Dynamic Design

A Lateral System Investigation and Redesign

University Hospitals
Case Medical Center
Cancer Hospital

11100 Euclid Avenue Cleveland, Ohio

April 7, 2009

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Structural Option
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http://www.engr.psu.edu/ae/thesis/portfolios/2009/dcm234

University Hospitals Case Medical Center Cancer Hospital

Cleveland, Ohio

PROJECT TEAM

Owner: University Hospitals, Case Medical Center
Construction Manager: Gilbane
Developer: Still researching
Architect: Cannon Design
Structural Engineer: Cannon Design
Lighting Consultant: Cannon Design
MEP Consultant: Cannon Design

ARCHITECTURAL DESIGN

- 9 story, 370,230 SF Cancer Hospital located on the University Hospitals Case Medical Center in Cleveland, Ohio
- Of the 92,000 SF of curtain wall, the entire east and west elevation constructed using a custom sloped – wall system
- A universal grid system consisting of 31'-6" modular bays has been incorporated into design to optimize floor space for varying use
- Roof system consists of sealed PVC assembly enveloping a 6-1/4" thick composite steel deck

STRUCTURAL SYSTEM

- Steel infrastructure consisting of composite beams and slabs
- Foundation consists of drilled piers transferring load to 3000psi caissons
- Concentrically braced "chevron" frames resist lateral loads controlled by wind

MECHANICAL SYSTEM

- Variable Air Volume System System
- 7 air handling units supplying between 15,000 and 60,000 cubic feet per minute to seven different designated building zones
- Hydronic Radiant Floor and Snow Melt System

ELECTRICAL SYSTEM

- 277/480V, 3 Phase, 4 Wire System for supplying mechanical and high powered research equipment
- 120/208V, 3 Phase, 4 Wire System for used by equipment requiring only standard loads
- Two 1200A bus duct risers

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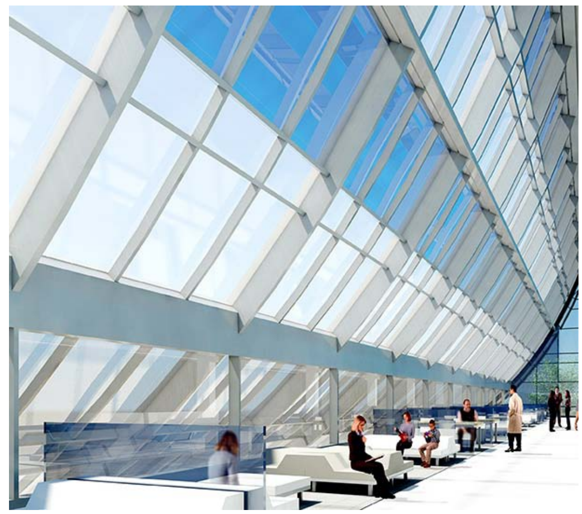
Daniel C. Myers *Structural*

Executive Summary

As an integral part of the University Hospital Vision 2010 expansion plan, a new Cancer Hospital will be constructed on The University Hospitals Case Medical Center Campus in Cleveland, Ohio. The Cancer Hospital is a 9 story, 370,230 SF research and patient care facility. Its infrastructure consists of steel and steel composite members which have been carefully arranged in order to conform to the modular architectural design system known as the *Universal Grid*, allowing full optimization of available space for varying use. Sloped curtain walls envelope the Cancer Hospital, consisting of exterior glazing and curved steel. The new Cancer Hospital will serve as an addition to the adjacent Case Medical Center which will combine medical services once spread through 7 different buildings.



The design of the Cancer Hospital has been evaluated in 3 previous *Technical Reports.* The lateral force resisting system analysis of *Technical Report 3* revealed several key areas of concern which were determined to be caused by the irregular geometry of the building. In order to provide a unique opportunity to further study the efficiency of this irregular design under more complex and dynamic seismic loads, the Cancer Hospital has been theoretically reproduced and relocated from Cleveland, Ohio to San Diego, California. The relocation of the design will allow the cancer research services provided primarily to the east coast by the Cleveland, Ohio location, to also be provided to the west coast, through the new San Diego, California location.

To maintain the feasibility of the theoretical location, all current Vision 2010 project requirements have been followed. To accomplish this, a thorough investigation has been conducted of 3 different commonly used seismic loading solutions in mid-rise buildings including the strengthening of the existing structure, the creation of a seismic isolation joint, and the use of a reinforced concrete core. Upon comparison of the results, the concrete shear wall core was found to be the optimal system and has been designed for strength and serviceablility under the new San Diego, California parameters. Lateral elements which have been redesigned include the concrete shear wall core, the steel eccentric braced frames, and the building foundations. A critical connection design has also been performed in accordance with the *Masters Requirement*. Loads used in the investigation and redesign have been determined in accordance with ASCE 7-05 and IBC 2006. ETABS models have been created and verified for accuracy for each investigation and the final design.

In addition to the design of the new lateral system, a building envelope study will be preformed in order to allow the use of the Cancer Hospital's most commanding architectural feature, the 92,000 SF curtain wall. Upon completion of the redesign of exisiting systems, a cost and schedule has also been performed finalizing the conclusion that the new Caner Hospital adheres to the Vision 2010 plan and is acceptible for service of the west coast.

Acknowledgements

My A.E. Classmates for all the help and support over the past five years. Our time together will be missed.

The Pennsylvania University A.E. Faculty for the unmesurable encouragement and valueable instructruction which make our futures so bright.

Dr. Memari for being a great personal thesis mentor.

I would like to thank *Cannon Design* for supplying all plans and building information request in a timely manner.

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Introduction

University Hospitals is a world renown health system specializnig in cutting edge treatment and research facilities for over 140 years. Currently, University Hospitals continues to lead in healthcare innovation with the addition of new cancer research facilities in various locations around the word under an expansion plan they have named Vision 2010. The Cleveland Case Medical Center Campus located in Cleveland, Ohio was identified under the Vision 2010 plan to receive a new Cancer Hospital.

The design of the Cancer Hospital has been analized in 3 previous *Technical Reports* including a lateral system analysis in *Technical Report 3*. This analysis indentified key areas of concern which were primarily associated with the irregularity of the builiding geometry and structural systems. The new Cancer Hospital will service mostly eastern coast clientele due to its location. In order to allow for services to be efficiently provided to the the entire United States, a location in San Diego, California has been proposed to provide service to the west coast. This will create a unique oportunity to study the behavior of the irregular geometry of the Cancer Hospital design under more complex dynamic earthquake loads and consequently facilitate the creation of a new lateral system design.

In order to maintain the feasiblility of such a porject, all the Vision 2010 constraints will be followed. This will consist of maintaining a design as close as possible to the origional structural and architectural plans in effort to decrease the amount of time needed for redesign. To accomplish this task, 3 common seismic load resisting structural system solutions will be evaluated including the strengthening of the existing structure, the creation of an seismic isolation joint, and the use of a reinforced concrete core. Upon a thorough evaluation of the solutions, the lateral system which is most efficient to carry the seismic loads and impacts the exisitng structure and architecture the least will be selected for design. In addition to the design of the new lateral system, a building envelope study will be preformed in order to allow the use of the Cancer Hospital's most commanding architectural feature, the 92,000 SF curtain wall. Upon completion of the redesign of exisiting systems, a cost analysis and schedule has be performed in order to ensure adherence to the Vision 2010 plan.

Building Background

Architecture

The University Hospitals Case Medical Center Cancer Hospital will integrate patient care and cancer research in a new and innovative way. Architecturally, the Cancer Hospital will reflect this cutting edge link by joining adjacent buildings together while serving as a primary gateway to the UHCMC campus located in Cleveland, Ohio (see Figure 1)

Figure 1

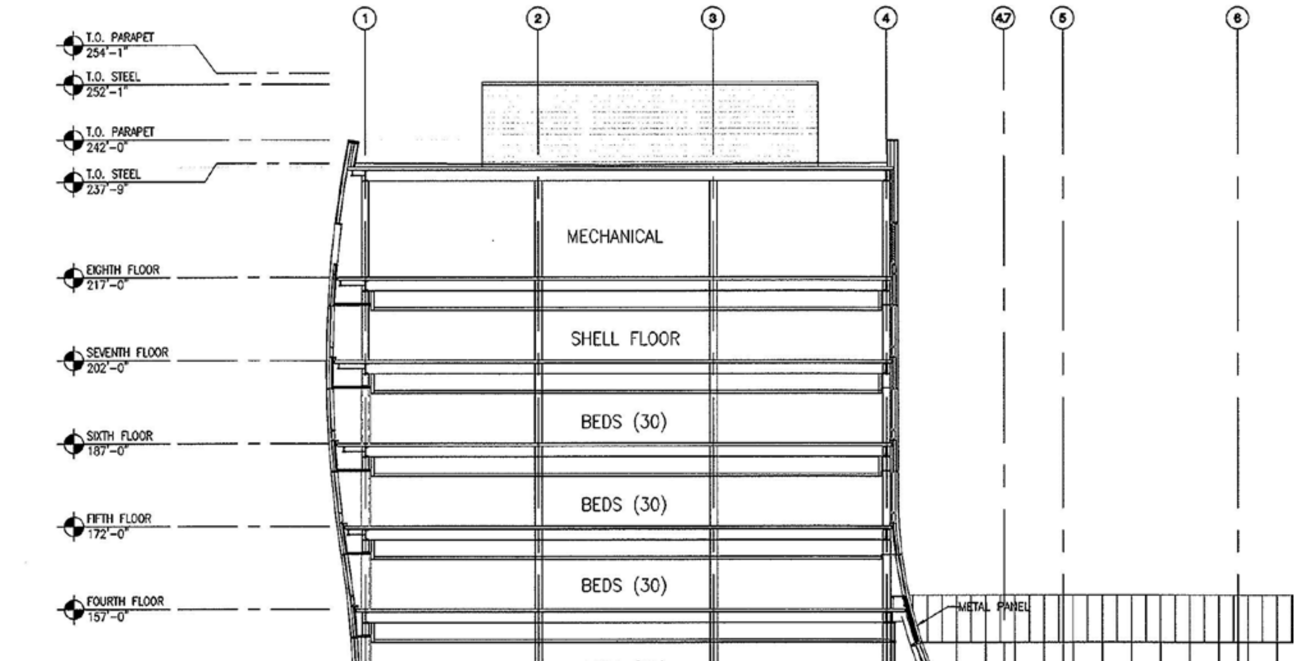

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Figure 2

The Cancer Hospital design fulfills the wishes of former facility cancer patients in creating an appealing and comfortable environment as opposed to the sterile feel of the past. This is accomplished through use of strong architectural accents including the Cancer Hospital's most dominating feature, its curved facade. A universal grid system consisting of 31'-6" modular bays has been incorporated into design to optimize floor space for varying uses. Clinical pods have been designed for treatment of specific patient populations (see Figure 2).

Medical services which were previously distributed among seven facilities will now be performed under one roof to optimize cancer research, education, and patient care while providing an architecturally appealing exterior as well as a warm and inviting natural interior.

Site Transportation

The existing site is located at the intersection of Cornell Road and Euclid Avenue on the University Hospitals Case Medical Center Campus located in Cleveland, Ohio. The site design utilizes access points from both main roads and integrates pedestrian and vehicular flow with the UHCMC campus (see Figure 3)

Transportation

Two main public entrances are located on the north and south sides of the new Cancer Hospital. 3 main corridors lead to the existing hospital adjacent to the east. A main tunnel below the entry drive has also been provided in order to facilitate flow to and from the Cancer Hospital from the rest of the UHCMC campus. Elevators are centralized in the building consisting of 4 for public use, 3 enlarged models for inpatient movement, and 3 for equipment relocation. 6 stairwells are placed at the corners of the building and at the centers of the north and south facing sides. The inpatient drop-off and main receiving area is located at the south entrance of the Cancer Hospital. The ambulance drop-off is located in the north east corner of the building, directly off the main road.

Figure 3

Construction

The Cancer Hospital encompasses 370,230 SF of the University Hospitals Case Medical Center Campus with its 9 above grade stories rising 172’-1” in height. The new Cancer Hospital and its four additional building counter parts makeup the UHCMC’s Vision 2010 project which is expected to be completed at a total cost of $1 billion under a single prime contract. The Cancer Hospital addition alone makes up $232 million of the Vision 2010 price.

Construction of the New Cancer Hospital will begin July 2008. The total time until completion is projected to be 17 months. This places the opening date at December 2010, which will comply with the Vision 2010 time constraints.

The design-bid-build project delivery method has been utilized for the construction of the Cancer Hospital. Special consultants and sub-contractors have been hired for specific items not covered under the scope of the general contractor. One of which, Wheaton & Sprague Engineering, a cladding consultant, has been awarded the task of competing the special construction and detailing required for the exterior curved façade.

Building Envelope

92,000 SF of curtain wall envelopes the new Cancer Hospital. The entire east and west elevations have been constructed using a custom sloped-wall system consisting of non-gravity bearing curved steel. The roof system consists of a sealed PVC assembly enveloping a 6-1/4" thick composite steel deck.

Mechanical System

A Variable-Air-Volume or VAV mechanical system is used in the new Cancer Hospital. 7 air handling units supply between 15,000 and 60,000 cfm to seven different designated building zones. A typical zone consists of a supply fan operating at 1720 rpm and supplying 25,000 cfm, as well as a return fan operating at 1100 rpm and returning 22,500 cfm. Both fan units in each zone comply with ASHREA standards for sound power level. The typical cooling coil has a capacity of 2081 MBH and pumps 255 gpm. *Air Enterprises* is the primary manufacturer for the equipment provided in the mechanical system. All units in the mechanical system have an emergency backup to be used if necessary. In addition to the primary mechanical system, a Hydronic Radiant Floor and Snow Melt System has also been incorporated into the new Cancer Hospital.

Electrical System

The electrical system in the Cancer Hospital is made up of 2 4000A main breakers. Current travels to the upper floors through 2 480Y/277V 1200A aluminum bus ducts. The main transformer size has been listed as per the electrical utility (owner). Each floor is equipped with a transformer for step down to a 208Y/120V distribution panel. This panel then distributes power to all assigned branch panels. In order adequately supply vital power to the hospital under any circumstance, a life safety branch, a critical branch, and an emergency standby branch pane has been provided in the system.

Lighting

Fluorescent lighting has been used throughout the Cancer Hospital in order to lower the overall energy consumption. Specific details and placement of luminaries are not listed on the provided schematic drawings and have been withheld by the owner.

Fire Protection

The new Cancer Hospital is falls fully under occupancy category I-2 with its primary use being a hospital. The building has both active and passive systems consisting of a full coverage sprinkler system, smoke compartments on each floor including a five story atrium, and fire walls placed as appropriate throughout. Standpipes are located at the base and each level above. All load bearing elements supporting more than one floor are fire rated for 2 hours with the exception of column members, which are rated at 3. A "Fire Command Center" is located at the center of the Cancer Hospital to allow for quick action and response to any fire related incidents.

Special Systems

Special consideration has been made in construction to accommodate high profile research and medical equipment located on the sub-basement floor. Protective partitions and enclosures have been used to shield occupants from hazards such as radiation produced due to this equipment.

Telecommunication

The telecommunication system includes standard phone jacks provided for patients, an intercom and loudspeaker system for public address, and a video intercom system at specific locations for broadcasting medical research and procedures.

Existing Structural System

Foundation

The Cancer Hospital consists of drilled piers transferring load to caissons for the gravity columns with the combined use of grade beams for the lateral force resisting frames. The drilled gravity piers/caissons range 30" to 60" in diameter depending on location. The drilled piers/caissons receiving lateral load are typically 66" in diameter. Along the south side, 36" thick spread footings, typically 48" by 72", have been used to carry gravity load along the existing adjacent Case Medical Center Hospital. The grade beams which carry the lateral load to the drilled piers/caissons are typically 24" by 24" and consist of Grade 60, #7 reinforcement bars. All foundations are made from concrete having a compressive strength of 4000psi with the exception of the caissons and spread footings, which have a strength of 3000psi.

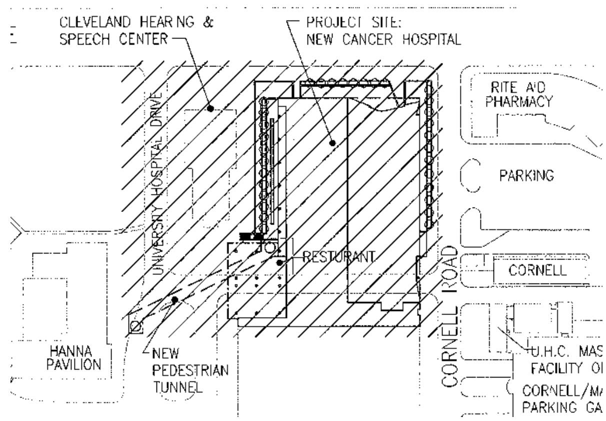


Figure 4

The soil on site has been classified as hard shale (see Figure 4). Thus, giving the caissons used in the foundation an end bearing capacity of 50kpf with a skin friction capacity of 10psi below the first 5' of shale. The typical minimum penetration depth for the gravity piers/caissons is 3'-0" and for the lateral, 16'-6".

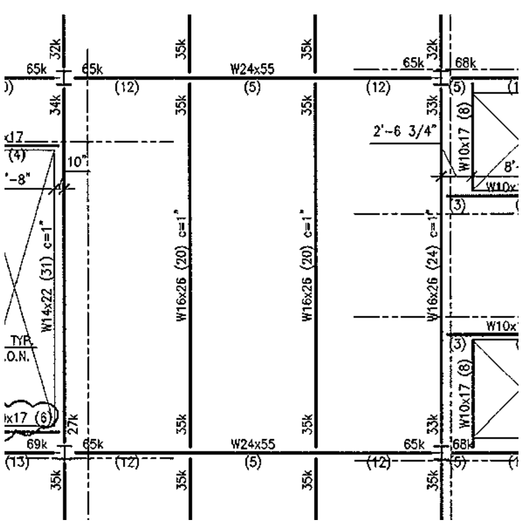


Figure 5

Framing

Being a primarily steel structure, the Cancer Hospital has a fairly typical composite steel beam and girder framing system (see Figure 5). The typical composite floor slab is 5-1/4" thick using 3000psi lightweight composite concrete, an 18 gauge 2" galvanized steel deck, and 3-1/2" metal studs. This composite floor slab is used on all but the 2nd and 8th floors. The second floor requiring a thicker slab with normal weight concrete due the vibration requirements of the surgery and imaging rooms and the 8th due to the increased load from the mechanical system. The slab used on these floors consists of 6-1/2" thick 3000psi normal weight concrete, an 18 gauge 2" galvanized steel deck, and 3-1/2" metal studs. Both decks are reinforced with 6x6 Welded Wire Fabric; W4.5xW4.5 for the first floor, W3.5xW3.5 for the second and eighth floors, and W2.1x2.1 for the remaining floors.

Bay sizes conform to the universal grid, having a typical size of 31’-6” by 31’-6”. Infill beams are typically W16x26 around the interior and W14x22 around the exterior framing into W24x68 girders (see Figure 5). For the larger breaks in the slab, such as the elevator shafts, HSS 8x4x1/4 tubes have been used. On the 4th and roof level, moment connections are utilized in conjunction with cantilevered beams in order to support the curved exterior façade. Smaller breaks used for mechanical, plumbing, etc., consist typically of W10x17. Columns consist of a typical W14 member decreasing in size with elevation and spliced every other floor starting with the second. All steel members conform to ASTM A-992, Grade 50 unless otherwise noted.

At the ground level, a 6” thick slab-on-grade is used with Grade 60 #5 reinforcement bars spaced @ 18”oc EW. The slab rests on a 10 mils min. vapor barrier on compacted granular material over a 2000psi mud slab. In the northeastern and southeastern section of the building special research equipment has been placed requiring a 12” thick slab-on-grade with Grade 60 #5 reinforcement bars placed @ 12”oc EW.

A 31’-0” by 63’-0” machine room is located on the 8th floor. Framing is similar to the rest of the structure however with shorter spans and larger members to account for the additional weight. Beams range from W21 beams to W40 beams depending on specific equipment.

Roof System

The roof of the Cancer Center is a sloped deck with a 63’-0’ by 63’-0” elevator penthouse perched at the southern corner. The roof slopes downward along the east and west sides of the building and allows drainage to the center third. The roof system consists of a 3”x20ga type ‘N’ galvanized steel deck. The roof deck rests typically rests on W14x22 beams framing into W21x44 girders with W18x35 beams being used to support mechanical equipment spaced uniformly across the building’s center. Roof decks lower than the top of the 8th level consist of 1.5”x20ga. type ‘B’ galvanized steel deck (see Figure 6).

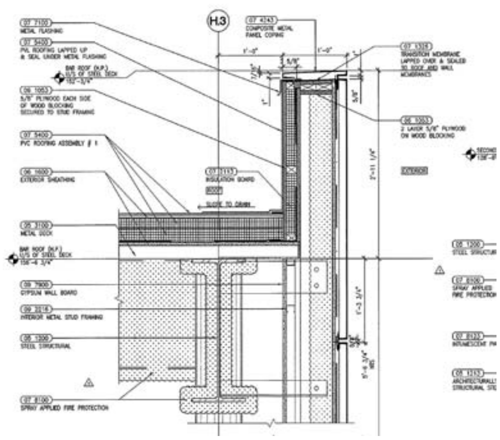


Figure 6

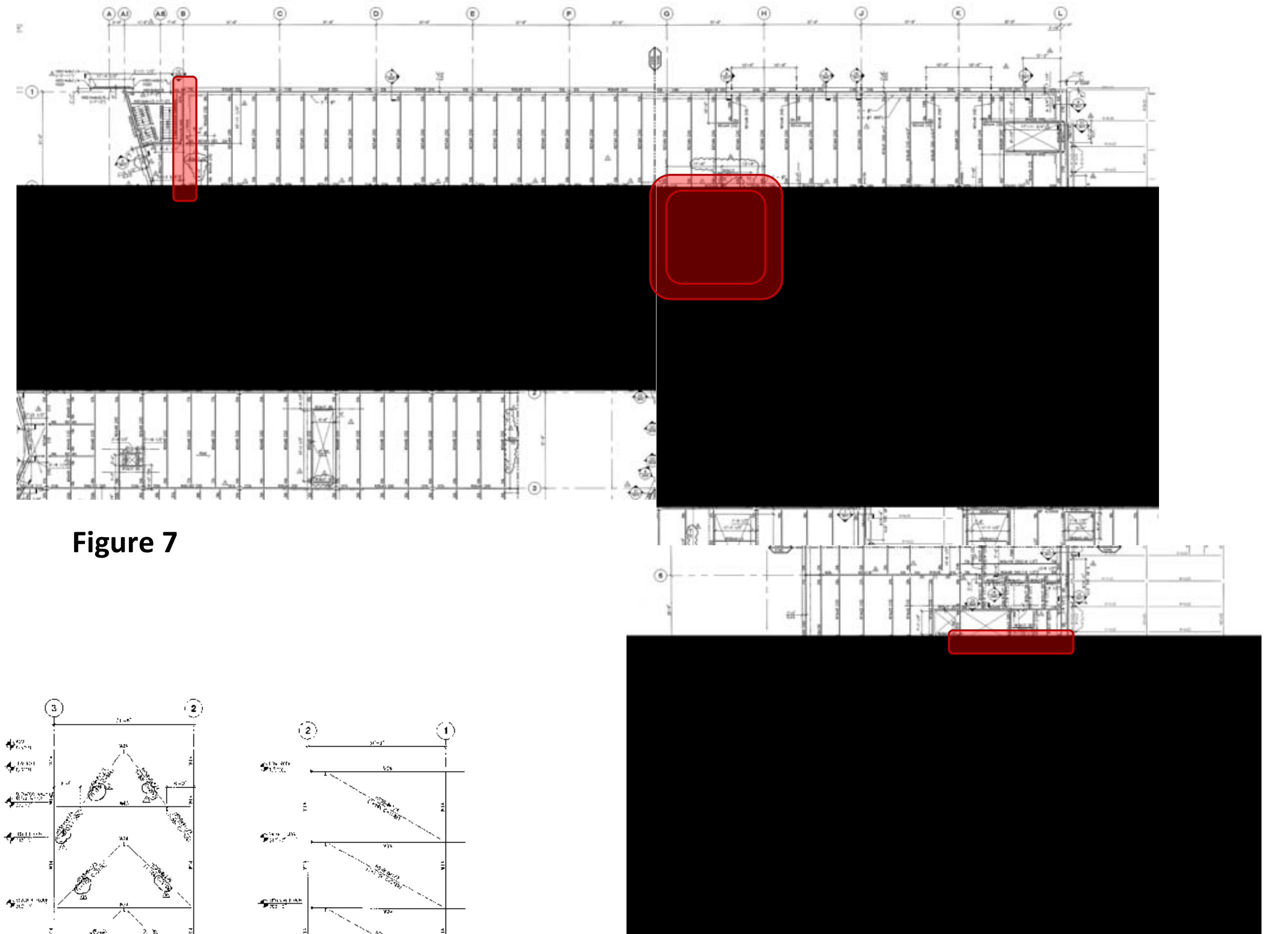


Figure 7

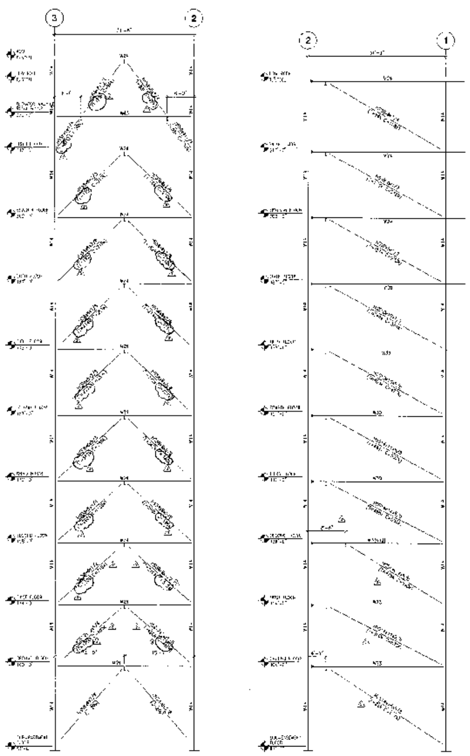


Figure 8

Lateral System

Lateral forces are resisted by a series of concentrically braced frames located at the center of the building near the main elevator core and along isolated points of the exterior bays (see Figure 7). This system consists of four chevron braces and two diagonal braces, which are used both in the north/south direction as well as the east/ west direction. Each brace typically consists of a 31'-6" wide W24 beam, a 15'-0' tall W14 column, and two HSS8 size diagonal members (see Figure 8). Structural brace members beyond the 8th floor increase in size due to increased lateral loads.

Code and Design Specifications

Existing Design Codes

Codified Ordinances of the City of Cleveland:

Land Use Code – Planning and Housing 6/3/07

Zoning Code 6/3/07

Land Use Code – Fire Prevention Code 6/3/07

Building Code 6/3/07

2007 Ohio Building Code (w/ 2006 International Building Code)

2006 International Mechanical Code

2006 International Plumbing Code

Design Codes and Specifications

IBC 2006 *International Building Code*

ASCE-7-05 *Design Code for Minimum Design Loads*

LRFD Specifications for Structural Steel Design – Unified Version, 2005

ACI 318-08 *Building Code Requirements for Structural Concrete, 2005*

LRFD Seismic Design Manual– Third Edition, 2008

Proposal

Problem Statement

Previous technical assignment have found the Cancer Hospital design to adhere to all drift limits and strength requirements as per all applicable codes given in its current location. However, the irregular “L” shape of the hospital causes a significant amount of torsion and drift from lateral loads. This movement greatly affects the efficiency of the Cancer Hospital, due the location of the imaging rooms, surgery rooms, and advanced researched equipment. The technical reports provide only a general amount of information on the response of the structure to increased movement, specifically dynamic loading.

Solution

High Seismic Region Design Relocation

In order to gain knowledge and experience in the seismic design of a movement sensitive and abnormally shaped structure, the Cancer Hospital design will be theoretically relocated to a high seismic region. This relocation will cause current loads to be higher and more dynamic. In-depth study of the ramifications of this new loading will be conducted in effort to create a building architecturally similar to the Cancer Hospital but with a structure designed to withstand movement in a high seismic region.

Lateral System Investigation

Once clear loads and conditions have been established from the theoretical relocation, 3 lateral system solutions will be investigated and compared for efficiency in design. These solutions will include an upsizing of the existing lateral system, the separation of the structure through use of a seismic expansion joint, and the use of a concrete core. All three solutions will be evaluated in regard to period, deflection, and strength.

Redesign of Existing Lateral System

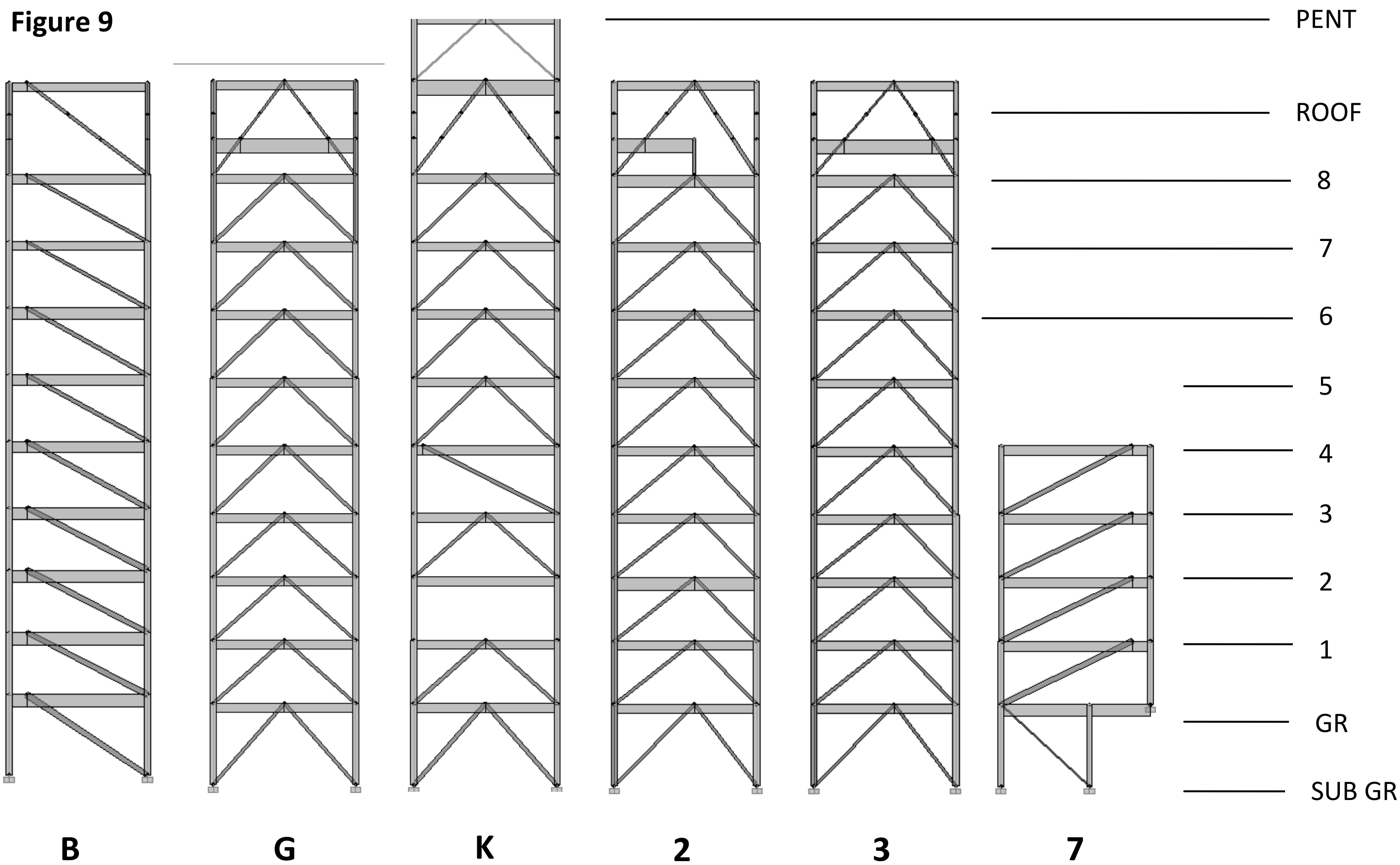
A new system will be designed upon selection of the optimal lateral solution in accordance with current codes and industry standards. Members and critical connections will be efficiently designed and checked to adhere to all strength and serviceability requirements.

High Seismic Region Relocation

Design Relocation

In previous technical reports, a sizeable amount of torsion was indentified to exist in the current Cancer Hospital design. This torsion was speculated to be caused by the irregular "L" shape of the building as well as inconsistencies in the lateral system. Bracing configurations are non-uniform between frames and non-existent at in isolated locations (see Figure 9).

Figure 9



In order to exemplify the effect of lateral loads on such an irregular configuration, the existing structural design has been theoretically relocated to San Diego, Ca. In San Diego, the structure will be exposed to dynamic seismic loads which will be shown in a later section to control both strength and serviceability in the new design (see Figure 10). This relocation will expand my current knowledge of seismic design and solutions which are commonly used to handle problems associated with irregular building configurations. The preservation of the original architectural design will be taken into account when selecting the most efficient system.

In addition the creation of a structural challenge, the relocation will also greatly affect the building envelope. The current design is exposed to large temperature variations due to its location in Cleveland, Ohio. The new location in San Diego, Ca may allow the exterior insulating system to be reduced due to warmer and more consistent temperatures year round.

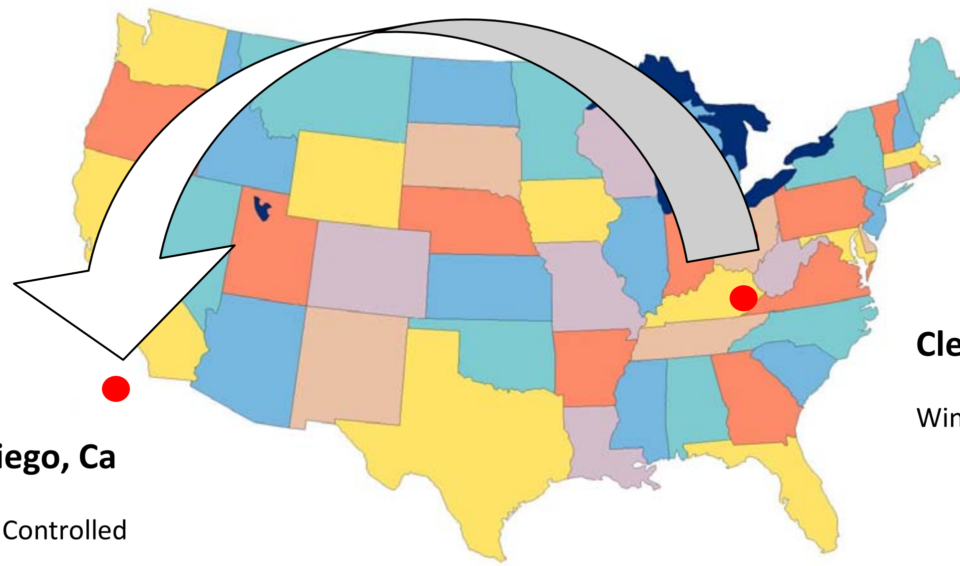


Figure 10

Load Calculation

Loads have been calculated for both wind and seismic forces in the new San Diego, Ca design location. Values have been analyzed in both the north/south and east/west directions for each and compared to determine the controlling loads.

Wind

All tables, figures, and equations for calculation of wind loads were done so in accordance with chapter 6 of ASCE 7-05. Method 2 of the Main Wind-Force Resisting Systems, also known as the Analytical Method, was used in determination of lateral wind pressures. For the approximate calculations of this report, Case I of ASCE7-05 Figure 6-9 has been assumed to be the most conservative and were analyzed in the both directions accordingly (see Figure 11).

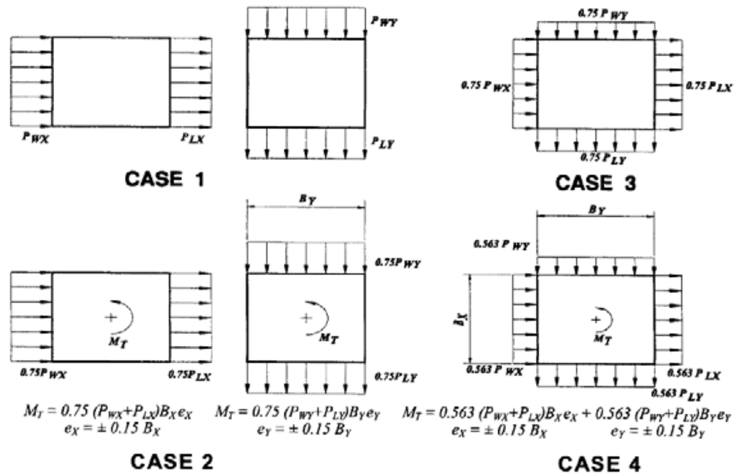


Figure 11

Different gust factors resulted due to flexibility (see Table 1). A conservative approach was taken in east/west direction in order to account for the vertical "L" shape caused by the lower 4 story, southern wing of the Cancer Hospital. Since the code is unclear about applying wind pressures to non-uniform shapes, a rectangular shape was used in calculation. This will cause the lateral forces to be larger than in actuality.

Wind Factors			
V	85mph	n	0.39
Kd	0.85	G	.85/.84
I	1.15	qz	18.08
Exp. Cat.	B	qi	20.25
Kzt	1	qh	20.25
Kh	1.2	Cp	0.8

Table 1

For this analysis internal pressures and roof top uplift pressures have been ignored. However, overturning moment has been determined. The maximum point load was calculated to be **178.18k** in the north/south direction and **169.40K** in the east /west direction at the roof level (see Tables 2-4).

Table 2

WIND ANALYSIS				
	Story	Tributary Height (ft)	Kz	qz (psf)
Windward	Penthouse	16.33	1.12	20.2
	High Roof	7.17	1.08	19.5
	Low Roof	13.58	1.06	19.2
	8	15	1.03	18.6
	7	15	0.99	17.9
	6	15	0.95	17.2
	5	15	0.89	16.1
	4	15	0.84	15.2
	3	14	0.77	13.9
	2	14	0.68	12.3
	1	14	0.57	10.3
Leeward		154.1	1.12	20.2
Side		154.1	1.12	20.2

Table 3

NORTH - SOUTH DIRECTION					
Story	Tributary Height (ft)	External Pressure qGC_p (psf)	Forces (k)	Story Shear (k)	Overturn Moment (ft-k)
Roof	20.75	19.23	179.18	89.59	89.59
8	15	17.88	120.44	210.03	5965.21
7	15	17.23	116.02	326.06	11329.75
6	15	16.27	109.55	435.60	18386.08
5	15	15.03	101.21	536.81	27023.06
4	15	13.84	93.19	630.00	37118.01
3	14	12.14	76.34	706.34	48086.49
2	14	10.82	68.00	774.34	59705.56
1	14	9.09	57.14	831.49	72200.64

EAST - WEST DIRECTION					
Story	Tributary Height (ft)	External Pressure qGC_p (psf)	Forces (k)	Story Shear (k)	Overturn Moment (ft-k)
Roof	20.75	19.12	169.40	84.70	84.70
8	15	17.78	113.87	198.56	5639.48
7	15	17.13	109.69	308.25	10711.09
6	15	16.17	103.57	411.82	17382.11
5	15	14.94	95.68	507.50	25547.46
4	15	13.76	88.10	595.60	35091.18
3	14	12.07	72.17	667.77	45460.72
2	14	10.75	64.29	732.06	56445.33
1	14	9.04	54.02	786.08	68258.12

Table 4

Seismic

All tables, figures, and equations used in calculation of seismic loads were done so in accordance with Chapter 12 of ASCE 7-05. After the design relocation, the Cancer Hospital was now found to fall under Seismic Design Category D causing a dramatic increase in lateral loads. The Equivalent Lateral Force Procedure will be used to calculate conservative user loads. These values will be used as a preliminary approach to investigation and design.

Due to the complexity and diversity of the gravity loads on each floor of the Cancer Hospital, a Load Key Diagram was obtained from the structural consultant in order to accurately calculate effective story weight to be used in analysis. Superimposed line and area dead loads from the diagram can be applied to each respective zoned area on each of the 9 levels. The penthouse level weight has been neglected due to its small amount of contribution to the period. After calculation, these loads were determined to include self-weight (see Figure 12). The dead load distribution is shown in the following Tables 5 through 6.

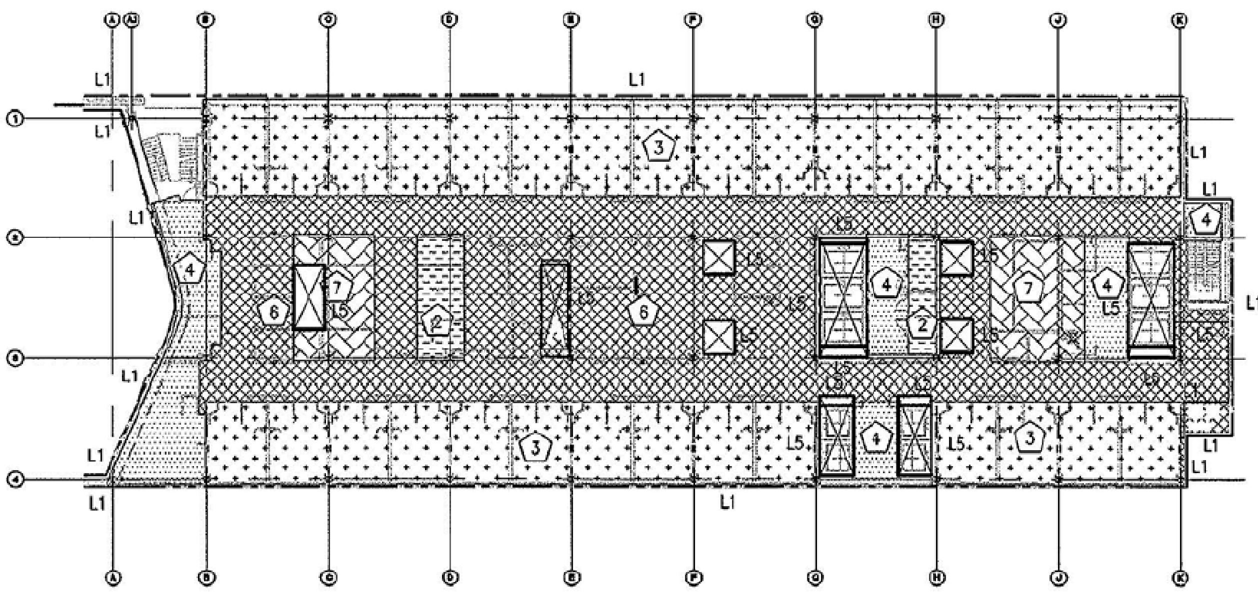


Figure 12

LEVEL 5 AND 6
LOAD KEY DIAGRAM

2

SCALE: 1/32=1'-0"

SUPERIMPOSED LOADS

SURFACE LOAD SCHEDULE

LABEL	PATTERN	DL (psf)	LL (psf)	REDUCTION TYPE	MASS DL (psf)
1		47	370	Unreducible	102.5
2		47	150	Unreducible	103.2
3		47	40	Reducible	75.7
4		41	100	Reducible	41
5		47	60	Reducible	102.5
6		47	60	Reducible	75.7
7		47	125	Unreducible	124
8		25	30	Unreducible	31
9		30	150	Unreducible	175
10		340	100	Unreducible	385
11		70	175	Unreducible	181
12		30	270	Unreducible	390
13		31	53	Unreducible	53
14		150	100	Unreducible	195
15		80	150	Unreducible	215

LINE LOAD SCHEDULE

LABEL	DL (k/ft)	MASS DL (k/ft)	LL
L1	.3	.3	
L2	.56	.42	
L3	.36	.36	
L4	.5	.5	
L5	.225	.225	
L6	.4	.4	

GRAVITY LOAD ROOF – LEVEL 5				
Level	**Description**	**Load**	**Area / Dist**	**Total(lb)**
Roof	Roof Load	25psf -41psf	28200 ft^2	800747
	Building Envelope	300plf - 500plf	1921 ft	713100
	Ceiling Partition	5psf	26791 ft^2	133955
	Suspended Mechanical Equipment	10psf	26791 ft^2	267910
	Interior Shafts	225plf	812ft	182700
8th	Floor Load	30psf -70psf	28315 ft^2	883095
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	26791 ft^2	133955
	Suspended Mechanical Equipment	10psf	26791 ft^2	267910
	Interior Shafts	225plf	812ft	182700
7th	Floor Load	47psf	28516 ft^2	1340252
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28516 ft^2	142580
	Suspended Mechanical Equipment	10psf	28516 ft^2	285160
	Interior Shafts	225plf	812ft	182700
6th	Floor Load	47psf	28518 ft^2	1340346
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28518 ft^2	142590
	Suspended Mechanical Equipment	10psf	28518 ft^2	285180
	Interior Shafts	225plf	812ft	182700
5th	Floor Load	47psf	28188 ft^2	1324836
	Building Envelope	300plf	814 ft	244200
	Ceiling Partition	5psf	28188 ft^2	140940
	Suspended Mechanical Equipment	10psf	28188 ft^2	281880
	Interior Shafts	225plf	812ft	182700

Table 5

GRAVITY LOAD LEVEL 4 – LEVEL 1				
Level	**Description**	**Load**	**Area / Dist**	**Total(lb)**
4th	Floor Load	47psf	28062 ft^2	1318914
	Building Envelope	300plf - 360 plf	1289 ft	409740
	Ceiling Partition	5psf	28062 ft^2	140310
	Suspended Mechanical Equipment	10psf	28062 ft^2	280620
	Interior Shafts	225plf	812ft	182700
3rd	Floor Load	47psf	40492 ft^2	1903124
	Building Envelope	300plf	1006 ft	301800
	Ceiling Partition	5psf	40492 ft^2	202460
	Suspended Mechanical Equipment	10psf	40492 ft^2	404920
	Interior Shafts	225plf	812ft	182700
2nd	Floor Load	47psf	41393 ft^2	1945471
	Building Envelope	300plf - 560plf	1006 ft	357180
	Ceiling Partition	5psf	41393 ft^2	206965
	Suspended Mechanical Equipment	10psf	41393 ft^2	413930
	Interior Shafts	225plf	812ft	182700
1st	Floor Load	47psf	41662 ft^2	1958114
	Building Envelope	300plf	336 ft	100800
	Ceiling Partition	5psf	41662 ft^2	208310
	Suspended Mechanical Equipment	10psf	41662 ft^2	416620

Table 6

GRAVITY LOAD	
Level	**Load(lb)**
Pent	211800
Roof	2098412
8	1711860
7	2194892
6	2195016
5	2174556
4	2332284
3	2995004
2	3106246
1	2683844
Total Wt.	21703914

Table 7

SEISMIC FACTORS			
Ss	1.576	I	1.5
S1	.62	Sds	1.051
Site Class	B	Sd1	.62
Occupancy Cat.	IV	Seismic Des. Cat.	D
Fa	1	Cs(x)	.109
Fv	1.5	Cs(y)	.122
Sms	1.576	Ta	1.366
Sm1	.930	K(x)	2.0
R	3.25	K(y)	1.85

Table 8

The original structural design includes eccentric braced frames as well as normal braced frames. The lower R value of 3.25 has been used in the calculation of the lateral forces (see Tables 6 – 7)

The fundamental period has been calculated in accordance with Chapter 12 of ASCE7-05. However, in order to accurately predict the behavior of the existing structure under the new parameters, an ETABS Model was created and analyzed. The preliminary model was used to estimate the fundamental periods to be used in user load calculation. In both the X and Y directions, the fundamental period from the ETABS dynamic analysis was found to be larger than the approximate fundamental period thereby controlling the seismic response coefficient.

<u>Fundamental Periods (sec)</u>

$T_x = 2.619$

$T_y = 2.203$

$T_z = 1.793$

Based on the equivalent lateral force procedure performed, the maximum force was found to be 751.26K at the roof level in the east/west direction (see Tables 9-10)).

SEISMIC FORCES - EAST/WEST						
Level	wx	hi	hik	wihik	Cvx	Story Force(k)
Pent	211800	144	20736	1.31E+11	0.03	79.54
Roof	2098412	132	17424	1.3093E+11	0.28	662.20
8	1711860	117	13689	1.3093E+11	0.18	424.42
7	2194892	102	10404	1.3093E+11	0.17	413.59
6	2195016	87	7569	1.3093E+11	0.13	300.90
5	2174556	72	5184	1.3093E+11	0.09	204.17
4	2332284	57	3249	1.3093E+11	0.06	137.24
3	2995004	42	1764	1.3093E+11	0.04	95.69
2	3106246	28	784	1.3093E+11	0.02	44.11
1	2683844	14	196.0	1.3093E+11	0.00	9.53

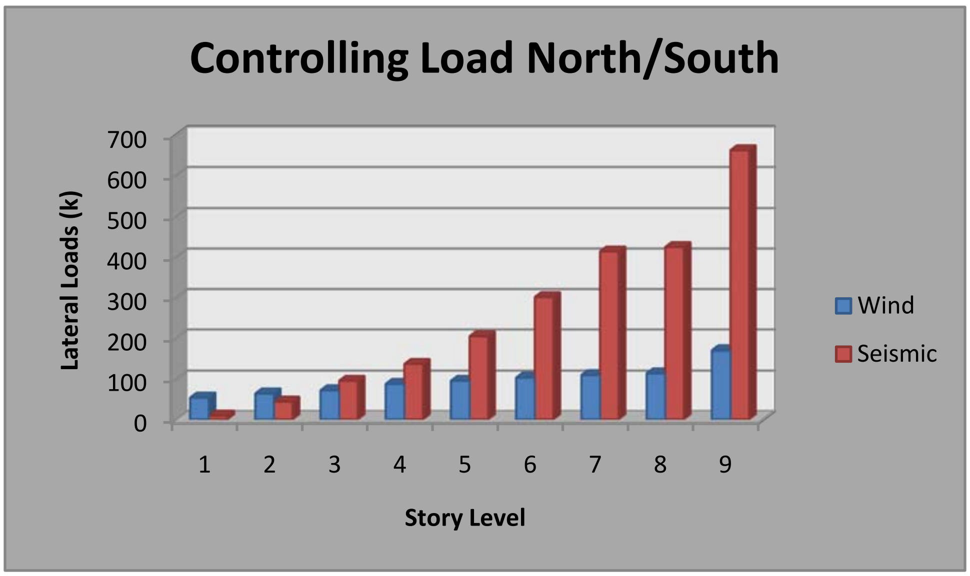
Table 9

SEISMIC FORCES - NORTH/SOUTH						
Level	wx	hi	hik	≅wihik	Cvx	Story Force(k)
Pent	211800	144	9839.4452	65960475881	0.03	89.07
Roof	2098412	132	8376.4842	65960475881	0.27	751.26
8	1711860	117	6701.0652	65960475881	0.17	490.29
7	2194892	102	5198.8863	65960475881	0.17	487.71
6	2195016	87	3873.5629	65960475881	0.13	363.40
5	2174556	72	2729.387	65960475881	0.09	253.67
4	2332284	57	1771.6115	65960475881	0.06	176.60
3	2995004	42	1006.9576	65960475881	0.05	128.90
2	3106246	28	475.60056	65960475881	0.02	63.14
1	2683844	14	131.9	65960475881	0.01	15.13

Table 10

Controlling Load

Wind loads were compared to seismic loads in both the north/south and east/west directions. As expected, seismic forces were found to control over wind in both orientations (see Figure 13). On the upper floors of the building, seismic forces exceeded the wind forces by a magnitude of 3 to 4 times. Through this approximate analysis, it has been determined that seismic loads control the behavior of the structure and no wind load effects will be further investigated. See Appendix A for detailed load calculations.



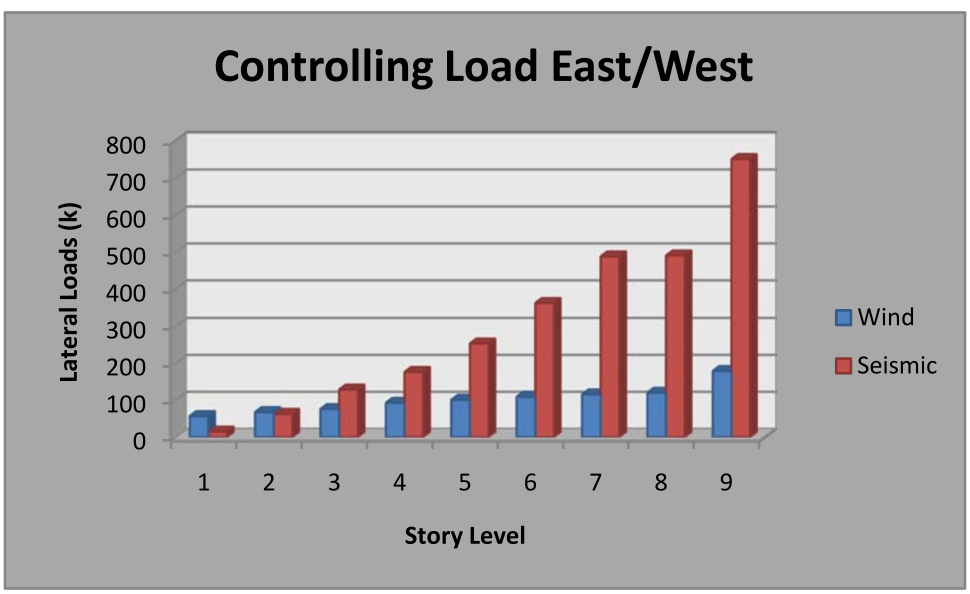


Figure 13

ETABS Model

As mentioned earlier, an *ETABS* model has been constructed using the existing design of the Cancer Hospital (see Figure 14). Even though the sub-ground level is actually below the surface and not susceptible to direct lateral loads, the lateral members have been modeled in these areas to increase consistency with the existing design. The diaphragms between braces have been modeled as rigid. In previous *Technical Report 3*, this model has been proven to be an accurate representation of the existing structure of the Cancer Hospital and has been used and modified in the investigation and design performed in this report.

Equivalent Lateral Force Procedure

Loads which have been calculated using the Equivalent Lateral Force Procedure have been inputted as user loads into the model of the existing structure. Using these loads, the model was analyzed in respect to period and deflection.

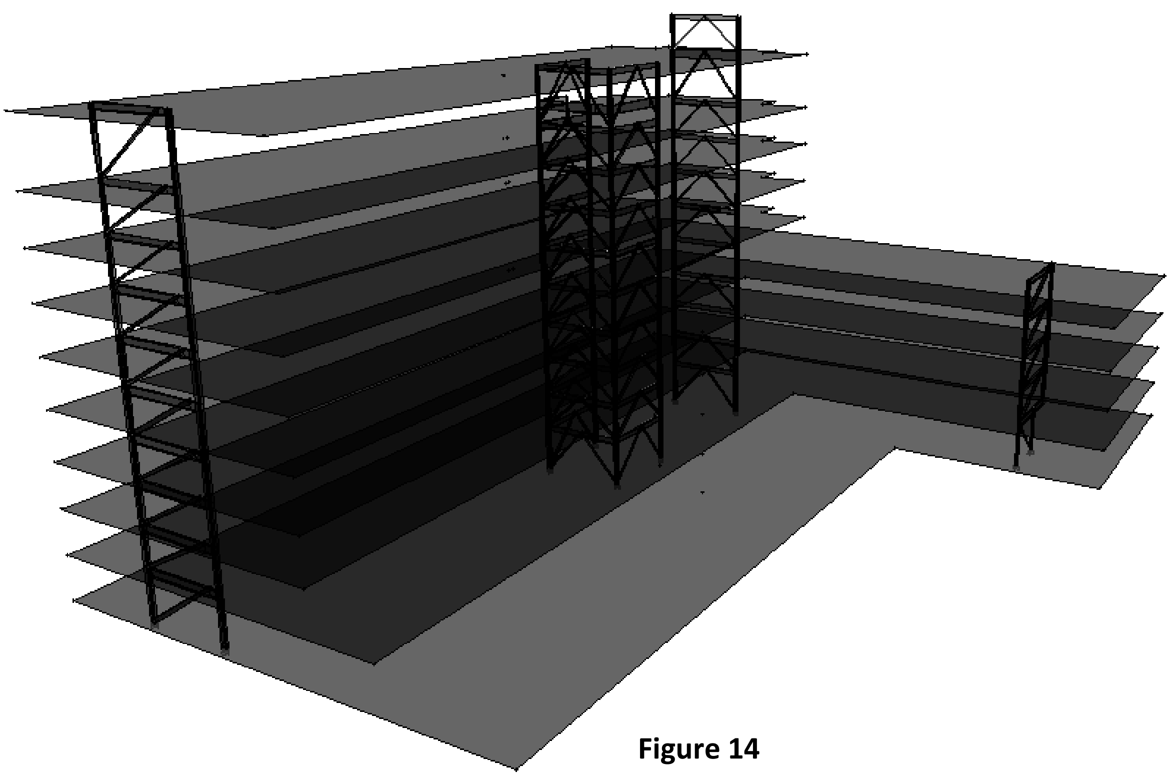


Figure 14

Modal Analysis

In order to obtain a more accurate depiction of the behavior of the existing structure under the new lateral loads, a modal analysis was performed using the dynamic solving capabilities of ETABS. Using the same given parameters as the Equivalent Lateral Force Procedure, the model response characteristics have been entered into the program and the existing structure has been re-analyzed.

Error (%)		
Story	Strength	Deflection
Roof	17.81%	117.07%
8	24.68%	32.65%
7	24.17%	27.16%
6	24.02%	32.91%
5	24.11%	28.25%
4	24.20%	28.21%
3	24.81%	28.74%
2	25.32%	28.59%
1	25.86%	34.77%

Table 11

ELF Analysis vs. Modal Analysis						
Story	Tributary Height (ft)	Forces(k)	Frame Shear (k)	Deflection (k)	ETABS Frame Shear (k)	ETABS Deflection (k)
Roof	20.75	751.26	751.26	14.81	346.10	18.02
8	15	490.29	1241.55	11.69	1843.36	15.52
7	15	487.71	1729.26	9.88	2374.11	13.03
6	15	363.40	2092.66	7.91	3119.04	10.41
5	15	253.67	2346.34	5.98	3270.11	7.88
4	15	176.60	2522.94	4.29	3514.24	5.66
3	14	128.90	2651.84	2.94	3721.45	3.91
2	14	63.14	2714.98	1.77	3802	2.37
1	14	15.13	2730.11	0.86	4185.18	1.16

Table 12

Comparison

Critical values for story shear and deflection have been found at each story using both the Equivalent Lateral Force Procedure and the Modal Analysis Method in ETABS (see Tables 11-12). These values were then compared in order to determine the validity of the approximate loads already established and determined that the Modal Analysis performed by ETABS is in fact an accurate depiction of the behavior of the existing structure under the increased seismic loads.

It was found that the error in story shear between the two methods averaged approximately 25% at all levels (see table ???). The load transfer to the members and the building response to the loads generated by the Modal Analysis were also drastically different. The error in this comparison averages around 30% on most levels (see table ???). The maximum deflection and story from the Modal Analysis yielded a more conservative value and has been determined to be more accurate than the approximations of the Equivalent Lateral Force Procedure.

Lateral Systems Investigation

Three common lateral system solutions have been analyzed with the existing structure and architecture of the Cancer Hospital. These solutions include an upsizing of the existing lateral system, the separation of the structure through use of a seismic isolation joint, and the use of a concrete core. These systems have been analyzed using the dynamic loads provided by a Modal Analysis conducted in an ETABS. Each lateral solution will have its own independent model and corresponding mass and diaphragm forces will be configured accordingly.

From the analysis, each system has been evaluated for required strength, drift, irregularity and feasibility. The most efficient solution was selected upon a rigorous comparison and the new design will be present in a later section of this report.

Existing Structure

The existing structure has been investigated in the new San Diego, CA, high seismic location in order to determine the immediate effects of the design relocation. As described in the previous background section, the existing lateral force resisting system consists of a mix between ordinary steel concentric braced frames and eccentric braced frames (see Figure 15). For the proposes of this investigation, the lower R value of 3.25 has been used in accordance with Chapter 12 of ASCE 7-05. The resulting lower Deflection Amplification Factor of 3.25 has also been used. In addition to a possibility for redesign, the existing structure will be used as a comparative figure for the other two systems.

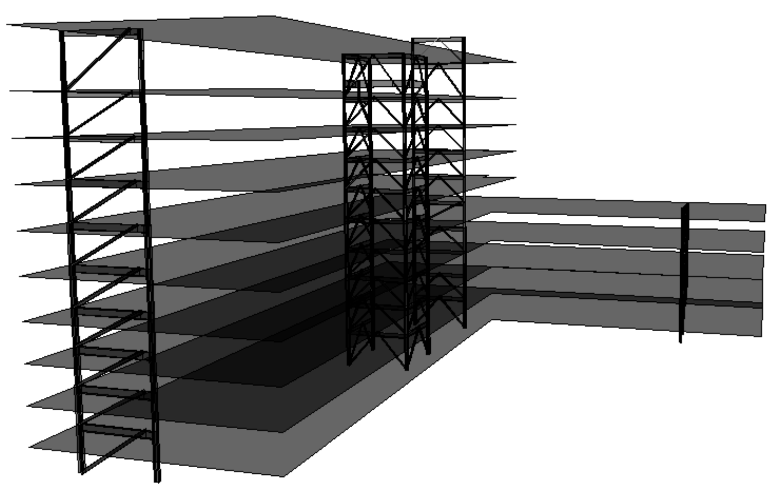


Figure 15

The time constraint that has been placed on the Cancer Hospital in its current location has greatly influenced the decision for the primary construction material used. Steel structures are generally able to be constructed faster due to a lack of need for formwork special labor. Although midrise buildings in San Diego, CA are typically constructed using concrete as the primary structural material, the structure will remain steel in order to attempt to keep the required schedule

Fundamental Periods (sec)

Tx = 2.619

Ty = 2.203

Tz = 1.793

Ax CALCULATION - E/W DIRECTION						
Level	δ_{53} (in)	δ_{61} (in)	δ_{60} (in)	δmax (in)	δavg (in)	Ax
Pent	0	0	0	0	0.00	0.00
Roof	0	25.81	23.82	25.81	24.82	0.75
8	0	21.57	19.73	21.57	20.65	0.76
7	0	18.64	16.91	18.64	17.78	0.76
6	0	15.59	13.97	15.59	14.78	0.77
5	0	12.03	10.51	12.03	11.27	0.79
4	5.62	9.07	7.68	9.07	7.46	1.03
3	3.73	6.19	5.19	6.19	5.04	1.05
2	2.08	3.69	3.04	3.69	2.94	1.10
1	1	1.55	1.3	1.55	1.29	1.00

Table 13

The unique advantageous characteristics of steel are relied upon in the development of the existing design. In response to the intense lateral loads, use of the ductility of steel will allow inelastic deformations leading to the formation and rotation of plastic hinges and the redistribution of bending moments (Rosenblueth 1980). This will allow higher loads to be resisted. The highly repetitious floor plan of the Cancer Hospital creates a sizeable amount of redundancy which in turn takes full advantage of the ductility of steel. This ductility is very advantageous in the event of an earthquake due to energy in which can be dissipated. Upon investigation conducted using the created ETABS model of the existing structure, much has been learned about the behavior of the lateral system under the new loading conditions. Although torsion appeared to be a reasonable concern in *Technical Report 3* , when placed in the context of the new seismic parameters the small amount of torsion has been found to be relatively insignificant. The existing structure has no irregularity under the new conditions and only a small amplification factor of 1.1 found at the second level of the north/south direction (see Table 13).

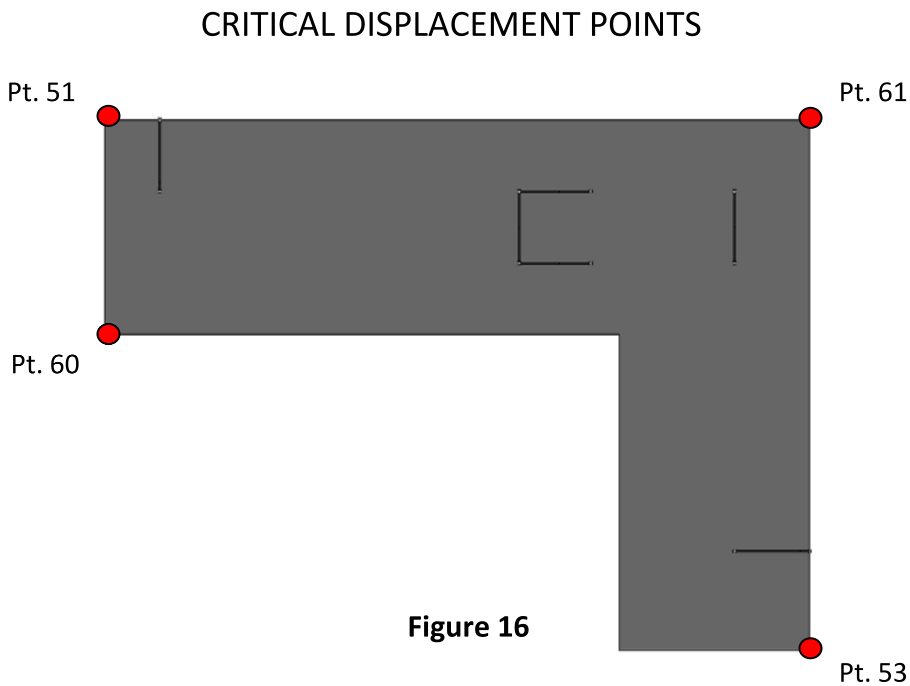


Figure 16

Deflections have been found to be extremely large in the existing structure under the new parameters. A maximum deflection of 25.81" has been found in the east/west direction (see Table 14). An allowable drift limit of 1.68" at the bottom 3 floors and 1.8" for the 4th through 8th floors. In both directions, the drift exceeds the allowable limit by a factor of approximately 3 at the critical point identified as point 51 (see Figure16). These increased deflection values have been associated with a large fundament period in which the building produces when subject to the new loading.

DRIFT FROM SEISMIC N/S - DIRECTION						
Level	Story Height (ft)	δ_{51}	Δ_{51}	δ_{61}	Δ_{61}	Code Allowable 0.010 hsx
Pent	162.58	0	0	0	0	2.9796
Roof	137.75	18.02	5.4	16.04	4.32	2.49
8th	117	15.52	5.3784	14.04	3.9312	1.8
7th	102	13.03	5.6592	12.22	3.996	1.8
6th	87	10.41	5.4648	10.37	4.1472	1.8
5th	72	7.88	4.7952	8.45	4.0176	1.8
4th	57	5.66	3.78	6.59	3.5856	1.8
3rd	42	3.91	3.3264	4.93	2.9808	1.68
2nd	28	2.37	2.6136	3.55	5.3568	1.68
1st	14	1.16	2.5056	1.07	2.3112	1.68
Ground	0	0	0	0	0	0

Table 14

The existing structure has also been analyzed in respect to strength demand. The story shear under the new seismic loads exceeds the previous wind demand by nearly 400% at most levels (see Tables 15-16). Under the applied conditions, the force in the members will be subject to a redundancy factor of 1.3, thereby significantly increasing the loads further. In order to support the loads found, columns will be forced to be increased from the optimal W14 size and several additional braced frames will be required to be placed around the structure. The large increase in size and quantity of the lateral force resisting members will significantly affect the architectural design of the Cancer Hospital.

Table 15

SHEAR N/S - DIRECTION				
Level	FRAME B	FRAME G	FRAME K	TOTAL
Pent	0	0	0	0
Roof	-121.70	-139.13	-85.26	-346.1
8th	-621.48	-823.81	-398.07	-1843.36
7th	-862.98	-931.49	-579.64	-2374.11
6th	-1065.36	-1047.28	-682.91	-2795.55
5th	-1247.29	-985.94	-885.80	-3119.04
4th	-1181.94	-1318.26	-769.91	-3270.11
3rd	-1262.47	-1233.67	-1018.09	-3514.24
2nd	-1242.37	-2299.54	-179.54	-3721.45
1st	-1338.95	-1372.81	-1090.50	-3802.25
GR	245.28	80.09	57.56	382.93

Table 16

SHEAR E/W - DIRECTION				
Level	FRAME 2	FRAME 3	FRAME 7	TOTAL
Pent	0	0	0	0
Roof	-176.83	-164.82	0.00	-341.65
8th	-951.24	-890.03	0.00	-1841.27
7th	-1210.13	-1151.68	0.00	-2361.82
6th	-1403.57	-1363.10	0.00	-2766.67
5th	-1584.12	-1516.20	0.00	-3100.32
4th	-2536.31	-2656.84	1928.09	-3265.06
3rd	-1227.70	-1311.76	-915.62	-3455.09
2nd	-1181.62	-1814.23	-635.33	-3631.18
1st	-1509.88	-1386.95	-809.20	-3706.03
GR	87.60	76.96	49.87	214.42

The existing lateral system has been found to have a strong resistance to torsion and distributes forces satisfactorily around its inherent corner. Although the fundamental is somewhat reasonable, story drift values greatly exceed associated limits and will require additional frames and massive member upsizing. The strength demand on the existing configuration also exceeds reasonable expectations and will similarly for an sizeable increase in quantity and size.

In order to use the existing system, several more braced frames will be required to be placed at optimal positions around the structure. Under Seismic Category D, the building height is limited to 35'. Use of this system will require the ordinary braced frames to be replaced with either eccentric or special concentric braced frames. Use of these special systems will also decrease the lateral demand on the building by increasing the Response Modification Coefficient and the Deflection Amplification Factor. In addition to the changes, it may be optimal to use moment frames in conjunction with the braced frames, creating a dual system and gaining an increase in resistance without further disrupting the architectural design. See Appendix A for detailed calculations.

Isolation Joint

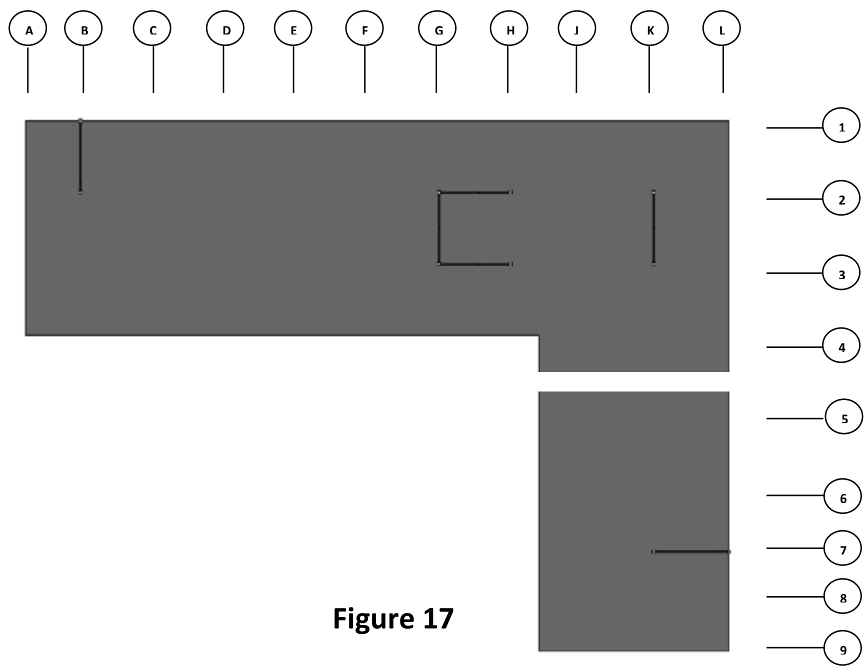


Figure 17

Similarly to the existing structure, the isolation joint design utilizes the advantageous characteristics of steel. This design will reduce the torsional effects of loads on the "L" shape" of the building as well as eliminate concentrated stresses caused by the inherent corner. The isolation joint will split the structure into two independent lateral systems; each with its own independent strength and serviceability characteristics (see Figure 17). As opposed to the "L shape", the two structures are now symmetrical. This will be beneficial for additions to the lateral systems. For purposes of this investigation, the tower structure has been identified as the primary element controlling design and only minimal initial analysis has been conducted on the extension.

With the separation of the "L Shape", the fundamental period of the tower portion increased by approximately 5% in both the east/west and the centriod when compared to the existing configuration. Deflection in the tower has also decreased nearly 10% at the roof level in the north/south direction at point 51. Story drift in this direction still greatly exceeds the required limit but now at a factor of approximately 2.5. The critical deflection direction remains in the east/west direction @ critical point 61 and only minimal reductions were found after the separation of the building (seeTable 17).

Fundamental Period	
Tx	2.776
Ty	2.206
Tz	1.931

DRIFT FROM SEISMIC N/S - DIRECTION						
Level	Story Height (ft)	δ_{51}	Δ_{51}	δ_{61}	Δ_{61}	Code Allowable 0.010 hsx
Pent	162.58	0	0	0	0	2.9796
Roof	137.75	16.36	4.9032	14.53	4.698	2.49
8th	117	14.09	4.86	12.73	4.2804	1.8
7th	102	11.84	5.1192	11.09	4.3326	1.8
6th	87	9.47	4.9464	9.43	4.4892	1.8
5th	72	7.18	4.3416	7.71	4.3587	1.8
4th	57	5.17	3.5424	6.04	3.915	1.8
3rd	42	3.53	3.1536	4.54	3.1581	1.68
2nd	28	2.07	2.052	3.33	6.3162	1.68
1st	14	1.12	2.4192	0.91	2.3751	1.68
Ground	0	0	0	0	0	0

Table 17

In addition to a decrease in drift and deflection, the strength required of the structure has also been reduced 10% in comparison to the former design (see Table 18-19). Similarly, this has been found to occur as a result of the removal of the inherent corner. A redundancy factor of 1.3 will also applied to both structures causing increased loads. The current lateral system in the tower has been found to be drastically insufficient to carry these desired loads even though a small reduction has occurred. Architectural plans will need to be altered to accommodate the necessary insertion of a much larger system.

SHEAR N/S - DIRECTION				
Level	FRAME B	FRAME G	FRAME K	TOTAL
Pent	0	0	0	0
Roof	109.51	127.15	76.11	312.78
8th	562.14	726.14	352.82	1665.87
7th	777.79	819.32	512.57	2138.58
6th	958.70	918.30	602.21	2513.94
5th	1121.17	863.09	781.62	2802.07
4th	1105.18	1240.82	680.66	3032.33
3rd	1233.12	1083.49	850.09	3188.71
2nd	934.69	2107.15	190.81	3323.26
1st	1299.96	1170.41	866.57	3375.79
GR	-192.08	-61.70	-38.01	-285.72

Table 18

SHEAR E/W - DIRECTION			
Level	FRAME 2	FRAME 3	TOTAL
Pent	0	0	0
Roof	163.56	148.71	312.27
8th	871.97	792.09	1664.06
7th	1104.15	1022.91	2127.06
6th	1272.76	1212.73	2485.49
5th	1435.65	1332.13	2767.78
4th	1397.16	1555.45	2952.61
3rd	1479.26	1652.38	3131.64
2nd	1220.29	2012.82	3233.11
1st	1663.69	1597.32	3261.01
GR	-80.66	-74.23	-154.88

Table 19

No irregularities were found in the tower section of the building. A reduction in torsion has been found when compared to the existing design. This is in part to the removal of stresses at the inherent corner and the new symmetrical shape of the structure. Even though a reduction has occurred and no torsional amplification is required, torsion in the building has been identified as an insignificant factor when compared to the remaining limiting characteristics in design.

In summary, the seismic isolation joint has provided a reduction in drift and lateral force. However, the magnitude of this reduction would need to be at least 100%-200% to make a significant difference. The reduction in torsion was estimated in *Technical Report 3* to allow for greatly reduced deflection and load values, however, this has been found not to be true. In order to use this system, a large increase in quantity and the size of the members will be required. In addition to the tower, the extension currently has lateral resisting elements in only the east/west direction and will require an orthogonal system. Similar to the existing system, all ordinary braced frames will be required to be either eccentric or special concentric. This will allow for a substantial decrease in lateral load due to an increased Response Modification Coefficient and Deflection Amplification Factor. Use of moment frames to produce dual systems will also be beneficial to this configuration and drastically reduce architectural impact. See Appendix B for detailed calculations.

Concrete Core

A concrete core has been selected as the 3rd lateral system solution. Although concrete is generally a less capable material in high seismic regions, its large stiffness characteristics make this material an optimal solution to reduce sizeable deflections. A design has been created utilizing a reinforced concrete core located in the elevator corridor at the former location of the braced frame core (see Figure 18). This reduces architectural interference and is consistent with existing designs. In addition to the core, the existing braced frames will remain in use to collect lateral load and control torsion. This design utilizes both the ductility of steel to dissipate energy and the stiffness of concrete to control drift.

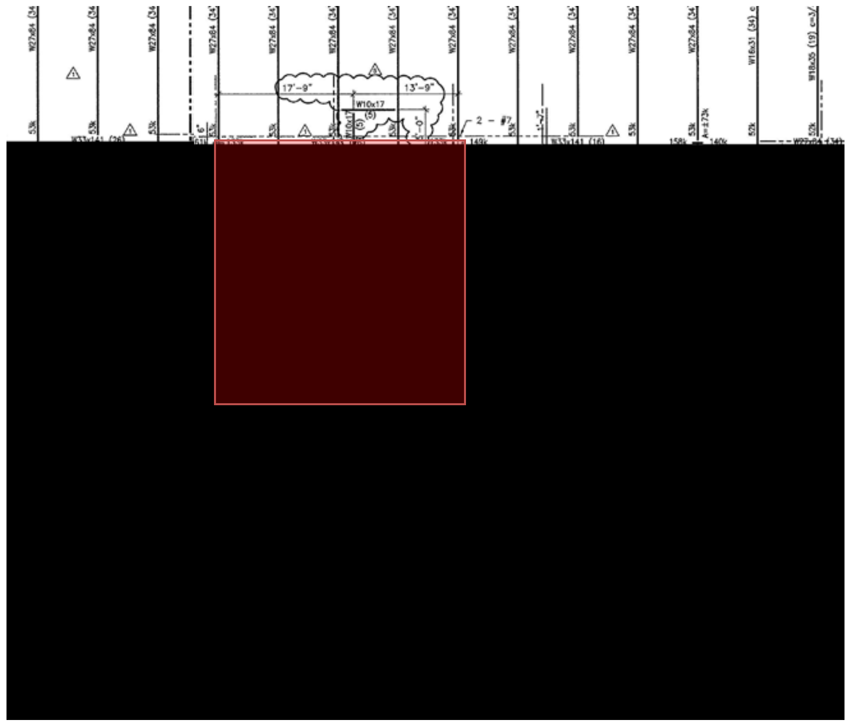


Figure 18

The concrete shear wall is modeled 18" inches thick and encloses all four sides of the elevator shaft. This continuity will allow the core to also resist out of plane forces. In order to accommodate the elevator shaft, coupling beams have been used to connect the walls in the east/west orientation. The concrete walls have been meshed into 24" units in order to accurately depict building behavior (see Figure 19).

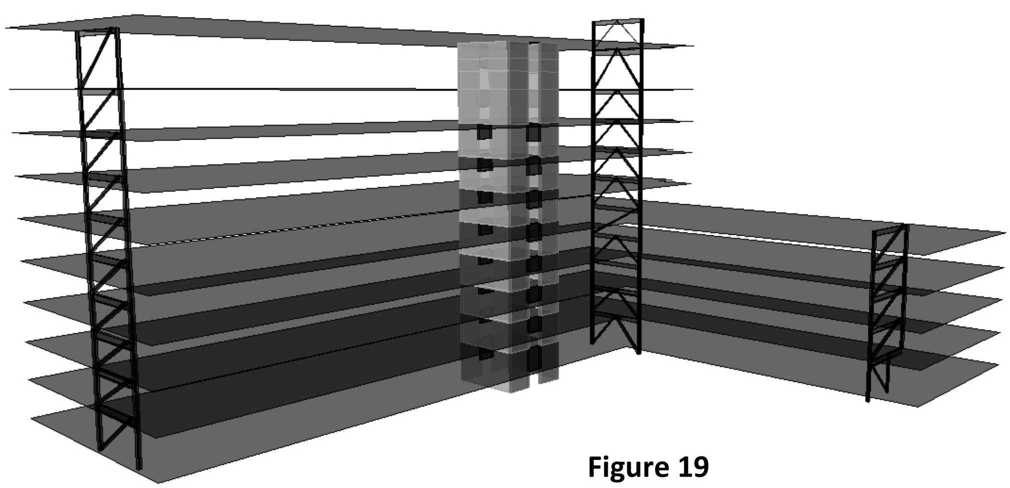


Figure 19

Fundamental Period	
Tx	1.261
Ty	1.606
Tz	0.673

The concrete core and braced frame configuration has reduced the fundamental period by 50%. This has occurred as a direct result of the addition of the stiffening characteristics of the concrete core. In the critical east/west direction for the existing system, deflection has been reduced nearly 600% and is now within manageable design levels. In the north/south direction deflection has been decreased approximately 33% at critical point 51, having a new controlling deflection of 11.99” (see Table 20). Drift values in the upper levels exceed limits by a factor of approximately 2 times. At lower levels drift levels are within reasonable values.

DRIFT FROM SEISMIC N/S - DIRECTION						
Level	Story Height (ft)	δ_{51}	Δ_{51}	δ_{61}	Δ_{61}	Code Allowable 0.010 h_{sx}
Pent	162.58	0	0	0	0	2.9796
Roof	137.75	11.99	5.34	-1.32	0.39	2.49
8th	117	10.21	4.8	-1.19	0.57	1.8
7th	102	8.61	4.89	-1	0.63	1.8
6th	87	6.98	4.89	-0.79	0.66	1.8
5th	72	5.35	4.5	-0.57	0.57	1.8
4th	57	3.85	3.6	-0.38	0.36	1.8
3rd	42	2.65	3.03	-0.26	1.26	1.68
2nd	28	1.64	2.7	0.16	0.66	1.68
1st	14	0.74	2.22	-0.06	0.18	1.68
Ground	0	0	0	0	0	0

Table 20

Changes in strength have also occurred as a result of the use of this system. An approximate average of 25% of the shear load has been reduced in the east/west direction. However, in the Y-Direction, loads have increased. Each shear wall takes on average between 30% and 40% of the load in a given direction (see Tables 21-22). Due to this relief of stress acquired by the braced frame members, more manageable sizes will be able to be used. A redundancy factor of 1.3 will be applied to the given loads used on lateral members causing significant increase.

SHEAR N/S - DIRECTION					
Level	FRAME B	SW G	SW H	FRAME K	TOTAL
Pent	0	0	0	1.44	1.44
Roof	97.06	97.93	140.28	14.99	350.26
8th	364.02	1638.45	57.78	-50.24	2010.01
7th	477.54	2296.57	44.64	-84.42	2734.33
6th	613.3	2662.71	12.81	-93.82	3195
5th	792.03	2676.38	298.34	-105.46	3661.29
4th	755.04	2371.98	808.13	-30.63	3904.52
3rd	740.26	2661.03	1018.39	-65.93	4353.75
2nd	764.59	3010.58	1057.02	-3.44	4828.75
1st	761.44	2967.08	1388.97	-58.71	5058.78
GR	-134.54	-1102.62	-669.78	3.81	-1903.13

Table 21

SHEAR E/W - DIRECTION				
Level	SW 2	SW 3	FRAME 7	TOTAL
Pent	0	0	0	0
Roof	111.18	105.31	0	216.49
8th	601.38	593.02	0	1194.4
7th	782.77	773.92	0	1556.69
6th	930.35	927.31	0	1857.66
5th	1039.45	1062.13	0	2101.58
4th	1108.86	940.48	172.63	2221.97
3rd	1176.07	1079.91	178.85	2434.83
2nd	1254.25	1219.88	163.06	2637.19
1st	1280.74	1273.89	157.83	2712.46
GR	-1638	-1619.71	-9.11	-3266.82

Table 22

Although massive improvements have been found in deflection and strength, a sizeable and significant amount of torsion now occurs in the structure as a result of the increased stiffness and location of the concrete core. Loads in the north/south direction cause a sizeable amount of accidental torsion and will require a torsional Amplification Factor of 3.0 to be applied, given the current configuration. An extreme torsional irregularity has been identified and will require additional stiffness from members at extreme points of the building.

The combined shear wall core and braced frame configuration has shown to dramatically reduce the period and deflection of the building as well as bring strength requirements to a more manageable design level. This system will provide minimal interference with the architectural design. Torsion has been identified as a serious concern for this design and will require an increase in the quantity of braced frames and a sizeable upsizing of members. The braced frames in this configuration will primarily be designed to resist these torsion loads. As with the previous designs, ordinary braced frames will be required to be designed as either special concentric or eccentric, thereby also raising Reponses Modification Coefficient and Deflection Amplification Factor. Use of moment frames to form dual systems will also have the possibility of be being beneficial to the design. In order to meet height requirements in Seismic Category D, the concrete core will have to consist of special shear walls. This will require the design of boundary elements. Attention will also need to be paid to ensure that any shear wall does not take greater than 60% of the shear in that direction. See Appendix C for detailed calculations.

System Comparison

The analysis of the existing system revealed an expected extreme change in the lateral loading of the structure. The existing lateral force resisting configuration has a strong resistance to torsion and had no irregularities. However, in regard to drift and strength, the values which were obtained through the seismic analysis showed a massive overload of the lateral members and the structure as a whole. Several additional upsized braced frames would need to be added in order to meet the strength and drift criteria of ASCE 7-05. This dramatic increase in size and member quantity will also greatly affect the current architectural design. Although upsizing is a common solution to such increased lateral loads, in this instance has been deemed through investigation to be inefficient.

The seismic isolation joint system provided a decrease in strength requirement and an increase in serviceability through the separation of the “L shape” into 2 independent structures. Although this decrease aides in creating a more designable structure, it is so minimal that all the downfalls of the existing design remain present. This has revealed that the initial thought that torsion caused by the “L Shape” was significantly impacting the lateral strength response of the structure was in fact not true. Use of this structural solution would also require massive upsizing of lateral members and a sizeable increase in quantity.

Unlike the study of the existing system and the use of a seismic isolation joint, the investigation of the combined concrete shear wall core and exterior braced frame system revealed a dramatic decrease in fundamental period and building drift. The use of the stiffening characteristics of reinforced concrete placed strength design values within a reasonable level with the possibility even distribution upon finalized design. However, severe torsional issues were identified in this design. The primary purpose of the braced frames in this configuration will be to resist this large amount of torsion and will need to be upsized and increased in quantity accordingly.

SYSTEM COMPARISON			
	Existing	Iso-Joint	Concrete Core
Period	OK	OK	GOOD
Deflection	BAD	BAD	OK
Story Drift	BAD	BAD	OK
Strength	BAD	BAD	OK
Irregularity	GOOD	GOOD	BAD
Torsion	GOOD	GOOD	BAD
Arch. Effect	BAD	BAD	GOOD

Table 23

The dramatic increase in serviceability and decrease in strength requirements revealing reasonable design values make the combined concrete shear wall core and braced frame configuration the optimal system (see Table 23). Special attention is required in the reduction of torsional forces in this design.

Redesign of Existing System

As previous stated, the combined concrete shear wall core and braced frame lateral system has been found to be the optimal design to for the reconfiguration of the existing Cancer Hospital to resist the high seismic load demand of San Diego, CA. The new design has been created and analyzed using the loads generated from the Modal Analysis conducted in ETABS. This lateral systems presented will provide detailed designs for the Shear Wall Core and associated coupling beams, the Perimeter Braced Frames and required critical connections, and the new Foundations under the revised loading. Each lateral system component has been designed in accordance with the proper industry codes and requirements, and checked for strength upon completion.

Steel Braced Frames

A severe amount of torsion has been found to exist in the new design during the investigative process and has been identified as the first critical aspect in initial design. In addition to torsion control, the steel braces also need to provide additional stiffness to create a more even load distribution across lateral elements and prevent the shear wall from taking too high of a percentage of the load. Several configurations for the addition of frames and upsizing of members have been tested and evaluated. Each design has been carefully coordinated with the existing in effort to reduce architectural impact.

The initial designs have been based primarily on deflection due to its control over strength when sizing members and determining frame additions. This deflection was monitored from the critical point of the building in which the maximum deflection occurs. This point has been previously identified as critical point 51 (see Figure 20). Upon reaching drift values acceptable for continued design, the members will be designed and checked for strength in conjunction with serviceability. The initial design process has been conducted using the lower R value of 6 provided by use of a special concretes wall in each direction. An accidental torsion factor of 1.0 has been assumed for proposes of design and this assumption will checked upon finalization.

CRITICAL DISPLACEMENT POINTS

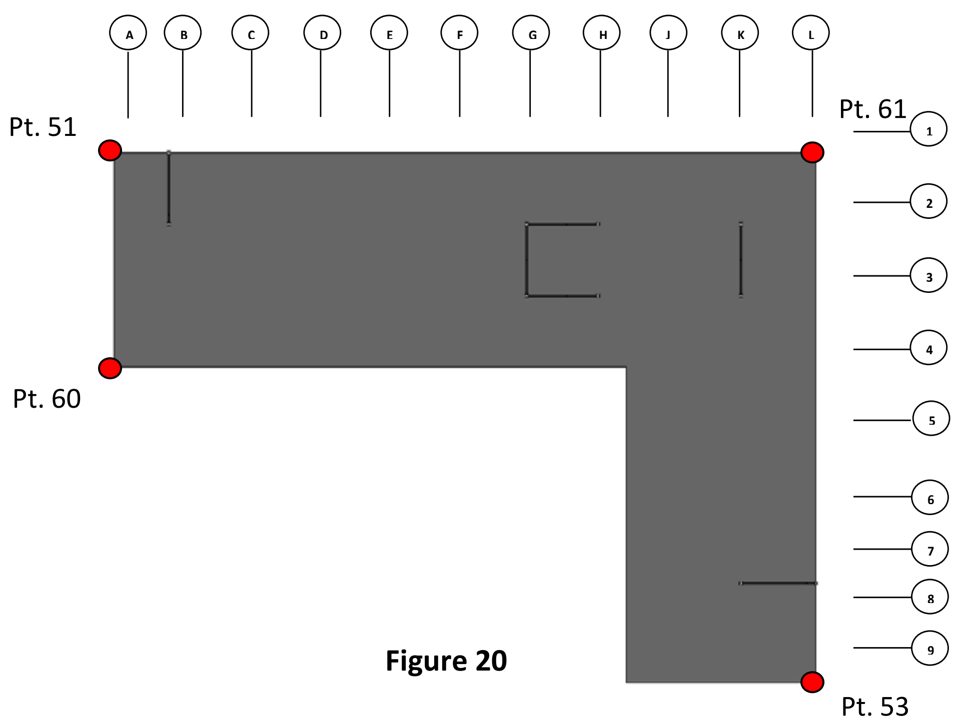
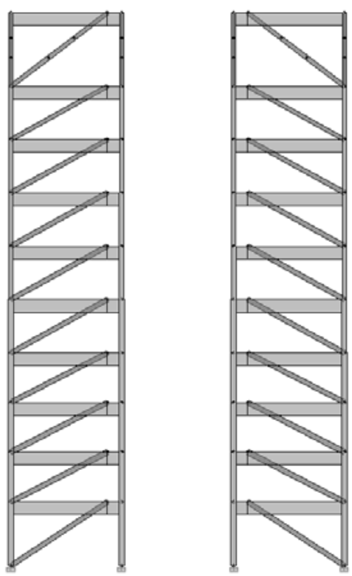


Figure 20

Symmetrical Perimeter Braced Frames

Initially a typical double brace system was tested along column line B and 7 with intent of the addition of intermediate moment frames to be added if necessary. These braced frames have been modeled as eccentric with a 4' link distance in order to comply with the seismic requirement of category D as mentioned in ASCE 7-05 Table 12.2-1. For testing purposes the original member sizes were kept in analysis (see Figure 21). A sizeable reduction has occurred as a result of this modification however drift values still needed to be reduced by a factor of two in order to become acceptable.

Story	Drift	Allowable
Roof	3.63	2.49
8	3.53	1.8
7	3.63	1.8
6	3.20	1.8
5	2.80	1.8
4	2.43	1.8
3	2.26	1.68
2	1.96	1.68
1	0.00	1.68



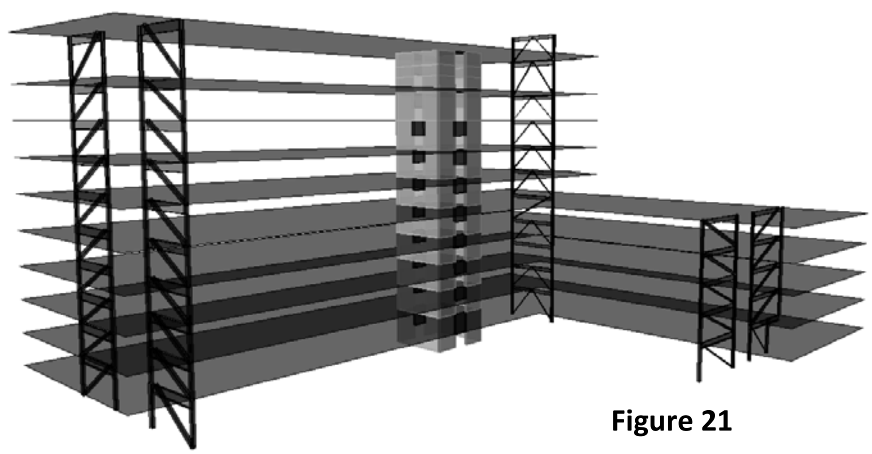


Figure 21

Story	Drift	Allowable
Roof	3.13	2.49
8	3.30	1.8
7	3.23	1.8
6	3.06	1.8
5	2.50	1.8
4	2.13	1.8
3	1.93	1.68
2	1.63	1.68
1	0.00	1.68

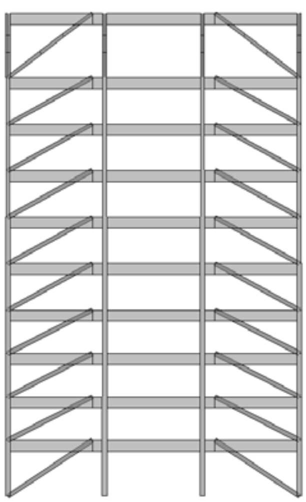
Symmetrical Perimeter Braced Frames w/ Enlarged Members

In order to test the magnitude of change that increasing the members in the lateral system would create, abnormally large size members where applied to the system. The beams consisted of W40 members, the braces at the maximum HSS16x16x1/2 size, and the columns at the upper limit of W14 members. This upsizing produce better drift results. However, more frames clearly needed to be added.

Symmetrical Perimeter Braced Frames w/ Enlarged Members and Moment Frames

Moment Frames have been added between the symmetrical braces creating dual systems in effort to further reduce drift values at the critical drift point (see Figure 22). In order to maintain a conservative figure for design, the members used in the moment frame were upsized to W40's. This modification resulted in a sizeable reduction in drift but still not enough to place values within a reasonable distance from the limits. For this reason, more braced frames need to be added.

Story	Drift	Allowable
Roof	2.56	2.49
8	2.80	1.8
7	2.83	1.8
6	2.80	1.8
5	2.30	1.8
4	1.96	1.8
3	1.80	1.68
2	1.60	1.68
1	0.00	1.68



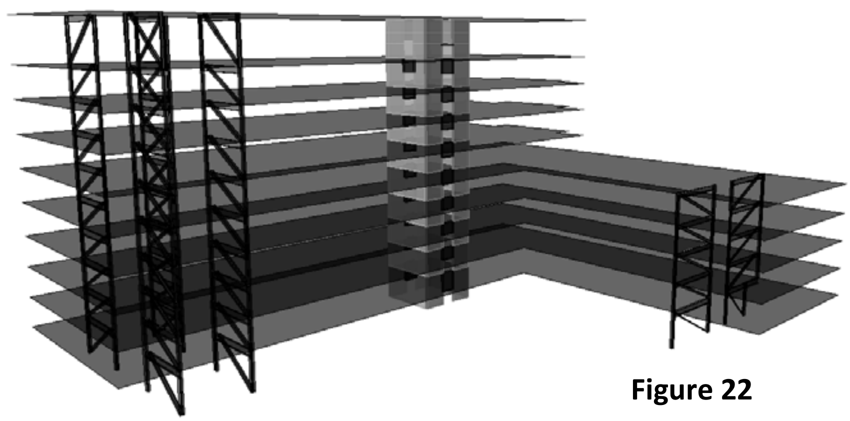


Figure 22

Symmetrical Perimeter Braced Frames w/ Enlarged Members and Moment Frames on 2 Column Lines

After coordinating with the architectural plans, it was found that the addition of the current configuration replicated on the immediate column behind the existing frame location at column line B would not disrupt the floor plan design. A new replicated dual system has been placed on column line C. This design yielded an over-conservative reduction in drift and has accomplished the intended displacement goals.

Story	Drift	Allowable
Roof	1.37	2.49
8	1.57	1.8
7	1.57	1.8
6	1.53	1.8
5	1.30	1.8
4	1.13	1.8
3	1.03	1.68
2	0.90	1.68
1	0.00	1.68

Story	Drift	Allowable
Roof	1.90	2.49
8	1.96	1.8
7	2.00	1.8
6	1.86	1.8
5	1.57	1.8
4	1.33	1.8
3	1.20	1.68
2	1.00	1.68
1	0.00	1.68

Symmetrical Perimeter Braced Frames w/ original members on 2 Column Lines

In order to reduce the over-conservative design now configured, the system was down-sized. Based on the original addition of moment frames, the removal was shown to have limited impact on the design. The symmetrical braced frames alone handled the seismic drift effectively and the dual system configuration has shown to be too conservative. Due to the addition of the shear wall core, the steel braced frame at K has become relatively ineffective and has been removed from the design. The size of the members have now been decreased back to the original dimensions (see Figure 23). The final design will be completely based on strength for overall efficiency with special attention paid to the top levels which exceed the drift limits.

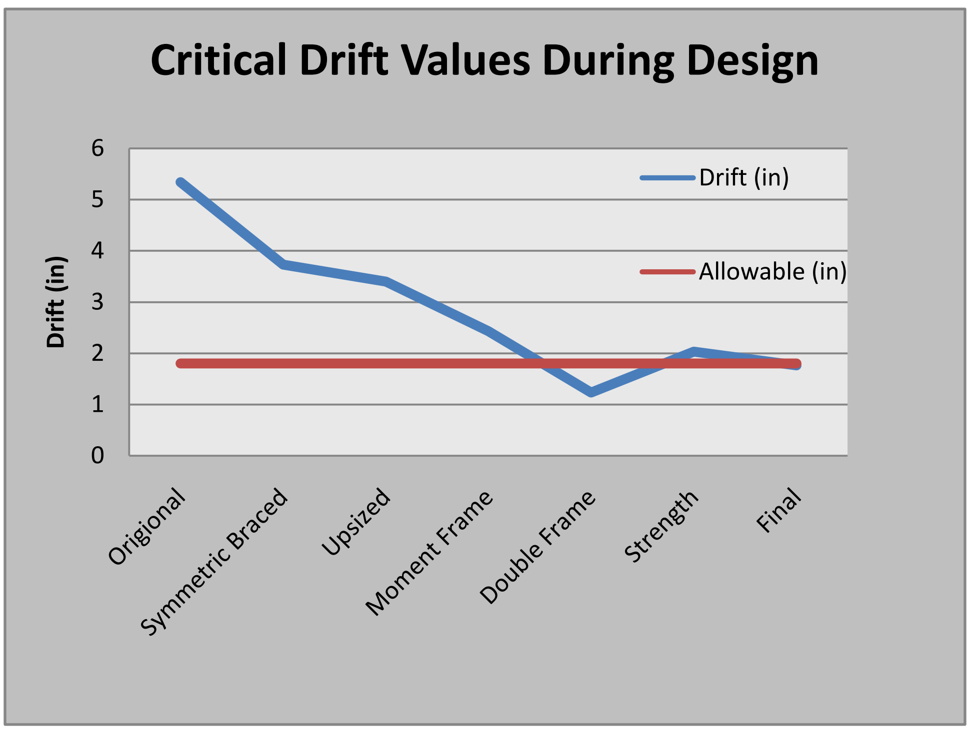
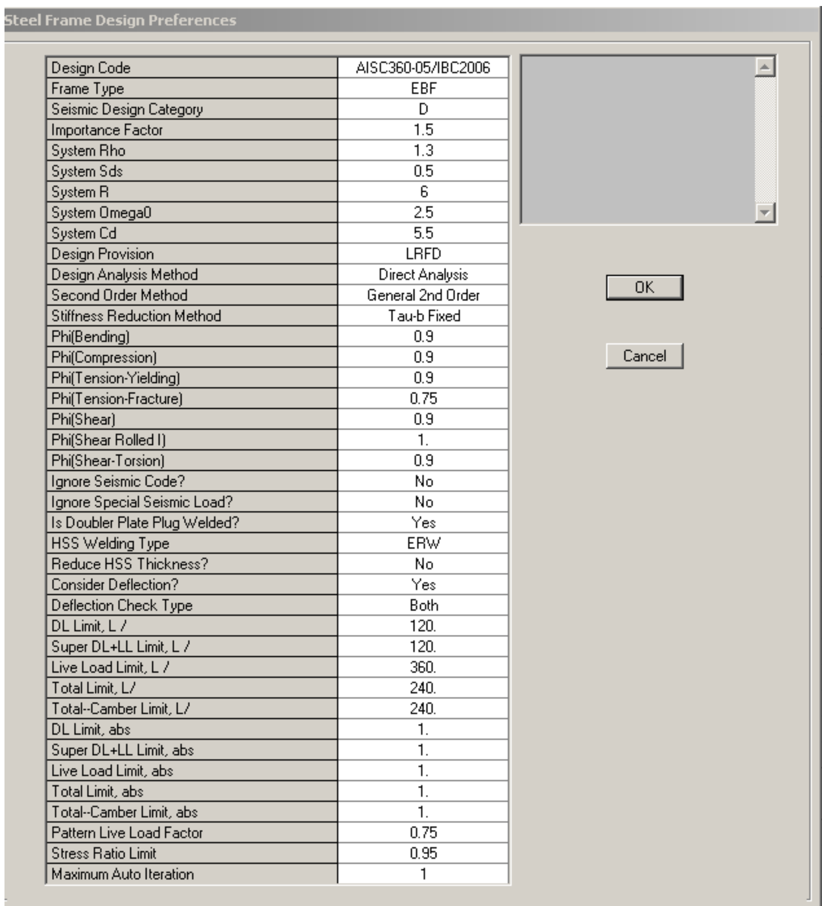


Figure 23

Story	Drift	Allowable
Roof	1.70	2.49
8	1.73	1.8
7	1.73	1.8
6	1.67	1.8
5	1.47	1.8
4	1.20	1.8
3	1.13	1.68
2	0.90	1.68
1	0.00	1.68

Final Perimeter Braced Frame Design

After achieving drift values within an acceptible design range, the new lateral steel system was checked for strength using ETABS. The design paremeters were set manualy in accordance with IBC 2006 and AISC360-05 for Eccentrically Braced Frames (see Figure24).



Design Code	AISC360-05/IBC2006
Frame Type	EBF
Seismic Design Category	D
Importance Factor	1.5
System Rho	1.3
System Sds	0.5
System R	6
System Omega0	2.5
System Cd	5.5
Design Provision	LRFD
Design Analysis Method	Direct Analysis
Second Order Method	General 2nd Order
Stiffness Reduction Method	Tau-b Fixed
Phi(Bending)	0.9
Phi(Compression)	0.9
Phi(Tension-Yielding)	0.9
Phi(Tension-Fracture)	0.75
Phi(Shear)	0.9
Phi(Shear Rolled I)	1.
Phi(Shear-Torsion)	0.9
Ignore Seismic Code?	No
Ignore Special Seismic Load?	No
Is Doubler Plate Plug Welded?	Yes
HSS Welding Type	ERW
Reduce HSS Thickness?	No
Consider Deflection?	Yes
Deflection Check Type	Both
DL Limit, L /	120.
Super DL+LL Limit, L /	120.
Live Load Limit, L /	360.
Total Limit, L/	240.
Total--Camber Limit, L/	240.
DL Limit, abs	1.
Super DL+LL Limit, abs	1.
Live Load Limit, abs	1.
Total Limit, abs	1.
Total--Camber Limit, abs	1.
Pattern Live Load Factor	0.75
Stress Ratio Limit	0.95
Maximum Auto Iteration	1

Figure 24

The redesign for strength yielded critical drift values within the required .010hsx limit at all levels accept for 7 and 8. The lateral frames were then manually changed and analyzed until the drift and strength values fell inside the industry design limits.

At the column line B and C frames, the finalized design consists of HSS16x16x1/2 braces, W36 and W33 beams, and large W14 columns at levels ground through 6 and HSS14x14x1/2 braces, W30 beams and mid size W14 columns at level 7 through the roof (see Figure 25). At column line 7 the finalized design consists of typical HSS12x12x3/8 braces, W27x84 beams, W14x132 columns.

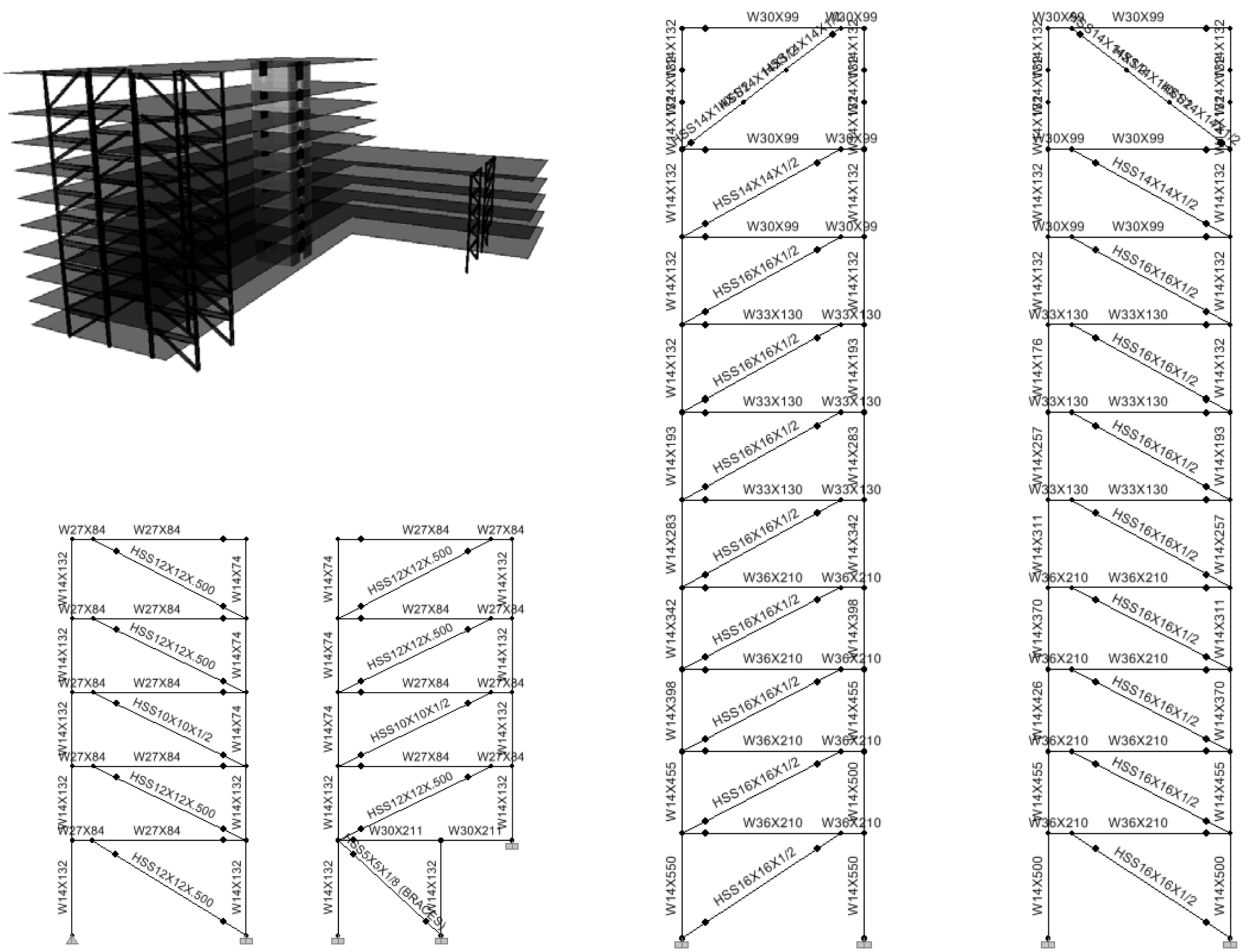


Figure 25

The concrete shear wall core has been kept to a thickness of 18" in order to minimize effect on the existing architectural design. The coupling beams have also been held to this dimension as well as the existing opening heights (see Figure 26). Both the shear wall and coupling beam design will be present in a later section of this report. Steel material types have been selected in the finalized design and associated properties have been applied in calculation (see Table 24).

Output for the design yielded a drift value inside allowable limits in both the north/south and east/west direction. Shear values are well distributed throughout lateral components and no irregularities exist in the new design of the building. No accident torsion exists in the new design and current loads will not be amplified. With the use of the shear wall core stiffness and the torsion resistance provided by the additional steel braced frames, a feasible and efficient design has been created. Further detailed calculations and finalized members selections are provided in Appendix D.

Final Design Steel Material Strengths

	Materal	Fy(ksi)	Fu(ksi)
Braces	A500 Gr. B	42	58
Colums	A992	50	65
Beams	A992	50	65

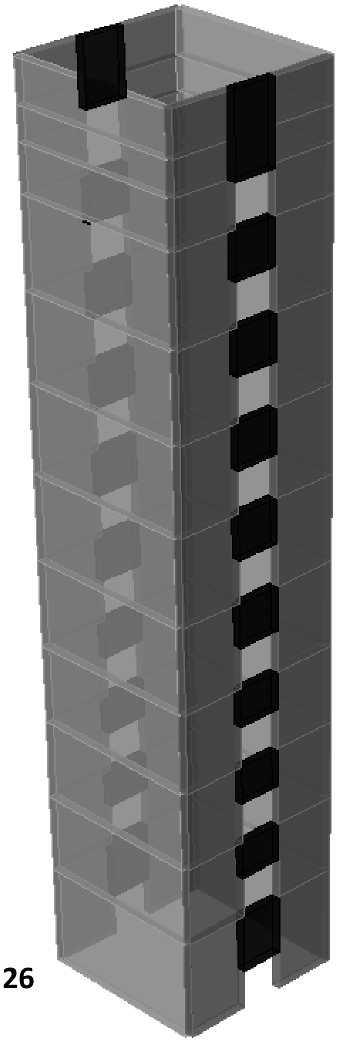


Figure 26

FINAL DESIGN CRITICAL VALUES				
Level	**Story Height (ft)**	**Δ_{max} (in)**	**V_{shear} (k)**	**Ax**
Pent	162.58	0	0	0
Roof	137.75	1.93	1.93	0.97
8th	117	1.53	1.53	0.95
7th	102	1.63	1.63	0.96
6th	87	1.67	1.67	0.96
5th	72	1.67	1.67	0.97
4th	57	1.60	1.60	0.96
3rd	42	1.30	1.30	0.96
2nd	28	1.10	1.10	0.99
1st	14	0.73	0.73	0.91
Ground	0	0	0	0

Table 24

Strength Check of Critical Steel Members

In order to verify the accuracy of the design conducted in ETABS, a manual strength check was performed for critical bracing members and columns in the north/south and east/west directions. The critical brace force of 347.51k was identified in the north/south direction located on the ground floor at the column line C frame (see Figure 27). In the east/west direction, a critical brace force of 166.38k has been identified on the third floor at the column line 7 frame (see Figure 28). Both braces were checked for axial capacity from controlling load combination 5 (D+1.0E+L+.2S) and amplified by a redundancy factor of 1.3 in accordance with Chapter 12 of ASCE7-05. Both critical have passed strength design conservatively. However, the excess strength is needed due to the strict drift control limitations.

All lateral columns have also been checked for strength due to the variance caused by differential gravity and lateral load relation throughout the Cancer Hospital. Similar to the brace strength check, the columns have been checked for axial capacity from controlling load combination 5 (D+1.0E+L+.2S) and amplified by a redundancy factor of 1.3 in accordance with Chapter 12 of ASCE7-05 and also found to conservatively pass. See Appendix D for detailed calculations.

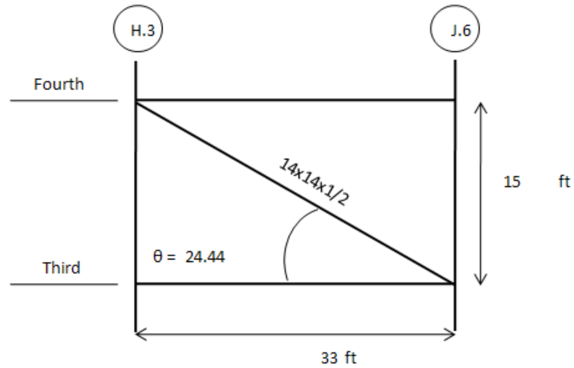


Figure 27

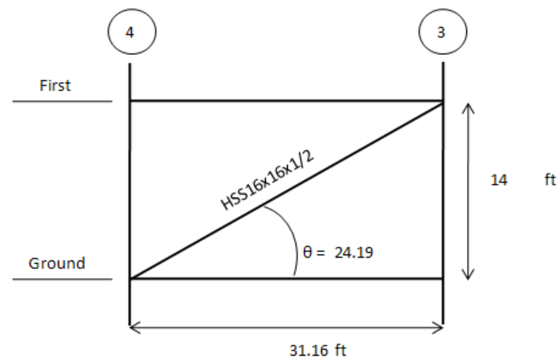


Figure 28

Slenderness Check

Each brace has been checked for slenderness to ensure efficient energy dissipation in accordance with AISC Seismic Design Manual and industry recommended provisions. The required limit in slenderness for Seismic Category D is established with the following equation:

$$\frac{\mathrm{KL}}{\mathrm{r}} \leq 200$$

However, industry standard recommends an efficient design in a high seismic region to follow a more stringent limit:

$$\frac{KL}{r} \leq \frac{720}{\sqrt{\mathrm{Fy}}}$$

All HSS14x14 and HSS16x16 members adhere conservatively to these limits with an approximate value of:

$$\frac{KL}{r} = 60$$

Width-to-Thickness Ratio Check

According to industry reccomendation and table B4.1 of the AISC Steel Construction Manual, in stiffened rectangular sections the width to thickness ration must not exceed the following in compression:

$$\frac{b}{t} \leq 1.4\sqrt{\frac{E}{Fy}}$$

HSS shapes used in the redesign of the Cancer Hospital have been selected in order to adhere to this limit

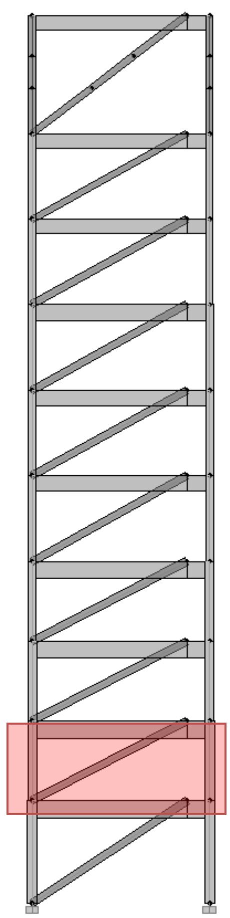


Figure 29

Critical Steel Connection Design (Master's Requirement)

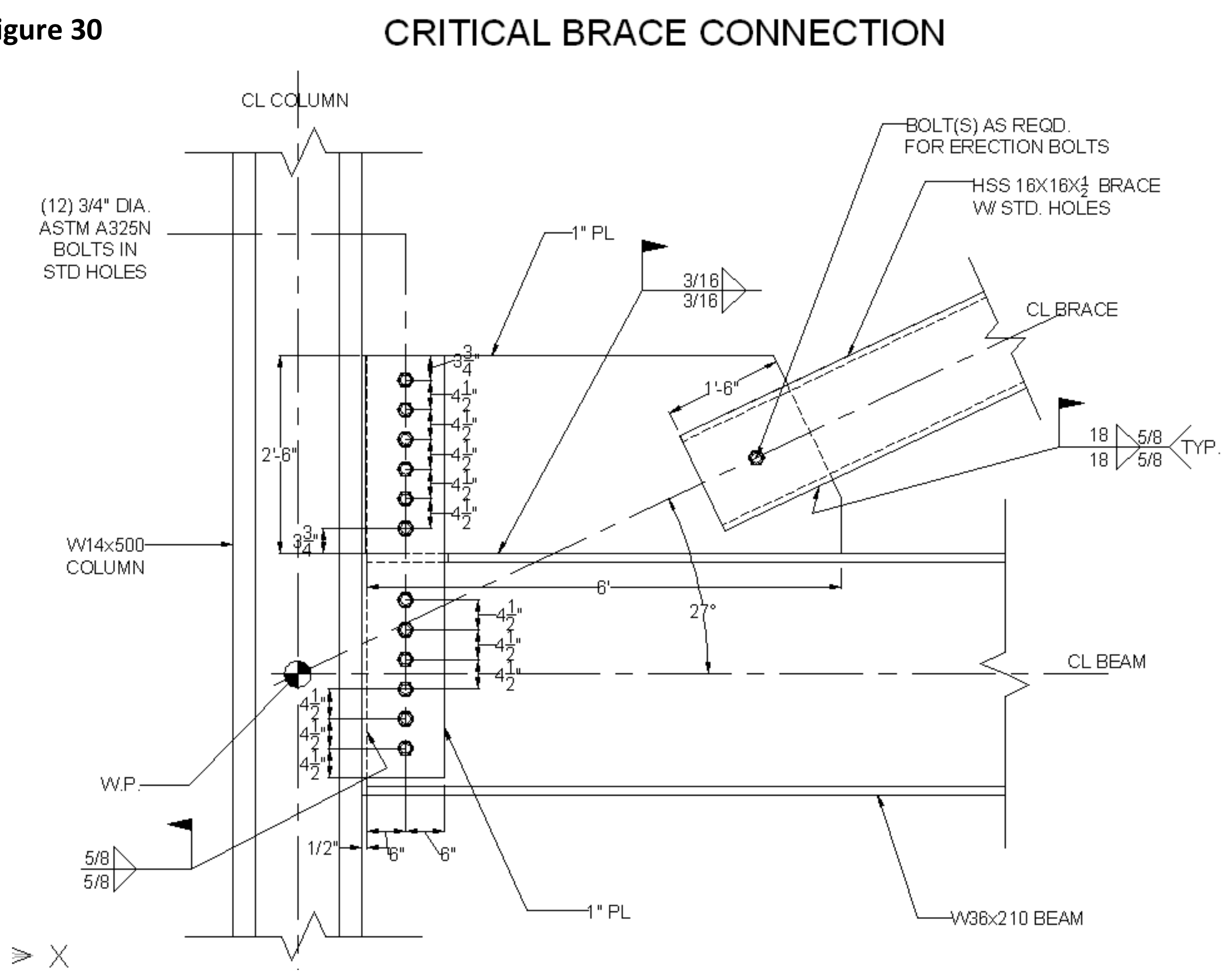
To ensure the lateral system response characteristics are consistent with the design calculations under the seismic load parameters, a critical connection has been identified and designed for maximum efficiency. This critical connection has been found to occur at the ground floor in the column line C frame (see Figure 29).

As previously mentioned, all frames have been designated as eccentric and will be designed as such in accordance with the *AISC Seismic Design Manual*. Each side of the braces will have a different typical connection and both will be analyzed. One connection will be an ordinary brace to beam/column connection – bolted and the other will be and eccentric welded connection.

The ordinary concentric connection which has been analyzed consists of a HSS16x16x1/2 brace framing into a W14x500 column and a W36x210 beam through a welded gusset plate bolted connection (see Figure 30). Loads for design have been amplified by a redundancy factor of 1.3 in accordance with Chapter 12 of ASCE 7-05. The HSS brace has been designed to connect to the plate through a 5/16" fillet weld on both sides with a ¾" bolt to be used in erection. After calculation, the thickness of the plate has been designed at 1" connected to the beam with 3/16" fillet welds. Calculations involved in the sizing of the plate included a Whitmore section and the incorporation of shear lag in the brace. 6 ¾" A-325N bolts have been design to complete the single connection to the column.

Similar to the ordinary concentric connection, the eccentric connection has been designed to frame into a W36x210 beam through a welded gusset plate. A similar geometric connection has been used in order to simplify design and the associated assumptions on strength have been used (see Figure 31). This primary difference is that this connection has been set back from the column by 4' and is intended to induce a failure mechanism in the beam in the event of an earthquake. In order to effectively transfer loads to the beam, stiffener plates have been designed for use in the beam web to prevent premature crippling and on the plate to prevent connection failure. The stiffeners on both the plate and the beam web are attached using a 3/16" weld and have been detailed to have a 1" by 1" notch. The stiffener plate dimensions have been designed to extend the full length of the beam with a width of 5-3/4" and a thickness of 1.25". The plate configuration has been dimensioned similarly to a W44x262 and the connection to the beam has been detailed and analyzed as such in order to gain maximum load transfer and for simplicity in design. See attached Appendix D for detailed calculations.

Figure 30



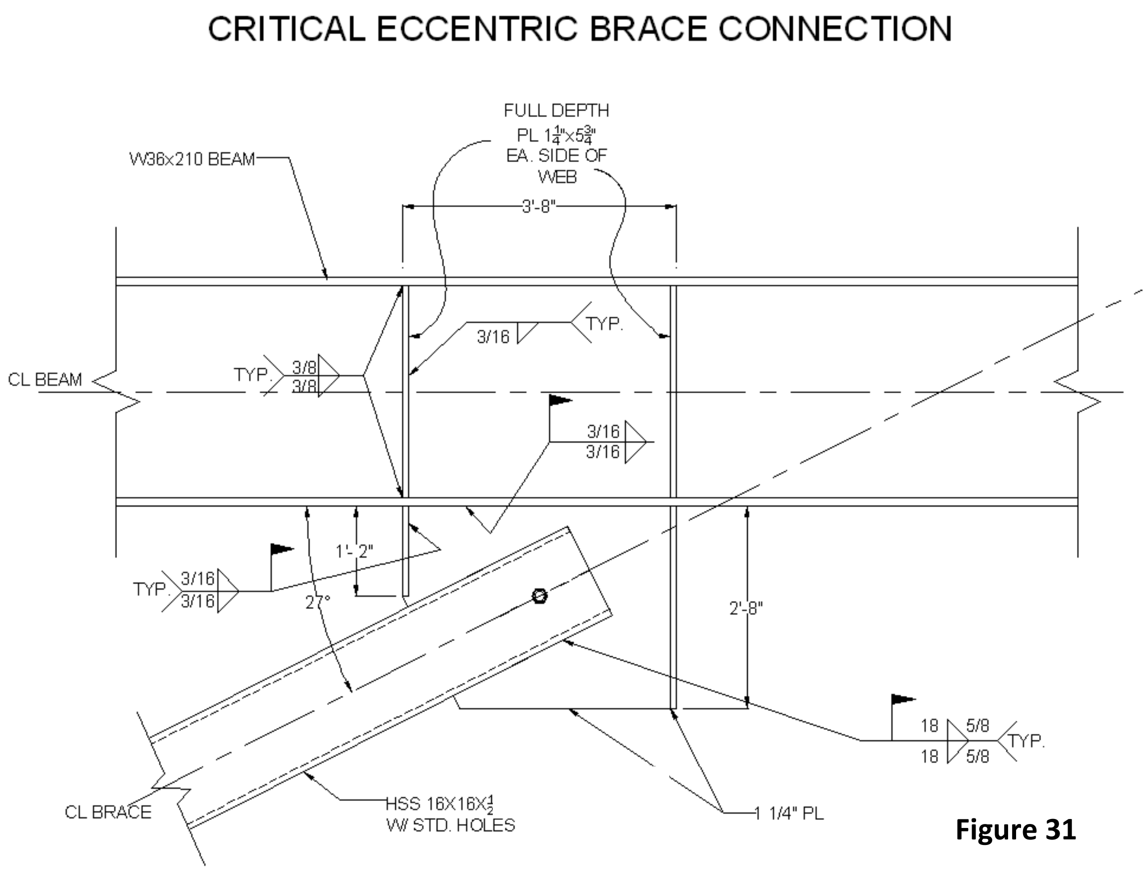


Figure 31

Concrete Shear Wall Core Design

In the initial design, the concrete shear walls were given an 18” thickness in order to minimize the effect on the existing architectural design and to more evenly distribute load to the braced frames located on the perimeter. The design of the core has required the design of the walls in both the north/south direction as well as the east/west direction. The east/west direction requires the additional design of coupling beams at various sizes due to the non uniformity of opening dimensions along the elevator shaft at different levels. Each shear wall has been designed to handle both gravity and lateral loads as well as their interaction in regard to both axial compression and uplift from overturning moment (see Figure 32). As with the steel design all applicable ASCE 7-05 load combinations have been considered and a redundancy factor of 1.3 has been applied to the required loads. See Appendix D for a detailed load chart used for shear wall design.

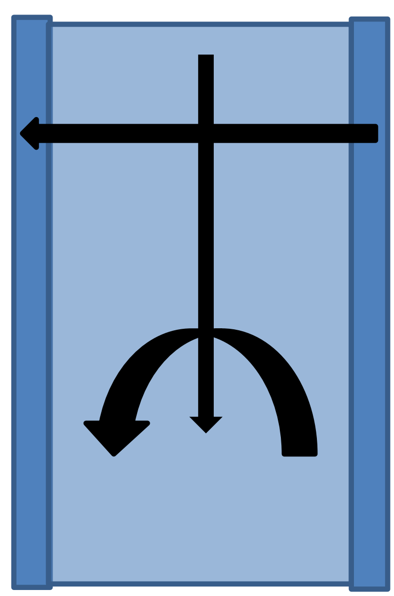


Figure 32

Applicable load combinations considered include:

1.4D

1.2D+1.6L

1.2D+1.0E+L

.9D+1.0E

Shear Walls

In order to comply with Chapter 12 of ASCE7-05, the reinforced concrete lateral element must be designed as a special shear wall in order to gain the R value of 6 and comply with the allowable height. The design of the reinforced concrete shear wall core has been done in accordance with ACI 318-05. Specific applicable sections which have been used include; chapter 7 and 12 for reinforcement, chapter 11 and 14 for shear wall design, and chapter 21 for special earthquake resistant structures.

Each shear extends up to a height of 155.75' to provide resistance to the lateral loads at all levels. The shear walls in both directions have been found to require boundary elements and have been designed as such. All concrete used in design has a strength of 4000psi and all reinforcement will have a yield strength of 60ksi.

Design of each shear wall has been conducted by:

- The determination of the need for a boundary element given the specified dimensions
- Sizing boundary element based on requirement of of ACI 318-05 21.9.6.4
- Determining transverse and longitudal reinforcement based on a trial design and the assumption that overturning moment will control design and minimum reinforcement will satisfy shear and flexural demand
- Check shear capacity with assumption using equation:

$$\phi Vn=Acv[(\alpha c)(f'c^2)+\rho t(fy)]$$

- Check flexural capacity using interaction diagram and equation:

$$Cu=(Pu/2)+(Mu/d)$$

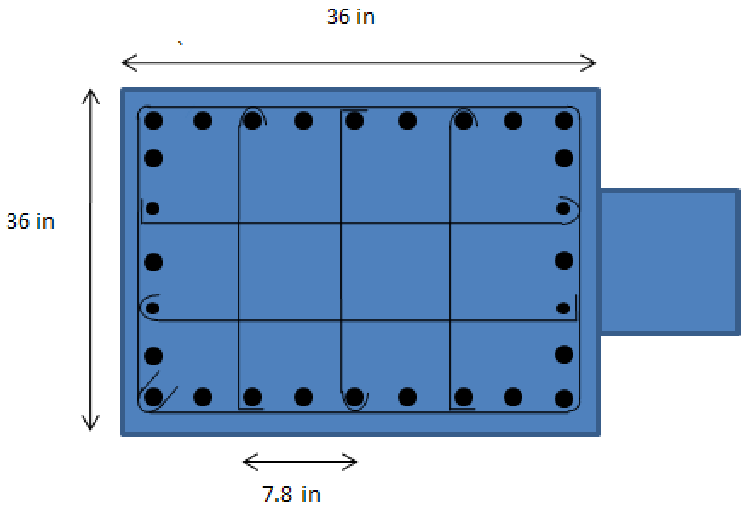
- Design Reinforcement for controlling axial load capacity using equation:

$$\phi Pn=.8\phi[.85(F'c)(As-Ast)+(Fy)(Ast)]$$

- Determine spacing and hoop design in accordance with ACI 318-05 Chapter 21

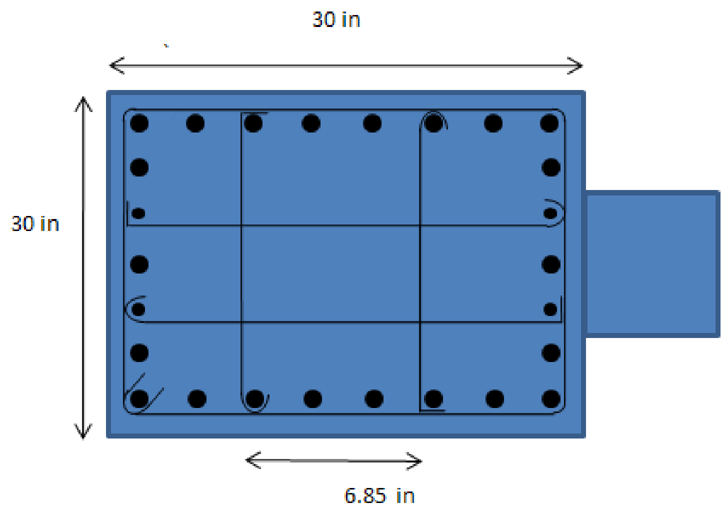
In the north/south direction, the walls stretch 31'-6" and have been designed with 3'x3' boundary elements in order to handle the given critical loads found at the base of the wall (see Figure 33). Both the shear walls along column line G and H have been designed to carry the found critical load in order to simplify construction. The factored critical axial load was found to be 1541k, the factored critical shear load was found to be 2910k and the critical factored overturning moment was found to be 68,678 ft-k. Considerations influencing the sizing and the placement of rebar in the shear wall design include minimum reinforcement, shear capacity, axial capacity, and flexural capacity. The finalized designs for shear walls G and H is presented in the following Figure 33. See Appendix D for detailed calculations and formulas used.

Typical SW G and H Boundary Element



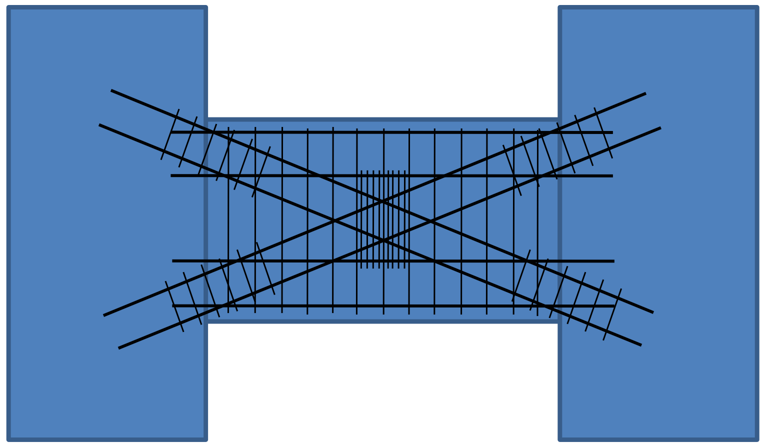
Boundary Element:	3" by 3" boundary element
Longitudal Shear:	2 curtains # 6's spaced 6" o.c.
Transverse Shear:	2 curtains # 6's spaced 6" o.c.
Axial:	24 #11's per boundary element
Stirrups:	5 #5's spaced spaced 6" o.c.
Hoop:	#5's spaced 6" o.c.

Typical SW 2 and 3 Boundary Element



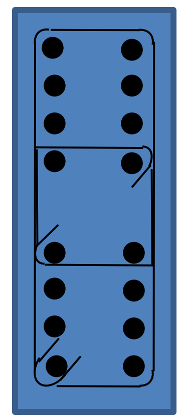
Boundary Element:	2.5" by 2.5" boundary element
Longitudal Shear:	2 curtains # 6's spaced 6" o.c.
Transverse Shear:	2 curtains # 6's spaced 6" o.c.
Axial:	22 #9's per boundary element
Stirrups:	4 #5's spaced spaced 6" o.c.
Hoop:	#5's spaced 6" o.c.

Figure 33



The east/west direction walls are also 31'-6" in length. However, due to the required 8.5' openings the wall has been split into to 2 walls with boundary elements and a coupling beam linking the 2. Each 11'-8" wall part has been designed to include a 2'-6'x2'-6" boundary element at each end in order to handle the given critical loads found at the base of the wall (see figure ??? and ???). As with the north/south direction shear wall design, both walls along column line 2 and 3 have been designed to carry the found critical load in order to simplify construction. The factored critical axial load was found to be 770k, the factored critical shear load was found to be 941k, and the critical factored overturning moment was found to be 15767 ft-k. Considerations influencing the sizing and the placement of rebar in the shear wall design include minimum reinforcement, shear capacity, axial capacity, and flexural capacity. The finalized design for shear walls 2 and 3 is presented in the previous Figure 33. See Appendix D for detailed calculations and formulas used.

Figure 34



Coupling Beams

The coupling beams in the shear walls along column lines 2 and 3 have also been designed in order to ensure proper load transfer (see Figure 34). Values for shear have been obtained from the Modal Analysis conducted using ETABS. 3 different sizes of beams exist in the new design. These 3 vary in depth between 82", 94", and 130". All coupling beams have been designed in accordance with ACI 318-05 21.7.7.2 and 21.7.7.3.

Diagonal reinforcement has been designed based on a clear length-to-overall depth ratios for each coupling beam (see Figure 34). According to ACI 318, diagonal reinforcement is only required when 4* √f'c*Acw is exceeded. However, diagonal reinforcement has been used on every level for ease in construction and for added redundancy.

In addition to diagonal reinforcement, transverse reinforcement has also sized and spaced according to ACI 318-05 21.4.4. The finalized design for each coupling beam at each respective level has been provided in the following Table 25. Detailed calculations are provided in Appendix D.

COUPLING BEAM DESIGN			
Level	**Member**	**Sturrups**	**Diagonal Rebar**
Roof	C130X18	9 #4 stirrups @ 4"oc	4 #5
8th	C94X18	7 #4 stirrups @ 4"oc	6 #9
7th	C94X18	7 #4 stirrups @ 4"oc	8 #11
6th	C94X18	7 #4 stirrups @ 4"oc	8 #14
5th	C94X18	7 #4 stirrups @ 4"oc	8 #14
4th	C94X18	7 #4 stirrups @ 4"oc	8 #14
3rd	C82X18	6 #4 stirrups @ 4"oc	8 #14
2nd	C82X18	6 #4 stirrups @ 4"oc	8 #11
1st	C82X18	6 #4 stirrups @ 4"oc	6 #9
Ground	C130X18	9 #4 stirrups @ 4"oc	6 #5

Table 25

Foundation Design

Under the increased lateral loads, special attention has been paid in the redesign of existing foundations at the base of lateral resisting walls and frames. As mentioned in the background information of this report, the existing foundations consist of drilled gravity piers and caissons. For the purposes of this redesign, the bearing capacity will based on friction only, due to an uncertainty of sub terrain conditions at the new location in San Diego, CA.

Certain key assumptions have been made in order to provide a conservative yet efficient design. San Diego is known for its clay rich soil. Due to this fact, a conservative figure of 50ksf has been used for soil bearing capacity. Also, an angle of friction of 30 degrees has been assumed. This magnitude is used under conditions where a foundation is placed on non-compacted, fairly loose soil. Unit weight will be assumed to 108pcf in accordance with the value for stiff clay. A typical value for Kht of .3 has been used for the purposes of this hypothetical design.

In order to determine the required caisson length and width, gravity loads though column takedowns where found at the anchoring point of every lateral member. Both the maximum downward compressive force and the maximum uplift force where found using gravity loads in conjunction with the lateral seismic loads previously calculated. The determination of these loads has been done in accordance with ASCE7-05 and a standard safety factor of 3.0 has been applied.

Applicable load combinations considered include:

1.4D

1.2D+1.6L

1.2D+1.0E+L

.9D+1.0E

Upon determination of the critical loads, the depth of the caisson was determined to be 50' in order to gain the required soil friction capacity. This soil friction has been calculated in accordance with the formula:

$$Tu = \sum_{H=H0}^{H=H0+Depth} K_{ht} * \rho 0 * TAN(\delta) * S * H$$

From the soil friction capacity, both the axial and uplift capacity have been determined. An allowable 25% reduction has been applied in accordance with ASCE7-05. Uplift was found to control on all braced frames and shear walls. This is as expected due to the large lateral forces on each. From the analysis performed 3 sizes of caisson are required (see Table 26). Detailed calculations are provided in Appendix D

Tu = Ultimate Load Capacity in Tension (Uplift)
Kht = Ratio of Horizontal Effective Stress on Element when in Tension
ρ0 = Effective Vertical Stress Over Length of Embedment (Depth)
H = Length of Segment

FOUNDATION DESIGN									
Frame	Axial Gravity (k)	Overturn Moment (k-in)	Reduction	Reduced Moment (k-ft)	Frame Width (ft)	Axial Lateral (k)	Upliftreq'd (k)	Length (ft)	Caisson ϕ (ft)
Ba	456.08	371722.50	0.25	23232.66	31.16	745.59	289.51	50	6
Bb	456.08	355749.60	0.25	22234.35	31.16	713.55	257.47	50	6
Ca	456.08	413658.10	0.25	25853.63	31.16	829.71	373.63	50	9
Cb	456.08	397173.30	0.25	24823.33	31.16	796.64	340.56	50	9
SW G	859.32	622799.50	0.25	38924.97	31.5	1235.71	376.39	50	9
SW H	859.32	536695.10	0.25	33543.44	31.5	1064.87	205.55	50	9
SW 2a	485.05	103899.80	0.25	6493.74	11.75	552.66	67.61	50	4
SW 2b	485.05	117209.80	0.25	7325.61	11.75	623.46	138.41	50	4
SW 3a	485.05	107137.10	0.25	6696.07	11.75	569.88	84.83	50	4
SW 3b	485.05	109583.20	0.25	6848.95	11.75	582.89	97.84	50	4
7a	72.8541	88061.60	0.25	5503.85	33	166.78	93.93	50	4
7b	72.8541	88610.30	0.25	5538.14	33	167.82	94.97	50	4

Table 26

Building Envelope Redesign

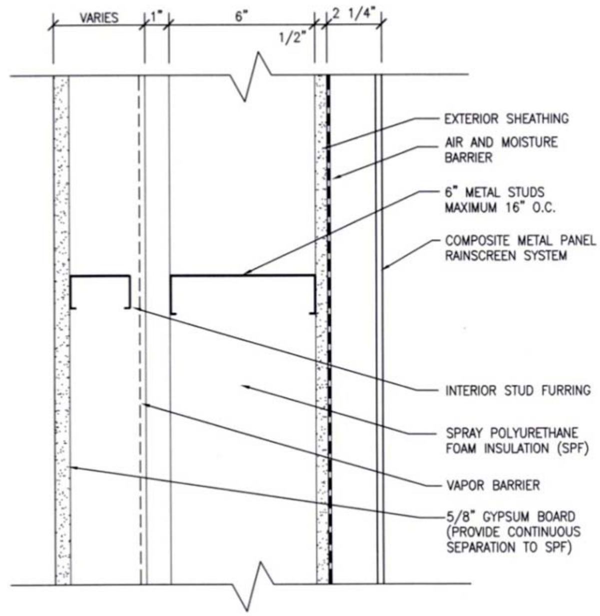


Figure 35

Arguably the most dominant and appealing architectural characteristic of the University Hospitals Case Medical Center Cancer Hospital is the 92,000 SF of curtain wall which envelopes the structure. Due to the large amount of fenestration on the building special care must be taken to ensure the maximum amount of thermal efficiency in non-curtain walls utilized around the structure (see Figure 35). The theoretical design relocation of the Cancer Hospital from Cleveland, OH to San Diego, CA has placed a different set of exterior conditions on the wall systems. In order to determine the most efficient design to be used under the new conditions, the existing wall system has been analyzed as well as 3 other commonly used walls systems. The 3 systems include a barrier wall system, a cavity wall system, and an EIFS system. After comparing the results, the optimal solution has been selected for use in the Cancer Hospital.

In addition to thermal wall analysis, the existing curtain wall has been modified to resist the new seismic loads. This new design has taken into account lateral pressures as well as recommended seismic fallout provisions. In accordance with the nation-wide response to terrorism in design, the new curtain wall system has been designed with consideration of blast loads.

Thermal Load and Moisture Analysis

Existing Wall System

The current building envelope of the Cancer Hospital consists of a combination of curtain wall and a typical barrier wall system. This barrier system consists of a metal screen façade mounted directly over an immediate air and moisture poly film barrier. 5/8" gypsum board serves a sheathing to the 2"x6" exterior metal studs. Between and around the metal studs 4" spray polyurethane foam has been utilized as the primary insulating element between the exterior elements and the typical 3'-1/2" interior furring. The interior furring is then sheathed with another 5/8" ply of gypsum board. In total, the existing wall system has a typical thickness of 13.3".

Using *Ham Toolbox,* an R value and condensation analysis has been performed under the given conditions in the existing Cleveland, OH location. The existing temperature and humidity conditions used were as follows in Chart 1.

Chart 1 :Climate Conditions in Cleveland, OH

	Winter		Summer	
	Temp (°F)	RH(%)	Temp (°F)	RH(%)
Indoor	70	25	75	50
Outdoor	1	76	88	65

Layer	Generic Material	Thick.	R Val.
1	aluminum clad.(unvntd), 1-1/2 ir	1.50	0.12
2	poly film, (10mil)	0.01	0.12
3	gypsum bd., 5/8 in., (#1)	0.63	0.46
4	ureth.(int.) insul., 4 in.	4.00	24.68
5	cavity, 3 in.	3.00	0.98
6	steel stud, 3-1/2 in.	3.54	0.12
7	gypsum bd., 5/8 in., (#1)	0.63	0.46
8			
9			
10			
11			
12			
	Total or (Layer 0)	13.30	26.94

Figure 36

The combined R value for the system was found to be 26.94 (see Figure 36). This falls within the reasonable industry standard of 20 to 30. No condensation was found to occur in this configuration in the summer. However, in the winter possible moisture issues have been found to occur towards the inside of the poly barrier (see Figure 37).

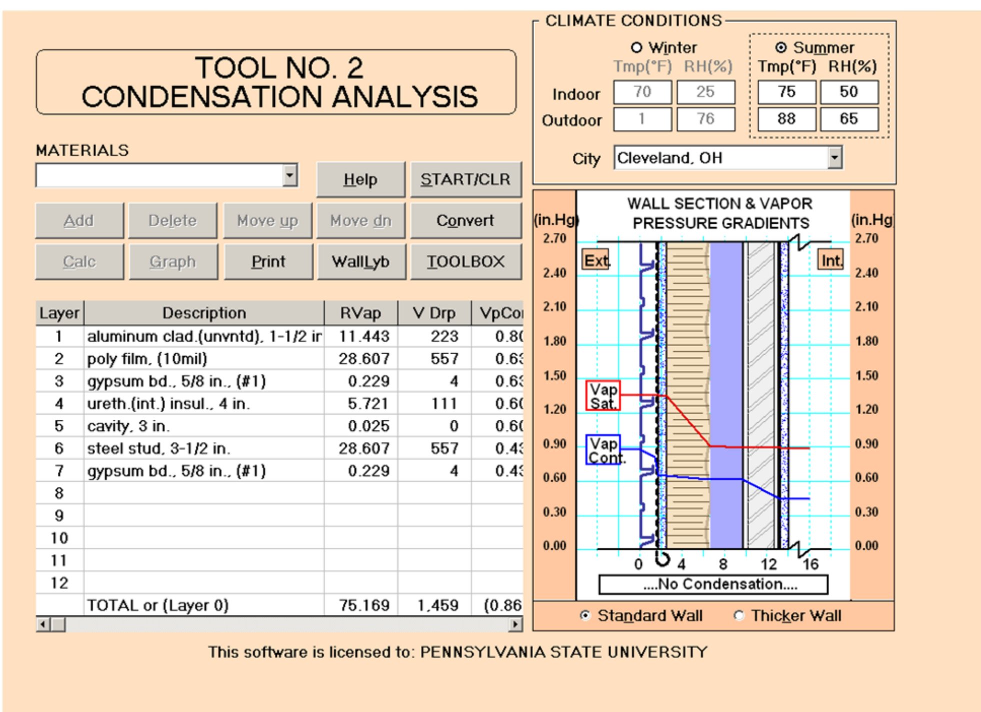


Figure 37

Barrier System in San Diego

The existing barrier system has been found to be relatively effective in Cleveland, OH. However, under the new San Diego, CA location, analysis is required to ensure efficiency. The new temperature and humidity are as follows in Chart 2.

Chart 2: Climate Conditions in San Diego, CA

	Winter		Summer	
	Temp (°F)	RH(%)	Temp (°F)	RH(%)
Indoor	70	25	75	50
Outdoor	45	60	88	70

Weather data from San Diego, CA has shown the region temperature to be relatively moderate with a much lower thermal fluxuation when compared to the climate in Cleveland, OH. For this reason, no condensation was found to occur with the given wall configuration. In order to increase efficiency of the system under the new conditions, the insulation has been decreased by 2”. Although not needed to protect against condensation, the poly film layer will still be used on the exterior of the wall in order to prevent moisture penetration from exterior conditions.

Cavity Wall

A typical cavity wall system has been investigated for use in the San Diego, CA climate (see Figure 38). The system consists of a typical ¾” external air barrier applied to a 4” brick wythe. A 4” cavity filled with 2” of rigid insulation has been placed between the brick and the 6” block wall holding it up. The interior surface consists of 5/8” gypsum board sheathing. A moisture barrier has been placed at the optimal position inside the 2” cavity in order make full utilization of the drainage plane. The total thickness with all components totals to 15.23”.

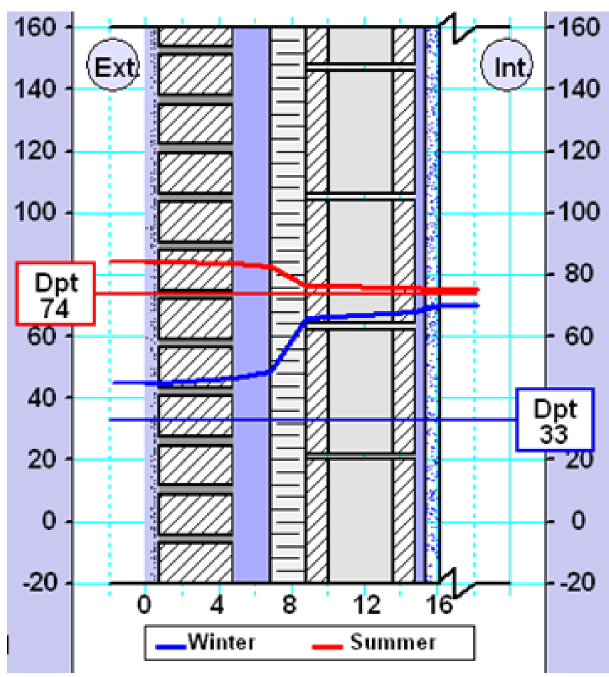


Figure 38

The cavity wall system provides an R value of 11.67. This value falls below typical acceptable standards. However, due to the mild fluxuations in the San Diego, CA climate, there is a low need for strong insulating characteristics. No condensation has been found to exist in this system under the given climate conditions (see Figure 39). This configuration has been found to be efficient. However, it varies the most from the existing design.

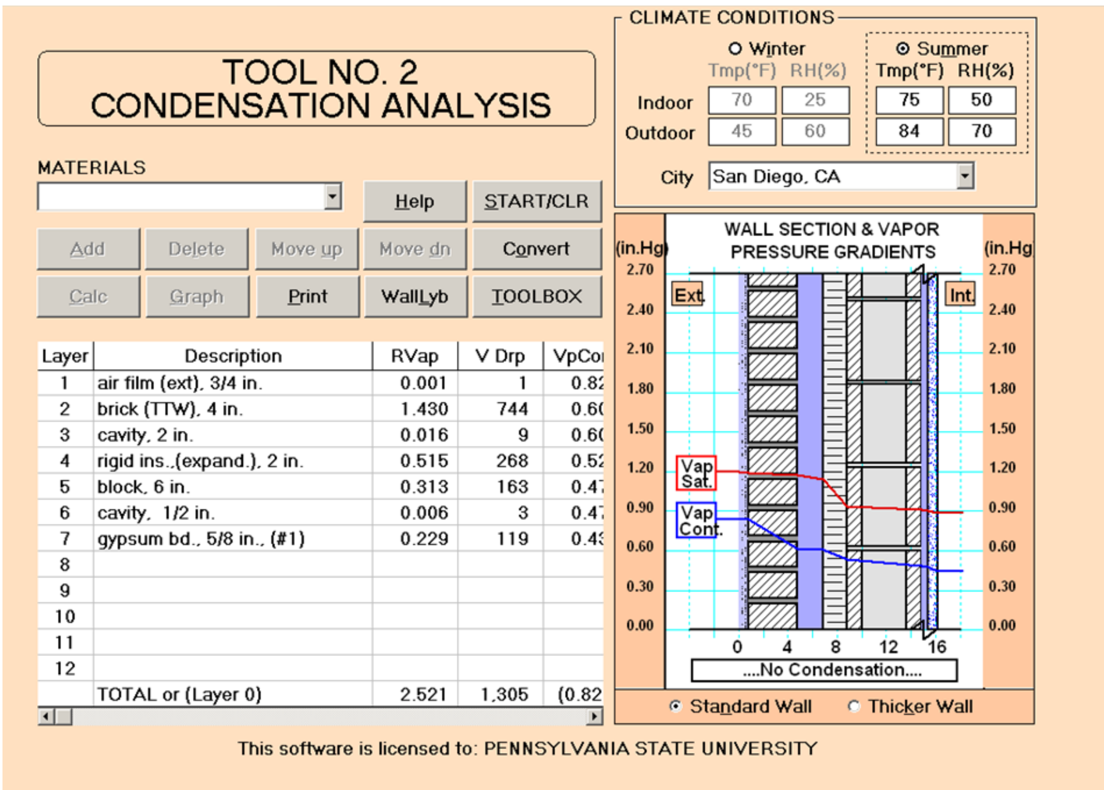


Figure 39

Exterior Insulation and Finish System (EIFS)

A typical wall system utilizing EIFS has been investigated for feasibility in the Cancer Hospital under the San Diego, CA climate conditions. The EIFS system is made up of a ¾" exterior air film on top of a 1-1/4" of EIFS layer. The interior section consists of a 5'-1/2" metal stud enclosed in 2 sheets of 5/8" gypsum board. The EIFS layer serves both as an insulator and an air and moisture barrier (see Figure 40)

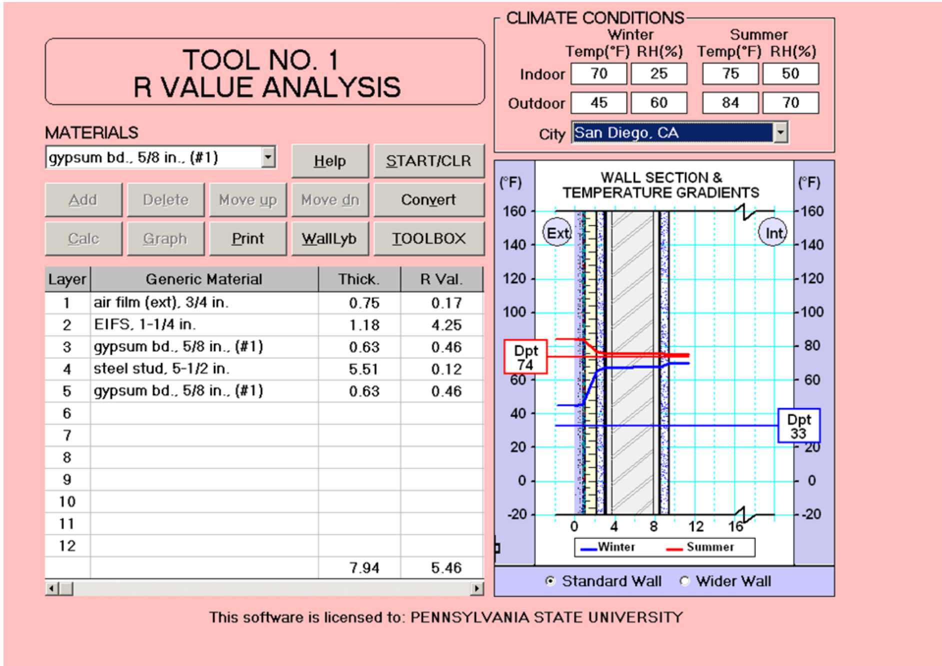


Figure 40

An R value of 5.46 was found to exist with the given elements. This value is generally very low. However, the EIFS system has been found to be optimal for the low fluxuating thermal loads of San Diego, CA. No condensation has been found to exist under summer or winter conditions (see Figure 41). This EIFS wall system spatially fits well with the existing design and decreases the need for wall space to a width of 8". However, any penetration of the EIFS barrier will rapidly decrease the efficiency of the system. Penetrations in many locations throughout the Cancer Hospital are necessary due to the irregular curved shape of the curtain wall.

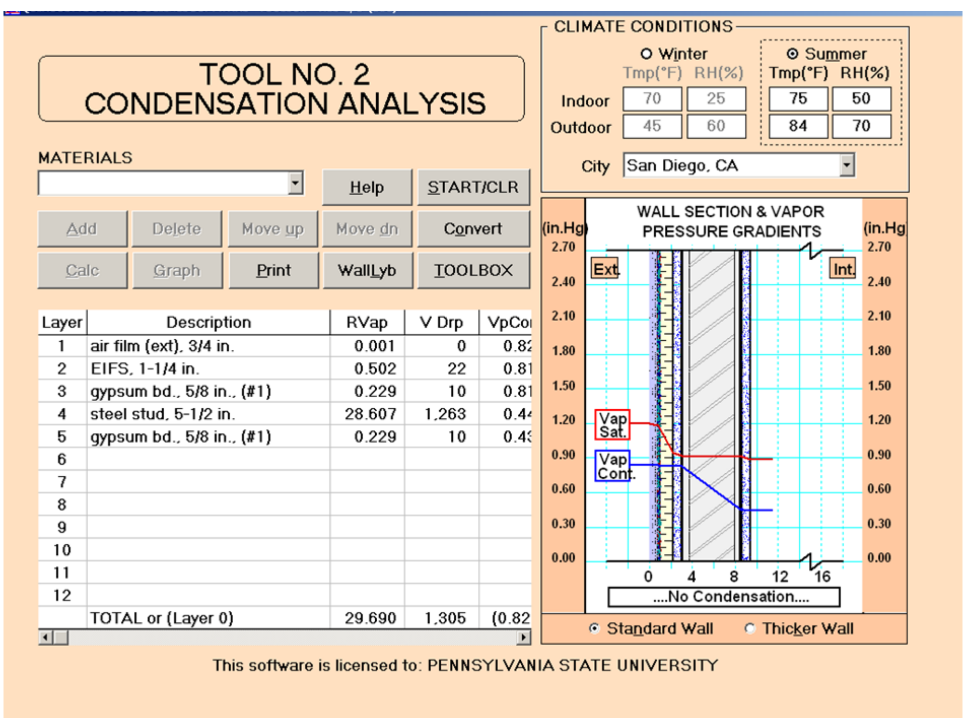


Figure 41

Wall System Comparison

The existing barrier wall system works well with the architectural design and provides adequate insulating and moisture prevention capabilities. However, the design is slightly conservative and relies heavily on the maintaining of an efficient exterior seal to prevent moisture penetration. This causes a significant efficiency concern over the lifespan of the wall. The cavity wall provides adequate thermal resistance and accomplishes the most effective moisture penetration resistance through use of a drainage plain. However, The cavity wall system varies from the original design the most out of the 3 systems investigated. The wall system using EIFS reduces the needed wall thickness and provides a more efficient value for thermal resistance for the climate of San Diego, CA. Although the EIFS system works well with the typical wall design throughout the Cancer Hospital, the many irregular points of interaction with the curved curtain wall will force penetration in the system and dramatically decrease efficiency.

System Selection

Taking into account the advantages and disadvantages of each system, the barrier wall system has been determined to be the most efficient and feasible for the design relocation. Although over conservative, the barrier system works well with the existing architecture and provides adequate thermal and moisture resistance. A maintenance and quality assurance plan will be required to ensure the efficiency of the external air and vapor barrier and prevent failure of the system over the lifespan.

Load Resistance Design

As previously stated, the increased load from seismic forces has been found to require a redesign of the existing curtain wall system. The current configuration has been checked for lateral resistance as well as seismic drift restraints and redesigned as necessary. In addition to the consideration of lateral pressures, a resistance to blast loads has been considered in the final design.

Lateral Force Resistance

The current curtain wall system consists of laminated glass units (LGU) spaced in conjunction with steel mullions and transparent spandrels. Each LGU consists of 2 ¼" plies of Annealed Glass (see Figure 42). A typical glass unit has been selected from the 8th floor and analyzed for lateral strength in accordance with *ASTM 1300.*

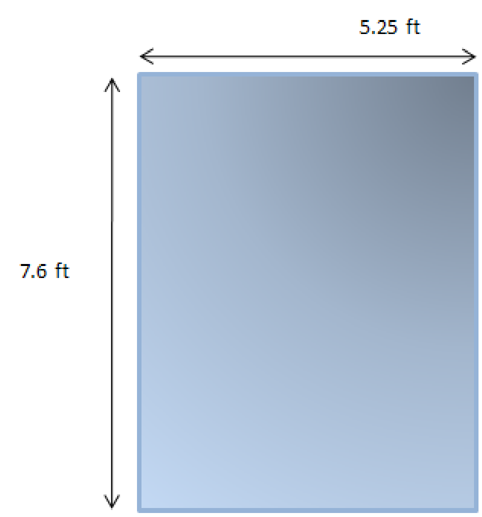


Figure 42

Seismic loads have been assumed to control the design of the LGUs and long term loading has been neglected. The selected window has a height of 7'-6" and a width of 5'-3" providing an aspect ratio of 1:1.5. From *ASTM 1300* for annealed laminated glass the current design was found to have a Glass Type Factor of .9 and a Load Share Factor of 2. From the *ASTM 130*0 load charts, the LGU was found to have a Non-Factored Load of 1.35 (see Figure 43).

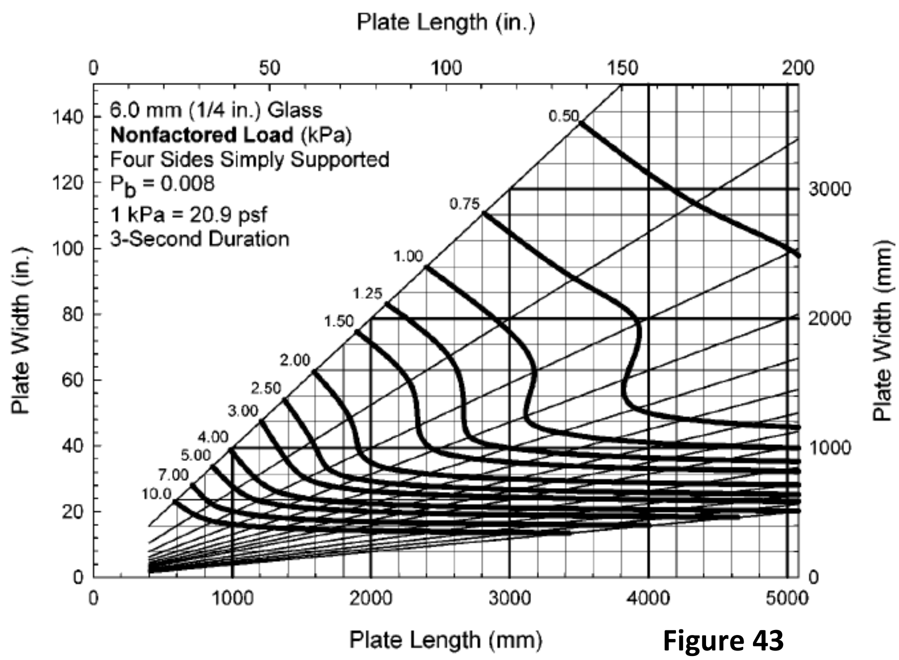


Figure 43

Form the load resistance equation, the lateral resistance capacity has been found to be 50.78psf. This is adequate to carry the required wind load of 41.85psf from Cleveland, OH. However, the seismic load of 156.47psf from San Diego, CA exceeds the capacity.

Load Resistance = Glass Type Factor x Load Share Factor x Non-Factored Load

In order to increase the strength capacity to resist lateral loads, a 2 ply ¼" Fully Tempered Laminated Glass Unit has been selected for use in the new design. The change in glass type increased the lateral load capacity to 198.55psf making the curtain wall system now adequate to carry the required seismic loads. See Appendix E for detailed calculations.

Seismic Drift Resistance

In addition to handling the lateral pressures from the given seismic loads, the glass units of the curtain wall have been designed to adhere to drift limits for seismic drift established in *Design of Architectural Glazing to Resist Earthquakes* by Richard A. Behr, P.E., F.ASCE (Behr 2006). Design consideration of seismic drift decreases the high vulnerability to damage of curtain walls during earthquakes.

The key consideration in this design aspect is to allow a drift clearance greater than the critical drift in which fallout occurs. The fallout drift or $\Delta_{fallout}$ is determined using the following equation where Dp represents the relative seismic displacement:

$$\Delta_{fallout} = 1.25 \times I \times Dp$$

$$Dp = \frac{\text{glass panel height}}{\text{story height}} \times \text{story drift}$$

Using these equations, $\Delta_{fallout}$ was found to be 1.71”. In order to accommodate this amount of drift, a 3/8” gap has been designed around the frame of each LGU (see Figure 44). See Appendix E for detailed calculations.

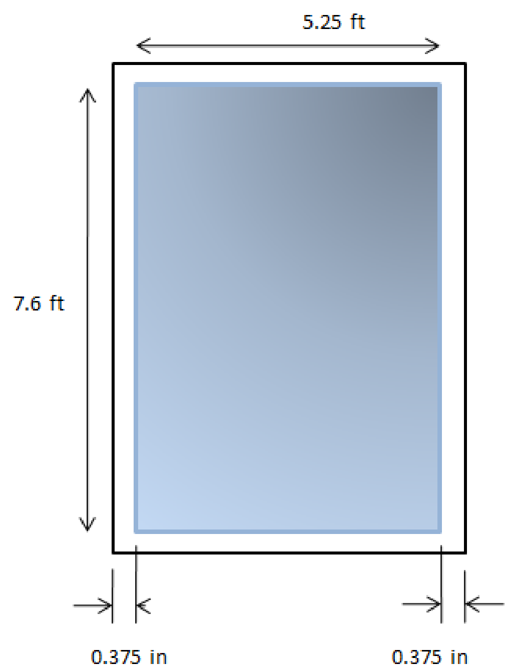


Figure 44

Blast Resistance

Designing for blast loads has become more common due to an increase in terrorist activity. The redesign of the curtain wall system of the Cancer Hospital has taken blast resistance into account in conjunction with design for lateral pressures (Norville 2006). Form the *Blast-Resistant Glazing Design* by H. Scott Norville, P.E., M.ASCE; and Edward J. Conrath, P.E., M.ASCE, lateral pressures have been established from blast base on the size of the charge and the standoff distance (see Figure 45).

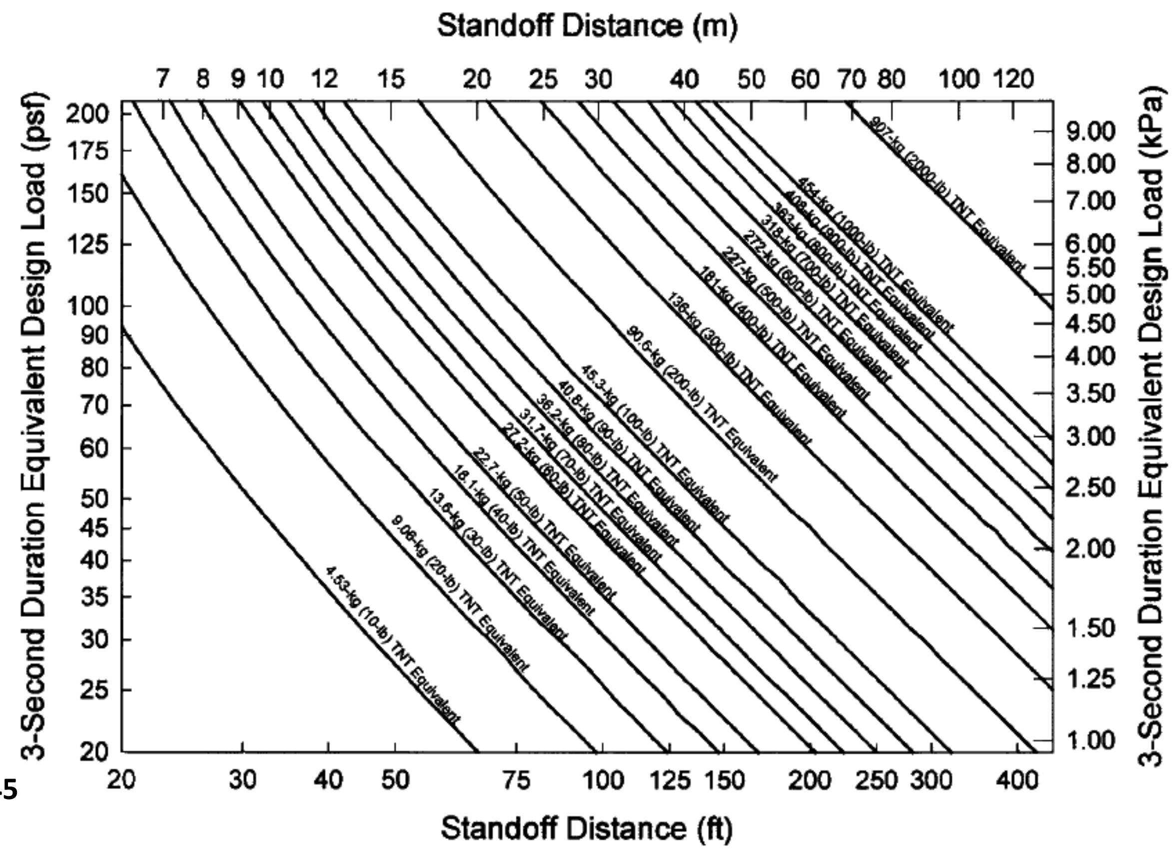


Figure 45

Minimum thickness has been established using the above chart and lateral capacity from ASTM 1300 (see table ???). Based on this acquired data, an increase in thickness to 5/16” will resist a charge of 100lb at 50’ and a 500lb charge at 100’.

100lb Charge

Dist	P_{table} (psf)	P_{actual} (psf)	T_{min}
50'	165	43.42	5/16"
100'	71	18.68	3/16"
200'	33	8.68	1/4"

500lb Charge

Dist	P_{table} (psf)	P_{actual} (psf)	T_{min}
50'	N/A	N/A	N/A
100'	180	47.37	5/16"
200'	85	22.37	1/4"

Final Curtain Wall Design

The final design of the individual glass units of the curtain wall will consist of 2 plies of 5/16” Laminated Fully Tempered Glass Units. This LGU design will be sufficient to carry the required lateral pressure, resist the seismic drift, and adequately resist blast load.

Schedule and Cost Analysis

The new Cancer Hospital design has added additional braced frames and a concrete shear wall core to the existing design. This has been found to have a major impact on not only the cost of the structure, but the construction schedule itself. It has been assumed that the Cancer Hospital would still be financed under the Vision 2010 project and must continue to adhere to the established construction time constraints. The effect on both cost and impact on schedule have been analyzed and a proposed solution in order to account for the additional time added has been found.

Existing Schedule

As previously stated, the current construction schedule of the Cancer Hospital begins July, 2008 and finishes December 2010 in accordance with the Vision 2010 constraints. In order to compare the impact of the new design on the existing schedule, a typical construction schedule has been created in effort to accurately portray the tasks and time periods of the original construction.

In order to create a realistic schedule, the Cancer Hospital floor plan has been split in parts to be constructed at a specified sequence. A single mobile crane has been selected for use in construction and crews will be assigned to each separate aspect. For instance, a crew will work on concrete components while a crew sequentially works on the steel structure. The sequenced sections vary by level and will be as follows (Figure 45):

Ground / Sub Floor **1st Level – 3rd Level**

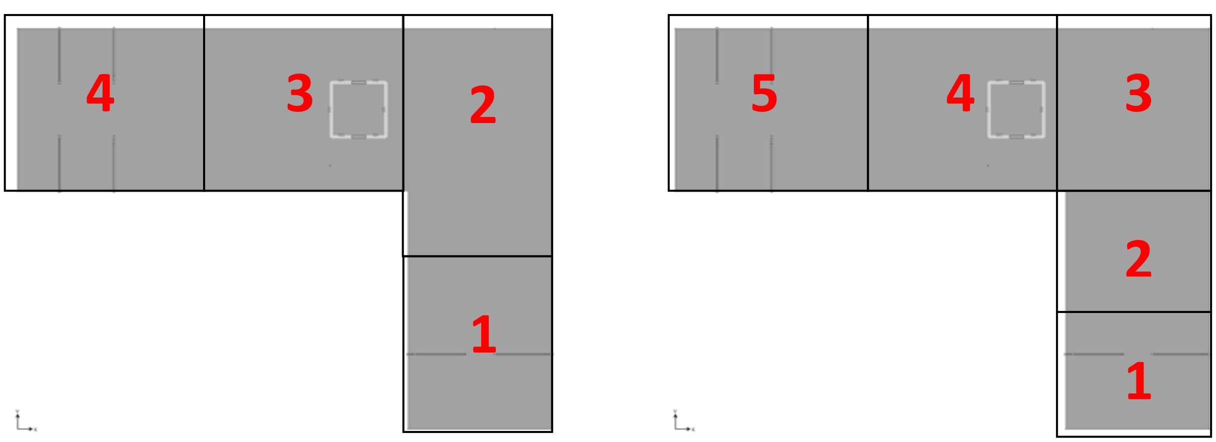


Figure 45

4th Level - Roof

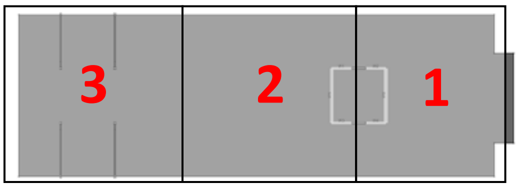


Figure 45

The schedule created has split work tasks into two main types, structural steel and concrete. In order to determine the amount of time in which each task will take, 30 pieces have been estimated to be able to be constructed per day. A member count then revealed the number of days it would take to finish a section when divided by this estimate. The same strategy has been applied with the composite concrete slab having an average completion rate of 140 cubic yards per day. The conservative concrete value has been obtained from *RS Means* for a 6" pumped slab.

The schedule begins by allowing drilling and pouring of caissons as well as associated foundations such as grade beams, footing, walls, and slab on grade. Upon completion of the slab on grade, the erection of the steel structure begins. A 3 floor window of time has been placed between the placement of floor slabs and the erection of the steel infrastructure. Concrete decks have been pumped for ease in construction and to better accommodate the sequential movements of crews.

Under the current tasks and conditions, the existing schedule has a completion date of March 12, 2009 with a total construction time of 179 days. This allows nearly 8 months for the completion of interior and MEP systems. The created schedule has been accepted as an accurate representation of the existing Cancer Hospital construction. See Appendix F for detailed schedule.

Revised Schedule

Under the new design, the construction schedule is similar to that of the existing with the exception of added brace frames and the shear wall core. Addition of steel members adds very little to the construction time. However, the concrete shear wall has been estimated to lengthen the construction time by 50 days. This increase in the critical path may cause the Vision 2010 date not to be achieved.

In order to prevent the any increase in construction time, a solution involving the addition of a second mobile crane has been utilized. The second mobile crane will only be needed for the construction of the lower extension of the "L shape". The utilization of the second crane on this section of the building will require its time in use to be approximately 17 days. This small amount of time will have little impact on the overall price, but has provided value in decreasing the overall construction time see (see Figure 45).

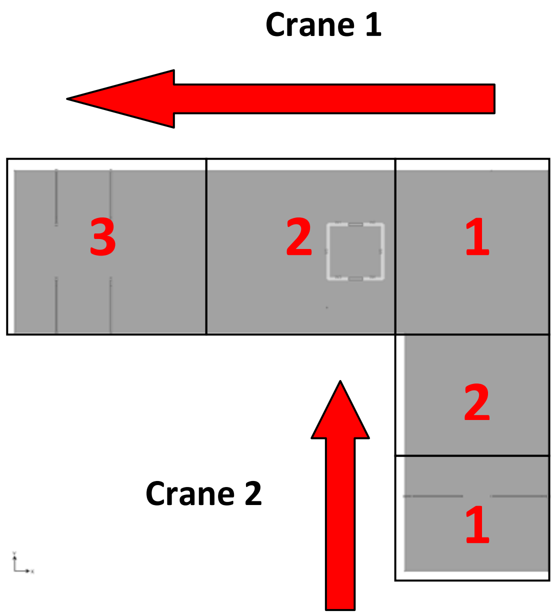


Figure 46

The new design schedule begins similar to the existing by creating foundations and pouring the slabs on grade. Immediately after the completion of the slab on grade, the concrete will begin to be constructed. Once the core reaches the 3rd floor, the steel gravity is started. The amount of lag time is necessary to allow curing of concrete and for the creation of the connections between the gravity system and the shear wall core.

The addition of the second mobile crane has reduced the critical path by 25 days even with the addition of the new steel members and shear wall core. The total completion time of the Cancer Hospital structure has now been found to be 154 days, placing the completion date on February, 5, 2009. This allows an additional month for the finalization of the building systems and ensures completion by the Vision 2010 deadline. See Appendix F for the detailed revised schedule.

Cost Analysis

A cost analysis has been performed in order to find the additional cost of the revision of the lateral system and evaluate the new design for constructability. The analysis has been performed by first determining the cost of the existing lateral system and then comparing this value to the increased figure from the new design.

Existing Lateral System Cost

In order to create the most accurate figure for the cost of the existing system, the members of the current lateral design have been counted and configured into their respective weight in tonnage. Lateral elements from each of the 6 steel braced frames were separated into 3 member types including columns, braces, and beams. Exact dimensions were found at each level and accounted for in the final steel tonnage. This value was then multiplied by a value of $4,275 per ton found in *RS Means.* This value includes cost of material, labor, and equipment for steel members in a 7 to 15 story building. From this figure and the performed member takeoff's, the estimated value of the existing lateral system was determined to be $1,553,483.28 (see Table 27). See Appendix F for detailed cost estimating data.

EXISTING LATERAL SYSTEM COST					
Level	**Steel Tonage**	**Steel Cost ($/ton)**	**CY of Concrete**	**Concrete Cost ($/CY)**	**Lateral Sys. Cost**
Frame @Line B	$79.44	$4,275.00	N/A	$139.50	$339,600.36
Frame @Line G	$53.26	$4,275.00	N/A	$139.50	$227,698.17
Frame @Line K	$101.65	$4,275.00	N/A	$139.50	$434,533.52
Frame @Line 2	$53.68	$4,275.00	N/A	$139.50	$229,466.75
Frame @Line 3	$52.79	$4,275.00	N/A	$139.50	$225,684.12
Frame @Line 7	$22.57	$4,275.00	N/A	$139.50	$96,500.37
				Total:	$1,553,483.28

Table 27

New Lateral System Cost

Similar to the method of calculation of the cost of the existing lateral system, the new revised lateral design has been analyzed. This takeoff for the new system proved to be slightly more complicated due to the use of 2 structural systems. All members of the additions of frames at column lines B, C, and 7 have been accounted for and converted to weight in tons. The shear wall system was split into two separate categories including steel reinforcement and concrete. The concrete used in the new system was found in cubic yards, while the steel reinforcement was converted to weight in tons.

EXISTING LATERAL SYSTEM COST					
Level	Steel Tonage	Steel Cost ($/ton)	CY of Concrete	Concrete Cost ($/CY)	Lateral Sys. Cost
Frames @Line B	$176.41	$4,275.00	N/A	$139.50	$754,146.90
Frames @LineC	$176.48	$4,275.00	N/A	$139.50	$754,432.26
Frames @Line 7	$35.19	$4,275.00	N/A	$139.50	$150,436.69
SW G and H	$98.18	$2,400.00	$573.13	$139.50	$315,586.23
SW 2 and 3	$129.33	$2,400.00	$360.86	$139.50	$360,731.48
				Total:	$2,335,333.56

Table 28

The price of rebar has been found to be $2,400 per ton in accordance with cost data obtained from *RS Means*. For the shear walls, a combined concrete and labor price has been found to be $139.5 per cubic yard also in accordance with *RS Means* cost data. The steel frames added in the new lateral system have increased the cost by only 7%. However, the addition if the shear wall core has increased the cost by an additional $676,317.00, bringing the total cost of the revised lateral system to $2,335,333.56 (see Table 28). See Appendix F for detailed cost estimating data.

Constructability Summary

After comparing data from the schedule and cost analysis, the new lateral system design has been found to be feasible under the new conditions. The use of a second mobile crane has reduced the critical path of the current construction schedule and will adhere to the Vision 2010 deadline with a minimal excess cost. The addition of the concrete shear wall core has increased the cost of the lateral system by nearly $700,000. However, when compared to the overall $232 Billion estimated budget of the current Cancer Hospital design, the increase in cost has been found to be practical given the added benefit to the structural system.

Conclusion

3 common seismic force resisting structural system solutions have been evaluated including; the strengthening of the existing structure, the creation of an seismic isolation joint, and the use of a reinforced concrete shear wall core. The reinforced concrete shear wall core was selected as the most efficient design which impacted the existing structural and architectural plans the least.

The new design uses the stiffness of the concrete shear wall core to provide strength and drift resistance. The design also includes eccentric steel braces which effectivly dissipate the energy from the seismic loads and resists the torsion caused by the addition of the concrete core. All critical elements of the new lateral system have been designed for strength and serviceability in accordance with apllicable industry codes and standards. The elements include the reinforced shear wall core, the eccentric braced frames and critical connection, and the new lateral foundation.

A new curtain wall design has been established which will include only minor changes to the exsisting plan in effort to reduce the impact on the origional architectural aethetic. The new design will now resist the required seismic load and associated drift, as well as a sizeable charge at close distance.

Analysis of the revised schedule yeilded a significant increase in construction time. However, with the additional mobile crane sequnced with the existing line items, a sizeable reduction was able to be produced. This reduction will allow for the current construction time to decreased by a month and aid in the completion of the new structure by the Vision 2010 deadline. A cost comparison determined the increase in price of the new lateral system to be reasonable and practicle given the added benefit to the structure.

Under the conditions presented, the relocation and reproduction of the existing design of the University Hospital Case Medical Center Cancer Hospital in Cleveland, Ohio to San Diego, California has been determined to be beneficial to the University Hospital’s Vision 2010 expansion project.

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