# STRUCTURAL DESIGN REPORT

STRUCTURAL DESIGN REPORT OF PROPOSED BASHAM RESIDENCE-FIRE DAMAGE REBUILD AT 6582 AVENIDA MARIPOSA JURUPA VALLEY, CA. 02509

**City of Jurupa Valley**
**Building Department**

Reviewed and Approved for Code Compliance

**By:** Brenda Yu **Date:** 11/22/2024

**Permit Number:** **B24-001535**

Approval of these plans shall not be construed to be a permit for or an approval of any violations of any City, County, State or Federal laws.

LICENSED ARCHITECT
NO. C-30183
REN. 9-30-23
STATE OF CALIFORNIA

PROJECT: 6582 AVENIDA MARIPOSA JURUPA VALLEY, CA. 02509

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# 1 PROJECT OVERVIEW

## 1.1 INTRODUCTION

This report is prepared to present the structural design calculations and recommendations for the Basham Residence-Fire Damage Rebuild at 6582 Avenida Mariposa Jurupa Valley, CA. 02509. (Location is given in Figure 1.1).


Figure 1-1: Project Location.

In this report basis for structural design such as design codes and standards, material properties, various type of loadings the structure is intended to withstand and their combination of action to be considered in the design are presented. Further the approach to be adopted in the structural analysis and the design and the assumptions associated are discussed. Structural analysis and design tools such as computer packages utilized by the designer to perform the structural designs are listed. The design criteria to be used in the performance verification of the proposed structural system under both gravity and the lateral loads are presented with references. The loadings adopted by the structural designer in the design of structural elements are presented. Finally detailed calculation of each structural elements with the design summary and verification of proposed element sizes to withstand the intended loading are present.

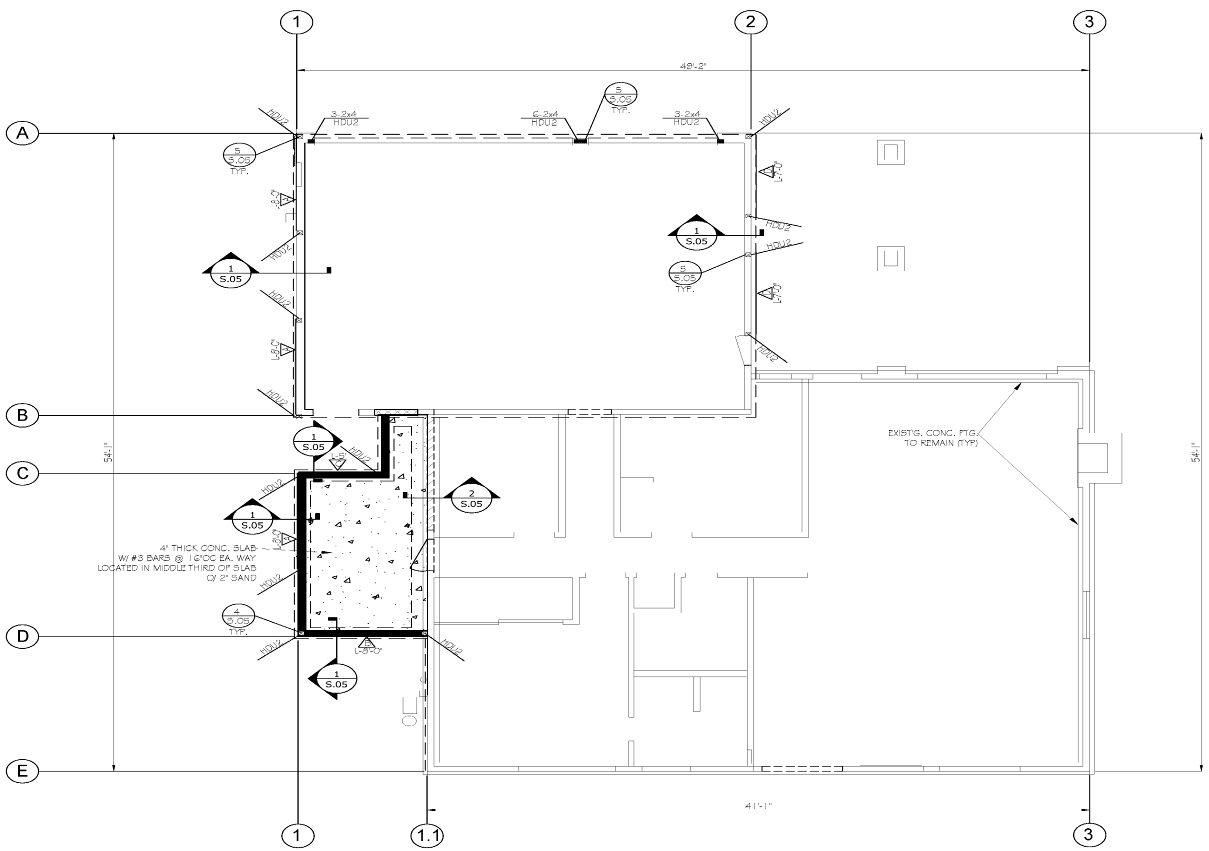



*Figure 1-2: Proposed foundation plan*

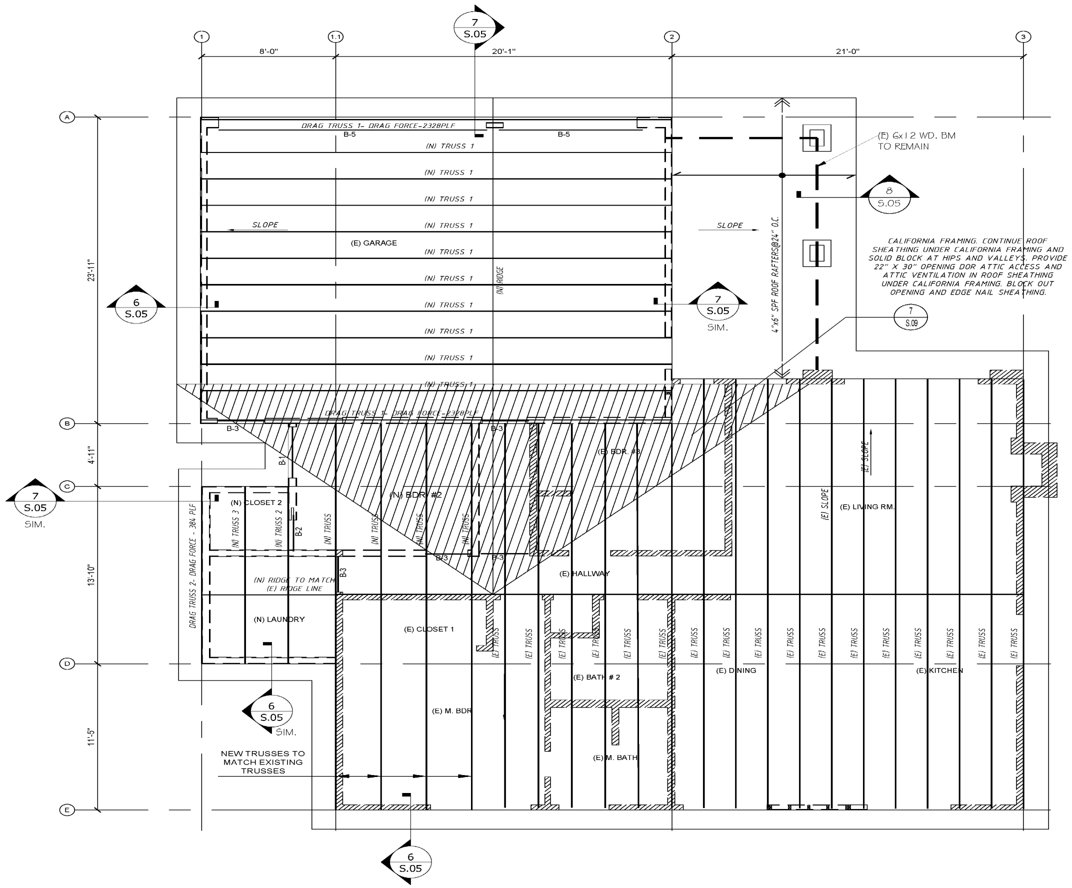



*Figure 1-3: Proposed Roof framing plan*

## 1.2 SCOPE

This report is only intended to address the following aspects and shall be read in combination with the documents referred to in section 1.3.

- Explain the basis of design adopted for the structural design.
  - Structural design codes and standards.
  - Material properties and loadings considered.
  - The basis for the calculation loadings.
- Structural concept proposed for superstructure and foundation.
- Present the detailed structural calculations of structural members.
- Detailed review of existing structural members.
- Proposed adequate sizes for proposed members.

## 1.3 SOURCE DOCUMENTS

- Relevant local and international standards.
  - Minimum design loads for buildings and other structures, American society of Civil Engineers.
- Architectural drawings from the client.
- National design specification for wood construction. (AWC-American wood council. (2018).

# 2 STRUCTURAL DESIGN CRITERIA

## 2.1 DESIGN CODES AND STANDARDS

Referring to Client's requirements specified in documents listed in section 1.3 of this report following design standards/ guidelines are designated for the structural design of proposed project.

- 2022 California Building Code Chapter 16; Minimum Design Loads for Buildings and Other Structure. (General Loads and Load combinations)
- 2022 California Building Code Chapter 19: American Concrete Institute; Building Code Requirements for Structural Concrete and Commentary
- National Design Specification for Wood Construction 2018 (NDS); American Wood Council. (Material properties of timber)
- California Building Code CBC 2022.
- California Residential Code CRC 2022
- https://hazards.atcouncil.org/ (Ground Snow Load)

## 2.2 STRUCTURAL MATERIAL PROPERTIES

**Timber**

All timber materials shall comply with National Design Specification of wood construction (NDS,2018). Material properties of the timber are as follows.

Table 2-1: Material Properties of Timber

| | **Fb**/psi (MPa) | **Fv** /psi (MPa) | **Fc**(per) /psi (MPa) | **Fc**(para)/ psi (MPa) | **E**/psi (MPa) |
|---|---|---|---|---|---|
| All Microllam's (LVL) | 2600 (17.9) | 285 (1.97) | 750 (5.17) | 2510 (17.3) | 2 X $10^6$ (13780) |
| Douglas Fir-Larch#2 | 900 | 575 | 625 | 1350 | 1.6 X $10^6$ |
| Abbreviations: psi – Pounds per Square Inch, per. – Perpendicular to Grain, para. – Parallel to Grain | | | | | |

## 2.3 DESIGN LOADING

Construction materials placed on the structure shall be placed in a manner not exceeding the design load specified. It is the property owner's responsibility to ensure that the design loads are not exceeded after construction. The design loads for the projects are:

- **Dead Loads**

Dead loads are based on material unit weights as presented in Table 2.2.

Table 2-2 Material Unit Weight

| Material | Density lb./ft$^3$ (kN/m$^3$) |
|---|---|
| Aluminum | 169.18 (2710) |
| Concrete | 143.5 (2300) |
| Steel | 490 (76.9) |

- **Super imposed Dead Loads**

Table 2.3 shows super imposed dead to be used in different areas as per intended usage. Super imposed dead loads values are obtained from Residential Structural design guide. These values compared with the employer's requirements and conservative values among those were selected.

Table 2-3: Loading values

| Loads | Value psf (kN/m$^2$) | Description |
|---|---|---|
| Floor Load | 20 (0.96) | Light-frame 2x12 wood floor with 3/4-inch wood structural panel sheathing with carpet, vinyl, or similar floor covering |
| Roof Load | 27 (1.29) | Light-frame wood roof with wood structural panel sheathing with Concrete roof tile roofing |
| Ceiling Load | 3.3, 4.2, 5.0 | 1/2", 5/8", 3/4" Gypsum Board |
| psf – Pounds per Square feet | | |

- **Stud Wall Load**

Table 2-4:Loading Value (8ft height walls)

| Description | Value (plf ) |
|---|---|
| 2x6 @ 16" O.C | 16.7 |
| 2x4 @ 16" O.C | 10.6 |

- **Live Load**

Live load values are obtained from ASCE/SEI 7 -16, Table 4.1. These values compared with the employer's requirements and conservative values among those were selected.

Table 2-5: Live load

| Description | | Value psf ($kN/m^2$) |
|---|---|---|
| Floor Loads | Office Usage | 50 (2.4) |
| | Balcony and Decks | 1.5 times the live load for the occupancy served. Not required to exceed 100 psf (4.79) |
| | Dining Rooms | 100 (4.79) |
| | Garages | 40 (1.92) |
| | Residentials Rooms | 40 (1.92) |
| Roof Load | All Roofs | 20 (0.96) |
| Stairs | Residential | 40 (1.92) |

Construction materials placed on the structure shall be placed in a manner not exceeding the design load specified. It is the property owner's responsibility to ensure that the design loads are not exceeded after construction. The design loads for the projects are:

- **Wind load calculation.**

Wind loads is determined based on the ASCE 7-16 chapter 26 regulations. Basic wind speed for Risk Category II selected as 96mph. Accordingly, the selected basic wind speed corresponds to Class 2 in the Saffir-Simpson Hurricane category.

- **Seismic load calculation.**

Structural system has been considered as a light-frame(wood)walls sheathed with wood structural panels rated for shear resistance. According to ASCE 7-16 Table 12.2- 1. Site class D has been chosen considering the critical situation, Response modification factor R=6.5, Importance factor I=1.0.

Table 2-6: Site Parameters

| | Long period(S1) | Short period (Ss) |
|---|---|---|
| Time(s) | 0.6 | 1.5 |

## 2.4 LOAD COMBINATION

Allowable stress method recommended in the design code is adopted in the design of wood elements. Combination of different loads will consider as per section of ASCE/SEI 7-16 and NDS 18. Basic load combinations to be used in the design is presented in Table 2.7. In special cases additional combination of relevant loading effects will be considered as per above stated code recommendations.

Table 2-7:Load combinations

| | Combination Abbreviation | Description |
|---|---|---|
| Allowable Stress Design | Com A1 | D |
| | Com A2 | D+L |
| | Com A3 | D+ S |
| | Com A4 | D + 0.75L + 0.75 S |
| | Com A5 | D + 0.6W |
| | Com A6 a | D + 0.75L + 0.75(0.6W) + 0.75 S |
| | Com A6 b | D + 0.75L + 0.75S |
| | Com A7 | 0.6D + 0.6W |
| Abbreviations: D – Dead loads, L – Live loads, S-Snow Load, W-Wind Load | | |

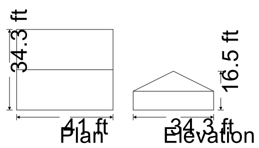
## 2.5 WIND LOAD CALCULATION.

- **WIND LOADING**

**In accordance with ASCE7-16**

**Using the directional design method**

Tedds calculation version 2.1.05




**Building data**

| | |
|---|---|
| Type of roof | Gable |
| Length of building | b = **41.00** ft |
| Width of building | d = **34.25** ft |
| Height to eaves | H = **8.00** ft |
| Pitch of roof | $\alpha_0$ = **26.4** deg |
| Mean height | h = **12.26** ft |

**General wind load requirements**

| | |
|---|---|
| Basic wind speed | V = **96.0** mph |
| Risk category | II |
| Velocity pressure exponent coef (Table 26.6-1) | $K_d$ = **0.85** |
| Ground elevation above sea level | $z_{gl}$ = **0** ft |
| Ground elevation factor | $K_e = \exp(-0.0000362 \times z_{gl}/1ft)$ = **1.00** |
| Exposure category (cl 26.7.3) | B |
| Enclosure classification (cl.26.12) | Enclosed buildings |
| Internal pressure coef +ve (Table 26.13-1) | $GC_{pi\_p}$ = **0.18** |
| Internal pressure coef –ve (Table 26.13-1) | $GC_{pi\_n}$ = **-0.18** |
| Gust effect factor | $G_f$ = **0.85** |
| Minimum design wind loading (cl.27.4.7) | $p_{min\_r}$ = **8** lb/ft$^2$ |

**Topography**

| | |
|---|---|
| Topography factor not significant | $K_{zt} = 1.0$ |
| Velocity pressure equation | $q = 0.00256 \times K_z \times K_{zt} \times K_d \times V^2 \times 1psf/mph^2$ |

**Velocity pressures table**

| z (ft) | $K_z$ (Table 26.10-1) | $q_z$ (psf) |
|---|---|---|
| 8.00 | 0.57 | 11.43 |
| 12.26 | 0.57 | 11.43 |
| 15.00 | 0.57 | 11.43 |
| 16.51 | 0.59 | 11.73 |

**Peak velocity pressure for internal pressure**

Peak velocity pressure – internal (as roof press.) $q_i$ = **11.43** psf

**Pressures and forces**

Net pressure $p = q \times G_f \times C_{pe} - q_i \times GC_{pi}$

Net force $F_w = p \times A_{ref}$

**Roof load case 1 - Wind 0, $GC_{pi}$ 0.18, $-c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ ($ft^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| A (-ve) | 12.26 | -0.23 | 11.43 | -4.30 | 784.08 | -3.37 |
| B (-ve) | 12.26 | -0.60 | 11.43 | -7.89 | 784.08 | -6.18 |

Total vertical net force $F_{w,v}$ = **-8.56** kips

Total horizontal net force $F_{w,h}$ = **1.25** kips

**Walls load case 1 - Wind 0, $GC_{pi}$ 0.18, $-c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ ($ft^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| A | 8.00 | 0.80 | 11.43 | 5.72 | 328.00 | 1.87 |
| B | 12.26 | -0.50 | 11.43 | -6.92 | 328.00 | -2.27 |
| C | 12.26 | -0.70 | 11.43 | -8.86 | 419.77 | -3.72 |
| D | 12.26 | -0.70 | 11.43 | -8.86 | 419.77 | -3.72 |

**Overall loading**

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H$ = **328.00** $ft^2$

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0)$ = **349.00** $ft^2$

Minimum overall horizontal loading $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0}$ = **8.04** kips

Leeward net force $F_l = F_{w,wB}$ = **-2.3** kips

Windward net force $F_w = F_{w,wA}$ = **1.9** kips

Overall horizontal loading $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min})$ = **8.0** kips

**Roof load case 2 - Wind 0, $GC_{pi}$ -0.18, $-0c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ ($ft^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| A (+ve) | 12.26 | 0.26 | 11.43 | 4.55 | 784.08 | 3.57 |
| B (+ve) | 12.26 | -0.60 | 11.43 | -3.77 | 784.08 | -2.96 |

Total vertical net force $F_{w,v}$ = **0.55** kips

Total horizontal net force $F_{w,h}$ = **2.91** kips

**Walls load case 2 - Wind 0, $GC_{pi}$ -0.18, $-0c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ ($ft^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| A | 8.00 | 0.80 | 11.43 | 9.83 | 328.00 | 3.22 |
| B | 12.26 | -0.50 | 11.43 | -2.80 | 328.00 | -0.92 |
| C | 12.26 | -0.70 | 11.43 | -4.74 | 419.77 | -1.99 |
| D | 12.26 | -0.70 | 11.43 | -4.74 | 419.77 | -1.99 |

**Overall loading**

Projected vertical plan area of wall $A_{vert\_w\_0} = b \times H$ = **328.00** $ft^2$

Projected vertical area of roof $A_{vert\_r\_0} = b \times d/2 \times \tan(\alpha_0)$ = **349.00** $ft^2$

| | |
|---|---|
| Minimum overall horizontal loading | $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_0} + p_{min\_r} \times A_{vert\_r\_0} = \mathbf{8.04}$ kips |
| Leeward net force | $F_l = F_{w,wB} = \mathbf{-0.9}$ kips |
| Windward net force | $F_w = F_{w,wA} = \mathbf{3.2}$ kips |
| Overall horizontal loading | $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{8.0}$ kips |

**Roof load case 3 - Wind 90, $GC_{pi}$ 0.18, $-c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ (ft$^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| A (-ve) | 12.26 | -0.90 | 11.43 | -10.80 | 234.38 | -2.53 |
| B (-ve) | 12.26 | -0.90 | 11.43 | -10.80 | 234.38 | -2.53 |
| C (-ve) | 12.26 | -0.50 | 11.43 | -6.92 | 468.77 | -3.24 |
| D (-ve) | 12.26 | -0.30 | 11.43 | -4.97 | 630.62 | -3.14 |

| | |
|---|---|
| Total vertical net force | $F_{w,v} = \mathbf{-10.25}$ kips |
| Total horizontal net force | $F_{w,h} = \mathbf{0.00}$ kips |

**Walls load case 3 - Wind 90, $GC_{pi}$ 0.18, $-c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ (ft$^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| $A_1$ | 15.00 | 0.80 | 11.43 | 5.72 | 415.17 | 2.37 |
| $A_2$ | 16.51 | 0.80 | 11.73 | 5.92 | 4.60 | 0.03 |
| B | 12.26 | -0.46 | 11.43 | -6.53 | 419.77 | -2.74 |
| C | 12.26 | -0.70 | 11.43 | -8.86 | 328.00 | -2.91 |
| D | 12.26 | -0.70 | 11.43 | -8.86 | 328.00 | -2.91 |

**Overall loading**

| | |
|---|---|
| Projected vertical plan area of wall | $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{419.77}$ ft$^2$ |
| Projected vertical area of roof | $A_{vert\_r\_90} = \mathbf{0.00}$ ft$^2$ |
| Minimum overall horizontal loading | $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{6.72}$ kips |
| Leeward net force | $F_l = F_{w,wB} = \mathbf{-2.7}$ kips |
| Windward net force | $F_w = F_{w,wA\_1} + F_{w,wA\_2} = \mathbf{2.4}$ kips |
| Overall horizontal loading | $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{6.7}$ kips |

**Roof load case 4 - Wind 90, $GC_{pi}$ -0.18, $+c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ (ft$^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| A (+ve) | 12.26 | -0.18 | 11.43 | 0.31 | 234.38 | 0.07 |
| B (+ve) | 12.26 | -0.18 | 11.43 | 0.31 | 234.38 | 0.07 |
| C (+ve) | 12.26 | -0.18 | 11.43 | 0.31 | 468.77 | 0.14 |
| D (+ve) | 12.26 | -0.18 | 11.43 | 0.31 | 630.62 | 0.19 |

| | |
|---|---|
| Total vertical net force | $F_{w,v} = \mathbf{0.43}$ kips |
| Total horizontal net force | $F_{w,h} = \mathbf{0.00}$ kips |

**Walls load case 4 - Wind 90, $GC_{pi}$ -0.18, $+c_{pe}$**

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ (ft$^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| $A_1$ | 15.00 | 0.80 | 11.43 | 9.83 | 415.17 | 4.08 |

| Zone | Ref. height (ft) | Ext pressure coefficient $c_{pe}$ | Peak velocity pressure $q_p$ (psf) | Net pressure p (psf) | Area $A_{ref}$ ($ft^2$) | Net force $F_w$ (kips) |
|---|---|---|---|---|---|---|
| $A_2$ | 16.51 | 0.80 | 11.73 | 10.04 | 4.60 | 0.05 |
| B | 12.26 | -0.46 | 11.43 | -2.42 | 419.77 | -1.01 |
| C | 12.26 | -0.70 | 11.43 | -4.74 | 328.00 | -1.56 |
| D | 12.26 | -0.70 | 11.43 | -4.74 | 328.00 | -1.56 |

**Overall loading**

Projected vertical plan area of wall — $A_{vert\_w\_90} = d \times H + d^2 \times \tan(\alpha_0) / 4 = \mathbf{419.77}$ $ft^2$

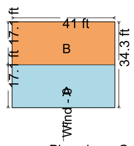
Projected vertical area of roof — $A_{vert\_r\_90} = \mathbf{0.00}$ $ft^2$

Minimum overall horizontal loading — $F_{w,total\_min} = p_{min\_w} \times A_{vert\_w\_90} + p_{min\_r} \times A_{vert\_r\_90} = \mathbf{6.72}$ kips

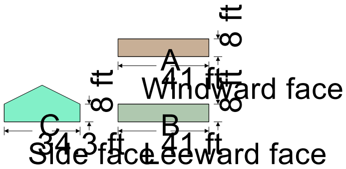
Leeward net force — $F_l = F_{w,wB} = \mathbf{-1.0}$ kips

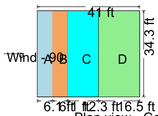
Windward net force — $F_w = F_{w,wA\_1} + F_{w,wA\_2} = \mathbf{4.1}$ kips

Overall horizontal loading — $F_{w,total} = \max(F_w - F_l + F_{w,h}, F_{w,total\_min}) = \mathbf{6.7}$ kips

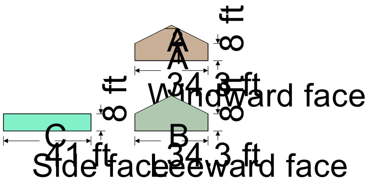


Plan view - Gable roof



Plan view - Gable roof


A
8 ft
34.3 ft
Windward face
8 ft
C
41 ft
Side face
B
34.3 ft
Leeward face

# 2.6 SEISMIC LOAD CALCULATION.

**SEISMIC FORCES (ASCE 7-16)**

Tedds calculation version 3.1.00

**Site parameters**

Site class D

Mapped acceleration parameters (Section 11.4.2)

at short period $S_S$ = **1.500**

at 1 sec period $S_1$ = **0.600**

**Alternate design spectral acceleration parameters (Chap 21)**

Design spectral response acceleration at period T (Sect 21.3) $S_a$ = 0.75

at short period (Sect 21.4) $S_{DSalt}$ = **1.000**

at 1 sec period (Sect 21.4) $S_{D1alt}$ = **0.717**

**Spectral response acceleration parameters**

at short period (Sect 21.4) $S_{MS} = 1.5 \times S_{DSalt}$ = **1.500**

at 1 sec period (Sect 21.4) $S_{M1} = 1.5 \times S_{D1alt}$ = **1.076**

**Seismic design category**

Occupancy category (Table 1-1) II

Seismic design category based on short period response acceleration (Table 11.6-1)
D

Seismic design category based on 1 sec period response acceleration (Table 11.6-2)
D

Seismic design category D

**Approximate fundamental period**

Height above base to highest level of building $h_n$ = **16.5** ft

From Table 12.8-2:

Structure type All other systems

Building period parameter $C_t$ $C_t$ = **0.02**

Building period parameter x x = **0.75**

Approximate fundamental period (Eq 12.8-7) $T_a = C_t \times (h_n)^x \times 1sec / (1ft)^x$= **0.164** sec

Building fundamental period (Sect 12.8.2) $T = T_a$ = **0.164** sec

Long-period transition period $T_L$ = **8** sec

**Seismic response coefficient**

Seismic force-resisting system (Table 12.2-1) A. Bearing_Wall_Systems
15. Light-frame (wood) walls sheathed with wood structural panels

Response modification factor (Table 12.2-1) R = **6.5**

Seismic importance factor (Table 1.5-2) $I_e$ = **1.000**

Seismic response coefficient (Sect 12.8.1.1)

Calculated (Eq 12.8-2) $C_{s\_calc} = S_{DSalt} / (R / I_e)$ = **0.1538**

Maximum ((Eq 12.8-3)) $C_{s\_max} = S_a / (R / I_e)$ = **0.1154**

Minimum:

Eq 9.5.5.2.1-3 $C_{s\_min1} = \max(0.044 \times S_{DSalt} \times I_e, 0.01)$ = **0.0440**

Eq 12.8-6 (where $S_1$ >= 0.6) $C_{s\_min2} = (0.5 \times S_1) / (R / I_e)$ = **0.0462**

$C_{s\_min}$ = **0.0462**

Seismic response coefficient $C_s$ = **0.1154**

**Seismic base shear (Sect 12.8.1)**

| | |
|---|---|
| Effective seismic weight of the structure | W = **92.3** kips |
| Seismic response coefficient | $C_s$ = **0.1154** |
| Seismic base shear (Eq 12.8-1) | V = $C_s \times$ W = **10.6** kips |

## WIND AND SEISMIC FORCE DISTRIBUTION.

- Design Seismic Base shear- **10.6 Kips**
- Design Wind Base shear- **8.0 Kips**

Seismic load is greater than the Wind load, hence the design governs by Seismic load.

| Roof Level | X direction | Y direction |
|---|---|---|
| Design Shear Force | 10.6kips | 10.6 kips |
| Wall length | 121.5 ft | 155.6 ft |
| Shear force(per unit length) | 87.24 lb/ft | 68.12 lb/ft |

# 3 STRUCTURAL CONCEPT AND DESIGN OF ELEMENTS

Proposed element arrangement and design verification of those structural elements are presented in this chapter. Finite element analysis for the structure was done to to check the stability of the structure.

## 3.1 DESIGNING OF STUD WALL-1

- Proposed Member : 2”X 6” Douglas-Fir-Larch (North)@16”

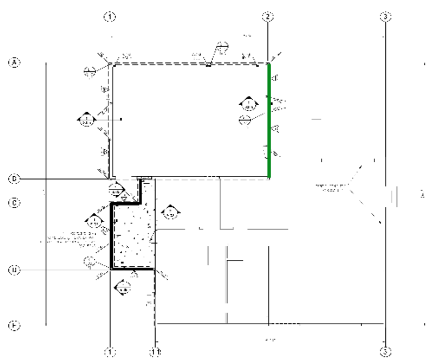


*Figure 3-1:Key Plan*

*Table 3-1:Wall Loading*

| Element | Loading | | | Tributary Width(ft) | Loading | | |
|---|---|---|---|---|---|---|---|
| | S. Dead ($lb/ft^2$) | Live ($lb/ft^2$) | Snow ($lb/ft^2$) | | S. Dead (lb/ft) | Live (lb/ft) | Snow (lb/ft) |
| **Roof** | 27 | 20 | - | 14.2 | 383.4 | 284 | - |
| **Ceiling** | 5 | - | - | 14.2 | 71.0 | - | - |
| **Total** | | | | | 454.4 | 284 | - |

- Seismic load = 1053.82 lbs
- Wind load= 902.4 lbs
- Seismic load is greater than the Wind load, hence the design governs by Seismic load.

*Table 3-2:Design Summary*

| | | Capacity | Maximum | Utilization | |
|---|---|---|---|---|---|
| Shear capacity | lbs | 7147 | 738 | 0.103 | ✔ |
| Chord capacity | $lb/in^2$ | 1285 | 49 | 0.038 | ✔ |
| Deflection | in | 1.920 | 0.090 | 0.047 | ✔ |

**Refer Annex A for detailed calculation.**

## 3.2 DESIGNING OF STUD WALL-2

- Proposed Member : 2”X 6” Douglas-Fir-Larch (North)@16”

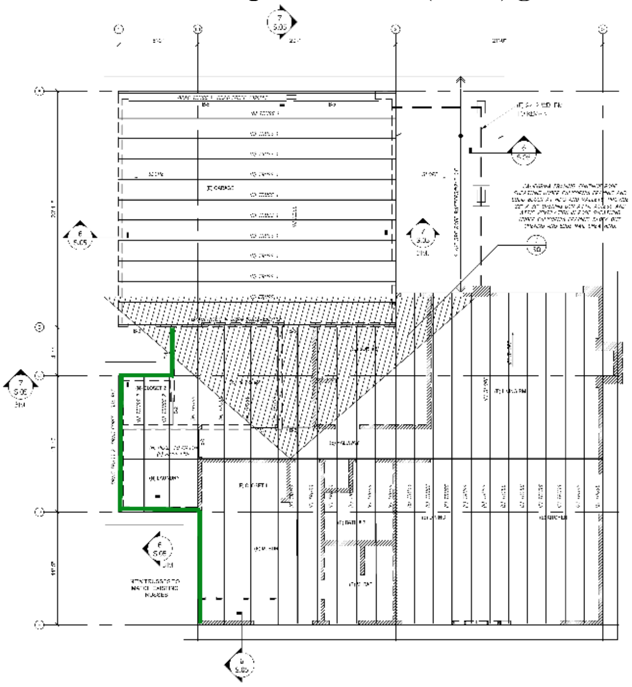


*Figure 3-2:Key Plan*

*Table 3-3:Wall Loading*

| Element | Loading | | | Tributary Width(ft) | Loading | | |
|---|---|---|---|---|---|---|---|
| | S. Dead ($lb/ft^2$) | Live ($lb/ft^2$) | Snow ($lb/ft^2$) | | S. Dead (lb/ft) | Live (lb/ft) | Snow (lb/ft) |
| **Roof** | 27 | 20 | - | 6.5 | 175.5 | 130 | - |
| **Ceiling** | 5 | - | - | 6.5 | 32.5 | - | - |
| **Total** | | | | | 208.0 | 130 | - |

- Seismic load = 2060.63 lbs
- Wind load= 416.0 lbs
- Seismic load is greater than the Wind load, hence the design governs by Seismic load.

*Table 3-4:Design Summary*

| | | Capacity | Maximum | Utilization | |
|---|---|---|---|---|---|
| Shear capacity | lbs | 9100 | 1442 | 0.159 | ✔ |
| Chord capacity | $lb/in^2$ | 1285 | 43 | 0.034 | ✔ |
| Collector capacity | $lb/in^2$ | 1040 | 12 | 0.012 | ✔ |

**Refer Annex B for detailed calculation.**

## 3.3 DESIGNING OF STUD WALL-3

- Proposed Member : 2”X 6” Douglas-Fir-Larch (North)@16”

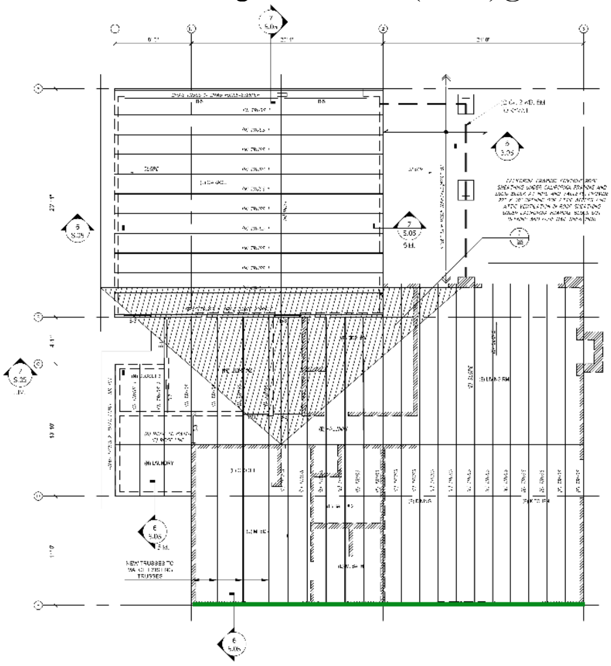


*Figure 3-3:Key Plan*

*Table 3-5:Wall Loading*

| Element | Loading | | | Tributary Width(ft) | Loading | | |
|---|---|---|---|---|---|---|---|
| | S. Dead ($lb/ft^2$) | Live ($lb/ft^2$) | Snow ($lb/ft^2$) | | S. Dead (lb/ft) | Live (lb/ft) | Snow (lb/ft) |
| **Roof** | 27 | 20 | - | 8.3 | 224.1 | 166.0 | - |
| **Ceiling** | 5 | - | - | 8.3 | 41.5 | - | - |
| **Total** | | | | | 265.6 | 166.0 | - |

- Seismic load = 3585.56 lbs
- Wind load=1632.0 lbs
- Seismic load is greater than the Wind load, hence the design governs by Seismic load.

*Table 3-6:Design Summary*

| | | Capacity | Maximum | Utilization |
|---|---|---|---|---|
| Shear capacity | lbs | 6825 | 2510 | 0.368 |
| Chord capacity | $lb/in^2$ | 1040 | 68 | 0.065 |
| Collector capacity | $lb/in^2$ | 1040 | 25 | 0.024 |

**Refer Annex C for detailed calculation.**

## 3.4 DESIGNING OF DRAG FORCE-1

- Roof Diaphragm calculation for drag truss design.

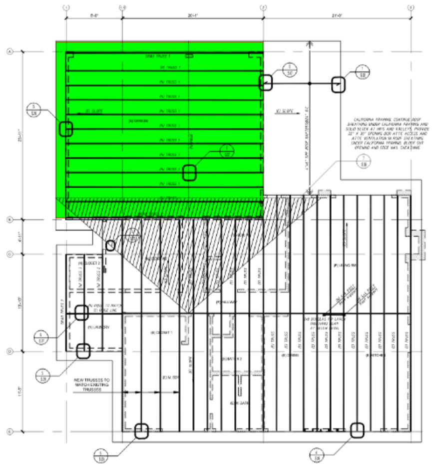


*Figure 3-4: Key plan*

**INPUT DATA & ANALYSIS**

| | | |
|---|---|---|
| Width | 23.5 | ft |
| Length | 28.2 | ft |
| Joist Spacing | 1.33 | ft |
| X-Dir Chord Area | 8.25 | $in^2$ |
| Y-Dir Chord Area | 8.25 | $in^2$ |
| Superimposed Loads | 27 | psf |
| Net Uplift Loads | 8 | psf |
| Seismic Factor | 1 | |
| Wind Factor | 1 | |
| Joist Direction | Parallel to Left and Right | |

| | | |
|---|---|---|
| Seismic Diaphragm Force X-Dir, $F_{pxE}$ = | 4.58 | kip |
| Seismic Diaphragm Force Y-Dir, $F_{pyE}$ = | 4.58 | kip |
| Wind Diaphragm Force X-Dir, $F_{pxW}$ = | 1.92 | kip |
| Wind Diaphragm Force Y-Dir, $F_{pyW}$ = | 1.60 | kip |
| Attachment Length Left, $L_{left}$ = | 23.50 | ft |
| Attachment Length Top, $L_{top}$ = | 28.20 | ft |
| Attachment Length Right, $L_{right}$ = | 23.50 | ft |
| Attachment Length Bottom, $L_{bot}$ = | 28.20 | ft |
| Deflection, L/ | 240 | |

Joists Span Up and Down

| Seismic | | |
|---|---|---|
| $w_X$ = | 194.9 | plf |
| $w_Y$ = | 162.4 | plf |
| $v_{Length}$ = | 97.5 | plf |
| $v_{Width}$ = | 81.2 | plf |
| $v_{Left}$ = | 97.5 | plf |
| $v_{Top}$ = | 81.2 | plf |
| $v_{Right}$ = | 97.5 | plf |
| $v_{Bottom}$ = | 81.2 | plf |
| **$P_{ChordX}$ =** | **0.7** | **kip** |
| **$P_{ChordY}$ =** | **0.7** | **kip** |

| Wind | | |
|---|---|---|
| $w_X$ = | 81.6 | plf |
| $w_Y$ = | 56.7 | plf |
| $v_{Length}$ = | 34.0 | plf |
| $v_{Width}$ = | 34.0 | plf |
| $v_{Left}$ = | 34.0 | plf |
| $v_{Top}$ = | 34.0 | plf |
| $v_{Right}$ = | 34.0 | plf |
| $v_{Bottom}$ = | 34.0 | plf |
| **$P_{ChordX}$ =** | **0.60** | **kip** |
| **$P_{ChordY}$ =** | **0.60** | **kip** |

Allowable Uniform Load

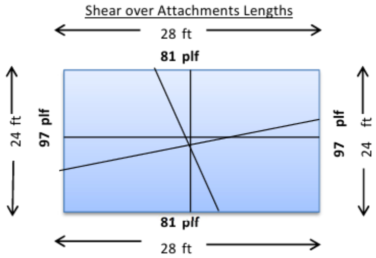
| | | | |
|---|---|---|---|
| Profile | Sheathing | | |
| Span | MULTIPLE | | |
| Thickness | 15/32 | | |
| Span | 1.3 | ft | |
| Allow. Uplift Load | 35 | psf | OK |

Allowable Diaphragm Shear Strength

| | | |
|---|---|---|
| Panel Edge | 6 | " O.C. |
| Panel Field | 12 | " O.C. |
| Nail | 10D | |
| Allow. Diaph. Shear | 180 | plf |

| | | |
|---|---|---|
| Left Shear Ratio | 0.54 | OK |
| Top Shear Ratio | 0.45 | OK |
| Right Shear Ratio | 0.54 | OK |
| Bottom Shear Ratio | 0.45 | OK |

Shear over Attachments Lengths




| | | |
|---|---|---|
| Diaph. Shear Stiffness, G' | 5 | kip/in |
| Flexibility Factor, F | 200 | $\mu$-in/lb |

Diaphragm Deflection

| | | | |
|---|---|---|---|
| X-Dir | $\Delta_w$ = | 0.095 | in |
| | $\Delta_f$ = | 1E-04 | in |
| | $\Delta_{Total}$ = | **0.096** | in |
| Y-Dir | $\Delta_w$ = | 0.137 | in |
| | $\Delta_f$ = | 2E-04 | in |
| | $\Delta_{Total}$ = | **0.138** | in |

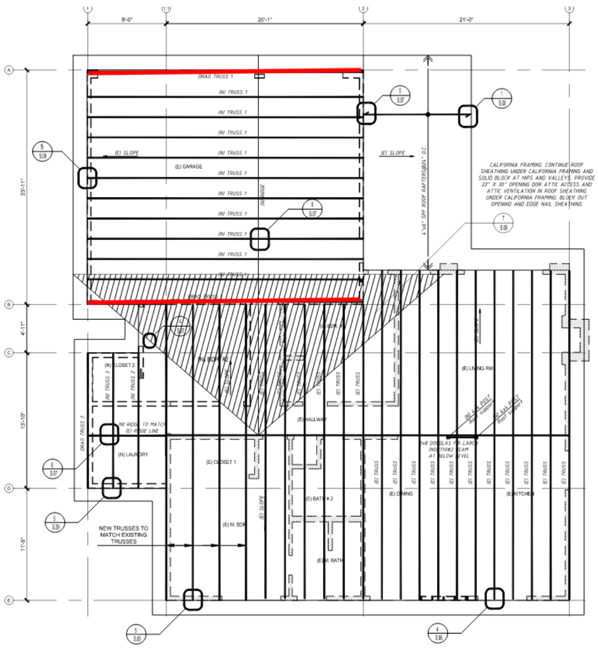



*Figure 3-5: Key plan*

- Horizontal Diaphragm forces

  97p/f x 24ft = 2328 lb

- Vertical Diaphragm forces

  81p/f x 28ft = 2268 lb

## 3.5 DESIGNING OF DRAG FORCE-2

- Roof Diaphragm calculation for drag truss design.

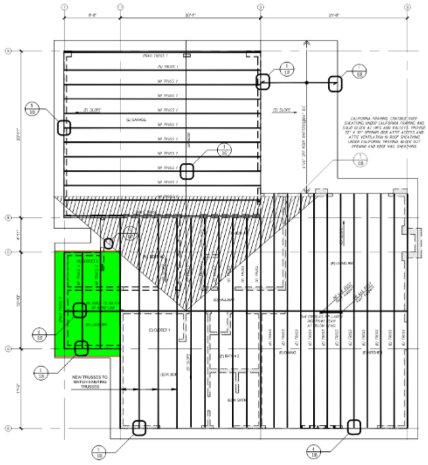


*Figure 3-6: Key plan*

**INPUT DATA & ANALYSIS**

| | | |
|---|---|---|
| Width | 8 | ft |
| Length | 13.8 | ft |
| Joist Spacing | 1.33 | ft |
| X-Dir Chord Area | 8.25 | $in^2$ |
| Y-Dir Chord Area | 8.25 | $in^2$ |
| Superimposed Loads | 27 | psf |
| Net Uplift Loads | 8 | psf |
| Seismic Factor | 1 | |
| Wind Factor | 1 | |
| Joist Direction | Parallel to Left and Right | |

| | | |
|---|---|---|
| Seismic Diaphragm Force X-Dir, $F_{pxE}$ = | 0.83 | kip |
| Seismic Diaphragm Force Y-Dir, $F_{pyE}$ = | 0.76 | kip |
| Wind Diaphragm Force X-Dir, $F_{pxW}$ = | 0.94 | kip |
| Wind Diaphragm Force Y-Dir, $F_{pyW}$ = | 0.54 | kip |
| Attachment Length Left, $L_{left}$ = | 8.00 | ft |
| Attachment Length Top, $L_{top}$ = | 13.80 | ft |
| Attachment Length Right, $L_{right}$ = | 8.00 | ft |
| Attachment Length Bottom, $L_{bot}$ = | 13.80 | ft |
| Deflection, L/ | 240 | |

Joists Span Up and Down

**Seismic**

| | | |
|---|---|---|
| $w_X$ = | 103.4 | plf |
| $w_Y$ = | 55.3 | plf |
| $\nu_{Length}$ = | 47.7 | plf |
| $\nu_{Width}$ = | 30.0 | plf |
| $\nu_{Left}$ = | 47.7 | plf |
| $\nu_{Top}$ = | 30.0 | plf |
| $\nu_{Right}$ = | 47.7 | plf |
| $\nu_{Bottom}$ = | 30.0 | plf |
| **$P_{ChordX}$ =** | **0.7** | **kip** |
| **$P_{ChordY}$ =** | **0.7** | **kip** |

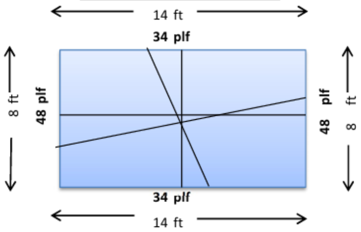
**Wind**

| | | |
|---|---|---|
| $w_X$ = | 117.3 | plf |
| $w_Y$ = | 39.4 | plf |
| $\nu_{Length}$ = | 34.0 | plf |
| $\nu_{Width}$ = | 34.0 | plf |
| $\nu_{Left}$ = | 34.0 | plf |
| $\nu_{Top}$ = | 34.0 | plf |
| $\nu_{Right}$ = | 34.0 | plf |
| $\nu_{Bottom}$ = | 34.0 | plf |
| **$P_{ChordX}$ =** | **0.60** | **kip** |
| **$P_{ChordY}$ =** | **0.60** | **kip** |

Allowable Uniform Load

| | | |
|---|---|---|
| Profile | Sheathing | |
| Span | MULTIPLE | |
| Thickness | 15/32 | |
| Span | 1.3 | ft |
| Allow. Uplift Load | 35 | psf OK |

Shear over Attachments Lengths




Allowable Diaphragm Shear Strength

| | | |
|---|---|---|
| Panel Edge | 6 | " O.C. |
| Panel Field | 12 | " O.C. |
| Nail | 10D | |
| Allow. Diaph. Shear | 180 | plf |

| | | |
|---|---|---|
| Diaph. Shear Stiffness, G' | 5 | kip/in |
| Flexibility Factor, F | 200 | $\mu$-in/lb |

| | | |
|---|---|---|
| Left Shear Ratio | 0.26 | OK |
| Top Shear Ratio | 0.19 | OK |
| Right Shear Ratio | 0.26 | OK |
| Bottom Shear Ratio | 0.19 | OK |

Diaphragm Deflection

| | | | |
|---|---|---|---|
| X-Dir | $\Delta_w$ = | 0.014 | in |
| | $\Delta_f$ = | 3E-06 | in |
| | $\Delta_{Total}$ = | **0.014** | in |
| Y-Dir | $\Delta_w$ = | 0.033 | in |
| | $\Delta_f$ = | 4E-05 | in |
| | $\Delta_{Total}$ = | **0.033** | in |

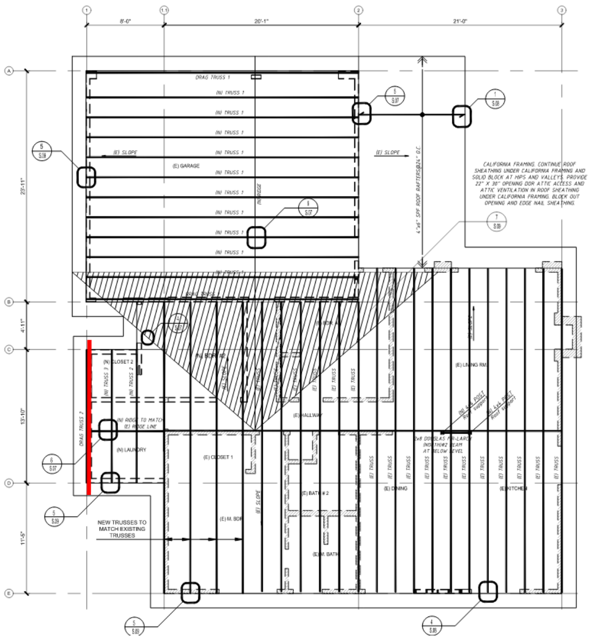


*Figure 3-7: Key plan*

- Horizontal Diaphragm forces

  48p/f x 8ft = 384 lb

- Vertical Diaphragm forces

  34p/f x 14ft = 476 lb

## 3.6 DESIGNING OF STRIP FOUDATION

- Proposed Member: 12” (width) x 12” (depth)

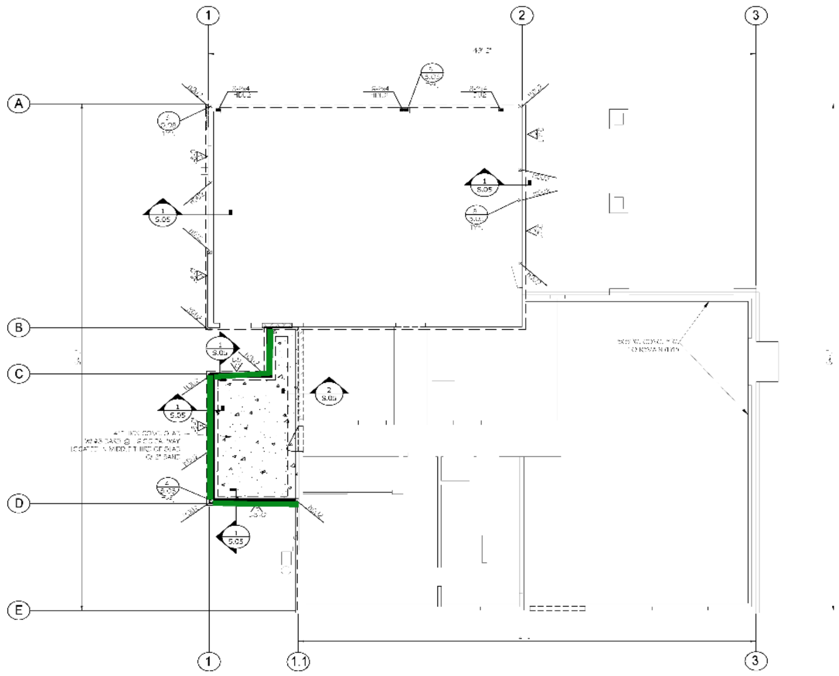



*Figure 3-8- Key Plan*

*Table 3-7- Foundation Loading*

| Element | Loading | | | | |
|---|---|---|---|---|---|
| | S. Dead (lb) | Live (lb) | Snow (lb) | Wind (lb/ft) | Seismic (lb/ft) |
| **Foundation** | 108.0 | 80.0 | - | 256.0 | 50.77 |

*Table 3-8- Design Summary*

| | | Applied | Resisting | FS (*Utiliz.) | |
|---|---|---|---|---|---|
| Uplift verification | kips | 0.5 | | | ✓ |
| Soil bearing | ksf | 0.46 | 1.5 | 0.307* | ✓ |

| | | Required | Provided | Utiliz. | | Note |
|---|---|---|---|---|---|---|
| Min. area of reinf.,bot. | $in^2$ | 0.000 | 0.440 | | ✓ | |
| Max. reinf. spacing, | in | 18.0 | 3.0 | | ✓ | |

**Refer Annex D for detailed calculation.**

## 3.7 DESIGNING OF FLOOR SLAB.

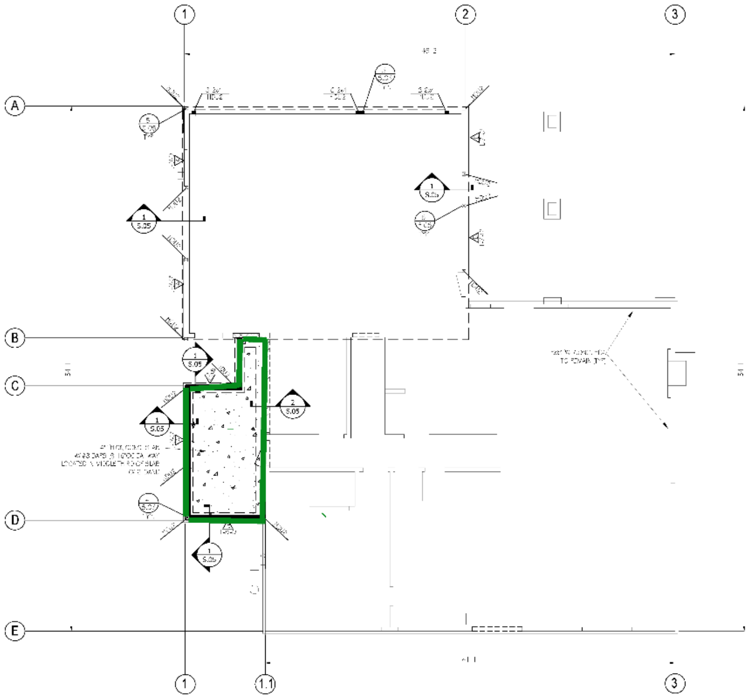



*Figure 3-9-Key Plan*

| | |
|---|---|
| Total Floor Live Load | = 105 psf |
| Compressive strength of concrete | = 2500 PSi |
| Allowable fiber strength (S) | $= 1.5 * 6 * \sqrt{2500}$ |
| | = 450 |
| W | $= 257.876 * S * \sqrt{\frac{Kh}{E}}$ |
| | $= 257.876 * 450 * \sqrt{\frac{100*4}{4*10^6}}$ |
| | = 1160.4 Psf > 105 psf |

So, 4’’ thick slab is good.

# ANNEX A

## WOOD SHEAR WALL DESIGN (NDS)-STUD WALL-1

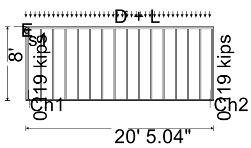
**In accordance with NDS2018 allowable stress design and the segmented shear wall method**

Tedds calculation version 1.2.04

**Panel details**

gypsum sheathing sheathing on both sides

| | |
|---|---|
| Panel height | h = **8** ft |
| Panel length | b = **20.42** ft |
| Total area of wall | A = h × b = **163.36** ft$^2$ |




**Panel construction**

| | |
|---|---|
| Nominal stud size | 2" x 6" |
| Dressed stud size | 1.5" x 5.5" |
| Cross-sectional area of studs | $A_s$ = **8.25** in$^2$ |
| Stud spacing | s = **16** in |
| Nominal end post size | 2 x 2" x 6" |
| Dressed end post size | 2 x 1.5" x 5.5" |
| Cross-sectional area of end posts | $A_e$ = **16.5** in$^2$ |
| Hole diameter | Dia = **1** in |
| Net cross-sectional area of end posts | $A_{en}$ = **13.5** in$^2$ |
| Nominal collector size | 2 x 2" x 6" |
| Dressed collector size | 2 x 1.5" x 5.5" |
| Service condition | Dry |
| Temperature | 100 degF or less |
| Vertical anchor stiffness | $k_a$ = **30000** lb/in |

**From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)**

| | |
|---|---|
| Species, grade and size classification | Douglas Fir-Larch (North), no.2 grade, 2" & wider |
| Specific gravity | G = **0.49** |
| Tension parallel to grain | $F_t$ = **500** lb/in$^2$ |
| Compression parallel to grain | $F_c$ = **1400** lb/in$^2$ |
| Modulus of elasticity | E = **1600000** lb/in$^2$ |
| Minimum modulus of elasticity | $E_{min}$ = **580000** lb/in$^2$ |

**Sheathing details**

Sheathing material 1/2" x 4' gypsum sheathing with blocking

Fastener type 0.120" nails at 4"centers

**From SDPWS Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Gypsum and Portland Cement Plaster**

Nominal unit shear capacity for seismic design $v_s$ = **350** lb/ft

Nominal unit shear capacity for wind design $v_w$ = **350** lb/ft

Apparent shear wall shear stiffness $G_a$ = **8.5** kips/in

**Combined unit shear capacities**

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 \times v_s$ = **700** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 \times v_w$ = **700** lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2}$ = **17** kips/in

**Loading details**

Dead load acting on top of panel D = **454.4** lb/ft

Floor live load acting on top of panel $L_f$ = **284** lb/ft

Self weight of panel $S_{wt}$ = **17** lb/ft$^2$

In plane seismic load acting at head of panel $E_q$ = **1053.82** lbs

Design spectral response accel. par., short periods $S_{DS}$ = **1.2**

**From IBC 2018 cl.1605.3.1 Basic load combinations**

| | |
|---|---|
| Load combination no.1 | D + 0.6W |
| Load combination no.2 | D + 0.7E |
| Load combination no.3 | D + 0.45W + 0.75$L_f$ + 0.75($L_r$ or S or R) |
| Load combination no.4 | D + 0.525E + 0.75$L_f$ + 0.75S |
| Load combination no.5 | 0.6D + 0.6W |
| Load combination no.6 | 0.6D + 0.7E |

**Adjustment factors**

Load duration factor – Table 2.3.2 $C_D$ = **1.60**

Size factor for tension – Table 4A $C_{Ft}$ = **1.30**

Size factor for compression – Table 4A $C_{Fc}$ = **1.10**

Wet service factor for tension – Table 4A $C_{Mt}$ = **1.00**

Wet service factor for compression – Table 4A $C_{Mc}$ = **1.00**

Wet service factor for modulus of elasticity – Table 4A

$C_{ME}$ = **1.00**

Temperature factor for tension – Table 2.3.3 $C_{tt}$ = **1.00**

Temperature factor for compression – Table 2.3.3

$C_{tc}$ = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

$C_{tE}$ = **1.00**

Incising factor – cl.4.3.8 $C_i$ = **1.00**

Buckling stiffness factor – cl.4.4.2 $C_T$ = **1.00**

Adjusted modulus of elasticity $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1565** psi

Reference compression design value $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2464** psi

For sawn lumber c = **0.8**Column stability factor – eqn.3.7-1 $C_P$ = $(1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{[(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c}$ = **0.52**

**From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios**

| | |
|---|---|
| Maximum shear wall aspect ratio | 2 |
| Shear wall length | b = **20.42** ft |
| Shear wall aspect ratio | h / b = **0.392** |

**Segmented shear wall capacity**

| | |
|---|---|
| Maximum shear force under seismic loading | $V_{s\_max} = 0.7 \times E_q =$ **0.738** kips |
| Shear capacity for seismic loading | $V_s = v_{sc} \times b / 2 =$ **7.147** kips |
| | $V_{s\_max} / V_s =$ **0.103** |

***PASS - Shear capacity for seismic load exceeds maximum shear force***

**Chord capacity for chords 1 and 2**

| | |
|---|---|
| Shear wall aspect ratio | h / b = **0.392** |
| Load combination 6 | |
| Shear force for maximum tension | $V = 0.7 \times E_q =$ **0.738** kips |
| Axial force for maximum tension | $P = (0.6 \times (D + S_{wt} \times h) - 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2 =$ **0.17** kips |
| Maximum tensile force in chord | $T = V \times h / (b) - P =$ **0.119** kips |
| Maximum applied tensile stress | $f_t = T / A_{en} =$ **9** lb/in$^2$ |
| Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1040** lb/in$^2$ |
| | $f_t / F_t' =$ **0.008** |

***PASS - Design tensile stress exceeds maximum applied tensile stress***

| | |
|---|---|
| Load combination 4 | |
| Shear force for maximum compression | $V = 0.525 \times E_q =$ **0.553** kips |
| Axial force for maximum compression | $P = ((D + S_{wt} \times h) + 0.525 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h) + 0.75 \times L_f) \times s / 2 =$ **0.585** kips |
| Maximum compressive force in chord | $C = V \times h / (b) + P =$ **0.802** kips |
| Maximum applied compressive stress | $f_c = C / A_e =$ **49** lb/in$^2$ |
| Design compressive stress | $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1285** lb/in$^2$ |
| | $f_c / F_c' =$ **0.038** |

***PASS - Design compressive stress exceeds maximum applied compressive stress***

**Hold down force**

| | |
|---|---|
| Chord 1 | $T_1 =$ **0.119** kips |
| Chord 2 | $T_2 =$ **0.119** kips |

**Seismic deflection**

| | |
|---|---|
| Design shear force | $V_{\delta s} = E_q =$ **1.054** kips |
| Deflection limit | $\Delta_{s\_allow} = 0.020 \times h =$ **1.92** in |
| Induced unit shear | $v_{\delta s} = V_{\delta s} / b =$ **51.61** lb/ft |
| Anchor tension force | $T_\delta = \max(0 \text{ kips}, v_{\delta s} \times h - (0.6 - 0.2 \times S_{DS}) \times (D + S_{wt} \times h) \times s / 2) =$ **0.271** kips |
| Shear wall elastic deflection – Eqn. 4.3-1 | $\delta_{swse} = 2 \times v_{\delta s} \times h^3 / (3 \times E \times A_e \times b) + v_{\delta s} \times h / (G_{ac}) + h \times T_\delta / (k_a \times b) =$ **0.028** in |
| Deflection ampification factor | $C_{d\delta} =$ **4** |
| Seismic importance factor | $I_e =$ **1.25** |
| Amp. seis. deflection – ASCE7 Eqn. 12.8-15 | $\delta_{sws} = C_{d\delta} \times \delta_{swse} / I_e =$ **0.09** in |
| | $\delta_{sws} / \Delta_{s\_allow} =$ **0.047** |

***PASS - Shear wall deflection is less than deflection limit***

# ANNEX B

## WOOD SHEAR WALL DESIGN (NDS)-STUD WALL -2

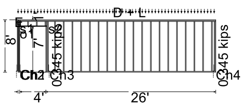
**In accordance with NDS2018 allowable stress design and the segmented shear wall method**

Tedds calculation version 1.2.04

**Panel details**

gypsum sheathing sheathing on both sides

| | |
|---|---|
| Panel height | h = **8** ft |
| Panel length | b = **30.25** ft |




**Panel opening details**

| | |
|---|---|
| Width of opening | $w_{o1}$ = **4** ft |
| Height of opening | $h_{o1}$ = **7** ft |
| Height to underside of lintel over opening | $l_{o1}$ = **7** ft |
| Position of opening | $P_{o1}$ = **0.25** ft |
| Total area of wall | $A = h \times b - w_{o1} \times h_{o1}$ = **214** ft$^2$ |

**Panel construction**

| | |
|---|---|
| Nominal stud size | 2" x 6" |
| Dressed stud size | 1.5" x 5.5" |
| Cross-sectional area of studs | $A_s$ = **8.25** in$^2$ |
| Stud spacing | s = **16** in |
| Nominal end post size | 2 x 2" x 6" |
| Dressed end post size | 2 x 1.5" x 5.5" |
| Cross-sectional area of end posts | $A_e$ = **16.5** in$^2$ |
| Hole diameter | Dia = **1** in |
| Net cross-sectional area of end posts | $A_{en}$ = **13.5** in$^2$ |
| Nominal collector size | 2 x 2" x 6" |
| Dressed collector size | 2 x 1.5" x 5.5" |
| Service condition | Dry |
| Temperature | 100 degF or less |
| Vertical anchor stiffness | $k_a$ = **30000** lb/in |

**From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)**

| | |
|---|---|
| Species, grade and size classification | Douglas Fir-Larch (North), no.2 grade, 2" & wider |
| Specific gravity | G = **0.49** |
| Tension parallel to grain | $F_t$ = **500** lb/in$^2$ |
| Compression parallel to grain | $F_c$ = **1400** lb/in$^2$ |
| Modulus of elasticity | E = **1600000** lb/in$^2$ |
| Minimum modulus of elasticity | $E_{min}$ = **580000** lb/in$^2$ |

**Sheathing details**

Sheathing material — 1/2" x 4' gypsum sheathing with blocking

Fastener type — 0.120" nails at 4"centers

**From SDPWS Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Gypsum and Portland Cement Plaster**

Nominal unit shear capacity for seismic design $v_s$ = **350** lb/ft

Nominal unit shear capacity for wind design $v_w$ = **350** lb/ft

Apparent shear wall shear stiffness $G_a$ = **8.5** kips/in

**Combined unit shear capacities**

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 \times v_s$ = **700** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 \times v_w$ = **700** lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2}$ = **17** kips/in

**Loading details**

Dead load acting on top of panel — D = **208** lb/ft

Floor live load acting on top of panel — $L_f$ = **130** lb/ft

Self weight of panel — $S_{wt}$ = **17** lb/ft$^2$

In plane seismic load acting at head of panel — $E_q$ = **2060.63** lbs

Design spectral response accel. par., short periods — $S_{DS}$ = **1.2**

**From IBC 2018 cl.1605.3.1 Basic load combinations**

Load combination no.1 — D + 0.6W

Load combination no.2 — D + 0.7E

Load combination no.3 — D + 0.45W + 0.75$L_f$ + 0.75($L_r$ or S or R)

Load combination no.4 — D + 0.525E + 0.75$L_f$ + 0.75S

Load combination no.5 — 0.6D + 0.6W

Load combination no.6 — 0.6D + 0.7E

**Adjustment factors**

Load duration factor – Table 2.3.2 — $C_D$ = **1.60**

Size factor for tension – Table 4A — $C_{Ft}$ = **1.30**

Size factor for compression – Table 4A — $C_{Fc}$ = **1.10**

Wet service factor for tension – Table 4A — $C_{Mt}$ = **1.00**

Wet service factor for compression – Table 4A — $C_{Mc}$ = **1.00**

Wet service factor for modulus of elasticity – Table 4A

$C_{ME}$ = **1.00**

Temperature factor for tension – Table 2.3.3 — $C_{tt}$ = **1.00**

Temperature factor for compression – Table 2.3.3

$C_{tc}$ = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

$C_{tE}$ = **1.00**

Incising factor – cl.4.3.8 — $C_i$ = **1.00**

Buckling stiffness factor – cl.4.4.2 — $C_T$ = **1.00**

Adjusted modulus of elasticity — $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi

Critical buckling design value — $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1565** psi

Reference compression design value — $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2464** psi

For sawn lumber — c = **0.8**

| | |
|---|---|
| Column stability factor – eqn.3.7-1 | $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) – \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.52** |

**From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios**

| | |
|---|---|
| Maximum shear wall aspect ratio | 2 |
| Segment 1 wall length | $b_1$ = **0.25** ft |
| Shear wall aspect ratio | $h / b_1$ = **32** |
| Segment 2 wall length | $b_2$ = **26** ft |
| Shear wall aspect ratio | $h / b_2$ = **0.308** |

**Segmented shear wall capacity - Strength distribution method**

| | |
|---|---|
| Maximum shear force under seismic loading | $V_{s\_max} = 0.7 \times E_q$ = **1.442** kips |
| Shear capacity for seismic loading | $V_s = v_{sc} \times (b_2) / 2$ = **9.1** kips |
| | $V_{s\_max} / V_s$ = **0.159** |

***PASS - Shear capacity for seismic load exceeds maximum shear force***

**Chord capacity for chords 1 and 2**

| | |
|---|---|
| Shear wall aspect ratio | $h / b_1$ = **32** |

***Segment not considered, shear wall aspect ratio exceeds maximum allowable.***

**Chord capacity for chords 3 and 4**

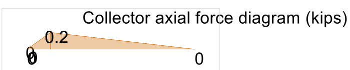
| | |
|---|---|
| Shear wall aspect ratio | $h / b_2$ = **0.308** |
| Load combination 6 | |
| Shear force for maximum tension | $V = 0.7 \times E_q$ = **1.442** kips |
| Axial force for maximum tension | $P = (0.6 \times (D + S_{wt} \times h) - 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2$ = **0.099** kips |
| Maximum tensile force in chord | $T = V \times (b_2 / (b_2)) \times (h / b_2) - P$ = **0.345** kips |
| Maximum applied tensile stress | $f_t = T / A_{en}$ = **26** lb/in$^2$ |
| Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1040** lb/in$^2$ |
| | $f_t / F_t'$ = **0.025** |

***PASS - Design tensile stress exceeds maximum applied tensile stress***

| | |
|---|---|
| Load combination 2 | |
| Shear force for maximum compression | $V = 0.7 \times E_q$ = **1.442** kips |
| Axial force for maximum compression | $P = ((D + S_{wt} \times h) + 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2$ = **0.268** kips |
| Maximum compressive force in chord | $C = V \times (b_2 / (b_2)) \times (h / b_2) + P$ = **0.712** kips |
| Maximum applied compressive stress | $f_c = C / A_e$ = **43** lb/in$^2$ |
| Design compressive stress | $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **1285** lb/in$^2$ |
| | $f_c / F_c'$ = **0.034** |

***PASS - Design compressive stress exceeds maximum applied compressive stress***

**Collector capacity**



Collector seismic design force factor $F_{Coll}$ = **1**

Maximum shear force on wall $V_{max} = F_{Coll} \times V_{s\_max}$ = **1.442** kips

Maximum force in collector $P_{coll}$ = **0.203** kips

Maximum applied tensile stress $f_t = P_{coll} / (2 \times A_s)$ = **12** lb/in$^2$

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1040** lb/in$^2$

$f_t / F_t'$ = **0.012**

***PASS - Design tensile stress exceeds maximum applied tensile stress***

Maximum applied compressive stress $f_c = P_{coll} / (2 \times A_s)$ = **12** lb/in$^2$

Column stability factor $C_P$ = **1.00**

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2464** lb/in$^2$

$f_c / F_c'$ = **0.005**

***PASS - Design compressive stress exceeds maximum applied compressive stress***

**Hold down force**

Chord 3 $T_3$ = **0.345** kips

Chord 4 $T_4$ = **0.345** kips

***DESIGN WARNING - Design using the strength distribution method to distribute shear loads to individual shear wall segments does not include a deflection check as results to that method are not reliable. Under seismic loads, a drift check is required as part of the design. Suggest using the equal deflections load distribution method to include the seismic drift check in this design.***

# ANNEX C

- **WOOD SHEAR WALL DESIGN (NDS)-STUD WALL-3**

**In accordance with NDS2018 allowable stress design and the segmented shear wall method**

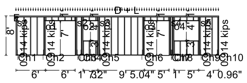
Tedds calculation version 1.2.04

**Panel details**

gypsum sheathing sheathing on both sides

Panel height $h$ = **8** ft

Panel length $b$ = **41.1** ft



**Panel opening details**

Width of opening $w_{o1}$ = **6** ft

Height of opening $h_{o1}$ = **7** ft

Height to underside of lintel over opening $l_{o1}$ = **7** ft

Position of opening $P_{o1}$ = **6** ft

Width of opening $w_{o2}$ = **3** ft

Height of opening $h_{o2}$ = **4** ft

Height to underside of lintel over opening $l_{o2}$ = **7** ft

Position of opening $P_{o2}$ = **13.6** ft

Width of opening $w_{o3}$ = **5** ft

Height of opening $h_{o3}$ = **7** ft

Height to underside of lintel over opening $l_{o3}$ = **7** ft

Position of opening $P_{o3}$ = **26.02** ft

Width of opening $w_{o4}$ = **5** ft

Height of opening $h_{o4}$ = **4** ft

Height to underside of lintel over opening $l_{o4}$ = **7** ft

Position of opening $P_{o4}$ = **32.02** ft

Total area of wall $A = h \times b - w_{o1} \times h_{o1} - w_{o2} \times h_{o2} - w_{o3} \times h_{o3} - w_{o4} \times h_{o4}$ = **219.8** ft$^2$

**Panel construction**

Nominal stud size 2" x 6"

Dressed stud size 1.5" x 5.5"

Cross-sectional area of studs $A_s$ = **8.25** in$^2$

Stud spacing $s$ = **16** in

Nominal end post size 2 x 2" x 6"

Dressed end post size 2 x 1.5" x 5.5"

Cross-sectional area of end posts $A_e$ = **16.5** in$^2$

Hole diameter Dia = **1** in

Net cross-sectional area of end posts $A_{en}$ = **13.5** in$^2$

Nominal collector size 2 x 2" x 6"

Dressed collector size 2 x 1.5" x 5.5"

Service condition Dry

Temperature 100 degF or less

Vertical anchor stiffness $k_a$ = **30000** lb/in

**From NDS Supplement Table 4A - Reference design values for visually graded dimension lumber (2" - 4" thick)**

Species, grade and size classification Douglas Fir-Larch (North), no.2 grade, 2" & wider
Specific gravity G = **0.49**
Tension parallel to grain $F_t$ = **500** lb/in$^2$
Compression parallel to grain $F_c$ = **1400** lb/in$^2$
Modulus of elasticity E = **1600000** lb/in$^2$
Minimum modulus of elasticity $E_{min}$ = **580000** lb/in$^2$

**Sheathing details**

Sheathing material 1/2" x 4' gypsum sheathing with blocking
Fastener type 0.120" nails at 4"centers

**From SDPWS Table 4.3C Nominal Unit Shear Capacities for Wood-Frame Shear Walls - Gypsum and Portland Cement Plaster**

Nominal unit shear capacity for seismic design $v_s$ = **350** lb/ft
Nominal unit shear capacity for wind design $v_w$ = **350** lb/ft
Apparent shear wall shear stiffness $G_a$ = **8.5** kips/in

**Combined unit shear capacities**

Combined nominal unit shear capacity for seismic design

$v_{sc} = 2 \times v_s$ = **700** lb/ft

Combined nominal unit shear capacity for wind design

$v_{wc} = 2 \times v_w$ = **700** lb/ft

Combined apparent shear wall shear stiffness $G_{ac} = G_{a1} + G_{a2}$ = **17** kips/in

**Loading details**

Dead load acting on top of panel D = **265.6** lb/ft
Floor live load acting on top of panel $L_f$ = **166** lb/ft
Self weight of panel $S_{wt}$ = **17** lb/ft$^2$
In plane seismic load acting at head of panel $E_q$ = **3585.56** lbs
Design spectral response accel. par., short periods $S_{DS}$ = **1.2**

**From IBC 2018 cl.1605.3.1 Basic load combinations**

Load combination no.1 D + 0.6W
Load combination no.2 D + 0.7E
Load combination no.3 D + 0.45W + 0.75$L_f$ + 0.75($L_r$ or S or R)
Load combination no.4 D + 0.525E + 0.75$L_f$ + 0.75S
Load combination no.5 0.6D + 0.6W
Load combination no.6 0.6D + 0.7E

**Adjustment factors**

Load duration factor – Table 2.3.2 $C_D$ = **1.60**
Size factor for tension – Table 4A $C_{Ft}$ = **1.30**
Size factor for compression – Table 4A $C_{Fc}$ = **1.10**
Wet service factor for tension – Table 4A $C_{Mt}$ = **1.00**
Wet service factor for compression – Table 4A $C_{Mc}$ = **1.00**
Wet service factor for modulus of elasticity – Table 4A

$C_{ME}$ = **1.00**

Temperature factor for tension – Table 2.3.3 $C_{tt}$ = **1.00**
Temperature factor for compression – Table 2.3.3

$C_{tc}$ = **1.00**

Temperature factor for modulus of elasticity – Table 2.3.3

| | |
|---|---|
| | $C_{tE}$ = **1.00** |
| Incising factor – cl.4.3.8 | $C_i$ = **1.00** |
| Buckling stiffness factor – cl.4.4.2 | $C_T$ = **1.00** |
| Adjusted modulus of elasticity | $E_{min}' = E_{min} \times C_{ME} \times C_{tE} \times C_i \times C_T$ = **580000** psi |
| Critical buckling design value | $F_{cE} = 0.822 \times E_{min}' / (h / d)^2$ = **1565** psi |
| Reference compression design value | $F_c^* = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i$ = **2464** psi |
| For sawn lumber | c = **0.8** |
| Column stability factor – eqn.3.7-1 | $C_P = (1 + (F_{cE} / F_c^*)) / (2 \times c) - \sqrt{([(1 + (F_{cE} / F_c^*)) / (2 \times c)]^2 - (F_{cE} / F_c^*) / c)}$ = **0.52** |

**From SDPWS Table 4.3.4 Maximum Shear Wall Aspect Ratios**

| | |
|---|---|
| Maximum shear wall aspect ratio | 2 |
| Segment 1 wall length | $b_1$ = **6** ft |
| Shear wall aspect ratio | $h / b_1$ = **1.333** |
| Segment 2 wall length | $b_2$ = **1.6** ft |
| Shear wall aspect ratio | $h / b_2$ = **5** |
| Segment 3 wall length | $b_3$ = **9.42** ft |
| Shear wall aspect ratio | $h / b_3$ = **0.849** |
| Segment 4 wall length | $b_4$ = **1** ft |
| Shear wall aspect ratio | $h / b_4$ = **8** |
| Segment 5 wall length | $b_5$ = **4.08** ft |
| Shear wall aspect ratio | $h / b_5$ = **1.961** |

**Segmented shear wall capacity - Strength distribution method**

| | |
|---|---|
| Maximum shear force under seismic loading | $V_{s\_max} = 0.7 \times E_q$ = **2.51** kips |
| Shear capacity for seismic loading | $V_s = v_{sc} \times (b_1 + b_3 + b_5) / 2$ = **6.825** kips |
| | $V_{s\_max} / V_s$ = **0.368** |

***PASS - Shear capacity for seismic load exceeds maximum shear force***

**Chord capacity for chords 1 and 2**

| | |
|---|---|
| Shear wall aspect ratio | $h / b_1$ = **1.333** |
| Load combination 6 | |
| Shear force for maximum tension | $V = 0.7 \times E_q$ = **2.51** kips |
| Axial force for maximum tension | $P = (0.6 \times (D + S_{wt} \times h) - 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2$ = **0.116** kips |
| Maximum tensile force in chord | $T = V \times (b_1 / (b_1 + b_3 + b_5)) \times (h / b_1) - P$ = **0.914** kips |
| Maximum applied tensile stress | $f_t = T / A_{en}$ = **68** lb/in$^2$ |
| Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1040** lb/in$^2$ |
| | $f_t / F_t'$ = **0.065** |

***PASS - Design tensile stress exceeds maximum applied tensile stress***

| | |
|---|---|
| Load combination 2 | |
| Shear force for maximum compression | $V = 0.7 \times E_q$ = **2.51** kips |
| Axial force for maximum compression | $P = ((D + S_{wt} \times h) + 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2$ = **0.313** kips |
| Maximum compressive force in chord | $C = V \times (b_1 / (b_1 + b_3 + b_5)) \times (h / b_1) + P$ = **1.342** kips |
| Maximum applied compressive stress | $f_c = C / A_e$ = **81** lb/in$^2$ |
| Design compressive stress | $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **1285** lb/in$^2$ |
| | $f_c / F_c'$ = **0.063** |

***PASS - Design compressive stress exceeds maximum applied compressive stress***

**Chord capacity for chords 3 and 4**

Shear wall aspect ratio $h / b_2 =$ **5**

***Segment not considered, shear wall aspect ratio exceeds maximum allowable.***

**Chord capacity for chords 5 and 6**

Shear wall aspect ratio $h / b_3 =$ **0.849**

Load combination 6

Shear force for maximum tension $V = 0.7 \times E_q =$ **2.51** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h) - 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2 =$ **0.116** kips

Maximum tensile force in chord $T = V \times (b_3 / (b_1 + b_3 + b_5)) \times (h / b_3) - P =$ **0.914** kips

Maximum applied tensile stress $f_t = T / A_{en} =$ **68** lb/in$^2$

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1040** lb/in$^2$

$f_t / F_t' =$ **0.065**

***PASS - Design tensile stress exceeds maximum applied tensile stress***

Load combination 2

Shear force for maximum compression $V = 0.7 \times E_q =$ **2.51** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h) + 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2 =$ **0.313** kips

Maximum compressive force in chord $C = V \times (b_3 / (b_1 + b_3 + b_5)) \times (h / b_3) + P =$ **1.342** kips

Maximum applied compressive stress $f_c = C / A_e =$ **81** lb/in$^2$

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1285** lb/in$^2$

$f_c / F_c' =$ **0.063**

***PASS - Design compressive stress exceeds maximum applied compressive stress***

**Chord capacity for chords 7 and 8**

Shear wall aspect ratio $h / b_4 =$ **8**

***Segment not considered, shear wall aspect ratio exceeds maximum allowable.***

**Chord capacity for chords 9 and 10**

Shear wall aspect ratio $h / b_5 =$ **1.961**

Load combination 6

Shear force for maximum tension $V = 0.7 \times E_q =$ **2.51** kips

Axial force for maximum tension $P = (0.6 \times (D + S_{wt} \times h) - 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2 =$ **0.116** kips

Maximum tensile force in chord $T = V \times (b_5 / (b_1 + b_3 + b_5)) \times (h / b_5) - P =$ **0.914** kips

Maximum applied tensile stress $f_t = T / A_{en} =$ **68** lb/in$^2$

Design tensile stress $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i =$ **1040** lb/in$^2$

$f_t / F_t' =$ **0.065**

***PASS - Design tensile stress exceeds maximum applied tensile stress***

Load combination 2

Shear force for maximum compression $V = 0.7 \times E_q =$ **2.51** kips

Axial force for maximum compression $P = ((D + S_{wt} \times h) + 0.7 \times 0.2 \times S_{DS} \times (D + S_{wt} \times h)) \times s / 2 =$ **0.313** kips

Maximum compressive force in chord $C = V \times (b_5 / (b_1 + b_3 + b_5)) \times (h / b_5) + P =$ **1.342** kips

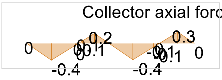
Maximum applied compressive stress $f_c = C / A_e =$ **81** lb/in$^2$

Design compressive stress $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P =$ **1285** lb/in$^2$

$f_c / F_c' =$ **0.063**

***PASS - Design compressive stress exceeds maximum applied compressive stress***

**Collector capacity**




| | |
|---|---|
| Collector seismic design force factor | $F_{Coll}$ = **1** |
| Maximum shear force on wall | $V_{max} = F_{Coll} \times V_{s\_max}$ = **2.51** kips |
| Maximum force in collector | $P_{coll}$ = **0.406** kips |
| Maximum applied tensile stress | $f_t = P_{coll} / (2 \times A_s)$ = **25** lb/in$^2$ |
| Design tensile stress | $F_t' = F_t \times C_D \times C_{Mt} \times C_{tt} \times C_{Ft} \times C_i$ = **1040** lb/in$^2$ |
| | $f_t / F_t'$ = **0.024** |

***PASS - Design tensile stress exceeds maximum applied tensile stress***

| | |
|---|---|
| Maximum applied compressive stress | $f_c = P_{coll} / (2 \times A_s)$ = **25** lb/in$^2$ |
| Column stability factor | $C_P$ = **1.00** |
| Design compressive stress | $F_c' = F_c \times C_D \times C_{Mc} \times C_{tc} \times C_{Fc} \times C_i \times C_P$ = **2464** lb/in$^2$ |
| | $f_c / F_c'$ = **0.010** |

***PASS - Design compressive stress exceeds maximum applied compressive stress***

**Hold down force**

| | |
|---|---|
| Chord 1 | $T_1$ = **0.914** kips |
| Chord 2 | $T_2$ = **0.914** kips |
| Chord 5 | $T_5$ = **0.914** kips |
| Chord 6 | $T_6$ = **0.914** kips |
| Chord 9 | $T_9$ = **0.914** kips |
| Chord 10 | $T_{10}$ = **0.914** kips |

***DESIGN WARNING - Design using the strength distribution method to distribute shear loads to individual shear wall segments does not include a deflection check as results to that method are not reliable. Under seismic loads, a drift check is required as part of the design. Suggest using the equal deflections load distribution method to include the seismic drift check in this design.***

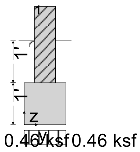
# ANNEX D

**Foundation analysis & design (ACI318) in accordance with ACI318-19 (22)-STRIP FOUNDATION**

Tedds calculation version 3.2.09

- **FOOTING ANALYSIS**

| | |
|---|---|
| Length of foundation | $L_x$ = **1** ft |
| Width of foundation | $L_y$ = **1** ft |
| Foundation area | $A = L_x \times L_y$ = **1** $ft^2$ |
| Depth of foundation | h = **12** in |
| Depth of soil over foundation | $h_{soil}$ = **12** in |
| Density of concrete | $\gamma_{conc}$ = **150.0** $lb/ft^3$ |




**Wall no.1 details**

| | |
|---|---|
| Width of wall | $l_{y1}$ = **6** in |
| position in y-axis | $y_1$ = **6** in |

**Soil properties**

| | |
|---|---|
| Gross allowable bearing pressure | $q_{allow\_Gross}$ = **1.5** ksf |
| Density of soil | $\gamma_{soil}$ = **120.0** $lb/ft^3$ |
| Angle of internal friction | $\phi_b$ = **30.0** deg |
| Design base friction angle | $\delta_{bb}$ = **30.0** deg |
| Coefficient of base friction | $\tan(\delta_{bb})$ = **0.577** |

**Foundation loads**

| | |
|---|---|
| Self weight | $F_{swt} = h \times \gamma_{conc}$ = **150** psf |
| Soil weight | $F_{soil} = h_{soil} \times \gamma_{soil}$ = **120** psf |

**Wall no.1 loads per linear foot**

| | |
|---|---|
| Dead load in z | $F_{Dz1}$ = **0.1** kips |
| Live load in z | $F_{Lz1}$ = **0.1** kips |
| Wind load in z | $F_{Wz1}$ = **0.3** kips |
| Seismic load in z | $F_{Ez1}$ = **0.1** kips |

**Footing analysis for soil and stability**

**Load combinations per ASCE 7-16**

1.0D (0.253)

1.0D + 1.0L (0.307)

**Combination 2 results: 1.0D + 1.0L**

**Forces on foundation per linear foot**

Force in z-axis $F_{dz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **0.5** kips

**Moments on foundation per linear foot**

Moment in y-axis, about y is 0 $M_{dy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **0.2** kip_ft

**Uplift verification**

Vertical force $F_{dz}$ = **0.46** kips

***PASS - Foundation is not subject to uplift***

**Stability against sliding**

Resistance due to base friction $F_{RFriction} = \max(F_{dz}, 0\text{ kN}) \times \tan(\delta_{bb})$ = **0.266** kips

**Bearing resistance**

**Eccentricity of base reaction**

Eccentricity of base reaction in y-axis $e_{dy} = M_{dy} / F_{dz} - L_y / 2$ = **0.000** in

**Strip base pressures**

$q_1 = F_{dz} \times (1 - 6 \times e_{dy} / L_y) / (L_y \times 1\text{ ft})$ = **0.46** ksf

$q_2 = F_{dz} \times (1 + 6 \times e_{dy} / L_y) / (L_y \times 1\text{ ft})$ = **0.46** ksf

Minimum base pressure $q_{min} = \min(q_1,q_2)$ = **0.46** ksf

Maximum base pressure $q_{max} = \max(q_1,q_2)$ = **0.46** ksf

**Allowable bearing capacity**

Allowable bearing capacity $q_{allow} = q_{allow\_Gross}$ = **1.5** ksf

$q_{max} / q_{allow}$ = **0.307**

***PASS - Allowable bearing capacity exceeds design base pressure***

- **FOOTING DESIGN (ACI318)**

**In accordance with ACI318-19 (22)**

**Material details**

Compressive strength of concrete $f'_c$ = **4000** psi

Yield strength of reinforcement $f_y$ = **60000** psi

Cover to reinforcement $c_{nom}$ = **3** in

Concrete type Normal weight

Concrete modification factor $\lambda$ = **1.00**

Wall type Concrete

**Analysis and design of concrete footing**

**Load combinations per ASCE 7-16**

1.4D (0.000)

1.2D + 1.6L + 0.5Lr (0.000)

**Combination 2 results: 1.2D + 1.6L + 0.5Lr**

**Forces on foundation per linear foot**

Ultimate force in z-axis $F_{uz} = \gamma_D \times A \times (F_{swt} + F_{soil}) + \gamma_D \times F_{Dz1} + \gamma_L \times F_{Lz1}$ = **0.6** kips

**Moments on foundation per linear foot**

Ultimate moment in y-axis, about y is 0 — $M_{uy} = \gamma_D \times (A \times (F_{swt} + F_{soil}) \times L_y / 2) + \gamma_D \times (F_{Dz1} \times y_1) + \gamma_L \times (F_{Lz1} \times y_1)$ = **0.3** kip_ft

**Eccentricity of base reaction**

Eccentricity of base reaction in y-axis — $e_{uy} = M_{uy} / F_{uz} - L_y / 2$ = **0.000** in

**Strip base pressures**

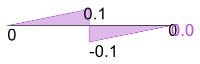
$q_{u1} = F_{uz} \times (1 - 6 \times e_{uy} / L_y) / (L_y \times 1\ ft)$ = **0.584** ksf

$q_{u2} = F_{uz} \times (1 + 6 \times e_{uy} / L_y) / (L_y \times 1\ ft)$ = **0.584** ksf

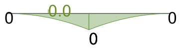
Minimum ultimate base pressure — $q_{umin} = \min(q_{u1},q_{u2})$ = **0.584** ksf

Maximum ultimate base pressure — $q_{umax} = \max(q_{u1},q_{u2})$ = **0.584** ksf

**Shear diagram (kips)**




**Moment diagram (kip_ft)**




**One-way shear design, y direction**

***One-way shear design does not apply. Shear failure plane fall outside extents of foundation.***