New Acute Care Hospital and Skilled Nursing Facility

San Francisco, CA

Thesis Report

Tim Ariosto – Structural Option

Faculty Advisor - Dr. Richard Behr

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New Acute Care Hospital and Skilled Nursing Facility

845 Jackson Street, San Francisco, CA

Tim Ariosto/Structural Option

www.engr.psu.edu/ae/thesis/portfolios/2010/tma5011/index.html

Building Statistics

Size:	92,000 S.F.
Number of Floors:	7 above ground/ 1 below
Project Cost:	$160 Billion
Construction Dates:	2010-2013
Delivery Method:	Integrated Delivery Process

Architecture

-Design **highlights** include 76 additional beds, several new health care departments, and a pharmacy

-**Addition to existing Chinese Hospital** in Chinatown area of San Francisco

-Facade made up of two types of **precast concrete panals** and glazing

MEP System

-Four **air handling units** located on roof level ranging from 20,000 - 36,000 CFM.

-3000A **277/480V 3-Phase 4-Wire** electrical system

-**Sprinkler system** installed throughout building

Project Team

Owner:	Chinese Hospital
Architect:	Jacobs Carter Burgess
Structural Engineer:	ARUP North America
Mechanical Engineer:	Mazzetti & Associates
Electrical Engineer:	FW Associates, Inc.
Plumbing Engineer:	SJ Engineers
Preconstruction Services:	DPR Construction

Structural System

-36" **mat slab foundation** for seismic dampening

-**Structural steel framing** with 3" composite steel deck and 3-1/4" concrete slab

-Perimeter **special steel moment frames** to resist lateral loads

-**Seismic design** for underpinning and crush zone considerations.

EXECUTIVE SUMMARY

The New Acute Care Hospital and Skilled Nursing Facility is a proposed addition to the Chinese Hospital in San Francisco, CA. This report involves an investigation into the implementation of Fluid Viscous Damper technology in the special steel moment frames of the hospital as a means of resisting the large seismic loads present in the region.

The design involved the redesign the lateral system using a response modification factor of 8 and to meet drift requirements for static loading conditions. Next, the amount of damping required by each frame to resist yielding in a major seismic event was determined using a nonlinear analysis. This total amount of damping was then used to determine the number and capacity of FVDs needed.

Fluid Viscous Dampers were found to be an effective means of reducing the formation of plastic hinges developed in the structure during a Maximum Considered Earthquake as defined by the geotechnical engineers on the project. The design involved (28) 55 kip damping devices located on diagonal braces throughout the moment frames of the structure. 8 devices were used on frames A and E, while 6 devices were used on frames 1 and 7.

Included in the report is an investigation into the architectural impact of the inclusion of the damper system. It was found that while the dampers could be incorporated into the system without major problems, the architectural restrictions of the project prevent a completely smooth implementation.

In addition, the total cost of the damper system was also determined. The device cost, including installation, was found to be approximately $115,900. This increase in cost was accompanied by a savings of $19,805.05 due to reductions in the steel moment frames. The net increase in cost was found to be $96,095.

ACKNOWLEDGEMENTS

I'd like to thank the following individuals for their support in this thesis project.

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INTRODUCTION

The New Acute Care Hospital and Skilled Nursing Facility will serve as an addition to the existing Chinese Hospital located in the historic Chinatown district of San Francisco (See Fig. 1). The site lies on the north flank of Nob Hill, at an elevation of approximately 110' above sea level. Due to the slope of the site, the ground floor of the site is located partially below grade.

This new addition will be connected directly to the existing Chinese Hospital, located at 845 Jackson Street. As part of the construction of this addition, the original portion of the hospital built in 1925 will be demolished. Then the new facility, which has seven stories above ground and one below will be constructed with a hard connection to a previous addition built in 1975. Therefore, the precast concrete panel exterior façade has been designed in a way that respects the 1975 design while providing a more modern look.

FIGURE 1: SITE VIEW OF NEW ACUTE CARE HOSPITAL (BLUE) LOCATED ADJACENT TO EXISTING CHINESE HOSPITAL. PHOTO COURTESY OF GOOGLE MAPS.

At approximately 92,000 SF, this new facility will provide additional patient rooms as well as well several new medical departments to serve the local community. Construction is expected to begin in 2010 and reach completion by Chinese New Year 2013. Throughout this thesis investigation, only the addition will be investigated.

FIGURE 2: EXTERIOR VIEW OF NEW ACUTE CARE HOSPITAL AND SURROUNDING BUILDINGS. PHOTO COURTESY OF JACOBS-CARTER BURGESS.

EXISTING STRUCTURAL SYSTEM

The structure of the New Acute Care hospital rests on a mat foundation and consists primarily of composite steel decking with steel framing. A perimeter moment frame system is used to resist lateral loading.

FOUNDATION SYSTEM

According to the geotechnical report provided by Treadwell & Rollo, the soil conditions on the site can be described as "very stiff to hard sandy clay and clay with gravel," which rests on "intensely fractured, low hardness, weak, deeply weathered shale." Because of this, the New Acute Care Facility has been designed to bear on a 36" mat foundation. Columns rest on concrete pedestals, typically sized at 3'-0" x 3'-0". Since the base of the structure will lie below the water table, the foundation was also designed for hydrostatic uplift.

The close proximity to nearby structures (see Figure 2), particularly the 1975 addition to the Chinese Hospital, provided a challenge to the designers. Underpinning was used to maintain the foundations of existing structures on either side of the building.

FRAMING SYSTEM

The New Acute Care Hospital uses steel columns (See Figure 3) to support the buildings gravity loads. These columns range in size from W14x445 near the base of the structure to W8x40's near the roof level. As the columns rise vertically through the structure they are spliced together, usually at a distance of 22'-0". Aside from those used in the lateral system, most of the columns are connected to beams and girders using pinned connections.

FLOOR SYSTEM

The floor system consists of a composite floor system using a 3" Verco W3 Formlock deck with an additional 3 ¼" of concrete resulting in a total thickness of 6 ¼". This slab then rests on W-shapes ranging from W10x12's used as beams to sizes as large as W24x207's which also serve in the buildings lateral system. ¾" ∅ shear studs were used to achieve composite action.

There are several different bay sizes used in the New Acute Care Hospital. Larger bays typically exist towards the plan east side of the building while smaller bay sizes are typically used in the western portion of the structure. In most cases, the bays varied from approximately 18'-0"x 17'-0" to 23'-10"x24'-0".
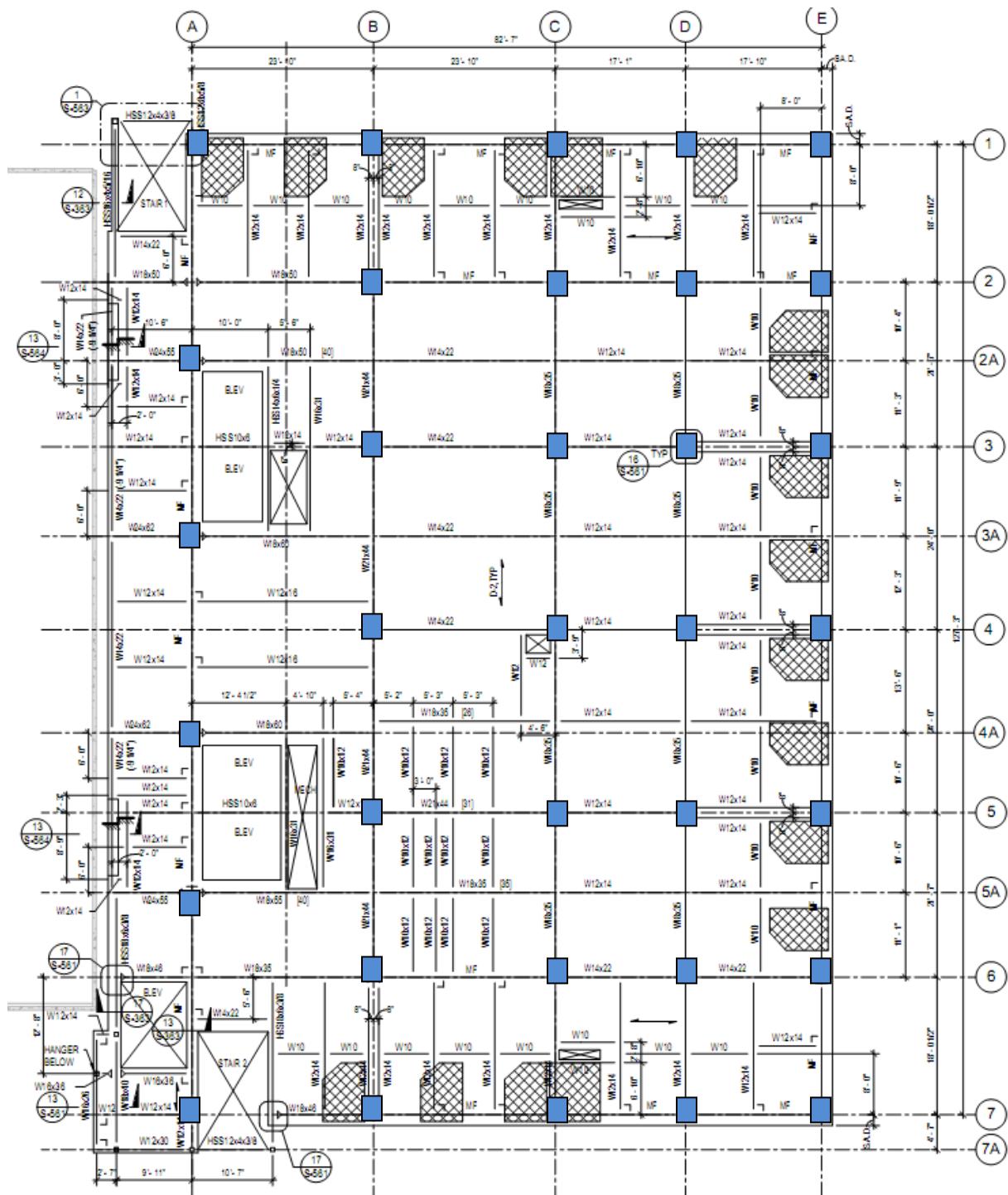
FIGURE 3: TYPICAL FRAMING PLAN WITH COLUMNS HIGHLIGHTED. DRAWINGS COURTESY OF JACOBS CARTER BURGESS.
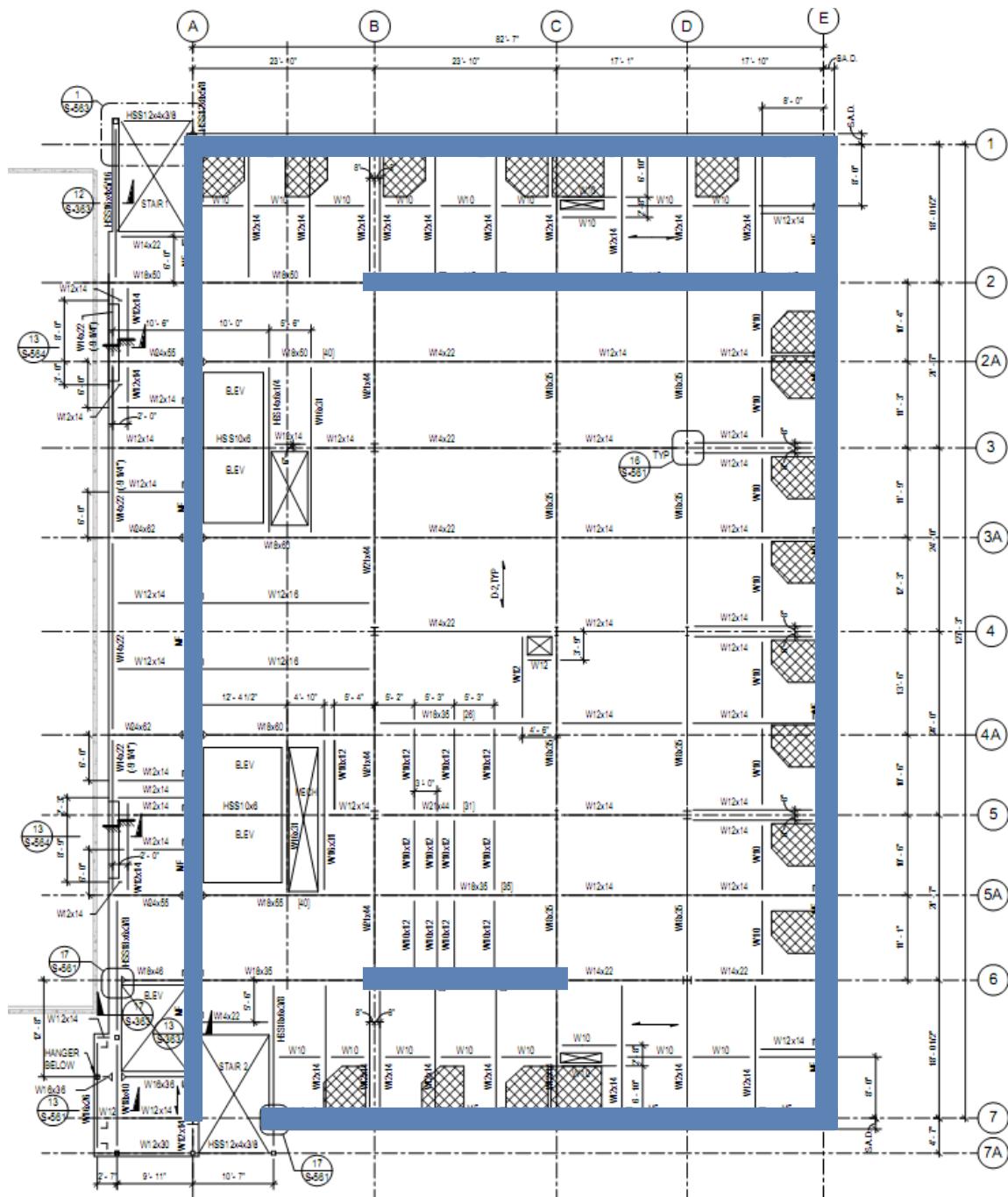
FIGURE 4: TYPICAL FRAMING PLANS WITH LATERAL SYSTEM HIGHLIGHTED IN BLUE. DRAWING COURTESY OF JACOBS CARTER BURGESS.

LATERAL SYSTEM

Lateral loads are transmitted through the structure primarily through the use of a series of special moment frames. There are 4 special moment frames running east-west, and 2 running north-south. One of the EW frames, located along gridline 2, terminates at the third floor level. See Figure 4 for the locations of the special moment frames.

Since brittle failure of connections in moment frames can potentially be a problem in regions of high seismic activity, the moment frame beams have been designed using Reduced Beam Sections (RBS). These RBS sections help to insure that yielding occurs in the reduced section of the beam rather than in the connection itself. See Figure 5 below.
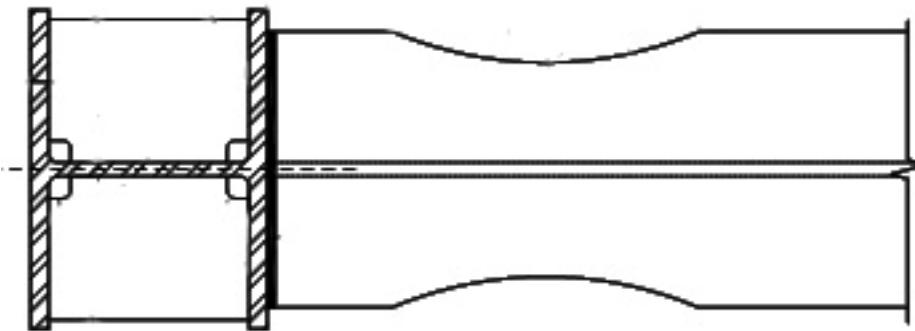
FIGURE 5: REDUCED BEAM SECTION

In addition to the steel moment frames, the basement walls also serve as shear walls for the basement level. These walls are 18" thick and composed of 4ksi concrete.

ROOF SYSTEM

The roof system is supported in a similar manner to the floors below, with a concrete filled metal deck supported by beams and girders. However, beams at this level are typically spaced much closer together, at a distance of approximately 10-12 feet. The sizes of these roof beams generally vary from W10x12's to W24x104's.

OTHER FEATURES

One of the unique structural features of the New Acute Care Hospital is its connection to the existing Chinese Hospital. The two structures are connected with a seismic gap that allows them to act independently. This size of this gap varies with story height so that a greater amount of movement is allowed at the upper floors.

A second unique feature of the New Acute Care Hospital is a result of the tight floor plan. There are several areas in which partition walls lie directly on beams. Since plumbing would normally

be routed through these partition walls, a system of two, parallel beams spaced at 16" were used to create a gap for the plumbing system. See Figure 6 below.
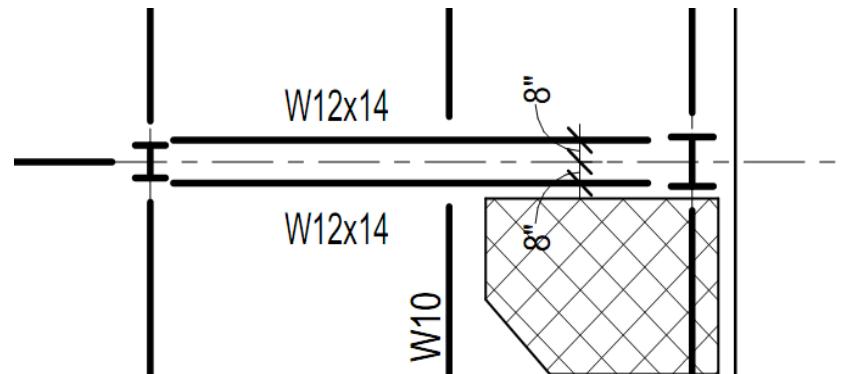
FIGURE 6: PARALLEL BEAMS USED FOR PLUMBING

MATERIALS USED

TABLE 1: MATERIAL PROPERTIES USED IN ORIGINAL AND THESIS DESIGN

Concrete

Location	Weight	Strength f'c (ksi)
Foundation	Normal	4000
Drilled Piers	Normal	4000
Slab-on-Grade Walls, Columns, and Piers	Normal	4000
Fill in Metal Deck and Curbs at Ground Floor	Normal	4500
Fill in Metal Deck at First Floor and Above, Top	Normal	4500
Fill in Stair Pans	Normal	2500
Fill in Over-Excavated Areas and Conduit Encas	Normal	1500

Structural Steel

Type	Standard	Grade
W-Shapes	ASTM A992	Grade 50
Other Shapes	ASTM A992	Grade 50
Plates for Built-Up Members	ASTM A572	Grade 50
Steel Channels, Angles, Base Plates, Shear Tabs	ASTM A36	Grade 36
Structural Steel Plates	ASTM A572	Grade 50
Steel Bars	ASTM A529	Grade 50
Square or Rectangular Steel Tubes	ASTM A500	Grade B
Round Steel Tubes	ASTM A500	Grade C
Pipe Sections	ASTM A53	Grade B

Reinforcing Steel

	ASTM A615	Grade 60

APPLICABLE CODES

ORIGINAL DESIGN CODES USED

In addition to the following codes, the California State Government requires that all new government and hospital buildings are approved by the Office of Statewide Health Planning and Development (OSHPD).

- 2007 California Administration Code
  - Part 1, Title 24, CCR
- 2001 California Building Code
  - Part 2, Title 24, CCR
  - (1997 UBC and 2001 CA Amendments)
- 2004 California Electrical Code
  - Part 3, Title 24, CCR
  - (2002 NEC and 2004 CA Amendments)
- 2001 California Fire Code
  - Part 4, Title 24, CCR
  - (2000 UMC and 2001 Amendments)

DESIGN CODES AND STANDARDS USED IN THESIS ANALYSIS

- American Society of Civil Engineers (ASCE)
  - ASCE7-05, Minimum Design Loads for Buildings and Other Structures
- International Building Code, 2006 Edition (IBC)
- American Institute of Steel Construction (AISC)
  - Steel Construction Manual, Thirteenth Edition (LRFD)
- American Concrete Institute
  - ACI 318-08, Building Code Requirements for Structural Concrete
- National Earthquake Hazards Reduction Program (NEHRP)
  - Recommended Provisions for New Buildings and Other Structures (2003)
  - Recommended Seismic Provisions (2009)
- Federal Emergency Management Agency (FEMA)
  - Effects of Strength and Stiffness Degradation on Seismic Response (FEMA P440A)
  - Prestandard and Commentary for the Seismic Rehabilitation of Existing Buildings (FEMA 356)

DESIGN LOADS

GRAVITY LOADS

TABLE 2: LIVE LOADS

Live Load (psf)		
Live Load	As Designed	Per ASCE 7
Treatment Rooms	80*+20(partitions)	60
Patient Room	80*+20(partitions)	40
Other Rooms (offices)	80*+20(partitions)	50
Storage Areas		
Fixed Racks	125	125
Mobile Racks	250	250
Corridors	100	80
Mechanical Rooms	125	
Roof (Mech)	125	
Roof (Other)	20*	20

*Reducible

The designed live loads were found to be larger than the minimum live loads specified by ASCE7-05. It is likely that these values were higher based on the more stringent requirements of OSHPD as well as the experience of the designers.

TABLE 3: DEAD LOADS

Floor Dead Loads	
Material	psf
6 1/4" Concrete Deck	50
Finishes	1
MEP and Misc.	20
Total	71

Exterior Wall Dead Loads	
Material	psf
5" Concrete Panels	50
6" Metals Studs and Wallboard	0.38
6" Batt Insulation	0.9
Total	51.28

Partition Wall Dead Loads (psf)	
Per ASCE7-05 12.7.2	10

Roof Dead Loads	
Material	psf
80 Mil. TPO Roof Membrane	5.5
5/8" Dens Deck	2.5
6 1/4" Concrete Deck	60.4
Total	68.4

Dead load values were determined from a combination of sources including but not limited to ASCE7-05, design aids, and manufacturer specifications

According to ASCE7-05 Figure 7-1, the ground snow load for San Francisco CA is 0 lb/ft^2. Therefore, no further investigation was taken into snow loads.

LATERAL LOADS

WIND LOADS

Wind loads were calculated as prescribed by ASCE7-05 Chapter 6. Although the New Acute Care Facility is an addition to an existing structure, it was modeled as an independent structure and the effects existing building on wind forces were neglected for the purpose of this analysis. This simplification was appropriate in that it allows for the possibility that the existing Chinese Hospital structure being demolished at a later date.

Microsoft Excel was used extensively in both the analysis and determination of net wind pressures, story forces, and overturning moments. The net wind pressures comprised of pressure of the windward, leeward, side, and internal area of the building. A detailed summary of the analysis can be found in Appendix A: Load Calculations. Once the net wind pressures were determined, the net wind loads were found. Wind loads were the largest in the NS direction resulting in a base shear of 151.59 kips and an overturning moment of 26,828.17 ft-kips (See Figure 7).

TABLE 4: WIND LOADS - NS DIRECTION

Wind Loads - NS Direction					
Floor Level	Floor Height (ft)	Elevation (ft)	Story Force (kips)	Total Story Shear (kips)	Overturning Moment Contribution (ft-k)
Ground	12.5	0	11.94	151.59	0
1	13.5	12.5	12.90	139.65	1745.59
2	13.5	26	15.12	126.75	3295.45
3	13.5	39.5	17.13	111.63	4409.44
4	13.5	53	18.60	94.50	5008.54
5	15	66.5	22.03	75.90	5047.29
6	15	81.5	23.50	53.87	4390.63
PH	18.5	96.5	30.38	30.38	2931.23
		Total Overturning Moment (ft-kips)			26828.17
		Total Shear (kips)			151.59
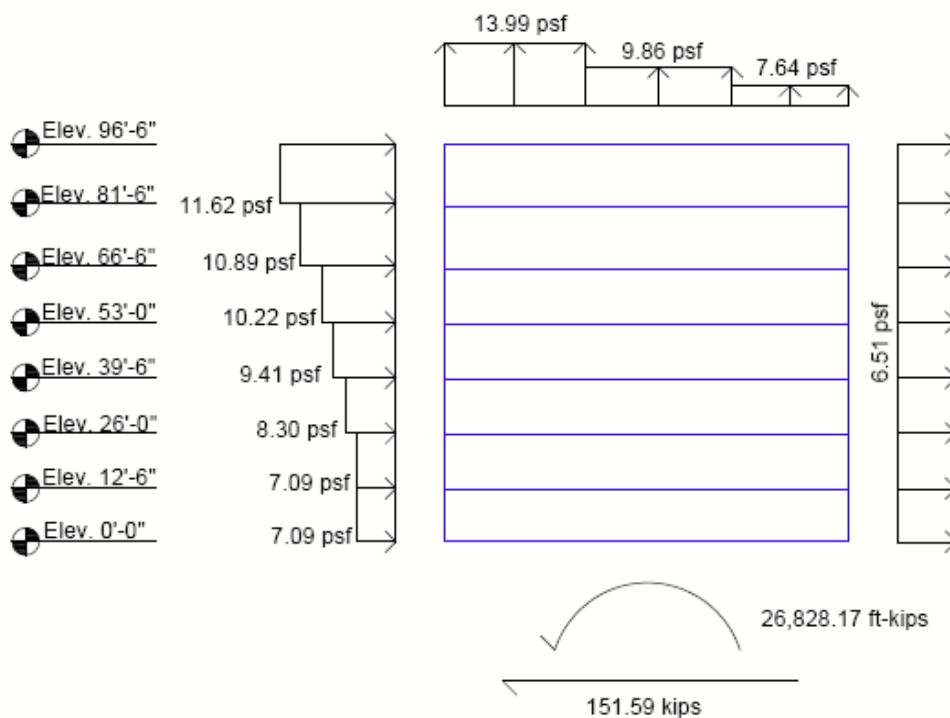
FIGURE 7: WIND LOADING DIAGRAM (NS) SHOWING WIND PRESSURES

TABLE 5: WIND LOADS – EW DIRECTION

Floor Level	Floor Height (ft)	Elevation (ft)	Story Force (kips)	Total Story Shear (kips)	Overturning Moment Contribution (ft-k)
Wind Loads - EW Direction					
Ground	12.5	0	8.29	105.28	0
1	13.5	12.5	8.96	96.98	1212.31
2	13.5	26	10.50	88.03	2288.69
3	13.5	39.5	11.90	77.53	3062.36
4	13.5	53	12.92	65.63	3478.43
5	15	66.5	15.30	52.71	3505.34
6	15	81.5	16.32	37.41	3049.29
PH	18.5	96.5	21.10	21.10	2035.74
		Total Overturning Moment (ft-kips)			18632.16
		Total Shear (kips)			105.28
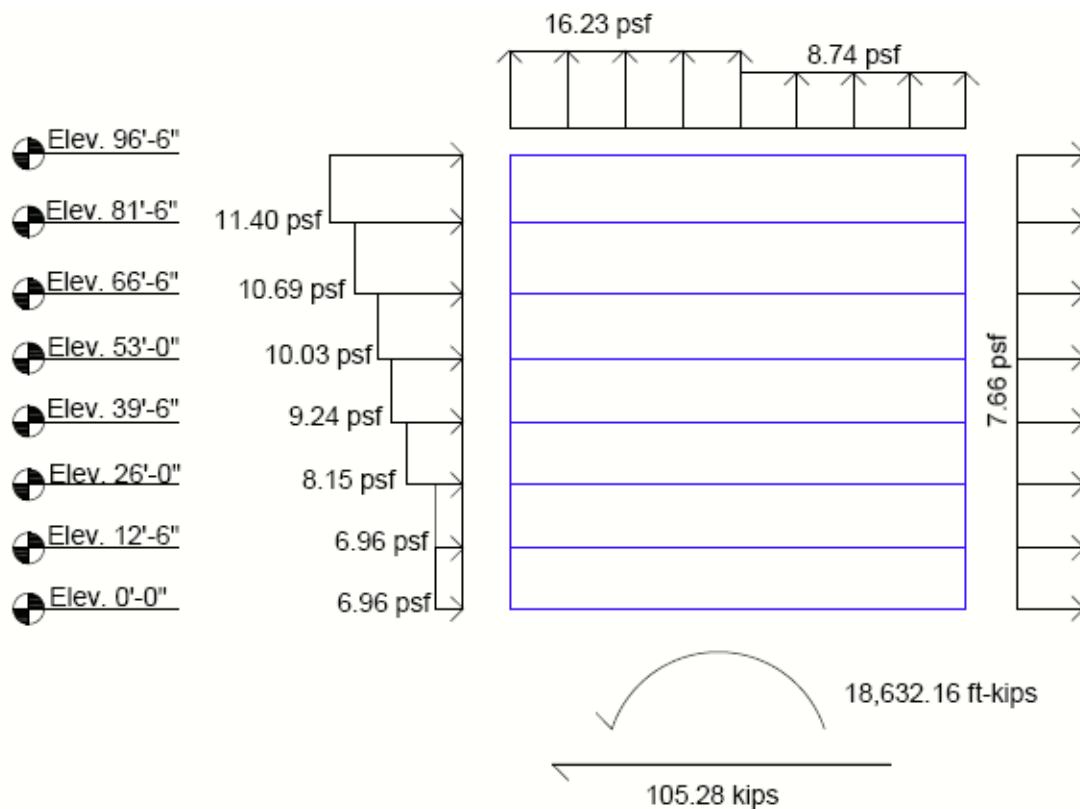
FIGURE 8: WIND LOADING DIAGRAM (EW) SHOWING WIND PRESSURES

SEISMIC LOADS

For this thesis analysis, the seismic loads were initially calculated using the Equivalent Lateral Force Method (ELFM) outlined in ASCE7-05 Chapter 12. Since a computer model was available at the time of the analysis, the fundamental period of the structure was compared with that calculated using the code ($T_a = C_t h_n^x$) which resulted in a period of 1.75sec. However, since the first mode period determined using ETABS, 2.08 sec., was greater than the code specified period, the code specified value was still used.

Since the New Acute Care Hospital uses special moment frames in both directions, the code specified period, T_a is independent of direction for this structure. Therefore, a single analysis holds for both directions. For a detailed set of calculation procedures, see Appendix A: Load Calculations.

The analysis resulted in a base shear of 771.49 kips and an overturning moment of 64,853.54 ft-kips.

TABLE 6: SEISMIC LOADS CALCULATED USING EQUIVALENT LATERAL FORCE METHOD

Seismic Loads						
Floor	h (ft)	w (kips)	wh^k	Story Force F$_i$ (kips)	Story Shear V$_i$ (kips)	Torsional Moment M$_z$ (ft-kips)
7	111.5	1945.12	2382226	215.34	0.00	1453.54
6	96.5	1839.94	1812149	163.81	215.34	13268.45
5	83	1850.11	1451666	131.22	379.15	10629.01
4	69.5	1840.6	1104953	99.88	510.37	8090.40
3	56	1865.87	808697.2	73.10	610.25	5921.23
2	42.5	1907.14	545238.4	49.29	683.35	3992.20
1	30	1881.67	318112.4	28.76	732.64	2329.20
Ground	15	1879.64	111698.9	10.10	761.40	817.85
Basement	0	0	0	0.00	771.49	0.00
Base Shear (kips)				771.49		
Overturning Moment (ft-kips)				64853.54		
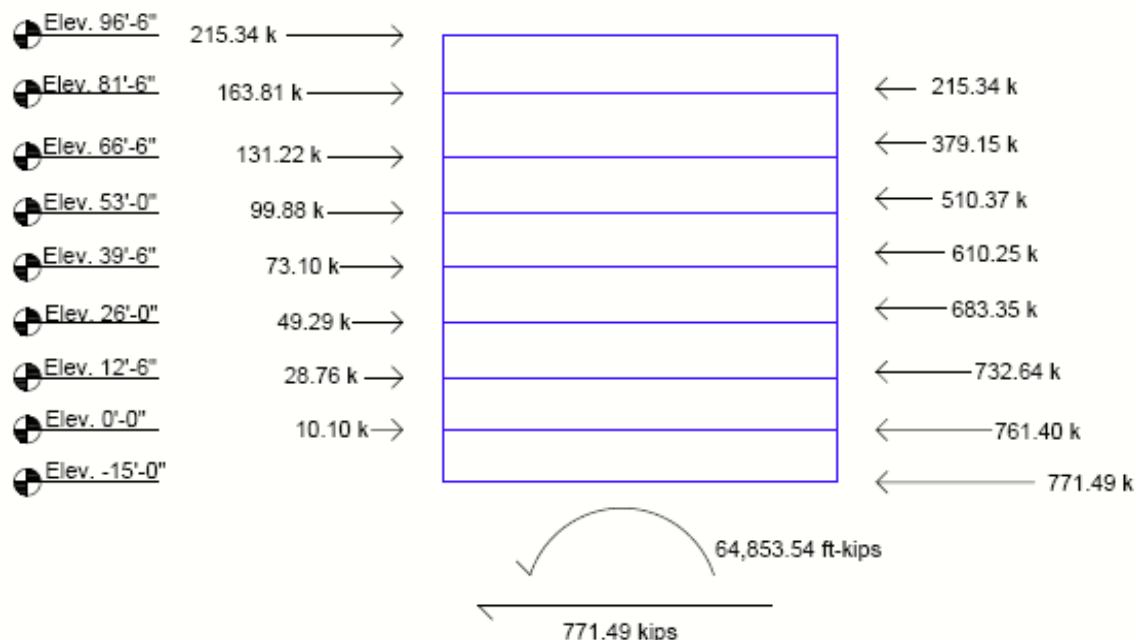
FIGURE 9: SEISMIC LOADING DIAGRAM (BOTH DIRECTIONS) SHOWING STORY FORCES (LEFT) AND STORY SHEARS (RIGHT)

LOAD COMBINATIONS

The ASCE7-05 load combinations are given in Figure 10 below. For the analysis of the lateral system, the key load combinations are 4 and 5 for general loading, and 6 and 7 for uplift. It can be seen that determination of the governing load case can be simplified to whether 1.6W is greater than 1.0E for the general loading conditions and uplift. Since seismic loads are greater than the wind loads by a large margin, cases 5 and 7 can be said to control strength design for general loading and uplift respectively. In addition, it is evident that the general loading combination for seismic will control strength design.

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

FIGURE 10: ASCE 7-05 LOAD COMBINATIONS

ETABs was used to confirm this assertion by comparing the story shears at the seventh floor for each load combination.

TABLE 7: 7TH FLOOR SHEAR FORCE OUTPUT FROM ETABS

Story	Load	Loc	P	VX	VY	T	MX	MY
STORY7	COMB401	Bottom	163.05	-45.41	-62.08	-201736	130307.6	-93249.3
STORY7	COMB402	Bottom	163.05	-45.41	-62.08	215429.6	130307.6	-93249.3
STORY7	COMB501	Bottom	163.05	-341.14	-341.14	107629.7	180538.4	-146481
STORY7	COMB601	Bottom	122.28	-45.41	-62.08	-201736	100524.3	-71980.3
STORY7	COMB701	Bottom	122.28	-341.14	-341.14	107629.7	150755.1	-125212

PROBLEM STATEMENT

As was previously shown, the structure of the New Acute Care Hospital and Skilled Nursing Facility is controlled by seismic loads. Unlike wind loads, seismic loads are primarily a strength issue. This is true to the extent that the prevailing design philosophy for the seismic design of structures is to allow only minor damage in moderate earthquakes, and some major dame in severe earthquakes. In fact, the primary concern for severe earthquakes is to save lives by preventing a complete collapse of the structure. However, following a severe earthquake, certain structures would be essential to the response effort. These structures include police departments, fire stations, prisons, and a range of other structures, including hospitals. Therefore, a unique challenge is presented to designers in how to design a structure that can not only withstand a severe earthquake, but also remain fully functional immediately following the event.

PROPOSED SOLUTION AND DESIGN PHILOSOPHY

When major seismic events occur, structural elements tend to deform beyond their elastic limits, which can potentially result in failure or collapse of these individual elements. This failure is a result of the structure dissipating energy developed during the earthquake through structural damage. One possible solution to this problem is by using Fluid Viscous Dampers (FVDs) to absorb this energy and dissipate it in the form of heat. This can be compared to how shock absorbers reduce the impact of sudden jerks and movements in a car.

FIGURE 11: TAYLOR DEVICES FVD, 50,000LB OUTPUT. PHOTO COURTESY OF TAYLOR DEVICES INC.

Fluid Viscous Damper technology was originally used for a variety of military applications, such as absorbing the recoil energy of weapons. After the cold war ended in 1990, this technology became public. Since that time, FVDs have been applied to numerous different building and bridge projects around the world.

The purpose of this thesis investigation will be to redesign the lateral system of the New Acute Care Hospital to make use of Fluid Viscous Damper technology. The lateral system will first be designed using a response modification factor of 8 and to meet drift requirements for static loading conditions. Next, the amount of damping required by each frame to resist yielding in a major seismic event will be determined using a nonlinear analysis. This total amount of damping will then be used to determine the number and capacity of FVDs needed.

Since this structure is considered an essential facility for the San Francisco Bay area, it must be able to remain operable under even the most severe seismic conditions. Therefore, an alternate design will be sought that can experience no yielding under a Maximum Considered Earthquake.

The criteria used to determine the success of a design will be the performance standards specified by the Federal Emergency Management Agency's *Prestandard and Commentary for the Seismic Rehabilitation of Existing Buildings* (FEMA 356). FEMA 356 lists three structural performance levels for seismic loads (See Figure 12 **Error! Reference source not found.** below) by which the formation of plastic hinges are evaluated. In the first level, Immediate Occupancy (IO), minor repairs may be needed, but are not typically required prior to reoccupying the building. In the second level, Life Safety (LS), there is a low risk of life-threatening injuries as a result of structural failure. At this stage, repair is possible, but may not be practical for economic reasons. In the third stage, Collapse Prevention (CP), the structure is on the verge of collapse. Structural repairs are not typically either possible or practical. Since the goal of this analysis is to achieve a design in which yielding does not occur, the LS and CP levels will be

Elements	Type	Structural Performance Levels		
		Collapse Prevention S-5	Life Safety S-3	Immediate Occupancy S-1
Steel Moment Frames	Primary	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Hinges form. Local buckling of some beam elements. Severe joint distortion; isolated moment connection fractures, but shear connections remain intact. A few elements may experience partial fracture.	Minor local yielding at a few places. No fractures. Minor buckling or observable permanent distortion of members.
	Secondary	Same as primary.	Extensive distortion of beams and column panels. Many fractures at moment connections, but shear connections remain intact.	Same as primary.
	Drift	5% transient or permanent	2.5% transient; 1% permanent	0.7% transient; negligible permanent

FIGURE 12: STRUCTURAL PERFORMACE LEVELS AS SPECIFICED BY FEMA 356

considered unacceptable.

DESIGN CONCEPT

There are several different variations in which Fluid Viscous Dampers may be incorporated into the building structure. Among these are as part of diagonal braces, chevron braces, horizontal braces, and toggle-brace dampers. Each of these configurations has its own set of benefits. For example, the toggle-brace damper system is effective if drift values are low enough that some amplification is necessary for the dampers to be effective. The design principles for any of these damper systems are identical, with changes occurring only in the specific behavior of the dampers. Diagonal braces are the configuration that is most commonly used due to the effectiveness by which damping can occur. Therefore, this is the only set-up that will be investigated.

FIGURE 13: DIAGONAL BRACE FVD CONFIGURATION; PHOTO COURTESY OF TAYLOR ET AL.

These diagonal fluid viscous dampers will be placed in the four exterior moment frames for several reasons. First, these four frames are the strongest frames. Secondly, these frames continue all the way through the structure, whereas frame 2 terminates at the third floor level. Finally, braces in these frames have the least impact on the architectural layout. For more information on the architectural impact of these dampers, see Architectural Impact on page 41.

COMPUTER MODELING

Two independent computer models were used in this analysis. A 2D model was created using SAP for the purposes of performing various analysis on individual frames, while a 3D model was created using ETABs to determine effects of loads on the complete lateral system.
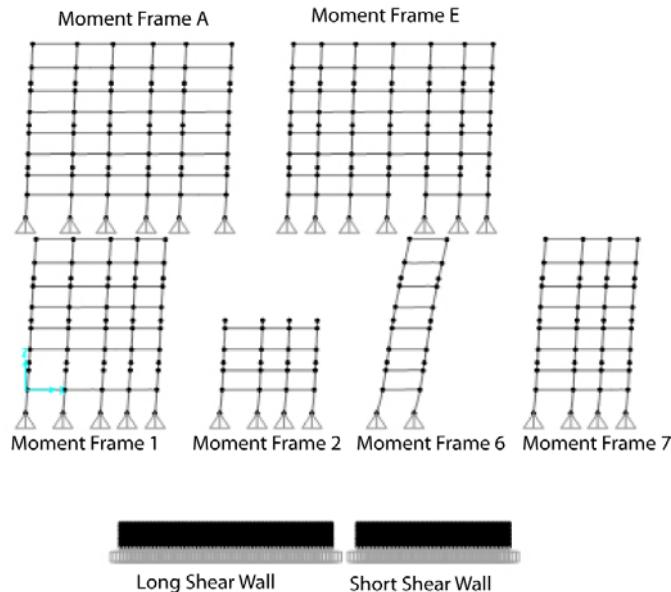
FIGURE 14: 2D MOMENT FRAME MODELS

While these models had several differences, they were created using a number of similar attributes. In addition to the geometric and material based constraints of the structure, there were several aspects of the special moment frames that were incorporated into both models.

There are 3 major attributes of special moment frames that were modeled using each software package. First, panel zones were explicitly modeled at beam-column connections to account for the yielding and deformations that occur at these areas due to buildup of shear forces due to moment transfer. This is required by ASCE 7 §12.7.3b. Secondly, the reduced properties of the beam sections due to the RBS's had to be taken into account. This was accomplished by modeling the beams using the RBS connection type in ETABs and 90% of the
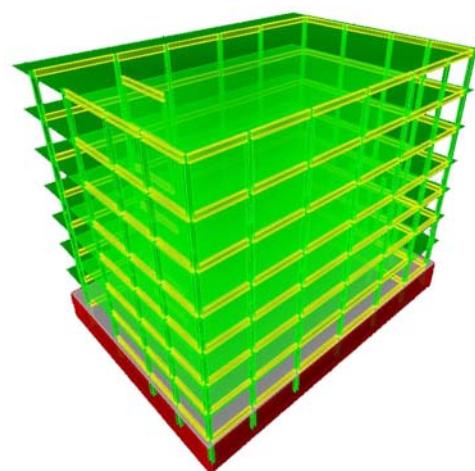
FIGURE 15: 3D LATERAL SYSTEM MODEL

section properties in SAP. Lastly, the column restraints at the base of the structure were modeled as "pinned" connections in order to achieve a conservative approximation of the column base fixity.

In addition to these requirements, the concrete shear walls at the basement level were assigned a modification of 70% of the moment of inertia as specified by ACI 318.08 §10.10.4.1 and ASCE7-05 §12.7.3a. This effectively "cracks" the section giving a reduced strength.

A detailed account of other modeling assumptions can be found in Appendix C: Computer Modeling.

STRUCTURAL REDESIGN OF LATERAL SYSTEM

STIFFNESS ANALYSIS

The center of rigidity of each floor was determined using the relative stiffness of each frame. This stiffness was taken as the ratio of the applied load to horizontal displacement it causes.

$$k_i = \frac{p}{\Delta_p}$$

Once the stiffness of each frame was found, the center of rigidity was found by dividing the sum of each elements stiffness times its location by the total stiffness in that direction.

$$\overline{X} = \frac{\sum k_{iy} x_i}{\sum k_{iy}} \qquad\qquad \overline{Y} = \frac{\sum k_{ix} y_i}{\sum k_{ix}}$$

After the center of rigidity was found, a thorough analysis was undertaken to determine the behavior of the structure under lateral loading. This was accomplished by applying a unit load to the center of mass of each floor. The load path was determined by adding the force in each frame developed due to direct forces to the torsional forces developed due to eccentricity.

$$F_i = F_{i\,direct} + F_{i\,torsion}$$

Since these forces were determined using a unit load, they can easily be used to express the percentage of the lateral load that each frame element carries. Since each of the moment frames do not share the same height, this analysis was performed in separate iterations for the basement, the ground floor through the third floor, and the fourth floor through the roof level. The results of the stiffness analysis, which show the percentage of lateral load distributed to each frame, can be found in Table 8. See Appendix B: Center of Rigidity Determination for a complete set of calculations.

TABLE 8: PERCENTAGE LOADS TO EACH FRAME BY LEVEL

$F_i = F_{idirect} \pm F_{itorsion}$ Basement	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.001005	-0.02633	-0.02532	-2.532	
Grid Line 2	0.000292	-0.01153	-0.01124	-1.124	
Grid Line 6	0.000699	0.007679	0.008378	0.838	
Grid Line 7	0	0.024461	0.024461	2.446	100.35
Basement - short high	0.499002	-0.02948	0.469522	46.952	
Basement - short low	0.499002	0.038668	0.537671	53.767	
Grid Line A	0.000743	0	0.000743	0.074	
Grid Line E	0.000796	0	0.000796	0.080	
Basement - long west	0.499231	0	0.499231	49.923	100.00
Basement - long east	0.499231	0	0.499231	49.923	

$F_i = F_{idirect} \pm F_{itorsion}$ Ground Floor - 3rd Floor	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.303399	0.04693	0.350329	35.033	
Grid Line 6	0.110249	-0.0171	0.09315	9.315	
Grid Line 7	0.292078	-0.06034	0.231736	23.174	100.00
Grid Line 2	0.294274	0.03052	0.324794	32.479	
Grid Line A	0.484076	0.063064	0.54714	54.714	
Grid Line E	0.515924	-0.06307	0.452852	45.285	100.00

$F_i = F_{idirect} \pm F_{itorsion}$ 4th Floor - Roof	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.458388	0.018751	0.477139	47.714	
Grid Line 6	0.145529	-0.00597	0.13956	13.956	99.11
Grid Line 7	0.396083	-0.02164	0.374443	37.444	
Grid Line A	0.48	0.081335	0.561335	56.133	99.87
Grid Line E	0.52	-0.08268	0.437316	43.732	

STATIC PUSHOVER ANALYSIS

After the relative stiffness each frame was determined, the appropriate percentages of the lateral loads were applied to each frame. These loads were simply the calculated story force multiplied by the percentage load found in Table 8 above. The values for the seismic load (Equivalent Lateral Force Method) can be found in Table 9 below.

TABLE 9: SEISMIC LOAD APPLIED TO EACH FRAME BASED ON STIFFNESS

Grid 1 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	46%	99.617
6	163.81	46%	75.778
5	131.22	46%	60.704
4	99.88	46%	46.206
3	73.10	34%	24.617
2	49.29	34%	16.597
1	28.76	34%	9.683
Ground	10.10	34%	3.400
Basement	0.00	-3%	0.000

Grid 7 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	37%	79.630
6	163.81	37%	60.574
5	131.22	37%	48.525
4	99.88	37%	36.935
3	73.10	22%	15.992
2	49.29	22%	10.782
1	28.76	22%	6.291
Ground	10.10	22%	2.209
Basement	0.00	2%	0.000

Grid 2 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	0%	0.000
6	163.81	0%	0.000
5	131.22	0%	0.000
4	99.88	0%	0.000
3	73.10	35%	25.402
2	49.29	35%	17.126
1	28.76	35%	9.992
Ground	10.10	35%	3.509
Basement	0.00	-1%	0.000

Grid A Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	57%	122.361
6	163.81	57%	93.079
5	131.22	57%	74.563
4	99.88	57%	56.755
3	73.10	56%	40.805
2	49.29	56%	27.512
1	28.76	56%	16.051
Ground	10.10	56%	5.636
Basement	0.00	0%	0.000

Grid 6 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	16%	34.185
6	163.81	16%	26.004
5	131.22	16%	20.831
4	99.88	16%	15.856
3	73.10	10%	7.242
2	49.29	10%	4.882
1	28.76	10%	2.849
Ground	10.10	10%	1.000
Basement	0.00	1%	0.000

Grid E Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	43%	92.689
6	163.81	43%	70.508
5	131.22	43%	56.482
4	99.88	43%	42.992
3	73.10	44%	32.478
2	49.29	44%	21.897
1	28.76	44%	12.775
Ground	10.10	44%	4.486
Basement	0.00	0%	0.000

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Next, the gravity and lateral loads were applied to each frame. The *1.2D + 1.0E + L* load combination, which was previously found to be the controlling load combination, was used, and the maximum moments were determined using SAP. A demand to capacity (DC) ratio was then determined for each member in which

$$DC = \frac{Maximum\ Moment}{S_{Girder}F_y}\ \text{for girders}$$

and

$$DC = \frac{Maximum\ Moment}{S_{Column}(F_y - \frac{P_u}{A_{Column}})}\ \text{for columns.}$$

It is important to note that the elastic section modulus, S, was used rather than the plastic section modulus, Z, since the members are intended to remain elastic. Throughout the structural redesign, members were designed so that the DC ratio for the columns remains lower than that for the beams. This insures that strong column-weak beam action is maintained. In other words, the beams will always fail prior to the columns.

Hinges were then added to the ends of both the beams and columns. These hinges were located away from member ends according to the provisions of *FEMA 356: Prestandard and Commentary for the Seismic Rehabilitation of Buildings*.

SAP was then used to subject each frame to a pushover analysis. In a pushover analysis, the frame is first subjected to an initial load condition from which the analysis begins. In this case, the full dead loads were used for the initial condition. A lateral load was then applied to the frame, and stresses and deflections are recorded. The load is gradually increased, and stresses and deflections are recorded in several different intervals. In addition, the formation of plastic hinges is denoted for each "step".

A pushover analysis gives several important pieces of information. First, it can be used to determine where failure is most likely to occur based on the formation of plastic hinges. For example, in Figure 16 below, it can be seen that plastic hinges form starting at the lower levels, and move upward throughout the structure. The yellow hinges indicate that collapse is occurring, and should be prevented. Since our goal was to insure that the structure meets the Immediate Occupancy performance level, the cyan and green hinges should also be avoided.

Plastic Hinges	
Immediate Occupancy	
Life Safety	
Collapse Prevention	
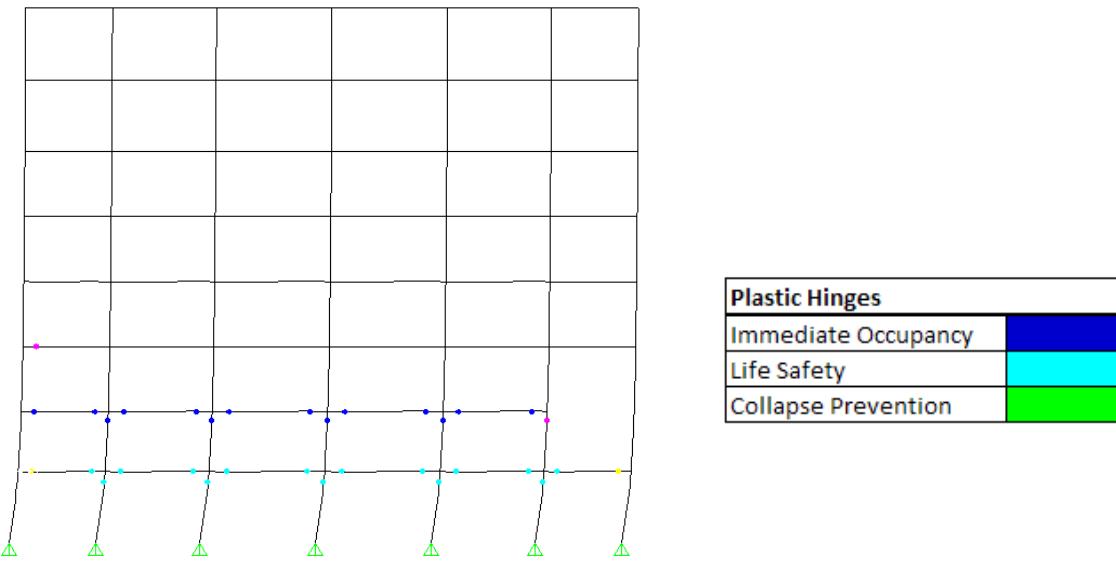
FIGURE 16: FORMATION OF PLASTIC HINGES ON FRAME E

In addition, a plot of base shear vs displacement gives what is known as a pushover curve. This pushover curve can be used to show how the structure as a whole (in this case, an individual frame) will behave under increased loading. The slope at any given point indicates the stiffness of the structure given a certain degree of loading. The sharp discontinuities indicate the formation of a plastic hinge, and therefore a loss of strength in the structure. Although the overall strength of the structure is indicated by the peak value for Base Shear, several plastic hinges are formed by this point, and elastic behavior is no longer occurring. Since the goal of the structural redesign is to ensure that the structure remains elastic, a point along the initial straight line segment of the curve must be selected as the system strength.
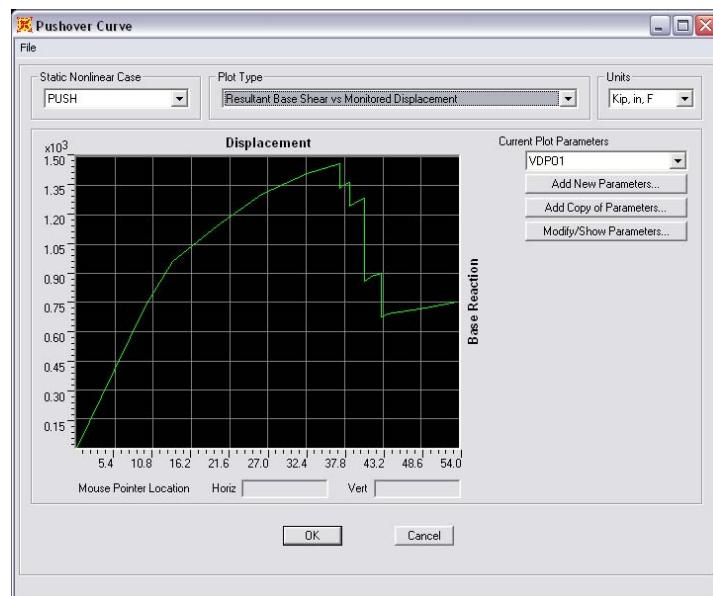
FIGURE 17: BASE SHEAR VS DISPLACEMENT PUSHOVER CURVE

CONVERSION INTO SDOF SYSTEM

There are several methods by which a point along the pushover curve can be selected. The design spectra from either ASCE7-05 or NEHRP can be used to determine the "demand" on the structure. However, a more accurate method is to use the Site-Specific Response Spectra (SSRS). This spectrum was determined specifically for the New Acute Care Hospital by the geotechnical engineers (Treadwell & Rollo), and therefore will result in a demand curve that is lower than that produced by code. For the purpose of comparison, two site specific demand spectra's were used.

The first is the Maximum Considered Earthquake (MCE) which refers to a seismic event having a Moment Magnitude of 7.9 located at a distance of 12.7 km from the building site. This equates to an event that has only a 2% probability of exceedance in 50 years.

The second spectrum plotted is for a Design Level Earthquake (DE). This corresponds to an event that is 2/3 of the MCE, therefore resulting in even lower values. However, since the goal of this design is determine a design to resist all yielding, the MCE will be used for the duration of the analysis. A plot of both spectrums, along with the IBC 2006 MCE and DE spectrums can be seen in Figure 18 below.
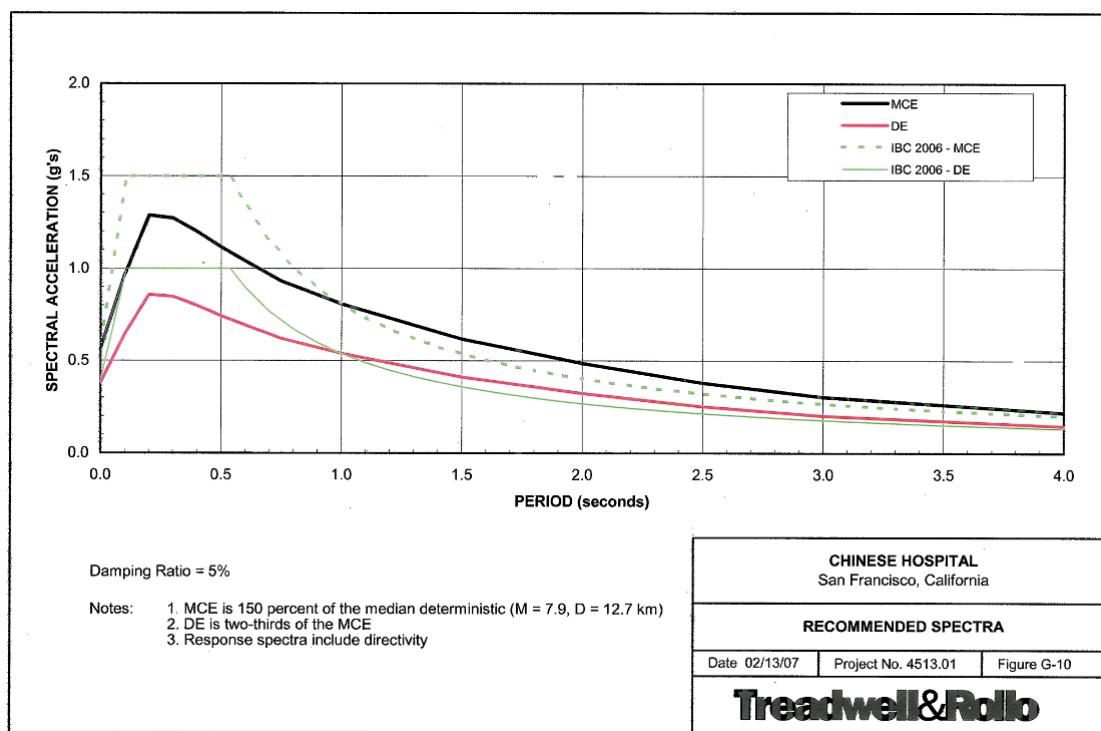
FIGURE 18: RESPONSE SPECTRUM DEVELOPED BY TREADWELL&ROLLO

The pushover curve previously developed was then converted into a plot of spectral acceleration vs displacement for the purpose of comparison with the SSRS. The point of intersection between the capacity curve and the MCE demand curve is known as the

performance point. This point is an indication of how the structure will behave as a whole, and can be used to convert the structural properties to a SDOF system.
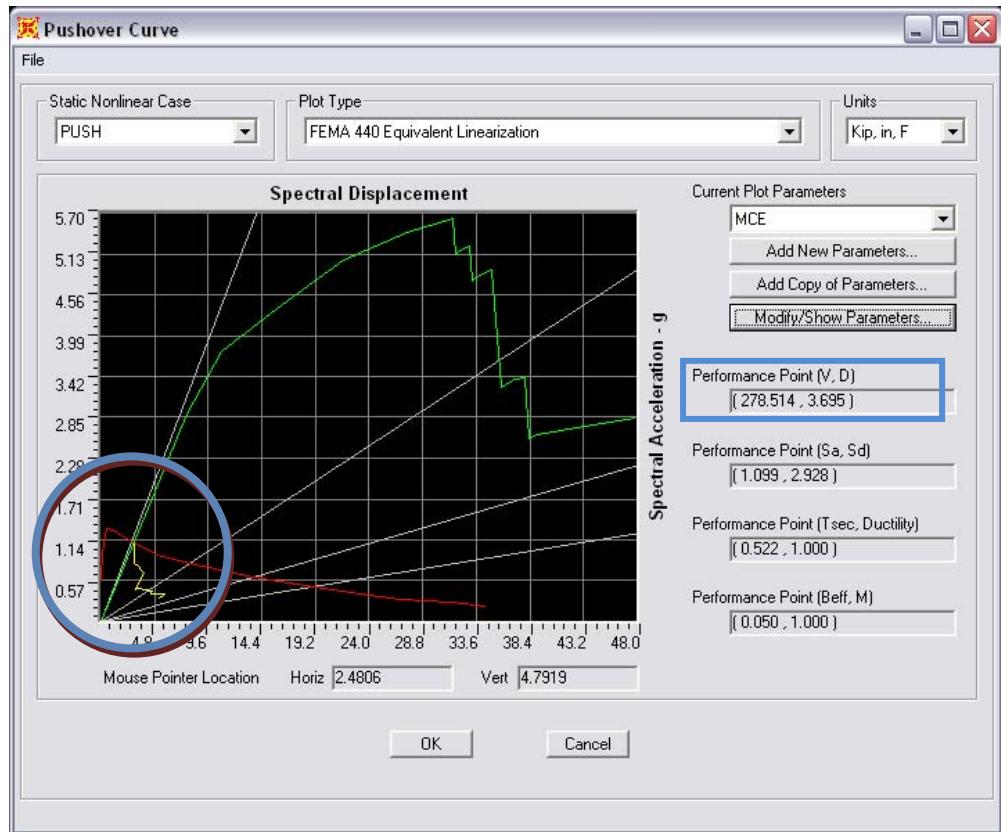
FIGURE 19: PUSHOVER CURVE SHOWING PERFORMACE POINT

As can be seen in Figure 19 above, the performance point can be taken as 278.514kips for base shear, and 3.695 in for maximum displacement. These values can then be used to determine frame strength and stiffness. The frame strength is simply the 278.514 kips. The stiffness would be 278.514kips/3.695in. This results in stiffness of 75.376 k/in. The secondary frame stiffness is 20% of the primary stiffness.

The frame weight was determined as being equivalent to the seismic weight, which is specified to be the total factored dead loads and live loads carried by each frame. For a SDOF system, this weight is assumed to be lumped at the top of the frame.

NONLIN SDOF ANALYSIS

In a nonlinear analysis, a time history model of a specific seismic event is used to test how a structure will behave given the velocity, acceleration, and ground movement of a real earthquake. Ideally speaking, the time-history selected should be taken from a location with similar distance, soil conditions, etc as the location of the building. Since this data was not available for the New Acute Care Hospital, data was taken from recorded time-histories of the Northridge earthquake. According to NEHRP Provisions §15.3.1.2, if at least seven time histories are used, then the average values of forces, displacements, and velocities may be used. However, if less than seven time histories are used, then the maximum values of the time histories must be used. In the case of this analysis, the values taken from the Arleta and Nordhoff Fire Station were used.

The Northridge Earthquake, which occurred January 17th, 1994, caused widespread damage to the Los Angeles area of CA. Although the magnitude of this earthquake was only a 6.7, the ground acceleration is known as being one of the highest recorded in North America. Due to the extensive amount of damage cause by the earthquake, the Northridge earthquake can be taken as a conservative ground motion for the San Francisco Bay area.

NONLIN32, which was used for the nonlinear analysis, is a program developed by Finley Charney at Virginia Tech for the dynamic analysis of SDOF structures. Although this program is typically used for academic purposes, it may be used in professional practice to determine the principals of how a structure will behave.

The nonlinear analysis was performed by inputting the SDOF properties of each frame into NONLIN. The earthquake was then run several different times, with various degrees of damping between 5% critical, which is assumed to be inherent to the structure, and 35% critical, which is the maximum allowed by NEHRP Provisions 15.2.4.2. In each run, the number of yield events incurred by the structure was determined. The point at which 0 yielding events occur was taken as the optimal amount of damping. For most of the frames, a percentage of damping in the high teens or low twenties was required to accomplish this. **Error! Reference source not found.** shows how regardless of frame strength (V), the optimal damping always ended up being around the same point.

The structure was then analyzed to determine if the members could be reduced in size, and therefore frame strength could be reduced as more of the load is absorbed by the damping. Beam and Column sizes were lowered in each frame. It was insured that the Demand to Capacity ratios were below 1 and that deflection and serviceability criteria were met. See Appendix D: Beam and Column Design for the complete set of calculations.

TABLE 10: ITERATIVE DAMPER ANALYSIS FOR FRAME E

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Optimal Damping Force (k)	Optimal Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	433.049	3.417	126.734	25.347	433.049	4	2	1	1	0	0	0	252.32	24
Smaller 1	412.267	3.366	122.480	24.496	412.267	4	2	1	1	0	0	0	255.57	25
Smaller 2	404.425	3.436	117.702	23.540	404.425	6	2	1	1	0	0	0	246.00	24
Smaller 3	390.408	3.508	111.291	22.258	390.408	5	3	1	1	0	0	0	234.29	23
Smaller 4	383.584	3.499	109.627	21.925	383.584	5	3	1	1	0	0	0	232.80	23
Smaller 5	372.695	3.506	106.302	21.260	372.695	6	2	1	1	0	0	0	223.05	22
Smaller 6	364.527	3.485	104.599	20.920	364.527	6	2	1	1	0	0	0	228.06	23

This became an iterative process of reducing the frame strength, and determining the amount of damping needed to prevent yielding. If the frame strength is plotted vs. the damping force required (see Figure 21), several interesting observations can be made.

First, as the frame strength is reduced, the damping force required is also reduced. This is because a reduction in frame strength also results in an increase in frame ductility. For the purposes of resisting seismic loads, extra ductility is a good thing.

Secondly, as frame strength is reduced, a point is eventually reached in which the damping force required begins to increase again. This is the point at which the frame has become overly ductile to the degree that extra damping is needed to prevent yielding.
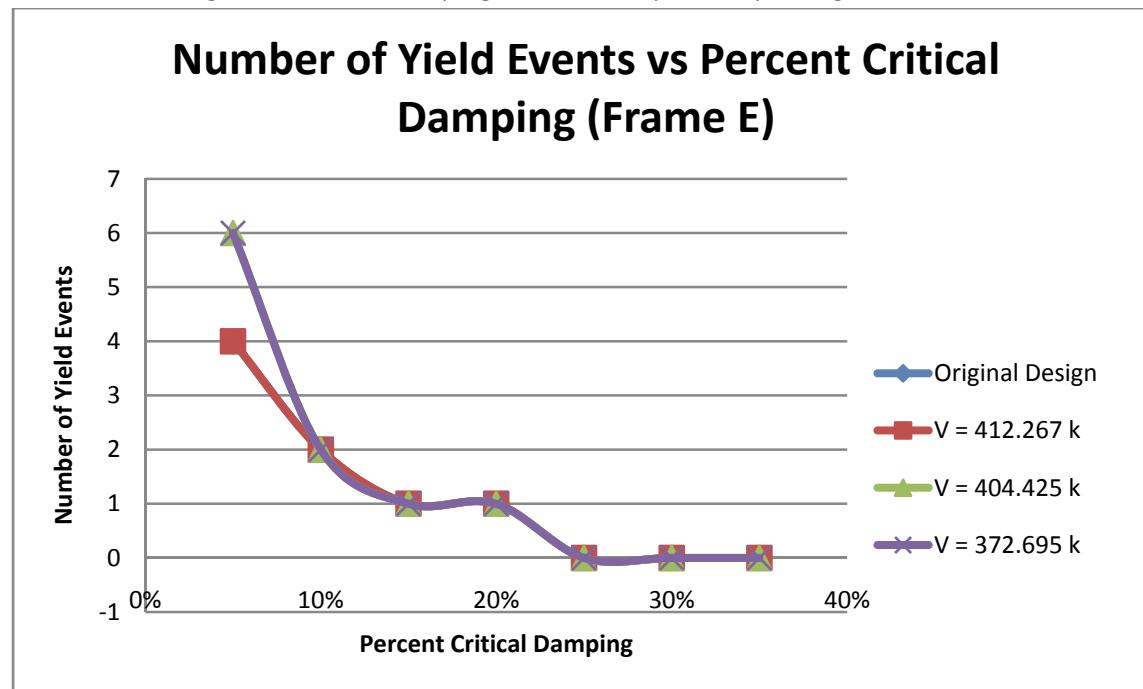
FIGURE 20: NUMBER OF YEILD EVENTS VS PERCENT DAMPING FOR FRAME E

Frame Strength vs Damping Force (Frame E)
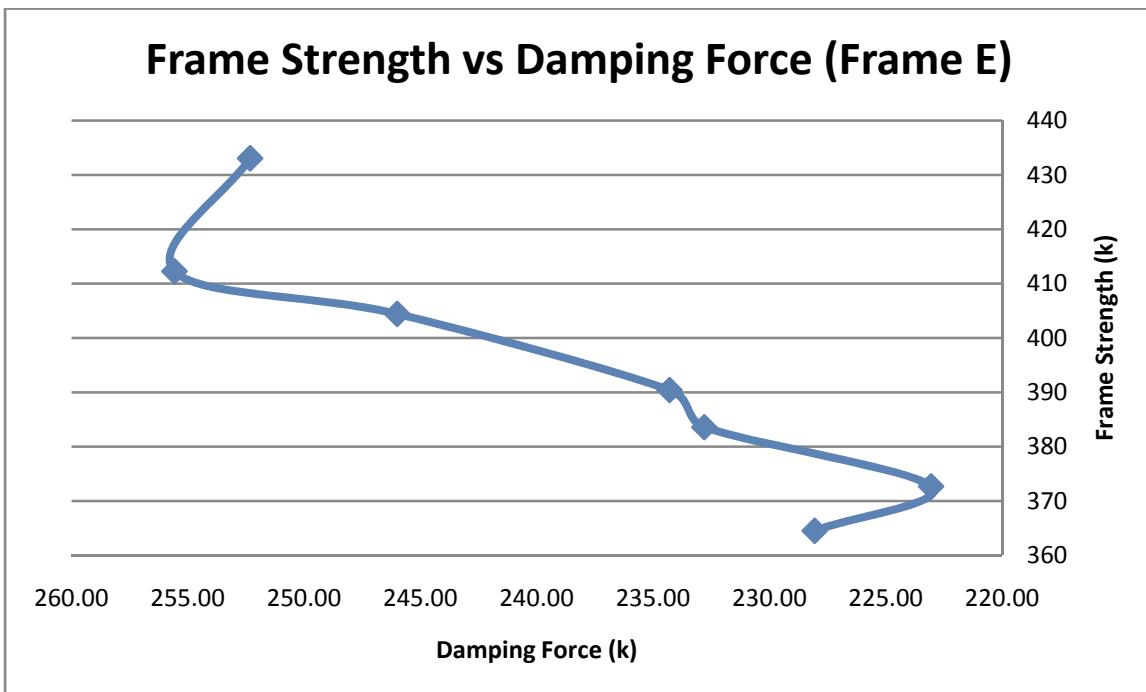
FIGURE 21: FRAME STRENTH VS DAMPING FORCE FOR FRAME E

A summary of the required amount of damping for each frame can be found in below. The set of calculations and figures can be found in Appendix E: Nonlinear Analysis

TABLE 11: DAMPING FORCE SUMMARY

Damping Force Summary		
Frame	Damping Force Required (kips)	Percent Critical Damping Required
1	133.003	18
7	120.726	24
A	172.404	21
E	223.054	22

Design of Damping Devices

The total amount of damping in each frame was then distributed throughout the frames. According to NEHRP Provisions §15.2.4.2, "in the direction of interest, the damping system has at least two damping devices in each story, configured to resist torsion…" This means that since a total of four frames (two in each direction) are being redesigned to include dampers, each frame must have at least one damper.

The fundamental equation used to describe the behavior of the dampers is

$$F = CV^\alpha$$

where: F is the damper force required for each individual device
C is the damping coefficient
V is the velocity across the damper
α is a velocity coefficient

With this in mind, the amount of damping force that each damper must carry is simply the total damping force needed by the frame (as determined by NONLIN) divided by eight stories.

The velocity across each damper device can be found using the response spectrum for the earthquake used in the analysis. Values for displacement, velocity, and acceleration are plotted against period on a tripartite graph (see Figure 22 below.) If the first mode period of the structure, 2.08 seconds, is drawn to the intersection of the 22% damped line, and then carried over to the Pseudo Velocity axis, then a value of 11in/sec can be read for the velocity across the dampers for the x direction (frames A and E). A similar process can be conducted for the z direction (frames 1 and 7) with a period of 1.93 seconds and resulting in a velocity of 11.5 in/sec.
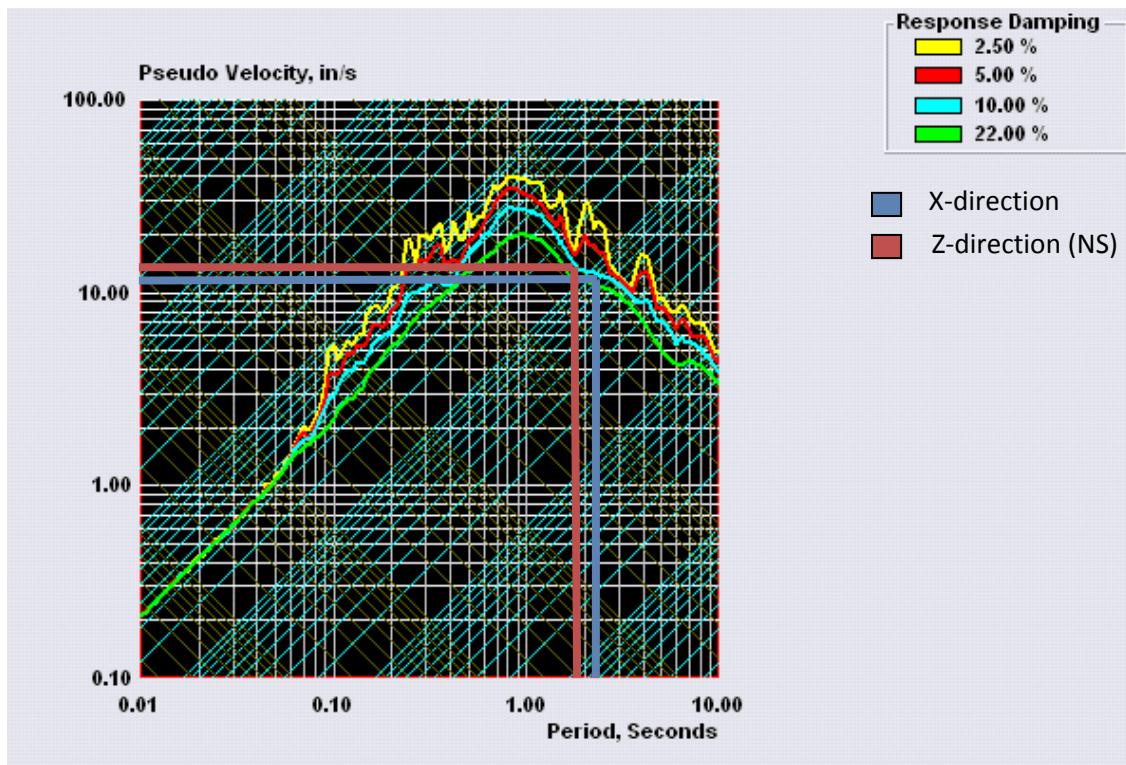
FIGURE 22: EARTHQUAKE RESPONSE SPECTRUM FOR NORTHRIDGE 1994

The velocity coefficient, α, can vary from 0.3-1.0. A value of 1.0 implies that the damper is acting completely linearly. A more economical design can be achieved by varying α. For the purposes of this initial design, α will be assumed to be 1.0. It is important to note that a constant exponent lower than 1.0 would result in larger requirements for the Damping Coefficient.

As can be seen above, the only variable that has not yet been determined is C, the damping coefficient. The damping coefficient is a product of the specific properties of the fluid within the damper. The required coefficient can be solved for using the remaining known factors of F, V, and α. A summary of the required damping coefficients is found in Table 12 below.

Specific product information on potential dampers which may be used for this application was obtained through the consultation of Craig Winters at Taylor Devices, Inc. Based on the required damping force per damper, each of the dampers fall under the 55 kip model category. (See Figure 23). As this figure shows, each device is approximately 28 inches when fully compressed, and 34 inches when fully extended. The maximum diameter of this device is 8 inches. The device uses inert silicone as the operating fluid.
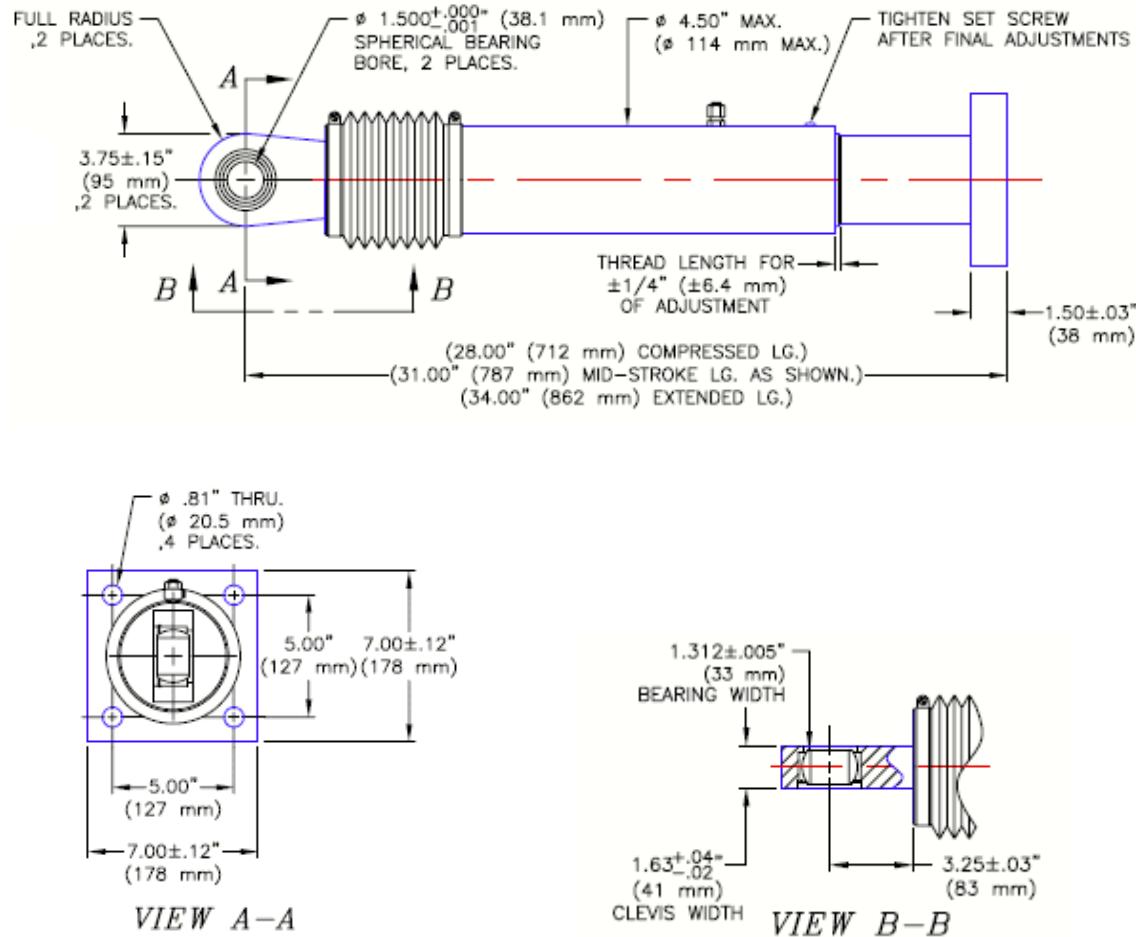
FIGURE 23: TAYLOR DEVICES 55 KIP FLUID VISCOUS DAMPER

TABLE 12: REQUIRED DAMPING COEFFICENT

Damping Coefficent F=CV$^\alpha$							
	Frame Damping Force (kips)	Indiviual Damper Force F (kips)	Damper Force Used (kips)	Velocity Across Damper V (in/s)	Constant Exponent α	Damping Coefficent C=F/V$^\alpha$ (k-sec/in)	Number Required
Frame A	172.40	21.55	55.00	12	1.0	4.58	8
Frame E	223.05	27.88	55.00	12	1.0	4.58	8
Frame 1	133.00	22.17	55.00	11	1.0	5.00	6
Frame 7	120.73	20.12	55.00	11	1.0	5.00	6

ARCHITECTURAL IMPACT

The current structural system of the New Acute Care Hospital is made of moment frames, which offer a considerable amount of freedom for the architecture. Since braces will be added to the exterior moment frames, there will be an impact to the façade of the structure for frames E, 1, 7 (See Figure 24, Figure 25 and Figure 26). In these figures, the damper locations are indicated by the red lines, whereas the blue rectangles indicate windows which will be affected by the dampers. In addition, the inclusion of dampers into the structure will have an impact on the spaces on the interior of the structure.

The impact of the dampers on the exterior frames (E, 1 , 7) can be solved fairly simply. In most cases, the use of chevron braced dampers eliminated any impact. See Figure 29. Although there were still isolated cases of braces blocking windows, the added performance of the damper warrants this disruption.

It should also be noted that no dampers were placed between the ground floor line and the 2nd floor line of grids 1 and 7 due to the regions on frame 1 which must remain open. The removal of these dampers has already been accounted for in Table 12 above.

The impact of the dampers on interior frame A is much more difficult to reconcile. Since the architectural layout is so tight in the current design, there are few places in which a diagonal brace can be placed which would not have a negative impact on the interior spaces. As Figure 27 shows, two of the bays on frame A are used as entrances to elevators (indicated by the orange line.) Of the three remaining bays, the two closest to the exterior of the structure would be easiest, as a door could be fit in the space underneath the top portion of the diagonal, whereas there is virtually no diagonal configuration that would be acceptable between columns 3A and 4A. However, these locations would be more likely to violate NEHRPs prescription of dampers being configured to resist torsion.

The other problem created by this design is that a dynamic soft story is created on the ground and first floor levels due to lack of dampers in those locations. This structural irregularity was resolved by closing off the last bay of the loggia and placing chevron braces in this location. See Figure 29.

The best solution is to place the dampers in between grids 3A and 4A as is shown in Figure 28. The diagonals of the dampers leave just enough room to fit a 7.5' doorway shown in Figure 27.

The results of this analysis show several things. In the case of the rehabilitation of an existing structure, it is possible to incorporate FVDs into moment frames without major disruption. However, it is very difficult to remove all disruption to an existing design. Therefore, FVD's should be incorporated into the design process as early as possible.
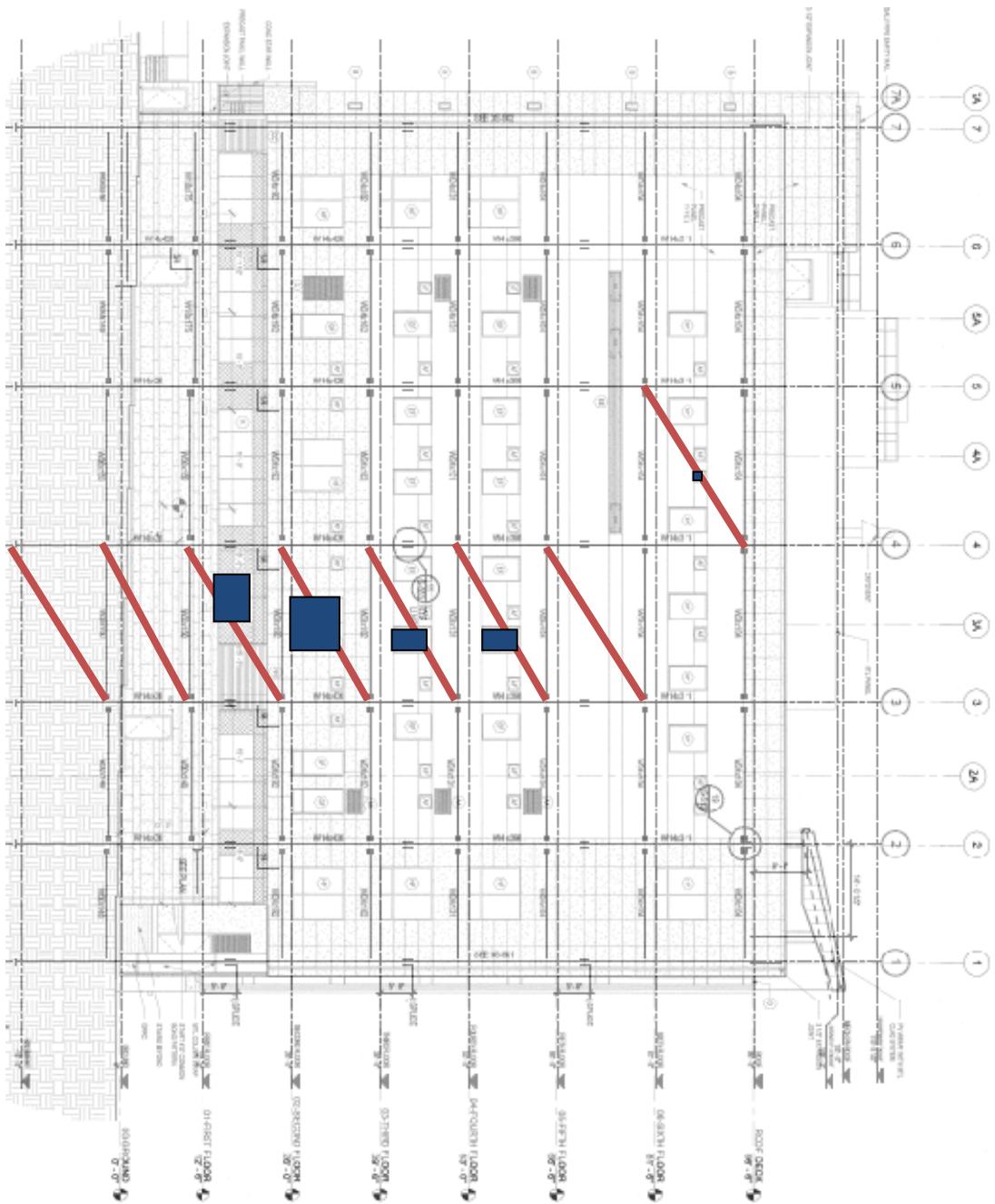
FIGURE 24: FRAME E DAMPER DEVICE IMPLEMENTATION AND IMPACT HIGHLIGHTED IN BLUE
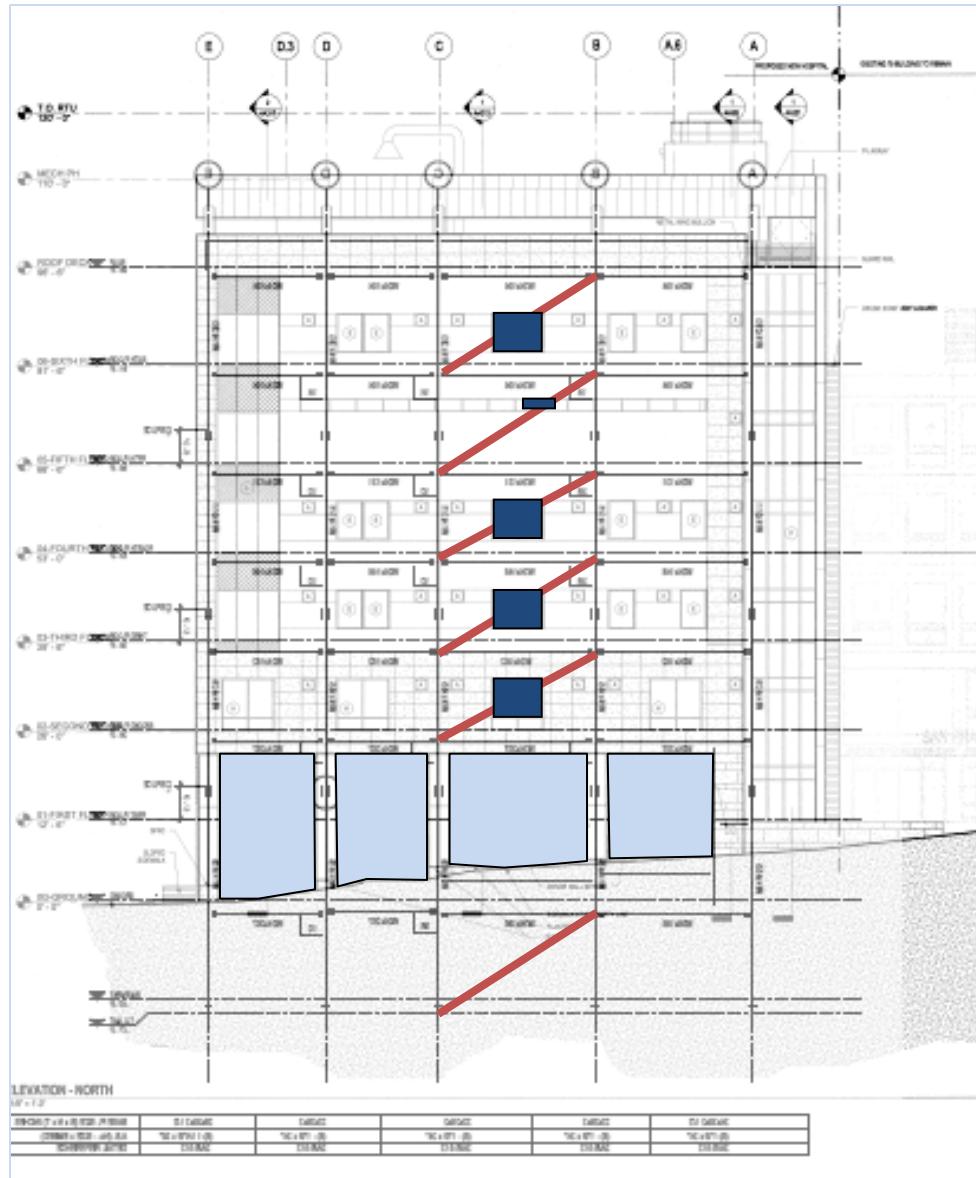
FIGURE 25: FRAME 1 SHOWING DAMPER IMPLEMENTATION AND IMPACT HIGHLIGHTED IN BLUE
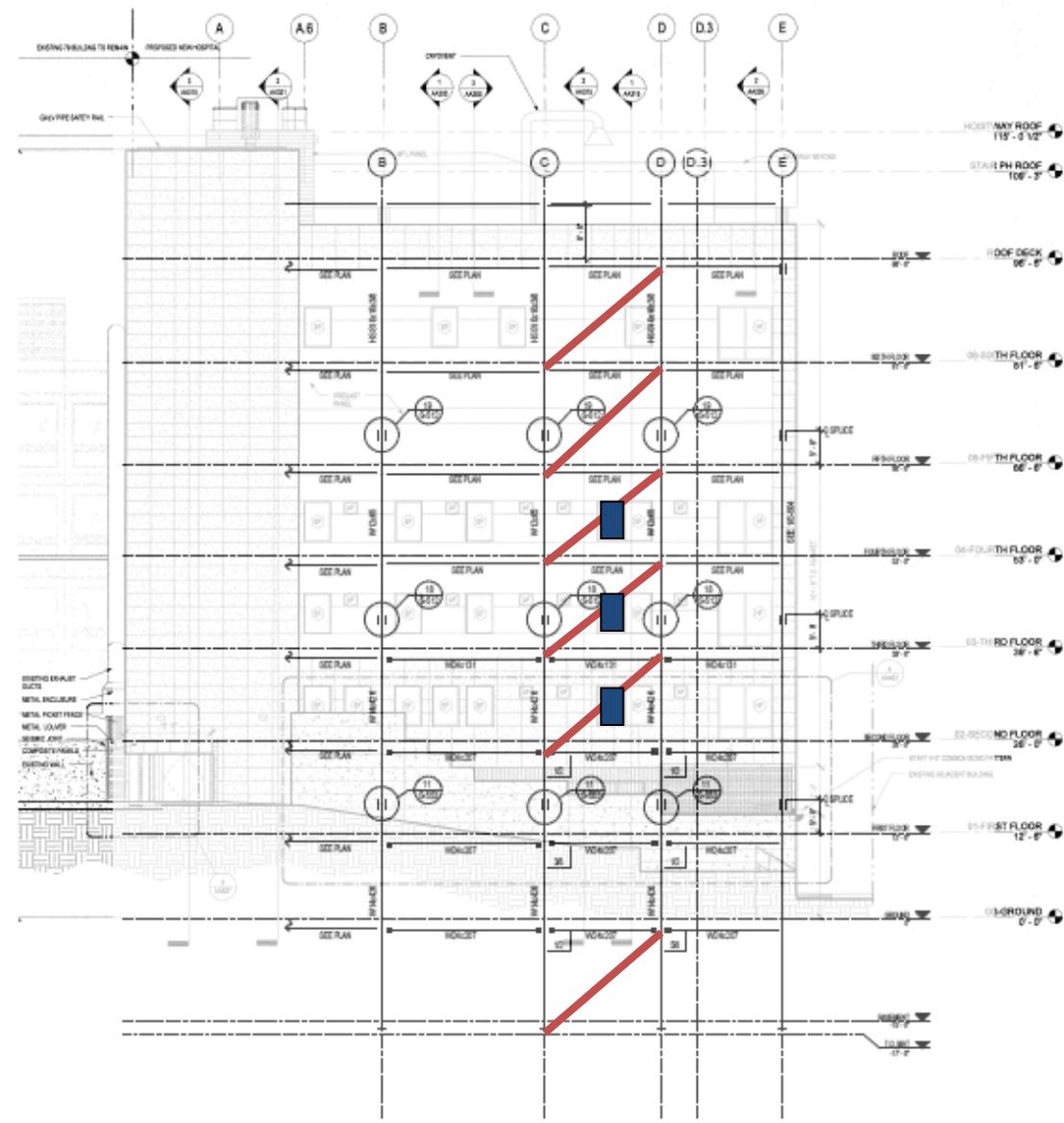
FIGURE 26: FRAME 7 SHOWING DAMPER IMPLEMENTATION AND IMPACT HIGHLIGHTED IN BLUE
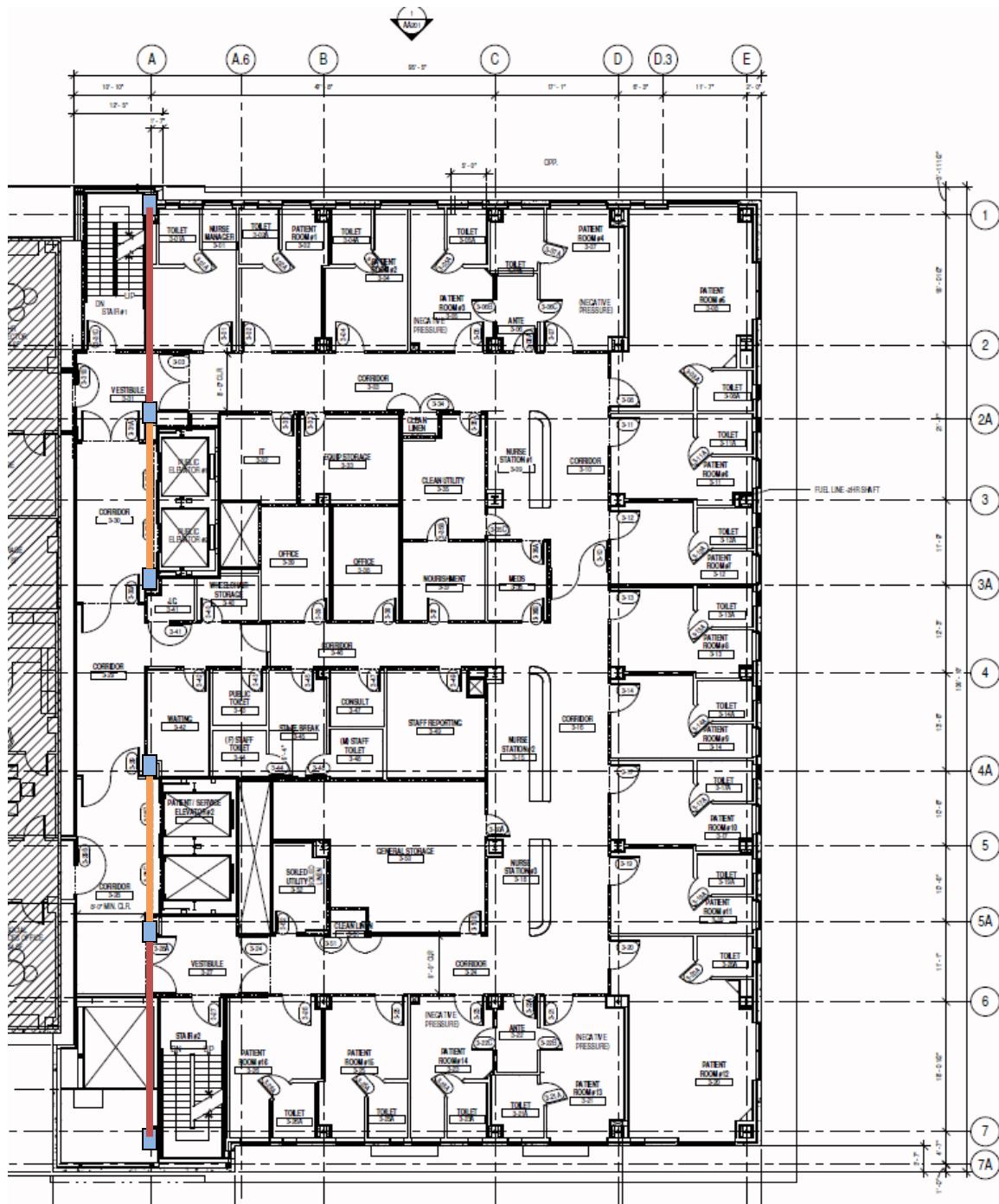
FIGURE 27: TYPICAL FLOORPLAN WITH COLUMNS IN BLUE, AREAS WHERE DAMPERS MAY NOT BE PLACED IN ORANGE, AND FINAL DAMPER PLACEMENT IN RED.
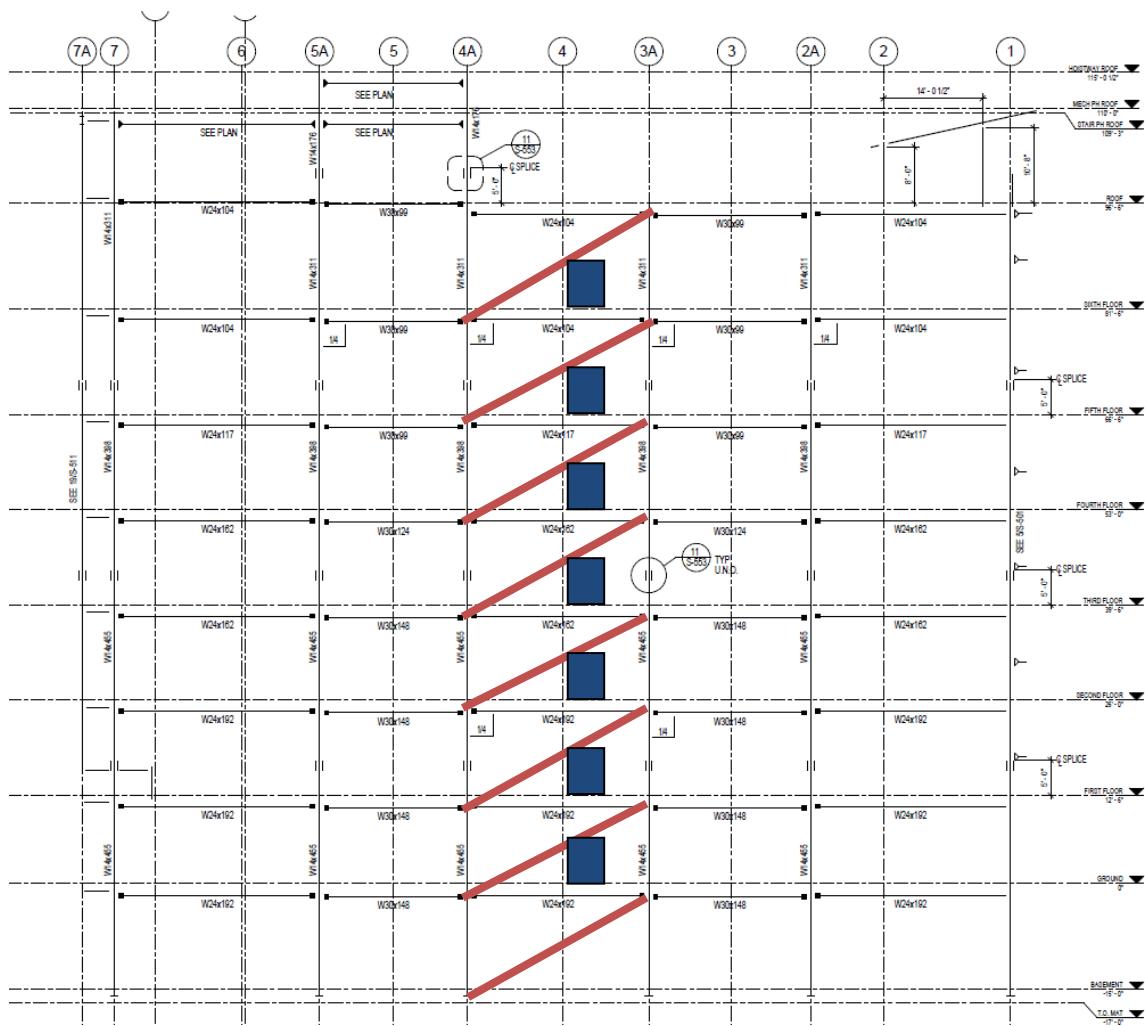
FIGURE 28: DAMPER IMPLEMENTATION IN FRAME A
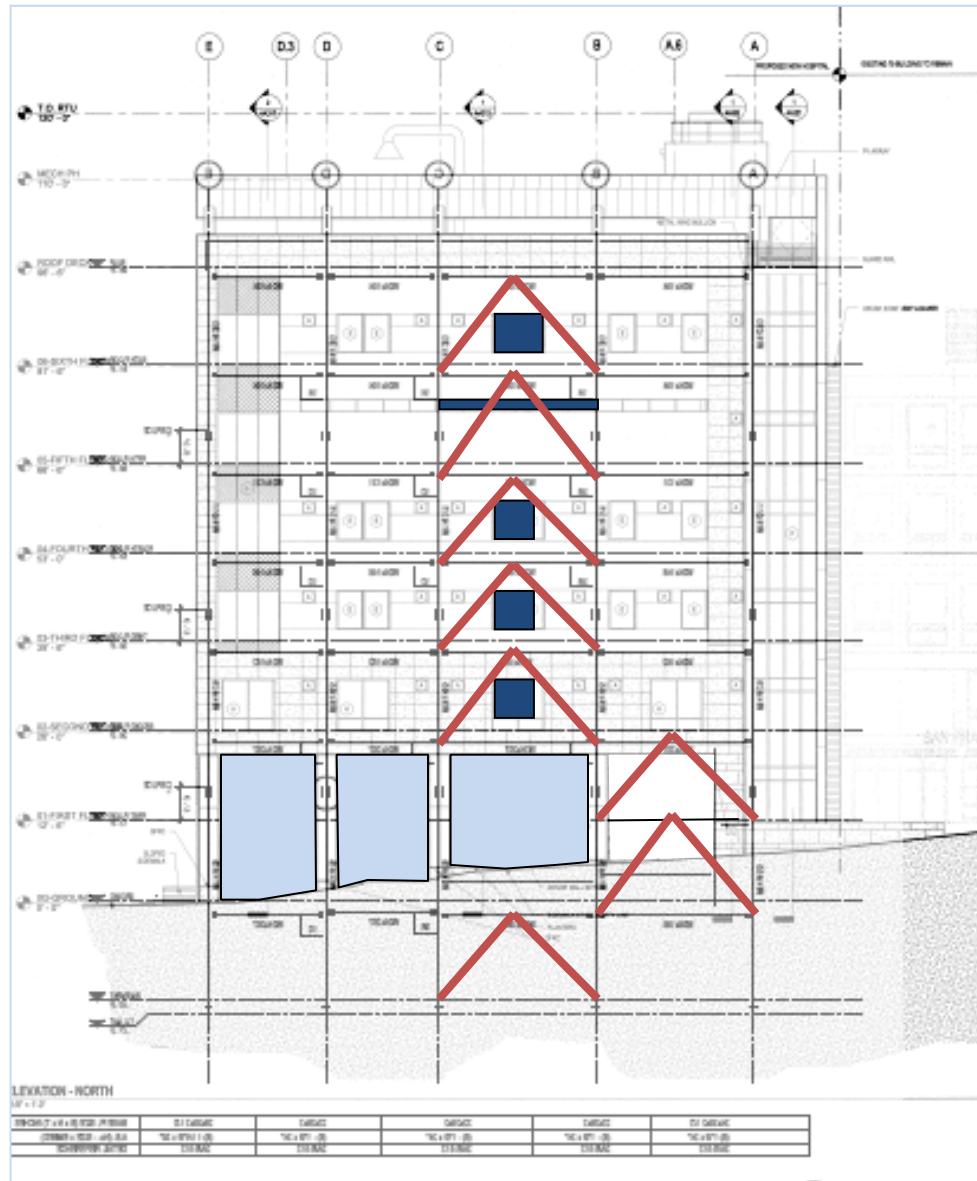
FIGURE 29: CORRECTION OF SOFT STORY EFFECT USING ADDITIONAL BRACES.

COST IMPACT

When the design of a structure for performance engineering is undertaken, it is always important to determine if the increase in performance warrants the increase in cost. In the case of this structure, there are several areas in which the cost can be assessed. Among these is the reduction in member sizes of the existing lateral system, cost of the damper devices themselves, the cost of potential structural repairs that can be avoided, and lastly any potential reduction in useable space due to the presence of the dampers.

The dampers, which were priced at $3600 per device, resulted in an increase in cost of $100,800. If the installation cost is estimated at 15% of the device cost ($540), then the total device cost will be $115920. This equates to an increase of less than 1% of the $160 million building cost.

TABLE 13: DAMPER COST SUMMARY

Damping Device Cost Summary					
	Constant Exponent α	Damping Coefficent $C=F/V^\alpha$ (k-	Number Required	Unit Cost ($)	Total Cost ($)
Frame A	0.5	6.22	8	3600	28800
Frame E	0.5	8.05	8	3600	28800
Frame 1	0.5	6.68	6	3600	21600
Frame 7	0.5	6.07	6	3600	21600
Total			28		100800

The inclusion of Fluid Viscous Dampers allowed the members in the structural system to be reduced. This reduction in size was most evident in both the columns and beams. Unit costs for steel were gathered from Cost Works and include both overhead and profit. See Table 14 below for a complete breakdown of savings due to weight. For a complete breakdown of the changes to the steel used in the lateral system, see Appendix D: Beam and Column Design.

TABLE 14: WEIGHT REDUCTION AND COSTS SAVINGS

Member Reductions					
Frame	Columns (kips)	Cost ($)	Beams (kips)	Cost ($)	Savings ($)
Frame 1	2.35		7.52		
Frame 7	18.95		6.11		
Frame A	3.91		0.00		
Frame E	18.71		28.36		
Totals	43.93	7939.89	41.99	11875.16	19815.05

The total cost of the addition of the dampers can be found by subtracting the cost savings of the steel reduction by the costs of the dampers. This results in a net increase of $96,095.

In addition to the immediate costs associated with the inclusion of the Fluid Viscous Dampers, there are also long term considerations to take into account. The real savings afforded by the dampers will not be evident until a major seismic event inevitable strikes the San Francisco Bay. In this situation, major repair and rehabilitation costs would be avoided. In addition, lives would be saved, both in terms of those in the hospital at the time of the earthquake, and those who are treated there afterwards. Due to the relatively low cost of the damper device implementation, it is clearly obvious that incorporation is worthwhile.

CONCLUSIONS

An investigation was undertaken into the design methodology and procedures for the implementation of Fluid Viscous Dampers into the structural system of the New Acute Care Hospital and Skilled Nursing Facility. It was found that this technology is useful for the prevention of plastic hinge formation in special steel moment frames.

The design of systems using FVDs involves several different types of analysis. The lateral system was first designed to remain elastic for static loading conditions. Next, the amount of damping required by each frame to resist yielding in a major seismic event was determined using a nonlinear analysis. This total amount of damping was then used to determine the number and capacity of FVDs needed.

In addition to the design of the dampers themselves, an investigation was also undertaken into the effects that of their use on the existing architectural layout, as well as the cost and schedule implications of their use.   It was determined that while FVD's could be incorporated into current design without major problems, a better practice would be to incorporate their use from the initial stages of design. In terms of cost, it was found that Fluid Viscous Dampers have marginal additional cost in comparison to the rest of the overall structure.

APPENDIX

APPENDIX A: LOAD CALCULATIONS

Wind Loads

JACOBS

Subject __Wind Analysis__   Project _____
_____   Sheet No. __1__ Of __3__
Authored by _____ Date _____   Checked by _____ Date _____

Use Method 2 since Building is not a Low-Rise and meets req of 6.5.1 & 6.5.2

○ Basic Wind Speed. ⟹ Using Table 6-1, $V = 85$ mph

○ Wind Directionality Factor ⟹ $K_d = 0.85$ ⟹ Using Table 6-4 (Buildings)

○ Occupancy Catagory = III (Health Care Facility) IBC 1604
  ↳ Importance Factor $I = 1.15$ (ASCE Table 6-1)

* Use Exposure C (Confirmed in Plans)

○ Topographic Factor

　　Nob Hill Elev = 315' ← Assume Bottom of Hill is Sea Level
　　Building Elev. = 110'

$$\frac{110}{315} = 0.349 \therefore \text{Not Top}\frac{1}{2} \text{ of Hill}$$

$$\therefore \text{Use } \underline{K_{zT} = 1.0}$$

○ Interpolate Table 6-3 to Find K_h & K_z at Building Elevations: See Excel Spreadsheet

● Velocity Pressures
　$q_z = 0.00256 \, K_z \, K_{zT} \, K_d \, V^2 \, I$ ⟩ See Excel for Graphs
　$q_h = 0.00256 \, K_h \, K_{zT} \, K_d \, V^2 \, I$

Gust Effect Factors

$$n_1 = \frac{75}{H} = \frac{75}{115} = 0.652 \text{ (Lower Bound } \begin{smallmatrix} \text{ASCE 7-05} \\ C6-17 \end{smallmatrix})$$

$$n_1 = \frac{100}{H} = \frac{100}{115} = 0.869 \text{ (Avg for } n_1 \begin{smallmatrix} \text{ASCE 7-05} \\ C6-18 \end{smallmatrix}) \; \boxed{\text{✗ Use for Calcs}}$$

In either case $n_1 \leq 1$ Hz ∴ Structure is Flexible

- $g_Q = g_v = 3.4$

- $g_R = \sqrt{2\ln(3600)n_1} + \dfrac{0.577}{\sqrt{2\ln(3600\,n_1)}}$

- $\bar{z} = 0.6h \geq z_{min}$

- $I_{\bar{z}} = c\left(\dfrac{33}{\bar{z}}\right)^{1/6}$

- $L_{\bar{z}} = \ell\left(\dfrac{\bar{z}}{33}\right)^{\bar{\epsilon}}$

- $Q = \sqrt{\dfrac{1}{1+0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}}$

- $\bar{V}_{\bar{z}} = \bar{b}\left(\dfrac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\dfrac{88}{60}\right)$

- $N_1 = \dfrac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}}$

- $R_n = \dfrac{7.47\,N_1}{(1+10.3N_1)^{5/3}}$

- $R_h = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$

- $\eta = 4.6n_1\,{}^h/\bar{V}_{\bar{z}}$

- $R_B = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$

- $\eta = 4.6n_1\,{}^B/\bar{V}_{\bar{z}}$

- $R_L = \dfrac{1}{\eta} - \dfrac{1}{2\eta^2}\left(1-e^{-2\eta}\right)$

- $\eta = 15.4n_1\,{}^L/\bar{V}_{\bar{z}}$

- $R = \sqrt{\dfrac{1}{B}R_n R_h R_B(0.53+0.47R_L)}$

$G_f = 0.925\left(\dfrac{1+1.7\,I_{\bar{z}}\sqrt{g_Q^2\,Q^2+g_R^2\,R^2}}{1+1.7g_v I_{\bar{z}}}\right)$

— SEE EXCEL SPREADSHEET
FOR CALCS

— SELECT VALUES CAN
BE FOUND IN ASCE7-05
TABLE 6-2.

APPROPRIATE VALUES
WERE CALCULATED
IN NS & EW DIRECTIONS.

- BUILDING IS FULLY ENCLOSED

- BUILDING HAS A PARAPET
  - PARAPET PRESSURES
    → DETERMINED USING k_z & q_z SPREADSHEET
    $$P_p = q_p q C_{pn}$$

- FIND EXTERNAL PRESSURE COEFFICIENTS C_p ⟹ SEE EXCEL SPREADSHEET
  - WALLS (USE ASCE FIG. 6-6)
    - INTERPOLATE L/B VALUES TO OBTAIN C_p
  - ROOF

$0 \to 0$ - USE FIG. 6-6

$h/2 \to 52.875'$ - RIDGE = 0°

$h = 105.75'$ - INTERPOLATE BETWEEN 1ˢᵗ C_p VALUES

$2h = 211.50'$ FOR EACH DISTANCE USING h/L

MEAN ROOF HEIGHT   50. FOR $0 \to h/2$

$$\frac{h}{L} \quad \frac{(0.78 - 0.5)}{(1 - .5)} = \frac{(x - -.9)}{(-1.3 - -.9)}$$

          - USE APPROPRIATE VALUES FOR
            NS & EW DIRECTIONS.
          - USE REDUCTION FACTORS AS
            APPROPRIATE VALUES INTERPOLATE

          - INTERNAL PRESSURE
            ± 0.18 FOR ENCLOSED STRUCTURES. (FIG 6-5)

<u>DESIGNED WIND PRESSURES</u>

WINDWARD WALLS $P_z = q_z q_f C_p - q_h (GC_{pi})$

LEEWARD WALLS
SIDE WALLS $\Big\}$ $P_h = q_h q_f C_p - q_h (GC_{pi})$
ROOFS

SEISMIC LOADS

EQUIVALENT LATERAL FORCE PROCEDURE

$S_s = 1.5 > 0.15$
$S_1 = 0.02 > 0.04$

↳ DETERMINED USING USGS WEBSITE

PENTHOUSE FLOOR SF = 2265 SF WHICH IS 18% OF A TYPICAL FLOOR. FLOORS < 20% TYPICAL FLOOR SF's CAN BE CONSIDERED A NON SUBSTANTIAL FLOOR & DISREGARDED
∴ $h = 111.5'$ = (FROM BASEMENT TO ROOF)

° A MAT FOUNDATION IS USED FOR SEISMIC DAMPING

° SITE CLASSIFIED AS SITE CLASS C ACCORDING TO GEOTEC REPORT.

USING TABLE 11.4-1   USING TABLE 11.4-2

$F_a = 1.0$ FOR $S_s > 1.25$ w/ SITE CLASS C $F_v = 1.3$ FOR $S_1 \geq 0.5$ w/ SITE CLASS C

∴ $S_{MS} = F_a S_s = \underline{1.5}$ ∴ $S_{M1} = F_v S_1 = 0.806$

$S_{DS} = 2 S_{ms}/3 = \underline{1}$ $S_{D1} = 2 S_{m1}/3 = \underline{0.537}$

SEISMIC DESIGN CATAGORY is IV (OCCUPANCY) BASED ON IBC TABLE 1604.5 (HOSPITAL / HEALTH CARE FACILITY)

CONDITIONS FOR USE OF SIMPLIFIED DESIGN PROCEDURE
1) ↗ 12.8.2.1 T_a IN EACH DIRECTION < $0.8 T_s$ ↗ 11.4.5

$T_a = C_t h_n^x$ (12.8-7) $T_s = \dfrac{S_{D1}}{S_{DS}} = \dfrac{0.537}{1} = 0.537$

$= (0.028)(111.5')^{0.8}$
$= 1.22$ SECS

OR

$T_a = 0.1 N$ (12.8-8)
$= 0.1(7) = 0.7$ SECS

$T_a > 0.8 T_s$ ∴ SIMPLIFIED DESIGN PROCEDURE MAY NOT BE USED.

SINCE ALL STEEL MOMENT FRAMES & < 12 STORIES

(T_a IS THE SAME IN EACH DIRECTION)

THE SDC IS DETERMINED AS THE MORE SEVERE OF SPC's GIVEN IN TABLES 11.6-1 & 11.6-2

11.6-1 ⟹ $S_{DS} = 1$ so SDC D

11.6-2 ⟹ $S_{D1} = 0.537$ so SDC D

[USE SEISMIC DESIGN CATAGORY D]

T WAS DETERMINED USING ETABS MODEL, GIVING

$\bar{T}_x = 2.04$ SECS

$\bar{T}_y = 1.89$ SECS   HOWEVER THE UPPER LIMIT THAT MAY BE

$\bar{T}_z = 1.29$ SECS   USED FOR T IS $C_U T_a = 1.75$

WHERE $C_U = 1.4$ (TABLE 12.8-1)

∴ USE $\boxed{T_a = 1.75 = T}$ THE HIGHER T_a VALUE WAS
                                    USED SINCE IT IS CLOSER
                                    TO THAT CALCULATED USING
$3.5 T_s = 1.879$ SECS $> T = 1.75$ SECS   ETABS ∴ CONSERVATIVE.

ACCORDING TO THE CRITERIA LISTED IN TABLE 12.3-1 (HORIZONTAL)
& TABLE 12.3-2 (VERTICAL IRREGULARITY), THE STRUCTURE CAN
BE CLASSIFIED AS REGULAR.

∴ $\boxed{\text{EQUIVALENT LATERAL FORCE METHOD CAN BE USED}}$

EQUIVALENT LATERAL FORCE METHOD

USING TABLE 12.2-1 FOR SPECIAL STEEL MOMENT FRAMES

$R = 8$ (RESPONSE MODIFICATION COEFFICIENT)

$\Omega_0 = 3$ (SYSTEM OVERSTRENGTH FACTOR)

$C_d = 5\frac{1}{2}$ (DEFLECTION AMPLIFICATION FACTOR) $\boxed{I = 1.5 \text{ USING} \\ \text{TABLE 11.5.1}}$

$T_L = 12$ SEC FROM TABLE 22-16 (SAN FRANCISCO)

$T = 1.75 < T_L$

ACCORDING TO 12.8.1.1 $C_s =$ SEISMIC RESPONSE COEFFICIENT

$$= \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{1}{\left(\frac{8}{1.5}\right)} = 0.1875 \quad \text{WHICH MUST BE} \\ \text{LESS THAN}$$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.537}{1.75\left(\frac{8}{1.5}\right)} = 0.058 \quad \text{(CONTROLS)}$$

ALSO, $S_1 = 0.62 g > 0.6 g$ SO

$$C_s \geq \frac{0.5 S_1}{\left(\frac{R}{I}\right)} = \frac{0.5(0.62)}{\left(\frac{8}{1.5}\right)} = 0.058 \quad \underline{GOOD}$$

THE TOTAL SEISMIC WEIGHT, CALCULATED FOR THE ENTIRE STRUCTURE
ACCORDING TO §12.7.2 IS

$W = 15,476,36^{lbs} \Rightarrow$ SAY $\underline{15.5}^{KIPS}$ INCLUDES PENTHOUSE WEIGHT

∴ SEISMIC BASE SHEAR $V = C_s W = 0.058(15.5) = \underline{0.897}^{KIPS}$

THE LATERAL SEISMIC FORCE INDUCED AT ANY LEVEL SHALL BE DETERMINED BY $F_x = C_{vx} V$ WHERE

$$C_{vx} = \frac{w_x h_x^k}{\Sigma w_i h_i^k}$$

$h_{i,x}$ = height of story from base

$$k \Rightarrow \frac{(1.75 - .5)}{(2.5 - .5)} = \frac{(k - 1)}{(2 - 1)} \quad \text{SEE } \S 12.8.3$$

$$k = 1.625$$

SINCE T_a ; WHICH IS INDEPENDENT OF FRAME DIRECTION WAS USED RATHER THAN T_β THIS ANALYSIS HOLDS FOR BOTH THE ND & EW DIRECTIONS.

Both Directions

T	1.516728745	
V_b	771.4919935	
k	1 T<.5	0.5
	1.508364373 .5 <T <2.5	
	2 T>2.5	2.5

Floor	h_i (ft)	h (ft)	w (kips)	wh^k	$C_{vx} = (w_x h_x^k)/(\Sigma w_i h_i^k)$	Story Force $F_i = C_{vx} V$ (kips)	Story Shear $V_j = \Sigma F_i$ (kips)	By (ft)	5%By (ft)	Ax	M_z (ft-k)	A_y	$M_z A_y$ (ft-k)
7	15	111.5	1945.12	2382226	0.279121049	215.34	0.00	135	6.75	1.0	1453.54	1.0	1453.54
6	13.5	96.5	1839.94	1812149	0.212326243	163.81	215.34	135	6.75	1.0	13268.45	1.0	13268.45
5	13.5	83	1850.11	1451666	0.170089025	131.22	379.15	135	6.75	1.0	10629.01	1.0	10629.01
4	13.5	69.5	1840.6	1104953	0.129465302	99.88	510.37	135	6.75	1.0	8090.40	1.0	8090.40
3	13.5	56	1865.87	808697.2	0.094753582	73.10	610.25	135	6.75	1.0	5921.23	1.0	5921.23
2	12.5	42.5	1907.14	545238.4	0.063884594	49.29	683.35	135	6.75	1.0	3992.20	1.0	3992.20
1	15	30	1881.67	318112.4	0.037272649	28.76	732.64	135	6.75	1.0	2329.20	1.0	2329.20
Ground	15	15	1879.64	111698.9	0.013087556	10.10	761.40	135	6.75	1.0	817.85	1.0	817.85
Basement	0	0	0	0	0	0.00	771.49	135	6.75	1.0	0.00	1.0	0.00
				8534740		556.15234					45048.34 k-ft	3754.02829 k-in	

Relative Stiffnesses of Lateral Elements - Basement		
Moment Frame	Drift per 1^k load (Δ_p) (in)	k=P/Δ_p (k/in.)
Grid Line 1	0.00341	293.26
Grid Line 2	0.00492	203.25
Grid Line 6	0.011	90.91
Grid Line 7	0.00512	195.31
Grid Line A	0.00338	295.86
Grid Line E	0.0035	283.29
Basement - long	4.46E-06	224064.5
Basement - short	6.44E-06	155231.3

Center of Rigidity - Basement				
X Direction	$k_{iy\,(k/in)}$	$x_{i\,(in)}$	$k_{iy}x_{i\,(k)}$	
MF A	295.86	122	36094.675	
MF E	283.29	1113	315297.45	
W_A	224064.53	0	0	
W_B	224064.53	1141.25	255713646	
SUM	448708.21		256065038	
x = $\Sigma k_{iy}x_i/\Sigma k_{iy}$		(in)		570.67
Y Direction	$k_{iy\,(k/in)}$	$x_{i\,(in)}$	$k_{iy}x_{i\,(k)}$	
MF 1	293.26	1550.75	454765.4	
MF 2	203.25	1336	271544.72	
MF6	90.91	244.5	22227.273	
MF7	195.31	27	5273.4375	
W_1	155231.29	1583.5	245808755	
W_2	155231.29	0	0	
SUM	311245.32		246562566	
y = $\Sigma k_{iy}x_i/\Sigma k_{iy}$		(in)		792.18

$\bar{y}$

47'-6¾"

131'-11½"   129'-2¾"

111'-4"

Center of Rigidity

Center of Mass

$\bar{x}$ 66'-0¼"

20'-4½"

y

2'-3"

x

10'-2"

92'-9"

95'-1¼"

Basement Lateral System

Center of Rigidity - Handcalcs

Center of Mass

ETABs Center of Rigidity

Direct Force - Basement			
Moment Frame	$k=P/\Delta_p$ (k/in.)	Σk_i (Basement)	$F_{iy}=(k_{iy}/\Sigma k_{jy})P_y$
Grid Line 1	293.26		0.000942815
Grid Line 6	90.91		0.000292273
Grid Line 7	195.31	311042	0.00062793
Grid Line 2	203.25		0.000653455
Basement - short (2)	155231.29		0.499068491
Grid Line A	295.86		0.000659355
Grid Line E	283.29	448708	0.000631337
Basement - long (2)	224064.53		0.499354654

Force due to eccentricity - Basement						
Moment Frame	$k=P/\Delta_p$ (k/in.)	d_i	$k_i d_i^2$ (Basement)	e_i	$\Sigma k_i d_i^2$ (Basement)	$F_{it}=(k_i d_i P_y e_x)/\Sigma k_j d_j^2$
Grid Line 1	293.26	652.50	124855205.28			0.027178576
Grid Line 2	203.25	437.50	38903709.35			0.012630295
Grid Line 6	90.91	654.25	38913005.68	104	728684941	0.008447955
Grid Line 7	195.31	871.50	148342236.33			0.024176754
Basement - short high	295.86	685.00	138823964.50			0.028785543
Basement - short low	295.86	898.5	238846819.53			0.037757387
Grid Line A	295.86	511.25	77330343.93			0
Grid Line E	283.29	479.75	65201150.8	0	102450752932	0
Basement - long west	155231.29	633.25	62248612620.3			0
Basement - long east	155231.29	508.00	40059608817.1			0

$F_i=F_{idirect} \pm F_{itorsion}$ Basement					
Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.0009	-0.0272	-0.0262	-2.6236	
Grid Line 2	0.0003	-0.0126	-0.0123	-1.2338	
Grid Line 6	0.0006	0.0084	0.0091	0.9076	100.24
Grid Line 7	0.0007	0.0242	0.0248	2.4830	
Basement - short high	0.4991	-0.0288	0.4703	47.0283	
Basement - short low	0.4991	0.0378	0.5368	53.6826	
Grid Line A	0.0007	0.0000	0.0007	0.0659	
Grid Line E	0.0006	0.0000	0.0006	0.0631	100.00
Basement - long west	0.4994	0.0000	0.4994	49.9355	
Basement - long east	0.4994	0.0000	0.4994	49.9355	

Relative Stiffnesses of Lateral Elements -Ground - 3rd		
Moment Frame	Drift per 1^k load (Δ_p) (in)	$k=P/\Delta_p$ (k/in.)
Grid Line 1	0.01437	69.59
Grid Line 2	0.0133	75.19
Grid Line 6	0.0355	28.17
Grid Line 7	0.01507	66.36
Grid Line A	0.00959	104.28
Grid Line E	0.0094	106.84
Basement - long		
Basement - short		

Center of Rigidity - Floors Ground -3				
X Direction	$k_{iy\ (k/in)}$	$x_{i\ (in)}$	$k_{iy}x_{i\ (k)}$	
MF A	104.28	122	12721.58	
MF E	106.84	1113	118910.3	
SUM	211.11		131631.8	
$x = \Sigma k_{ix}x_i/\Sigma k_{ix}$		(in)		623.51
Y Direction	$k_{iy\ (k/in)}$	$x_{i\ (in)}$	$k_{iy}x_{i\ (k)}$	
MF 1	69.59	1550.75	107915.8	
MF 2	75.19	1336	100451.1	
MF6	28.17	244.5	6887.324	
MF7	66.36	27	1791.639	
SUM	239.30		217045.9	
$y = \Sigma k_{iy}x_i/\Sigma k_{iy}$		(in)		906.99

Floors Ground - 3 Lateral System

Center of Rigidity - Handcalcs

Center of Mass

ETABs Center of Rigidity

Direct Force Floors Ground-3			
Moment Frame	$k=P/\Delta_p$ (k/in.)	Σk_i (Floors 1-3))	$F_{iy}=(k_{iy}/\Sigma k_{jy})P_y$
Grid Line 1	69.59		0.2908
Grid Line 6	28.17		0.1177
Grid Line 7	66.36	239.30	0.2773
Grid Line 2	75.19		0.3142
Grid Line A	104.28		0.4939
Grid Line E	106.84	211.11	0.5061

Force due to eccentricity - Floors Ground-3						
Moment Frame	$k=P/\Delta_p$ (k/in.)	d_i	$k_i d_i^2$ (Floors 1-3))	e_i	$\Sigma k_i d_i^2$ (Floors 1-3))	$F_{it}=(k_i d_i P_y e_x)/\Sigma k_j d_j^2$
Grid Line 1	69.59	652.50	29628132			0.0460
Grid Line 6	28.17	654.25	12057551			0.0187
Grid Line 7	66.36	871.50	50398955	107.75	106476084.8	0.0585
Grid Line 2	75.19	437.50	14391447			0.0333
Grid Line A	104.28	511.25	27255116	62.50	51844866.3	0.0643
Grid Line E	106.84	479.75	24589750			0.0618

$F_i=F_{idirect} \pm F_{itorsion}$ Ground Floor - 3rd Floor					
Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.2908	0.0460	0.3368	33.6750	
Grid Line 6	0.1177	-0.0187	0.0991	9.9062	
Grid Line 7	0.2773	-0.0585	0.2188	21.8770	100.21
Grid Line 2	0.3142	0.0333	0.3475	34.7483	
Grid Line A	0.4939	0.0643	0.5582	55.8199	
Grid Line E	0.5061	-0.0618	0.4443	44.4279	100.25

Relative Stiffnesses of Lateral Elements - Floors 4 - Roof		
Moment Frame	Drift per 1^k load (Δ_p) (in)	$k=P/\Delta_p$ (k/in.)
Grid Line 1	0.03121	32.04
Grid Line 6	0.0841	11.89
Grid Line 7	0.03539	28.26
Grid Line A	0.02289	43.69
Grid Line E	0.0217	46.04

Center of Rigidity - Floors 4 - Roof				
X Direction	$k_{iy\ (k/in)}$	$x_{i\ (in)}$	$k_{iy}x_{i\ (k)}$	
MF A	43.69	122	5329.83836	
MF E	46.04	1113	51243.0939	
SUM	89.73		56572.9323	
$x = \Sigma k_{ix}x_i/\Sigma k_{ix}$		(in)		630.50
Y Direction	$k_{iy\ (k/in)}$	$x_{i\ (in)}$	$k_{iy}x_{i\ (k)}$	
MF 1	32.04	1550.75	49687.6001	
MF6	11.89	244.5	2907.25327	
MF7	28.26	27	762.927381	
SUM	72.19		53357.7808	
$y = \Sigma k_{iy}x_i/\Sigma k_{iy}$		(in)		739.15
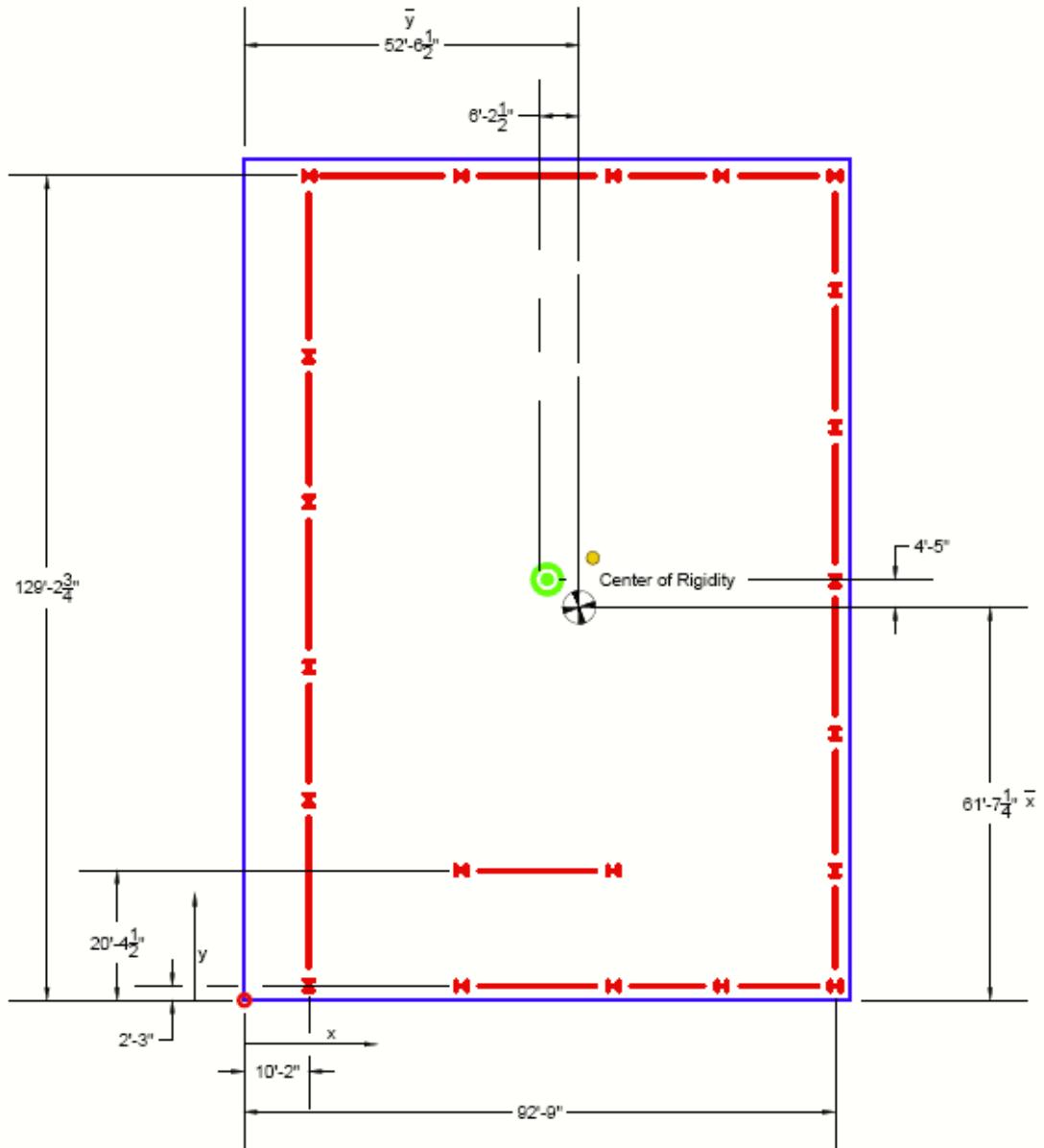
Floors 4-7 Lateral System

Center of Rigidity - Handcalcs

Center of Mass

ETABs Center of Rigidity

Direct Force Floors 4-Roof

Moment Frame	$k=P/\Delta_p$ (k/in.)	Σki (4-Roof)	$F_{iy}=(k_{iy}/\Sigma k_{jy})P_y$
Grid Line 1	32.04		0.4439
Grid Line 6	11.89	72.19	0.1647
Grid Line 7	28.26		0.3914
Grid Line A	43.69	89.73	0.4869
Grid Line E	46.04		0.5131

Force due to eccentricity - Floors 4-Roof

Moment Frame	$k=P/\Delta_p$ (k/in.)	d_i	$k_i d_i^2$ (Floors 4-7))	e_i	$\Sigma k_i d_i^2$ (Floors 4-7))	$F_{it}=(k_i d_i P_y e_x)/\Sigma k_j d_j^2$
Grid Line 1	37.45	652.50	15945927			0.01875
Grid Line 6	11.89	654.25	5089692	35.00	45615303.0	0.00597
Grid Line 7	32.36	871.50	24579684			0.02164
Grid Line A	54.95	511.25	14361350	81.25	28061353.3	0.08133
Grid Line E	59.52	479.75	13700004			0.08268

$F_i=F_{idirect} \pm F_{itorsion}$ 4th Floor - Roof

Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.4439	0.0188	0.4626	46.2605	
Grid Line 6	0.1647	-0.0060	0.1587	15.8748	99.11
Grid Line 7	0.3914	-0.0216	0.3698	36.9789	
Grid Line A	0.4869	0.0813	0.5682	56.8221	99.87
Grid Line E	0.5131	-0.0827	0.4304	43.0430	

Grid 1 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	46%	99.617
6	163.81	46%	75.778
5	131.22	46%	60.704
4	99.88	46%	46.206
3	73.10	34%	24.617
2	49.29	34%	16.597
1	28.76	34%	9.683
Ground	10.10	34%	3.400
Basement	0.00	-3%	0.000

Grid 7 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	37%	79.630
6	163.81	37%	60.574
5	131.22	37%	48.525
4	99.88	37%	36.935
3	73.10	22%	15.992
2	49.29	22%	10.782
1	28.76	22%	6.291
Ground	10.10	22%	2.209
Basement	0.00	2%	0.000

Grid 2 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	0%	0.000
6	163.81	0%	0.000
5	131.22	0%	0.000
4	99.88	0%	0.000
3	73.10	35%	25.402
2	49.29	35%	17.126
1	28.76	35%	9.992
Ground	10.10	35%	3.509
Basement	0.00	-1%	0.000

Grid A Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	57%	122.361
6	163.81	57%	93.079
5	131.22	57%	74.563
4	99.88	57%	56.755
3	73.10	56%	40.805
2	49.29	56%	27.512
1	28.76	56%	16.051
Ground	10.10	56%	5.636
Basement	0.00	0%	0.000

Grid 6 Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	16%	34.185
6	163.81	16%	26.004
5	131.22	16%	20.831
4	99.88	16%	15.856
3	73.10	10%	7.242
2	49.29	10%	4.882
1	28.76	10%	2.849
Ground	10.10	10%	1.000
Basement	0.00	1%	0.000

Grid E Moment Frames			
Floor	$F_i=C_{vx}V$	$F_i\%$	$F_{i\,applied}$ (kips)
7	215.34	43%	92.689
6	163.81	43%	70.508
5	131.22	43%	56.482
4	99.88	43%	42.992
3	73.10	44%	32.478
2	49.29	44%	21.897
1	28.76	44%	12.775
Ground	10.10	44%	4.486
Basement	0.00	0%	0.000

2D Model

- MOMENT FRAME ELEVATION 1
  - RBS MOMENT CONNECTIONS USED.

  { PORTIONS OF THE BEAM FLANGE
    ARE SELECTIVELY TRIMMED IN
    REGION ADJACENT TO
    COLUMN BEAM CONNECTION
    TO ENSURE YIELDING & HINGE
    FORMATION w/in REDUCED
    AREA }

  - NO MOMENT CONNECTIONS @ 1st FLOOR
    LEVEL.

  - DETERMINATION OF Δ_p USING 2D MODELING
    - ~~IGNORE BASEMENT~~
    - USE EXPLICIT PANEL ZONE MODELING TO ACCOUNT FOR
      PANEL ZONE DEFORMATIONS
        (REQ. BY | ASCE 7 §12.7.3b |)

    - USE CONSTRAINTS TYPE "EQUAL" AT EACH LEVEL FOR X TRANS
    - MAT STRENGTHS
        STEEL
        CONCRETE
    - BASEMENT WALLS EXIST @ BASEMENT LEVEL ONLY
        - PORTIONS OF CONCRETE WALLS ON 1st GROUND FLOOR
          IGNORED
        - MODELED AS "THICK SHELL" w/ MEMBRANE $t = 18"$
                                          BENDING $t = 1.8"$
        - "JOGS" IN WALL WERE
          SIMPLIFIED TO CREATE A RECTANGLE.

- ASSUME CENTER OF MASS LOCATED @ GEOMETRIC
  CENTER OF STRUCTURE

3D Model

<u>LATERAL SYSTEM MODELING</u>

• MATERIAL PROPERTIES (MEMBERS MODELED w/ 0 MASS TO CONTROL MASS
    • CONCRETE $f'_c = 4ksi$ $E = 3,600,000 psi$ $\nu = 0.2$ LOCATION)

    • STEEL $F_y = 50 ksi$ $E = 29,000,000 psi$ $\nu = 0.3$

• FRAME SECTIONS
    • SHAPE SECTIONS TAKEN FROM ETABS DATABASE & AISC 13 PROPERTIES.

• BASEMENT WALL SECTIONS
    • "MEMBRANE" ELEMENT → CONCRETE MATERIAL → MEMBRANE 18"
                                 BENDING " 18"
• MODEL GEOMETRY.   → WALL I_g "CRACKED" TO $0.7 I_g$
                                           ACI 318.08 §10.10.4.1
    • FLOORPLAN SIMPLIFIED TO A RECTANGULAR SHAPE OF DIMENSIONS
         82'-7" × 131'-11.5"

    • FLOORS   MODELED   PENTHOUSE SF = 2265 SF ⟹ 18% SF FOR
         BASEMENT (-15'-0")   TYPICAL FLOOR
         ROOF   (96'-6")   • ∴ CAN BE IGNORED

       TOTAL HEIGHT (111'-6")

    • COLUMN SPLICES LOCATED 5' ABOVE FLOORS 1, 3, & 5

    • COLUMN ORIENTATIONS ADJUSTED APPROPRIATELY.

    • INSERTION POINTS WERE DISREGARDED AT THIS STAGE OF
     ANALYSIS

    • JOINT RESTRAINTS @ BASE MODELED AS <u>PINNED</u>. THIS IS A
     CONSERVATIVE APPROXIMATION OF COLUMN/BASE FIXITY. SEE NEHRP pg 18.

    • BASEMENT WALLS MESHED w/ MAX SIZE = 24" & RESTRAINTS WERE
     MODELED ON EDGES.

    • FLOOR DIAPHRAGMS

         • GROUND FLOOR DIAPHRAGM.
            • 6.25" CONCRETE MEMBRANE, MESHED @ 48"
            • HELPS TO ACCOUNT FOR SHEAR REVERSAL EFFECT.
        • OTHER FLOORS
           • RIGID DIAPHRAGM.

    • EXPLICIT PANEL ZONE MODELING USED @ ALL BEAM-COLUMN
     CONNECTIONS TO ACCOUNT FOR "SOFTENING" OF THESE AREAS
     DUE TO LOCALIZED STRESSES (REQ'D BY ASCE 7, §12.7.3b)
         • MODELED USING ELASTIC PROPERTIES FROM COLUMN.

    • BEAMS WERE MODELED AS REDUCED BEAM SECTION
     USING PROGRAM DEFAULTS FOR CUTOUTS.

- ADDITIONAL MASS PER FLOOR

  - $\dfrac{1879.64^{KIPS}}{12,325\,SF} = 0.1525^{KSF}\left(\dfrac{1}{32.2^{ft/sec^2}}\right)\left(\dfrac{1'}{(12''\cdot 1')^3}\right) = 2.74 \times 10^{-6}$

  - ADDITIONAL MASS ADDED TO DIAPHRAGMS.

Appendix D: Beam and Column Design

Frame 1: Beam Design

Floor	L (in)	Original Size	Weight (lbs/ft)	S (in³)	M_{max} (k-in)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in³)	Ratio	w (k/in)	I (in⁴)	Δ_{max} (in)	$\Delta_{allowed}$ (in)
Frame 1 Beams															
1	286	W36	150	504	17072	0.678	510	W36	135	439	0.778	0.158	6260	0.015	0.715
	286	W36	150	504	12807	0.509	382	W36	135	439	0.583	0.158	6260	0.015	0.715
	205	W24	207	531	12033	0.453	359	W24	192	491	0.490	0.158	6260	0.004	0.513
	214	W24	207	531	15477	0.583	462	W24	192	491		0.158	6260	0.005	0.535
2															
3	286	W24	207	531	10583	0.399	316	W24	176	450	0.470	0.158	5680	0.017	0.715
	286	W24	207	531	8940	0.337	267	W24	176	450	0.397	0.158	5680	0.017	0.715
	205	W24	207	531	9443	0.356	282	W24	176	450	0.420	0.158	5680	0.004	0.513
	214	W24	207	531	11058	0.417	330	W24	176	450	0.491	0.158	5680	0.005	0.535
4	286	W24	162	414	7596	0.367	227	W24	146	370	0.411	0.158	4580	0.021	0.715
	286	W24	162	414	6803	0.329	203	W24	146	370	0.368	0.158	4580	0.021	0.715
	205	W24	162	414	7025	0.340	210	W24	146	370	0.380	0.158	4580	0.005	0.513
	214	W24	162	414	7841	0.379	234	W24	146	370	0.424	0.158	4580	0.006	0.535
5	286	W24	146	371	6136	0.331	183	W24	131	329	0.373	0.158	4020	0.024	0.715
	286	W24	146	371	5354	0.289	160	W24	131	329	0.325	0.158	4020	0.024	0.715
	205	W24	146	371	5301	0.286	158	W24	131	329	0.322	0.158	4020	0.006	0.513
	214	W24	146	371	6170	0.333	184	W24	131	329	0.375	0.158	4020	0.007	0.535
6	286	W24	131	328	5436	0.331	162	W24	117	291	0.374	0.158	3540	0.027	0.715
	286	W24	131	328	4833	0.295	144	W24	117	291	0.332	0.158	3540	0.027	0.715
	205	W24	131	328	4694	0.286	140	W24	117	291	0.323	0.158	3540	0.007	0.513
	214	W24	131	328	5390	0.328	161	W24	117	291	0.370	0.158	3540	0.008	0.535
7	286	W24	104	257	3893	0.303	116	W24	104	258	0.302	0.158	3100	0.031	0.715
	286	W24	104	257	3690	0.287	110	W24	104	258	0.286	0.158	3100	0.031	0.715
	205	W24	104	257	3369	0.262	101	W24	104	258	0.261	0.158	3100	0.008	0.513
	214	W24	104	257	2503	0.195	75	W24	104	258	0.194	0.158	3100	0.010	0.535
8	286	W24	104	257	2503	0.195	75	W24	104	258	0.194	0.158	3100	0.031	0.715
	286	W24	104	257	2660	0.207	79	W24	104	258	0.206	0.158	3100	0.031	0.715
	205	W24	104	257	2139	0.166	64	W24	104	258	0.166	0.158	3100	0.008	0.513
	214	W24	104	257	2144	0.167	64	W24	104	258	0.166	0.158	3100	0.010	0.535
	Total Weight (kips)	84.90						Total Weight (kips)	77.39						

Weight Reduction (kips)   -7.52

Frame 1: Column Design

Frame 1 Columns														
Floor	h (ft)	Original Size	Weight (lbs/ft)	S (in^3)	A (in^2)	M_{max} (k-in)	P_u (K)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in^3)	A (in^2)	Ratio
1	15	W14	455	756	134	9496	479	0.271	409	W14	455	756	134	0.271
		W14	455	756	134	13338	432	0.377	570	W14	455	756	134	0.377
		W14	455	756	134	12441	447	0.353	533	W14	455	756	134	0.353
		W14	455	756	134	12147	306	0.337	509	W14	455	756	134	0.337
		W14	455	756	134	8745	542	0.252	381	W14	455	756	134	0.252
2	12.5	W14	455	756	134	7576	352	0.212	320	W14	455	756	134	0.212
		W14	455	756	134	10793	353	0.301	456	W14	455	756	134	0.301
		W14	455	756	134	10149	374	0.284	430	W14	455	756	134	0.284
		W14	455	756	134	10103	246	0.277	420	W14	455	756	134	0.277
		W14	455	756	134	6733	398	0.189	286	W14	455	756	134	0.189
3.1	5									W14	455			
										W14	455			
										W14	455			
										W14	455			
										W14	455			
3.2	8.5	W14	426	706	125	8091	352	0.243	343	W14	426	706	125	0.243
		W14	455	757	134	11207	353	0.313	473	W14	426	706	125	0.336
		W14	455	757	134	10952	374	0.307	464	W14	426	706	125	0.330
		W14	455	757	134	11221	246	0.308	466	W14	426	706	125	0.331
		W14	426	706	125	8285	398	0.251	354	W14	426	706	125	0.251
4	13.5	W14	426	706	125	3937	263	0.116	164	W14	426	706	125	0.116
		W14	455	757	134	5980	288	0.165	250	W14	426	706	125	0.178
		W14	455	757	134	6074	297	0.168	254	W14	426	706	125	0.181
		W14	455	757	134	6704	189	0.182	276	W14	426	706	125	0.196
		W14	426	706	125	4117	289	0.122	173	W14	426	706	125	0.122
5.1	5	W14	426	706	125	3658	194	0.107	151	W14	426	706	125	0.107
		W14	455	757	134	5173	230	0.142	214	W14	426	706	125	0.152
		W14	455	757	134	5616	147	0.152	230	W14	426	706	125	0.163
		W14	455	757	134	5600	152	0.151	229	W14	426	706	125	0.163
		W14	426	706	125	3725	206	0.109	154	W14	426	706	125	0.109
5.2	8.5	W14	311	506	91.4	3256	194	0.134	136	W14	311	506	91.4	0.134
		W14	311	506	91.4	4404	230	0.183	185	W14	311	506	91.4	0.183
		W14	311	506	91.4	4752	147	0.194	196	W14	311	506	91.4	0.194
		W14	311	506	91.4	4731	152	0.193	196	W14	311	506	91.4	0.193
		W14	311	506	91.4	3286	206	0.136	138	W14	311	506	91.4	0.136
6	13.5	W14	311	506	91.4	3162	162	0.129	131	W14	311	506	91.4	0.130
		W14	311	506	91.4	4177	162	0.171	173	W14	311	506	91.4	0.171
		W14	311	506	91.4	4487	115	0.182	184	W14	311	506	91.4	0.182
		W14	311	506	91.4	4429	116	0.179	182	W14	311	506	91.4	0.180
		W14	311	506	91.4	3152	139	0.128	130	W14	311	506	91.4	0.129
7.1	5	W14	311	506	91.4	2274	80	0.091	93	W14	311	506	91.4	0.091
		W14	311	506	91.4	3063	110	0.124	126	W14	311	506	91.4	0.124
		W14	311	506	91.4	3355	81	0.135	137	W14	311	506	91.4	0.135
		W14	311	506	91.4	3271	79	0.131	133	W14	311	506	91.4	0.132
		W14	311	506	91.4	2238	79	0.090	91	W14	311	506	91.4	0.090
7.2	10	W14	283	459	83.3	2895	80	0.129	118	W14	283	459	83.3	0.129
		W14	283	459	83.3	3501	110	0.157	144	W14	283	459	83.3	0.157
		W14	283	459	83.3	3777	81	0.168	154	W14	283	459	83.3	0.168
		W14	283	459	83.3	3673	79	0.163	150	W14	283	459	83.3	0.163
		W14	283	459	83.3	2838	79	0.126	116	W14	283	459	83.3	0.126
8	15	W14	283	459	83.3	2465	35	0.108	99	W14	283	459	83.3	0.108
		W14	283	459	83.3	2545	56	0.112	103	W14	283	459	83.3	0.112
		W14	283	459	83.3	2982	43	0.131	121	W14	283	459	83.3	0.131
		W14	283	459	83.3	2581	40	0.114	104	W14	283	459	83.3	0.114
		W14	283	459	83.3	2144	33	0.094	86	W14	283	459	83.3	0.094
		Total Weight (kips)	199.7815							Total Weight (kips)	197.4325			

Weight Reduction (kips)   -2.349

Frame 7: Beam Design

Frame 7 Beams														
Floor	L (in)	Original Size	Weight (lbs/ft)	S (in³)	M_{max} (k-in)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in³)	Ratio	w (k/in)	I (in⁴)	Δ_{max} (in)	$\Delta_{allowed}$ (in)
1	286	W24	176	451	10919	0.484	326	W24	176	450	0.485	0.158	5680	0.0167	0.715
	205	W24	176	451	9284	0.412	277	W24	176	450	0.413	0.158	5680	0.0044	0.513
	214	W24	176	451	11738	0.521	350	W24	176	450	0.522	0.158	5680	0.0052	0.535
2	286	W24	176	451	8278	0.367	247	W24	162	414	0.400	0.158	5170	0.0184	0.715
	205	W24	176	451	7339	0.326	219	W24	162	414	0.355	0.158	5170	0.0048	0.513
	214	W24	176	451	8680	0.385	259	W24	162	414	0.419	0.158	5170	0.0058	0.535
3	286	W24	176	451	7665	0.340	229	W24	146	371	0.413	0.158	4580	0.0207	0.715
	205	W24	176	451	6562	0.291	196	W24	146	371	0.354	0.158	4580	0.0055	0.513
	214	W24	176	451	7844	0.348	234	W24	146	371	0.423	0.158	4580	0.0065	0.535
4	286	W24	176	451	7550	0.335	225	W24	146	371	0.407	0.158	4580	0.0207	0.715
	205	W24	176	451	6391	0.284	191	W24	146	371	0.345	0.158	4580	0.0055	0.513
	214	W24	176	451	7636	0.339	228	W24	146	371	0.412	0.158	4580	0.0065	0.535
5	286	W24	162	414	6461	0.312	193	W24	146	371	0.348	0.158	4580	0.0207	0.715
	205	W24	162	414	5318	0.257	159	W24	146	371	0.287	0.158	4580	0.0055	0.513
	214	W24	162	414	6390	0.309	191	W24	146	371	0.345	0.158	4580	0.0065	0.535
6	286	W24	131	328	5519	0.336	165	W24	117	291	0.379	0.158	3540	0.0268	0.715
	205	W24	131	328	4654	0.284	139	W24	117	291	0.320	0.158	3540	0.0071	0.513
	214	W24	131	328	5381	0.328	161	W24	117	291	0.370	0.158	3540	0.0084	0.535
7	286	W24	104	257	4015	0.312	120	W24	104	258	0.311	0.158	3100	0.0306	0.715
	205	W24	104	257	3400	0.264	101	W24	104	258	0.264	0.158	3100	0.0081	0.513
	214	W24	104	257	3705	0.288	111	W24	104	258	0.287	0.158	3100	0.0096	0.535
8	286	W24	104	257	2595	0.202	77	W24	104	258	0.201	0.158	3100	0.0306	0.715
	205	W24	104	257	2218	0.172	66	W24	104	258	0.172	0.158	3100	0.0081	0.513
	214	W24	104	257	2181	0.170	65	W24	104	258	0.169	0.158	3100	0.0096	0.535
Total Weight (kips) 70.7938							Total Weight (kips) 64.68375								

Weight Reduction (kips)   -6.11

Frame 7: Column Design

Frame 7 Columns														
Floor	h (ft)	Original Size	Weight (lbs/ft)	S (in³)	A (in²)	M_{max} (k-in)	P_u (K)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in³)	A (in²)	Ratio
1	15	W14	426	706	125	9648	499	0.297	419	W14	370	607	109	0.350
		W14	426	706	125	11649	473	0.357	504	W14	370	607	109	0.420
		W14	426	706	125	12054	343	0.361	510	W14	370	607	109	0.424
		W14	426	706	125	9978	572	0.311	439	W14	370	607	109	0.367
2	12.5	W14	426	706	125	4538	409	0.138	194	W14	370	607	109	0.162
		W14	426	706	125	6893	407	0.209	295	W14	370	607	109	0.245
		W14	426	706	125	7393	291	0.220	310	W14	370	607	109	0.257
		W14	426	706	125	4836	458	0.148	209	W14	370	607	109	0.174
3 .1	5	W14	426	706	125	3741	336	0.112	158	W14	370	607	109	0.131
		W14	426	706	125	5352	342	0.160	226	W14	370	607	109	0.188
		W14	426	706	125	5689	246	0.168	237	W14	370	607	109	0.196
		W14	426	706	125	3845	369	0.116	163	W14	370	607	109	0.136
3.2	8.5	W14	398	656	117	4019	336	0.130	171	W14	342	558	101	0.154
		W14	398	656	117	5617	342	0.182	239	W14	342	558	101	0.216
		W14	398	656	117	5925	246	0.189	247	W14	342	558	101	0.223
		W14	398	656	117	4129	369	0.134	176	W14	342	558	101	0.160
4	13.5	W14	398	656	117	3763	267	0.120	158	W14	342	558	101	0.142
		W14	398	656	117	5770	189	0.182	239	W14	342	558	101	0.215
		W14	398	656	117	5755	188	0.181	238	W14	342	558	101	0.214
		W14	398	656	117	3833	288	0.123	161	W14	342	558	101	0.146
5.1	5	W14	398	656	117	3787	199	0.120	157	W14	342	558	101	0.141
		W14	398	656	117	5552	157	0.174	228	W14	342	558	101	0.205
		W14	398	656	117	5518	153	0.173	227	W14	342	558	101	0.204
		W14	398	656	117	3803	208	0.120	158	W14	342	558	101	0.142
5.2	8.5	W14	311	506	91.4	3415	199	0.141	143	W14	283	459	83.3	0.156
		W14	311	506	91.4	4888	157	0.200	202	W14	283	459	83.3	0.221
		W14	311	506	91.4	4829	153	0.197	200	W14	283	459	83.3	0.218
		W14	311	506	91.4	3397	208	0.141	142	W14	283	459	83.3	0.156
6	13.5	W14	311	506	91.4	3161	138	0.129	130	W14	283	459	83.3	0.142
		W14	311	506	91.4	4481	120	0.182	184	W14	283	459	83.3	0.201
		W14	311	506	91.4	4395	117	0.178	180	W14	283	459	83.3	0.197
		W14	311	506	91.4	3100	140	0.126	128	W14	283	459	83.3	0.140
7.1	5	W14	311	506	91.4	2358	82	0.095	96	W14	283	459	83.3	0.105
		W14	311	506	91.4	3415	83	0.137	139	W14	283	459	83.3	0.152
		W14	311	506	91.4	3333	80	0.134	136	W14	283	459	83.3	0.148
		W14	311	506	91.4	2281	79	0.092	93	W14	283	459	83.3	0.101
7.2	10	W14	283	460	83.3	2929	82	0.130	120	W14	257	415	75.6	0.144
		W14	283	460	83.3	3779	83	0.168	154	W14	257	415	75.6	0.186
		W14	283	460	83.3	3700	80	0.164	151	W14	257	415	75.6	0.182
		W14	283	460	83.3	2838	79	0.126	116	W14	257	415	75.6	0.140
8	15	W14	283	460	83.3	2595	37	0.114	105	W14	257	415	75.6	0.126
		W14	283	460	83.3	3026	44	0.133	122	W14	257	415	75.6	0.148
		W14	283	460	83.3	2668	40	0.117	108	W14	257	415	75.6	0.130
		W14	283	460	83.3	2181	33	0.096	88	W14	257	415	75.6	0.106
		Total Weight (k)	160.252							Total Weight (k)	141.3			

Weight Reduction (kips)   -18.952

Frame A: Beam Design

Floor	L (in)	Original Size	Weight (lbs/ft)	S (in³)	M_{max} (k-in)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in³)	Ratio	w (k/in)	I (in⁴)	Δ_{max} (in)	$\Delta_{allowed}$ (in)
Frame A Beams															
1	340.5	W24	192	491	10905	0.444	326	W24	192	491	0.444	0.107	6260	0.021	0.851
	276	W30	148	435	10108	0.465	302	W30	148	435	0.465	0.107	6680	0.008	0.690
	306	W24	192	491	9188	0.374	274	W24	192	491	0.374	0.107	6260	0.013	0.765
	252	W30	148	435	10626	0.488	317	W30	148	435	0.488	0.107	6680	0.006	0.630
	349.5	W24	192	491	9310	0.379	278	W24	192	491	0.379	0.107	6260	0.023	0.874
2	340.5	W24	192	491	7856	0.320	234	W24	192	491	0.320	0.107	6260	0.021	0.851
	276	W30	148	435	7582	0.348	226	W30	148	435	0.348	0.107	6680	0.008	0.690
	306	W24	192	491	7027	0.286	210	W24	192	491	0.286	0.107	6260	0.013	0.765
	252	W30	148	435	7770	0.357	232	W30	148	435	0.357	0.107	6680	0.006	0.630
	349.5	W24	192	491	7488	0.305	224	W24	192	491	0.305	0.107	6260	0.023	0.874
3	340.5	W24	192	491	7370	0.300	220	W24	192	491	0.300	0.107	6260	0.021	0.851
	276	W30	148	435	6937	0.319	207	W30	148	435	0.319	0.107	6680	0.008	0.690
	306	W24	192	491	6557	0.267	196	W24	192	491	0.267	0.107	6260	0.013	0.765
	252	W30	148	435	7124	0.327	213	W30	148	435	0.327	0.107	6680	0.006	0.630
	349.5	W24	192	491	6639	0.270	198	W24	192	491	0.270	0.107	6260	0.023	0.874
4	340.5	W24	162	414	6449	0.312	193	W24	162	414	0.312	0.107	5170	0.025	0.851
	276	W30	148	435	6973	0.320	208	W30	148	435	0.320	0.107	6680	0.008	0.690
	306	W24	162	414	5687	0.275	170	W24	162	414	0.275	0.107	5170	0.016	0.765
	252	W30	148	435	7094	0.326	212	W30	148	435	0.326	0.107	6680	0.006	0.630
	349.5	W24	162	414	5960	0.288	178	W24	162	414	0.288	0.107	5170	0.028	0.874
5	340.5	W24	162	414	6160	0.298	184	W24	162	414	0.298	0.107	5170	0.025	0.851
	276	W30	124	355	5726	0.323	171	W30	124	355	0.323	0.107	5360	0.010	0.690
	306	W24	162	414	5620	0.272	168	W24	162	414	0.272	0.107	5170	0.016	0.765
	252	W30	124	355	5875	0.331	175	W30	124	355	0.331	0.107	5360	0.007	0.630
	349.5	W24	162	414	5443	0.263	162	W24	162	414	0.263	0.107	5170	0.028	0.874
6	340.5	W24	117	291	5689	0.391	170	W24	117	291	0.391	0.107	3540	0.036	0.851
	276	W30	99	269	5689	0.423	170	W30	99	269	0.423	0.107	3990	0.014	0.690
	306	W24	117	291	5131	0.352	153	W24	117	291	0.352	0.107	3540	0.024	0.765
	252	W30	99	269	5842	0.435	174	W30	99	269	0.435	0.107	3990	0.010	0.630
	349.5	W24	117	291	5014	0.344	150	W24	117	291	0.344	0.107	3540	0.040	0.874
7	340.5	W24	104	257	2578	0.200	77	W24	104	257	0.200	0.107	3100	0.042	0.851
	276	W30	99	269	2381	0.177	71	W30	99	269	0.177	0.107	3990	0.014	0.690
	306	W24	104	257	2348	0.183	70	W24	104	257	0.183	0.107	3100	0.027	0.765
	252	W30	99	269	2256	0.168	67	W30	99	269	0.168	0.107	3990	0.010	0.630
	349.5	W24	104	257	2533	0.197	76	W24	104	257	0.197	0.107	3100	0.046	0.874
8	340.5	W24	104	257	2332	0.181	70	W24	104	257	0.181	0.107	3100	0.042	0.851
	276	W30	99	269	2086	0.155	62	W30	99	269	0.155	0.107	3990	0.014	0.690
	306	W24	104	257	2152	0.167	64	W24	104	257	0.167	0.107	3100	0.027	0.765
	252	W30	99	269	2056	0.153	61	W30	99	269	0.153	0.107	3990	0.010	0.630
	349.5	W24	104	257	2353	0.183	70	W24	104	257	0.183	0.107	3100	0.046	0.874
	Total Weight (kips)		146.247					Total Weight (kips)	146.247						

Weight Reduction (kips)   0

Frame A: Column Design

Frame A Columns														
Floor	h (ft)	Original Size	Weight (lbs/ft)	S (in³)	A (in²)	M_{max} (k-in)	P_u (K)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in³)	A (in²)	Ratio
1	15	W14	455	756	134	10713	396	0.301	455	W14	455	756	134	0.301
		W14	455	756	134	13177	390	0.370	560	W14	455	756	134	0.370
		W14	455	756	134	13162	359	0.368	556	W14	455	756	134	0.368
		W14	455	756	134	13234	385	0.371	562	W14	455	756	134	0.371
		W14	455	756	134	13072	453	0.371	561	W14	455	756	134	0.371
		W14 (w)	455	304	134	5731	373	0.399	243	W14 (w)	455	304	134	0.399
2	12.5	W14	455	756	134	4336	321	0.120	182	W14	455	756	134	0.120
		W14	455	756	134	7075	336	0.197	298	W14	455	756	134	0.197
		W14	455	756	134	7156	308	0.198	300	W14	455	756	134	0.198
		W14	455	756	134	7218	326	0.201	303	W14	455	756	134	0.201
		W14	455	756	134	7011	382	0.197	297	W14	455	756	134	0.197
		W14 (w)	455	304	134	4249	305	0.293	178	W14 (w)	455	304	134	0.293
3.1	5	W14	455	756	134	3519	262	0.097	146	W14	455	756	134	0.097
		W14	455	756	134	5342	285	0.148	223	W14	455	756	134	0.148
		W14	455	756	134	5479	262	0.151	228	W14	455	756	134	0.151
		W14	455	756	134	5486	275	0.151	229	W14	455	756	134	0.151
		W14	455	756	134	5314	322	0.148	223	W14	455	756	134	0.148
		W14 (w)	455	304	134	3238	248	0.221	135	W14 (w)	455	304	134	0.221
3.2	8.5	W14	455	756	134	4073	262	0.112	170	W14	426	706	134	0.120
		W14	455	756	134	5973	285	0.165	250	W14	426	706	134	0.177
		W14	455	756	134	6097	262	0.168	254	W14	426	706	134	0.180
		W14	455	756	134	6111	275	0.169	255	W14	426	706	134	0.181
		W14	455	756	134	5879	322	0.163	247	W14	426	706	134	0.175
		W14 (w)	426	283	134	3461	248	0.254	144	W14 (w)	426	283	134	0.254
4	13.5	W14	455	756	134	3313	206	0.090	137	W14	426	706	134	0.097
		W14	455	756	134	5763	162	0.156	236	W14	426	706	134	0.167
		W14	455	756	134	5560	217	0.152	230	W14	426	706	134	0.163
		W14	455	756	134	5562	226	0.152	230	W14	426	706	134	0.163
		W14	455	756	134	5354	263	0.147	223	W14	426	706	134	0.158
		W14 (w)	426	283	134	3178	195	0.231	131	W14 (w)	426	283	134	0.231
5.1	5	W14	455	756	134	3137	155	0.085	128	W14	426	706	134	0.091
		W14	455	756	134	5350	137	0.144	218	W14	426	706	134	0.155
		W14	455	756	134	6097	262	0.168	254	W14	426	706	134	0.180
		W14	455	756	134	6111	275	0.169	255	W14	426	706	134	0.181
		W14	455	756	134	5879	322	0.163	247	W14	426	706	134	0.175
		W14 (w)	426	283	134	2876	147	0.208	118	W14 (w)	426	283	134	0.208
5.2	8.5	W14	398	656	117	3097	155	0.097	127	W14	398	656	117	0.097
		W14	398	656	117	4870	137	0.152	199	W14	398	656	117	0.152
		W14	398	656	117	4715	166	0.148	194	W14	398	656	117	0.148
		W14	398	656	117	4936	114	0.153	201	W14	398	656	117	0.153
		W14	398	656	117	4879	114	0.152	199	W14	398	656	117	0.152
		W14 (w)	311	199	91.4	2646	147	0.275	109	W14 (w)	311	199	91.4	0.275
6	13.5	W14	398	656	117	3063	105	0.095	125	W14	398	656	117	0.095
		W14	398	656	117	4858	104	0.151	198	W14	398	656	117	0.151
		W14	398	656	117	4707	124	0.147	192	W14	398	656	117	0.147
		W14	398	656	117	4950	88	0.153	201	W14	398	656	117	0.153
		W14	398	656	117	4837	90	0.150	197	W14	398	656	117	0.150
		W14 (w)	311	199	91.4	2646	147	0.275	109	W14 (w)	311	199	91.4	0.275
7.1	5	W14	398	656	117	3996	58	0.123	161	W14	398	656	117	0.123
		W14	398	656	117	5432	74	0.168	220	W14	398	656	117	0.168
		W14	398	656	117	5327	71	0.164	216	W14	398	656	117	0.164
		W14	398	656	117	5457	65	0.168	221	W14	398	656	117	0.168
		W14	398	656	117	5431	69	0.168	220	W14	398	656	117	0.168
		W14 (w)	311	199	91.4	2490	57	0.253	101	W14 (w)	311	199	91.4	0.253
7.2	10	W14	311	506	91.4	2303	58	0.092	93	W14	311	506	91.4	0.092
		w14	311	506	91.4	3056	74	0.123	124	W14	311	506	91.4	0.123
		W14	311	506	91.4	2999	78	0.121	122	W14	311	506	91.4	0.121
		W14	311	506	91.4	3096	65	0.124	126	W14	311	506	91.4	0.124
		W14	311	506	91.4	3066	69	0.123	125	W14	311	506	91.4	0.123
		W14 (w)	283	179	83.3	1275	57	0.144	52	W14 (w)	283	179	83.3	0.144
8	15	W14	311	506	91.4	2313	28	0.092	93	W14	311	506	91.4	0.092
		w14	311	506	91.4	2732	38	0.109	110	W14	311	506	91.4	0.109
		W14	311	506	91.4	2566	36	0.102	103	W14	311	506	91.4	0.102
		W14	311	506	91.4	2767	33	0.110	111	W14	311	506	91.4	0.110
		W14	311	506	91.4	2930	35	0.117	118	W14	311	506	91.4	0.117
		W14 (w)	283	179	83.3	2194	28	0.247	88	W14 (w)	283	179	83.3	0.247
		Total Weight (k)	269.729							Total Weight (k)	265.814			

Weight Reduction (kips)   -3.915

Frame E: Beam Design

Frame E Beams															
Floor	L (in)	Original Size	Weight (lbs/ft)	S (in³)	M_{max} (k-in)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in³)	Ratio	w (k/in)	I (in⁴)	Δ_{max} (in)	$\Delta_{allowed}$ (in)
1	216.5	W30	148	436	7791	0.357	233	W30	148	436	0.357	0.158	6680	0.1963545	0.541
	259	W30	148	436	7084	0.325	211	W30	148	436	0.325	0.158	6680	0.2555071	0.648
	288	W36	150	504	10934	0.434	326	W30	148	436	0.502	0.158	6680	0.4876816	0.720
	288	W36	150	504	9326	0.370	326	W30	148	436	0.428	0.158	6680	0.4159438	0.720
	259	W30	148	436	9370	0.430	280	W30	148	436	0.430	0.158	6680	0.3379964	0.648
	216.5	W30	148	436	7099	0.326	212	W30	148	436	0.326	0.158	6680	0.1789146	0.541
2	216.5	W18	175	345	4848	0.281	145	W18	143	282	0.344	0.158	2750	0.296825	0.541
	259	W18	175	345	4169	0.242	124	W18	143	282	0.296	0.158	2750	0.3652527	0.648
	288	W36	150	504	7358	0.292	220	W30	132	380	0.387	0.158	5770	0.379949	0.720
	288	W36	150	504	6506	0.258	327	W30	132	380	0.342	0.158	5770	0.3359579	0.720
	259	W30	148	435	6893	0.317	206	W30	132	380	0.363	0.158	5770	0.2878281	0.648
3	216.5	W24	192	491	5208	0.212	155	W24	131	329	0.317	0.158	4020	0.2181262	0.541
	259	W24	192	491	5069	0.206	151	W24	131	329	0.308	0.158	4020	0.3038378	0.648
	288	W24	192	491	5006	0.204	149	W24	131	329	0.304	0.158	4020	0.3710234	0.720
	288	W24	192	491	5125	0.209	328	W24	131	329	0.312	0.158	4020	0.379854	0.720
	259	W24	192	491	4935	0.201	147	W24	131	329	0.300	0.158	4020	0.2957775	0.648
	216.5	W24	192	491	4959	0.202	148	W24	131	329	0.301	0.158	4020	0.2077014	0.541
4	216.5	W24	162	414	4691	0.227	140	W24	117	291	0.322	0.158	3540	0.223101	0.541
	259	W24	162	414	4587	0.222	137	W24	117	291	0.315	0.158	3540	0.3122222	0.648
	288	W24	162	414	4548	0.220	136	W24	117	291	0.313	0.158	3540	0.382774	0.720
	288	W24	162	414	4642	0.224	329	W24	117	291	0.319	0.158	3540	0.3906741	0.720
	259	W24	162	414	4468	0.216	133	W24	117	291	0.307	0.158	3540	0.3041238	0.648
	216.5	W24	162	414	4638	0.224	138	W24	117	291	0.319	0.158	3540	0.2205864	0.541
5	216.5	W24	131	328	3672	0.224	110	W24	94	222	0.331	0.158	2700	0.2289431	0.541
	259	W24	131	328	4038	0.246	121	W24	94	222	0.364	0.158	2700	0.3603189	0.648
	288	W24	131	328	4034	0.246	120	W24	94	222	0.363	0.158	2700	0.4451602	0.720
	288	W24	131	328	4107	0.250	330	W24	94	222	0.370	0.158	2700	0.4531925	0.720
	259	W24	131	328	3948	0.241	118	W24	94	222	0.356	0.158	2700	0.3523574	0.648
	216.5	W24	131	328	3706	0.226	111	W24	94	222	0.334	0.158	2700	0.2311215	0.541
6	216.5	W24	104	257	3107	0.242	93	W24	84	196	0.317	0.158	2370	0.2207149	0.541
	259	W24	104	257	3512	0.273	105	W24	84	196	0.358	0.158	2370	0.3570329	0.648
	288	W24	104	257	3551	0.276	106	W24	84	196	0.362	0.158	2370	0.4464386	0.720
	288	W24	104	257	3606	0.280	331	W24	84	196	0.368	0.158	2370	0.453354	0.720
	259	W24	104	257	3444	0.268	103	W24	84	196	0.351	0.158	2370	0.3501684	0.648
	216.5	W24	104	257	3141	0.244	94	W24	84	196	0.321	0.158	2370	0.2231298	0.541
7	216.5	W24	104	257	2361	0.184	70	W24	84	196	0.241	0.158	2370	0.1677231	0.541
	259	W24	104	257	2745	0.213	82	W24	84	196	0.280	0.158	2370	0.2791062	0.648
	288	W24	104	257	2826	0.220	84	W24	84	196	0.288	0.158	2370	0.3552005	0.720
	288	W24	104	257	2880	0.224	332	W24	84	196	0.294	0.158	2370	0.3620508	0.720
	259	W24	104	257	2680	0.208	80	W24	84	196	0.273	0.158	2370	0.2724793	0.648
	216.5	W24	104	257	2392	0.186	71	W24	84	196	0.244	0.158	2370	0.1699557	0.541
8	216.5	W24	104	257	1427	0.111	43	W24	84	196	0.146	0.158	2370	0.1014049	0.541
	259	W24	104	257	2013	0.156	60	W24	84	196	0.205	0.158	2370	0.2046185	0.648
	288	W24	104	257	2110	0.164	63	W24	84	196	0.215	0.158	2370	0.2651948	0.720
	288	W24	104	257	2165	0.168	333	W24	84	196	0.221	0.158	2370	0.2721339	0.720
	259	W24	104	257	1950	0.152	58	W24	84	196	0.199	0.158	2370	0.198279	0.648
	216.5	W24	104	257	1484	0.115	44	W24	84	196	0.151	0.158	2370	0.1054352	0.541
	Total Weight (kips)	134.25						Total Weight (kips)	105.8909						

Weight Reduction (kips)   -28.3591

Frame E: Column Design

Frame E Columns														
Floor	h (ft)	Original Size	Weight (lbs/ft)	S (in³)	A (in²)	M_{max} (k-in)	P_u (K)	Ratio	S_{req}	New Design	Weight (lbs/ft)	S (in³)	A (in²)	Ratio
1	15	W14 (w)	455	304	134	4748	408	0.333	202	W14 (w)	370	241	109	0.426
		W14	426	706	125	9622	420	0.292	413	W14	398	656	117	0.316
		W14	426	706	125	10135	434	0.309	436	W14	398	656	117	0.334
		W14	426	706	125	8392	476	0.257	363	W14	398	656	117	0.278
		W14	426	706	125	7776	470	0.238	336	W14	398	656	117	0.258
		W14	426	706	125	9863	358	0.296	419	W14	398	656	117	0.320
		W14 (w)	426	283	125	4893	343	0.366	207	W14 (w)	455	304	134	0.339
2	12.5	W14 (w)	455	304	134	3043	324	0.210	128	W14 (w)	370	241	109	0.268
		W14	426	706	125	3901	354	0.117	165	W14	398	656	117	0.127
		W14	426	706	125	5367	361	0.161	228	W14	398	656	117	0.174
		W14	426	706	125	5650	386	0.171	241	W14	398	656	117	0.184
		W14	426	706	125	5182	387	0.156	221	W14	398	656	117	0.169
		W14	426	706	125	4414	305	0.131	186	W14	398	656	117	0.142
		W14 (w)	426	283	125	1600	268	0.118	67	W14 (w)	455	304	134	0.110
3.1	5	W14 (w)	455	304	134	1967	367	0.137	83	W14 (w)	370	241	109	0.175
		W14	426	706	125	2864	243	0.084	119	W14	398	656	117	0.091
		W14	426	706	125	3878	333	0.116	164	W14	398	656	117	0.125
		W14	426	706	125	5190	334	0.155	219	W14	398	656	117	0.168
		W14	426	706	125	5169	300	0.154	217	W14	398	656	117	0.166
		W14	426	706	125	2478	239	0.073	103	W14	398	656	117	0.079
		W14 (w)	426	283	125	319	268	0.024	13	W14 (w)	455	304	134	0.022
3.2	8.5	W14 (w)	426	283	125	2693	267	0.199	113	W14 (w)	342	221	101	0.257
		W14	426	706	125	4454	243	0.131	185	W14	370	607	109	0.154
		W14	426	706	125	4870	333	0.146	206	W14	370	607	109	0.171
		W14	426	706	125	4146	334	0.124	175	W14	370	607	109	0.146
		W14	426	706	125	4319	300	0.129	181	W14	370	607	109	0.151
		W14	426	706	125	4461	239	0.131	186	W14	370	607	109	0.154
		W14 (w)	398	262	117	2398	268	0.192	101	W14 (w)	426	283	125	0.177
4	13.5	W14 (w)	426	283	125	2515	207	0.184	104	W14 (w)	342	221	101	0.237
		W14	426	706	125	3781	207	0.111	156	W14	370	607	109	0.130
		W14	426	706	125	3530	245	0.104	147	W14	370	607	109	0.122
		W14	426	706	125	3369	278	0.100	141	W14	370	607	109	0.117
		W14	426	706	125	3533	251	0.104	147	W14	370	607	109	0.122
		W14	426	706	125	3750	203	0.110	155	W14	370	607	109	0.128
		W14 (w)	398	262	117	2561	209	0.203	106	W14 (w)	426	283	125	0.187

Frame E: Column Design (Continued)

5.1	5	W14 (w)	426	283	125	2221	152	0.161	91	W14 (w)	342	221	101	0.207
		W14	426	706	125	3540	171	0.103	146	W14	370	607	109	0.120
		W14	426	706	125	3327	198	0.097	137	W14	370	607	109	0.114
		W14	426	706	125	3173	222	0.093	132	W14	370	607	109	0.109
		W14	426	706	125	3334	202	0.098	138	W14	370	607	109	0.114
		W14	426	706	125	3524	167	0.103	145	W14	370	607	109	0.120
		W14 (w)	398	262	117	2182	153	0.171	90	W14 (w)	426	283	125	0.158
5.2	8.5	W14 (w)	311	199	91.4	1946	152	0.202	80	W14 (w)	283	179	83.3	0.226
		W14	398	656	117	3282	171	0.103	135	W14	342	558	101	0.122
		W14	398	656	117	3111	198	0.098	129	W14	342	558	101	0.116
		W14	398	656	117	2953	222	0.094	123	W14	342	558	101	0.111
		W14	398	656	117	3117	202	0.098	129	W14	342	558	101	0.116
		W14	398	656	117	3272	167	0.103	135	W14	342	558	101	0.121
		W14 (w)	311	199	91.4	1957	153	0.204	81	W14 (w)	311	199	83.3	0.204
6	13.5	W14 (w)	311	199	91.4	1728	104	0.178	71	W14 (w)	311	179	83.3	0.198
		W14	398	656	117	2926	131	0.091	120	W14	398	558	101	0.108
		W14	398	656	117	2781	149	0.087	114	W14	398	558	101	0.103
		W14	398	656	117	2607	166	0.082	107	W14	398	558	101	0.097
		W14	398	656	117	2783	152	0.087	114	W14	398	558	101	0.103
		W14	398	656	117	2918	128	0.091	119	W14	398	558	101	0.107
		W14 (w)	311	199	91.4	1749	105	0.180	72	W14 (w)	311	199	83.3	0.180
7.1	5	W14 (w)	311	199	91.4	1379	61	0.140	56	W14 (w)	311	179	83.3	0.156
		W14	398	656	117	2182	89	0.068	89	W14	398	558	101	0.080
		W14	398	656	117	2057	101	0.064	84	W14	398	558	101	0.075
		W14	398	656	117	1913	111	0.059	78	W14	398	558	101	0.070
		W14	398	656	117	2059	102	0.064	84	W14	398	558	101	0.075
		W14	398	656	117	2175	88	0.067	88	W14	398	558	101	0.079
		W14 (w)	311	199	91.4	1393	62	0.142	56	W14 (w)	311	199	83.3	0.142
7.2	10	W14 (w)	283	179	83.3	1482	61	0.168	60	W14 (w)	257	161	75.6	0.187
		W14	311	506	91.4	2581	89	0.104	105	W14	311	506	91.4	0.104
		W14	311	506	91.4	2471	101	0.100	101	W14	311	506	91.4	0.100
		W14	311	506	91.4	2351	111	0.095	96	W14	311	506	91.4	0.095
		W14	311	506	91.4	2473	102	0.100	101	W14	311	506	91.4	0.100
		W14	311	506	91.4	2575	88	0.104	105	W14	311	506	91.4	0.104
		W14 (w)	283	179	83.3	1496	62	0.170	61	W14 (w)	283	179	83.3	0.170
8	15	W14 (w)	283	179	83.3	1205	26	0.135	48	W14 (w)	257	161	75.6	0.151
		W14	311	506	91.4	1806	47	0.072	73	W14	311	506	91.4	0.072
		W14	311	506	91.4	1799	51	0.072	73	W14	311	506	91.4	0.072
		W14	311	506	91.4	1626	55	0.065	66	W14	311	506	91.4	0.065
		W14	311	506	91.4	1801	52	0.072	73	W14	311	506	91.4	0.072
		W14	311	506	91.4	1793	46	0.072	72	W14	311	506	91.4	0.072
		W14 (w)	283	179	83.3	1226	26	0.138	49	W14 (w)	283	179	83.3	0.138
		Total Weight (k)	301.1645							Total Weight (k)	282.4545			

Weight Change (kips)   -18.71

APPENDIX E: NONLINEAR ANALYSIS

Frame A NONLIN Damper Analysis

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Damping Force (k)	Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	408.225	3.6	113.396	22.679	408.225	6	2	1	1	0	0	0	173.898	21
Smaller 1	403.317	3.59	112.345	22.469	403.317	6	2	2	1	0	0	0	172.404	21
Smaller 2	392.599	3.528	111.281	22.256	392.599	6	2	1	1	0	0	0	241.2	21

Frame A NONLIN Damper Analysis

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Damping Force (k)	Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	272.035	2.402	113.254	22.651	272.035	13	6	3	2	1	1	1	169.786	21
Smaller 1	268.764	2.396	112.172	22.434	268.764	13	7	3	2	1	1	1	242.368	21
Smaller 2	261.616	2.354	111.137	22.227	261.616	13	7	3	2	2	1	1	241.01	21

Frame E NONLIN Damper Analysis - Maximum Considered Earthquake

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Optimal Damping Force (k)	Optimal Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	433.049	3.417	126.734	25.347	433.049	4	2	1	1	0	0	0	252.32	24
Smaller 1	412.267	3.366	122.480	24.496	412.267	4	2	1	1	0	0	0	255.57	25
Smaller 2	404.425	3.436	117.702	23.540	404.425	6	2	1	1	0	0	0	246.00	24
Smaller 3	390.408	3.508	111.291	22.258	390.408	5	3	1	1	0	0	0	234.29	23
Smaller 4	383.584	3.499	109.627	21.925	383.584	5	3	1	1	0	0	0	232.80	23
Smaller 5	372.695	3.506	106.302	21.260	372.695	6	2	1	1	0	0	0	223.05	22
Smaller 6	364.527	3.485	104.599	20.920	364.527	6	2	1	1	0	0	0	228.06	23

Frame E NONLIN Damper Analysis - Design Level Earthquake

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Optimal Damping Force (k)	Optimal Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	288.581	2.278	126.682	25.336	288.581	12	8	7	5	5	4	4	338.61	24
Smaller 1	274.739	2.241	122.597	24.519	274.739	6	6	5	2	2	2	2	233.22	25
Smaller 2	269.502	2.288	117.789	23.558	269.502	8	6	5	2	2	2	2	230.47	24
Smaller 3	260.156	2.336	111.368	22.274	260.156	9	7	5	2	2	2	2	218.73	23
Smaller 4	255.609	2.330	109.703	21.941	255.609	9	7	5	2	2	2	2	216.83	23
Smaller 5	248.355	2.335	106.362	21.272	248.355	9	7	6	2	2	2	2	206.49	22
Smaller 6	242.915	2.321	104.660	20.932	242.915	10	7	6	3	2	2	2	211.21	23

Frame 1 NONLIN Damper Analysis - Maximum Considered Earthquake

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Damping Force (k)	Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	278.514	3.695	75.376	15.075	278.514	5	2	1	0	0	0	0	147.942	20
Smaller 1	273.078	3.692	73.965	14.793	273.078	5	2	1	0	0	0	0	146.621	20
Smaller 2	265.536	3.756	70.696	14.139	265.536	6	2	1	0	0	0	0	133.003	18
Smaller 3	251.713	3.622	69.496	13.899	251.713	6	2	1	0	0	0	0	142.262	20

Frame 1 NONLIN Damper Analysis - Design Level Earthquake

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Damping Force (k)	Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	185.607	2.463	75.358	15.072	185.607	9	9	4	2	2	2	2	205.235	20
Smaller 1	181.984	2.461	73.947	14.789	181.984	9	7	4	2	2	2	2	136.609	20
Smaller 2	176.961	2.503	70.700	14.140	176.961	9	7	5	2	2	2	2	123.119	18
Smaller 3	167.741	2.414	69.487	13.897	167.741	11	7	5	2	2	2	2	131.056	20

Frame 7 NONLIN Damper Analysis - Maximum Considered Earthquake

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Damping Force (k)	Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	227.939	3.313	68.801	13.760	227.939	7	2	1	1	1	0	0	154.875	26
Smaller 1	222.587	3.377	65.913	13.183	222.587	4	2	1	1	0	0	0	126.316	25
Smaller 2	218.638	3.368	64.916	12.983	218.638	4	2	1	1	0	0	0	125.863	25
Smaller 3	209.516	3.451	60.712	12.142	209.516	4	2	1	1	0	0	0	120.726	24
Smaller 4	201.846	3.403	59.314	11.863	201.846	4	2	1	1	0	0	0	122.713	25

Frame 7 NONLIN Damper Analysis - Design Level Earthquake

Iteration	Performance Point Info		Equivalent Frame Properties			Yield Events Based on Percent Damping							Damping Force (k)	Percent Damping
	V (k)	D (in)	Stiffness 1 (k/in)	Stiffness 2 (k/in)	Strength (k)	5%	10%	15%	20%	25%	30%	35%		
Original Design	151.905	2.208	68.798	13.760	151.905	10	7	4	3	2	2	2	124.824	26
Smaller 1	148.333	2.249	65.955	13.191	148.333	9	7	4	2	2	2	2	120.634	25
Smaller 2	145.703	2.243	64.959	12.992	145.703	9	7	4	2	2	2	2	119.822	25
Smaller 3	139.617	2.296	60.809	12.162	139.617	6	6	5	2	2	2	2	114.308	24
Smaller 4	134.511	2.267	59.334	11.867	134.511	6	6	5	2	2	2	2	115.821	25

APPENDIX F: REFERENCES

Gilani, A. S., Miyamoto, H. K., & Kohagura, T. (2006). Seismic Rehabilitation of a Nine-Story Hospital Building Using Fluid Viscous Dampers. *Structures* .

McNamara, R. J., & Taylor, D. P. (2003). Fluid Viscous Dampers for High-Rise Buildings. *The Structural Design of Tall and Special Buidlings* . Wiley InterScience.

Miyamoto, H. K., & Scholl, R. E. (1998, November). Steel Pyramid. *Modern Steel Construction* .

Taylor, D., & Duflot, P. (n.d.). Fluid Viscous Dampers Used for Seismic Energy Dissipation in Structures.