Technical Report

May 2020

Performance Based Seismic Design of Lateral Force Resisting System

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This report has been prepared at the end of the “ongoing phase” spent at the University of California in San Diego by the Experienced Researcher Kenan Michel, as part of CIC-BREL project (Cracked and Inelastic Calculation of BRacing Elements). This project has received funding from the European Union’s Horizon 2020 research and innovation program under agreement No 744332 – CIC – BREL.

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Acknowledgement

This project has been carried out mainly under the guidance of Professor José Restrepo and commendable mentoring of Rodolfo Sanchez, Teaching Assistant.

Basic information and a lot of support are taken from the classes of Prof. Benson Shing in the class Earthquake Engineering and Prof. Joel Conte in the class Nonlinear Structural Analysis.

I would also like to thank the Department of Structural Engineering, UCSD for availing this opportunity to learn these courses.

Moreover, fellow classmates, Molly Pobiell and Dhyey Bhavsar played an important role in developing and writing some parts of this report when working as a project team, I highly appreciate their contribution and work.

PART I: General Information, Site and Loading

1. General Information About the Building

A fourteen-story residential building will be constructed in Downtown LA, California. A typical plan view of the building is shown in Figure I.1. Floor to floor height is 12' for all floors except for the first level, in which the story height is 16'. The North-South (N-S) and East-West (E-W) lateral force resisting system for the building consists of one core wall. Since the core wall will be framing the elevator shafts and stairwells, the architect provided specific centerline dimensions for the walls. The specified centerline length of the walls is 10'-6" in the E-W direction and 24' in the N-S direction, with a wall thickness of 30" at stories 1-5 and 22" at stories 6-14, as shown in Figure I.2. The coupling beams spanning 7' between the c-shaped walls are 42" deep with the same width of the walls. The slabs are 8" reinforced concrete supported by 28" square columns. The contribution to the lateral resistance from the gravity framing system will be ignored. For this project, the lateral force resisting system in the N-S and E-W directions will be designed and detailed. The model of ETABS will be already provided.

Figure I.1 Typical Floor Framing Plan

Figure I.2 Typical core wall plan view and dimensions.

Note: wall thickness t = of 30" at stories 1-5 and t = 22" at stories 6-14.

1.1. Specified Material Properties:

Concrete compressive strength (cylinder):	$f'_c = 7$ ksi
Concrete unit weight:	$w = 150$ pcf
Steel reinforcement yield strength Grade 60 ASTM A615:	$f_y = 60$ ksi

1.2. Site Information:

Location coordinates: 34.05° N, 118.26° W
 550 South Hope St, Los Angeles, CA 90071 (Downtown LA)
 Site soil class: D "Stiff soil"
 Code reference document: ASCE 7-16
 Risk category: non-essential facility (II), $I_e = 1.0$

1.3. Geometry (Figure I.1):

Typical floor height = 12'

First floor height = 16'

RC concrete slab thickness = 8"

Columns = 28"x28" square (maximum stress from unfactored DL < $0.25*(f'_c)$)

Wall centerline dimensions: 24' NS, 10'-6" EW, t= 30" (stories 1-5) - 22" (stories 6-14)

Coupling beam dimensions: 30"-22" wide x 42" deep (Figure I.2)

2. Site Seismicity and Design Coefficients

2.1. USGS Results

Site information was input into USGS website (Figure I.3) to obtain the mapped acceleration parameters per ASCE 7-16 §11.4.1.

Figure I.3 Map of site location. Location coordinates: 34.05° N, 118.26° W

Mapped acceleration parameters

$$\begin{aligned} S_s &= 1.966 \text{ g} & S_{MS} &= 1.970 \text{ g} & S_{DS} &= 2/3 * S_s = 1.311 \text{ g} \\ S_1 &= 0.700 \text{ g} & S_{M1} &= 1.190 \text{ g} & S_{D1} &= 2/3 * S_1 = 0.793 \text{ g} \end{aligned}$$

Seismic Design Category E

2.2. Site Response Spectra

Using this information, the design response spectrum and maximum considered earthquake spectrum are plotted per ASCE 7-16 §11.4 (see Figures I.4 and I.5)

Figure I.4 Design response spectrum

11.4.6 Design response spectrum

Figure I.5 Design response spectrum for our site ($\zeta=5\%$)

2.3. Design Coefficients And Factors For Seismic Force-Resisting Systems

An assumption is made that only walls resist lateral loading and that the columns and beams are for gravity only. From ASCE 7-16 Table 12.2-1, the design coefficients and factors can be determined. The same coefficients are used for the N-S and E-W lateral force resisting systems.

Bearing Wall System	Response Modification Coefficient, R	Overstrength Factor, Ω_0	Deflection Amplification Factor, C_d
Special reinforced concrete shear wall	6	2.5	5

3. Loading

3.1. Determination Of Seismic Forces

For gravity loading, the core walls will be subjected to a dead load from the weight of the structure and the self-weight of the walls themselves. Additionally, a superimposed dead load will act on the floors.

Superimposed Floor Dead Load

Covering	1 psf
Built-in partition	15 psf
Ceiling	3 psf
Mechanical electrical	3 psf
Façade	15 psf
Misc.	3 psf
Sum	40 psf

Structural seismic mass and seismic weight can be determined from the ETABS model (Figure I.6) and using the equivalent lateral force procedure outlined in ASCE 7-16 §12.8, the seismic base shear (V) in X and Y directions considering redundancy factor $\rho = 1.3$ (Seismic Design Category E in ASCE 7-16 chapter 12.3.4.2) as well as the fundamental periods in each direction are determined.

W_{tot}	22541 kip
C_{sx}	0.135 g
V_x	3051 kip
T_x	1.27 s
C_{sy}	0.129 g
V_y	2913 kip
T_y	1.33 s

3.2. Modal Response Spectrum Analysis

The lateral forces for the building analysis followed section 12.9 in ASCE 7-16. A modal response spectrum analysis method was performed in order to obtain the lateral force distribution. First, a modal response spectrum analysis was performed in both directions. According to ASCE 7-16 12.9.1, the analysis shall include a sufficient number of modes

to obtain a combined modal mass participation of at least 90% of the mass in each direction, a maximum of 30 modes were computed in the ETABS model with such purpose. **Modes 2 and 1** for the E-W and N-S respectively resulted the fundamental periods in each direction. The corresponding base shears **V_{tx}** and **V_{ty}**, were obtained and compared to 100% of the base shear obtained from the equivalent lateral force procedure. V_t in both directions was found to be less than V from ELF procedure, so the forces were increased by V_x/V_{tx} and V_y/V_{ty}, for the E-W and N-S directions, respectively.

EW (X)		NS (Y)	
Mode	Period (s)	Mode	Period (s)
2	1.27	1	1.33

V_{tx}: 2446 kip < V_x = 3051 kip

V_{ty}: 2308 kip < V_y = 2913 kip

Amplification factor

V_x /V_{tx}: 1.25

V_y /V_{ty}: 1.26

Figure I.6 Model of ETABS

3.3. Seismic Load Effects And Combinations

The seismic load effect, E, is defined in ASCE 7-16 §12.4.2 as:

$$E = E_h + E_v \quad \text{or} \quad E = E_h - E_v \quad (\text{depending on the load combination})$$

where horizontal seismic load effect is defined by ASCE 7-16 §12.4.2.1 as:

$$E_h = \rho Q E$$

Per §12.3.4.2, the redundancy factor, ρ , is taken as 1.3. The vertical seismic load effect is defined by §12.4.2.1 as:

$$E_v = 0.2 S_{DS} D$$

The basic combinations used for this analysis are defined by §2.3.2:

$$\frac{1.2D + 1.0E + 1.0L}{0.9D + 1.0E}$$

or as defined by §2.3.6 equations 6 and 7 as:

$$\frac{1.2D + E_h + E_v + 1.0L + 0.2S}{0.9D + E_h - E_v}$$

Substituting the values for E into the loading combinations:

$$\begin{aligned} & (1.2 + 0.2S_{DS}) D + L \pm 1.0E_h \\ & (0.9 - 0.2S_{DS}) D \pm 1.0E_h \end{aligned}$$

Using $S_{DS} = 1.311$, the final seismic load combinations used for analysis are:

$$\begin{aligned} & (1.462) D + L \pm 1.0E_h \\ & (0.638 D) \pm 1.0E_h \end{aligned}$$

PART II: Core Wall Design - Linear Model

4. Model of ETABS

4.1. Geometry

The analysis was run with stiffness modifiers equal to $0.5 \cdot I_g$ in both c-shaped walls, according with the suggested for the ACI 318-14, whereas the coupling beams used stiffness modifiers equal to $0.3 \cdot I_g$.

Note: From that analysis, the trailing wall experiences tensile axial force from lateral loading in the E-W direction. Since the shear capacity of the wall is greatly reduced if it is experiencing a tensile axial force, spurious shear distribution between trailing and leading walls is expected in the linear analysis.

Figure II.1 Extruded elevation of the corewall

4.2. Gravity Loads

Figure II.2 Typical plan view of model.

Figure II.3 Elevation D (left) and Elevation 2(right)

Figure II.4 Superimposed dead load (SDL) (ksf).

Figure II.5 Live load (Live) (ksf).

Note: Dead load (Dead) from self-weight is consider automatically in the ETABS model.

4.3. Seismic Loads

Unless otherwise indicated, results will be presented in kips (k) and kip-feet (k-ft). A positive axial load is tension, and a negative axial load is compression. The sign convention for the moments is shown in Figure II.6.

Figure II.6 Sign convention

4.4. Tabulated Selected Results From ETABS Analysis

Moments NS and shear forces EW at the base of the C-shaped wall at the left.

No	LC	Pu (kip)	Mu, NS (kip-ft)	Vu, EW (kip)	$\delta u/hw$
1	0.9D - 0.2SDS + 1.0EX Max	7384	33485	1529	0.0060
2	0.9D - 0.2SDS + 1.0EX Min	-14767	-33151	-1480	0.0060
3	1.2D + 0.2SDS + 0.5L + 1.0EX Max	2146	33724	1564	0.0060
4	1.2D + 0.2SDS + 0.5L + 1.0EX Min	-20005	-32912	-1445	0.0060

Notes:

Pu (-) is compression in the C-shaped wall.

For sign convention of Mu see Figure II.6.

If Vu, EW is (+) the shear force goes to the East.

Moments EW and shear forces NS at the base of the C-shaped wall at the left.

No	LC	Pu (kip)	Mu, NS (kip-ft)	Vu, EW (kip)	$\delta u/hw$
1	0.9D - 0.2SDS + 1.0EX Max	-3691	144880	1442	0.0066
2	0.9D - 0.2SDS + 1.0EX Min	-3691	-144880	-1442	0.0066
3	1.2D + 0.2SDS + 0.5L + 1.0EX Max	-8930	144880	1442	0.0066
4	1.2D + 0.2SDS + 0.5L + 1.0EX Min	-8930	-144880	-1442	0.0066

Notes:

Pu (-) is compression in the C-shaped wall.

For sign convention of Mu see Figure II.6.

If Vu, NS is (+) the shear force goes to the North.

Shear forces in the coupling beams (face N or S)

Story	0.9D - 0.2SDS + 1.0EX Vu (kip)	1.2D + 0.2SDS + 0.5L + 1.0EX Vu (kip)
14	131	134
13	192	195
12	260	263
11	322	325
10	372	375
9	413	416
8	447	450
7	474	477
6	486	489
5	611	615
4	620	624
3	626	630
2	606	610
1	508	512

5. P-M Interaction Diagrams

A P-M interaction diagram in excel is developed for C shape core wall for both cases of loading, N-S and EW. Each of them covers the case when the bending moment is positive and negative.

5.1. N-S Direction

For N-S loading, input data and the geometry are shown in figure II.7

Figure II.7 C shape core wall, geometry and material in N-S direction

For the N-S direction, when putting the values of P_u and $M_{u,NS}$ we get a maximum reinforcement ratio of 1.4% as shown in table II.1 and figure II.8. In figure II.8, the limit values of P_u and $M_{u,NS}$ are shown in red dots.

Load / No.	LC 1	LC 2	LC 3	LC 4
P _u	7384	-14767	2146	-20005
M _{u,ns}	33485	-33151	33724	-32912
V _{u,ew}	1529	-1480	1564	-1445
δ _u / h _w	0.0066	0.0066	0.0066	0.0066
cMax	23.74	23.74	23.74	23.74
ro required	1.4	0.4	0.8	0.4
ro max	1.4	1.4	1.4	1.4
c [inch]	4	12.9	25	18
Boundary	no	no	yes	no

Table II.1 Loads and required reinforcement ratios, C shape wall in N-S direction

Figure II.8 P-M Interaction diagram, C shape wall in N-S direction

5.2. E-W Direction

In E-W direction the input values and geometry are shown in figure II.9.

Figure II.9 C shape core wall, geometry and material in E-W direction

For the E-W direction, we the values of P_u and $M_{u,EW}$ we get a maximum reinforcement ratio of 1.0% as shown in table II.2 and figure II.10. In figure II.10, the limit values of P_u and $M_{u,EW}$ are shown in red dots.

Load / No.	LC 1	LC 2	LC 3	LC 4
P_u	-3691	-3691	-8930	-8930
$M_{u,ns}$	144880	-144880	144880	-144880
$V_{u,ew}$	1442	-1442	1442	-1442
δ_u / h_w	0.006	0.006	0.006	0.006
cMax	26.11	58.89	58.89	58.89
ro required	1	0.4	0.4	1
ro max	1	1	1	1

Table II.2 – Loads and required reinforcement ratios, C shape wall in E-W direction

Figure II.10 P-M Interaction diagram, C shape wall in E-W direction

6. Lateral Force Resisting System, Linear

6.1. Longitudinal Reinforcement

The dimensions of the shear wall are already given thus the next step in designing the shear walls is to determine the required longitudinal reinforcement. From the P-M envelopes developed before. The demands are plotted in red in figures II.8 for NS and figure II.10 for EW directions.

The required reinforcement ratio in the N-S direction $\rho_{l \max NS}$ was found 1.4%, see figure II.8 and table II.1. The required longitudinal reinforcement ratio in E-W direction $\rho_{l \max EW}$ was found to be 1%, see figure II.10 and table II.2. The required reinforcement ratio is then $\{\max(\rho_{l \max EW}, \rho_{l \max NS}) = 1.4\%$ for the whole cross section.

This value is bigger than the minimum longitudinal reinforcement ratio of 0.25% per ACI 318-14 chapter. 18.10.2.1 and the minimum reinforcement ratio of 0.42% per chapter. 9.6.1.2. It is also less than the maximum reinforcement ratio of 2.5% per chapter 18.6.3.1.

Choosing the required bars is done in both directions N-S and E-W. In N-S direction the shape and required reinforcement areas are shown in figure II.11

NS direction

Lw1	in	141
Lw2	in	318
t1	in	30
t2	in	30

Required		
ρ		1.40%
As1	in2	108.4
As2	in2	29.6
As3	in2	29.6
Ast	in2	226.8

Chosen			
As1	84 # 10	in2	106.7
As2	24 # 10	in2	30.5
As3	24 # 10	in2	30.5
Ast		in2	228.6

Figure II.11 Required reinforcement areas for N-S direction

In E-W direction the shape and required reinforcement areas are shown in figure II.12

EW direction

Lw1	in	141
Lw2	in	318
t1	in	30
t2	in	30

Required		
ρ		1.40%
As1	in2	46.6
As2	in2	66.8
As3	in2	66.8
As4	in2	46.6
Ast	in2	226.8

Chosen			
As1	38 # 10	in2	48.3
As2	52 # 10	in2	66.0
As3	52 # 10	in2	66.0
As4	38 # 10	in2	48.3
Ast		in2	228.6

Figure II.12 Required reinforcement areas for E-W direction

By considering both directions, with the available perimeter of the section to distribute the bars on it, the resulting longitudinal reinforcement should look like the figure II.13:

Figure II.13 Distributing longitudinal bars in the section, and bar spacing

6.2. Shear Reinforcement

The procedure to design the c-shape wall for shear is shown as follows:

- According to ACI 318-14, chapter 18.10.2.2 two curtain of shear reinforcement will be used.
- According to ACI 318-14, chapter 18.10.2.3 development length are defined in table II.3:

18.10.2.3	development length				
	(a) extend long. reinf. beyond flexure at least 0.8 Lw=0.8*				
		Lw1 = [inch]	141		
		0.8 Lw1 =	113		
	(b) development length calculated from 25.4.2.4 as				
		fy [psi]	60000		
		fc' [psi]	7000		
		lamda	1		
		psi_t	1		
		psi_e	1		
		psi_s	1		
		cb	2.7		
		Ktr	0.16	Atr=	0.4
		(cb+Ktr)/db=	2.25	(cb+Ktr)/db=	2.25
		db	1.27	< 2.5	2.2519685
		Ld [inch]	30		
		Ld [inch]	38		

$$\ell_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\psi_t \psi_e \psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b$$

(25.4.2.3a)

1.25 fy 18.10.2.3 (b)

Table II.3 – Development length calculation of long. Reinf.

>> Development length 113" (in the foundation)

- According to ACI 318-14, chapter 18.10.3 shear demand and the parts of the c-shape wall that carry the shear in each direction are shown in table II.4:

18.10.3 Vu values are given as

	E-W	N - S
max	1564	1442
min	-1480	-1442
Max (abs)	1564	1442

Lw1	in	141
Lw2	in	318
t1	in	30
t2	in	30

Table II.4 – Shear demand and section geometry

- According to ACI 318-14, chapter 18.10.4 shear capacity and the design of shear reinforcement is shown in table II.5:

Shear NS			Shear EW		
hw	172.00	ft	hw	172.00	ft
lw2	26.50	ft	lw1	11.75	ft
t2	2.50	ft	2*t1	5.00	ft
hw/lw	6.49		hw/lw	14.64	
Acv	66.25	ft^2	Acv	58.75	ft^2
α	2	*ACI 318-14 ch18.10.4.1 hw/lw>2	α	2	*ACI 318-14 ch18.10.4.1 hw/lw>2
λ	1		λ	1	
f'c	7000	psi	f'c	7000	psi
fy	60	ksi	fy	60	ksi
ϕ	0.6	ch. 21.2.4.1	ϕ	0.6	ch. 21.2.4.1
Vn,concrete	1,596	kips	Vn,concrete	1,416	kips
Vu	1442	kips	Vu	1564	kips
Vn,steel,req	807	kips	Vs,req	1,191	kips
pt,req	0.141%	Ast/Avc	pt,req	0.235%	Ast/Avc
pt,min	0.250%	Ast/Avc	pt,min	0.250%	Ast/Avc
pt	0.250%	Ast/Avc	pt	0.250%	Ast/Avc
Ab, #4	0.2	in^2	Ab, #4	0.2	in^2
no of curtains	2		no of curtains	4	
s,req	5.33	in	s,req	5.33	in
s	5	in	s	5	in
pt,provided	0.267%		pt,provided	0.267%	
Vn	3,122.75	kips	Vn	2,769.23	kips
ϕVn	1,873.65	Kips	ϕVn	1,661.54	Kips
DCR	0.77		SF	0.94	
10AcwSqrt(f'c)	7,981.74	kips	10AcwSqrt(f'c)	7,078.14	Kips
Vn too large?	Pass	ch. 18.10.4.5	Vn too large?	Pass	ch. 18.10.4.5
use 2#4 @ 5" o.c.			use 2 x 2#4 @ 5" o.c.		

Table II.5 – Shear design of c-shape walls

The reinforcement of 2#4 @ 5" o.c. and 2 x 2#4 @ 5" o.c. will be used parallel to the surface of the wall cross section. Perpendicular to the surface the provisions of ACI 318-14 chapter 25.7.5.3 will be used; using **#3 hooks** will be required at each alternate longitudinal bar.

6.3. Boundary Elements

According to ACI 318-14 chapter 18.10.6.2, boundary element is required in wall when $hw/lw = (16' \cdot 1 + 12' \cdot 13)/10' - 6" > 2$ when the compression zone c satisfies:

$$c \geq \frac{\ell_w}{600(1.5\delta_u/h_w)} \quad (18.10.6.2)$$

Values for c for the load cases in N-S direction and the requirement of boundary elements are summarized in table II.6. It shows that BE are required at edge of the flanges of the c-shape walls. The dimensions of these boundary elements are defined using ACI 318-14 chapter 18.10.6.4

Load / No.	LC 1	LC 2	LC 3	LC 4
Pu	7384	-14767	2146	-20005
Mu,ns	33485	-33151	33724	-32912
Vu,ew	1529	-1480	1564	-1445
δ_u / h_w	0.0066	0.0066	0.0066	0.0066
cMax	23.74	23.74	23.74	23.74
ro required	1.4	0.4	0.8	0.4
ro max	1.4	1.4	1.4	1.4
c [inch]	21.7	16.1	33.1	18.9
Boundary	no	no	yes	no
0.1 Lw1 [inch]	14.1	14.1	14.1	14.1
c/2 =			16.55	
c-0.1Lw1=			19	
L_BE [inch] =	Max (c/2, c0.1Lw1)		19	required
Prof. Advice	Lw1 / 3		70.5	
flanged section	33.1	compressed width 18.10.6.4 (d)		
choose L_BE [inch]	30			chosen
b_be [inch]	hu/16	hu=16'*12	12	>12 inch ok
hw = [inch]	2064			
Lw1 = [inch]	141			
hw/Lw1=	14.6			

Table II.6: Load cases, and the need for boundary element, N-S moment

Similar check for the c values for the load cases in E-W direction shows that BE are not required, see table II.7.

Load / No.	LC 1	LC 2	LC 3	LC 4
Pu	-3691	-3691	-8930	-8930
Mu,ns	144880	-144880	144880	-144880
Vu,ew	1442	-1442	1442	-1442
δ_u / h_w	0.006	0.006	0.006	0.006
cMax	26.11	58.89	58.89	58.89
ro required	1	0.4	0.4	1
ro max	1	1	1	1
ro used	1.4	1.4	1.4	1.4
C	6	8	17	18
BE required	No	No	No	No

Table II.7: Load cases, and the need for boundary element, E-W moment

6.3.1. Transverse Reinforcement Of Boundary Elements

According to ACI 318-14 chapter 18.10.6.2 (b), where special boundary elements are required by (a), the special boundary element transverse reinforcement shall extend vertically above and below the critical section at least the greater of ℓ_w and $M_u/4V_u$. The extend of this transverse reinforcement is shown in table II.8 for E-W direction. The required height is 11.8 ft, choose 12 ft in the first floor (from total height of 16 ft), and over the whole height of the second floor (height is 12ft).

Moment N-S and shear EW				
Load / No.	LC 1	LC 2	LC 3	LC 4
Pu	7384	-14767	2146	-20005
Mu,ns	33485	-33151	33724	-32912
Vu,ew	1529	-1480	1564	-1445
$M_u/4V_u$ [ft]	5.5	5.6	5.4	5.7
Lw1 [ft]	11.8	11.8	11.8	11.8
Max(Lw1, $M_u/4V_u$) [ft]	11.8	11.8	11.8	11.8

Table II.8: Extend of transversal rebar in boundary element

The design of shear rebar in boundary elements is achieved in accordance with ACI 318-14 chapter 18.10.6.4 (e). The chosen rebar diameter could be #3 or #4, #4 has been chosen to get the same hoops as in the wall. All the requirements of chapter 18.7.5.2 have been checked and met.

Smax is the minimum value of: a $1/3$ least dimension of the boundary i.e. $30''/3=10''$

b $6d_b$ of long. bar i.e. $6 \times 1.27'' = 7.62''$

c $S_o = 4 + (14 - h_x)/3 = 4 + (14 - 4.9 \times 2)/3 = 5.4''$ ($4'' < 5.4'' < 6''$)

Amount of transverse rebar is defined by 18.10.6.4 (f)

Table 18.10.6.4(f)—Transverse reinforcement for special boundary elements

Transverse reinforcement	Applicable expressions	
A_{sh}/sb_c for rectilinear hoop	Greater of	$0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (a)
		$0.09 \frac{f'_c}{f_{yt}}$ (b)

Choose #4 for transverse rebar,

$$A_{sh} = 4 * 0.2 = 0.80 \text{ inch}^2 \quad S=5'' \quad bc=24.7'' \quad A_g=30*30=900 \text{ inch}^2$$

$$A_{ch}=24.7*24.7=610 \text{ inch}^2 \quad f'_c=7 \text{ ksi} \quad f_{yt}=60 \text{ ksi}$$

$$\text{Max} \left\{ 0.3 \left(\frac{900}{610} - 1 \right) \frac{7}{60}, 0.09 \frac{7}{60} \right\} = 0.0166$$

$$\frac{A_{sh}}{S b_c} = \frac{0.8}{S * 24.7} = 0.0166 \gg S = \frac{0.8}{0.0166 * 24.7} = 1.95 \text{ inch} \cong 2 \text{ inch}$$

Amount of transverse reinforcement is **#4 @ 2" o.c.**

Figure II.14 Drafting Boundary element, reinforcements and spacings

6.4. Coupling Beams

Designing the coupling beams is achieved according to ACI 318-14 chapter 18.10.7. Calculation is done according to chapter 18.10.7.4 (a):

$$V_n = 2A_{vd}f_y \sin \alpha \leq 10 \sqrt{f'_c} A_{cw} \quad (18.10.7.4)$$

$$A_{vd} = \frac{V_n}{2 f_y \sin \alpha}$$

Max Vu = 610 kips, given from ETABS see chapter 4.4 in this report.

$V_n = Vu / \Phi = 610 / 0.85 = 718$ kips, chapter 21.2.4.3

$\alpha = \arctg \frac{42 - 5.5 \times 2}{84} = 20 \text{ degrees}$ see figure II.15

$A_{vd} = \frac{718}{2 \times 60 \times \sin 20} = 17.49 \text{ inch}^2$ use **12 # 11** db=1.41 inch

$A_{vd,provided} = 12 \times 1.56 = 18.72 \text{ inch}^2 > 17.49 \text{ inch}^2$ ok.

Figure II.15 – Calculating α in the equation 18.10.7.4 of ACI 318-14

Embedding length is calculated according to 18.10.7.4 (b) from 25.4 as follows:

(b) development length calculated from 25.4.2.4 as			
$l_d = \left(\frac{3}{40} \frac{f_y}{\lambda \sqrt{f'_c}} \frac{\Psi_t \Psi_e \Psi_s}{\left(\frac{c_b + K_{tr}}{d_b} \right)} \right) d_b$ <p>(25.4.2.3a)</p>	fy [psi]	60000	
	fc' [psi]	7000	
	lamda	1	
	psi_t	1.3	
	psi_e	1	
	psi_s	1	
	cb	2.7	
	Ktr	2.816	Atr=4*0.44
	(cb+Ktr)/db=	2.5	(cb+Ktr)/db=
	db	1.41	< 2.5
1.25 fy	18.10.2.3 (b)	Ld [inch]	39
		Ld [inch]	49

Horizontal length is 49" x cos 20 = 46" **choose 50"**

Figure II.16 Drafting coupling beam, reinforcements and spacings

The design of shear rebar in coupling beams is achieved in accordance with ACI 318-14 chapter 18.7.5.2 have been checked and met.

Smax is the minimum value of: a 1/3 least dimension of the boundary i.e. 30"/3=10"

b 6db of long. bar i.e. 6*1.41" = 8.46"

c So=4+(14-hx)/3 = 4+(14-5.7*2)/3=4.86" (4"<5.4<6")

Amount of transverse rebar is defined by 18.10.6.4 (f)

Table 18.10.6.4(f)—Transverse reinforcement for special boundary elements

Transverse reinforcement	Applicable expressions	
A_{sh}/sb_c for rectilinear hoop	Greater of	$0.3 \left(\frac{A_g}{A_{ch}} - 1 \right) \frac{f'_c}{f_{yt}}$ (a)
		$0.09 \frac{f'_c}{f_{yt}}$ (b)

A_{sh} horizontal:

2 hoops #6 (4 legs)

$$A_{sh} = 4 * 0.44 = 1.76 \text{ inch}^2 \quad S=5" \quad bc=30-2.7*2=24.6" \quad Ag=30*42=1260 \text{ inch}^2$$

$$A_{ch}=(30-2.7*2)*(42-2.7*2)=900 \text{ inch}^2 \quad f'_c=7 \text{ ksi} \quad f_{yt}=60 \text{ ksi}$$

$$\text{Max}\left\{ 0.3 \left(\frac{1260}{900} - 1 \right) \frac{7}{60} = 0.014, 0.09 \frac{7}{60} = 0.0105 \right\} = 0.014$$

$$\frac{A_{sh}}{S bc} = \frac{1.76}{5*24.6} = 0.014 > S = \frac{1.76}{0.014 * 24.6} = 5.11 \text{ inch}$$

Use 2 hoops (4 legs) # 6 @ 5" o.c.

Smax should be also min (6", 6 db=6x1.41=8.46)=6" > 5" ok.

A_{sh} Vertical

two big hoops #6 and 2 small hoops #4

$$A_{sh} = 4 * 0.44 + 4 * 0.2 = 2.56 \text{ inch}^2 \quad bc=42-2.7*2=36.6" \quad Ag=30*42=1260 \text{ inch}^2$$

$$A_{ch}=(30-2.7*2)*(42-2.7*2)=900 \text{ inch}^2 \quad f'_c=7 \text{ ksi} \quad f_{yt}=60 \text{ ksi}$$

$$\text{Max}\left\{ 0.3 \left(\frac{1260}{900} - 1 \right) \frac{7}{60} = 0.014, 0.09 \frac{7}{60} = 0.0105 \right\} = 0.014$$

$$\frac{A_{sh}}{S bc} = \frac{2.56}{5*24.6} = 0.014 > S = \frac{2.56}{0.014 * 36.6} = 5.00 \text{ inch}$$

Use 2 # 6 @ 5" o.c. and 2 # 4 @ 5" o.c.

Smax should be also min (6", 6 db=6x1.41=8.46)=6" > 5" ok.

7. Detailing

See attachment

PART III: Site Response Spectra and Input Ground Motions

8. Performance Levels

The building should be verified for seismic response for two performance levels:

- a) Fully operational at earthquake intensities corresponding to 69% in 50 years of exposure (resulting in a return period of 43 years), and
- b) Near collapse at earthquake intensities corresponding to 2% in 50 years of exposure (resulting in a return period of 2,475 years).

In this report, the recorded ground motion of M7.3 Landers Earthquake of 28 June 1992 at the Joshua Tree Fire Station site were processed according to the provisions of ASCE 7-16 to design for the 2% in 50 years response.

8.1. ASCE 7-16 Target Spectra

The first step was to find the ASCE 7-16 target spectra. The site class was defined using the site specific low-strain shear wave velocity V_{30} presented in Table III.1.

Name	Thick...	Classification	P...	Description	Modulus Reduct...	Damping Curve	Density	Max. Shear Wave ...
Sand	10.0 ...	USCS SP		Dark gray and y...	Sand (Idriss 199...	Sand (Idriss 1990)	120.0...	776.2 feet per seco...
Sand	5.0 f...	USCS SP		White and brow...	Sand (Idriss 199...	Sand (Idriss 1990)	120.0...	855.9 feet per seco...
Sand	5.0 f...	USCS SP		White and brow...	Sand (Idriss 199...	Sand (Idriss 1990)	120.0...	774.7 feet per seco...
Sandy Clay	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al 1...	Clay (Sun et al ...	120.0...	702.1 feet per seco...
Silty Clay	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	772.4 feet per seco...
Silty Clay	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	1027.2 feet per sec...
Silty Clay	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	997.1 feet per seco...
Sand	5.0 f...	USCS SP		White dense sa...	Sand (Idriss 199...	Sand (Idriss 1990)	120.0...	911.2 feet per seco...
Clayed Sand	5.0 f...	USCS SP		Pale to white cla...	Sand (Idriss 199...	Sand (Idriss 1990)	120.0...	934.7 feet per seco...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	888.0 feet per seco...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	980.5 feet per seco...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	1092.5 feet per sec...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	1075.5 feet per sec...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	120.0...	1336.3 feet per sec...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	130.0...	1523.3 feet per sec...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	130.0...	2202.7 feet per sec...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	130.0...	2204.7 feet per sec...
Silty Caly	5.0 f...	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	130.0...	2204.7 feet per sec...
Silty Caly	Infinite	USCS CL		Light reddish bro...	Clay (Sun et al ...	Clay (Sun et al ...	130.0...	2205.0 feet per sec...

Table III.1 Soil Shear Wave Velocities from Specific Site Study

According to ASCE 7-16 chapter 20.4.1, the average velocity was calculated by Equation 1 as 1046.6 ft/s, which corresponds to site class D, stiff soil.

$$v_s = \frac{\sum_{i=1}^n d_i}{\sum_{i=1}^n \frac{d_i}{v_{si}}} \quad (1)$$

Using the site class and corresponding tables and equations from ASCE 7-16, the response spectrum parameters were determined and summarized in Table III.2.

S_s	S_1	F_a	F_v	S_{MS}	S_{M1}	S_{DS}	S_{D1}	T_L	Site class	Seismicity
1.966 g	0.700 g	1.0	1.7	1.970 g	1.190 g	1.311 g	0.793 g	8 s	D	high

Table III.2 – ASCE 7-16 Acceleration Response Spectrum Parameters

According to ASCE 7-16, chapter 11.4.6, the pseudo-acceleration elastic design spectrum for the risk-targeted MSE_R is simplified to the piecewise relationship shown in Figure III.1.

Figure III.1 ASCE 7-16 Design Response Spectrum

1. For periods less than or equal to T_0 :

$$S_a = 0.6 \frac{S_{MS}}{T_0} T + 0.4 S_{MS}$$

2. For periods greater than T_0 and less than or equal to T_S :

$$S_a = S_{MS}$$

3. For periods greater than T_S and less than or equal to T_L :

$$S_a = \frac{S_{M1}}{T}$$

4. For periods greater than T_L :

$$S_a = \frac{S_{M1} T_L}{T^2}$$

Where:

$$T_0 = 0.2 \frac{S_{M1}}{S_{DS}} = 0.1210$$

$$T_S = \frac{S_{M1}}{S_{DS}} = 0.6049$$

Note that the MSE_R parameters (S_{MS} , S_{M1}) are used in the relationships above instead of the Design Earthquake (DE) parameters (S_{DS} , S_{D1}). The resulting response spectrum for 2% in 50 years is shown in Figure III.2.

Figure III.2 Target Acceleration Response Spectrum, 2% in 50 years

8.2. Site Response Spectra

8.2.1. Ground Motion Conditioning

The Ground motions representing 2% in 50 years for M7.3 Landers Earthquake of 28 June 1992 at the Joshua Tree Fire Station site can be seen below in Figure III.3 for the x direction and Figure III.4 for the y direction.

Figure III.3 Ground Acceleration for Joshua Tree in X-direction

Figure III.4 Ground Acceleration for Joshua Tree in Y-direction

WAVEGEN is a tool that modifies the input/raw ground motion by adding wavelets into raw ground motion. It requires acceleration data and target spectrum as an input. WAVEGEN adds sine/cosine (harmonic) wavelets to raw acceleration data in order to match with target spectrum and gives “compatible.out” file as an output. This “compatible.out” file is to be used as conditioned ground motion for obtaining response spectra oriented between 0 to 180 degrees. Besides, the above two files *WAVEGEN* requires damping ratio as an input.

Once the Ground motions were conditioned to the target acceleration, the maximum-direction (RotD100) could be found. The two orthogonal x and y acceleration time series were combined together using Equation 2 to find 90 separate acceleration time series angles of $0^\circ \leq \theta < 180^\circ$ with $\Delta\theta = 2^\circ$.

$$a_{ROT}(t; \theta) = ax(t)\cos(\theta) + ay(t)\sin(\theta) \quad (2)$$

The pseudo acceleration spectra was then found for each of the angles using Newmark's Integration Method with $\gamma = 0.5$ and $\beta = 0.25$ (Constant Average Acceleration Method) to find and plot the maximum of pseudo acceleration time histories for each period. It can be seen in Figure III.5, plotted with the ASCE target spectra.

Figure III.5 Cumulative Response from 0 to 180 degrees

The envelope of all 90 directions is RotD100 and it is shown plotted in Figure III.6 with the target spectrum and 90% of the target spectrum.

Figure III.6 Envelope of Unscaled Cumulative Response, Unscaled RotD100

8.2.2.Amplitude Scaling

Once RotD100 was obtained for the Joshua Tree data, it needed to have its amplitude scaled down according to ASCE 7-16 16.2.3.2 so that it was either equal to or above the 90% target response spectra. The period range for scaling is defined in ASCE 7-16 16.2.3.1 as:

$$\min (T_{90}, 0.2 * T_2) \geq T \leq 2 * T_1$$

Where T_1 is the largest first translational mode period, T_2 is the smallest first translational mode period, and T_{90} is the boundary period including 90% of the mass participation. This range was thus calculated as:

$$0.125 \text{ s} \geq T \leq 2.8 \text{ s}$$

Within this period scaling range, a scaling factor of 0.9585 was required to scale RotD100 down to rest on the 90% target spectra as seen in Figure III.7.

Figure III.7 Envelope of Scaled Cumulative Response, Scaled RotD100

8.2.3. Pseudo Acceleration and Displacement Response Spectra

The scaling factor of 0.9585 was then applied to the Joshua Tree x and y conditional ground motion accelerations. The pseudo acceleration response spectra and displacement response spectra for these scaled motions were plotted with the target spectra as seen in Figures III.8 and III.9. Comparing the scaled acceleration and displacement responses with the target, it was observed that they are very similar and essentially right on top of one other.

Figure III.8 Target and Scaled Conditioned Acceleration Response Spectra

Figure III.9 Target and Scaled Conditioned Displacement Response Spectra

PART IV: Non-Linear Model

9. Variant 1 of Non-Linear Model

In this part of the project, the importance of uncertainty that arises from the use of a damping model in non-linear time-history analysis is to be understood. The non-linear time history analysis is carried out in ETABS. The model provided in ETABS has core walls modeled as frame elements with distributed plasticity (Displacement-based formulations) and sections discretized with concrete and steel fibers. The coupling beams are modeled using diagonal nonlinear truss elements. This idealization of model is more representative of the actual behavior of a coupling beam than models based on springs commonly used in practice.

9.1. Complete Core Wall Design for Combined Axial-Flexure

In this section, the design is done partially separate for Level 1 to 5 and for Level 6 to 14. For level 1 to 5, the design is kept exactly similar to the design of core wall in Part I except for the design of coupling beams. For level 6 to 14, identical longitudinal and transverse reinforcement are redesigned for maximum load values. The design of coupling beams is further divided into three sections namely, level 1 to 4, level 5 to 8 and level 9 to 14. Coupling beams for level 1 to 4 are designed with maximum load coming on that levels and likewise it is done for level 5 to 8 and level 9 to 14.

After identifying critical load combinations for Level 6 to 14 from ETABS model, values are plotted on P-M interaction diagrams for E-W and N-S. From the diagram, the values of minimum reinforcement ratio is coming out to be 0.80%. Critical Load combinations acting on the walls are tabulated below in Table IV.1 and Table IV.2 for Core Wall AB and CD.

Core Wall AB				
	Load Combo	Pu (kips)	Mu_NS (k-ft)	Mu_EW (k-ft)
1	1.5D+L+E_EW	709.1	13816.2	0
2	0.64D+E_EW	3755.5	13727.3	0
3	1.5D+L+E_NS	-5407.7	0	73458.5
4	0.64D+E_NS	-2227.7	0	73458.5

Table IV.1 Critical Load Combination for Core Wall AB

Core Wall CD				
	LC	Pu (kips)	Mu_NS (k-ft)	Mu_EW (k-ft)
1	1.5D+L+E_EW	-11287.6	-13816.2	0
2	0.64D+E_EW	-8107.7	-13727.3	0
3	1.5D+L+E_NS	-5407.7	0	-73458.5
4	0.64D+E_NS	-2227.7	0	-73458.5

Table IV.2 Critical Load Combination for Core Wall CD

The values of Axial Load and Moment combinations are plotted on P-M interaction diagrams of EW and NS direction respectively. P-M Interaction diagrams for E-W and N-S direction are shown below in Figure IV.1 and Figure IV.2.

Figure IV.1 P-M Interaction Diagram in E-W Direction

Figure IV.2 P-M Interaction diagram for N-S direction

Design of Longitudinal and Transverse Reinforcement

Design of Longitudinal reinforcement is carried out using obtained reinforcement ratio of 0.8 % from P-M Interaction diagram and gross area of the section of wall. From the calculation, #8 bars at 7 in. OC are to be provided in 2 curtains. Similarly, transverse reinforcement is designed for maximum shear demand in E-W and N-S direction respectively. From the calculation, it is found that minimum reinforcement of 0.25 % is sufficient. Transverse reinforcement of #4 bars at 6 in. OC in 4 curtains are provided in E-W direction and #4 bars at 6 in. OC in 2 curtains are provided in N-S direction.

Detailed calculation is shown in **Appendix B**.

Design of Coupling Beams

Design of coupling beams are done for 3 different categories based on groups of coupling beams, i.e, Level 1 to 4, Level 5 to 8 and Level 9 to 14. It was assigned to find the maximum coupling beam shear on each of the floors and design the coupling beams

accordingly. The maximum shears for each of the floor categories can be seen in Table IV.3.

TABLE: Spandrel Forces						
Story	Spandrel	Output Case	Case Type	Step Type	Location	V2
Story3	S1	1.2D+0.2SDS+0.5L+1.0EQX	Combination	Min	Left	-630.41
Story5	S1	1.2D+0.2SDS+0.5L+1.0EQX	Combination	Min	Left	-614.742
Story9	S1	1.2D+0.2SDS+0.5L+1.0EQX	Combination	Min	Left	-416.178

Table IV.3 Coupling Beam Shear Forces

Design Summary

Design detail summary of shear wall and coupling beams based on calculations similar to that done in Part II shown in Table IV.4.

Design Detail Summary of Core Wall		
Shear Wall	Longitudinal Reinforcement	#11 bars @ 7 in. (Level 1 to 5) #8 bars @ 7 in. (Level 6 to 14)
	Transverse Reinforcement	#4 bars @ 5 in. (Level 1 to 5) #4 bars @ 6 in. (Level 6 to 14)
Coupling Beams	Longitudinal Reinforcement	12 #11 bars in 3 layers (Level 1 to 4) 12 #11 bars in 3 layers (Level 5 to 8) 8 #11 bars in 2 layers (Level 9 to 14)
	Transverse Reinforcement	5 legs vs 6 legs with 2 bundled #4 bars each leg (Level 1 to 4) 4 legs vs 6 legs with 2 bundled #4 bars each leg (Level 5 to 8) 4 legs vs 6 legs with 2 bundled #4 bars each leg (Level 9 to 14)

Table IV.4 Design Summary

Detailed Drawings

Detailed Drawings are attached in **Appendix A**.

9.2. Modal Analysis

Modal Analysis is the study of dynamic properties of the structures under consideration in frequency domain. Modal analysis is also of great importance to study about resonating behavior of structure. It identifies the periods at which the structure/system may resonate at its natural

frequency. It also talks about the various modes in which structure/system may vibrate under the given earthquake conditions which eventually helps in identifying possible zones of damage.

Modal Response in X - Direction

Figure IV.3 Modal Response in X - Direction

The modal properties of the structure in X-direction for Mode 1, Mode 2 and Mode 3 are tabulated below in Table IV.5, Table IV.6 and Table IV.7 respectively.

ϕ_i	m_i	$m_i \cdot \phi_i$	$m_i \cdot \phi_i^2$	h_i/H	$m_i \cdot \phi_i \cdot h_i/H$	$\Gamma_i \cdot \phi_i$	Period	T_i/T_1	H_e/H	$M_{e,n}/M_t$	Mode
	kip-s ² /in					Modal contribution vector					
7.89E-03	3.424109	0.0270	0.00021330	1.00	0.02703	1.52	0.952	1.000	0.736	0.643	Mode 1
7.18E-03	3.995999	0.0287	0.00020609	0.93	0.02670	1.38					
6.45E-03	3.995999	0.0258	0.00016611	0.86	0.02217	1.24					
5.70E-03	3.995999	0.0228	0.00012968	0.79	0.01800	1.10					
4.94E-03	3.995999	0.0197	0.00009755	0.72	0.01423	0.95					
4.19E-03	3.995999	0.0168	0.00007027	0.65	0.01091	0.81					
3.47E-03	3.995999	0.0139	0.00004814	0.58	0.00806	0.67					
2.79E-03	3.995999	0.0112	0.00003113	0.51	0.00571	0.54					
2.18E-03	3.995999	0.0087	0.00001891	0.44	0.00384	0.42					
1.65E-03	4.135863	0.0068	0.00001124	0.37	0.00254	0.32					
1.21E-03	4.275727	0.0052	0.00000628	0.30	0.00157	0.23					
8.22E-04	4.275727	0.0035	0.00000289	0.23	0.00082	0.16					
4.85E-04	4.275727	0.0021	0.00000101	0.16	0.00034	0.09					
2.15E-04	4.512978	0.0010	0.00000021	0.09	0.00009	0.04					
1.83E-05	0.949005	0.0000	0.00000000	0.00	0.00000	0.00					

Table IV.5 Modal Properties of Structure in X-Direction for Mode 1

ϕ_i	m_i	$m_i \cdot \phi_i$	$m_i \cdot \phi_i^2$	h_i/H	$m_i \cdot \phi_i \cdot h_i/H$	$\Gamma_i \cdot \phi_i$	Period	T_i/T_1	H_e/H	$M_{e,n}/M_t$	Mode
	kip-s ² /in					Modal contribution vector					
-0.00653638	3.424109	-0.0224	0.0001463	1.00	-0.02	-0.75	0.266	0.279	0.460	0.383	Mode 2
-0.0044928	3.995999	-0.0180	0.0000807	0.93	-0.02	-0.51					
-0.00230143	3.995999	-0.0092	0.0000212	0.86	-0.01	-0.26					
-8.7921E-05	3.995999	-0.0004	0.0000000	0.79	0.00	-0.01					
0.001976109	3.995999	0.0079	0.0000156	0.72	0.01	0.23					
0.003720376	3.995999	0.0149	0.0000553	0.65	0.01	0.42					
0.005000467	3.995999	0.0200	0.0000999	0.58	0.01	0.57					
0.005715673	3.995999	0.0228	0.0001305	0.51	0.01	0.65					
0.005823418	3.995999	0.0233	0.0001355	0.44	0.01	0.66					
0.005367195	4.135863	0.0222	0.0001191	0.37	0.01	0.61					
0.004580074	4.275727	0.0196	0.0000897	0.30	0.01	0.52					
0.003551583	4.275727	0.0152	0.0000539	0.23	0.00	0.41					
0.002386194	4.275727	0.0102	0.0000243	0.16	0.00	0.27					
0.001219945	4.512978	0.0055	0.0000067	0.09	0.00	0.14					
0	0.949005	0.0000	0.0000000	0.00	0.00	0.00					

Table IV.6 Modal Properties of Structure in X-Direction for Mode 2

ϕ_i	m_i	$m_i \cdot \phi_i$	$m_i \cdot \phi_i^2$	h_i/H	$m_i \cdot \phi_i \cdot h_i/H$	$\Gamma_i \cdot \phi_i$	Period	T_i/T_1	H_e/H	$M_{e,n}/M_t$	Mode
	kip-s ² /in					Modal contribution vector					
0.00592612	3.424109	0.02	0.000	1.00	0.02	0.37	0.128	0.134	0.254	0.700	Mode 3
0.002851723	3.995999	0.01	0.000	0.93	0.01	0.18					
-0.00052276	3.995999	0.00	0.000	0.86	0.00	-0.03					
-0.00345001	3.995999	-0.01	0.000	0.79	-0.01	-0.22					
-0.00516316	3.995999	-0.02	0.000	0.72	-0.01	-0.33					
-0.00521476	3.995999	-0.02	0.000	0.65	-0.01	-0.33					
-0.00362943	3.995999	-0.01	0.000	0.58	-0.01	-0.23					
-0.00090834	3.995999	0.00	0.000	0.51	0.00	-0.06					
0.002101153	3.995999	0.01	0.000	0.44	0.00	0.13					
0.004420705	4.135863	0.02	0.000	0.37	0.01	0.28					
0.00542805	4.275727	0.02	0.000	0.30	0.01	0.34					
0.005356233	4.275727	0.02	0.000	0.23	0.01	0.34					
0.004249252	4.275727	0.02	0.000	0.16	0.00	0.27					
0.002440581	4.512978	0.01	0.000	0.09	0.00	0.15					
0	0.949005	0.00	0.000	0.00	0.00	0.00					

Table IV.7 Modal Properties of Structure in X-Direction for Mode 3

Y - Direction

The modal properties of the structure in Y-direction for Mode 1, Mode 2 and Mode 3 are tabulated below in Table IV.8, Table IV.9 and Table IV.10 respectively.

ϕ_i	m_i	$m_i \cdot \phi_i$	$m_i \cdot \phi_i^2$	h_i/H	$m_i \cdot \phi_i \cdot h_i/H$	$\Gamma_i \cdot \phi_i$	Period	T_i/T_1	$H_{e,i}/H$	$M_{e,i}/M_t$	Mode
	kip-s ² /in					Modal contribution vector					
8.21E-03	3.424109	0.0281	0.00023106	1.00	0.02813	1.54	1.078	1.000	0.750	0.608	Mode 1
7.37E-03	3.995999	0.0294	0.00021684	0.93	0.02738	1.38					
6.51E-03	3.995999	0.0260	0.00016942	0.86	0.02239	1.22					
5.66E-03	3.995999	0.0226	0.00012789	0.79	0.01787	1.06					
4.82E-03	3.995999	0.0192	0.00009270	0.72	0.01388	0.90					
4.00E-03	3.995999	0.0160	0.00006406	0.65	0.01042	0.75					
3.24E-03	3.995999	0.0129	0.00004186	0.58	0.00752	0.61					
2.53E-03	3.995999	0.0101	0.00002567	0.51	0.00518	0.48					
1.92E-03	3.995999	0.0077	0.00001471	0.44	0.00339	0.36					
1.41E-03	4.135863	0.0058	0.00000824	0.37	0.00217	0.26					
1.01E-03	4.275727	0.0043	0.00000432	0.30	0.00130	0.19					
6.54E-04	4.275727	0.0028	0.00000183	0.23	0.00065	0.12					
3.67E-04	4.275727	0.0016	0.00000058	0.16	0.00026	0.07					
1.54E-04	4.512978	0.0007	0.00000011	0.09	0.00006	0.03					
0.00E+00	0.949005	0.0000	0.00000000	0.00	0.00000	0.00					

Table IV.8 Modal Properties of Structure in Y-Direction for Mode 1

ϕ_i	m_i	$m_i * \phi_i$	$m_i * \phi_i^2$	h_i/H	$m_i * \phi_i * h_i/H$	$\Gamma_i * \phi_i$	Period	T_i/T_1	H_e/H	$M_{e,n}/M_t$	Mode
	kip-s ² /in					Modal contribution vector					
0.006725274	3.424109	0.0230	0.0001549	1.00	0.02	-0.79	0.259	0.240	0.420	0.355	Mode 2
0.004321755	3.995999	0.0173	0.0000746	0.93	0.02	-0.51					
0.001866449	3.995999	0.0075	0.0000139	0.86	0.01	-0.22					
-0.00049597	3.995999	-0.0020	0.0000010	0.79	0.00	0.06					
-0.0025951	3.995999	-0.0104	0.0000269	0.72	-0.01	0.31					
-0.00427236	3.995999	-0.0171	0.0000729	0.65	-0.01	0.50					
-0.00540343	3.995999	-0.0216	0.0001167	0.58	-0.01	0.64					
-0.00591745	3.995999	-0.0236	0.0001399	0.51	-0.01	0.70					
-0.00580693	3.995999	-0.0232	0.0001347	0.44	-0.01	0.69					
-0.00514437	4.135863	-0.0213	0.0001095	0.37	-0.01	0.61					
-0.00422824	4.275727	-0.0181	0.0000764	0.30	-0.01	0.50					
-0.00317647	4.275727	-0.0136	0.0000431	0.23	0.00	0.38					
-0.002079	4.275727	-0.0089	0.0000185	0.16	0.00	0.25					
-0.00104588	4.512978	-0.0047	0.0000049	0.09	0.00	0.12					
0	0.949005	0.0000	0.0000000	0.00	0.00	0.00					

Table IV.9 Modal Properties of Structure in Y-Direction for Mode 2

ϕ_i	m_i	$m_i * \phi_i$	$m_i * \phi_i^2$	h_i/H	$m_i * \phi_i * h_i/H$	$\Gamma_i * \phi_i$	Period	T_i/T_1	H_e/H	$M_{e,n}/M_t$	Mode
	kip-s ² /in					Modal contribution vector					
0.005954985	3.424109	0.02	0.000	1.00	0.02	0.40	0.116	0.108	0.267	0.761	Mode 3
0.002470995	3.995999	0.01	0.000	0.93	0.01	0.17					
-0.00106136	3.995999	0.00	0.000	0.86	0.00	-0.07					
-0.00386501	3.995999	-0.02	0.000	0.79	-0.01	-0.26					
-0.00525649	3.995999	-0.02	0.000	0.72	-0.02	-0.36					
-0.00492511	3.995999	-0.02	0.000	0.65	-0.01	-0.33					
-0.00303662	3.995999	-0.01	0.000	0.58	-0.01	-0.21					
-0.00019588	3.995999	0.00	0.000	0.51	0.00	-0.01					
0.002719178	3.995999	0.01	0.000	0.44	0.00	0.18					
0.004801919	4.135863	0.02	0.000	0.37	0.01	0.33					
0.005573622	4.275727	0.02	0.000	0.30	0.01	0.38					
0.00536154	4.275727	0.02	0.000	0.23	0.01	0.36					
0.004239383	4.275727	0.02	0.000	0.16	0.00	0.29					
0.002481421	4.512978	0.01	0.000	0.09	0.00	0.17					
0	0.949005	0.00	0.000	0.00	0.00	0.00					

Table IV.10 Modal Properties of Structure in Y-Direction for Mode 3

	Mode	Period (seconds)
X-Direction	1	0.952
	2	0.266
	3	0.128
Y-Direction	1	1.078
	2	0.259
	3	0.116

Table IV.11 Summary of Period for various modes in X and Y Directions

Cumulative Modal Participation Mass Ratio

The plot of cumulative modal mass participation ratio vs. period is plotted for both linear and non-linear model in X and Y directions. Surprisingly, non-linear model seems to have lesser period making it stiffer than linear model. This behavior is due to the fact that for linear model, stiffness modifiers are incorporated for conservative estimates whereas for nonlinear model full stiffness (uncracked section initially) of the structure is taken into account.

Figure IV.5 Cumulative Modal Mass Participation vs Period in X-Direction

Figure IV.6 Cumulative Modal Mass Participation vs Period in Y-Direction

Besides, it is also observed that upto 60 percent mass of the building take part into first translational mode, then upto 90 percent mass of the building take part into second translational mode and approximately 5 percent mass from the remaining 10 percent mass takes part into third translational mode.

9.3. Influence of the Damping Model on the Nonlinear Dynamic Response

In this section, the effect of damping model on nonlinear response of a building is analyzed. Rayleigh (proportional) damping is generally available in all non-linear dynamic analysis programs. This damping model is convenient as it is a linear combination of the, usually banded, stiffness and mass matrices. Damping coefficients for this model are determined by selecting damping ratios for only two modes.

1. Set damping ratio of 2.0% in the first translational mode in the NS direction and in the first translational mode in EW direction. Plot the damping ratio (Y-axis) from 0 to 25% and the frequency range (X-axis) from 0 to 20 Hz.
2. Set damping ratio of 2.0% in the average frequency of the first translational mode in the NS direction and the first translational mode in the EW direction, and 5% at a circular frequency of 125.6 rad/s ($f = 20$ Hz, $T = 0.05$ s). Plot the damping ratio (Y-axis) from 0 to 10% and the frequency range (X-axis) from 0 to 20 Hz.

Relation between Damping ratio and modal frequency:

The damping ratio of a structure/system in n th mode is calculated as below:

$$\zeta_n = \frac{\alpha}{2\omega_n} + \frac{\beta\omega_n}{2} \quad (1)$$

The Rayleigh damping coefficients α and β are determined from the given damping ratio for the i th and j th mode respectively. In the matrix form, it can be written as:

$$\frac{1}{2} \begin{bmatrix} 1/\omega_i & \omega_i \\ 1/\omega_j & \omega_j \end{bmatrix} \begin{bmatrix} \alpha \\ \beta \end{bmatrix} = \begin{bmatrix} \zeta_i \\ \zeta_j \end{bmatrix} \quad (2)$$

Upon solving the above matrix equation, we can get the values of α and β given that both modes have same damping ratio.

Hence, we can get the equations of α and β as follows:

$$\alpha = \zeta \frac{2\omega_i\omega_j}{\omega_i + \omega_j}$$

$$\beta = \zeta \frac{2}{\omega_i + \omega_j}$$

(3)

The equations are solved in MATLAB and values obtained are tabulated below in Table VI.12.

Case	α (sec^{-1})	β (sec)
Case - 1	0.1238	0.0032
Case - 2	0.2184	0.0008

Table IV.12 Values of Coefficients for Case - 1 and Case – 2

Using these two coefficients, values of damping ratio are calculated for a given frequency range (0 to 20 Hz) by using Equation 1. The plot for both cases is shown below in Figure IV.7.

Figure IV.7 Damping Ratio vs Frequency

The frequency for Mode 1 and Mode 2 in X and Y Direction is tabulated below which is evident from Table IV.13.

	X - Direction	Y - Direction
	Frequency (Hz)	Frequency (Hz)
Mode 1	1.0504	0.9276
Mode 2	3.7594	3.8610

Table IV.13 Modal Frequencies in X and Y Direction

The corresponding damping ratio for each mode in each direction is tabulated below in Table IV.14.

	Damping Ratio ($\zeta\%$)	
	1st Translational Mode	2nd Translational Mode
EW (X Direction)		
Case - 1	2.00	4.05
Case - 2	1.91	1.38
NS (Y Direction)		
Case - 1	2.00	4.15
Case - 2	2.10	1.39

Table IV.14 Damping Ratio for Mode 1 and Mode 2 in EW and NS Direction

From Table IV.14 it is evident that for both cases 1st Translational Mode has more or less same damping ratios. However, for 2nd translational mode, Case-1 seems to be overdamped compared to Case-2.

Nonlinear Time History Analysis

In this section, nonlinear time history analysis is performed using MCE conditioned ground motion pairs of JOSHUA earthquake for cases mentioned above. Integration scheme used was constant average acceleration with output time step size of $T = 0.005$ seconds.

Lateral Floor Displacement

Figure IV.8 Relative Lateral Floor Displacement - EW

Figure IV.9 Relative Lateral Floor Displacement - NS

Interstory Drift Ratio

Figure IV.10 Interstory Drift Ratio - EW

Figure IV.11 Interstory Drift Ratio - NS

Total Floor Acceleration

Figure IV.12 Total Floor Acceleration - EW

Figure IV.13 Total Floor Acceleration - NS

Observations on Diagrams

1. Maximum relative displacement is seen in the middle floors 6-8 in EW (X) direction and the upper stories in NS (Y) direction.
2. Similar trend could be seen in the interstory drift ratio. Minimum / maximum values are -1.5 inch / 2.0 inches in EW (X) direction, and -2.5 inch / 2.0 inches in NS (Y) direction.
3. Maximum floor acceleration occurs in the last story in either direction EW (X) and NS (Y).
4. Maximum and minimum accelerations has almost the same absolute values, in direction EW (X) it is 1g for case 1 and 1.7g for case 2, in the direction NS (Y) it is 1.4g for case 1 and 2.2g for case 2. Case 2 shows higher acceleration values in all stories, in both directions, than case 1.

10. Variant 2 of Non-Linear Model

In this part of the project, another ETABS 2018 model is to be used. This model is similar to the model used in Part III except for the coupling beams. Here, the coupling beams are modeled using vertical nonlinear link elements (equivalent shear spring model) connected to rigid elements cantilevering from the walls. The equivalent springs are widely used in practice, however models based on diagonal nonlinear truss elements (equivalent truss model) are more representative of the actual behavior of the coupling beams. Your Professor will discuss both methods in class.

10.1. Influence of the Coupling Beam Model on the Nonlinear Dynamic Response

In this section, the effect of the model for the coupling beams on the response of a core wall will be seen. Using the damping Case 2 described in III.4, the nonlinear ETABS models provided of Part III (equivalent truss model) and Part IV (equivalent shear spring) will be updated with the flexural reinforcement in the walls and the diagonal reinforcement in the coupling beams computed in III.2. The two ETABS models will be subjected to the same MCE_R scaled ground motion pair.

Equivalent Truss Model

- The beam is modeled as two truss elements located along the centroidal axis of the diagonal reinforcement in the beam.
- The cross-section of each truss contains the total steel area in the diagonal as well as an equivalent area of concrete ($A_{c,eq}$), determined by matching the stiffness of the model to the un-cracked section of the beam.
- Additionally, the capacity in tension of the concrete fibers must be adjusted to match the actual shear stress at the onset of cracking (V_{cr}).

According to Timoshenko beam theory, the stiffness of a deep beam can be computed as follows:

The equivalent truss model under the same cracking force:

From equilibrium in the transverse direction of the equivalent truss model, the equivalent concrete area is calculated by:

$$A_{c,eq} = \frac{6 \cdot I_g \cdot D^3}{L^3 \cdot \left(1 + 2.88 \cdot \left(\frac{H}{L}\right)^2\right)} \cdot \frac{1}{(\gamma H)^2}$$

Equivalent Shear Spring Model

In this model the coupling beam is modeled using two rigid elements connected by a spring representing the shear in the transverse direction.

This model is constituted by a single material model, representing the steel reinforcement, and does not capture the initial stiffness and onset of cracking of the beam section.

The properties of the spring element are determined by matching the initial yield of reinforcement in the beam (Δ_y, V_y). While in the truss model the onset of yield state is not computed explicitly, the shear spring formulation requires explicit values for Δ_y and V_y .

To model the shear spring using a fiber section of steel with a unit area, use the following properties for the material model (e.g. Steel02)

$$E_s^{\text{spring}} = \frac{4 \cdot A_s \cdot \tan^2(\alpha)}{D} \cdot E_s$$

$$F_y^{\text{spring}} = 2 \cdot A_s \cdot f_{ye} \cdot \sin(\alpha)$$

Figure IV.14 and Figure IV.15 show the negative and positive envelopes in the EW direction for relative lateral floor displacement and interstory drift ratio of the truss and spring models respectively:

Figure IV.14 Relative Lateral Floor Displacement, E-W Direction

Figure IV.15 Interstory Drift Ratio, E-W Direction

From the above graphs, both the Truss and Spring models appear to have similar relative displacements and drift ratios. It can be seen in Figure IV.14, especially in the negative drift where the core wall reinforcement changes at floor 9 and the building drifts more drastically between floors 9 and 14.

Figure IV.16 shows graph of the the core wall shear force (total shear force).

Figure IV.16 Core Wall Total Shear Force - E-W direction

From Figure IV.16, it is evident that the truss and spring models provide similar total shear force results until you get to the bottom of the building, where the Truss model shows larger shears. This is because the Truss model, unlike the spring model, is able to show the lengthening in the coupling beams. The coupling beams are able to expand at the top of the building, but once you get to the base, the walls are much stiffer closer to the stiff foundation and unable to spread apart, thus causing an increase in the shear force and a redistribution between the two walls that will be seen in later figures.

Figure IV.17 and Figure IV.18 show the hysteretic responses given by truss and spring models for the south side coupling beams in Story 1 and Story 4 respectively:

Figure IV.17 Hysteretic Responses Given by Truss Model and Spring Model in Story 1

Figure IV.18 Hysteretic Responses Given by Truss Model and Spring Model in Story 4

As seen from the above plots, the bauschinger effect is more clear in the truss model which is more precise, however the spring model is more often used in industry for

simplicity. In Story 1, the maximum shear force in the truss model is ± 1500 kips (more precise value), while in the spring model it is ± 900 kips (approximate value). Linear hysteresis response of the spring model suggests that one spring can not predict the actual behavior of the coupling beam. Energy Dissipation is more or less linearly confined for Story 4 compared to Story 1 in which Truss Model simulates non-linear behavior by incorporating yielding and then moving into nonlinear range. At story 1, the truss model provides a better estimation of energy dissipation compared to the spring model.

The following Figure IV.19 and Figure IV.20 show the envelope of the c-shaped wall shear forces in the EW and NS directions for the truss and spring models. Shear forces that were used for the design (linear analysis) are shown with them for comparison:

Figure IV.19 C-shaped Wall Enveloped Shear - EW

Figure IV.20 C-shaped Wall Enveloped Shear - NS

The EW direction is the direction of the coupling beams, thus in Figure IV.19, the Truss model shows an extreme jump in enveloped shear at the base where almost all of the base shear is going into one of the C-shaped walls due to the redistribution in shear at the stiff base of the building as discussed previously. The spring model is unable to represent the lengthening in the beams, or the redistribution of shear to the walls, and thus shows a much lower enveloped shear at the base.

The NS direction is perpendicular to the coupling beams and the walls have no redistribution of the shear force at the base, thus in Figure IV.20, the truss and spring models show very similar results.

In both figures, it is clear that the design shear from linear analysis used for the core wall design is far under designing the walls when you look at the nonlinear results.

The following Figures IV.21 to IV.24 show the shear forces in each of the c-shaped walls in the EW direction for the truss and spring models, at both the positive peak base shear force and the negative base shear force, plotted with both the shear force from the linear model and the design shear forces chosen for the floor sections:

Figure IV.21 Shear Force Distribution at Positive Peak Base Shear - Truss Model

Figure IV.22 - Shear Force Distribution at Negative Peak Base Shear - Truss Model

Figure IV.23 - Shear Force Distribution at Positive Peak Base Shear - Spring Model

Figure IV.24 - Shear Force Distribution at Negative Peak Shear - Spring Model

Figures IV.21 and IV.22 for the truss model shows that the shear force in walls C1 and C2 are the same until the base where there is a huge evident redistribution of shear into the leading wall and far less shear in the trailing wall. This behavior is not seen in the Spring model shown in Figure IV.23 and IV.24 where the shear stays almost the same across the height of the core wall.

The Design linear shear forces are again seen to be unconservative for the bottom 4 floors and closer yet conservative for the top floors. There is an evident discrepancy between the linear and nonlinear results.

Figures IV.23 and IV.24 for the spring model doesn't show the difference seen in the truss model, this model can't capture the redistribution of shear forces the truss model can show.

Similar to the truss model, the Design linear shear forces are seen to be unconservative for the bottom 4 floors and closer yet conservative for the top floors. There is an evident discrepancy between the linear and nonlinear results.

10.2. Estimated Roof Displacement

To gain confidence in the ETABS model results, the roof displacement from the nonlinear model in Part III will be compared to displacements corresponding to MCE_R level calculated from the following Equation:

$$\Delta_N = \Gamma^*_1 S_d(T_p, \zeta = 5\%) 1/\beta C_R$$

Δ_N = Estimated roof displacement in the direction considered (NS or EW).

Γ^*_1 = Modal contribution factor for mode 1 in the direction considered (NS or EW).

S_d = spectral displacement from the response spectrum ($\zeta=5\%$) at period T_p

T_p = predominant period which is assumed to range from $1.5T_1$ to $1.7T_1$

T_1 = period of the first mode in the direction considered (NS or EW).

$\beta = 0.8$ from Table 17.5-1 of ASCE 7-16 considering $\zeta=2\%$

C_R = displacement coefficient S_{di}/S_d , assumed as 1 (equal displacement)

	T1	Tp_1.5	Tp_1.7	$\Delta_N_{1.5}$ (in)	$\Delta_N_{1.7}$ (in)	Δ - ETABS (in)
EW	0.95	1.43	1.62	22.81	31.25	23
NS	1.08	1.62	1.83	31.33	35.57	26.9

Table IV.15 Estimated vs Model Roof Displacement

The predominant period range and results can be seen in Table IV.15 above. The EW ETABS roof displacement of 23 in. is in the estimated displacement range of 22.81 in. - 31.25 in, while the the NS ETABS roof displacement of 26.9 in. is just outside the estimated displacement range of 31.33 in. to 35.57 in. The NS displacement is close enough to the range to put it in the probability distribution of the estimation and the results are still able to provide a level of confidence in the results.

PART V: Design Verification

11. General

Part V of the project is intended to verify the Fully Operational and Life Safety performance objectives of the core wall designed previously. Non-linear time-history analysis was used to determine usage ratios. Usage ratios are defined as the demand obtained from the nonlinear analysis over the damage limit state for a specific parameter. The damage limit states are listed in V.2. At this stage it was assumed that the shear reinforcement was sufficient.

11.1. Performance Objectives

1. **Fully operational** was checked against ground motions representing the seismic hazard with 69% probability of exceedance in 50 years of exposure (i.e. a return period $T_p = -E / \ln (1 - PE) = 43$ years, where the exposure time is $E = 50$ years and the probability of exceedance is $PE = 69\%$):
 - Gypsum board partition walls: Interstory drift ratio shall not exceed 0.6%.
 - Strain in any longitudinal core wall reinforcing bar and in the diagonal reinforcement in the coupling beams shall not exceed 1% (i.e. damage limit state sii, residual cracks likely to remain large).
 - Compressive strain at the extreme corner fibers at each wall at base shall not be less than -0.4% (i.e. damage limit state cii, onset of concrete cover spalling).
2. **Life Safety** was checked against ground motions representing the seismic hazard with 2% probability of exceedance in 50 years of exposure (i.e. a return period $T_p = -E / \ln (1 - PE) = 2475$ years, where the exposure time is $E = 50$ years and the probability of exceedance is $PE = 2\%$):
 - Strain amplitude (maximum tensile strain minus the minimum compressive strain) in any longitudinal core wall reinforcing bar and in the diagonal reinforcement in the coupling beams shall not exceed $(14 - 1.33S_h/d_b)/100$ and $\epsilon_{cc} \leq -0.004$, this is the reinforcement damage limit-state siv, longitudinal bar fracture.
 - Compressive strains at the core wall base shall not be less than -0.4%. Shall not exceed the crushing strain.
 - Maximum tensile strain in the core wall longitudinal reinforcement away from the plastic hinge region at the wall base shall not exceed 1% (i.e.

damage limit state sii). To simplify the work, only the extreme-most four lumped reinforcing bars in each wall at Level 6 will be checked.

11.2. Model For Time-History Analyses

The provided ETABS fiber model (Nonlinear model of Part III, equivalent truss model in the coupling beams) was modified by updating the core wall longitudinal reinforcement and the diagonal reinforcement in the coupling beams according to Part III.2. Expected material properties for the steel reinforcement and for the concrete were used. Damping Case 2 described in section III.4 was used as well.

In order for the shell elements of the walls to be better capable of capturing the strain values in the first floor, the wall shell elements in this floor have been divided horizontally into two, i.e. a finer mesh has been used.

11.3. Performance Level Verification

Nonlinear time-history analysis was performed for the building to verify the two performance levels adopted in design.

11.4. Fully Operational Performance Level Verification

Nonlinear analysis has been performed for the building with three ground motion pairs scaled to match the response spectra for the seismic hazard with 69% probability of exceedance in 50 years of exposure (2x Covina, 2x Pasadena, 2x Pomana earthquake motions).

Plots of strain time history for longitudinal bars at critical locations (corners of the wall) in three ground motions are shown in Figure V.1, Figure V.2 and Figure V.3 respectively for full operational case ground motions.

Figure V.1 Strain Time History for Corner Longitudinal Bars - Covina

Figure V.2 Strain Time History for Corner Longitudinal Bars - Pasadena

Figure V.4 Location of Critical Points on Core Wall

Critical points of the core wall are labeled from 1 to 8 and can be seen on Figure V.4

Table V.1 shows the strain values and usage ratios for S_{ii} (i.e. the maximum tensile strain for each lumped reinforcement and for each ground motion over a limit of 0.01). Table V.2 shows the usage ratios for the unconfined concrete (limit state C_{ii} , onset of concrete cover spalling) at the eight critical points at the core wall base.

Point	Strain Values			Usage Ratio, Sii		
	Covina (10 ⁻⁴)	Pasadena	Pomona	Covina	Pasadena	Pomona
1	5.28	0.0010	0.0010	0.0528	0.1015	0.1035
2	6.77	0.00098	0.0011	0.0677	0.0980	0.1127
3	7.21	0.0010	0.00096	0.0721	0.1032	0.0962
4	5.26	0.0094	0.00084	0.0526	0.0940	0.0841
5	5.24	0.00101	0.00088	0.0524	0.1016	0.0886
6	5.56	0.00104	0.00098	0.0556	0.1039	0.0987
7	7.92	0.00108	0.00106	0.0792	0.1086	0.1060
8	5.77	0.00104	0.00089	0.0577	0.1045	0.089
			Max	0.792	0.1086	0.1127

Table V.1 Usage Ratios for Longitudinal Bars at Critical Locations, Sii

Poin t	Strain Values			Usage Ratio, Cii		
	Covina(10 ⁻⁴)	Pasadena(10 ⁻⁴)	Pomona(10 ⁻⁴)	Covina	Pasadena	Pomona
1	-3.42	-5.25	-3.71	0.0855	0.1312	0.0928
2	-3.27	-3.10	-3.56	0.0817	0.0775	0.0890
3	-3.92	-5.59	-5.08	0.0980	0.1398	0.1270
4	-2.44	-3.38	-3.14	0.0610	0.0845	0.0785
5	-2.59	-3.34	-3.17	0.0648	0.0860	0.0793
6	-4.58	-5.72	-4.73	0.1145	0.1430	0.1183
7	-2.6	-3.09	-3.52	0.065	0.0773	0.0880
8	-3.41	-5.31	-4.16	0.0852	0.1328	0.1040
			Max	0.1145	0.1430	0.1270

Table V.2 Usage Ratio for Unconfined Concrete at Critical Locations, Cii

As shown in Table V.1 and Table V.2, Usage ratio for longitudinal reinforcement and unconfined concrete at critical locations (corners of core wall) were calculated respectively. Upon comparison with limit values of strain tension (0.01) and in compression (-0.004), it was found that extreme corners of the wall remain safe under ground motions provided for full occupancy condition. For longitudinal reinforcement, considering maximum tensile strain, maximum usage ratio came out to be 0.1127 at point 2, whereas in the case of unconfined concrete, the maximum usage ratio came out to be 0.1430 at point 6.

Figures V.5 and V.6 show the NS and EW interstory drift ratio envelopes for the three input ground motion pairs, with the interstory drift limit chosen for performance objective (0.6%).

Figure V.5 EW Interstory Drift Ratio Envelopes with Interstory Drift Limit

Figures V.5 and V.6 show that the maximum drift ratio in both walls in both directions due to the three earthquake motions is less than the limit drift ratio of 0.6%, and thus passes the limit state.

Figure V.6 NS Interstory Drift Ratio Envelopes with Interstory Drift Limit

Figures V.7 and Figure V.8 show the usage ratios for each level (the envelope of drift ratio at each level divided by the damage limit drift ratio stated in V.1.1). They are plotted with the usage ratio limit of one.

Figure V.7 EW Usage Ratios of Interstory Drift Ratio Envelopes

Figure V.8 NS Usage Ratios of Interstory Drift Ratio Envelopes

In Figure V.7 and Figure V.8 the maximum usage ratio in EW directions is 0.5 and 0.65 in NS direction, both less than 1. Thus, the usage ratio of interstory drift ratio meets the criteria.

Table V.3 below shows the usage ratios for the damage limit state s_{ii} (residual cracks likely to remain large) in coupling beams. In this table the tensile strains in the outermost longitudinal bar are compared with the limit, $\epsilon_s = 1\%$.

Story	Strain (Max. Tensile)			Usage Ratio		
	Covina	Pasadena	Pomona	Covina	Pasadena	Pomona
14	0.00018	0.00014	0.00006	0.0182	0.0136	0.0063
13	0.00050	0.00045	0.00026	0.0496	0.0449	0.0263
12	0.00071	0.00073	0.00051	0.0710	0.0731	0.0513
11	0.00082	0.00094	0.00067	0.0818	0.0935	0.0667
10	0.00089	0.00107	0.00077	0.0888	0.1072	0.0770
9	0.00092	0.00117	0.00084	0.0921	0.1168	0.0835

8	0.00089	0.00116	0.00089	0.0891	0.1157	0.0891
7	0.00087	0.00118	0.00092	0.0874	0.1177	0.0920
6	0.00084	0.00119	0.00099	0.0842	0.1193	0.0988
5	0.00093	0.00119	0.00104	0.0931	0.1188	0.1039
4	0.00100	0.00119	0.00107	0.0999	0.1193	0.1066
3	0.00100	0.00113	0.00105	0.0997	0.1129	0.1047
2	0.00092	0.00101	0.00099	0.0924	0.1006	0.0994
1	0.00081	0.00085	0.00080	0.0812	0.0847	0.0802
			Max	0.0999	0.1193	0.1066

Table V.3 Usage Ratio for Damage Limit State in Coupling Beams, Sii

From Table V.3, the maximum usage ratio is 0.1193 (11.9%) at story 6. Thus, the structure passes the Sii limit state criteria.

11.5. Life Safety Performance Level Verification

Nonlinear analysis was performed on the building model with the same three ground motion pairs scaled to match the response spectra for the seismic hazard with 2% probability of exceedance in 50 years of exposure.

Strain time histories were taken from the ETABS model from three points on each wall (P, Q, and R), whose location can be seen in Figure V.9. With the strain as the Z value and the X and Y values being taken from the defined origin (Figure V.9), vectors were formed from these points and used to form planes for each wall separately at each time step. These planes were used to calculate the maximum tensile strain and minimum compressive strain contours along the core walls as seen in Figures V.10 and Figure V.11. The strain amplitude was also calculated as maximum tensile strain minus the minimum compressive strain and its strain contour is seen in Figure V.12.

Figure V.9 Vectors with Center Points on the Core Wall

The strain amplitude was compared to the Siii limit state for the onset of long bar buckling:

$$\varepsilon_s - \varepsilon_{cc} \geq \frac{10 - \frac{sh}{db}}{100} = 0.064$$

and was also compared to the Ciii limit state for deep concrete cover spalling:

$$\varepsilon_{cc} \geq -0.004$$

While the entire core wall met the criteria for limit state Siii, there was an edge of the wall that did not meet the Ciii limit state and would thus require boundary elements, as seen in Figure 13. However, the strains at the extreme corners of the 6th floor showed that the steel was yielding and forming a plastic hinge (discussed in more detail below), and it was decided that the best course of action would be to extend the 1.4% longitudinal reinforcement up to the 8th floor to mitigate the mid building plastic hinging. The model was changed and rerun with the new reinforcement, new strains were taken, and the new contour plots can be seen on the bottom of Figures 10, 11, and 12. This change caused more of the edges of the core walls to require reinforcement as seen in the revised Figure 13. The zone that did not pass the Ciii limit state extended a maximum of 20 inches into the core wall. Since core wall reinforcement was previously detailed to be 24 inches at every corner based on the linear analysis, it did not need to be revised after the non-linear analysis.

Figure V.10 Maximum Strain Contours for the original model (top) and revised model (bottom)

Figure V.11 Minimum Strain Contours for the original model (top) and revised model (bottom)

Figure V.12 Strain Amplitude Contours for the original model (top) and revised model (bottom)

V

Figure V.13: Locations requiring boundary elements (black) for the -0.004 minimum strain limit for the original model (top) and revised model (bottom)

Strains were extracted from the extreme fibers of the contours and compared to the limit state Siv, Bar Fracture, and Civ, Crushing of the confined concrete core. The Siv limit state for the onset of long bar buckling is shown below:

$$\varepsilon_s - \varepsilon_{cc} \geq \frac{14 - \frac{4Sh}{3db}}{100} = 0.092$$

The Ciii limit state for deep concrete cover spalling was based on the rectangular cross section equation and was calculated from the boundary element reinforcement:

$$\varepsilon_{cc} \geq -(0.004 + 2\sqrt{\rho_{sx}\rho_{sy}}) = -0.026$$

The usage ratios are seen tabulated in Tables V.4 and Table V.5.

	Wall C1		Wall C2	
Point	Strain	Usage Ratio	Strain	Usage Ratio
Top left	0.018	0.220	0.0221	0.270
Top Right	0.0205	0.251	0.0152	0.186
Bottom Left	0.0149	0.182	0.0221	0.270
Bottom Right	0.0204	0.249	0.0127	0.155

Table V.4 Usage Ratio for Reinforcement at Wall corners on Level 1, Siv

	Wall C1		Wall C2	
Point	Strain	Usage Ratio	Strain	Usage Ratio
Top left	-0.0025	0.098	-0.0032	0.125
Top Right	-0.0053	0.208	-0.0014	0.055
Bottom Left	-0.0018	0.071	-0.0036	0.141
Bottom Right	-0.004	0.157	0.0025	0.098

Table V.5 Usage Ratio for Reinforcement at Wall corners on Level 1, Civ

From Table V.4 and V.5 it can be seen that the core walls met the Civ and Siv criteria and there was no bar fracture or crushing of the confined concrete core.

Usage Ratio for all eight-lumped reinforcement at the wall corners at Level 6 were also obtained for two reinforcement scenarios and comparative study is presented here.

In the first case, the model was provided with 1.4% longitudinal reinforcement in stories 1 to 5 and 0.8% longitudinal reinforcement between story 6 to 14. Usage ratios for this case are calculated at level 6. In the second case, the model was provided with 1.4% longitudinal reinforcement farther up, between story 1 to 8 and usage ratios for level 6 were again calculated. This comparison gives rise to an idea that differentiates performance based design to conventional seismic design. Usage Ratios for both cases are shown below in Table V.6.

Point	Usage Ratio, 0.8% Long. R/F			Usage Ratio, 1.4% Long. R/F		
	Castaico	Joshua	Yermo	Castaico	Joshua	Yermo
1	1.3864	0.9109	1.0524	0.5303	0.2377	0.2700
2	1.2607	0.8905	0.9573	0.5194	0.2090	0.3098
3	0.5347	1.4576	0.6989	0.2051	0.2615	0.2004
4	0.4732	1.2828	0.7590	0.2338	0.2807	0.1691
5	0.7364	1.0174	0.9858	0.2295	0.2161	0.3018
6	0.6073	1.1792	1.0879	0.2020	0.2285	0.3577
7	0.6913	0.9360	1.0064	0.2107	0.3343	0.1986
8	0.7577	1.0554	1.2165	0.2561	0.3342	0.2295
Max	1.3864	1.4576	1.2165	0.5303	0.3343	0.3577

Table V.6 Usage Ratio for Reinforcement at Wall corners on Level 6, Sii

From Table V.4, it is evident that when incorporating performance based design, the usage ratio significantly reduces due to provision of higher percentage of longitudinal reinforcement. When the model is provided with 1.4% longitudinal reinforcement only up to level 5, plastic hinge forms at level 6, which is evident from tensile strain based usage ratios greater than 1. These drop by approximately 60% for Castaico, 80% for Joshua and 70% for Yermo. From this observation one can conclude that the longitudinal

reinforcement curtailment should be done in a manner that it allows plastic hinge to form at base and not at upper levels.

Table V.7 shows usage ratio for damage limit state in coupling beams for the diagonal reinforcement. It compares the strain value for each ground motion with critical value ϵ_{iv} . The limit for coupling beam was calculated as 0.082.

Story	Strain (Max. Tensile)			Usage Ratio		
	Castaico	Joshua	Yermo	Castaico	Joshua	Yermo
14	0.00123	0.00197	0.00134	0.01503	0.02401	0.01640
13	0.00198	0.00350	0.00206	0.02415	0.04272	0.02515
12	0.00325	0.00485	0.00293	0.03963	0.05909	0.03571
11	0.00450	0.00585	0.00406	0.05489	0.07131	0.04955
10	0.00551	0.00606	0.00478	0.06726	0.07394	0.05836
9	0.00549	0.00569	0.00467	0.06694	0.06940	0.05724
8	0.00635	0.00496	0.00635	0.07744	0.06051	0.07744
7	0.00776	0.00503	0.00429	0.09459	0.06139	0.05240
6	0.00939	0.00546	0.00547	0.11453	0.06656	0.06674
5	0.01097	0.00631	0.00700	0.13380	0.07700	0.08538
4	0.01212	0.00807	0.00888	0.14789	0.09848	0.10835
3	0.01215	0.00945	0.00983	0.14828	0.11520	0.11988
2	0.01109	0.01029	0.00967	0.13535	0.12552	0.11797
1	0.00852	0.00838	0.00654	0.10395	0.10227	0.07971
			Maximum	0.14828	0.12552	0.11988

Table V.7 Usage Ratio for damage limit state in coupling beams, ϵ_{iv}

From Table V.7, the maximum value of usage ratio comes out to be 0.148 from ground motion Castaico which was below 1. Thus the structure's coupling beams passed the ϵ_{iv} limit state for life safety.

PART VI: Capacity Design of Force Controlled Elements and Regions and Design of Acceleration-Sensitive Nonstructural Elements

12. General

The non-linear time-history analysis results for the building obtained in Part V will be used to design the force-controlled elements and regions that will remain elastic. All the design forces will be obtained from the envelope of the building response to the MCE_R level ground motions (i.e. $1.5 \times DE$).

12.1. Design Verification

As part of Design verification following plots are to be made.

- 1) Roof Drift Ratio vs. Overturning Moment Ratio in NS and EW Directions.
- 2) Roof Drift Ratio vs. Normalized Base Shear Ratio in NS and EW Directions.
- 3) Normalized elevation of resultant lateral force vs. absolute base shear ratio for time step where the absolute base shear ratio is larger than 70% of the peak or moment is larger than 85% of the peak.

Here are some of the relations worth noting.

Roof Drift Ratio = Roof Lateral Displacement / Roof Height

Overturning Moment Ratio = Overturning Moment / (Building Weight x Height)

Base Shear Ratio = Base Shear / Building Weight

Normalized elevation of Resultant lateral force = (Moment / Shear) / Building Height

12.1.1. Full Occupancy Case

Plots for Normalized Drift vs Normalized Moment are shown below. Normalized Drift is defined as roof lateral displacement over roof height. Normalized Moment is overturning moment divided by the building weight and by the roof height.

Figure VI.1 Normalized Moment vs Drift plots for Full Occupancy case in EW and NS for all three ground motions

Plots for Normalized Shear vs Normalized Drift

Figure VI.2 Normalized Shear vs Normalized Drift plots for Full Occupancy in EW and NS for all three ground motions

Plots for Normalized Lateral Force Elevation to Absolute Base Shear Ratio in EW and NS Direction are shown in the figures below.

Figure VI.3 Normalized Lateral Force Elevation vs Absolute Base Shear Ratio in EW

Figure VI.4 Normalized Lateral Force Elevation vs Absolute Base Shear Ratio in NS

12.1.2. Life Safety Case

Plots for Normalized Drift vs Normalized Moment

Figure VI.5 Normalized Moment vs Drift plots for Life Safety case in EW and NS

Plots for Normalized Shear vs Normalized Drift

Figure VI. 6 Normalized Shear vs Normalized Drift plots for Life Safety case in EW and NS

12.1.3. Observations on Plots

Looking at the plots of normalized moment with normalized drift for both full occupancy and life safety case, a similar non-linear elastic behavior is observed. From Full occupancy case, crack opening leading to reduction in the stiffness of the structure can be observed clearly from hysteretic behavior. Besides, moment seems more sensitive to higher modes compared to shear response of the structure. In both Full occupancy and Life safety case, normalized shear can be best described with linear response compared to moment response.

From the plots of normalized lateral force elevation to absolute base shear ratio predictive response is obtained. However, for the case when absolute base shear reduces to a very low value, this, in turn, increases the value of lateral force elevation to values above 1. This can be explained with the variation of shear across the building height. Since it doesn't have any physical significance, these values need to be eliminated.

Plots for Normalized Lateral Force Elevation to Absolute Base Shear Ratio in EW and NS Direction.

Figure VI.7 Normalized Lateral Force Elevation vs Absolute Base Shear Ratio in EW

Figure VI.8 Normalized Lateral Force Elevation vs Absolute Base Shear Ratio in NS

12.2. Acceleration response spectra at roof level

For each performance level one ground motion has been selected, Covina for Full Occupancy and Joshua for Life Safety, then the Linear acceleration response spectra for the response at the ground and roof levels (using the latest nonlinear model) in the North-South direction of the building ($PSa_{ground}(T, \xi=0\%)$ and $PSa_{roof}(T, \xi=0\%)$) has been computed. Plots of response spectra as well as the ratio $PSa_{roof} / PSa_{ground}$, versus the corresponding period are shown in the figures below.

To determine pseudo-acceleration response spectra at roof and ground level, joint absolute acceleration data and ground motion data in N-S direction were provided as an input to Newmark Integrator that calculates PSa. For full occupancy case covina ground motion and for life safety case pasadena ground motion were used.

Full Occupancy	Life Safety
Covina	Pasadena

Figure VI.9 PSA for Covina Ground Motion Roof Level (left); PSA Ground Level (right)

Figure VI.10 PSA for Joshua Ground Motion Roof Level (left); PSA Ground Level (right)

12.2.1. Observations on Plots

Comparative plot of Pseudo-acceleration ratio at roof to ground level for full occupancy and life safety case is presented below. Comparing the values of PSA Ratio for Mode-1, Mode-2 and Mode-3, it seems that Mode-1 is the most influential to acceleration response followed by Mode-2 and Mode-3 respectively.

Peak period has shifted from 1.38 second in the Full Occupancy case to 2.2 seconds in the Life Safety case, due to yielding and change in the stiffness in the building.

Figure VI.11 Comparison of PSA Ratio between Full Occupancy and Life Safety Case

From Figure VI.11, it is evident that for life safety type ground motion (more intensive) shift in time period for all three modes has occurred. Hence, when the structure enters non-linear range, it is governed by predominant or effective period which depends on the intensity of the ground motion.

12.3. Core Wall

The envelope of the maximum and minimum axial forces in the core walls is shown in Figure VI.12 for the Life Safety case (2x YERMO, 2x JOSHUA, 2x CASTAICO).

Figure VI.12 Maximum and Minimum Axial Forces in the Core Walls, Life Safety Case

The envelope of the maximum bending moments in EW and NS directions are shown in Figures VI.13 and VI.14 respectively.

Figure VI.13 Maximum and Minimum Bending Moment in Core Walls EW, Life Safety Case

Figure VI.14 Maximum and Minimum Bending Moment in Core Walls NS, Life Safety Case

The influence of coupling beams could be seen on the bending moment in the EW direction, “Christmas Tree” effect.

The envelope of the shear forces in NS and EW directions, together with shear force obtained using the ACI 318-14 and ASCE 7-16 prescriptive design requirements, are shown in Figures VI.15 and VI.16.

Figure VI.15 Maximum and Minimum Shear in Core Walls EW, Life Safety Case with ASCE 7-16 Prescriptive Design

Figure VI.16 Maximum and Minimum Shear in Core Walls NS, Life Safety Case with ASCE 7-16 Prescriptive Design

Big deviation could be noticed between the shear force obtained using ACI 318-14 and ASCE 7-16 prescriptive design and the values obtained using performance based design (more than 300%). Buildings designed using prescriptive design procedures will meet the standard but will not prevent a shear failure in the case of MCE.

The core wall has been redesigned for shear for floors 1-5, and for floors 6-14 as shown in the Table VI.1 and Table VI.2 respectively.

FLOORS 1-5					
Shear NS			Shear EW		
hw	172	ft	hw	172	ft
lw	26.67	ft	lw	11.83	ft
t	2.67	ft	t	5.33	ft
hw/lw	6.45		hw/lw	14.54	
Acv	71.11	ft ²	Acv	63.11	ft ²
α	2	*ACI 318-14 ch18.10.4.1 hw/lw>2	α	2	*ACI 318-14 ch18.10.4.1 hw/lw>2
λ	1		λ	1	
f'c	10000	psi	f'c	6000	psi
f _y	60	ksi	f _y	60	ksi
ϕ	1		ϕ	1	
V _{n,concrete}	2,048.00	kips	V _{n,concrete}	1,407.90	kips
V _u	9486.993	kips	V _u	5728.658	kips
V _{n,steel,req}	7,438.99	kips	V _{s,req}	4,320.76	kips
pt,req	1.211%	Ast/Avc	pt,req	0.792%	Ast/Avc
pt,min	0.250%	Ast/Avc	pt,min	0.250%	Ast/Avc
pt	1.211%	Ast/Avc	pt	0.792%	Ast/Avc
Ab, #5	0.31	in ²	Ab, #5	0.31	in ²
no of curtains	4		no of curtains	4	
s,req	3.20	in	s,req	2.45	in
s	3.00	in	s	2	in
pt,provided	1.292%		pt,provided	0.969%	
V _n	9,984.01	kips	V _n	6,690.29	kips
ϕV_n	9,984.01	Kips	ϕV_n	6,690.29	Kips
DCR	0.95	OK	DCR	0.86	OK
10AcwSqrt(f'c)	10,240.01	kips	10AcwSqrt(f'c)	7,039.51	Kips
V _n too large?	Pass		V _n too large?	Pass	

Table VI.1 Shear Reinforcement Redesign (Floor 1-5)

FLOORS 6-14

Shear NS			Shear EW		
hw	172	ft	hw	172	ft
lw	26.67	ft	lw	11.83	ft
t	2.00	ft	t	4.00	ft
hw/lw	6.45		hw/lw	14.54	
Acv	53.33	ft^2	Acv	47.33	ft^2
α	2	*ACI 318-14 ch18.10.4.1 hw/lw>2	α	2	*ACI 318-14 ch18.10.4.1 hw/lw>2
λ	1		λ	1	
f'c	6000	psi	f'c	6000	psi
fy	60	ksi	fy	60	ksi
ϕ	1		ϕ	1	
Vn,concrete	1,189.78	kips	Vn,concrete	1,055.93	kips
Vu	4309.981	kips	Vu	2996.704	kips
Vn,steel,req	3,120.20	kips	Vs,req	1,940.78	kips
pt,req	0.677%	Ast/Avc	pt,req	0.475%	Ast/Avc
pt,min	0.250%	Ast/Avc	pt,min	0.250%	Ast/Avc
pt	0.677%	Ast/Avc	pt	0.475%	Ast/Avc
Ab, #5	0.31	in^2	Ab, #5	0.31	in^2
no of curtains	2		no of curtains	4	
s,req	3.82	in	s,req	5.44	in
s	3.00	in	s	5	in
pt,provided	0.861%		pt,provided	0.517%	
Vn	5,157.79	kips	Vn	3,168.88	kips
ϕVn	5,157.79	Kips	ϕVn	3,168.88	Kips
DCR	0.84	OK	DCR	0.95	OK
10AcwSqrt(f'c)	5,948.91	kips	10AcwSqrt(f'c)	5,279.64	Kips
Vn too large?	Pass		Vn too large?	Pass	

Table VI.2 Shear Reinforcement Redesign (Floors 6 - 14)

12.4. Design Detail Comparison

Core Wall Design		Linear Model	Non-linear Model
Core Wall Design	Longitudinal	#11 bars at 7 in. OC (Floors 1 - 5) #8 bars at 7 in. OC (Floors 6 - 14)	#11 bars at 7 in. OC (Floors 1 - 7) #8 bars at 7 in. OC (Floors 8 - 14)
	Transverse (E-W)	#4 bars at 5 in. OC (Floors 1 - 5) #4 bars at 6 in. OC (Floors 6 - 14)	#5 bars at 2 in. OC (Floors 1 - 5) #5 bars at 5 in. OC (Floors 6 - 14)
	Transverse (N-S)	Same as E-W	2 bundled #5 bars at 3 in. OC (Floors 1 - 5) #5 bars at 3 in. OC (Floors 6 - 14)
Boundary Elements	Longitudinal	#11 bars at 6 in. OC	#11 bars at 6 in. OC
	Transverse	24 in. long Longitudinal: #11 bars at 6 in. OC Transverse: #4 bars at 3 in. OC	24 in. long Longitudinal: #11 bars at 6 in. OC Transverse: #5 bars at 2 in. OC
f'_c	Floors 1-2	6000 psi	10000 psi

Table VI.3 Design Detail Comparison of Linear and Nonlinear Model

12.5. Detailed Drawing

Detailed Drawing is attached in Appendix A.

12.6. Diaphragm

Using the latest nonlinear model (Model V with 3 pairs of ground motions for Life Safety and 3 pairs of ground motions for Full Occupancy), the maximum of the peak NS and the maximum of the peak EW roof total accelerations (in g) for the three MCE_R ground motions have been obtained. Stringer-panel analysis for the entire roof diaphragm in the NS and EW directions was performed. The stringer and panel demands calculated from the NS and EW total roof accelerations were overlaid and the design envelopes were computed.

The panels and stringers (chords and collectors) connecting the diaphragm and the core wall were designed for the largest NS and EW acceleration.

Table VI.4 shows the maximum and minimum roof acceleration in EW and NS directions (maximum of 4 corner joints of the core walls), for the three pairs of Life Safety ground motions (2x YERMO, 2x JOSHUA, 2x CASTAICO), in in/sec² and in g consequently.

	EW (Ux) [in / sec ²]	NS (Uy) [in / sec ²]
Max	520.4	511.9
Min	-502.9	-508.2

	EW (Ux) [g]	NS (Uy) [g]
Max	1.35	1.33
Min	-1.30	-1.32

Table VI.4 maximum and minimum roof acceleration for Life Safety

Table VI.5 shows the maximum and minimum roof acceleration in EW and NS directions (maximum of 4 corner joints of the core walls), for the three pairs of Full Occupancy ground motions (2x POMONA, 2x COVINA, 2x PASADENA), in in/sec² and in g consequently.

	EW (Ux) [in / sec ^2]	NS (Uy) [in / sec ^2]
Max	120.4	131.4
Min	-502.9	-508.2

	EW (Ux) [g]	NS (Uy) [g]
Max	0.31	0.34
Min	-0.26	-0.33

Table VI. 5 maximum and minimum roof acceleration for Full Occupancy

From both tables above, the maximum roof acceleration in EW direction is 520.4 in/sec² (1.35 g), in NS direction is 511.9 in/sec² (1.33 g).

The floor mass could be calculated by:

- RC plate mass:

$$\text{area} * \text{thickness} * \text{unit weight of RC} = 90 * 90 * 8/12 * 150 / 1000 = 810 \text{ kips}$$
 - Superimposed mass:

$$\text{Area} * \text{sum of superimposed dead load} = 90 * 90 * 40 / 1000 = 324 \text{ kips}$$
-
- Sum **=1134 kips**

ETABS gives 14th floor mass as 3.424 kip-s²/in, by multiplying by $g=386.089 \text{ in/sec}^2$ gives 1322 kips. This is because ETABS considers the weight of walls and columns as well.

The total earthquake force in EW direction is

$$F = ma = 1134 \text{ kips} * 1.35g = 1531 \text{ kips}$$

The total earthquake force in NS direction is

$$F = ma = 1134 \text{ kips} * 1.33g = 1508 \text{ kips}$$

The force is distributed on each panel according to each area as:

$$F_{\text{panel}} = F * \text{Panel Area} / \text{Total Area}$$

Figure VI.17 Panel numbers

The panel areas, forces in EW and NS directions are listed in the Table VI.6 below:

Panel	Area [ft2]	Panel force EW [kips]	Panel force NS [kips]
1	1023	193	190
2	924	174	171
3	1023	193	190
4	744	140	138
5	672	127	125
6	744	140	138
7	1023	193	190
8	924	174	171
9	1023	193	190
Sum	8100	1529	1503

Table VI.6 Associated Panel Forces in each Direction EW and NS

The following figures shows the calculations of the Panel and Stringers in EW (and WE) direction:

Figure VI.18 Panel Forces, EW

Figure VI.19 Forces Applied on Corners of the Panels (Panel forces divided by four), EW

Figure VI.20 Forces Applied on Corners of the Panels, Shear Flow Along the Edges, EW

Figure VI.21 Axial Force Diagram for Stringers 1 & 4, EW

Figure VI.22 Axial Force Diagram for Stringers 2 & 3, EW

Figure VI.23 Axial Force Diagram for Stringers A & D, EW

Figure VI.24 Axial Force Diagram for Stringers C & B, EW

The following figures shows the calculations of the Panel and Stringers in NS direction:

	31'	28'	31'	Units
33'	189.9	171.5	189.9	kips
24'	138.1	124.7	138.1	
33'	189.9	171.5	189.9	

Figure VI.25 Panel Forces, NS

Figure VI.26 Forces Applied on Corners of the Panels (Panel forces divided by four), NS

Figure VI.27 Forces Applied on Corners of the Panels, Shear Flow Along the Edges, NS

Figure VI.28 Axial Force Diagram for Stringers 1 & 4, NS

Figure VI.29 Axial Force Diagram for Stringers 2 & 3, NS

Figure VI.30 Axial Force Diagram for Stringers A & D, NS

Figure VI.31 Axial Force Diagram for Stringers C & B, NS

Panel Design:

Maximum shear flow from the figures above is 4.52 kip/ft.

Panel Shear reinforcement

ϕ	0.75
f'_c	6 ksi
f_y	60 ksi
t	8 inch

demand	shear flow	V_u/L	4.52	kip/ft
	shear stress	v_u	47.1	psi

capacity	Concrete	v_c	154.9	psi
	without reinforcement	$\phi * v_c / 2$	58.1	psi

no shear reinforcement is needed

No additional shear reinforcement is needed in the slab.

Stringer Design:

Maximum tensile force from the figures above is 223.5 kips, the required reinforcement is calculated as follows:

stringer (collector)		
ϕ	1	
Tu	223.5	kips
fy	60	ksi

As	3.73	in ²
choose	10#6	
As	4.4	OK
ro_lim	2.00%	
width_allow	27.50	in
Width	28	in
ro	1.96%	OK

stringer (collector)		
ϕ	1	
Tu	120.0	kips
fy	60	ksi

As	2.00	in ²
choose	#6	
n	6	
As	2.64	OK
ro_lim	1.50%	
width_alow	22	in
Width	36	in
ro	0.92%	OK

Stringer Reinforcement at peak load, and at a distance of 8' from edge

For constructional reasons, the width of the stringer is chosen so that the reinforcement ratio is less than 2% (preferable 1.5%). Detailing is shown in Figure VI.32 for the stringer in the middle of the slab and at a distance of 8' from slab edge.

Figure VI.32 Stringer Detailing at the Center of the Building (top), and at 8' Distance from the Slab Edge (bottom)

12.7. Fire Sprinkler System

The sprinkler system shall remain functional at Life Safety. A fire sprinkler piping system has been designed and will be placed hanging from the roof. The performance objective for the piping system has been checked for both Fully Operational and Life Safety levels, and in the context of this project, only for the building response in the NS direction.

The fundamental period of this system and effective viscous damping ratio have been determined from a field test. Displacement record for freely vibrating sprinkler is shown in Figure VI.33

Figure VI.33 Displacement Record for Freely Vibrating Sprinkler in NS Direction

From this figure the period could be calculated from average time between four consequent peaks. Damping could be calculated from equation:

$$\zeta = \frac{1}{2\pi j} \ln \frac{u_i}{u_{i+j}} \quad (\text{Chopra, Equation 2.2.14})$$

Where:

- ζ is the damping ratio
- u_i is the displacement at peak number i
- u_{i+j} is the displacement at peak number $i + j$

	Peak 1	Peak 5
Time [sec.]	0.096	1.716
Amplitude	0.1275	0.0308

Delta-T [sec.]	Nr. of peaks	Period [sec.]	amping
1.620	4	0.405	5.65%

Note that the calculated damping ratio depends on the range of the movement, in case of 8 peaks, the damping would be 5.21% instead of 5.65% with 4 peaks.

	Peak 1	Peak 9
Time [sec.]	0.096	3.060
Amplitude	0.1275	0.0093

Delta-T [sec.]	Nr. of peaks	Period [sec.]	Damping
2.964	8	0.371	5.21%

A preliminary analysis shows that the threaded connections can satisfactorily, that is without leaking, resist 18 mm relative floor-pipe displacement demands. With the calculated period of 0.405 second and **effective** viscous damping ratio of 5.65 %, the displacement response and the usage ratio of the piping system has been defined as follows:

Full Occupancy motion (e.g. Covina)

Max. Displacement = 8.88 mm

Usage Ratio = $8.88/18 = 49\%$ OK

Life Safety motion (e.g. Joshua)

Max. Displacement = 61.34 mm

Usage Ratio = $61.34/18 = 341\%$ N.G.

The excessive displacement could be mitigated by adding additional seismic bracing on the piping to limit the movement of the pipe.

12.8. Overhanging Projector

Considering a projector hanging of the slab level 14 of the building (roof). A free vibration test for displacement was carried out for the projector, only NS direction will be considered, Figure VI.34. The fundamental period (T_n , sec) and the percentage of critical damping from the test data (ξ) has been calculated as below:

$$\zeta = \frac{1}{2\pi j} \ln \frac{\ddot{u}_i}{\ddot{u}_{i+j}} \quad (\text{Chopra 5th edition, Equation 2.2.14})$$

Where:

ζ is the damping ratio

$\ddot{u}_i$ is the acceleration at peak number i

$\ddot{u}_{i+j}$ is the acceleration at peak number $i + j$

Figure VI.34 Acceleration Record for Freely Vibrating Projector in NS Direction

	Peak 1	Peak 5
Time [sec.]	0.160	0.772
Amplitude	0.337	0.195

Delta-T [sec.]	Nr. of peaks	Period [sec.]	Damping
0.612	4	0.153	2.18%

	Peak 1	Peak 11
Time [sec.]	0.160	1.672
Amplitude	0.337	0.099

Delta-T [sec.]	Nr. of peaks	Period [sec.]	Damping
1.512	10	0.151	1.95%

The calculated damping ratio depends on the range of the movement considered too, in case of 4 peaks the damping is 2.18%, in case of 10 peaks it is 1.95%.

Using the spectra, the maximum seismic force in the projector corresponding to its period $T_n=0.15$ sec and weight (36 lb) has been calculated. With this force and the weight, factorized tensile and shear forces in the anchorage system could be determined. This is shown in the table and figure below.

PART VII: Conclusion

An idea of performance based seismic design, that stems from analyzing nonlinear response of the structure, is well justified given the merits in terms of better predictability of structure performance compared to linear model. In conclusion, few learnings worth taking note of are discussed below:

- Nonlinear, performance based seismic design, although time consuming and expensive process, was the correct choice for the structure analysis. Had only linear design been used, the transverse reinforcement would not have been enough to withstand an MCE level earthquake, and a plastic hinge would have formed at floor 6 instead of at the bottom of the building as intended.
- Nonlinear behavior is far more accurate at predicting the response of concrete, as it is a nonlinear material when used for seismic design.
- Nonlinear response is governed by the predominant period which is intensity dependent and hence, one can not uniquely predict modal properties of the structure while considering nonlinear response of the structure.
- The structure subjected to nonlinear response, with effective coupling beam model, can better predict the distribution of shear between coupled lateral force resisting system. Hence, the design of lateral force resisting system for shear demand can be done for higher shear demand.
- Resilience Base Seismic Design (RBSD) also proved why nonstructural components need to be properly detailed for earthquakes if they are to be operational after an earthquake. Even if the building structure manages to withstand the earthquake, the building cannot be operational with broken sprinklers, generators, utilities, etc.
- The cost of damage after an earthquake could be so high that the building may need to be demolished. This reasoning can help engineers explain to owners the economic benefits of PBS and RBSD.
- Despite of all advanced computational framework, human error must be considered. Everyone in this project got different results, although everyone started with the same basic model. This uncertainty, which looks random at this moment, it will be gradually reduced with better human machine interface that will be able to post-process the data more efficiently. The review process for PBS needs to be extensive if it is to be used correctly. Hopefully one day PBS will become more common in practice, and the human error could be decreased as it becomes more developed.

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Mr. Richman
14 Story Bldg in LA

No.	Description	Date

Plan View, Level 2

Project number	Project Number	S.001
Date	Issue Date	
Drawn by	Author	
Checked by	Checker	
Scale 1" = 20'-0"		

4/24/2020 2:34:15 AM

② Section 2
1/4" = 1'-0"

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14 Story Bldg in LA

No.	Description	Date

Boundary Element

Project number	Project Number	S.002
Date	Issue Date	
Drawn by	Author	
Checked by	Checker	
		Scale As indicated

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No.	Description	Date
1	Revision 1	Date 1

Walls Reinforcement

Project number	Project Number	S.003
Date	Issue Date	
Drawn by	Author	
Checked by	Checker	
Scale 1/4" = 1'-0"		

4/24/2020 2:34:15 AM

① Section 4
1/4" = 1'-0"

③ South
1/8" = 1'-0"

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No.	Description	Date

Coupling Beams		
Project number	Project Number	S.004
Date	Issue Date	
Drawn by	Author	
Checked by	Checker	
		Scale As indicated