September 29, 2020

Urban Catalyst
99 S. Almaden Blvd Suite 840
San Jose, CA 95113

Attention: Mr. Matt Bernardis 20-185

Subject: Structural Evaluation of Josefa Tankhouse
491-499 W. San Carlos St. and 270-280 Josefa St.
San Jose, California

Mr. Bernardis:

Thank you for selecting Peoples Associates for your structural engineering needs. Pursuant to our August 7, 2020 Proposal, we have prepared the following Structural Evaluation Report. The following is a brief summary of our findings. Please refer to the detailed Report below for specific conditions, conclusions, and recommendations.

The Josefa Tankhouse was likely constructed in the early 1900's. Its construction is typical for buildings of this vintage, but the current condition of the building is considered poor with significant signs of distress noted. The lateral load resisting system for the building relies on a wood braced frame at the upper level of the water tower and a combination of horizontal and vertical board siding shear walls at the garage and water tower ground level. The structure contains discontinuous elements and horizontal irregularities in the lateral force resisting system.

Overall, based on the site observation and our evaluation, the structure exhibits significant damage and continuing deterioration as shown by the missing rafter tail, broken rafter, fascia and eave, bowed roof framing and wall top plates, peeling siding with large gaps and holes, damaged floor framing, severely rotted major post, severe slab cracks and inadequate major beam connections to post. Structural calculation shows that shearwall shear strength is highly deficient (300% above code allowable stress). We expect that the structure will experience continued deterioration and instability coupled by differential settlement due to absence of footing and evident settlement of walls and slabs. Due to this unstable characteristic of the structure, it does not meet the structural provisions of the code for occupancy of any type. If occupancy of the structure is desired, major re-structuring (replacement of all vertical and lateral load resisting systems) will be needed.

1. **SITE OBSERVATION** - PASE conducted a site visit on August 31, 2020 to observe the current condition and any signs of distress in the existing structure.

   The following are the signs of distress noted during the site visit.

   1.1. The exterior horizontal sheathing/siding is peeling from the structure. Many holes and gaps were observed. Other walls appear to have sustained significant water damage. Images below are taken from inside the structure.

1.2. At the south façade, exterior fascia at the water tower was missing and several rafter tails at the garage were already broken. This is a strong indication that the framing is severely deteriorating.

1.3. The southern portion of the roof shows serious signs of deterioration and sustained damage. Horizontal lumber roof sheathing was missing and supporting rafter was broken.

1.4. Significant bowing in the roof framing was observed in the northeastern corner of the garage.

1.5. Attic floor framing in the tower felt spongy with many floorboards loose or broken. Looking at the framing from the bottom, it appears that the framing has previously sustained some water damage.

1.6. Per contractor exploratory work, the north and west walls of the garage structure do not have a concrete footing while the south and east walls bear on a concrete footing. Structures with no footing will experience differential settlement and this is aggravated by the presence of footing at select location that will magnify the difference in soil pressure underneath. Differential settlement can lead to unlevel building, cracking of the concrete slab and foundations, doors and windows getting out of plumb, skewed wood framing, cracking of wall finishes, etc. See items 1.7 and 1.8 below for some of these observed symptoms.

1.7. The floor slab inside the building was not level and badly cracked throughout suggesting significant differential settlement has occurred over the years. Should this building be rehabilitated, continued differential settlement can result in additional unwanted slab steps and possible tripping hazards to future users.

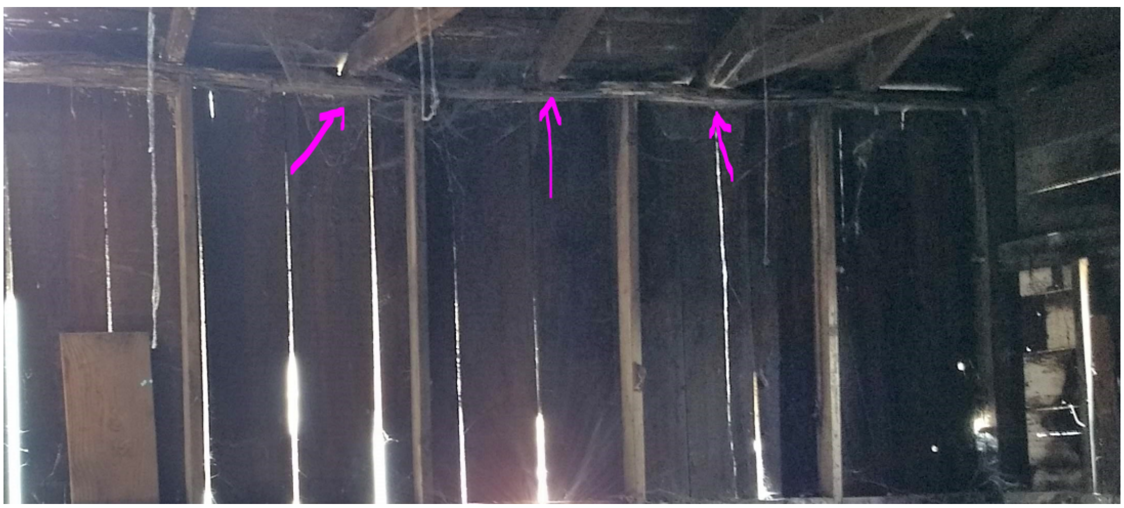
1.8. The north wall of the garage (side with no concrete footing) appears to have settled and the wall sill plate is now sitting lower than the garage slab. See photo from item 1.4 above. This appears to be a pronounced effect of differential settlement between the garage and water tower.

1.9. Wall framing on south side of garage shows significant signs of deterioration. Photo below shows a main building post with rot. When tested with a scratch awl, the 3" long scratch awl went in completely with relative ease.

1.10. The main structural beam supporting the west wall of the water tower and attic framing is supported by a wood corbel. From this photo, the corbel is a double 2x face nailed to the post. In the photo we can see (4) face nails installed on the outer member. If we assume the inner 2x is fastened with a total of (8) 16d nails to the post, this connection will be inadequate. There would need to be roughly two times as many nails, or utilize different connectors, to adequately support the full code level design loading.

1.11. The north bearing wall at the garage is comprised of 2x4 at about 52” o.c. aligned with the roof trusses. Intermediate roof rafters are supported on a single top plate spanning the 52”. The south bearing wall appears to have had the studs replaced at some point in time. However, these studs are 2x4 @ 32” o.c. and are not aligned directly under the roof trusses or rafters. See item 1.12 below.

1.12. Typical structural wood bearing walls utilize double 2x top plates to support the joists above, but only single 2x top plates were provided at the garage. Since the trusses and rafters were not aligned with the studs and the single top plate is having to span such a far distance between studs (see item 1.11 above), structural analysis shows that these top plates are not adequate (d/c = 1.21, 21% overstressed). Existing wall top plates appear to be bowing as a result.



1.13. Wall sill plates do not have anchor bolts to the concrete foundation as required by CBC section 2308.3.1.

1.14. Per CBC section 2308.4.2.3, joists shall be supported laterally at the ends by solid blocking, rim joist, stud, or other means. This lateral bracing is not provided at the attic platform joist framing.

1.15. There is a discontinuous gravity and lateral load path at the interface between garage and water tower. See image above in section 1.14.

1.16. Exterior wall framing sits directly on grade and does not appear to be protected against decay and termites per CBC section 2304.12. In the photo below, there are signs of water damage and intrusion of vegetation into the building.

1.17. Diaphragm at water tower attic level consist of horizontally sheathed single layer of lumber. Although this system is allowed per current code, end joints of boards are required to be staggered, but this was not the case. Additionally, there was no clear lateral load path to the surrounding shearwalls.

2. **METHODLOGY** - PASE went to the site on August 31, 2020 to understand the existing structural system and map the building loads to be used in evaluating the structure.

   2.1. Vertical (Gravity) Analysis - Beams, joist, post, and studs were checked using CBC 2019 requirements.

   2.2. Lateral (Seismic) Analysis – The evaluation of the lateral force resisting element of the structure is based on a modified ASCE 41 Tier 3 evaluation comparing the structural performance to 75% of 2019 CBC code level forces.

   2.3. Lateral (Seismic) Analysis – In addition to the ASCE 41 method, we also conducted a FEMA P-154 rapid visual screening for the lateral resistance.

3. **FEMA P-154 RAPID VISUAL SCREENING**

Peoples Associates conducted a FEMA P-154 Rapid Visual Screening (RVS) for the subject building in addition to the structural evaluation using CBC 2019 (See Appendix for RVS data collection form). The purpose of this screening is to estimate the building's probability of collapse in the event of a risk-targeted maximum considered earthquake (MCE_R) ground motion. The building's Final Score obtained by this RVS is an estimate and is based on limited observed and analytical data.

The Rapid Visual Screening for the subject building yielded a Final Score, S=0.7 implying that there is a chance of 1 in $10^{0.7}$, or 1 in 5, that the building will collapse if such ground motions occur. The Final Score S=0.7 is the lowest available score that this building type can receive based on the RVS screening. The results of the RVS screening further support our findings in the structural evaluation reported in the sections below.

4. CONCLUSIONS - VERTICAL LOAD RESISTING SYSTEM

4.1. In addition to the site observations, we run the structural calculation using 2019 CBC and found that the Tower Attic joists are overstressed by 11% and the Wall Top Plate supporting the trusses at the garage bearing walls are overstressed by 21%. These calculations did not include any reduction in member capacity due to the observed damage and deterioration. We expect that the amount of overstress will substantially increase once actual testing is done.

Based on the site observation mentioned above, the structure exhibits significant damage and continuing deterioration as shown by missing rafter tail, broken rafter, fascia and eave, bowed roof framing and wall top plates and peeling siding with large gaps and holes. The structure also shows damaged floor framing, severely rotted major post, severe slab cracks and inadequate major beam connections to post. We expect that the structure will experience continued deterioration and instability coupled by differential settlement due to absence of footing and evident settlement of walls and slabs. These unstable characteristics of the structure will require major structural replacement if occupancy of the structure is desired. Preservation work will be impractical based on the condition of the structure.

5. CONCLUSIONS – LATERAL LOAD RESISTING SYSTEM

5.1. Shear sheathing for the lateral force resisting system at ground level is comprised of a mixture of single-layer horizontally and vertically sheathed lumber shear walls. Horizontally and vertically sheathed lumber shear walls have limited unit shear capacity and stiffness compared to those provided by wood structural panel shear walls of the same dimensions. As a result, horizontal/vertical lumber sheathing is not permitted under current code provisions.

5.2. Even if we assume that vertical and horizontal board sheathing is allowed in this seismic region and the existing sheathing is in optimal condition (much of it is compromised), structural analysis shows that this system has demand-to-capacity (d/c) ratios as follows:

5.2.1. West wall: d/c = 4.26

5.2.2. East wall: d/c = 3.05

5.2.3. North wall: d/c = 1.00 (Acceptable)

5.2.4. South wall: d/c = 1.57

Due to the extent of compromised lumber sheathing and supporting framing, we expect that the actual d/c ratios will increase significantly.

5.3. At the interface between the water tower and the garage, there is an out-of-plane offset irregularity with an unaddressed load path to transfer lateral forces out of the discontinuous lateral force resisting element.

5.4. At the south side of the building near the entrance, there is a reentrant corner resulting in an in-plane discontinuity in the vertical lateral force resisting element and an out-of-plane offset irregularity. Collector element to transfer the lateral load out of the tower into the garage shearwall is nonexistent.

5.5. No shearwall anchor bolts or tiedowns are present to adequately anchor the structure against lateral loads.

5.6. No concrete footing at north and west sides of the garage structure.

Based on the evaluation mentioned above, the structure has a highly deficient shearwall shear strength to the range of 300% overstressed per code requirement. The absence of any anchor bolts nor tiedowns will also greatly limit its resistance under lateral forces. In addition, the absence of footing will cause instability to the structure in a code level seismic event.

Please feel free to call if you need clarification regarding the report. We look forward assisting Urban Catalyst on this and other projects.

Sincerely,

Kevin Chan, P.E.
Project Engineer

Dave Y. Lo, S.E
Senior Project Manager

APPENDIX A:
SUPPORTING CALCULATIONS

ATC Hazards by Location

Search Information

Address: 295 Josefa St, San Jose, CA 95110, USA

Coordinates: 37.3259396, -121.8988055

Elevation: 95 ft

Timestamp: 2020-09-02T16:25:51.013Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D

Basic Parameters

Name	Value	Description
S_S	1.5	MCE_R ground motion (period=0.2s)
S_1	0.6	MCE_R ground motion (period=1.0s)
S_{MS}	1.5	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.96	Coefficient of risk (0.2s)
CR_1	0.935	Coefficient of risk (1.0s)
PGA	0.52	MCE_G peak ground acceleration
F_{PGA}	1.1	Site amplification factor at PGA
PGA_M	0.572	Site modified peak ground acceleration
T_L	12	Long-period transition period (s)
SsRT	2.091	Probabilistic risk-targeted ground motion (0.2s)

SsUH	2.178	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.773	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.827	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.52	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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GOVERNING CODES & DESIGN CRITERIA

ANALYSIS SATISFACTORY

California Building Code, 2019 Edition
*ASCE 7-16 Minimum Design Loads for Buildings and Other Structures

SEISMIC COEFFICIENTS

Site Criteria and Seismic Design Category

Importance Factor --->	I =	1	(*Section 11.5-1)
Risk Category	=	II	(*Table 1-5-1)
Site Class:	=	D-Default	(*Table 20.3-1)
- Where Site Class D is selected as the default site class per Section 11.4.3, the value of Fa shall not be less than 1.2.			Section 11.4.4
Mapped Short Term Response Spectral Acceleration --------------------------->	S_S =	1.500	(*Fig. 22-1)
Mapped 1s period Spectral Response Acceleration ----------------------------->	S_1 =	0.600	(*Fig. 22-2)
Site Coefficients --->	F_a =	1.200	(Table 11.4-1)
	F_v =	1.700	(Table 11.4-2)
Modified MCE Spectral Response Acceleration at 0.2s --------------->	$S_{MS} = F_a S_S$ =	1.80	(Eqn 11.4-1)
Modified MCE Spectral Response Acceleration at 1s ----------------->	$S_{M1} = F_v S_1$ =	1.02	(Eqn 11.4-2)
Design Spectral Acceleration at 0.2s ------------------------------------>	$S_{DS} = (2/3) S_{MS}$ =	1.20	(Eqn 11.4-3)
Design Spectral Acceleration at 1s --------------------------------------->	$S_{D1} = (2/3) S_{M1}$ =	0.68	(Eqn 11.4-4)

Seismic Design Category: **D** (*Table 11.6-1 and 11.6-2)

Site-Specific Ground Motion Procedures: Section 11.4.8

Site Response Analysis Provided per Section 21.1	NO		
Site-Specific Ground Motion Hazard Analysis Required:	YES		
Site-Specific Ground Motion Hazard Analysis Provided:	NO	Exception:	**2**

EXCEPTIONS	Applicable	Satisfied
1 Structures on Site Class E sites with S_S greater than or equal to 1.0, provided the site coefficient Fa is taken as equal to that of Site Class C.	FALSE	**NO**
2 Structures on Site Class D sites with S_1 greater than or equal to 0.2, provided the value of the seismic response coefficient Cs is determined by Eq. (12.8-2) for values of T ≤ 1.5Ts and taken as equal to 1.5 times the value computed in accordance with either Eq. (12.8-3) for TL ≥T > 1.5Ts or Eq. (12.8-4) for T > TL.	TRUE	(12.8-2)
3 Structures on Site Class E sites with S_1 greater than or equal to 0.2, provided that T is less than or equal to Ts and the equivalent static force procedure is used for design.	FALSE	**NO**

Seismic Base Shear (Equivalent Lateral Force Procedure Section 12.8*)

Structural System per Table 12.2-1

A. 17. Light-frame walls with shear panels of all other materials

Response Modification Factor --> R = 2.00 (Table 12.2-1)

Ω_0 = 2.50 (Table 12.2-1)

Fundamental Period Worksheet (*Section 12.8.2)

Using Approximate Method:

Structural Type --------------------------------> All other structural Systems

	C_t =	0.02	(*Table 12.8-2)
	x =	0.75	(*Table 12.8-2)
	C_u =	1.4	Section 12.8.2, Table 12.8-1
Height of structure -------------------------------->	h_n =	20	ft
Number of Stories -------------------------------->	N =	2	
Approximate Fundamental Period	$T_a = C_t h_n^x$ =	0.189	(*Eqn 12.8-7)
Approximate Fundamental Period	$T_a = 0.1N$ =	n.a.	(*Eqn 12.8-8)
Using properly substantiated analysis:	T =	NOT USED	
	T_s =	0.57	
Fundamental Period of the Structure:	T =	0.189	sec

Long Period Transition Period -->	T_L =	12	(*Figure 22-12)
Seismic Response Coefficient, Cs =	$S_{DS} / (R / I)$ =	**0.60**	(*Eqn 12.8-2)
Upper Limit (for T <= TL):	$S_{D1} / (T (R / I))$ =	N/A	(*Eqn 12.8-3)
Upper Limit (for T > TL):	$S_{D1} T_L / (T^2 (R / I))$ =	N/A	(*Eqn 12.8-4)
Lower Limit:	$(0.044\ S_{DS}\ I >= 0.01)$ =	0.05	(*Eqn 12.8-5)
Lower Limit (S_1 >= 0.6):	$0.5 \times S_1 / (R / I)$ =	0.15	(*Eqn 12.8-6)

S_{DS} reduction per 12.8.1.3. Requirement is satisfied if the structure is regular, T< 0.5s, N <= 5 stories, ρ is 1.0, Risk Category is I or II, Site Class is not E or F: **Requirements not met.**

☐ No irregularities per 12.3.2 — S_{DS} = 1.20 (*Section 12.8.1.3)

Seismic Base Shear (Strength): $V = C_s W$ = **0.60** W (*Eqn 12.8-1)

Redundancy Factor:	For Diaphragm Design ------------------------->	ρ =	1.00	(*Section 12.3.4.1)
	For SDC D through F ---------------------------->	ρ =	1.00	(*Section 12.3.4.2)

Earthquake Load:

Strength Design:	$E_h = \rho V$ =	0.600 W
	$E_v = .2 {*} S_{DS} D$ =	0.240 D
Allowable Stress Design:	$E_h / 1.4$ =	0.429 W
	$E_v / 1.4$ =	0.171 D

(Ev = 0 for foundation design)

Hazards by Location

Search Information

Address: 295 Josefa St, San Jose, CA 95110, USA

Coordinates: 37.3259396, -121.8988055

Elevation: 95 ft

Timestamp: 2020-09-02T16:21:51.156Z

Hazard Type: Wind

ASCE 7-16

MRI 10-Year	63 mph
MRI 25-Year	70 mph
MRI 50-Year	74 mph
MRI 100-Year	79 mph
Risk Category I	86 mph
Risk Category II	92 mph
Risk Category III	98 mph
Risk Category IV	102 mph

ASCE 7-10

MRI 10-Year	72 mph
MRI 25-Year	79 mph
MRI 50-Year	85 mph
MRI 100-Year	91 mph
Risk Category I	100 mph
Risk Category II	110 mph
Risk Category III-IV	115 mph

ASCE 7-05

ASCE 7-05 Wind Speed	85 mph

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are interpolated from data provided in ASCE 7 and rounded up to the nearest whole integer. Per ASCE 7, islands and coastal areas outside the last contour should use the last wind speed contour of the coastal area – in some cases, this website will extrapolate past the last wind speed contour and therefore, provide a wind speed that is slightly higher. NOTE: For queries near wind-borne debris region boundaries, the resulting determination is sensitive to rounding which may affect whether or not it is considered to be within a wind-borne debris region.

Mountainous terrain, gorges, ocean promontories, and special wind regions shall be examined for unusual wind conditions.

MecaWind v2341

Calculations Prepared by:
Date: Sep 01, 2020

File Location:
Y:\Jobs\20-Jobs\20-185 Josefa St Water Tower Evaluation\calc\
20-185_MecaWind_Garage.wnd

Basic Wind Parameters

```
Wind Load Standard          = ASCE 7-16    Exposure Category           = B
Wind Design Speed           = 93.0 mph     Risk Category               = II
Structure Type              = Building     Building Type               = Enclosed
```

General Wind Settings

```
          = ASCE 7-16 Wind Parameters                              =
Incl_LF   = Include ASD Load Factor of 0.6 in Pressures            = True
DynType   = Dynamic Type of Structure                              = Rigid
NF        = Natural Frequency of Structure (Mode 1)                = 1.000 Hz
NF        = Natural Frequency of Structure                         = 1.000 Hz
Zg        = Altitude (Ground Elevation) above Sea Level            = 0.000 ft
Bdist     = Base Elevation of Structure                            = 0.000 ft
GenElev   = Specify the Elevations For Wind Pressures              = Mean Roof Ht
SDB       = Simple Diaphragm Building                              = False
MWFRS     = Analysis Procedure being used for MWFRS                = Ch 27 Pt 1
C&C       = Analysis Procedure being used for C&C                  = Ch 30 Pt 1
Reacs     = Show the Base Reactions in the output                  = False
MWFRSType = MWFRS Method Selected                                  = Ch 27 Pt 1
```

Topographic Factor per Fig 26.8-1

```
Topo      = Topographic Feature                                    = None
Kzt       = Topographic Factor                                     = 1.000
```

Building Inputs

```
RoofType: Building Roof Type  = Gabled           : Gabled            =
W       : Width Perp to Ridge = 16.167 ft   L       : Length Along Ridge  = 29.667 ft
EHt     : Eave Height         = 10.000 ft   RE      : Roof Entry Method   = Slope
Slope   : Slope of Roof       = 5.5 :12     OH      : Specify Roof to Wall intersections and Overhangs= None
Parapet : Type of Parapet     = None        Theta   : Roof Slope          = 24.62 Deg
Par     : Is there a Parapet  = False       OH_ALL  : None                = 0.000 ft
OH_ALL  : None                = 0.000 ft    OH_ALL  : None                = 0.000 ft
```

Exposure Constants per Table 26.11-1:

```
Alpha: Const from Table 26.11-1= 7.000      Zg:    Const from Table 26.11-1= 1200.000 ft
At:    Const from Table 26.11-1= 0.143      Bt:    Const from Table 26.11-1= 0.840
Am:    Const from Table 26.11-1= 0.250      Bm:    Const from Table 26.11-1= 0.450
C:     Const from Table 26.11-1= 0.300      Eps:   Const from Table 26.11-1= 0.333
```

Overhang Inputs:

```
Std       = Overhangs on all sides are the same                    = True
OHType    = Type of Roof Wall Intersections                        = None
```

Main Wind Force Resisting System (MWFRS) Calculations per Ch 27 Part 1:

```
h         = Mean Roof Height above grade                           = 11.852 ft
Kh        = Z < 15 ft [4.572 m]--> (2.01 * (15/zg)^(2/Alpha) {Table 26.10-1}= 0.575
Kzt       = Topographic Factor is 1 since no Topographic feature specified  = 1.000
Kd        = Wind Directionality Factor per Table 26.6-1            = 0.85
Zg        = Elevation above Sea Level                              = 0.000 ft
Ke        = Ground Elevation Factor: Ke = e^-(0.0000362*Zg) {Table 26.9-1}  = 1.000
GCPi      = Ref Table 26.13-1 for Enclosed Building                = +/-0.18
RA        = Roof Area                                              = 527.60 sq ft
LF        = Load Factor based upon ASD Design                      = 0.60
qh        = (0.00256 * Kh * Kzt * Kd * Ke * V^2) * LF              = 6.49 psf
qin       = For Negative Internal Pressure of Enclosed Building use qh*LF  = 6.49 psf
qip       = For Positive Internal Pressure of Enclosed Building use qh*LF  = 6.49 psf
```

Gust Factor Calculation:

```
Gust Factor Category I Rigid Structures - Simplified Method
G1        = For Rigid Structures (Nat. Freq.>1 Hz) use 0.85        = 0.85
Gust Factor Category II Rigid Structures - Complete Analysis
Zm        = 0.6 * Ht                                               = 30.000 ft
Izm       = Cc * (33 / Zm) ^ 0.167                                 = 0.305
Lzm       = L * (Zm / 33) ^ Epsilon                                = 309.993
Q         = (1 / (1 + 0.63 * ((B + Ht) / Lzm)^0.63))^0.5           = 0.937
G2        = 0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))              = 0.888
Gust Factor Used in Analysis
G         = Lessor Of G1 Or G2                                     = 0.850
```

MWFRS Wind Normal to Ridge (Ref Fig 27.3-1}

```
h         = Mean Roof Height Of Building                           = 11.852 ft
RHt       = Ridge Height Of Roof                                   = 13.705 ft
B         = Horizontal Dimension Of Building Normal To Wind Direction  = 29.667 ft
L         = Horizontal Dimension Of building Parallel To Wind Direction  = 16.167 ft
L/B       = Ratio Of L/B used For Cp determination                 = 0.545
h/L       = Ratio Of h/L used For Cp determination                 = 0.733
Slope     = Slope of Roof                                          = 24.62 Deg
Roof_LW   = Roof Coefficient (Leeward)                             = -0.6, -0.6
Roof_WW   = Roof Coefficient (Windward)                            = 0.09, -0.4

Cp_WW     = Windward Wall Coefficient (All L/B Values)              = 0.80
```

Cp_LW	= Leward Wall Coefficient using L/B	= -0.50
Cp_SW	= Side Wall Coefficient (All L/B values)	= -0.70
GCpn_WW	= Parapet Combined Net Pressure Coefficient (Windward Parapet)	= 1.50
GCpn_LW	= Parapet Combined Net Pressure Coefficient (Leeward Parapet)	= -1.00

Wall Wind Pressures based On Positive Internal Pressure (+GCPi) - Normal to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
10.00	0.575	1.000	6.49	0.18	3.24	-3.93	-5.03	7.17	9.60

Wall Wind Pressures based on Negative Internal Pressure (-GCPi) - Normal to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
10.00	0.575	1.000	6.49	-0.18	5.58	-1.59	-2.69	7.17	9.60

Notes Wall Pressures:
Kz = Velocity Press Exp Coeff   Kzt = Topographical Factor
qz = 0.00256*Kz*Kzt*Kd*V^2   GCPi = Internal Press Coefficient
Side = qh * G * Cp_SW - qip * +GCPi   Windward = qz * G * Cp_WW - qip * +GCPi
Leeward = qh * G * Cp_LW - qip * +GCPi   Total = Windward Press - Leeward Press
* Minimum Pressure: Para 27.1.5 no less than 9.60 psf (Incl LF) applied to Walls
+ Pressures Acting TOWARD Surface   - Pressures Acting AWAY from Surface

Roof Wind Pressures for Positive & Negative Internal Pressure (+/- GCPi) - Normal to Ridge
All wind pressures include a load factor of 0.6

Roof Var	Start Dist ft	End Dist ft	Cp_min	Cp_max	GCPi	Pressure Pn_min* psf	Pressure Pp_min* psf	Pressure Pn_max psf	Pressure Pp_max psf
Roof_LW	N/A	N/A	-0.600	-0.600	0.180	-2.14	-4.48	-2.14	-4.48
Roof_WW	N/A	N/A	0.090	-0.400	0.180	1.66	-0.67	-1.04	-3.37

Notes Roof Pressures:
Start Dist = Start Dist from Windward Edge   End Dist = End Dist from Windward Edge
Cp_Max = Largest Coefficient Magnitude   Cp_Min = Smallest Coefficient Magnitude
Pp_max = qh*G*Cp_max - qip*(+GCPi)   Pn_max = qh*G*Cp_max - qin*(-GCpi)
Pp_min* = qh*G*Cp_min - qip*(+GCPi)   Pn_min* = qh*G*Cp_min - qin*(-GCPi)
OH = Overhang   X = Dir along Ridge   Y = Dir Perpendcular to Ridge   Z = Vertical
* The smaller uplift pressures due to Cp_Min can become critical when wind is combined with roof live load or snow load; load combinations are given in ASCE 7
+ Pressures Acting TOWARD Surface   - Pressures Acting AWAY from Surface

MWFRS Wind Parallel to Ridge (Ref Fig 27.3-1)

h	= Mean Roof Height Of Building	= 11.852 ft
RHt	= Ridge Height Of Roof	= 13.705 ft
B	= Horizontal Dimension Of Building Normal To Wind Direction	= 16.167 ft
L	= Horizontal Dimension Of building Parallel To Wind Direction	= 29.667 ft
L/B	= Ratio Of L/B used For Cp determination	= 1.835
h/L	= Ratio Of h/L used For Cp determination	= 0.400
Slope	= Slope of Roof	= 24.62 Deg
Roof	= Roof Coeff (0 to h/2) (0.000 ft to 5.926 ft)	= -0.18, -0.9
Roof	= Roof Coeff (h/2 to h) (5.926 ft to 11.852 ft)	= -0.18, -0.9
Roof	= Roof Coeff (h to 2h) (11.852 ft to 23.705 ft)	= -0.18, -0.5
Roof	= Roof Coeff (>2h) (>23.705 ft)	= -0.18, -0.3

Cp_WW	= Windward Wall Coefficient (All L/B Values)	= 0.80
Cp_LW	= Leward Wall Coefficient using L/B	= -0.33
Cp_SW	= Side Wall Coefficient (All L/B values)	= -0.70
GCpn_WW	= Parapet Combined Net Pressure Coefficient (Windward Parapet)	= 1.50
GCpn_LW	= Parapet Combined Net Pressure Coefficient (Leeward Parapet)	= -1.00

Wall Wind Pressures based On Positive Internal Pressure (+GCPi) - Parallel to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
13.70	0.575	1.000	6.49	0.18	3.24	-3.01	-5.03	6.25	9.60
10.00	0.575	1.000	6.49	0.18	3.24	-3.01	-5.03	6.25	9.60

Wall Wind Pressures based on Negative Internal Pressure (-GCPi) - Parallel to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
13.70	0.575	1.000	6.49	-0.18	5.58	-0.67	-2.69	6.25	9.60
10.00	0.575	1.000	6.49	-0.18	5.58	-0.67	-2.69	6.25	9.60

Notes Wall Pressures:

```
Kz       = Velocity Press Exp Coeff        Kzt      = Topographical Factor
qz       = 0.00256*Kz*Kzt*Kd*V^2           GCPi     = Internal Press Coefficient
Side     = qh * G * Cp_SW - qip * +GCPi    Windward = qz * G * Cp_WW - qip * +GCPi
Leeward  = qh * G * Cp_LW - qip * +GCPi    Total    = Windward Press - Leeward Press
* Minimum Pressure: Para 27.1.5 no less than 9.60 psf (Incl LF) applied to Walls
+ Pressures Acting TOWARD Surface          - Pressures Acting AWAY from Surface
```

Roof Wind Pressures for Positive & Negative Internal Pressure (+/- GCPi) - Parallel to Ridge

All wind pressures include a load factor of 0.6

Roof Var	Start Dist ft	End Dist ft	Cp_min	Cp_max	GCPi	Pressure Pn_min* psf	Pressure Pp_min* psf	Pressure Pn_max psf	Pressure Pp_max psf
Roof (+Y)	0.000	5.926	-0.180	-0.900	0.180	0.18	-2.16	-3.80	-6.13
Roof (-Y)	0.000	5.926	-0.180	-0.900	0.180	0.18	-2.16	-3.80	-6.13
Roof (+Y)	5.926	11.852	-0.180	-0.900	0.180	0.18	-2.16	-3.80	-6.13
Roof (-Y)	5.926	11.852	-0.180	-0.900	0.180	0.18	-2.16	-3.80	-6.13
Roof (+Y)	11.852	23.705	-0.180	-0.500	0.180	0.18	-2.16	-1.59	-3.93
Roof (-Y)	11.852	23.705	-0.180	-0.500	0.180	0.18	-2.16	-1.59	-3.93
Roof (+Y)	23.705	29.667	-0.180	-0.300	0.180	0.18	-2.16	-0.49	-2.82
Roof (-Y)	23.705	29.667	-0.180	-0.300	0.180	0.18	-2.16	-0.49	-2.82

Notes Roof Pressures:

```
Start Dist = Start Dist from Windward Edge  End Dist = End Dist from Windward Edge
Cp_Max     = Largest Coefficient Magnitude  Cp_Min   = Smallest Coefficient Magnitude
Pp_max     = qh*G*Cp_max - qip*(+GCPi)      Pn_max   = qh*G*Cp_max - qin*(-GCPi)
Pp_min*    = qh*G*Cp_min - qip*(+GCPi)      Pn_min*  = qh*G*Cp_min - qin*(-GCPi)
OH = Overhang    X = Dir along Ridge    Y = Dir Perpendcular to Ridge  Z = Vertical
* The smaller uplift pressures due to Cp_Min can become critical when wind is combined
  with roof live load or snow load;  load combinations are given in ASCE 7
+ Pressures Acting TOWARD Surface          - Pressures Acting AWAY from Surface
```

Components and Cladding (C&C) Calculations per Ch 30 Part 1:

```
h/W       = Ratio of mean roof height to building width                    = 0.733
h/L       = Ratio of mean roof height to building length                   = 0.400
h         = Mean Roof Height above grade                                   = 11.852 ft
Kh        = Z < 15 ft [4.572 m]--> (2.01 * (15/zg)^(2/Alpha) {Table 26.10-1}= 0.575
Kzt       = Topographic Factor is 1 since no Topographic feature specified  = 1.000
Kd        = Wind Directionality Factor per Table 26.6-1                    = 0.85
GCPi      = Ref Table 26.13-1 for Enclosed Building                        = +/-0.18
LF        = Load Factor based upon ASD Design                              = 0.60
qh        = (0.00256 * Kh * Kzt * Kd * Ke * V^2) * LF                      = 6.49 psf
LHD       = Least Horizontal Dimension:  Min(B, L)                         = 16.167 ft
a1        = Min(0.1 * LHD,  0.4 * h                                        = 1.617 ft
a         = Max(a1, 0.04 * LHD, 3 ft [0.9 m])                              = 3.000 ft
h/B       = Ratio of mean roof height to least hor dim: h / B              = 0.733
```

Wind Pressures for C&C Ch 30 Pt 1

All wind pressures include a load factor of 0.6

Description	Zone	Width ft	Span ft	Area sq ft	1/3 Rule	Ref Fig	GCp Max	GCp Min	p Max psf	p Min psf
Zone 1	1	10.000	1.000	10.00	No	30.3-2C	0.535	-1.500	9.60	-10.90
Zone 2e	2e	10.000	1.000	10.00	No	30.3-2C	0.535	-1.500	9.60	-10.90
Zone 2n	2n	10.000	1.000	10.00	No	30.3-2C	0.535	-2.500	9.60	-17.39
Zone 2r	2r	10.000	1.000	10.00	No	30.3-2C	0.535	-2.500	9.60	-17.39
Zone 3e	3e	10.000	1.000	10.00	No	30.3-2C	0.535	-2.500	9.60	-17.39
Zone 3r	3r	10.000	1.000	10.00	No	30.3-2C	0.535	-2.947	9.60	-20.29
Zone 4	4	10.000	1.000	10.00	No	30.3-1	1.000	-1.100	9.60	-9.60
Zone 5	5	10.000	1.000	10.00	No	30.3-1	1.000	-1.400	9.60	-10.25
Zone 5	5	20.000	1.000	20.00	No	30.3-1	0.947	-1.294	9.60	-9.60

```
Area      = Span Length x Effective Width
1/3 Rule = Effective width need not be less than 1/3 of the span length
GCp       = External Pressure Coefficients taken from Figures 30.3-1 through 30.3-7
p         = Wind Pressure:  qh*(GCp - GCpi)  [Eqn 30.3-1]*
* Per Para 30.2.2 the Minimum Pressure for C&C is 9.60 psf [0.460 kPa] {Includes LF}
```

MecaWind v2341

Calculations Prepared by:
Date: Sep 01, 2020

File Location:
Y:\Jobs\20-Jobs\20-185 Josefa St Water Tower Evaluation\calc\
20-185_MecaWind_Tower.wnd

Basic Wind Parameters

```
Wind Load Standard            = ASCE 7-16    Exposure Category            = B
Wind Design Speed             = 93.0 mph     Risk Category                = II
Structure Type                = Building     Building Type                = Enclosed
```

General Wind Settings

```
          = ASCE 7-16 Wind Parameters                                    =
Incl_LF   = Include ASD Load Factor of 0.6 in Pressures                  = True
DynType   = Dynamic Type of Structure                                    = Rigid
NF        = Natural Frequency of Structure (Mode 1)                      = 1.000 Hz
NF        = Natural Frequency of Structure                               = 1.000 Hz
Zg        = Altitude (Ground Elevation) above Sea Level                  = 0.000 ft
Bdist     = Base Elevation of Structure                                  = 0.000 ft
GenElev   = Specify the Elevations For Wind Pressures                    = Mean Roof Ht
SDB       = Simple Diaphragm Building                                    = False
MWFRS     = Analysis Procedure being used for MWFRS                      = Ch 27 Pt 1
C&C       = Analysis Procedure being used for C&C                        = Ch 30 Pt 4
Reacs     = Show the Base Reactions in the output                        = False
MWFRSType = MWFRS Method Selected                                        = Ch 27 Pt 1
```

Topographic Factor per Fig 26.8-1

```
Topo      = Topographic Feature                                          = None
Kzt       = Topographic Factor                                           = 1.000
```

Building Inputs

```
RoofType: Building Roof Type  = Gabled             : Gabled              =
W       : Width Perp to Ridge = 13.000 ft   L      : Length Along Ridge  = 13.000 ft
EHt     : Eave Height         = 20.700 ft   RE     : Roof Entry Method   = Slope
Slope   : Slope of Roof       = 0.0 :12     OH     : Specify Roof to Wall intersections and Overhangs= Overhang
Parapet : Type of Parapet     = None        Theta  : Roof Slope          = 0.0 Deg
Par     : Is there a Parapet  = False       OH_ALL : Overhang            = 1.500 ft
OH_ALL  : Overhang            = 1.500 ft    OH_ALL : Overhang            = 1.500 ft
```

Exposure Constants per Table 26.11-1:

```
Alpha: Const from Table 26.11-1= 7.000       Zg:   Const from Table 26.11-1= 1200.000 ft
At:    Const from Table 26.11-1= 0.143       Bt:   Const from Table 26.11-1= 0.840
Am:    Const from Table 26.11-1= 0.250       Bm:   Const from Table 26.11-1= 0.450
C:     Const from Table 26.11-1= 0.300       Eps:  Const from Table 26.11-1= 0.333
```

Overhang Inputs:

```
Std       = Overhangs on all sides are the same                          = True
OHType    = Type of Roof Wall Intersections                              = Overhang
OH        = Overhang of Roof Beyond Wall                                 = 1.500 ft
```

Main Wind Force Resisting System (MWFRS) Calculations per Ch 27 Part 1:

```
h         = Mean Roof Height above grade                                 = 20.700 ft
Kh        = 15 ft [4.572 m]< Z <Zg -->(2.01*(Z/zg)^(2/Alpha) {Table 26.10-1}= 0.630
Kzt       = Topographic Factor is 1 since no Topographic feature specified  = 1.000
Kd        = Wind Directionality Factor per Table 26.6-1                  = 0.85
Zg        = Elevation above Sea Level                                    = 0.000 ft
Ke        = Ground Elevation Factor: Ke = e^-(0.0000362*Zg) {Table 26.9-1}  = 1.000
GCPi      = Ref Table 26.13-1 for Enclosed Building                      = +/-0.18
RA        = Roof Area                                                    = 256.00 sq ft
LF        = Load Factor based upon ASD Design                            = 0.60
qh        = (0.00256 * Kh * Kzt * Kd * Ke * V^2) * LF                    = 7.12 psf
qin       = For Negative Internal Pressure of Enclosed Building use qh*LF  = 7.12 psf
qip       = For Positive Internal Pressure of Enclosed Building use qh*LF  = 7.12 psf
```

Gust Factor Calculation:

```
Gust Factor Category I Rigid Structures - Simplified Method
G1        = For Rigid Structures (Nat. Freq.>1 Hz) use 0.85              = 0.85
Gust Factor Category II Rigid Structures - Complete Analysis
Zm        = 0.6 * Ht                                                     = 30.000 ft
Izm       = Cc * (33 / Zm) ^ 0.167                                       = 0.305
Lzm       = L * (Zm / 33) ^ Epsilon                                      = 309.993
Q         = (1 / (1 + 0.63 * ((B + Ht) / Lzm)^0.63))^0.5                 = 0.930
G2        = 0.925*((1+1.7*lzm*3.4*Q)/(1+1.7*3.4*lzm))                    = 0.884
Gust Factor Used in Analysis
G         = Lessor Of G1 Or G2                                           = 0.850
```

MWFRS Wind Normal to Ridge (Ref Fig 27.3-1}

```
h         = Mean Roof Height Of Building                                 = 20.700 ft
RHt       = Ridge Height Of Roof                                         = 20.700 ft
B         = Horizontal Dimension Of Building Normal To Wind Direction    = 13.000 ft
L         = Horizontal Dimension Of building Parallel To Wind Direction  = 13.000 ft
L/B       = Ratio Of L/B used For Cp determination                       = 1.000
h/L       = Ratio Of h/L used For Cp determination                       = 1.592
Slope     = Slope of Roof                                                = 0.0 Deg
OH_Bot    = Overhang Bottom (Windward Face Only)                         = 0.8, 0.8
OH_Top    = **Overhang Top  Coeff (0 to h/2) (0.000 ft to 8.000 ft)      = -0.18, -1.169
OH_Top    = **Overhang Top  Coeff (0 to h/2) (0.000 ft to 1.500 ft)      = -0.18, -1.169
```

```
OH_Top    = **Overhang Top  Coeff (0 to h/2) (8.000 ft to 10.350 ft)        = -0.18, -1.169
OH_Top    = Overhang Top  Coeff (h/2 to h) (10.350 ft to 16.000 ft)         = -0.18, -0.7
OH_Top    = Overhang Top  Coeff (h/2 to h) (14.500 ft to 16.000 ft)         = -0.18, -0.7
Roof      = **Roof  Coeff (0 to h/2) (1.500 ft to 8.000 ft)                 = -0.18, -1.169
Roof      = **Roof  Coeff (0 to h/2) (8.000 ft to 10.350 ft)                = -0.18, -1.169
Roof      = Roof  Coeff (h/2 to h) (10.350 ft to 14.500 ft)                 = -0.18, -0.7
**Includes Reduction Factor 0.9 For roof area, applied To Cp=-1.3 For h/L>=1 & (0 To h/2)

Cp_WW     = Windward Wall Coefficient (All L/B Values)                      = 0.80
Cp_LW     = Leward Wall Coefficient using L/B                               = -0.50
Cp_SW     = Side Wall Coefficient (All L/B values)                          = -0.70
GCpn_WW   = Parapet Combined Net Pressure Coefficient (Windward Parapet)     = 1.50
GCpn_LW   = Parapet Combined Net Pressure Coefficient (Leeward Parapet)      = -1.00
```

Wall Wind Pressures based On Positive Internal Pressure (+GCPi) - Normal to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
20.70	0.630	1.000	7.12	0.18	3.56	-4.30	-5.51	7.86	9.60

Wall Wind Pressures based on Negative Internal Pressure (-GCPi) - Normal to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
20.70	0.630	1.000	7.12	-0.18	6.12	-1.74	-2.95	7.86	9.60

```
Notes Wall Pressures:
Kz       = Velocity Press Exp Coeff        Kzt      = Topographical Factor
qz       = 0.00256*Kz*Kzt*Kd*V^2           GCPi     = Internal Press Coefficient
Side     = qh * G * Cp_SW - qip * +GCPi    Windward = qz * G * Cp_WW - qip * +GCPi
Leeward  = qh * G * Cp_LW - qip * +GCPi    Total    = Windward Press - Leeward Press
* Minimum Pressure: Para 27.1.5 no less than 9.60 psf (Incl LF) applied to Walls
+ Pressures Acting TOWARD Surface          - Pressures Acting AWAY from Surface
```

Roof Wind Pressures for Positive & Negative Internal Pressure (+/- GCPi) - Normal to Ridge
All wind pressures include a load factor of 0.6

Roof Var	Start Dist ft	End Dist ft	Cp_min	Cp_max	GCPi	Pressure Pn_min* psf	Pressure Pp_min* psf	Pressure Pn_max psf	Pressure Pp_max psf
OH_Bot	N/A	N/A	0.800	0.800	0.000	4.84	4.84	4.84	4.84
OH_Top (+X-Y)	0.000	8.000	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (-X-Y)	0.000	8.000	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (-Y)	0.000	1.500	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (+X+Y)	8.000	10.350	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (-X+Y)	8.000	10.350	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (+X+Y)	10.350	16.000	-0.180	-0.700	0.000	-1.09	-1.09	-4.23	-4.23
OH_Top (-X+Y)	10.350	16.000	-0.180	-0.700	0.000	-1.09	-1.09	-4.23	-4.23
OH_Top (+Y)	14.500	16.000	-0.180	-0.700	0.000	-1.09	-1.09	-4.23	-4.23
Roof (-Y)	1.500	8.000	-0.180	-1.169	0.180	0.19	-2.37	-5.79	-8.35
Roof (+Y)	8.000	10.350	-0.180	-1.169	0.180	0.19	-2.37	-5.79	-8.35
Roof (+Y)	10.350	14.500	-0.180	-0.700	0.180	0.19	-2.37	-2.95	-5.51

```
Notes Roof Pressures:
Start Dist = Start Dist from Windward Edge  End Dist = End Dist from Windward Edge
Cp_Max     = Largest Coefficient Magnitude  Cp_Min   = Smallest Coefficient Magnitude
Pp_max     = qh*G*Cp_max - qip*(+GCPi)      Pn_max   = qh*G*Cp_max - qin*(-GCpi)
Pp_min*    = qh*G*Cp_min - qip*(+GCPi)      Pn_min*  = qh*G*Cp_min - qin*(-GCPi)
OH = Overhang   X = Dir along Ridge   Y = Dir Perpendcular to Ridge  Z = Vertical
* The smaller uplift pressures due to Cp_Min can become critical when wind is combined
  with roof live load or snow load;  load combinations are given in ASCE 7
+ Pressures Acting TOWARD Surface          - Pressures Acting AWAY from Surface
```

MWFRS Wind Parallel to Ridge (Ref Fig 27.3-1)

```
h         = Mean Roof Height Of Building                                    = 20.700 ft
RHt       = Ridge Height Of Roof                                            = 20.700 ft
B         = Horizontal Dimension Of Building Normal To Wind Direction       = 13.000 ft
L         = Horizontal Dimension Of building Parallel To Wind Direction     = 13.000 ft
L/B       = Ratio Of L/B used For Cp determination                          = 1.000
h/L       = Ratio Of h/L used For Cp determination                          = 1.592
Slope     = Slope of Roof                                                   = 0.0 Deg
OH_Bot    = Overhang Bottom (Windward Face Only)                             = 0.8, 0.8
OH_Top    = **Overhang Top  Coeff (0 to h/2) (0.000 ft to 1.500 ft)         = -0.18, -1.169
OH_Top    = **Overhang Top  Coeff (0 to h/2) (1.500 ft to 10.350 ft)        = -0.18, -1.169
OH_Top    = Overhang Top  Coeff (h/2 to h) (10.350 ft to 14.500 ft)         = -0.18, -0.7
OH_Top    = Overhang Top  Coeff (h/2 to h) (14.500 ft to 16.000 ft)         = -0.18, -0.7
Roof      = **Roof  Coeff (0 to h/2) (1.500 ft to 10.350 ft)                = -0.18, -1.169
Roof      = **Roof  Coeff (h/2 to h) (10.350 ft to 14.500 ft)               = -0.18, -0.7
**Includes Reduction Factor 0.9 For roof area, applied To Cp=-1.3 For h/L>=1 & (0 To h/2)

Cp_WW     = Windward Wall Coefficient (All L/B Values)                      = 0.80
Cp_LW     = Leward Wall Coefficient using L/B                               = -0.50
Cp_SW     = Side Wall Coefficient (All L/B values)                          = -0.70
```

GCpn_WW = Parapet Combined Net Pressure Coefficient (Windward Parapet) = 1.50
GCpn_LW = Parapet Combined Net Pressure Coefficient (Leeward Parapet) = -1.00

Wall Wind Pressures based On Positive Internal Pressure (+GCPi) - Parallel to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
20.70	0.630	1.000	7.12	0.18	3.56	-4.30	-5.51	7.86	9.60

Wall Wind Pressures based on Negative Internal Pressure (-GCPi) - Parallel to Ridge
All wind pressures include a load factor of 0.6

Elev ft	Kz	Kzt	qz psf	GCPi	Windward Press psf	Leeward Press psf	Side Press psf	Total Press psf	Minimum Pressure* psf
20.70	0.630	1.000	7.12	-0.18	6.12	-1.74	-2.95	7.86	9.60

Notes Wall Pressures:
Kz = Velocity Press Exp Coeff   Kzt = Topographical Factor
qz = 0.00256*Kz*Kzt*Kd*V^2   GCPi = Internal Press Coefficient
Side = qh * G * Cp_SW - qip * +GCPi   Windward = qz * G * Cp_WW - qip * +GCPi
Leeward = qh * G * Cp_LW - qip * +GCPi   Total = Windward Press - Leeward Press
* Minimum Pressure: Para 27.1.5 no less than 9.60 psf (Incl LF) applied to Walls
+ Pressures Acting TOWARD Surface   - Pressures Acting AWAY from Surface

Roof Wind Pressures for Positive & Negative Internal Pressure (+/- GCPi) - Parallel to Ridge
All wind pressures include a load factor of 0.6

Roof Var	Start Dist ft	End Dist ft	Cp_min	Cp_max	GCPi	Pressure Pn_min* psf	Pressure Pp_min* psf	Pressure Pn_max psf	Pressure Pp_max psf
OH_Bot	N/A	N/A	0.800	0.800	0.000	4.84	4.84	4.84	4.84
OH_Bot	N/A	N/A	0.800	0.800	0.000	4.84	4.84	4.84	4.84
OH_Top (-X+Y)	0.000	1.500	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (-X-Y)	0.000	1.500	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (-Y)	1.500	10.350	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (+Y)	1.500	10.350	-0.180	-1.169	0.000	-1.09	-1.09	-7.07	-7.07
OH_Top (-Y)	10.350	14.500	-0.180	-0.700	0.000	-1.09	-1.09	-4.23	-4.23
OH_Top (+Y)	10.350	14.500	-0.180	-0.700	0.000	-1.09	-1.09	-4.23	-4.23
OH_Top (+X+Y)	14.500	16.000	-0.180	-0.700	0.000	-1.09	-1.09	-4.23	-4.23
OH_Top (+X-Y)	14.500	16.000	-0.180	-0.700	0.000	-1.09	-1.09	-4.23	-4.23
Roof (+Y)	1.500	10.350	-0.180	-1.169	0.180	0.19	-2.37	-5.79	-8.35
Roof (-Y)	1.500	10.350	-0.180	-1.169	0.180	0.19	-2.37	-5.79	-8.35
Roof (+Y)	10.350	14.500	-0.180	-0.700	0.180	0.19	-2.37	-2.95	-5.51
Roof (-Y)	10.350	14.500	-0.180	-0.700	0.180	0.19	-2.37	-2.95	-5.51

Notes Roof Pressures:
Start Dist = Start Dist from Windward Edge   End Dist = End Dist from Windward Edge
Cp_Max = Largest Coefficient Magnitude   Cp_Min = Smallest Coefficient Magnitude
Pp_max = qh*G*Cp_max - qip*(+GCPi)   Pn_max = qh*G*Cp_max - qin*(-GCpi)
Pp_min* = qh*G*Cp_min - qip*(+GCPi)   Pn_min* = qh*G*Cp_min - qin*(-GCpi)
OH = Overhang   X = Dir along Ridge   Y = Dir Perpendcular to Ridge   Z = Vertical
* The smaller uplift pressures due to Cp_Min can become critical when wind is combined
with roof live load or snow load; load combinations are given in ASCE 7
+ Pressures Acting TOWARD Surface   - Pressures Acting AWAY from Surface

Components and Cladding (C&C) Calculations per Ch 30 Part 4:

h = Mean Roof Height = 20.700 ft
LF = Load Factor based upon ASD Design = 0.60
Kzt = Topographic Factor is 1 since no Topographic feature specified = 1.000
EAF = Exposure Adjusment Factor per Table 30.7-2 = 0.693
LHD = Least Horizontal Dimension: Min(B, L) = 13.000 ft
a1 = Min(0.1 * LHD, 0.4 * h = 1.300 ft
a = Max(a1, 0.04 * LHD, 3 ft [0.9 m]) = 3.000 ft
2a = Parameter used to define zone width: 2*a = 6.000 ft
V = Velocity has been increased to meet the min per Table 30.6-2 = 110.0 mph
Lamba = Adjustment factor per Table 30.6-2 to Fig 30.4-1 pressures = 0.899

C&C entries with Zones which are Not Applicable to Ch 30 Pt 4 and/or Building Selections

Description ft	Zone	Width ft	Span Length ft
Zone 3r	3r	10.000	1.000

Wind Pressures for Components and Cladding per Fig 30.4-1
All wind pressures include a load factor of 0.6

Description ft	Zone	Width ft	Span ft	Area ft	1/3 Rule	Ptable Pos psf	Ptable Neg psf	p Pos psf	p Neg psf
Zone 1	1	10.000	1.000	10.000	No	9.60	-18.71	9.60	-18.71
Zone 2e	2e	10.000	1.000	10.000	No	9.60	-9.60	9.60	-9.60

Zone 2n	2n	10.000	1.000	10.000	No	9.60	-9.60	9.60	-9.60
Zone 2r	2r	10.000	1.000	10.000	No	9.60	-9.60	9.60	-9.60
Zone 3e	3e	10.000	1.000	10.000	No	9.60	-9.60	9.60	-9.60
Zone 4	4	10.000	1.000	10.000	No	11.75	-12.72	11.75	-12.72
Zone 5	5	10.000	1.000	10.000	No	11.75	-15.69	11.75	-15.69
Zone 5	5	20.000	1.000	20.000	No	11.21	-14.67	11.21	-14.67

Ptable = Pressure taken from Fig 30.4-1
p = Wind Pressure: Ptable * Lambda * Kzt * LF [Eqn 30.7-1 & Table 30.6-2 Note 5]
* Per Para 30.2.2 the Minimum Pressure for C&C is 9.60 psf [0.460 kPa] {Includes LF}
Pressures on overhangs include Pressure from the top and bottom surface of overhang

Design Loads - Wood Superstructure

	Dead Load	Live Load	
TOWER ROOF			
Roofing	1.0 psf		
1x Sheathing	2.2 psf		
2x12 @ 12" o.c.	5.6 psf		
Miscellaneous	0.7 psf		
	9.5 psf	20.0 psf	(Reducible)
PITCHED ROOF			
Roofing	1.0 psf		
1x Sheathing	2.2 psf		
Slope Adjust (5.5:12)	0.3 psf		
2x4 Trusses @ 52" o.c.	1.2 psf		
2x4 Rafters Btwn Trusses	0.4 psf		
Miscellaneous	0.4 psf		
	5.5 psf	20.0 psf	(Reducible)
ATTIC PLATFORM			
1x Sheathing	2.2 psf		
2x6 @ 20" o.c.	1.8 psf		
Miscellaneous	0.6 psf		
	4.5 psf	40.0 psf	
EXTERIOR WALLS	4x		
1x Sheathing	2.2 psf		
2x4 Studs @ 24" o.c. (VARIES)	1.0 psf		
Top & Bottom Plates	0.5 psf		
Miscellaneous	0.9 psf		
	4.5 psf		

Wood Beam

Lic. # : KW-06009713 Peoples Associates Structural Engineers

DESCRIPTION: Attic Joists - 2x6 @ 20" o.c.

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

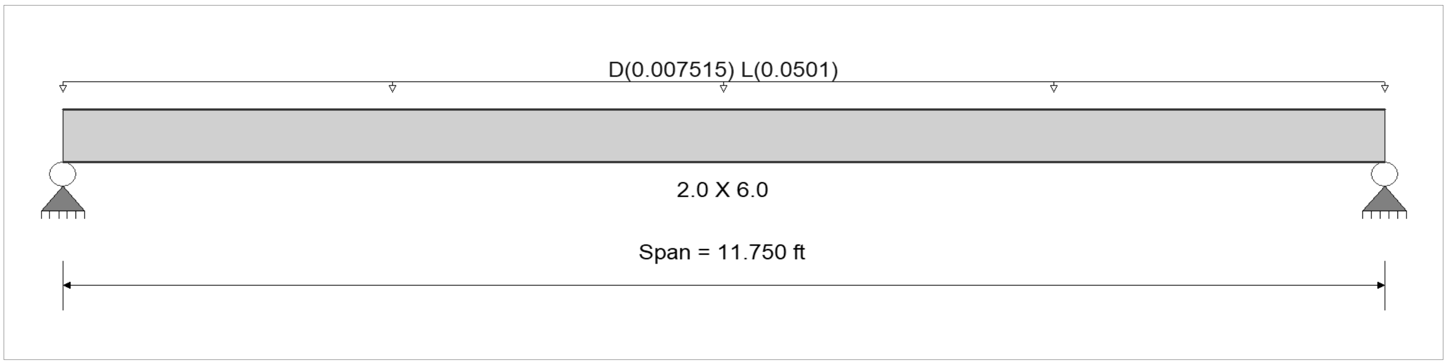
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

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

Fb +	900.0	psi
Fb -	900.0	psi
Fc - Prll	1,350.0	psi
Fc - Perp	625.0	psi
Fv	180.0	psi
Ft	575.0	psi

E : Modulus of Elasticity

Ebend- xx	1,600.0	ksi
Eminbend - xx	580.0	ksi
Density	31.210	pcf



Applied Loads

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

Uniform Load : D = 0.00450, L = 0.030 ksf, Tributary Width = 1.670 ft, (ROOF)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	**1.105** : 1	Maximum Shear Stress Ratio	=	**0.216** : 1
Section used for this span		**2.0 X 6.0**	Section used for this span		**2.0 X 6.0**
fb: Actual	=	994.31 psi	fv: Actual	=	38.91 psi
Fb: Allowable	=	900.00 psi	Fv: Allowable	=	180.00 psi
Load Combination		+D+L	Load Combination		+D+L
Location of maximum on span	=	5.875 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	0.375 in	Ratio =	375 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.431 in	Ratio =	326 >=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 Only													0.00	0.00	0.00	0.00
Length = 11.750 ft	1	0.160	0.031	0.90	1.000	1.00	1.00	1.00	1.00	1.00	0.13	129.69	810.00	0.04	5.08	162.00
+D+L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.750 ft	1	1.105	0.216	1.00	1.000	1.00	1.00	1.00	1.00	1.00	0.99	994.31	900.00	0.31	38.91	180.00
+D+0.750L					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.750 ft	1	0.692	0.135	1.25	1.000	1.00	1.00	1.00	1.00	1.00	0.78	778.15	1125.00	0.24	30.45	225.00
+0.60D					1.000	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 11.750 ft	1	0.054	0.011	1.60	1.000	1.00	1.00	1.00	1.00	1.00	0.08	77.82	1440.00	0.02	3.05	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+L	1	0.4315	5.918		0.0000	0.000

DESCRIPTION: Attic Joists - 2x6 @ 20" o.c.

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.338	0.338
Overall MINimum	0.294	0.294
D Only	0.044	0.044
+D+L	0.338	0.338
+D+0.750L	0.265	0.265
+0.60D	0.026	0.026
L Only	0.294	0.294

Wood Beam

DESCRIPTION: Garage - Single top plate supporting truss

CODE REFERENCES

Calculations per NDS 2018, IBC 2018, CBC 2019, ASCE 7-16
Load Combination Set : ASCE 7-16

Material Properties

Analysis Method : Allowable Stress Design
Load Combination : ASCE 7-16

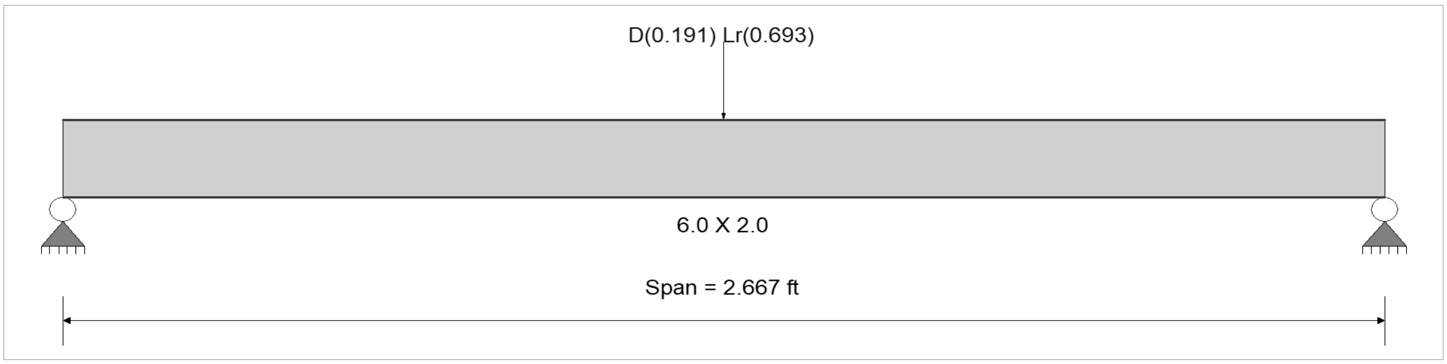
Wood Species : Douglas Fir-Larch
Wood Grade : No.2

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

Fb +	900.0	psi
Fb -	900.0	psi
Fc - Prll	1,350.0	psi
Fc - Perp	625.0	psi
Fv	180.0	psi
Ft	575.0	psi

E : Modulus of Elasticity

Ebend- xx	1,600.0	ksi
Eminbend - xx	580.0	ksi
Density	31.210	pcf



Applied Loads

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

Beam self weight calculated and added to loads
Point Load : D = 0.1910, Lr = 0.6930 k @ 1.333 ft, (truss)

DESIGN SUMMARY

Design N.G.

Maximum Bending Stress Ratio	=	**1.214** : 1	Maximum Shear Stress Ratio	=	**0.247** : 1
Section used for this span		**6.0 X 2.0**	Section used for this span		**6.0 X 2.0**
fb: Actual	=	1,774.89 psi	fv: Actual	=	55.64 psi
Fb: Allowable	=	1,462.50 psi	Fv: Allowable	=	225.00 psi
Load Combination		+D+Lr	Load Combination		+D+Lr
Location of maximum on span	=	1.334 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	0.074 in	Ratio =	430 >=240
Max Upward Transient Deflection	0.000 in	Ratio =	0 <240
Max Downward Total Deflection	0.095 in	Ratio =	335 >=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 Only													0.00	0.00	0.00	0.00
Length = 2.667 ft	1	0.369	0.076	0.90	1.300	1.00	1.00	1.00	1.00	1.00	0.13	388.93	1053.00	0.10	12.32	162.00
+D+Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.667 ft	1	1.214	0.247	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.59	1,774.89	1462.50	0.45	55.64	225.00
+D+0.750Lr					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.667 ft	1	0.977	0.199	1.25	1.300	1.00	1.00	1.00	1.00	1.00	0.48	1,428.40	1462.50	0.36	44.81	225.00
+0.60D					1.300	1.00	1.00	1.00	1.00	1.00			0.00	0.00	0.00	0.00
Length = 2.667 ft	1	0.125	0.026	1.60	1.300	1.00	1.00	1.00	1.00	1.00	0.08	233.36	1872.00	0.06	7.39	288.00

Overall Maximum Deflections

Load Combination	Span	Max. "-" Defl	Location in Span	Load Combination	Max. "+" Defl	Location in Span
+D+Lr	1	0.0953	1.334		0.0000	0.000

DESCRIPTION: Garage - Single top plate supporting truss

Vertical Reactions

Support notation : Far left is #1

Values in KIPS

Load Combination	Support 1	Support 2
Overall MAXimum	0.446	0.445
Overall MINimum	0.347	0.346
D Only	0.099	0.099
+D+Lr	0.446	0.445
+D+0.750Lr	0.359	0.359
+0.60D	0.059	0.059
Lr Only	0.347	0.346

BY	KMC	DATE	9/17/2020	PEOPLES ASSOCIATES	SHEET NO.		OF	
CHKD. BY		DATE		STRUCTURAL ENGINEERS	JOB NO.	20-185		

Keystone Tankhouse **Urban Catalyst**

LATERAL ANALYSIS Building No./Seg No.

Seismic Loads

Element	Area (ft^2)	Unit Weight (psf)	Weight (lbs)		
Elements Tributary to Tower Level					
Tower Roof	156	9.5	1482		
Walls	250	4.5	1125		
		Total Weight Tributary to Tower:	2607	lbs	
		Area:	156	sq ft	16.71 psf
Elements Tributary to Roof/Attic Level					
Garage Roof	272	4.5	1224		
Attic Framing	182	4.5	819		
Walls	710	4.5	3195		
		Total Weight Tributary to Roof/Attic:	5238	lbs	
		Area:	454	sq ft	11.54 psf
		Total Weight of Structure (W) =	7845	lbs	

Forces in the North/South Direction

Shear Walls*

E_h = p * V = 0.600

V = 0.75*(E_h)*W/1.4 = 2522 lbs

Forces in the East/West Direction

Shear Walls*

E_h = p * V = 0.600

V = 0.75*(E_h)*W/1.4 = 2522 lbs

*See Design Criteria for calculation of Base Shear Coefficients

*75% of 2019 CBC code level forces used for ASCE 41 Tier 3 evaluation

Keystone Tankhouse **Urban Catalyst**

LATERAL ANALYSIS

Distribution of Forces

Level	Weight, W (lbs)	Cum W (lbs)	Height, h (ft)	W*h	% W*h (%)	Fx (lbs)	Cum Fx (lbs)	Floor Load (psf)
Tower	2607	2607	20.7	53877	50%	**1258**	1258	8.1
Roof/Attic	5238	7845	10.3	54124	50%	**1264**	2522	2.8
Total	7845			108001	100%	2522		

Wind Loads

Level	Elevation (ft)	Wind Pressure (psf)
Tower	20.7	9.6
Roof/Attic	10.3	9.6

	North/South		East/West	
Diaphragm Level	Area	Force	Area	Force
Area Trib to Tower	67 sf	643 lbs	67 sf	643 lbs
Area Trib to Roof/Attic	221 sf	2126 lbs	149 sf	1434 lbs
		2769 lbs		2076 lbs

Governing Lateral Loads

Level	North/South		East/West	
Tower	1258 lbs (S)	SEISMIC	1258 lbs (S)	SEISMIC
Roof/Attic	2126 lbs (W)	WIND	1434 lbs (W)	WIND

Diaphragm Forces**

Level	Upper (lbs)	Actual (lbs)	Lower (lbs)	Diaphragm (lbs)	Fpx (psf)	Diaph Factor, λ
Tower	894	1258	447	894	5.7	0.71
Roof/Attic	1796	1684	898	1684	3.7	1.33

S_{DS} = 1.20

I = 1.00

Keystone Tankhouse **Urban Catalyst**

NORTH/SOUTH LATERAL DESIGN (FLEXIBLE DIAPHRAGM)

TOWER FRAMING LEVEL
DESIGN FOR SHEAR: Shear Applied to Wall Elements at the Tower Level

Area = 156
V = 1.26 k → 8.06 psf 0

Grid Line	N/S Direction Area (ft^2)	Length (ft)	Force (lbs)	Force Above (lbs)	Total Force (lbs)	Shear/ft (lbs/ft)	h/w ratio	3-1/2-1 Rat. (%) F Inc.	Adj Shear/ft (lbs/ft)	SW Type Min. Req.	SW Type Used	Wall Capacity (plf)	Demand to Capacity
1.0													
2.0	78	12.50	629	0	629	50	0.74	1.00	50	Horizontal Sheathing		50	✖ 1.01
3.0	78	12.50	629	0	629	50	0.74	1.00	50	Horizontal Sheathing		50	✖ 1.01
Totals=>	156	25	1258	0	1258								

TOWER FRAMING LEVEL
DESIGN FOR SHEAR: Shear Applied to Wall Elements at the Tower Level

Area = 156
V = 1.26 k → 8.06 psf 0

Grid Line	N/S Direction Area (ft^2)	Length (ft)	Force (lbs)	Force Above (lbs)	Total Force (lbs)	Shear/ft (lbs/ft)	h/w ratio	3-1/2-1 Rat. (%) F Inc.	Adj Shear/ft (lbs/ft)	SW Type Min. Req.	SW Type Used	Wall Capacity (plf)	Demand to Capacity
A	78	12.50	629	0	629	50	0.74	1.00	50	Horizontal Sheathing		50	✖ 1.01
A.1													
B	78	12.50	629	0	629	50	0.74	1.00	50	Horizontal Sheathing		50	✖ 1.01
Totals=>	156	25	1258	0	1258								

Keystone Tankhouse **Urban Catalyst**

NORTH/SOUTH LATERAL DESIGN (FLEXIBLE DIAPHRAGM)

ROOF FRAMING LEVEL
DESIGN FOR SHEAR: Shear Applied to Wall Elements at the Roof Level

Area = 454
V = 2.13 k → 4.68 psf 0

Grid Line	N/S Direction Area (ft^2)	Length (ft)	Force (lbs)	Force Above (lbs)	Total Force (lbs)	Shear/ft (lbs/ft)	h/w ratio	3-1/2-1 Rat. (%) F Inc.	Adj Shear/ft (lbs/ft)	SW Type Min. Req.	SW Type Used	Wall Capacity (plf)	Demand to Capacity
1.0	227	6.33	1063	287	1349	213	1.63	1.00	213	Horizontal Sheathing		50	4.26
2.0													
3.0	227	13.33	1063	971	2034	153	0.77	1.00	153	Horizontal Sheathing		50	3.05
Totals=>	454	20	2126	1258	3384								

ROOF FRAMING LEVEL
DESIGN FOR SHEAR: Shear Applied to Wall Elements at the Roof Level

Area = 454
V = 1.43 k → 3.16 psf 0

Grid Line	N/S Direction Area (ft^2)	Length (ft)	Force (lbs)	Force Above (lbs)	Total Force (lbs)	Shear/ft (lbs/ft)	h/w ratio	3-1/2-1 Rat. (%) F Inc.	Adj Shear/ft (lbs/ft)	SW Type Min. Req.	SW Type Used	Wall Capacity (plf)	Demand to Capacity
A													
A.1	227	19.00	717	629	1346	71	0.54	1.00	71	Vertical Sheathing		45	1.57
B	227	30.00	717	629	1346	45	0.34	1.00	45	Vertical Sheathing		45	1.00
Totals=>	454	49	1434	1258	2692								

APPENDIX B:
RAPID VISUAL SCREENING

Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA P-154 Data Collection Form

Level 1
VERY HIGH Seismicity

Address: 276 Josefa St., San Jose, CA
Zip: 95110
Other Identifiers:
Building Name: Keystone Tankhouse
Use:
Latitude: 37.326207 **Longitude:** -121.898725
S_S: 1.5 **S_1:** 0.6
Screener(s): KMC **Date/Time:** 08/31/2020 1:30 pm

No. Stories: Above Grade: 2 Below Grade: 0 **Year Built:** EARLY 1900's [X] EST
Total Floor Area (sq. ft.): 550 **Code Year:**
Additions: ☐ None ☐ Yes, Year(s) Built:

Occupancy: Assembly, Commercial, Emer. Services, ☐ Historic, ☐ Shelter, Industrial, Office, School, ☐ Government, [Utility] (selected), Warehouse, Residential, # Units:

Soil Type: ☐ A Hard Rock ☐ B Avg Rock ☐ C Dense Soil [X] D Stiff Soil ☐ E Soft Soil ☐ F Poor Soil DNK *If DNK, assume Type D.*

Geologic Hazards: Liquefaction: Yes/No/DNK Landslide: Yes/No/DNK Surf. Rupt.: Yes/No/DNK

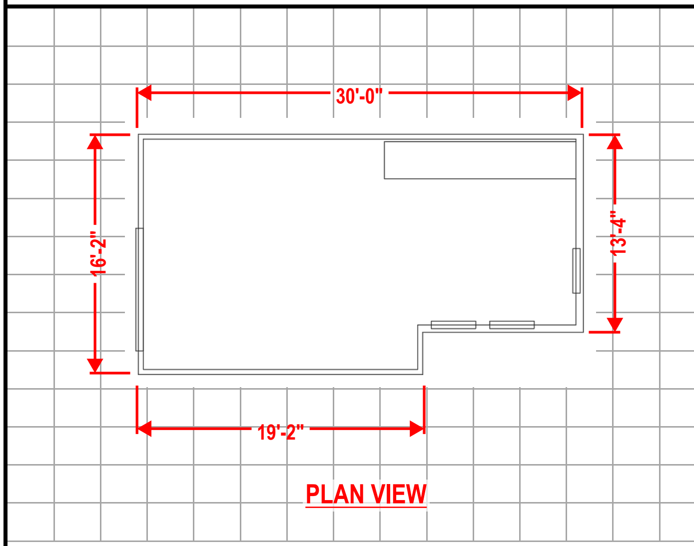
Adjacency: ☐ Pounding ☐ Falling Hazards from Taller Adjacent Building

Irregularities: [X] Vertical (type/severity) [Severe] Out-of-plane setback
[X] Plan (type) Re-entrant corner w/ in-plane discontinuity of lateral force resisting system

Exterior Falling Hazards: ☐ Unbraced Chimneys ☐ Heavy Cladding or Heavy Veneer ☐ Parapets ☐ Appendages ☐ Other:

COMMENTS:
- Existing building appears to be in poor condition
- Significant deterioration of wood observed both in the lateral and gravity system.
- Existing main lateral force resisting system type is not allowed for this seismic region under current code.
- Non-existent concrete footing for portion of building
- Extensively cracked foundation slab
- Evidence of differential settlement of foundation observed

☐ Additional sketches or comments on separate page



SKETCH

BASIC SCORE, MODIFIERS, AND FINAL LEVEL 1 SCORE, S_{L1}

FEMA BUILDING TYPE	Do Not Know	W1	W1A	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM	MH
Basic Score		**2.1**	**1.9**	**1.8**	**1.5**	**1.4**	**1.6**	**1.4**	**1.2**	**1.0**	**1.2**	**0.9**	**1.1**	**1.0**	**1.1**	**1.1**	**0.9**	**1.1**
Severe Vertical Irregularity, V_{L1}		-0.9	-0.9	-0.9	-0.8	-0.7	-0.8	-0.7	-0.7	-0.7	-0.8	-0.6	-0.7	-0.7	-0.7	-0.7	-0.6	NA
Moderate Vertical Irregularity, V_{L1}		-0.6	-0.5	-0.5	-0.4	-0.4	-0.5	-0.4	-0.3	-0.4	-0.4	-0.3	-0.4	-0.4	-0.4	-0.4	-0.3	NA
Plan Irregularity, P_{L1}		-0.7	-0.7	-0.6	-0.5	-0.5	-0.6	-0.4	-0.4	-0.4	-0.5	-0.3	-0.5	-0.4	-0.4	-0.4	-0.3	NA
Pre-Code		-0.3	-0.3	-0.3	-0.3	-0.2	-0.3	-0.2	-0.1	-0.1	-0.2	0.0	-0.2	-0.1	-0.2	-0.2	0.0	0.0
Post-Benchmark		1.9	1.9	2.0	1.0	1.1	1.1	1.5	NA	1.4	1.7	NA	1.5	1.7	1.6	1.6	NA	0.5
Soil Type A or B		0.5	0.5	0.4	0.3	0.3	0.4	0.3	0.2	0.2	0.3	0.1	0.3	0.2	0.3	0.3	0.1	0.1
Soil Type E (1-3 stories)		0.0	-0.2	-0.4	-0.3	-0.2	-0.2	-0.2	-0.1	-0.1	-0.2	0.0	-0.2	-0.1	-0.2	-0.2	0.0	-0.1
Soil Type E (> 3 stories)		-0.4	-0.4	-0.4	-0.3	-0.3	NA	-0.3	-0.1	-0.1	-0.3	-0.1	NA	-0.1	-0.2	-0.2	0.0	NA
Minimum Score, S_{MIN}		*0.7*	*0.7*	*0.7*	*0.5*	*0.5*	*0.5*	*0.5*	*0.5*	*0.3*	*0.3*	*0.3*	*0.2*	*0.2*	*0.3*	*0.3*	*0.2*	*1.0*

FINAL LEVEL 1 SCORE, $S_{L1} \geq S_{MIN}$: [S.L1 = 0.2] < [S.min = 0.7]... Therefore, use S.min = 0.7

EXTENT OF REVIEW

Exterior: [X] Partial [X] All Sides ☐ Aerial
Interior: ☐ None [X] Visible [X] Entered
Drawings Reviewed: ☐ Yes [X] No
Soil Type Source:
Geologic Hazards Source:
Contact Person:

LEVEL 2 SCREENING PERFORMED?

[X] Yes, Final Level 2 Score, S_{L2} 0.7 ☐ No
Nonstructural hazards? [X] Yes ☐ No

OTHER HAZARDS

Are There Hazards That Trigger A Detailed Structural Evaluation?
☐ Pounding potential (unless S_{L2} > cut-off, if known)
☐ Falling hazards from taller adjacent building
☐ Geologic hazards or Soil Type F
[X] Significant damage/deterioration to the structural system

ACTION REQUIRED

Detailed Structural Evaluation Required?
☐ Yes, unknown FEMA building type or other building
[X] Yes, score less than cut-off
[X] Yes, other hazards present
☐ No

Detailed Nonstructural Evaluation Recommended? *(check one)*
[X] Yes, nonstructural hazards identified that should be evaluated
☐ No, nonstructural hazards exist that may require mitigation, but a detailed evaluation is not necessary
☐ No, no nonstructural hazards identified ☐ DNK

Where information cannot be verified, screener shall note the following: EST = Estimated or unreliable data OR DNK = Do Not Know

Legend: MRF = Moment-resisting frame; BR = Braced frame; RC = Reinforced concrete; SW = Shear wall; URM INF = Unreinforced masonry infill; TU = Tilt up; MH = Manufactured Housing; LM = Light metal; FD = Flexible diaphragm; RD = Rigid diaphragm

Rapid Visual Screening of Buildings for Potential Seismic Hazards — **Level 2 (Optional)**

FEMA P-154 Data Collection Form — **VERY HIGH Seismicity**

Optional Level 2 data collection to be performed by a civil or structural engineering professional, architect, or graduate student with background in seismic evaluation or design of buildings.

Bldg Name: Keystone Tankhouse	**Final Level 1 Score:**	S_{L1} = 0.2		*(do not consider S_{MIN})*
Screener: KMC	**Level 1 Irregularity Modifiers:**	*Vertical Irregularity, V_{L1}* = -0.9		*Plan Irregularity, P_{L1}* = -0.7
Date/Time: 08/31/2020 1:30 pm	**ADJUSTED BASELINE SCORE:**	*$S' = (S_{L1} - V_{L1} - P_{L1})$* = 1.8		

STRUCTURAL MODIFIERS TO ADD TO ADJUSTED BASELINE SCORE

Topic		Statement *(If statement is true, circle the "Yes" modifier; otherwise cross out the modifier.)*	Yes	Subtotals
Vertical Irregularity, V_{L2}	Sloping Site	W1 building: There is at least a full story grade change from one side of the building to the other.	-0.9	
		Non-W1 building: There is at least a full story grade change from one side of the building to the other.	-0.2	
	Weak and/or Soft Story *(circle one maximum)*	W1 building cripple wall: An unbraced cripple wall is visible in the crawl space.	-0.5	
		W1 house over garage: Underneath an occupied story, there is a garage opening without a steel moment frame, and there is less than 8' of wall on the same line (for multiple occupied floors above, use 16' of wall minimum).	-0.9	
		W1A building open front: There are openings at the ground story (such as for parking) over at least 50% of the length of the building.	-0.9 (circled)	
		Non-W1 building: Length of lateral system at any story is less than 50% of that at story above or height of any story is more than 2.0 times the height of the story above.	-0.7	
		Non-W1 building: Length of lateral system at any story is between 50% and 75% of that at story above or height of any story is between 1.3 and 2.0 times the height of the story above.	-0.4	
	Setback	Vertical elements of the lateral system at an upper story are outboard of those at the story below causing the diaphragm to cantilever at the offset.	-0.7	
		Vertical elements of the lateral system at upper stories are inboard of those at lower stories.	-0.4 (circled)	
		There is an in-plane offset of the lateral elements that is greater than the length of the elements.	-0.2	
	Short Column/ Pier	C1,C2,C3,PC1,PC2,RM1,RM2: At least 20% of columns (or piers) along a column line in the lateral system have height/depth ratios less than 50% of the nominal height/depth ratio at that level.	-0.4	
		C1,C2,C3,PC1,PC2,RM1,RM2: The column depth (or pier width) is less than one half of the depth of the spandrel, or there are infill walls or adjacent floors that shorten the column.	-0.4	
	Split Level	There is a split level at one of the floor levels or at the roof.	-0.4	
	Other Irregularity	There is another observable severe vertical irregularity that obviously affects the building's seismic performance.	-0.7	V_{L2} = -0.9
		There is another observable moderate vertical irregularity that may affect the building's seismic performance.	-0.4	*(Cap at -0.9)*
Plan Irregularity, P_{L2}		Torsional irregularity: Lateral system does not appear relatively well distributed in plan in either or both directions. *(Do not include the W1A open front irregularity listed above.)*	-0.5	
		Non-parallel system: There are one or more major vertical elements of the lateral system that are not orthogonal to each other.	-0.2	
		Reentrant corner: Both projections from an interior corner exceed 25% of the overall plan dimension in that direction.	-0.2	
		Diaphragm opening: There is an opening in the diaphragm with a width over 50% of the total diaphragm width at that level.	-0.2	
		C1, C2 building out-of-plane offset: The exterior beams do not align with the columns in plan.	-0.2	P_{L2} = -0.5
		Other irregularity: There is another observable plan irregularity that obviously affects the building's seismic performance.	-0.5 (circled)	*(Cap at -0.7)*
Redundancy		The building has at least two bays of lateral elements on each side of the building in each direction.	+0.2	
Pounding		Building is separated from an adjacent structure by less than 1.5% of the height of the shorter of the building and adjacent structure and: The floors do not align vertically within 2 feet. *(Cap total pounding modifiers at -0.9)*	-0.7	
		One building is 2 or more stories taller than the other.	-0.7	
		The building is at the end of the block.	-0.4	
S2 Building		"K" bracing geometry is visible.	-0.7	
C1 Building		Flat plate serves as the beam in the moment frame.	-0.3	
PC1/RM1 Bldg		There are roof-to-wall ties that are visible or known from drawings that do not rely on cross-grain bending. *(Do not combine with post-benchmark or retrofit modifier.)*	+0.2	
PC1/RM1 Bldg		The building has closely spaced, full height interior walls (rather than an interior space with few walls such as in a warehouse).	+0.2	
URM		Gable walls are present.	-0.3	
MH		There is a supplemental seismic bracing system provided between the carriage and the ground.	+0.5	
Retrofit		Comprehensive seismic retrofit is visible or known from drawings.	+1.2	M = ______

FINAL LEVEL 2 SCORE, $S_{L2} = (S' + V_{L2} + P_{L2} + M) \geq S_{MIN}$: [S.L2 = 0.4] < [S.min = 0.7]... Therefore, use S.min = 0.7 *(Transfer to Level 1 form)*

There is observable damage or deterioration or another condition that negatively affects the building's seismic performance: ☒ Yes ☐ No

If yes, describe the condition in the comment box below and indicate on the Level 1 form that detailed evaluation is required independent of the building's score.

OBSERVABLE NONSTRUCTURAL HAZARDS

Location	Statement *(Check "Yes" or "No")*	Yes	No	Comment
Exterior	There is an unbraced unreinforced masonry parapet or unbraced unreinforced masonry chimney.			
	There is heavy cladding or heavy veneer.			
	There is a heavy canopy over exit doors or pedestrian walkways that appears inadequately supported.			
	There is an unreinforced masonry appendage over exit doors or pedestrian walkways.			
	There is a sign posted on the building that indicates hazardous materials are present.			
	There is a taller adjacent building with an unanchored URM wall or unbraced URM parapet or chimney.			
	Other observed exterior nonstructural falling hazard:	X		Falling siding
Interior	There are hollow clay tile or brick partitions at any stair or exit corridor.			
	Other observed interior nonstructural falling hazard:			

Estimated Nonstructural Seismic Performance (*Check appropriate box and transfer to Level 1 form conclusions*)

☒ Potential nonstructural hazards with significant threat to occupant life safety → Detailed Nonstructural Evaluation recommended

☐ Nonstructural hazards identified with significant threat to occupant life safety → But no Detailed Nonstructural Evaluation required

☐ Low or no nonstructural hazard threat to occupant life safety → No Detailed Nonstructural Evaluation required

Comments:

October 29, 2020

Urban Catalyst
99 S. Almaden Blvd Suite 840
San Jose, CA 95113

Attention: Mr. Matt Bernardis 20-233

Subject: Structural Observation and Rapid Visual Screening
499 & 497 W. San Carlos St.
San Jose, California

Mr. Bernardis:

Thank you for selecting Peoples Associates for your structural engineering needs. As per your request, Peoples Associates has completed the FEMA P-154 Rapid Visual Screening (RVS) Level 1 & 2 for the subject project. This letter is to serve as a report for our findings for the RVS for the subject project.

The site consists of a one-story wood framed building with two addresses (i.e. 499 & 497 W. San Carlos) that is likely constructed in the early 1900's. We expect that its construction is typical for buildings of this vintage although no invasive structural survey and testing was done to confirm this. The current condition of the building is showing some signs of distress as noted in our site observation. We expect that the lateral load resisting system for the building relies on wood let-in braces. The front façade exhibits big storefronts, doors and windows with no clear lateral force resisting system. The rear wall also has several windows and doors with no space for a shearwall. The crawl space partial basement underneath is enclosed with 3'-6" tall cripple studwall and 3' tall concrete stemwall. The ground floor is framed with 2"x7.5" floor joist @ 16" o.c. (24" o.c. at very rear) over 4"x4" or 6"x6" beam spaced approximately from 8' to 10' with 6"x6" posts spaced from 6' to 8' o.c. The structure contains horizontal and vertical irregularities in the lateral force resisting system.

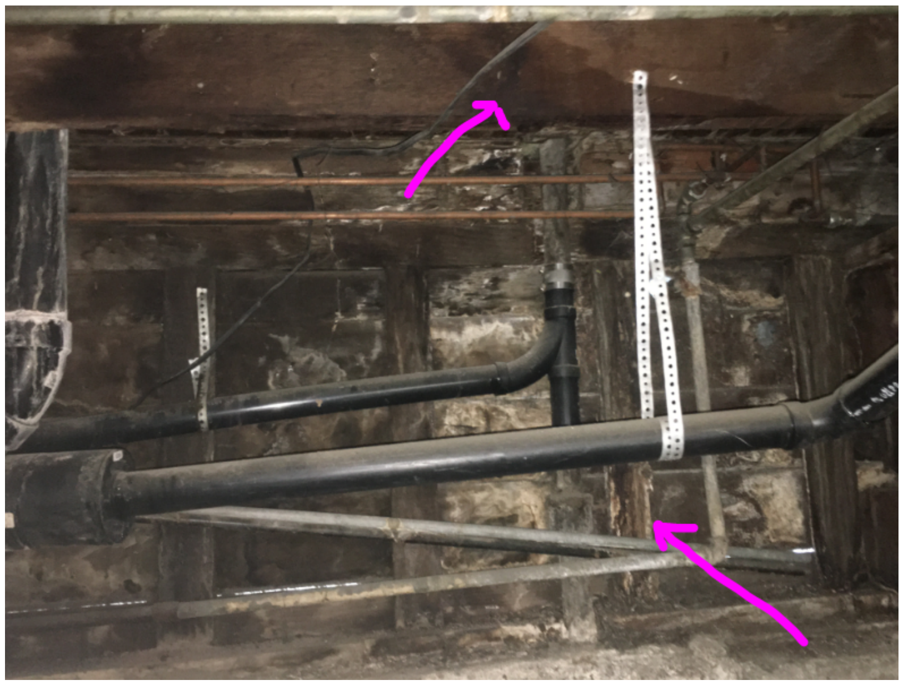
1. **SITE OBSERVATION** - PASE conducted a site visit on October 20, 2020 to observe the current condition and identify signs of distress in the existing structure. Unit '497', which is a residential space, was not available during our observation. We only observed the commercial space unit '499'. The roof framing are all covered with ceiling finish while the wall framing are covered with interior and exterior wall finishes, thus no observation was done to the roof framing and wall framing.

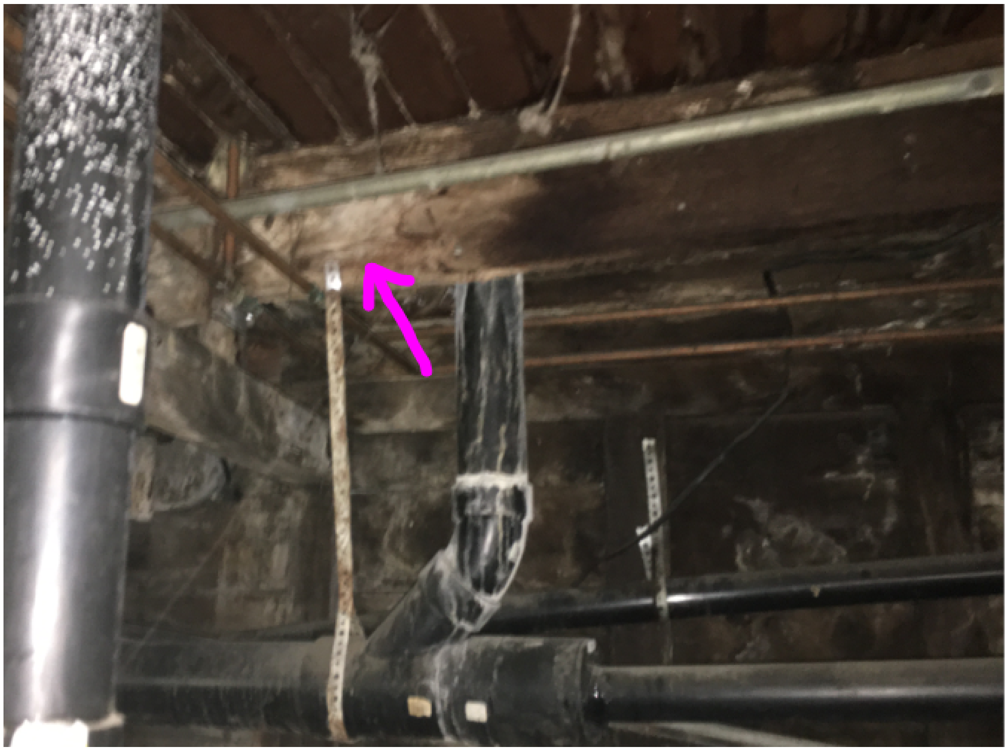
   The following are the signs of distress or deficiency noted during the site visit.

   1.1. The front façade of the building is essentially full of wall openings with no space for shearwall. This building has an open front that relies on interior wall for lateral resistance. Due to the age of the building, it is highly likely that roof diaphragms are not sufficient to transfer the load from the exterior wall to the interior wall.

   1.2. The rear stair is deteriorating showing bigger gaps between treads. The beam and studs are showing water damage. A newer floor sheathing is installed above the joist which indicate that a floor repair was done at one point.

1.3. Floor joists and sill plates are showing signs of water exposure and deterioration.





1.4. All posts are not protected from water that accumulate in the basement. The photo below shows standing water from pipe leak above. Bottom of post shows deterioration likely from previous exposure to moisture.

1.5. At the back wall, sidings are deteriorating showing warping and bigger gaps.

1.6. Portion of the Ceiling at the front shows some water stain and bumpiness that might indicate possible water intrusion that may also affect the roof framing above.

1.7. Unit 499 front deck is in poor condition. The trimmer shows some crack and the siding shows big corner gap that indicate swelling or warping of wood underneath. Some flooring are not level and broken

1.8. Exterior molding at the roof overhang underside is separating from the ceiling.

1.9. Wall sill plates do not have anchor bolts to the concrete foundation as required by CBC section 2308.3.1.

2. FEMA P-154 RAPID VISUAL SCREENING

Peoples Associates conducted a FEMA P-154 Rapid Visual Screening (RVS) for the subject. This procedure has been developed by FEMA to identify, inventory and screen buildings that are potentially seismically hazardous. The purpose of this screening is to estimate the building's probability of collapse in the event of a risk-targeted maximum considered earthquake (MCE_R) ground motion. The building's Final Score obtained by this RVS is an estimate and is based on a very limited observed and analytical data.

The Rapid Visual Screening for the subject building yielded a Final Score of S=0.7 implying that there is a chance of 1 in $10^{0.7}$, or 1 in 5, that the building will collapse if such ground motions occur. Note that 0.7 is the lowest available score for this building type and a score of 2.0 is considered as the acceptable score (1 in 100 probability of collapse).

3. CONCLUSIONS

Based on the RVS result (0.7 versus 2.0), the building is considered a potentially seismically hazardous structure. Please note that the RVS is an initial screening phase of a multi-phase procedure for identifying potentially hazardous buildings. The absence of any anchor bolts and tiedowns at the basement and the unclear lateral resisting system at the front will significantly limit the building resistance to lateral forces.

Based on the items noted in our site observation, the building shows previous prolonged exposure to water at several location in the basement and that some of the joist, beam and stud show signs of deterioration. The front deck of unit 497 shows broken floor decking that may indicate some damage to the floor framing underneath. We recommend fixing the broken floor decking at unit 497. We also recommend repairing the deteriorated framing members in the basement.

Please feel free to call us if you need clarification regarding the report. We look forward in assisting Urban Catalyst on this and other projects.

Sincerely,

Dave Y. Lo, S.E
Senior Project Manager

Rapid Visual Screening of Buildings for Potential Seismic Hazards
FEMA P-154 Data Collection Form

Level 1
VERY HIGH Seismicity

Address: 499 & 497 W. San Carlos St., San Jose, CA
Zip: 95110
Other Identifiers:
Building Name: 499 & 497 W. San Carlos
Use:
Latitude: 37.3259945 **Longitude:** -121.8985611
S_S: 1.5 **S_1:** 0.6
Screener(s): DYL **Date/Time:** 10/20/2020 10:30 am

No. Stories: Above Grade: 1 Below Grade: 1 **Year Built:** EARLY 1900's [X] EST
Total Floor Area (sq. ft.): 1700 **Code Year:**
Additions: ☐ None ☐ Yes, Year(s) Built:

Occupancy: Assembly, **Commercial** (marked), Emer. Services, [X] Historic, ☐ Shelter, Industrial, Office, School, ☐ Government, Utility, Warehouse, Residential, # Units:

Soil Type: ☐A Hard Rock ☐B Avg Rock ☐C Dense Soil [X]D Stiff Soil ☐E Soft Soil ☐F Poor Soil DNK *If DNK, assume Type D.*

Geologic Hazards: Liquefaction: Yes/No/(DNK) Landslide: Yes/No/(DNK) Surf. Rupt.: Yes/No/(DNK)

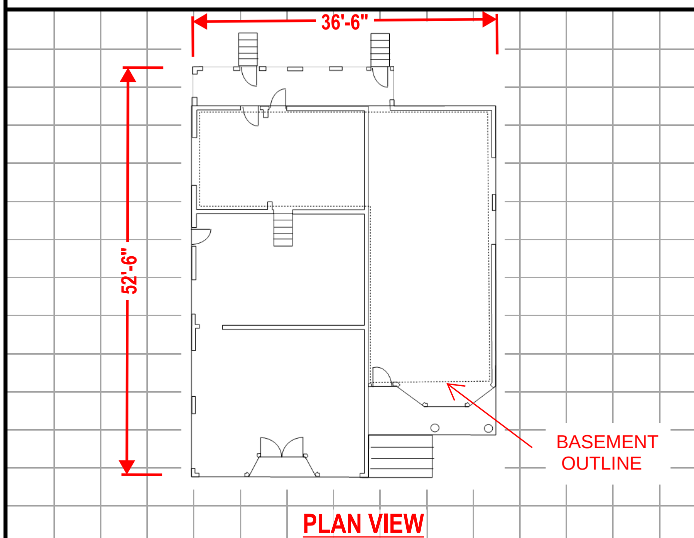
Adjacency: ☐ Pounding ☐ Falling Hazards from Taller Adjacent Building

Irregularities: [X] Vertical (type/severity) [Moderate] Unbraced cripple wall
[X] Plan (type) Re-entrant corner , Torsion

Exterior Falling Hazards: ☐ Unbraced Chimneys ☐ Heavy Cladding or Heavy Veneer ☐ Parapets ☐ Appendages ☐ Other:

COMMENTS:
- Existing building appears to be in poor condition
- Floor joist,beam, stud and sill have water damage.
- Front facade has no clear lateral force resisting system.
- Interior lateral force resisting system (E-W) is not continuous at the crawl space
- Damaged Floor

☐ Additional sketches or comments on separate page



SKETCH

BASIC SCORE, MODIFIERS, AND FINAL LEVEL 1 SCORE, S_{L1}

FEMA BUILDING TYPE	Do Not Know	W1 (marked)	W1A	W2	S1 (MRF)	S2 (BR)	S3 (LM)	S4 (RC SW)	S5 (URM INF)	C1 (MRF)	C2 (SW)	C3 (URM INF)	PC1 (TU)	PC2	RM1 (FD)	RM2 (RD)	URM	MH
Basic Score		**2.1**	**1.9**	**1.8**	**1.5**	**1.4**	**1.6**	**1.4**	**1.2**	**1.0**	**1.2**	**0.9**	**1.1**	**1.0**	**1.1**	**1.1**	**0.9**	**1.1**
Severe Vertical Irregularity, V_{L1}		-0.9	-0.9	-0.9	-0.8	-0.7	-0.8	-0.7	-0.7	-0.7	-0.8	-0.6	-0.7	-0.7	-0.7	-0.7	-0.6	NA
Moderate Vertical Irregularity, V_{L1}		-0.6 (marked)	-0.5	-0.5	-0.4	-0.4	-0.5	-0.4	-0.3	-0.4	-0.4	-0.3	-0.4	-0.4	-0.4	-0.4	-0.3	NA
Plan Irregularity, P_{L1}		-0.7 (marked)	-0.7	-0.6	-0.5	-0.5	-0.6	-0.4	-0.4	-0.4	-0.5	-0.3	-0.5	-0.4	-0.4	-0.4	-0.3	NA
Pre-Code		-0.3 (marked)	-0.3	-0.3	-0.3	-0.2	-0.3	-0.2	-0.1	-0.1	-0.2	0.0	-0.2	-0.1	-0.2	-0.2	0.0	0.0
Post-Benchmark		1.9	1.9	2.0	1.0	1.1	1.1	1.5	NA	1.4	1.7	NA	1.5	1.7	1.6	1.6	NA	0.5
Soil Type A or B		0.5	0.5	0.4	0.3	0.3	0.4	0.3	0.2	0.2	0.3	0.1	0.3	0.2	0.3	0.3	0.1	0.1
Soil Type E (1-3 stories)		0.0	-0.2	-0.4	-0.3	-0.2	-0.2	-0.2	-0.1	-0.1	-0.2	0.0	-0.2	-0.1	-0.2	-0.2	0.0	-0.1
Soil Type E (> 3 stories)		-0.4	-0.4	-0.4	-0.3	-0.3	NA	-0.3	-0.1	-0.1	-0.3	-0.1	NA	-0.1	-0.2	-0.2	0.0	NA
Minimum Score, S_{MIN}		*0.7*	*0.7*	*0.7*	*0.5*	*0.5*	*0.5*	*0.5*	*0.5*	*0.3*	*0.3*	*0.3*	*0.2*	*0.2*	*0.3*	*0.3*	*0.2*	*1.0*

FINAL LEVEL 1 SCORE, $S_{L1} \geq S_{MIN}$: [S.L1 = 0.5] < [S.min = 0.7]... Therefore, use S.min = 0.7

EXTENT OF REVIEW

Exterior: [X] Partial ☐ All Sides ☐ Aerial
Interior: ☐ None [X] Visible [X] Entered
Drawings Reviewed: ☐ Yes [X] No
Soil Type Source:
Geologic Hazards Source:
Contact Person:

LEVEL 2 SCREENING PERFORMED?

[X] Yes, Final Level 2 Score, S_{L2} 0.7 ☐ No
Nonstructural hazards? ☐ Yes [X] No

OTHER HAZARDS

Are There Hazards That Trigger A Detailed Structural Evaluation?

☐ Pounding potential (unless S_{L2} > cut-off, if known)
☐ Falling hazards from taller adjacent building
☐ Geologic hazards or Soil Type F
☐ Significant damage/deterioration to the structural system

ACTION REQUIRED

Detailed Structural Evaluation Required?

☐ Yes, unknown FEMA building type or other building
[X] Yes, score less than cut-off
☐ Yes, other hazards present
☐ No

Detailed Nonstructural Evaluation Recommended? *(check one)*

☐ Yes, nonstructural hazards identified that should be evaluated
☐ No, nonstructural hazards exist that may require mitigation, but a detailed evaluation is not necessary
☐ No, no nonstructural hazards identified ☐ DNK

Where information cannot be verified, screener shall note the following: EST = Estimated or unreliable data OR DNK = Do Not Know

Legend: MRF = Moment-resisting frame, BR = Braced frame, RC = Reinforced concrete, SW = Shear wall, URM INF = Unreinforced masonry infill, TU = Tilt up, MH = Manufactured Housing, LM = Light metal, FD = Flexible diaphragm, RD = Rigid diaphragm

Rapid Visual Screening of Buildings for Potential Seismic Hazards **Level 2 (Optional)**

FEMA P-154 Data Collection Form **VERY HIGH Seismicity**

Optional Level 2 data collection to be performed by a civil or structural engineering professional, architect, or graduate student with background in seismic evaluation or design of buildings.

Bldg Name: 499 & 497 W. San Carlos	**Final Level 1 Score:**	S_{L1} = 0.5	(do not consider S_{MIN})
Screener: DYL	**Level 1 Irregularity Modifiers:**	Vertical Irregularity, V_{L1} = -0.6	Plan Irregularity, P_{L1} = -0.7
Date/Time: 10/20/2020 10:30 am	**ADJUSTED BASELINE SCORE:**	$S' = (S_{L1} - V_{L1} - P_{L1})$ = 1.8	

STRUCTURAL MODIFIERS TO ADD TO ADJUSTED BASELINE SCORE

Topic		Statement (If statement is true, circle the "Yes" modifier; otherwise cross out the modifier.)	Yes	Subtotals
Vertical Irregularity, V_{L2}	Sloping Site	W1 building: There is at least a full story grade change from one side of the building to the other.	-0.9	
		Non-W1 building: There is at least a full story grade change from one side of the building to the other.	-0.2	
	Weak and/or Soft Story (circle one maximum)	W1 building cripple wall: An unbraced cripple wall is visible in the crawl space.	-0.5	
		W1 house over garage: Underneath an occupied story, there is a garage opening without a steel moment frame, and there is less than 8' of wall on the same line (for multiple occupied floors above, use 16' of wall minimum).	-0.9	
		W1A building open front: There are openings at the ground story (such as for parking) over at least 50% of the length of the building.	-0.9	
		Non-W1 building: Length of lateral system at any story is less than 50% of that at story above or height of any story is more than 2.0 times the height of the story above.	-0.7	
		Non-W1 building: Length of lateral system at any story is between 50% and 75% of that at story above or height of any story is between 1.3 and 2.0 times the height of the story above.	-0.4	
	Setback	Vertical elements of the lateral system at an upper story are outboard of those at the story below causing the diaphragm to cantilever at the offset.	-0.7	
		Vertical elements of the lateral system at upper stories are inboard of those at lower stories.	-0.4	
		There is an in-plane offset of the lateral elements that is greater than the length of the elements.	-0.2	
	Short Column/ Pier	C1,C2,C3,PC1,PC2,RM1,RM2: At least 20% of columns (or piers) along a column line in the lateral system have height/depth ratios less than 50% of the nominal height/depth ratio at that level.	-0.4	
		C1,C2,C3,PC1,PC2,RM1,RM2: The column depth (or pier width) is less than one half of the depth of the spandrel, or there are infill walls or adjacent floors that shorten the column.	-0.4	
	Split Level	There is a split level at one of the floor levels or at the roof.	-0.4	
	Other Irregularity	There is another observable severe vertical irregularity that obviously affects the building's seismic performance.	-0.7	V_{L2} = -1.7
		There is another observable moderate vertical irregularity that may affect the building's seismic performance.	-0.4	(Cap at -0.9)
Plan Irregularity, P_{L2}		Torsional irregularity: Lateral system does not appear relatively well distributed in plan in either or both directions. (Do not include the W1A open front irregularity listed above.)	-0.5	
		Non-parallel system: There are one or more major vertical elements of the lateral system that are not orthogonal to each other.	-0.2	
		Reentrant corner: Both projections from an interior corner exceed 25% of the overall plan dimension in that direction.	-0.2	
		Diaphragm opening: There is an opening in the diaphragm with a width over 50% of the total diaphragm width at that level.	-0.2	
		C1, C2 building out-of-plane offset: The exterior beams do not align with the columns in plan.	-0.2	P_{L2} = -0.5
		Other irregularity: There is another observable plan irregularity that obviously affects the building's seismic performance.	-0.5	(Cap at -0.7)
Redundancy		The building has at least two bays of lateral elements on each side of the building in each direction.	+0.2	
Pounding		Building is separated from an adjacent structure by less than 1.5% of the height of the shorter of the building and adjacent structure and: The floors do not align vertically within 2 feet. (Cap total pounding modifiers at -0.9)	-0.7	
		One building is 2 or more stories taller than the other.	-0.7	
		The building is at the end of the block.	-0.4	
S2 Building		"K" bracing geometry is visible.	-0.7	
C1 Building		Flat plate serves as the beam in the moment frame.	-0.3	
PC1/RM1 Bldg		There are roof-to-wall ties that are visible or known from drawings that do not rely on cross-grain bending. (Do not combine with post-benchmark or retrofit modifier.)	+0.2	
PC1/RM1 Bldg		The building has closely spaced, full height interior walls (rather than an interior space with few walls such as in a warehouse).	+0.2	
URM		Gable walls are present.	-0.3	
MH		There is a supplemental seismic bracing system provided between the carriage and the ground.	+0.5	
Retrofit		Comprehensive seismic retrofit is visible or known from drawings.	+1.2	M = ______

FINAL LEVEL 2 SCORE, $S_{L2} = (S' + V_{L2} + P_{L2} + M) \geq S_{MIN}$: [S.L2 = -0.4] < [S.min = 0.7]... Therefore, use S.min = 0.7 (Transfer to Level 1 form)

There is observable damage or deterioration or another condition that negatively affects the building's seismic performance: ☐ Yes ☐ No

If yes, describe the condition in the comment box below and indicate on the Level 1 form that detailed evaluation is required independent of the building's score.

OBSERVABLE NONSTRUCTURAL HAZARDS

Location	Statement (Check "Yes" or "No")	Yes	No	Comment
Exterior	There is an unbraced unreinforced masonry parapet or unbraced unreinforced masonry chimney.			
	There is heavy cladding or heavy veneer.			
	There is a heavy canopy over exit doors or pedestrian walkways that appears inadequately supported.			
	There is an unreinforced masonry appendage over exit doors or pedestrian walkways.			
	There is a sign posted on the building that indicates hazardous materials are present.			
	There is a taller adjacent building with an unanchored URM wall or unbraced URM parapet or chimney.			
	Other observed exterior nonstructural falling hazard:			
Interior	There are hollow clay tile or brick partitions at any stair or exit corridor.			
	Other observed interior nonstructural falling hazard:			

Estimated Nonstructural Seismic Performance (*Check appropriate box and transfer to Level 1 form conclusions*)

- ☐ Potential nonstructural hazards with significant threat to occupant life safety → Detailed Nonstructural Evaluation recommended
- ☐ Nonstructural hazards identified with significant threat to occupant life safety → But no Detailed Nonstructural Evaluation required
- X Low or no nonstructural hazard threat to occupant life safety → No Detailed Nonstructural Evaluation required

Comments:

ATC Hazards by Location

Search Information

Address: 499 W San Carlos St, San Jose, CA 95110, USA

Coordinates: 37.3259945, -121.8985611

Elevation: 94 ft

Timestamp: 2020-10-23T01:06:23.812Z

Hazard Type: Seismic

Reference Document: ASCE7-16

Risk Category: II

Site Class: D-default

Basic Parameters

Name	Value	Description
S_S	1.5	MCE_R ground motion (period=0.2s)
S_1	0.6	MCE_R ground motion (period=1.0s)
S_{MS}	1.8	Site-modified spectral acceleration value
S_{M1}	* null	Site-modified spectral acceleration value
S_{DS}	1.2	Numeric seismic design value at 0.2s SA
S_{D1}	* null	Numeric seismic design value at 1.0s SA

* See Section 11.4.8

Additional Information

Name	Value	Description
SDC	* null	Seismic design category
F_a	1.2	Site amplification factor at 0.2s
F_v	* null	Site amplification factor at 1.0s
CR_S	0.96	Coefficient of risk (0.2s)
CR_1	0.935	Coefficient of risk (1.0s)
PGA	0.521	MCE_G peak ground acceleration
F_{PGA}	1.2	Site amplification factor at PGA
PGA_M	0.625	Site modified peak ground acceleration

T_L	12	Long-period transition period (s)
SsRT	2.092	Probabilistic risk-targeted ground motion (0.2s)
SsUH	2.18	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
SsD	1.5	Factored deterministic acceleration value (0.2s)
S1RT	0.773	Probabilistic risk-targeted ground motion (1.0s)
S1UH	0.827	Factored uniform-hazard spectral acceleration (2% probability of exceedance in 50 years)
S1D	0.6	Factored deterministic acceleration value (1.0s)
PGAd	0.521	Factored deterministic acceleration value (PGA)

* See Section 11.4.8

The results indicated here DO NOT reflect any state or local amendments to the values or any delineation lines made during the building code adoption process. Users should confirm any output obtained from this tool with the local Authority Having Jurisdiction before proceeding with design.

Disclaimer

Hazard loads are provided by the U.S. Geological Survey Seismic Design Web Services.

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