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STRUCTURAL & EARTHQUAKE ENGINEERING

Structural Evaluation & Condition Assessment

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City of Manzanita
Elementary School and Quonset Hut
Manzanita, Oregon

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APPENDICES

Executive Summary

The Elementary School and Quonset Hut are located in Manzanita, Oregon and are both single-story, wood-framed structures. The Elementary School is approximately 5,478 square feet and the Quonset Hut is approximately 4,160 square feet. An American Society of Civil Engineers (ASCE) 41-17 seismic evaluation and a 2014 Oregon Structural Specialty Code (OSSC) structural evaluation were performed on the two buildings. In addition, a condition assessment was performed. Several structural and nonstructural components were found to be deficient. A conceptual strengthening scheme to address the deficiencies is summarized below.

Elementary School

- Roof framing will be strengthened at select locations of snow drift by adding additional 2x framing members. Rigid insulation and a membrane roof system will be installed over new plywood.
- New shear walls will be added by installing plywood sheathing and infilling openings, where required. Hold-downs and enlarged footings will be added at select shear walls. The concrete masonry unit (CMU) wall at the original hallway will be demolished and rebuilt as a wood shear wall.
- Deteriorated foundation walls along the west side will be repaired with new concrete and dowels.
- Deteriorated wood framing will be replaced. New siding will be added to the west side and wherever new plywood sheathing was installed.
- Corroded anchor bolts and connections to footings will be replaced with post-installed epoxy anchor bolts.
- The masonry chimney will be demolished to below the roof sheathing. Remaining masonry walls will be strengthened with HSS 4x4 strongbacks.
- Cracked masonry walls will be strengthened with fiber reinforced polymer (FRP).
- Post-installed anchors will be installed at exterior post bases.

Quonset Hut

- Corroded anchor bolts and connections to footings will be replaced with post-installed epoxy anchor bolts and galvanized angles, where required.
- CMU piers will be demolished and rebuilt as wood-framed piers and braced to the structure.
- New shear walls will be added by installing plywood sheathing.
- Add new plywood to the mezzanine. Increase plywood nailing at the mezzanine walls and add anchor bolts from walls to the foundation.

Both Buildings

- Strengthen the roof diaphragms and the connection of the diaphragms to the new shear walls. Diaphragms will be strengthened by installing new plywood sheathing. The transfer of forces from the roof diaphragm to the shear walls will be strengthened with new blocking, straps, and clips.

In general, the Elementary School is in poor condition and appears to have suffered from many years of deferred maintenance and neglect. The Quonset Hut is in fair condition for a building of its age. There are deferred maintenance items noted, but the Quonset Hut is in far better shape than the Elementary School.

We have prepared a conceptual estimate of probable construction costs for all strengthening schemes. A summary of the strengthening scheme costs is provided in the table below.

Summary of Project Costs by Strengthening Option			
Construction Category	**Strenghten Elementary School**	**Strengthen Quonset Hut**	**Demolish Both Buildings**
Structural Strengthening	$29,940	$14,319	-
Condition Assessment	$646,055	$116,306	-
Demolition	$58,889	$14,036	$172,310
Margins & Adjustments	$587,744	$115,697	$137,810
Total	**$1,322,628**	**$260,358**	**$310,120**
Total Area	5,478 Sq. Ft.	4,160 Sq. Ft.	9,638 Sq. Ft.
$/Sq. Ft.	$241	$63	$32

1. Project Background

A structural evaluation and building condition assessment of the Elementary School and the Quonset Hut located in Manzanita, Oregon have been performed to determine the expected structural building performance. The purpose of our structural evaluation is to identify the structural deficiencies that exist at the Elementary School and the Quonset Hut. The structural evaluation and building condition assessment are the basis for the conceptual strengthening schemes to address the identified deficiencies. In addition, the evaluation will be used to develop a cost estimate for the options to either renovate or demolish the structures.

Our work is based on the following:

1. A review of available original construction documents for the Elementary School, dated July 10, 1948. No construction drawings were available for the Quonset Hut. Relevant drawings have been included in Appendix A for reference.

2. A site visit by Brian Knight, P.E., S.E., of WRK Engineers on September 13, 2018 to verify the structural system shown in the original construction drawings and to assess the as-built conditions of the structural systems.

2. Evaluation Criteria and Methodology

2.1 Wind, Snow, and Live Loads

The structural evaluation was performed to current loading requirements based on the 2014 Oregon Structural Specialty Code (OSSC) for wind, snow, and live loads (as appropriate). Live loads are only evaluated at elevated floor levels, which only occur at the Quonset Hut mezzanine. The live load is dependent on the intended occupancy/usage of the space. Loading criteria for wind, snow, and live loads are provided below. The live loads are for probable mezzanine occupancy usage.

- Snow Load = 25 PSF
- Design Wind Speed = 135 MPH
- Exposure Category = B
- Risk Category = II
- Live Load = 40 PSF (Classroom), 50 PSF (Office), and 125 PSF (Light Storage)

2.2 Seismic Loads

The seismic evaluation was performed using ASCE Standard 41-17, "Seismic Evaluation and Retrofit of Existing Buildings", published by the American Society of Civil Engineers (ASCE). ASCE 41 is the nationally recognized Standard for seismic assessment and strengthening of existing buildings. The goal of ASCE 41 is to identify "weak links" in a building's lateral force resisting system that can lead to significant failure and/or collapse.

The ASCE 41 methodology utilizes a series of checklists to screen for possible seismic deficiencies (i.e. Tier 1 Screening). Checklists are included in the Standard for all of the major structural systems, nonstructural elements, and geological and site hazards. The evaluating engineer addresses each statement and determines whether it is compliant or non-compliant. Compliant statements identify conditions that are acceptable. Non-compliant statements identify conditions that need mitigation or further investigation (i.e. Tier 2 or Tier 3 evaluation).

Our evaluation used an ASCE 41 Tier 1 screening to determine potential deficiencies. We then used the Tier 2 deficiency procedure to further evaluate the identified deficiencies and to determine strengthening measures. The nominal capacity of components is compared with the expected force demands determined by the Tier 2 analysis procedure to determine a demand-capacity ratio (DCR). Structural elements are identified as having adequate strength when the DCR > 1.0.

For each building, a performance objective must be selected. The performance objective consists of a target building performance level and a corresponding seismic hazard level. Common seismic hazard levels and their mean return periods are outlined in Table 1 below. There are two different basic performance objectives: Basic Performance Objective for Existing Buildings (BPOE) and Basic Performance Objective Equivalent to New Building Standards (BPON). The BPOE evaluates based on Basic Safety Earthquake-1E (BSE-1E) or BSE-2E. The BPON evaluates based on either BSE-1N or BSE-2N.

TABLE 1 – Seismic Hazard Levels and Return Period

Seismic Hazard Level	Probability of Exceedance in 50 Years	Mean Return Period
BSE-1E	50 Percent	225 Years
BSE-2E	5 Percent	975 Years
BSE-1N	10 Percent	475 Years
BSE-2N	2 Percent	2,475 Years

Our evaluation is based on Life Safety (LS) Structural Performance considering the BSE-1N earthquake event with a 10% probability of occurrence in 50 years (475-year) mean recurrence period). In other words, we are evaluating the building for compliance with a level consistent with the Oregon State Building Code for new Risk Category II Buildings. Please note that our evaluation does not incorporate a reduced hazard level (i.e. using 75% of BSE-1N).

The intent of the LS performance level is:

> *After a seismic event, the building will experience extensive damage to structural and nonstructural components, but some margin against partial or total structural collapse will remain. The basic vertical and lateral force resisting systems of the building retain some residual strength and stiffness from their pre-earthquake state. The risk of life-threatening injury as a result of structural damage is low, and structural repairs to the damaged structure are advised before re-occupancy.*

In other words, the LS performance level objective is meant to ensure that re-occupancy of the building is possible after structural repairs are made. However, repairs may not be economically feasible and may take between six and twelve months to complete.

Please note that ASCE 41-17 does not have a Tier 1 checklist for the LS performance level. The ASCE 41-17 Tier 1 checklist is for the Collapse Prevention (CP) performance level evaluated at the BSE-2N earthquake event. The BSE-2N is a larger earthquake than the BSE-1N. The lower CP performance level evaluated at the larger BSE-2N earthquake ill identify deficiencies that will also be present at the higher LS performance level evaluated at the lower BSE-1N earthquake.

3. Site Description and Seismicity

The buildings are located on a level site with the ground floor at grade.

The seismic soil coefficients used for evaluation are based on the current classifications from the ASCE 7-10 provisions. The site soil classification is assumed to be Class D.

Using maps developed by the United States Geological Survey (USGS), the site-specific parameter for short period spectral acceleration, S_{XS}, is 0.888 g. The spectral response acceleration parameter at a one second period, S_{X1}, is 0.681 g. Amplification factors used to account for the soil conditions at the site are F_V = 1.500 and F_a = 1.000. Based on ASCE 41-17, Tables 2-4, the buildings are located in an area of high seismicity.

4. Elementary School Evaluation

4.1 Building Description

The building was originally constructed in 1948 as a single-story structure. The structure has a total area of approximately 5,478 square feet between two buildings connected by a covered walkway. The structural system does not appear to have been substantially altered since the time of original construction. See Appendix A for original construction drawings.

The Elementary School is a wood bearing wall structure. The west building is rectangular in plan, approximately 40 feet by 135 feet, with three separate roof levels. The east building is rectangular in plan, approximately 26 feet by 52 feet, with two separate roof levels. There are no adjacent structures built directly against the elementary school structure. The only adjacent structure is the Quonset Hut located approximately seven feet away to the east.

The roof structure consists of 1x wood decking supported by either 4x wood roof joists at 3'-3" on center or 2x wood roof joists at 16" on center. Roof joists are supported at each end by wood stud bearing walls or wood posts and beams above windows and doors. The exterior walls are constructed with 2x wood studs at 16" on center. The walls are supported by below-grade concrete stem walls with continuous strip footings. Interior and exterior wood posts are supported by the below-grade concrete stem walls as well. The floor consists of a concrete slab-on-grade and an elevated concrete slab over the below-grade mechanical tunnel.

The building's lateral force resisting system consists of wood shear walls. The ASCE 41 Model Building Type is W2. The perimeter wood walls have several openings for windows and doors. The wood shear walls transfer lateral loads to the foundation.

The roof diaphragm is straight 1x decking that spans between wood shear walls. The diaphragm is considered flexible relative to the shear walls that support it.

The exterior face of the building is clad with a mixture of cedar wood shingles and light-gauge metal siding. The interior partitions are primarily wood stud walls with a plaster finish. However, there are hollow 8" concrete masonry unit (CMU) block partition walls around the boiler room and hallway at the original main entrance.

4.2 Building Condition Assessment

The Elementary School was assessed using only the visual condition of the structural components, building systems, and building envelope during the September 13, 2018 site visit. The building condition assessment is based on what was accessible to view with non-destructive methods. The condition of components in the attic was not assessed. The condition assessment of the mechanical, electrical, and plumbing (MEP) systems and the building envelope are outside of our professional expertise. However, the condition assessment of these systems is based on our professional opinion and experience with similar buildings.

The west building wall is significantly deteriorated due to the apparent failure of the envelope protection system, notably a lack of paint and failure of caulking. As such, we estimate 80% of the wood framing is likely experiencing dry rot and may need to be replaced along with the entire siding system. Photos of the current conditions are shown below.

Figure 1: Deteriorated Siding

Figure 2: Deteriorated Window Sill

In general, the Elementary School is in poor condition and appears to have suffered from many years of deferred maintenance and neglect. The observations of the condition assessment are summarized in Table 2 below.

TABLE 2 – Elementary School Building Condition Assessment	
Component	**Building Condition Assessment**
Foundation	The foundation along the west side of the building is severely deteriorated and exposing the exterior wall sill plate's anchor bolts. Exposed anchor bolts are severely corroded and need to be replaced. The foundation along the east wall closest to the Quonset Hut shows significant cracking. See Figures 3 & 4.
Exterior Walls	Exterior finishes along the west side of the building show significant deterioration and water damage. Exterior window sills show evidence of water damage and potential water infiltration and dry rot of wood studs. Wood posts along the west side show significant water damage in some locations.
Masonry Walls	Masonry walls have significant cracking.
Masonry Chimney	The masonry chimney has significant cracking and poor waterproofing at the base. The masonry chimney presents a structural hazard.

TABLE 2 – Elementary School Building Condition Assessment [continued]	
Component	**Building Condition Assessment**
Roof/Ceiling	The roof shows locations of ponding due to inadequate roof slope provided. The gutters and downspouts are backed up and are not draining water adequately off the roof. Locations observed of water staining on ceiling tiles, which indicates that roofing system repairs are needed. See Figure 5.
MEP Systems	The mechanical system appears to be inoperable and likely needs to be replaced. The electrical system appears to be inadequate and needs to be upgraded. The plumbing system appears to be partially inoperable and likely needs to be repaired or replaced. See Figure 6

Figure 3: Foundation Deterioration

Figure 4: Exposed Anchor Bolts

Figure5: Roof Ponding and Moss Buildup

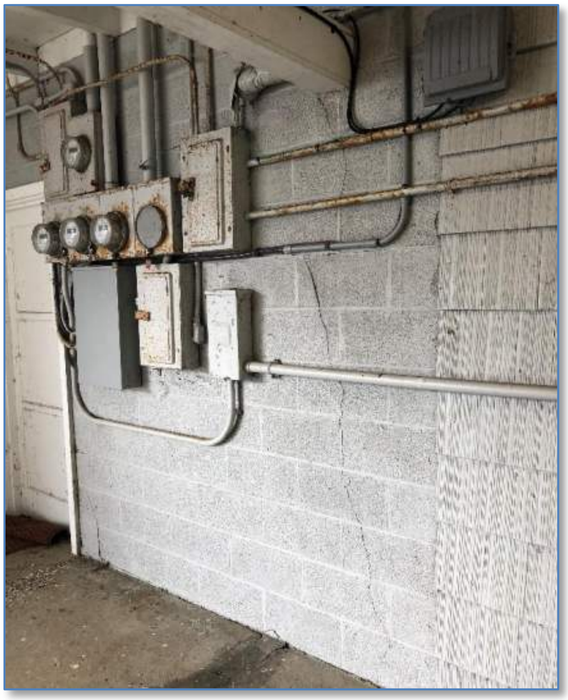


Figure 6: Cracked CMU Wall and Outdated Electrical

4.3 Structural Evaluation and Deficiencies

Based on our structural evaluation, we have identified structural deficiencies in the lateral and gravity systems. We evaluated the roof joists and wood posts under the current snow load criteria as provided in Section 2. The roof joists at the lower roofs on the west building have inadequate strength to resist the snow drifts expected to accumulate. The bearing walls are deemed structurally adequate based on observation of their construction, stud height, and spacing and were not evaluated.

For the lateral system, the 2014 OSSC wind loading was compared to the seismic loads calculated using ASCE 41-17. The structural demand on the lateral resisting system from wind loads is significantly less than the demand from seismic loads. Therefore, the structural evaluation of the lateral resisting system was based primarily on seismic loads.

Based on the gravity and lateral evaluation, numerous deficiencies were identified and are presented in Table 3 on the next page. Structural calculations are provided in Appendix C.

TABLE 3 – Elementary School Structural Deficiencies (Life Safety Performance Level)	
Structural Component Type	**Structural Deficiency**
LATERAL DEFICIENCIES	
Load Path	The diaphragm does not contain a complete load path to the shear walls. There is no apparent load path via blocking between the diaphragm and the shear walls to transfer in-plane seismic forces to the shear walls. In addition, there is an inadequate load path from the upper roof diaphragm of the west building to the lower shear walls due to the windows above the covered walkway.
Wall Stress Check	The straight-sheathed shear walls do not have adequate strength to resist expected seismic in-plane shear forces.
Narrow Wood Shear Walls	Some of the shear walls have an aspect ratio (height/length ratio) greater than 1.5-to-1.
Straight Roof Sheathing	The 1x6 wood straight-sheathed diaphragms have aspect ratios greater than 2-to-1, which may result in large displacements. In addition, the diaphragms have insufficient strength to resists seismic shear forces.
Wood Posts	Wood posts do not have a positive connection to the foundation.
Proportions	Unreinforced hollow CMU walls exceed the height-to-thickness ratio of 13 and have the potential for damage caused by out-of-plane forces.
GRAVITY DEFICIENCIES	
Roof Joists	The existing 4x12 at 3'-3" on center and 2x8 at 16" on center wood roof joists have inadequate strength to resist the current drift snow load at locations of snow drift at the southwestern lower roofs.

4.4 Conceptual Strengthening Scheme

Based on the building condition assessment, we have developed conceptual remediation measures to address the deficiencies in Section 4.2. The conceptual remediation measures are presented in Table 4 below.

TABLE 4 – Elementary School Building Condition Assessment Conceptual Remediation	
Component	**Remediation Measure**
Foundation	At deteriorated foundations, chip away the exposed concrete until quality concrete is reached. Place new concrete footing with #4 dowels at 12" on center into the existing slab and the existing continuous footing. Replace corroded sill plate anchor bolts with new 5/8-inch-diameter epoxy anchor bolts with 6" embedment into the footing. See Figure 5 in Appendix B for more information.

TABLE 4 – Elementary School Building Condition Assessment Conceptual Remediation [continued]	
Component	**Remediation Measure**
Exterior Walls	Demolish and rebuild the exterior wood stud wall along the west side of the building with ½" plywood sheathing and studs to match the original wall width. Replace water damaged posts with new Douglas Fir-Larch #1 posts sized to match the existing posts. Replace siding along the entire building with Hardi-plank lap siding. Remove and replace metal siding where it occurs.
Masonry Walls	Add a 60-mil fiber reinforced polymer (FRP) wrap to the exterior of cracked CMU walls.
Masonry Chimney	Demolish the existing masonry chimney to below the new plywood sheathing. Install new ½" plywood sheathing over the top of the masonry chimney.
Roof/Ceiling	Install new rigid insulation (4-inch-thick maximum) and membrane roof system over new ½" plywood sheathing. Replace gutters and downspouts.
MEP Systems	Replace mechanical system. Upgrade electrical system to meet current code requirements. Replace plumbing system where required.

Based on the structural evaluation, we have developed strengthening measures to address the deficiencies identified in Section 4.3. The conceptual strengthening measures are presented in Table 5 below.

TABLE 5 – Elementary School Conceptual Structural Strengthening Measures (Life Safety Performance Level)	
Structural Deficiency	**Structural Strengthening Measure**
LATERAL	
Load Path	Infill windows above the covered walkway with wood-framed shear walls and sheath exterior face with ½" plywood. In addition, add blocking and clips between the shear walls and the diaphragm to transfer in-plane seismic forces from the diaphragm to the shear walls.
Wall Stress Check	Remove shiplap at new shear walls where it occurs and add ½" plywood sheathing with fully blocked panel edges. Demo the existing masonry wall along the hallway at the original entrance and rebuild as 2x8 wood stud shear wall with ½" plywood sheathing. Add hold-downs at each end of selected shear walls. Additional studs may be needed at hold-downs. There are approximately 24 hold-down locations. Increase footing sizes at selected shear walls to resist the overturning demand on the shear walls.
Narrow Wood Shear Walls	New narrow wood shear walls have been designed to have capacity for the seismic demand. See "Wall Stress Check" strengthening measure.

TABLE 5 – Elementary School Conceptual Structural Strengthening Measures [continued] (Life Safety Performance Level)	
Structural Deficiency	**Structural Strengthening Measure**
LATERAL	
Straight Roof Sheathing	Add ½" plywood over the existing straight sheathing over the entire roof area with 10d at 4" on center at the edges and 12" on center in the field. At select locations on Figure 2, provide Simpson CMST16 along the edge of the diaphragm over the wall below.
Wood Posts	Add two Simpson RPBZ with 3/8-inch-diameter expansion anchor bolts at each exterior post location. There are approximately six locations.
Proportions	Install HSS 4x4x3/16 strongbacks at 5'-0" on center maximum. Provide 5/8-inch-diameter epoxy anchor bolts at 36" on center along full height of strongback and 6" minimum from top and bottom of HSS. Provide 6" embedment of epoxy anchor bolts in the masonry wall.
GRAVITY	
Roof Joists	Sister Douglas Fir-Larch #2 2x12 to the existing 4x12 roof joists and Douglas Fir-Larch #2 2x8 to the existing 2x8 roof joists at locations where snow drift is expected to accumulate.

5. Quonset Hut Evaluation

5.1 Building Description

The Quonset Hut has a total area of approximately 4,160 square feet. The building has a wood-framed mezzanine, 12 feet by 17 feet, located inside at the back of the structure. There are two 14-foot-high, exterior, nonstructural CMU piers at the front of the structure. There is also a 960-square-foot storage structure attached to the back of the Quonset Hut that is included in the total Quonset Hut area.

The Quonset Hut is a single-story glulam arch structure that is semi-circular in cross-section with a wood-bearing-wall-framed storage structure, 30 feet by 32 feet, attached. The building is rectangular in plan, approximately 40 feet by 110 feet. The Quonset Hut is approximately seven feet to the east of the Elementary School. There is an adjacent small, rectangular utility shed to the east of the Quonset Hut.

The roof of the Quonset Hut is composed of metal decking supported by 2x8 glulam arches at 2'-6" on center. The roof of the attached storage structure consists of metal roofing over particle board sheathing supported by pre-manufactured wood trusses at 2'-0" on center. The trusses are supported by wood stud bearing walls that are constructed out of 2x studs at 16" on center. At locations where the trusses bear on the wood stud walls over openings, they are supported by wood beam headers. The Quonset Hut arched structure has wood stud bearing walls at the north and south ends.

Figure 7: Quonset Hut Interior

The floor system is a concrete slab-on-grade with assumed thickened footings under bearing walls. The walls and glulam arches are supported by a continuous concrete foundation.

The Quonset Hut arched structure resists lateral forces through the semi-circular metal diaphragm in the longitudinal direction (north-south) and through the glulam arches in the transverse direction (east-west). In addition, there is plywood sheathing in the longitudinal direction that extends eight feet up on the arch on both sides.

The lateral force resisting system at the attached storage structure consists of wood shear with particle board sheathing. The ASCE 41 Model Building Type is W2. The perimeter wood walls have some openings for windows and doors. The lateral force resisting system transfers loads to the foundation through anchor bolts.

The roof diaphragm of the Quonset Hut is semi-circular metal decking with multiple skylight openings. The roof diaphragm of the storage structure is particle board sheathing that spans between wood shear walls. The diaphragm is considered flexible relative to the shear walls that support it.

The exterior face of the building consists of a mixture of metal siding and cedar wood shingles. The interior partitions are wood stud walls with plywood sheathing. There are two exterior hollow CMU block piers at the front of the Quonset Hut.

5.2 Building Condition Assessment

The Quonset Hut was assessed on the visual condition of the structural components during the September 13, 2018 site visit. The condition assessment is based on what was accessible to view with non-destructive methods.

In general, the Quonset Hut is in fair condition for a building of its age. There are deferred maintenance items noted, but the Quonset Hut is in far better shape than the

Elementary School. The observations of the condition assessment are summarized in Table 6 below.

TABLE 6 – Quonset Hut Building Condition Assessment	
Component	**Building Condition Assessment**
Foundation	At the Quonset Hut, the connections between the glulam arches and the foundation are severely corroded. In addition, there are trees growing close to the foundation that have potentially damaged the foundation. See Figure 8
Exterior Walls	At the storage structure, there is evidence of water damage where exterior walls meet the grade, including rusted metal siding and rotted wood framing around openings. At the Quonset Hut, there is inadequate clear distance between the bottom of the exterior finishes and the final grade.
Masonry Piers	Nonstructural masonry piers at the front of the Quonset Hut are free-standing and unbraced. The piers present a falling hazard during a seismic event.
Roof	Metal decking at the storage structure is significantly corroded from water damage. The gutters appear to be damaged and the downspouts do not extend to the ground. There is noticeable metal siding corrosion. See Figure 9

Figure 8: Rusted Glulam Arch to Foundation Connection

Figure 9: Storage Structure Deterioration

5.3 Structural Evaluation and Deficiencies

For the Quonset Hut gravity system, we evaluated the glulam arches under the current snow load criteria as provided in Section 2. The storage structure's pre-manufactured wood trusses were not evaluated and are assumed to be adequate based on observations of their construction and spacing. The bearing walls are deemed structurally adequate based on observation of their construction, stud height, and spacing. The Quonset Hut mezzanine floor's joist framing was not accessible to view and is assumed to be 2x12 joists at 24" on center. The framing is adequate for live loads corresponding to the classroom or office occupancy use (40-50 PSF). However, if the mezzanine is to be used for light storage, the live load required is 125 PSF and the assumed framing is inadequate. Additional 2x12 joists will be required to be installed centered in between the existing joists to achieve a storage use.

For the lateral system, the 2014 OSSC wind loading was compared to the seismic loads calculated using ASCE 41-17. The structural demand on the lateral resisting system from wind loads is significantly less than the demand from seismic loads. Therefore, the structural evaluation of the lateral resisting system was based primarily on seismic loads. The glulam arch connections were evaluated for the combination of the lateral and gravity loads. The metal roof decking size and fastening pattern of the glulam arch is unknown, but is assumed to be structurally adequate based on observation.

Based on the gravity and lateral evaluation, lateral deficiencies were identified and are presented in Table 7 on the next page. No gravity deficiencies were identified at the Quonset Hut structure. Structural calculations are provided in Appendix C.

TABLE 7 – Quonset Hut Structural Deficiencies (Life Safety Performance Level)	
Structural Component Type	**Structural Deficiency**
LATERAL DEFICIENCIES	
Load Path	The storage structure roof diaphragm does not contain a complete load path to the shear walls. There is no apparent load path via blocking between the diaphragm and the shear walls to transfer in-plane seismic forces to the shear walls.
Mezzanine	The interior mezzanine is braced by interior wood stud walls with plywood sheathing. The provided load path is inadequate and needs to be strengthened.
Wall Stress Check	The storage structure's particle-board-sheathed shears walls do not have adequate strength to resist the expected seismic in-plane shear forces.
Narrow Wood Shear Walls	Some of the shear walls have an aspect ratio (height/length ratio) greater than 1.5-to-1.
Other Diaphragms	The storage structure's roof diaphragm consists of a system other than wood, metal deck, concrete, or horizontal bracing. The particle board diaphragm has insufficient strength to resist seismic shear forces.

5.4 Conceptual Strengthening Scheme

Based on the building condition assessment, we have developed conceptual remediation measures to address the deficiencies identified in Section 5.2. The conceptual remediation measures are presented in Table 8 below.

TABLE 8 – Quonset Hut Building Condition Assessment Conceptual Remediation	
Component	**Remediation Measure**
Foundation	At the Quonset Hut, install new galvanized L6x4x3/8 on each side of the existing glulam arch with a ½-inch-diameter thru-bolt and a ½-inch-diameter anchor bolt with 3" minimum embedment into the footing.
Exterior Walls	At the storage structure, replaced rotted wood framing at openings where it occurs. Replace metal siding at the storage structure.
Masonry Piers	At the Quonset Hut, demo the existing CMU piers. Rebuild as 2x8 wood piers with plywood sheathing and frame into the structure.
Roof	At the storage structure, remove the existing metal roofing. Install new plywood per the comment in Table 9. Replace gutters and downspouts.

Based on the structural evaluation, we have developed strengthening measures to address the deficiencies identified in Section 5.3. The conceptual strengthening measures are presented in Table 9 below.

TABLE 9 – Quonset Hut Conceptual Structural Strengthening Measures (Life Safety Performance Level)	
Structural Deficiency	**Structural Strengthening Measure**
LATERAL	
Load Path	At the storage structure, add blocking and clips between the shear walls and the diaphragm to transfer in-plane seismic forces from the diaphragm to the shear walls.
Mezzanine	Add ½" plywood sheathing to the mezzanine diaphragm with 10d at 6" on center at the edges and 12" n center in the field. Strengthen plywood sheathing nailing to 10d at 6" on center at the edges at the interior wood stud walls. Provide 5/8-inch-diameter adhesive anchor bolts at 3'-0" on center along the sill plate.
Wall Stress Check	At the storage structure, remove the particle board sheathing at new selected shear walls and add ½" plywood sheathing with fully blocked panel edges and nailing per Figure 4 in Appendix B.
Narrow Wood Shear Walls	At the storage shed, new narrow wood shear walls have been designed to have capacity for seismic demand. See "Wall Stress Check" strengthening measure.
Other Diaphragms	Remove metal roofing. Overlay new ½" plywood sheathing with 10d at 4" on center at the edges and 12" on center in the field. Block all panel edges.

6. Demolition

We have provided a conceptual estimate of the probable construction costs associated with the option to demolish the Elementary School and the Quonset Hut. The demolition option accounts for the complete demolition and disposal of the buildings and foundations, the removal of utility lines back to the point of connection with the utility, and rough grading of the site after demolition. We have not accounted for hazardous materials abatement and assume a local qualified abatement contractor will be consulted for this portion of the demolition.

7. Conceptual Estimate of Probable Construction Costs

We have developed an estimate of probable construction costs for remediation of the Elementary School and the Quonset Hut. Expected construction costs include contingencies for design, construction, and other marginal adjustments.

MEP systems and the building envelope are outside of our professional expertise. However, the estimated cost for MEP systems and the building envelope remediation are based on our professional opinion and experience with similar buildings. We have also included an allowance for the replacement of doors and windows. The contingency costs account for the uncertainty in the MEP and building envelope remediation estimate. If a more accurate and detailed cost estimate for these systems is desirable, a qualified consultant should be contacted.

In addition, the cost estimate does not account for hazardous materials abatement. We assume the City of Manzanita will use the asbestos study from 2017 to gather bids from local qualified abatement contractors to determine this portion of the building renovation or demolition expense as this work will be needed for either option.

7.1 Remediation Cost Estimate

A summary of the expected costs of the remediation option for the Elementary School and the Quonset Hut is provided in Table 10 below. See Appendix D for the complete cost estimate.

TABLE 10 – Remediation Cost Estimate		
Construction Category	**Elementary School**	**Quonset Hut**
Structural Strengthening	$29,940	$14,319
Condition Remediation	$646,055	$116,306
Demolition	$58,889	$14,036
Margins & Adjustments	$587,744	$115,697
Total	**$1,322,628**	**$260,358**
Total Area	5,478 Sq. Ft.	4,160 Sq. Ft.
$/Sq. Ft.	$241	$63

7.2 Demolition Cost Estimate

A summary of the expected costs of the demolition option for the Elementary School and the Quonset Hut is provided in Table 11 below.

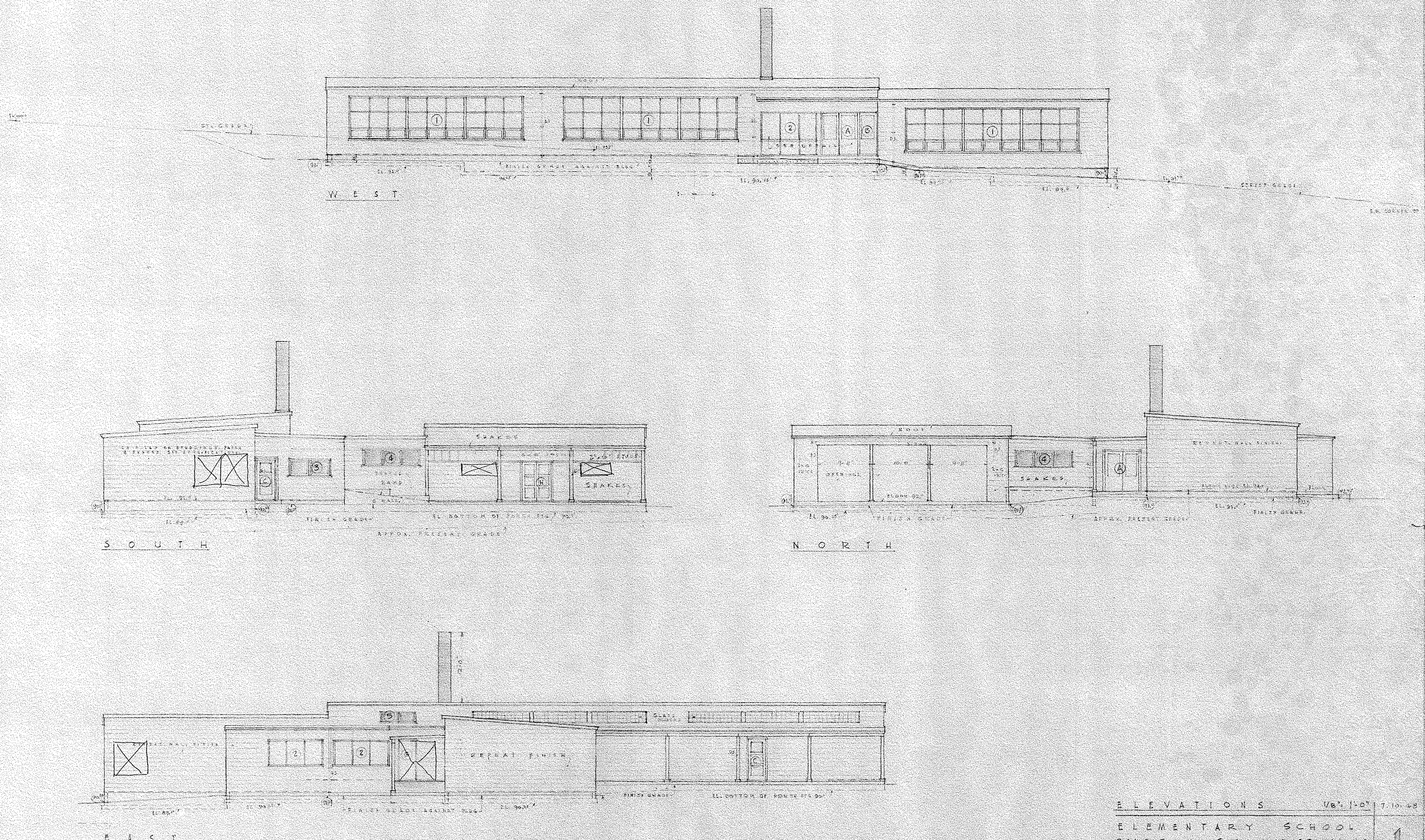
TABLE 11 – Demolition Cost Estimate			
Construction Category	**Elementary School**	**Quonset Hut**	**Both Buildings**
Building Demolition	$89,545	$39,015	$128,560
Site Utilities	$22,000	$10,000	$32,000
Earthwork	$8,250	$35,000	$11,750
Contingencies	$95,809	$42,000	$137,810
Total	**$215,604**	**$94,515**	**$310,120**
Total Area	5,478 Sq. Ft.	4,160 Sq. Ft.	9,638 Sq. Ft.
$/Sq. Ft.	$39	$23	$32

8. Limitations

The opinions and recommendations presented in this report were developed with the care commonly used as the state of practice of the profession. No other warranties are included, either expressed or implied, as to the professional advice included in this report. This report has been prepared for the City of Manzanita to be used solely in its structural evaluation of the buildings included herein. This report has not been prepared for use by other parties and may not contain sufficient information for purposes of other parties or uses.

Appendix A

ORIGINAL CONSTRUCTION DRAWINGS

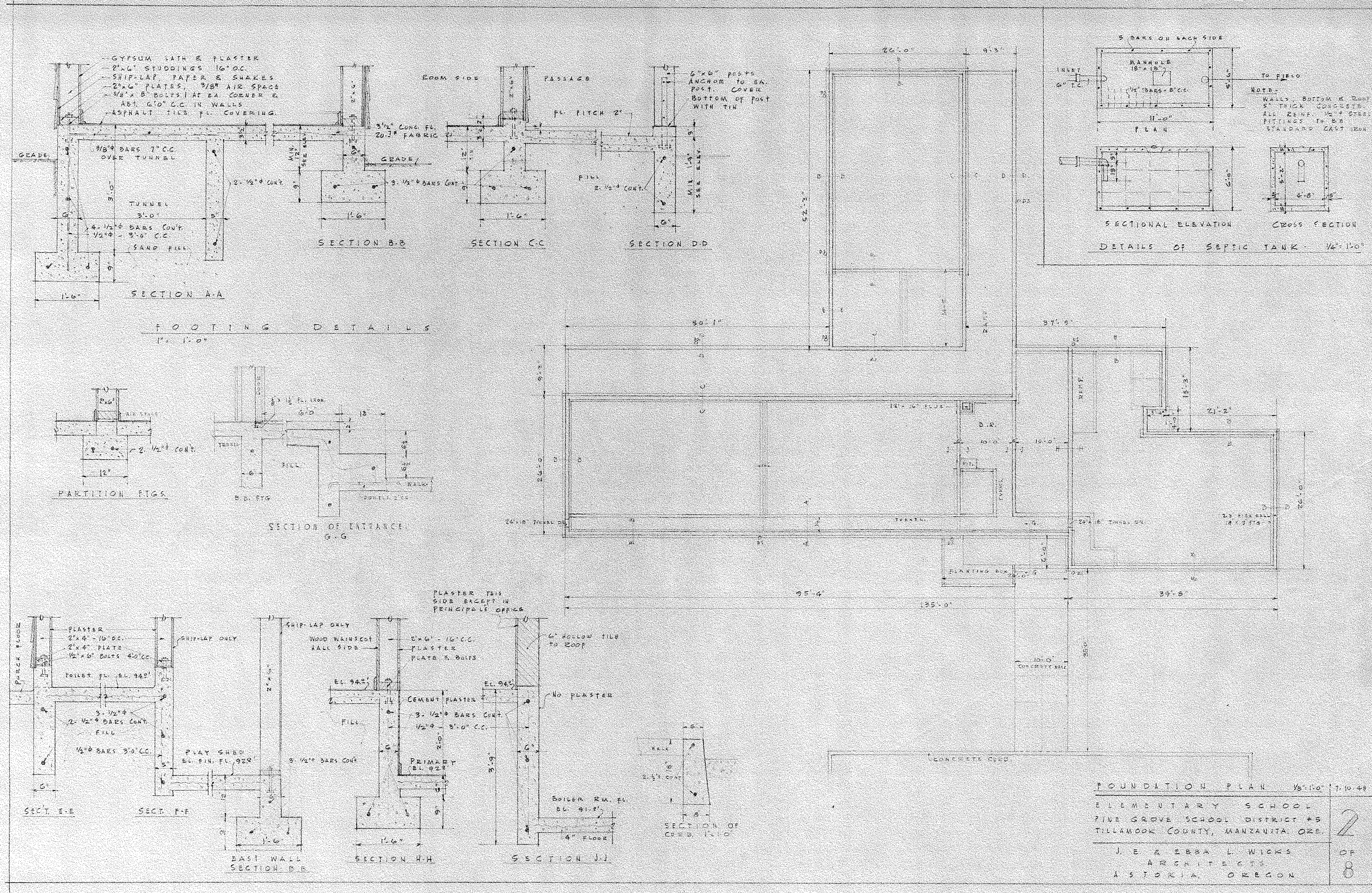


ELEVATIONS 1/8" = 1'-0" 7.10.48

ELEMENTARY SCHOOL
PINE GROVE SCHOOL DISTRICT #5
TILLAMOOK COUNTY, MANZANITA, ORE.

J. E. & EBBA L. WICKS
ARCHITECTS
ASTORIA, OREGON

1 OF 8


GYPSUM LATH & PLASTER
2"x6" STUDDINGS 16" O.C.
SHIP-LAP PAPER & SHAKES
2"x6" PLATES. 3/8" AIR SPACE
5/8" x 8" BOLTS | AT EA. CORNER & ABT. 6'0" C.C. IN WALLS
ASPHALT TILE FL. COVERING
GRADE
3/8"ϕ BARS 7" C.C. OVER TUNNEL
TUNNEL
SAND FILL
SECTION A-A
3½" CONC. FL.
GRADE
SECTION B-B
ROOM SIDE
PASSAGE
FL. PITCH 2"
SECTION C-C
6"x6" POSTS ANCHOR TO EA. POST. COVER BOTTOM OF POST WITH TIN
FILL
SECTION D-D
FOOTING DETAILS
1" = 1'-0"
PARTITION FTGS.
SECTION OF ENTRANCE G-G
PLASTER
SHIP-LAP ONLY
TOILET FL. EL. 94.0'
FILL
SECT. E-E
SECT. F-F
PLAY SHED
EAST WALL SECTION B-B
WOOD WAINSCOT HALL SIDE
CEMENT PLASTER
PRIMARY
SECTION H-H
PLASTER THIS SIDE EXCEPT IN PRINCIPAL'S OFFICE
6" HOLLOW TILE TO ROOF
NO PLASTER
BOILER RM. FL.
4" FLOOR
SECTION J-J
SECTION OF CURB 1"=1'-0"
TUNNEL
PLANTING BOX
CONCRETE WALK
CONCRETE CURB
135'-0"
5 BARS ON EACH SIDE
MANHOLE
INLET
TO FIELD
NOTE:
WALLS, BOTTOM & ROOF 5" THICK CONCRETE. ALL REINF. ½"ϕ STEEL. FITTINGS TO BE STANDARD CAST IRON.
PLAN
SECTIONAL ELEVATION
CROSS SECTION
DETAILS OF SEPTIC TANK 1/4" = 1'-0"
FOUNDATION PLAN 1/8" = 1'-0" 7-10-48
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2 OF 8

WINDOW SCHEDULE

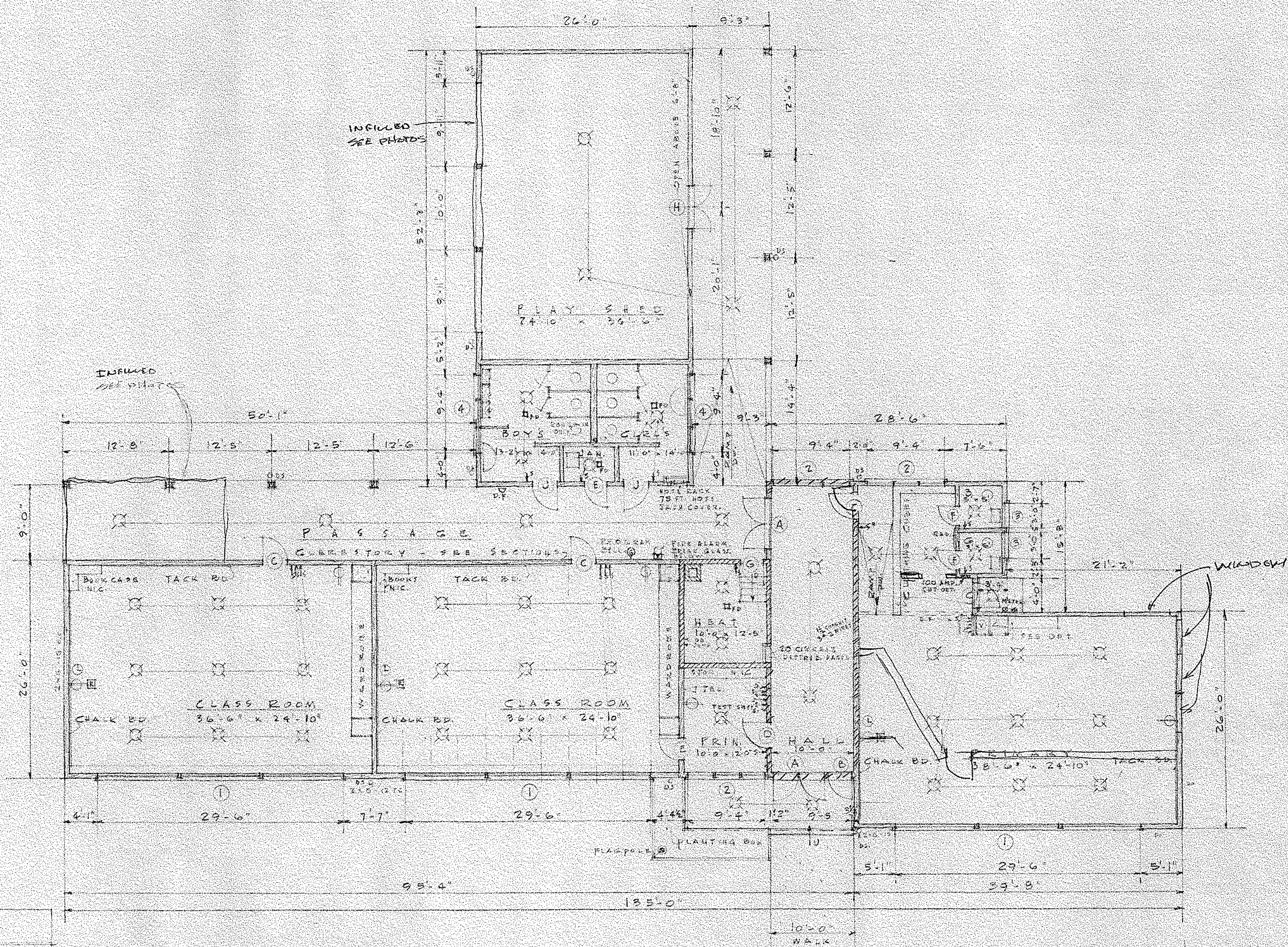
MARK	NO.	SIZE	OPERATION		GLASS
			FIXED	AWNING	
1	3	29'-6" x 7-3 x 1 5/8	18	9	D.S.B.
2	3	9-4 x 4-2" x 1 5/8	2	1 CASEMENT	3/16" CRYSTAL
3	2	3'-3" x 2-6" x 1 5/8	—	2	INDUSTREX
4	2	9'-4" x 2'-6" x 1 5/8	—	3	"
5	1	5-6 x 2'-6" x 1 5/8	—	2	"

DOOR SCHEDULE

MARK	NO.	SIZE	STYLE	GLASS	WOOD
A	2 PR.	6'-0" x 6'-8" x 2"	FLUSH	3/16" CRYSTAL	FIR
B	1	3'-0" x 6'-8" x 2"	" "	" "	"
C	4	3'-0" x 6'-8" x 1 3/4"	3 PANEL	TOP LIGHT DSB	"
D	1	2'-8" x 6'-8" x 1 3/4	3 PANEL	NONE	"
E	2	2'-8" x 6'-8" x 1 3/4"	" "	"	"
F	2	2'-4 x 6'-8" x 1 3/8"	" "	"	"
G	1	2'-8" x 6'-8"	IRON CLAD	"	—
H	1 PR.	5'-0" x 6'-8" x 1 3/4	3 PANEL	"	FIR
J	2	3'-0" x 6'-8" x 1 3/4"	" "	"	"

ELECTRICAL SYMBOLS.

- DUPLEX CONVENIENCE OUTLET
- RADIO & " "
- CLOCK
- TELEPHONE
- S SWITCH
- S3 3-WAY SWITCH
- CEILING OUTLET, CAPACITY - CLASS RM. OUTLETS WIRED FOR 300 WATTS EACH. HALL & SMALL RMS. 200 WATTS EA.



ROOM FINISH SCHEDULE

ROOM	FLOOR		WALLS		WAINSCOTE		CEILING		TRIM	
	MATERIAL	FINISH	MAT.	FIN.	MAT.	FIN.	MAT.	FIN.	MAT.	FIN.
CLASS	CONCRETE	ASPHALT TILE	PLASTER	PAINT	NONE		OPEN TIMBER FIBERBD.	REZ NAT.	FIR	PAINT
PRINCIPAL	"	" "	" 1 WALL FIR.TILE	"	NONE		FIBERBD.	"	"	"
HALL	"	" "	PLASTER	"	FIR	PAINT	"	"	"	"
PRIMARY TOILETS	"	" "	"	"	"	"	"	"	"	"
TOILETS	"	—	"	"	PLASTER	"	"	"	"	"
HEAT	"	—	HOLLOW TILE	NAT.	NONE		OPEN TIMBER	WATER PAINT	"	"
PLAY SHED	"	—	OPEN TIMBER	"	"		" "	NAT.	PAINT DOORS & WINDOWS	

FLOOR PLAN. 1/8" = 1'-0" | 7-10-48

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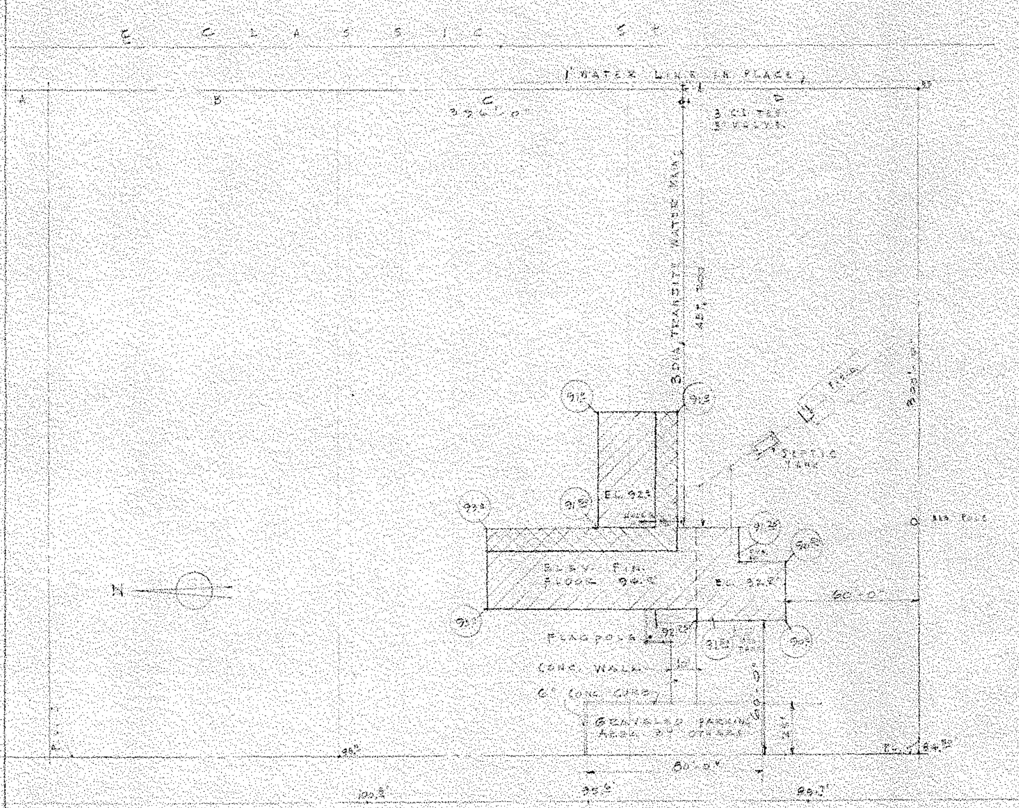
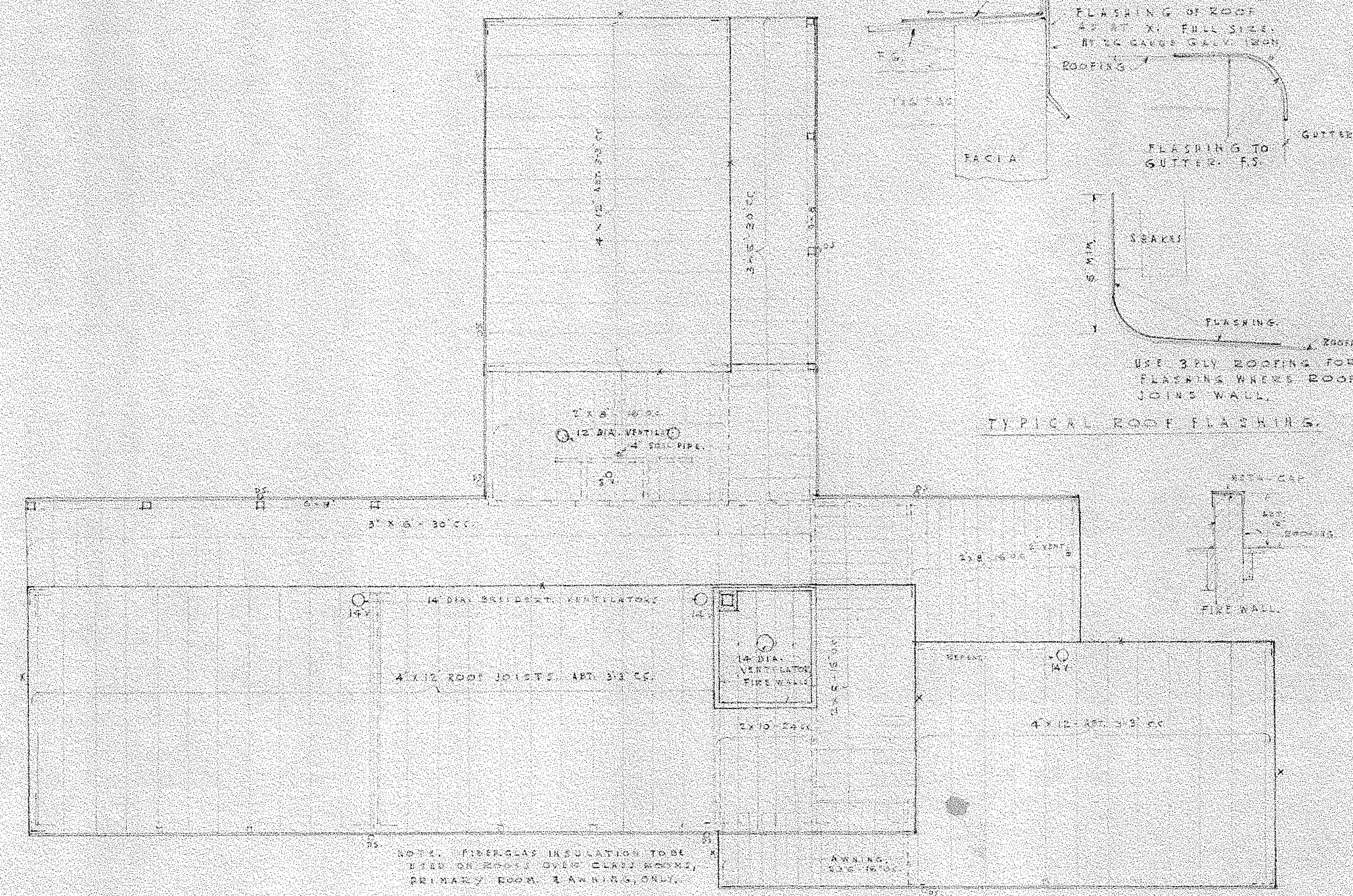


PLOT PLAN
SCALE 1" = 40'-0"



ROOF PLAN - 1/8" = 1'-0"

TYPICAL ROOF FLASHING.

ROOF & PLOT PLANS | 1.10.48

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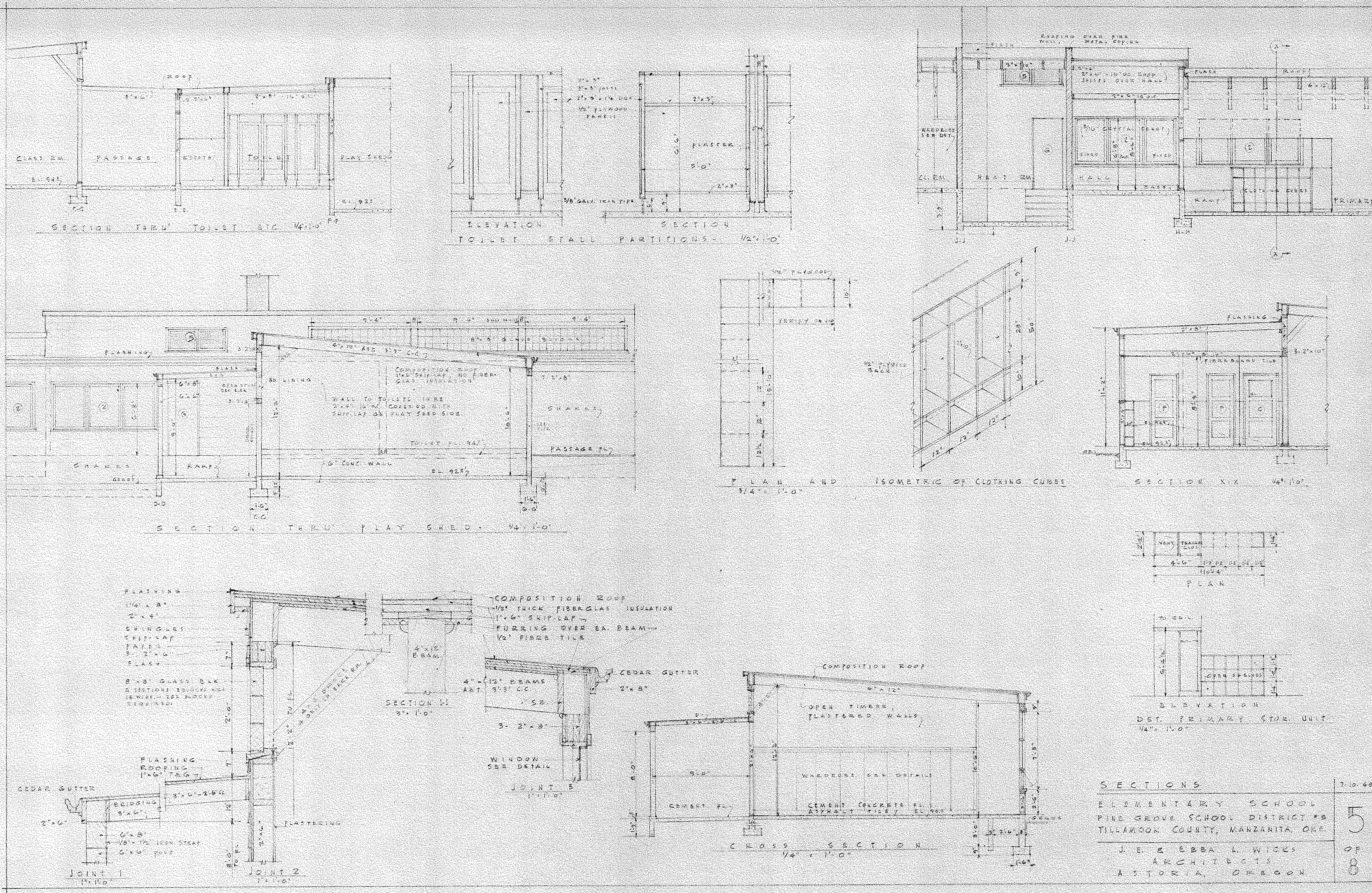
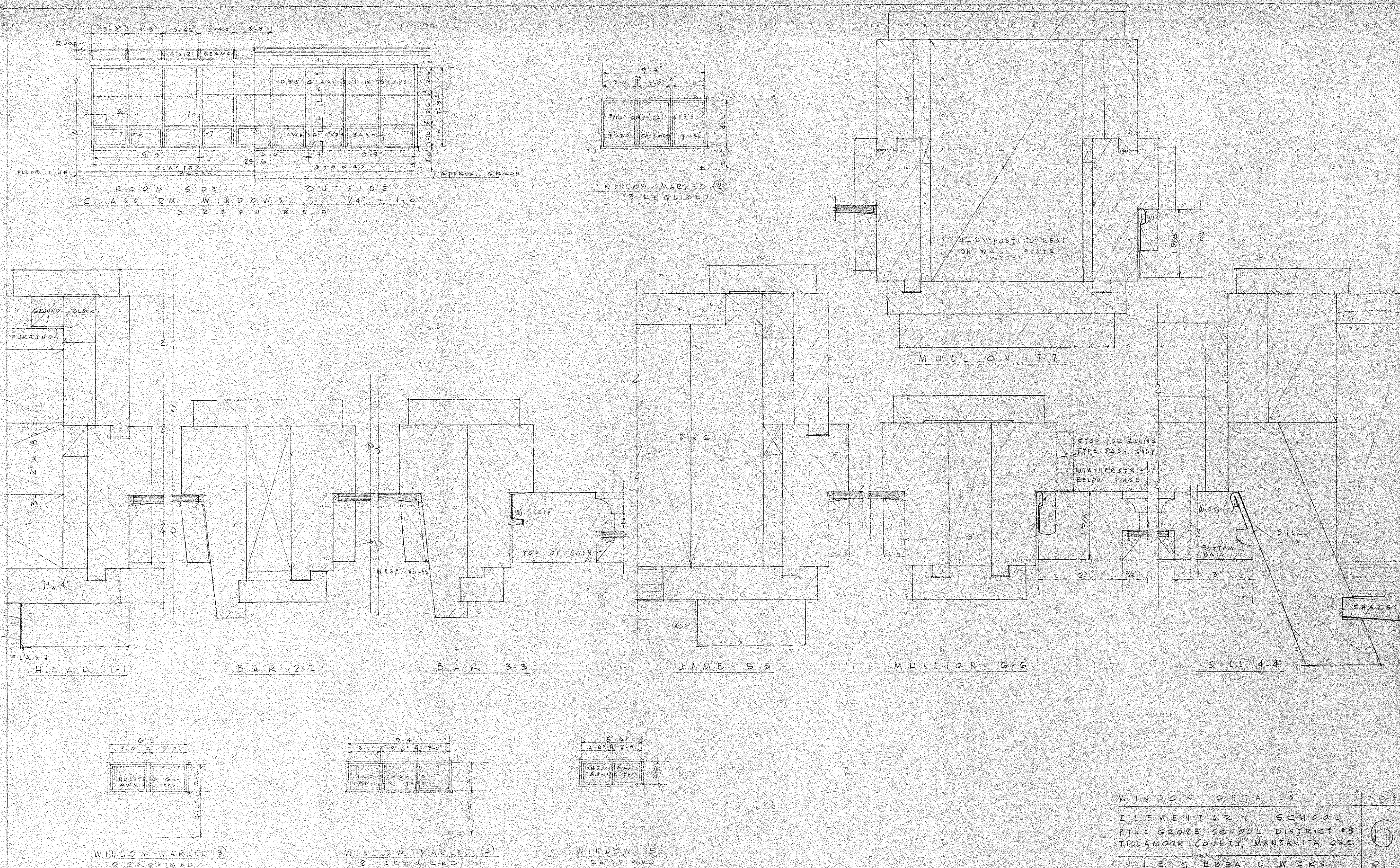

SECTION THRU' TOILET ETC. 1/4"=1'-0"
CLASS RM.
PASSAGE
TOILET
PLAY SHED
ELEVATION
SECTION
TOILET STALL PARTITIONS - 1/2"=1'-0"
PLASTER
3/8" GALV. IRON PIPE
HEAT RM.
HALL
PRIMARY
RAMP
CLOTHING CUBES
SECTION THRU' PLAY SHED - 1/4"=1'-0"
FLASHING
NO LINING
WALL TO TOILETS TO BE 2"x4" 16" o.c. COVERED WITH SHIP-LAP ON PLAY SHED SIDE.
SHAKES
PASSAGE FL.
RAMP
PLAN AND ISOMETRIC OF CLOTHING CUBES 3/4"=1'-0"
1/2" PLYWOOD
1/2" PLYWOOD BACK
SECTION X-X 1/4"=1'-0"
PLAN
ELEVATION
OPEN SHELVES
DET. PRIMARY STOR. UNIT
1/4"=1'-0"
FLASHING
SHINGLES
SHIP-LAP
PAPER
FLASH
8"x8" GLASS BLK.
COMPOSITION ROOF
1/2" THICK FIBERGLAS INSULATION
1"x6" SHIP-LAP
FURRING OVER EA. BEAM
1/2" FIBRE TILE
4"x12" BEAM
SECTION 1-1
3"=1'-0"
4"x12" BEAMS ABT. 3'-3" C.C.
CEDAR GUTTER
2"x8"
WINDOW SEE DETAIL
JOINT 3
1"=1'-0"
FLASHING
ROOFING
1"x6" T&G
CEDAR GUTTER
BRIDGING
PLASTERING
JOINT 1
1"=1'-0"
JOINT 2
1"=1'-0"
COMPOSITION ROOF
OPEN TIMBER, PLASTERED WALLS
WARDROBE SEE DETAILS
CEMENT FL.
CEMENT CONCRETE FL. ASPHALT TILE
CROSS SECTION
1/4"=1'-0"
SECTIONS
7-10-48
ELEMENTARY SCHOOL
PINE GROVE SCHOOL DISTRICT #5
TILLAMOOK COUNTY, MANZANITA, ORE.
5
J. E. & EBBA L. WICKS
ARCHITECTS
ASTORIA, OREGON
OF
8



WINDOW DETAILS | 7-10-48

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PINE GROVE SCHOOL DISTRICT #5
TILLAMOOK COUNTY, MANZANITA, ORE.

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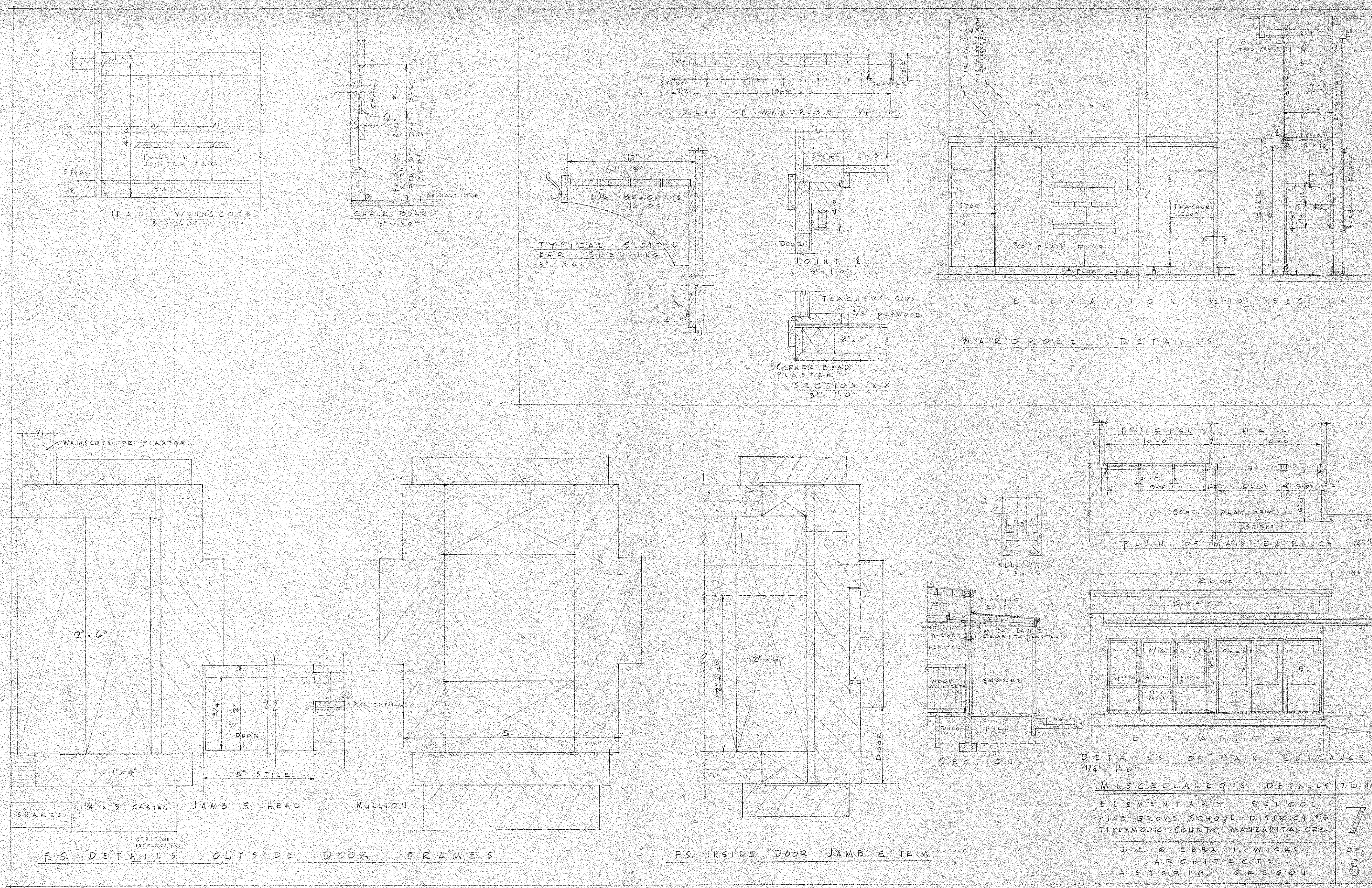

MISCELLANEOUS DETAILS 7.10.48

ELEMENTARY SCHOOL
PINE GROVE SCHOOL DISTRICT #3
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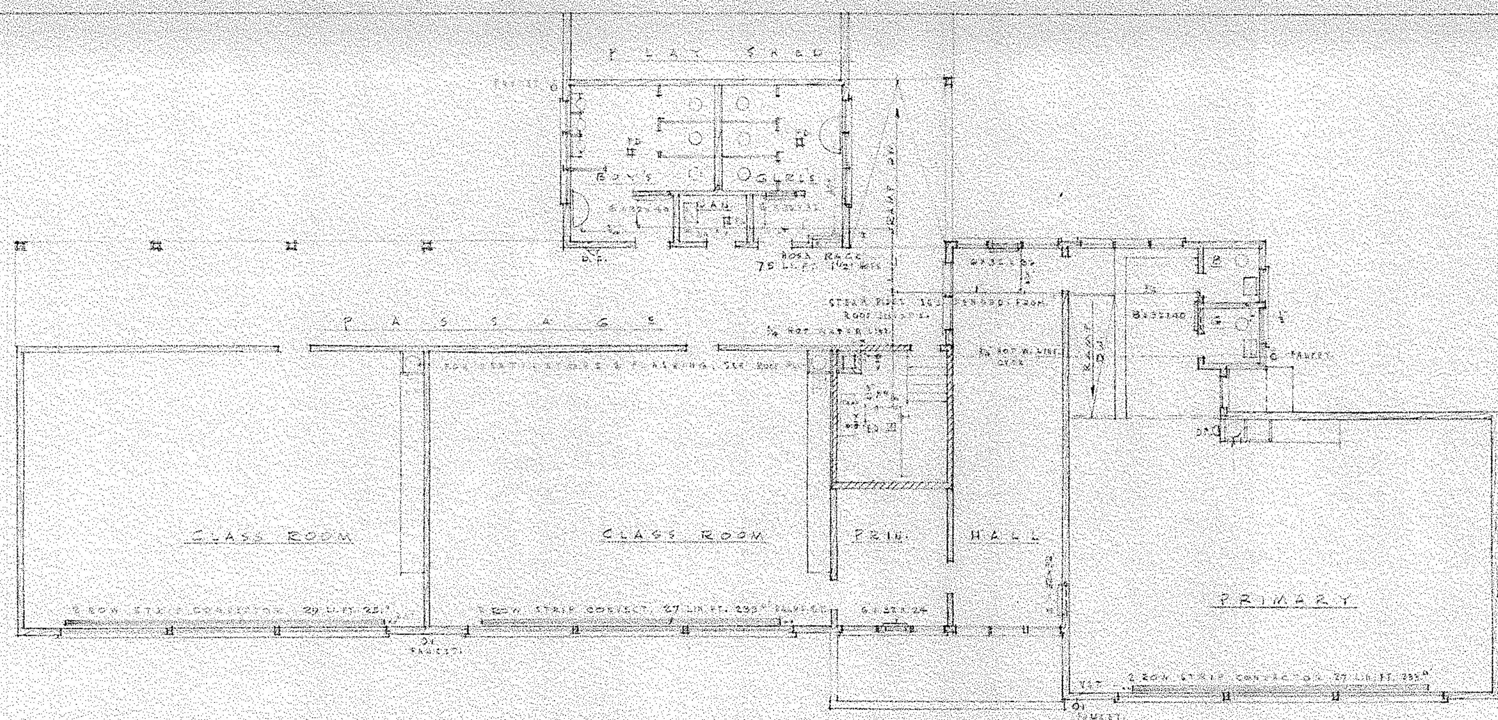
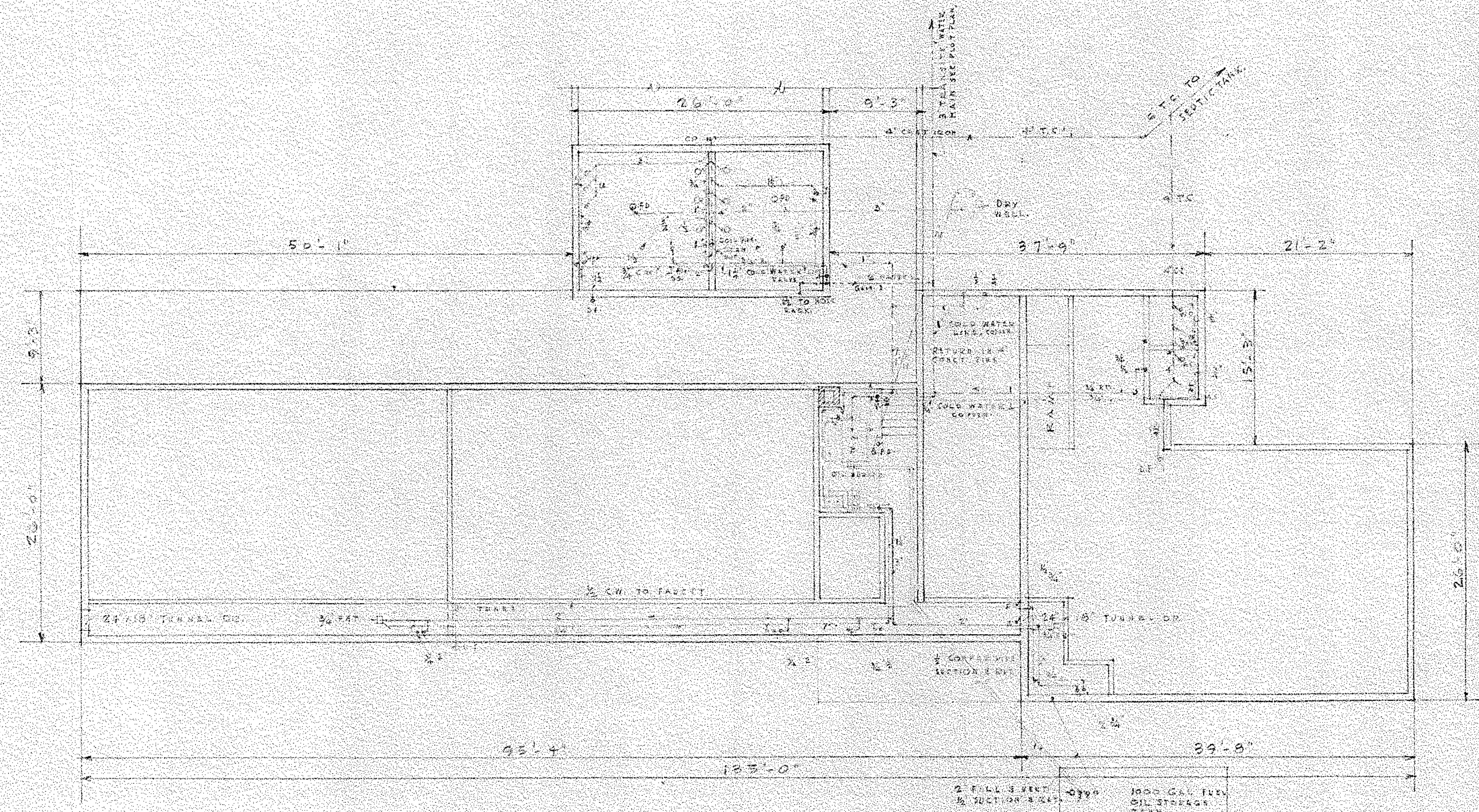
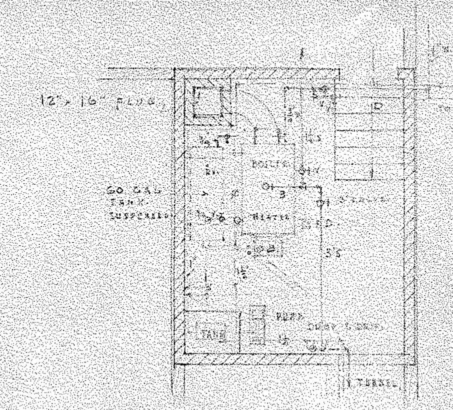


FLOOR PLAN



FOUNDATION PLAN



PLAN OF BOILER RM.
1/4" = 1'-0"

CONVECTOR SCHEDULE								
ROOM	NO. CON.	TYPE	WIDTH	HGT.	LENGTH	VALVE	TRAP	E.D.R.
CL. RM. 1	1	2 ROWS STRIP CON.	–	–	29'-0"	2"	3/4"	251
CL. RM. 2	1	2 " " "	–	–	27'-0"	2"	3/4"	233
PRIMARY	1	2 " " "	–	–	27'-0"	2"	3/4"	233
" SMALL RM.	1	CABINET CONVECTOR	8	32	40"	3/4"	1/2"	60
HALL	2	" "	6	32	32"	3/4"	1/2"	78
PRINCIPAL	1	" "	6	32	24"	3/4"	1/2"	28.8
GIRLS TOILET	1	" "	6	32	32"	3/4"	1/2"	39
BOYS "	1	" "	6	32	40"	3/4"	1/2"	48.8
			TOTAL RADIATION					971.6

PLUMBING AND
HEATING PLAN 1/8" = 1'-0"

7.14.48

ELEMENTARY SCHOOL
PINE GROVE SCHOOL DISTRICT #5
TILLAMOOK COUNTY, MANZANITA, ORE.

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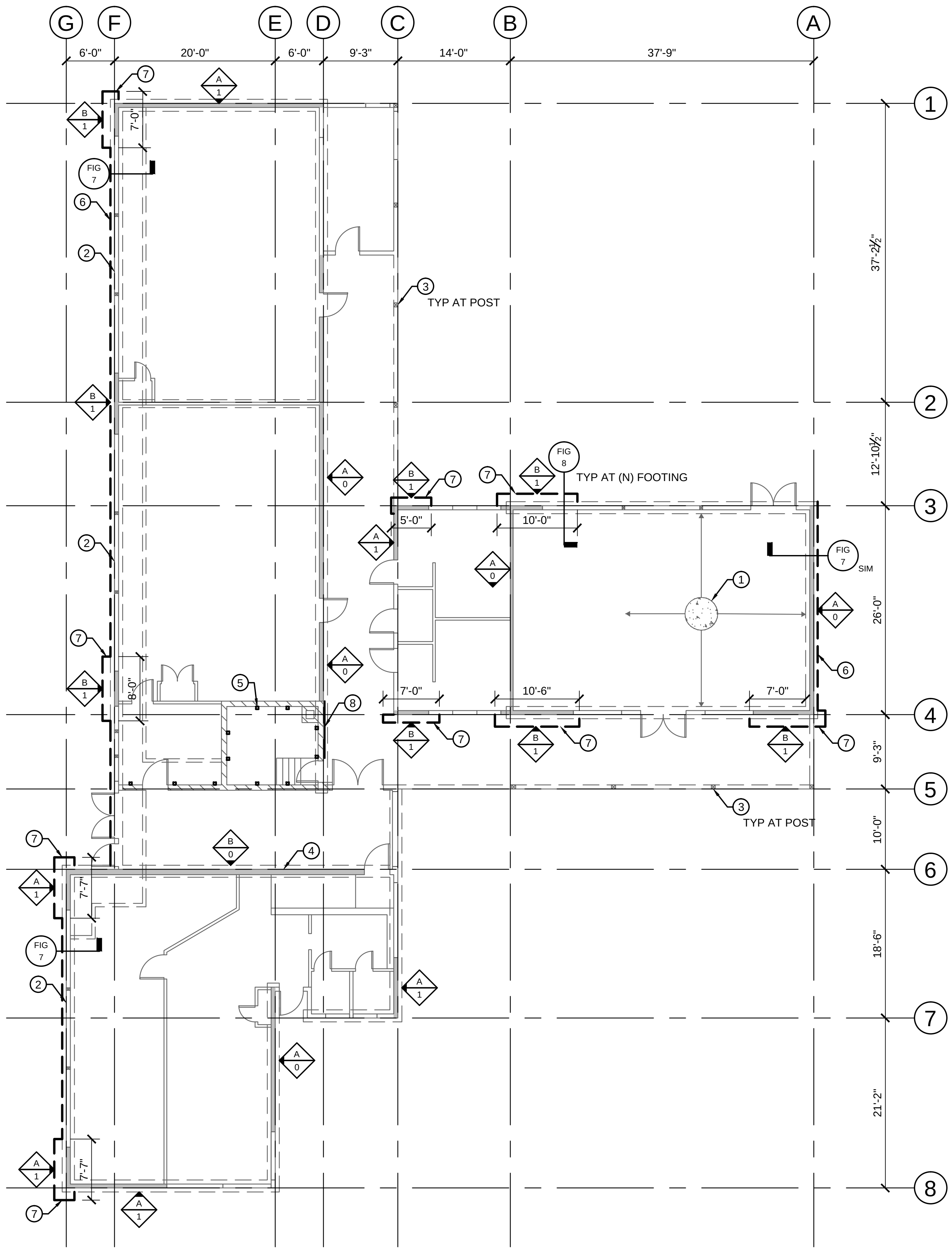
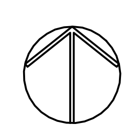
Appendix B

CONCEPTUAL STRENGTHENING SCHEME



1 FOUNDATION PLAN
SCALE: 1/16"=1'-0"



B1

PLAN NOTES:

1. EXISTING CONDITIONS ARE SHOWN WITHOUT GUARANTEE OF ACCURACY. VERIFY IN FIELD.
2. SEE FIGURE 4 FOR SHEAR WALL AND HOLD DOWN SCHEDULE.
3. REMOVE ALL (E) WOOD SIDING/SHINGLES AND REPLACE WITH (N) HARDIPLANK LAP SIDING. REMOVE SHIPLAP SHEATHING AT SHEAR WALLS. REMOVE (E) METAL SIDING AND REPLACE WITH (N) METAL SIDING IN KIND.

SHEET KEYED NOTES:

1. (E) CONC. SOG, TYPICAL ALL BUILDINGS.
2. REMOVE (E) SHIPLAP SHEATHING AND WALL FRAMING ALONG GRIDLINE F AND G. REBUILD WALL W/ 2x6 AT 16"OC STUDS AND $^{15}/_{32}$" PLYWOOD SHEATHING.
3. INSTALL (2) SIMPSON RPBZ W/ $^{3}/_{8}$" DIA EXPANSION AB.
4. DEMO (E) MASONRY WALL AND REBUILD AS 2x8 WOOD STUD WALL.
5. INSTALL HSS 4x4x$^{3}/_{16}$ STRONGBACK W/ $^{5}/_{8}$" DIA EPOXY ANCHOR AT 36" OC AND 6" MINIMUM FROM TOP AND BOTTOM OF STRONGBACK. SPACE HSS AT 5'-0" OC MAX.
6. REPAIR DETERIORATED FOOTING ALONG GRIDLINES A, F AND G. SEE FIGURE 5.
7. NEW FOOTING EXTENSION UNDER SHEAR WALL. SEE FIGURE 6.
8. INDICATES TYFO BCC FRP SYSTEM OVER CRACKED CMU WALL.

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CITY OF MANZANITA
ELEMENTARY SCHOOL STRUCTURE
STRUCTURAL EVALUATION

DATE: 10/15/2018
JOB NUMBER: 180111.00
PAGE REFERENCE:

SHEET NO.
FIGURE 1

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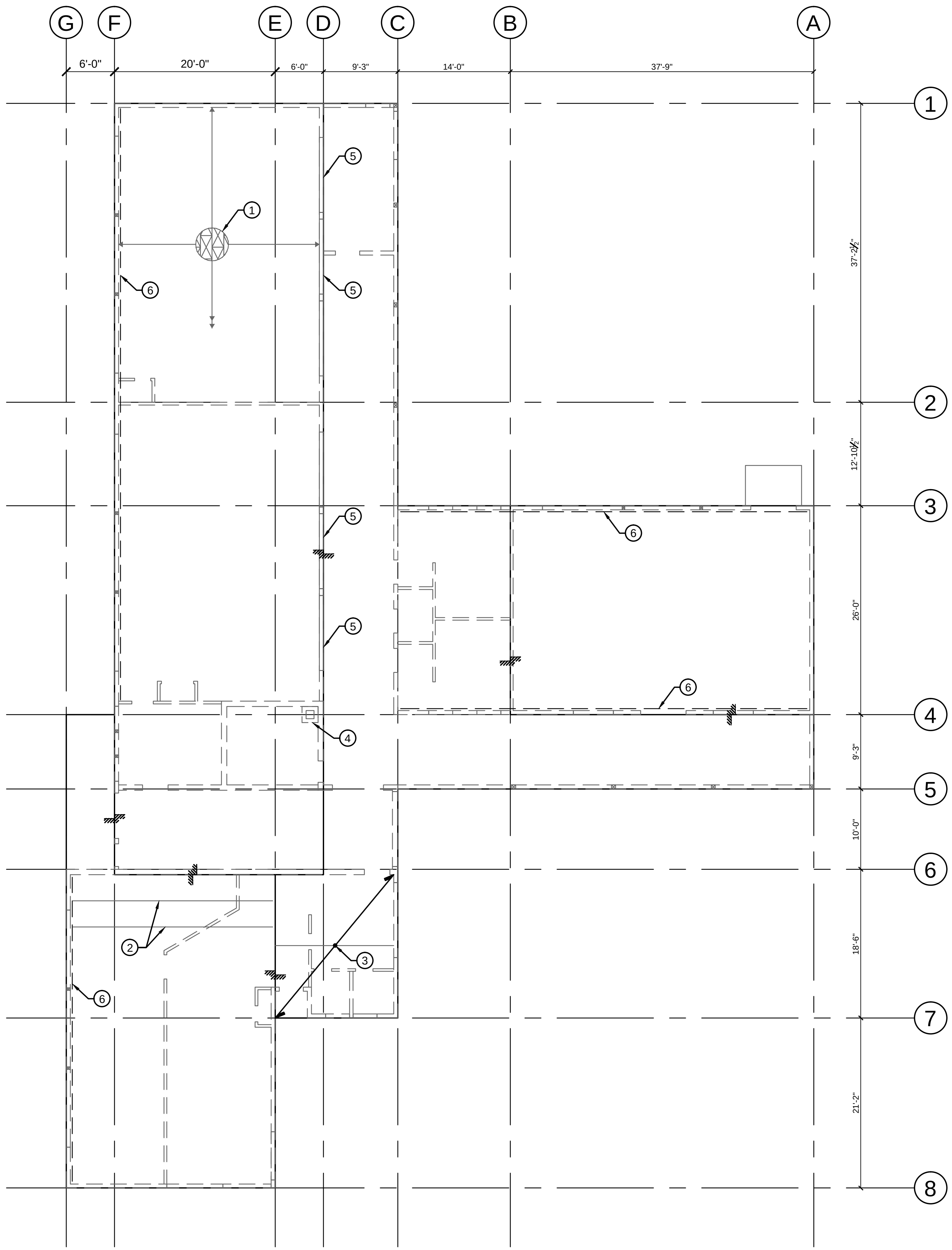

1 ROOF PLAN
SCALE: 1/16"=1'-0"

NORTH

B2

PLAN NOTES:

1. EXISTING CONDITIONS ARE SHOWN WITHOUT GUARANTEE OF ACCURACY. VERIFY IN FIELD.

SHEET KEYED NOTES:

1. INSTALL $^{15}/_{32}$" PLYWOOD SHEATHING W/ 10d AT 4" OC AT EDGES AND 12" OC IN THE FIELD OVER (E) 1x STRAIGHT SHEATHING. INSTALL (N) 4" RIGID INSULATION AND MEMBRANE ROOF SYSTEM OVER PLY SHTHG. TYPICAL ALL ROOFS.
2. SISTER DF-L #2 2x12 ALONG SIDE (E) 4x12
3. SISTER DF-L #2 2x8 ALONGSIDE (E) 2x8 FRAMING
4. DEMO (E) MASONRY CHIMNEY TO BELOW PLYWOOD SHTHG. SHEATH OVER TOP
5. INFILL WINDOWS ABOVE COVERED WALKWAY W/ 2x STUDS AT 16" OC TO MATCH WALL WIDTH. INSTALL PLYWOOD SHTHG PER SHEAR WALL TYPE A. SEE FIGURE 4.
6. INSTALL (2) SIMPSON CMST12

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ELEMENTARY SCHOOL STRUCTURE
STRUCTURAL EVALUATION

DATE: 10/15/2018
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FIGURE 2

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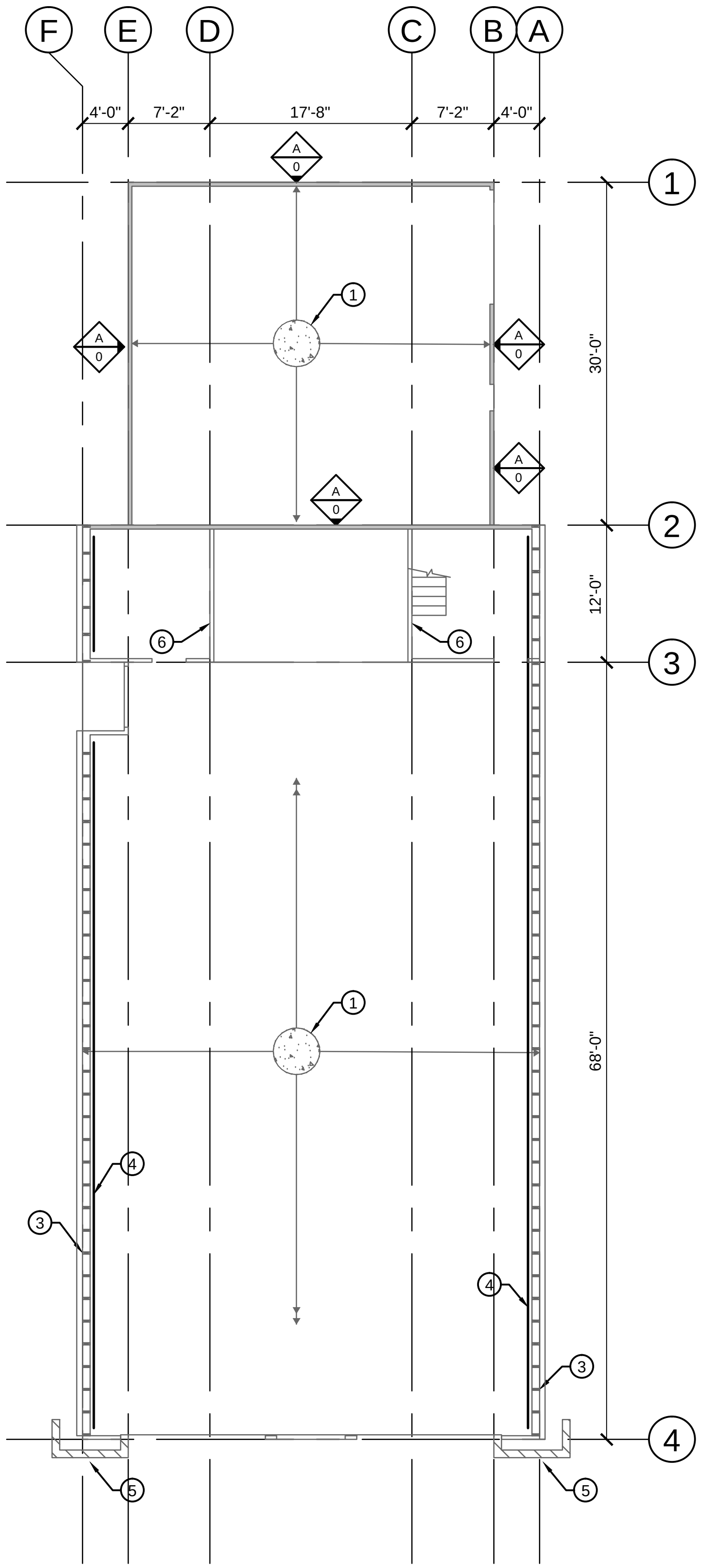
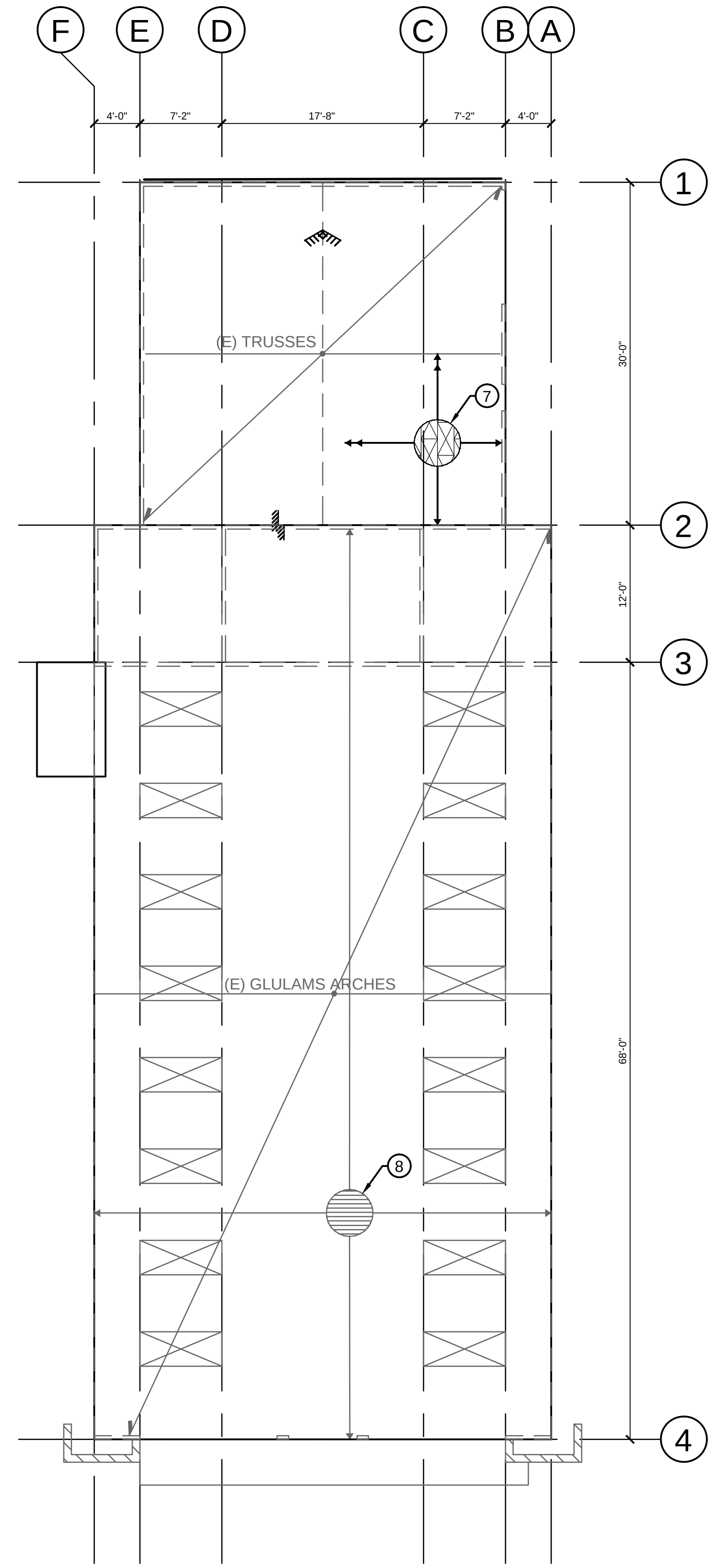


1 FOUNDATION PLAN
SCALE: 1/16"=1'-0"



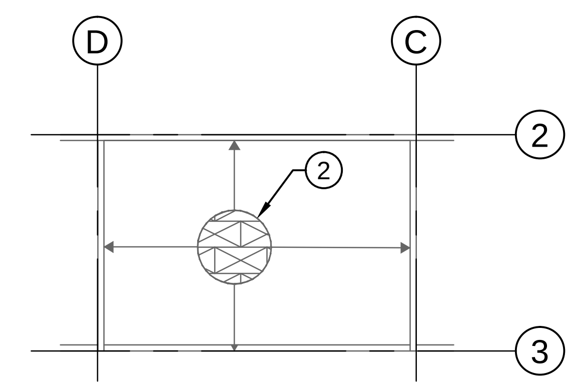
2 ROOF PLAN
SCALE: 1/16"=1'-0"

PLAN NOTES:

1. EXISTING CONDITIONS ARE SHOWN WITHOUT GUARANTEE OF ACCURACY. VERIFY IN FIELD
2. SEE FIGURE 4 FOR SHEARWALL AND HOLD DOWN SCHEDULE

SHEET KEYED NOTES:

1. (E) CONC SOG
2. (E) MEZZANINE ABOVE. INSTALL $\frac{15}{32}$" PLYWOOD SHTHG W/ 10d AT 6" OC AT PANEL EDGES AND 12" OC IN FIELD
3. GALVANIZED L6x4x$\frac{3}{8}$ EACH SIDE OF (E) GLULAM ARCH W/ $\frac{1}{2}$" DIAMETER THRU-BOLT AND $\frac{1}{2}$" DIAMETER AB W/ 3" MIN EMBEDMENT INTO FOOTING.
4. REPLACE SHTHG W/ $\frac{15}{32}$" PLYWOOD SHTHG 8'-0" UP ARCH
5. DEMO (E) CMU PIERS AND REBUILD AS 2x8 WOOD PIERS
6. STRENGTHEN EXISTING PLY SHEATHING NAILING TO 10d AT 6" OC AT PANEL EDGES AND STRENGTHEN SILL PLATE ANCHOR BOLTS TO $\frac{5}{8}$" DIAMETER EPOXY ANCHOR BOLTS AT 3'-0" OC.
7. REMOVE (E) METAL DECK AND (E) SHTHG. REPLACE W/ $\frac{15}{32}$" PLY SHTHG W/ 10d AT 4" OC AT EDGES AND 12" OC IN THE FIELD. BLOCK ALL PANEL EDGES. INSTALL (N) METAL DECK.
8. (E) METAL DECK



3 MESSANINE PLAN
SCALE: 3/32"=1'-0"

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QUONSET HUT
STRUCTURAL EVALUATION

DATE: 10/15/2018
JOB NUMBER: 180111.00
PAGE REFERENCE:

SHEET NO.
FIGURE 3

SHEAR WALL SCHEDULE										
MARK	SHEATHING	SIDE	PANEL NAILING			PANEL BLOCKING	SILL PLATE	ADHESIVE ANCHOR BOLTS	SILL NAILING	VALUE (PLF)
			SIZE	EDGE	FIELD					
A	15/32" APA RATED SHEATHING WITH STUDS AT 16" OC	ONE	8d	6" OC	12" OC	2x	2x	5/8" x 6" EMBED AT 40" OC	16d AT 6" OC	520
B	15/32" APA RATED SHEATHING WITH STUDS AT 16" OC	ONE	8d	3" OC	12" OC	2x	2x	5/8" x 6" EMBED AT 32" OC	16d AT 3" OC	980

NOTES:

1. 8d NAIL = 2 ½" x 0.131" COMMON.
2. IF AB SPACING IS GREATER THAN SHEAR WALL LENGTH INSTALL (1) AB WITHIN 12" OF EACH END.
3. NAIL SIZES SHOWN ARE FOR COMMON NAILS OR GALVANIZED BOX. POWER DRIVEN NAILS SHALL COMPLY WITH ESR 1539 FOR RECOMMENDED SPACING AND INSTALLATION TO COMPLY WITH THE ABOVE SHEAR WALL SCHEDULE.
4. SILL PLATE ANCHORS SHALL INCLUDE A STEEL PLATE WASHER NOT LESS THAN 0.229"x3"x3" IN SIZE PER AF&PA SDPWS SECTION 4.3.6.4.3. THE HOLE IN THE PLATE WASHERS SHALL BE PERMITTED TO BE DIAGONALLY SLOTTED W/ A WIDTH OF UP TO 3/16" LARGER THAN THE BOLT DIAMETER AND A SLOT LENGTH NOT TO EXCEED 1 3/4", PROVIDED A STANDARD CUT WASHER IS PLACED BETWEEN THE PLATE WASHER AND THE NUT. THE PLATE WASHER SHALL EXTEND TO WITHIN 1/2" OF THE EDGE OF THE BOTTOM PLATE ON THE SIDE(S) WITH SHEATHING.
5. WHERE SHEAR DESIGN VALUES EXCEED 700 POUNDS PER LINEAR FOOT (350 PLF ASD), ALL FRAMING MEMBERS RECEIVING EDGE NAILING FROM ABUTTING PANELS SHALL NOT BE LESS THAN A SINGLE 3-INCH NOMINAL MEMBER, OR TWO 2-INCH NOMINAL MEMBERS LAMINATED TOGETHER WITH 16d NAILS AT 6" OC TO TRANSFER THE DESIGN SHEAR VALUES BETWEEN FRAMING MEMBERS. WOOD STRUCTURAL PANEL JOINT AND SILL PLATE NAILING SHALL BE STAGGERED IN ALL CASES. SEE 1/S7.1 FOR ADDITIONAL INFORMATION.
6. SHEAR WALL NAILING MUST BE INSTALLED SUCH THAT THE NAIL HEAD OR CROWN IS FLUSH WITH THE SURFACE OF SHEATHING. OVERDRIVEN OR OVER PENETRATED NAILS WILL NOT BE ALLOWED OR COUNTED AS APPROPRIATE NAILING.
7. ALL PANEL EDGES SHALL BE BLOCKED AS NOTED.

HOLDOWN SCHEDULE							
MARK	HOLDOWN	WOOD MEMBER	WOOD FASTENER	ANCHOR BOLT	ANCHOR BOLT EMBEDMENT (IN)	COMMENTS	VALUE (LBS)
0	NONE REQD						
1	(S) HDU2-SDS2.5	(1) 4x	(6) (S) SDS SCREWS	5/8" DIA EPOXY AB	8"	N/A	3,075

NOTES:

1. DOUBLE STUDS ARE REQUIRED AT HOLDOWNS UNLESS NOTED OTHERWISE. DOUBLE STUDS SHALL BE LAMINATED TOGETHER WITH 16d NAILS AT 6" OC.
2. PROVIDE HOLDOWN NOTED WITHIN 6" FROM EACH END OF EACH SHEAR WALL SHOWN ON PLANS.
3. INSTALL ANCHORS PER MANUFACTURER'S RECOMMENDATIONS.
4. ANCHOR BOLTS SHALL BE ASTM F1554 GRADE 36. ASTM A36 THREADED ROD MAY BE USED AT CONTRACTOR'S OPTION.

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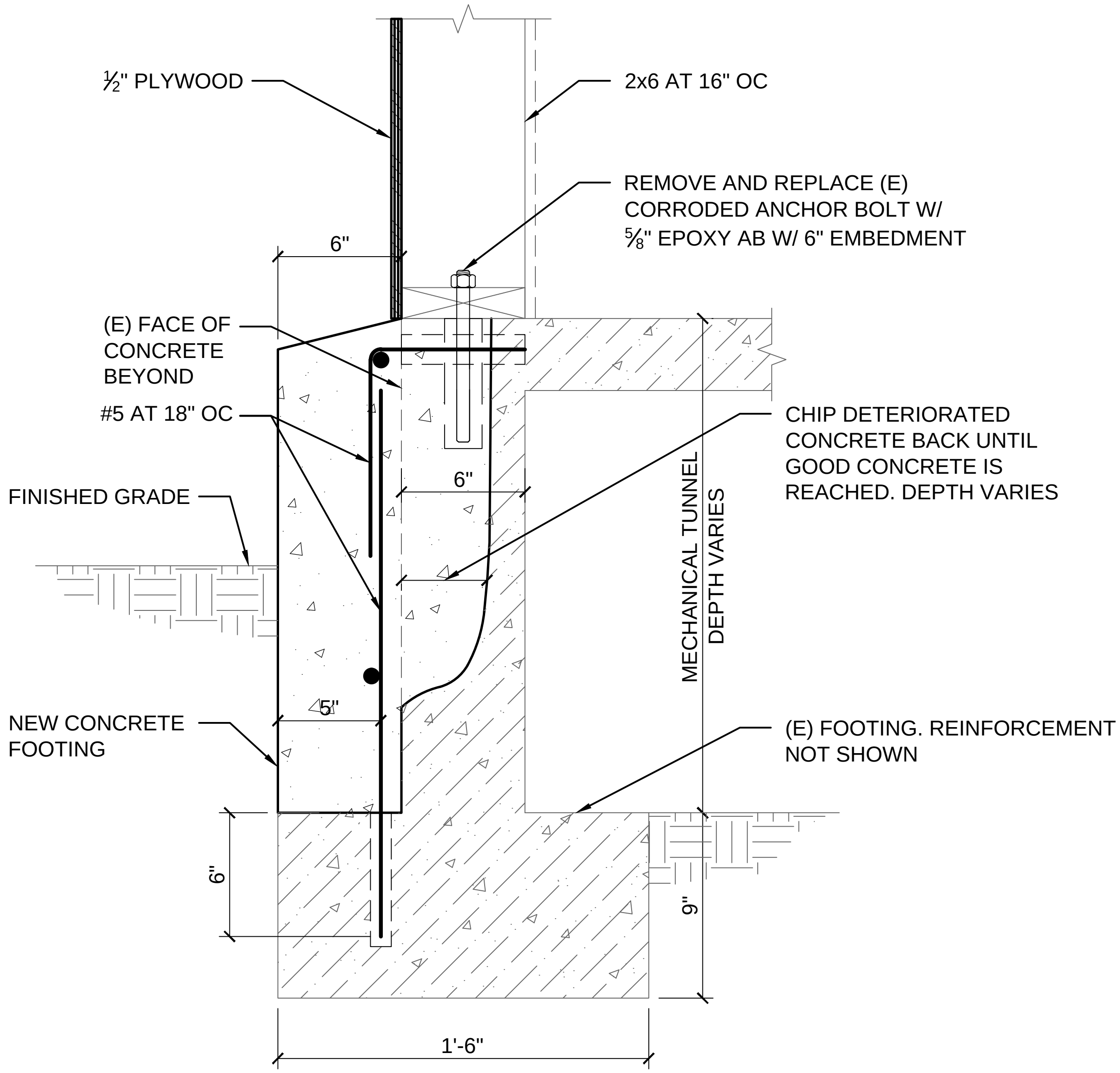
360.695.9731 www.wrkengrs.com

SHEAR WALL SCHEDULE

CITY OF MANZANITA
ELEMENTARY SCHOOL STRUCTURE
STRUCTURAL ELEVATION

SCALE: FULL SCALE
DATE: 10/15/2018
PROJECT NO: 18111.00

FIGURE 4



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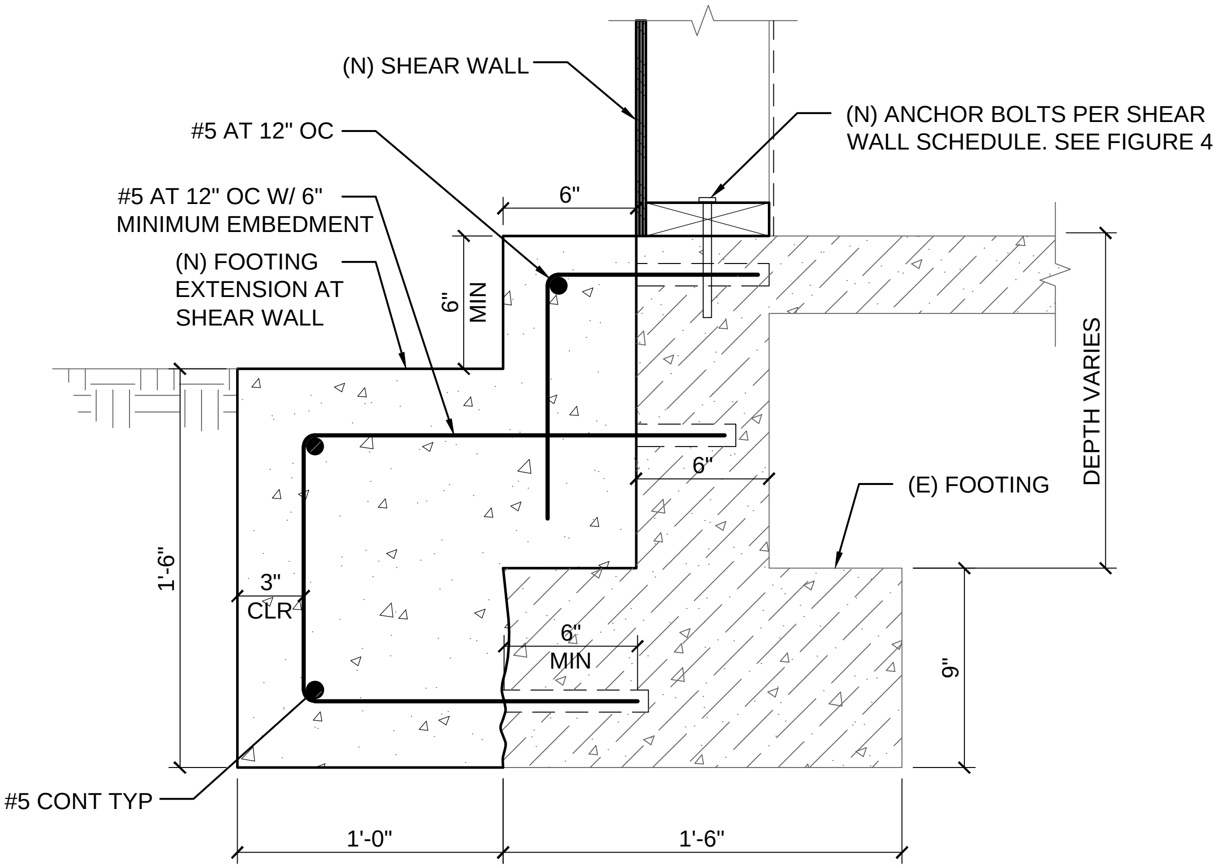
DETERIORATED FOOTING REPAIR

CITY OF MANZANITA
ELEMENTARY SCHOOL STRUCTURE
STRUCTURAL ELEVATION

SCALE: 1 1/2" = 1'-0"
DATE: 10/15/2018
PROJECT NO: 18111.00

FIGURE 5



wrk engineers 215 w. 12th street, suite 202, vancouver, washington 98660 360.695.9731 www.wrkengrs.com	NEW SHEAR WALL FOOTING EXTENSION CITY OF MANZANITA ELEMENTARY SCHOOL STRUCTURE STRUCTURAL ELEVATION	SCALE: 1 1/2" = 1'-0" DATE: 10/15/2018 PROJECT NO: 18111.00 FIGURE 6

Appendix C

STRUCTURAL CALCULATIONS

I. Elementary School Structure

Design Maps Summary Report

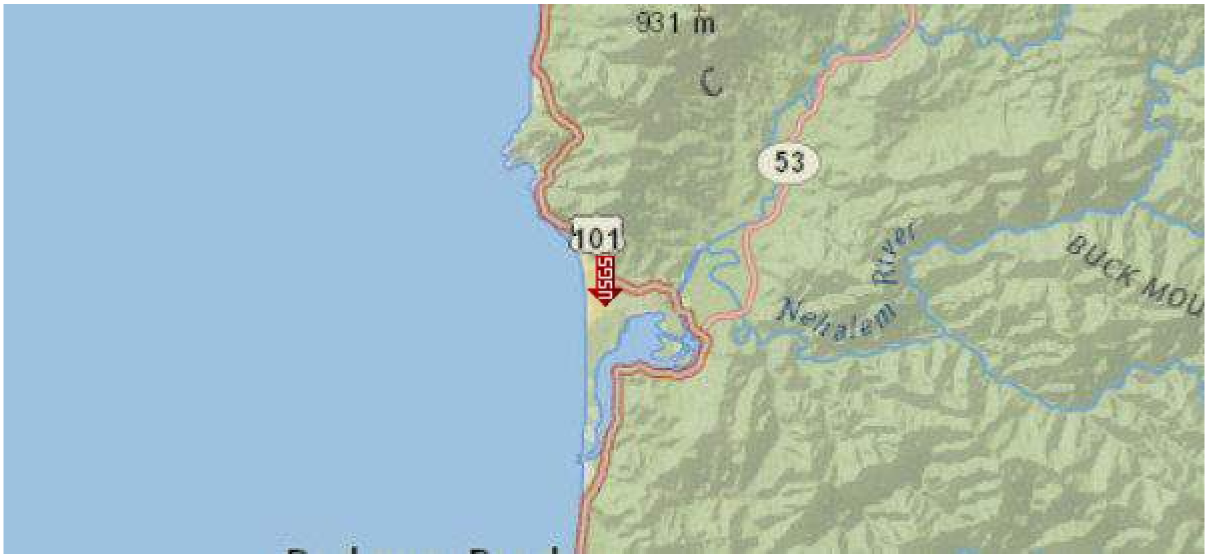
User–Specified Input

Report Title City of Manzanita School Evaluation
Fri September 28, 2018 15:20:08 UTC

Building Code Reference Document ASCE 41-13 Retrofit Standard, BSE-1N
(which utilizes USGS hazard data available in 2008)

Site Coordinates 45.7207°N, 123.93061°W

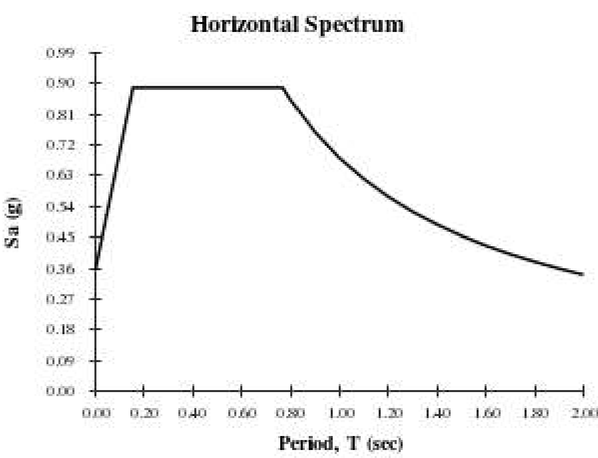
Site Soil Classification Site Class D – “Stiff Soil”

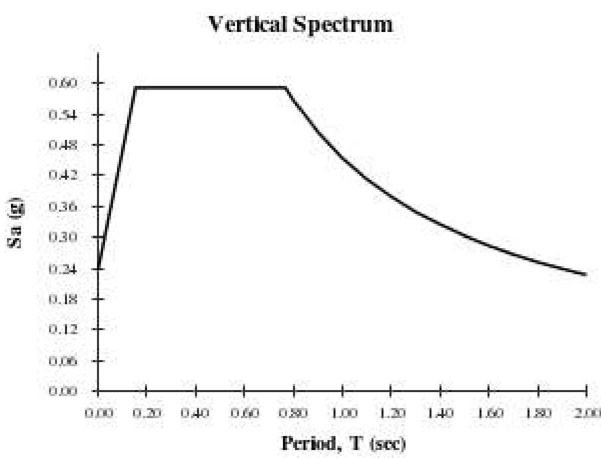


USGS–Provided Output

$S_{XS,BSE\text{-}1N}$ 0.888 g

$S_{X1,BSE\text{-}1N}$ 0.681 g





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Design Maps Summary Report

FOR TIER 1 SCREENING ONLY

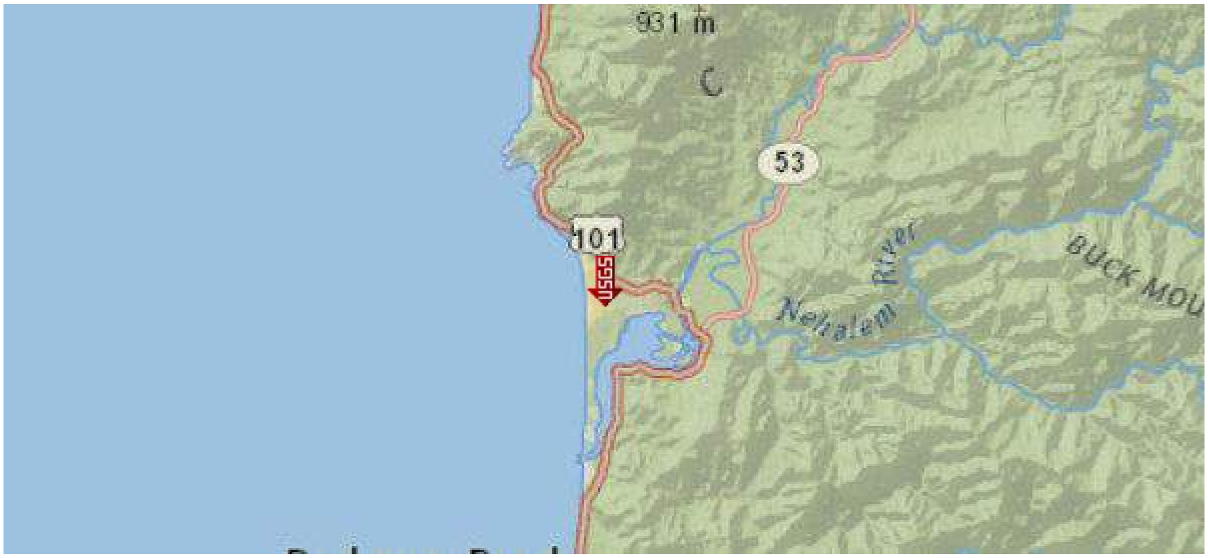
User–Specified Input

Report Title City of Manzanita School Evaluation
Fri September 28, 2018 15:21:37 UTC

Building Code Reference Document ASCE 41-13 Retrofit Standard, BSE-2N
(which utilizes USGS hazard data available in 2008)

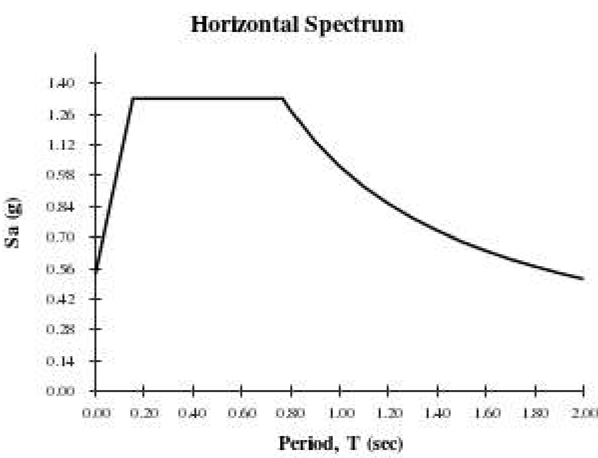
Site Coordinates 45.7207°N, 123.93061°W

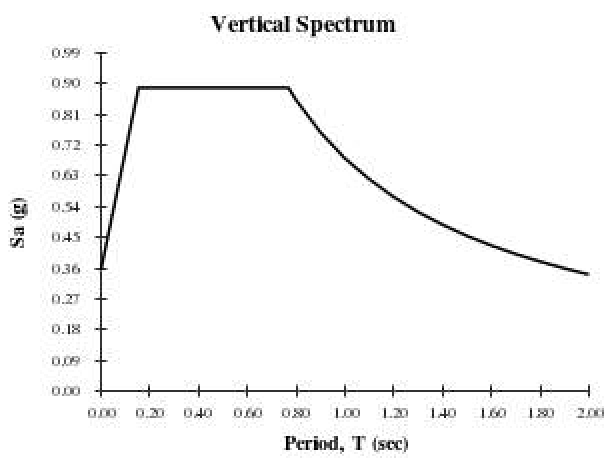
Site Soil Classification Site Class D – "Stiff Soil"



USGS–Provided Output

$S_{S,BSE\text{-}2N}$	1.332 g	$S_{XS,BSE\text{-}2N}$	1.332 g
$S_{1,BSE\text{-}2N}$	0.681 g	$S_{X1,BSE\text{-}2N}$	1.022 g





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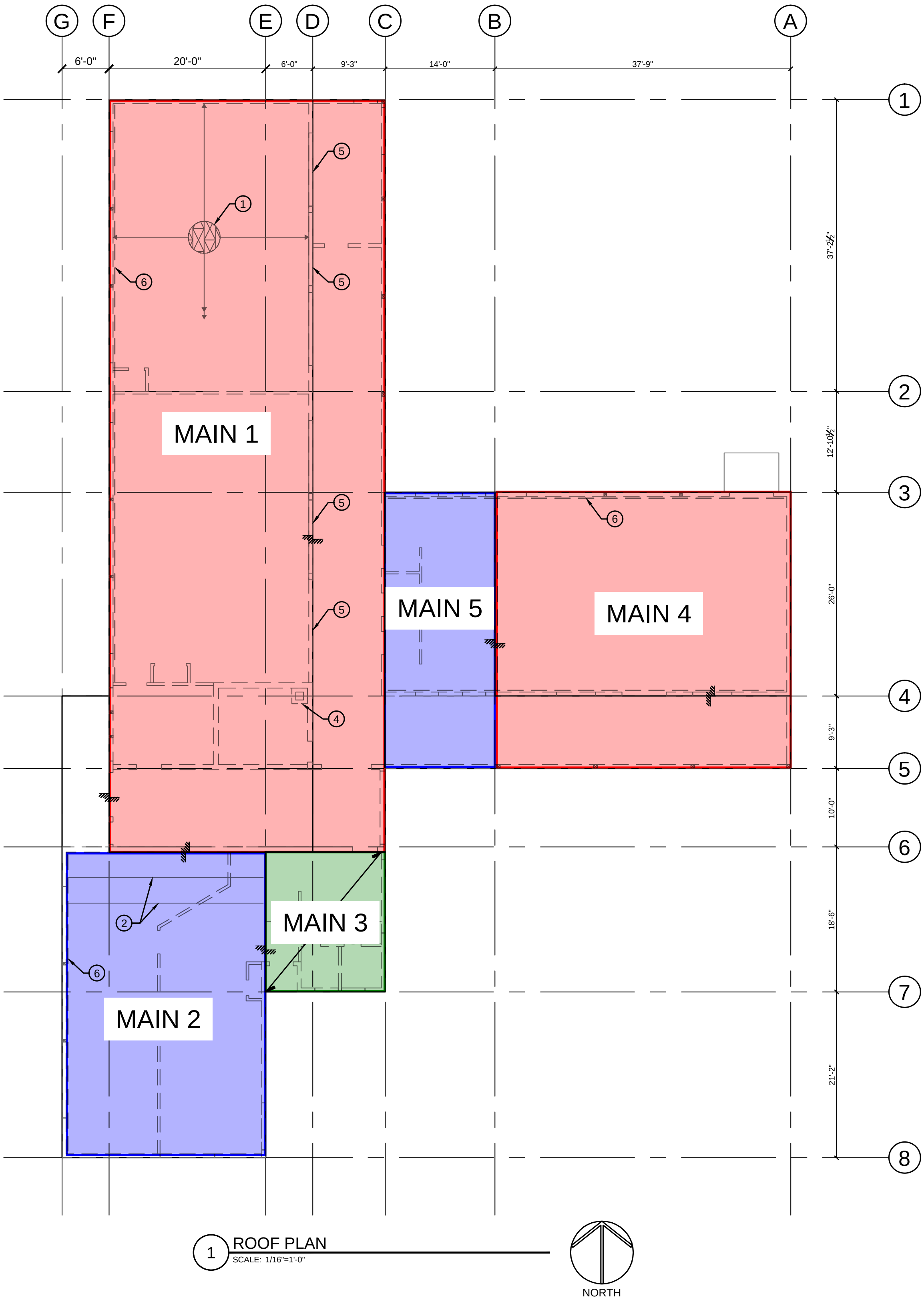


1 ROOF PLAN
SCALE: 1/16"=1'-0"

Main 1

Material	Qty	Tributary	Units	Weight	Units	Total Weight (kips)
Roof						
1x6 Shiplap Decking	1	2496	sf	3	psf	7.5
Insulation	1	2496	sf	1.0	psf	2.5
Framing	1	2496	sf	3.0	psf	7.5
Ceiling	1	2496	sf	2	psf	5.0
Acoustical Tile	1	2496	sf	1.0	psf	2.5
M & E	1	2496	lf	3.0	plf	7.5
Misc	1	2496	sf	2.0	psf	5.0
Canopy						
Canopy	1	984	sf	12.0	psf	11.8
Walls						
East/West Wall						
8" CMU Walls	1	130	sf	30.0	psf	3.9
Wood Walls	1	1252	sf	15.0	psf	18.8
North/South Wall						
8" CMU Walls	1	254	sf	30.0	psf	7.6
Wood Walls	1	629	sf	15.0	psf	9.4
				Total Load =		**89.0** kips

Base Shear - Main 1

Floor	Incr. Ht. ft.	Height ft	Elevation ft	Diaphragm Seis Weight kips	N-S Wall Seis Weight kips	E-W Wall Seis Weight kips	N-S Direction Diaphragm Seis Weight kips	E-W Direction Diaphragm Seis Weight kips
Roof	13.00	13.00	13.00	49	17	23	66	72
Summation:				49	17	23	66	72

Seismic Design Parameters (site Specific):

	BSE-1N
Sxs =	0.888
Sx1 =	0.681
Cm =	1
C_1 =	1.4
C_2 =	1

Approximate Code Period :

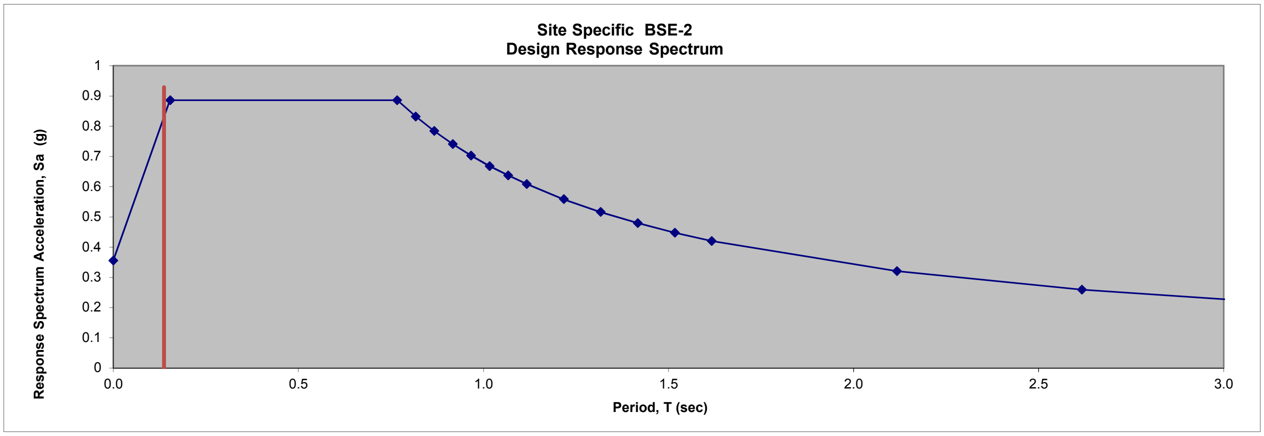
	BSE-1N	
B =	0.75	
Ct =	0.020	
T_L =	12.00	
To =	0.153	
Ts =	0.767	
Period Used to Estimate Base Shear; T =	**0.137**	seconds
k =	1.000	
b =	0.050	
B_1 =	1.002	
Response Spectrum Acceleration, Sa =	**0.829**	
psuedo lateral force, V = $C_1C_2C_mS_a$ W =	**1.161**	W

E-W Direction OUTPUT						
Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
89	1,157	1.000	103	1.16	72	83
89	1,157	1.000	103			

psuedo lateral force, V= **103** **kips**

N-S Direction OUTPUT						
Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
89	1,157	1.000	103	1.16	66	77
89	1,157	1.000	103			

psuedo lateral force, V= **103** **kips**



Project No.	18111.00	Date:	10/01/18
SUBJECT: Elementary School Building Load Takeoff			
Design:	KS	Section:	
PROJECT: City of Manzanita Structural Evaluation			
Checked:		Page:	of

Main 2

Material	Qty	Tributary	Units	Weight	Units	Total Weight (kips)
Roof						
1x6 Shiplap Decking	1	1040	sf	3	psf	3.1
Insulation	1	1040	sf	1.0	psf	1.0
Framing	1	1040	sf	3.0	psf	3.1
Ceiling	1	1040	sf	2	psf	2.1
Acoustical Tile	1	1040	sf	1.0	psf	1.0
M & E	1	1040	lf	3.0	plf	3.1
Misc	1	1040	sf	2.0	psf	2.1
Canopy						
Walls						
East/West Wall						
Wood Walls	1	654	sf	15.0	psf	9.8
North/South Wall						
Wood Walls	1	372.0	sf	15.0	psf	5.6
				Total Load =		**31.0** kips

Base Shear - Main 2

Floor	Incr. Ht. ft.	Height ft	Elevation ft	Diaphragm Seis Weight kips	N-S Wall Seis Weight kips	E-W Wall Seis Weight kips	N-S Direction Diaphragm Seis Weight kips	E-W Direction Diaphragm Seis Weight kips
Roof	13.00	13.00	13.00	16	6	10	21	25
Summation:				16	6	10	21	25

Seismic Design Parameters (site Specific):

	BSE-1N
Sxs =	0.888
Sx1 =	0.681
Cm =	1
C_1 =	1.4
C_2 =	1

Approximate Code Period :

	BSE-1N	
B =	0.75	
Ct =	0.020	
T_L =	12.00	
To =	0.153	
Ts =	0.767	
Period Used to Estimate Base Shear; T =	**0.137**	seconds
k =	1.000	
b =	0.050	
B_1 =	1.002	
Response Spectrum Acceleration, Sa =	**0.829**	
psuedo lateral force, V = $C_1C_2C_mS_a$ W =	**1.161**	W

E-W Direction OUTPUT

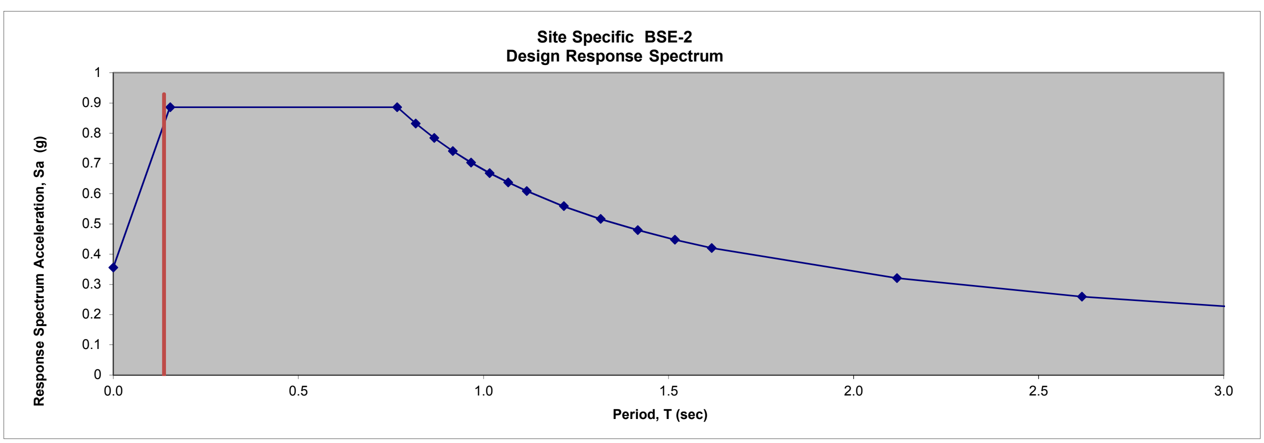
Total Weight kips	wihi^k kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
31	403	1.000	36	1.16	25	29
31	403	1.000	36			

psuedo lateral force, V= **36** **kips**

N-S Direction OUTPUT

Total Weight kips	wihi^k kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
31	403	1.000	36	1.16	21	25
31	403	1.000	36			

psuedo lateral force, V= **36** **kips**



wrk engineers

SUBJECT: Elementary School Building Load Takeoff	Project No. 18111.00	Date: 10/01/18
PROJECT: City of Manzanita Structural Evaluation	Design: KS	Section:
	Checked:	Page: of

Main 3

Material	Qty	Tributary	Units	Weight	Units	Total Weight (kips)
Roof						
1x6 Shiplap Decking	1	304	sf	3	psf	0.9
Insulation	1	304	sf	1.0	psf	0.3
Framing	1	304	sf	3.0	psf	0.9
Ceiling	1	304	sf	2	psf	0.6
Acoustical Tile	1	304	sf	1.0	psf	0.3
M & E	1	304	lf	3.0	plf	0.9
Misc	1	304	sf	2.0	psf	0.6
Canopy						
Walls						
East/West Wall						
Wood Walls	1	180	sf	15.0	psf	2.7
North/South Wall						
Wood Walls	1	130.0	sf	15.0	psf	2.0
				Total Load =		**9.2** kips

Base Shear - Main 3

Floor	Incr. Ht. ft.	Height ft	Elevation ft	Diaphragm Seis Weight kips	N-S Wall Seis Weight kips	E-W Wall Seis Weight kips	N-S Direction Diaphragm Seis Weight kips	E-W Direction Diaphragm Seis Weight kips
Roof	10.00	10.00	10.00	5	2	3	7	7
Summation:				5	2	3	7	7

Seismic Design Parameters (site Specific):

	BSE-1N
Sxs =	0.888
Sx1 =	0.681
Cm =	1
C_1 =	1.4
C_2 =	1

Approximate Code Period :

	BSE-1N	
B =	0.75	
Ct =	0.020	
T_L =	12.00	
To =	0.153	
Ts =	0.767	
Period Used to Estimate Base Shear; T =	**0.112**	seconds
k =	1.000	
b =	0.050	
B_1 =	1.002	
Response Spectrum Acceleration, Sa =	**0.744**	
psuedo lateral force, $V = C_1C_2C_mS_a$ W =	**1.042**	W

E-W Direction OUTPUT

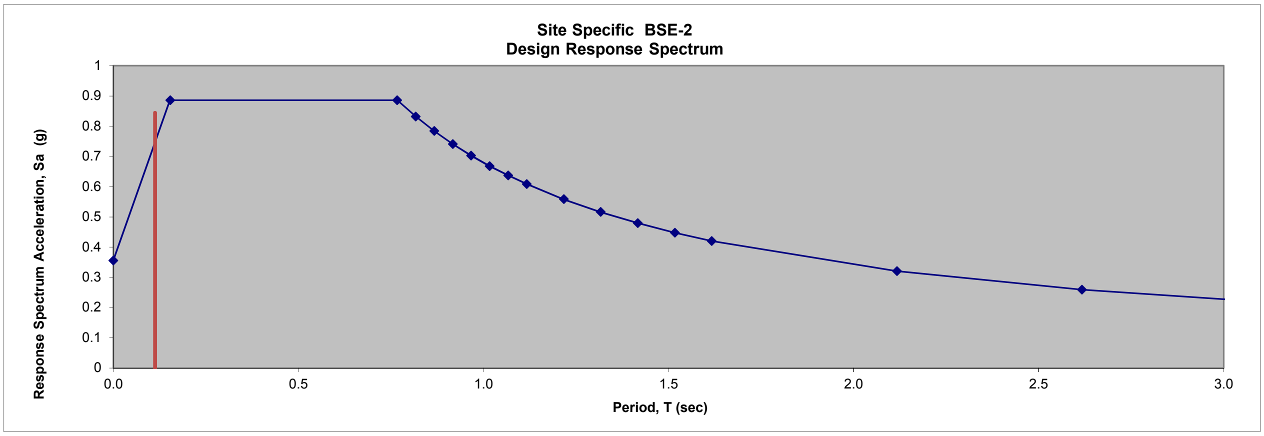
Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
9	92	1.000	10	1.04	7	8
9	92	1.000	10			

psuedo lateral force, V= **10** **kips**

N-S Direction OUTPUT

Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
9	92	1.000	10	1.04	7	7
9	92	1.000	10			

psuedo lateral force, V= **10** **kips**



Main 4

Material	Qty	Tributary	Units	Weight	Units	Total Weight (kips)
Roof						
1x6 Shiplap Decking	1	1368	sf	3	psf	4.1
Insulation	1	1368	sf	1.0	psf	1.4
Framing	1	1368	sf	3.0	psf	4.1
Ceiling	1	1368	sf	2	psf	2.7
Acoustical Tile	1	1368	sf	1.0	psf	1.4
M & E	1	1368	lf	3.0	plf	4.1
Misc	1	1368	sf	2.0	psf	2.7
Canopy						
Walls						
East/West Wall						
Wood Walls	1	221	sf	15.0	psf	3.3
North/South Wall						
Wood Walls	1	494.0	sf	15.0	psf	7.4
				Total Load =		**31.2** kips

wrk engineers	SUBJECT: Base Shear - Main 4	Project No. 18111.00	Date: 10/01/18
	PROJECT: City of Manzanita School Structural Evaluation	Design: KS	Section:
		Checked:	Page: of

Base Shear - Main 4

Floor	Incr. Ht. ft.	Height ft	Elevation ft	Diaphragm Seis Weight kips	N-S Wall Seis Weight kips	E-W Wall Seis Weight kips	N-S Direction Diaphragm Seis Weight kips	E-W Direction Diaphragm Seis Weight kips
Roof	13.00	13.00	13.00	21	7	3	28	24
Summation:				21	7	3	28	24

Seismic Design Parameters (site Specific):

	BSE-1N
Sxs =	0.888
Sx1 =	0.681
Cm =	1
C_1 =	1.4
C_2 =	1

Approximate Code Period :

	BSE-1N	
B =	0.75	
Ct =	0.020	
T_L =	12.00	
To =	0.153	
Ts =	0.767	
Period Used to Estimate Base Shear; T =	**0.137**	seconds
k =	1.000	
b =	0.050	
B_1 =	1.002	
Response Spectrum Acceleration, Sa =	**0.829**	
psuedo lateral force, V = $C_1C_2C_mS_a$ W =	**1.161**	W

E-W Direction OUTPUT

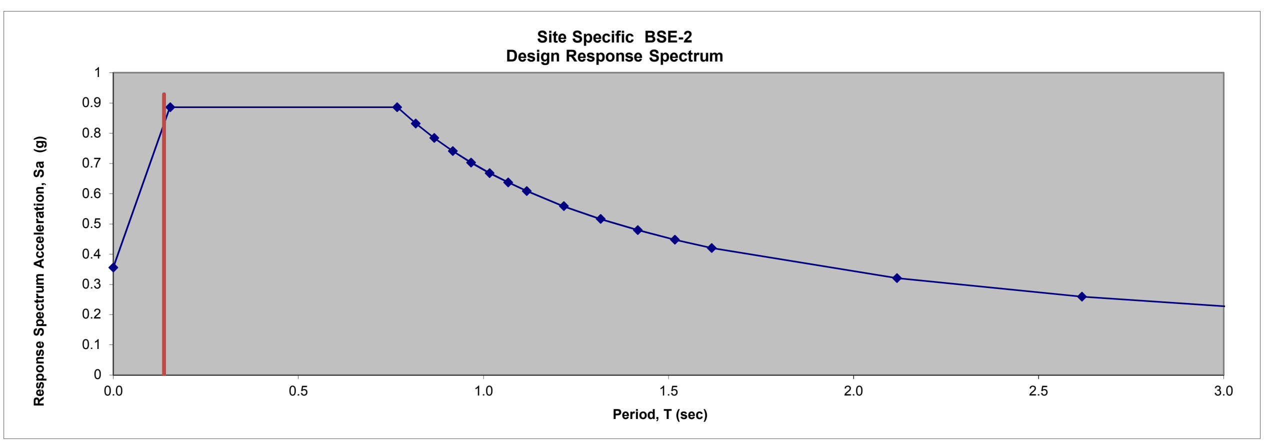
Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
31	406	1.000	36	1.16	24	28
31	406	1.000	36			

psuedo lateral force, V= **36** **kips**

N-S Direction OUTPUT

Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
31	406	1.000	36	1.16	28	32
31	406	1.000	36			

psuedo lateral force, V= **36** **kips**



Main 5

Material	Qty	Tributary	Units	Weight	Units	Total Weight (kips)
Roof						
1x6 Shiplap Decking	1	720	sf	3	psf	2.2
Insulation	1	720	sf	1.0	psf	0.7
Framing	1	720	sf	3.0	psf	2.2
Ceiling	1	720	sf	2	psf	1.4
Acoustical Tile	1	720	sf	1.0	psf	0.7
M & E	1	720	lf	3.0	plf	2.2
Misc	1	720	sf	2.0	psf	1.4
Canopy						
Walls						
East/West Wall						
Wood Walls	1	387	sf	15.0	psf	5.8
North/South Wall						
Wood Walls	1	245.0	sf	15.0	psf	3.7
				Total Load =		**20.3** kips

Base Shear - Main 5

Floor	Incr. Ht. ft.	Height ft	Elevation ft	Diaphragm Seis Weight kips	N-S Wall Seis Weight kips	E-W Wall Seis Weight kips	N-S Direction Diaphragm Seis Weight kips	E-W Direction Diaphragm Seis Weight kips
Roof	10.00	10.00	10.00	11	4	6	14	17
Summation:				11	4	6	14	17

Seismic Design Parameters (site Specific):

	BSE-1N
Sxs =	0.888
Sx1 =	0.681
Cm =	1
C_1 =	1.4
C_2 =	1

Approximate Code Period :

	BSE-1N	
B =	0.75	
Ct =	0.020	
T_L =	12.00	
To =	0.153	
Ts =	0.767	
Period Used to Estimate Base Shear; T =	**0.112**	seconds
k =	1.000	
b =	0.050	
B_1 =	1.002	
Response Spectrum Acceleration, Sa =	**0.744**	
psuedo lateral force, $V = C_1C_2C_mS_a$ W =	**1.042**	W

E-W Direction OUTPUT

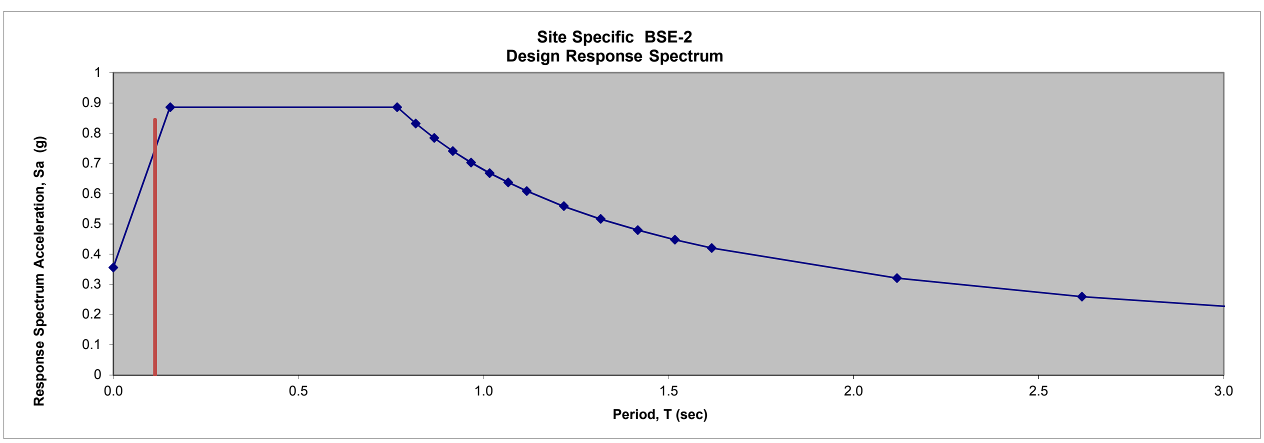
Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
20	203	1.000	21	1.04	17	17
20	203	1.000	21			

psuedo lateral force, V= **21** **kips**

N-S Direction OUTPUT

Total Weight kips	$w_ih_i^k$ kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
20	203	1.000	21	1.04	14	15
20	203	1.000	21			

psuedo lateral force, V= **21** **kips**



DIAPHRAGM DESIGN - Upper Roof

East/West Loading

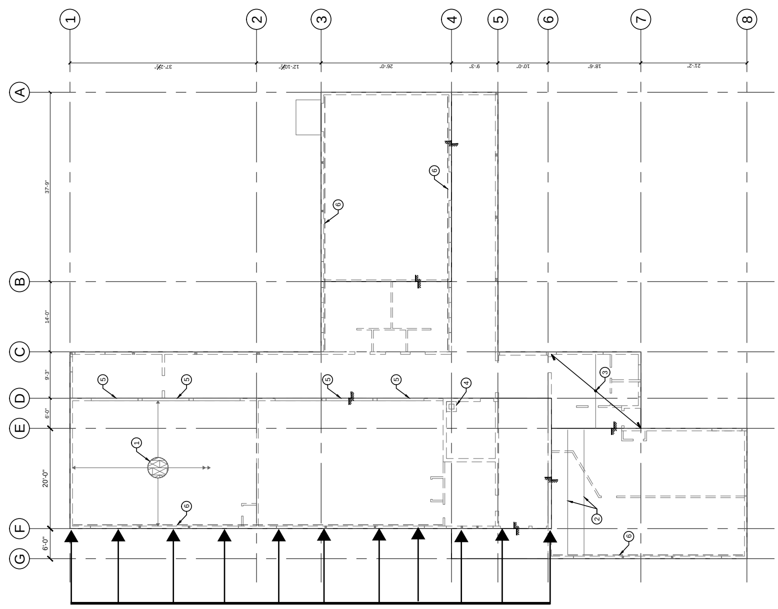
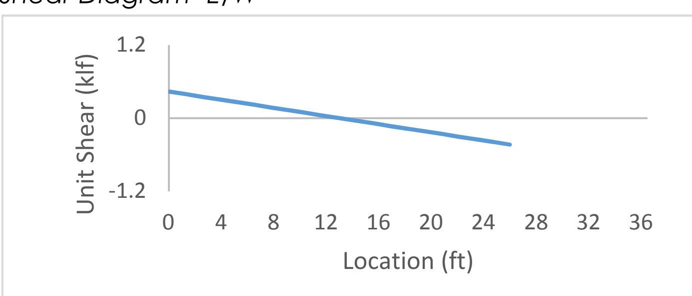


Diagram Loading

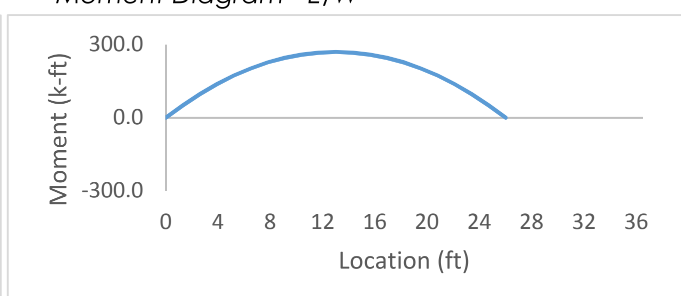
$$F_{px} = \frac{\Sigma F_i}{\Sigma w_i} w_{px}$$ *ASCE 41-17 Eqn. 7-26*

F_{px} =	83	kips	*See seismic loading for the Elementary School Structure*
L =	26	ft	*Diaphragm span*
B =	95.33	ft	*Diaphragm depth*
E =	3.19	klf	*Distributed Load [F_{px} / L]*

Shear Diagram -E/W



Moment Diagram - E/W



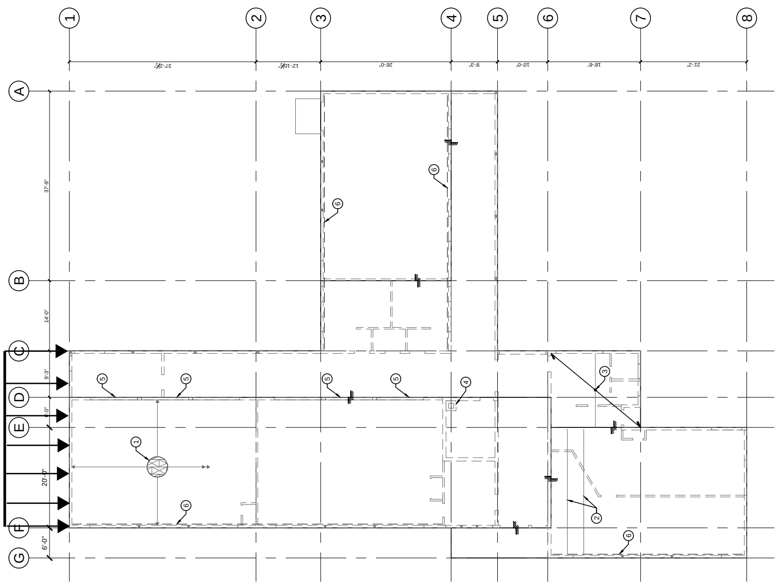
Diaphragm Shear

Q_{UD} =	0.44	klf	*Applied Unit Shear*
m =	2.5		*ASCE 41-17 Tbl. 12-3*
κ =	0.75		*ASCE 41-17 Sect. 6.2.4*
Q_{CE} =	1.155	klf	*Expected Strength*
$m\kappa Q_{CE}$ =	2.16563	klf	*Design Strength*
DCR =	0.20	< 1.0, OK	

NORTH/SOUTH LOADING CONTROLS
USE 1/2" SHTHG WITH 10d AT 4" OC
AT EDGES AND 12" OC IN THE FIELD

DIAPHRAGM DESIGN - Upper Roof

North/South Loading

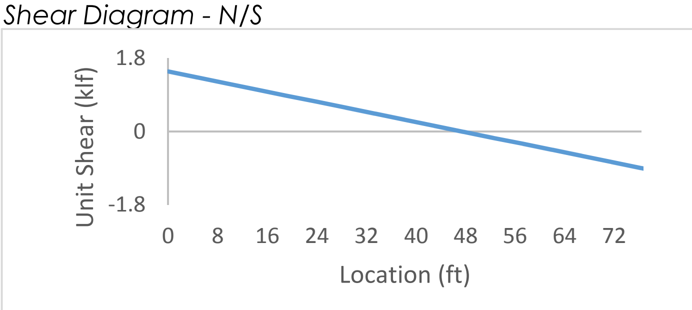


Diagrammg Loading

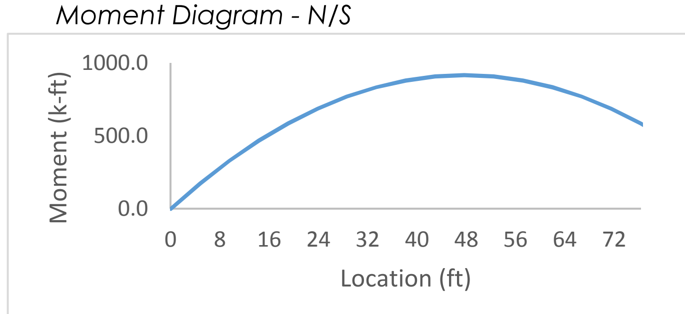
$$F_{px} = \frac{\Sigma F_i}{\Sigma w_i} w_{px}$$ *ASCE 41-17 Eqn. 7-26*

F_{px} =	77	kips	*See seismic loading for the Elementary School Structure*
L =	95.33	ft	*Diaphragm span*
B =	26	ft	*Diaphragm depth*
E =	0.81	klf	*Distributed Load [F_{px} / L]*

Shear Diagram - N/S



Moment Diagram - N/S



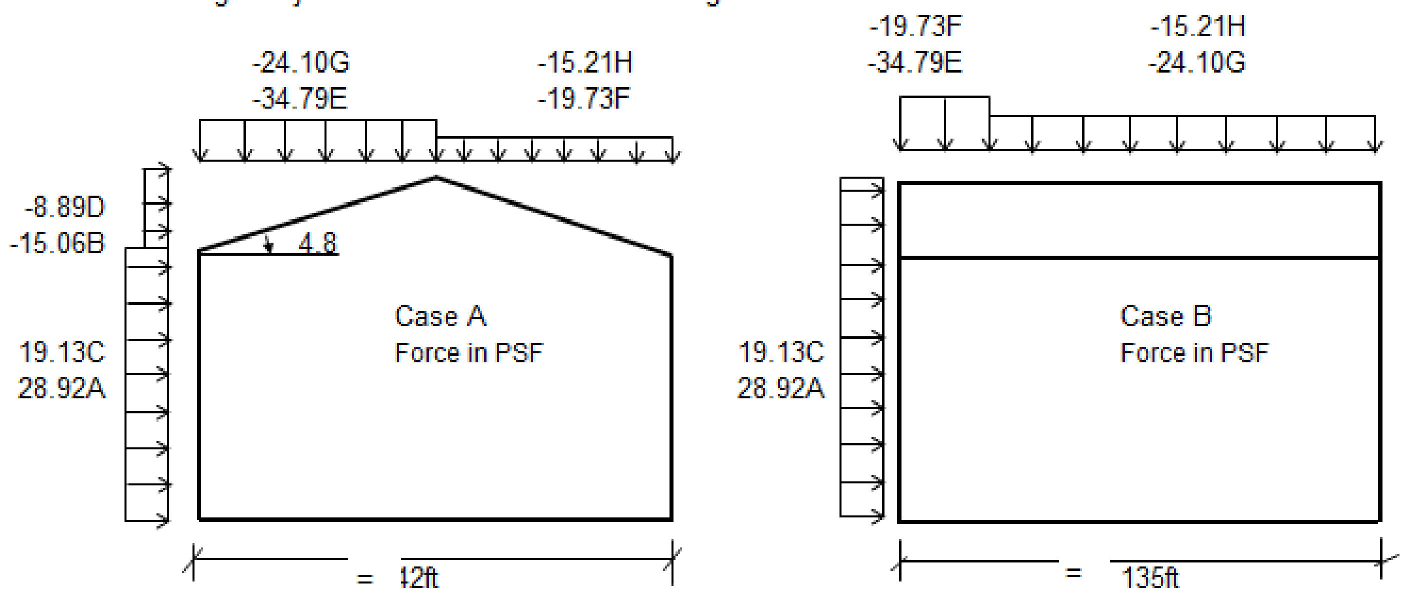
Diaphragm Shear

Q_{UD} =	1.48	klf	*Applied Unit Shear*
m =	2.5		*ASCE 41-17 Tbl. 12-3*
κ =	0.75		*ASCE 41-17 Sect. 6.2.4*
Q_{CE} =	1.155	klf	*Expected Strength*
$m\kappa Q_{CE}$ =	2.17	klf	*Design Strength*
DCR =	0.68	< 1.0, OK	

GOVERNING CASE
USE 1/2" SHTHG WITH 10d AT 4" OC
AT EDGES AND 12" OC IN THE FIELD

28.4 Envelope Procedure,
MWFRS For Low-Rise Building. Part 2: Enclosed Simple Diaphragm Building (≤ 60 ft)

Roof Height h =	12	feet	
Roof Pitch =	1.00 :12	= 4.76	Degree
Building & Structure Risk Category =	II, standard	IBC T-1604.5	
Wind Speed V =	135	MPH	
Topography factor K_{zt} =	1.00	26.8, Figure 26.8-1	
Exposure	B		
Height Adjustment factor λ =	1	Fig 28.6-1	



Plus and minus signs signify pressures acting toward and away from projected surfaces, respectively.
For Case B use ϴ = 0°
Total horizontal load shall not be less than that determined by assume p_s = 0 in zones B & D

a= 10% of least horizontal dimension or 0.4h, whichever smaller, but bot less than either 4% of least horizontal dimention or 3ft.

10 % of least dimension=	4.2 ft	⇐
40 % of the eave height =	4.8 ft	
4 % of least dimension or 3 ft=	3.0 ft	
therefore a =	4.2 ft	

SUBJECT: Elementary School Structure - Wind Loading	Project No. 18111.00	Date: 9/28/18
PROJECT: City of Manzanita Structural Evaluation	Design: KS	Section:
	Checked:	Page:

WIND BASE SHEAR

MAIN 1 DIAPHRAGM: $V = \omega HL$, ω = WIND PRESSURE (PSF)
H = HEIGHT (FT)
L = WALL LENGTH (FT)

E/W: (28.92 PSF)(96 FT)(14 FT)/1000 = 38.9 KIPS (CONTROLS)

N/S: (28.92 PSF)(26 FT)(14 FT)/1000 = 10.5 KIPS

$\Rightarrow V_{SEISMIC} = 103$ KIPS > 38.9 KIPS (SEISMIC CONTROLS)

MAIN 2 DIAPHRAGM:

E/W: (28.92 PSF)(40 FT)(13 FT)/1000 = 15 KIPS

$\Rightarrow V_{SEISMIC} = 36$ KIPS (SEISMIC CONTROLS)

$\Rightarrow$ SEISMIC CONTROLS IN ALL DIRECTIONS AT EACH DIAPHRAGM.

SUBJECT: Shear Wall Distribution		Project No. 18111.00	Date: 10/9/18
PROJECT: City of Manzanita Structural Evaluation		Design: KS	Section:
		Checked:	Page:

Shear Wall Force Distribution

Wall	Wall Length	Shear Force	Unit Shear	m-factor	k-factor	Design Shear	Height	Ar	SW Type	Nominal Capacity	Shear DCR
1	26.0 ft	51.5 k	1981 plf	3.8	0.75	463 plf	13.0 ft	0.5:1<3.5:1 OK	A	520	0.89
3-1	5.2 ft	17.3 k	3341 plf	3.8	0.75	782 plf	13.0 ft	2.51:1<3.5:1 OK	B	917	0.85
3-2	4.0 ft	3.3 k	827 plf	3.8	0.75	193 plf	10.0 ft	2.5:1<3.5:1 OK	B	919	0.21
4-1	6.0 ft	10.6 k	1769 plf	3.8	0.75	414 plf	13.0 ft	2.17:1<3.5:1 OK	B	960	0.43
4-2	7.0 ft	24.2 k	3458 plf	3.8	0.75	809 plf	13.0 ft	1.86:1<3.5:1 OK	B	980	0.83
4-3	4.0 ft	4.9 k	1220 plf	3.8	0.75	285 plf	10.0 ft	2.5:1<3.5:1 OK	B	919	0.31
6	37.0 ft	79.5 k	2149 plf	3.8	0.75	503 plf	13.0 ft	0.35:1<3.5:1 OK	B	980	0.51
8	12.5 ft	18.0 k	1440 plf	3.8	0.75	337 plf	12.0 ft	0.96:1<3.5:1 OK	A	520	0.65
A	26.0 ft	18.0 k	692 plf	3.8	0.75	162 plf	13.0 ft	0.5:1<3.5:1 OK	A	520	0.31
B	26.0 ft	25.9 k	995 plf	3.8	0.75	233 plf	13.0 ft	0.5:1<3.5:1 OK	A	520	0.45
C-1	6.8 ft	13.1 k	1922 plf	3.8	0.75	450 plf	10.0 ft	1.46:1<3.5:1 OK	A	520	0.86
C-2	5.0 ft	5.0 k	1000 plf	3.8	0.75	234 plf	10.0 ft	2:1<3.5:1 OK	A	520	0.45
D-1	35.0 ft	50.6 k	1447 plf	3.8	0.75	338 plf	13.0 ft	0.37:1<3.5:1 OK	A	520	0.65
D-2	9.8 ft	14.1 k	1447 plf	3.8	0.75	338 plf	13.0 ft	1.33:1<3.5:1 OK	A	520	0.65
E	17.3 ft	23.0 k	1327 plf	3.8	0.75	310 plf	12.0 ft	0.69:1<3.5:1 OK	A	520	0.60
F-1	4.1 ft	9.8 k	2393 plf	3.8	0.75	560 plf	13.0 ft	3.19:1<3.5:1 OK	B	835	0.67
F-2	7.6 ft	18.1 k	2393 plf	3.8	0.75	560 plf	13.0 ft	1.72:1<3.5:1 OK	B	980	0.57
F-3	4.3 ft	10.4 k	2393 plf	3.8	0.75	560 plf	13.0 ft	3:1<3.5:1 OK	B	857	0.65
G-1	5.1 ft	9.0 k	1772 plf	3.8	0.75	414 plf	12.0 ft	2.36:1<3.5:1 OK	A	496	0.83
G-2	5.1 ft	9.0 k	1772 plf	3.8	0.75	414 plf	12.0 ft	2.36:1<3.5:1 OK	A	496	0.83

Shear Wall Overturning

Wall	Roof Trib	DL	MOT	MR	M DCR	Holddown
1	1.6 ft	180 plf	670 k-ft	61 k-ft	1.09	YES
3-1	13.0 ft	351 plf	225 k-ft	5 k-ft	4.75	YES
3-2	0.7 ft	130 plf	33 k-ft	1 k-ft	3.15	YES
4-1	17.5 ft	419 plf	138 k-ft	8 k-ft	1.82	YES
4-2	17.5 ft	419 plf	315 k-ft	10 k-ft	3.04	YES
4-3	4.5 ft	188 plf	49 k-ft	2 k-ft	3.23	YES
6	6.0 ft	246 plf	1034 k-ft	168 k-ft	0.61	NO
8	1.6 ft	168 plf	216 k-ft	13 k-ft	1.63	YES
A	1.6 ft	180 plf	234 k-ft	61 k-ft	0.38	NO
B	8.8 ft	287 plf	336 k-ft	97 k-ft	0.34	NO
C-1	11.7 ft	295 plf	131 k-ft	7 k-ft	1.89	YES
C-2	7.6 ft	234 plf	50 k-ft	3 k-ft	1.69	YES
D-1	17.5 ft	419 plf	658 k-ft	256 k-ft	0.25	NO
D-2	17.5 ft	419 plf	183 k-ft	20 k-ft	0.91	NO
E	13.0 ft	339 plf	276 k-ft	51 k-ft	0.54	NO
F-1	13.0 ft	351 plf	127 k-ft	3 k-ft	4.31	YES
F-2	13.0 ft	351 plf	236 k-ft	10 k-ft	2.32	YES
F-3	13.0 ft	351 plf	135 k-ft	3 k-ft	4.06	YES
G-1	13.0 ft	339 plf	108 k-ft	4 k-ft	2.45	YES
G-2	13.0 ft	339 plf	108 k-ft	4 k-ft	2.45	YES

Footing Overturning - Increased Footing Size

Wall	Ftg Width	Ftg DL	Ftg Length	MOT Ftg	Mr Ftg	DCR Ftg
1	1.5 ft	244 plf	26.0 ft	5 k-ft	82 k-ft	0.06
3-1	2.5 ft	356 plf	10.0 ft	16 k-ft	18 k-ft	0.89
3-2	1.5 ft	244 plf	5.0 ft	2 k-ft	3 k-ft	0.66
4-1	2.5 ft	356 plf	7.0 ft	6 k-ft	9 k-ft	0.63
4-2	2.5 ft	356 plf	10.5 ft	19 k-ft	20 k-ft	0.96
4-3	1.5 ft	244 plf	7.0 ft	3 k-ft	6 k-ft	0.50
8	1.5 ft	244 plf	26.0 ft	7 k-ft	82 k-ft	0.09
C-1	1.5 ft	244 plf	6.8 ft	6 k-ft	6 k-ft	0.97
C-2	1.5 ft	244 plf	9.0 ft	2 k-ft	10 k-ft	0.19
F-1	2.5 ft	356 plf	7.0 ft	9 k-ft	9 k-ft	1.00
F-2	1.5 ft	244 plf	10.0 ft	12 k-ft	12 k-ft	0.98
F-3	2.5 ft	356 plf	8.0 ft	9 k-ft	11 k-ft	0.80
G-1	2.5 ft	356 plf	6.0 ft	6 k-ft	6 k-ft	0.89
G-2	2.5 ft	356 plf	6.0 ft	6 k-ft	6 k-ft	0.89

ROOF DIAPHRAGM CHORD FORCES

E/W DIRECTION:

$$83^K/26\ FT = 3.19\ ^K/_{FT} \rightarrow T/C = \frac{wl^2}{8d} = \frac{(3.19)(26)^2}{8(96)} = 2808^\# \Rightarrow \frac{2808^\#}{(2.5)(0.75)} = 1498^\#$$

N/S DIRECTION:

$$77^K/96 = 0.802\ ^K/_{FT} \rightarrow T/C = \frac{wl^2}{8d} = \frac{(0.802)(96)^2}{8(26)} = 35{,}535^\# \Rightarrow \frac{35{,}535^\#}{(2.5)(0.75)} = 19{,}952^\#$$

$\Rightarrow$ $f_{c,EW} = f_{t,EW} = 1498^\#/(2)(1.5)(5.5) = 90.8$ PSI
$f_{c,NS} = f_{t,NS} = 19{,}952^\#/(2)(1.5)(5.5) = 1209$ PSI } DOUBLE 2x6 TOP PLATE

F_c' (DF-L NO 1) = (1.6)(1500 psi) = 2400 psi > 1209 ✓OK

F_t' = (DF-L No 1) = (1.6)(675 psi) = 1080 psi < 1209 ⇒ X NO GOOD.

⇒ TRIPLE 2x6 PER DETAILS ⇒ $19{,}952^\#/(3)(8.25\ in^2)$ = 806 psi ⇒ ✓OK

NAILING:

$19{,}952^\#/26'$ = 767 PLF

1/2" SHTG AT 4" o.c NAILING = 770 PLF ✓OK

COLLECTOR:

LINE 5N: (14 FT)(770 PLF) = 10,780$^\#$ ⇒ 10.8^K

WORST SHEAR INTO WALL = 14^K ⇒ COLLECTOR REQUIRED

$F_p = 0.4 S_{xs} X W_p$

$= 0.4(0.888)(1.3)(30 \text{ psf}) = 13.8 \text{ psf}$ (CONTROLS)

MIN $F_p = 0.1(1.3)(30 \text{ PSF}) = 3.9 \text{ PSF}$

STRONGBACK

MAX HEIGHT: 14 FT

MAX SPAN WALL: 5 FT

$F_p = (13.8 \text{ PSF})(5 \text{ FT}) = 69 \text{ PLF}$

$M_u = \dfrac{(69 \text{ PLF})(14 \text{ FT})^2}{8} = 1691 \text{ LB-FT}$

$\Rightarrow$ USE HSS 4x4x 3/16

$M_u/\Omega = 8.42 \text{ K-FT} \Rightarrow M_n = 8.42 \text{ K-FT} * 1.67$

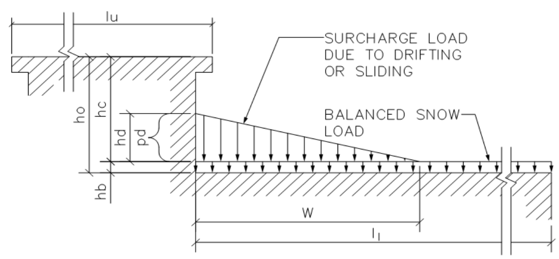
$M_n = 14.1 \text{ K-FT} = 14{,}100 \text{ LB-FT} > 1691 \text{ LB-FT}$ ✓OK

2012 IBC Snow Loading for Low Roofs and Decks

Design per 2012 International Building Code - ASCE 7-10 Sections 7.7-7.8, 7.9

Input Data:

Risk Category =	II		(Table 1.5-1)
Ground Snow Load, p_g =	25.0 psf		(Figure 7-1, Table 7-1 for AK)
Roof Exposure =	Fully Exposed		(Table 7-2)
Terrain Category =	B		(Sec. 26.7)
Thermal Factor, C_t =	1.0		(Table 7-3)
Roof Slope Factor, C_s =	1.0		(Sec. 7.4.1-7.4.4 and Figure 7-2)
Length of Upper Roof, l_u =	104.0 ft		
Length of Lower Roof, l_l =	40.0 ft		
Height of Obstruction, h_o =	2.0 ft		
Is upper roof a slippery surface?	No		(Sec. 7.9)
Lower Roof Slope =	1.000:12	Minimum p_f applies for slopes < 3:12	
Upper Roof Slope =	1.000:12	Include Sliding Snow for slopes > 2:12	



Configuration of Snow Drift on Lower Roof

Results:

Exposure Factor, C_e =	0.9		(Table 7-2)
Importance Factor, I_s =	1.00		(Table 1.5-2)
Flat Roof Snow Load =	15.8 psf	= 0.7*Ce*Ct*Is*pg	(Eqn. 7.3-1)
p_m =	20.0 psf	pg > 20 psf, pm=20*Is	(Sec. 7.3.4)
Balanced Flat Roof Snow Load, pf =	**25.0 psf**	maximum of pm and Flat Rood Snow Load	(Sec. 7.3.4)

Drifting Snow Case **ASCE 7-10 Section 7.7**

Snow Density, γ =	17.3 pcf	= 0.13pg + 14 ≤ 30	(Eqn. 7.7-1)
Height of Balanced Snow, h_b =	1.4 ft	= pf/γ	(Sec. 7.1)
Clear Height, h_c =	0.6 ft	= ho - hb	(Sec. 7.1)
Leeward Drift Height, h_{dl} =	3.4 ft	= 0.43(lu^0.33)((pg+10)^.25)-1.5, where lu ≥ 20'	(Figure 7-9)
Windward Drift Height, h_{dw} =	1.6 ft	= 0.75*(0.43(ll^0.33)((pg+10)^.25)-1.5), where ll ≥ 20'	(Figure 7-9)
Design Drift Height, h_d =	0.6 ft	= min(hc and max(hdl and hdw))	
h_c / h_b =	0.38	hc/hb ≥ 0.2, Snow Drifts are applied	(Sec. 7.7.1)
Drift Width(Case a), w_a =	2.2 ft	hd ≤ hc then w_a = 4hd -- hd > hc then w_a = 4hd^2/hc	(Sec. 7.7.1)
Drift Width (Case b), w_b =	4.4 ft	= 8hc	(Sec. 7.7.1)
Controlling Drift Width, w =	**2.2 ft**	= min(wa and wb) and ≤ Ll	(Sec. 7.7.1)
Drift Snow Load, pd =	**9.5 psf**	= γhd	(Sec. 7.7.1)
Drift Snow Load, pd,end =	**0.0 psf**	=pd*(w-Ll)/w Note: pd Decreases to 0 linearly	
Total Snow Load, $p_{(total)}$ =	**34.5 psf**	= pd+pf **Controls**	

Sliding Snow Case **ASCE 7-10 Section 7.9**

Sliding Snow, p_{ss} =	0.0 psf	(0.4*pf*W(15-s)/(15*Ws)) Applied for lu or 15ft Maximum	(Sec. 7.9)
Sliding Snow Extent, w_s =	0.0 ft		(Sec. 7.9)
Max Snow Load, p_d =	**0.0 psf**	Slope < 2:12 **Does Not Apply**	(Sec. 7.9)

2012 IBC Snow Loading for Low Roofs and Decks

Design per 2012 International Building Code - ASCE 7-10 Sections 7.7-7.8, 7.9

Input Data:

Risk Category = II (Table 1.5-1)

Ground Snow Load, p_g = 25.0 psf (Figure 7-1, Table 7-1 for AK)

Roof Exposure = Fully Exposed (Table 7-2)

Terrain Category = B (Sec. 26.7)

Thermal Factor, C_t = 1.0 (Table 7-3)

Roof Slope Factor, C_s = 1.0 (Sec. 7.4.1-7.4.4 and Figure 7-2)

Length of Upper Roof, l_u = 26.0 ft

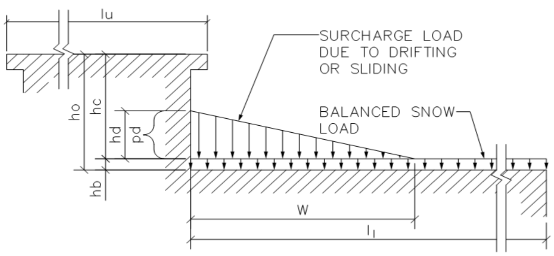
Length of Lower Roof, l_l = 9.3 ft

Height of Obstruction, h_o = 4.0 ft

Is upper roof a slippery surface? No (Sec. 7.9)

Lower Roof Slope = 1.000:12 Minimum p_f applies for slopes < 3:12

Upper Roof Slope = 1.000:12 Include Sliding Snow for slopes > 2:12



Configuration of Snow Drift on Lower Roof

Results:

Exposure Factor, C_e = 0.9 (Table 7-2)

Importance Factor, I_s = 1.00 (Table 1.5-2)

Flat Roof Snow Load = 15.8 psf = 0.7*Ce*Ct*Is*pg (Eqn. 7.3-1)

p_m = 20.0 psf pg > 20 psf, pm=20*Is (Sec. 7.3.4)

Balanced Flat Roof Snow Load, pf = 25.0 psf maximum of pm and Flat Rood Snow Load (Sec. 7.3.4)

Drifting Snow Case **ASCE 7-10 Section 7.7**

Snow Density, γ = 17.3 pcf = 0.13pg + 14 ≤ 30 (Eqn. 7.7-1)

Height of Balanced Snow, h_b = 1.4 ft = pf/γ (Sec. 7.1)

Clear Height, h_c = 2.6 ft = ho - hb (Sec. 7.1)

Leeward Drift Height, h_{dl} = 1.6 ft = 0.43(lu^0.33)((pg+10)^.25)-1.5, where lu ≥ 20' (Figure 7-9)

Windward Drift Height, h_{dw} = 1.0 ft = 0.75*(0.43(ll^0.33)((pg+10)^.25)-1.5), where ll ≥ 20' (Figure 7-9)

Design Drift Height, h_d = 1.6 ft = min(hc and max(hdl and hdw))

h_c / h_b = 1.76 hc/hb ≥ 0.2, Snow Drifts are applied (Sec. 7.7.1)

Drift Width(Case a), w_a = 6.4 ft hd ≤ hc then w_a = 4hd -- hd > hc then $w_a = 4hd^2/hc$ (Sec. 7.7.1)

Drift Width (Case b), w_b = 20.4 ft = 8hc (Sec. 7.7.1)

Controlling Drift Width, w = 6.4 ft = min(wa and wb) and ≤ Ll (Sec. 7.7.1)

Drift Snow Load, pd = 27.6 psf = γhd (Sec. 7.7.1)

Drift Snow Load, pd,end = 0.0 psf =pd*(w-Ll)/w Note: pd Decreases to 0 linearly

Total Snow Load, $p_{(total)}$ = 52.6 psf = pd+pf **Controls**

Sliding Snow Case **ASCE 7-10 Section 7.9**

Sliding Snow, p_{ss} = 0.0 psf (0.4*pf*W(15-s)/(15*Ws)) Applied for lu or 15ft Maximum (Sec. 7.9)

Sliding Snow Extent, w_s = 0.0 ft (Sec. 7.9)

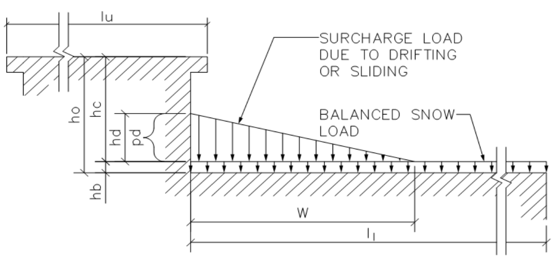
Max Snow Load, p_d = 0.0 psf Slope < 2:12 **Does Not Apply** (Sec. 7.9)

2012 IBC Snow Loading for Low Roofs and Decks

Design per 2012 International Building Code - ASCE 7-10 Sections 7.7-7.8, 7.9

Input Data:

Risk Category =	II		(Table 1.5-1)
Ground Snow Load, p_g =	25.0 psf		(Figure 7-1, Table 7-1 for AK)
Roof Exposure =	Fully Exposed		(Table 7-2)
Terrain Category =	B		(Sec. 26.7)
Thermal Factor, C_t =	1.0		(Table 7-3)
Roof Slope Factor, C_s =	1.0		(Sec. 7.4.1-7.4.4 and Figure 7-2)
Length of Upper Roof, l_u =	104.0 ft		
Length of Lower Roof, l_l =	18.0 ft		
Height of Obstruction, h_o =	4.0 ft		
Is upper roof a slippery surface?	No		(Sec. 7.9)
Lower Roof Slope =	.000:12	Minimum p_f applies for slopes < 3:12	
Upper Roof Slope =	1.000:12	Include Sliding Snow for slopes > 2:12	



Configuration of Snow Drift on Lower Roof

Results:

Exposure Factor, C_e =	0.9		(Table 7-2)
Importance Factor, I_s =	1.00		(Table 1.5-2)
Flat Roof Snow Load =	15.8 psf	= 0.7*Ce*Ct*Is*pg	(Eqn. 7.3-1)
p_m =	20.0 psf	pg > 20 psf, pm=20*Is	(Sec. 7.3.4)
Balanced Flat Roof Snow Load, pf =	**25.0 psf**	maximum of pm and Flat Rood Snow Load	(Sec. 7.3.4)

Drifting Snow Case **ASCE 7-10 Section 7.7**

Snow Density, γ =	17.3 pcf	= 0.13pg + 14 ≤ 30	(Eqn. 7.7-1)
Height of Balanced Snow, h_b =	1.4 ft	= pf/γ	(Sec. 7.1)
Clear Height, h_c =	2.6 ft	= ho - hb	(Sec. 7.1)
Leeward Drift Height, h_{dl} =	3.4 ft	= 0.43(lu^0.33)((pg+10)^.25)-1.5, where lu ≥ 20'	(Figure 7-9)
Windward Drift Height, h_{dw} =	1.0 ft	= 0.75*(0.43(ll^0.33)((pg+10)^.25)-1.5), where ll ≥ 20'	(Figure 7-9)
Design Drift Height, h_d =	2.6 ft	= min(hc and max(hdl and hdw))	
h_c / h_b =	1.76	hc/hb ≥ 0.2, Snow Drifts are applied	(Sec. 7.7.1)
Drift Width(Case a), w_a =	10.2 ft	hd ≤ hc then w_a = 4hd -- hd > hc then w_a = $4hd^2/hc$	(Sec. 7.7.1)
Drift Width (Case b), w_b =	20.4 ft	= 8hc	(Sec. 7.7.1)
Controlling Drift Width, w =	**10.2 ft**	= min(wa and wb) and ≤ Ll	(Sec. 7.7.1)
Drift Snow Load, pd =	**44.0 psf**	= γhd	(Sec. 7.7.1)
Drift Snow Load, pd,end =	**0.0 psf**	=pd*(w-Ll)/w — Note: pd Decreases to 0 linearly	
Total Snow Load, $p_{(total)}$ =	**69.0 psf**	= pd+pf — **Controls**	

Sliding Snow Case **ASCE 7-10 Section 7.9**

Sliding Snow, p_{ss} =	0.0 psf	(0.4*pf*W(15-s)/(15*Ws)) Applied for lu or 15ft Maximum	(Sec. 7.9)
Sliding Snow Extent, w_s =	0.0 ft		(Sec. 7.9)
Max Snow Load, p_d =	**0.0 psf**	Slope < 2:12 — **Does Not Apply**	(Sec. 7.9)

Title Block Line 1
You can change this area
using the "Settings" menu item
and then using the "Printing &
Title Block" selection.
Title Block Line 6

Project Title:
Engineer:
Project Descr:

Project ID:

Printed: 28 SEP 2018, 9:41AM

Wood Beam

ita Elementary School Structural Evaluation\Calcs\Gravity\2018-09-27_WRK18111.00 - Gravity Framing - V1.0.ec6
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10

Lic. # : KW-06010783 **Licensee : WRK Engineers**

Description : 6x8 Beam

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

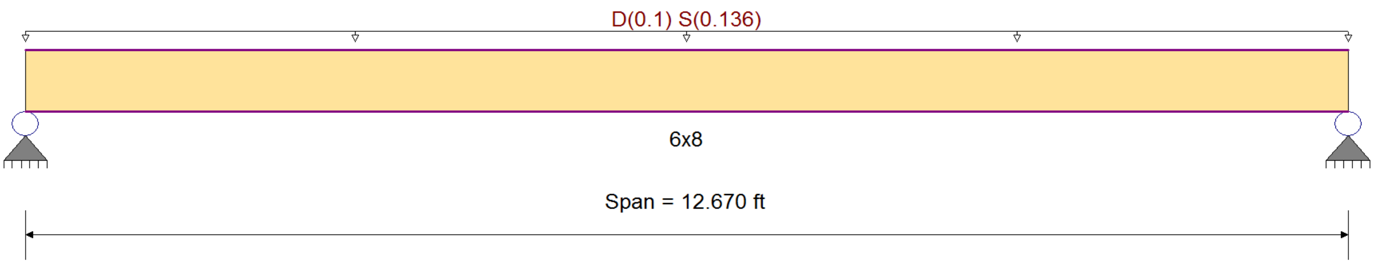
Wood Species : Douglas Fir - Larch
Wood Grade : No.1

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

Fb +	1350	psi
Fb -	1350	psi
Fc - Prll	925	psi
Fc - Perp	625	psi
Fv	170	psi
Ft	675	psi

E : Modulus of Elasticity

Ebend- xx	1600	ksi
Eminbend - xx	580	ksi
Density	31.2	pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.10, S = 0.1360 , Tributary Width = 1.0 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.737**: 1	Maximum Shear Stress Ratio	=	**0.261** : 1
Section used for this span		**6x8**	Section used for this span		**6x8**
fb : Actual	=	1,143.84 psi	fv : Actual	=	51.07 psi
FB : Allowable	=	1,552.50 psi	Fv : Allowable	=	195.50 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	6.335 ft	Location of maximum on span	=	12.069 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.256 in	Ratio =	593 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.462 in	Ratio =	329 >=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 = 12.670 ft	1	0.419	0.148	0.90	1.000	1.00	1.00	1.00	1.00	1.00	2.19	508.73	1215.00	0.62	22.71	153.00
+D+L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.377	0.134	1.00	1.000	1.00	1.00	1.00	1.00	1.00	2.19	508.73	1350.00	0.62	22.71	170.00
+D+Lr+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.301	0.107	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.19	508.73	1687.50	0.62	22.71	212.50
+D+S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.737	0.261	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.91	1,143.84	1552.50	1.40	51.07	195.50
+D+0.750Lr+0.750L+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Description : 6x8 Beam

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
Length = 12.670 ft	1	0.301	0.107	1.25	1.000	1.00	1.00	1.00	1.00	1.00	2.19	508.73	1687.50	0.62	22.71	212.50
+D+0.750L+0.750S+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.635	0.225	1.15	1.000	1.00	1.00	1.00	1.00	1.00	4.23	985.06	1552.50	1.21	43.98	195.50
+D+0.60W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.236	0.084	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.19	508.73	2160.00	0.62	22.71	272.00
+D+0.70E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.236	0.084	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.19	508.73	2160.00	0.62	22.71	272.00
+D+0.750Lr+0.750L+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.236	0.084	1.60	1.000	1.00	1.00	1.00	1.00	1.00	2.19	508.73	2160.00	0.62	22.71	272.00
+D+0.750L+0.750S+0.450W+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.456	0.162	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.23	985.06	2160.00	1.21	43.98	272.00
+D+0.750L+0.750S+0.5250E+H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.456	0.162	1.60	1.000	1.00	1.00	1.00	1.00	1.00	4.23	985.06	2160.00	1.21	43.98	272.00
+0.60D+0.60W+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.141	0.050	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.31	305.24	2160.00	0.37	13.63	272.00
+0.60D+0.70E+0.60H					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 12.670 ft	1	0.141	0.050	1.60	1.000	1.00	1.00	1.00	1.00	1.00	1.31	305.24	2160.00	0.37	13.63	272.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.4617	6.381		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.552	1.552
Overall MINimum	0.862	0.862
+D+H	0.690	0.690
+D+L+H	0.690	0.690
+D+Lr+H	0.690	0.690
+D+S+H	1.552	1.552
+D+0.750Lr+0.750L+H	0.690	0.690
+D+0.750L+0.750S+H	1.336	1.336
+D+0.60W+H	0.690	0.690
+D+0.70E+H	0.690	0.690
+D+0.750Lr+0.750L+0.450W+H	0.690	0.690
+D+0.750L+0.750S+0.450W+H	1.336	1.336
+D+0.750L+0.750S+0.5250E+H	1.336	1.336
+0.60D+0.60W+0.60H	0.414	0.414
+0.60D+0.70E+0.60H	0.414	0.414
D Only	0.690	0.690
Lr Only		
L Only		
S Only	0.862	0.862
W Only		
E Only		
H Only		

Wood Column

Description : 6x6 Post

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method : Allowable Stress Design
End Fixities: Top & Bottom Pinned
Overall Column Height: 9.0 ft
(Used for non-slender calculations)

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

Fb +	1,200.0 psi	Fv	170.0 psi
Fb -	1,200.0 psi	Ft	825.0 psi
Fc - Prll	1,000.0 psi	Density	31.20 pcf
Fc - Perp	625.0 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1,600.0	1,600.0	1,600.0 ksi
Minimum	580.0	580.0	

Wood Section Name: **6x6**
Wood Grading/Manuf.: Graded Lumber
Wood Member Type: Sawn

Exact Width	**5.50** in
Exact Depth	**5.50** in
Area	30.250 in^2
Ix	76.255 in^4
Iy	76.255 in^4

Allow Stress Modification Factors

Cf or Cv for Bending	1.0
Cf or Cv for Compression	1.0
Cf or Cv for Tension	1.0
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 NDS 15.3.2
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 9.0 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 9.0 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 58.988 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 9.0 ft, Xecc = 0.50 in, Yecc = 0.50 in, D = 1.250, S = 1.70 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.1209** : 1
	Load Combination	+D+S+H
	Governing NDS Forumla	Comp Only, fc/Fc'
	Location of max.above base	0.0 ft
	At maximum location values are . . .	
	Applied Axial	3.009 k
	Applied Mx	0.0 k-ft
	Applied My	0.0 k-ft
	Fc : Allowable	822.75 psi
PASS	Maximum Shear Stress Ratio =	**0.003464** : 1
	Load Combination	+D+S+H
	Location of max.above base	9.0 ft
	Applied Design Shear	0.6772 psi
	Allowable Shear	195.50 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.01366 k	Bottom along Y-Y	0.01366 k
Top along X-X	0.01366 k	Bottom along X-X	0.01366 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y -0.009124 in at 5.255 ft above base
for load combination : +D+S+H
Along X-X -0.009124 in at 5.255 ft above base
for load combination : +D+S+H

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	0.788	0.06102	PASS	0.0 ft	0.001876	PASS	9.0 ft
+D+L+H	1.000	0.759	0.05702	PASS	0.0 ft	0.001688	PASS	9.0 ft
+D+Lr+H	1.250	0.687	0.05038	PASS	0.0 ft	0.001350	PASS	9.0 ft
+D+S+H	1.150	0.715	0.1209	PASS	0.0 ft	0.003464	PASS	9.0 ft
+D+0.750Lr+0.750L+H	1.250	0.687	0.05038	PASS	0.0 ft	0.001350	PASS	9.0 ft
+D+0.750L+0.750S+H	1.150	0.715	0.1038	PASS	0.0 ft	0.002965	PASS	9.0 ft
+D+0.60W+H	1.600	0.596	0.04534	PASS	0.0 ft	0.001055	PASS	9.0 ft
+D+0.70E+H	1.600	0.596	0.04534	PASS	0.0 ft	0.001055	PASS	9.0 ft
+D+0.750Lr+0.750L+0.450W+H	1.600	0.596	0.04534	PASS	0.0 ft	0.001055	PASS	9.0 ft
+D+0.750L+0.750S+0.450W+H	1.600	0.596	0.08951	PASS	0.0 ft	0.002131	PASS	9.0 ft
+D+0.750L+0.750S+0.5250E+H	1.600	0.596	0.08951	PASS	0.0 ft	0.002131	PASS	9.0 ft
+0.60D+0.60W+0.60H	1.600	0.596	0.02721	PASS	0.0 ft	0.000633	PASS	9.0 ft

Wood Column

Description : 6x6 Post

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D+0.70E+0.60H	1.600	0.596	0.02721	PASS	0.0 ft	0.000633	PASS	9.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H	-0.006	0.006	-0.006	0.006	1.309				
+D+L+H	-0.006	0.006	-0.006	0.006	1.309				
+D+Lr+H	-0.006	0.006	-0.006	0.006	1.309				
+D+S+H	-0.014	0.014	-0.014	0.014	3.009				
+D+0.750Lr+0.750L+H	-0.006	0.006	-0.006	0.006	1.309				
+D+0.750L+0.750S+H	-0.012	0.012	-0.012	0.012	2.584				
+D+0.60W+H	-0.006	0.006	-0.006	0.006	1.309				
+D+0.70E+H	-0.006	0.006	-0.006	0.006	1.309				
+D+0.750Lr+0.750L+0.450W+H	-0.006	0.006	-0.006	0.006	1.309				
+D+0.750L+0.750S+0.450W+H	-0.012	0.012	-0.012	0.012	2.584				
+D+0.750L+0.750S+0.5250E+H	-0.012	0.012	-0.012	0.012	2.584				
+0.60D+0.60W+0.60H	-0.003	0.003	-0.003	0.003	0.785				
+0.60D+0.70E+0.60H	-0.003	0.003	-0.003	0.003	0.785				
D Only	-0.006	0.006	-0.006	0.006	1.309				
Lr Only									
L Only									
S Only	-0.008	0.008	-0.008	0.008	1.700				
W Only									
E Only									
H Only									

Maximum Deflections for Load Combinations

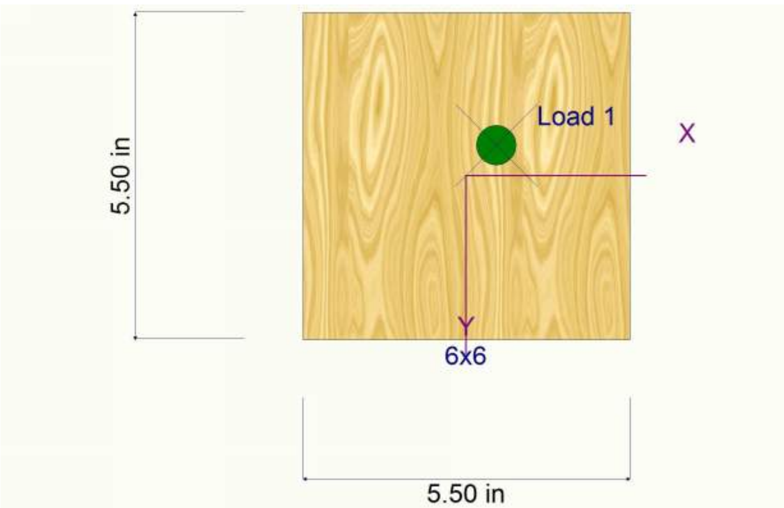
Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
+D+L+H	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
+D+Lr+H	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
+D+S+H	-0.0091 in	5.255 ft	-0.009 in	5.255 ft
+D+0.750Lr+0.750L+H	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
+D+0.750L+0.750S+H	-0.0078 in	5.255 ft	-0.008 in	5.255 ft
+D+0.60W+H	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
+D+0.70E+H	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
+D+0.750Lr+0.750L+0.450W+H	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
+D+0.750L+0.750S+0.450W+H	-0.0078 in	5.255 ft	-0.008 in	5.255 ft
+D+0.750L+0.750S+0.5250E+H	-0.0078 in	5.255 ft	-0.008 in	5.255 ft
+0.60D+0.60W+0.60H	-0.0023 in	5.255 ft	-0.002 in	5.255 ft
+0.60D+0.70E+0.60H	-0.0023 in	5.255 ft	-0.002 in	5.255 ft
D Only	-0.0039 in	5.255 ft	-0.004 in	5.255 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	-0.0053 in	5.255 ft	-0.005 in	5.255 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

Wood Column

Lic. # : KW-06010783 **Licensee : WRK Engineers**

Description : 6x6 Post

Sketches



Wood Column

Description : 4x6 post

Code References

Calculations per NDS 2015, IBC 2015, CBC 2016, ASCE 7-10
Load Combinations Used : ASCE 7-10

General Information

Analysis Method :	Allowable Stress Design	Wood Section Name	**4x6**
End Fixities	Top & Bottom Pinned	Wood Grading/Manuf.	Graded Lumber
Overall Column Height	10 ft	Wood Member Type	Sawn

(Used for non-slender calculations)

Wood Species	Douglas Fir - Larch		
Wood Grade	Select structural		
Fb +	1500 psi	Fv	180 psi
Fb -	1500 psi	Ft	1000 psi
Fc - Prll	1700 psi	Density	31.2 pcf
Fc - Perp	625 psi		

E : Modulus of Elasticity . . .	x-x Bending	y-y Bending	Axial
Basic	1900	1900	1900 ksi
Minimum	690	690	

Exact Width	**3.50** in
Exact Depth	**5.50** in
Area	19.250 in^2
Ix	48.526 in^4
Iy	19.651 in^4

Allow Stress Modification Factors	
Cf or Cv for Bending	1.30
Cf or Cv for Compression	1.10
Cf or Cv for Tension	1.30
Cm : Wet Use Factor	1.0
Ct : Temperature Factor	1.0
Cfu : Flat Use Factor	1.0
Kf : Built-up columns	1.0 *NDS 15.3.2*
Use Cr : Repetitive ?	No

Brace condition for deflection (buckling) along columns :
X-X (width) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0
Y-Y (depth) axis : Unbraced Length for X-X Axis buckling = 10 ft, K = 1.0

Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Column self weight included : 41.708 lbs * Dead Load Factor
AXIAL LOADS . . .
Axial Load at 10.0 ft, Xecc = 0.50 in, Yecc = 0.50 in, D = 2.60, S = 3.250 k

DESIGN SUMMARY

Bending & Shear Check Results

PASS	Max. Axial+Bending Stress Ratio =	**0.9583** : 1
	Load Combination	+D+S+H
	Governing NDS Forumla	np + Mxx + Myy, NDS Eq. 3.9-
	Location of max.above base	9.933 ft
	At maximum location values are . . .	
	Applied Axial	5.892 k
	Applied Mx	-0.2421 k-ft
	Applied My	-0.2421 k-ft
	Fc : Allowable	457.741 psi
PASS	Maximum Shear Stress Ratio =	**0.009176** : 1
	Load Combination	+D+S+H
	Location of max.above base	10.0 ft
	Applied Design Shear	1.899 psi
	Allowable Shear	207.0 psi

Maximum SERVICE Lateral Load Reactions . .

Top along Y-Y	0.02438 k	Bottom along Y-Y	0.02438 k
Top along X-X	0.02438 k	Bottom along X-X	0.02438 k

Maximum SERVICE Load Lateral Deflections . . .
Along Y-Y -0.02956 in at 5.839 ft above base
for load combination : +D+S+H
Along X-X -0.0730 in at 5.839 ft above base
for load combination : +D+S+H

Other Factors used to calculate allowable stresses . . .

Bending	Compression	Tension

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+D+H	0.900	0.267	0.3052	PASS	0.0 ft	0.005211	PASS	10.0 ft
+D+L+H	1.000	0.242	0.3026	PASS	0.0 ft	0.004690	PASS	10.0 ft
+D+Lr+H	1.250	0.197	0.2984	PASS	0.0 ft	0.003752	PASS	10.0 ft
+D+S+H	1.150	0.213	0.9583	PASS	9.933 ft	0.009176	PASS	10.0 ft
+D+0.750Lr+0.750L+H	1.250	0.197	0.2984	PASS	0.0 ft	0.003752	PASS	10.0 ft
+D+0.750L+0.750S+H	1.150	0.213	0.7001	PASS	9.933 ft	0.007901	PASS	10.0 ft
+D+0.60W+H	1.600	0.156	0.2949	PASS	0.0 ft	0.002931	PASS	10.0 ft
+D+0.70E+H	1.600	0.156	0.2949	PASS	0.0 ft	0.002931	PASS	10.0 ft
+D+0.750Lr+0.750L+0.450W+H	1.600	0.156	0.2949	PASS	0.0 ft	0.002931	PASS	10.0 ft
+D+0.750L+0.750S+0.450W+H	1.600	0.156	0.5865	PASS	9.933 ft	0.005679	PASS	10.0 ft
+D+0.750L+0.750S+0.5250E+H	1.600	0.156	0.5865	PASS	9.933 ft	0.005679	PASS	10.0 ft
+0.60D+0.60W+0.60H	1.600	0.156	0.1769	PASS	0.0 ft	0.001759	PASS	10.0 ft

Wood Column

Description : 4x6 post

Load Combination Results

Load Combination	C_D	C_P	Maximum Axial + Bending Stress Ratios: Stress Ratio	Status	Location	Maximum Shear Ratios: Stress Ratio	Status	Location
+0.60D+0.70E+0.60H	1.600	0.156	0.1769	PASS	0.0 ft	0.001759	PASS	10.0 ft

Maximum Reactions

Note: Only non-zero reactions are listed.

Load Combination	X-X Axis Reaction k @ Base	X-X Axis Reaction @ Top	Y-Y Axis Reaction @ Base	Y-Y Axis Reaction @ Top	Axial Reaction @ Base	My - End Moments k-ft @ Base	My - End Moments @ Top	Mx - End Moments @ Base	Mx - End Moments @ Top
+D+H	-0.011	0.011	-0.011	0.011	2.642				
+D+L+H	-0.011	0.011	-0.011	0.011	2.642				
+D+Lr+H	-0.011	0.011	-0.011	0.011	2.642				
+D+S+H	-0.024	0.024	-0.024	0.024	5.892				
+D+0.750Lr+0.750L+H	-0.011	0.011	-0.011	0.011	2.642				
+D+0.750L+0.750S+H	-0.021	0.021	-0.021	0.021	5.079				
+D+0.60W+H	-0.011	0.011	-0.011	0.011	2.642				
+D+0.70E+H	-0.011	0.011	-0.011	0.011	2.642				
+D+0.750Lr+0.750L+0.450W+H	-0.011	0.011	-0.011	0.011	2.642				
+D+0.750L+0.750S+0.450W+H	-0.021	0.021	-0.021	0.021	5.079				
+D+0.750L+0.750S+0.5250E+H	-0.021	0.021	-0.021	0.021	5.079				
+0.60D+0.60W+0.60H	-0.007	0.007	-0.007	0.007	1.585				
+0.60D+0.70E+0.60H	-0.007	0.007	-0.007	0.007	1.585				
D Only	-0.011	0.011	-0.011	0.011	2.642				
Lr Only									
L Only									
S Only	-0.014	0.014	-0.014	0.014	3.250				
W Only									
E Only									
H Only									

Maximum Deflections for Load Combinations

Load Combination	Max. X-X Deflection	Distance	Max. Y-Y Deflection	Distance
+D+H	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
+D+L+H	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
+D+Lr+H	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
+D+S+H	-0.0730 in	5.839 ft	-0.030 in	5.839 ft
+D+0.750Lr+0.750L+H	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
+D+0.750L+0.750S+H	-0.0629 in	5.839 ft	-0.025 in	5.839 ft
+D+0.60W+H	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
+D+0.70E+H	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
+D+0.750Lr+0.750L+0.450W+H	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
+D+0.750L+0.750S+0.450W+H	-0.0629 in	5.839 ft	-0.025 in	5.839 ft
+D+0.750L+0.750S+0.5250E+H	-0.0629 in	5.839 ft	-0.025 in	5.839 ft
+0.60D+0.60W+0.60H	-0.0195 in	5.839 ft	-0.008 in	5.839 ft
+0.60D+0.70E+0.60H	-0.0195 in	5.839 ft	-0.008 in	5.839 ft
D Only	-0.0324 in	5.839 ft	-0.013 in	5.839 ft
Lr Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
L Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
S Only	-0.0406 in	5.839 ft	-0.016 in	5.839 ft
W Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
E Only	0.0000 in	0.000 ft	0.000 in	0.000 ft
H Only	0.0000 in	0.000 ft	0.000 in	0.000 ft

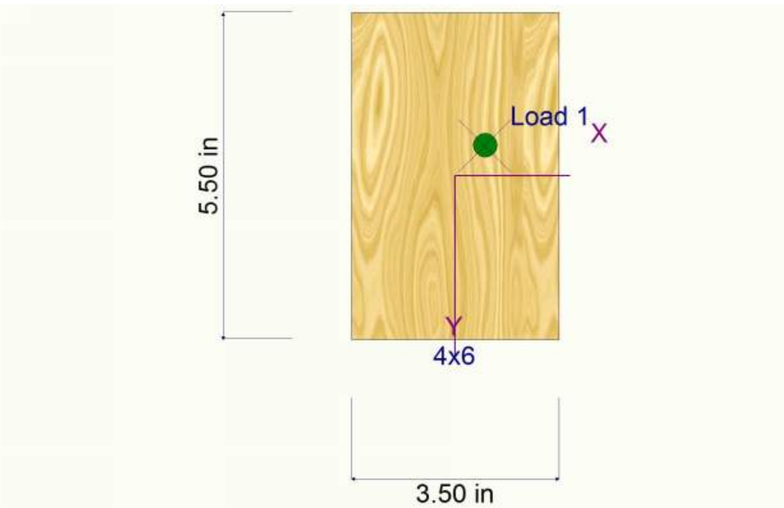
Wood Column

ita Elementary School Structural Evaluation\Calcs\Gravity\2018-09-27_WRK18111.00 - Gravity Framing - V1.0.ec6
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10

Lic. # : KW-06010783 **Licensee : WRK Engineers**

Description : 4x6 post

Sketches



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Title Block Line 6

Project Title:
Engineer:
Project Descr:

Project ID:

Printed: 28 SEP 2018, 11:05AM

Wood Beam

ita Elementary School Structural Evaluation\Calcs\Gravity\2018-09-27_WRK18111.00 - Gravity Framing - V1.0.ec6
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10

Lic. # : KW-06010783 **Licensee : WRK Engineers**

Description : 2x6 at 16"oc

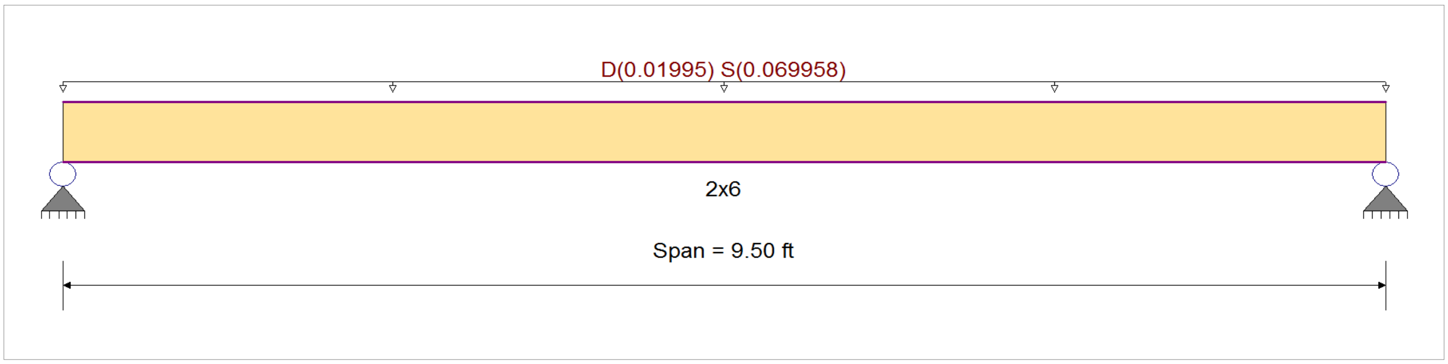
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,500.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb -	1,500.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	1,700.0 psi	Eminbend - xx	690.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: Select structural	Fv	180.0 psi		
		Ft	1,000.0 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.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.05260 ksf, Tributary Width = 1.330 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.732** 1	Maximum Shear Stress Ratio	=	**0.346** : 1
Section used for this span		**2x6**	Section used for this span		**2x6**
fb : Actual	=	1,641.43 psi	fv : Actual	=	71.68 psi
FB : Allowable	=	2,242.50 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	4.750 ft	Location of maximum on span	=	9.049 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.326 in	Ratio =	349 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.428 in	Ratio =	266 >=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 = 9.50 ft	1	0.222	0.105	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.25	389.12	1755.00	0.09	16.99	162.00
+D+L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.200	0.094	1.00	1.300	1.00	1.00	1.00	1.00	1.00	0.25	389.12	1950.00	0.09	16.99	180.00
+D+Lr+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.160	0.076	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.25	389.12	2437.50	0.09	16.99	225.00
+D+S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.732	0.346	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.03	1,641.43	2242.50	0.39	71.68	207.00
+D+0.750Lr+0.750L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Description : 2x6 at 16"oc

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
Length = 9.50 ft	1	0.160	0.076	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.25	389.12	2437.50	0.09	16.99	225.00
+D+0.750L+0.750S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.592	0.280	1.15	1.300	1.00	1.00	1.00	1.00	1.00	0.84	1,328.35	2242.50	0.32	58.01	207.00
+D+0.60W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.125	0.059	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.25	389.12	3120.00	0.09	16.99	288.00
+D+0.70E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.125	0.059	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.25	389.12	3120.00	0.09	16.99	288.00
+D+0.750Lr+0.750L+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.125	0.059	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.25	389.12	3120.00	0.09	16.99	288.00
+D+0.750L+0.750S+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.426	0.201	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.84	1,328.35	3120.00	0.32	58.01	288.00
+D+0.750L+0.750S+0.5250E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.426	0.201	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.84	1,328.35	3120.00	0.32	58.01	288.00
+0.60D+0.60W+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.075	0.035	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.15	233.47	3120.00	0.06	10.20	288.00
+0.60D+0.70E+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.50 ft	1	0.075	0.035	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.15	233.47	3120.00	0.06	10.20	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.4278	4.785		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.436	0.436
Overall MINimum	0.332	0.332
+D+H	0.103	0.103
+D+L+H	0.103	0.103
+D+Lr+H	0.103	0.103
+D+S+H	0.436	0.436
+D+0.750Lr+0.750L+H	0.103	0.103
+D+0.750L+0.750S+H	0.352	0.352
+D+0.60W+H	0.103	0.103
+D+0.70E+H	0.103	0.103
+D+0.750Lr+0.750L+0.450W+H	0.103	0.103
+D+0.750L+0.750S+0.450W+H	0.352	0.352
+D+0.750L+0.750S+0.5250E+H	0.352	0.352
+0.60D+0.60W+0.60H	0.062	0.062
+0.60D+0.70E+0.60H	0.062	0.062
D Only	0.103	0.103
Lr Only		
L Only		
S Only	0.332	0.332
W Only		
E Only		
H Only		

Wood Beam

Description : 2x8 at 16"oc

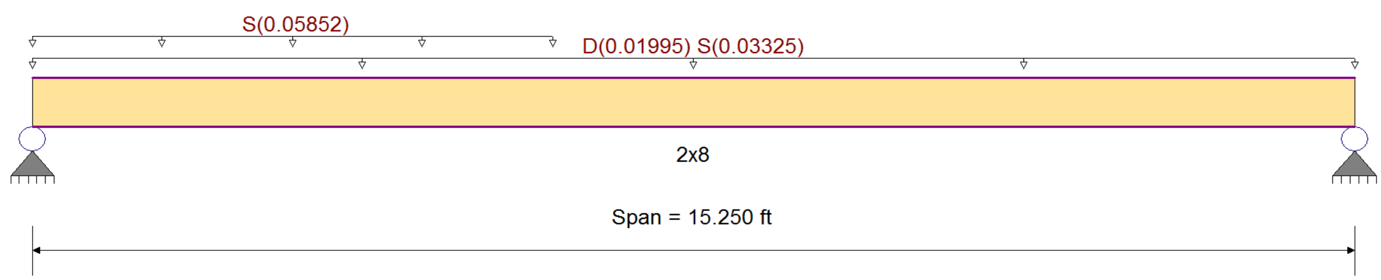
CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1000 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb -	1000 psi	Ebend- xx	1700 ksi
		Fc - Prll	1500 psi	Eminbend - xx	620 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625 psi		
Wood Grade	: No.1	Fv	180 psi		
		Ft	675 psi	Density	31.2 pcf

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



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 1.330 ft

Uniform Load : S = 0.0440 ksf, Extent = 0.0 -->> 6.0 ft, Tributary Width = 1.330 ft, (Drifted Snow)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	1.446 : 1	Maximum Shear Stress Ratio	=	0.428 : 1
Section used for this span		2x8	Section used for this span		2x8
fb : Actual	=	1,995.02 psi	fv : Actual	=	88.58 psi
FB : Allowable	=	1,380.00 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	6.401 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	0.799 in	Ratio =	228 <240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	1.136 in	Ratio =	161 <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 = 15.250 ft	1	0.548	0.134	0.90	1.200	1.00	1.00	1.00	1.00	1.00	0.65	592.16	1080.00	0.16	21.75	162.00
+D+L+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.493	0.121	1.00	1.200	1.00	1.00	1.00	1.00	1.00	0.65	592.16	1200.00	0.16	21.75	180.00
+D+Lr+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.395	0.097	1.25	1.200	1.00	1.00	1.00	1.00	1.00	0.65	592.16	1500.00	0.16	21.75	225.00

Wood Beam

Lic. # : KW-06010783 **Licensee : WRK Engineers**

Description : 2x8 at 16"oc

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+S+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	1.446	0.428	1.15	1.200	1.00	1.00	1.00	1.00	1.00	2.18	1,995.02	1380.00	0.64	88.58	207.00
+D+0.750Lr+0.750L+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.395	0.097	1.25	1.200	1.00	1.00	1.00	1.00	1.00	0.65	592.16	1500.00	0.16	21.75	225.00
+D+0.750L+0.750S+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	1.189	0.347	1.15	1.200	1.00	1.00	1.00	1.00	1.00	1.80	1,640.83	1380.00	0.52	71.87	207.00
+D+0.60W+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.308	0.076	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.65	592.16	1920.00	0.16	21.75	288.00
+D+0.70E+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.308	0.076	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.65	592.16	1920.00	0.16	21.75	288.00
+D+0.750Lr+0.750L+0.450W+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.308	0.076	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.65	592.16	1920.00	0.16	21.75	288.00
+D+0.750L+0.750S+0.450W+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.855	0.250	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.80	1,640.83	1920.00	0.52	71.87	288.00
+D+0.750L+0.750S+0.5250E+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.855	0.250	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.80	1,640.83	1920.00	0.52	71.87	288.00
+0.60D+0.60W+0.60H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.185	0.045	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.39	355.30	1920.00	0.09	13.05	288.00
+0.60D+0.70E+0.60H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.185	0.045	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.39	355.30	1920.00	0.09	13.05	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	1.1359	7.458		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.706	0.493
Overall MINimum	0.536	0.323
+D+H	0.170	0.170
+D+L+H	0.170	0.170
+D+Lr+H	0.170	0.170
+D+S+H	0.706	0.493
+D+0.750Lr+0.750L+H	0.170	0.170
+D+0.750L+0.750S+H	0.572	0.412
+D+0.60W+H	0.170	0.170
+D+0.70E+H	0.170	0.170
+D+0.750Lr+0.750L+0.450W+H	0.170	0.170
+D+0.750L+0.750S+0.450W+H	0.572	0.412
+D+0.750L+0.750S+0.5250E+H	0.572	0.412
+0.60D+0.60W+0.60H	0.102	0.102
+0.60D+0.70E+0.60H	0.102	0.102
D Only	0.170	0.170
Lr Only		
L Only		
S Only	0.536	0.323
W Only		
E Only		
H Only		

Title Block Line 1
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Title Block Line 6

Project Title:
Engineer:
Project Descr:

Project ID:

Printed: 4 OCT 2018, 1:36PM

Wood Beam

ita Elementary School Structural Evaluation\Calcs\Gravity\2018-09-27_WRK18111.00 - Gravity Framing - V1.0.ec6
ENERCALC, INC. 1983-2017, Build:10.17.12.10, Ver:10.17.12.10

Lic. # : KW-06010783 **Licensee : WRK Engineers**

Description : 2x8 at 16"oc w/ sistered 2x8

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination :ASCE 7-10

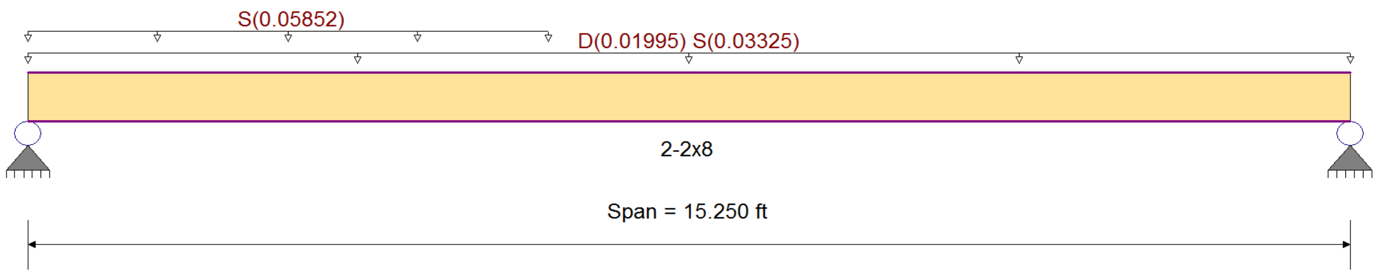
Wood Species : Douglas Fir - Larch
Wood Grade : No.2

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

Fb +	900	psi
Fb -	900	psi
Fc - Prll	1350	psi
Fc - Perp	625	psi
Fv	180	psi
Ft	575	psi

E : Modulus of Elasticity

Ebend- xx	1600	ksi
Eminbend - xx	580	ksi
Density	31.2	pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 1.330 ft

Uniform Load : S = 0.0440 ksf, Extent = 0.0 -->> 6.0 ft, Tributary Width = 1.330 ft, (Drifted Snow)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.828 1**	Maximum Shear Stress Ratio	=	**0.219 : 1**
Section used for this span		**2-2x8**	Section used for this span		**2-2x8**
fb : Actual	=	1,027.98 psi	fv : Actual	=	45.44 psi
FB : Allowable	=	1,242.00 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	6.456 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	0.425 in	Ratio =	431 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.622 in	Ratio =	294 >=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 = 15.250 ft	1	0.337	0.074	0.90	1.200	1.00	1.00	1.00	1.00	1.00	0.72	327.36	972.00	0.17	12.02	162.00
+D+L+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.303	0.067	1.00	1.200	1.00	1.00	1.00	1.00	1.00	0.72	327.36	1080.00	0.17	12.02	180.00
+D+Lr+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.242	0.053	1.25	1.200	1.00	1.00	1.00	1.00	1.00	0.72	327.36	1350.00	0.17	12.02	225.00

Wood Beam

Description : 2x8 at 16"oc

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+S+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.828	0.219	1.15	1.200	1.00	1.00	1.00	1.00	1.00	2.25	1,027.98	1242.00	0.66	45.44	207.00
+D+0.750Lr+0.750L+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.242	0.053	1.25	1.200	1.00	1.00	1.00	1.00	1.00	0.72	327.36	1350.00	0.17	12.02	225.00
+D+0.750L+0.750S+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.685	0.179	1.15	1.200	1.00	1.00	1.00	1.00	1.00	1.86	851.08	1242.00	0.54	37.08	207.00
+D+0.60W+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.189	0.042	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.72	327.36	1728.00	0.17	12.02	288.00
+D+0.70E+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.189	0.042	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.72	327.36	1728.00	0.17	12.02	288.00
+D+0.750Lr+0.750L+0.450W+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.189	0.042	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.72	327.36	1728.00	0.17	12.02	288.00
+D+0.750L+0.750S+0.450W+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.493	0.129	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.86	851.08	1728.00	0.54	37.08	288.00
+D+0.750L+0.750S+0.5250E+H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.493	0.129	1.60	1.200	1.00	1.00	1.00	1.00	1.00	1.86	851.08	1728.00	0.54	37.08	288.00
+0.60D+0.60W+0.60H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.114	0.025	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.43	196.41	1728.00	0.10	7.21	288.00
+0.60D+0.70E+0.60H					1.200	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 15.250 ft	1	0.114	0.025	1.60	1.200	1.00	1.00	1.00	1.00	1.00	0.43	196.41	1728.00	0.10	7.21	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.6223	7.458		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.724	0.511
Overall MINimum	0.536	0.323
+D+H	0.188	0.188
+D+L+H	0.188	0.188
+D+Lr+H	0.188	0.188
+D+S+H	0.724	0.511
+D+0.750Lr+0.750L+H	0.188	0.188
+D+0.750L+0.750S+H	0.590	0.430
+D+0.60W+H	0.188	0.188
+D+0.70E+H	0.188	0.188
+D+0.750Lr+0.750L+0.450W+H	0.188	0.188
+D+0.750L+0.750S+0.450W+H	0.590	0.430
+D+0.750L+0.750S+0.5250E+H	0.590	0.430
+0.60D+0.60W+0.60H	0.113	0.113
+0.60D+0.70E+0.60H	0.113	0.113
D Only	0.188	0.188
Lr Only		
L Only		
S Only	0.536	0.323
W Only		
E Only		
H Only		

Wood Beam

Description : 3x6 at 30" oc

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

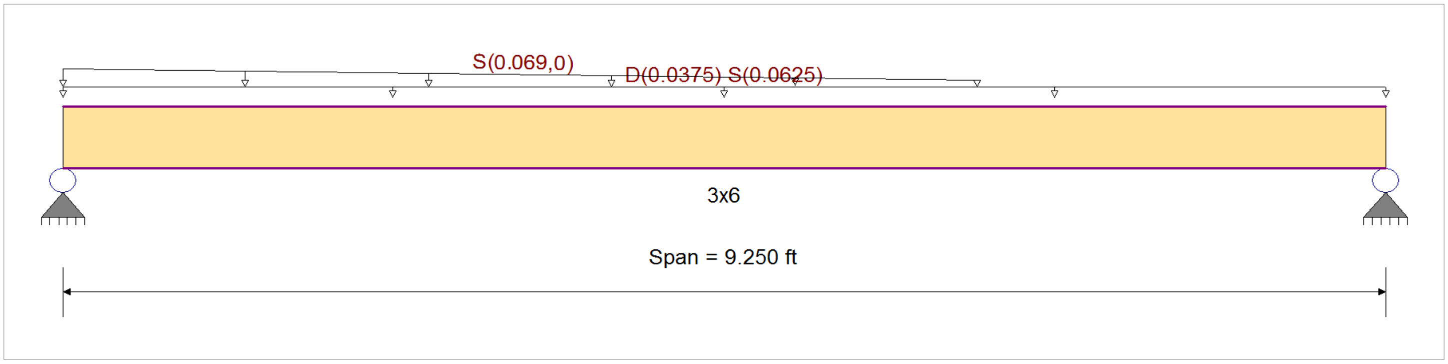
Wood Species : Douglas Fir - Larch
Wood Grade : Select structural

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

Fb +	1,500.0	psi
Fb -	1,500.0	psi
Fc - Prll	1,700.0	psi
Fc - Perp	625.0	psi
Fv	180.0	psi
Ft	1,000.0	psi

E : Modulus of Elasticity

Ebend- xx	1,900.0	ksi
Eminbend - xx	690.0	ksi
Density	31.20	pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 2.50 ft

Varying Uniform Load : S= 0.02760->0.0 ksf, Extent = 0.0 -->> 6.40 ft, Trib Width = 2.50 ft, (Snow Drift)

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.565 : 1**	Maximum Shear Stress Ratio	=	**0.301** : 1
Section used for this span		**3x6**	Section used for this span		**3x6**
fb : Actual	=	1,267.72 psi	fv : Actual	=	62.37 psi
FB : Allowable	=	2,242.50 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	4.355 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	0.211 in	Ratio =	525 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.313 in	Ratio =	354 >=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 = 9.250 ft	1	0.235	0.114	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.43	412.18	1755.00	0.17	18.49	162.00
+D+L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.211	0.103	1.00	1.300	1.00	1.00	1.00	1.00	1.00	0.43	412.18	1950.00	0.17	18.49	180.00
+D+Lr+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.169	0.082	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.43	412.18	2437.50	0.17	18.49	225.00

Wood Beam

Description : 3x6 at 30" oc

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+S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.565	0.301	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.33	1,267.72	2242.50	0.57	62.37	207.00
+D+0.750Lr+0.750L+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.169	0.082	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.43	412.18	2437.50	0.17	18.49	225.00
+D+0.750L+0.750S+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.470	0.248	1.15	1.300	1.00	1.00	1.00	1.00	1.00	1.11	1,053.50	2242.50	0.47	51.40	207.00
+D+0.60W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.132	0.064	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.43	412.18	3120.00	0.17	18.49	288.00
+D+0.70E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.132	0.064	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.43	412.18	3120.00	0.17	18.49	288.00
+D+0.750Lr+0.750L+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.132	0.064	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.43	412.18	3120.00	0.17	18.49	288.00
+D+0.750L+0.750S+0.450W+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.338	0.178	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.11	1,053.50	3120.00	0.47	51.40	288.00
+D+0.750L+0.750S+0.5250E+H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.338	0.178	1.60	1.300	1.00	1.00	1.00	1.00	1.00	1.11	1,053.50	3120.00	0.47	51.40	288.00
+0.60D+0.60W+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.079	0.039	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.26	247.31	3120.00	0.10	11.09	288.00
+0.60D+0.70E+0.60H					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 9.250 ft	1	0.079	0.039	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.26	247.31	3120.00	0.10	11.09	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	0.3130	4.591		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.646	0.527
Overall MINimum	0.459	0.340
+D+H	0.187	0.187
+D+L+H	0.187	0.187
+D+Lr+H	0.187	0.187
+D+S+H	0.646	0.527
+D+0.750Lr+0.750L+H	0.187	0.187
+D+0.750L+0.750S+H	0.531	0.442
+D+0.60W+H	0.187	0.187
+D+0.70E+H	0.187	0.187
+D+0.750Lr+0.750L+0.450W+H	0.187	0.187
+D+0.750L+0.750S+0.450W+H	0.531	0.442
+D+0.750L+0.750S+0.5250E+H	0.531	0.442
+0.60D+0.60W+0.60H	0.112	0.112
+0.60D+0.70E+0.60H	0.112	0.112
D Only	0.187	0.187
Lr Only		
L Only		
S Only	0.459	0.340
W Only		
E Only		
H Only		

Wood Beam

Lic. # : KW-06010783 **Licensee : WRK Engineers**

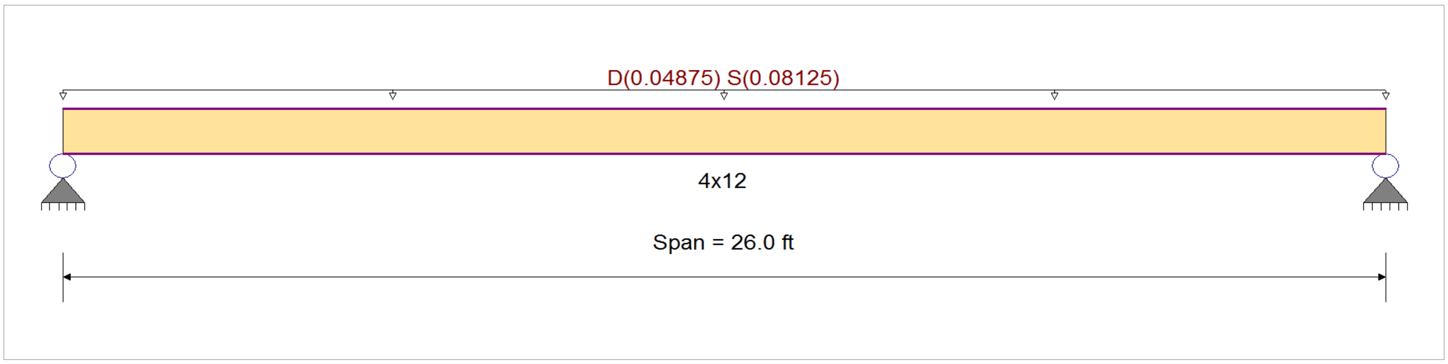
Description : 4x12 at 3'-3" oc

CODE REFERENCES

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

Material Properties

Analysis Method :	Allowable Stress Design	Fb +	1,500.0 psi	*E : Modulus of Elasticity*	
Load Combination :	ASCE 7-10	Fb -	1,500.0 psi	Ebend- xx	1,900.0 ksi
		Fc - Prll	1,700.0 psi	Eminbend - xx	690.0 ksi
Wood Species	: Douglas Fir - Larch	Fc - Perp	625.0 psi		
Wood Grade	: Select structural	Fv	180.0 psi		
		Ft	1,000.0 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.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 3.250 ft

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	**1.003**: 1	Maximum Shear Stress Ratio	=	**0.310** : 1
Section used for this span		**4x12**	Section used for this span		**4x12**
fb : Actual	=	1,902.67 psi	fv : Actual	=	64.10 psi
FB : Allowable	=	1,897.50 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	13.000 ft	Location of maximum on span	=	25.146 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	1.065 in	Ratio =	292 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	1.816 in	Ratio =	171 <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 = 26.0 ft	1	0.530	0.164	0.90	1.100	1.00	1.00	1.00	1.00	1.00	4.84	786.74	1485.00	0.70	26.50	162.00
+D+L+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.477	0.147	1.00	1.100	1.00	1.00	1.00	1.00	1.00	4.84	786.74	1650.00	0.70	26.50	180.00
+D+Lr+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.381	0.118	1.25	1.100	1.00	1.00	1.00	1.00	1.00	4.84	786.74	2062.50	0.70	26.50	225.00
+D+S+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	1.003	0.310	1.15	1.100	1.00	1.00	1.00	1.00	1.00	11.71	1,902.67	1897.50	1.68	64.10	207.00
+D+0.750Lr+0.750L+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Lic. # : KW-06010783 **Licensee : WRK Engineers**

Description : 4x12 at 3'-3" oc

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
Length = 26.0 ft	1	0.381	0.118	1.25	1.100	1.00	1.00	1.00	1.00	1.00	4.84	786.74	2062.50	0.70	26.50	225.00
+D+0.750L+0.750S+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.856	0.264	1.15	1.100	1.00	1.00	1.00	1.00	1.00	9.99	1,623.69	1897.50	1.44	54.70	207.00
+D+0.60W+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.298	0.092	1.60	1.100	1.00	1.00	1.00	1.00	1.00	4.84	786.74	2640.00	0.70	26.50	288.00
+D+0.70E+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.298	0.092	1.60	1.100	1.00	1.00	1.00	1.00	1.00	4.84	786.74	2640.00	0.70	26.50	288.00
+D+0.750Lr+0.750L+0.450W+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.298	0.092	1.60	1.100	1.00	1.00	1.00	1.00	1.00	4.84	786.74	2640.00	0.70	26.50	288.00
+D+0.750L+0.750S+0.450W+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.615	0.190	1.60	1.100	1.00	1.00	1.00	1.00	1.00	9.99	1,623.69	2640.00	1.44	54.70	288.00
+D+0.750L+0.750S+0.5250E+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.615	0.190	1.60	1.100	1.00	1.00	1.00	1.00	1.00	9.99	1,623.69	2640.00	1.44	54.70	288.00
+0.60D+0.60W+0.60H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.179	0.055	1.60	1.100	1.00	1.00	1.00	1.00	1.00	2.90	472.04	2640.00	0.42	15.90	288.00
+0.60D+0.70E+0.60H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.179	0.055	1.60	1.100	1.00	1.00	1.00	1.00	1.00	2.90	472.04	2640.00	0.42	15.90	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	1.8157	13.095		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	1.801	1.801
Overall MINimum	1.056	1.056
+D+H	0.745	0.745
+D+L+H	0.745	0.745
+D+Lr+H	0.745	0.745
+D+S+H	1.801	1.801
+D+0.750Lr+0.750L+H	0.745	0.745
+D+0.750L+0.750S+H	1.537	1.537
+D+0.60W+H	0.745	0.745
+D+0.70E+H	0.745	0.745
+D+0.750Lr+0.750L+0.450W+H	0.745	0.745
+D+0.750L+0.750S+0.450W+H	1.537	1.537
+D+0.750L+0.750S+0.5250E+H	1.537	1.537
+0.60D+0.60W+0.60H	0.447	0.447
+0.60D+0.70E+0.60H	0.447	0.447
D Only	0.745	0.745
Lr Only		
L Only		
S Only	1.056	1.056
W Only		
E Only		
H Only		

Wood Beam

Description : 4x12 w/ sistered 2x12

CODE REFERENCES

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

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-10

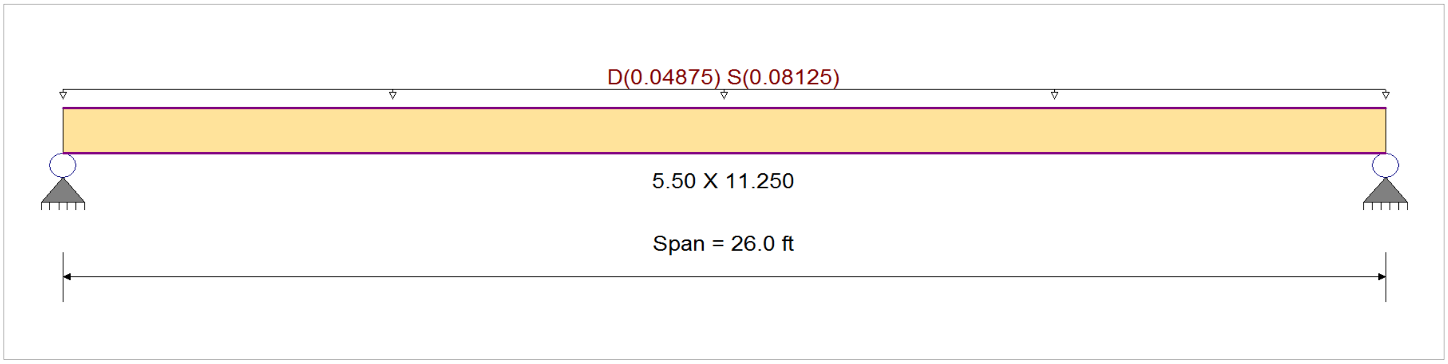
Wood Species : Douglas Fir - Larch
Wood Grade : No.1

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

Fb +	1000	psi
Fb -	1000	psi
Fc - Prll	1500	psi
Fc - Perp	625	psi
Fv	180	psi
Ft	675	psi

E : Modulus of Elasticity

Ebend- xx	1700	ksi
Eminbend - xx	620	ksi
Density	31.2	pcf



Applied Loads

Service loads entered. Load Factors will be applied for calculations.

Beam self weight calculated and added to loads
Uniform Load : D = 0.0150, S = 0.0250 ksf, Tributary Width = 3.250 ft

DESIGN SUMMARY

Design OK

Maximum Bending Stress Ratio	=	**0.991** : 1	Maximum Shear Stress Ratio	=	**0.204** : 1
Section used for this span		**5.50 X 11.250**	Section used for this span		**5.50 X 11.250**
fb : Actual	=	1,253.40 psi	fv : Actual	=	42.23 psi
FB : Allowable	=	1,265.00 psi	Fv : Allowable	=	207.00 psi
Load Combination		+D+S+H	Load Combination		+D+S+H
Location of maximum on span	=	13.000 ft	Location of maximum on span	=	25.146 ft
Span # where maximum occurs	=	Span # 1	Span # where maximum occurs	=	Span # 1

Maximum Deflection

Max Downward Transient Deflection	0.757 in	Ratio =	411 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	1.337 in	Ratio =	233 >=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 = 26.0 ft	1	0.549	0.113	0.90	1.100	1.00	1.00	1.00	1.00	1.00	5.25	543.26	990.00	0.75	18.30	162.00
+D+L+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.494	0.102	1.00	1.100	1.00	1.00	1.00	1.00	1.00	5.25	543.26	1100.00	0.75	18.30	180.00
+D+Lr+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.395	0.081	1.25	1.100	1.00	1.00	1.00	1.00	1.00	5.25	543.26	1375.00	0.75	18.30	225.00
+D+S+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.991	0.204	1.15	1.100	1.00	1.00	1.00	1.00	1.00	12.12	1,253.40	1265.00	1.74	42.23	207.00
+D+0.750Lr+0.750L+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00

Wood Beam

Description : 4x12 w/ sistered 2x12

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
Length = 26.0 ft	1	0.395	0.081	1.25	1.100	1.00	1.00	1.00	1.00	1.00	5.25	543.26	1375.00	0.75	18.30	225.00
+D+0.750L+0.750S+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.850	0.175	1.15	1.100	1.00	1.00	1.00	1.00	1.00	10.40	1,075.86	1265.00	1.50	36.24	207.00
+D+0.60W+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.309	0.064	1.60	1.100	1.00	1.00	1.00	1.00	1.00	5.25	543.26	1760.00	0.75	18.30	288.00
+D+0.70E+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.309	0.064	1.60	1.100	1.00	1.00	1.00	1.00	1.00	5.25	543.26	1760.00	0.75	18.30	288.00
+D+0.750Lr+0.750L+0.450W+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.309	0.064	1.60	1.100	1.00	1.00	1.00	1.00	1.00	5.25	543.26	1760.00	0.75	18.30	288.00
+D+0.750L+0.750S+0.450W+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.611	0.126	1.60	1.100	1.00	1.00	1.00	1.00	1.00	10.40	1,075.86	1760.00	1.50	36.24	288.00
+D+0.750L+0.750S+0.5250E+H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.611	0.126	1.60	1.100	1.00	1.00	1.00	1.00	1.00	10.40	1,075.86	1760.00	1.50	36.24	288.00
+0.60D+0.60W+0.60H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.185	0.038	1.60	1.100	1.00	1.00	1.00	1.00	1.00	3.15	325.95	1760.00	0.45	10.98	288.00
+0.60D+0.70E+0.60H					1.100	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 26.0 ft	1	0.185	0.038	1.60	1.100	1.00	1.00	1.00	1.00	1.00	3.15	325.95	1760.00	0.45	10.98	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+S+H	1	1.3369	13.095		0.0000	0.000

Vertical Reactions

Support notation : Far left is #1 Values in KIPS

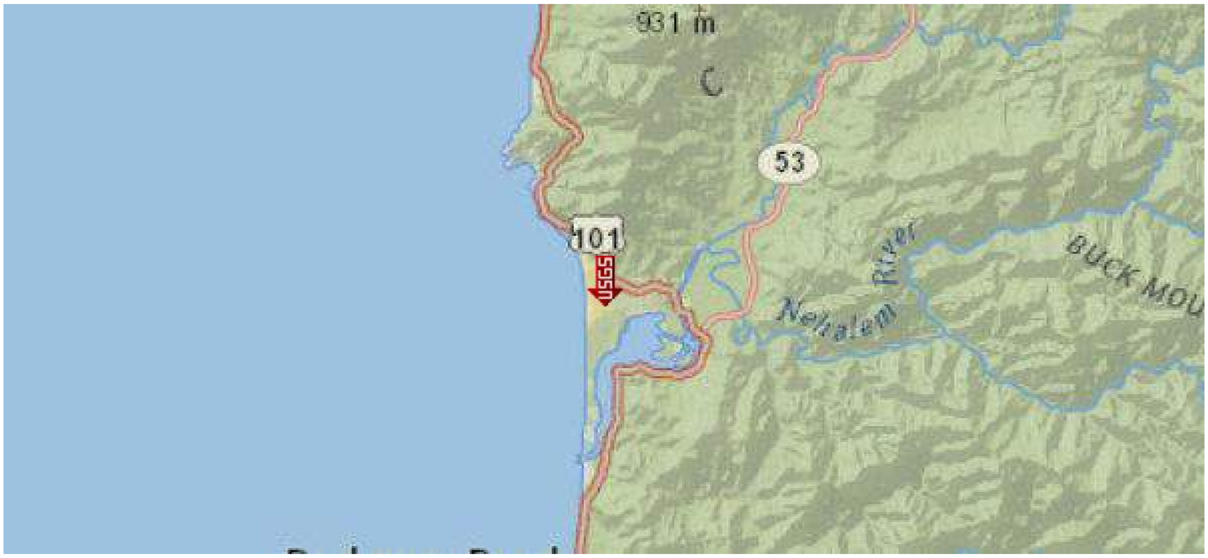
Load Combination	Support 1	Support 2
Overall MAXimum	1.864	1.864
Overall MINimum	1.056	1.056
+D+H	0.808	0.808
+D+L+H	0.808	0.808
+D+Lr+H	0.808	0.808
+D+S+H	1.864	1.864
+D+0.750Lr+0.750L+H	0.808	0.808
+D+0.750L+0.750S+H	1.600	1.600
+D+0.60W+H	0.808	0.808
+D+0.70E+H	0.808	0.808
+D+0.750Lr+0.750L+0.450W+H	0.808	0.808
+D+0.750L+0.750S+0.450W+H	1.600	1.600
+D+0.750L+0.750S+0.5250E+H	1.600	1.600
+0.60D+0.60W+0.60H	0.485	0.485
+0.60D+0.70E+0.60H	0.485	0.485
D Only	0.808	0.808
Lr Only		
L Only		
S Only	1.056	1.056
W Only		
E Only		
H Only		

II. Quonset Hut

Design Maps Summary Report

User–Specified Input

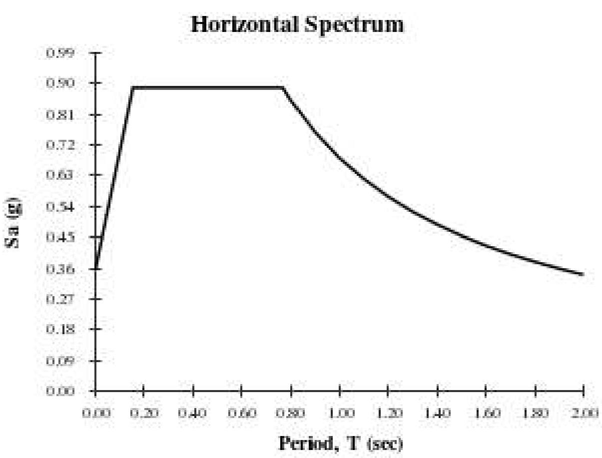
Report Title	City of Manzanita School Evaluation Fri September 28, 2018 15:20:08 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-1N (which utilizes USGS hazard data available in 2008)
Site Coordinates	45.7207°N, 123.93061°W
Site Soil Classification	Site Class D – "Stiff Soil"

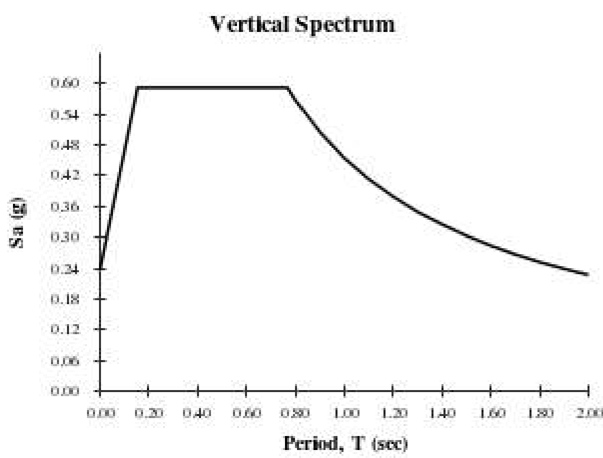


USGS–Provided Output

$S_{XS,BSE-1N}$ 0.888 g

$S_{X1,BSE-1N}$ 0.681 g



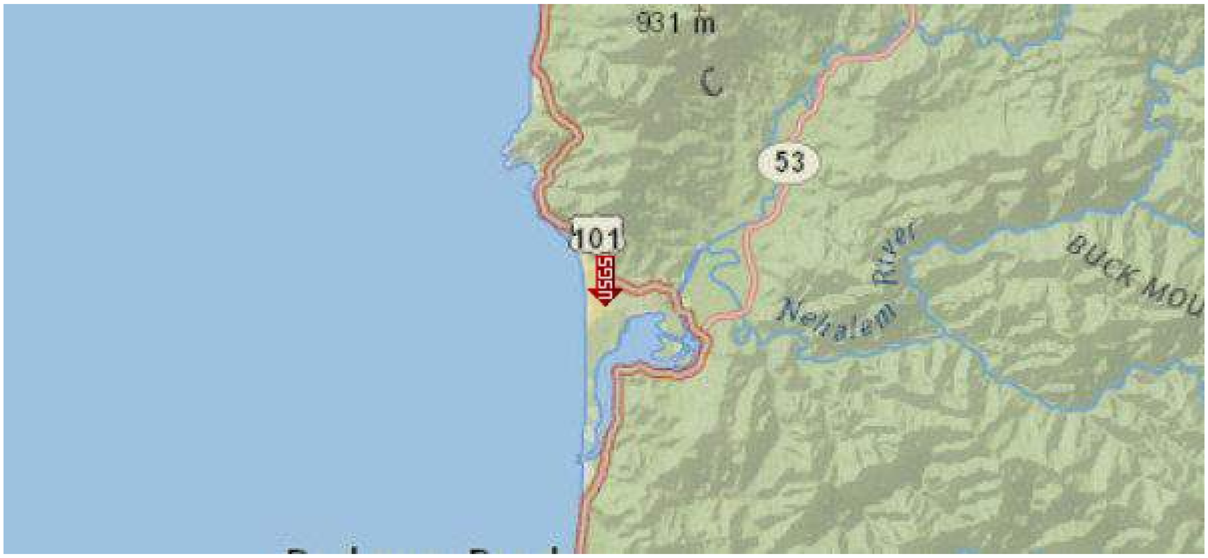


Design Maps Summary Report

FOR TIER 1 SCREENING ONLY

User–Specified Input

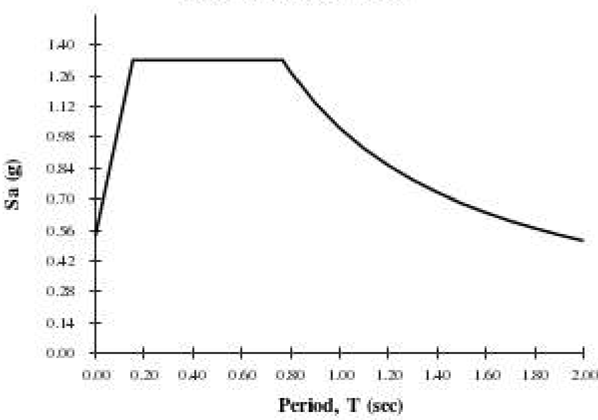
Report Title	City of Manzanita School Evaluation Fri September 28, 2018 15:21:37 UTC
Building Code Reference Document	ASCE 41-13 Retrofit Standard, BSE-2N (which utilizes USGS hazard data available in 2008)
Site Coordinates	45.7207°N, 123.93061°W
Site Soil Classification	Site Class D – "Stiff Soil"



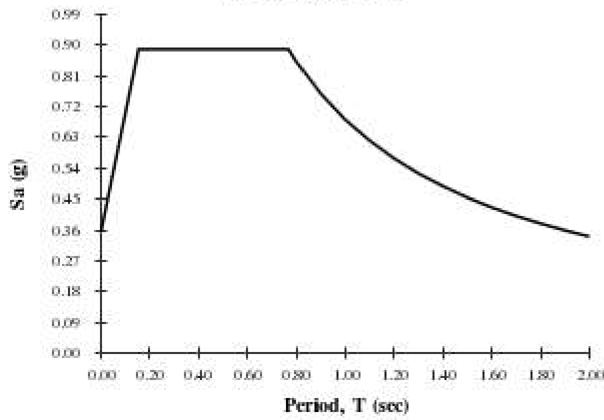
USGS–Provided Output

$S_{S,BSE-2N}$	1.332 g	$S_{XS,BSE-2N}$	1.332 g
$S_{1,BSE-2N}$	0.681 g	$S_{X1,BSE-2N}$	1.022 g

Horizontal Spectrum



Vertical Spectrum



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SUBJECT: Quonset Hut Load Takeoff	Project No. 18111.00	Date: 10/01/18
PROJECT: City of Manzanita Structural Evaluation	Design: KS	Section:
	Checked:	Page: of

Quonset Hut

Material	Qty	Tributary	Units	Weight	Units	Total Weight (kips)
Roof						
Metal Deck	1	3040	sf	3	psf	9.1
Purlins	1	3040	sf	3	psf	9.1
Glulam Arches	1	3040	sf	4	psf	12.2
Misc	1	3040	sf	2	psf	6.1
Canopy						
Walls						
East/West Wall						
Wood Walls	1	252	sf	10.0	psf	2.5
North/South Wall						
Wood Walls	1	1197	sf	10.0	psf	12.0
				Total Load =		**51.0** kips

Base Shear

Floor	Incr. Ht. ft.	Height ft	Elevation ft	Diaphragm Seis Weight kips	N-S Wall Seis Weight kips	E-W Wall Seis Weight kips	N-S Direction Diaphragm Seis Weight kips	E-W Direction Diaphragm Seis Weight kips
Roof	21.00	21.00	21.00	36	12	3	48	39
Summation:				36	12	3	48	39

Seismic Design Parameters (site Specific):

	BSE-1N
Sxs =	0.888
Sx1 =	0.681
Cm =	1
C_1 =	1.4
C_2 =	1

Approximate Code Period :

	BSE-1N	
B =	0.75	
Ct =	0.020	
T_L =	12.00	
To =	0.153	
Ts =	0.767	
Period Used to Estimate Base Shear; T =	**0.196**	seconds
k =	1.000	
b =	0.050	
B_1 =	1.002	
Response Spectrum Acceleration, Sa =	**0.886**	
psuedo lateral force, $V = C_1C_2C_mS_a$ W =	**1.240**	W

E-W Direction OUTPUT

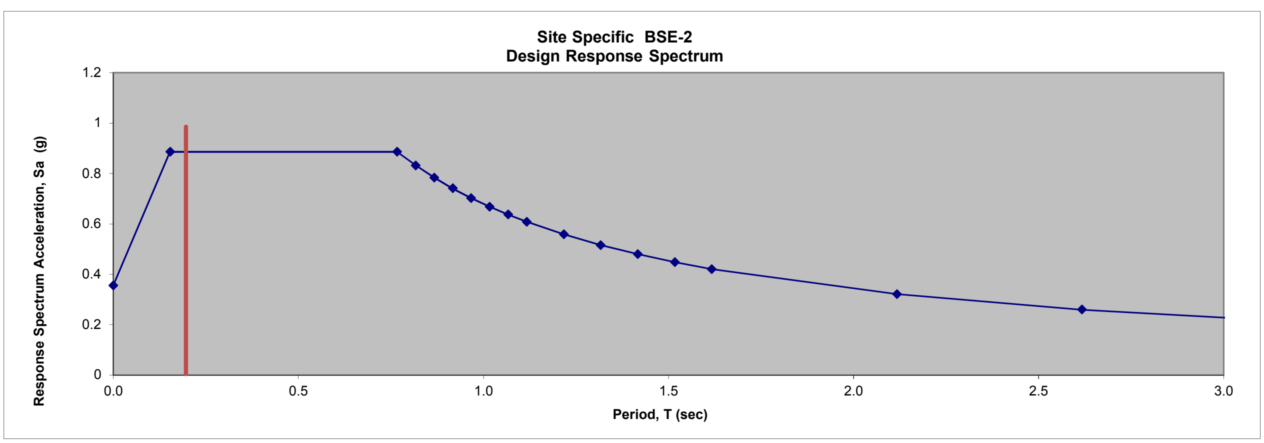
Total Weight kips	wih_i^k kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
51	1,070	1.000	63	1.24	39	48
51	1,070	1.000	63			

psuedo lateral force, V= 63 kips

N-S Direction OUTPUT

Total Weight kips	wih_i^k kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
51	1,070	1.000	63	1.24	48	60
51	1,070	1.000	63			

psuedo lateral force, V= 63 kips



Storage

Material	Qty	Tributary	Units	Weight	Units	Total Weight (kips)
Roof						
Metal Deck	1	960	sf	3	psf	2.9
Particle Board	1	960	sf	2	psf	1.9
Pre-manufactured Trusses	1	960	sf	4	psf	3.8
Misc	1	960	sf	3	psf	2.9
Canopy						
Walls						
East/West Wall						
Wood Walls	1	360	sf	12.0	psf	4.3
North/South Wall						
Wood Walls	1	576	sf	12.0	psf	6.9

Total Load = **22.8** **kips**

Base Shear

Floor	Incr. Ht. ft.	Height ft	Elevation ft	Diaphragm Seis Weight kips	N-S Wall Seis Weight kips	E-W Wall Seis Weight kips	N-S Direction Diaphragm Seis Weight kips	E-W Direction Diaphragm Seis Weight kips
Roof	13.00	13.00	13.00	12	7	4	18	16
Summation:				12	7	4	18	16

Seismic Design Parameters (site Specific):

	BSE-1N
Sxs =	0.888
Sx1 =	0.681
Cm =	1
C_1 =	1.4
C_2 =	1

Approximate Code Period :

	BSE-1N	
B =	0.75	
Ct =	0.020	
T_L =	12.00	
To =	0.153	
Ts =	0.767	
Period Used to Estimate Base Shear; T =	**0.137**	seconds
k =	1.000	
b =	0.050	
B_1 =	1.002	
Response Spectrum Acceleration, Sa =	**0.829**	
psuedo lateral force, $V = C_1C_2C_mS_a$ W =	**1.161**	W

E-W Direction OUTPUT

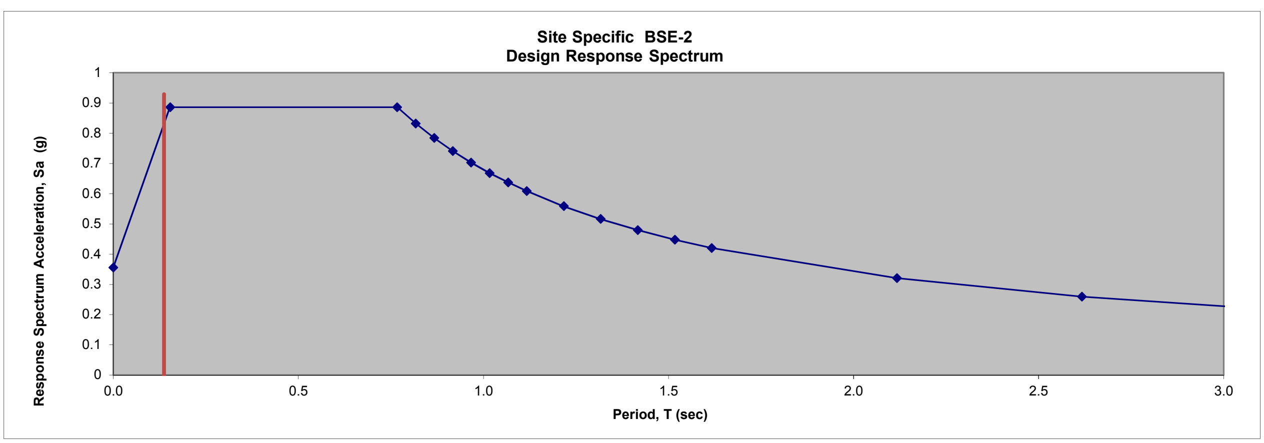
Total Weight kips	wihi^k kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
23	296	1.000	26	1.16	16	18
23	296	1.000	26			

psuedo lateral force, V= **26** **kips**

N-S Direction OUTPUT

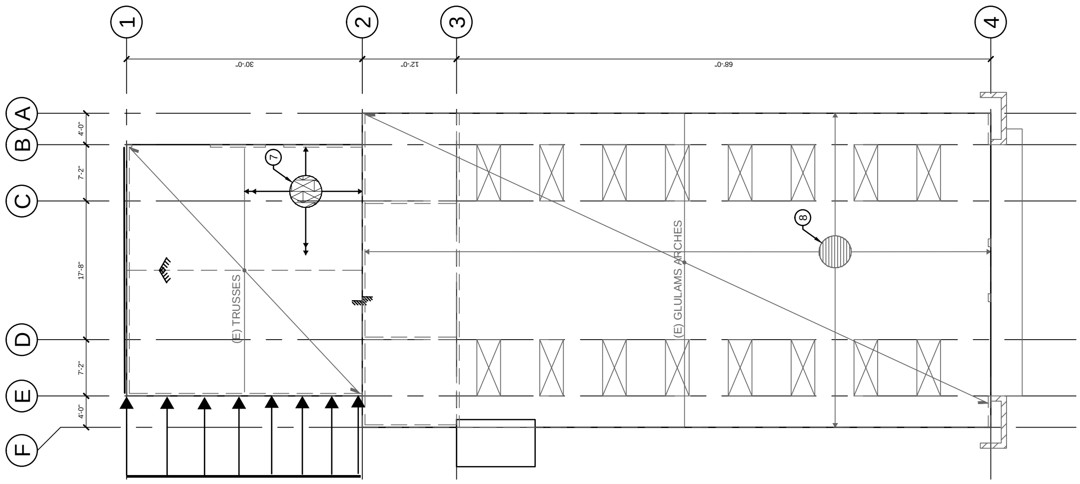
Total Weight kips	wihi^k kip-ft	Cvx	Story Force (Fx) kips	$\frac{\Sigma F_i}{\Sigma w_i}$	Diaph. Seis. Weight (wpx) kips	Diaphragm Force (Fpx) kips
23	296	1.000	26	1.16	18	21
23	296	1.000	26			

psuedo lateral force, V= **26** **kips**



DIAPHRAGM DESIGN - Lower Roof

East/West Loading

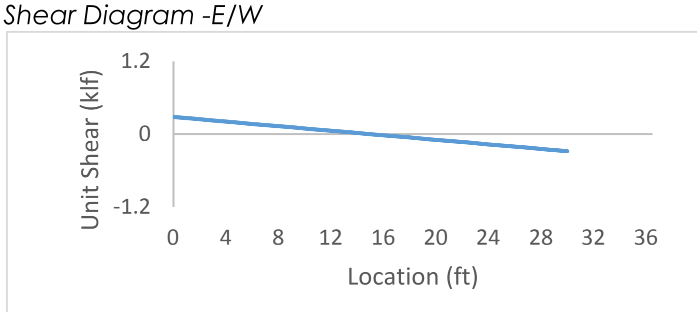


Diagrams Loading

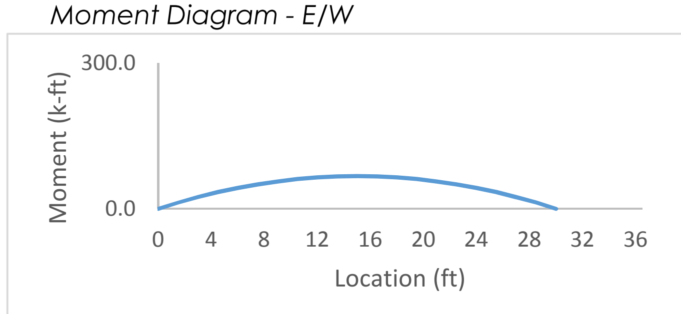
$$F_{px} = \frac{\Sigma F_i}{\Sigma w_i} w_{px}$$ *ASCE 41-17 Eqn. 7-26*

F_{px} =	18	kips	*See seismic loading for the Quonset Hut*
L =	30	ft	*Diaphragm span*
B =	32	ft	*Diaphragm depth*
E =	0.60	klf	*Distributed Load [F $_{px}$ / L]*

Shear Diagram -E/W



Moment Diagram - E/W



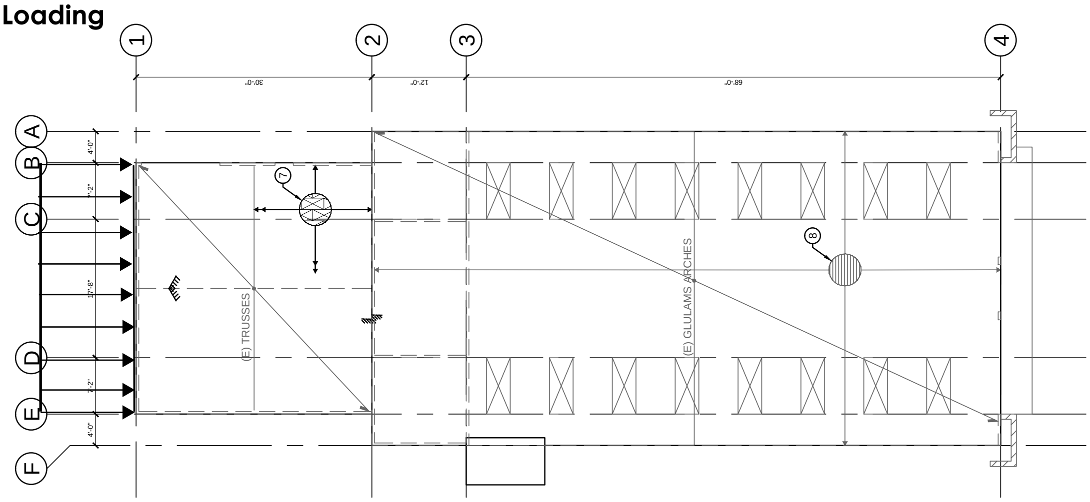
Diaphragm Shear

Q_{UD} =	0.28	klf	*Applied Unit Shear*
m =	2.5		*ASCE 41-17 Tbl. 12-3*
κ =	0.75		*ASCE 41-17 Sect. 6.2.4*
Q_{CE} =	1.155	klf	*Expected Strength*
mκQ_{CE} =	2.16563	klf	*Design Strength*
DCR =	0.13	< 1.0, OK	

GOVERNING CASE
USE 1/2" SHTHG WITH 10d AT 4" OC
AT EDGES AND 12" OC IN THE FIELD

DIAPHRAGM DESIGN - Upper Roof

North/South Loading

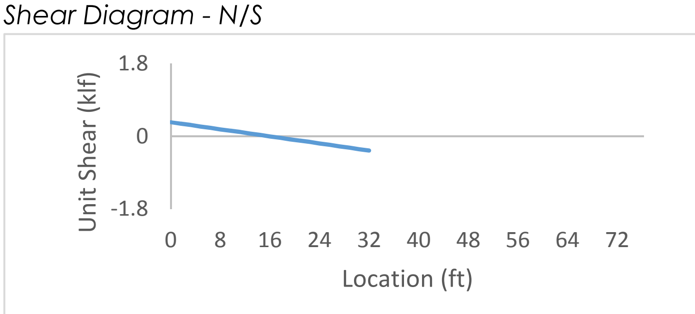


Diagrhragm Loading

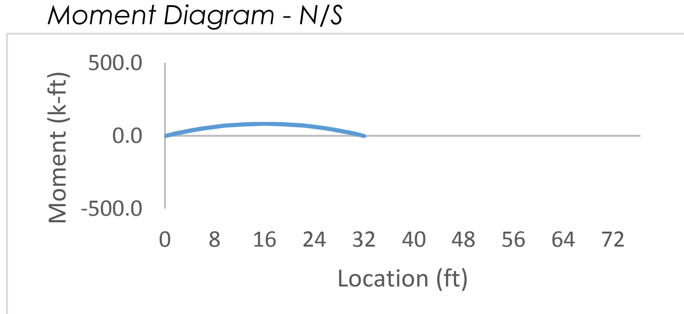
$$F_{px} = \frac{\Sigma F_i}{\Sigma w_i} w_{px}$$ *ASCE 41-17 Eqn. 7-26*

F_{px} =	21	kips	*See seismic loading for the original building*
L =	32	ft	*Diaphragm span*
B =	30	ft	*Diaphragm depth*
E =	0.66	klf	*Distributed Load [F $_{px}$ / L]*

Shear Diagram - N/S



Moment Diagram - N/S



Diaphragm Shear

Q_{UD} =	0.35	klf	*Applied Unit Shear*
m =	2.5		*ASCE 41-17 Tbl. 12-3*
κ =	0.75		*ASCE 41-17 Sect. 6.2.4*
Q_{CE} =	1.155	klf	*Expected Strength*
$m\kappa Q_{CE}$ =	2.17	klf	*Design Strength*
DCR =	0.16	< 1.0, OK	

EAST/WEST LOADING CONTROLS
USE 1/2" SHTHG WITH 10d AT 4" OC
AT EDGES AND 12" OC IN THE FIELD

wrk engineers

SUBJECT: Wind Loading - Quonset Hut N/S Direction

PROJECT: City of Manzanita School Structural Evaluation

Project No. 18111.00 Date:

Design: KS Section:

Checked: Page:

27 Directional Procedure, Part 1:Enclosed and Partially Enclosed Rigid Buildings. (All Heights)

27.4. MWFRS

Velocity pressure $q_z = .00256\ K_z\ K_{zt}\ K_d\ V^2$ (27.3-1)

Exposure B Roof Height h = 21 feet

Roof Pitch = 0.00 :12

Exposure coefficient K_z = Section 27.3.1, shall be determined from Table 27.3-1

Topography factor K_{zt} = 1.00 26.8.2, Figure 26.8-1

Directionality factor K_d = 0.85 26.6, Table 26.6-1

Building & Structure Risk Category = II, standard IBC T-1604.5

Wind Speed V = 135 mph

q_z = 39.66 K_z psf

Internal Pressure Coefficient (GC_{pi}) = ± 0.18 Table 26.11-1, for Enclosed Buildi

Gust effect factor G = 0.85 26.9

Pressures for MWFRS p = $qGC_p - q_i\ (GC_{pi})$ (27.4-1)

Wall and Roof External pressure Coefficients C_p from Fig. 27.4-1

Wind Normal to Ridge (┴ to 38) L/B = 0.48 h/L = 21/38 = 0.55 θ= 0.0

Windward wall C_p = 0.80 Windward roof C_p =

Leeward wall C_p = -0.50 for L/B = 0.48 Leeward roof C_p =

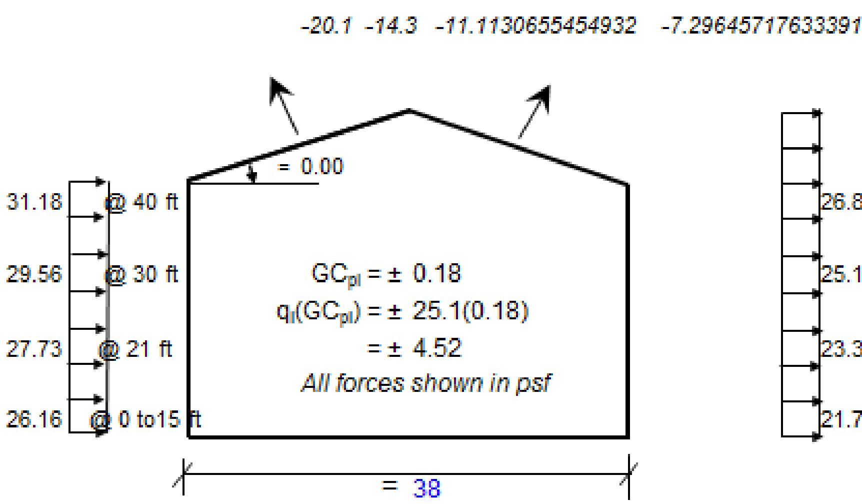
Side wall C_p = -0.70 or Roof C_p = -0.94 -0.67 -0.52

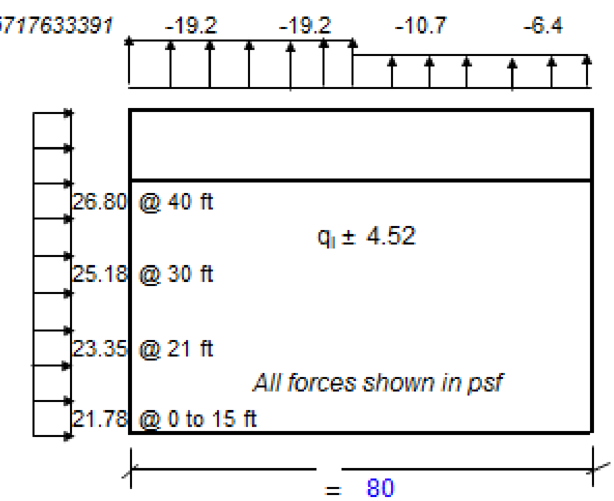
Wind Parallel to Ridge (┴ to 80) L/B = 2.11

Windward wall C_p = 0.80 h/L = 21/80 = 0.26

Leeward wall C_p = -0.29 for L/B = 2.11 Roof C_p = -0.90 -0.90 -0.50

Side wall C_p = -0.70 for dist 0 11 21





wrk engineers

SUBJECT: QUONSET HUT - WIND LOAD.

PROJECT: City of Manzanita School Structural Evaluation

Project No. 18111.00

Design: KS

Checked:

Date:

Section:

Page:

ASCE 7-10

ARCHED ROOF:

$r = 21\text{ft} / 38\text{ft} = 0.55$

FIGURE 27.4-3 $\Rightarrow C_p = 1.4r = 0.77$

$p = q G C_p - q_i (G C_{pi})$ (27.4-1)

$q = 0.00256 K_z K_{zt} K_d V^2$

$= 0.00256(0.62)(1.0)(0.85)(135)^2 = 24.6$ psf

$GC_{pi} = \pm 0.18$

$G = 0.85$

$p = 24.6\text{psf}\ (0.85 * 0.77 + 0.18) = 20.5$ psf

$V_{E\text{-}W} = (20.5\text{PSF})(21\text{FT}/2)(80\text{FT}) = 17.2$ KIPS

$\Rightarrow$ WIND SHEAR << SEISMIC SHEAR. $\Rightarrow$ SEISMIC CONTROLS

$V_{N\text{-}S} = (23.35\text{ PSF})(21\text{FT}/2)(38\text{FT}) = 9.3$ KIPS

$\Rightarrow$ SEISMIC CONTROLS

SHEAR FORCE DISTRIBUTION

LINE 1 AND 2:

$V_{SEISMIC} = 26^{KIPS}/2 = 13$ KIPS

$Q_{UD} = (13\text{ KIPS})/32.0\text{ FT} = 406$ PLF

$mKQ_{CE} = (3.8)(0.75)(1.5 * 520\text{ PLF}) = 2223$ PLF ⇒ DCR = 0.18 ✓

⇒ SHEAR WALL TYPE A

$M_{OT} = (13\text{ KIPS})(12\text{ FT}) = 156$ K-FT

DEAD LOAD RESISTING = (12 PSF)(32 FT)(1 FT) + (12 PSF)(32 FT)(12 FT) = 4,992 LBS

$M_S = \dfrac{(4{,}992\text{ LBS})(32\text{ FT}/2)}{1000} = 79.8$ K-FT

ACCEPTANCE:

$\dfrac{M_{OT}}{J C_1 C_2} < 0.9 M_S \Rightarrow DCR = \dfrac{(156\text{ K-FT})/(8 * 1.4)}{0.9(79.8\text{ K-FT})} = 0.19 < 1.0$ ⇒ NO HOLD DOWNS.

LINE B:

$Q_{UD} = 13\text{ KIPS}/17\text{ FT} = 765\text{ PLF} < 2223$ PLF ⇒ SHEAR WALL TYPE A OK

7' PIER:

$M_{OT} = (765\text{ PLF})(7\text{ FT})(12\text{ FT}) = 64.3$ K-FT

DEAD LOAD RESISTING = (12 PSF)(7 FT)(16 FT) + (12 PSF)(7 FT)(12 FT) = 2352 LBS

$M_S = \dfrac{(2352\text{ LBS})(7\text{ FT}/2)}{1000} = 8.2$ K-FT.

$DCR = \dfrac{(64.3\text{ K-FT})/(8 * 1.4)}{0.9(8.2\text{ K-FT})} = 0.78$ ⇒ NO HOLD DOWN REQUIRED

COLLECTOR

765 PLF < 770 PLF (1/2" PLYWOOD w/ 10d @ 4" o.c.) ⇒ NO COLLECTOR REQUIRED

⇒ DIAPHRAGM AND CHORDS OK BY COMPARISON TO SCHOOL BUILDING.

wrk engineers

SUBJECT: GLULAM ARCH CONNECTION

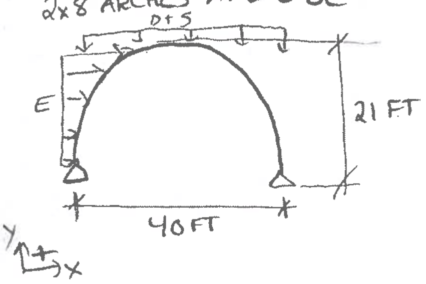
PROJECT: City of Manzanita School Structural Evaluation

Project No. 18111.00 **Design:** KS **Checked:**

Date: 10/04/18 **Section:** **Page:**

GLULAM ARCH

2x8 ARCHES AT 2'-0" OC



	D	S	E
R_x:	206#	430#	-1613#
R_y:	456#	950#	-791#

GRAVITY LOADING

DEAD: (2 FT)(12 PSF) = 24 PLF (PROJECTED)

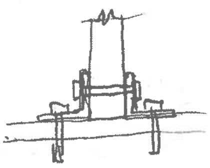
SNOW: (2 FT)(25 PSF) = 50 PLF (PROJECTED)

SEISMIC LOADING

(63 KIPS)/80 FT = 788 PLF

(788 PLF)(2 FT) = 1576 # TO EACH ARCH

CONNECTION CHECK



THRU-BOLT: 1/2" BOLT

$m = 2.8$

$Q_{UD} = Q_G + Q_E$ (ASCE 41-17 EQ 7-34)

$Q_E = 1613^{\#}$

$Q_G = 0.9 Q_D = 0.9(206^{\#})$ (EQ 7-2)

$Q_{UD}: 1428^{\#}$

$Q_{CE} = (1.5) Z' \Rightarrow Z = 790^{\#}$ (NDS 2015 TABLE 12I)

$Z' = (3.32)(1.0)(1.0)(790^{\#}) : 2683^{\#}$

$mKQ_{CE} = (2.8)(0.75)(1.5)(2683^{\#}) = 8451^{\#} > 1428^{\#}$ ✓OK

BOLT TO FOOTING:

SEE HILTI CALCULATION.

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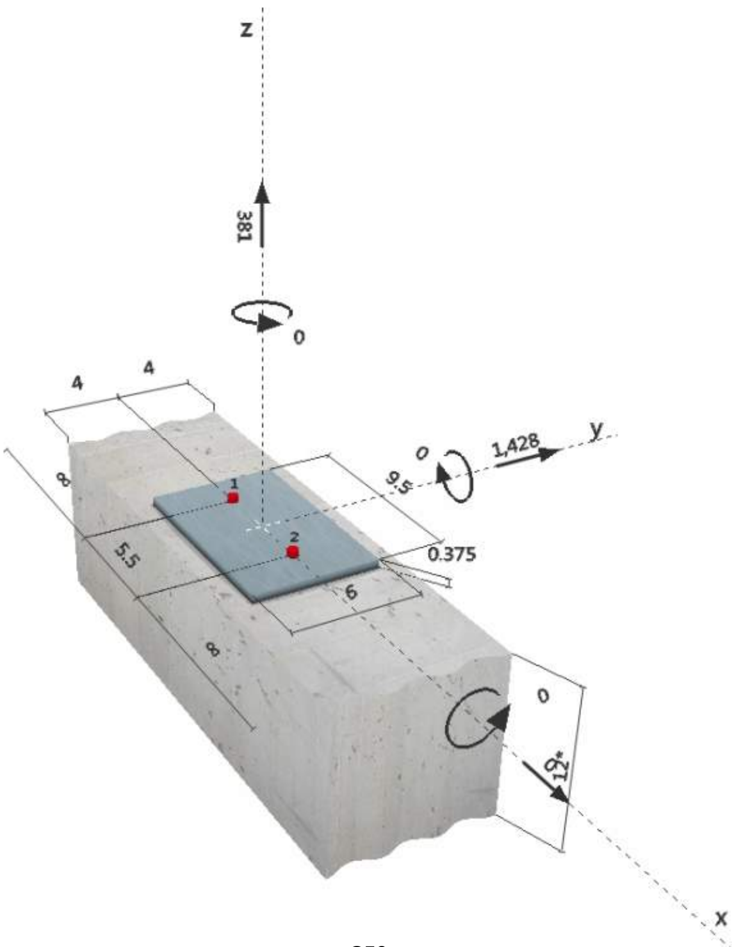
Specifier's comments:

1 Input data

Anchor type and diameter:	**KWIK HUS-EZ (KH-EZ) 1/2 (2 1/4)**
Effective embedment depth:	$h_{ef,act}$ = 1.520 in., h_{nom} = 2.250 in.
Material:	Carbon Steel
Evaluation Service Report:	ESR-3027
Issued I Valid:	2/1/2016 | 12/1/2017
Proof:	Design method ACI 318-14 / Mech.
Stand-off installation:	e_b = 0.000 in. (no stand-off); t = 0.375 in.
Anchor plate:	l_x x l_y x t = 9.500 in. x 6.000 in. x 0.375 in.; (Recommended plate thickness: not calculated
Profile:	no profile
Base material:	cracked concrete, 2500, f_c' = 2,500 psi; h = 12.000 in.
Installation:	**hammer drilled hole, Installation condition: Dry**
Reinforcement:	tension: condition B, shear: condition B; no supplemental splitting reinforcement present edge reinforcement: none or < No. 4 bar

Geometry [in.] & Loading [lb, in.lb]



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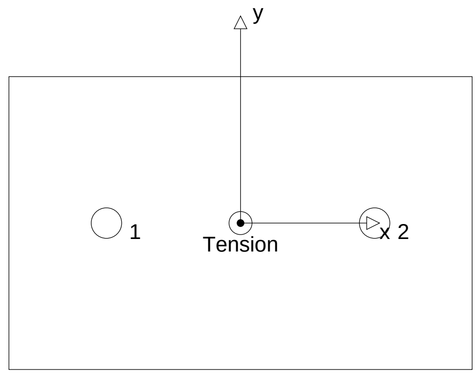
2 Load case/Resulting anchor forces



Load case: Design loads

Anchor reactions [lb]
Tension force: (+Tension, -Compression)

Anchor	Tension force	Shear force	Shear force x	Shear force y
1	190	714	0	714
2	190	714	0	714

max. concrete compressive strain: - [‰]
max. concrete compressive stress: - [psi]
resulting tension force in (x/y)=(0.000/0.000): 381 [lb]
resulting compression force in (x/y)=(0.000/0.000): 0 [lb]

3 Tension load

	Load N_{ua} [lb]	Capacity ϕ N_n [lb]	Utilization $\beta_N = N_{ua}/\phi N_n$	Status
Steel Strength*	190	11,778	2	OK
Pullout Strength*	N/A	N/A	N/A	N/A
Concrete Breakout Strength**	381	2,071	19	OK

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

3.1 Steel Strength

N_{sa} = ESR value refer to ICC-ES ESR-3027
$\phi\ N_{sa} \geq N_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,N}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

N_{sa} [lb]
18,120

Results

N_{sa} [lb]	ϕ_{steel}	$\phi\ N_{sa}$ [lb]	N_{ua} [lb]
18,120	0.650	11,778	190

3.2 Concrete Breakout Strength

$N_{cbg} = \left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\, \psi_{ed,N}\, \psi_{c,N}\, \psi_{cp,N}\, N_b$	ACI 318-14 Eq. (17.4.2.1b)
$\phi\ N_{cbg} \geq N_{ua}$	ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)	
$A_{Nc0} = 9\, h_{ef}^2$	ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\, e_N'}{3\, h_{ef}}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5 h_{ef}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5 h_{ef}}{c_{ac}}\right) \leq 1.0$	ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\, \lambda_a \sqrt{f_c'}\, h_{ef}^{1.5}$	ACI 318-14 Eq. (17.4.2.2a)

Variables

h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]	$\psi_{c,N}$
1.520	0.000	0.000	4.000	1.000

c_{ac} [in.]	k_c	λ_a	f_c' [psi]
2.750	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
41.59	20.79	1.000	1.000	1.000	1.000	1,593

Results

N_{cbg} [lb]	$\phi_{concrete}$	$\phi\ N_{cbg}$ [lb]	N_{ua} [lb]
3,186	0.650	2,071	381

4 Shear load

	Load V_{ua} [lb]	Capacity ϕ V_n [lb]	Utilization $\beta_V = V_{ua}/\phi V_n$	Status
Steel Strength*	714	5,547	13	OK
Steel failure (with lever arm)*	N/A	N/A	N/A	N/A
Pryout Strength**	1,428	2,230	65	OK
Concrete edge failure in direction y+**	1,428	2,524	57	OK

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

4.1 Steel Strength

V_{sa} = ESR value refer to ICC-ES ESR-3027
$\phi\ V_{steel} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

Variables

$A_{se,V}$ [in.^{2}]	f_{uta} [psi]
0.16	112,540

Calculations

V_{sa} [lb]
9,245

Results

V_{sa} [lb]	ϕ_{steel}	$\phi\ V_{sa}$ [lb]	V_{ua} [lb]
9,245	0.600	5,547	714

4.2 Pryout Strength

$V_{cpg} = k_{cp} \left[\left(\frac{A_{Nc}}{A_{Nc0}}\right) \psi_{ec,N}\ \psi_{ed,N}\ \psi_{c,N}\ \psi_{cp,N}\ N_b\right]$ ACI 318-14 Eq. (17.5.3.1b)
$\phi\ V_{cpg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1
A_{Nc} see ACI 318-14, Section 17.4.2.1, Fig. R 17.4.2.1(b)
$A_{Nc0} = 9\ h_{ef}^2$ ACI 318-14 Eq. (17.4.2.1c)
$\psi_{ec,N} = \left(\frac{1}{1 + \frac{2\ e'_N}{3\ h_{ef}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.4)
$\psi_{ed,N} = 0.7 + 0.3 \left(\frac{c_{a,min}}{1.5h_{ef}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.5b)
$\psi_{cp,N} = \text{MAX}\left(\frac{c_{a,min}}{c_{ac}}, \frac{1.5h_{ef}}{c_{ac}}\right) \leq 1.0$ ACI 318-14 Eq. (17.4.2.7b)
$N_b = k_c\ \lambda_a \sqrt{f'_c}\ h_{ef}^{1.5}$ ACI 318-14 Eq. (17.4.2.2a)

Variables

k_{cp}	h_{ef} [in.]	$e_{c1,N}$ [in.]	$e_{c2,N}$ [in.]	$c_{a,min}$ [in.]
1	1.520	0.000	0.000	4.000

$\psi_{c,N}$	c_{ac} [in.]	k_c	λ_a	f'_c [psi]
1.000	2.750	17	1.000	2,500

Calculations

A_{Nc} [in.^{2}]	A_{Nc0} [in.^{2}]	$\psi_{ec1,N}$	$\psi_{ec2,N}$	$\psi_{ed,N}$	$\psi_{cp,N}$	N_b [lb]
41.59	20.79	1.000	1.000	1.000	1.000	1,593

Results

V_{cpg} [lb]	$\phi_{concrete}$	$\phi\ V_{cpg}$ [lb]	V_{ua} [lb]
3,186	0.700	2,230	1,428

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4.3 Concrete edge failure in direction y+

$V_{cbg} = \left(\frac{A_{Vc}}{A_{Vc0}}\right) \psi_{ec,V}\, \psi_{ed,V}\, \psi_{c,V}\, \psi_{h,V}\, \psi_{parallel,V}\, V_b$ ACI 318-14 Eq. (17.5.2.1b)

$\phi\ V_{cbg} \geq V_{ua}$ ACI 318-14 Table 17.3.1.1

A_{Vc} see ACI 318-14, Section 17.5.2.1, Fig. R 17.5.2.1(b)

$A_{Vc0} = 4.5\, c_{a1}^2$ ACI 318-14 Eq. (17.5.2.1c)

$\psi_{ec,V} = \left(\frac{1}{1+\frac{2e_V'}{3c_{a1}}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.5)

$\psi_{ed,V} = 0.7 + 0.3\left(\frac{c_{a2}}{1.5c_{a1}}\right) \leq 1.0$ ACI 318-14 Eq. (17.5.2.6b)

$\psi_{h,V} = \sqrt{\frac{1.5c_{a1}}{h_a}} \geq 1.0$ ACI 318-14 Eq. (17.5.2.8)

$V_b = \left(7\left(\frac{l_e}{d_a}\right)^{0.2}\sqrt{d_a}\right) \lambda_a \sqrt{f_c'}\, c_{a1}^{1.5}$ ACI 318-14 Eq. (17.5.2.2a)

Variables

c_{a1} [in.]	c_{a2} [in.]	e_{cV} [in.]	$\psi_{c,V}$	h_a [in.]
4.000	-	0.000	1.000	12.000

l_e [in.]	λ_a	d_a [in.]	f_c' [psi]	$\psi_{parallel,V}$
1.520	1.000	0.500	2,500	1.000

Calculations

A_{Vc} [in.^{2}]	A_{Vc0} [in.^{2}]	$\psi_{ec,V}$	$\psi_{ed,V}$	$\psi_{h,V}$	V_b [lb]
105.00	72.00	1.000	1.000	1.000	2,473

Results

V_{cbg} [lb]	$\phi_{concrete}$	$\phi\ V_{cbg}$ [lb]	V_{ua} [lb]
3,606	0.700	2,524	1,428

5 Combined tension and shear loads

β_N	β_V	ζ	Utilization $\beta_{N,V}$ [%]	Status
0.184	0.640	5/3	54	OK

$\beta_{NV} = \beta_N^{\zeta} + \beta_V^{\zeta} <= 1$

6 Warnings

- The anchor design methods in PROFIS Anchor require rigid anchor plates per current regulations (ETAG 001/Annex C, EOTA TR029, etc.). This means load re-distribution on the anchors due to elastic deformations of the anchor plate are not considered - the anchor plate is assumed to be sufficiently stiff, in order not to be deformed when subjected to the design loading. PROFIS Anchor calculates the minimum required anchor plate thickness with FEM to limit the stress of the anchor plate based on the assumptions explained above. The proof if the rigid base plate assumption is valid is not carried out by PROFIS Anchor. Input data and results must be checked for agreement with the existing conditions and for plausibility!
- Condition A applies when supplementary reinforcement is used. The Φ factor is increased for non-steel Design Strengths except Pullout Strength and Pryout strength. Condition B applies when supplementary reinforcement is not used and for Pullout Strength and Pryout Strength. Refer to your local standard.
- Refer to the manufacturer's product literature for cleaning and installation instructions.
- Checking the transfer of loads into the base material and the shear resistance are required in accordance with ACI 318 or the relevant standard!
- Hilti post-installed anchors shall be installed in accordance with the Hilti Manufacturer's Printed Installation Instructions (MPII). Reference ACI 318-14, Section 17.8.1.

Fastening meets the design criteria!

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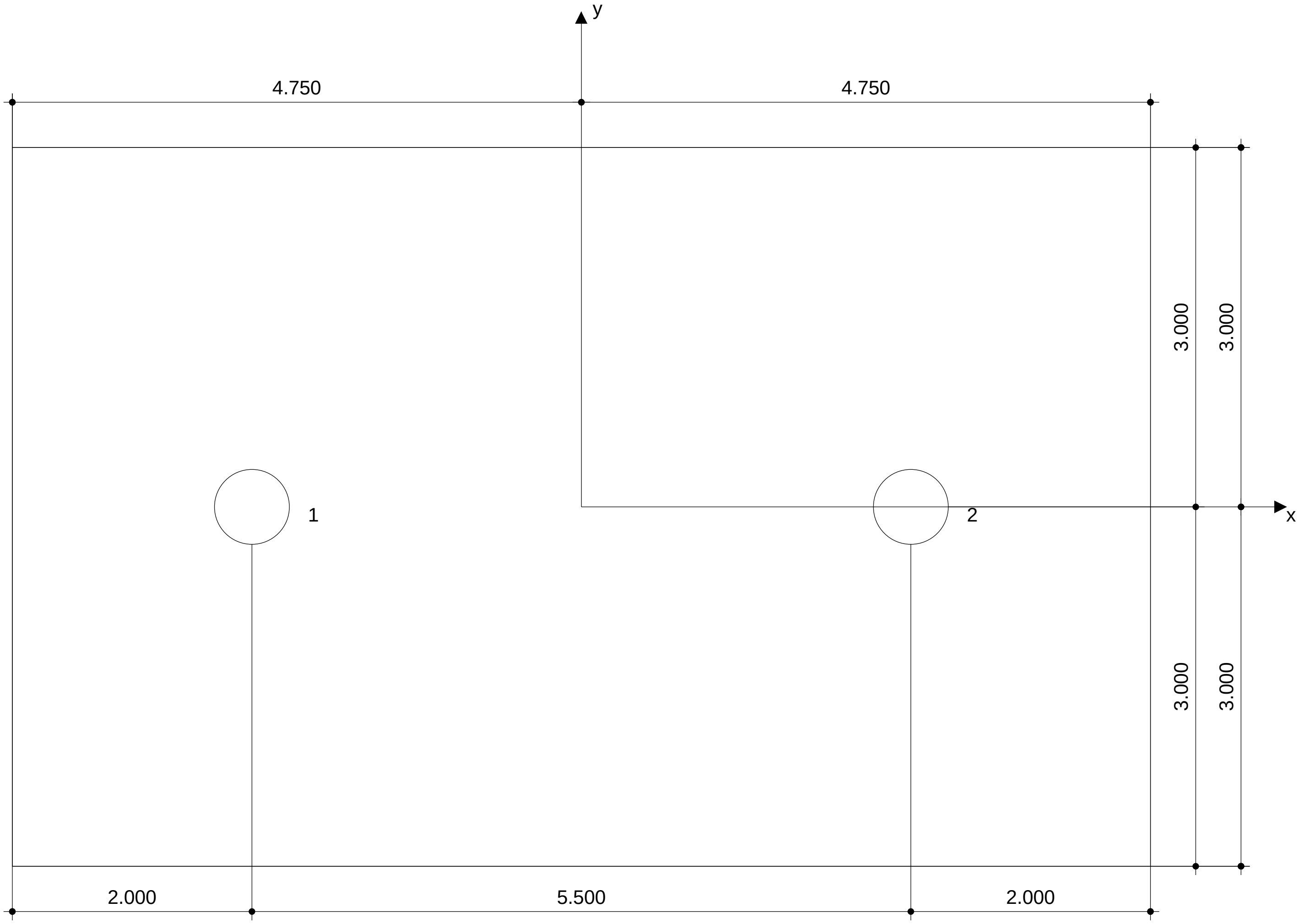
7 Installation data

Anchor plate, steel: -
Profile: no profile
Hole diameter in the fixture: d_f = 0.625 in.
Plate thickness (input): 0.375 in.
Recommended plate thickness: not calculated
Drilling method: Hammer drilled
Cleaning: Manual cleaning of the drilled hole according to instructions for use is required.

Anchor type and diameter: KWIK HUS-EZ (KH-EZ) 1/2 (2 1/4)
Installation torque: 540.001 in.lb
Hole diameter in the base material: 0.500 in.
Hole depth in the base material: 2.625 in.
Minimum thickness of the base material: 4.500 in.

7.1 Recommended accessories

Drilling	Cleaning	Setting
• Suitable Rotary Hammer	• Manual blow-out pump	• Torque wrench
• Properly sized drill bit		



Coordinates Anchor in.

Anchor	x	y	c_{-x}	c_{+x}	c_{-y}	c_{+y}
1	-2.750	0.000	-	-	4.000	4.000
2	2.750	0.000	-	-	4.000	4.000

C64

8 Remarks; Your Cooperation Duties

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Appendix D

CONCEPTUAL ESTIMATE OF PROBABLE CONSTRUCTION COSTS

I. ELEMENTARY SCHOOL STRUCTURE

CONCEPTUAL STRENGTHENING AND REMEDIATION

SUMMARY

Location	GFA SF	Cost/SF	Total Cost
A STRUCTURAL STRENGTHENING	5,478	$ 5.47	$ 29,939.81
B BUILDING CONDITION REMEDIATION	5,478	$ 46.74	$ 256,054.80
C MEP	5,478	$ 71.19	$ 390,000.00
D DEMOLITION	5,478	$ 10.75	$ 58,889.16
ESTIMATED NET COST	**5478**	**$ 134.15**	**$ 734,883.77**

MARGINS & ADJUSTMENTS

Phasing & Temporary Work	2%			$ 14,697.68
General Conditions	10%			$ 74,958.14
Bonds & Insurance	3%			$ 24,736.19
Overhead & Profit	5%			$ 42,463.79
Design Contingency	20%			$ 178,347.91
Construction Contingency	20%			$ 214,017.50
Escalation to 3Q2019	3%			$ 38,523.15
ESTIMATED NET COST		**5478**	**$ 241.44**	**$ 1,322,628.13**

GFA: 5478 SF

A STRUCTURAL STRENGTHENING

Description	Unit	Qty	Rate	Total
05 Metals				
HSS Strongbacks	Ea	11	$ 393.25	$ 4,325.75
Hold Downs	Ea	24	$ 125.16	$ 3,003.84
		$ 1.34	**/SF**	**$ 7,329.59**
06 Wood, Plastics, and Composites				
Plywood Roof Sheathing 1/2"	SF	6420	$ 1.74	$ 11,170.80
New Roof Framing	LF	277	$ 5.21	$ 1,443.17
New Wall Studs	LF	1375	$ 2.15	$ 2,956.25
Plywood Wall Sheathing 1/2"	SF	3200	$ 2.20	$ 7,040.00
		$ 4.13	**/SF**	**$ 22,610.22**
	TOTAL	**$ 5.47**	**/SF**	**$ 29,939.81**

GFA: 5478 SF

B BUILDING CONDITION REMEDIATION

Description	Unit	Qty	Rate	Total
03 Concrete				
Concrete	CY	25	$ 353.86	$ 8,846.50
		$ 1.61	**/SF**	**$ 8,846.50**
05 Metals				
5/8" Epoxy Anchor Bolts	Ea	215	$ 10.20	$ 2,193.00
		$ 0.40	**/SF**	**$ 2,193.00**
06 Wood, Plastics, and Composites				
FRP Reinforcing	SF	100	$ 93.50	$ 9,350.00
		$ 1.71	**/SF**	**$ 9,350.00**
07 Thermal and Mositure Protection				
New Roof	SF	6420	$ 12.00	$ 77,040.00
New Siding	SF	5830	$ 4.91	$ 28,625.30
		$ 19.29	**/SF**	**$ 105,665.30**
08 Openings				
Allowance for New Windows	Item			$ 100,000.00
		$ 18.25	**/SF**	**$ 100,000.00**
09 Finishings				
Allowance for Repair of Finishes	Item			$ 30,000.00
		$ 5.48	**/SF**	**$ 30,000.00**
	TOTAL	**$ 46.74**	**/SF**	**$ 256,054.80**

GFA: 5478 SF

C MEP

Description	Unit	Qty	Rate	Total
22 Plumbing				
Allowance for Plumbing Demo	Item			$ 15,000.00
Allowance for Plumbing Repair	Item			$ 75,000.00
		$ 16.43	**/SF**	**$ 90,000.00**
23 Heating, Ventilation and Air Conditioning				
Allowance for HVAC Demo	Item			$ 30,000.00
Allowance for HVAC Repair	Item			$ 150,000.00
		$ 32.86	**/SF**	**$ 180,000.00**
26 Electrical				
Allowance for Electrical Demo	Item			$ 20,000.00
Allowance for Electrical Repair	Item			$ 100,000.00
		$ 21.91	**/SF**	**$ 120,000.00**
	TOTAL	**$ 71.19**	**/SF**	**$ 390,000.00**

GFA: 5478 SF

D DEMOLITION

Description	Unit	Qty	Rate	Total
02 Existing Conditions				
Allowance for Misc. Demo	SF	5478	$ 2.20	$ 12,051.60
Demo Roof	SF	6420	$ 5.17	$ 33,191.40
Demo Masonry Wall	SF	535	$ 1.02	$ 545.70
Demo Wood Stud Wall	SF	1620	$ 1.06	$ 1,717.20
Demo Siding	SF	5830	$ 1.49	$ 8,686.70
Demo Shiplap	SF	3200	$ 0.68	$ 2,176.00
Demo Masonry Chimney	CF	54	$ 9.64	$ 520.56
		$ 10.75	**/SF**	**$ 58,889.16**
	TOTAL	**$ 10.75**	**/SF**	**$ 58,889.16**

II. QUONSET HUT

CONCEPTUAL STRENGTHENING AND REMEDIATION

SUMMARY

Location	GFA SF	Cost/SF	Total Cost
A STRUCTURAL STRENGTHENING	4,160	$ 3.44	$ 14,319.48
B BUILDING CONDITION REMEDIATION			$ 21,148.40
C MEP			$ 95,000.00
D DEMOLITION			$ 14,036.24
ESTIMATED NET COST	**4,160**	**$ 34.74**	**$ 144,504.12**

MARGINS & ADJUSTMENTS

Phasing & Temporary Work	2%		$ 2,890.08
General Conditions	10%		$ 14,739.42
Bonds & Insurance	3%		$ 4,864.01
Overhead & Profit	5%		$ 8,349.88
Design Contingency	20%		$ 35,069.50
Construction Contingency	20%		$ 42,083.40
Escalation to 3Q2019	3%		$ 7,575.01
ESTIMATED NET COST	**4160**	**$ 62.52**	**$ 260,075.43**

GFA: 4160 SF

A STRUCTURAL STRENGTHENING

Description	Unit	Qty	Rate	Total
05 Metals				
5/8" Epoxy Anchor Bolts	Ea	40	$ 10.20	$ 408.00
		$ 0.10	**/SF**	**$ 408.00**
06 Wood, Plastics, and Composites				
Plywood Roof/Floor Sheathing	SF	1164	$ 1.74	$ 2,025.36
Plywood Wall Sheathing	SF	3900	$ 1.97	$ 7,683.00
Misc. Framing	SF	5064	$ 0.83	$ 4,203.12
		$ 3.34	**/SF**	**$ 13,911.48**
	TOTAL	**$ 3.44**	**/SF**	**$ 14,319.48**

GFA: 4160 SF

B BUILDING CONDITION REMEDIATION

Description	Unit	Qty	Rate	Total
05 Metals				
Misc. Allowance for Connections	Item			$ 8,000.00
5/8" Epoxy Anchor Bolts	Ea	240	$ 10.20	$ 2,448.00
		$ 0.59	**/SF**	**$ 2,448.00**
06 Wood, Plastics, and Composites				
Wall Framing	SF	336	$ 2.50	$ 840.00
		$ 0.20	**/SF**	**$ 840.00**
07 Thermal and Mositure Protection				
New Roof	SF	960	$ 5.00	$ 4,800.00
New Siding	SF	2500	$ 4.91	$ 12,275.00
New Gutters/Downspouts	LF	84	$ 9.35	$ 785.40
		$ 4.29	**/SF**	**$ 17,860.40**
	TOTAL	**$ 5.08**	**/SF**	**$ 21,148.40**

		GFA:	4160 SF		
C MEP					
Description		Unit	Qty	Rate	Total
22 Plumbing					
	Allowance for Plumbing Demo	Item			$ 5,000.00
			$ 1.20	**/SF**	**$ 5,000.00**
23 Heating, Ventilation and Air Conditioning					
	Allowance for HVAC Demo	Item			$ 10,000.00
	Allowance for HVAC Repair	Item			$ 35,000.00
			$ 10.82	**/SF**	**$ 45,000.00**
26 Electrical					
	Allowance for Electrical Demo	Item			$ 10,000.00
	Allowance for Electrical Repair	Item			$ 35,000.00
			$ 10.82	**/SF**	**$ 45,000.00**
		TOTAL	**$ 22.84**	**/SF**	**$ 95,000.00**

GFA: 4160 SF

D DEMOLITION

Description	Unit	Qty	Rate	Total
02 Existing Conditions				
Allowance for Misc. Demo	SF	4160	$ 2.20	$ 9,152.00
Demo Roof	SF	960	$ 1.24	$ 1,190.40
Demo Gutters/Downspouts	LF	84	$ 2.18	$ 183.12
Demo Walls	SF	1600	$ 1.98	$ 3,168.00
Demo Masonry Piers	SF	336	$ 1.02	$ 342.72
		$ 3.37	**/SF**	**$ 14,036.24**
	TOTAL	**$ 3.37**	**/SF**	**$ 14,036.24**

III. DEMOLITION

SUMMARY

Location	GFA SF	Cost/SF	Total Cost
A ELEMENTARY SCHOOL STRUCTURE DEMO	5,478	$ 21.88	$ 119,838.96
B QUONSET HUT STRUCTURE DEMO	4160	$ 12.65	$ 52,626.10
ESTIMATED NET COST	**9,638**	**$ 17.89**	**$ 172,465.06**

MARGINS & ADJUSTMENTS

Phasing & Temporary Work	2%		$ 3,449.30
General Conditions	10%		$ 17,591.44
Bonds & Insurance	3%		$ 5,805.17
Overhead & Profit	5%		$ 9,965.55
Design Contingency	20%		$ 41,855.30
Construction Contingency	20%		$ 50,226.36
Escalation to 3Q2019	3%		$ 9,040.75
ESTIMATED NET COST	**9,638**	**$ 32.21**	**$ 310,398.93**

	GFA:	5478 SF		

A ELEMENTARY SCHOOL STRUCTURE DEMO

Description	Unit	Qty	Rate	Total
02 Existing Conditions				
Building Demo	CF	72595	$ 0.41	$ 29,763.95
Slab Demo	SF	5478	$ 0.92	$ 5,039.76
Foundation Demo	CY	75	$ 530.47	$ 39,785.25
Electric Utility Removal Allowance	Item			$ 10,000.00
Sewer/Water Utility Removal Allowance	Item			$ 12,000.00
MEP Removal Allowance	Item			$ 15,000.00
Site Work	SY	1100	$ 7.50	$ 8,250.00
		$ 21.88	**/SF**	**$ 119,838.96**
	TOTAL	**$ 21.88**	**/SF**	**$ 119,838.96**

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STRUCTURAL & EARTHQUAKE ENGINEERING

GFA: 4160 SF

B QUONSET HUT

Description	Unit	Qty	Rate	Total
02 Existing Conditions				
Building Demo	CF	61790	$ 0.41	$ 25,333.90
Slab Demo	SF	4160	$ 0.92	$ 3,827.20
Electric Utility Removal Allowance	Item			$ 5,000.00
Sewer/Water Utility Removal Allowance	Item			$ 5,000.00
MEP Removal Allowance				$ 10,000.00
Site Work	SY	462	$ 7.50	$ 3,465.00
		$ 12.65	**/SF**	**$ 52,626.10**
	TOTAL	**$ 12.65**	**/SF**	**$ 52,626.10**

Appendix E

ASCE 41-17 STRUCTURAL CHECKLISTS

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-6. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low and Moderate Seismicity			
Seismic-Force-Resisting System			
C NC N/A U	REDUNDANCY: The number of lines of shear walls in each principal direction is greater than or equal to 2.	5.5.1.1	A.3.2.1.1
C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft All other conditions 100 lb/ft	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2

Table 17-6 *(Continued)*. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-2. Collapse Prevention Basic Configuration Checklist

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
Low Seismicity			
Building System—General			
C NC N/A U	LOAD PATH: The structure contains a complete, well-defined load path, including structural elements and connections, that serves to transfer the inertial forces associated with the mass of all elements of the building to the foundation.	5.4.1.1	A.2.1.1
C NC N/A U	ADJACENT BUILDINGS: The clear distance between the building being evaluated and any adjacent building is greater than 0.25% of the height of the shorter building in low seismicity, 0.5% in moderate seismicity, and 1.5% in high seismicity.	5.4.1.2	A.2.1.2
C NC N/A U	MEZZANINES: Interior mezzanine levels are braced independently from the main structure or are anchored to the seismic-force-resisting elements of the main structure.	5.4.1.3	A.2.1.3
Building System—Building Configuration			
C NC N/A U	WEAK STORY: The sum of the shear strengths of the seismic-force-resisting system in any story in each direction is not less than 80% of the strength in the adjacent story above.	5.4.2.1	A.2.2.2
C NC N/A U	SOFT STORY: The stiffness of the seismic-force-resisting system in any story is not less than 70% of the seismic-force-resisting system stiffness in an adjacent story above or less than 80% of the average seismic-force-resisting system stiffness of the three stories above.	5.4.2.2	A.2.2.3
C NC N/A U	VERTICAL IRREGULARITIES: All vertical elements in the seismic-force-resisting system are continuous to the foundation.	5.4.2.3	A.2.2.4
C NC N/A U	GEOMETRY: There are no changes in the net horizontal dimension of the seismic-force-resisting system of more than 30% in a story relative to adjacent stories, excluding one-story penthouses and mezzanines.	5.4.2.4	A.2.2.5
C NC N/A U	MASS: There is no change in effective mass of more than 50% from one story to the next. Light roofs, penthouses, and mezzanines need not be considered.	5.4.2.5	A.2.2.6
C NC N/A U	TORSION: The estimated distance between the story center of mass and the story center of rigidity is less than 20% of the building width in either plan dimension.	5.4.2.6	A.2.2.7
Moderate Seismicity (Complete the Following Items in Addition to the Items for Low Seismicity)			
Geologic Site Hazards			
C NC N/A U	LIQUEFACTION: Liquefaction-susceptible, saturated, loose granular soils that could jeopardize the building's seismic performance do not exist in the foundation soils at depths within 50 ft (15.2 m) under the building.	5.4.3.1	A.6.1.1
C NC N/A U	SLOPE FAILURE: The building site is located away from potential earthquake-induced slope failures or rockfalls so that it is unaffected by such failures or is capable of accommodating any predicted movements without failure.	5.4.3.1	A.6.1.2
C NC N/A U	SURFACE FAULT RUPTURE: Surface fault rupture and surface displacement at the building site are not anticipated.	5.4.3.1	A.6.1.3
High Seismicity (Complete the Following Items in Addition to the Items for Moderate Seismicity)			
Foundation Configuration			
C NC N/A U	OVERTURNING: The ratio of the least horizontal dimension of the seismic-force-resisting system at the foundation level to the building height (base/height) is greater than $0.6S_a$.	5.4.3.3	A.6.2.1
C NC N/A U	TIES BETWEEN FOUNDATION ELEMENTS: The foundation has ties adequate to resist seismic forces where footings, piles, and piers are not restrained by beams, slabs, or soils classified as Site Class A, B, or C.	5.4.3.4	A.6.2.2

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.

Table 17-6. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
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Seismic-Force-Resisting System			
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C NC N/A U	SHEAR STRESS CHECK: The shear stress in the shear walls, calculated using the Quick Check procedure of Section 4.4.3.3, is less than the following values: Structural panel sheathing 1,000 lb/ft Diagonal sheathing 700 lb/ft Straight sheathing 100 lb/ft $\boxed{\text{All other conditions 100 lb/ft}}$	5.5.3.1.1	A.3.2.7.1
C NC N/A U	STUCCO (EXTERIOR PLASTER) SHEAR WALLS: Multi-story buildings do not rely on exterior stucco walls as the primary seismic-force-resisting system.	5.5.3.6.1	A.3.2.7.2
C NC N/A U	GYPSUM WALLBOARD OR PLASTER SHEAR WALLS: Interior plaster or gypsum wallboard is not used for shear walls on buildings more than one story high with the exception of the uppermost level of a multi-story building.	5.5.3.6.1	A.3.2.7.3
C NC N/A U	NARROW WOOD SHEAR WALLS: Narrow wood shear walls with an aspect ratio greater than 2-to-1 are not used to resist seismic forces.	5.5.3.6.1	A.3.2.7.4
C NC N/A U	WALLS CONNECTED THROUGH FLOORS: Shear walls have an interconnection between stories to transfer overturning and shear forces through the floor.	5.5.3.6.2	A.3.2.7.5
C NC N/A U	HILLSIDE SITE: For structures that are taller on at least one side by more than one-half story because of a sloping site, all shear walls on the downhill slope have an aspect ratio less than 1-to-1.	5.5.3.6.3	A.3.2.7.6
C NC N/A U	CRIPPLE WALLS: Cripple walls below first-floor-level shear walls are braced to the foundation with wood structural panels.	5.5.3.6.4	A.3.2.7.7
C NC N/A U	OPENINGS: Walls with openings greater than 80% of the length are braced with wood structural panel shear walls with aspect ratios of not more than 1.5-to-1 or are supported by adjacent construction through positive ties capable of transferring the seismic forces.	5.5.3.6.5	A.3.2.7.8
Connections			
C NC N/A U	WOOD POSTS: There is a positive connection of wood posts to the foundation.	5.7.3.3	A.5.3.3
C NC N/A U	WOOD SILLS: All wood sills are bolted to the foundation.	5.7.3.3	A.5.3.4
C NC N/A U	GIRDER–COLUMN CONNECTION: There is a positive connection using plates, connection hardware, or straps between the girder and the column support.	5.7.4.1	A.5.4.1
High Seismicity (Complete the Following Items in Addition to the Items for Low and Moderate Seismicity)			
Connections			
C NC N/A U	WOOD SILL BOLTS: Sill bolts are spaced at 6 ft (1.8 m) or less with acceptable edge and end distance provided for wood and concrete.	5.7.3.3	A.5.3.7
Diaphragms			
C NC N/A U	DIAPHRAGM CONTINUITY: The diaphragms are not composed of split-level floors and do not have expansion joints.	5.6.1.1	A.4.1.1
C NC N/A U	ROOF CHORD CONTINUITY: All chord elements are continuous, regardless of changes in roof elevation.	5.6.1.1	A.4.1.3
C NC N/A U	DIAPHRAGM REINFORCEMENT AT OPENINGS: There is reinforcing around all diaphragm openings larger than 50% of the building width in either major plan dimension.	5.6.1.5	A.4.1.8
C NC N/A U	STRAIGHT SHEATHING: All straight-sheathed diaphragms have aspect ratios less than 2-to-1 in the direction being considered.	5.6.2	A.4.2.1
C NC N/A U	SPANS: All wood diaphragms with spans greater than 24 ft (7.3 m) consist of wood structural panels or diagonal sheathing.	5.6.2	A.4.2.2

Table 17-6 *(Continued)*. Collapse Prevention Structural Checklist for Building Type W2

Status	Evaluation Statement	Tier 2 Reference	Commentary Reference
C NC N/A U	DIAGONALLY SHEATHED AND UNBLOCKED DIAPHRAGMS: All diagonally sheathed or unblocked wood structural panel diaphragms have horizontal spans less than 40 ft (12.2 m) and have aspect ratios less than or equal to 4-to-1.	5.6.2	A.4.2.3
C NC N/A U	OTHER DIAPHRAGMS: The diaphragms do not consist of a system other than wood, metal deck, concrete, or horizontal bracing.	5.6.5	A.4.7.1

Note: C = Compliant, NC = Noncompliant, N/A = Not Applicable, and U = Unknown.