographically, the site is relatively flat with a gentle incline of the lot to the west corner. The site is adjacent to similar residential properties to the north, west and south. The property fronts West Avenida Cadiz to the east.

We understand that the proposed development of the site will include construction of a single-story addition wrapping the northern corner of the existing structure. We further understand that the interior of the existing structure will be remodeled, a deck will be constructed west of the structure, and the existing garage slab will likely be replaced.

1.2 Evaluation & Laboratory Testing

A site visit was performed on March 28, 2016 to observe the site geotechnical conditions. The in-situ soils in the area of the proposed building addition were probed from the surface during our site visit with a T-handled probe and found to be generally firm and unyielding at a depth of approximately 6 inches below the surface. Two hand-excavated test pits were excavated adjacent to the existing stem wall foundation of the structure in the crawl space beneath the residence. The test pits were excavated in order to measure the embedment depth of the perimeter stem wall. The footing embedment observed indicated 27 to 37 inches of embedment for the perimeter stem wall on the exterior of the structure and approximately 9 inches on the interior.

Laboratory testing was performed on a representative bulk sample to evaluate the soil characteristics and to aid in the development of our foundation design recommendations. Laboratory test results are presented in Appendix B.

1.3 Site Geologic Conditions

The site is located on the southwestern border of the Peninsular Ranges at the southwestern-most portion of the Los Angeles Basin. Specifically, the site lies on the western flank of the sedimentary basin known as the Capistrano Embayment, an early Cenozoic Seaway, which trended northerly between the Peninsular Ranges and a hypothetical Catalina uplift off the Southern California coast. Locally, the Capistrano Embayment refers to the flat-bottomed structural trough formed by the downward displacement along the west side of the Christianitos Fault and down warping along the east side of the San Joaquin Hills. The embayment was subsequently in-filled with marine siltstone and clayey siltstone bedrock of the late Miocene to early Pliocene (approximately 5 to 15 million years old) Capistrano Formation. This sedimentary unit, in excess of 3,000 feet thick near the center of the embayment was uplifted, folded and eroded in Pliocene and post-Pliocene times (approximately 2 to 3 million years ago) producing the low, rolling ridges observed today. More recently, the local geology has also been influenced by a rapid drop in sea level resulting in a series of wave-cut marine platforms along the coast. This caused extensive erosion, which in turn, created numerous steep-sided drainage channels and over-steepened slopes.

Per the regional geologic map of the area, the site is underlain by Quaternary older marine and non-marine terrace deposits above a wave-cut platform, cut into underlying Tertiary Capistrano Formation bedrock material at depth (CDMG, 1999). Minor artificial fill is also likely present on the site. Based on our field observations, the material observed included fine sand, clayey silt and silty clay.

The site is not located within a mapped Earthquake Fault-Rupture Hazard Zone per compiled maps released by the CDMG (2000 and 2007), and no known active or potentially active faults cross the site. The site is not located within a mapped zone considered susceptible to seismically-induced slope instability or within a mapped zone considered susceptible to seismically-induced liquefaction (CDMG, 2002b).

1.4 Geologic Structure

Geologic structure was not identified in the subject site geotechnical evaluations and is not likely to be an issue for the proposed remodel and addition. The regional geologic map of the area (CDMG, 1999) indicates that the Capistrano Formation in the vicinity of the site dips gently and variably, with bedding attitudes from 5 degrees to the north to 12 degrees to the south. Where bedding is present in the site terrace deposits, it is anticipated to be generally flat-lying.

1.5 Landslides and Slope Stability

Sloping conditions are not present in the vicinity of the site. Document research and field observations do not indicate the presence of landslides on the site or in the immediate vicinity (CDMG, 1999). Review of the Seismic Hazards Zone Map (CDMG, 2002b) and the Seismic Hazard Zone Report (CDMG, 2002a) for the San Clemente 7.5 Minute Quadrangle indicates that the site is not located within a mapped area considered potentially susceptible to seismically-induced slope instability.

1.6 Groundwater

Shallow groundwater was not encountered during our subsurface evaluation and is generally not anticipated in the site vicinity. Groundwater is not expected to be encountered for the proposed project.

Groundwater and/or groundwater seepage conditions may occur in the future due to changes in land use and/or following periods of heavy rain. Seasonal fluctuations of groundwater elevations should be expected over time. In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local landscape irrigation or precipitation especially during rainy seasons.

1.7 <u>Faulting</u>

California is located on the boundary between the Pacific and North American Lithospheric Plates. The average motion along this boundary is on the order of 50-mm/yr in a right-lateral sense. The majority of the motion is expressed at the surface along the northwest trending San Andreas Fault Zone with lesser amounts of motion accommodated by sub-parallel faults located predominantly west of the San Andreas Fault including the San Jacinto, Elsinore, and Newport-Inglewood Faults. Within Southern California, a large bend in the San Andreas Fault north of the San Gabriel Mountains has resulted in a transfer of a portion of the right-lateral motion between the plates into left-lateral displacement and vertical uplift. Compression south and west of the bend has resulted in folding, left-lateral, reverse thrust faulting, and regional uplift creating the east-west trending Transverse Ranges and several east-west trending faults. Further south within the Los Angeles Basin, “blind thrust” faults are believed to have developed below the surface also as a result of this compression, which have resulted in earthquakes such as the 1994 Northridge event along faults with little to no surface expression.

Prompted by damaging earthquakes in Northern and Southern California, State legislation and policies concerning the classification and land-use criteria associated with faults have been developed. Their purpose was to prevent the construction of urban developments across the trace of active faults. The result is the Alquist-Priolo Earthquake Fault Zoning Act, which was most recently revised in 2007 (CGS, 2007). According to the State Geologist, an active fault is defined as one, which has had surface displacement within the Holocene Epoch (roughly the last 11,000 years). A potentially active fault is defined as any fault, which has had surface displacement during Quaternary time (last 1,600,000 years), but not within the Holocene. Earthquake Fault Zones have been delineated along the traces of active faults within California. Where developments for human occupation are proposed within these zones, the state requires detailed fault investigations be performed so that engineering geologists can mitigate the hazards associated with active faulting by identifying the location of active faults and allowing for a setback from the zone of previous ground rupture.

The subject site is not located within a Fault Rupture Hazard Zone and there are no active or potentially active faults mapped on the site. The possibility of damage due to ground rupture, as a result of faulting, is considered very low since active faults are not known to cross the site.

Secondary effects of seismic shaking resulting from large earthquakes on the major faults in the Southern California region include soil liquefaction, dynamic settlement, ground lurching, shallow ground rupture, and seiches and tsunamis. These secondary effects of seismic shaking are a possibility throughout portions of the Southern California region and are dependant on the distance between the site and causative fault and the onsite geology. Parameters for seismic design are included in the sections below. The major active nearby faults that could produce these secondary effects include the off-shore Newport-Inglewood Fault Zone, the Whittier Fault, and the Elsinore Fault Zone. The presence of a blind thrust fault has been interpolated from limited data, to exist at a depth of approximately eight miles below the uplifted local hills; however, the San Joaquin Hills Blind Thrust Fault does not have a known location of surface rupture. A discussion of these secondary effects and their potential impact on the site is provided in the following sections.

1.7.1 ***Lurching and Shallow Ground Rupture***

Soil lurching refers to the rolling motion on the ground surface by the passage of seismic surface waves. Effects of this nature are not likely to be significant where the thickness of soft sediments do not vary appreciably under structures. Ground rupture due to active faulting is not likely to occur onsite due to the absence of known active fault traces. Ground cracking due to shaking from distant seismic events is not considered a significant hazard, although it is a possibility at any site.

1.7.2 ***Liquefaction and Dynamic Settlement***

Liquefaction and liquefaction-induced dynamic settlement of soils can be caused by strong vibratory motion due to earthquakes. Liquefaction is typified by a buildup of pore-water pressure in the affected soil layer to a point where a total loss of shear strength may occur, causing the soil to behave as a liquid. Liquefaction primarily occurs in loose, saturated, granular soils while cohesive soils such as silty clays and clays are generally not considered susceptible to soil liquefaction. The effect of liquefaction may be manifested at the ground surface by rapid settlement and/or sand boils. Based on our review of the State of California Seismic Hazard Zones for the San Clemente 7.5 Minute Quadrangle (CDMG, 2002b), the site is not located within a zone mapped as having a potential for liquefaction or earthquake induced landslides.

Based on the hard/dense nature of the material below the site and lack of shallow groundwater, there is a very low potential for liquefaction to be triggered during the design earthquake.

1.7.3 ***Lateral Spreading***

Lateral spreading is a type of liquefaction-induced ground failure associated with the lateral displacement of surficial blocks of sediment resulting from liquefaction in a subsurface layer. Once liquefaction transforms the subsurface layer into a fluid mass, gravity plus the earthquake inertial forces may cause the mass to move down-slope towards a free face (such as a river channel or an embankment). Lateral spreading may cause large horizontal displacements and such movement typically damages pipelines, utilities, bridges, and structures.

Based on the very low potential for site liquefaction, the potential for lateral spreading is also considered to be very low.

1.7.4 ***Tsunamis and Seiches***

Based on the elevation of the site, with respect to sea level, there is a low possibility of damage to the site during a large tsunami event. The site is not located within the Tsunami Inundation Area delineated on the Tsunami Inundation Map for Emergency Planning San Clemente Quadrangle (CEMA, 2009).

1.8 *__Expansive Soil Characteristics__*

Based on the lab results of ASTM D4829 Expansion Index of soils, the soil sampled during our field exploration yielded an Expansion Index of 80. An expansion index that falls within the range of 51-90 is classified as having a "medium" potential for expansion.

1.9 *__Corrosivity Potential__*

Based on our experience in the area the onsite soils should be considered as having a designated sulfate exposure class of "S2" per ACI 318-14, Table 19.3.1.1. As a result, per Table 19.3.2.1 the minimum compressive strength of structural concrete shall be 4,500 psi, the maximum water to cement ratio shall be 0.45 and the cementitious material type under ASTM C-150 shall be Type V.

2.0 ANALYSIS

2.1 Seismic Design Criteria

The site seismic characteristics were evaluated per the guidelines set forth in Chapter 16, Section 1613 of the 2016 California Building Code (CBC). Representative site coordinates of latitude 33.4214 degrees (north) and longitude -117.6126 degrees (west) were utilized in our analyses. The maximum considered earthquake (MCE) spectral response accelerations (S_{MS} and S_{M1}) and adjusted design spectral response acceleration parameters (S_{DS} and S_{D1}) for Site Class D are provided in Table 1 below.

TABLE 1

Seismic Design Parameters

Selected Parameters from 2016 CBC, Section 1613 - Earthquake Loads	Seismic Design Values
Site Class per Chapter 20 of ASCE 7	D
Risk-Targeted Spectral Acceleration for Short Periods (S_S)*	1.270g
Risk-Targeted Spectral Accelerations for 1-Second Periods (S_1)*	0.481g
Site Coefficient F_a per Table 1613.3.3(1)	1.000
Site Coefficient F_v per Table 1613.3.3(2)	1.519
Site Modified Spectral Acceleration for Short Periods (S_{MS}) for Site Class D [Note: $S_{MS} = F_aS_S$]	1.270g
Site Modified Spectral Acceleration for 1-Second Periods (S_{M1}) for Site Class D [Note: $S_{M1} = F_vS_1$]	0.731g
Design Spectral Acceleration for Short Periods (S_{DS}) for Site Class D [Note: $S_{DS} = (^2/_3)S_{MS}$]	0.847g
Design Spectral Acceleration for 1-Second Periods (S_{D1}) for Site Class D [Note: $S_{D1} = (^2/_3)S_{M1}$]	0.487g
Mapped Risk Coefficient at 0.2 sec Spectral Response Period, C_{RS} (per ASCE 7)	0.964
Mapped Risk Coefficient at 1 sec Spectral Response Period, C_{R1} (per ASCE 7)	1.008

* From USGS, 2017

Section 1803.5.12 of the 2016 CBC (per Section 11.8.3 of ASCE 7) states that the maximum considered earthquake geometric mean (MCE_G) Peak Ground Acceleration (PGA) should be used for geotechnical evaluations such as liquefaction potential. The PGA_M for the site is equal to 0.497g.

A deaggregation of the PGA based on a 2,475-year average return period indicates that an earthquake magnitude of 6.7 at a distance of approximately 8.6 miles (13.8 km) from the site would contribute the most to this ground motion (USGS, 2008).

3.0 FINDINGS AND CONCLUSIONS

Based on the results of our geotechnical evaluation, it is our opinion that the proposed building addition and remodel is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the site design, grading, and construction.

The following is a summary of the primary geotechnical factors, which may affect future development of the site.

- Based on our review of pertinent geologic maps, the site is underlain by older marine and non-marine terrace deposits underlain by Capistrano Formation bedrock at depth.
- Based on our evaluation there is a very low potential for earthquake-induced liquefaction and landslides.
- Active or potentially active faults are not known to exist on or in the immediate vicinity of the site.
- The proposed redevelopment will likely be subjected to strong seismic ground shaking during its design life from one of the regional faults.
- For foundation design, site soils should be considered to have "Medium" expansion potential. Mitigation measures are required for foundations and site improvements, such as concrete flatwork, to minimize the impacts of expansive soils.
- Based on test results throughout the city, the City of San Clemente requires structural concrete be designed for corrosive soils.
- From a geotechnical point of view, provided the geotechnical recommendations and parameters provided herein are appropriately incorporated into the design and construction of the project, the proposed site grading and construction are not anticipated to impact the adjacent properties and improvements.

4.0 RECOMMENDATIONS

The following recommendations are to be considered preliminary, and should be confirmed upon completion of grading and earthwork operations. In addition, they should be considered minimal from a geotechnical viewpoint, as there may be more restrictive requirements from the architect, structural engineer, building codes, governing agencies, or the City.

It should be noted that the following geotechnical recommendations are intended to provide sufficient information to develop the site in general accordance with the 2016 C.B.C. requirements. With regard to the potential occurrence of potentially catastrophic geotechnical hazards such as fault rupture, earthquake-induced landslides, liquefaction, etc. the following geotechnical recommendations should provide adequate protection for the proposed development to the extent required to reduce seismic risk to an "acceptable level". The "acceptable level" of risk is defined by the California Code of Regulations as "that level that provides reasonable protection of the public safety, though it does not necessarily ensure continued structural integrity and functionality of the project" [Section 3721(a)]. Therefore, repair and remedial work of the proposed improvement may be required after a significant seismic event. With regards to the potential for less significant geologic hazards to the proposed development, the recommendations contained herein are intended as a reasonable protection against the potential damaging effects of geotechnical phenomena such as expansive soils, fill settlement, groundwater seepage, etc. It should be understood that although our recommendations are intended to maintain the structural integrity of the proposed development and structures given the site geotechnical conditions, they cannot, however, preclude the potential for some cosmetic distress or nuisance issues to develop as a result of the site geotechnical conditions.

The geotechnical recommendations contained herein must be confirmed to be suitable or modified based on the actual as-graded conditions.

The following sections include our geotechnical recommendations for site preparation, foundation design, and site drainage. These recommendations are based upon our evaluation of the near-surface soils and our understanding of the proposed construction.

4.1 Site Earthwork

We anticipate that earthwork at the site will generally consist of site preparation, remedial grading, construction of footings for the proposed addition construction of a replacement slab for the garage and improvements. We recommend that earthwork onsite be performed in accordance with the following recommendations, the 2016 CBC and the City of San Clemente grading requirements. The following recommendations should be considered preliminary and may be revised based on the actual conditions encountered during site grading and construction.

4.1.1 Site Preparation

Prior to grading of areas to receive structural fill, engineered structures or improvements, the areas should be cleared of surface obstructions, any existing debris and potentially compressible or otherwise unsuitable material. Debris should be removed and properly disposed of off-site. Holes resulting from the removal of buried obstructions, which extend below proposed removal bottoms, should be replaced with suitable compacted fill material. Areas to receive fill and/or surface improvements should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and recompacted to at least 90 percent relative compaction (based on American Standard of Testing and Materials [ASTM] Test Method D1557).

4.1.2 Removal and Recompaction

It is anticipated that at the depth of proposed foundation excavation competent existing material will be encountered. The excavation bottom should be observed by the project geotechnical consultant to confirm suitable materials are present. If unsuitable materials are encountered, over-excavation may be required. The actual depth and lateral extents of over-excavation should be determined by the geotechnical consultant, based on subsurface conditions encountered. Removals shall not extend past a 1:1 (horizontal to vertical) plane extended downward and away from the bottom edge of any existing structural footing.

In general, the recommended removal bottom should extend sufficiently beyond the area of proposed grading and improvements so that a 1:1 (horizontal to vertical) projection down from the outer edge of the grading and/or improvements will intercept the removal bottom.

If due to property line constraints and/or the presence of existing improvements and structures, recommended removal of potentially compressible soils may not be completely achievable and no structural improvements are proposed, a reduced lateral extent of removals may be considered at the geotechnical consultant's discretion. In areas where structural improvements are proposed and the recommended 1:1 (horizontal to vertical) projection from the outer edge of the proposed improvements cannot be achieved, the proposed footings may be deepened to achieve the recommended projection and/or a reduced foundation bearing pressure may be provided. If such constraints exist, they should be further addressed at the grading plan and foundation plan review stage of the project.

From a geotechnical perspective, material that is removed may be placed as fill, provided the material is relatively free of organic material and/or deleterious debris, is moisture-conditioned or dried (as needed) to obtain near-optimum moisture content, and then recompacted prior to additional fill placement or construction.

4.1.3 Removal Bottoms and Subgrade Preparation

If over-excavation is necessary, the over-excavated removal bottom areas and areas to receive compacted fill should be scarified to a minimum depth of 6 inches, brought to a near-optimum moisture condition, and re-compacted per project requirements. Removal bottoms and areas to receive fill should be observed and accepted by the geotechnical consultant prior to subsequent fill placement.

4.1.4 Material for Fill

From a geotechnical perspective, the onsite soils are generally considered suitable for use as general compacted fill, provided they are screened of organic materials, construction debris and any oversized material (6 inches or more in greatest dimension). Import soil, if required, should be similar to onsite soils where possible to reduce differential bearing conditions.

4.1.5 Fill Placement and Compaction

Material to be placed as fill (where applicable) should be brought to near-optimum moisture content (generally within optimum and 2 percent above optimum moisture content) and recompacted to at least 90 percent relative compaction (per ASTM Test Method D1557). Moisture conditioning of site soils will likely be required in order to achieve adequate compaction. The optimum lift thickness to produce a uniformly compacted fill will depend on the type and size of compaction equipment used. In general, fill should be placed in uniform lifts not exceeding 6 inches in loose thickness. Each lift should be thoroughly compacted and accepted prior to subsequent lifts. Generally, placement and compaction of fill should be performed in accordance with local grading ordinances and with observation and testing by the geotechnical consultant. Oversized material, as previously defined, should be removed from site fills.

4.1.6 Temporary Stability of Removal Excavations

Due to the rather shallow anticipated remedial removal depths, temporary backcut slope instability is not anticipated to be a concern. We expect temporary backcut slopes to be grossly stable at a 1:1 (horizontal to vertical) inclination or flatter; however, excavations must be made in accordance with Cal OSHA and OSHA requirements. Vehicular traffic, stockpiles, and equipment storage should be set back from the perimeter of excavations a distance equivalent to a 1:1 projection from the bottom of the excavation. Soil conditions should be mapped and frequently checked by a representative of LGC Geotechnical, not only to confirm the geologic conditions but to also help provide early warning of potential failures. The contractor will be responsible for providing the "competent person" required by Cal/OSHA standards to evaluate soil conditions. Close coordination with the geotechnical engineer should be maintained to facilitate construction while providing safe excavations. Excavation safety is the responsibility of the contractor.

Once an excavation has been initiated, it should be backfilled as soon as practical. Prolonged exposure of excavations may result in some localized instability. Excavations should be planned so that they are not initiated without sufficient time to shore/fill them prior to weekends, holidays, or forecasted rain.

From a geotechnical point of view, provided the geotechnical recommendations and parameters presented herein are appropriately incorporated into the design and construction of the project, the proposed site grading and construction is not anticipated to impact the adjacent properties and improvements. Remedial grading is anticipated to extend up to approximately three feet below existing grades. Temporary perimeter backcuts at 1:1 (horizontal to vertical) inclinations or flatter, initiated at the site property lines should be sufficient to achieve the recommended removals below the proposed improvements while maintaining suitable support for the adjacent properties.

4.1.7 Trench Backfill and Compaction

The onsite soils may generally be suitable as trench backfill, provided the soils are screened of rocks and other material greater than 8 inches in maximum dimension and organic matter. If trenches are shallow, or if the use of conventional equipment may result in damage to the utilities, sand having a Sand Equivalent (SE) of 30 or greater may be used to bed and shade the pipes. Sand backfill may be densified by jetting or flooding and then tamping to ensure adequate compaction. Otherwise, trench backfill should be compacted in uniform lifts (generally not exceeding 12 inches in loose thickness) by mechanical means to at least 90 percent relative compaction (per ASTM Test Method D1557). A representative from LGC Geotechnical should observe, probe, and test the backfill to verify compliance with the project specifications.

4.2 Foundation Recommendations

Per your request, we have provided geotechnical design parameters for a rigid slab-on-grade conventionally reinforced slab foundation. The site may be considered suitable for the support of the proposed structure using a rigid slab-on-grade conventionally reinforced slab foundation designed in accordance with Section 1808 of the 2016 C.B.C. It should be noted that, as with many structures in Southern California, risk does remain that the proposed structures could suffer some damage as a result of an earthquake. Repair and remedial work may be required after a seismic event.

The following sections summarize our foundation recommendations. The proposed foundations should be designed by the foundation engineer in accordance with the following recommendations. The following recommendations may be superseded by the requirements of the foundation engineer, structural engineer and/or local jurisdictions. Proposed foundations should be designed to accommodate estimated site static settlements.

4.2.1 Provisional Conventional Foundation Design Parameters

Given that the correlated expansion index exceeds 20, the foundation systems shall be designed for effects of expansive soil. Conventional foundations may be designed in accordance with Wire Reinforcement Institute (WRI) procedure for slab-on-ground foundations per Section 1808 of the 2016 CBC to resist expansive soils. The following preliminary soil parameters may be used:

- Effective Plasticity Index: 25
- Climatic Rating: Cw = 15
- Reinforcement: Per structural designer.
- Moisture condition subgrade soils to 120 percent of optimum moisture content to a depth of 18 inches prior to trenching for footings.

4.2.2 Foundation Subgrade Preparation and Maintenance

Moisture-conditioning of slab subgrade soils is recommended prior placement of concrete steel. The subgrade moisture condition of the building pad soils should be maintained at the recommended moisture content up to the time of concrete placement. This moisture content should be maintained around the immediate perimeter of the slab during construction and up to occupancy of the building structures.

The geotechnical parameters provided assume that if the areas adjacent to the foundation are planted and irrigated, these areas will be designed with proper drainage and adequately maintained so that ponding, which causes significant moisture changes below the foundation, does not occur. Our recommendations do not account for excessive irrigation and/or incorrect landscape design. Plants should only be provided with sufficient irrigation for life and not overwatered to saturate subgrade soils. Sunken planters placed adjacent to the foundation should either be designed with an efficient drainage system or liners to prevent moisture infiltration below the foundation. Some lifting of the perimeter foundation beam should be expected even with properly constructed planters.

In addition to the factors mentioned above, future owners/property management personnel should be made aware of the potential negative influences of trees and/or other large vegetation. Roots that extend near the vicinity of foundations can cause distress to foundations. Future owners (and the owner's landscape architect) should not plant trees/large shrubs closer to the foundations than a distance equal to half the mature height of the tree or 20 feet, whichever is more conservative, unless specifically provided with root barriers to prevent root growth below the building foundation.

It is the homeowner's responsibility to perform periodic maintenance during hot and dry periods to ensure that adequate watering has been provided to keep soil from separating or pulling back from the foundation. Future owners and property management personnel should be informed and educated regarding the importance of maintaining a constant level of soil-moisture. The owners should be made aware of the potential negative consequences of both excessive watering, as well as allowing potentially expansive soils to become too dry. Expansive soils can undergo shrinkage during drying, and swelling during the rainy winter season, or when irrigation is resumed. This can result in distress to building structures and hardscape improvements. The builder should provide these recommendations to future homeowners and property management personnel.

4.2.3 __Slab Underlayment Guidelines__

The following is for informational purposes only since slab underlayment (e.g., moisture retarder, sand or gravel layers for concrete curing and/or capillary break) is unrelated to the geotechnical performance of the foundation and thereby not the purview of the geotechnical consultant. Post-construction moisture migration should be expected below the foundation. The foundation engineer/architect should determine whether the use of a capillary break (sand or gravel layer), in conjunction with the vapor retarder, is necessary or required by code. Sand layer thickness and location (above and/or below vapor retarder) should also be determined by the foundation engineer/architect.

4.2.4 __Existing Stem Wall Footings__

The findings of our evaluation indicate that the existing stem wall footing for the residence have less than 12 inches of minimum embedment required by the current building code. To achieve this minimum embedment, we recommend placement of either 6 inches of compacted fill adjacent to the interior side of the stem wall and extending at least 3 horizontal feet away from the stem wall. Alternatively, 6 inches of 2-sack slurry cement may be placed adjacent to the interior side of the stem wall and extending at least 3 horizontal feet away from the stem wall.

Alternatively, the existing stem wall may be deepened to achieve the minimum required embedment. Recommendations for deepening should be provided by the project foundation engineer.

4.3 ***Soil Bearing Pressure***

An allowable soil bearing pressure of 1,500 pounds per square foot (psf) may be used for the design of footings having a minimum width of 12 inches and minimum embedment of 12 inches below lowest adjacent ground surface. This value may be increased by 300 psf for each additional foot of embedment or 100 psf for each additional foot of foundation width to a maximum value of 2,500 psf. An allowable soil bearing pressure of 1,200 psf may be used for a mat slab a minimum of 6 inches below lowest adjacent grade. These allowable bearing pressures are applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. Bearing values indicated are for total dead loads and frequently applied live loads and may be increased by ⅓ for short duration loading (i.e., wind or seismic loads).

In utilizing the above-mentioned allowable bearing capacity and provided our earthwork recommendations are implemented, foundation settlement due to structural loads is anticipated to be 1-inch. Differential settlement may be taken as half of the total settlement (i.e., ½-inch over a horizontal span of 40 feet).

Resistance to lateral loads can be provided by friction acting at the base of foundations and by passive earth pressure. For concrete/soil frictional resistance, an allowable coefficient of friction of 0.35 may be assumed with dead-load forces. An allowable passive lateral earth pressure of 250 psf per foot of depth (or pcf) to a maximum of 2,500 psf may be used for the sides of footings poured against properly compacted fill. This passive pressure is applicable for level (ground slope equal to or flatter than 5H:1V) conditions only. The passive pressure may be increased by one-third due to wind or seismic forces. We recommend that the upper foot of passive resistance be neglected if finished grade will not be covered with concrete or asphalt. Frictional resistance and passive pressure may be used in combination without reduction. The provided allowable passive pressures are based on a factor of safety of 1.5 and 1.1 for static and seismic loading conditions, respectively.

4.4 ***Foundation Setback from Slopes***

Per the 2016 CBC, planned building and retaining wall foundations adjacent to slopes should be setback a minimum horizontal distance of H/3 from the face of the descending slopes, or 40 feet (whichever is less), where H is the height of the slope. This distance is measured horizontally from the outside bottom edge of the footing to the slope face.

4.5 Nonstructural Concrete Flatwork

Nonstructural concrete flatwork (such as walkways, patios, etc.) has a high potential for cracking due to changes in soil volume related to soil-moisture fluctuations. To reduce the potential for excessive cracking and lifting, concrete should be designed in accordance with the minimum guidelines outlined in Table 2 on the following page. These guidelines will reduce the potential for irregular cracking and promote cracking along construction joints, but will not eliminate all cracking or lifting. Thickening the concrete and/or adding additional reinforcement will further reduce cosmetic distress.

TABLE 2

Nonstructural Concrete Flatwork for Medium Expansion Potential

	Homeowner Sidewalks	Private Drives	Patios/Entryways	City Sidewalk Curb and Gutters
Minimum Thickness (in.)	4 (nominal)	5 (full)	5 (full)	City/Agency Standard
Presaturation	Wet down prior to placing	Presoak to 12 inches	Presoak to 12 inches	City/Agency Standard
Reinforcement	—	No. 3 at 24 inches on centers	No. 3 at 24 inches on centers	City/Agency Standard
Thickened Edge (in.)	—	8 x 8	—	City/Agency Standard
Crack Control Joints	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	Saw cut or deep open tool joint to a minimum of 1/3 the concrete thickness	City/Agency Standard
Maximum Joint Spacing	5 feet	10 feet or quarter cut whichever is closer	6 feet	City/Agency Standard
Aggregate Base Thickness (in.)	—	—	2	City/Agency Standard

To reduce the potential for flatwork to separate from the building foundation, the builder may elect to install dowels to tie these two elements together.

4.6 Control of Surface Water and Drainage Control

Positive drainage of surface water away from structures is very important. Water should not be allowed to pond adjacent to buildings or to flow freely down a graded slope. Per Section 1804.3 of the 2016 CBC, positive drainage may be accomplished by providing drainage away from buildings at a gradient of at least 5 percent for earthen surfaces for a distance of at least 10 feet away from the face of a wall. If a distance of 10 feet cannot be achieved, an alternative of a gradient of at least 5 percent to an area drain or swale having a gradient of 2 percent is acceptable. Where necessary, drainage paths may be shortened by use of area drains and collector pipes. Eave gutters are recommended and should reduce water infiltration into the subgrade soils if the downspouts are properly connected to appropriate outlets.

Planters with open bottoms adjacent to buildings should be avoided. Planters should not be designed adjacent to buildings unless provisions for drainage, such as catch basins, liners, and/or area drains, are made. Overwatering must be avoided.

4.7 Freestanding Walls

To reduce the potential for unsightly cracks, due to differential settlement or possibly expansive soils, we recommend the inclusion of construction joints at a maximum of 20 feet on-center. This spacing may be altered by the structural engineer based upon the wall reinforcement. If the soil-moisture content below the wall foundation varies significantly, some wall movement should be expected; however, this movement is unlikely to cause more than cosmetic distress. Allowable soil bearing values for wall footing design are provided in Section 4.3.

4.8 Soil Corrosivity

Although not corrosion engineers (LGC Geotechnical is not a corrosion consultant), several governing agencies in Southern California require the geotechnical consultant to determine the corrosion potential of soils to buried concrete and metal facilities.

Based on our experience in the area the onsite soils should be considered as having a designated sulfate exposure class of “S2” per ACI 318-14, Table 19.3.1.1. As a result, per Table 19.3.2.1 the minimum compressive strength of structural concrete shall be 4,500 psi, the maximum water to cement ratio shall be 0.45 and the cementitious material type under ASTM C-150 shall be Type V.

4.9 Subsurface Water Infiltration

Recent regulatory changes in some jurisdictions have recommended that low flow runoff be infiltrated rather than discharged via conventional storm drainage systems. In general, the vast majority of geotechnical distress issues are directly related to improper drainage. In general, distress in the form of movement of improvements could occur as a result of soil saturation and loss of soil support, expansion, internal soil erosion, collapse and/or settlement. Infiltrated water may enter underground utility pipe zones and migrate along the pipe backfill, potentially impacting other improvements located far away from the point of infiltration.

We do not recommend that water be intentionally infiltrated at this site.

4.10 Water Intrusion

We understand that periodic water intrusion into the crawl space area beneath the residence has been reported. We also understand that the recent installation of rain gutters on the property appear to have addressed this problem.

At the owner’s option, the precaution of installing a “French Drain” system to intercept potential water intrusion may be performed in accordance with the following recommendations.

In general, we recommend construction of a "French Drain" style subdrain system consisting of a one-foot-wide trench, excavated as deep as practical while still allowing for gravity flow to a suitable outlet location. The system may be constructed locally in the area where water intrusion has been observed, or may extend to multiple sides of the residence or around the entire perimeter. To avoid undermining existing footings, trenches should be excavated such that a 1:1 (horizontal to vertical) upward projection from the bottom of the trench lies above the bottom of adjacent footings.

If gravity flow to the street is not achievable, it may be necessary to install an onsite sump pit collection point and sump pump to discharge accumulated water. The sump pit should be installed at the lowest point of the system. Once installed, it is imperative that the sump pump be maintained operational and functioning in perpetuity.

A concrete cut-off wall should be constructed where the perforated pipe transitions to a non-perforated outlet pipe to the sump pit or outlet point. The cut-off wall should have a minimum width of 8 inches and should be notched into the bottom and sides of the trench wall for the French Drain portion of the system a minimum of 6 inches. The cut-off wall should extend at least 1-foot above the subdrain pipe.

A typical "French Drain" style subdrain system detail is depicted on Figure 2. Clean-outs should be considered in several locations along the subdrain system (such as at the end, any angle points in the pipe, and where the subdrain transitions to the non-perforated outlet pipe).

The location of the recommended drainage system may be modified to avoid conflict with existing and/or proposed improvements where necessary. Area drains and the recommended "French Drain" system must not be tied together.

4.11 ***Geotechnical Plan Review***

Precise grading plans, foundation plans, wall plans, and final project drawings should be reviewed by this office prior to construction to verify that our geotechnical recommendations, provided herein, have been appropriately incorporated.

4.12 ***Geotechnical Observation/Testing During Grading and Construction***

The recommendations provided in this report are based on limited subsurface evaluation, field observations, and geotechnical analysis. The interpolated subsurface conditions should be checked in the field during grading and construction by a representative of LGC Geotechnical.

Geotechnical observation and/or testing should be performed by a field representative from our office at the following stages:

- During remedial grading operations;
- During fill placement and compaction;
- After footing excavation and prior to placing concrete and/or reinforcement;
- After drainage and/or planter liner installation, prior to backfill;
- Excavation, backfill and construction of the French Drain; and

- When any unusual soil conditions are encountered during any construction operation subsequent to issuance of this report.

5.0 LIMITATIONS

Our services were performed using the degree of care and skill ordinarily exercised, under similar circumstances, by reputable engineers and geologists practicing in this or similar localities. No other warranty, expressed or implied, is made as to the conclusions and professional advice included in this report. The samples taken and submitted for laboratory testing, the observations made and the in-situ field testing performed are believed representative of the entire project; however, soil and geologic conditions revealed by excavation may be different than our preliminary findings. If this occurs, the changed conditions must be evaluated by the project soils engineer and geologist and design(s) adjusted as required or alternate design(s) recommended.

This report is issued with the understanding that it is the responsibility of the owner, or of his/her representative, to ensure that the information and recommendations contained herein are brought to the attention of the architect and/or project engineer and incorporated into the plans, and the necessary steps are taken to see that the contractor and/or subcontractor properly implements the recommendations in the field. The contractor and/or subcontractor should notify the owner if they consider any of the recommendations presented herein to be unsafe.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can and do occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. Therefore, the findings, conclusions, and recommendations presented in this report can be relied upon only if LGC Geotechnical has the opportunity to observe the subsurface conditions during grading and construction of the project, in order to confirm that our preliminary findings are representative for the site.

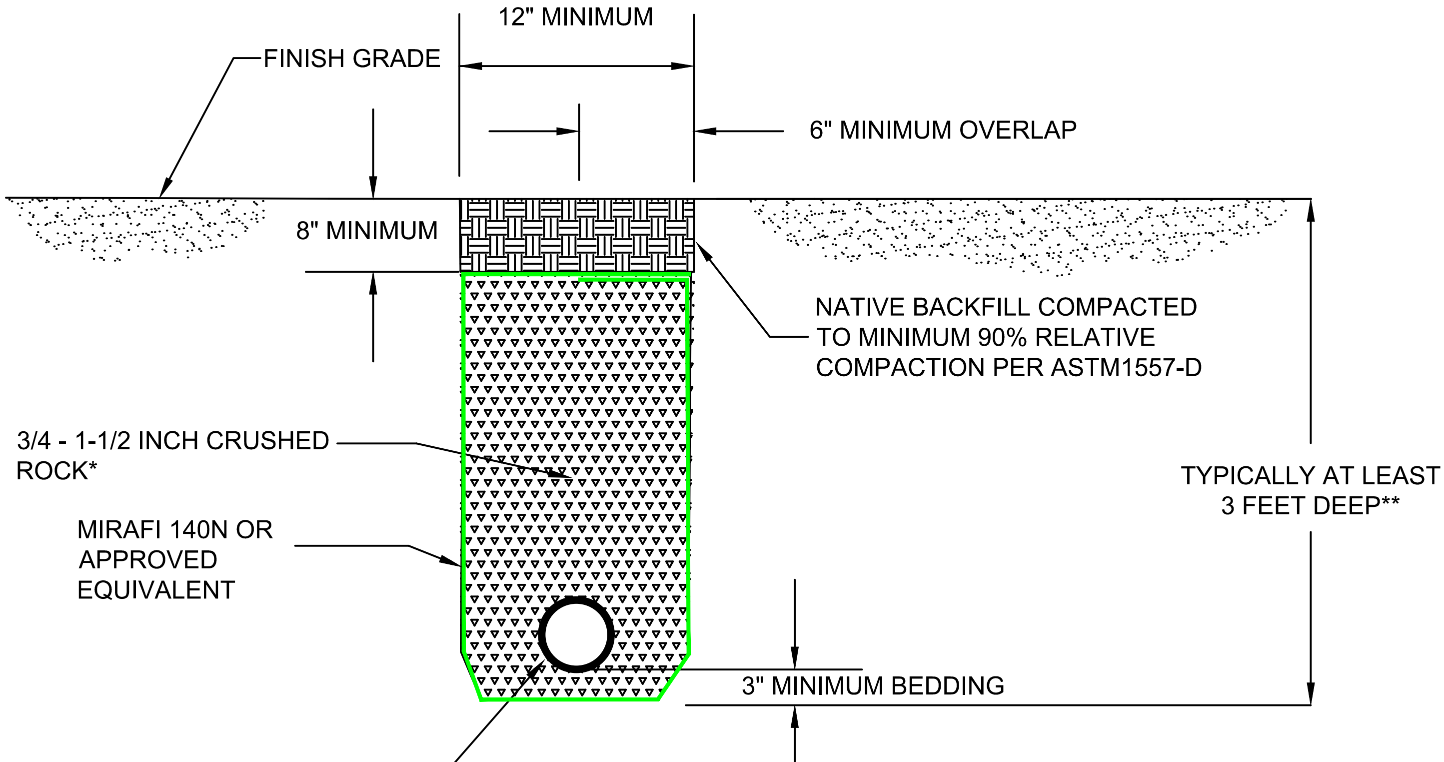
In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and modification, and should not be relied upon after a period of 3 years.

LGC Geotechnical, Inc.

FIGURE 1
Site Location Map

APPENDIX C

PROJECT NAME	Beauchemin Residence
PROJECT NO.	17032-01
ENG. / GEOL.	BRG / KBC
SCALE	Not to Scale
DATE	April 2017



CONTINUOUS 4 INCH DIAMETER SCHEDULE 40 (OR SDR 35) PERFORATED PVC PIPE FOR LENGTH OF TRENCH, WITH PERFORATIONS ORIENTED DOWNWARD, NON-PERFORATED PVC PIPE BETWEEN END OF FRENCH DRAIN AND SUITABLE OUTLET, MINIMUM 2% FALL FOR PERFORATED PIPE AND 1% FOR NON-PERFORATED (SOLID) PIPE

SPECIFICATION FOR CLATRANS CLASS 2 PERMEABLE MATERIAL

U.S. STANDARD SIEVE SIZE	% PASSING
1"	100
3/4"	90-100
3/8"	40-100
No. 4	25-40
No. 8	18-33
No. 30	5-15
No. 50	0-7
No. 200	0-3

SAND EQUIVALENT > 75

* IF CLASS 2 PERMEABLE MATERIAL (SEE GRADATION TO LEFT) IS USED IN PLACE OF 3/4" - 1-1/2" CRUSHED ROCK, FILTER FABRIC MAY BE DELETED. CLASS 2 PERMEABLE MATERIAL SHOULD BE COMPACTED TO 90 PERCENT RELATIVE COMPACTION BASED ON ASTM D1557

**OR AS DEEP AS POSSIBLE WHILE STILL ALLOWING OUTLET TO SUMP PUMP SYSTEM

LGC Geotechnical, Inc.

FIGURE 2
Typical French Drain Detail

APPENDIX C

PROJECT NAME	*Beauchemin Residence*
PROJECT NO.	*17032-01*
ENG. / GEOL.	*BRG / KBC*
SCALE	*Not To Scale*
DATE	*April 2017*

Appendix A
References

APPENDIX A

References

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Appendix B
Laboratory Testing Procedures and Test Results

APPENDIX C

Laboratory Testing Procedures and Test Results

The laboratory testing program was formulated towards providing data relating to the relevant engineering properties of the soils with respect to residential construction. Samples considered representative of site conditions were tested in general accordance with American Society for Testing and Materials (ASTM) procedure and/or California Test Methods (CTM), where applicable. The following summary is a brief outline of the test type and a table summarizing the test results.

Expansion Index: The expansion potential of selected samples was evaluated by the Expansion Index Test, Standard ASTM D4829. Specimens are molded under a given compactive energy to approximately the optimum moisture content and approximately 50 percent saturation or approximately 90 percent relative compaction. The prepared 1-inch-thick by 4-inch-diameter specimens are loaded to an equivalent 144 psf surcharge and are inundated with tap water until volumetric equilibrium is reached. The results of these tests are presented in the table below.

Sample Location	Compacted Dry Density (pcf)	Expansion Index	Expansion Potential*
B-1	95.3	80	Medium

* ASTM D4829

Appendix C
General Earthwork and Grading Specifications for Rough Grading

General Earthwork and Grading Specifications for Rough Grading

1.0 General

1.1 Intent

These General Earthwork and Grading Specifications are for the grading and earthwork shown on the approved grading plan(s) and/or indicated in the geotechnical report(s). These Specifications are a part of the recommendations contained in the geotechnical report(s). In case of conflict, the specific recommendations in the geotechnical report shall supersede these more general Specifications. Observations of the earthwork by the project Geotechnical Consultant during the course of grading may result in new or revised recommendations that could supersede these specifications or the recommendations in the geotechnical report(s).

1.2 The Geotechnical Consultant of Record

Prior to commencement of work, the owner shall employ a qualified Geotechnical Consultant of Record (Geotechnical Consultant). The Geotechnical Consultant shall be responsible for reviewing the approved geotechnical report(s) and accepting the adequacy of the preliminary geotechnical findings, conclusions, and recommendations prior to the commencement of the grading.

Prior to commencement of grading, the Geotechnical Consultant shall review the "work plan" prepared by the Earthwork Contractor (Contractor) and schedule sufficient personnel to perform the appropriate level of observation, mapping, and compaction testing.

During the grading and earthwork operations, the Geotechnical Consultant shall observe, map, and document the subsurface exposures to verify the geotechnical design assumptions. If the observed conditions are found to be significantly different than the interpreted assumptions during the design phase, the Geotechnical Consultant shall inform the owner, recommend appropriate changes in design to accommodate the observed conditions, and notify the review agency where required.

The Geotechnical Consultant shall observe the moisture-conditioning and processing of the subgrade and fill materials and perform relative compaction testing of fill to confirm that the attained level of compaction is being accomplished as specified. The Geotechnical Consultant shall provide the test results to the owner and the Contractor on a routine and frequent basis.

1.3 The Earthwork Contractor

The Earthwork Contractor (Contractor) shall be qualified, experienced, and knowledgeable in earthwork logistics, preparation and processing of ground to receive fill, moisture-conditioning and processing of fill, and compacting fill. The Contractor shall review and accept the plans, geotechnical report(s), and these Specifications prior to commencement of grading. The Contractor shall be solely responsible for performing the grading in accordance with the project plans and specifications. The Contractor shall prepare and submit to the owner and the Geotechnical Consultant a work plan that indicates the sequence of earthwork grading, the number of "equipment" of work and the estimated quantities of daily earthwork

contemplated for the site prior to commencement of grading. The Contractor shall inform the owner and the
Geotechnical Consultant of changes in work schedules and updates to the work plan at least 24 hours in advance of such changes so that appropriate personnel will be available for observation and testing. The Contractor shall not assume that the Geotechnical Consultant is aware of all grading operations.

The Contractor shall have the sole responsibility to provide adequate equipment and methods to accomplish the earthwork in accordance with the applicable grading codes and agency ordinances, these Specifications, and the recommendations in the approved geotechnical report(s) and grading plan(s). If, in the opinion of the Geotechnical Consultant, unsatisfactory conditions, such as unsuitable soil, improper moisture condition, inadequate compaction, insufficient buttress key size, adverse weather, etc., are resulting in a quality of work less than required in these specifications, the Geotechnical Consultant shall reject the work and may recommend to the owner that construction be stopped until the conditions are rectified. It is the contractor's sole responsibility to provide proper fill compaction.

2.0 Preparation of Areas to be Filled

2.1 Clearing and Grubbing

Vegetation, such as brush, grass, roots, and other deleterious material shall be sufficiently removed and properly disposed of in a method acceptable to the owner, governing agencies, and the Geotechnical Consultant.

The Geotechnical Consultant shall evaluate the extent of these removals depending on specific site conditions. Earth fill material shall not contain more than 1 percent of organic materials (by volume). Nesting of the organic materials shall not be allowed.

If potentially hazardous materials are encountered, the Contractor shall stop work in the affected area, and a hazardous material specialist shall be informed immediately for proper evaluation and handling of these materials prior to continuing to work in that area.

As presently defined by the State of California, most refined petroleum products (gasoline, diesel fuel, motor oil, grease, coolant, etc.) have chemical constituents that are considered to be hazardous waste. As such, the indiscriminate dumping or spillage of these fluids onto the ground may constitute a misdemeanor, punishable by fines and/or imprisonment, and shall not be allowed. The contractor is responsible for all hazardous waste relating to his work. The Geotechnical Consultant does not have expertise in this area. If hazardous waste is a concern, then the Client should acquire the services of a qualified environmental assessor.

2.2 Processing

Existing ground that has been declared satisfactory for support of fill by the Geotechnical Consultant shall be scarified to a minimum depth of 6 inches. Existing ground that is not satisfactory shall be over-excavated as specified in the following section. Scarification shall continue until soils are broken down and free of oversize material and the working surface is reasonably uniform, flat, and free of uneven features that would inhibit uniform compaction.

2.3 Over-excavation

In addition to removals and over-excavations recommended in the approved geotechnical report(s) and the grading plan, soft, loose, dry, saturated, spongy, organic-rich, highly fractured or otherwise unsuitable ground shall be over-excavated to competent ground as evaluated by the Geotechnical Consultant during grading.

2.4 Benching

Where fills are to be placed on ground with slopes steeper than 5:1 (horizontal to vertical units), the ground shall be stepped or benched. Please see the Standard Details for a graphic illustration. The lowest bench or key shall be a minimum of 15 feet wide and at least 2 feet deep, into competent material as evaluated by the Geotechnical Consultant. Other benches shall be excavated a minimum height of 4 feet into competent material or as otherwise recommended by the Geotechnical Consultant. Fill placed on ground sloping flatter than 5:1 shall also be benched or otherwise over-excavated to provide a flat subgrade for the fill.

2.5 Evaluation/Acceptance of Fill Areas

All areas to receive fill, including removal and processed areas, key bottoms, and benches, shall be observed, mapped, elevations recorded, and/or tested prior to being accepted by the Geotechnical Consultant as suitable to receive fill. The Contractor shall obtain a written acceptance from the Geotechnical Consultant prior to fill placement. A licensed surveyor shall provide the survey control for determining elevations of processed areas, keys, and benches.

3.0 Fill Material

3.1 General

Material to be used as fill shall be essentially free of organic matter and other deleterious substances evaluated and accepted by the Geotechnical Consultant prior to placement. Soils of poor quality, such as those with unacceptable gradation, high expansion potential, or low strength shall be placed in areas acceptable to the Geotechnical Consultant or mixed with other soils to achieve satisfactory fill material.

3.2 Oversize

Oversize material defined as rock, or other irreducible material with a maximum dimension greater than 8 inches, shall not be buried or placed in fill unless location, materials, and placement methods are specifically accepted by the Geotechnical Consultant. Placement operations shall be such that nesting of oversized material does not occur and such that oversize material is completely surrounded by compacted or densified fill. Oversize material shall not be placed within 10 vertical feet of finish grade or within 2 feet of future utilities or underground construction.

3.3 Import

If importing of fill material is required for grading, proposed import material shall meet the requirements of the geotechnical consultant. The potential import source shall be given to the Geotechnical Consultant at least 48 hours (2 working days) before importing begins so that its suitability can be determined and appropriate tests performed.

4.0 Fill Placement and Compaction

4.1 Fill Layers

Approved fill material shall be placed in areas prepared to receive fill (per Section 3.0) in near-horizontal layers not exceeding 8 inches in loose thickness. The Geotechnical Consultant may accept thicker layers if testing indicates the grading procedures can adequately compact the thicker layers. Each layer shall be spread evenly and mixed thoroughly to attain relative uniformity of material and moisture throughout.

4.2 Fill Moisture Conditioning

Fill soils shall be watered, dried back, blended, and/or mixed, as necessary to attain a relatively uniform moisture content at or slightly over optimum. Maximum density and optimum soil moisture content tests shall be performed in accordance with the American Society of Testing and Materials (ASTM Test Method D1557).

4.3 Compaction of Fill

After each layer has been moisture-conditioned, mixed, and evenly spread, it shall be uniformly compacted to not less than 90 percent of maximum dry density (ASTM Test Method D1557). Compaction equipment shall be adequately sized and be either specifically designed for soil compaction or of proven reliability to efficiently achieve the specified level of compaction with uniformity.

4.4 Compaction of Fill Slopes

In addition to normal compaction procedures specified above, compaction of slopes shall be accomplished by backrolling of slopes with sheepsfoot rollers at increments of 3 to 4 feet in fill elevation, or by other methods producing satisfactory results acceptable to the Geotechnical Consultant. Upon completion of grading, relative compaction of the fill, out to the slope face, shall be at least 90 percent of maximum density per ASTM Test Method D1557.

4.5 Compaction Testing

Field tests for moisture content and relative compaction of the fill soils shall be performed by the Geotechnical Consultant. Location and frequency of tests shall be at the Consultant's discretion based on field conditions encountered. Compaction test locations will not necessarily be selected on a random basis. Test locations shall be selected to verify adequacy of compaction levels in areas that are judged to be prone to inadequate compaction (such as close to slope faces and at the fill/bedrock benches).

4.6 Frequency of Compaction Testing

Tests shall be taken at intervals not exceeding 2 feet in vertical rise and/or 1,000 cubic yards of compacted fill soils embankment. In addition, as a guideline, at least one test shall be taken on slope faces for each 5,000 square feet of slope face and/or each 10 feet of vertical height of slope. The Contractor shall assure that fill construction is such that the testing schedule can be accomplished by the Geotechnical Consultant. The Contractor shall stop or slow down the earthwork construction if these minimum standards are not met.

4.7 Compaction Test Locations

The Geotechnical Consultant shall document the approximate elevation and horizontal coordinates of each test location. The Contractor shall coordinate with the project surveyor to assure that sufficient grade stakes are established so that the Geotechnical Consultant can determine the test locations with sufficient accuracy. At a minimum, two grade stakes within a horizontal distance of 100 feet and vertically less than
5 feet apart from potential test locations shall be provided.

5.0 Subdrain Installation

Subdrain systems shall be installed in accordance with the approved geotechnical report(s), the grading plan, and the Standard Details. The Geotechnical Consultant may recommend additional subdrains and/or changes in subdrain extent, location, grade, or material depending on conditions encountered during grading. All subdrains shall be surveyed by a land surveyor/civil engineer for line and grade after installation and prior to burial. Sufficient time should be allowed by the Contractor for these surveys.

6.0 Excavation

Excavations, as well as over-excavation for remedial purposes, shall be evaluated by the Geotechnical Consultant during grading. Remedial removal depths shown on geotechnical plans are estimates only. The actual extent of removal shall be determined by the Geotechnical Consultant based on the field evaluation of exposed conditions during grading. Where fill-over-cut slopes are to be graded, the cut portion of the slope shall be made, evaluated, and accepted by the Geotechnical Consultant prior to placement of materials for construction of the fill portion of the slope, unless otherwise recommended by the Geotechnical Consultant.

7.0 Trench Backfills

7.1 The Contractor shall follow all OHSA and Cal/OSHA requirements for safety of trench excavations.

7.2 All bedding and backfill of utility trenches shall be done in accordance with the applicable provisions of Standard Specifications of Public Works Construction. Bedding material shall have a Sand Equivalent greater than 30 (SE>30). The bedding shall be placed to 1 foot over the top of the conduit and densified by jetting. Backfill shall be placed and densified to a minimum of 90 percent of maximum from 1 foot above the top of the conduit to the surface.

7.3 The jetting of the bedding around the conduits shall be observed by the Geotechnical Consultant.

7.4 The Geotechnical Consultant shall test the trench backfill for relative compaction. At least one test should be made for every 300 feet of trench and 2 feet of fill.

7.5 Lift thickness of trench backfill shall not exceed those allowed in the Standard Specifications of Public Works Construction unless the Contractor can demonstrate to the Geotechnical Consultant that the fill lift can be compacted to the minimum relative compaction by his alternative equipment and method.

APPENDIX D : PRESENTATION SLIDES

BEAUCHEMIN RESIDENCE

CAL POLY SAN LUIS OBISPO
HANNAH ROGERS

APPENDIX D

OWNERS - BEAUCHEMIN FAMILY

ARCHITECT - JAMES GLOVER HOME

GEOTECH - LGC GEOTECHNICAL

STRUCTURAL - COASTLINE ENGINEERING INC.

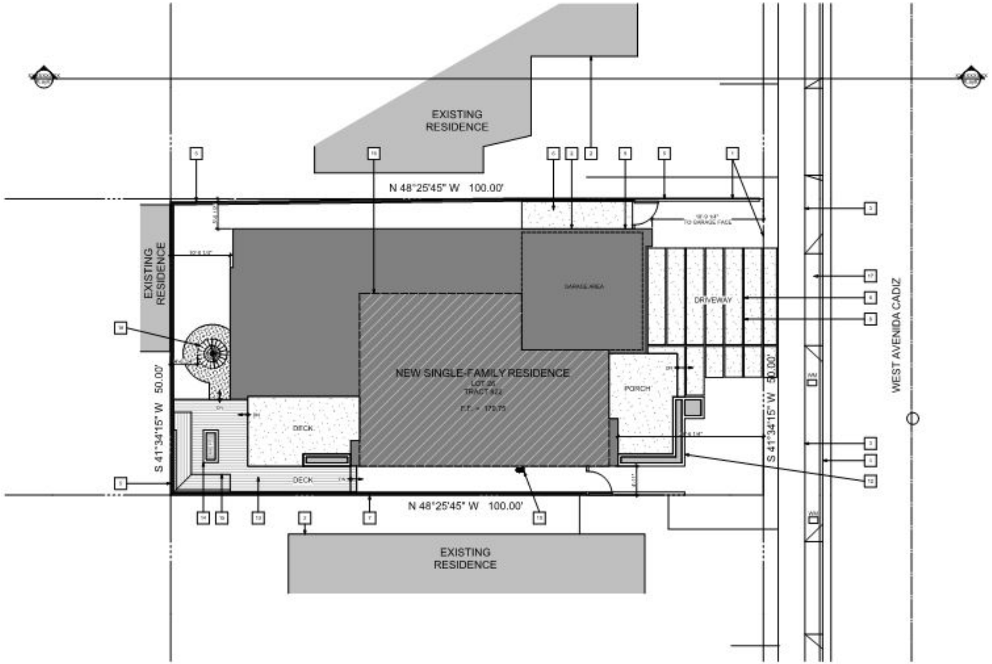
CONTRACTOR - TBD

APPENDIX D

SCOPE

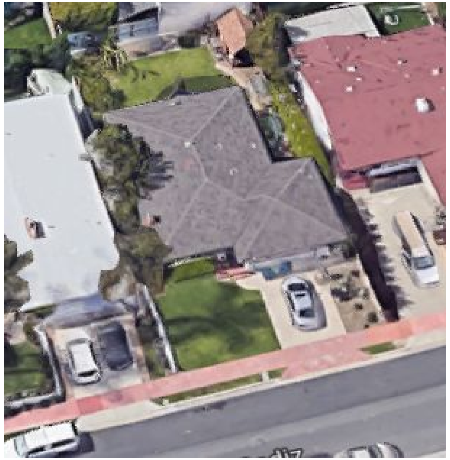
USING ARCH. DESIGN:

- GRAVITY CALCULATIONS
- LATERAL CALCULATIONS
- COMPLETE SET OF CONSTRUCTION DOCUMENTS



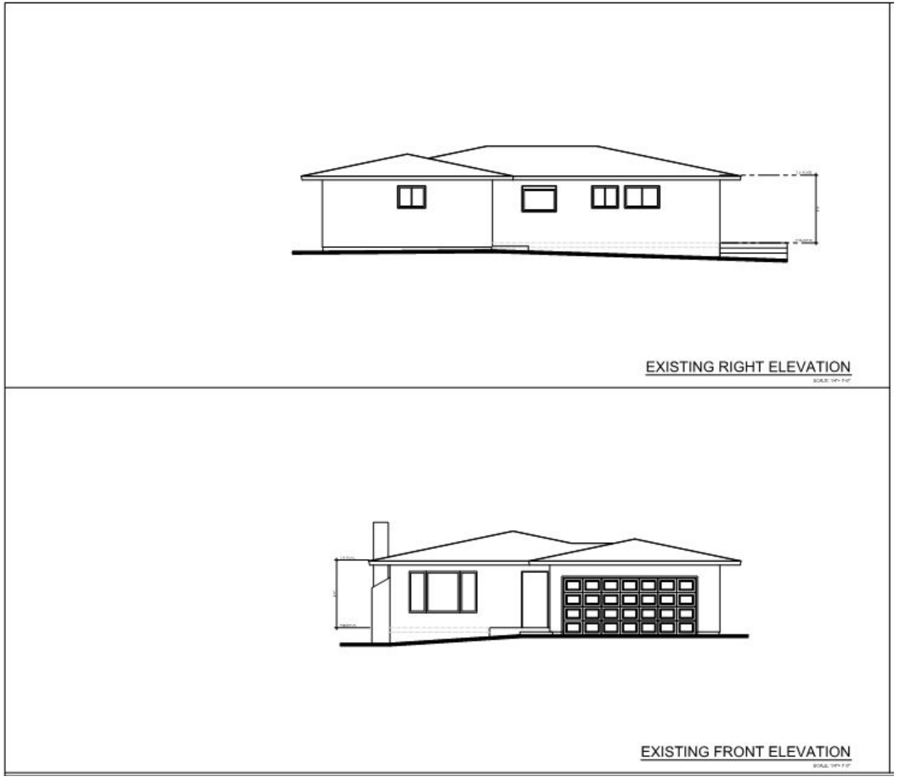
APPENDIX D

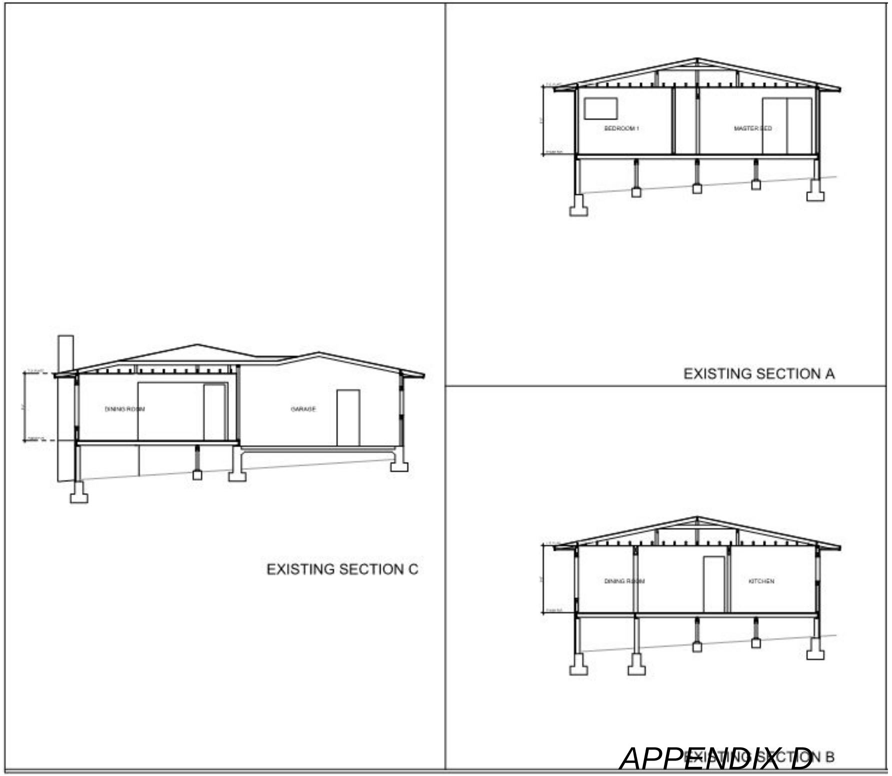
EXISTING STRUCTURE



APPENDIX D

AS BUILT

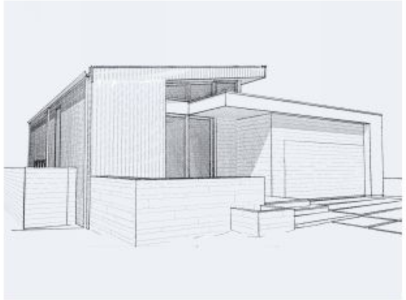




APPENDIX D

PROPOSED STRUCTURE

FRONT PERSPECTIVE



FRONT PERSPECTIVE

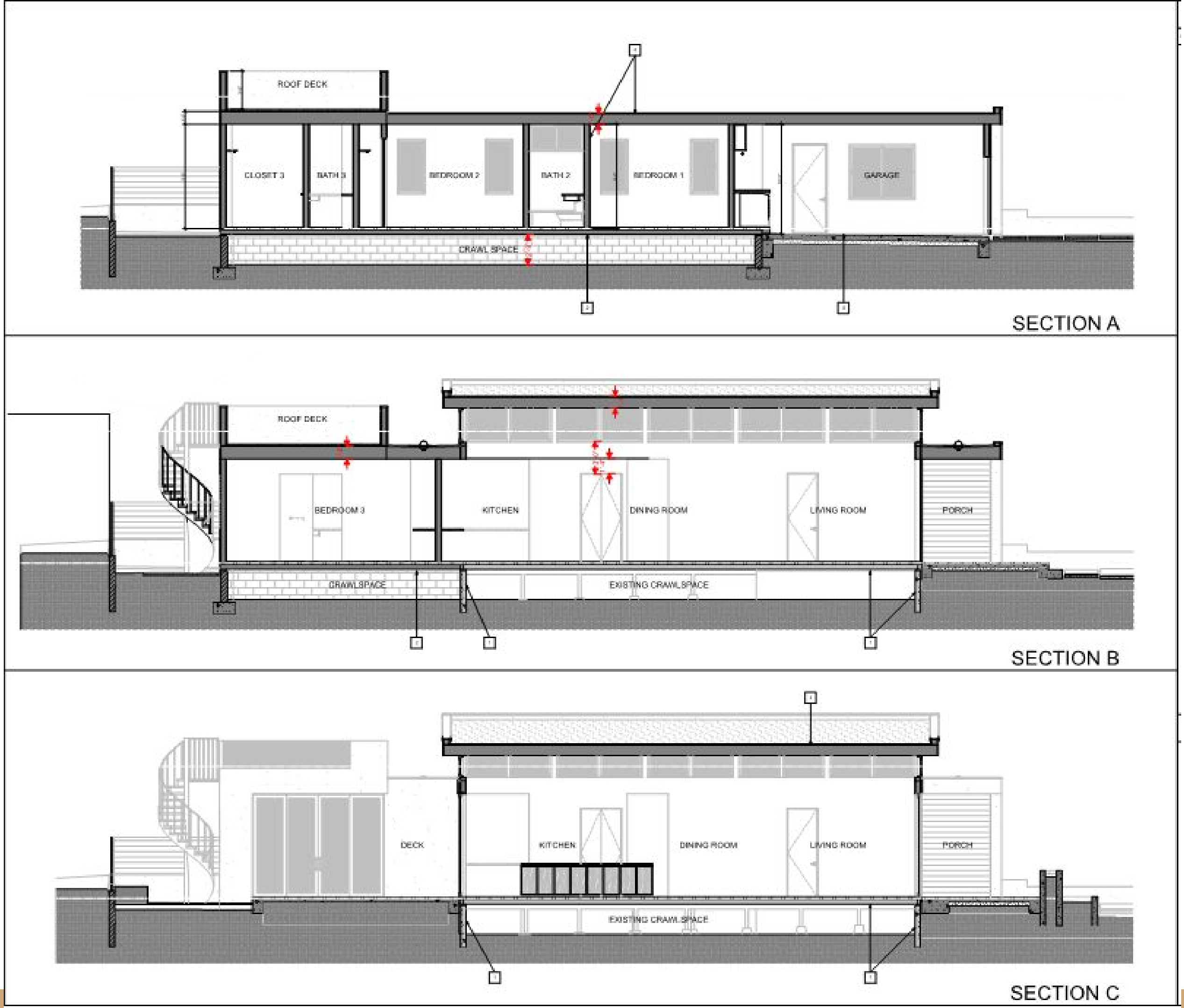
REAR PERSPECTIVE

REAR PERSPECTIVE

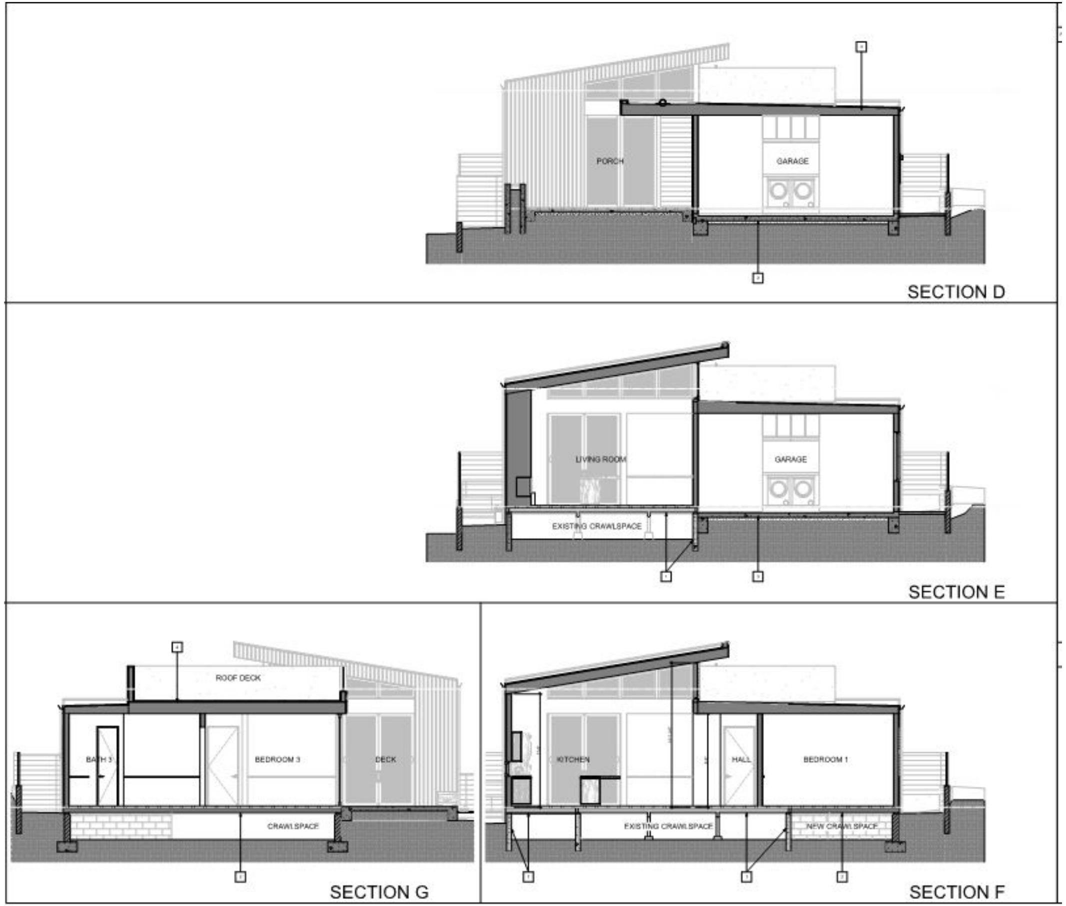
APPENDIX D

ARCHITECTURAL

- PLANS & SECTIONS (PDF & AUTOCAD PROVIDED)
  - DIMENSIONS
  - DEPTH
  - PLATE HEIGHTS
  - WALL LAYOUT
  - EXISTING PLANS


ROOF DECK
CLOSET 3
BATH 3
BEDROOM 2
BATH 2
BEDROOM 1
GARAGE
CRAWL SPACE
SECTION A
ROOF DECK
BEDROOM 3
KITCHEN
DINING ROOM
LIVING ROOM
PORCH
CRAWLSPACE
EXISTING CRAWLSPACE
SECTION B
DECK
KITCHEN
DINING ROOM
LIVING ROOM
PORCH
EXISTING CRAWLSPACE
SECTION C

APPENDIX D


PORCH
GARAGE
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LIVING ROOM
GARAGE
EXISTING CRAWLSPACE
SECTION E
ROOF DECK
BATH 3
BEDROOM 3
DECK
CRAWLSPACE
SECTION G
KITCHEN
HALL
BEDROOM 1
EXISTING CRAWLSPACE
NEW CRAWLSPACE
SECTION F

APPENDIX D

STRUCTURAL

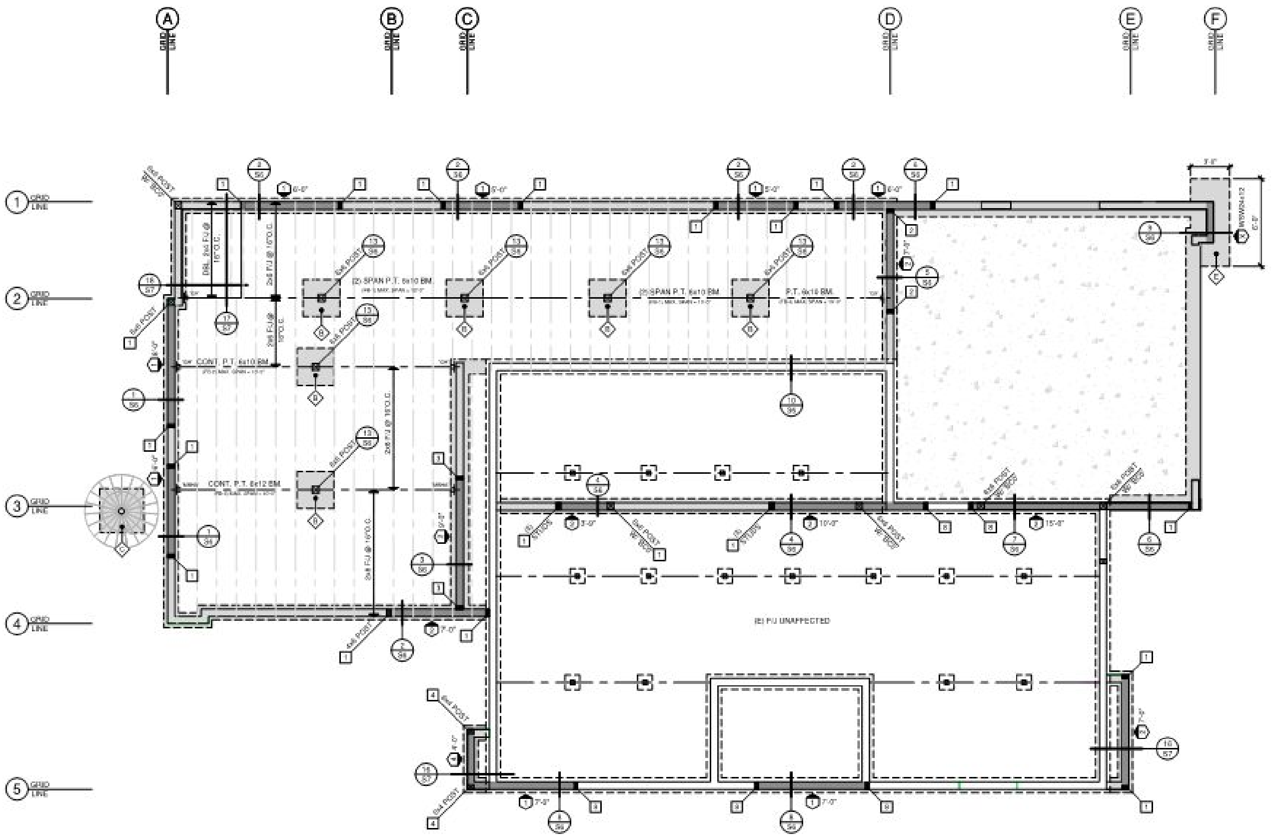
- RAISED WOOD FLOOR
  - EXISTING/NEW
  - “REMODEL” = AT LEAST FLOOR SHEATHING REMAINS
- MONOSLOPE/FLAT ROOF (RIPPED RAFTERS)
  - SPECIFIC WEIGHTS
- CLEAR STORY WINDOWS
  - LATERAL DIAPHRAGM LOADS
  - BEAM DEPTH AVAILABLE (LIMITED)
- GEOTECHNICAL CONCERNS
- ARCHITECTURAL CONCERNS

APPENDIX D

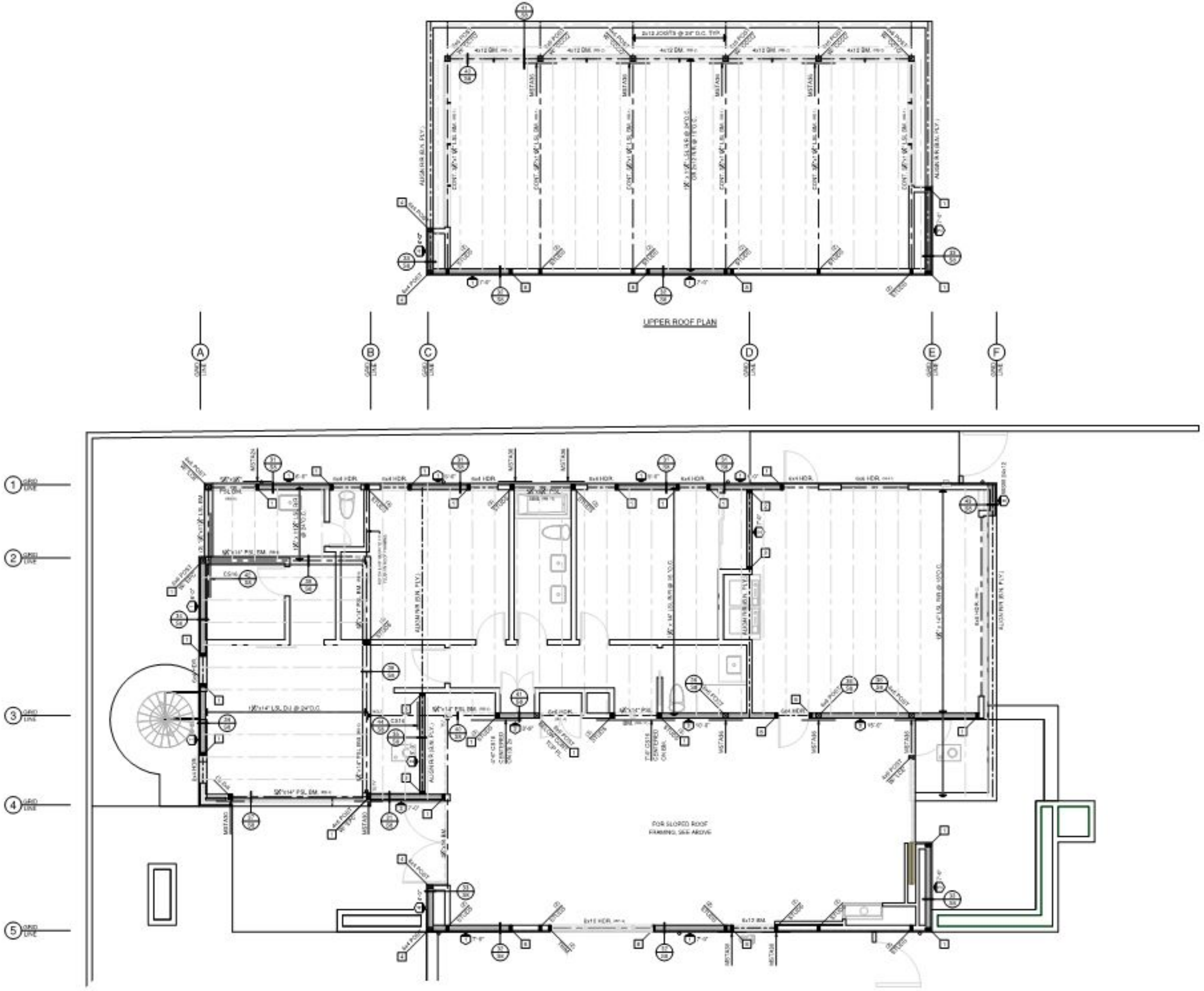
PROCESS

- GRAVITY
  - DETERMINE ALLOWABLE DEPTH
  - MATERIAL LOADS
  - TRIBUTARY AREA
  - DESIGN MEMBERS
  - HARDWARE SELECTED FOR CONTINUOUS LOAD PATH
  - FOUNDATION ELEMENTS TO GET LOAD TO GROUND
- LATERAL
  - DEAD LOAD FROM MATERIAL WEIGHTS
  - TRIBUTARY TO SELECTED WALLS (FOR SHEAR RESISTANCE)
  - TYPE OF SHEAR WALL DETERMINED, HOLDOWNS SELECTED
  - CHECK EXISTING FOUNDATION
  - CHECK SHEAR FLOW VIA STRAPS, CONTINUOUS MEMBERS

APPENDIX D



APPENDIX D

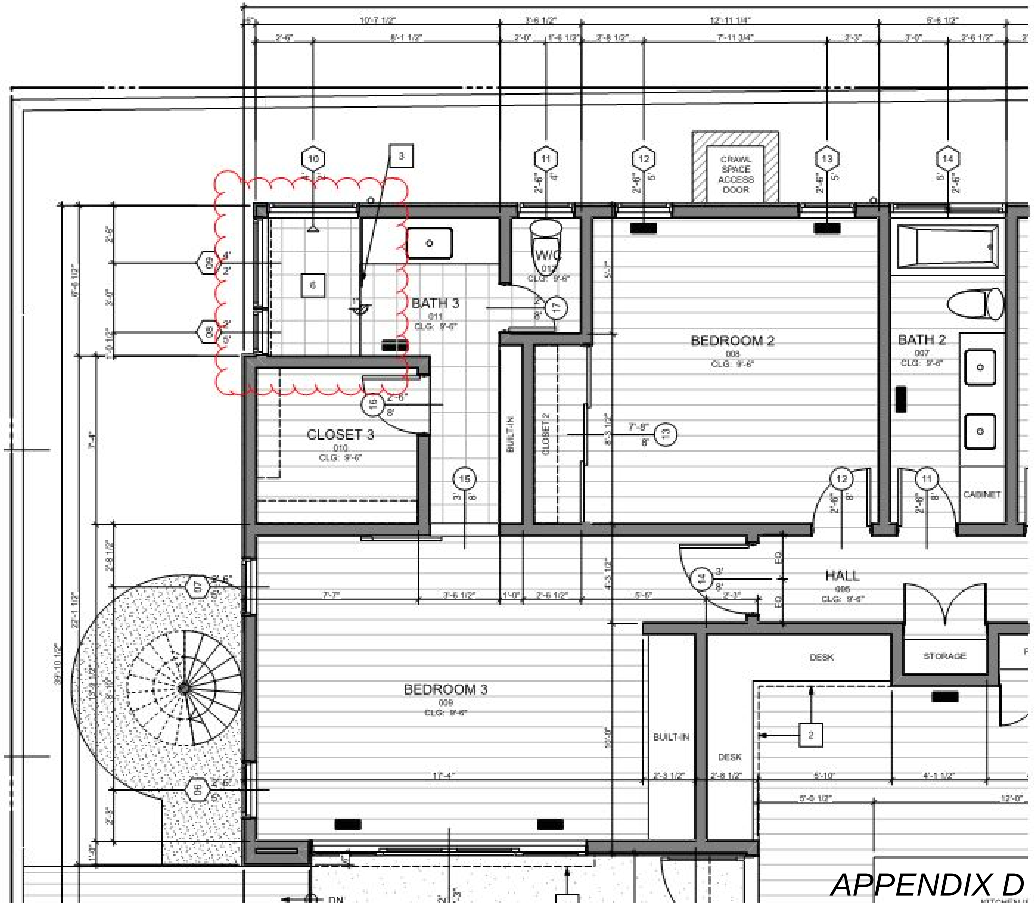


APPENDIX D

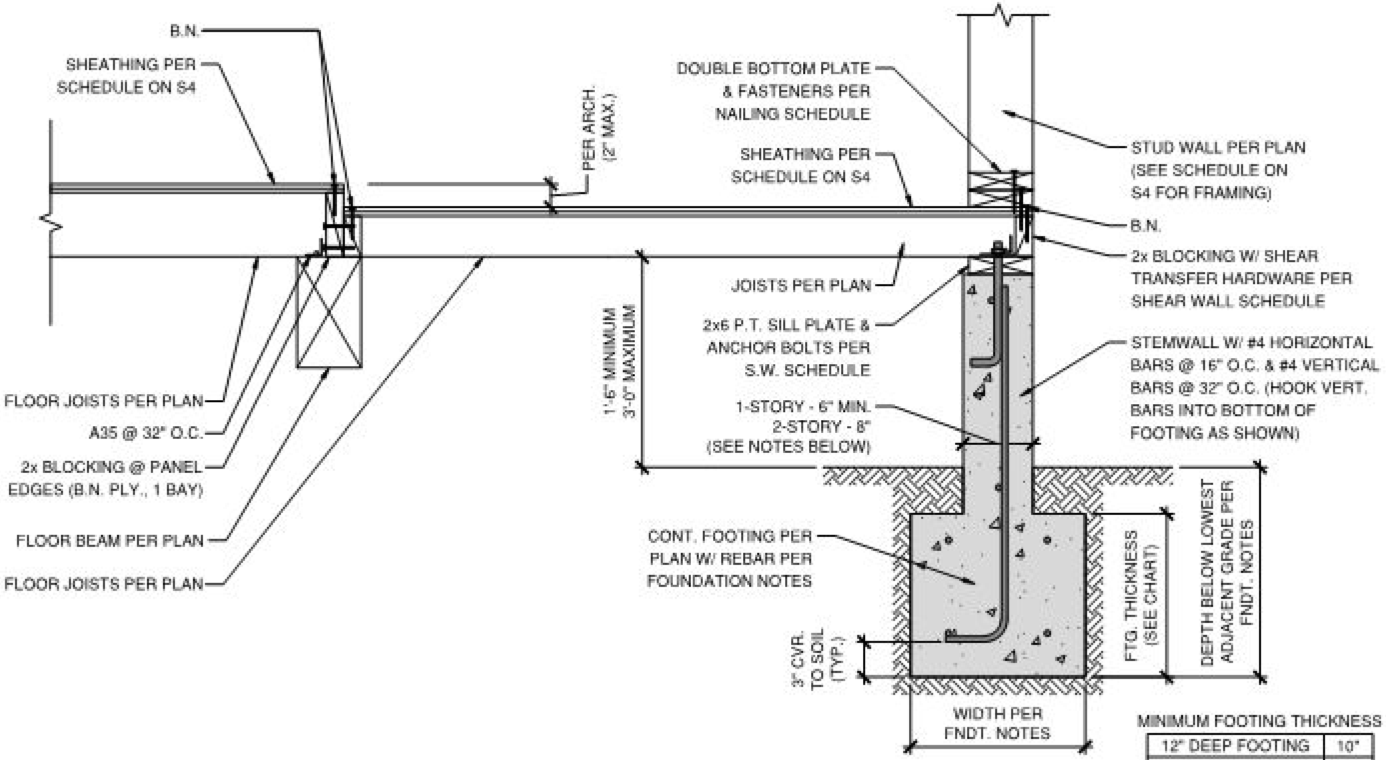
SPECIAL CONSIDERATIONS

APPENDIX D

SHOWER FLOOR



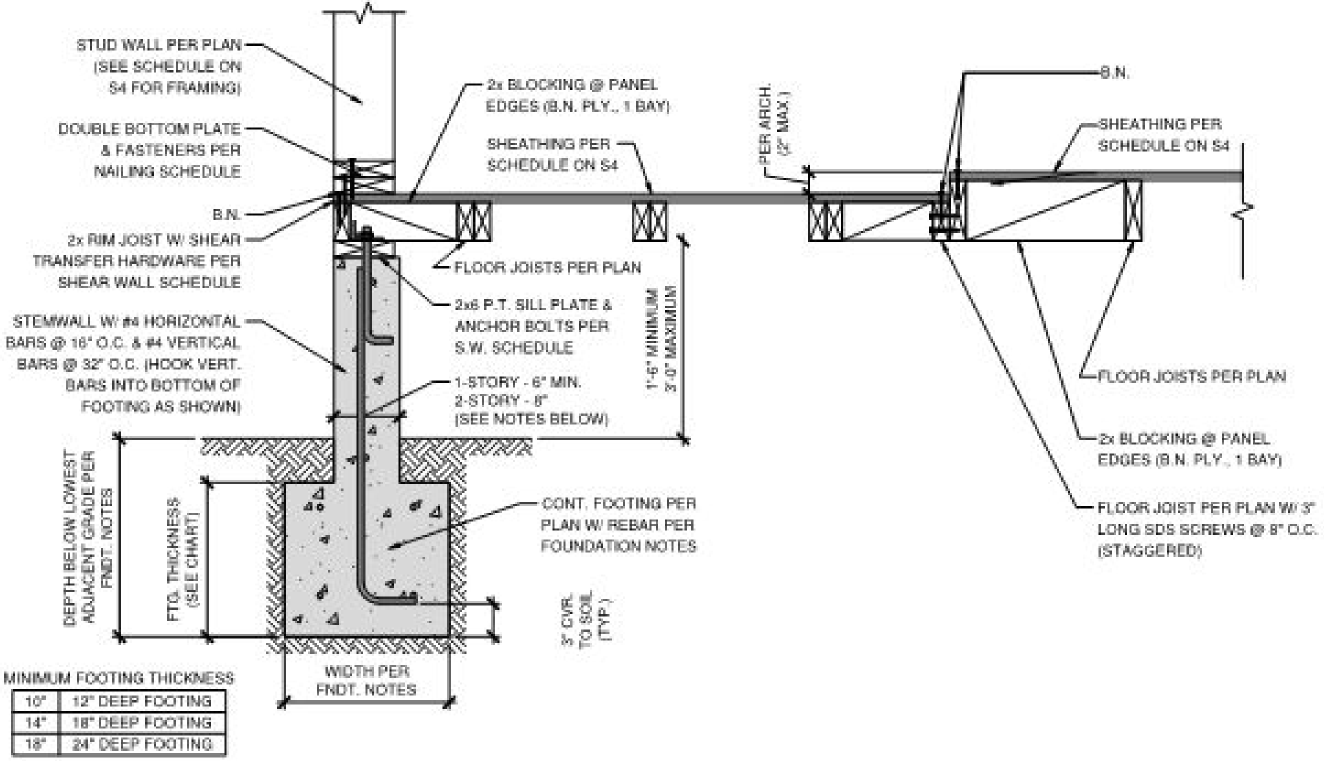
APPENDIX D



MINIMUM FOOTING THICKNESS

12" DEEP FOOTING	10"
18" DEEP FOOTING	14"
24" DEEP FOOTING	18"

APPENDIX D

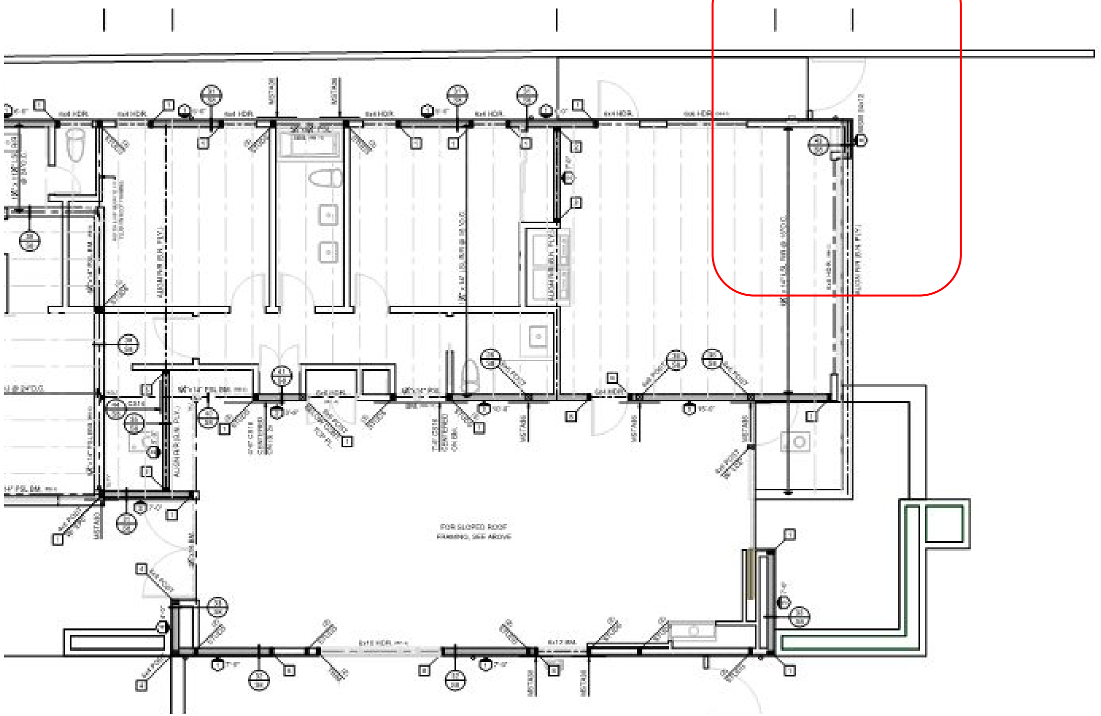


MINIMUM FOOTING THICKNESS

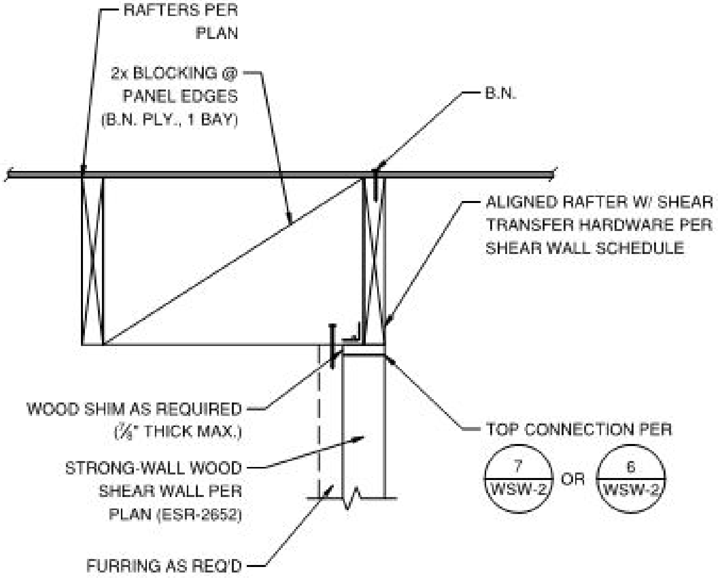
10"	12" DEEP FOOTING
14"	18" DEEP FOOTING
18"	24" DEEP FOOTING

APPENDIX D

LATERAL - STRONG WALL



APPENDIX D


RAFTERS PER
PLAN
2x BLOCKING @
PANEL EDGES
(B.N. PLY., 1 BAY)
B.N.
ALIGNED RAFTER W/ SHEAR
TRANSFER HARDWARE PER
SHEAR WALL SCHEDULE
WOOD SHIM AS REQUIRED
(⅞" THICK MAX.)
STRONG-WALL WOOD
SHEAR WALL PER
PLAN (ESR-2652)
FURRING AS REQ'D
TOP CONNECTION PER
7
WSW-2
OR
6
WSW-2

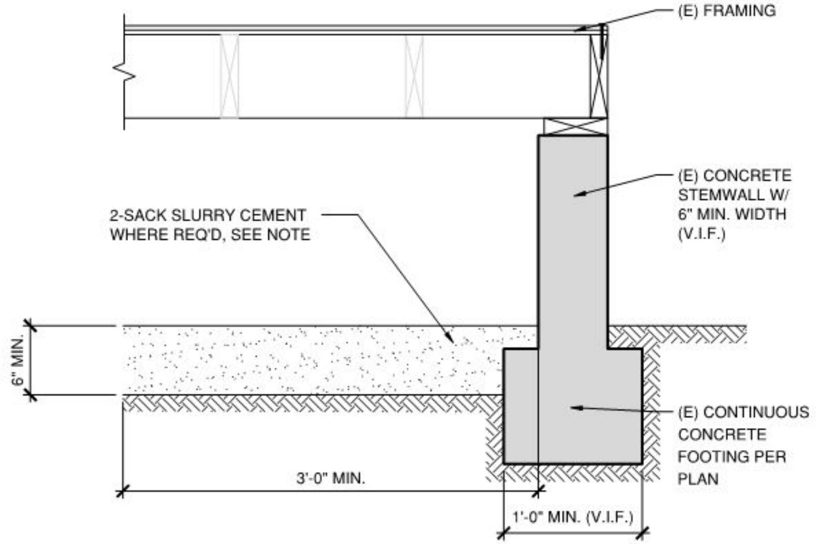
APPENDIX D

GEOTECHNICAL

RECOMMENDATIONS:

- ➔ 1,500 PSF BEARING PRESSURE
- ➔ 12” MIN. FOOTING WIDTH & DEPTH (BELOW GRADE)
- ➔ EXPANSIVE SOIL - RETROFIT FOR EXISTING FOOTINGS REQUIRED
- ➔ ALL STEM WALL CONTINUOUS FOOTINGS (CONCERN FOR SOME HOLDOWNS)

STEM WALL MODIFICATION



NOTES:

- WHERE EXISTING FOOTING EMBEDMENT IS LESS THAN 12" PLACE 6" OF 2-SACK SLURRY CEMENT ADJACENT TO THE INTERIOR SIDE OF THE STEM WALL AND EXTEND FOR AT LEAST 3 HORIZONTAL FEET AWAY FROM THE STEM WALL
- ALTERNATIVELY, MAY PLACE 6" OF COMPACTED FILL ADJACENT TO THE INTERIOR SIDE OF THE STEM WALL AND EXTEND FOR AT LEAST 3 HORIZONTAL FEET AWAY FROM THE STEM WALL.

APPENDIX D

CONCLUSIONS

- SYSTEM ADEQUATELY RESISTED IN BOTH GRAVITY & LATERAL
- ARCHITECTURAL FEATURES ACKNOWLEDGED & ACCOMMODATED STRUCTURALLY
- CONSTRUCTION FEASIBILITY ACCOMMODATED (STILL IN PROCESS)
  - PLAN CHECK RESPONSES IN PROGRESS
- COST EVALUATION KEPT IN MIND, DUE TO RESIDENTIAL NATURE
  - NAILS V. BOLTS
  - EPOXY HARDWARE @ MINIMUM

APPENDIX D

QUESTIONS/COMMENTS

APPENDIX D