New Acute Care Hospital and Skilled Nursing Facility

San Francisco, CA

Thesis Report

Tim Ariosto – Structural Option

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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.

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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.

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

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.

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” Ø 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”.

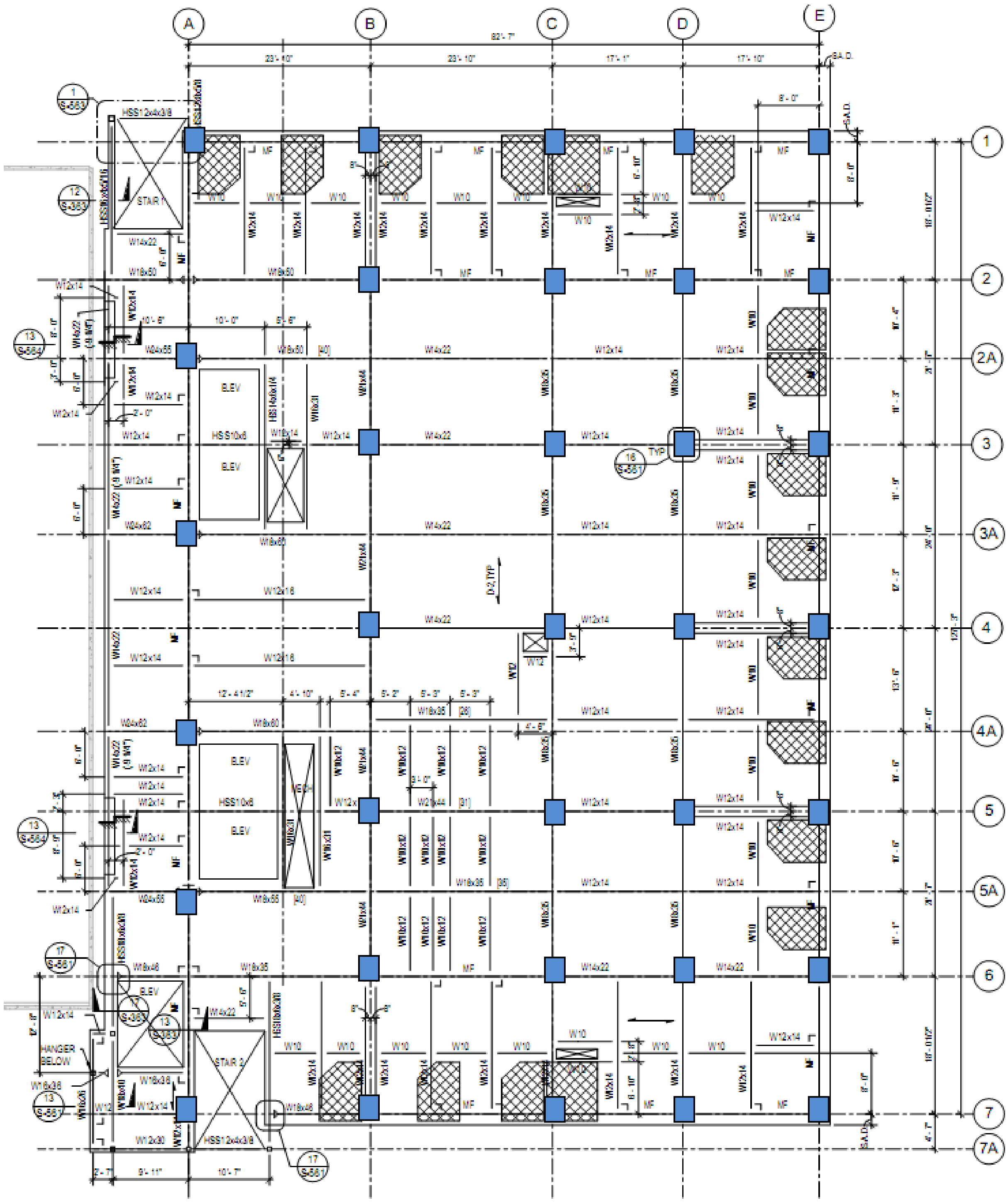

FIGURE 3: TYPICAL FRAMING PLAN WITH COLUMNS HIGHLIGHTED. DRAWINGS COURTESY OF JACOBS CARTER BURGESS.

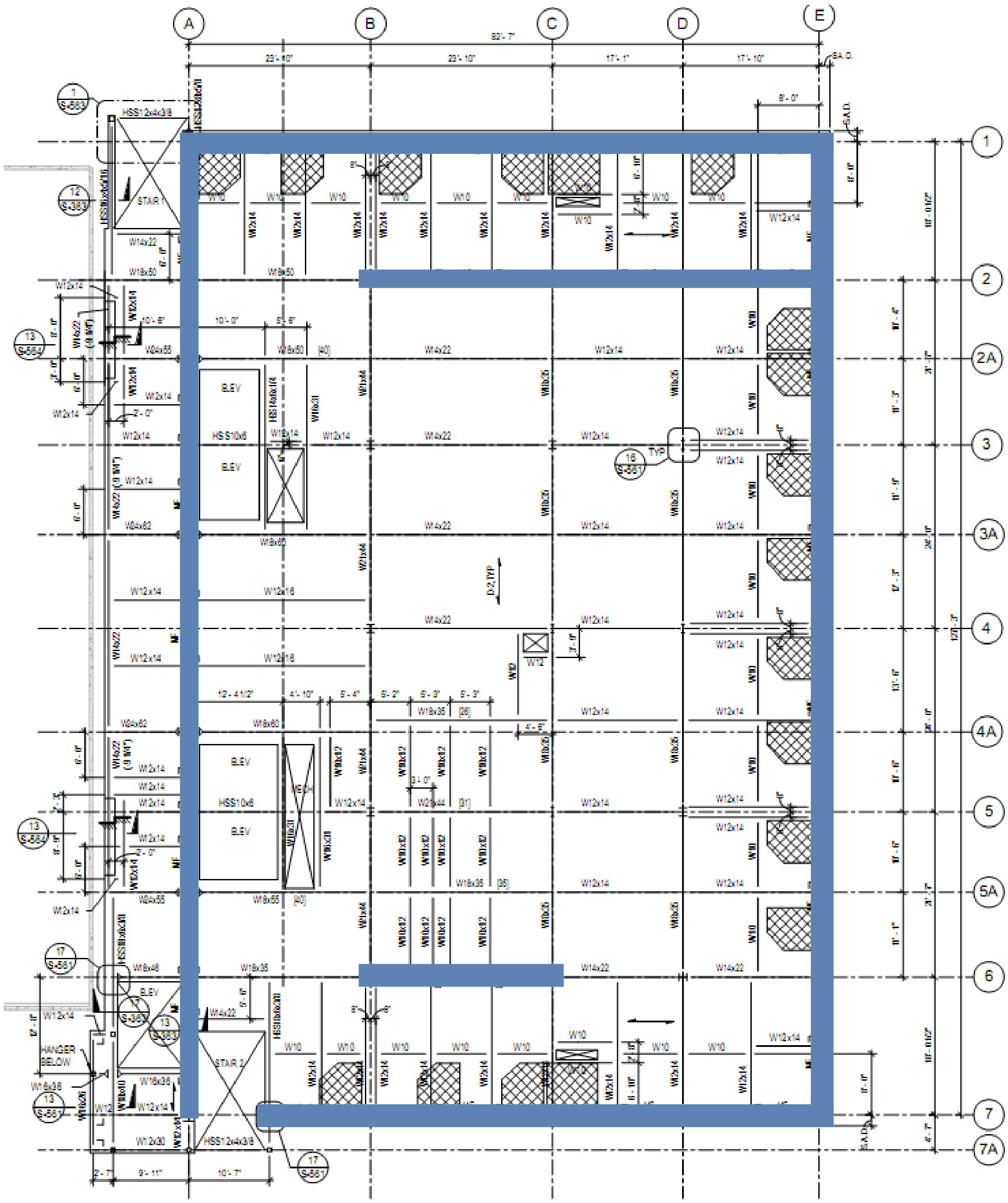

FIGURE 4: TYPICAL FRAMING PLANS WITH LATERAL SYSTEM HIGHLIGHTED IN BLUE. DRAWING COURTESY OF JACOBS CARTER BURGESS.

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.

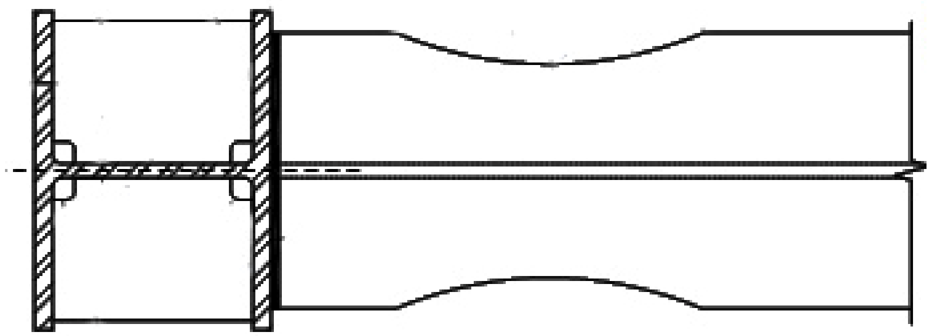

FIGURE 5: REDUCED BEAM SECTION

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.

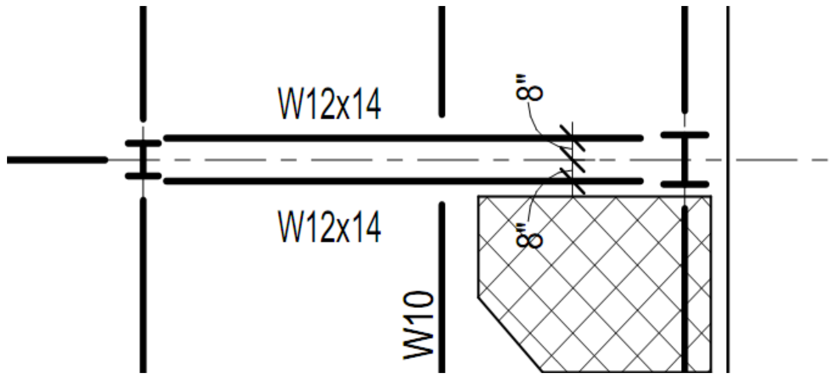

FIGURE 6: PARALLEL BEAMS USED FOR PLUMBING

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

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

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

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

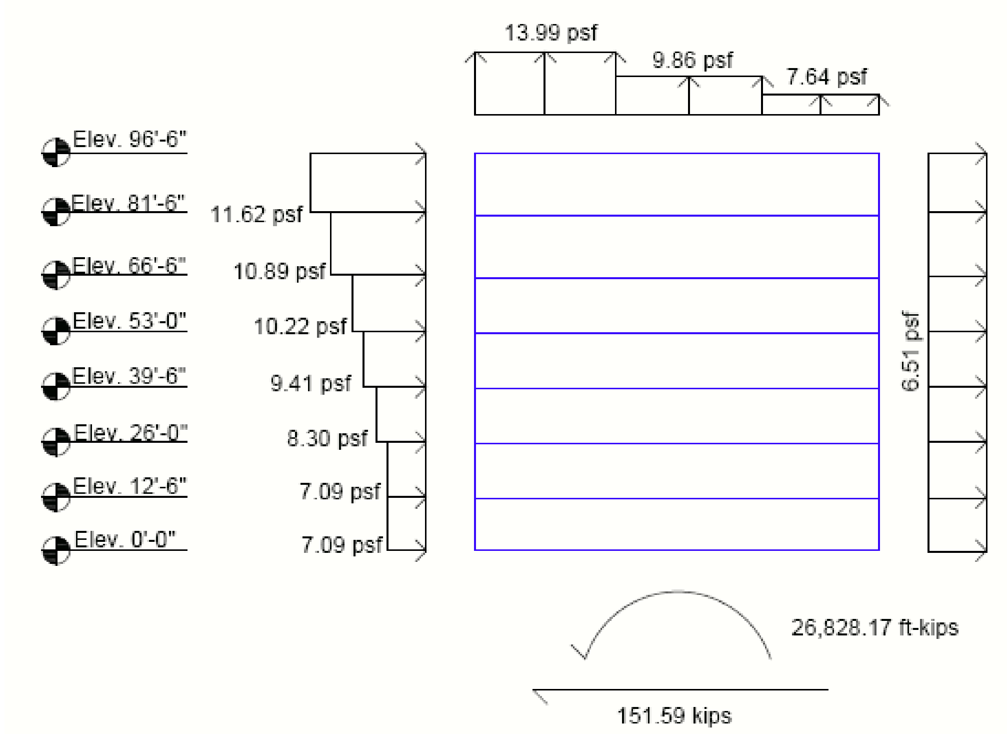

FIGURE 7: WIND LOADING DIAGRAM (NS) SHOWING WIND PRESSURES

TABLE 5: WIND LOADS – EW DIRECTION

Wind Loads - EW Direction					
Floor Level	Floor Height (ft)	Elevation (ft)	Story Force (kips)	Total Story Shear (kips)	Overturning Moment Contribution (ft-k)
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

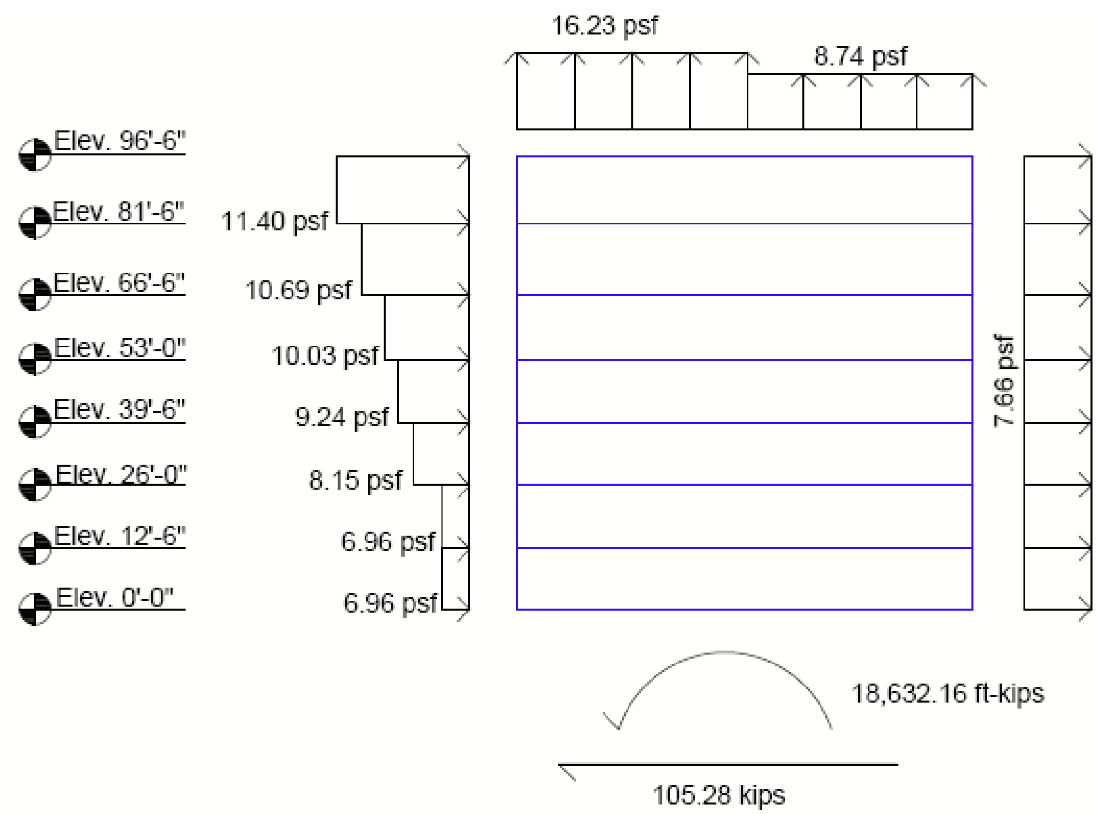

FIGURE 8: WIND LOADING DIAGRAM (EW) SHOWING WIND PRESSURES

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		

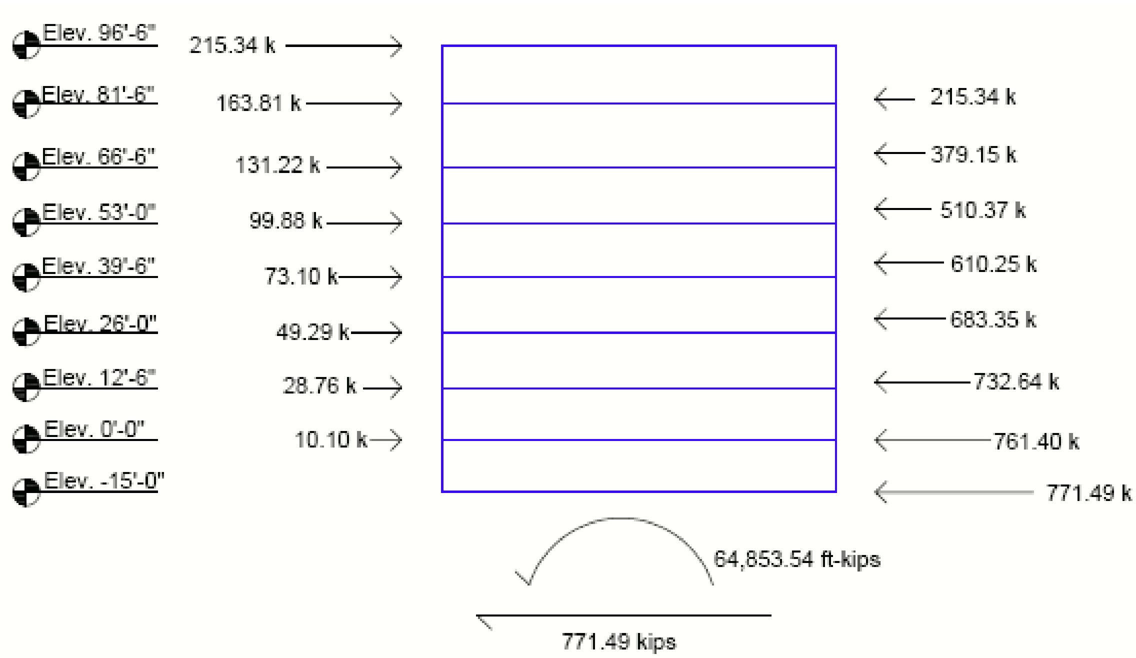

FIGURE 9: SEISMIC LOADING DIAGRAM (BOTH DIRECTIONS) SHOWING STORY FORCES (LEFT) AND STORY SHEARS (RIGHT)

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. $\boxed{1.2D + 1.0E + L + 0.2S}$
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

		Structural Performance Levels		
Elements	**Type**	**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 40.

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.

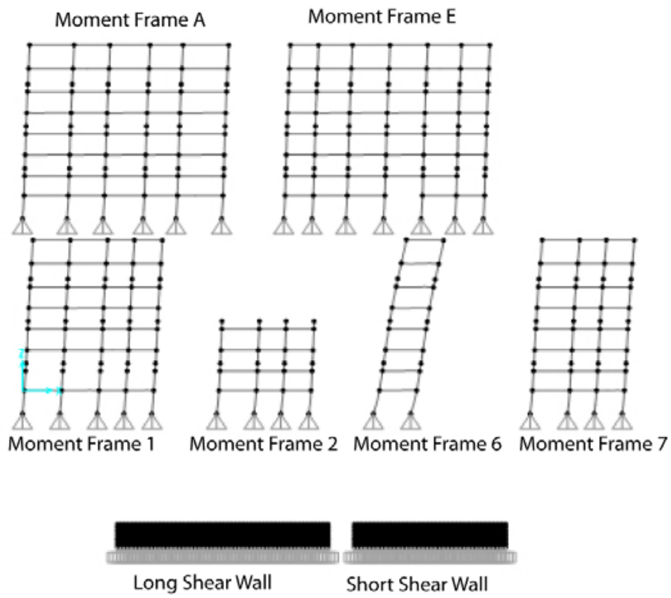

FIGURE 14: 2D MOMENT FRAME MODELS

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.

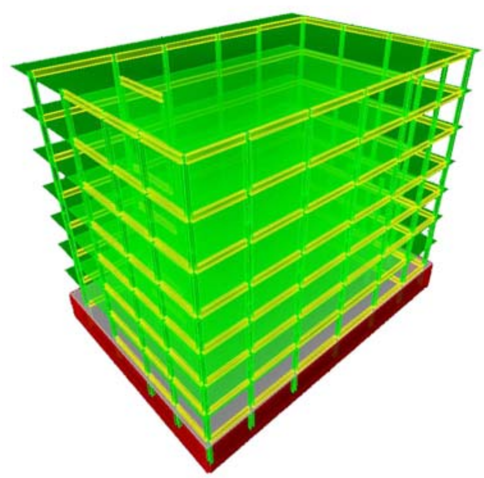

FIGURE 15: 3D LATERAL SYSTEM MODEL

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 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.

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

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.

$$\boldsymbol{\bar{X} = \frac{\sum k_{iy} x_i}{\sum k_{iy}}} \qquad \boldsymbol{\bar{Y} = \frac{\sum k_{ix}\, y_i}{\sum k_{ix}}}$$

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.

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

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**					
Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.001005	-0.02633	-0.02532	-2.532	100.35
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	
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	100.00
Grid Line E	0.000796	0	0.000796	0.080	
Basement - long west	0.499231	0	0.499231	49.923	
Basement - long east	0.499231	0	0.499231	49.923	

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

$F_i = F_{idirect} \pm F_{itorsion}$ **4th Floor - Roof**					
Moment Frame	$F_{idirect}$	$F_{itorsion}$	F_i	% Load	Check
Grid Line 1	0.458388	0.018751	0.477139	47.714	99.11
Grid Line 6	0.145529	-0.00597	0.13956	13.956	
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.

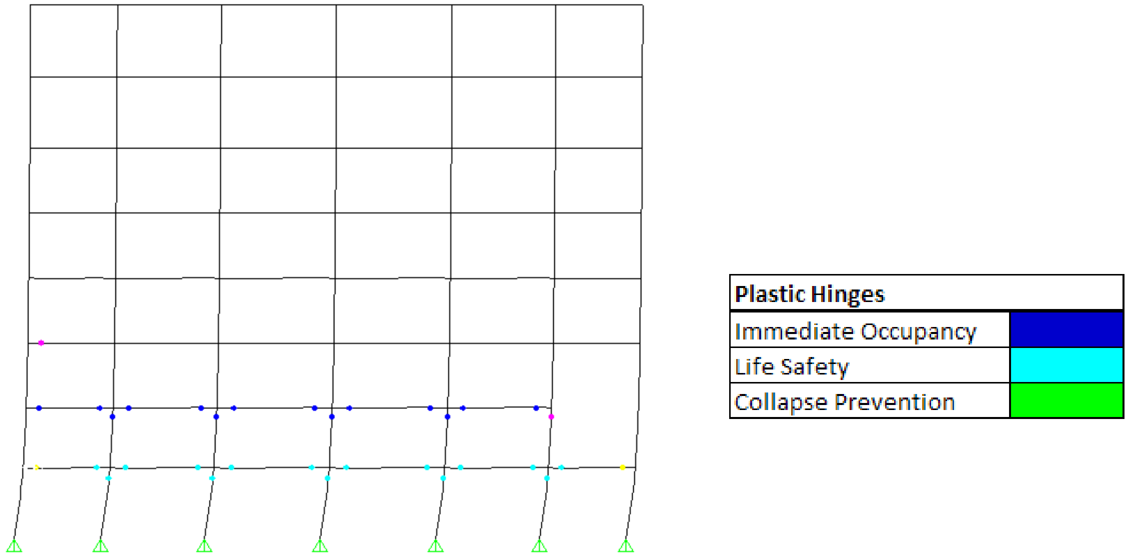

FIGURE 16: FORMATION OF PLASTIC HINGES ON FRAME E

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.

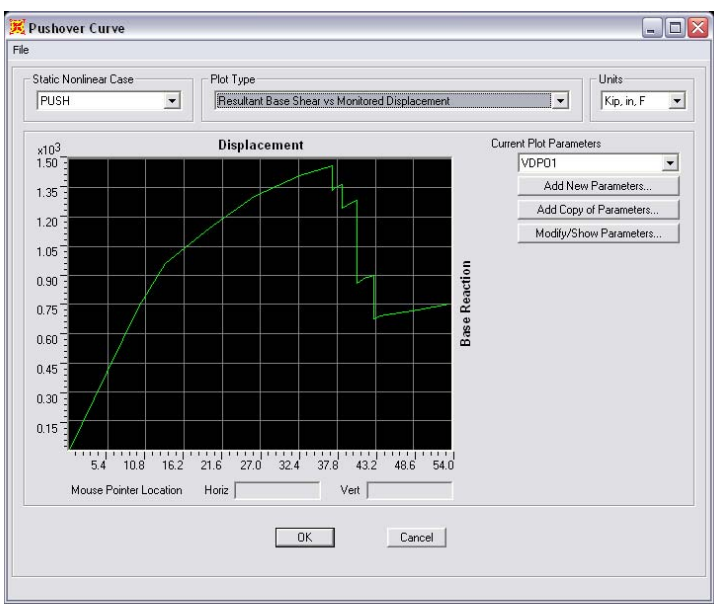

FIGURE 17: BASE SHEAR VS DISPLACEMENT PUSHOVER CURVE

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.

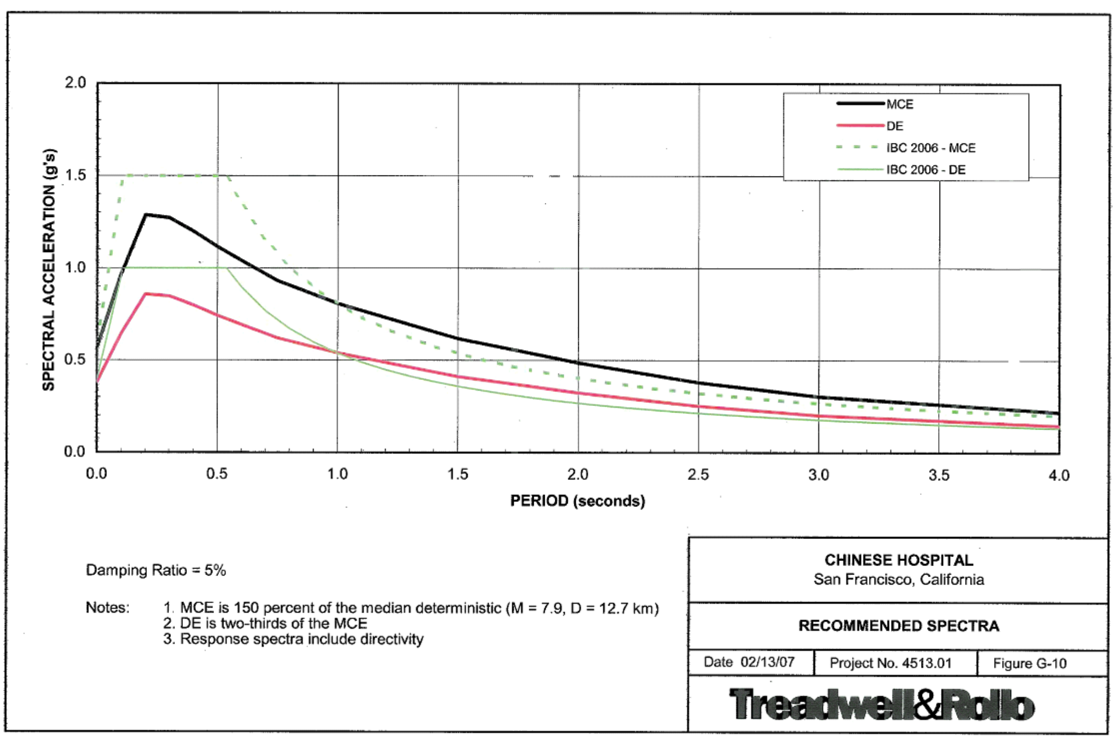

FIGURE 18: RESPONSE SPECTRUM DEVELOPED BY TREADWELL&ROLLO

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.

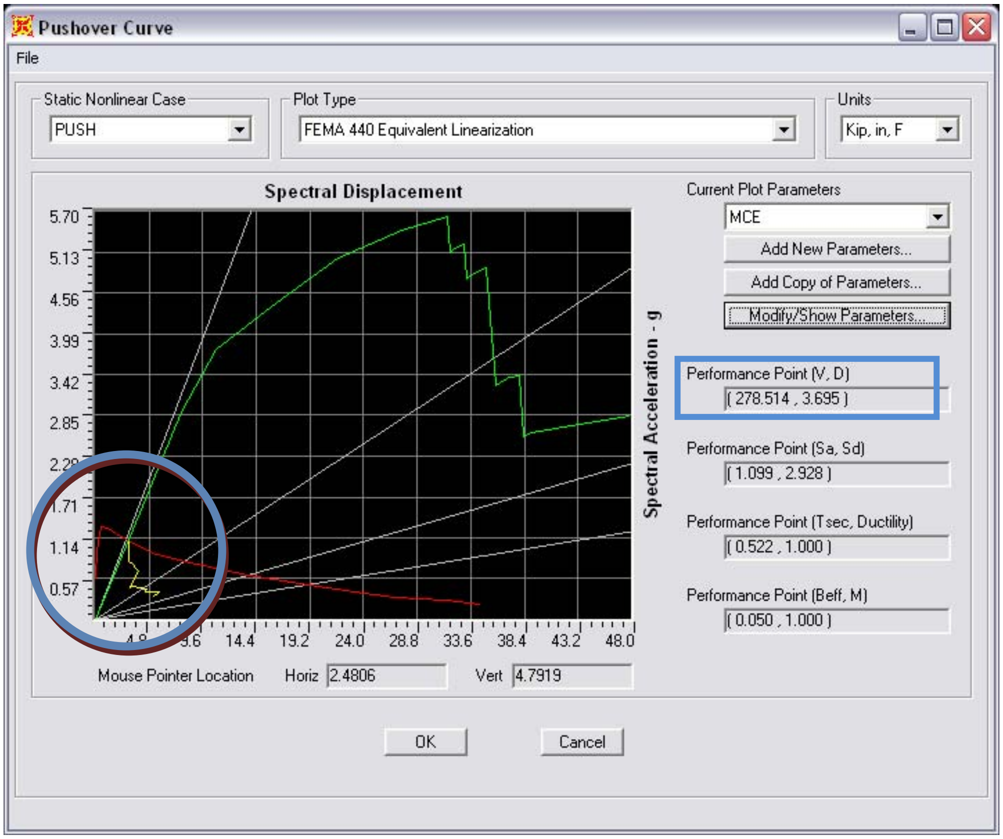

FIGURE 19: PUSHOVER CURVE SHOWING PERFORMACE POINT

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

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

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.

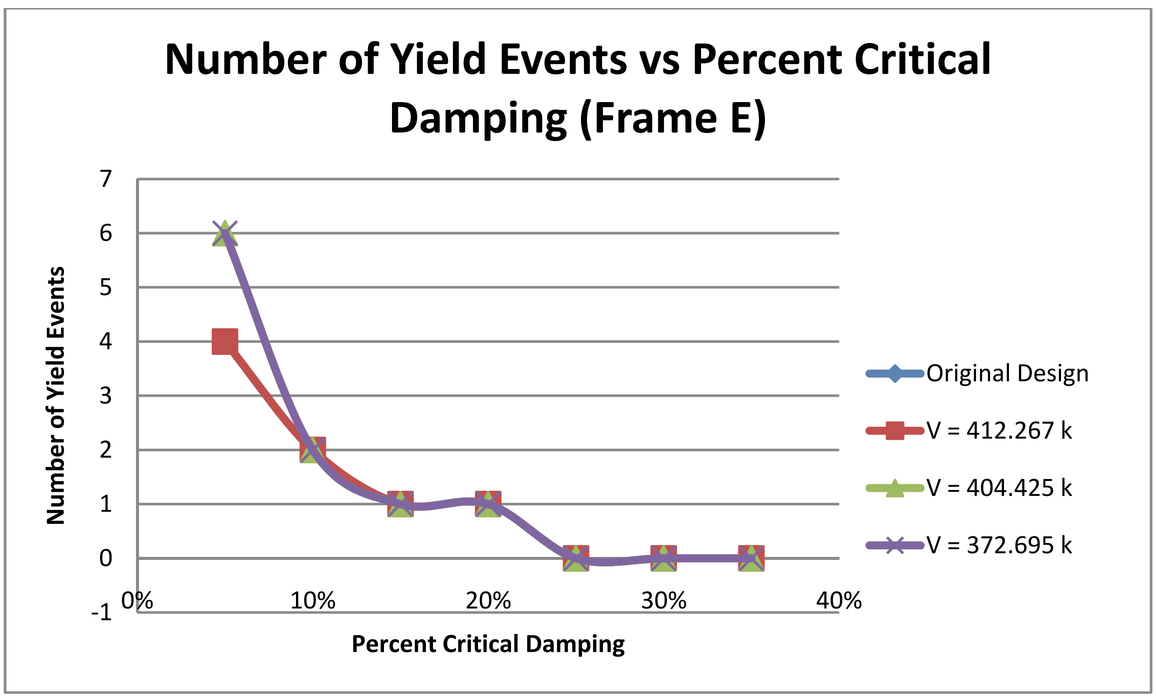

FIGURE 20: NUMBER OF YEILD EVENTS VS PERCENT DAMPING FOR FRAME E

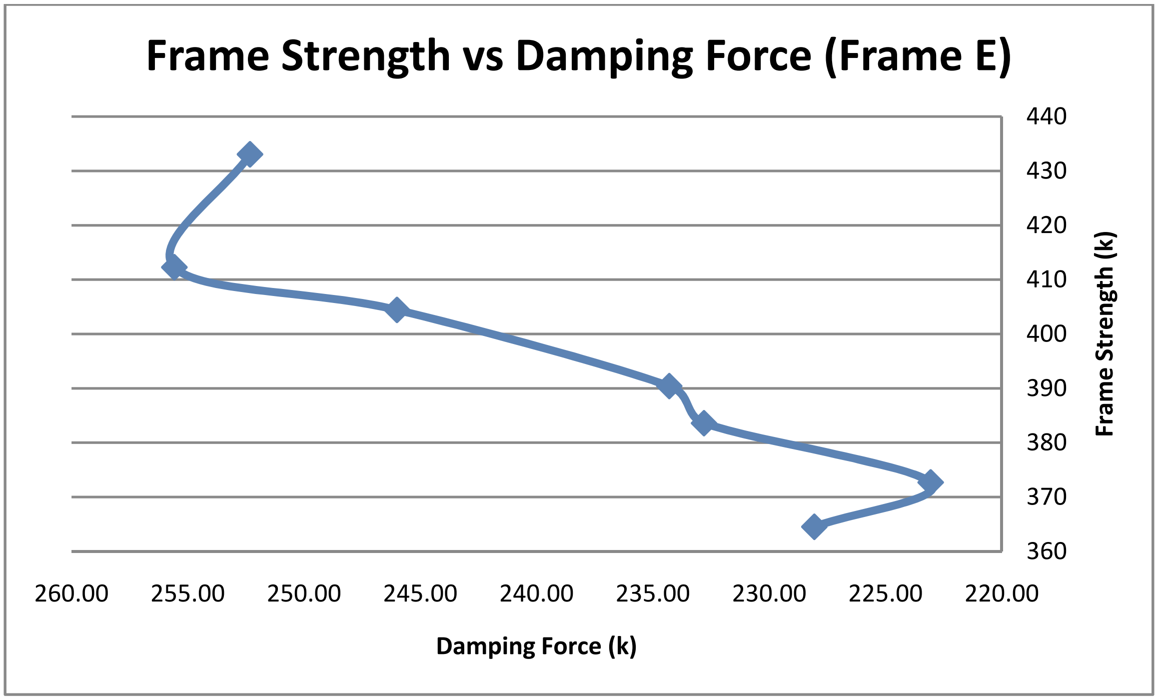

FIGURE 21: FRAME STRENTH VS DAMPING FORCE FOR FRAME E

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.

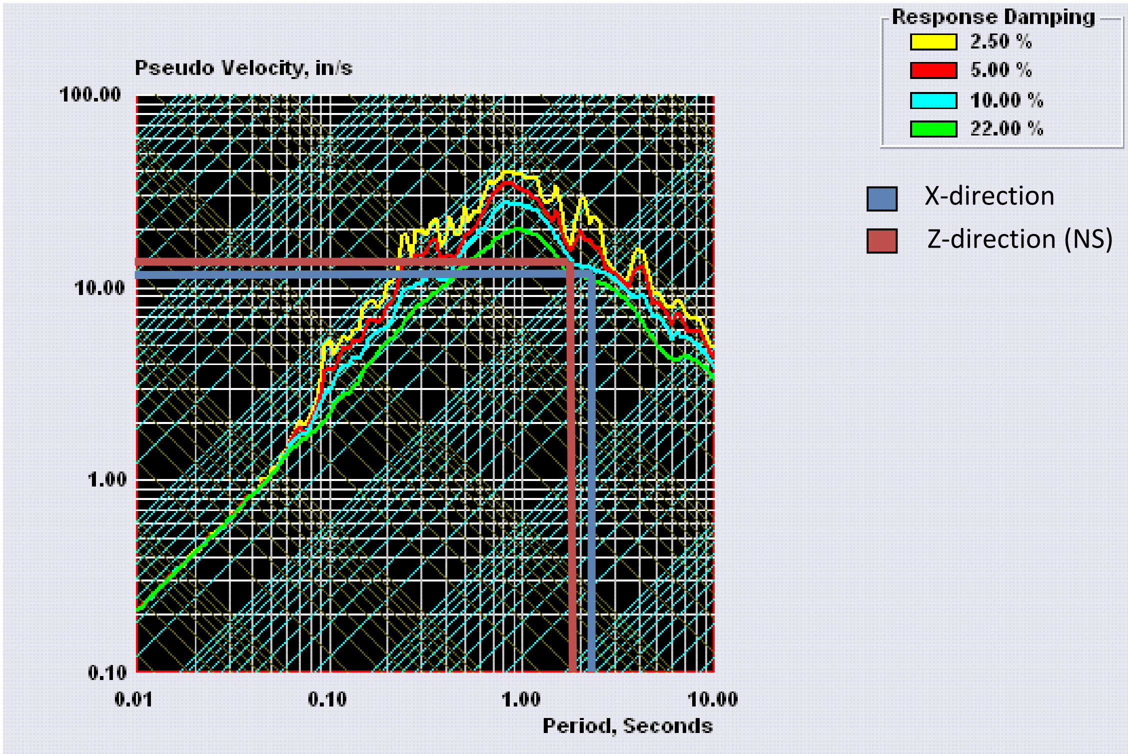

FIGURE 22: EARTHQUAKE RESPONSE SPECTRUM FOR NORTHRIDGE 1994

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.

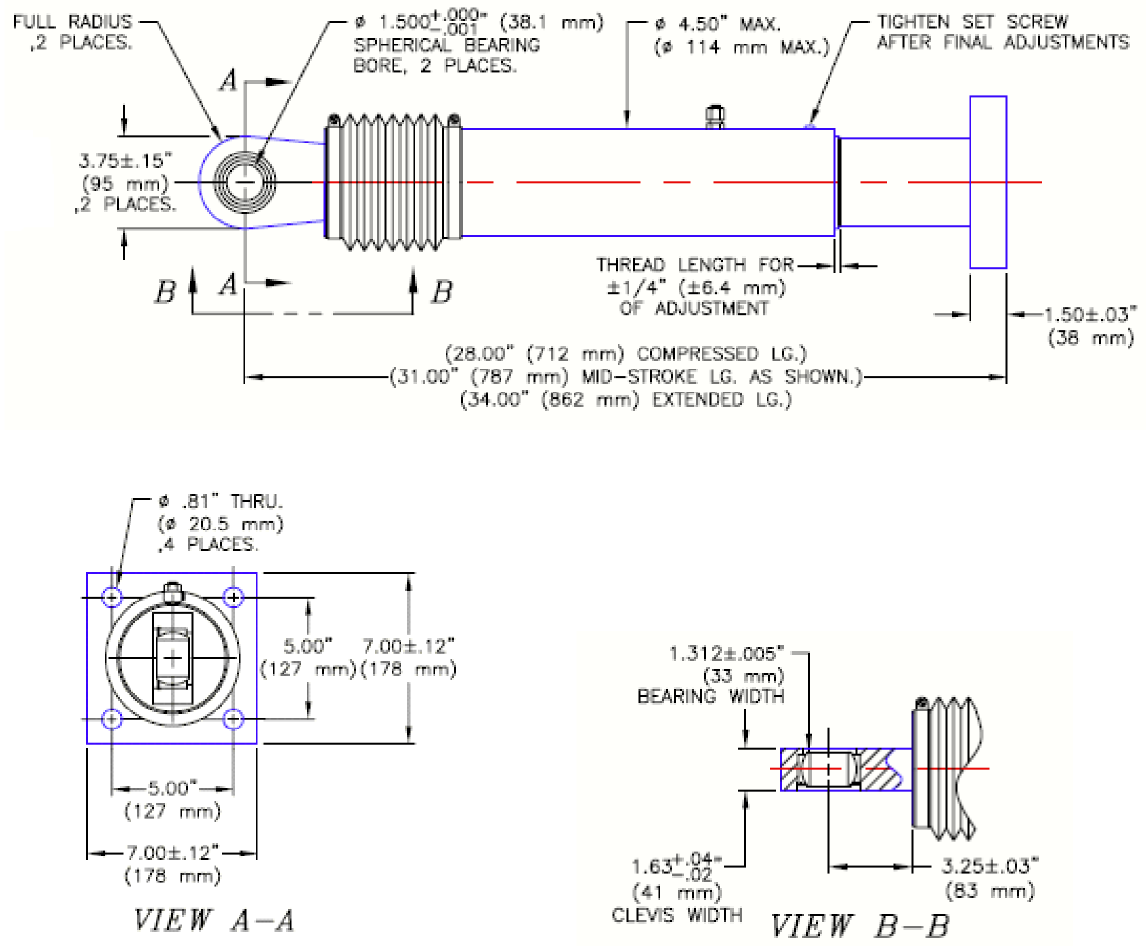

FIGURE 23: TAYLOR DEVICES 55 KIP FLUID VISCOUS DAMPER

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.

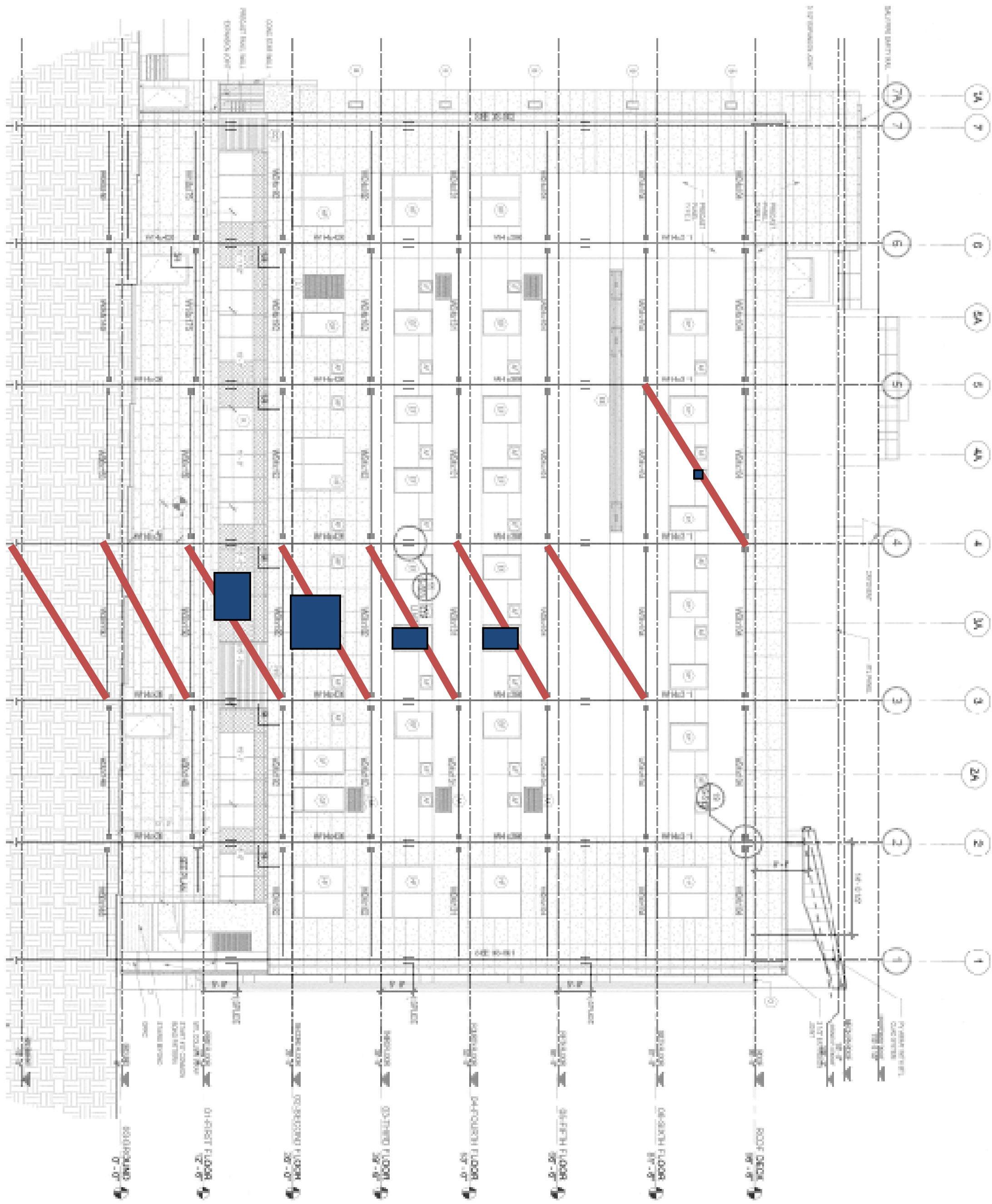

FIGURE 24: FRAME E DAMPER DEVICE IMPLEMENTATION AND IMPACT HIGHLIGHTED IN BLUE

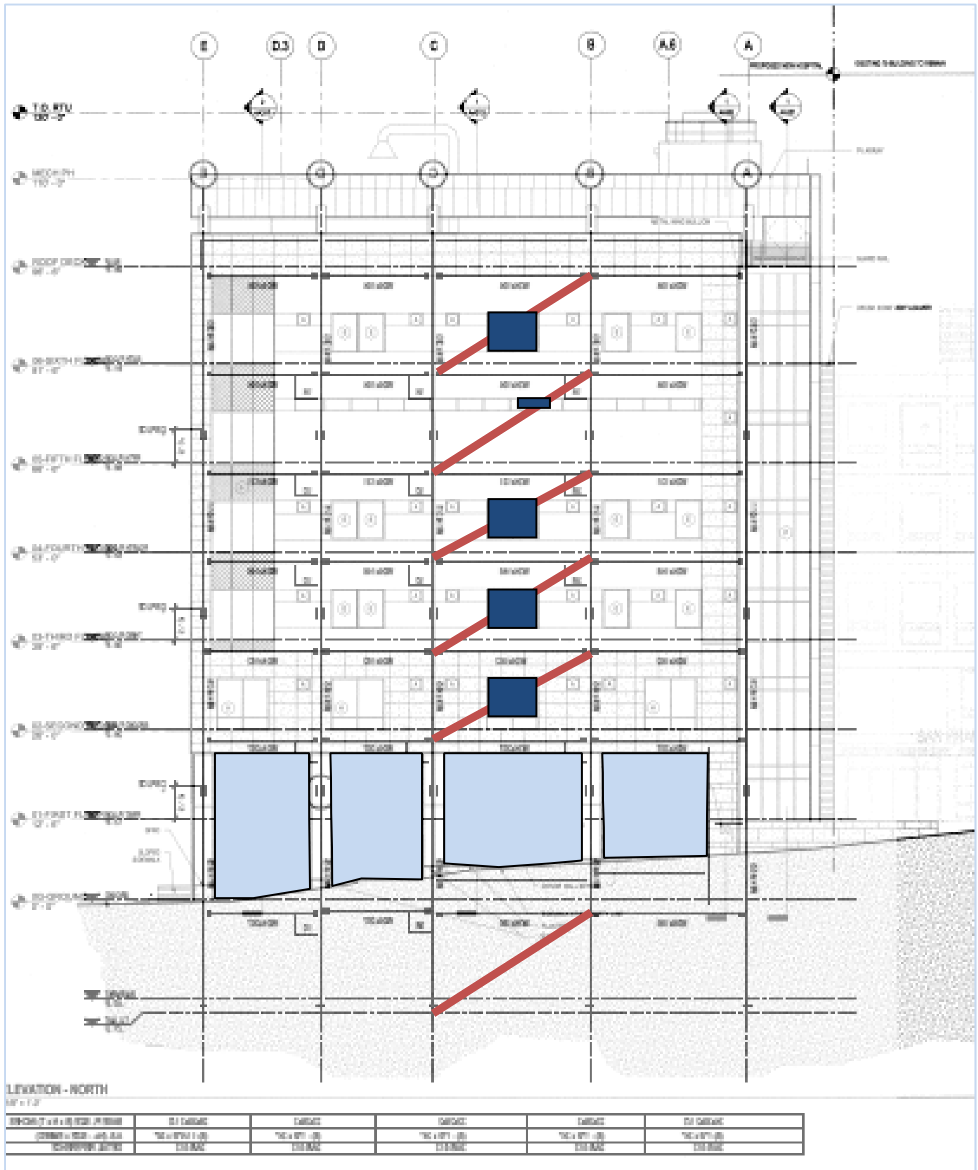

FIGURE 25: FRAME 1 SHOWING DAMPER IMPLEMENTATION AND IMPACT HIGHLIGHTED IN BLUE

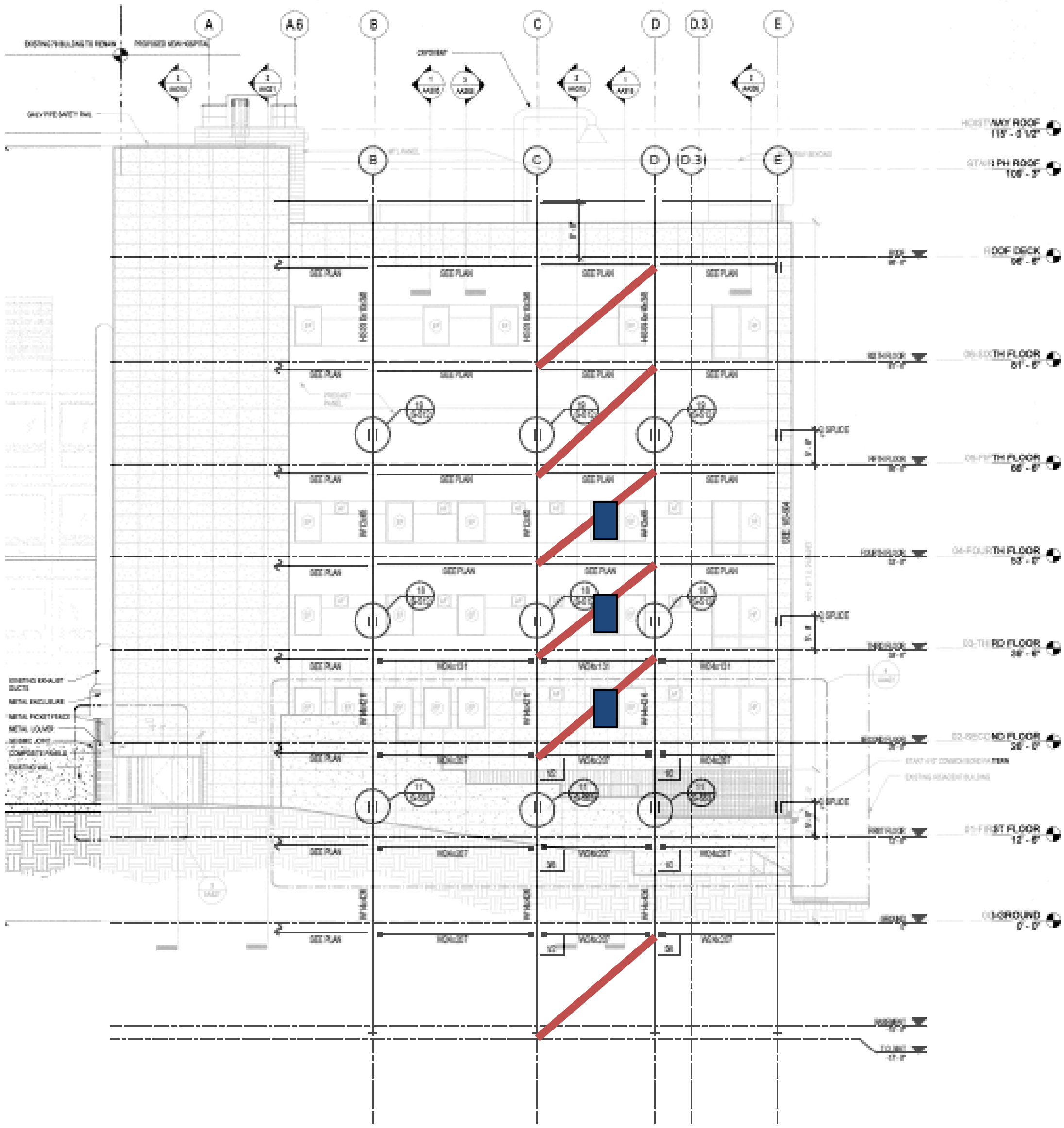

FIGURE 26: FRAME 7 SHOWING DAMPER IMPLEMENTATION AND IMPACT HIGHLIGHTED IN BLUE

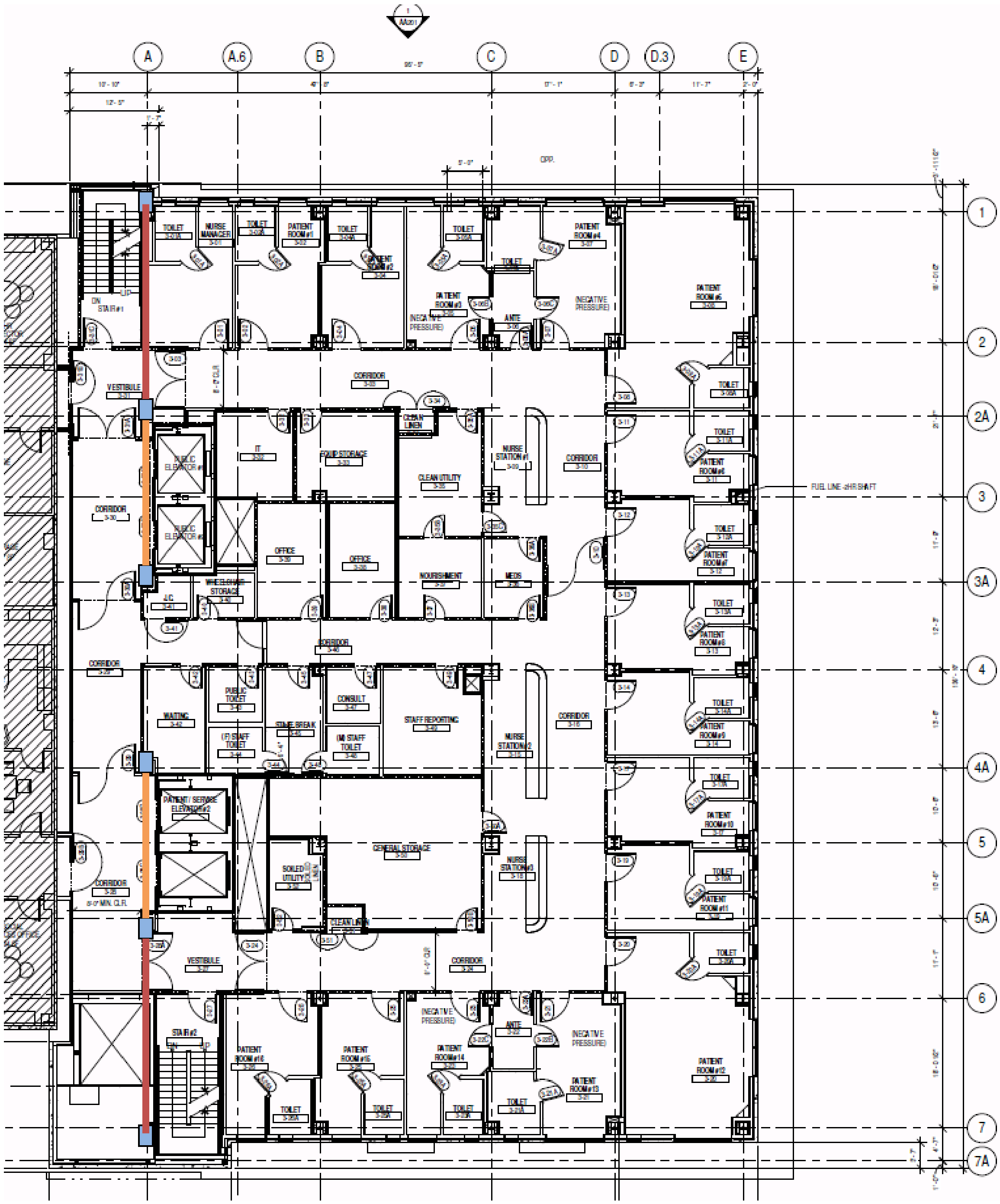

FIGURE 27: TYPICAL FLOORPLAN WITH COLUMNS IN BLUE, AREAS WHERE DAMPERS MAY NOT BE PLACED IN ORANGE, AND FINAL DAMPER PLACEMENT IN RED.

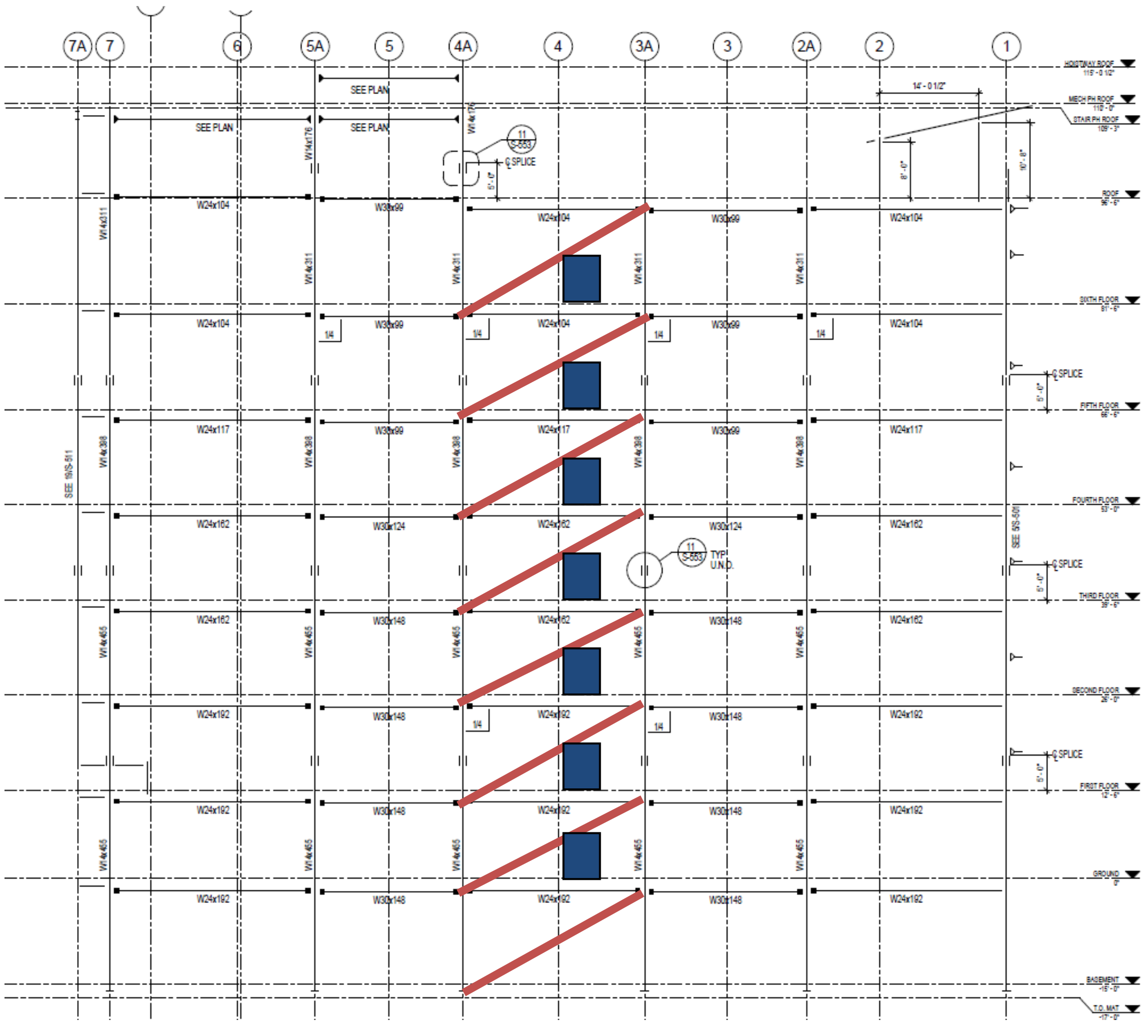

FIGURE 28: DAMPER IMPLEMENTATION IN FRAME A

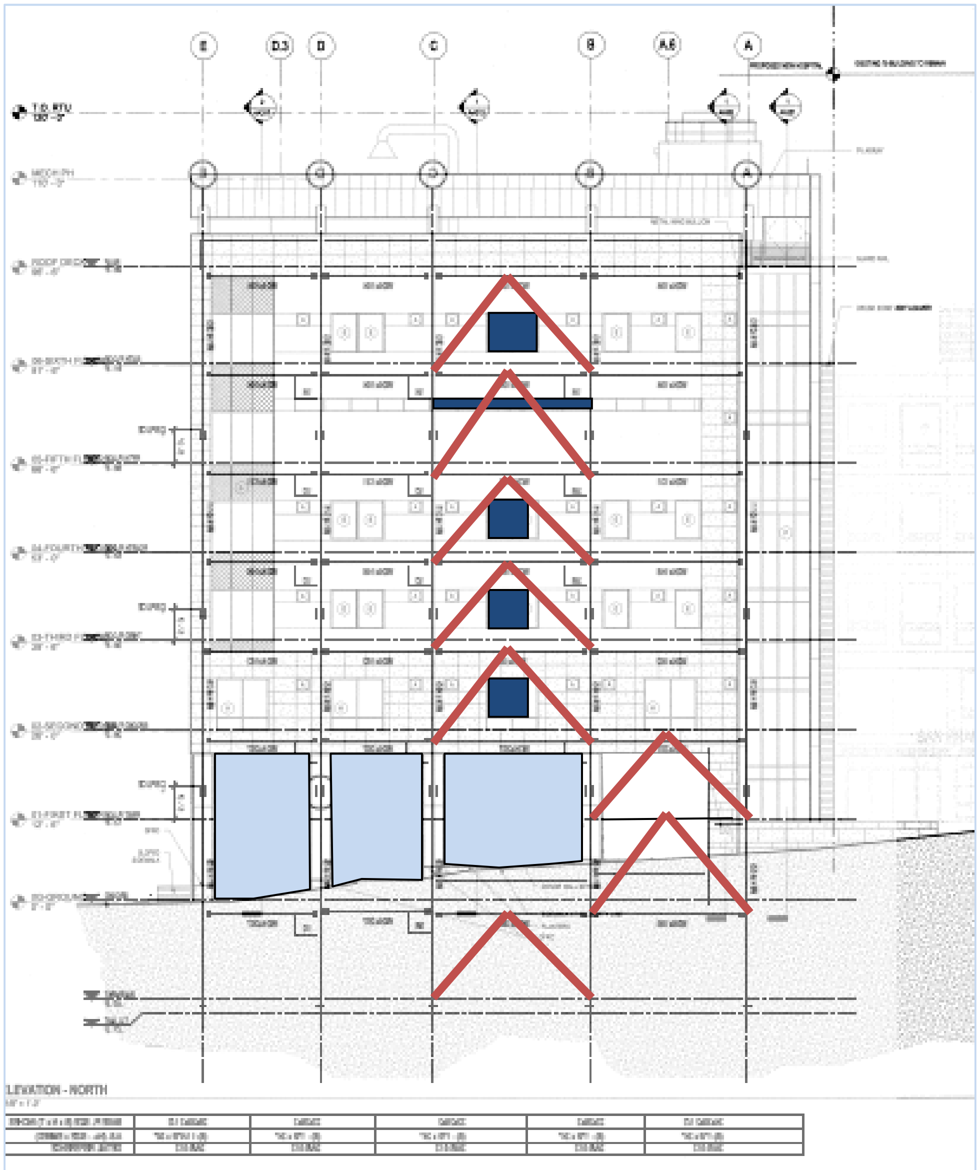

FIGURE 29: CORRECTION OF SOFT STORY EFFECT USING ADDITIONAL BRACES.

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.