SMILOW CANCER HOSPITAL | NEW HAVEN, CONNECTICUT

DANIELE R. NAVARRETE | STRUCTURAL

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EXECUTIVE SUMMARY

Smilow Cancer Center

Located in the middle of New Haven, the addition of the Smilow Cancer Hospital to the Yale-New Haven Hospital complex will feature a state-of-the-art building with the latest equipment for the treatment of the disease. The several areas of specialization are separated among the sixteen stories of the building, with the larger equipment (i.e. MRIs, ultrasound, operating rooms) housed primarily on the lower floors and the 112 inpatient rooms—all single rooms—starting on the eleventh floor. As for the exterior, the façade emulates that of the surrounding buildings in the complex with its glass and terra cotta curtain walls. For ease of installation, a unitized curtain wall panel system was used: the glass and terra cotta come in pre-installed panels ready to be attached to the structure.

Being one of the most comprehensive cancer facilities in the New England region, the city of New Haven extended its Medical Zone to allow the construction of the Smilow Cancer Hospital back in 2006—despite some opposition from a few local residents. Those opposed to the new building were mostly concerned about issues such as traffic, parking, and "architectural integration with the neighborhood." The design of the building follows the 2005 Connecticut State Building Code which adopts mostly from BOCA National Building Code.

Structural Proposal

The highly critical nature of a hospital warrants an investigation into designing structural members against progressive collapse. In the case of Smilow Cancer Center, failure of even a few critical members on the ground level could easily lead to very catastrophic results. Damage to the hospital could cost up to the millions of dollars, not to mention the tragic and severe loss of life that could ensue. For this reason, this thesis explores the feasibility of replacing the steel framing system with a reinforced concrete frame system. The proposed system will be considered for progressive collapse focusing on typical corner, exterior, and interior columns on the first floor.

Breadth Proposal

Other than the superstructure, the exterior cladding/envelope of the building will also be considered in the redesign of Smilow Cancer Center. The pros and cons of installing a blast-resistant curtain wall system—at least at ground level—will be analyzed. Hopefully, this type of curtain wall would partially alleviate the effects of any destructive events that may occur. Of course, this system would have to be compared to the existing unitized curtain wall panel system in terms of blast-resistance and thermal performance.

Results

Past experiences with progressive, or disproportionate, collapse of structures prove that a relatively small, localized failure of one member can very well lead to large scale damage and loss of life. Based on the results of the structural analysis and design for Smilow Cancer Hospital, it is recommended that owners, developers, and engineers of more prominent building projects consider progressive collapse in the design of their structure. Employing the indirect method of providing adequate tie forces is a very efficient way of dealing with the possibility of localized column failure. Since steel reinforcing used in traditional concrete design can also act as ties, the only added cost is the detailing of tie connections, splices, and anchorage. And if it happens that the structure is already detailed for seismic loads, then the requirements for progressive collapse design are already met. Without a doubt, the potential of preserving human life far outweighs the small cost and effort of detailing a few rebar splices.

With the threat of terrorist attacks becoming more and more a reality, the idea of designing building glazing for blast loads has become more viable and warranted. Again, as with progressive collapse design, there is no way to put a price on human life. It is simply a judgment call on the design team’s part. The risk and probability of an explosive attack must be assessed as accurately as possible, and the need for blast-resistant glazing must be determined accordingly. The design of blast-resistant glazing for Smilow Cancer Hospital shows that one modification in the type of glass used can drastically change the blast resistance of a glazing panel system.

While maybe not as crucial as blast resistance, the effect of heat flow through glazing systems is an important issue that concerns both our environment and our economy. The use of high-performance glazing such as low-emissivity and insulating glass units has become almost standard practice as owners are seeing the benefits of spending a little more in the short run. More and more developers are looking ahead and considering the longer term life-cycle costs of their buildings. The cost analysis of switching from normal to low-E glazing done for Smilow Cancer Hospital shows how paying the small premium in the beginning of the project can lead to significant savings down the line.

SMILOW CANCER HOSPITAL | NEW HAVEN, CONNECTICUT

DANIELE R. NAVARRETE | STRUCTURAL

"...the most comprehensive cancer care facility between Boston and New York City, offering patients state-of-the-art care and treatment."

PROJECT TEAM

owner:
YALE-NEW HAVEN HOSPITAL

architect:
SHEPLEY BULFINCH RICHARDSON & ABBOTT

construction manager:
TURNER CONSTRUCTION COMPANY

structural engineer:
SPIEGEL ZAMECNIK & SHAH

mechanical/ electrical engineer:
BR+A CONSULTING ENGINEERS, INC.

civil engineer:
TIGHE & BOND CONSULTING ENGINEERS

GENERAL INFORMATION

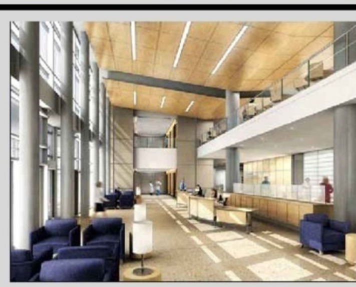
location:
20 York Street, New Haven, CT

size: 497,000 sq. ft.

height: 14 stories; 235 ft.

project cost: $253M

dates of construction:
Sept. 2006 - Feb. 2010



ARCHITECTURE

Part of the Yale-New Haven Hospital Complex

Unitized curtain wall panel system: glass + terra cotta

Two-story lobby with three-story glass awning overhanging front of building

Combination of cast-in-place concrete roof deck and metal decking; fully adhered thermoplastic polyolefin (TPO) sheet membrane roofing

STRUCTURE

4 ft thick concrete mat slab foundation over entire footprint (8 ft thick at shear wall locations)

4 concrete shear walls

Steel framing: combination of moment, lateral braced, and gravity frames

MECHANICAL

8 air-handling units:
6 on 5th floor + 2 on roof
70,000 cfm per AHU

LIGHTING/ELECTRICAL

480/277V 3 Phase, 4 wire system

208/120V 3 Phase, 4 wire system

Three 2000 kW/2500 kVA diesel generators

Low voltage lighting system: 208/120V, combination of incandescent, fluorescent, metal halide, and halogen loads

http://www.engr.psu.edu/ae/thesis/portfolios/2009/don5000

Final Thesis Report

SMILOW CANCER CENTER – YALE-NEW HAVEN HOSPITAL

20 York Street, New Haven, Connecticut

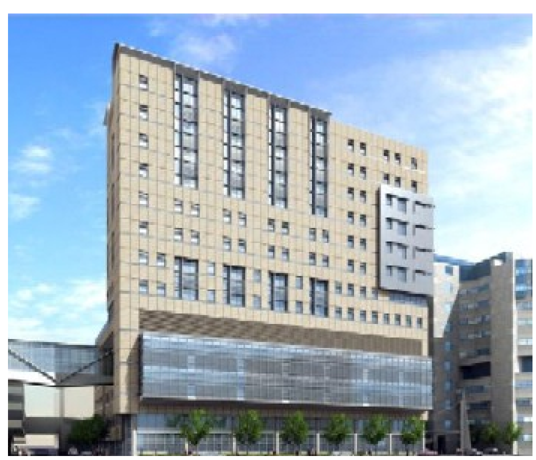
INTRODUCTION

As a hypothetical situation, during the early stages in the design of Smilow Cancer Center, the cost of structural steel has risen significantly. This development has suddenly made concrete the material of choice for the sixteen-story structure. And so, for the purposes of this thesis, the steel frame and concrete shear wall system of the existing design will be replaced with an entirely concrete system. This system will still utilize the four C-shaped shear walls around the core of the building, but it will substitute a reinforced concrete frame for the steel frame originally designed.

Also, because of the critical nature of the hospital, the new concrete frame will be analyzed for the effects of progressive, or disproportionate, collapse. In the case of Smilow Cancer Center, failure of even a few critical members on the ground level could easily lead to very catastrophic results. Damage to the hospital could cost up to the millions of dollars, not to mention the tragic and severe loss of life that could ensue. This thesis explores the feasibility of designing typical interior and corner columns and their surrounding elements for progressive collapse.

In addition to the superstructure, the exterior cladding/envelope of the building will also be considered in the redesign of Smilow Cancer Center. The pros and cons of installing a blast-resistant curtain wall system—at least at ground level—will be analyzed. Hopefully, this type of curtain wall would partially alleviate the effects of any destructive events that may occur. Of course, this system would have to be compared to the existing unitized curtain wall panel system in terms of blast-resistance and thermal performance.

OVERVIEW: Smilow Cancer Center



Located in the middle of New Haven, the addition of the Smilow Cancer Hospital to the Yale-New Haven Hospital complex will feature a state-of-the-art building with the latest equipment for the treatment of the disease. The several areas of specialization are separated among the sixteen stories of the building, with the larger equipment (i.e. MRIs, ultrasound, operating rooms) housed primarily on the lower floors and the 112 inpatient rooms—all single rooms—starting on the eleventh floor. As for the exterior, the façade emulates that of the surrounding buildings in the complex with its glass and terra cotta curtain walls. For ease of installation, a unitized curtain wall panel system was used: the glass and terra cotta come in pre-installed panels ready to be attached to the structure. The hospital's roof is a combination of cast-in-place concrete roof deck and metal (steel) decking. The insulation and waterproofing are comprised of fully adhered thermoplastic polyolefin (TPO) sheet membrane roofing over mechanically attached insulation and cover board.

Construction on the 497,000 square foot project began in September of 2006 and is projected to be completed by early 2009. Overall cost is estimated at about $253 million. The architect is Shepley Bulfinch Richardson & Abbott of Boston, and Turner Construction Company is the construction manager [see "Building Statistics Part 1" for a full list of the primary project team]. Structural design was headed by Spiegel Zamecnik & Shah of New Haven, CT. The design of the building follows the 1999 Connecticut State Building Code which adopts mostly from "The BOCA National Building Code/1996." Other codes and standards used in the design of the structure are listed below:

- ASCE 7-02: Load combinations for consideration of future vertical expansion
- ACI 318-02: "Building Code Requirements for Structural Concrete"
- ACI 315-latest edition: "Details and Detailing of Concrete Reinforcement"
- AISC LRFD Steel Manual (2nd Edition): "LRFD Specification for Structural Steel Buildings"
- AISC 341-02: Seismic Provisions for Structural Steel Buildings
- Latest Specifications of the Steel Deck Institute
- "Specification for Welded Steel Wire Fabric for Concrete Reinforcement" (Latest Edition) by the Wire Reinforcement Institute

The hospital's structure and curtain walls were designed for wind loads using the Main Wind Force Resisting System (MWFRS) method and Components and Cladding (C&C) method as prepared by RWDI, Inc. of Guelph, Ontario. As for seismic loads, the structural engineer used the Equivalent Lateral Force Procedure (ELFP).

EXISTING STRUCTURAL SYSTEM

Summary

The structural system of Smilow Cancer Center consists of a concrete slab on metal deck floor system supported on a steel framing system (moment, lateral braced, and regular gravity frames) and four reinforced concrete (RC) shear walls. On the first level, concrete beams of varying sizes run along three edges of the building. The floor slab and steel beams act in composite action with each other, while the moment frames and shear walls share the lateral load. The whole structure rests on a 4-foot thick mat slab foundation (the slab is 8 feet thick at shear wall locations). A relatively simple structure, the footprint of the building through the first five levels is almost square (210 ft. x 176 ft.). At the beginning of the seventh floor[1], however, the northeast “corner” of the building ends in a rooftop garden, and the rest of the building rises to the roof as an L-shape. Typical framing plans for levels 1 through 5 and levels 7 through 17 are shown in Figures 1a and 1b below.

Normal weight concrete is used for the shear walls and the foundation, while lightweight concrete is used for the floor slabs. Concrete strength ranges from 3000 psi to 8000 psi depending on the location and use. All reinforcement is A615 Grade 60 steel. A range of steel W-shapes are used for the framing system, but all are of the standard A992 grade steel (F_y = 50 ksi). Additionally, Hollow Structural Shapes (HSS) conform to ASTM A500 Grade B, while all other steel shapes (i.e. plates, channels, etc.) conform to ASTM A36 (F_y = 36 ksi).

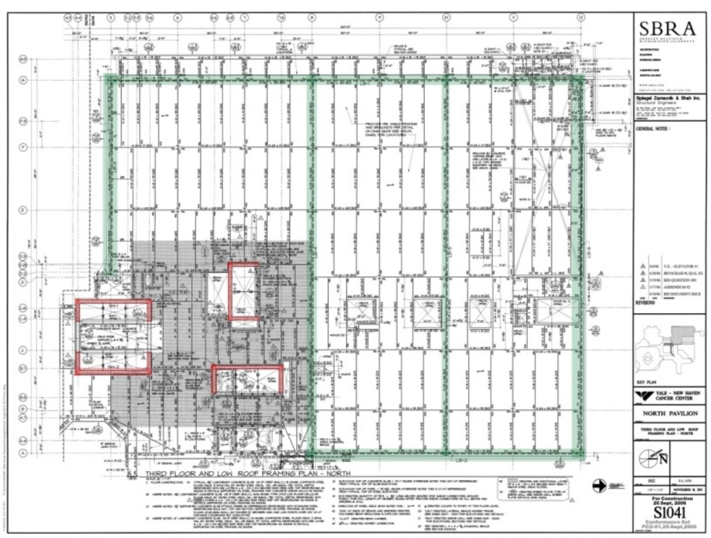


Figure 1a: Typical Framing Plan for Levels 1-5. Green denotes Moment Frames; Red denotes Shear Walls.

[1] Floors 5 & 6 are combined to house mechanical equipment. There is no floor labeled 13 for superstition reasons.

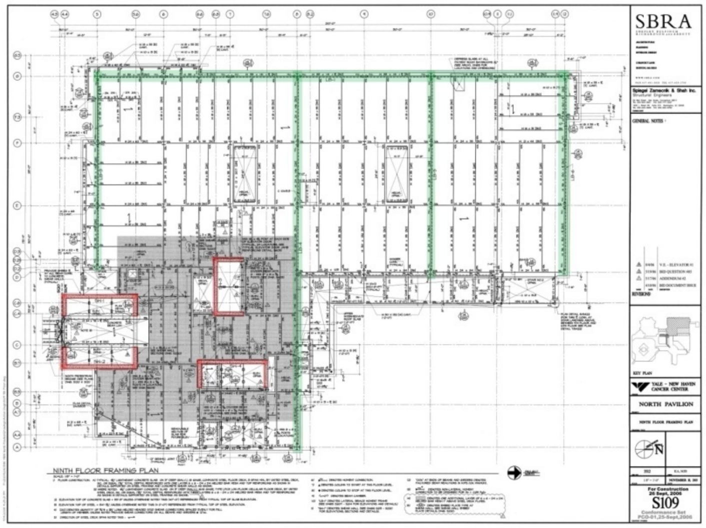


Figure 1b: Typical Framing Plan for Levels 7-17. Green denotes Moment Frames; Red denotes Shear Walls.

Foundation + Columns

As mentioned above, the foundation for Smilow Cancer Center is a 4-foot thick mat slab with different types of column base plates down at the basement level. These columns vary from W-shapes, HSS, and even cruciform columns consisting of a wide flange plus two T-shapes—all of which are encased in concrete. Some columns are regular reinforced concrete columns. Starting on the first floor, the columns continue up the structure as regular steel columns.

Floor Slabs + Beams

The typical floor slab for Smilow Cancer Center is a 4-1/2" thick lightweight concrete slab on a 3" deep, galvanized, 18 gage composite steel floor deck with a 3 span minimum. Reinforcement consists of one layer of 6 x 6 – D4 x D4 welded bar mesh and top reinforcing bars. The slab is supported on steel framing and concrete shear walls at some locations. As per ASCE 05, the floor slabs are considered as rigid diaphragms when taking into account lateral loads.

The hospital's typical bay [shown in Fig. 2] is a 30 ft. x 30 ft. square with W-shape columns at the corners, W24 girders along the perimeter, and two W18/21/24 beams spaced evenly at 10 ft. on-center. As discussed in the following section, most of the beams frame into simple gravity columns, while moment frames and shear walls are dispersed throughout the structure to effectively resist lateral loads.

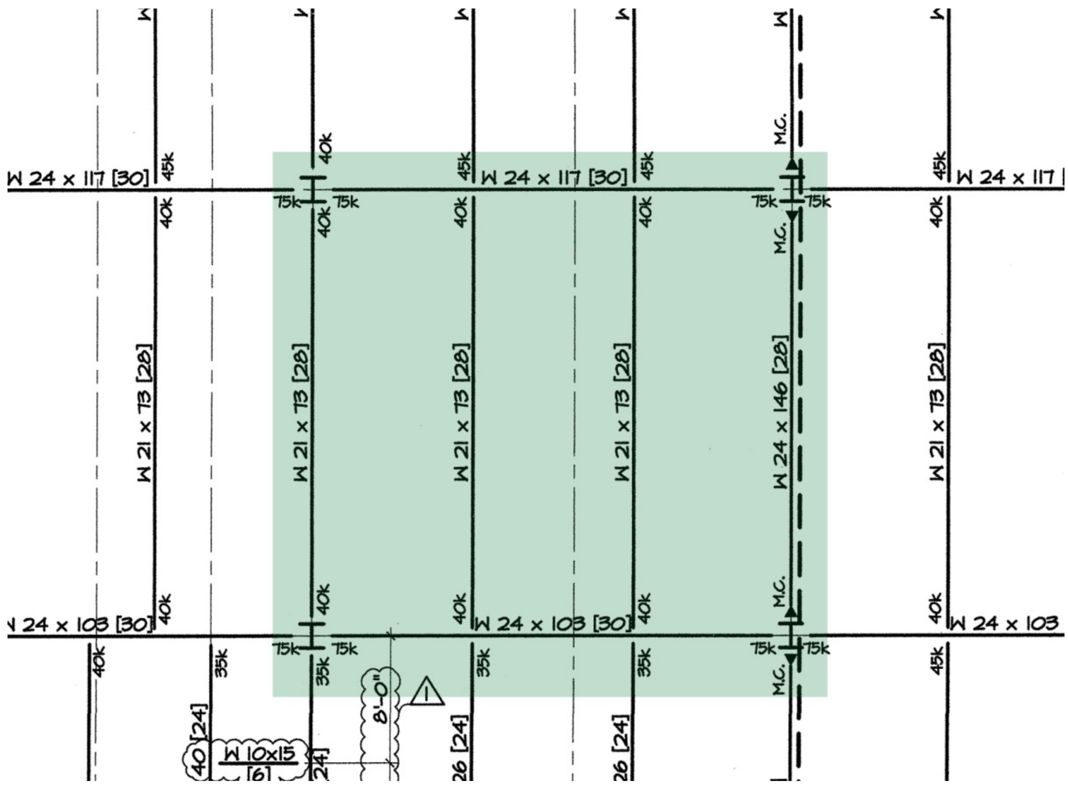


Figure 2: Typical Bay Plan – The typical 30'x30' bay is shown here highlighted in green.

Lateral Resisting System

Smilow Cancer Center's lateral resisting system is a combination of six primary moment frames, several smaller lateral braced frames on the roof, and four C-shaped RC shear walls. Four of the six main moment frames are located at the edges of the building, while the remaining two run along the east-west direction at approximately one-third points of the building's length. The four shear walls are all located towards the southeast quadrant of the building, strategically placed around central elevator and mechanical openings. All four shear walls rise up to either the sixteenth or seventeenth floor, ending where the lateral braced frames of the roof begin.

BUILDING LOADS

Gravity Loads

As part of previous Technical Report requirements, gravity loads were determined as per ASCE 7-05 and a few assumptions on the student's part. Structural drawings included in the construction documents (CD) provided some insight into code compliant loads. Table 1 below summarizes loads by type and material.

Table 1: Gravity Loads

FLOOR LOADS			
Type	**Material/Occupancy**	**Load**	**Reference**
Dead Load	Normal Weight Concrete	145 pcf	[Assumed]
	Light Weight Concrete	110 pcf	[Assumed]
	Steel	per shape	AISC 13th Edition
	Partitions	20 psf	[Assumed]
	Superimposed	10 psf	CD: S605 – S606
Live Load	Common Areas	100 psf	CD: S605 – S606
	Lobbies	100 psf	CD: S605 – S606
	Corridors (1F)	100 psf	ASCE 7-05
	Corridors (Above 1F)	80 psf	ASCE 7-05
	Operating Rooms	80 psf	CD: S605 – S606
	Exam Rooms	80 psf	CD: S605 – S606
	Mechanical	150 psf	CD: S605 – S606
	Stairs	100 psf	CD: S605 – S606
ROOF LOADS			
Dead Load	Normal Weight Concrete	145 pcf	[Assumed]
	Light Weight Concrete	110 pcf	[Assumed]
	Steel	per shape	AISC 13th Edition
	Superimposed	25 psf	CD: S605 – S606
Live Load	Roof Live Load	33 PSF	CD: S605 – S606

Lateral Loads

As per ASCE 7-08, lateral loads—specifically wind and seismic—were calculated to compare against design loads used by the structural engineer. The methods used for calculating wind and seismic loads were the Main Wind Force Resisting System (MWFRS) and the Equivalent Lateral Force Procedure (ELFP), respectively. Other references include IBC 2006 and the United States Geological Service website, www.usgs.gov. Refer to the following tables for a summary of wind and seismic loads:

Table 2a: Wind Load N-S (short face of building)

Location	Height (ft.)	K_z	q_z	p_z (psf)	P_z (kips)	Overturning Moment, M_o (ft-kips)
Windward	30	0.70	25.2	16.6	42.45	1273.59
	40	0.76	27.4	18.1	46.09	1843.68
	50	0.81	29.2	19.3	49.12	2456.22
	60	0.85	30.6	20.2	51.55	3093.01
	70	0.89	32.1	21.2	53.98	3778.33
	80	0.93	33.5	22.1	56.40	4512.16
	90	0.96	34.6	22.8	58.22	5239.93
	100	0.99	35.7	23.5	60.04	6004.08
	120	1.04	37.5	24.7	63.07	7568.78
	140	1.09	39.3	25.9	66.11	9254.78
	160	1.13	40.7	26.9	68.53	10965.03
	180	1.17	42.2	27.8	70.96	12772.32
	200	1.2	43.2	28.5	72.78	14555.35
	235	1.26	45.4	30.0	76.42	17957.67
Leeward	ALL	1.26	45.4	-16.9	-42.98	-5050.59

Table 2b: Wind Load E-W (long face of building)

Location	Height (ft.)	K_z	q_z	p_z (psf)	P_z (kips)	Overturning Moment, M_o (ft-kips)
Windward	30	0.70	25.2	16.6	53.05	1591.59
	40	0.76	27.4	18.0	57.60	2304.02
	50	0.81	29.2	19.2	61.39	3069.50
	60	0.85	30.6	20.2	64.42	3865.29
	70	0.89	32.1	21.1	67.45	4721.72
	80	0.93	33.5	22.1	70.48	5638.78
	90	0.96	34.6	22.8	72.76	6548.26
	100	0.99	35.7	23.5	75.03	7503.22
	120	1.04	37.5	24.7	78.82	9458.60
	140	1.09	39.3	25.9	82.61	11565.56
	160	1.13	40.7	26.8	85.64	13702.84
	180	1.17	42.2	27.8	88.67	15961.38
	200	1.2	43.2	28.5	90.95	18189.61
	235	1.26	45.4	29.9	95.50	22441.43
Leeward	ALL	1.26	45.4	-18.7	-59.68	-7012.95

Table 3: Seismic Loads

Floor Level	Height (ft)	Story Weight, W (kips)	Vertical Distribution Factor, C_{vx}	Lateral Seismic Force, F_x (kips)	Story Shear, V_x (kips)	Overturning Moment, M_o (ft-kips)
1	0	2157.55	0.000	0.00	0.00	0.00
2	15	2157.55	1.000	268.19	268.19	4022.81
3	30.5	2157.55	0.789	211.65	479.84	6455.27
4	45.5	2157.55	0.624	167.39	647.23	7616.24
5	60.5	2157.55	0.515	138.02	785.25	8350.35
7	80.5	1618.16	0.396	106.30	891.54	8556.75
8	95.5	1618.16	0.353	94.56	986.10	9030.63
9	110.5	1618.16	0.316	84.81	1070.92	9371.84
10	125.5	1618.16	0.286	76.72	1147.64	9628.89
11	140.5	1618.16	0.261	69.96	1217.60	9829.52
12	155.5	1618.16	0.240	64.25	1281.85	9990.56
14	170.5	1618.16	0.221	59.37	1341.22	10122.76
15	185.5	1618.16	0.206	55.17	1396.39	10233.29
16	200.5	1618.16	0.192	51.51	1447.89	10327.14
17	217.5	1618.16	0.183	48.98	1496.87	10653.26
Roof	232	30.60	0.004	1.04	1497.92	241.36

STRUCTURAL DEPTH STUDY: PROGRESSIVE COLLAPSE DESIGN

REINFORCED CONCRETE DESIGN

The ultimate goal in structural design of buildings—regardless of the material—is to design a safe, efficient, and cost-effective structure. The redesign of Smilow Cancer Hospital into a purely reinforced concrete (RC) structure is no different. While keeping the four existing shear walls in the original design, the steel frames supporting the rest of the structure were replaced with RC frames. Structural elements were designed by following the load path: starting with the floor slabs, then the beams and girders, and finally the columns.

As much as possible, the layout and configuration of the existing steel frame design were kept constant: the typical bay size of 30' x 30' is still used, and intermediate beams spaced at 10' within the bays are still used. Initially, typical member sizes and reinforcement were determined using hand calculations based on equations and guidelines from the American Concrete Institute's "318-05: Building Code Requirements for Structural Concrete and Commentary" (ACI 318-05). These trial members were then developed into a 3D computer model of the structure using the ETABS Program. This model was then analyzed for the gravity and lateral loads given in the previous section.

A concrete compressive strength (f'_c) of 4000 psi was assumed for slab, beam, and column concrete, while steel reinforcement yield strength (f_y) was assumed at 60,000 psi (Grade 60 Steel). Superimposed dead and live loads were obtained from loading diagrams in structural drawings. The critical values of 35 psf DL and 100 psf LL were assumed for the whole building. For the computer analysis of the structure, the following ASCE load combinations were considered:

$$\mathbf{1.2D + 1.6W + L + 0.5S}$$
$$\mathbf{1.2D + E + L + 0.2S}$$

where,
$D = Dead\ Load$
$L = Live\ Load$
$W = Wind\ Load$
$S = Snow\ Load$
$E = Seismic\ (Earthquake)\ Load$

Typical Member Sizes

Since this thesis focuses more on progressive collapse design, only typical member sizes were calculated. Also, hand calculations consider only gravity loads. Flexural members were designed and checked for moment capacity, live load deflection, and shear capacity. Columns were designed using design aid charts from the text, *Design of Concrete Structures*, 13th edition, by Nilson, Darwin, and Dolan (NDD). These charts consider both axial load and moments created due to eccentric loading.

Note: See pages 35-49 in Appendix for hand calculations.

ONE-WAY FLOOR SLAB DESIGN

Because of intermediate beams within the bays, the floor slab was considered to be a one-way slab with a span length of 10’ and a width of 30’. Designing a one-foot wide section of the slab as a simple beam under bending moment and shear resulted in a required thickness of 3 inches. However, ACI 318-05 gives values for “minimum thicknesses of one-way slabs unless deflections are calculated” in Table 9.5a:

$$\boldsymbol{Minimum\ Thickness = h_{min} = \frac{L}{28} = \frac{10' * 12}{28} = 4.29" \cong 5"}$$

Hence, the typical floor slab used in the new design is a 5” thick slab with #5 bars at 6” spacing for flexure.

BEAM DESIGN

The beams spanning between columns and within bays in the N-S direction have a span length of 30’ and a tributary width of 10’. Checking for flexural and shear capacities as well as live load deflection limits, the beams were sized at 12” x 25”. Note that the 25” height includes the thickness of the slab. Flexural reinforcing consists of four #9 bars, and shear reinforcing consists of #5 stirrups at 12” spacing.

GIRDER DESIGN

The girders running in the E-W direction have a span length of 30’ and are modeled to carry two point loads from the beam reactions along the girders’ third points. As with all other elements, the self-weight of the member was also considered. The standard flexural, shear, and deflection checks yielded a girder size of 16” x 30”. Note that the 30” height includes the thickness of the slab. Flexural reinforcing consists of five #10 bars, and shear reinforcing consists of #5 stirrups at 12” spacing.

INTERIOR COLUMN DESIGN

As previously mentioned, typical interior columns were sized using design aid charts from NDD. An axial load of 250 kips per story on typical interior columns was obtained from structural drawings. Also, a spreadsheet [shown below] was developed to *approximate* required gross areas for columns on different floors. A few assumptions were made to simplify the design process:

- Square column: $b = h$
- Reinforcement on four faces
- Reinforcement ratio limited to 4-5%. Assume $\rho = 3.5\%$
- Spiral reinforcement used as confinement. $\Phi = 0.70$
- Load eccentricity of $e = 0.10h$
- $\gamma = 0.80$
- Assume $\boldsymbol{K_n} = \frac{\boldsymbol{P_u}}{\boldsymbol{\Phi * f'_c * A_g}}$ controls column design.

$$\therefore \boldsymbol{A_{g,req'd}} = (\boldsymbol{P_u})/(\boldsymbol{\Phi} * \boldsymbol{f'_c} * \boldsymbol{K_n})$$

where,

$K_n = 1.08$ *(from design aid chart)*

These assumptions, the design aid charts, and the spreadsheet result in column sizes ranging from 16” x 16” on the upper levels to 40” x 40” on lower floors.

Table 4: Approximation of Required Column Gross Areas

Floor Level	# Supported Stories	Axial Load, P_u (kips)	Gross Area Required (in^2)	Square Column Dimension (in)	Col. Dimensions used (in x in)
1	15	3750	1240.08	35.21	40 x 40
2	14	3500	1157.41	34.02	40 x 40
3	13	3250	1074.74	32.78	36 x 36
4	12	3000	992.06	31.50	36 x 36
5	11	2750	909.39	30.16	30 x 30
7	10	2500	826.72	28.75	30 x 30
8	9	2250	744.05	27.28	30 x 30
9	8	2000	661.38	25.72	30 x 30
10	7	1750	578.70	24.06	24 x 24
11	6	1500	496.03	22.27	24 x 24
12	5	1250	413.36	20.33	24 x 24
14	4	1000	330.69	18.18	24 x 24
15	3	750	248.02	15.75	16 x 16
16	2	500	165.34	12.86	16 x 16
17	1	250	82.67	9.09	16 x 16

Axial Load = **250** **kips/story**

Computer Analysis of Structure

After initial sizes were determined for typical structural members, a 3D computer model of the redesigned structure was created using the ETABS computer program. The model was analyzed for both gravity and lateral loads using appropriate ASCE load combinations.

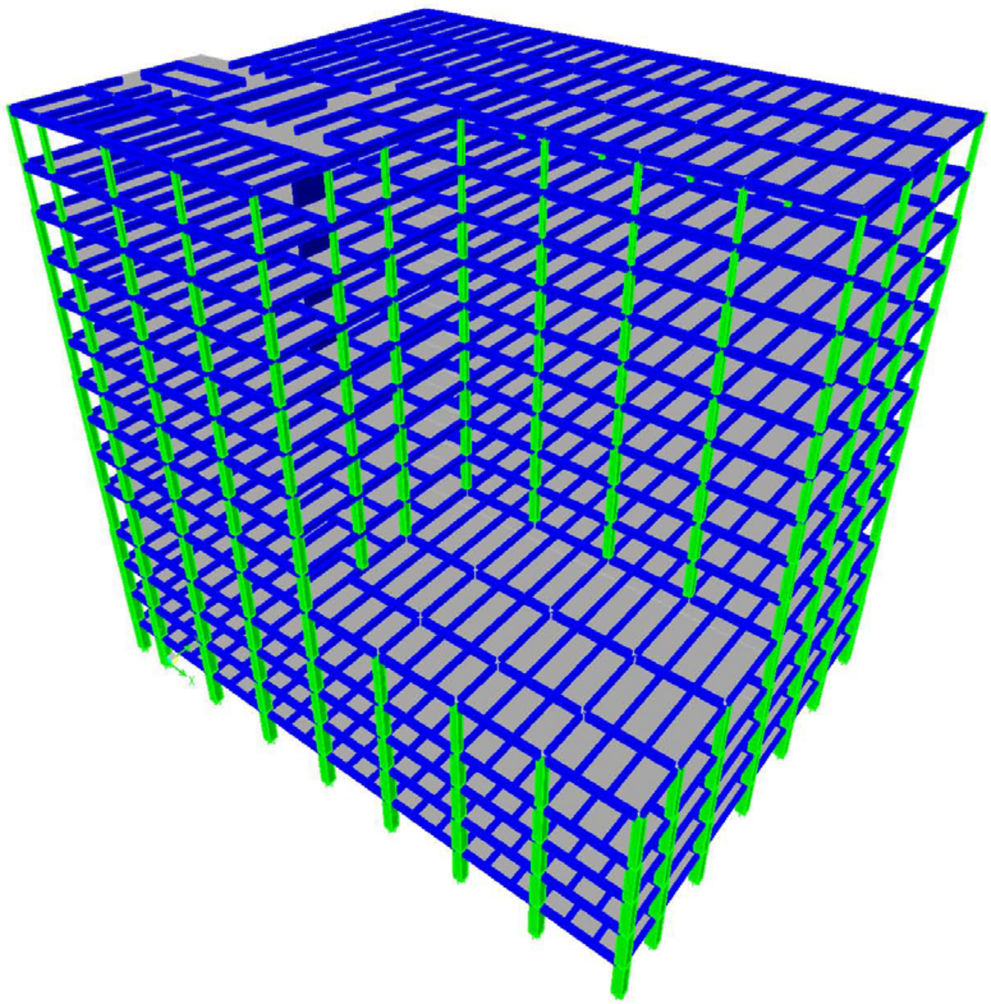


Figure 3: 3D Computer Model of New RC Structure

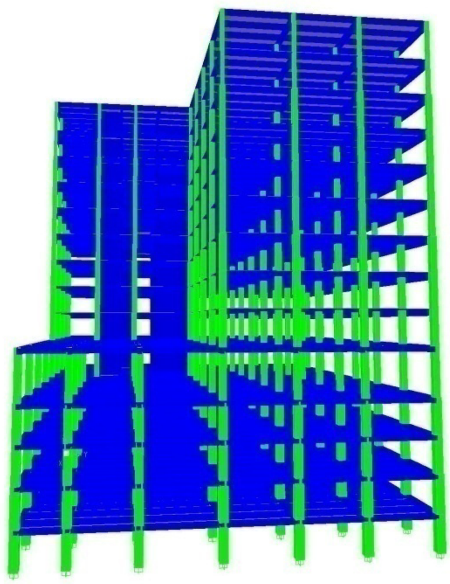


Figure 4a: View of North End of Structure

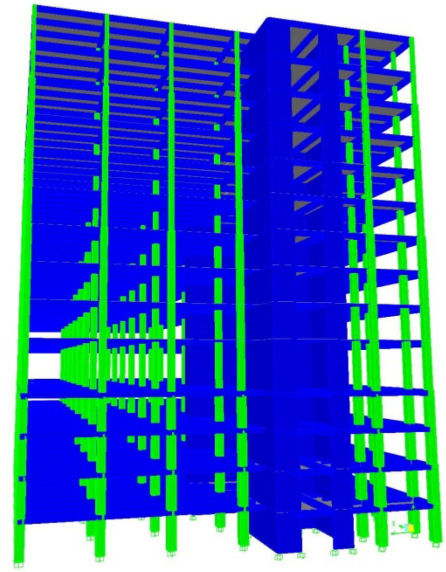


Figure 4b: View of South End of Structure

Running a check on the model shows that all of the typical member sizes are adequate for the load case of **1.2D + 1.0E + L**. The few members that did not pass were beams framing into shear walls. According to the program, "shear stress due to shear force and torsion together exceeds maximum allowed." These few members were neglected since they were not a part of the progressive collapse analysis of the structure.

BACKGROUND: Progressive Collapse

Information on progressive collapse theory and design was mostly taken from the "Unified Facilities Criteria (UFC) 4-023-03 Design of Buildings to Resist Progressive Collapse" document published by the United States Department of Defense (DoD).

In their ASCE 7-02 standard, the American Society of Civil Engineers (ASCE) defines progressive collapse as "the spread of an initial local failure from element to element, eventually resulting in the collapse of an entire structure or a disproportionately large part of it." (ASCE 7-02) In simpler terms, progressive collapse can be likened to a chain reaction in the failure of critical structural components of a building. Picture a simple three-bay, one-story frame where each member has sufficient capacity under gravity loads. But suppose a local failure occurs in one of the interior columns: the remaining members are suddenly carrying the additional load of the failed member. Without considering this effect during the design process, the remaining members would most likely fail as well, leading to a "progressive collapse" of the building.

One of the more infamous instances of this type of failure was the Ronan Point Apartment Tower collapse in England in 1968. A gas explosion near the corner of the 18^{th} floor knocked out some load-bearing precast concrete panels supporting the floors above. The loss of these few critical members resulted in the collapse of the entire corner of the building, from the 22^{nd} floor all the way to the ground. Thus, the disproportionate nature of a progressive collapse is evident in the catastrophic failure of a large part of a building caused by a relatively small event (UFC 4-023-03).

Events such as the Ronan Point collapse have sparked the development of guidelines and standard practices concerning progressive collapse design. Nowadays, the reason behind progressive collapse design is not accidental explosions, but something more unfortunate: deliberate terrorist attacks. This idea should not be a strange one, considering the involvement of the U.S. government, and more specifically the Department of Defense. The UFC guidelines were, after all, originally intended for government facilities.

UFC 4-023-03 outlines two general approaches to progressive collapse design: direct and indirect. The direct method relies on the concept of an "alternate path," where the structure is designed so that it can bridge across a removed element (usually a column). This typically means slabs and beams and girders must be designed for longer spans as vertical structural members are removed. This method is not considered to be the most efficient way of designing for progressive collapse, since members are usually oversized. The indirect method, on the other hand, relies on a "catenary" response of the structure. Catenary comes from the Latin word for "chain" and generally refers to the shape a cable—or in this case a yielding beam—will take when supported only at its two ends. In the indirect method for progressive collapse, the goal is to develop adequate tie forces within the beams and slabs so that they will still be able to support their own weight in the event of column removal. Note that the primary purpose of this method is to allow time for evacuation in the event of column failure; it is not necessarily meant to be a permanent solution.

For new and existing construction, UFC 4-023-03 classifies design requirements into four categories based on the level of protection (LOP) required: Very Low (VL), Low (L), Medium (M), and High (H). For construction requiring VLLOP to LLOP, only the indirect approach need be considered. The structure is to be analyzed/designed conventionally then checked for adequate horizontal and vertical tie forces. For MLOP and HLOP buildings, however, tie forces must be considered *as well as* an alternate path analysis. According to UFC 4-023-03, "it is expected that the majority of new and existing DoD facilities will be assigned VLLOP or LLOP ratings…"

ANALYSIS/DESIGN PROCEDURE

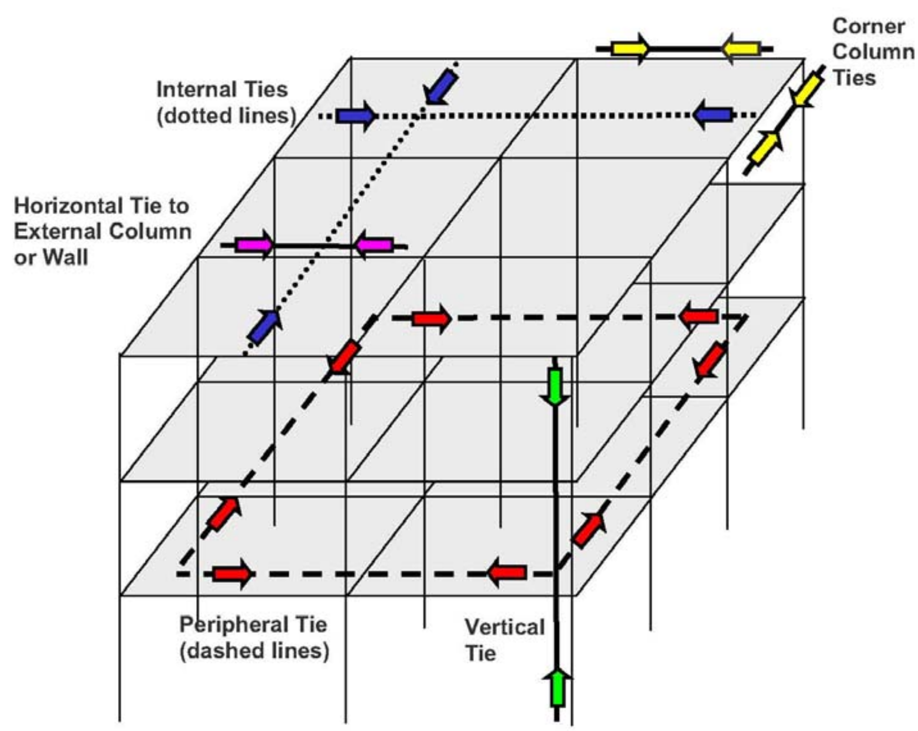
Since Smilow Cancer Center is not even a DoD facility, it is fair to assign the hospital a LLOP rating. Hence, the design to resist progressive collapse will require the application of only the

Tie Force criteria. The inherent efficiency in this design approach is that existing flexural reinforcement in the slabs, beams, and columns can be considered as ties. Note that the tie forces referred to are *not* synonymous with the reinforcement ties for conventional RC design as defined in ACI 318 codes.

But before tie forces can be considered, several blast threat scenarios must first be established. Since Smilow Cancer Hospital does not have an underground parking garage, an intentional explosive attack in the basement is not highly likely. However, an interior column failure on the first floor is considered, since a large part of the ground level is open to the public. Also, because Smilow Cancer Hospital is located in downtown New Haven, its exterior and corner columns are highly vulnerable to explosive attacks. Therefore, the second scenario considered is failure of a corner column, as in the Ronan Point Apartment Tower collapse.

Based on the layout of the new RC structure, several types of horizontal ties need to be provided: internal, peripheral, and ties to edge columns, corner columns. Vertical ties are also required in columns. Figure 5 below illustrates the location and configuration of these ties:

Figure 5: Schematic of Tie Forces in a Frame Structure (reproduced from UFC 4-023-03)



Note: The required External Column, External Wall, and Corner Column tie forces may be provided partly or wholly by the same elements that are used to meet the Peripheral or Internal tie requirement.

Required horizontal and vertical tie forces are given in Chapter 4 of UFC 4-023-03. This chapter also gives the appropriate over-strength (Ω) and strength reduction (Φ) factors. These formulas and factors are summarized below:

Over-strength Factor, Ω

An over-strength factor greater than 1.0 is justified by several assumptions/simplifications made during typical structural design. For one, the values for material strengths are usually conservative to begin with—not to mention the fact that these values are adjusted with reduction factors. Also, almost all structural members are sized larger than what the load requires. Table 5 below gives over-strength values for reinforced concrete:

Table 5: Over-Strength Factors for Reinforced Concrete

Reinforced Concrete	Over-Strength Factor, Ω
Concrete Compressive Strength	1.25
Reinforcing Steel (ultimate and yield strength)	1.25

Strength Reduction Factor, Φ

The strength reduction factor for RC tie forces is 0.75.

Basic Strength, F_t

The basic strength is to be calculated using the following calculation:

$$\boldsymbol{F_t \leq \begin{cases} 4.5 + 0.9n_o \\ 13.5 \end{cases}}$$

where,

$$n_o \equiv Number\ of\ Stories$$

Peripheral Ties

Peripheral ties, as the name implies, are to be provided within 3.9 ft of building edges or within perimeter walls. They are to be designed for the tie strength shown below:

$$\boldsymbol{R_u = 1.0 * F_t}$$

Internal Ties

Internal ties must be distributed at each floor level along both directions of the building. Existing reinforcement that could possibly act as internal ties are the flexural bars provided for the slab in the E-W direction and the beams in the N-S direction. The following equation gives the required design strength for internal ties:

$$\boldsymbol{R_u} \geq \begin{cases} \boldsymbol{\frac{D+L}{156.6}\ \frac{lr}{16.4}\ \frac{Ft}{3.3}} \\ \boldsymbol{\frac{Ft}{3.3}} \end{cases}$$

where,

$D \equiv Dead\ Load\ (psf)$
$L \equiv Live\ Load\ (psf)$
$l_r \equiv Distance\ between\ supports\ (ft)$
$F_t \equiv Basic\ Strength$

Horizontal Ties to Columns

Horizontal ties to columns are to be placed within horizontal members (slabs and beams) framing into vertical load-bearing members (columns and walls). These ties must meet the required tensile strength given by the equations below:

$$\boldsymbol{R_u} \geq \begin{cases} \boldsymbol{0.03[4(D+L)]A_t} \\ \boldsymbol{R'_u} \leq \begin{cases} \boldsymbol{2.0\ F_t} \\ \boldsymbol{\frac{l_s}{8.2}} \end{cases} \end{cases}$$

where,

$A_t \equiv Tributary\ Area\ (ft^2)$
$l_s \equiv floor\ to\ floor\ height\ (ft)$

Vertical Ties

Columns and any load-bearing walls must have continuous vertical ties from the lowest to the highest level. The ties must have a design strength in tension equal to the largest factored vertical load on the column or wall from any one story, using conventional load combinations. Or,

$$\boldsymbol{R_u = A_t * (D + L)}$$

Corner Column Ties

Corner columns must have horizontal ties into the surrounding beams/slab at each floor and roof level in each of two directions. The ties must be designed for the same tensile strength as horizontal ties to columns:

$$\boldsymbol{R_u \geq \begin{cases} 0.03[4(D+L)]A_t \\ R'_u \leq \begin{cases} 2.0\ F_t \\ \dfrac{l_s}{8.2} \end{cases} \end{cases}}$$

where,

$A_t \equiv Tributary\ Area\ (ft^2)$
$l_s \equiv floor\ to\ floor\ height\ (ft)$

Nominal Tie Capacity

The design tensile strength of all types of ties is to be calculated as,

$$\boldsymbol{R_n = \Omega * \Phi * A_s * f_y}$$

Using the above equations, the required steel reinforcement area was calculated for each type of tie. This was then compared with the area of steel provided during conventional structural design (i.e. flexural rebar in the slab and beams). The table below summarizes the results of this comparison:

Table 6: Required Tie Forces and Steel Areas

Tie Location	Tie Force (kips)	Required Steel Area (in^2)	Provided Steel Area (in^2)
Peripheral	13.5	0.24	0.31
Internal (N-S)	6.45 per foot width	0.12 per foot width	0.27 per foot width
Internal (E-W)	6.45 per foot width	0.12 per foot width	0.62 per foot width
Horizontal (N-S)	24.7	0.44	4.00
Horizontal (E-W)	24.7	0.44	6.35
Vertical	121.5	2.16	10.16
Corner Column	24.7	0.44	6.35

Note: See pages 50 through 54 in Appendix for hand calculations.

As evident from the table, the existing reinforcement in the slab, beams, girders, and columns is more than adequate to meet the required tie forces. The bigger issue is detailing this reinforcement to be continuous and properly anchored where necessary. According to Chapter 4 of UFC 4-023-03, detailing the reinforcement to meet the requirements of Chapter 21 of ACI 318-02 (Seismic Design Provisions chapter) will ensure the continuity and proper anchorage of the ties.

Tie Detailing

The following details of rebar splices and hooks were produced following the Seismic Design Provisions chapter of the 2005 edition of ACI 318:

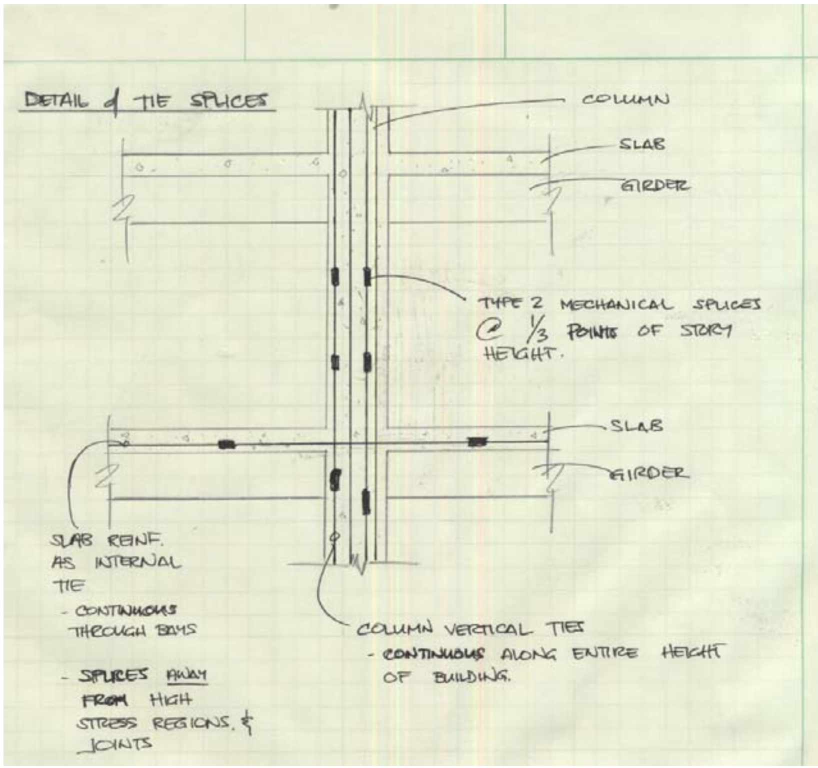


Figure 6: Detailing Requirements for Vertical and Horizontal Ties

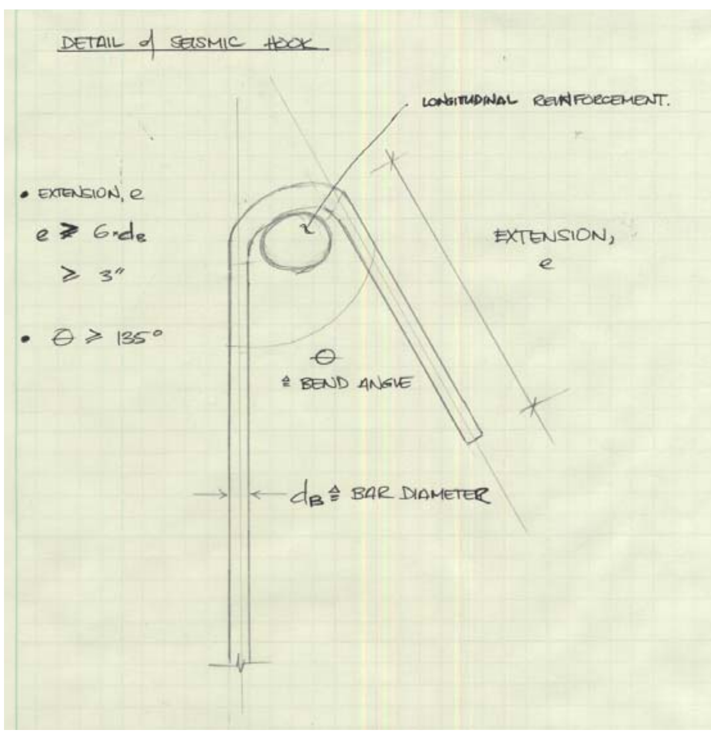


Figure 7: Detail of Seismic Hook

DEPTH STUDY CONCLUSION

Past experiences with progressive, or disproportionate, collapse of structures prove that a relatively small, localized failure of one member can very well lead to large scale damage and loss of life. Based on the results of the structural analysis and design for Smilow Cancer Hospital, it is recommended that owners, developers, and engineers of more prominent building projects consider progressive collapse in the design of their structure. Employing the indirect method of providing adequate tie forces is a very efficient way of dealing with the possibility of localized column failure. Since steel reinforcing used in traditional concrete design can also act as ties, the only added cost is the detailing of tie connections, splices, and anchorage. And if it happens that the structure is already detailed for seismic loads, then the requirements for progressive collapse design are already met. Without a doubt, the potential of preserving human life far outweighs the small cost and effort of detailing a few rebar splices.

STRUCTURAL DEPTH STUDY: BLAST-RESISTANT GLAZING

BACKGROUND: Blast-Resistant Glazing

Recently, unfortunate national and international events have forced the concept of blast-resistant design for building curtain walls to become a bigger priority for high-profile facilities. Whether it is an accidental explosion or a deliberate attack, many of the injuries and deaths result from the flying glass shards created when windows shatter from the blast pressure. As it pertains to glazing in curtain walls, the primary goal of blast-resistant design is to mitigate the formation of this flying debris by properly detailing the glazing itself, the surrounding frame, and the attachment between the two. In blast situations, glass fracture is not only acceptable but expected; it is the manner in which the glass fractures that dictates the design.

As outlined in the document “Blast-Resistant Glazing Design” by H. Scott Norville and Edward J. Conrath, there are two commonly accepted methods of designing blast-resistant glazing. The first method, which uses the Unified Facilities Criteria (UFC) published by the U.S. Department of Defense, is mostly restricted to government facilities and is not discussed in this thesis. The second method, however, is appropriate for public use and utilizes the American Society for Testing and Materials (ASTM) document, ASTM F 2248. This method simplifies the loads created during blast scenarios by equating them to 3-second design loads, which is the load duration used in designing for wind and other lateral pressures. The ASTM F 2248 has a relatively simple chart [shown in Fig. 8] that relates standoff distance and charge sizes to equivalent 3-second design loads. The designer can then take these equivalent loads and design the glazing according to the method given in another ASTM document, ASTM E 1300. Charts in the ASTM E 1300 standard give the base capacities, or non-factored loads (NFL), for standard thicknesses of annealed (AN) monolithic glass. These NFL’s are then multiplied by adjustment factors that account for other glass types (i.e. heat-strengthened (HS) and fully-tempered (FT)) and configurations (i.e. laminated glass and insulating glass units).

After determining the capacity of the glazing, the designer can then compare this value to the 3-second design load obtained from the ASTM F 2248 standard and make any necessary adjustments to the design. It should be noted that blast design for glazing and curtain walls in general is complicated and still under development. Many conservative assumptions and simplifications are made. Also, many aspects of the procedure are restricted from public knowledge due to security and political reasons.

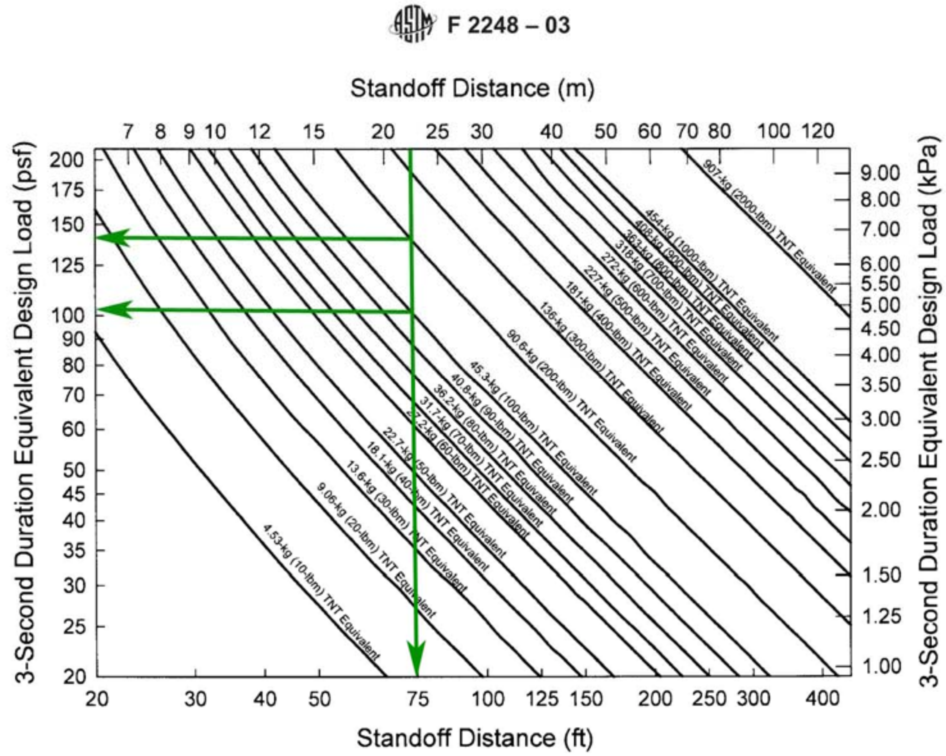


Figure 8: Chart Relating Standoff Distance and Charge Mass to 3-sec Equivalent Load

ANALYSIS/DESIGN PROCEDURE

The design of blast-resistant glazing for Smilow Cancer Hospital is limited to the first story. After all, this is where most of the glazing in the building is located, and this is also the level most susceptible to blast threats. Construction documents indicate the use of symmetrical insulating glass units (IGU) consisting of two ¼-inch lites with a ½-inch intermediate air space. Critical panel dimensions are 6'-9" x 5'-0" [see page 55 in Appendix for diagrams].

Since Smilow Cancer Hospital is within downtown New Haven, the idea of standoff distance is virtually non-existent. A large portion of its glazing faces the sidewalks of major public streets. Nonetheless, a minimum standoff distance of 75 ft. is assumed for calculations. This assumption seems reasonable in the sense that such a high-profile facility would most likely have some sort of security detail. In terms of the blast magnitudes, calculations assume charge sizes of 100 lb and 200 lb. This is a fairly reasonable assumption since "most intentional blasts in the United States are relatively small … generally on the order of 10 lb or less" (Norville & Conrath). In contrast, *very* infrequent larger blasts such as the Oklahoma City bombing are on the order of 1000 lb or more. These types of blasts usually result in significant damage, injuries, and deaths.

With fairly reasonable assumptions for standoff distance and charge sizes, equivalent 3-second design loads of **100 psf** and **140 psf** are determined using the chart from ASTM F 2248. To put things in perspective, typical design wind loads range from 20 psf to 30 psf.

Using the charts and modifiers found in the ASTM E 1300 standard, the following parameters were determined:

Non-Factored Load, NFL (¼-inch glass) = 35.5 psf

Glass Type Factor, GTF (HS Glass) = 1.8

Load Share Factor, LSF (2 identical lites) = 2.0

Load Resistance = Non-Factored Load x Adjustment Factors

$$\boldsymbol{LR = NFL * GTF * LSF = 35.5\, psf * 1.8 * 2.0}$$
$$\boldsymbol{LR = 128\, psf} > 100\, psf$$
$$\boldsymbol{LR = 128\, psf} < 140\, psf$$

∴ Existing design is adequate for 100-lb charge but not for 200-lb charge.

To ensure that the building's glazing would be adequate for blast pressures from a 200-lb charge, the type of glass used was changed from heat-strengthened (HS) to fully-tempered (FT). The only change in the calculation of load resistance is a different Glass Type Factor of **3.6**. The following calculation shows the new load resistance value:

$$\boldsymbol{LR = NFL * GTF * LSF = 35.5\, psf * 3.6 * 2.0}$$
$$\boldsymbol{LR = 256\, psf > 140\, psf}$$

∴ An IGU consisting of (2) ¼-inch FT lites is acceptable for the calculated blast loads. Furthermore, the calculated Load Resistance of 256 psf is beyond the range of the ASTM F 2248 chart, so allowable blast loads for this configuration is indefinite.

Note: Also see pages 55 – 58 in the Appendix for hand calculations.

DEPTH STUDY CONCLUSION

With the threat of terrorist attacks becoming more and more a reality, the idea of designing building glazing for blast loads has become more viable and warranted. Again, as with progressive collapse design, there is no way to put a price on human life. It is simply a judgment call on the design team's part. The risk and probability of an explosive attack must be assessed as accurately as possible, and the need for blast-resistant glazing must be determined accordingly. The design of blast-resistant glazing for Smilow Cancer Hospital shows that one modification in the type of glass used can drastically change the blast resistance of a glazing panel system.

BREADTH STUDY I: GLAZING THERMAL PERFORMANCE

BACKGROUND: Heat Flow Through Glazing

Ever since the emergence of curtain walls in the early 1900's, glazing has become a more dominant part of the modern building façade. The use of structural steel and reinforced concrete frames has allowed architects to freely design non-structural exterior walls. And as it happened, many architects were keen on the idea of using glass as a major part of the building's façade. As evident in many major cities in the United States, skyscrapers with their slick glass façades dominate the skyline. In fact, most modern building envelope systems are at least 50% glazing.

Aside from the aesthetic issues surrounding mostly-glass façades, the thermal performance of glazing has become a significant subject of study and research in the design community. After all, glazing is responsible for 2 quadrillion BTUs of heating and cooling energy in commercial buildings. This represents over 12% of all energy use in commercial buildings (Carmody et al., 2004). So, it is fair to state that designing glazing systems for efficient thermal performance is a worthwhile task.

In typical curtain wall heat flow design, the three modes of heat transfer are considered: conduction, convection, and radiation. Conduction is the flow of heat through a material or between materials via direct molecular contact. A fitting example is touching cold steel or a hot potato. Convection is the transfer of heat by the movement or flow of liquid or gas (fluid) molecules. One example would be blowing on a hot cup of tea to cool it down. Finally, radiation is heat transfer via electromagnetic waves through a gas or vacuum. The simplest example of radiant heat transfer is warm sunshine.

ANALYSIS/DESIGN PROCEDURE

As with blast resistance design, only the first floor glazing of Smilow Cancer Hospital was considered for its thermal performance. Since the first floor houses most of the public spaces in the building, it is the only level that uses mostly glass for its façade. On the other hand, the upper floors are mostly operating and patient rooms which require more privacy. Hence the use of the terra cotta and glass curtain wall panels on these floors. The typical glazing panel consists of symmetrical insulating glass units (IGU) made of two ¼-inch lites with a ½-inch intermediate air space. Assuming the glazing runs along the entire perimeter of the first floor with a total height of 10'-9", a total glass area of about 7800 sq ft was determined.

Typical summer and winter conditions for the Connecticut area were determined using the Heat, Air & Moisture (HAM) Toolbox, a computer program used to determine heat flows through different wall systems. The program gives average temperatures of 86 °F and 7 °F for summer and winter, respectively. Design indoor temperatures were given as 75 °F and 70 °F for summer and winter, respectively.

Since convective heat flow in IGU's is virtually non-existent (movement of air within the unit is negligible), only conductive and radiant heat flow were considered in the thermal performance analysis and design:

Conduction

Conductive heat flow through any assembly of materials depends on three basic parameters: the area of the assembly, the temperature difference between the two sides of the assembly, and the overall coefficient of heat transmission, or the U-value. The following equation relates these parameters to conductive heat flow through the assembly:

$$\boldsymbol{Q_c = A * U * \Delta T \equiv Conductive\ Heat\ Flow}$$

$$\begin{aligned} &where, \\ &A \equiv surface\ area\ of\ enclosure \\ &U \equiv coefficient\ of\ heat\ transmission \\ &\Delta T = T_{outside} - T_{inside} \end{aligned}$$

The U-value of a wall/window assembly depends on the conductivity, k, of each material. These conductivities are usually tabulated in reference sources for common building materials and even for air spaces. The following table gives the values of the conductivity, conductance, and thermal resistance for each material and for the whole assembly:

Table 7: Conductive Properties of Existing Glazing

Layer	Conductivity, k (W/m*K)	Thickness (m)	Thickness (in)	Conductance, C (W/m^2*K)		Resistance, R (m^2*K/W)	
				Summer	Winter	Summer	Winter
Exterior Air Film	N/A	N/A		23.00	34.00	0.0435	0.0294
Glass Lite 1	0.96	0.0064	0.25	151.18		0.0066	
Air Space	N/A	0.0127	0.5	7.14	5.00	0.1401	0.2000
Glass Lite 2	0.96	0.0064	0.25	151.18		0.0066	
Interior Air Film	N/A	N/A		8.30		0.1205	

	Summer	Winter
$\sum R_{SI}$	0.32	0.36
$\sum R$ (hr*ft^2*°F/BTU)	**1.80**	**2.06**

	Summer	Winter
U (BTU/hr*ft2*°F)	**0.56**	**0.49**

Relevant Equations/Conversions:

- $\boldsymbol{C = \frac{k}{thickness}}$
- $\boldsymbol{R = \frac{1}{C}}$
- $\boldsymbol{R = 5.678 * R_{SI}}$
- $\boldsymbol{U = \frac{1}{\sum R}}$

$\therefore \boldsymbol{Q_C \cong 48,000\ \frac{BTU}{hr}}$ [Summer]

$\boldsymbol{Q_C \cong 241,000\ \frac{BTU}{hr}}$ [Winter]

Radiation

For building-related heat flow calculations, radiation theory can be greatly simplified as derived from the Stefan-Boltzmann equation:

$$\boldsymbol{Q_R = A * F_E * F_A * \sigma * (T_s^4 - T_a^4) \equiv Radiant\ Heat\ Flow}$$

where,

$A \equiv surface\ area\ of\ enclosure = 7800\ ft^2 = 725\ m^2$
$F_E \equiv emissivity\ factor = 0.92$
$F_A \equiv angle\ factor\ (assume\ as\ 1.0)$
$T_S \equiv surface\ temperature\ [K]$
$T_A \equiv ambient\ temperature\ [K]$
$\sigma \equiv Stefan - Boltzmann\ Constant = 5.67 * 10^{-8} \frac{W}{m^2 * K^4}$

For simplicity, SI units were used in the calculations to find radiant heat flow in Watts (Joules per second) and then converted to BTU's. The temperatures of the outside and inside surfaces of the glazing panel were determined using the principle of temperature gradients through different materials. A standard value for the emissivity of normal, unfrosted glass was determined to be 0.92 from various manufacturers' catalogs. These values were then used to calculate the following results:

$$\therefore \boldsymbol{Q_R = 9222\ W \cong 31,500\ \frac{BTU}{hr}} \qquad \text{[Summer]}$$

$$\boldsymbol{Q_R = 1793\ W \cong 6,100\ \frac{BTU}{hr}} \qquad \text{[Winter]}$$

Total Heat Flow

$$\boldsymbol{Q_T = Q_C + Q_R}$$

$$\boldsymbol{Q_{T,Summer} = 48,000 + 31,500 \cong 79,500\ \frac{BTU}{hr}}$$

$$\boldsymbol{Q_{T,Winter} = 241,000 + 6,100 \cong 247,000\ \frac{BTU}{hr}}$$

Approximate Heating & Cooling Costs

To put things into perspective, the cost to replace/remove the heat lost/gained through the first floor glazing was calculated using average retail prices of electricity in Connecticut as reported on the Energy Information Administration's (EIA) website:

$$Cooling\ Cost = \frac{\$0.1593}{kWh} * \frac{kWh}{3412\ BTU} * \frac{79{,}500\ BTU}{hr} = \$3.71\ per\ hour$$

$$\boldsymbol{Cooling\ Cost \cong \$2,700\ per\ month}$$

$$Heating\ Cost = \frac{\$0.1593}{kWh} * \frac{kWh}{3412\ BTU} * \frac{247{,}000\ BTU}{hr} = \$11.53\ per\ hour$$

$$\boldsymbol{Heating\ Cost \cong \$8,300\ per\ month}$$

Comparison of Existing & Low-Emissivity Glazing

A relatively new innovation in the design of high-performance glazing is the concept of low-emissivity (Low-E) coatings. These coatings are very thin metallic layers that are either applied during manufacturing or sprayed on after the glazing has been installed. A low-E coating, as the name suggests, reduces the emissivity of the glass, thereby reducing radiant heat transfer through the system. The coating also serves to "insulate" the glass panel, which reduces the overall coefficient of heat transmission (U-value). Replacing even just one of the glass lites with a low-E glass lite significantly reduces the heat flow through the assembly, consequently reducing heating and cooling costs of the building.

New heat flow values and costs associated with the modified low-E system are shown below:

Table 8: Conductive Properties of Low-E Glazing

<table>
<tr><th rowspan="2">Layer</th><th rowspan="2">Conductivity, k (W/m*K)</th><th rowspan="2">Thickness (m)</th><th rowspan="2">Thickness (in)</th><th colspan="2">Conductance, C (W/m²*K)</th><th colspan="2">Resistance, R (m²*K/W)</th></tr>
<tr><th>Summer</th><th>Winter</th><th>Summer</th><th>Winter</th></tr>
<tr><td>Exterior Air Film</td><td>N/A</td><td colspan="2">N/A</td><td>23.00</td><td>34.00</td><td>0.0435</td><td>0.0294</td></tr>
<tr><td>Glass Lite 1 (Low-E)</td><td>0.96</td><td>0.0064</td><td>0.25</td><td colspan="2">1.62</td><td colspan="2">0.6164</td></tr>
<tr><td>Air Space</td><td>N/A</td><td>0.0127</td><td>0.5</td><td>7.14</td><td>5.00</td><td>0.1401</td><td>0.2000</td></tr>
<tr><td>Glass Lite 2</td><td>0.96</td><td>0.0064</td><td>0.25</td><td colspan="2">151.18</td><td colspan="2">0.0066</td></tr>
<tr><td>Interior Air Film</td><td>N/A</td><td colspan="2">N/A</td><td colspan="2">8.30</td><td colspan="2">0.1205</td></tr>
</table>

$\sum R_{SI}$	0.93	0.97
$\sum R$ (hr*ft²*°F/BTU)	**5.26**	**5.52**

U (BTU/hr*ft2*°F)	**0.19**	**0.18**

Conductive Heat Flow:

$$\boldsymbol{Q_C \cong 16,300\ \frac{BTU}{hr}} \qquad \text{[Summer]}$$

$$\boldsymbol{Q_C \cong 88,500\ \frac{BTU}{hr}} \qquad \text{[Winter]}$$

Radiant Heat Flow:

$$\boldsymbol{Q_R \cong 25,300\ \frac{BTU}{hr}} \qquad \text{[Summer]}$$

$$\boldsymbol{Q_R \cong 4920\ \frac{BTU}{hr}} \qquad \text{[Winter]}$$

Total Heat Flow:

$$Q_T \cong 41,600 \frac{BTU}{hr} \qquad \text{[Summer]}$$

$$Q_T \cong 93,400 \frac{BTU}{hr} \qquad \text{[Winter]}$$

Cooling Cost Comparison:

$$Cooling\ Cost \cong \$1,400\ per\ month$$

$$Original\ Cooling\ Cost \cong \$2,700\ per\ month$$

48% decrease

Heating Cost Comparison:

$$Heating\ Cost \cong \$3,100\ per\ month$$

$$Original\ Heating\ Cost \cong \$8,300\ per\ month$$

63% decrease

Note: Also see pages 59 – 65 in the Appendix for hand calculations.

CONCLUSION

While maybe not as crucial as blast resistance, the effect of heat flow through glazing systems is an important issue that concerns both our environment and our economy. The use of high-performance glazing such as low-emissivity and insulating glass units has become almost standard practice as owners are seeing the benefits of spending a little more in the short run. More and more developers are looking ahead and considering the longer term life-cycle costs of their buildings. The cost analysis of switching from normal to low-E glazing done for Smilow Cancer Hospital shows how paying the small premium in the beginning of the project can lead to significant savings down the line.

BREADTH STUDY II: ARCHITECTURE

BACKGROUND: Unitized Curtain Wall Systems

Ever since the emergence of curtain walls in the early 1900's, glazing has become a more dominant part of the modern building façade. The use of structural steel and reinforced concrete frames has allowed architects to freely design non-structural exterior walls. And as it happened, many architects were keen on the idea of using glass as a major part of the building's façade. As evident in many major cities in the United States, skyscrapers with their slick glass façades dominate the skyline. In fact, most modern building envelope systems are at least 50% glazing.

A growing trend in the curtain wall industry is the idea of unitized curtain wall systems. These panelized systems, as the name implies, utilizes pre-assembled panels of curtain wall which can be attached to the main structure. Furthermore, these panels are available in many different materials, allowing architects and designers a variety of options for the aesthetic of their buildings. One of the main advantages of this type of system is the increased speed and ease of installation, which in turn leads to increased savings. A glaring drawback, however, is the lack of customization available with pre-assembled panels. Also, the panels themselves come at an initial cost premium compared to conventional stick built curtain wall systems.

As described earlier in the report, most of Smilow's curtain wall is of the unitized, factory-assembled type. The upper floors of the building are enclosed by unitized terra cotta and glass panels. However, the two-story glass lobby/atrium located on the building's west face was designed as a stick built aluminum and glass curtain wall system. This section of the report investigates the possibility of replacing the stick built system with a unitized glass panel system. It will determine whether it is possible and/or reasonable to keep the overall dimensions and layout of the lobby the same with the change in curtain wall system.

CURTAIN WALL DESIGN

The current design for the lobby/atrium area is shown in the figures below. The vestibule is a one-story glass "box" measuring 50'-9" by 16'-0". The height of the glass panels is 9'. The lobby is a two-story box with dimensions 80' by 50' (with only 30' as glass). Glass covers three sides of the lobby with a total height of 29'-2".

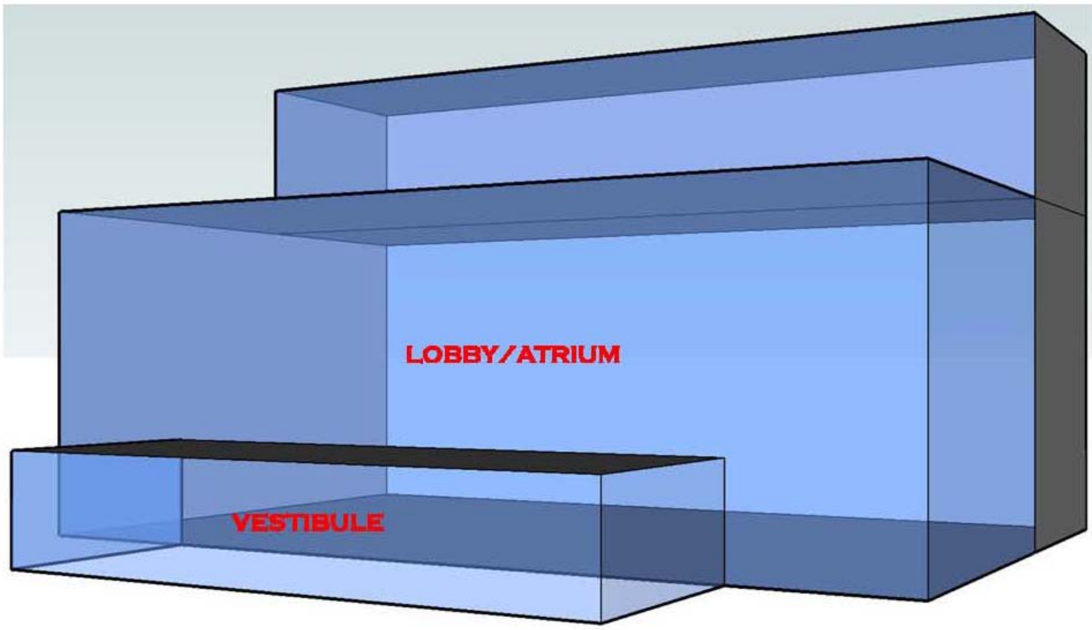


Figure 2: West View of Entry

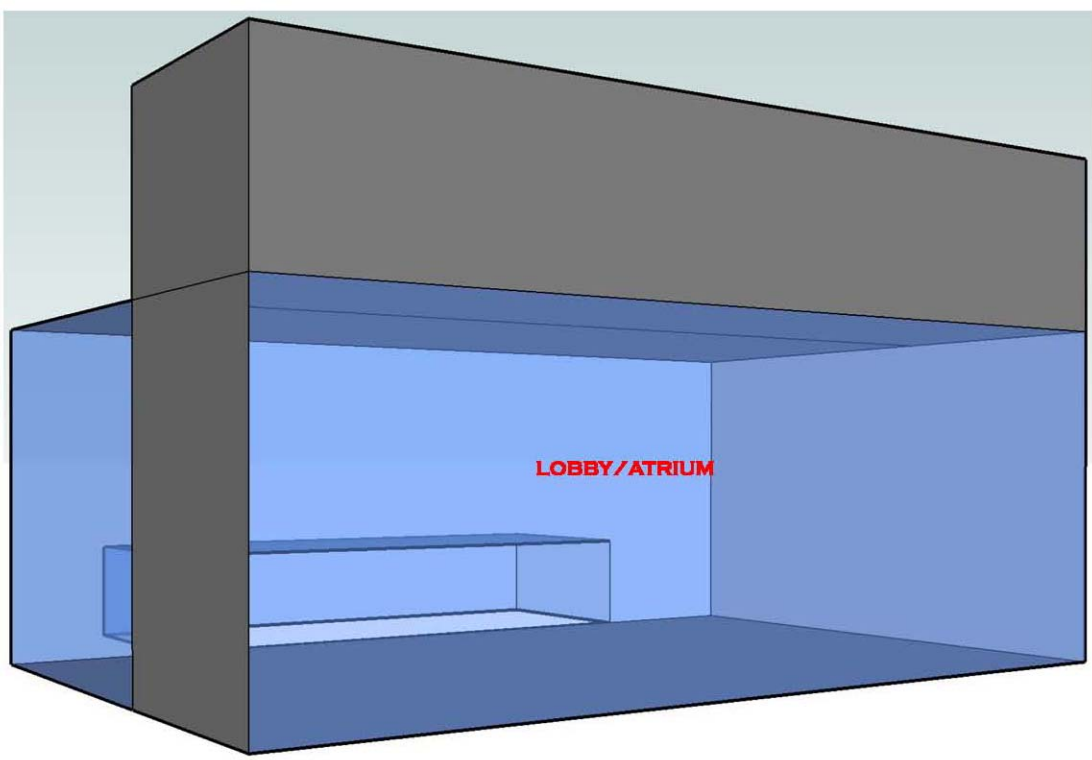


Figure 3: East View of Entry

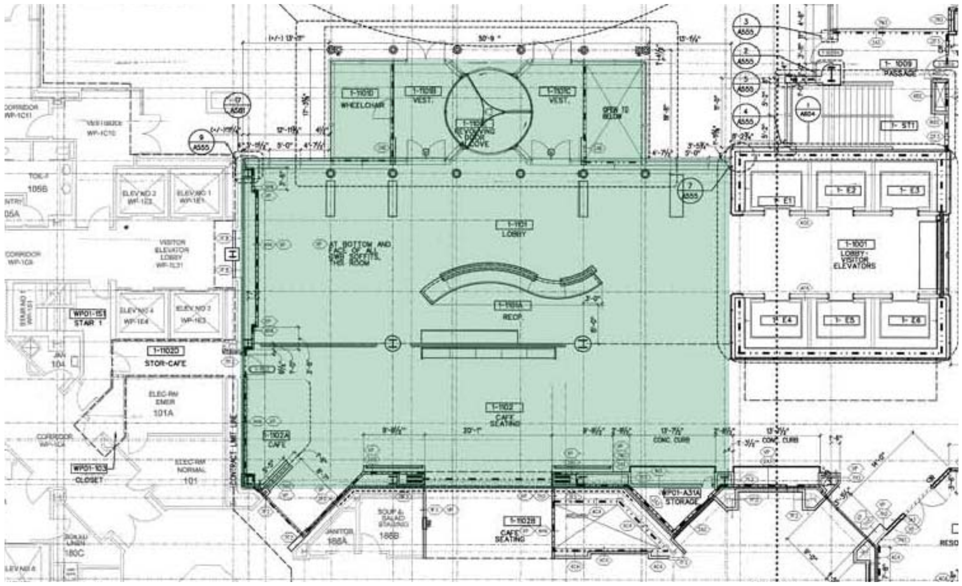


Figure 4: Plan of Entry

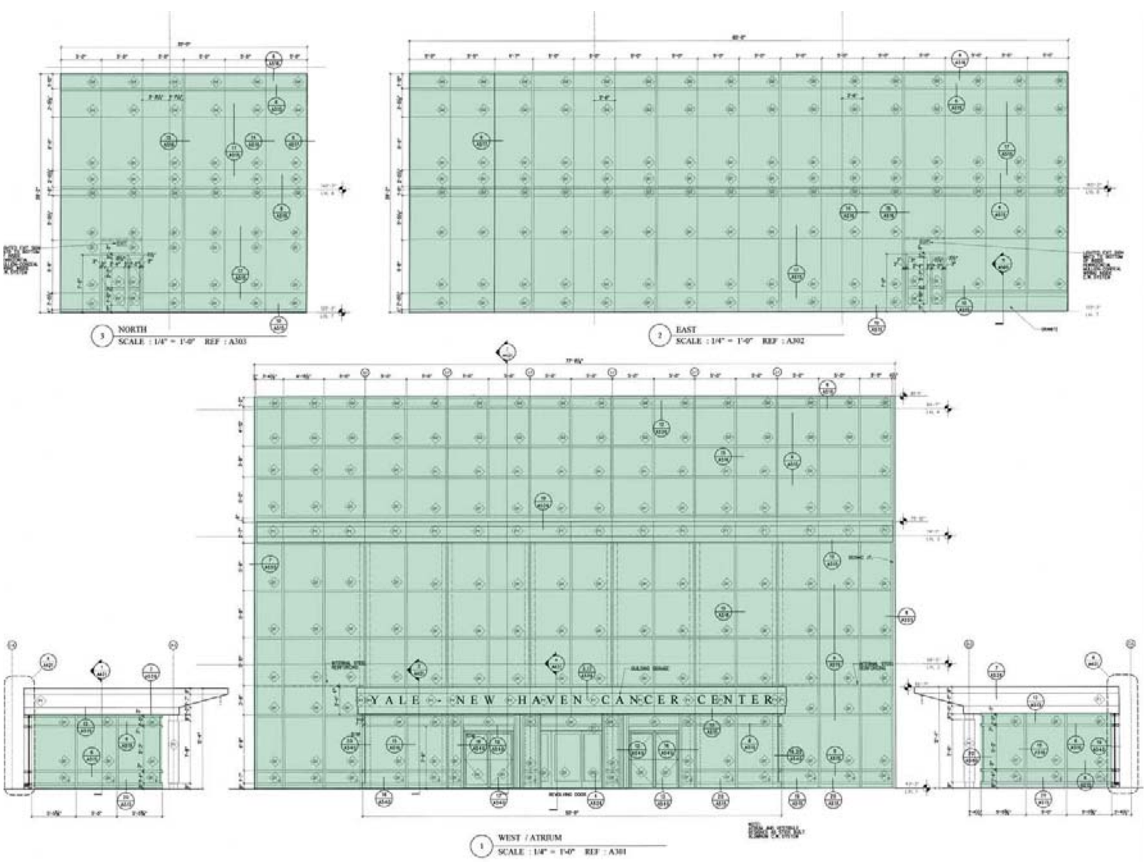
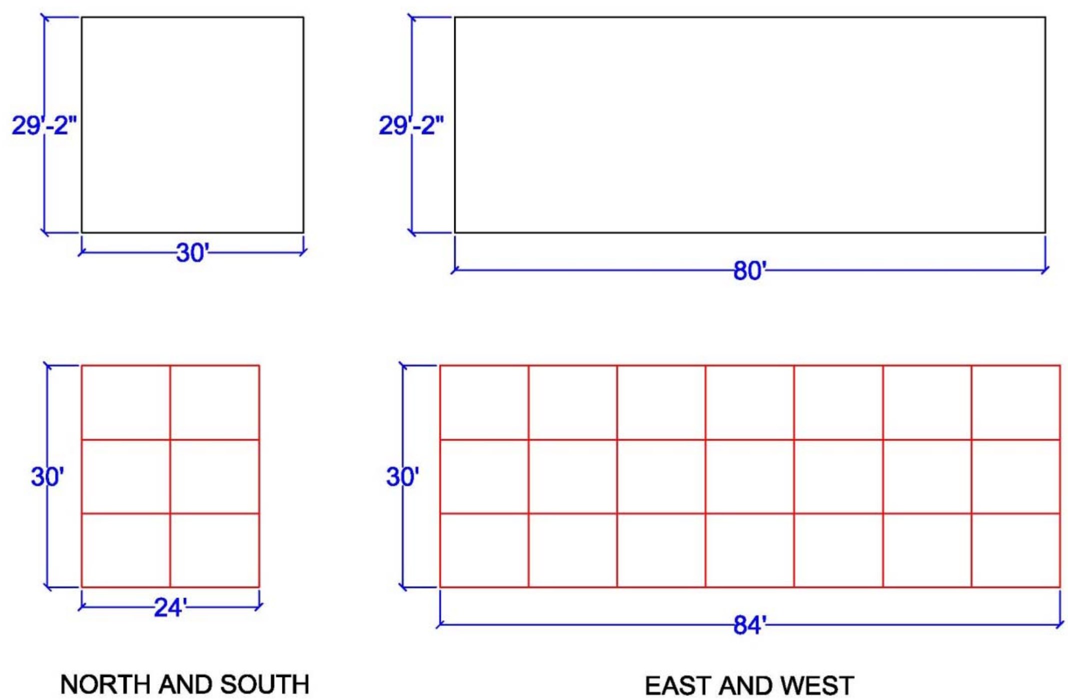


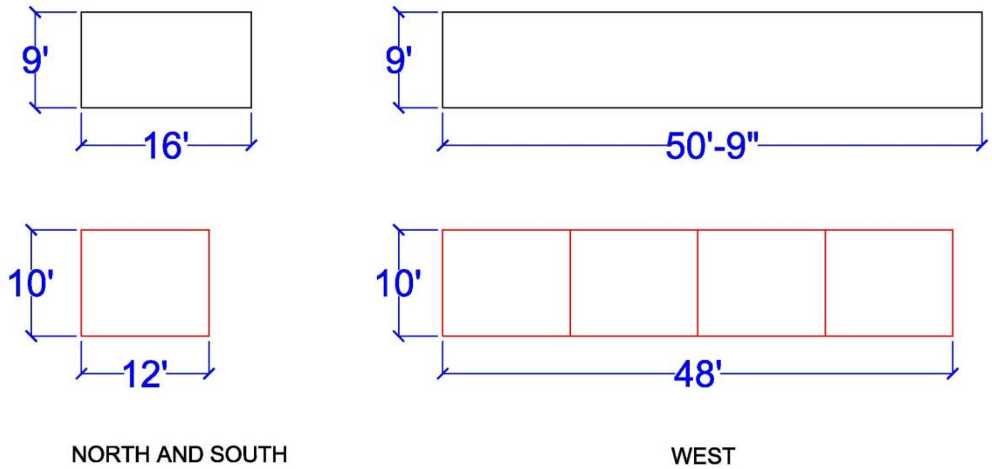
Figure 5: Elevations of Entry

The decision to use a stick built system for the vestibule and lobby areas may have been due to the dimensions of the space. Because most unitized panel manufacturers have standard panel sizes, unitized systems are better suited for modular design. Of course, the “standard” panel size varies from manufacturer to manufacturer. Most manufacturers will even produce custom sizes depending on the project, which would certainly lead to increased costs. This report uses Trainor Glass Company’s TCW-300P system, which has a standard panel size of 12’ wide by 10’ high. By only using this standard panel size, the vestibule/lobby glass curtain walls would have to change in size as shown below:

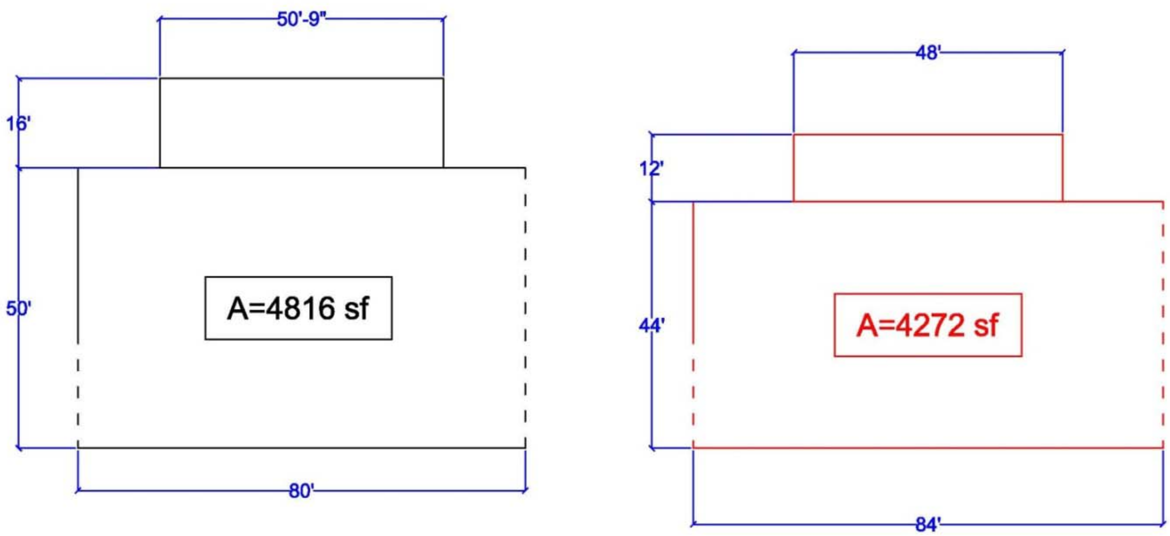
*Note: Black indicates existing design; red indicates proposed design with unitized panels. Dashed lines in plans indicate non-glass façade.



LOBBY/ATRIUM GLAZING ELEVATIONS



VESTIBULE GLAZING ELEVATIONS



EXISTING AND PROPOSED FLOOR PLANS

CONCLUSION

As shown in the diagrams, using only standard size unitized panels would lead to a decrease in usable area of the entry space. Obviously, modular panel sizes do not work well for smaller projects and spaces. This is most likely the reason that the entry space was designed with a stick built system rather than the unitized system used for the rest of the hospital. Also, the entry lobby acts much like a focal point for the Yale-New Haven Hospital Complex since it connects existing structures to the Smilow addition. Therefore, it must have a certain layout so that the flow between new and existing spaces will be a smooth one. To try and accomplish this goal with only the standard panel sizes of glazing would present too many problems. Also, for such a small part of the whole project, the differences in cost and construction time between stick built and unitized systems would probably be negligible.

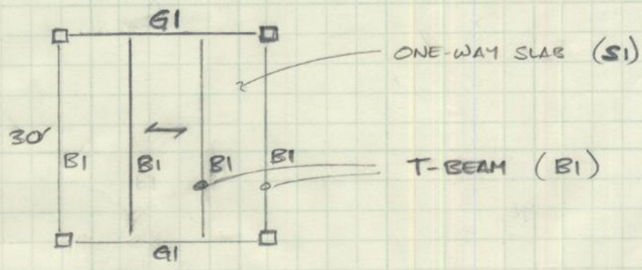
All told, utilizing a stick built system was probably the better choice for the entry lobby of Smilow Cancer Hospital. The strict requirements of the space would prevent the structure from being designed efficiently with standard size glazing panels. Nevertheless, the use of unitized curtain wall systems is a very viable alternative to stick systems given the right project conditions.

APPENDIX : STRUCTURAL SYSTEM REDESIGN

1

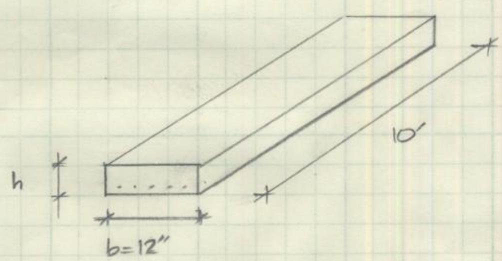
STRUCTURAL SYSTEM REDESIGN:

- TYPICAL INTERIOR BAY:



- TYPICAL LOADS: [FROM "TYPICAL LOAD DIAGRAMS," S605 - S606 OF CONSTRUCTION DOCUMENTS]
  - 35 psf DL
  - 100 psf LL
  - ADDITIONAL 10% DL FOR MEMBER SELF-WEIGHT (ASSUMED)
  - LC: 1.2D + 1.6L
- TYPICAL FLOOR SLAB DESIGN

ONE-FOOT WIDE SECTION:



$f'_c = 4000$ psi

$f_y = 60{,}000$ psi

$w_u = [1.2(35) + 1.6(100)] \times 1.10$ ↳ SELF-WEIGHT EFFECTS

$= 222.2\ lbs/ft^2 \times 1ft$

$= 222.2\ lbs/ft$

CONT'D →

2

STRUCTURAL SYSTEM REDESIGN

□ TYPICAL FLOOR SLAB DESIGN (cont'd.)

$$M_u = \frac{w_u L^2}{8} = \frac{222.2\,(10')^2}{8} = 2777.5 \text{ ft-lbs} \approx 2.8 \text{ ft-k}$$

▷ ESTIMATE $d = h - \underset{(CLR)}{3/4''}$

FROM $\phi M_n \geq M_u$,

$$M_u \leq \phi A_s f_y d \left(1 - \frac{A_s f_y}{1.7 f'_c b d}\right)$$

ASSUME $\rho = 0.0125$ & RECALL $f_y = 60{,}000$ psi, $f'_c = 4000$ psi

$\Rightarrow A_s = 0.0125\, b \cdot d$

$\Rightarrow M_u \leq \dfrac{b \cdot d^2}{20}$ WHERE M_u IS IN ft-k & b, d ARE IN in.

$$\rightarrow d \geq \sqrt{\frac{20 M_u}{b}} = \sqrt{\frac{20(2.8)}{12''}}$$

$d \geq 2.16''$

$h \geq 2.16'' + \underset{(CLR)}{0.75''}$

$h \geq 2.91'' \approx 3''$

▷ DEFLECTION CONTROL

FROM ACI 318-05 TABLE 9.5(a)

MIN. THICKNESS OF ONE-WAY SLABS UNLESS DEFLECTIONS ARE CALCULATED: (BOTH ENDS CONTINUOUS)

$$h_{min} = L/28 = \frac{10' \times 12}{28} = 4.29'' \approx 5''$$

3

STRUCTURAL SYSTEM REDESIGN:

□ TYPICAL FLOOR SLAB DESIGN (cont'd)

▷ SHEAR CONTROL

$$V_u = \frac{w_u L}{2} = \frac{222.2\,(10')}{2} = 1111 \text{ lbs}$$

$$V_c = 2\sqrt{f'_c}\; b_w \cdot d$$

$$= 2\sqrt{4000}\;(12)(h - 0.75'')$$

$$= 1518\,(h - 0.75'')$$

$$\phi V_n = 0.5\,(0.75)\left[1518\,(h - 0.75)\right]$$

$$= 569.25\,(h - 0.75)$$

FOR $\phi V_n \geq V_u$

$$569.25\,(h - 0.75) \geq 1111$$

$$h \geq 2.7'' \approx 3''$$ [NO STIRRUPS REQ'D]

∴ USE $h = 5''$; $d = 4.25''$

▷ DESIGN FLEXURAL REINF.

ASSUMING $\rho = 0.0125$,

$$A_s = \rho b d = 0.0125\,(12'')(4.25'')$$

$$A_{s\,(req.)} = 0.64 \text{ in}^2 \text{ per ft. SLAB WIDTH}$$

USE #5 ($A_s = 0.31$) @ $s = 6''$

$\Rightarrow A_{s\,(prov)} = 0.62 \text{ in}^2$ per ft. SLAB WIDTH $\approx 0.64 \text{ in}^2$

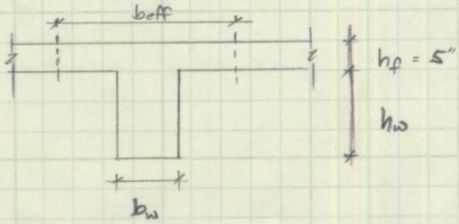
∴ FINAL SLAB DESIGN:

$h = 5''$; $d = 4.25''$

FLEX. REINF: #5 @ 6" O.C.

STRUCTURAL SYSTEM DESIGN

☐ TYPICAL T-BEAM DESIGN.



$f'_c = 4000$ psi

$f_y = 60{,}000$ psi

▸ TRY $b_w = 10"$, $h_w = 12"$

$h = 5" + 12" = 17"$

$d = 17" - 1.5" = 15.5"$
(CLR)

$b_{eff} \le b_w + 16h_f = 10" + 16(5) = 90"$
$\le b_w + \Sigma(\frac{1}{2}\text{ CLR DIST}) = 10" + 2(\frac{1}{2} \times 110") = 120"$
$\le 0.25L = 0.25(30' \times 12") = 90"$
$\therefore b_{eff} = 90"$

▸ CHECK T-BEAM BEHAVIOR :

T-BEAM BEHAVIOR OCCURS WHEN $a = h_f$

$\Rightarrow M_u \overset{?}{>} M_{u,T\text{-}BM}$

$$w_u = \left\{ \underbrace{[1.2(35) + 1.6(100)]}_{\text{SUPER-IMPOSED}} + \underbrace{\left[1.2(150\text{ pcf})\left(\frac{5}{12}\right)\right]}_{\text{SLAB WEIGHT}} \right\} \times \underbrace{10\text{ ft}}_{\text{TRIB. WIDTH}}$$

$$+ \underbrace{\left[1.2(150\text{ pcf})\left(\frac{10"}{12}\right)\left(\frac{12"}{12"}\right)\right]}_{\text{BEAM WEIGHT}}$$

$w_u = 2920\ \text{lbs/ft}$

$$M_u = \frac{w_u L^2}{8} = \frac{2920(30')^2}{8}\left(\frac{1}{1000}\right) = 328.5\ \text{ft-k} = 3942\ \text{in-k}$$

$$M_u \overset{?}{>} M_{u_{T\text{-}BM}} = \phi\, 0.85 f'_c \cdot b_{eff} \cdot h_f\,(d - h_f/2)$$

$$= 0.9(0.85)(4)(90)(5)\left(15.5 - \frac{5}{2}\right)$$

$$\underset{(3942)\ \text{in-k}}{M_u} < M_{u_{T\text{-}BM}} = 17901\ \text{in-k}$$

∴ NO T-BEAM BEHAVIOR WITH DIMENSIONS CHOSEN.
DESIGN AS RECTANGULAR BEAM.

cont'd.⟶

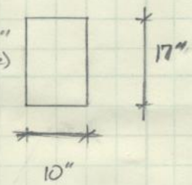
5

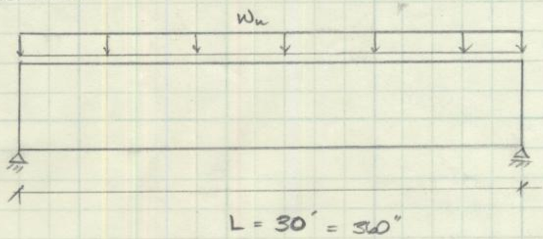
STRUCTURAL SYSTEM REDESIGN:

TYPICAL BEAM DESIGN (cont'd.)

▸ RECTANGULAR BEAM SECTION: → INCLUDES SLAB THICKNESS (5")

$d = 17" - 1.5"$ (CLR)
$= 15.5"$





$w_u = 2920$ lbs/ft
$M_u = 3942$ in-k
$= 328.5$ ft-k
} FROM PREVIOUS CALCS.

▸ CHECK NOMINAL FLEXURAL CAPACITY

· ASSUME $f_y = 60{,}000$ psi , $f'_c = 4000$ psi.

· MAX. REINF. RATIO, ρ_{max}

$$\rho_{max} = 0.85\beta_1 \frac{f'_c}{f_y}\left(\frac{\epsilon_u}{\epsilon_u + 0.004}\right)$$

$$= 0.85(0.85)\frac{4}{60}\left(\frac{0.003}{0.003+0.004}\right)$$

$$\rho_{max} = 0.0206 = 2.06\%$$

· TRY $\rho_{PROV.} = 0.0125 < \rho_{max}$

$\rightarrow A_s = \rho b d = 0.0125(10")(15.5")$

$A_s = 1.94\ in^2$

$\rightarrow$ USE (3) #8 : $A_s = 3(0.79 in^2) = 2.37 in^2$

· CHECK CAPACITY:

$$\phi M_n = \phi\left[A_s f_y\left(d - \frac{A_s f_y}{2(0.85)f'_c b}\right)\right] \geq M_u = 3942 \text{ in-k}$$

$$= 0.9\left[2.37(60)\left(15.5 - \frac{2.37(60)}{2(0.85)(4)(10)}\right)\right] \geq 3942 \text{ in-k}$$

$$= 1716 \text{ in-k} < M_u$$

∴ NO GOOD

STRUCTURAL SYSTEM REDESIGN

☐ TYPICAL BEAM DESIGN (cont'd.)

▷ INCREASE BEAM DIMENSIONS. (SAY $b = 12''$)

ASSUMING $\rho = 0.0125$ & $f_y = 60{,}000\,psi$, $f'_c = 4000\,psi$,

$\rightarrow M_u \leq \dfrac{b \cdot d^2}{20}$ WHERE M_u IS IN ft-k & b, d ARE IN in.

$\Rightarrow d \geq \sqrt{\dfrac{20 M_u}{b}}$

$\geq \sqrt{\dfrac{20(328.5)}{12}}$

$d \geq 23.4$ in ≈ 23.5 in

$h \geq 23.5\text{ in} + \underset{\text{CLR}}{1.5\text{ in}} = 25''$ (INCLUDING SLAB THICKNESS)

$\Rightarrow A_s = \rho b d$

$= 0.0125(12'')(23.4'')$

$A_s = 3.51\ in^2$

$\Rightarrow$ USE (4) #9 : $A_s = 4(1.00 in^2) = 4.00\ in^2$

▷ CHECK NOMINAL CAPACITY:

$\phi M_n = \phi\left[4.00(60)\left(23.4 - \dfrac{4.00(60)}{2(0.85)(4)(12)}\right)\right]$

$= 0.9\left[4910\ \text{in-k}\right]$

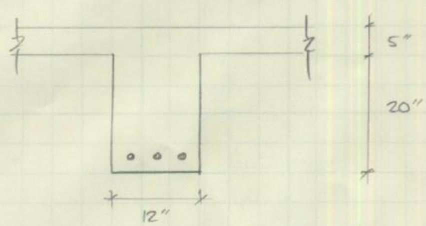
$\phi M_n = 4419^{\text{in-k}} > 4246^{\text{in-k}} = M_u$

* FIND NEW M_u DUE TO INCREASE IN SIZE:

$M_u = \dfrac{3145(30)^2}{8} = 354$ ft-k $= 4246^{\text{in-k}}$

∴ GOOD FOR FLEXURE.

▷ TENTATIVE DESIGN:



STRUCTURAL SYSTEM REDESIGN

□ TYPICAL BEAM DESIGN (cont'd.)

▷ DEFLECTION CONTROL

LIMIT LIVE LOAD DEFLECTION : $\Delta_{LL} \leq \frac{L}{360}$ [ACI 318-05]

$$\Delta_{LL} \leq \frac{30' \times 12''}{360} = 1''$$

$$\Delta_{LL} = \frac{5}{384} \frac{w_{LL} L^3}{EI} \leq 1''$$

$$w_{LL} = 100 \text{ lbs/ft}^2 \times \underset{\text{TRIB WIDTH}}{10 \text{ ft}} = 1000 \text{ lbs/ft}$$

$$I_B = \frac{bh^3}{12} = \frac{12''(25'')^3}{12} = 15625 \text{ in}^4$$

$$E_c = 57000\sqrt{f'_c} = 57000\sqrt{4000}$$

$$E_c \approx 3,605,000 \text{ psi}$$

$$\Delta_{LL} = \frac{5}{384}\left(\frac{1000(360'')^3}{3,605,000(15625)}\right)$$

$$\Delta_{LL} = 0.011'' < 1.0''$$

∴ GOOD FOR DEFLECTION

▷ SHEAR CONTROL

$$V_u = \frac{w_u L}{2} = \frac{3145 \text{ lbs/ft}(30')}{2} = 47,175 \text{ lbs}$$

$$V_c = 2\sqrt{f'_c}\, bd = 2\sqrt{4000}\,(12)(23.4) = 35,519 \text{ lbs}$$

$$\phi V_n = 0.5\phi V_c = 0.5(0.75)(35,519) = 13,320 \text{ lbs}$$

$$\underset{\text{(CONCRETE ALONE)}}{\phi V_n} = 13,320 \text{ lbs} < 47,175 \text{ lbs} = V_u$$

∴ SHEAR REINF. REQ'D.

8

STRUCTURAL SYSTEM REDESIGN

□ TYPICAL BEAM DESIGN (cont'd.)

▷ DESIGN SHEAR REINF.

$$V_{s_{REQ'D}} = \frac{V_u}{\phi} - V_c \leq 8\sqrt{f'_c}\, b_w \cdot d$$

$$= \frac{47{,}175}{0.75} - 35{,}519 \leq 8\sqrt{4000}\,(12'')(23.4'')$$

$$V_{s_{REQ'D}} = 27{,}381 \text{ lbs} \leq 142{,}075 \text{ lbs} \quad \therefore OK$$

• MAX. STIRRUP SPACING:

$$V_s = 27{,}381 \text{ lbs} \leq 4\sqrt{f'_c}\, b_w d = 71{,}038 \text{ lbs}$$

$$\rightarrow S_{max} \leq d/2 = 11.7'' \quad \text{* GOVERNS}$$
$$\leq 24''$$

• DETERMINE STIRRUP SPACING (USE $A_v = 2(0.31\text{ in}^2)$ ⊔ #5)

$$S = A_v f_{yt} \frac{d}{V_s} < S_{max}$$

$$= 0.62(60{,}000)\frac{(23.4)}{(27{,}381)} \leq 11.7''$$

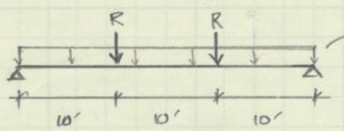
$$S = 31.8'' > S_{max}$$

⇒ USE $S = 12''$ (CLOSE ENOUGH)

∴ SHEAR REINF: ⊔ #5 @ 12" O.C.

STRUCTURAL SYSTEM REDESIGN

☐ TYPICAL GIRDER DESIGN



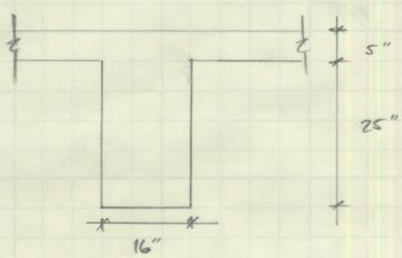
$f'_c = 4000$ psi

$f_y = 60{,}000$ psi

R ≙ REACTION FROM BEAMS

R = 43,800 lbs
= 43.8 kips

▷ TRIAL DIMENSIONS, $b = 16"$, $h = 30"$ (INCLUDES SLAB THICKNESS)
$d = 28.5"$



▷ DESIGN MOMENT

$$M_u = 43.8^k(10') + \overbrace{1.2\left[0.150\ {}^k/_{ft^3}\left(\tfrac{16}{12}\right)\left(\tfrac{25}{12}\right)\right]\left(\frac{30'^2}{8}\right)}^{\frac{w_u L^2}{8}}$$

(FROM BEAM REACTIONS) (GIRDER SELF-WEIGHT)

$M_u = 438$ ft-k + 52.3 ft-k

$M_u = 490.3$ ft-k = 5884 in-k

* CHECK $M_u \leq \dfrac{bd^2}{20}$ (ASSUME $\rho = 0.0125$)

$$d \geq \sqrt{\frac{20(490.3)}{16}}$$

$d \geq 24.8$ in

28.5 in > 24.8 in ✓

STRUCTURAL SYSTEM REDESIGN

II TYPICAL GIRDER DESIGN (cont'd.)

▸ CHECK FLEXURAL CAPACITY

- ASSUMING $\rho = 0.0125$,

$$A_s = \rho bd = 0.0125(16'')(28.5'') = 5.7 \text{ in}^2$$

$$\rightarrow \text{TRY } (5)\#10 \; ; \; A_s = 5(1.27 \text{ in}^2) = 6.35 \text{ in}^2$$

- CHECK CAPACITY:

$$\phi M_n = 0.9\left[6.35(60)\left(28.5 - \frac{6.35(60)}{2(0.85)(4)(16)}\right)\right] \geq M_u = 5884^{\text{ in-k}}$$

$$= 8572^{\text{ in-k}} > 5884^{\text{ in-k}}$$

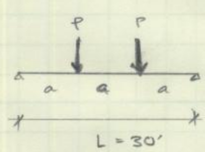
∴ GOOD FOR FLEXURE

▸ DEFLECTION CONTROL

- LIMIT LIVE LOAD DEFLECTION:

$$\Delta_u \leq \frac{L}{360} = \frac{30' \times 12}{360} = 1''$$

FOR LOAD CONFIG. SHOWN,



$$\Delta_{max} = \frac{PL^3}{28EI}$$

FIND P_{LL} FROM BEAMS:

$$P_{LL} = 1.6(100 \text{ lbs/ft}^2) \times 10' \times (15' + 15')$$

$$= 48{,}000 \text{ lbs}$$

$$E_c = 3{,}605{,}000 \text{ psi}$$

$$I_G = \frac{16''(30'')^3}{12} = 36000 \text{ in}^4$$

$$\Delta_{LL} = \frac{48000(30 \times 12)^3}{28(3{,}605{,}000)(36000)} = 0.62 \text{ in} < 1 \text{ in}$$

∴ GOOD FOR DEFLECTION.

11

STRUCTURAL SYSTEM REDESIGN

- TYPICAL GIRDER DESIGN (cont'd.)

▸ SHEAR CONTROL

$$V_u = 94{,}350 \text{ lbs} + \frac{500 \text{ lbs/ft}(30')}{2} = \underline{101{,}850 \text{ lbs}}$$

$\underbrace{(\text{P FROM BEAMS})}_{3145 \text{ lbs/ft}(30\text{ft})}$ $\quad (\tfrac{1}{2}$ SELF-WEIGHT$)$

$$V_c = 2\sqrt{f'_c}\,bd = 2\sqrt{4000}\,(16'')(28.5'') = \underline{57{,}680 \text{ lbs}}$$

$$\phi V_n = 0.5\phi V_c = 0.5(0.75)(57{,}680 \text{ lbs}) = \underline{21{,}630 \text{ lbs}}$$

$\phi V_n < V_u \quad \therefore$ SHEAR REINF. REQ'D.

▸ DESIGN SHEAR REINF.

$$V_{s_{req'd}} = \frac{V_u}{\phi} - V_c \leq 8\sqrt{f'_c}\,bd$$

$$= \frac{101{,}850}{0.75} - 57{,}680 \leq 8\sqrt{4000}\,(16)(28.5)$$

$$= 78{,}120 \text{ lbs} \leq 230{,}720 \text{ lbs} \quad \therefore \text{OK}$$

- MAX. STIRRUP SPACING:

$$V_s = 78{,}120 \text{ lbs} < 4\sqrt{f'_c}\,bd = 115{,}360 \text{ lbs}$$

$\Rightarrow S_{max} \leq d/2 = 14.3''$ GOVERNS

$\leq 24''$

- DETERMINE STIRRUP SPACING (USE $A_v = 2(0.31 \text{ in}^2)$ ⊔ #5)

$$s = A_v f_{yt} \frac{d}{V_s} \leq S_{max} = 14.3''$$

$$= 0.62(60{,}000)\left(\frac{28.5}{78{,}120}\right)$$

$$= 13.6 \text{ in} < S_{max} \quad ✓$$

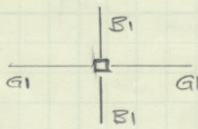
$\therefore \rightarrow$ USE $S = 12''$

$\therefore$ SHEAR REINF. ⊔ #5 @ 12" o.c.

12

STRUCTURAL SYSTEM REDESIGN

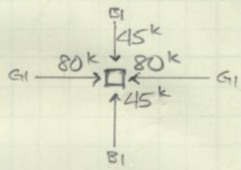
☐ TYPICAL INTERIOR COLUMN DESIGN:



* INITIALLY DESIGN FOR GRAVITY LOADS ONLY, THEN CHECK FOR LATERAL STIFFNESS REQUIREMENTS (COMPUTER ANALYSIS).

▸ TYPICAL COLUMN LOADS:

TAKEN FROM PAGES S102.1 – S117 OF STRUCTURAL DRAWINGS.

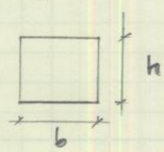


$\rightarrow P_u = 80^k + 80^k + 45^k + 45^k$

$P_u = 250^k$

▸ BASIC COLUMN DIMENSIONS / PARAMETERS:

TRY SQUARE COLUMN:



$b = h$

PRACTICAL REINF. RATIO LIMIT: (REINF. ON FOUR FACES)

$\rho \leq 4\text{-}5\% \longrightarrow$ USE $\rho_g = 3.5\% = 0.035$

ASSUME $f'_c = 4\,ksi$ & $f_y = 60\,ksi$ & SPIRAL REINF. USED ($\phi = 0.70$)

▸ TRIAL DIMENSIONS:

$b = h = 16''$ * ASSUME LOAD ECCENTRICITY OF $e = 0.10h = 1.6''$

$d' = \underset{(clr)}{1.5''} + \tfrac{1}{2}\underset{(\#8\,d_b)}{(1.0'')} = 2''$

$* \gamma = \frac{h - 2d'}{h} = \frac{16'' - 2(2'')}{16''} = 0.75$

$* e/h = \frac{1.6}{16} = 0.10$

$\rightarrow$ FROM COLUMN DESIGN AID CHARTS:

$K_n = 1.06 = \frac{P_u}{\phi f'_c A_g} \Rightarrow A_g = \frac{250}{0.70(4)(1.06)} = 84\,in^2$

$R_n = 0.11 = \frac{P_u e}{\phi f'_c A_g h} \Rightarrow A_g = \frac{250(1.6)}{0.70(4)(16)(0.11)} = 81.2\,in^2$

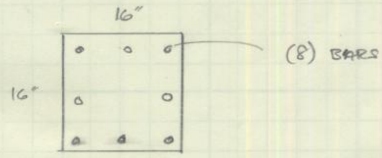
STRUCTURAL SYSTEM REDESIGN

☐ TYPICAL INTERIOR COLUMN DESIGN (cont'd.)

▸ TRIAL DIMENSIONS ARE SUFFICIENT.

$A_{g\,PROV.} = 256\ in^2 > 81.2\ in^2 = A_{g\,req'd}$

$A_{STEEL\ REQ'D} = \rho bh = 0.035(16 \times 16) = 8.96\ in^2$



USE (8) #10 BARS: $A_{STEEL} = 8(1.27) = 10.16\ in^2 > 8.96\ in^2$

▸ CHECK COLUMNS ON LOWER LEVELS.

$P_u = 16\ \text{STORIES}\ (250^k/\text{STORY})$
$= 4000k$

• ASSUME 24" × 24" COLUMN:

$d' = \underset{(CLR)}{1.5"} + \underset{(\#8)}{\tfrac{1}{2}(1.0")} = 2"$

$\gamma = \frac{h-2d'}{h} = \frac{24-4}{24} = 0.83$

$e/h = \frac{2.4}{24} = 0.10$

→ FROM COLUMN DESIGN AID CHARTS:

$K_n = 1.08 = \frac{P_u}{\phi f'_c A_g} \Rightarrow A_g = \frac{4000}{0.70(4)(1.08)} = 1373\ in^2$

$R_n = 0.11 = \frac{P_u e}{\phi f'_c A_g h} \Rightarrow A_g = \frac{4000(2.4)}{0.70(4)(0.11)(24)} = 1300\ in^2$

$A_{PROV} = (24 \times 24) = 576\ in^2 << A_{g\,req'd}$ ∴ NO GOOD

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STRUCTURAL SYSTEM REDESIGN

□ TYPICAL INTERIOR COLUMN DESIGN (cont'd.)

- CHECK COLUMNS ON LOWER LEVELS

$\Rightarrow A_{g\,req'd} = 1373 \text{ in}^2 = bh = b^2$ for SQUARE COLUMN

$\Rightarrow b = h = 37''$

$\therefore$ USE $40'' \times 40''$ COLUMNS ON LOWER FLOORS

$\Rightarrow A_{STR\,REQ'D.} = \rho bh = 0.035\,(40 \times 40) = 56 \text{ in}^2$

$\therefore$ USE (36) # 11 : $A_{STR} = 36\,(1.56 \text{ in}^2) = 56.2 \text{ in}^2$

* FOR INTERMEDIATE FLOORS, USE SPREADSHEET DEVELOPED TO APPROXIMATE COLUMN SIZES.

ASSUMPTIONS :

$\rho = 0.035$

$\frac{e}{h} = 0.10$

$\gamma \cong 0.80$

* ASSUME COLUMN DESIGN IS CONTROLLED BY

$$K_n = \frac{P_u}{\phi f'_c A_g}$$

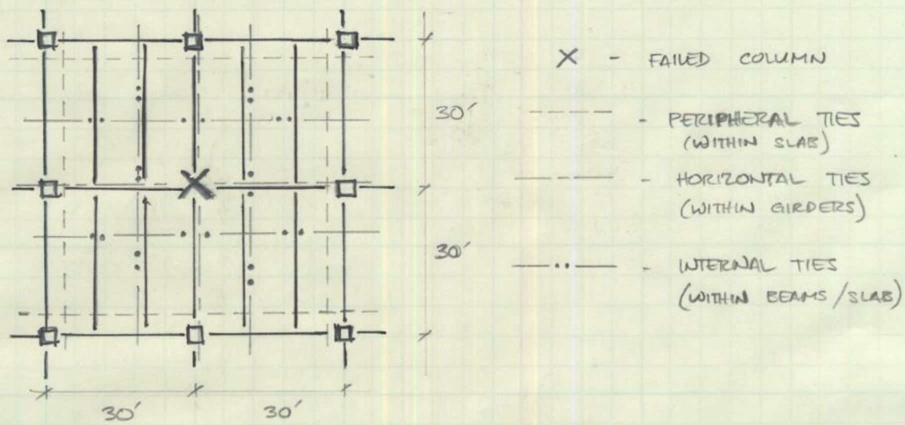
APPENDIX : PROGRESSIVE COLLAPSE DESIGN

1

PROGRESSIVE COLLAPSE DESIGN

□ INDIRECT METHOD

SCENARIO 1 : INTERIOR COLUMN FAILURE



▷ PERIPHERAL TIES

REQUIRED DESIGN STRENGTH = $1.0\,F_t$ (kips)

$$F_t = \min \begin{cases} 4.5 + 0.9 n_o = 4.5 + 0.9(16 \text{ stories}) = 18.9^k \\ 13.5^k \quad * \text{ GOVERNS} \end{cases}$$

$$1.0 F_t \leq \phi\,\Omega\, f_y A_s$$

$$\Rightarrow A_s \geq \frac{13.5^k}{0.75(1.25)(60^{ksi})} = 0.24\,in^2$$

$A_{s,PROV} = 0.31\,in^2$ (#5 FLEX. REINF IN SLAB - EW DIRECTION)

* ADD #5 BARS IN SLAB FOR NS DIR.

2

PROGRESSIVE COLLAPSE DESIGN

▫ INDIRECT METHOD : SCENARIO 1 (cont'd)

▷ INTERNAL TIES

REQUIRED DESIGN STRENGTH (FOR BOTH N-S & E-W DIR.)

$$= \max \begin{cases} \frac{D+L}{156.6}\left(\frac{L_r}{16.4}\right)\left(\frac{F_t}{3.3}\right) = \frac{35+100}{156.6}\left(\frac{30'}{16.4}\right)\left(\frac{13500}{3.3}\right) = 6451 \text{ lbs/ft-width} \; * \text{ GOVERNS} \\ \frac{F_t}{3.3} = \frac{13500}{3.3} = 4091 \text{ lbs/ft-width} \end{cases}$$

$\Rightarrow$ 6451 lbs/ft-width $\leq \phi \Omega f_y A_s = 0.75(1.25)(60{,}000) A_s$

$\Rightarrow$ $A_s \geq 0.12$ in²/ft-width

$A_{s,PROV}$ (N-S) $= \frac{2(4.00 \text{ in}^2)}{30 \text{ ft}} = 0.27$ in²/ft-width $\rightarrow$ (FLEX. REINF. IN BEAMS)

$A_{s,PROV}$ (E-W) $= 0.62$ in²/ft-width $\longrightarrow$ (FLEX. REINF. IN SLAB)

∴ EXISTING FLEXURAL REINF. IN BOTH DIRECTIONS IS ADEQUATE FOR REQUIRED TIE FORCES.

▸ HORIZONTAL TIES TO COLUMNS

REQUIRED DESIGN STRENGTH

$$= \max \begin{cases} 0.03A_t(4)(D+L) = 0.03(30' \times 30')(4)(35+100) = 14.6^k \\ \min \begin{cases} 2.0F_t = 2.0(13.5) = 27^k \\ \frac{l_s'}{8.2}F_t = \frac{15'}{8.2}(13.5) = 24.7^k \; * \text{ GOVERNS} \end{cases} \end{cases}$$

$\Rightarrow$ $A_s \geq \frac{24.7^k}{0.75(1.25)(60^{ksi})} = 0.44 \text{ in}^2$

$A_{s,PROV}$ (N-S) $= 4.0 \text{ in}^2$ (FLEX REINF IN BEAMS)

$A_{s,PROV}$ (E-W) $= 6.35 \text{ in}^2$ (FLEX REINF. IN GIRDERS)

∴ EXISTING FLEXURAL REINF. IN BOTH DIRECTIONS IS ADEQUATE.

3

PROGRESSIVE COLLAPSE DESIGN

□ INDIRECT METHOD : SCENARIO 1 (cont'd.)

▶ VERTICAL COLUMN TIES

REQUIRED DESIGN STRENGTH = $A_t(D+L) = (30' \times 30')(35 + 100)$

$= 121.5^k$

→ $A_s \geq \dfrac{121.5^k}{0.75(1.25)(60\,ksi)} = 2.16\ in^2$

→ $A_{s,PROV} = 10.16\ in^2$ (8x #10 BARS)

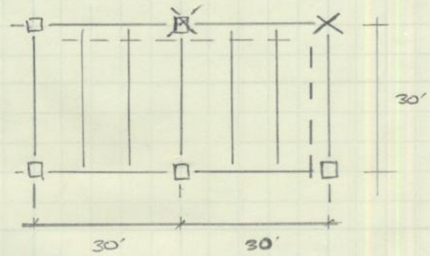
∴ EXISTING COLUMN REINF. IS ADEQUATE FOR REQUIRED TIE FORCES.

4

PROGRESSIVE COLLAPSE DESIGN

□ INDIRECT METHOD

SCENARIO 2: EXTERIOR/CORNER COLUMN FAILURE



▸ EXTERNAL/CORNER COLUMN TIES

REQUIRED DESIGN STRENGTH

EXT. COL. A_t

$$= \max \begin{cases} 0.03(D+L)(A_t) = 0.03(35+100)(30' \times 15') = 1.82^{k} \\ \min \begin{cases} 2.0F_t = 27^{k} \\ \dfrac{l_s}{8.2}F_t = 24.7^{k} \end{cases} \end{cases}$$

* GOVERNS

$$\Rightarrow A_s \geq \frac{24.7^{k}}{0.75(1.25)(60^{ksi})} = 0.44\,in^2$$

$$\rightarrow A_{s,PROV} = 6.35\,in^2$$ (5 #10 AS FLEX. REINF. IN GIRDERS)

∴ EXISTING GIRDER REINF. IS ADEQUATE FOR REQUIRED TIE FORCES

APPENDIX : BLAST-RESISTANT GLAZING DESIGN

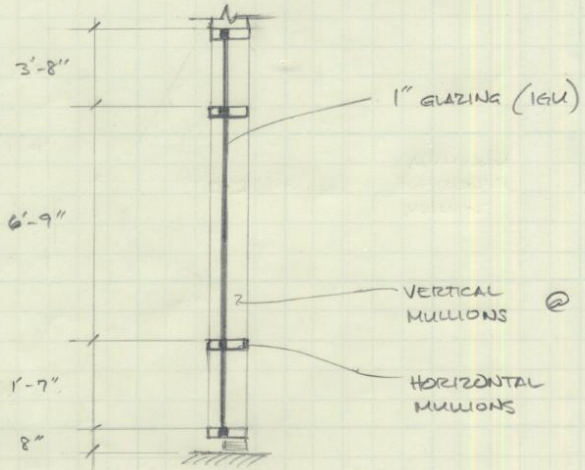
1

BLAST-RESISTANT GLASS DESIGN

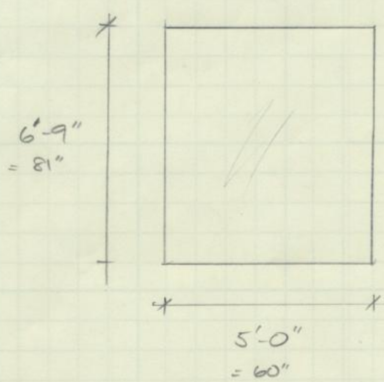
☐ EXISTING GROUND LEVEL GLAZING ANALYSIS

UNITIZED PRE-FRAMED ALUMINUM CURTAIN WALL W/ 1" CLEAR GLAZING. (TYP.)

▷ FIRST FLOOR CURTAIN WALL SECTION:



▷ CRITICAL GLASS PANEL SIZE:



GLAZING PANEL AREA:

$A_G = 6.75' \times 5.0' = 81'' \times 60''$

$A_G = 33.75\ ft^2 = 4860\ in^2$

ASPECT RATIO:

$\frac{6.75'}{5'} = 1.35$

2

BLAST-RESISTANT GLASS DESIGN

☐ EXISTING GLAZING ANALYSIS (cont'd.)

▶ DETERMINE EQUIVALENT 3-SECOND DURATION DESIGN LOAD

FROM FIGURE 3, NORVILLE & CONRATH:

ASSUME 100lb & 200 lb CHARGES.
ASSUME STANDOFF DISTANCE OF 75 ft
(CLOSE PROXIMITY TO STREET)

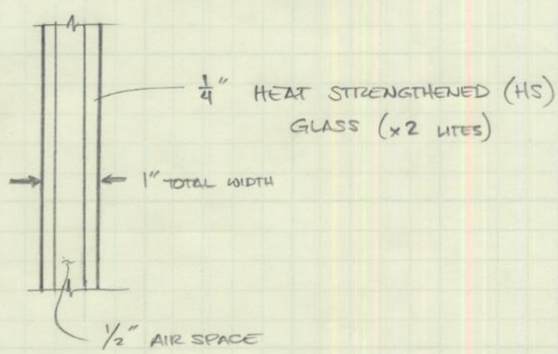
→ 100lb CHARGE EQUIVALENT LOAD = 100 psf

→ 200 lb CHARGE EQUIVALENT LOAD = 140 psf

▷ DETERMINE CAPACITY OF EXISTING GLAZING UNIT:

IGU's USED FOR FIRST FLOOR PANELS:

SECTION:



FROM ASTM E1300-04,

→ NFL (NON-FACTORED LOAD) FOR BOTH LITES: [FIG. A1.6]

= 1.70 kPa = 35.5 psf

→ GTF (GLASS TYPE FACTOR) [TABLE 2]

GTF1 = GTF2 = 1.8

→ LSF (LOAD SHARE FACTOR) [TABLE 5]

LS1 = LS2 = 2.00

3

BLAST-RESISTANT GLASS DESIGN

□ EXISTING GLAZING ANALYSIS (cont'd.)

▷ DETERMINE CAPACITY OF EXISTING GLAZING UNIT:

⇒ LOAD RESISTANCE (CAPACITY) OF IGU:

LR = NFL × GTF × LS

= 35.5 psf × 1.8 × 2.0

LR = 128 psf > 100 psf ∴ OK FOR 100 lb CHARGE

LR = 128 psf < 140 psf ∴ NO GOOD FOR 200 lb CHARGE

□ DESIGN GLAZING FOR 200 lb CHARGE @ 75 ft. STANDOFF DISTANCE.

▷ TRY CHANGING GLASS TYPE TO FULLY TEMPERED (FT).

* ASSUME SAME THICKNESSES/CONFIGURATION AS EXISTING DESIGN.

→ NFL = 35.5 psf

→ GTF 1 = GTF 2 = 3.6

→ LS 1 = LS 2 = 2.00

⇒ LR = NFL × GTF × LS

= 35.5 psf × 3.6 × 2.0

LR = 256 psf > 140 psf ∴ GOOD FOR 200 lb CHARGE

▷ DETERMINE MAXIMUM ALLOWABLE CHARGE / LOAD FOR NEW DESIGN:

FROM FIG. 3, NORVILLE & CONRATH:

* @ LR = 256 psf, ALLOWABLE CHARGE IS INDEFINITE

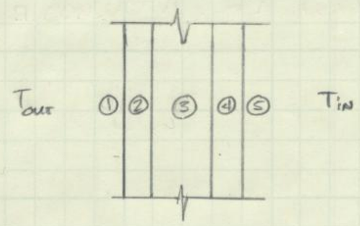
∴ CURRENT DESIGN OF (2) LITES FT GLASS @ 1/4" THICKNESS

IN IGU CONFIGURATION IS ACCEPTABLE FOR BLAST LOADS.

APPENDIX : GLAZING THERMAL PERFORMANCE

1

HEAT GAIN/LOSS OF EXISTING SYSTEM



SECTION OF TYPICAL INSULATING GLASS UNIT

① EXTERIOR AIR FILM
② 1/4" GLASS LITE 1
③ 1/2" AIR SPACE
④ 1/4" GLASS LITE 2
⑤ INTERIOR AIR FILM

▹ CONDITIONS FOR NEW HAVEN, CT:

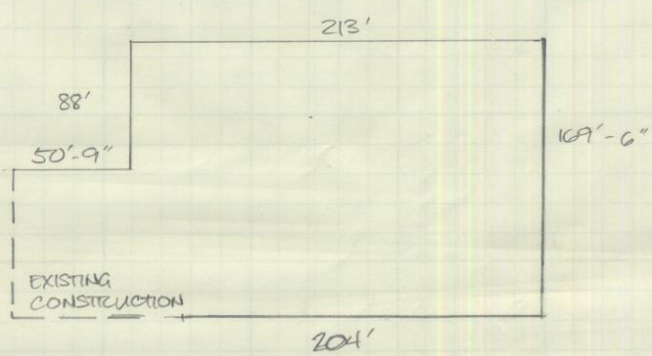
SUMMER:	T_{OUT} =	86°F	=	30°C	=	303 K
	T_{IN} =	75°F	=	24°C	=	297 K
WINTER:	T_{OUT} =	7°F	=	14°C	=	287 K
	T_{IN} =	70°F	=	21°C	=	294 K

$\Delta T_{SUMMER} = 6K = 11°F$

$\Delta T_{WINTER} = -7K = -63°F$

▹ OVERALL GLAZING AREA (1ST FLOOR ONLY)

- HEIGHT OF GLAZING = 10'-9" = 129"
- BUILDING FOOTPRINT:



P = 213' + 88' + 50.75' + 204' + 169.5'

P = 725.25'

A_{TOT} = 10.75' × 725.25'

$\boxed{A_{TOT} \approx 7800 \text{ sq.ft.}}$

2

HEAT GAIN/LOSS OF EXISTING SYSTEM

▷ CONDUCTION HEAT FLOW

$Q_c = AU \cdot \Delta T$

$$Q_{c_{SUMMER}} = 7800\,ft^2 \left(0.56 \frac{BTU}{hr \cdot ft^2 \cdot °F}\right)(11°F)$$

$$= 48048\ BTU/hr$$

$$Q_{c_{WINTER}} = 7800\,ft^2 \left(0.49 \frac{BTU}{hr \cdot ft^2 \cdot °F}\right)(-63°F)$$

$$= -240{,}786\ BTU/hr$$

▷ RADIANT HEAT FLOW

STEFAN-BOLTZMANN EQUATION:

$$Q_R = A \cdot F_E \cdot F_A\, \sigma \left(T_s^4 - T_a^4\right)$$

$A = 7800$ sq.ft $= 725\ m^2$

$F_E = 0.92$ FOR GLASS [www.thermoworks.com]

$F_A \triangleq$ ANGLE FACTOR (ASSUME AS 1.0)

$\sigma \triangleq$ STEFAN-BOLTZMANN CONSTANT

$= 5.67 \times 10^{-8}\ W/m^2 \cdot K^4$

$T_s \triangleq$ SURFACE TEMPERATURE (K)

$T_a \triangleq$ AMBIENT TEMPERATURE (K)

HEAT GAIN/LOSS OF EXISTING SYSTEM

- RADIANT HEAT FLOW (cont'd.)

  * FIND SURFACE TEMPERATURES:

SUMMER:

SURFACE TEMPERATURE of INSIDE LITE:

$$\Delta T_s = \frac{\Delta T_{TOTAL}}{\Sigma R} \cdot R_{LAYER}$$

$$= \frac{6K}{0.32}(0.197)$$

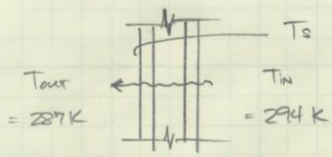
$$\Delta T_s = 3.7K$$

$$\therefore T_s = 303K - 3.7K$$

$$T_s = 299.3K$$

WINTER:

SURFACE TEMPERATURE of OUTSIDE LITE:



$$\Delta T_s = \frac{7K}{0.36}(0.334)$$

$$\Delta T_s = 6.5K$$

$$\therefore T_s = 294^K - 6.5^K$$

$$T_s = 287.5K$$

$$Q_{R_{SUMMER}} = 725m^2(0.92)(1.0)(5.67\times10^{-8}\ W/m^2\cdot K^4)(299.3^4 - 297^4)$$

$$= 9222\ W \approx 31{,}500\ BTU/hr$$

$$Q_{R_{WINTER}} = 725m^2(0.92)(1.0)(5.67\times10^{-8}\ W/m^2\cdot K^4)(287.5^4 - 287^4)$$

$$= 1793\ W \approx 6100\ BTU/hr$$

4

HEAT GAIN/LOSS OF EXISTING SYSTEM

▸ TOTAL HEAT FLOW THROUGH EXISTING SYSTEM.

SUMMER:

$$Q_{TOT} = Q_C + Q_R$$
$$= 48{,}048 \text{ BTU/hr} + 31500 \text{ BTU/hr}$$
$$= 79{,}548 \text{ BTU/hr} \quad \text{(HEAT GAIN)}$$

WINTER:

$$Q_{TOT} = Q_C + Q_R$$
$$= 240{,}786 \text{ BTU/hr} + 6100 \text{ BTU/hr}$$
$$\approx 247{,}000 \text{ BTU/hr} \quad \text{(HEAT LOSS)}$$

▸ APPROXIMATE HEATING & COOLING COSTS

COOLING:

* ELECTRICITY COST FOR CT, 2008: 15.93 cents per kW·hr.

[ENERGY INFORMATION ADMINISTRATION (EIA) WEBSITE]

$$\Rightarrow \text{COOLING COST} = \frac{\$0.1593}{\cancel{kW\cdot hr}}\left(\frac{\cancel{kW}}{3412\,\cancel{BTU}}\right)\left(\frac{1\,\cancel{hr}}{1}\right)\left(\frac{79{,}548\,\cancel{BTU}}{hr}\right)$$
$$= \$3.71 \text{ per } \cancel{hour} \left(\frac{24\,\cancel{hrs}}{1\,\cancel{day}}\right)\left(\frac{30\,\cancel{days}}{month}\right)$$
$$\boxed{\approx \$2700 \text{ per month}}$$

HEATING:

$$\Rightarrow \text{HEATING COST} = \frac{\$0.1593}{\cancel{kWh}}\left(\frac{\cancel{kWh}}{3412\,\cancel{BTU}}\right)\left(\frac{247{,}000\,\cancel{BTU}}{hr}\right)$$
$$= \$11.53 \text{ per hour}$$
$$\approx \boxed{\$8300 \text{ per month.}}$$

5

COMPARISON OF LOW-E GLASS

▸ ADJUST R-VALUES & F_E FOR NEW TYPE OF GLAZING.

SOFT COAT LOW-E INSULATED GLASS:

$R\text{-VALUE} = 3.50 \ \frac{hr \cdot ft^2 \cdot °F}{BTU}$

$F_E = 0.74$

$R_{\text{EXISTING GLASS}} = 0.04 \ \frac{hr \cdot ft^2 \cdot °F}{BTU}$

$F_{E_{\text{EXISTING}}} = 0.92$

→ CALCULATE U-VALUE OF NEW SYSTEM.

[SEE EXCEL SPREADSHEET]

$\Sigma R = 5.26 \ \frac{hr \cdot ft^2 \cdot °F}{BTU}$

$U = 0.19 \ \frac{BTU}{hr \cdot ft^2 \cdot °F}$

[SUMMER]

$\Sigma R = 5.52 \ \frac{hr \cdot ft^2 \cdot °F}{BTU}$

$U = 0.18 \ \frac{BTU}{hr \cdot ft^2 \cdot °F}$

[WINTER]

→ CALCULATE NEW HEAT GAINS/LOSSES:

CONDUCTIVE:

$$Q_{c_{\text{SUMMER}}} = 7800 \ ft^2 (0.19)(11°F) = 16{,}302 \ BTU/hr$$

$$Q_{c_{\text{WINTER}}} = 7800 \ ft^2 (0.18)(-63°F) = -88{,}452 \ BTU/hr$$

RADIANT: (ASSUME SURFACE TEMPERATURES ARE CONSTANT)

$$Q_{R_{\text{SUMMER}}} = 725 \ m^2 (0.74)(1.0)(5.67 \times 10^{-8} \ W/m^2 \cdot K^4)(299.3^4 - 297^4)$$

$$= 7417 \ W \approx 25{,}300 \ BTU/hr$$

$$Q_{R_{\text{WINTER}}} = \frac{0.74}{0.92}(1793 W) = 1442 \ W$$

$$\approx 4920 \ BTU/hr$$

6

COMPARISON OF LOW-E GLASS

▸ TOTAL HEAT FLOW THROUGH LOW-E SYSTEM:

SUMMER

$$Q_{TOT} = Q_C + Q_R$$
$$= 16{,}302 + 25{,}300$$
$$= 41602 \ BTU/hr \quad \text{(HEAT GAIN)}$$

$$Q_{TOT} = Q_C + Q_R$$
$$= 88{,}452 \ BTU/hr + 4920 \ BTU/hr$$
$$= 93{,}400 \ BTU/hr \quad \text{(HEAT LOSS)}$$

▸ APPROXIMATE HEATING & COOLING COSTS

COOLING:

* ELECTRICITY COST: 15.93 cents per kWh.

$$\rightarrow \text{COOLING COST} = \frac{\$0.1593}{\cancel{kWh}}\left(\frac{\cancel{kWh}}{3412\ \cancel{BTU}}\right)\left(\frac{41{,}602\ \cancel{BTU}}{hr}\right)$$
$$= \$1.94 \text{ per } \cancel{hour}\left(\frac{24\ hrs}{\cancel{day}}\right)\left(\frac{30\ \cancel{days}}{month}\right)$$
$$\approx \boxed{\$1400 \text{ per month}}$$

HEATING:

$$\rightarrow \text{HEATING COST} = \frac{\$0.1593}{\cancel{kWh}}\left(\frac{1\ \cancel{kWh}}{3412\ \cancel{BTU}}\right)\left(\frac{93{,}400\ \cancel{BTU}}{hr}\right)$$
$$= \$4.36 \text{ per } \cancel{hour}\left(\frac{24\ hours}{\cancel{day}}\right)\left(\frac{30\ \cancel{days}}{month}\right)$$
$$\approx \boxed{\$3100 \text{ per month}}$$

APPENDIX : PROJECT TEAM

Owner:

Yale-New Haven Hospital // New Haven, CT // www.ynhh.org

Construction Manager:

Turner Construction Company // Milford, CT // www.turnerconstruction.com

Architect:

Shepley Bulfinch Richardson & Abbott // Boston, MA // www.sbra.com

Structural Engineer:

Spiegel Zamecnik & Shah // New Haven, CT // www.szsdc.com

Mechanical/Electrical Engineer:

BR+A Consulting Engineers, Inc. // Boston, MA // www.brplusa.com

Plumbing/Fire Protection Engineer, Code Consultant:

R.W. Sullivan Consulting Engineers, Inc. // Boston, MA // www.rwsullivan.com

Civil Engineer:

Tighe & Bond Consulting Engineers // Norwalk, CT // www.tighebond.com

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My family and friends

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