Food and Drug Administration

Center for Drug Evaluation and Research

Office Building 2

FINAL REPORT

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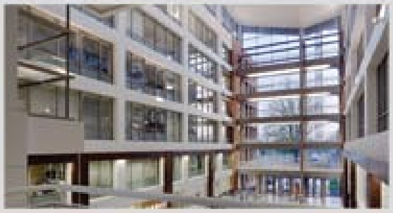
Food & Drug Administration
Center for Drug Evaluation and Research
Office Building 2, White Oak, MD






Project Information

project size:	330,000 gsf
floors/height:	6 stories/85 ft
architect:	kling stubbins
	rtkl associates
structural/mep:	rtkl associates

Architecture

- composed of two seperate office wings
- office wings utilize a brick facade and punch windows
- rising six-story atrium connects the wings
- atrium curtain wall system promotes an inviting entrance
- aluminum panel wall system accents brick facade

Structure

- concrete moment resisting frame
- 9.4" two-way slab floor system
- interior columns 2'x2' with 5'x'5 drop panel typ.
- atrium roof supported by W310 and W360 beams

Mechanical

- 5 outdoor air handling units - 100% outdoor air
- (1) 236 liter/s, (3) 990 liter/s, (1) 1133 liter/s
- 43.68 kW heating coil typical

Electrical

- office lighting: semi-indirect 277V flourescent
- hall lighting: recessed compact flourescent
- primary 480V delta, secondary 208/120V transformers

Michael O. Spear | Structural Option | Architectural Engineering
www.engr.psu.edu/ae/thesis/portfolios/2009/mos122/

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EXECUTIVE SUMMARY

This thesis report focused on the structural redesign of Center for Drug Evaluation and Research Office Building 2 (CDER2). While originally employing a two-way flat plate, moment frame structural system, this thesis explored the implementation of a composite metal deck, steel moment frame structural system. After determining the required metal deck and lightweight concrete slab using the Vulcraft design guide, RAM Structural System was utilized in the design of the composite gravity beams, as well as the steel moment frames.

Additional loading scenarios were explored as the progressive collapse effects of a blasted column were explored. Critical columns to be removed, plus the necessary load combinations, for an alternative path, static analysis were determined through referencing various design guides and standards. Two designs were performed, both of which considered identical loading and column removal cases. The first design was based on virtual work findings that considered the plastic behavior of the moment frame girders. These minimal design member sizes were then compared against those determined in the initial analysis, which considered traditional loading. The second progressive collapse analysis employed RAM for a completely elastic design. Again the required minimum member sizes were compared against those determined in the initial analysis. These two analyses resulted in two different designs to inhibit progressive collapse.

In learning from the unfortunate experiences of past disasters, it has become clear that other design concerns need to be addressed regarding terrorism attacks besides the structural implications alone. For many buildings, glass and flying debris were the primary cause for the majority of the injuries. As a result, the atrium curtain wall and office window were both designed to resist blast loading. The conductance and heat transfer properties of these new units were then analyzed for summer and winter design conditions.

BUILDING INFORMATION

Center for Drug Evaluation and Research Office Building 2

Owner:
GSA/FDA (General Services Administration/Food and Drug Administration)

Architect:
Kling Stubbins
2301 Chestnut Street
Philadelphia, PA 19103
http://www.klingstubbins.com/

Architect:
RTKL Associates Inc.
901 South Bond Street
Baltimore, MD 21231
http://www.rtkl.com/

Structure/Interior/Mechanical/Electrical:
RTKL Associates Inc.
901 South Bond Street
Baltimore, MD 21231
http://www.rtkl.com/

Acoustics:
Shen Milson & Wilke, Inc.
3300 N. Fairfax Dr., Suite 302
Arlington, VA 22201
http://www.smwinc.com/index.html

Site/Civil/Environmental:
Greenhorne & O'Mara, Inc
http://www.g-and-o.com/main.asp

Topographic Surveying:
A. Morton Thomas & Associates, Inc
http://www.amtengineering.com/

BUILDING INFORMATION

The Center for Drug Evaluation and Research Office Building 2 (CDER 2) is a new six story office building for the FDA. This office, located in White Oak, Maryland - north of Washington DC (Figure 1) - is responsible for, as the name implies, the investigation into drugs for humans.

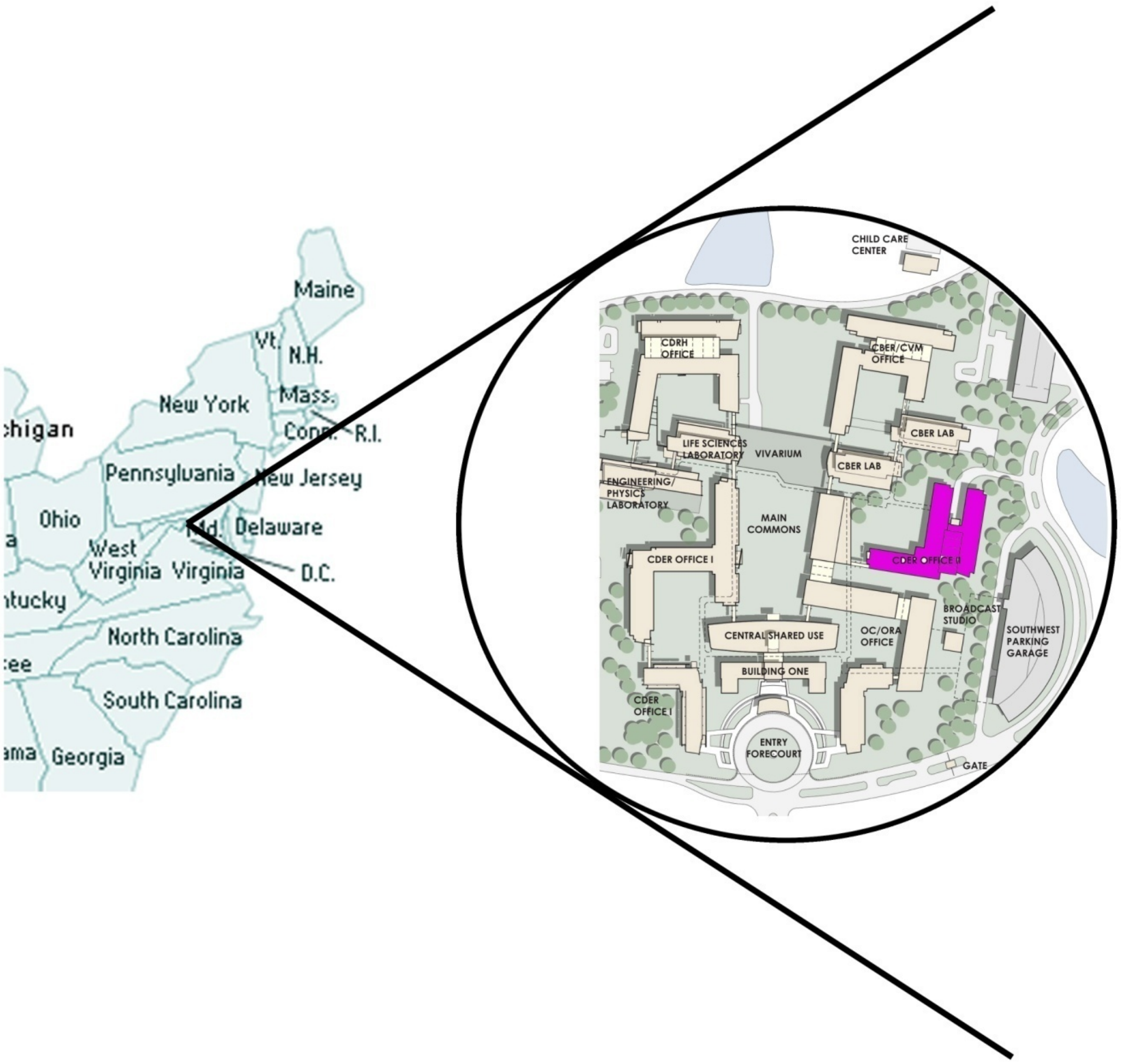

Figure 1: Site Location and Campus

BUILDING INFORMATION

CDER 2 is composed of two separate office wings (Figure 2) connected by an atrium that reaches through the center of the building. The first floor of the atrium is the main entrance which rises up the full six stories, creating a dramatic entrance to CDER 2. Opposite the entrance is the main elevator lobby; bridging the entrance and elevators are the security desks and offices, as well as a few more welcoming spaces such as the large reception area and coffee bar (Figure 3). As the elevators rise, they empty into lobbies which are accessible to both office wings.

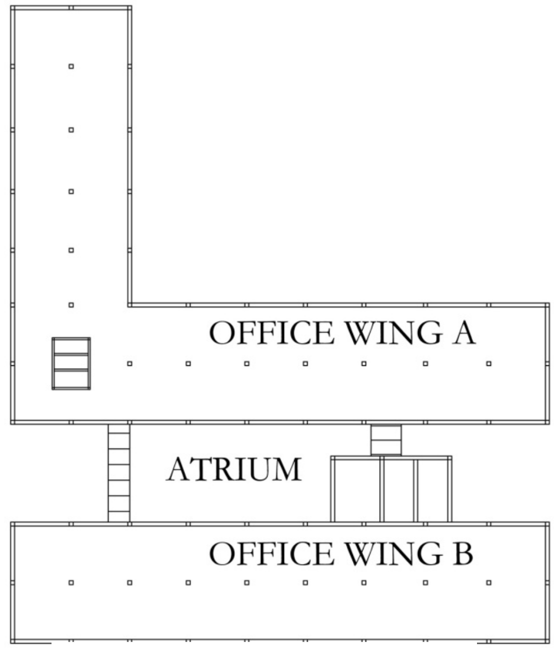

Figure 2: Typical Plan

The floor plans for both office wings A and B are fairly consistent throughout all levels, with story height e to the geometry of the office wings, being long and narrow, most offices have a view of either the exterior or the grand interior lobby. The offices themselves are also quite consistent in size.

Figure 3: Interior Atrium

BUILDING INFORMATION

The primary exterior wall system, the one used on office wings A and B, includes a brick façade which anchors into the structural concrete beams and supporting CMU wall. The interior of this system consists of gypsum board on metal studs. Punch windows, typically about six feet wide by seven feet tall, occur at each office location.

Figure 4: CDER2 Elevation

The brick façade is occasionally accented by an aluminum panel and mullion system. The mullions anchor into a metal stud framing system, whose interior is also gypsum board. The aluminum panels are usually utilized near areas of egress that are visible from the exterior: stairs, elevators, and exits are a few examples (Figure 4).

The atrium's curtain wall system stands out from the rest of the building and draws people to the entrance. The curtain wall consists of low-e insulated glass, supported by horizontal and vertical aluminum mullions which anchor into the structural concrete. Almost the entire atrium utilizes the system, even the sides that are framed in by the office wings A and B (Figure 5). This provides excellent views both from the atrium to the outside and from the many offices that overlook the atrium.

Figure 5: Atrium curtain wall

BUILDING INFORMATION

The air supply for CDER 2 is provided through an underfloor low-pressure air plenum. There are five rooftop air handling units responsible for supplying 100% outdoor air, (3) 990 L/s, (1) 1133 L/s, and (1) 236 L/s. Additionally, there are five return air handling units, all around 1500 L/s. The supply air is directed to each floor by 700x600 mm ductwork. The air is then directed to the supply floor diffusers through the distribution plenum box located beneath each floor.

There are seven additional indoor air handling units located in the basement. They service the atrium, mechanical and electrical rooms, and the basement.

The hot and cold water supply enters CDER 2 in the basement from the tunnel that connects CDER 2 to the other buildings in the campus. From the tunnel, the water is direct to the six water pumps located in the basement of wing B; the three cold water pumps are 10KW pumps capable of 32.2 L/s, while the three hot water pumps are 5.4KW pumps capable of 17.3 L/s.

CDER 2 utilizes a wet sprinkler system for fire protection; the system is completely separate from the function of the other mechanical systems. The water supply for the sprinkler system is supplied from the same tunnel that connects CDER 2 with the campus. A separate room in the basement holds the 56 KW, 47 L/s fire pump and controlling equipment. Each level has the wet sprinkler system located in the main corridor of each office wing, as well as the atrium bridges connecting the wings.

The most typical lighting in CDER 2 occurs in the offices; wall mounted and pendant mounted 2400mm long fluorescents are utilized. Typically the fluorescent lamp has a color temperature of 3500K and a CRI of 85. The fixture housing consists of a steel rectangular profile with a semi-specular aluminum reflector. The lighting in the main corridors of the office wings primarily consists of a low profile compact fluorescent downlight.

The elevator lobby employs a few types of lighting systems. The lobby itself is primarily lit by a 0.9meter diameter circular recessed fluorescent fixture with a thick white acrylic lens; a compact fluorescent downlight within a frosted glass cylinder is also used. Additionally, two types of accent lighting are utilized in the lobby; one is a low voltage, incandescent wallwasher with a glass lens, which directs its light onto the wall adjacent to the elevators. The other is a low voltage, adjustable accent light recessed within the raised floor. Its light is directed on the wall of the conference room that faces the lobby.

Similar to the mechanical supply, the primary feeders enter CDER 2 through the tunnel which connects to the campus. Each office wing has its own transformer; wing A utilizes a 2000 KVA forced air, three phase cast coil dry-type transformer that steps the voltage down from 13.8 KV to 480\277V. Wing B utilizes a 1500 KVA natural ventilation, three phase cast coil dry-type transformer that steps the voltage down from 13.8 KV to 480\277V.

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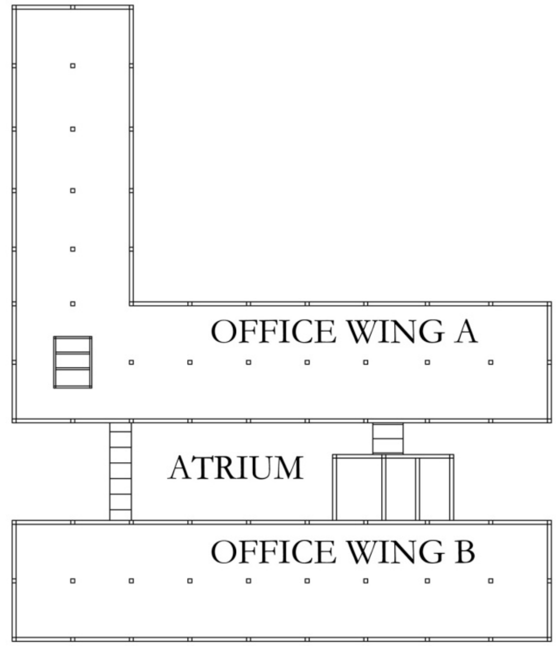
EXISTING STRUCTURAL SYSTEM

CDER2 utilizes a two-way flat slab floor system with drop panels located at the interior columns. Structurally, bay sizes are very typical and almost square being about 30’x31’ (Figure 6). The floor slab itself is made up 9.4 inch (240mm), f’c = 4000 psi (28 MPa); at the drop panels, the slab gains an addition seven inches (180 mm). Void of interior beams, the reinforcement of the two way slab consists of primarily of #4-#7 ASTM GRADE 400 steel bars. An even distribution of reinforcement is regular throughout most of the slab, multiple layers of reinforcement are common near columns as well as the deep exterior beams. While there are little variations in the floor system, these differences primarily occur around the areas of egress and in mechanical spaces.



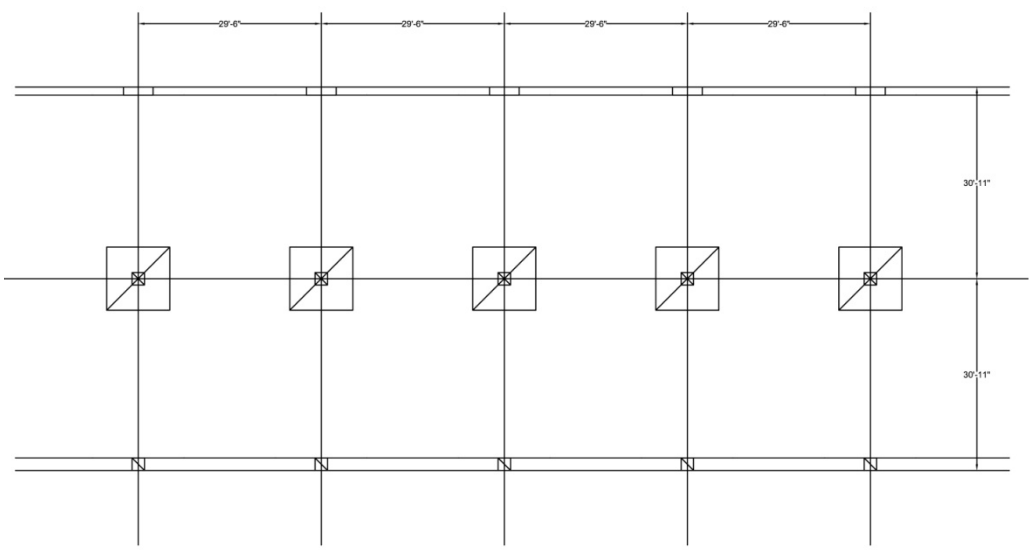


Figure 6: Typical Bays

EXISTING STRUCTURAL SYSTEM

With a concrete moment frame, the lateral and gravity system of CDER2 are the same. A uniform grid of columns with deep exterior beams and a two-way flat slab, establishes the structural system.

There are two main column geometries: the interior square columns and the long and lean exterior columns. The interior columns are about 24"x24" (600mm x 600mm) using 4000 psi (28MPa) concrete (Figure 7). In addition to being found at the interiors of the office wings, these 24"x24" columns are also located where the wings are bordered by the atrium. The exterior columns are about 16"x58" (400mm x 1460mm) and are also composed of 4000 psi concrete. Reinforcement within the columns is fairly consistent with #9 bars, but varies in arrangement and number depending on the loading and level.

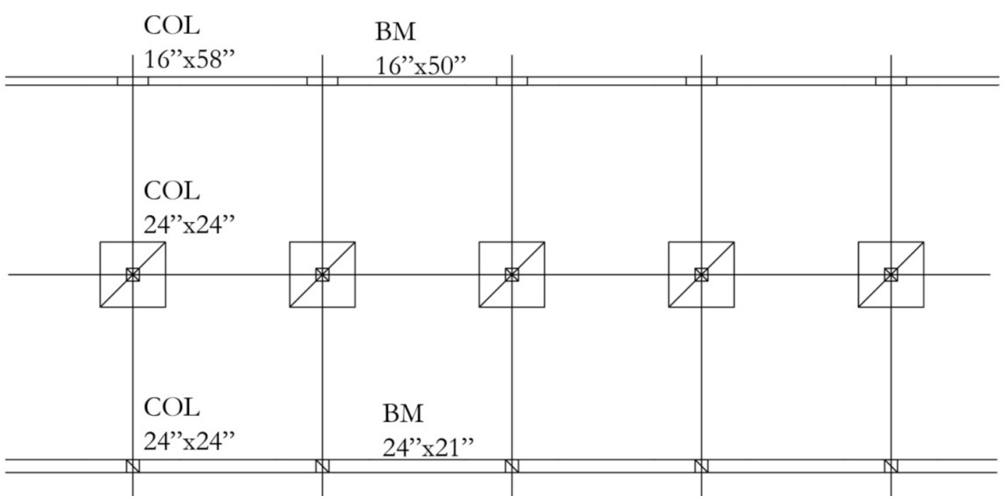


Figure 7: Typical Bays

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STRUCTURAL DEPTH: INHIBITING PROGRESSIVE COLLAPSE

One challenge facing the designers of CDER2, was the potential for terrorist attacks and other threats on the building's structure and its occupants. As a result of disasters, such as the Oklahoma City Federal Building bombing as well as the 9/11 World Trade Center collapse, the methodology for designing federal buildings has changed. In the case of CDER2, the designers had to employ the "GSA Progressive Collapse Analysis and Guidelines" when designing the ground level structural elements.

For the purpose of this thesis, the problem of progressive of collapse was reevaluated for an alternate steel framing system. Due to the relative ease of accessibility, as compared to the other levels of the building, only ground level structural threats were addressed; all blasts will be assumed to have originated on the ground level or site level and any "blasted-out" columns will be constrained to the first floor only. This parallels the challenges faced by the original designers. Additionally by considering an alternate steel framing system, a new gravity and lateral resisting structural systems will was implemented.

STEEL DESIGN

In order to develop a steel framing solution that was equivalent to the original concrete system, the same design loads were employed for the steel redesign; please note that the seismic loads were also reevaluated to accommodate the new building weight:

Office:	SDL = 20psf (mechanical, ceiling, access floor) LL = 80psf
Public/Egress:	SDL = 20psf (mechanical, ceiling, access floor) LL = 100psf
Roof:	SDL = 42psf (mechanical, ceiling, roofing, insulation, paver) LL = 32psf

STEEL DESIGN

Referencing ASCE 7-05, the following LRFD load combinations were considered when analyzing CDER2:

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

A composite steel deck with composite steel beams and girders is an alternative steel floor system that was developed for CDER2. The Vulcraft Steel Roof and Floor Deck Catalog was employed for the design. In order to achieve a two hour fire rating for the system, a slab depth of three and one quarter inches of lightweight concrete was used.

Due to the current column layout, a desired deck span was either about ten feet or fifteen feet. The selection of the 2VLI 2", 20 gauge metal deck - spanning ten feet was controlled by the max unshored length during construction (Figure 8, 9). This was a given value in the design catalog.

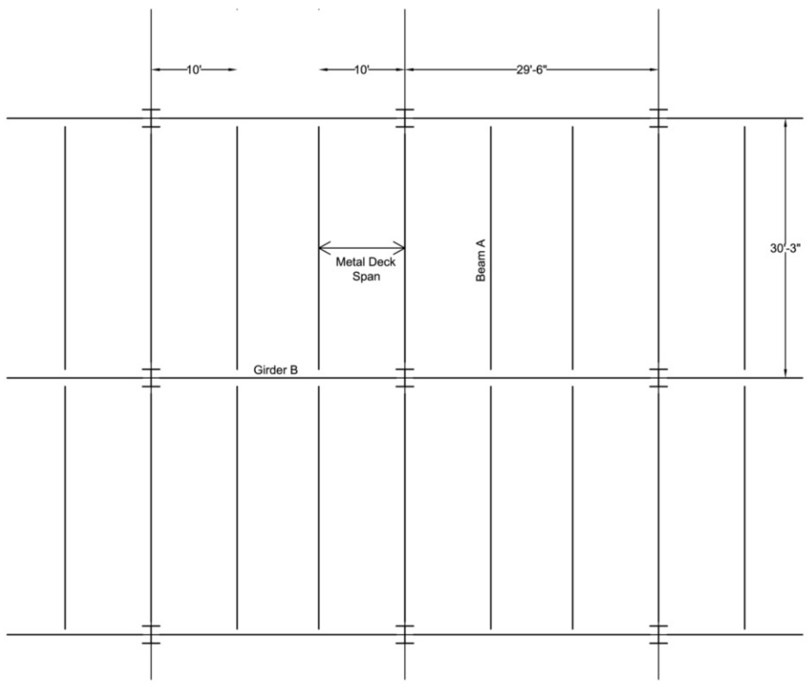

Figure 8: Composite Steel Floor System

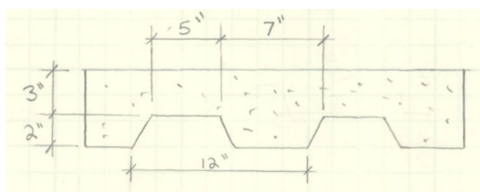

Figure 9: 2VLI 2" Metal Deck

STEEL DESIGN

In developing the lateral force resisting system, progressive collapse concerns also needed to be considered. For that reason, a steel moment frame system was chosen as the lateral force resisting system for its redundancy. In order to have minimal impact on the existing architecture, the original 30'x30' bay size remained (Figure 8).

At this point, with the steel deck and slab thickness having been calculated, as well as the beam and column layout having been determined, the computer software RAM Structural System was employed to assist in the design of the steel members according to AISC LRFD standards. The beams, spaced ten feet on center and which span thirty feet from girder to girder, were designed as composite steel beams responsible for gravity loads only. The beams were assumed to have the top flange fully braced by metal decking. The girders, spanning column to column, make up the lateral system and utilize moment connections. Finishing the moment frames, the columns were assumed to be spliced every two floors.

Both seismic and lateral loads were considered when designing the steel moment frame. The wind loads were determined using Chapter 6 of ASCE 7-05:

Wind Speed =	90mph
Kd =	0.85
Occupancy =	II
Importance =	1
Exposure =	B

The due to the new building structural system, the building weights, and thus seismic loads were a function of the chosen member sizes. In order to prevent multiple hand calculation iterations, RAM was employed to calculate and apply the necessary seismic loads:

Site Class =	C	Fa =	1.2
Importance =	1	Fv =	1.7
Ss =	0.155g	TL =	8.0s
S1 =	0.050g	R =	3

STEEL DESIGN

The steel members were given initial trial sizes based solely on gravity loads. Then as the lateral loads were applied, multiple iterations were performed to determine the required sizes of the moment beams and columns; all demand loads were checked against the member capacities and the interaction equations H1-1a, H1-1b.

The designed steel moment frame system consists of (Figure 11):

typical gravity beam =	W16x26
typical interior girder =	W21x57, W21x50
typical exterior girder =	W21x50
base interior column =	W12x106
base exterior column =	W12x72
base atrium column =	W12x120

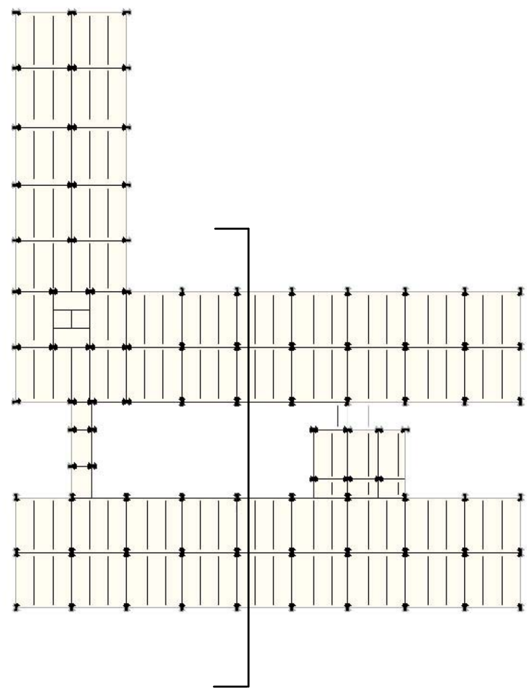

Figure 10: New Steel Plan

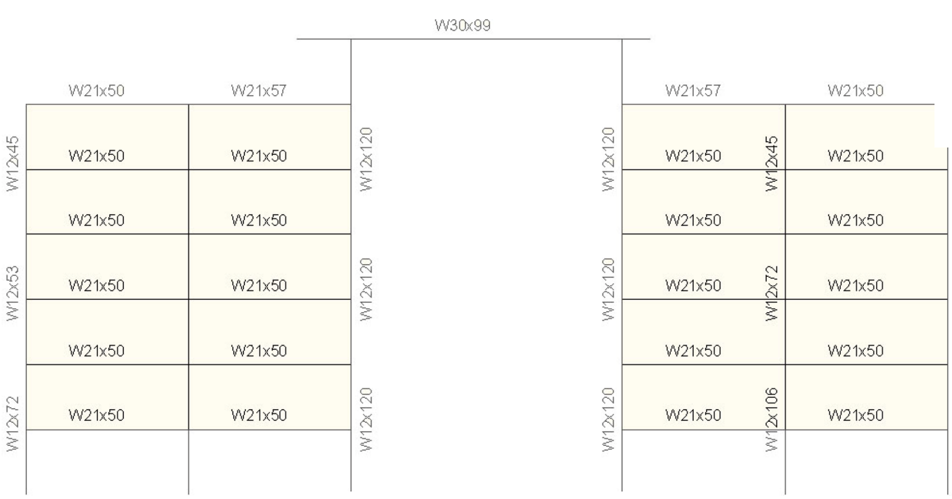

Figure 11: New Steel Section

PROGRESSIVE COLLAPSE

Once a new steel structural system had been developed for standard loading conditions, the progressive collapse analysis began by researching and implementing various design codes, standards, and papers: AISC Blast Guide, UFC Design of Buildings to Resist Progressive Collapse, etc.

Assuming a low level of protection design requirement, the alternate path method is not required, but was still utilized in the analysis of CDER2. Also, a linear static analysis procedure was employed. This is a geometric formulation in which the materials are treated as linear elastic, with the exception of discrete hinges. A full static load is applied at one time to the structure, from which a critical load bearing element has been removed.

The critical load bearing elements are typically in the form of columns; as a minimum, corner columns need to be removed. Also, external columns located on the middle of the long and short sides need to be removed. Internal columns must be removed at critical locations as determined with engineering judgment (Figure 12).

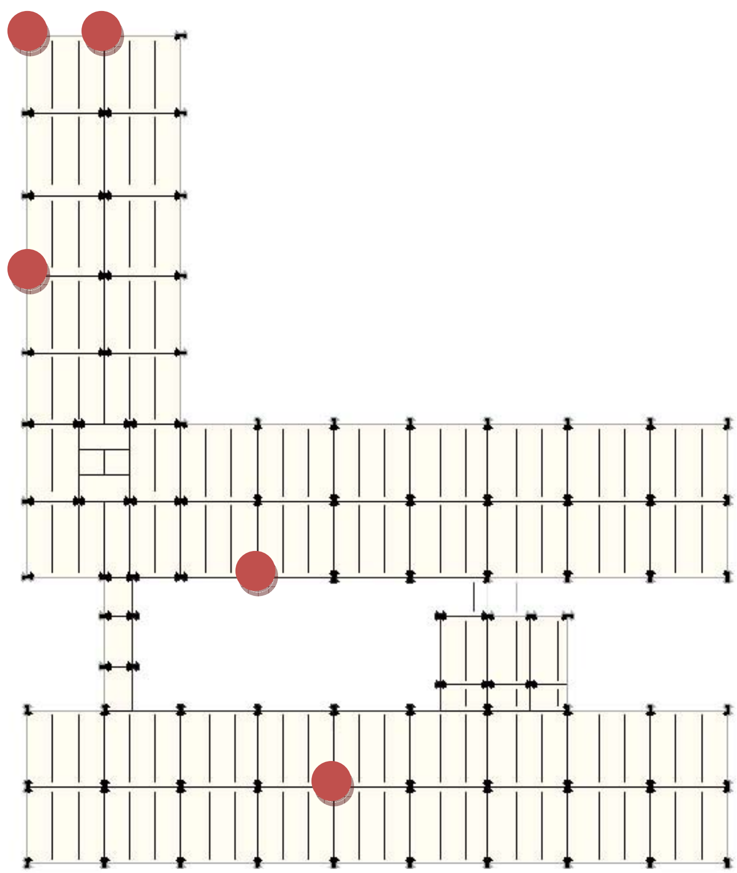

Figure 12: Critical Column Locations

PROGRESSIVE COLLAPSE

In those bays that are immediately affected by the removal of the critical element, including both adjacent bays and those on floors above the element (Figure 13, 14), the following load combination must be employed:

2.0 [(0.9 or 1.2)D + (0.5L or 0.2S)] + 0.2W

The factor of two, applied to the dead and live loads, is to approximate the dynamic amplification of the load when the critical element is instantly removed. To all other bays, the following load combination is applied:

(0.9 or 1.2)D + (0.5L or 0.2S) + 0.2W

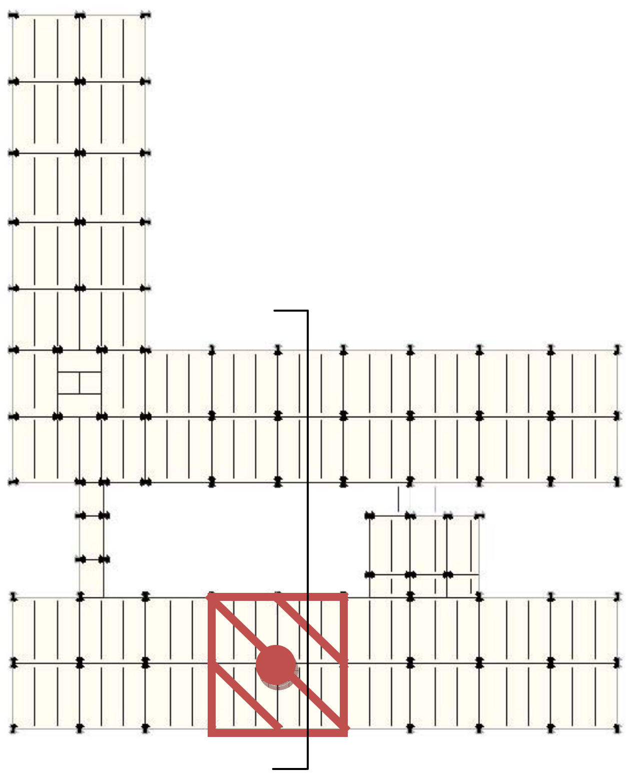

Figure 13: Critical Loading Plan

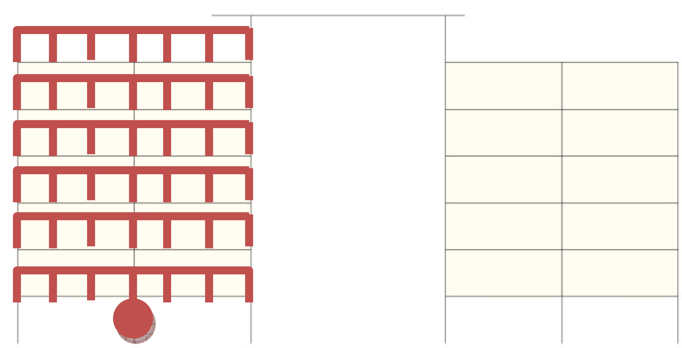

Figure 14: Critical Loading Section

PROGRESSIVE COLLAPSE

In order to develop preliminary trial sizes, the plastic moment of the girders required to prevent formation of a collapse mechanism can be determined from a virtual work analysis (Figure 15); the internal work is equal to the plastic moment of the girders multiplied by the rotation of the girder. The external work is the gravity load on the removed column multiplied by the displacement. This procedure neglects the catenary action of the beams and, therefore, is a conservative design.

$$P * \delta = \sum Mp * \Theta$$

$$\delta = L * \Theta$$

$$P = \sum Mp * \Theta / \delta$$

$$P = \sum Mp * 1/L$$ Note: L for all girders is the same

$$P * L = \sum Mp$$

$$P * L = N * Mp$$ N = # plastic hinges (●)

$$Mp = P * L / N$$

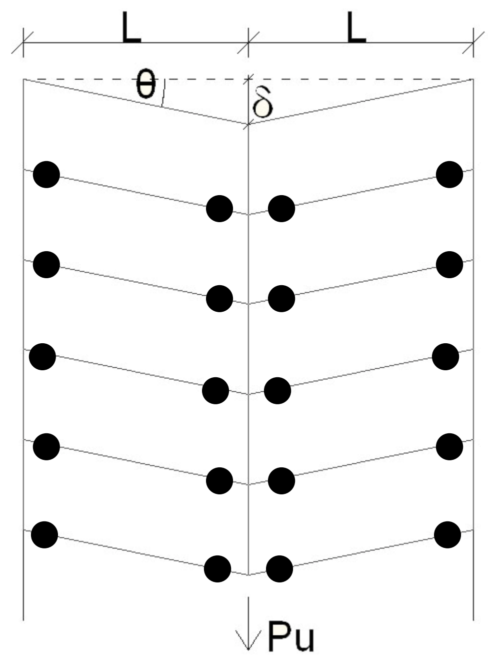

Figure 15: Virtual Work

For the girder size estimation, the RAM model was loaded for a critical column failure, as described above. However, the column was not removed. Instead, the RAM analysis is run in order to determine the demand loads on the column; in particular, the axial load is desired. This axial load is the "P" value in the above equation. With the length known at thirty feet and "N" equaling two plastic hinges per beam, the required Mp value can be determined. A W-shape with the necessary plastic section modulus is chosen.

PROGRESSIVE COLLAPSE

In order for the girder size estimation to be valid, the bending capacity of both the connection and the column must be greater than that of the girder; this will ensure that the plastic hinge forms in the girder and not the corresponding connection or column. In order to ensure strong column – weak beam attributes:

$$\sum M_{pc} / \sum M_{pb} > 1.0$$

$$\sum M_{pc} = \sum Z_c (F_y - P_u/A_g) \qquad \text{(column)}$$

$$\sum M_{pb} = \sum (1.1\, R_y F_y Z_b + M_{uv}) \qquad \text{(beam)}$$

M_p = plastic moment capacity
Z = plastic section modulus
R_y = ratio of expected to nominal yield strength (assume 1.1)
M_{uv} = additional moment due to shear (assume 15% addition)

In order to ensure that the column has sufficient strength, its plastic capacity is decreased by a portion of the axial load, while the plastic moment capacity of the beam is increased by a number of factors.

The trial steel moment frame system, based on various critical column removals, consists of:

typical gravity beam =	W18x35
typical interior girder =	W21x83
typical exterior girder =	W21x57, W21x62
base interior column =	W14x233
base exterior column =	W14x233, W14x159
base atrium column =	W14x233

PROGRESSIVE COLLAPSE

When comparing the required beam sizes of the plastic progressive collapse analysis to the required beam sizes of the traditional load combination analysis, it is clear that the plastic progressive collapse sizes control the design. As a result, the corresponding column sizes are also utilized. If the beams required by the original LRFD design were larger than those necessary in the progressive collapse analysis, an additional strong column – weak beam check would have been needed; larger beams would necessitate larger columns to ensure that plastic hinges form in the beams and not the columns.

Next, these plastic design sizes were to be confirmed using computer software. Unfortunately, this design could not be checked in RAM or ETABS as intended. Plastic behavior was not able to be modeled correctly. Therefore, separate purely elastic design and analysis were performed for progressive collapse loading utilizing the computer software RAM Structural System.

As with the plastic design, the following load combinations were employed:

2.0 [(0.9 or 1.2)D + (0.5L or 0.2S)] + 0.2W	for immediately affected bays
(0.9 or 1.2)D + (0.5L or 0.2S) + 0.2W	for all other bays (Figure 16, 17)

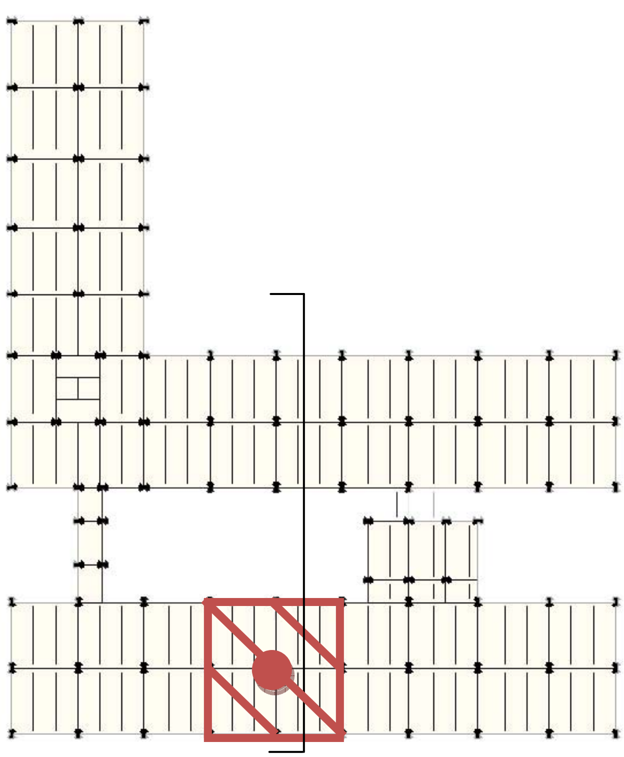

Figure 16: Critical Loading Plan

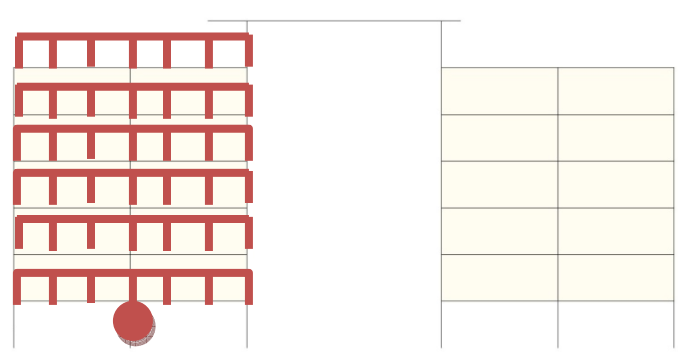

Figure 17: Critical Loading Section

PROGRESSIVE COLLAPSE

The steel moment frame was reanalyzed for the same critical columns as in the plastic analysis. Due to the nature of the program, RAM did not allow for the simple removal and analysis of a column. In order to achieve the effect of a column that had been "blasted-out", a one half inch standard pipe, with an axial load capacity of only 200lb, was placed at the desired locations. The minimal axial capacity of the pipe resulted in an immediately failed base column (Figure 18). This also allowed the program to perform the analysis without any errors.

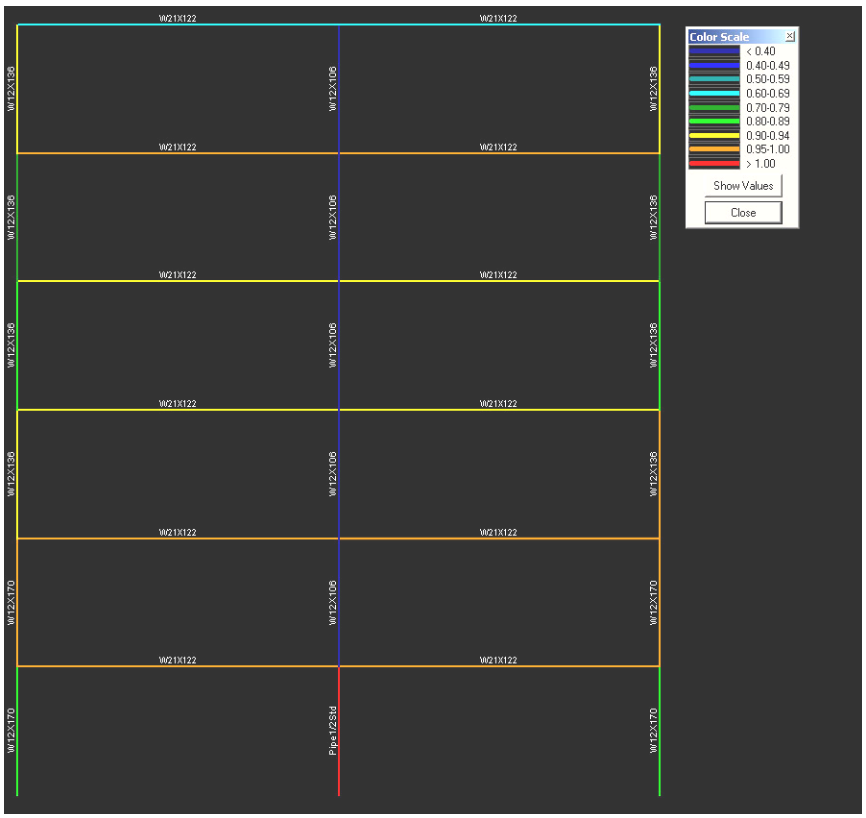

Figure 18: Ram Failed ½" STD Pipe (red column at base)

As a result of the RAM analysis being performed under elastic assumptions and because the loading concerns life safety and not service conditions, deflection was not considered when performing the elastic design.

PROGRESSIVE COLLAPSE

After many iterations of multiple critical column choice removals, the elastic progressive collapse resulted in controlling member sizes of:

typical gravity beam =	W18x35
typical interior girder =	W21x122
typical exterior girder =	W21x83, W21x73
base interior column =	W12x190
base exterior column =	W12x170
base atrium column =	W14x176

When comparing the required sizes for the elastic progressive collapse analysis to the required sizes for the traditional load combination analysis, the progressive collapse loading conditions controlled the design of the steel moment frame.

In both the plastic and elastic progressive collapse analysis and design, the progressive collapse loading controlled the design over the traditional loading. This makes sense due to the extreme nature of the event and loading. The dynamic effect of the loading essentially doubles the gravity loads acting on an already weakened section of the structure. In any building, this degree of loading will most likely control the design.

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ENCLOSURE BREADTH: BLAST GLAZING

When considering the effects of a potential blast or explosion, other building components besides just the structural system need to be considered. In particular, the central glass curtain wall system, enclosing the atrium, is the visible gateway into CDER2 and could be a potential target for a terrorist attack. In the Oklahoma City Federal Building bombing, broken glass accounted for a majority of the sustained injuries. Therefore, the glass curtain wall and punch windows of CDER2 will be designed to resist a blast load and to minimize glass related injuries. Changing the glazing system could result in other design modifications.

In order to determine the necessary glass size to resist a blast, ASTM F 2248-03 was utilized. The guide provides a procedure to convert a TNT charge mass, at a known standoff distance, into an equivalent three second equivalent design pressure. Then using ASTM E 1300-04, a laminated glass unit is designed which has a load resistance greater than the equivalent blast load.

For CDER2, the glass will be insulating glass and both lites will be laminated glass of the same thickness. Due to the nature of the loading, this design will not employ a "sacrificial lite", as frequently employed in high wind loading locations. For this blast loading design, both lites will be assumed to fracture, not just the exterior lite. Therefore, using two equivalent lites, which can equally share the loading, will produce the most efficient design.

In order to determine an equivalent three second blast design pressure, the standoff distance and charge size, in equivalent TNT pounds, are required. According to the RTKL Campus Security Site Plan, the given standoff distance is seventy five feet. Without a detailed site plan to analyze, this given standoff distance was used.

While there is not a defined method for establishing the charge size, the United States Department of Transportation provides a basic guide (Figure 19).

ENCLOSURE BREADTH: BLAST GLAZING

Device	Description	Charge Weight (TNT Equiv. lbs)
	Pipe Bomb	5
	Suitcase	50
	Compact Sedan	220
	Full Size Sedan	500
	Passenger / Cargo Van	1,000
	Box Truck	4,000
	Semi-Trailer	40,000

Figure 19: Charge Size Guide

Using the guide as well as the basic site information, a charge size of two hundred pounds of TNT, or approximately a small vehicle, is assumed for the design.

Entering the chart provided in ASTM F 224-03, a three second equivalent design pressure of 6.3 kPa is determined.

When considering the existing atrium's curtain wall system, the largest opening will be the most critical for the design pressures. Base on the current mullion layout, the largest glass opening will be 2.25m x 1.5m

The punch windows, located regularly at each office, have very consistent dimensions of 1.28m x 1.46m.

Before entering the glass nonfactored load charts within ASTM E 1300-04, the glass type needs to be considered. In addition to providing laminated lites, heat strengthened glass is a chosen for a few reasons. First, it is considerably stronger than regular annealed glass; therefore, less glass thickness/material will be required. Also, the heat strengthened glass provides a straighter, cleaner look than fully tempered – which can be less attractive due to the roller waves on the glass surface.

The load resistance of the glass is a function of a few variables:

LR = 2 x 1.8 x NFL

2 – due to IGU with two equivalent lites; load sharing occurs
GTF – glass type factor = 1.8 for heat strengthened glass
NFL – nonfactored load

The nonfactored load is acquired from charts within ASTM E 1300-04; the NFL is a function of the glass dimensions and thickness. Various charts exist for different

ENCLOSURE BREADTH: BLAST GLAZING

thicknesses of annealed and laminated glass (Figure 20). For this design, it is assumed that all four sides of the glass are supported by the mullions.

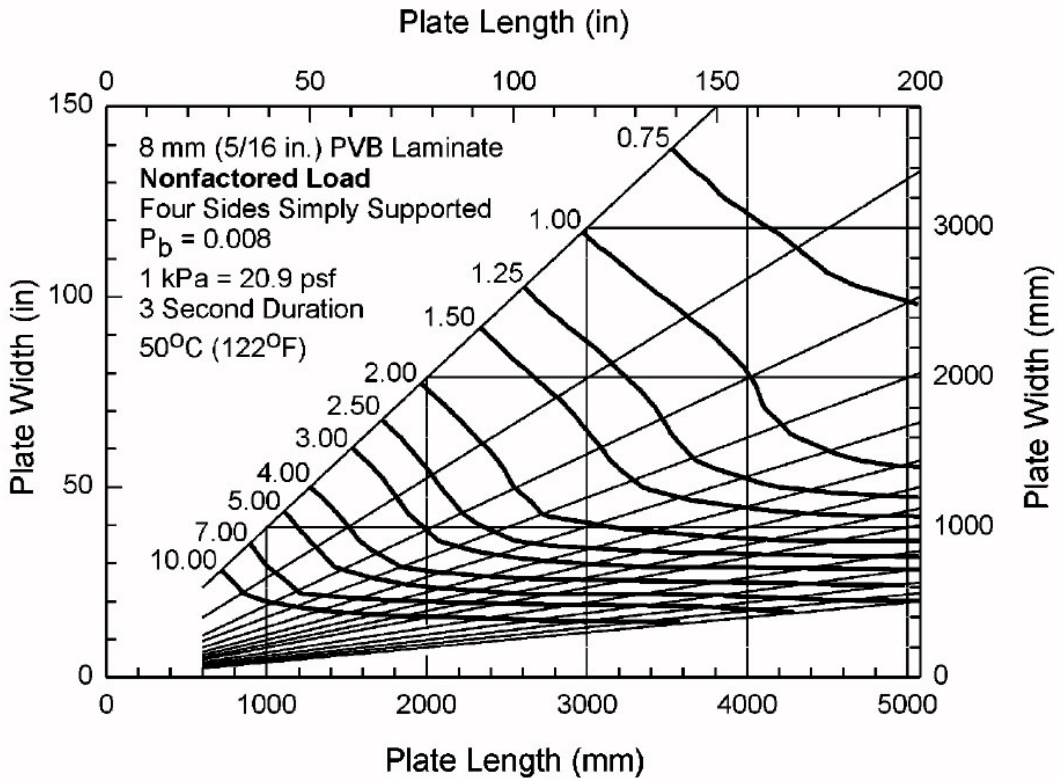


Figure 20: Sample NFL Chart

After multiple glass thickness trials, the atrium's curtain wall was designed as (2) 3/16" heat strengthened, laminated insulating glass unit; its load resistance was determined to be 7.56 kPa, which – at a seventy five foot standoff distance – can withstand an approximate 250 lb TNT equivalent charge.

The typical office punch windows actually met the required load at a thickness of 3/16"; although the ASTM E 2248-03 method was employed, the UFC (DoD 2003) prescribes the use of laminated glass of minimum thickness 1/4". Therefore conservatively, the office windows were designed as (2) 1/4" heat strengthened, laminated insulating glass units. The load resistance was determined to be 9.0 kPa, which is able to withstand an equivalent three hundred pound TNT charge at seventy five feet.

ENCLOSURE BREADTH: BLAST GLAZING

The thermal resistances of the blast-resistant glass units were also evaluated. By comparing the conductance of the IGU components, the overall R-value and system conductance were determined (Figure 21, 22). "Building Science for Building Enclosures", Straube & Burnett, 2003, was referenced for material conductivity values. Additionally by calculating the temperature difference and the area of the unit, as well as the entire glass system, the heat change was calculated (Figure 23, 24).

MATERIAL	THICKNESS (m)	CONDUCTIVITY (W/m-K)	CONDUCTANCE (W/m^2-K)	RESISTANCE	
air film	n/a	n/a	23	0.043	Assume: Emissivity = 0.9
glass	0.008	0.8	100	0.010	
air space	0.02	n/a	1.75	0.571	Assume: Emissivity = 0.05
glass	0.008	0.8	100	0.010	
air film	n/a	n/a	8.3	0.120	
			Total =	0.755	
			U =	1.324	(W/m^2-K)

Figure 21: Curtain Wall Conductance

MATERIAL	THICKNESS (m)	CONDUCTIVITY (W/m-K)	CONDUCTANCE (W/m^2-K)	RESISTANCE	
air film	n/a	n/a	23	0.043	Assume: Emissivity = 0.9
glass	0.006	0.8	133	0.008	
air space	0.02	n/a	1.75	0.571	Assume: Emissivity = 0.05
glass	0.006	0.8	133	0.008	
air film	n/a	n/a	8.3	0.120	
			Total =	0.750	
			U =	1.333	(W/m^2-K)

Figure 22: Office Window Conductance

ENCLOSURE BREADTH: BLAST GLAZING

In order to perform heat change calculations on the glass systems, the temperature difference as well as surface area needed to be calculated. In order to establish design condition temperatures, the Quirouette Building Science Sofware – HAM Toolbox was utilized. For Washington DC, which is very close to the CDER2 site of White Oak, MD, the design conditions are:

Summer		Winter	
Tout =	35 ^{0}C	Tout =	-9.4 ^{0}C
Tin =	23.9 ^{0}C	Tin =	21.1 ^{0}C

HEAT CHANGE Q

AREA (m^2)	ΔT (K)	R (m^2-K/W)	Q (W)	
3.375	11.1	0.755	28	per unit
182	11.1	0.755	2673	system

Figure 23: Curtain Wall Summer Heat Change

HEAT CHANGE Q

AREA (m^2)	ΔT (K)	R (m^2-K/W)	Q (W)	
1.87	11.1	0.750	28	per unit
956	11.1	0.750	14134	system

Figure 24: Office Window Summer Heat Change

~end of section~

CONCLUSION

This thesis report focused on the structural redesign of Center for Drug Evaluation and Research Office Building 2 (CDER2). While originally employing a two-way flat plate, moment frame structural system, this thesis explored the implementation of a composite metal deck, steel moment frame structural system. After determining the required metal deck and lightweight concrete slab using the Vulcraft design guide, RAM Structural System was utilized in the design of the composite gravity beams, as well as the steel moment frames.

Additional loading scenarios were explored as the progressive collapse effects of a blasted column were explored. Critical columns to be removed, plus the necessary load combinations, for an alternative path, static analysis were determined through referencing various design guides and standards. Two designs were performed, both of which considered identical loading and column removal cases. The first design was based on virtual work findings that considered the plastic behavior of the moment frame girders. These minimal design member sizes were then compared against those determined in the initial analysis, which considered traditional loading. The second progressive collapse analysis employed RAM for a completely elastic design. Again the required minimum member sizes were compared against those determined in the initial analysis. These two analyses resulted in two different designs to inhibit progressive collapse.

In learning from the unfortunate experiences of past disasters, it has become clear that other design concerns need to be addressed regarding terrorism attacks besides the structural implications alone. For many buildings, glass and flying debris were the primary cause for the majority of the injuries. As a result, the atrium curtain wall and office window were both designed to resist blast loading. The conductance and heat transfer properties of these new units were then analyzed for summer and winter design conditions.

MAE DISCUSSION

I did my best to incorporate much of my MAE coursework into this thesis. While working on my structural depth, I employed the computer program RAM Structural System. While learning and implementing this new program, I used the knowledge that I had required in AE 597a. While the program may not have been the same as used in the class, the same underlying principles are the same. Dr. Lepage once told us to respond "yes" if we are asked if we know a particular structural program; because in AE 597a, we learned the core components of structural computer modeling and any nuances that vary from program to program could be learned in a short time.

Additional AE 597a coursework that was implemented in this thesis was actually the virtual work method for estimating the necessary plastic beam size. In the initial weeks of the course, we spent a lot of time on virtual work problems and the development of plastic hinges.

Although no particular design methods were discussed in AE 534, we did discuss multiple case studies of progressive collapse. In one class, we even had a guest speaker come in and discuss the partial collapse that occurred on his project. Many of the conceptual ideas that I developed for hindering progressive collapse stemmed from discussions in Building Failures.

It was also from AE 534 lectures on previous failures, explosion and other human related failures, that led me to pursue the blast resistance glazing breadth. It was mention quite a few times in class that many injuries in building failures are not structurally related, but are rather related to flying debris.

While the enclosures breadth idea stemmed from AE 534, the knowledge about the glass design process, as well as the corresponding thermal analysis was all acquired in AE 542, Building Enclosures. Both design methods and calculations were derived from course work.

ACKNOWLEDGEMENTS

I would like to thank the following firms, professionals, professors, and individuals for their support throughout the year with this thesis:

RTKL Associates Inc.

Mr. Peter Malmquist

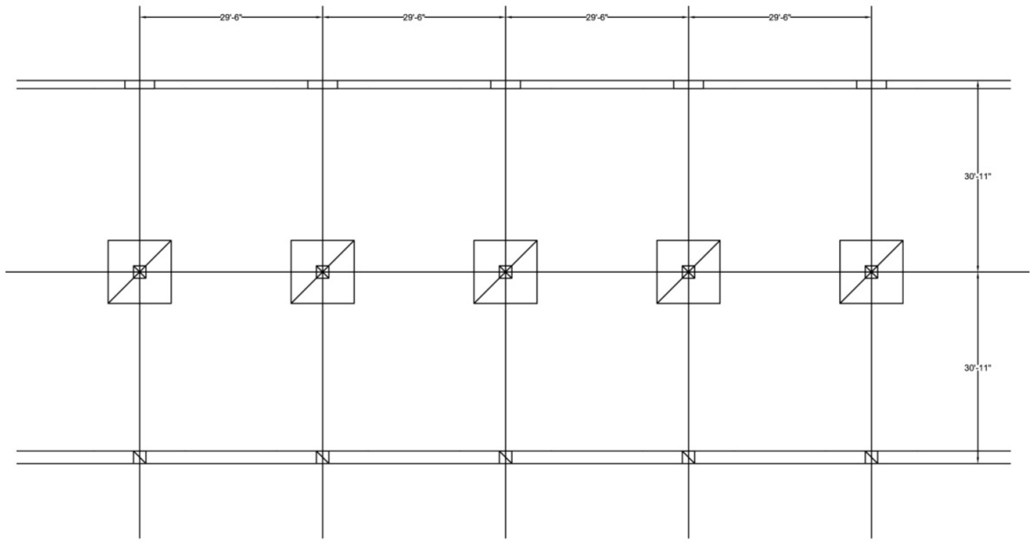
Prof. M. Kevin Parfitt
Prof. Robert Holland
Dr. Louis Geschwinder
All AE structural faculty

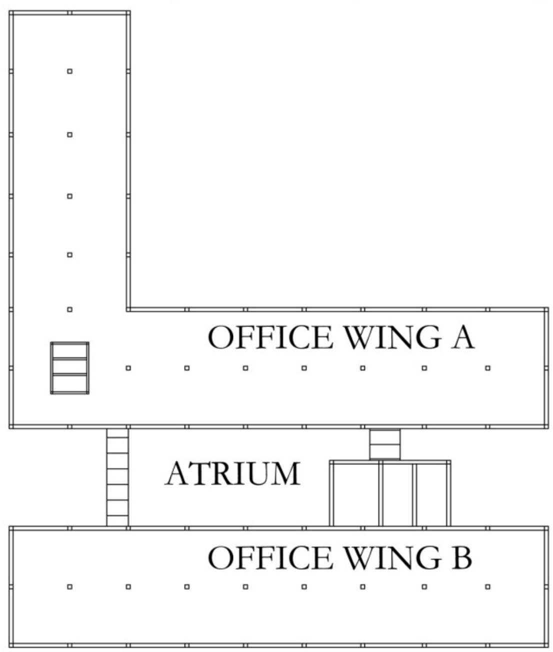
Thank you to all of my AE friends who were always willing to help and answer questions.

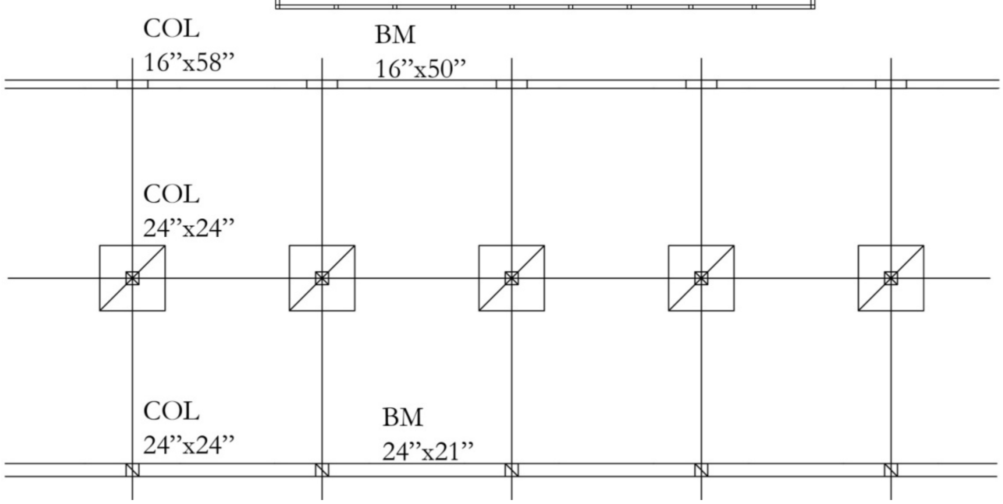
To my roommates, thank you for being patient with me throughout the entire year.

And to my family, thank you so much for always being there to support me through all of the highs and lows that this year has brought.

APPENDIX A: EXISTING SYSTEM



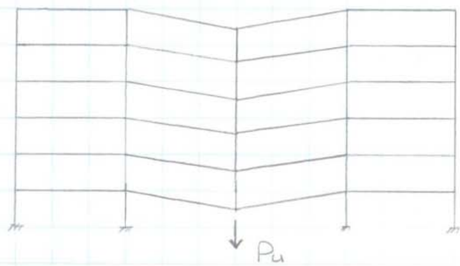


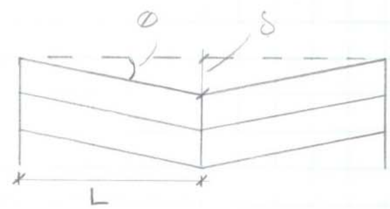


APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR
AE THESIS
COLLAPSE: LONG SIDE MIDDLE

BEAM ESTIMATION:





FROM VIRTUAL WORK:

$P\delta = \sum M_p \varnothing$ $\qquad \delta = L\varnothing \Rightarrow L = \frac{\delta}{\varnothing}$

$P = \sum M_p \, \varnothing/\delta$

$P = \sum M_p \, 1/L$ $\qquad L = 30' = $ CONSTANT FOR ALL BEAMS

$PL = \sum M_p$

$PL = N M_p$ $\qquad N =$ NUMBER OF PLASTIC HINGES ASSUMING ALL BEAMS SAME SIZE

$M_p = \frac{PL}{N}$

FROM RAM: $P_u = 607^k$

$N = 6$ PH/floor × 6 floors = 36 PH

$M_p = \frac{(607)(30)}{36} = 509$ k-ft / beam

$\frac{F_y Z}{12} = 509$

$Z \geq 122 \text{ in}^3$ $\qquad$ W21 × 57 $\quad Z_x = 129 \text{ in}^3$

$\varnothing M_n = 484$ k-ft

TRY W21 × 57 BEAMS

APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR
AE THESIS
COLLAPSE: LONG SIDE MIDDLE

STRONG COLUMN, WEAK BEAM: BEAM = W 21 x 57
$Z_x = 129 \text{ in}^3$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} \geq 1.0$$

$$\Sigma M_{pc} = \Sigma \left[Z_c \left(F_y - P_u / A_g \right) \right]$$

$$= 2\left[Z_c \left(50 - \frac{506}{A_g} \right) \right]$$

NOTE: CRITICAL COLUMN $P_u = 506^k$

$$\Sigma M_{pb} = \Sigma \left[1.1 R_y F_y Z_b + M_{uv} \right]$$

R_y = RATIO OF EXPECTED TO NOMINAL YIELD STRENGTH
= 1.1 ASSUMPTION

M_{uv} = ADDITIONAL MOMENT DUE TO SHEAR
= 15 % ADDITIONAL MOMENT

$$= 2\left[1.1(1.1)(50)(129)(1.15) \right]$$
$$= 17950$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} \geq 1.0$$

$$\frac{2\left[Z_c \left(50 - \frac{506}{A_g} \right) \right]}{17950} \geq 1.0$$

$$\frac{Z_c \left(50 - \frac{506}{A_g} \right)}{8975} \geq 1.0$$

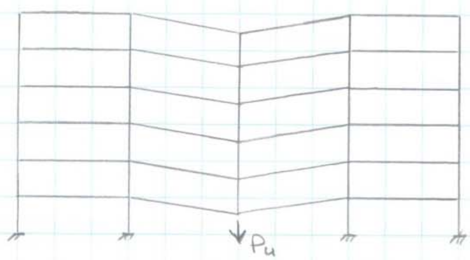
FOR WEAK AXIS TRY:

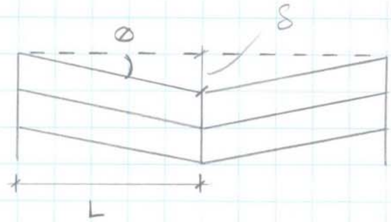
W 12 x 152	$A_g = 44.7 \text{ in}^2$	$Z_y = 111 \text{ in}^3$	0.48 < 1.0
W 12 x 190	$A_g = 55.6 \text{ in}^2$	$Z_y = 143 \text{ in}^3$	0.65 < 1.0
W 14 x 193	$A_g = 56.8 \text{ in}^2$	$Z_y = 180 \text{ in}^3$	0.82 < 1.0
W 14 x 223	$A_g = 68.5 \text{ in}^2$	$Z_y = 221 \text{ in}^3$	1.05 > 1.0 ✓ OK

APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR
AE THESIS
COLLAPSE : INTERIOR

BEAM ESTIMATION :





FROM VIRTUAL WORK :

$P\delta = \sum M_p \, \varnothing$ $\qquad \delta = L\varnothing \Rightarrow L = \frac{\delta}{\varnothing}$

$P = \sum M_p \, \varnothing/\delta$

$P = \sum M_p \, 1/L$ $\qquad L = 30'$ = CONSTANT FOR ALL BEAMS

$PL = \sum M_p$

$PL = N M_p$ $\qquad N$ = NUMBER OF PLASTIC HINGES ASSUMING ALL BEAMS SAME SIZE

$M_p = \frac{PL}{N}$

FROM RAM : $P_u =$

$N = 8$ PH/floor × 6 floors = 48 PH

$M_p = \frac{(1218)(30)}{48} = 761$ k-ft / beam

$\frac{F_y Z}{12} = 761$

$Z \geq 183 \text{ in}^3$ $\qquad$ W21x83 $\quad Z_x = 196 \text{ in}^3$

$\varnothing M_n = 735$ k-ft

TRY W21x83 BEAMS

APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR
AE THESIS
COLLAPSE : INTERIOR

STRONG COLUMN, WEAK BEAM :

BEAM = W 21 x 83
$Z_x = 196 \text{ in}^3$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1.0$$

$$\Sigma M_{pc} = \Sigma\left[Z_c\left(F_y - P_u/A_g\right)\right]$$
$$= 2\left[Z_c\left(50 - 1010/A_g\right)\right]$$

$$\Sigma M_{pb} = \Sigma\left[1.1\, R_y F_y Z_b + M_{uv}\right]$$

R_y = RATIO OF EXPECTED TO NOMINAL YIELD STRENGTH
= 1.1 ASSUMPTION

M_{uv} = ADDITIONAL MOMENT DUE TO SHEAR
= 15% ADDITIONAL MOMENT ASSUMPTION

$$= 2\left[1.1(1.1)(50)(196)(1.15)\right]$$
$$= 27273$$

$$\frac{\Sigma M_{pc}}{\Sigma M_{pb}} > 1.0$$

$$\frac{2\left[Z_c\left(50 - 1010/A_g\right)\right]}{27273} > 1.0$$

$$\frac{Z_c\left(50 - 1010/A_g\right)}{13637} > 1.0$$

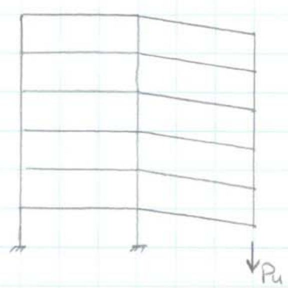
FOR WEAK AXIS TRY :

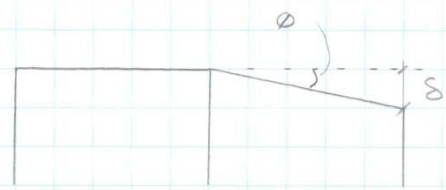
W 14 x 211	$A_g = 62.0 \text{ in}^2$	$Z_y = 199 \text{ in}^3$	0.98 < 1.0
W 14 x 233	$A_g = 68.5 \text{ in}^2$	$Z_y = 221 \text{ in}^3$	1.14 > 1.0 ∴ OK

APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR
AE THESIS
COLLAPSE: CORNER

BEAM ESTIMATION:





FROM VIRTUAL WORK:

$M_p = \frac{PL}{N}$ NOTE: SEE SHORT-SIDE-MIDDLE FOR DERIVATION

FROM RAM: $P_u =$

$N =$ NUMBER OF PLASTIC HINGES

$= \left(4 \frac{PH}{story}\right)(6 \text{ storys}) = 24$ plastic hinges

$M_p = \frac{(346)(30)}{24}$

$M_p = 433$ k-ft / beam

$\frac{F_y Z}{12} = 433$

$Z \geq 103 \text{ in}^3$ W 21 × 50 $Z_x = 110 \text{ in}^3$

$\phi M_n = 413$ k-ft

TRY W 21 × 50 BEAMS

APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR
AE THESIS
COLLAPSE: CORNER

STRONG COLUMN, WEAK BEAM: BEAM = W 21 x 50
$Z_x = 110 \text{ in}^3$

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.0$$

$$\sum M_{pc} = \sum\left[Z_c\left(F_y - \frac{P_u}{A_g}\right)\right] = 2\left[Z_c\left(50 - \frac{346}{A_g}\right)\right]$$

$$\sum M_{pb} = \sum\left[1.1\, R_y F_y Z_b + M_{uv}\right]$$

R_y = RATIO OF EXPECTED TO NOMINAL YIELD STRENGTH
= 1.1 ASSUMPTION

M_{uv} = ADDITIONAL MOMENT DUE TO SHEAR
= 15% ADDITIONAL MOMENT ASSUMPTION

$$\sum M_{pb} = 1\left[1.1(1.1)(50)(110)(1.15)\right] = 7653$$

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.0$$

$$\frac{2\left[Z_c\left(50 - \frac{346}{A_g}\right)\right]}{7653} \geq 1.0$$

$$\frac{Z_c\left(50 - \frac{346}{A_g}\right)}{3827} \geq 1.0$$

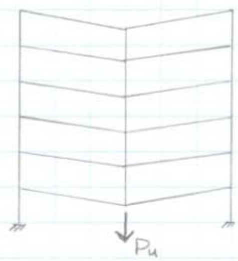
TRY: FOR WEAK AXIS

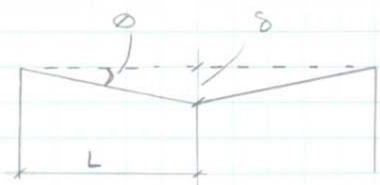
W 12 x 170	$A_g = 50 \text{ in}^2$	$Z_y = 126 \text{ in}^3$	1.42 > 1.0	∴ OK
W 12 x 152	$A_g = 44.7 \text{ in}^2$	$Z_y = 111 \text{ in}^3$	1.22 > 1.0	∴ OK
W 12 x 136	$A_g = 39.9 \text{ in}^2$	$Z_y = 98 \text{ in}^3$	1.06 > 1.0	∴ OK
W 14 x 132	$A_g = 38.8 \text{ in}^2$	$Z_y = 113 \text{ in}^3$	1.06 > 1.0	∴ OK
W 14 x 120	$A_g = 35.3 \text{ in}^2$	$Z_y = 102 \text{ in}^3$	0.92 < 1.0	

APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR
AE THESIS
COLLAPSE : SHORT SIDE MIDDLE

BEAM ESTIMATION:





FROM VIRTUAL WORK:

$$P\delta = \Sigma M_p \phi$$
$$P = \Sigma M_p \frac{\phi}{\delta}$$
$$P = \Sigma M_p \frac{1}{L}$$

$\delta = L\phi \Rightarrow L = \frac{\delta}{\phi}$

$L = 30'$ = CONSTANT FOR ALL BEAMS

$$PL = \Sigma M_p$$
$$PL = N M_p$$
$$M_p = \frac{PL}{N}$$

N = NUMBER OF PLASTIC HINGES ASSUMING ALL BEAMS SAME SIZ

FROM RAM: $P_u = 665$ K
$N = 6 \frac{PH}{floor} \times 6$ floors $= 36$ PH

$$M_p = \frac{(665)(30)}{36}$$
$$M_p = 554 \text{ K-ft / beam}$$
$$\frac{F_y Z}{12} = 554$$
$$Z \geq 133 \text{ in}^3$$

W 21 x 62 $Z_x = 144$ in^3
$\phi M_n = 540$ K-ft

TRY W 21 x 62 BEAMS

APPENDIX B: STRUCTURAL DEPTH

MICHAEL SPEAR 1
AE THESIS
COLLAPSE : SHORT SIDE MIDDLE

STRONG COLUMN, WEAK BEAM : BEAM = W 21 x 62
$Z_x = 144 \text{ in}^3$

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.0$$

$$\sum M_{pc} = \sum \left[Z_c \left(F_y - \frac{P_u}{A_g} \right) \right]$$

$$\sum M_{pb} = \sum \left[1.1 R_y F_y Z_b + M_{uv} \right]$$

R_y = RATIO OF EXPECTED TO NOMINAL YIELD STRENGTH
= 1.1 ASSUMPTION

M_{uv} = ADDITIONAL MOMENT DUE TO SHEAR
= 15 % ADDITION MOMENT

$$\sum M_{pb} = 2 \left[1.15 (1.1)(1.1)(50)(144) \right]$$
$$= 20{,}038$$

$$\sum M_{pc} = 2 \left[Z_c \left(50 - \frac{P_u}{A_g} \right) \right]$$

NOTE: BASE COLUMN MOST CRITICAL
$P_u = 547^k$

$$\frac{\sum M_{pc}}{\sum M_{pb}} \geq 1.0$$

$$\frac{2 \left[Z_c \left(50 - \frac{547}{A_g} \right) \right]}{20{,}038} \geq 1.0$$

$$\frac{Z_c \left(50 - \frac{547}{A_g} \right)}{10019} \geq 1.0$$

TRY : W 12 x 170	$A_g = 50.0 \text{ in}^2$	$Z_x = 275 \text{ in}^3$	1.07	< 1.0 ✓OK
W 12 x 152	$A_g = 44.7 \text{ in}^2$	$Z_x = 243 \text{ in}^3$	0.91	< 1.0
W 14 x 159	$A_g = 46.7 \text{ in}^2$	$Z_x = 287 \text{ in}^3$	1.10	> 1.0 ✓OK
W 14 x 145	$A_g = 42.7 \text{ in}^2$	$Z_x = 260 \text{ in}^3$	0.97	< 1.0

APPENDIX B: STRUCTURAL DEPTH

$\sum M_{pc} / \sum M_{pb} \geq 1.0$

$\sum M_{pc} = \sum [Z_c (F_y - P_u/A_g)]$ Column Properties

$\sum M_{pb} = \sum [1.1 R_y F_y Z_b + M_{uv}]$ Beam Properties

R_y = ratio of expected to nominal yield strength
= 1.1 assumed value

M_{uv} = additional moment due to shear
= 15% assumed additional moment

	Beam Properties			Column Properties					Interaction
	Size	**Zb**	**∑Mpb**	**Size**	**Ag**	**Zc**	**Pu**	**∑Mpb**	**∑Mpc / ∑Mpb**
RAM Elastic	W21x83	196	13637	W12x152	44.7	243	506	9399	0.69
Output	W21x83	196	13637	W12x170	50	275	506	10967	0.80
	W21x83	196	13637	W12x190	55.8	311	506	12730	0.93
	W21x83	196	13637	W12x210	61.8	348	506	14551	1.07
	W21x93	221	15376	W12x210	61.8	348	506	14551	0.95
	W21x93	221	15376	W12x230	67.7	386	506	16415	1.07
	W21x93	221	15376	W14x211	62	390	506	16317	1.06
long side	W21x57	129	8975	W12x152	44.7	111	506	4293	0.48
middle	W21x57	129	8975	W12x190	55.8	143	506	5853	0.65
	W21x57	129	8975	W14x193	56.8	180	506	7396	0.82
	W21x57	129	8975	W14x223	68.5	221	506	9418	1.05
short side	W21x62	144	10019	W14x159	46.7	287	547	10988	1.10
middle	W21x62	144	10019	W14x145	42.7	260	547	9669	0.97
corner	W21x50	110	7653	W14x132	38.8	113	547	4057	1.06
	W21x50	110	7653	W14x120	35.3	102	547	3519	0.92
middle	W21x83	196	13637	W14x120	35.3	102	1010	2182	0.32
	W21x83	196	13637	W14x159	46.7	146	1010	4142	0.61
	W21x83	196	13637	W14x233	68.5	221	1010	7791	1.14
	W21x83	196	13637	W14x211	62	199	1010	6708	0.98

Strong Column – Weak Beam Calculations

APPENDIX C: ENCLOSURE BREADTH

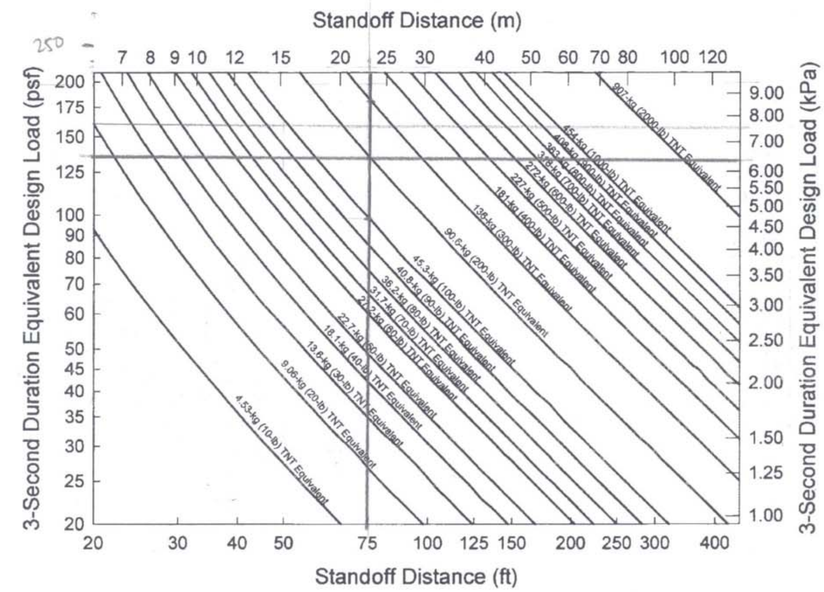


Fig. 3. Chart that relates standoff distance and charge size to equivalent 3-s duration equivalent design loading from ASTM F 2248-03. (Reprinted with permission from ASTM F 2248-03, copyright ASTM International, 100 Barr Harbor Dr., West Conshohocken, PA 19428.)

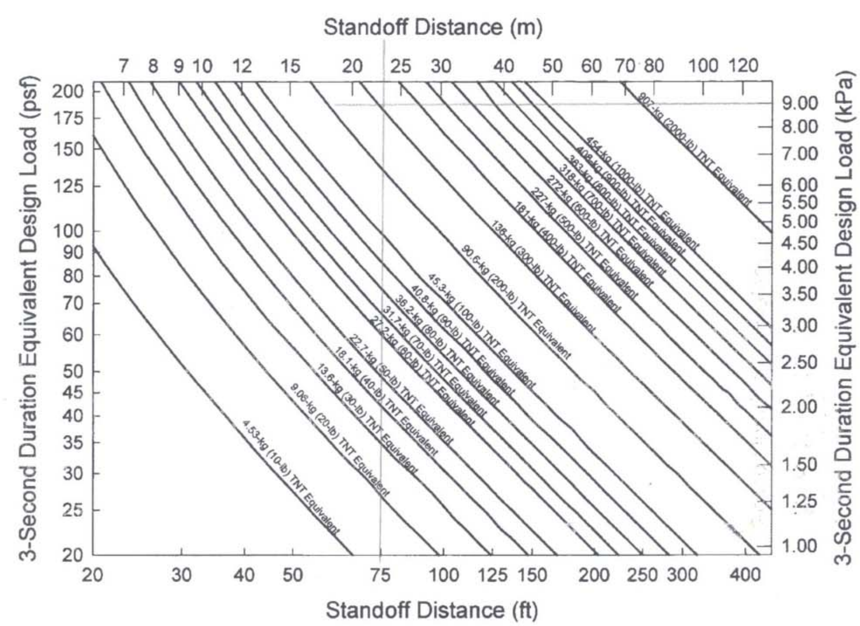


Fig. 3. Chart that relates standoff distance and charge size to equivalent 3-s duration equivalent design loading from ASTM F 2248-03. (Reprinted with permission from ASTM F 2248-03, copyright ASTM International, 100 Barr Harbor Dr., West Conshohocken, PA 19428.)

APPENDIX C: ENCLOSURE BREADTH

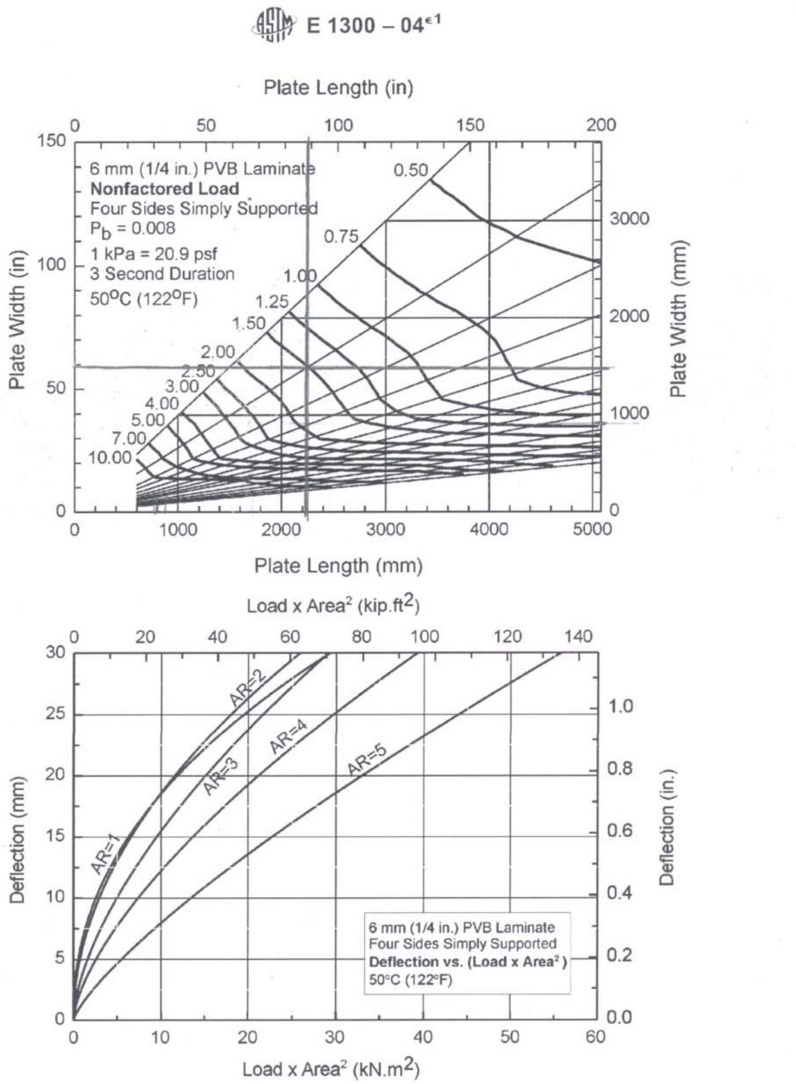


FIG. A1.28 (upper chart) Nonfactored Load Chart for 6.0 mm (¼ in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 6.0 mm (¼ in.) Laminated Glass with Four Sides Simply Supported

Curtain Wall Trial Size

APPENDIX C: ENCLOSURE BREADTH

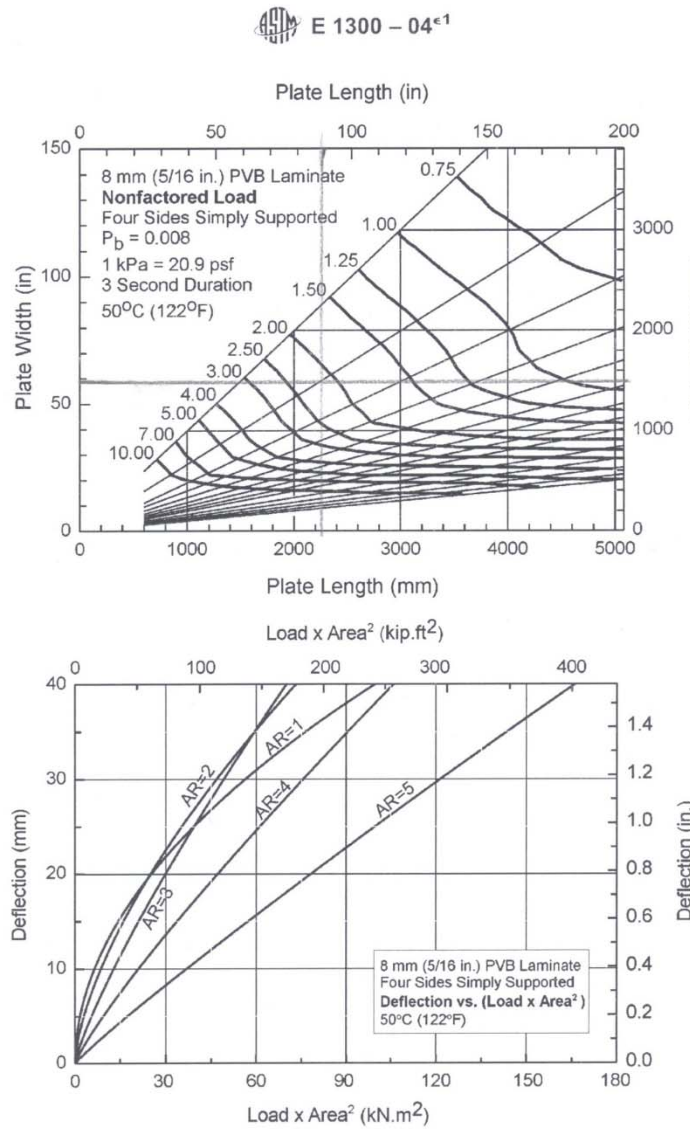


FIG. A1.29 (upper chart) Nonfactored Load Chart for 8.0 mm (5/16 in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 8.0 mm (5/16 in.) Laminated Glass with Four Sides Simply Supported

Curtain Wall Trial Size

APPENDIX C: ENCLOSURE BREADTH

E 1300 – 04ε1

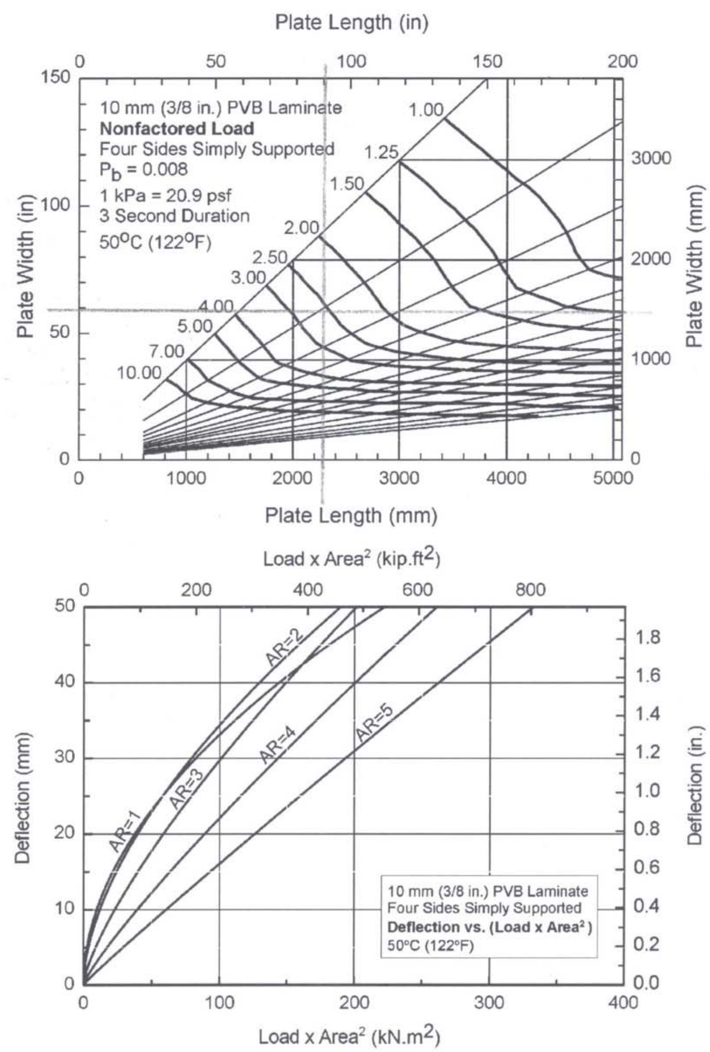


FIG. A1.30 (upper chart) Nonfactored Load Chart for 10.0 mm (3/8 in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 10.0 mm (3/8 in.) Laminated Glass with Four Sides Simply Supported

Curtain Wall Trial Size

APPENDIX C: ENCLOSURE BREADTH

E 1300 – 04$^{\epsilon 1}$

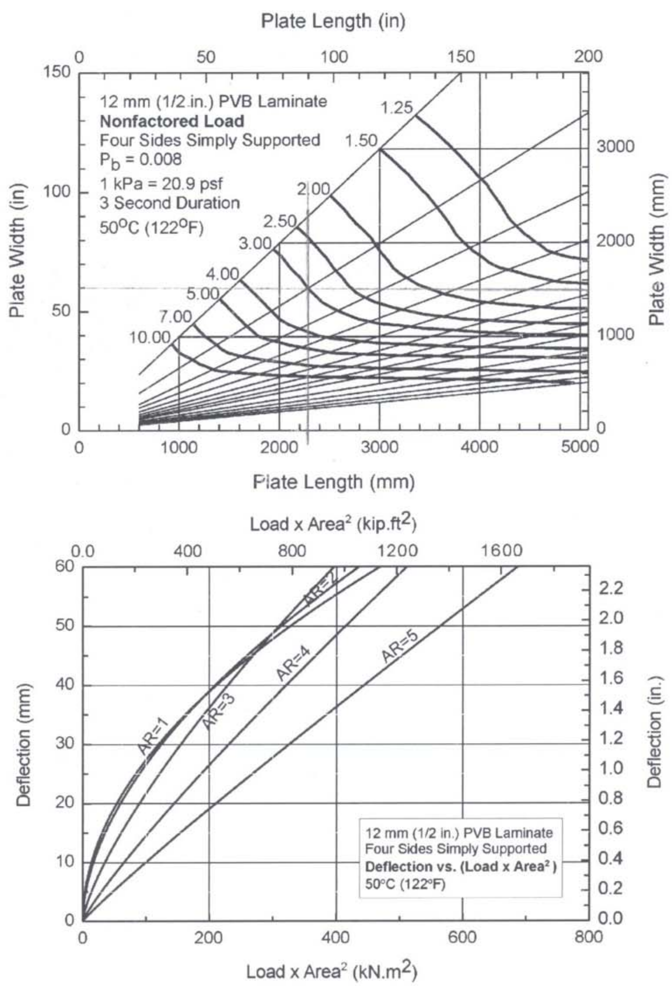


FIG. A1.31 (upper chart) Nonfactored Load Chart for 12.0 mm (½ in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 12.0 mm (½ in.) Laminated Glass with Four Sides Simply Supported

Curtain Wall Trial Size

APPENDIX C: ENCLOSURE BREADTH

E 1300 – 04^ε1

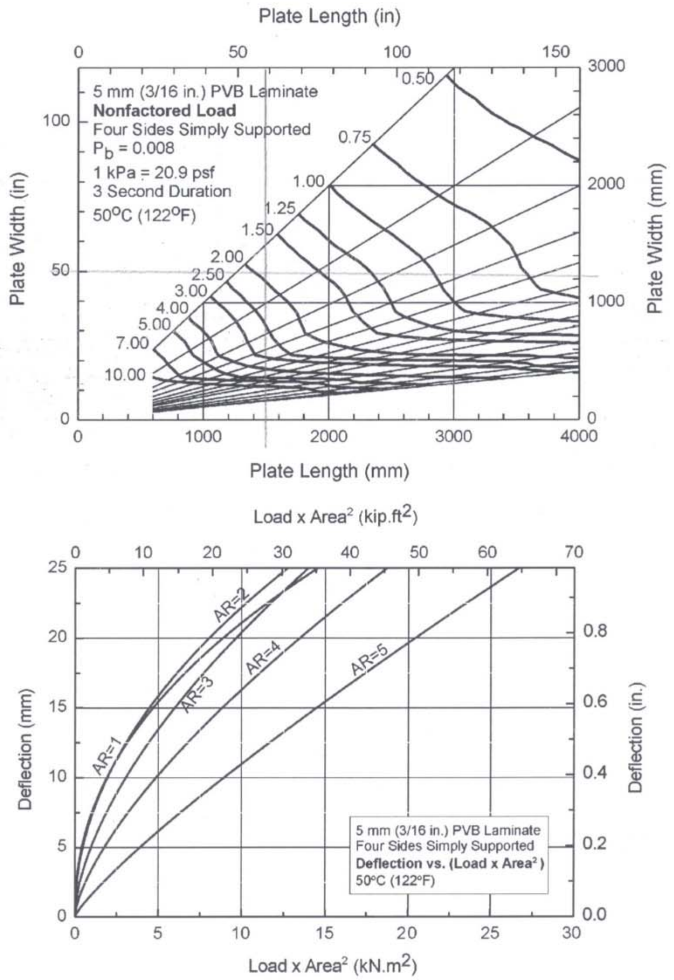


FIG. A1.27 (upper chart) Nonfactored Load Chart for 5.0 mm (3/16 in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 5.0 mm (3/16 in.) Laminated Glass with Four Sides Simply Supported

Office Window Trial Size

APPENDIX C: ENCLOSURE BREADTH

E 1300 – 04[ε1]

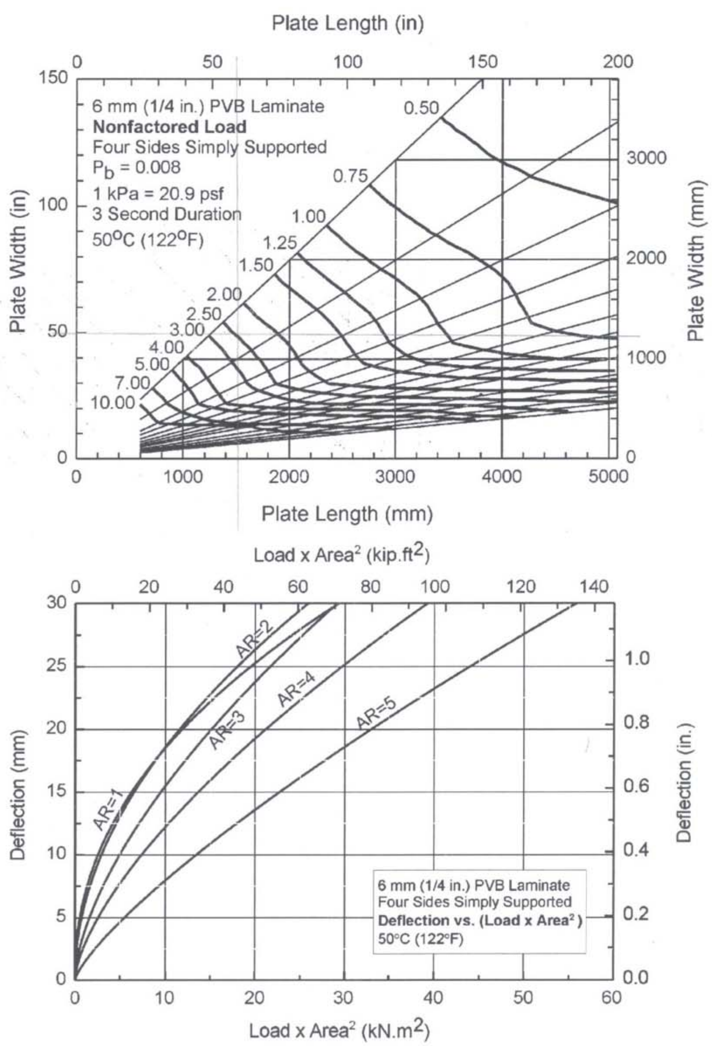


FIG. A1.28 (upper chart) Nonfactored Load Chart for 6.0 mm (¼ in.) Laminated Glass with Four Sides Simply Supported
(lower chart) Deflection Chart for 6.0 mm (¼ in.) Laminated Glass with Four Sides Simply Supported

Office Window Trial Size

APPENDIX C: ENCLOSURE BREADTH

Michael Spear
AE Thesis
Curtain Wall Calculation

Sources employed: ASTM F 2248-03 with ASTM E 1300-04

Glass will be insulating glass and both lites will be laminated glass of the same thickness. Due to the nature of the loading, this design will not employ a "sacrificial lite", as frequently utilized in high wind loading locations. For this blast loading, both lites will be assumed to fracture, not just the exterior lite. Therefore, using two equivalent liteswill produce the most efficient design.

glass size	(m)	(mm)	(ft)	(in)
w=	2.25	2250	7.4	89
h=	1.5	1500	4.9	59

Determination of 3-s Equivalent Design Load

standoff distance (ft) =	75	given by RTKL Campus Security/Site plan
equivalent charge (lb TNT) =	200	approximate small car
p (kpa) =	6.3	ASTM F 2248-03

Heat Strengthened, Laminated Glass

thick (in)	LR (kPa)		
0.5	10.8		
0.375	9.36		
0.3125	7.56	ASTM E 1300-04	thickness optimization
0.25	5.4		

Note: Heat strengthened glass chosen for multiple reasons:
1) considerably stronger than annealed, therefore less glass thickness required
2) "cleaner" look than fully tempered, less roller waves on glass surface

Note: At the chosen size of (2) 3/16" HS, laminated IGU, the unit can withstand an equivalent charge size of 250 (lb TNT).

APPENDIX C: ENCLOSURE BREADTH

Michael Spear
AE Thesis
Office Window Calculation

Sources employed: ASTM F 2248-03 with ASTM E 1300-04

Glass will be insulating glass and both lites will be laminated glass of the same thickness. Due to the nature of the loading, this design will not employ a "sacrificial lite", as frequently utilized in high wind loading locations. For this blast loading, both lites will be assumed to fracture, not just the exterior lite. Therefore, using two equivalent liteswill produce the most efficient design.

glass size	(m)	(mm)	(ft)	(in)
w=	1.278	1278	4.2	50
h=	1.466	1466	4.8	58

Determination of 3-s Equivalent Design Load

standoff distance (ft) =	75	given by RTKL Campus Security/Site plan
equivalent charge (lb TNT) =	200	approximate small car
p (kpa) =	6.3	ASTM F 2248-03

Heat Strengthened, Laminated Glass

thick (in)	LR (kPa)		
0.25	9	ASTM E 1300-04	Note: Although the ASTM E 2248-03 method was employed,
0.1875	6.48		DoD 2003 requires a minimum IGU lite thickness of 1/4".

Note: Heat strengthened glass chosen for multiple reasons:
1) considerably stronger than annealed, therefore less glass thickness required
2) "cleaner" look than fully tempered, less roller waves on glass surface

Note: At the chosen size of (2) 1/4" HS, laminated IGU, the unit can withstand an equivalent charge size of 300 (lb TNT).

APPENDIX C: ENCLOSURE BREADTH

Michael Spear
AE Thesis
Enclosure: Thermal Behavior
Sources: "Building Science for Building Enclosures", Straube & Burnett, 2005
Quirouette Building Science Software - HAM Toolbox
Office Windows
(2) 1/4" HS, laminated IGU

		Washington DC			
SUMMER		Design Conditions			
	Tout =	35	°C		
	Tin =	23.9	°C		
MATERIAL	THICKNESS (m)	CONDUCTIVITY (W/m-K)	CONDUCTANCE (W/m^2-K)	RESISTANCE	
air film	n/a	n/a	23	0.043	Assume: Emissivity = 0.9
glass	0.006	0.8	133	0.008	
air space	0.02	n/a	1.75	0.571	Assume: Emissivity = 0.05
glass	0.006	0.8	133	0.008	
air film	n/a	n/a	8.3	0.120	
			Total =	0.750	
			U =	1.333	(W/m^2-K)
HEAT CHANGE Q					
AREA (m^2)	ΔT (K)	R (m^2-K/W)	Q (W)		
1.87	11.1	0.750	28	per unit	
956	11.1	0.750	14134	system	

WINTER		Design Conditions			
	Tout =	-9.4	°C		
	Tin =	21.1	°C		
MATERIAL	THICKNESS	CONDUCTIVITY (W/m-K)	CONDUCTANCE (W/m^2-K)	RESISTANCE	
air film	n/a	n/a	23	0.043	Assume: Emissivity = 0.9
glass	0.006	0.8	133	0.008	
air space	0.02	n/a	1.59	0.629	Assume: Emissivity = 0.05
glass	0.006	0.8	133	0.008	
air film	n/a	n/a	8.3	0.120	
			Total =	0.808	
			U =	1.238	(W/m^2-K)
HEAT CHANGE Q					
AREA (m^2)	ΔT (K)	R (m^2-K/W)	Q (W)		
1.87	-30.5	0.808	-71	per unit	
956	-30.5	0.808	-36073	system	

APPENDIX C: ENCLOSURE BREADTH

Michael Spear						
AE Thesis						
Enclosure: Thermal Behavior						
Sources:	"Building Science for Building Enclosures", Straube & Burnett, 2005					
	Quirouette Building Science Software - HAM Toolbox					
Atrium Curtain Wall						
(2) 3/16" HS, laminated IGU						
			Washington DC			
	SUMMER		Design Conditions			
		Tout =	35	°C		
		Tin =	23.9	°C		
	MATERIAL	THICKNESS (m)	CONDUCTIVITY (W/m-K)	CONDUCTANCE (W/m^2-K)	RESISTANCE	
	air film	n/a	n/a	23	0.043	Assume: Emissivity = 0.9
	glass	0.008	0.8	100	0.010	
	air space	0.02	n/a	1.75	0.571	Assume: Emissivity = 0.05
	glass	0.008	0.8	100	0.010	
	air film	n/a	n/a	8.3	0.120	
				Total =	0.755	
				U =	1.324	(W/m^2-K)
	HEAT CHANGE Q					
	AREA (m^2)	ΔT (K)	R (m^2-K/W)	Q (W)		
	3.375	11.1	0.755	28	per unit	
	182	11.1	0.755	2673	system	
	WINTER		Design Conditions			
		Tout =	-9.4	°C		
		Tin =	21.1	°C		
	MATERIAL	THICKNESS	CONDUCTIVITY (W/m-K)	CONDUCTANCE (W/m^2-K)	RESISTANCE	
	air film	n/a	n/a	23	0.043	Assume: Emissivity = 0.9
	glass	0.008	0.8	100	0.010	
	air space	0.02	n/a	1.59	0.629	Assume: Emissivity = 0.05
	glass	0.008	0.8	100	0.010	
	air film	n/a	n/a	8.3	0.120	
				Total =	0.813	
				U =	1.230	(W/m^2-K)
	HEAT CHANGE Q					
	AREA (m^2)	ΔT (K)	R (m^2-K/W)	Q (W)		
	3.375	-30.5	0.813	-70	per unit	
	182	-30.5	0.813	-6824	system	