Crossroads at Westfields
Building II

Chantilly, Va

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Structural option

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AE Senior Thesis
Final Report
Spring 2009

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EXECUTIVE SUMMARY

Crossroads at Westfields Building II is a five story office building in Chantilly, Virginia. The project was originally designed in 2005 but the project has been on hold since. On completion of the Technical Reports, the overall design of the building complied with all of the applicable codes however it was concluded that it may not be the most economical solution. After modeling the building, it was found that the moment frames were oversized in the original design and the members could be optimized if the lateral design is altered.

A study was conducted to find the most economical lateral system by comparing three alternative systems to the original. Due to architectural restrictions the structural system was limited to the original system of composite steel framing and the use of moment frames. However, the finalized lateral resisting system was designed with separate Response Modification Factors in each direction and the use of (2) two-story "X" braces combined with moment frames. The original design had four moment frames in each direction. Overall, the new design used 13% less steel than the original design and the overall structure cost just under 21% less. Not only was the amount of steel reduced but the overall cost of the foundations was also reduced with no effect to the schedule. Designing a more economical lateral system in terms of the amount of steel used was the first goal of the depth that was met.

The second part of the depth was to design a portion of the building to mitigate the risk of Progressive Collapse. For the purpose of this thesis Building II will occupy a hypothetical client of government or 'high profile' stature. With the building now being considered 'performance based' or high profile it could be subject to abnormal loading from an explosion or blast from a terrorist attack. Following recommendations from the GSA, the building was analyzed with a linear static approach. Two methods were analyzed and compared; an Indirect method and Direct method. Both methods coupled with the new lateral design proved to be more cost efficient than the original design which was the intended goal.

The architecture breadths are based off a similar premise that Building II is considered a 'high-profile' type of building. The scope of the breadth was to analyze the original site layout and redesign to mitigate the risk of an attack. The one fallback when designing a site to have a hardened perimeter and certain setbacks is the amount of land available. Fortunately for Building II, the site provided enough land for a sufficient setback distance. The only additional costs came from hardening the perimeter with fences, bollards, etc. With the goal to keep the Architecture of the façade untouched, the glazing and precast panel connections were evaluated and redesigned to better resist abnormal blast loadings.

Overall, I would conclude that most of the goals from the proposal were met including redesigning a more cost efficient structure and using the savings to design against progressive collapse.

ACKNOWLEDGEMENTS

- My family for all the support they've provided over the past five years while I've attended Penn State University

- Thesis Advisor and Consultants

    - Dr. Andres LePage, Assistant Professor of Architectural Engineering
    - Prof. M. Kevin Parfitt, Associate Professor of Architectural Engineering
    - Prof. Robert Holland, Associate Professor of AE and Arch.

- Other Faculty Assistance

    - Dr. Louis Geschwinder, Professor Emeritus of Architectural Engineering
    - All of my professors in the Architectural Engineering Department from the past five years

- Outside Assistance

    - Mike Weiss, P.E. of Structura, Inc.
    - Don Bockoven of Structura, Inc.
    - Charlie Angelilli of Structura, Inc.
    - Eric Inman, R.A. of Hickok Cole Architects
    - David Morse, P.E., PhD

- The Alter Group and Structura, Inc. for providing me permission and the drawings for The Crossroads at Westfields Building II as my thesis project.

- My roommates Jamison Morse and David Sivin for keeping me sane throughout the past two semesters.

REFERENCES

Liu, R., Davison, B., and Tyas, A. (2005). “A Study of progressive collapse in multi-storey steel frames”, J. Arch. Eng. Structures 2005, ACSE 171, 1-9.

Baldridge, S.M., and Humay, F.K. (2005). “Multi-Hazard Approach to Progressive Collapse Mitigation”, J. Arch. Eng. Structures 2005, ASCE 171, 1-12.

Gould, N.C., Winn, V., and Drevinsky, D. (2005). “Progressive Collapse Analysis, Retrofit Design, and Costs for Existing Structures”, J. Arch. Eng. Structures 2006, ASCE, 201, 1-10.

Ellingwood, B.R., (2005). “Strategies for Mitigating Risk of Progressive Collapse”, J. Arch. Eng. Structures 2005, ASCE, 171, 1-6.

Shipe, J.A., and Carter, C.J., “Defensive Design: Blast and Progressive Collapse Resistance in Steel Buildings”, J. Arch. Eng. Structures 2004, ASCE, 137, 1-9.

Lew, H.S., “Best Practices Guidelines For Mitigation of Building Progressive Collapse”, < www.pwri.go.jp/eng/ujnr/joint/35/paper/72lew.pdf >

Department of Defense, (2005). “United Facilities Criteria (UFC)”, Dod Minimum Antiterrorism Standards for buildings. < www.tisp.org/files/pdf/dodstandards.pdf >

GSA (2003). “Progressive Collapse Analysis and Design Guidelines for new federal buildings. < www.gsa.gov >

GSA (2007). “The Site Security Design Guide” <http://www.gsa.gov/Portal/gsa/ep/channelView.do?pageTypeId=17109&channelPage=%2Fep%2Fchannel%2FgsaOverview.jsp&channelId=-24284 >

ISC (2005) “Performance Based Design” < gsa.gov >

FEMA (December 2003), “Primer for design of commercial buildings to mitigate terrorist attacks”, < www.fema.gov >

Old Castle Glass (2009). < http://www.oldcastleglass.com/forced_entry_introduction.php#1 >

Midwest Precast (2009). “Designers notebook blast considerations < http://www.midwestprecast.net/ >

SidePlate (2009). < www.sideplate.com >

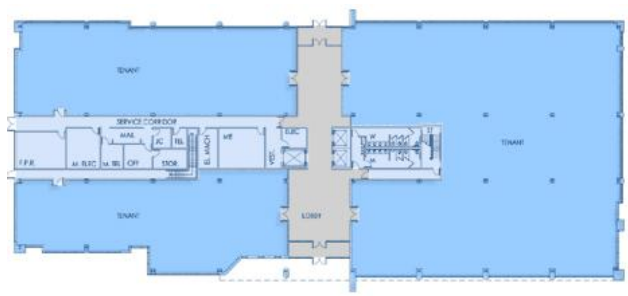
ARCAT (2009). < http://www.arcat.com/details/divs/dd_div02.shtml >

BACKGROUND

SITE AND GENERAL ARCHITECTURE

The Crossroads at Westfields are two identical office buildings mirroring each other on site. Although the project is currently on hold, these two buildings will offer over 300,000 GSF of office space to future tenants. Located in the Westfields Corporate Center of Chantilly, Virginia, the site is at the crossing of the Stonecroft Blvd. and Lee Rd., hence the name. Building II, identical to Building I, is a 5-story office building with floor plans that offer spans of over 41 feet. The long spans in the exterior bays create a large open floor allowing the tenant to easily adapt the space to their needs. The structure consists of composite steel beam framing on each floor and is combined with ordinary moment frames to resist lateral loading. The roof is supported by joists and steel decking that will support future mechanical units.

SITE PLAN



TYPICAL FLOOR PLAN

In prior Technical Reports, the existing design for Building II was analyzed to check several aspects of the buildings structural systems. All of the systems met the applicable code and requirements. This included an analysis of gravity design loads and lateral forces in compliance with ASCE 7-05, an assessment of multiple floor systems comparing cost and ease of constructability, and finally a complete analysis of the buildings lateral system. On completion of these reports, the overall design complied with all of the applicable codes however it was found that it may not be the most cost efficient solution.

INTRODUCTION

This report will conduct a study to find the most economical lateral system by comparing three alternative systems to the original. The architectural design of the façade is a combination of glass and precast panels limiting the lateral system on the exterior of the building to moment frames. The floor plan consists of large open spaces created by the spans of over 41' ft in the exterior bays. Due to these architectural restrictions the structural system is mostly limited to original system of composite steel framing and the use of moment frames. An investigation will be conducted to analyze the architectural plan to distinguish the possibility of using braced frames somewhere in the building.

Located west of Washington DC, Building II is located in the Westfields Corporate Center of Chantilly, Va. For the purpose of this project and the second part of the structural depth, Building II will be considered a 'high-profile' or 'performance-based'. With the building now being considered 'high profile' it may be at risk to abnormal loading from an explosion or blast from a terrorist attack which could potentially lead to progressive collapse. Following recommendations from the GSA and the DoD, Building II will be analyzed to mitigate the risk of progressive collapse.

The architecture breadth is based off the same premise that Building II is considered a 'high-profile' type of building. The scope of the breadth is to analyze the original site layout and redesign to mitigate the risk of an attack. With the goal to keep the Architecture of the façade untouched, the glazing and precast panel connections are going to be evaluated and redesigned to better resist abnormal blast loadings. The final breadth will be a cost analysis of the lateral redesign, both methods of progressive collapse design and the additional costs associated with the site redesign.

EXISTING STRUCTURAL SYSTEMS

FOUNDATION SYSTEMS

The Foundation system consists of reinforced cast-in-place concrete spread footings. According to the Geotechnical report recommendations prepared by ECS, Ltd the allowable soil bearing values vary throughout the site. Foundations bearing on the natural 'weathered rock' soil classification will be designed with an allowable soil bearing of 6000 psf while foundations bearing on engineered fill will be designed for soil bearing of 3000 psf. The concrete strength shall be 3000 psi.

According to recommendations in the Geotechnical Report, the Slab on Grade will bear on the natural soil. The slab is a 4" thick cast-in-place concrete with 6x6–10/10 welded wire mesh (WWM), laid on a 6-mil fiberglass reinforced polyethylene vapor barrier and 4" of washed gravel. Interior SOG will have a compressive strength of 3000 psi, while exterior SOG will have a strength of 4500 psi.

FLOOR SYSTEMS

A typical floor in the Building II consists of 3" 20 gauge composite steel deck with 3-1/4" lightweight concrete slab totaling a total slab thickness of 6-1/4". The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi. The floor is supported by A992 wide flange beams with studs dimensioned at ¾" in diameter and 5 ¼" in length. The beams are spaced at 10' o/c and span 41'-8" in a typical exterior bay and 30'-0" in a typical interior bay, as you can see in Figure 2 below. Depending on the floor, the beams will be cambered from an 1" to 1½" and will vary in size and weight. Typical interior girders are W24-62 spanning 30'-0", while typical exterior girders vary in size and also span 30'-0".

ROOF SYSTEM

As seen in Figure 3, the roof system is comprised of 1-1/2" 22 gauge Type B wide rib galvanized roof deck, on K series bar joists and steel girders. Light-gage framing makes up the 4' parapet and the screen wall encompassing the roof. Precast panels frame into each floor including the roof.

Rooftop Mechanical pads for future tenant equipment shall be constructed similar to the typical floor system consisting of 3" 20 gauge composite steel deck with 3-1/4" lightweight concrete slab totaling a total slab thickness of 6-1/4". The slab shall be reinforced with 6X6-10/10 WWM and have a compressive strength of 3000 psi.

COLUMN SYSTEM

Having a very uniform design layout the column system consists of typical exterior bays of 30'-0" x 41'-8" and interior bays of 30'-0" x 30'-0". All of the columns consist of either a gravity resisting member or a combined lateral and gravity resisting member. Each columns is spliced at 4 feet past the third floor, regardless of its resisting system. All columns vary in size depending on location and load resistance capabilities.

LATERAL SYSTEM

The lateral resisting system for wind and seismic loads consists of a number of structural steel moment frames running in both directions. Lateral loading is transferred from precast panels (connected at each floor) to each individual floor. Once transferred into the floor system, the load is transferred into composite beams which make up the framing and then into the columns. The columns and beams are connected by a moment connection seen in Figure 1. the columns transfer the rest of the load into the foundation.

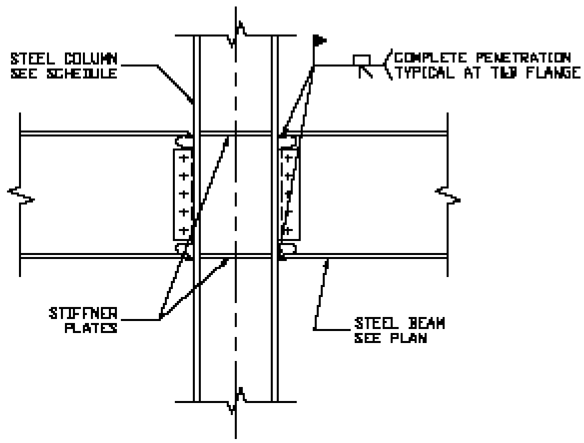


FIGURE 1 – Typical Beam to Column Moment connection

Figure 2 clearly shows the four moment frames positioned in each direction, North-South and East-West, supporting the building laterally. In both directions the moment frames are positioned symmetrically about the center axis. The North-South (Frames 1-4) lateral system is 2 sets of parallel moment frames anchoring each end bay. The East-West (Frames 5-8) lateral system is a set of 2 moment frames on each exterior side of the building. The beam sizes vary.

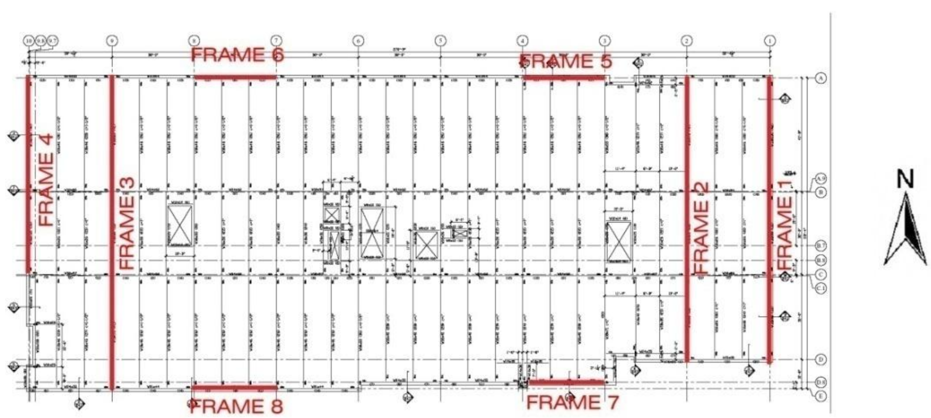


FIGURE 2a – Typical Floor plan with moment frames

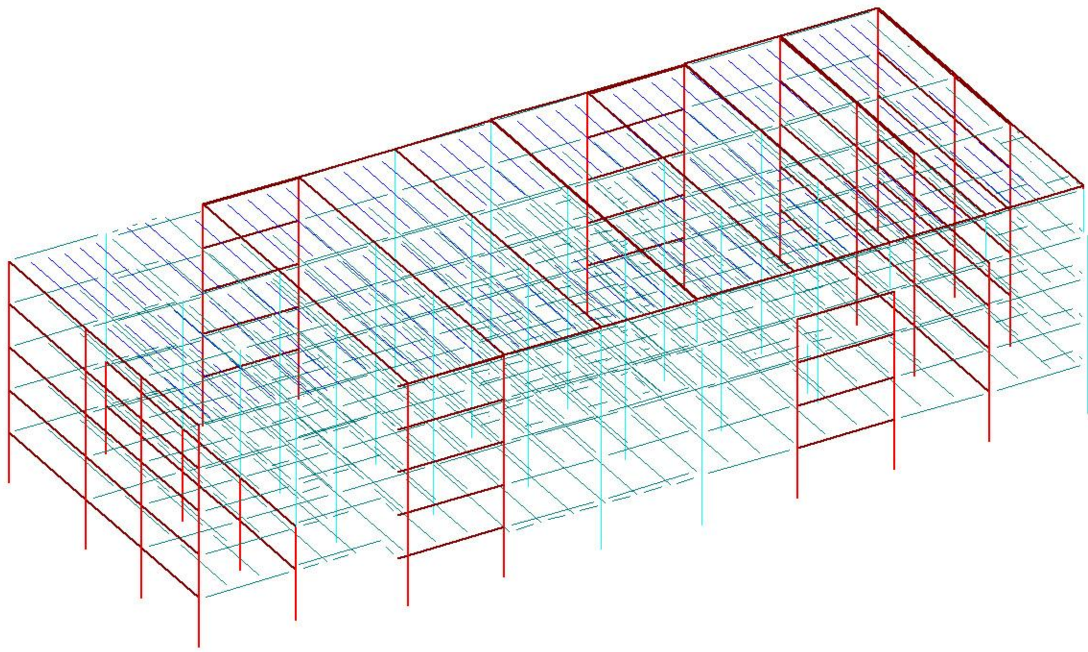


FIGURE 2b – Overall 3D RAM Model with highlighted moment frames

STRUCTURAL DEPTH

LATERAL REDESIGN STUDY

BACKGROUND

After modeling and analyzing the original lateral design in the prior technical reports it was concluded that the members of the moment frames were oversized. As stated in the Background portion, one of the key architectural features of the Building II was the open floor plan created by the long spans of the composite framing. Due to this and the fact that all of the exterior façades are mostly windows, the lateral design was almost exclusively limited to moment frames. However, after a study of the architectural floor plan, I was able to find locations next to two stairways in the building in which I could place two-story "X" braces without affecting the floor plan design. Only the North-South direction (long side) had the ability to add braces without disrupting the floor plan, the East-West (short side) only has three bays and none of which presented the option to add braces.

DESIGN ASPECTS

The design lateral loads seemed a little high under the assumption that the structure was rigid. After investigating the design of the building I found that the original structure was a flexible even though the initial "rule-of-thumb" suggested that the building was "probably" rigid. The rule of thumb states that the if the building's shorter width (115') exceeds four times the building's height (68' x 4 = 272' > 115') the structure is "probably" rigid. This is found in the earthquake design code with a typical preconception towards higher estimates of fundamental frequencies. However, the commentary of ASCE 7-05 states that the natural frequency for wind is $n_1=22.2/H^{.8}$. This approximation was far more accurate to the results of the RAM output, stating that the building was flexible with a period of 2.8 seconds in the north-south direction which wind happened to control. This verified the original design.

Another notable design aspect was the original design used the ASCE 7-02 and IBC 2003 as the governing code which used USGS maps from 2000. These maps gave spectral response accelerations of Ss (0.2 sec) = .183 and S1 (1 sec) = .064, resulting in a Seismic Design Category of B. This requires Equivalent Lateral-Force Analysis. However, for this thesis project ASCE 7-05 was used as the governing code which uses USGS maps from 2003 producing spectral response accelerations of Ss = .158 and S1= .051, resulting in a Seismic Design Category of A. There is no further analysis needed if your building is in Seismic Design Category A, the response coefficient is simply .01 and you design the base shear to 10% of the buildings weight. For this reason, I conducted all of the analysis using the 2000 USGS maps to be able to compare the redesign to the original design.

DESIGN APPROACH

The original lateral system was designed with wind forces (factored) controlling in the north-south direction and seismic controlling in the east-west direction. This was somewhat surprising considering the low seismic region. Knowing the limitations architecturally the design options were laid out in each direction. In the north-south direction, wind controlled and there is no way to reduce the design force from wind because it's based on solely on location. This was the direction that I was able to add braces in the middle bay and moment frames in the adjacent bays at column lines 3 and 8. The original design had two moment frames anchoring the end bays, as seen in figure 3. By adding these moment frames/braced frames to column line 3 and 8 and removing the original moment frames I was able to keep the lateral members symmetrical preventing torsion. In the short direction, or the east-west direction the use of braced frames was not possible without altering the architecture of the building. Since the controlling forces were seismic, I was able to reduce the base shear by increasing the Response Modification Factor (R) which was originally R=3 (system not specifically detailed for seismic resistance).

SUMMARY OF OPTIONS

Original Design (R=3, both directions)

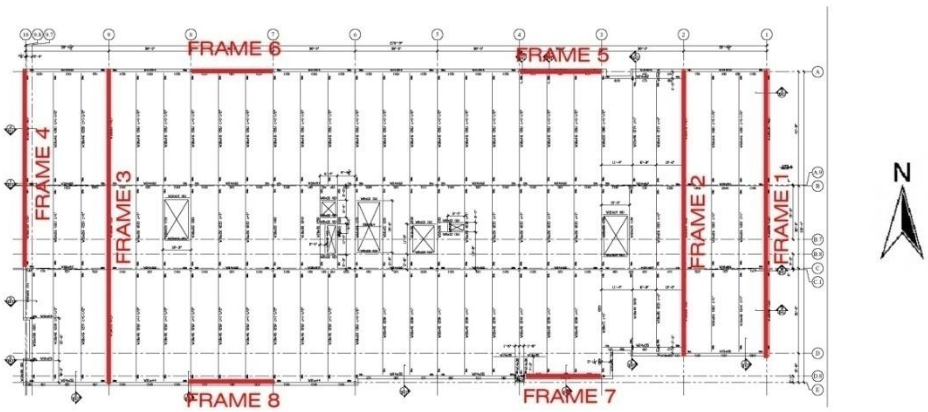


FIGURE 3

Figure 4 – Frame 1 in original design

Design Base Shears		
	North-South	East-West
Wind	342.0	144.0
Seismic	210.0	210.0

Table 1 – Design Base Shears for Original Design

Option A-1 (R=8, both directions)

This design consists of the same moment frame configuration but instead of an R=3 (system not detailed seismically) an R=8 (special steel moment frames) which add more ductility to the connection. It also adds time to fabrication and erection because the connections must meet AISC seismic specifications for the respected type of connection and frame. This design reduced the member size in the east-west direction but didn't reduce any members in the north-south direction, eliminating it from a possible redesign option. These connections are used primarily in high seismic regions.

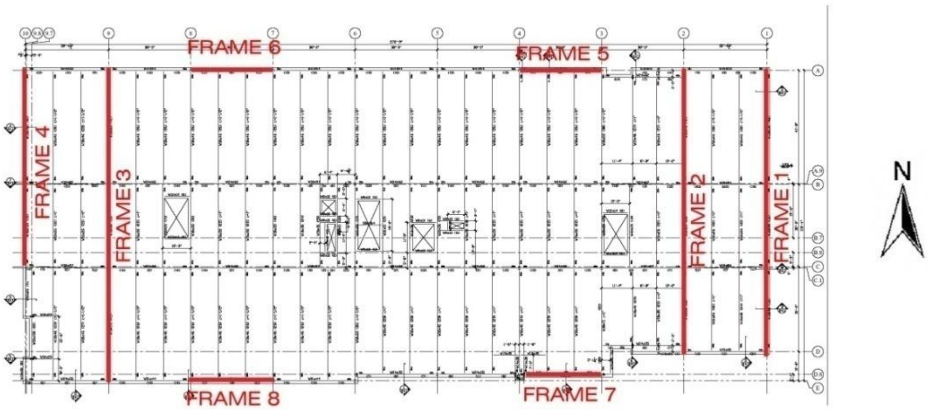


FIGURE 5 – Option A-1

Design Base Shears		
	North-South	East-West
Wind	314.6	142.0
Seismic	124.0	124.0

Table 2 – Design Base Shears for Option A-1

Option B-1 (N-S direction R=6, E-W direction R=4.5)

This design consists of braced frames located in column lines 3 and 8, and moments frames anchoring the perimeters in the north-south direction with an R=6 (dual system with IMF – special steel concentrically braced frames). Using this system however you can use OMF in lieu of IMF in my Seismic Design Category. So the outer moment frames could be designed for an R=3 and the Braces could be designed for an R=6. However, two problems stood out; 1) the wind base shear still wasn't reduced therefore not reducing member size and 2) the braces took 91% of the shear distribution when only allotted 75% by code. For the moment frames to resist 25% of the forces the member sizes must increase to increase stiffness. This eliminated this option in the north-south direction. The east-west direction an R=4.5 (Intermediate Moment Frames) lowered the seismic base shear to a point where it controlled the upper floor design shear and wind forces controlled the lower levels. This lowered member size and didn't impact the schedule too much. This is a good balance and proved to be the most cost efficient design in the east-west direction.

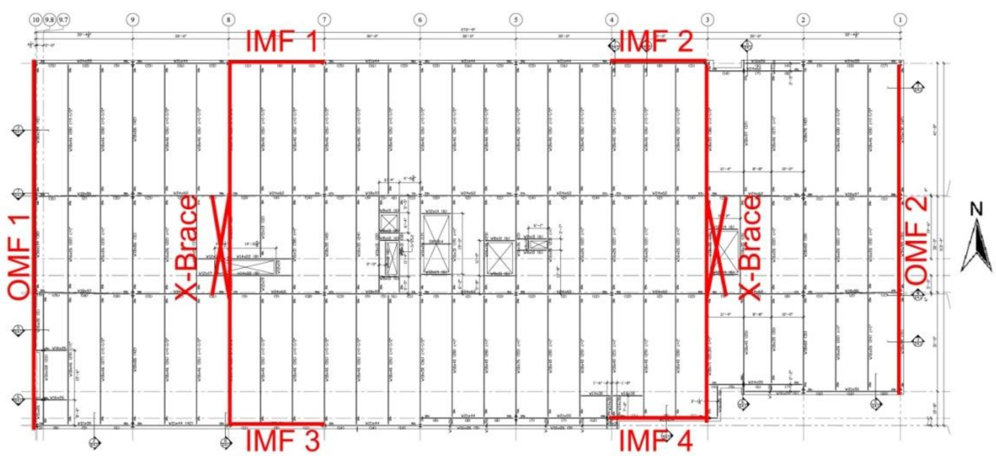


FIGURE 6 – Option B-1

Design Base Shears		
	North-South	East-West
Wind	314.6	144.0
Seismic	142.2	142.2

Table 3 – Design Base Shears for Option B-1

Option B-2 (N-S R=3, E-W R=4.5)

This design consists of braced frames located in the inner bays of column lines 3 and 8 with moment frames in the outer bays for the north-south direction. An R=3 is sufficient because wind controls and the brace takes over 90% of the load. The east-west direction is the same design concept as the previous example, R=4.5 (IMF).

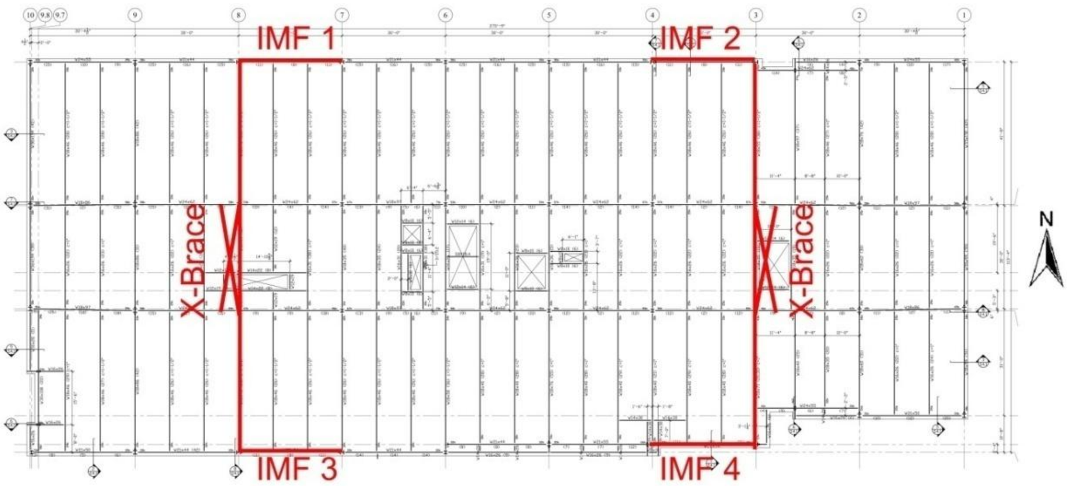


FIGURE 7a- Option B-2

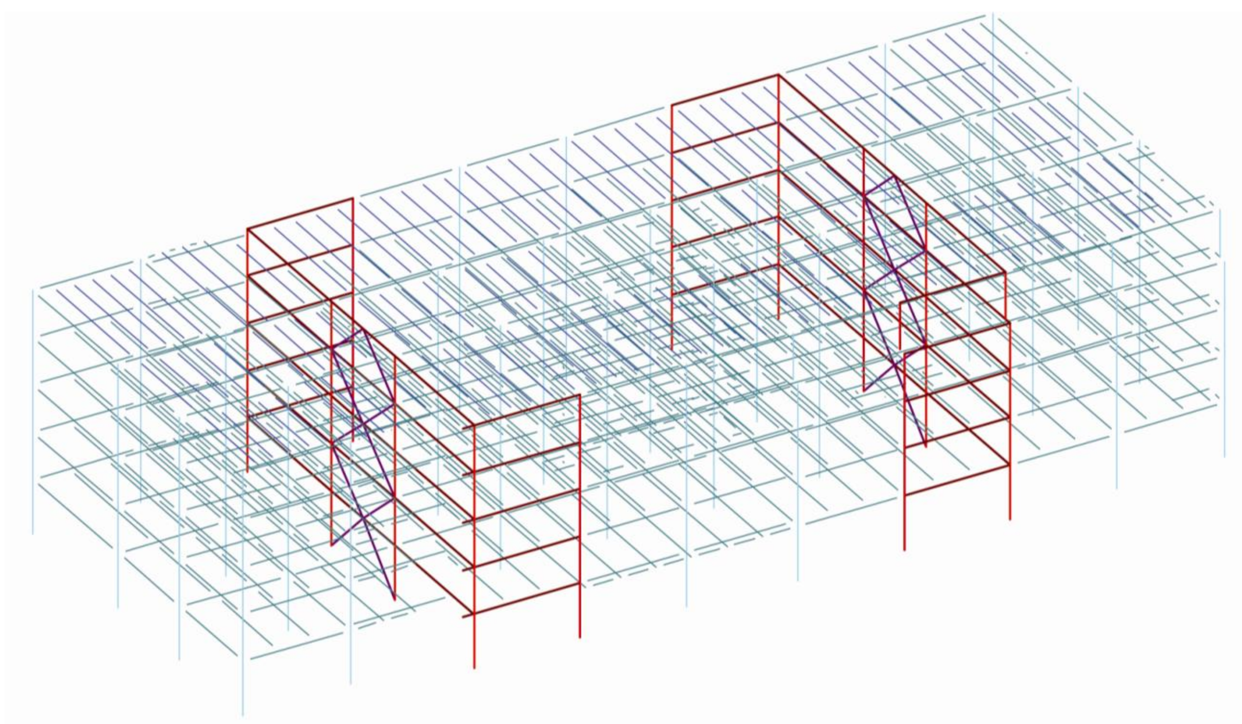


FIGURE 7b – Option B-2 (Final Design)

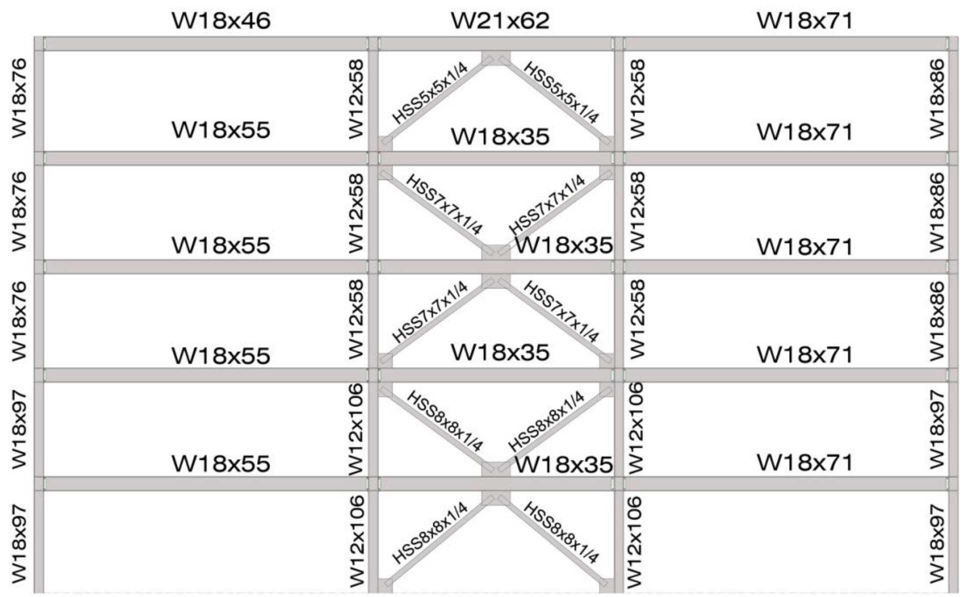


FIGURE 8 – Braced Frame in lateral redesign

Design Base Shears		
	North-South	East-West
Wind	314.6	144.0
Seismic	142.2	142.2

Table 4 – Design Base Shears for Option B-2

IMPACT ON FOUNDATION

The original foundation design was spread footing ranging in thickness from 30" to 42". Designing the new lateral resisting system with braced frames, the majority of the load is taken by the brace. Therefore, the majority of the overturning moment is also taken by the brace resulting in a higher uplift force. The new foundations were designed in RAM and to resist the increase in uplift force friction piles and pile caps were used. The final pile design is summarized in the Appendix B and the cost breakdown can be found in the construction breadth.

CONCLUSIONS

Since increasing the R value in the north-south direction is not cost efficient due to controlling wind forces, the most economical solution was to keep an R value of 3 in the north-south direction. An R value of 4.5 (IMF) will be used in the east-west keeping the original layout due to architectural restrictions. After this option was narrowed down and eventually resolved as the most efficient it was analyzed and checked with code (see Appendix B for calculations). The new design resulted in 13% less steel, obtaining the goal to design a more cost efficient structure. The takeoffs can be seen in the tables 5 and 6 below. Since the brace takes the majority of the load the foundation was checked to see if the foundations could handle the overturning moment of the brace. The results from RAM showed that there was uplift and the original design of all spread footings wasn't adequate. Piles and Pile Caps were designed in RAM and the results were factored into the overall cost comparison of the structure. A full cost analysis of the structure was completed and can be found in the construction breadth.

Original Design Takeoff						
	Beams	Columns	Joists	Braces		
Gravity members (lbs)	813,457.00	88,509.00	58,000.00	0.00	480.0	tons
Lateral members (lbs)	210,003.00	173,127.00	-	0.00	191.6	tons
Total Weight (lbs)	1,023,460.00	261,636.00	58,000.00	0.00		
Tons of Steel	511.7	130.8	29.0	0.0	671.5	tons

TABLE 5 – Original Design Takeoff for Steel members

Lateral Redesign Takeoff						
	Beams	Columns	Joists	Braces		
Gravity members (lbs)	813,457.00	88,509.00	58,000.00		480.0	tons
Lateral members (lbs)	129,539.00	65,588.00		8,686.00	97.6	tons
Total Weight (lbs)	942,996.00	154,097.00	58,000.00	8,686.00		
Tons of Steel	471.5	77.0	29.0	4.3	581.9	tons

TABLE 6 – Lateral Redesign Takeoff for Steel members

PROGRESSIVE COLLAPSE STUDY

SIGNIFICANCE

On April 19th,1995 the Murrah Building in Oklahoma City was bombed and the results were catastrophic. The blast caused a portion of the building to collapse which resulted in 168 causalities mostly from the building collapsing and not from the blast effects. This was the major progressive collapse event in US history and with the increase of international terrorist attacks (September 11th attacks) the chances have increased that other structures may be targeted in the future.

FIGURE 9 – Murrah Federal Building after bombing (FEMA Primer 2003)

BACKGROUND

The Current code in the U.S. is written by International Code Council which adheres to the American National Standards Institute. Within the code publications there is very little mention to the mitigation of progressive collapse. In fact, the only standard that even references progressive collapse is the American Society of Civil Engineers which deals with design loads. The current edition, ASCE 7-05 provides a basic direction but gives no specific design criteria, stating:

"Except for specially designed protective systems, it is usually impractical for a structure to be designed to resist general collapse caused by gross misuse of a large part of the system or severe abnormal loads acting directly on a large portion of it. However, precautions can be taken in the design of structures to limit the effects of local collapse to prevent or minimize progressive collapse" (Baldridge).

Although there is no real code pertaining to the potential collapse of buildings, US government agencies such as the General Services Administration and the Department of Defense have looked extensively into progressive collapse developing design criteria and guidelines to reduce the risk. The guidelines presented by these agencies include preventing collapse in new buildings and methods for assessing risk in existing buildings (Gould). The GSA has its own set of requirements for GSA facilities and meets the provisions set forth by the Interagency Security Committee (ISC). DoD facilities must meet requirements set forth by the Unified Facilities Criteria (UFC).

DESIGN APPROACH

For the Purpose of this thesis report the applicable code was ASCE 7-05. There are two different design approaches defined in the code: Direct Design method and Indirect Design method:

- Direct Design Approach – provide "explicit consideration of resistance to progressive collapse during the design process
    - Alternate Path – structure must be capable of bridging over a missing structural element, localizing damage.
    - Specific Local Resistance – which requires a part of the building to sufficient strength to resist the load or blast
- Indirect Design Approach – provide resistance to progressive collapse "implicitly through the provision of minimum levels of strength, continuity, and strength."
    - Plan layout
    - Integrated system of ties
    - Redundancy
    - Ductile detailing
    - Reinforcement for blast and load reversal

These are simply just approaches and provide no criteria or code to adhere to. As stated in the introduction, Building II is hypothetically being considered a 'high-profile' building in which it qualifies for additional design criteria to mitigate the risk of progressive collapse. Determining the threat level is first also important because it determines the approach taken. The Indirect method is more cost effective and is typically used when the threat is low. The Direct method is much more costly and is used when the threat is considered high.

For this report, I will consider both methods to compare the costs implications. For the indirect method I will consider my building DoD facility and with a Low Level of Protection (LLOP). The design process can be seen by the flowchart below.

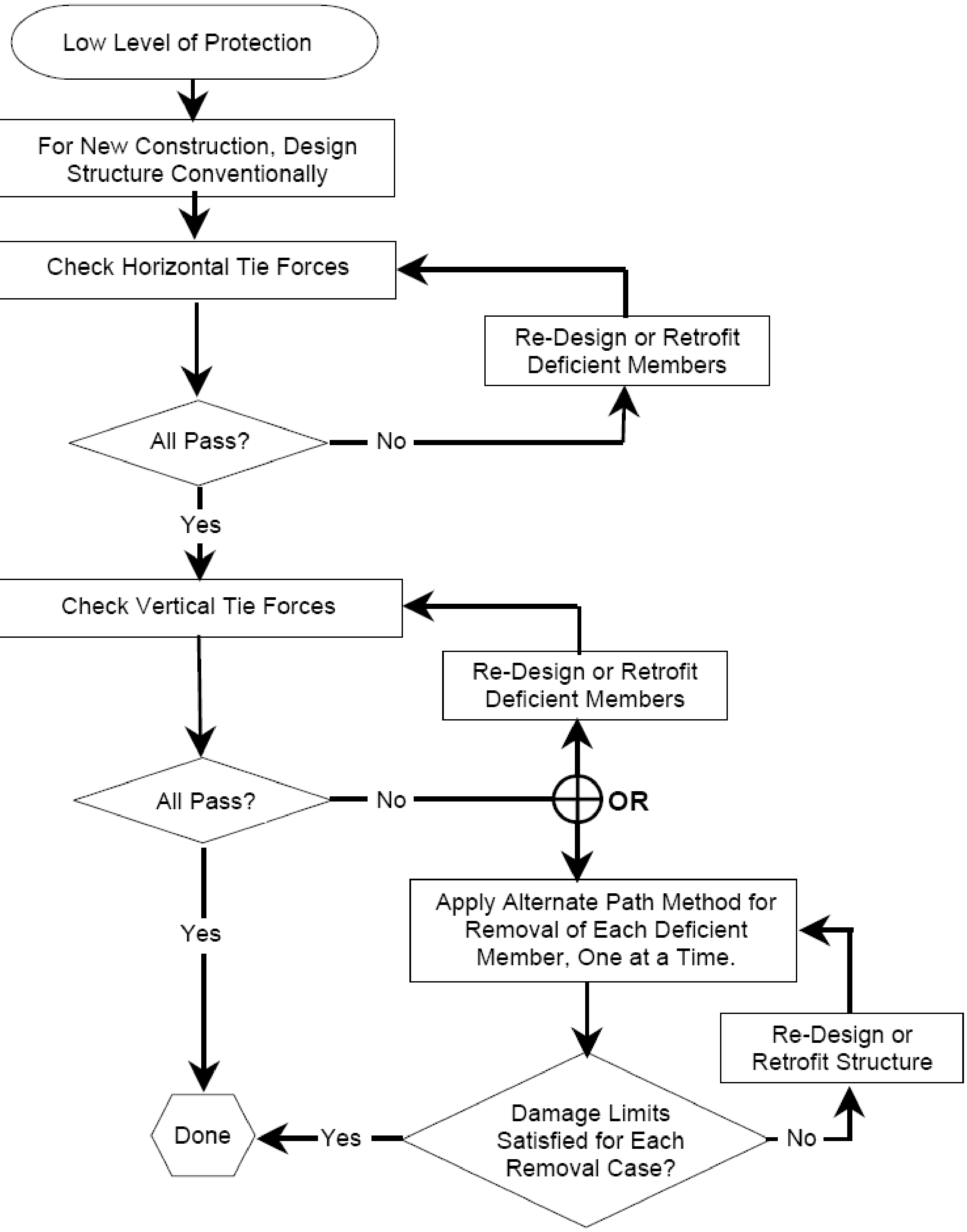


Figure 10 - DoD Design process flowchart (UFC 2005)

As for the Direct Method, Building II is a GSA Facility and the threat was defined as a high level of protection. The Design Process can be seen in the Flowchart below.

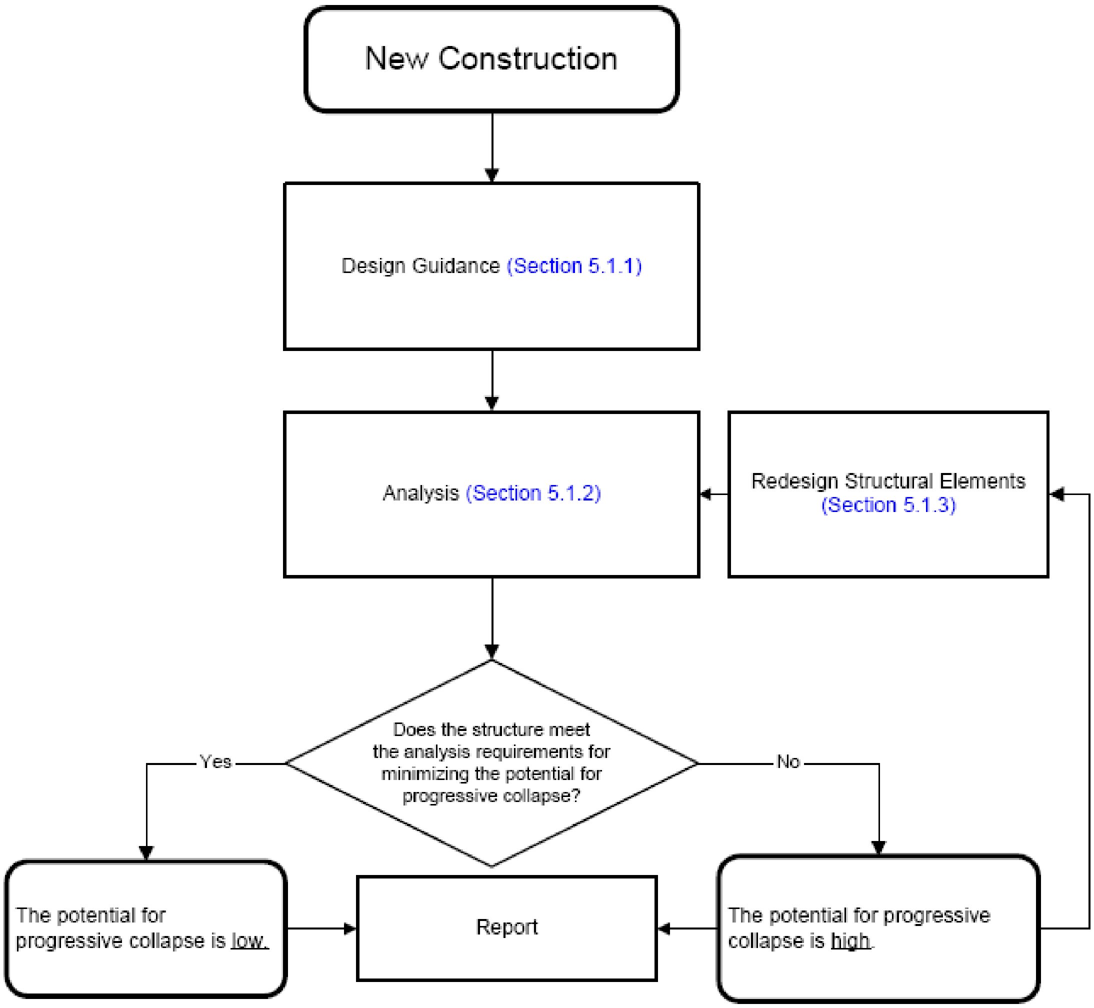


Figure 11 - GSA design process flowchart (GSA 2003)

DESIGN STRATEGIES

Indirect Method – DoD facility with design for a Low Level of Protection

The goal of this method is to effectively tie the structure together, making sure all of the ties meet the required design strengths. Since the assumption was to have a Low Level of Protection all tie forces must be checked. Any vertical tie forces that do not meet the tie force capacity, an Alternative Load Path analysis is required. Examples of Horizontal Tie forces can be seen in the figure 10. A summary of the Tie Forces can be seen in figure 11 and calculations can be found in Appendix C.

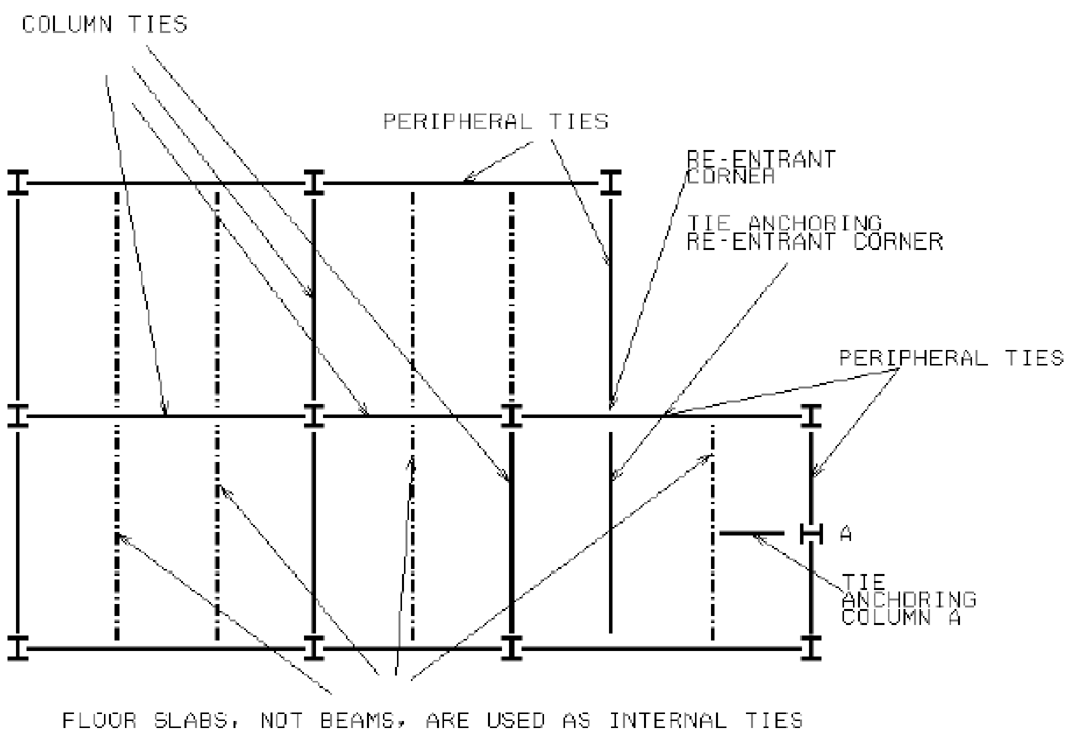


FIGURE 12 – Example of general tying of a steel framed building (UFC 2005)

Tie Force Requirements	
Internal Tie Force	42.2 K
Peripheral Tie Force	15.3 K
Horizontal Tie Force	42.2 K
Vertical Tie Force	113.9 K

Table 7 – Tie force requirements

Direct Method – GSA facility with a design for a High threat level

The guidelines provided by the GSA are to mitigate progressive collapse by concentrating on the detailing of local connections and global configurations of the structure. Alternative Load Path is a minimum requirement in the GSA requirements stating it is critical that girders and beams must be designed to span two full spans (two full bays). The Guidelines also state that there must be continuity against a removed column and that the beams must deform flexurally well past their elastic limit without collapsing. This figure shows the inability of a typical moment connection scheme to resist collapse after the removal of a column.

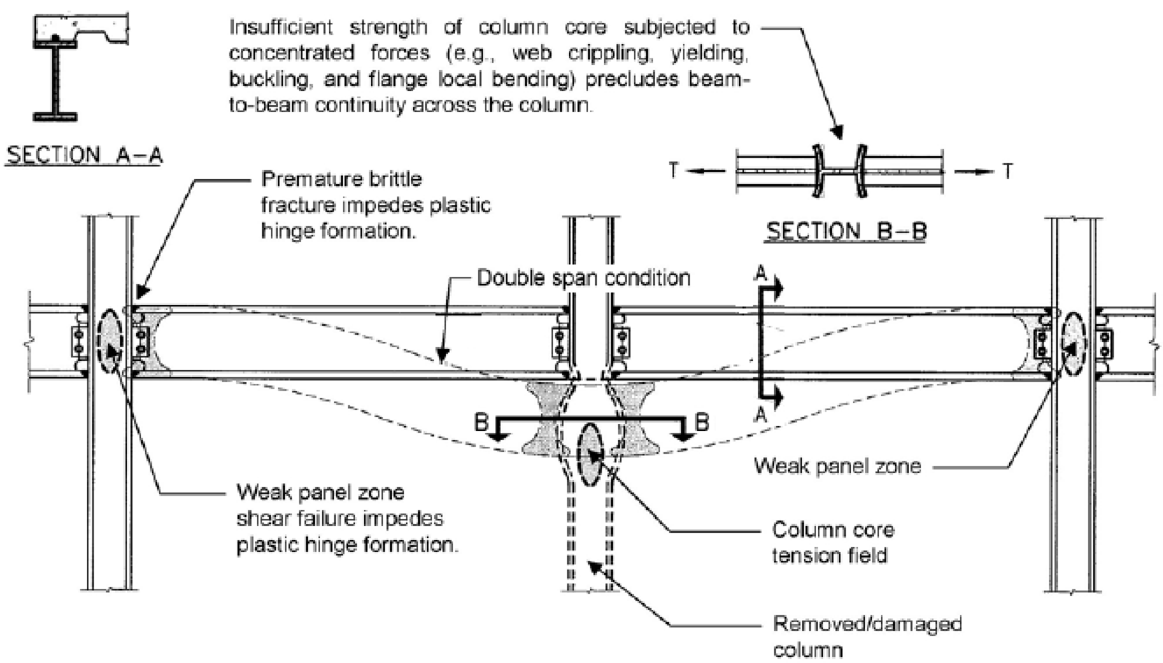


FIGURE 13 – Example of a typical moment frame's inability to protect against progressive collapse (GSA 2003)

To analyze the exterior considerations of a steel framed structure, the GSA outlines the following procedure:

1 Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the short side of the building.
2 Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at or near the middle of the long side of the building.
3 Analyze for the instantaneous loss of a column for one floor above grade (1 story) located at the corner of the building.

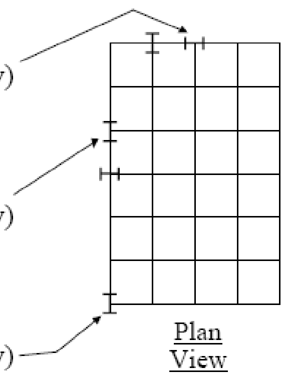


FIGURE 14 – Exterior considerations for analyzing for alternative load path (GSA 2003)

However, for Building II's hypothetical case an analysis will be conducted locally or a Specific Load Path. The back of Building II has the smallest setback distance and therefore would have the highest risk of a explosion at ground level. Three bays will be taken into consideration, or two columns, and redesigned. This happens to be the second point outlined, the removal of a column at or near the middle of the buildings long side. The following load case was applied:

$$\text{Load} = 2(\text{DL} + 0.25\text{LL})$$

For exterior considerations, the GSA states that the collapse area resulting from the instantaneous removal of a vertical member at grade level is limited to the smaller of:

1. The structural bays directly associated with the instantaneously removed vertical member in the floor level directly above the instantaneously removed vertical member

   or

2. 1,800 ft^2 at the floor level directly above the instantaneously removed vertical member

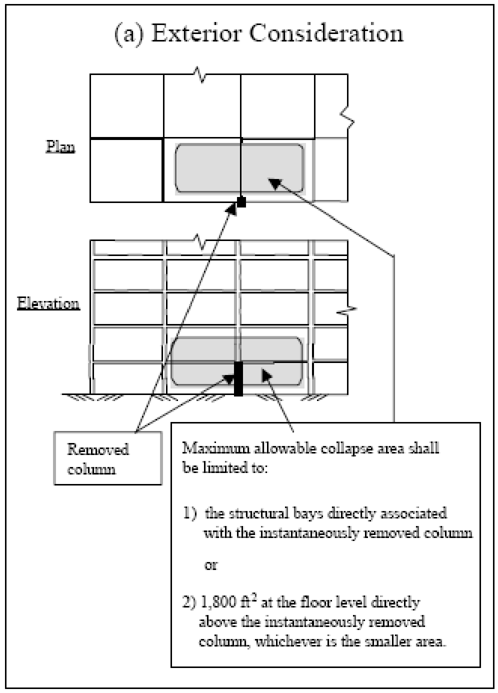


FIGURE 15 – Exterior considerations for analyzing for alternative load path (GSA 2003)

Due to the abnormally long spans of Building II the structural bays affected by the removal of an exterior column exceeded 1800 ft^2 so the extent was limited in this case.

As for acceptance criteria, the GSA's requirements are indicated by **D**emand-**C**apacity **R**atios or DCR. These ratios are determined by:

$$DCR = \frac{Q_{UD}}{Q_{CE}}$$

Where, Q_{UD} is the acting or demand force determined in the component and Q_{CE} is the expected ultimate, un-factored capacity of the component.

To analyze Building II for progressive collapse, a step by step procedure was followed which can seen in the Appendix C. Using virtual work, a plastic analysis was conducted on a one bay frame in Building II. DCR's were taken off an acceptance criteria chart found in a Appendix C and the expected ultimate capacity, Q_{CE} was determined. According to expected capacity, the members were chosen according to their plastic capacity. Figures 16 and 17 represent the final design for 3 adjacent bays.

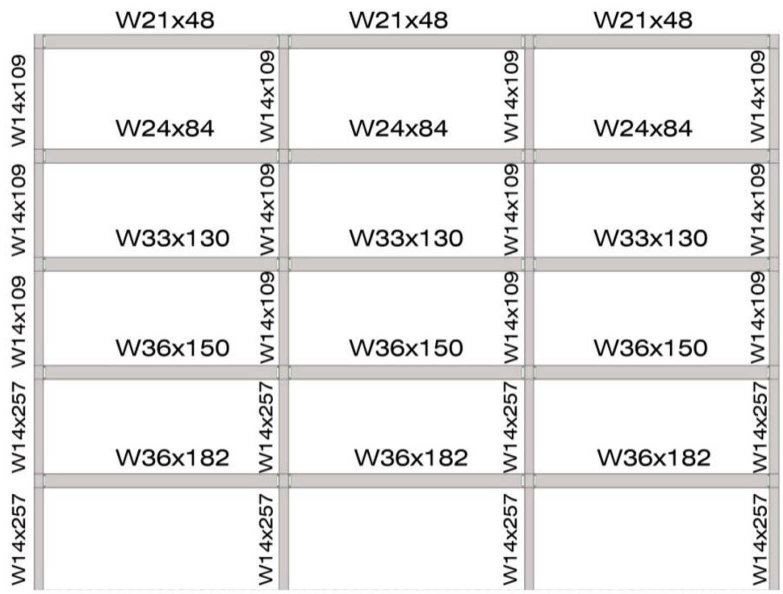


FIGURE 16 – Progressive Collapse Design

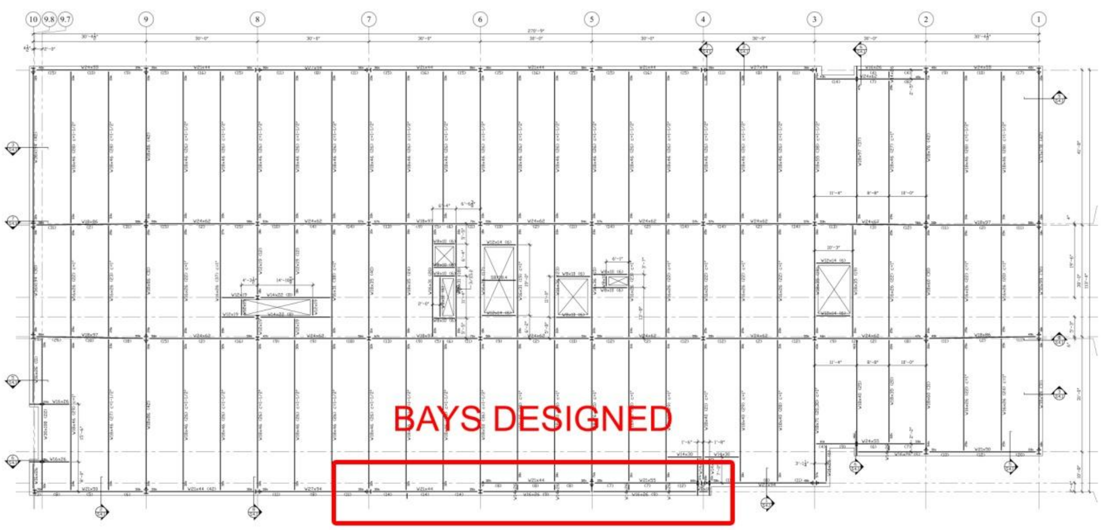


FIGURE 17 – Bays designed for Progressive Collapse

SidePlate moment connections were used as final connection type the direct method. A typical moment connection will not adequately meet the requirements for a continuous load path if a vertical element is removed instantaneously. In the occasion that a column is removed by a blast, SidePlate's steel frame connections form a steel box that achieves beam-to-beam continuity. Also, the steel plates add robustness to the structure helping defend the structures integrity against blast loads. Figure 18 shows the SidePlate connection. The SidePlate moment connection system have been extensively tested and exceeds all of the criteria for the GSA and DoD for designing against progressive collapse.

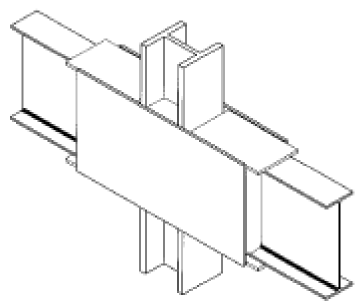


FIGURE 18 – SidePlate moment connection

CONCLUSION

After redesigning the building for mitigating the risk of progressive collapse a material takeoff was conducted to see if the design could be accomplished with the extra costs saved from the new lateral design. The Direct Method used 3.75% less steel than the original design therefore obtaining the goal. The indirect method used no more steel than the lateral redesign, other than whatever was necessary for additional moment connections. A full cost analysis was completed and can be found in the construction breadth of the report.

Progressive Collapse (Direct) + Lateral Redesign Takeoff						
	Beams	Columns	Joists	Braces		
Gravity members (lbs)	865,191.00	139,337.00	58,000.00		531.3	tons
Lateral members (lbs)	129,539.00	91,966.00		8,686.00	110.8	tons
Total Weight (lbs)	994,730.00	231,303.00	58,000.00	8,686.00		
Tons of Steel	497.4	115.7	29.0	4.3	646.4	tons

TABLE 8 – Material Takeoff for Progressive Collapse Design

CONCLUSION of STRUCTURAL DEPTH

After meeting both goals for the structural depth of this report, the original lateral design doesn't seem to be the most optimum solution. The lateral system was redesigned using Intermediate moment frames in the east-west direction (R =4.5) and braced frames in the north-south direction (R=3). The redesign used 13% less steel, successfully accomplishing the intent of the depth. The comparison was made strictly on the amount of steel used and not the overall cost analysis. This can be found in the construction breadth later in the report.

The second part of the depth encompassed designing the structure to mitigate the risk for progressive collapse with two different scenarios. The first, was an indirect method following guidelines from the Department of Defense with a low level of threat. No additional steel was needed other than additional moment connections around the perimeter of the building. The second, was a direct method following guidelines from the General Services Administration with a high level of threat. The guidelines are to design the entire building for an alternate load path in the case a vertical member is removed instantaneously by a blast load. However, for the purpose of this report a specific load path was analyzed limiting the analysis to one bay and the design to three adjacent bays. In both cases, the design was added into the lateral system redesign and in both cases less steel was used than the original design.

ARCHITECTURAL BREADTH

SITE DESIGN

BACKGROUND

Similar to the second part of the structural depth this breadth is based off the same premise that Building II is considered a 'high-profile' type of building. That being the case, the scope is to analyze the original site layout and redesign it to mitigate the risk of a possible attack. To accomplish this goal, the GSA's Site Security Design Guide was used which provides the criteria to design a secure site and safe public environment. With the initial goal to keep the architecture of the façade untouched, the glazing and precast panel connections are going to be evaluated and redesigned to better resist abnormal blast loadings.

DESIGN PRINCIPLES

The easiest way to prevent an attack of a 'high-profile' or federal building is to prevent the threat from ever approaching the target. To provide a secure site the GSA recommends integrated security measures into the site architecture creating a safe effective public space. Hopefully, many of these measures will never be used to prevent an attack and if integrated properly can be used for the purpose of the public, similar to Figure 19. Not only is the integration of these types of security elements into the design a challenge but balancing the amount of risk with the high costs is even more of a challenge. Some of these challenges include the determination of threats and vulnerabilities which are very hard to predict, decisions about what to protect, and selection of countermeasures which are usually extremely expensive. To achieve this balance between aesthetics and security the GSA has established four principles or hallmarks:

FIGURE 19 – Use of monument as perimeter barrier

1. Strategic Reduction of Risk – defines priorities, weighs resources available to site design, facility design and property management

2. Comprehensive Site Design – meets site requirements while maximizing functionality, aesthetics, and total project value for the users

3. Collaborative Participation – a multidisciplinary team that integrates diverse expertise to create innovative and effective solutions

4. Long-Term Development Strategy – A phased, incremental strategy for implementations of security improvements over time.

DESIGN GUIDELINES

To allow the multidisciplinary teams to effectively achieve collaboration the GSA has developed site "zones". By breaking the site into these zones the design team can better recognize the relationships between the zones and how they affect each other in the design process. The site is broken into 6 zones:

Zone 1 – Neighborhood
Zone 2 – Stnadoff Perimeter
Zone 3 – Site Access and Parking
Zone 4 – Site
Zone 5 – Building Envelope
Zone 6 – Management and Building Operations

A breakdown of each zone can be seen in the Appendix D.

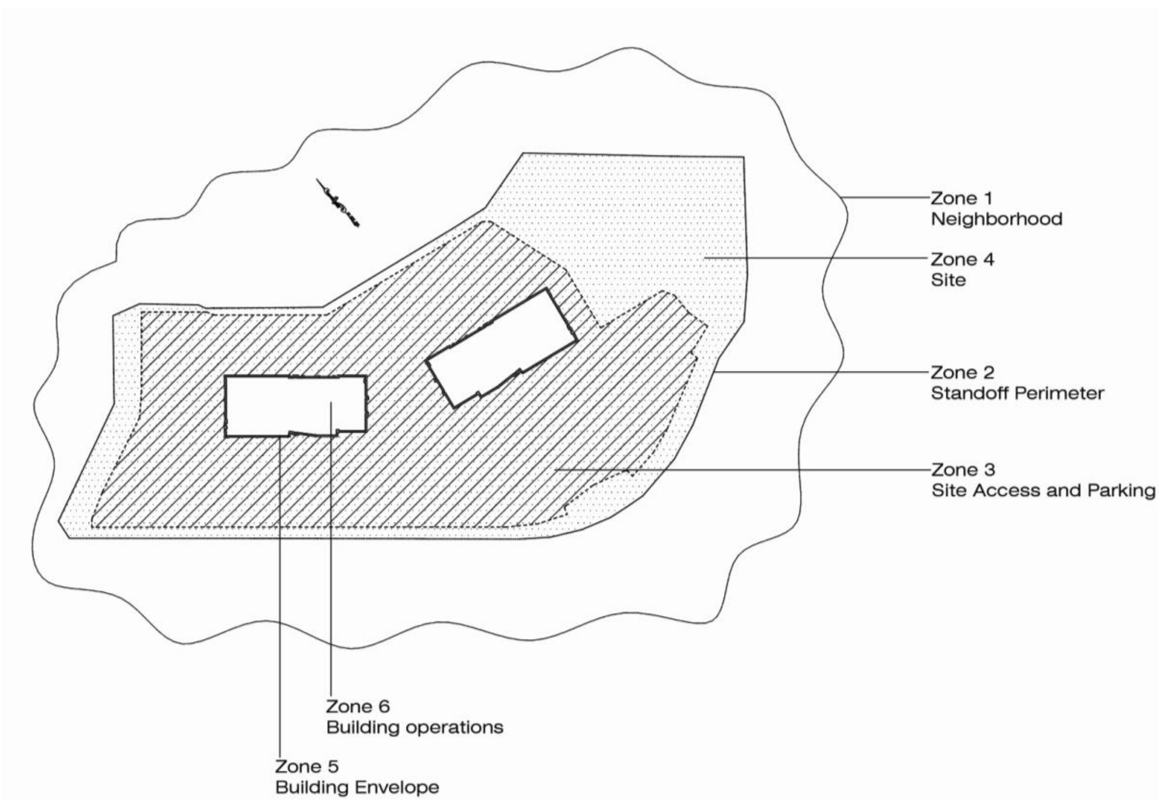


FIGURE 20 – Conceptual Zone Plan

DESIGN APPROARCH

Once built, Building II will be located in the Westfileds Corporate Center of Chantilly, Va. The property owned by the Alter Group is very extensive allowing for a wide perimeter setback. Standoff perimeter is the easiest way to prevent an attack on a building but purchasing land is one of the most expensive parts of a project. The problem with most "high-profile" buildings is the building is usually set in a city setting and the available land isn't enough to obtain a proper setback. Designers then must compensate by hardening the façade, envelope and structure to meet security requirement which is very expensive. Fortunately for the Building II, the available setback is 135' in the back, and over 200' in the front and on the side. Even though the building is setback further the front of the building is at higher risk because of the public road access and no barrier. The back of the building is setback 135 ft. from the property line however there is a 6 ft change in elevation with a retaining wall, preventing the pressures from blast.

FIGURE 21 – SITE REDESIGN

To start the design approach a threat level must be assumed and this is usually determined by the type of building. As previously stated, Building II will be hypothetically a “high-profile” corporate office building with a “High” level of protection required. Figure 22 shows the level of protection vs. standoff distance and explosive weight for a “typical generic conventional construction”. The critical standoff for Building II is 230 ft (front of building and distance to nearest road access) which would limits the blast weight to 220 lbs of equivalent TNT charge. For purposes of this breadth, a 500 lb. equivalent TNT blast will be used. This obviously requires changes to the façade to compensate for the additional standoff distance required which is approximately 300 ft. according to Figure 22.

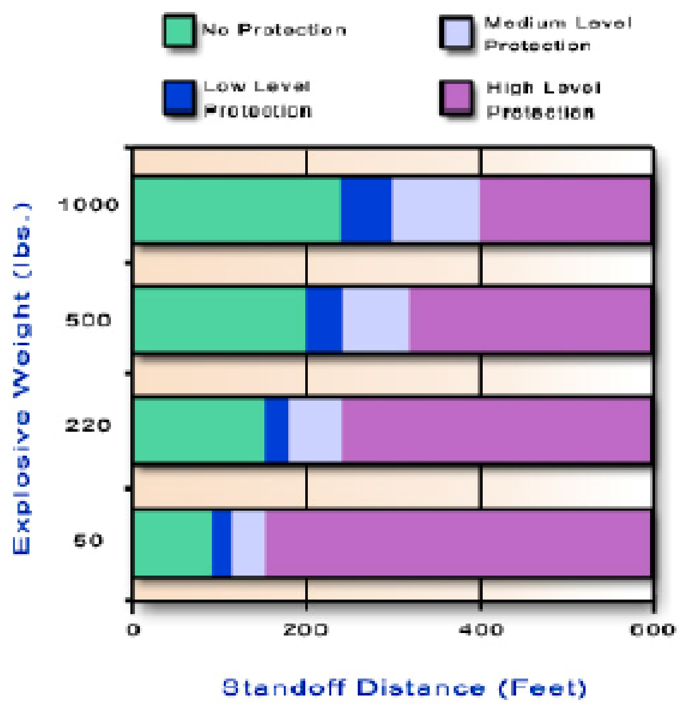


FIGURE 22 – Protection Level vs. Standoff and Explosive Weight
(ISC performance based design)

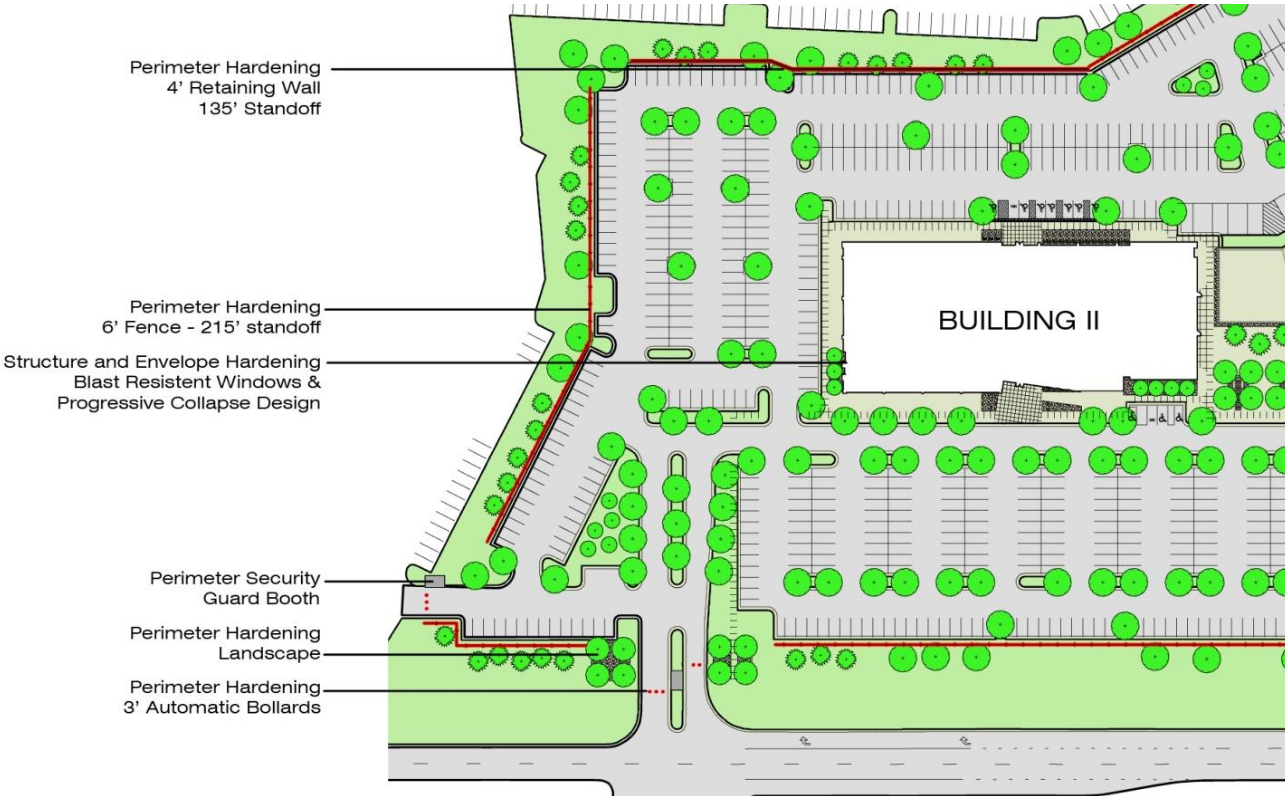


FIGURE 23a – Site redesign with hardened perimeter

Since the level of protection is considered 'high' the decision was made to secure the perimeter. Since the site is so extensive, securing the perimeter will cost a lot of money. There are already retaining walls used throughout the site so a comparison was conducted to see the cost implications of using a retaining wall vs. a security fence to secure the perimeter. Figure 23a shows the measures taken to harden the perimeter and Figure 23b shows the secure access point. As you can see from the Table 9 below, the fence is more cost efficient and a much more logical choice. Figures 24 and 25 show the other materials and objects used to harden the perimeter. The key was to secure the access points of the site by using automatic anti-ram bollards and guard booths. The full specs can be found in Appendix D. Natural landscaping is a great way to save costs and fortunately the site has a large pond in the northeast corner, which acts as a natural barrier.

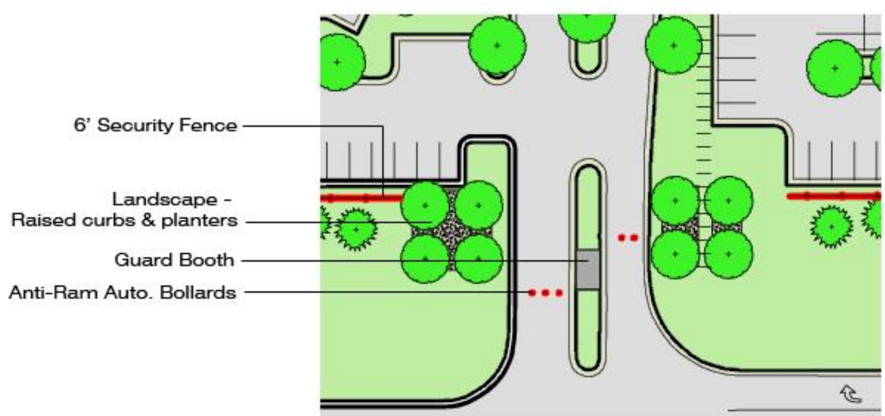


FIGURE 23b – Access point to Site

Hardened Perimeter Comparison					
Retaining Wall	quantity	unit	unit price	amount	
Excavation	640.4	CY	$30	$19,212	
Backfill	320.2	CY	$20	$6,404	
footing concrete	320.2	CY	$350	$112,070	
wall (12")	14410.0	SF	$30	$432,300	
wall drain tile	2882.0	LF	$25	$72,050	
				$642,036	} compred to
Security Fence	1926.0	LF	$130	$250,380	

TABLE 9 – Hardened perimeter comparison

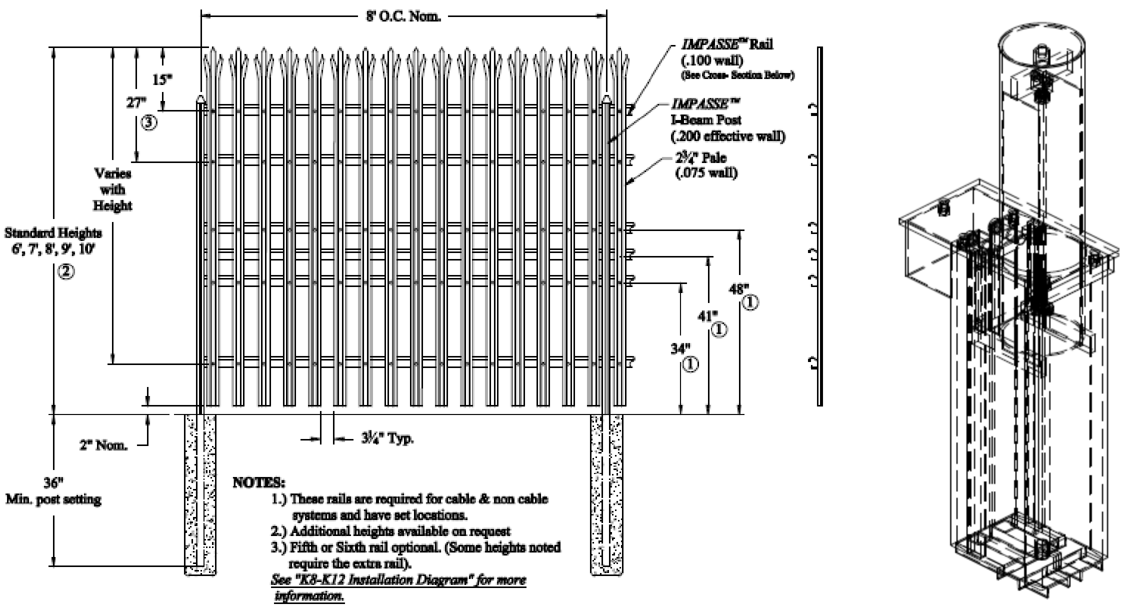


FIGURE 24 – Security Fence and Anti-ram automatic bolllard

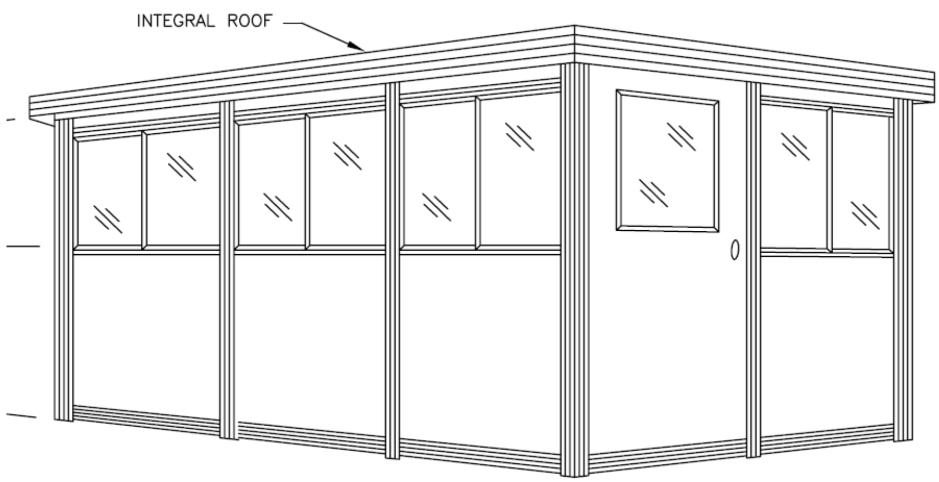


FIGURE 25 – Guard booth module

FAÇADE REDESIGN

WINDOW DESIGN APPROACH

According the GSA, the type of façade and level of protection determine the minimum standoff distance. The façade can either be 'frangible' which has an ultimate, unfactored flexural capacity that is less than 1.0 psi or 'non-frangible' which has an ultimate, unfactored flexural capacity over more than 1.0 psi. As stated in the site design of the breadth, the level of protection is assumed to be 'high'. To see the calculations of the strength of the façade please see Appendix D. After determining that the façade surface was 'non-frangible' the required standoff is 130 ft., according to Figure 26. All of the standoff distances of Building II meet this requirement.

Construction Type	Minimum Defended Standoff Distance (ft)*		
	ISC Required Level of Protection		
	Low and Medium/low	Medium	Higher
Steel Construction			
Rigid frame structure with a ***non-frangible facade*** (FEMA 310 Building Type: S4)	25	40	130
Rigid frame structure with a ***frangible facade*** (FEMA 310 Building Type: S1, S5, RM2)	25	35	100
Lightweight steel framed structures (i.e., Butler style buildings, etc.) (FEMA 310 Building Type: S1A, S2, S2A, S3, S5A)	55	105	165

FIGURE 26 – Minimum defended standoffs for various types of construction (GSA 2003)

As for the materials of the façade, there are several unknowns that must be assumed similar to the site design. Assuming a 'high' level of protection and a 500 lb TNT equivalent charge similar to the site design the window were designed according to ASTM F 2248-03. There are a couple different glazing options when it comes to blast resistant windows depending on the pressure. The window assembly recommended by ASTM is a laminated glass unit which is two separate plies of glass separated with a innerlayer of polyvinyl butyral (PVB). The design concept behind a laminated glass unit is for the inner PVB layer to act as an "adhering net" for the outer layer once it's been compromised. The buildup can be seen in Figure 28. A similar glazing system is the "sacrificial ply" which is designed to have the outer ply break or "sacrifice" itself and design the inner layer to resist the blast load. The typical glass plies used are heat-strengthened or fully tempered glass which are stronger and safer when the break than the typical annealed monolithic ply as seen in Figure 27.

Figure 27 – Impact performance of Glazing Materials (Old Castle)

Figures 29 and 30 show the design pressure indicated and the minimum thickness required to resist a blast for Building II. The final design for the window fenestrations are (2) 1/8” heat strengthened plies with a .030” layer of PVB in between. Figure 28 shows the glazing buildup and the specs can be found in the Appendix D as well as the design calculations.

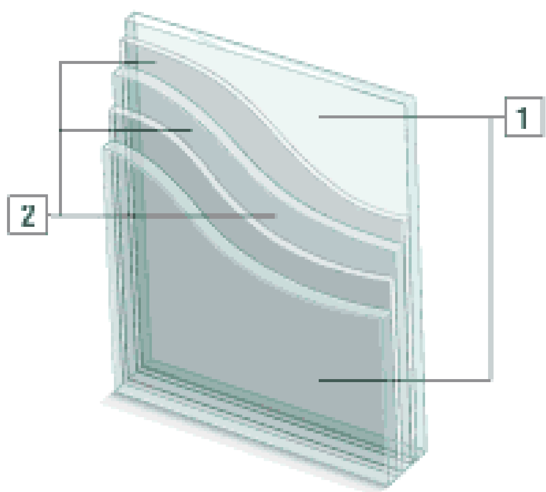


FIGURE 28 – Glazing buildup (Old Castle)

1. represents (2) layers of glass
2. inner-layer material

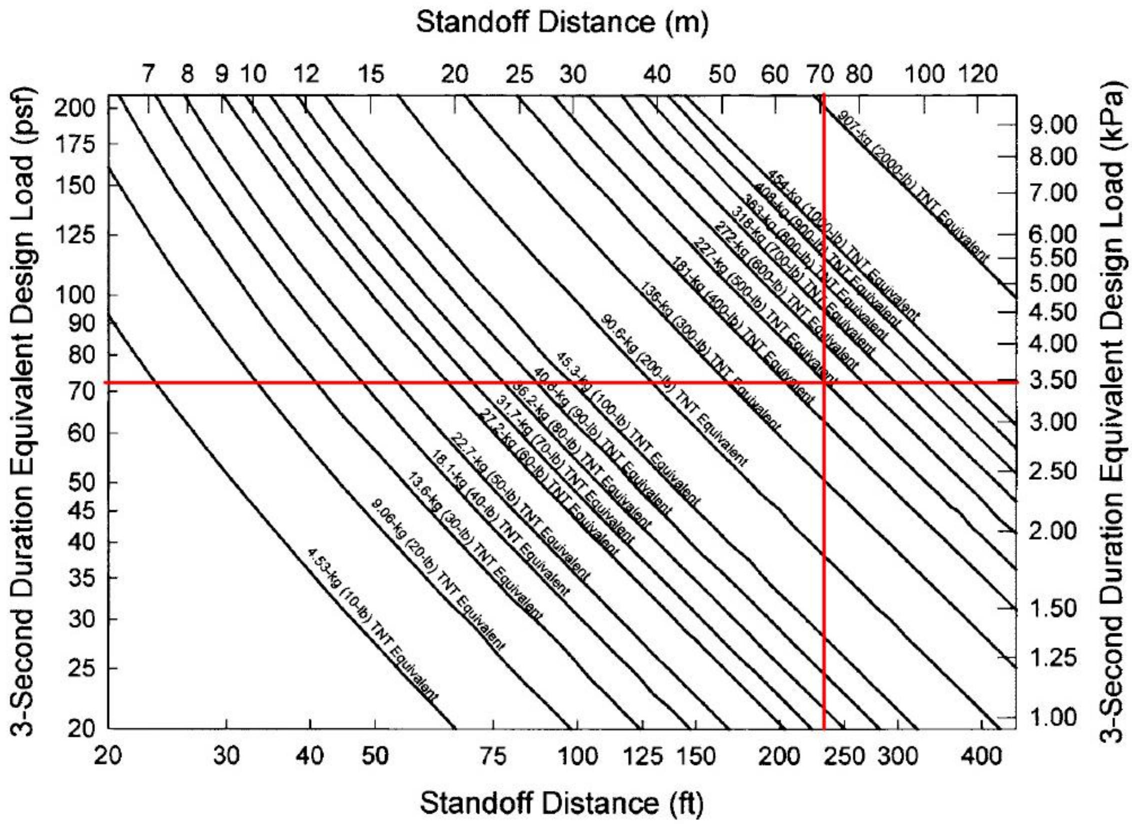


FIGURE 29 – standoff distance vs. equivalent design load (ASTM 2248-03)

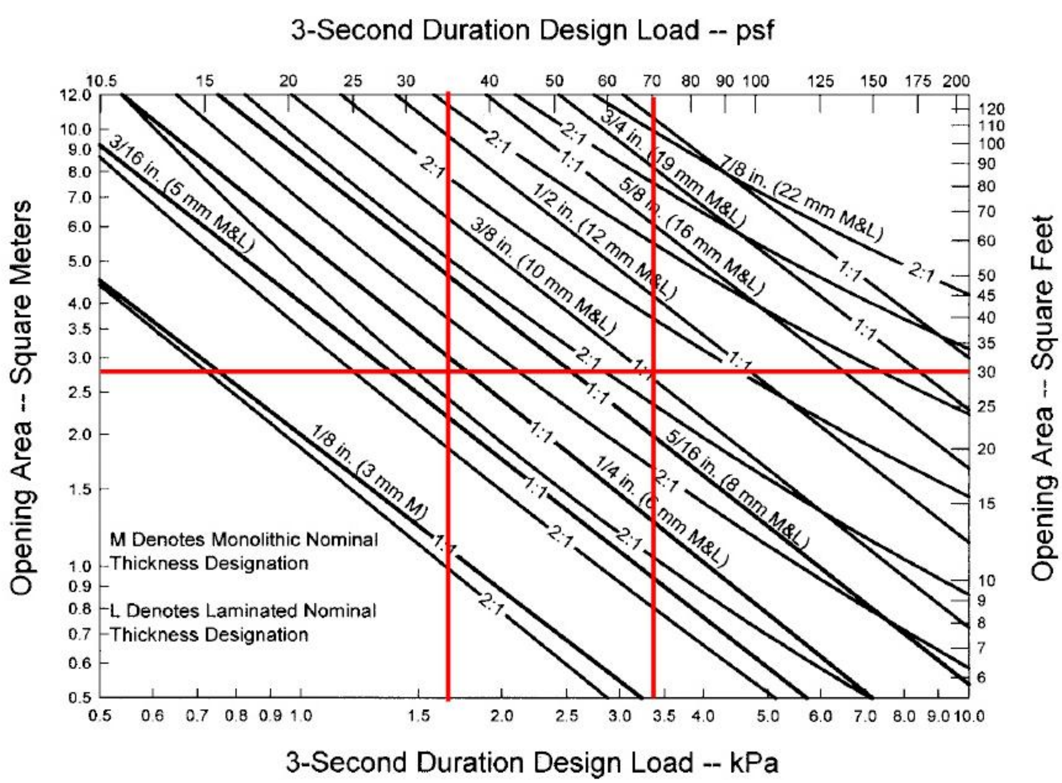


FIGURE 30 – Fenestration opening vs. Design blast load (ASTM 2248-03)

PRECAST PANELS CONNECTION

Once the design pressures were determined, the reinforcement can be designed for the precast panels. The original connection type as seen in Figure 31 was designed to resist lateral pressures of wind and transfer them into the respective diaphragms. The pressures from a blast will be much higher and therefore the reinforcement should be changed along with alterations to the connections. Figure 32 shows an alternative solution to the connection type that will absorb energy from the blast load by deforming plastically. Changing the connection type will allow the panels to act like springs to damp the forces from the blast.

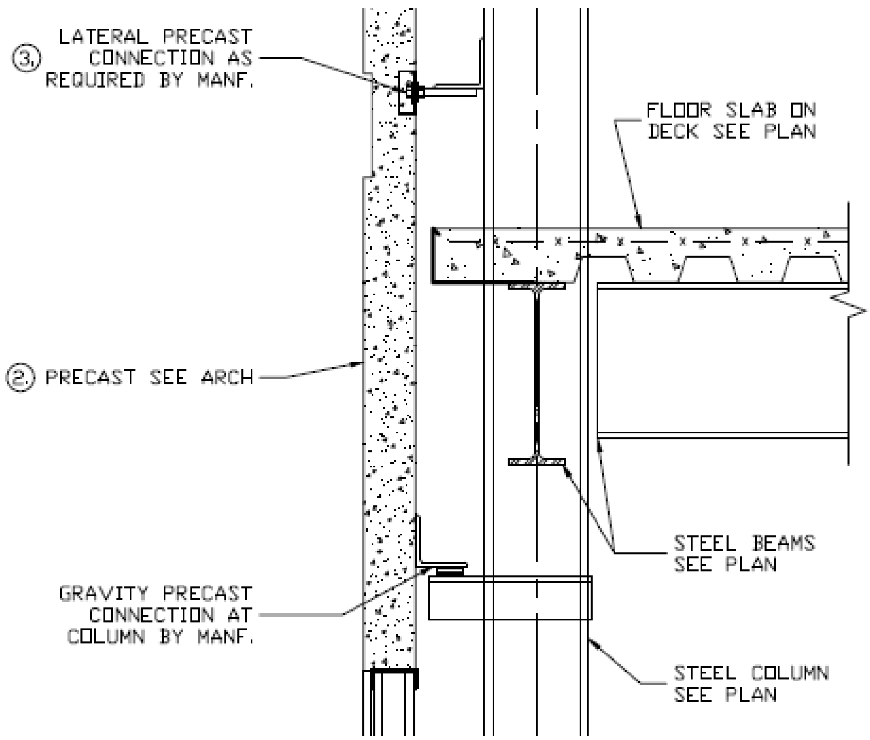


FIGURE 31 – Original Precast connection

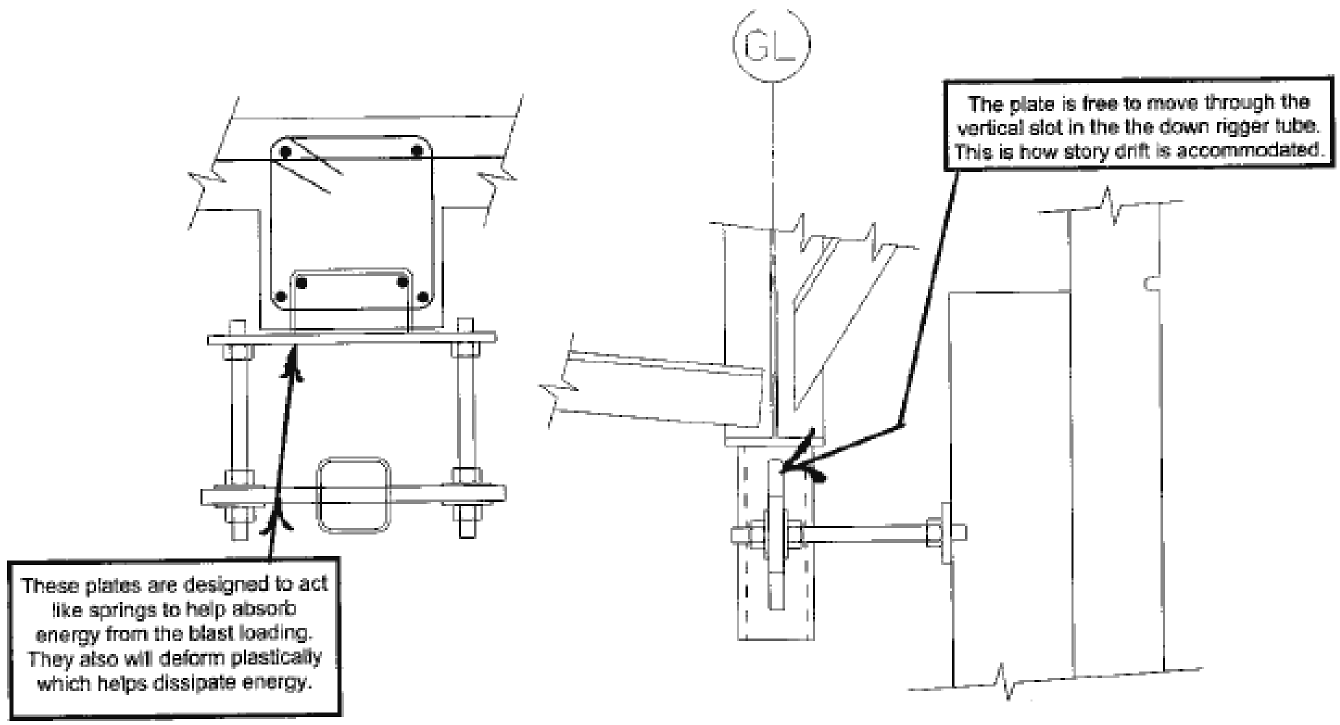


FIGURE 31 – Precast panel connection (Midwest precast)

CONCLUSION

This breadth evaluated the integration of the site and building as a function to resist the threat associated with a potential attack. The safety of the occupants is the number one concern when it comes to designing a building that has the high potential of attacked. Obviously, the designer can a fortify the site and which would surely meet all requirements and keep occupants safe. However, that would not be cost efficient and it is the designers to challenge to balance a design that is cost efficient and still meets the security requirements. Figure 32 shows the relationship between standoff distance and incremental component cost. As you can see from the chart, any standoff under 50 ft. the cost increases exponentially. The total protection cost is the top purple curve and is a function of all of the other curves. Progressive Collapse design is a straight line because it is an independent threat. To protect Building II the perimeter was secured and that happens to be the most expensive way to achieve the security measures. After analyzing the site and determining the threat and risk levels a balanced design was incorporated. Since the site was so extensive and the perimeter had such a large radius, securing it was a necessity to counter the threat level.

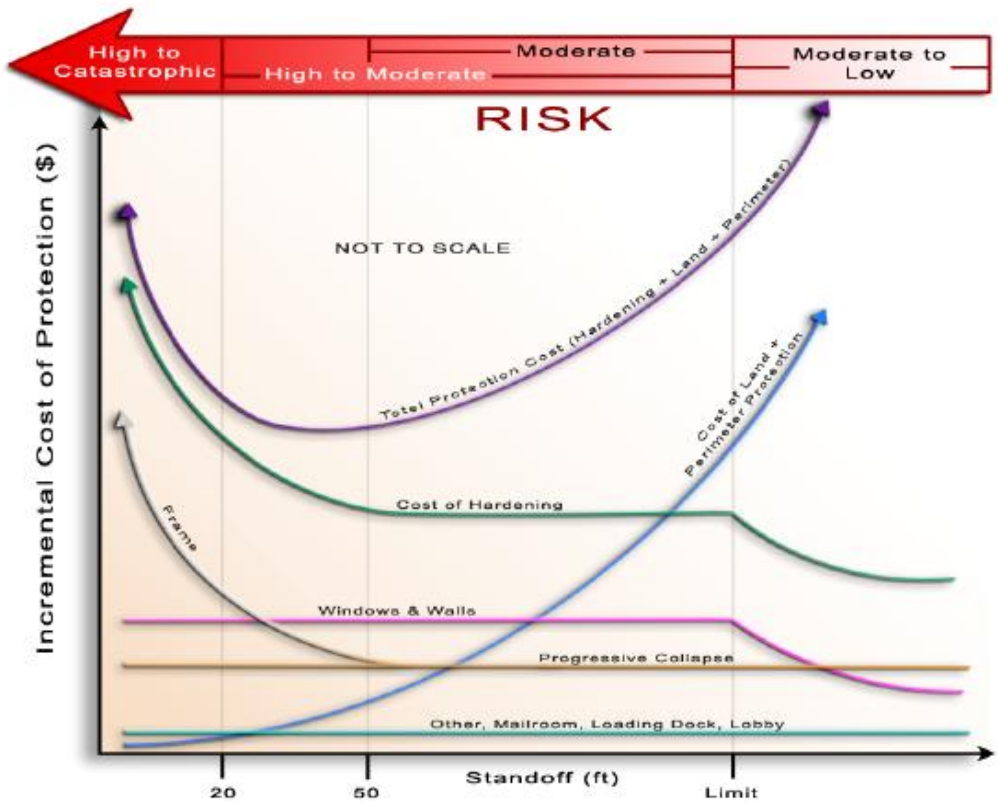


FIGURE 32 – Standoff vs. Cost of protection

CONSTRUCTION BREADTH

The purpose of this breadth was to breakdown the costs and schedule implications of re-designing the lateral system. A comparison was also completed for the hypothetical situation that Building II was a 'high-profile' building and required a 'High' level of protection. The charts below summarize the cost breakdowns and number of weeks required for construction.

Original Design Cost Summary						
	Quantity	Unit	Rate	Cost	Duration	Unit
Tons of Steel	671.5	Tons	$1,850	$1,242,364	-	-
Gravity Fab. & Erection	480.0	Tons	$400	$191,993	8	weeks
Lateral Fab. & Erection	191.6	Tons	$1,000	$191,565		
Connections R=3	120	Ea	-	-		
Foundation	1090.0	Cy	$275	$299,750	4	weeks
			Total cost and Duration	$1,925,672	12	weeks

TABLE 10 – Original Design Cost summary

Lateral Redesign Cost Summary						
	Quantity	Unit	Rate	Cost	Duration	
Tons of Steel	581.9	Tons	$1,850	$1,076,496	-	-
Gravity Fab. & Erection	480.0	Tons	$400	$191,993	9	weeks
Lateral Fab & Erection	55.1	Tons	$1,000	$55,145		
R=3	52.0	Ea		-		
Lateral Fab & Erection	42.4	Tons	$1,200	$50,903		
R=4.5	40.0	Ea	-	-		
Lateral Fab & Erection	4.3	Tons	$500	$2,165		
Brace	40.0	Ea	-	-		
Foundation: Spread	395.0	CY	$275	$108,625	2	weeks
Pile Cap	81.0	Cy	$275	$22,275	1/2	week
Piles	600.0	VLF	$10	$6,000	1/2	week
Total cost and Duration				$1,513,601	12	weeks

Cost +/-	% +/-
-$412,071	-21.40%

TABLE 11 – Lateral Redesign Cost Summary

Progressive Collapse (Direct) + Lateral Redesign Cost Summary						
	Quantity	Unit	Rate	Cost	Duration	
Tons of Steel	646.4	Tons	$1,850	$1,195,765	-	-
Gravity Fab. & Erection	531.3	Tons	$400	$212,506	10	weeks
Lateral Fab & Erection	47.2	Tons	$1,000	$47,206		
R=3	52.0	Ea	-	-		
Lateral Fab & Erection	36.3	Tons	$1,200	$43,575		
R=4.5	40.0	Ea	-	-		
Lateral Fab & Erection	27.2	Tons	$1,400	$38,128		
SidePlate Conn.	30.0	Ea	-	-		
Lateral Fab & Erection	4.3	Tons	$500	$2,150		
Brace	40.0	Ea	-	-		
Foundation: Spread	500.0	CY	$275	$137,500	2	weeks
Pile Cap	81.0	Cy	$275	$22,275	1/2	week
Piles	600.0	VLF	$10	$6,000	1/2	week
				Total cost and Duration	$1,705,104	13 weeks

Cost +/-	% +/-
-$220,568	-11.45%

TABLE 12 – Direct Method Cost Summary

Progressive Collapse (Indirect) + Lateral Redesign Cost Summary						
	Quantity	Unit	Rate	Cost	Duration	
Tons of Steel	581.9	Tons	$1,850	$1,076,496	-	-
Gravity Fab. & Erection	390.0	Tons	$400	$156,000		
Lateral Fab & Erection	145.2	Tons	$1,000	$145,200	11	weeks
R=3	200.0	Ea	-	-		
Lateral Fab & Erection	42.4	Tons	$1,200	$50,880		
R=4.5	40.0	Ea		-		
Lateral Fab & Erection	4.3	Tons	$500	$2,150		
Brace	40.0	Ea	-	-		
Foundation: Spread	395.0	CY	$275	$108,625	2	weeks
Pile Cap	81.0	Cy	$275	$22,275	1/2	week
Piles	600.0	VLF	$10	$6,000	1/2	week
				Total cost and Duration	$1,567,626	14 weeks

Cost +/-	% +/-
-$358,046	-18.59%

TABLE 13 – Indirect Method Cost Summary

As you can see from the charts above, the lateral redesign saved over $400,000 dollars or 21% to the overall structure. This included total tonnage of steel, the fabrication and erection and foundation costs. The new lateral design uses 13% less steel which reduces the overall weight of the building. The decrease in weight results in smaller foundation costs as well. Since the new design incorporates braced frames, which take the majority of the lateral load, piles and pile caps were used as the foundation for braced frames and moment frames. The piles were precast concrete and the pile caps were significantly smaller than the original spread footings used. These changes to the foundation saved a week on construction time and compensated for extra fabrication time required for the intermediate moment frames. Therefore, schedule was not impacted and the new lateral system proved to be a better design all around.

The second part of the structural depth encompassed designing the structure to mitigate the risk of progressive collapse. The original goal was to accomplish this by using the lateral redesign and integrate the progressive collapse design with the costs saved. Two methods were completed, a direct method and an indirect method. Both were accomplished without exceeding the original costs of the building. As stated previously, the progressive collapse design was concentrated to a specific load path in lieu of an alternate load path. If the entire exterior of the building was to be designed to meet GSA's alternate load path criteria the costs would have surpassed the original design. As far as construction implications, the increased number of connections and extra fabrication time adds multiple weeks onto the construction process as you can see from the charts.

Summary of Cost Analysis				
	Total Cost of Structure	+/- Costs	+/- %	Total Project +/-%
Original Design	$1,925,672	-	-	-
Lateral Redesign	$1,513,601	-$412,071	-21.40%	-2.83%
Direct Method PC*	$1,705,104	-$220,568	-11.45%	-1.52%
Indirect Method PC	$1,567,626	-$358,046	-18.59%	-2.46%

* Specific Load Path in lieu of Alternative Load Path

TABLE 14 – Overall cost summary of structural depth

Although the indirect method proved to be only slightly more cost efficient than the direct method it must be noted that the direct method only considered three bays. If the actual criteria were followed, then the entire perimeter would have analyzed redesigned increasing costs significantly. The costs results correspond with the level of threat. The more risk a building is exposed to the more costs are related to obtain the required level of safety.

A summary of the additional site costs is shown in table 15 and 16 below.

Additional Site Costs					
	quantity	unit	unit price	amount	
Bollards	12.0	Ea	$600	$7,200	Redesign
Guard Booth	3.0	booth	$25,000	$75,000	
Security Fence	1926.0	LF	$130	$250,380	
Additional Site Costs	------------------------->			$332,580	
Original Total	------------------------->			$3,972,996	Original
New Total	------------------------->			$4,305,576	
				8.37%	

TABLE 15 – Additional Site Costs to Secure Perimeter

Additional Façade costs					
	Quantity	Unit	Unit Price	Amount	
Glazing & Curtain Wall	28177.0	SF	$75	$2,113,275	original
Glazing & Curtain Wall	28177.0	SF	$150	$4,226,550	Redesign
				100%	

TABLE 16 – Additional Façade Costs (RS Means – Cost Works)

Reviewing Figure 32, and comparing costs for Building II it was concluded that securing the perimeter was more cost efficient in lieu of additional hardening to the façade. In Building II's case, hardening the façade outweighs the securing the perimeter in cost. This is due to the large amount of glass that the façade contains. Securing the perimeter is less than $1,000,000 while hardening the façade is over $4,000,000 due to the expensive costs of glazing.

OVERALL CONCLUSION

Several studies were conducted in this report to find the most economical lateral system. The comparison of three alternative systems to the original design was completed and a redesign was finalized. Due to architectural restrictions the structural system was limited to original system of composite steel framing and the use of moment frames. However, the finalized lateral resisting system was designed with separate Response Modification Factors in each direction and the use of (2) two-story “X” braces combined with moment frames. This is compared to the original design which had four oversized moment frames in each direction. Overall, the new design used 13% less steel than the original design and the overall structure cost about 21% less. Not only was the amount of steel reduced but the overall cost of the foundations was also reduced with no effect to the schedule. This achieved the original goal.

The second part of the depth was to design a portion of the building to mitigate the risk of Progressive Collapse. For the purpose of this thesis the building occupied a hypothetical client of government or ‘high profile’ stature. With the building now being considered ‘performance based’ or high profile it was subjected to abnormal loading from an explosion or blast from a terrorist attack. Following recommendations from the GSA, the building was analyzed with a linear static approach. Two methods were analyzed and compared; an Indirect method and Direct method. Both methods coupled with the new lateral design proved to be more cost efficient than the original design which was the intended goal.

The architecture breadths were based off a similar premise that Building II was considered a ‘high-profile’ type of building. The scope of the breadth was to analyze the original site layout and redesign to mitigate the risk of an attack. Additional costs came from hardening the perimeter with fences, bollards, etc. With the goal to keep the Architecture of the façade untouched, the glazing and precast panel connections were evaluated redesigned to better resist abnormal blast loadings. To achieve a balanced design, it was concluded after a cost analysis that securing the site was more cost efficient than adding additional hardening to the building.

Overall, I would conclude that most of the goals from the proposal were met including redesigning a more cost efficient structure and using the savings to design against progressive collapse.

APPENDICES

APPENDIX A

LATERAL REDESIGN (R-VALUE STUDY)

ORIGINAL DESIGN

VI. Seismic Loads: IBC 2003

Seismic Use Group :	I
Importance Factor (I) :	1.00
Site Class :	C
Ss (0.2 sec) =	18.30 %g
S1 (1.0 sec) =	6.40 %g

Zip Code search for Ss & S1.

Latitude & Longitude from address	
then Ss & S1 from latitude & longitude using 1997 USGS.	then Ss & S1 from latitude & longitude using 2002 USGS.

Fa =	1.200	Sms =	0.220	Sds =	0.146	Design Category =	A
Fv =	1.700	Sm1 =	0.109	Sd1 =	0.073	Design Category =	B

Seismic Design Category =	**B**
Number of Stories:	5
Structure Type:	Moment-resisting frame systems of steel
Plan Structural Irregularities:	No plan Irregularity
Vertical Structural Irregularities:	No vertical Irregularity
Flexible Diaphragms:	No
Building System:	**Structural steel systems not specifically detailed for seismic resistance**
Seismic resisting system:	**Structural steel systems not specifically detailed for seismic resistance**
System Building Height Limit:	**Height not limited**
Actual Building Height (hn) =	68.0 ft

DESIGN COEFFICIENTS AND FACTORS

		IBC2003 Simplified Analysis
Response Modification Factor (R) =	3	3
System Over-Strength Factor (π □) =	3	3
Deflection Amplification Factor (Cd) =	3	3
Sds =	0.146	
Sd1 =	0.073	

Code Reference Section for Detailing : AISC & AISI

π = redundancy coefficient

Seismic Load Effect (E) = π<3 Q_E +/- $0.2S_{DS}$ D = π Q_E +/- 0.029D

Q_E = horizontal seismic force

Special Seismic Load Effect (Em) = π o Q_E +/- $0.2S_{DS}$ D = 3.0 Q_E +/- 0.029D

D = dead load

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis (Seismic Category A only) Method Not Permitted

Simplified Analysis Method Not Permitted

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) =	0.028			Cu =	1.70
Approx fundamental period (Ta) =	$C_T h_n^x$ =	0.819 sec	x= 0.80	Tmax = CuTa =	1.392
User calculated fundamental period (T) =		2.83 sec 8		Use T =	1.392
Seismic response coef. (Cs) =	SdsI/R =	0.049			
need not exceed Cs =	Sd1 I /RT =	0.017			
but not less than Cs =	0.044SdsI =	0.006			
USE Cs =		0.017			

Design Base Shear V = 0.017W

Model, Linear & Nonlinear Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift = 0.020hsx where hsx is the story height below level x

OPTION A-1

VI. Seismic Loads: ASCE 7- 05

Occupancy Category: II
Importance Factor (I) : 1.00

Site Class : C

Zip Code search for Ss & S1.

Latitude & Longitude from address	
then Ss & S1 from latitude & longitude using 1997 USGS.	then Ss & S1 from latitude & longitude using 2002 USGS.

Ss (0.2 sec) = 18.30 %g
S1 (1.0 sec) = 6.40 %g

Fa =	1.200	Sms =	0.220	Sds =	0.146	Design Category =	A
Fv =	1.700	Sm1 =	0.109	Sd1 =	0.073	Design Category =	B

Seismic Design Category = **B**

Number of Stories: 5
Structure Type: Moment-resisting frame systems of steel
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: No
Building System: **Moment-resisting Frame Systems**
Seismic resisting system: **Special steel moment frames**
System Building Height Limit: **Height not limited**
Actual Building Height (hn) = 68.0 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Factor (R) = 8
System Over-Strength Factor (Ω_o) = 3
Deflection Amplification Factor (Cd) = 5.5
Sds = 0.146
Sd1 = 0.073

Seismic Load Effect (E) = ρQ_E +/- $0.2S_{DS}$ D = ρ Q_E +/- 0.029D
Special Seismic Load Effect (E) = $\Omega_o Q_E$ +/- $0.2S_{DS}$ D = 3.0 Q_E +/- 0.029D

ρ = redundancy coefficient
Q_E = horizontal seismic force
D = dead load

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis (Seismic Category A only) Method Not Permitted

Simplified Analysis Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) = 0.028 Cu = 1.70
Approx fundamental period (Ta) = $C_T h_n^x$ = 0.819 sec x= 0.80 Tmax = CuTa = 1.392
User calculated fundamental period (T) = 2.83 sec Use T = 1.392
Long Period Transition Period (TL) = ASCE7 map = 8
Seismic response coef. (Cs) = SdsI/R = 0.018
need not exceed Cs = Sd1 I /RT = 0.007
but not less than Cs = 0.010
USE Cs = 0.010
Design Base Shear V = 0.010W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift = 0.020hsx where hsx is the story height below level x

VI. Seismic Loads: ASCE 7- 05

Occupancy Category: II
Importance Factor (I) : 1.00
Site Class : C

Ss (0.2 sec) = 18.30 %g
S1 (1.0 sec) = 6.40 %g

Zip Code search for Ss & S1.

Latitude & Longitude from address	
then Ss & S1 from latitude & longitude using 1997 USGS.	then Ss & S1 from latitude & longitude using 2002 USGS.

Fa =	1.200	Sms =	0.220	Sds =	0.146	Design Category =	A
Fv =	1.700	Sm1 =	0.109	Sd1 =	0.073	Design Category =	B

Seismic Design Category = **B**
Number of Stories: 5
Structure Type: Moment-resisting frame systems of steel
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: No
Building System: **Dual Systems w/ intermediate Moment Frames Capable of Resisting >= 25% of Seismic Forces**
Seismic resisting system: **Special steel concentrically braced frames (see code footnote)**
System Building Height Limit: **Height not limited**
Actual Building Height (hn) = 68.0 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Factor (R) = 6
System Over-Strength Factor (ﾉ □) = 2
Deflection Amplification Factor (Cd) = 5
Sds = 0.146
Sd1 = 0.073

ﾉ = redundancy coefficient
Seismic Load Effect (E) = ﾉ□3 Q_E +/- 0.2S_{DS} D = ﾉ Q_E +/- 0.029D
Q_E = horizontal seismic force
Special Seismic Load Effect (E) = ﾉ o Q_E +/- 0.2S_{DS} D = 2.0 Q_E +/- 0.029D
D = dead load

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis (Seismic Category A only) Method Not Permitted

Simplified Analysis Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted
Building period coef. (C_T) = 0.028 Cu = 1.70
Approx fundamental period (Ta) = $C_T h_n^x$ = 0.819 sec x= 0.80 Tmax = CuTa = 1.392
User calculated fundamental period (T) = 2.83 sec Use T = 1.392
Long Period Transition Period (TL) = ASCE7 map = 8
Seismic response coef. (Cs) = SdsI/R = 0.024
need not exceed Cs = Sd1 I /RT = 0.009
but not less than Cs = 0.010
USE Cs = 0.010
Design Base Shear V = 0.010W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift = 0.020hsx where hsx is the story height below level x

OPTION B-2 (Final Design)

VI. Seismic Loads: ASCE 7- 05

Occupancy Category:	II		
Importance Factor (I) :	1.00		
Site Class :	C		
Ss (0.2 sec) =	18.30 %g		
S1 (1.0 sec) =	6.40 %g		

Zip Code search for Ss & S1.

Latitude & Longitude from address

then Ss & S1 from latitude & longitude using 1997 USGS.	then Ss & S1 from latitude & longitude using 2002 USGS.

Fa =	1.200	Sms =	0.220	Sds =	0.146	Design Category =	A
Fv =	1.700	Sm1 =	0.109	Sd1 =	0.073	Design Category =	B

Seismic Design Category = **B**

Number of Stories: 5
Structure Type: Moment-resisting frame systems of steel
Horizontal Struct Irregularities: No plan Irregularity
Vertical Structural Irregularities: No vertical Irregularity

Flexible Diaphragms: No
Building System: **Dual Systems w/ intermediate Moment Frames Capable of Resisting >= 25% of Seismic Forces**
Seismic resisting system: **Special steel concentrically braced frames (see code footnote)**
System Building Height Limit: **Height not limited**
Actual Building Height (hn) = 68.0 ft

DESIGN COEFFICIENTS AND FACTORS

Response Modification Factor (R) = 4.5
System Over-Strength Factor (0 □) = 3
Deflection Amplification Factor (Cd) = 4
Sds = 0.146
Sd1 = 0.073

0 = redundancy coefficient
Seismic Load Effect (E) = 0□□3 Q_E +/- 0.2S_{DS} D = 0 Q_E +/- 0.029D
Special Seismic Load Effect (E) = 0 o Q_E +/- 0.2S_{DS} D = 3.0 Q_E +/- 0.029D

Q_E = horizontal seismic force
D = dead load

PERMITTED ANALYTICAL PROCEDURES

Index Force Analysis (Seismic Category A only) Method Not Permitted

Simplified Analysis Use Equivalent Lateral Force Analysis

Equivalent Lateral-Force Analysis - Permitted

Building period coef. (C_T) =	0.028			Cu = 1.70
Approx fundamental period (Ta) =	$C_T h_n^x$ =	0.819 sec	x= 0.80	Tmax = CuTa = 1.392
User calculated fundamental period (T) =		2.83 sec		Use T = 1.392
Long Period Transition Period (TL) =	ASCE7 map =	8		
Seismic response coef. (Cs) =	SdsI/R =	0.033		
need not exceed Cs =	Sd1 I /RT =	0.012		
but not less than Cs =		0.010		
USE Cs =		0.012		

Design Base Shear V = 0.012W

Model & Seismic Response Analysis - Permitted (see code for procedure)

ALLOWABLE STORY DRIFT

Structure Type: All other structures

Allowable story drift = 0.020hsx where hsx is the story height below level x

APPENDIX B

LATERAL CALCULATIONS

DESIGN WIND PRESSURES

WIND CALCULATIONS – ASCE 7-05, METHOD 2

- BASIC WIND, $V = 90$ MPH (FIGURE 1) (6.5.4)
- EXPOSURE B (6.5.6.3)
- OCCUPANCY CATEGORY II – NON-HURRICANE
- IMPORTANCE FACTOR, $I = 1.0$ (6.5.5)
- DIRECTIONALITY FACTOR, $K_D = .85$ (6.5.4)
- TOPOGRAPHIC FACTOR, $K_{zt} = 1.0$ (6.5.7)
  → NO HILLS, RIDGES, ESCARPMENTS

→ DESIGN WIND PRESSURE ON PARAPET

PARAPET h = 76.5"

↳ BY LINEAR INTERPOLATION $K_H = .919$

- VELOCITY PRESSURE q_P ON PARAPET

$$q_P = .00256(K_h)(K_{zt})(K_d)(V^2)I$$

$$= .00256(.919)(1.0)(.85)(90^2)(1.0)$$

$$\boxed{q_P = 16.20 \text{ PSF}}$$

- COMBINED NET PRESSURE COEFFICIENT, GC_{pn} (6.5.12.2.4)

$\boxed{GC_{pn} = 1.5}$ WINDWARD PARAPET

$\boxed{GC_{pn} = -1.0}$ LEEWARD PARAPET

- COMBINED NET DESIGN PRESSURE P_P

$$P_P = q_P GC_{pn}$$

$= 16.20(1.5) = +24.30$ PSF ON WINDWARD PARAPET

$= 16.20(-1.0) = -16.20$ PSF ON LEEWARD PARAPET

FORCES: $24.3(8.5') = 206.6$ PLF (WW)

$-16.2(8.5') = 137.7$ PLF (LW)

WIND CALCS | ASCE 7-05

GUST EFFECT FACTOR - FLEXIBLE STRUCTURE

$T_x = 2.89$, $n_1 = .346 < 1$ ∴ FLEXIBLE
$T_y = 2.02$, $n_1 = .496$

$$G_f = .925\left(\frac{1 + 1.7 I_z \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_z}\right)$$

$\epsilon = 1/3$
$l = 320$
$z_{min} = 30' < 40.8 \Rightarrow 40.8$
$c = .3$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{.577}{\sqrt{2\ln(3600 n_1)}}$$

$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = .3\left(\frac{33}{40.8}\right)^{1/6} = .29$$

$$L_{\bar{z}} = l\left(\frac{\bar{z}}{33}\right)^{\epsilon} = 343$$

$g_{R_x} = 3.928$

g_Q & $g_v = 3.4$

$g_{R_y} =$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (.53 + .47 R_L)}$$

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

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

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

$\bar{\alpha} = 1/4$
$\bar{b} = .45$

$$R_l = \frac{1}{\eta} - \frac{1}{2\eta^2}(1 - e^{-2\eta})$$

$Q = .84$ (PREVIOUS CALCULATION)

R_l: R_h setting $\eta = 4.6 n_1 h / \bar{V}_{\bar{z}}$
R_l: R_B setting $\eta = 4.6 n_1 B / \bar{V}_{\bar{z}}$
R_l: R_L setting $\eta = 15.4 n_1 L / \bar{V}_{\bar{z}}$

$\beta = .01$

$N_{1x} = 1.90$

$N_{1y} = 2.72$

$$\bar{V}_{\bar{z}} = 0.45\left(\frac{40.8}{33}\right)^{.25}(90)\left(\frac{88}{60}\right) = 62.6$$

X - DIRECTION		Y - DIRECTION	
N_1	1.90	N_1	2.72
R_n	.092	R_n	.074
R_B	.284	R_B	.210
R_L	.042	R_L	.029
R_h	.417	R_h	.323
R	.773	R	.523

$$G_{f_x} = .925\left(\frac{1 + 1.7(.29)\sqrt{3.4^2(.84)^2 + (3.93^2)(.773)^2}}{1 + 1.7(3.4)(.29)}\right) = 1.055$$

$$G_{f_y} = .925\left(\frac{1 + 1.7(.29)\sqrt{3.4^2(.84)^2 + 3.93^2(.523^2)}}{1 + 1.7(3.4)(.29)}\right) = .949$$

VELOCITY PRESSURES q_z & q_h

$q_z = .00256\, K_z K_{zt} K_d V^2 I$

$= .00256\, K_z (1.0)(.85) 90^2 (1.0)$

$q_z = 12.63\, K_z$

↳ VARIES SEE TABLE

PRESSURE COEFFIENTS, C_p (CASE 2 APPLIES
EXPOSURE B)

• WALL PRESSURE COEFFIENTS, C_p

EAST - WEST WIND

• $L/B = \frac{275}{115} = 2.39$

WINDWALL WALL:	$C_p = .8$
LEEWARD WALL:	$C_p = -.28$
SIDE WALL:	$C_p = -.7$

NORTH - SOUTH WIND

• $L/B = 115/275 = .42$

$C_p = .8$
$C_p = -.5$
$C_p = -.7$

• ROOF PRESSURE COEFFIENTS, C_p

EAST - WEST

$h/L = 68/275 = .25 \leq .5$

$0 \to h/2$:	$C_p = -.9$,
$h/2 \to h$:	$C_p = -.9$,
$h \to 2h$:	$C_p = -.5$,
$> 2h$:	$C_p = -.3$

NORTH - SOUTH

$h/L = 68/115 = .591$

$C_p = -.93$
$C_p = -.86$
$C_p = -.54$
INTERPOLATED

$q_i = q_h = 15.6$ PSF (6.5.12.2.1)

• INTERNAL PRESSURE COEFFICIENTS (GC_{pi})

FIGURE 6.5 → ENCLOSED BLDG'S → $\boxed{GC_{pi} = \pm .18}$

DESIGN PRESSURES, p_z & p_h

EAST-WEST (X-DIRECTION)

WINDWARD: $p_z = q_z G C_p - q_h (G C_{pi})$
$= q_z (1.055)(.8) - 15.6(\pm .18)$

$p_z = .84 q_z \pm 2.80$ (EXT. & INT. PRESSURE)

LEEWARD, SIDE, ROOF: $p_h = q_h G C_p - q_h G C_{pi}$
$= 15.6(1.055) C_p - 15.6(\pm .18)$

$p_h = 16.5 C_p \pm 2.80$ (EXT & INT PRESS)

<u>SEE TABLE</u>

NORTH-SOUTH (Y-DIRECTION

WINDWARD: $p_z = q_z G C_p - q_h (G C_{pi})$
$= q_z (.949)(.8) - 15.6(\pm .18)$

$p_z = .76 q_z \pm 2.80$

LEEWARD, SIDE & ROOF: $p_h = q_h G C_p - q_h G C_{pi}$
$= 15.6(.949) C_p - 15.6(\pm .18)$

$p_h = 14.8 C_p \pm 2.80$

<u>SEE TABLE</u>

DESIGN WIND PRESSURES

Design Wind Pressures, p in the E-W Direction						
Location	Height above ground z(ft)	q (psf)	External pressure qGCp (psf)	Internal pressure qGCp (psf)	Net Pressure p (psf)	
					(+GCpi)	(-GCpi)
Windward	0-15	10.05	8.66	± 2.80	5.86	11.46
	20	10.93	9.42	± 2.80	6.62	12.22
	25	11.99	10.34	± 2.80	7.54	13.14
	30	12.34	10.64	± 2.80	7.84	13.44
	40	13.40	11.55	± 2.80	8.75	14.35
	50	14.28	12.31	± 2.80	9.51	15.11
	60	14.98	12.92	± 2.80	10.12	15.72
	68	15.60	13.45	± 2.80	10.65	16.25
Leeward	ALL	15.60	-4.71	± 2.80	-7.51	-1.91
Side	ALL	15.60	-11.77	± 2.80	-14.57	-8.97
Roof	68	15.60	-15.14 °	± 2.80	-17.94	-12.34
	68	15.60	-8.41 †	± 2.80	-11.21	-5.61
	68	15.60	-5.05 ‡	± 2.80	-7.85	-2.25

° from windward edge to 68 ft

† from 68 to 136 ft

‡ from 136 to 275 ft

Design Wind Pressures, p in the N-S Direction						
Location	Height above ground z(ft)	q (psf)	External pressure qGCp (psf)	Internal pressure qGCp (psf)	Net Pressure p (psf)	
					(+GCpi)	(-GCpi)
Windward	0-15	10.05	6.91	± 2.80	4.11	9.71
	20	10.93	7.52	± 2.80	4.72	10.32
	25	11.99	8.25	± 2.80	5.45	11.05
	30	12.34	8.49	± 2.80	5.69	11.29
	40	13.40	9.22	± 2.80	6.42	12.02
	50	14.28	9.82	± 2.80	7.02	12.62
	60	14.98	10.31	± 2.80	7.51	13.11
	68	15.60	10.73	± 2.80	7.93	13.53
Leeward	ALL	15.60	-6.71	± 2.80	-9.51	-3.91
Side	ALL	15.60	-9.39	± 2.80	-12.19	-6.59
Roof	68	15.60	-12.48 °	± 2.80	-15.28	-9.68
	68	15.60	-11.54 †	± 2.80	-14.34	-8.74
	68	15.60	-7.24 ‡	± 2.80	-10.04	-4.44

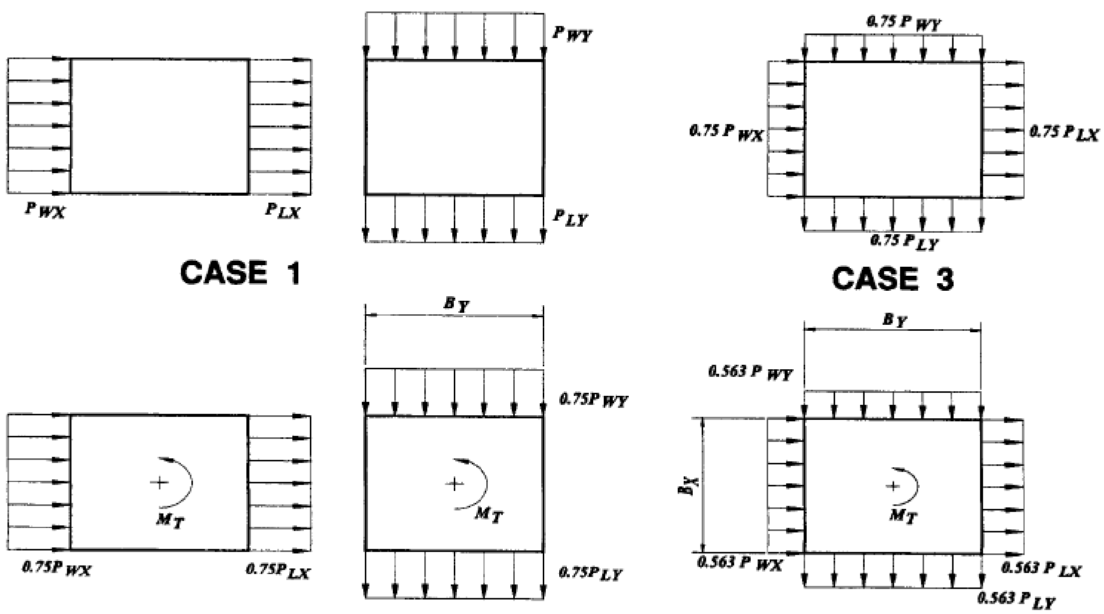
° from windward edge to 34 ft

† from 34 to 68 ft

‡ from 68 to 115 ft

CONTROLLING WIND CASES

Main Wind Force Resisting System–Method 2		All Heights
Figure 6-9	Design Wind Load Cases	



$M_T = 0.75\,(P_{WX}+P_{LX})B_X e_X$ $\quad e_X = \pm\,0.15\,B_X$

$M_T = 0.75\,(P_{WY}+P_{LY})B_Y e_Y$ $\quad e_Y = \pm\,0.15\,B_Y$

CASE 2

$M_T = 0.563\,(P_{WX}+P_{LX})B_X e_X + 0.563\,(P_{WY}+P_{LY})B_Y e_Y$ $\quad e_X = \pm\,0.15\,B_X \quad e_Y = \pm\,0.15\,B_Y$

CASE 4

Case 1. Full design wind pressure acting on the projected area perpendicular to each principal axis of the structure, considered separately along each principal axis.

Case 2. Three quarters of the design wind pressure acting on the projected area perpendicular to each principal axis of the structure in conjunction with a torsional moment as shown, considered separately for each principal axis.

Case 3. Wind loading as defined in Case 1, but considered to act simultaneously at 75% of the specified value.

Case 4. Wind loading as defined in Case 2, but considered to act simultaneously at 75% of the specified value.

Notes:

1. Design wind pressures for windward and leeward faces shall be determined in accordance with the provisions of 6.5.12.2.1 and 6.5.12.2.3 as applicable for building of all heights.
2. Diagrams show plan views of building.
3. Notation:
   - P_{WX}, P_{WY}: Windward face design pressure acting in the x, y principal axis, respectively.
   - P_{LX}, P_{LY}: Leeward face design pressure acting in the x, y principal axis, respectively.
   - e (e_X, e_Y) : Eccentricity for the x, y principal axis of the structure, respectively.
   - M_T: Torsional moment per unit height acting about a vertical axis of the building.

CASE 1	
Px (E-W)	Total Px
Parapet	34.5
Roof	14.5
5.0	26.9
4.0	24.7
3.0	22.3
2.0	21
Base	143.9
Py (N-S)	Total Py
Parapet	45.5
Roof	34.7
5.0	64.8
4.0	60.3
3.0	55.7
2.0	53.7
Base	314.7

CASE 2				
	.75Px (E-W)	Mt	Torsion, Px	Total Px
Parapet	25.9	446.3	1.15	29.8
Roof	10.9	187.6	1.15	12.5
5.0	20.2	348.0	1.15	23.2
4.0	18.5	319.6	1.15	21.3
3.0	16.7	288.5	1.15	19.2
2.0	15.8	271.7	1.15	18.1
Base				124.1
	.75Py (N-S)	Mt	Torsion, Px	Total Py
Parapet	34.1	2580.7	1.275	43.5
Roof	26.0	1968.1	1.275	33.2
5.0	48.6	3675.4	1.275	62.0
4.0	45.2	3420.1	1.275	57.7
3.0	41.8	3159.2	1.275	53.3
2.0	40.3	3045.8	1.275	51.4
Base				300.9

Controls

*confirmed by RAM analysis

CASE 3	
.75Px (E-W)	Total Px
Parapet	25.88
Roof	10.88
5.0	20.18
4.0	18.53
3.0	16.73
2.0	15.75
Base	107.9
.75Py (N-S)	Total Py
Parapet	34.13
Roof	26.03
5.0	48.60
4.0	45.23
3.0	41.78
2.0	40.28
Base	236.0

CASE 4				
	.563Px (E-W)			Total Px
Parapet	19.4	335.1	1.15	22.3
Roof	8.2	140.8	1.15	9.4
5.0	15.1	261.2	1.15	17.4
4.0	13.9	239.9	1.15	16.0
3.0	12.6	216.6	1.15	14.4
2.0	11.8	203.9	1.15	13.6
Base				93.2
	.563Py (N-S)	Mt	Torsion, Px	Total Py
Parapet	25.6	441.9	1.275	32.7
Roof	19.5	337.0	1.275	24.9
5.0	36.5	629.3	1.275	46.5
4.0	33.9	585.6	1.275	43.3
3.0	31.4	540.9	1.275	40.0
2.0	30.2	521.5	1.275	38.5
Base				225.9

DESIGN WIND PRESURES

Design Wind Pressures E-W					
Floor	Height	q (psf)	windward q	leeward q	total pressure q
	76.50	16.20			16.20
Roof	68.00	15.60	13.45	4.61	18.06
5	54.00	14.56	12.56	4.61	17.17
4	40.75	13.46	11.61	4.61	16.22
3	27.50	11.63	10.03	4.61	14.64
2	14.25	10.05	8.66	4.61	13.27

Design Wind Pressures N-S					
Floor	Height	q (psf)	windward q	leeward q	total pressure q
	76.50	16.20			16.20
Roof	68.00	15.60	10.73	7.40	18.13
5	54.00	14.56	10.02	7.40	17.42
4	40.75	13.46	9.26	7.40	16.66
3	27.50	11.63	8.00	7.40	15.40
2	14.25	10.05	6.91	7.40	14.31

DESIGN WIND FORCES

Lateral Forces E-W Direction, Width = 115'						
	Force	Factored Force	Story Shear	Factored Shear	Moment	Factored Moment
Floor	Fx, (k)	Fx * 1.6 (K)	V, (k)	V, (k)	M (ft-k)	M, (ft-k)
Parapet	34.6	55.3	-	-	2349.4	787.7
Roof	14.5	23.3	34.6	55.3	988.8	331.5
5.0	26.9	43.0	14.5	78.5	1452.4	613.2
4.0	24.7	39.5	41.4	121.6	1007.1	563.5
3.0	22.3	35.7	66.2	161.1	613.6	508.7
2.0	21.0	33.6	88.5	196.8	299.1	478.6
-	-	-	109.5	230.4	-	-
Base Shear	144.0	230.4		Overturning Moment	6710.3	3283.2

Lateral Forces N-S Direction, Width = 275'						
	Force	Factored Force	Story Shear	Factored Shear	Moment	Factored Moment
Floor	Fx, (k)	Fx * 1.6 (K)	V, (k)	V, (k)	M (ft-k)	M, (ft-k)
Parapet	45.4	72.7				
Roof	34.7	55.4	45.4	72.7	2356.3	3770.1
5.0	64.8	103.7	80.1	128.1	3498.3	5597.2
4.0	60.3	96.4	144.9	231.8	2456.0	3929.5
3.0	55.7	89.1	205.1	328.2	1532.2	2451.6
2.0	53.7	86.0	260.9	417.4	765.6	1224.9
-	-	-	314.6	503.3	-	-
Base Shear	314.6	503.3		Overturning Moment	10608.4	16973.4

TORSION CALCUATIONS WIND DESIGN

Torsion Rigidity (J) & C (COG) - Controlling NS									
Level	Frame 9 (C)	R*C²	Frame 10 (C)	R*C²	Brace 11 (C)	R*C²	Brace 12 (C)	R*C²	J=ΣR*C²
roof	75.00	3.96	75.00	6.65	75.00	63.67	75.00	72.55	146.81
5.00	75.00	3.96	75.00	6.65	75.00	63.67	75.00	72.55	146.81
4.00	75.00	3.96	75.00	6.65	75.00	63.67	75.00	72.55	146.81
3.00	75.00	3.96	75.00	6.65	75.00	63.67	75.00	72.55	146.81
2.00	75.00	3.96	75.00	6.65	75.00	63.67	75.00	72.55	146.81

Torsion Rigidity (J) & C (COR) - Controlling EW									
Level	Frame 5 (C)	R*C²	Frame 6 (C)	R*C²	Frame 7 (C)	R*C²	Frame 8 (C)	R*C²	J=ΣR*C²
Roof	54.09	84.71	54.09	76.72	59.96	100.65	59.96	99.68	361.77
5.00	57.87	96.97	57.87	87.82	57.12	91.34	57.12	90.46	366.59
4.00	58.18	98.01	58.18	88.76	57.47	92.47	57.47	91.57	370.81
3.00	57.35	95.23	57.35	86.25	58.21	94.86	58.21	93.95	370.29
2.00	57.21	94.77	57.21	85.83	58.71	96.50	58.71	95.57	372.66

Direct Shear (V*Ri / ΣR)										
Level	Controlling N-S					Controlling E-W				
	V (k)	Frame 9	Frame 10	Brace 11	Brace 12	V (k)	Frame 5	Frame 6	Frame 7	Frame 8
roof	127.10	3.43	5.75	55.12	62.80	90.40	23.60	21.38	22.82	22.60
5	228.90	6.17	10.36	99.26	113.11	137.60	35.93	32.54	34.74	34.40
4	323.60	8.72	14.65	140.33	159.90	177.00	46.21	41.85	44.68	44.25
3	411.30	11.08	18.62	178.36	203.24	214.30	55.95	50.67	54.10	53.58
2	496.00	13.37	22.45	215.09	245.09	248.50	64.88	58.76	62.73	62.13

Torsional Shear V*e*Ri*C / ΣR*C²										
Level	Controlling N-S					Controlling E-W				
	V (k)	Frame 9	Frame 10	Brace 11	Brace 12	V (k)	Frame 5	Frame 6	Frame 7	Frame 8
roof	Case 1 for Wind load cases controls therefore acciedntal torison does not need to be accouted for					90.40	2.26	2.05	2.42	2.40
5						137.60	3.63	3.29	3.46	3.43
4						177.00	4.64	4.20	4.43	4.39
3						214.30	5.55	5.02	5.44	5.39
2						248.50	6.37	5.77	6.32	6.26

Total Shear (Direct ± Torsional)										
Level	Controlling N-S					Controlling E-W				
	Total V (k)	Frame 9	Frame 10	Brace 11	Brace 12	Total V (k)	Frame 5	Frame 6	Frame 7	Frame 8
roof	127.10	3.43	5.75	55.12	62.80	90.40	25.86	23.42	25.24	25.00
5	228.90	6.17	10.36	99.26	113.11	137.60	39.55	35.82	38.20	37.83
4	323.60	8.72	14.65	140.33	159.90	177.00	50.85	46.06	49.11	48.64
3	411.30	11.08	18.62	178.36	203.24	214.30	61.50	55.70	59.54	58.97
2	496.00	13.37	22.45	215.09	245.09	248.50	71.25	64.53	69.06	68.39

WIND CALCULATIONS

Center of Rigidity - RAM Output		
Level	x (ft)	y (ft)
Roof	135.9	60.91
5	135.36	57.13
4	135.34	56.82
3	135.69	57.65
2	137.01	57.79

Center of Mass - RAM Output		
Level	x (ft)	y (ft)
Roof	136.18	61.77
5	134.95	59.63
4	134.88	59.63
3	134.97	59.58
2	135.04	59.13

Center of Geometry - Hand Calculated					
Level	Σa	Σa*x	Σa*y	x (ft)	y (ft)
Roof	30879	4172020	1717514	135.11	55.62
5	30879	4172020	1717514	135.11	55.62
4	30879	4172020	1717514	135.11	55.62
3	30879	4172020	1717514	135.11	55.62
2	30879	4172020	1717514	135.11	55.62

Eccentricity, e (5%building width) - RAM Output		
Level	x (ft)	y (ft)
Roof	13.64	5.77
5	13.64	5.77
4	13.64	5.77
3	13.64	5.77
2	13.64	5.77
	5% of 274'	5% of 115'

SEISMIC DESIGN FORCES

Seismic Force Story Distribution						
Floor	w_x	h_x	k	$w_x h_x^k$	$\Sigma\, w_i h_i^k$	C_{vx}
Base	--	--	--	--	--	--
2	2740.50	14.25	1.46	131154.82	2625347.90	0.050
3	2701.80	27.50	1.46	336760.85	2625347.90	0.128
4	2692.00	40.25	1.46	584270.80	2625347.90	0.223
5	2685.70	54.00	1.46	894175.60	2625347.90	0.341
Roof	1457.90	68.00	1.46	678985.83	2625347.90	0.259

Seismic Design Forces			
Floor	F_x (Kips)	Story Shear Vx	Moment (k-ft)
Roof	36.82	-	2503.59
5	48.49	36.82	2618.24
4	31.68	85.30	1275.18
3	18.26	116.98	502.17
2	7.11	135.25	101.34
-	-	142.36	-
Base	142.36	Overtunring Moment (k-ft)	7000.52

DRIFT CRITERIA

Wind Drift - NS Direction							
Level	Story height (Ft)	Story Drift (in)	Allowable drift (h/400)		Total Drift (in)	Allowable total (H/400)	
roof	68	0.249	0.600	ok	2.060	2.914	ok
5	54	0.347	0.568	ok	1.811	2.314	ok
4	40.75	0.467	0.568	ok	1.464	1.746	ok
3	27.5	0.525	0.568	ok	0.997	1.179	ok
2	14.25	0.472	0.611	ok	0.472	0.611	ok

* Serviceability = 0.7*Wind Force (ASCE 7-05 commentary)

Wind Drift - EW Direction							
Level	Story height (Ft)	Story Drift (in)	Allowable drift (h/400)		Total Drift (in)	Allowable total (H/400)	
roof	68	0.167	0.600	ok	0.909	2.914	ok
5	54	0.167	0.568	ok	0.742	2.314	ok
4	40.75	0.191	0.568	ok	0.575	1.746	ok
3	27.5	0.192	0.568	ok	0.384	1.179	ok
2	14.25	0.192	0.611	ok	0.192	0.611	ok

* Serviceability = 0.7*Wind Force (ASCE 7-05 commentary)

Seismic Drift - NS Direction							
Level	Story Height (ft)	Strory Drift (in)	Allowable drift (.020h)		Total Drift (in)	Allowable total (.02h)	
roof	68	0.208	3.360	ok	2.059	16.320	ok
5	54	0.358	3.180	ok	1.851	12.960	ok
4	40.75	0.502	3.180	ok	1.493	9.780	ok
3	27.5	0.537	3.180	ok	0.991	6.600	ok
2	14.25	0.454	3.420	ok	0.454	3.420	ok

Seismic Drift - EW Direction							
Level	Story Height (ft)	Strory Drift (in)	Allowable drift (.020h)		Total Drift (in)	Allowable total (.02h)	
roof	68	0.123	3.360	ok	0.713	16.320	ok
5	54	0.141	3.180	ok	0.590	12.960	ok
4	40.75	0.162	3.180	ok	0.449	9.780	ok
3	27.5	0.156	3.180	ok	0.287	6.600	ok
2	14.25	0.131	3.420	ok	0.131	3.420	ok

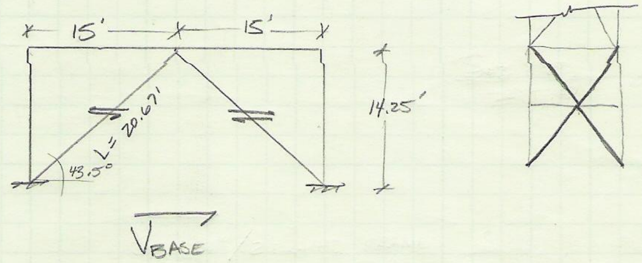
BRACING DESIGN

Bracing Design

Wind Load (Y Direction) Base Shear: 314.7

Controlling Wind Load Case = $1.6(314.7) = 503.52^k$

1st - Story & 2nd Story



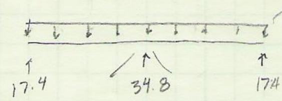
Lateral - Axial

$$\frac{503.52}{2\ BF} = 251.76^k/\text{Braced Frame} \Rightarrow 125.9^k \text{ Per Brace}$$

$$125.9^k / \cos 43.53° = 173.6^k \Rightarrow \text{Axial Force Per Brace}$$

Gravity - Axial

$1.2D + .5L = 1.2(1.65^k/ft) + .5(.678) = 2.32^k/ft$



$17.4^k/\sin 43.5 = 25.3^k$ (1/2 Load Goes to Upper Level Bracing) ∴ Use 12.65^k As Add. Gravity

Total Axial Per Brace = $173.6^k + 12.65^k = \underline{186.25^k}$

- $KL = 20.67'$, $186^k \Rightarrow \boxed{\text{HSS } 8 \times 8 \times 1/4}$ $\phi_c P_n = 193^k > 186.25^k$

$\phi_t P_n = 294^k > 186.25^k$

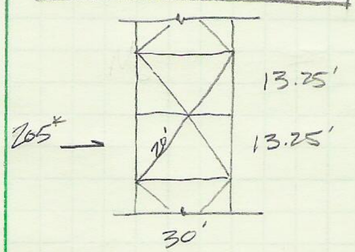
∴ ok

* Most Economical Shape Based on A_g.

Use HSS 8 x 8 x 1/4 For Story 1 & 2 Braces

BRACING DESIGN CONTINUED

$\boxed{\text{STORIES 3 \& 4}}$



LATERAL LOADS

$\frac{205(1.6)}{2 \text{ B.F.}} = 164^k = 82^k$ PER BRACE

$82^k\left(\frac{20'}{15'}\right) = \underline{109^k}$ (AXIAL FORCE PER BRACE)

GRAVITY LOADS

SAME AS STORY 1 => 17.4^k

$17.4^k\left(\frac{20}{13.25}\right) = 26.3^k$ (1/2 TO EACH STORY)

TOTAL AXIAL LOAD PER BRACE

$109^k + 13.2^k = \underline{122.2^k}$

$KL = 20'$, 122.2^k => $\boxed{\text{HHS } 7 \times 7 \times 1/4}$ * MOST ECON. BY A_g.

$\phi_c P_n = 153^k > 122.2^k$
$\phi_T P_n = 255^k > 122.2^k$ ∴ OK

USE HSS 7×7×1/4 FOR STORIES 3 & 4 BRACES

$\boxed{\text{STORY 5 (ROOF) + PARAPET FORCES}}$

LATERAL LOAD

$\frac{80.1(1.6)}{2} = 64^k$ => 32^k PER BRACE

$32^k\left(\frac{20.5}{15'}\right) = 43.7^k$ (AXIAL PER BRACE)

GRAVITY LOAD

$2.65^k\left(\frac{20.5}{14'}\right) = 3.88^k$

$\boxed{\text{HSS } 5 \times 5 \times 1/4}$

TOTAL AXIAL LOAD PER BRACE

$43.7^k + 3.88^k = \underline{47.6^k}$

$\phi_c P_n = 60^k > 47.6^k$
$\phi_t P_n = 178^k > 47.6^k$
∴ OK

BRACED FRAME COLUMN CHECK : GRAVITY ONLY : W12×87

AXIAL LOADS (FROM RAM)

$D_L = 323.33^k$
$L_L = 182.30^k$
$R_L = 29.65^k$
(GRAVITY)

$KL = 14.25'$

$W = 132.86^k$ (WIND)

$P_u = 1.2D + .5L + 1.6W$

$P_u = 1.2(323.33) + .5(182.30) + 1.6(132.86)$

$P_u = 691.7$, $KL = 14.25'$

W12×87 $\phi P_n = 916^k > 691.7^k$ ∴ OK

INTERACTION EQ.:

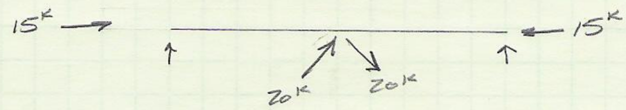
$p = 1.09E^{-3}$
$by = 3.92E^{-3}$
$bx = 1.87E^{-3}$

$pP = 1.09E^{-3}(691) = .753 < 1$ ∴ H1-1a

$.753 + 1.87E^{-3}(30.6) = .81 < 1$ ∴ ok

∴ W12×87 IS ADEQUATE

BEAM CHECK @ ROOF LEVEL (DUE TO UNEVEN VERTICAL LOAD)



$Q_b = P_{ty} - P_{cy}$

$= 20 - (-20) = 40^k$ (UNBALANCED VERT. LOAD)

AXIAL FORCE IN BEAM = 15^k

MOMENTS

$M_D = \frac{.25(30)^2}{8} = 28.1'^k$

$M_L = \frac{.4(30)^2}{8} = 45'^k$ $M_u = 1.2(28) + .5(45) + 300$

$M_{Qb} = \frac{40(30)}{4} = 300'^k$ $M_u = 356'^k$

$\Rightarrow \phi M_u = 387$, W21x62 $KL = 15'$

COLUMN CHECK - BRACED FRAME

```
RAM SColumn V1.01                              Column Design Results
                              B~Øè3
Job Name:                                          3/31/09   21:37
Comments:
Steel Code:  ASD 9th Ed.

    Fy (ksi) =  50.00                   Column Size = W14X109

INPUT DESIGN PARAMETERS:                       X-Axis    Y-Axis
    Lu (ft) ..............................  13.25      0.00
    K ....................................   1.00      1.00
    Braced Against Joint Translation.......    No        No
    Loaded Between Joints..................    No        No
    Ends Fixed Against Rotation............    No        No

COLUMN LOADS:                Design
    Axial   (kip) .......    432.0
    Top Mx (kip-ft)......    245.0
        My (kip-ft)......      0.0
    Bot Mx (kip-ft)......    245.0
        My (kip-ft)......      0.0

CALCULATED PARAMETERS:
    Allowable Stress Increase Factor.. 1.00
    fa  (ksi) =  13.50   Fa  (ksi) =  27.68
    fbx (ksi) =  16.99   Fbx (ksi) =  33.00
                        Fbx1 (ksi) =  33.00
    fby (ksi) =   0.00   Fby (ksi) =  37.50

    Single curvature about X-Axis
    Single curvature about Y-Axis

    Cb     =   1.00
    KL/Rx =  25.54    KL/Ry =    0.00
    F'ex  = 228.89    F'ey  =    0.00
    Cmx   =    0.85   Cmy   =    0.85

INTERACTION EQUATION:
    fa/Fa  = 0.49
    Eq H1-1: 0.49 + 0.47 + 0.00 = 0.95
    Eq H1-2: 0.45 + 0.51 + 0.00 = 0.96
```

IMF Connection Calculations

INTERMEDIATE MOMENT FRAME CONNECTION

- PARAMETRIC LIMITATIONS FOR PREQUALIFICATIONS (TABLE 6.1 - SEISMIC MANUAL)

END PLATE & BOLT DESIGN

- 4-BOLT UNSTIFFENED
- BEAM W27 x 84

$d = 26.7''$
$A_g = 24.8\ in^2$
$b_f = 10''$, $t_f = .640''$

COLUMN W18 x 97

$d = 18.6'$
$b_f = 11.1''$
$t_f = .870''$

$M_f = 1.2(97) + .5(123) + 1.6(244) = 568'^k$

$V_u = 1.2(17) + .5(23) + 1.6(16) = 57.5^k$

BOLT DIAMETER

$d_{b,req} = \sqrt{\dfrac{2(568)(12)}{\pi(90\,ksi)(28.38 + 23.74)(.9)}} = 1.01 \rightarrow 1\tfrac{1}{8}''\ \phi$ A325 BOLTS

END PL THICKNESS

$t_p = \sqrt{\dfrac{1.11(568)(12)}{(1.0)(36)\,618.9}} = .58 \Rightarrow 5/8''$ THICK PL

$Y_p = \dfrac{10.5}{2}\left[23.74\left(\dfrac{1}{2} + 3.43\right) + 28.38\left(\dfrac{1}{2}\right) - \dfrac{1}{2}\right] + \dfrac{2}{4.5}\left[23.74(2 + 3.43)\right] = 618.9$

$s = \tfrac{1}{2}\sqrt{10.5 \cdot 4.5} = 3.43$

FACTORED BEAM FORCE

$$F_{fu} = \frac{568(12)}{26.7 - .64} = 25.59^{K}$$

SHEAR YIELDING OF EXT. PORTION OF PL (4E)

$$\frac{25.59}{2} < 1.0(.6)(36)(10.5)(5/8)$$

$12.8^{K} < 141.75^{K}$ ∴ OK ✓

RUPTURE

$$\frac{25.59}{2} < (0.9)(.6)(58)(10.5 - 2(1\tfrac{1}{8}))(5/8)$$

$12.8^{K} < 161.5^{K}$ ∴ OK ✓

BOLT SHEAR RUPTURE

$V_u \leq .9(2)(48\ ksi)(.994)$

$57.5 \leq 86^{K}$ ∴ OK ✓

BOLT - BEARING/TEARNOUT

$V_u \leq .9(2)\underbrace{(1.2)(1.5)(5/8)(58)}_{} + .9(2)(\quad)$

$57.5 \leq 235^{K}$ ∴ OK ✓

WELDS:

FLANGE TO END PL: $\phi R_n = (1.392)(4)(2)(10'')(1.5) = 167^{K} > 25.6^{K}$ ∴ OK ($\#16^{ths}$)

WEB TO END PL: $\phi R_n = (1.392)(4)(2)(23\tfrac{5}{8}'')(1.0) = 263.1^{K} > 25.6^{K}$ ∴ OK

USE 1/4" FILLET WELDS ON BOTH FLANGE & WEB TO PL

COLUMN SIDE DESIGN

t_{cf} req'd: $\sqrt{\dfrac{1.11(568)(12)}{(1.0)(50)(226.9)}} = .817'' < .870''$ ∴ OK ✓

$Y_c = \frac{11.1}{2}\left[23.74\left(\frac{1}{3.53}\right) + 28.38\left(\frac{1}{3.53}\right)\right] + \frac{2}{4.5}\left[23.74\left(3.53 + \frac{3(4.64)}{4}\right) + 28.38\left(3.53 + \frac{4.64}{4}\right) + \frac{4.64^2}{2}\right] + \frac{4.5}{2}$

$= 226.9$

$S = \frac{1}{2}\sqrt{11.1(4.5)} = 3.53$

LOCAL COLUMN WEB YIELD STRENGTH

$\phi_d R_n \geq F_{fu}$

$R_n = C_t\left(6k_c + t_{bf} + 2t_p\right)F_{yc}\, t_{cw}$

$R_n = .5(6(1.27) + .640 + 2(.625))(50)(.535)$

$R_n = 127.2^k \geq 25.6^k$ ∴ OK ✓

CHECK UNSTIFF. COLUMN WEB BUCKLING

$\phi R_n \geq F_{fu}$

$\phi = .75$

$.75(146.3^k) > 25.6^k$

$109.7 > 25.6^k$

∴ OK ✓

$d_c/2 >$ DIST. TO END OF COLUMN

∴ $R_n = \dfrac{12\, t_{cw}^3\sqrt{E F_{yc}}}{h}$

$R_n = \dfrac{12(.535)^3\sqrt{29000 \cdot 50}}{15.125}$

$R_n = 146.3^k$

$d_c = 18.6''$

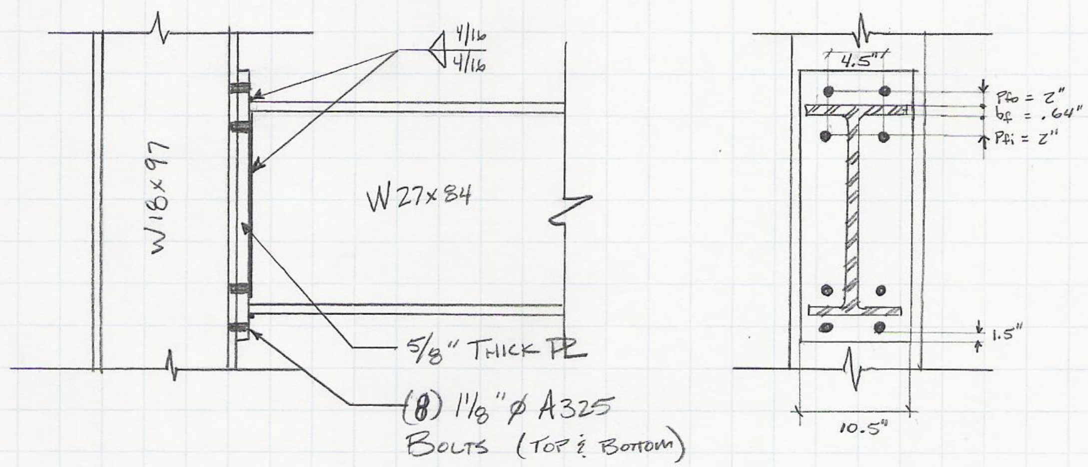
$N = .64'' + 0 = .64''$ $\quad .64/18.6 = .03 < 2$

$$\phi R_n = .8(.535)^2\left[1 + 3\left(\frac{.64}{18.6}\right)\left(\frac{.535}{.870}\right)^{1.5}\right]\sqrt{\frac{29000(50)(.870)}{.535}}$$

$\phi R_n = 369^k > 25.6^k \therefore$ ok ✓

DETAILED CONNECTION NEXT PAGE

TYPICAL INTERMEDIATE MOMENT FRAME CONNECTION



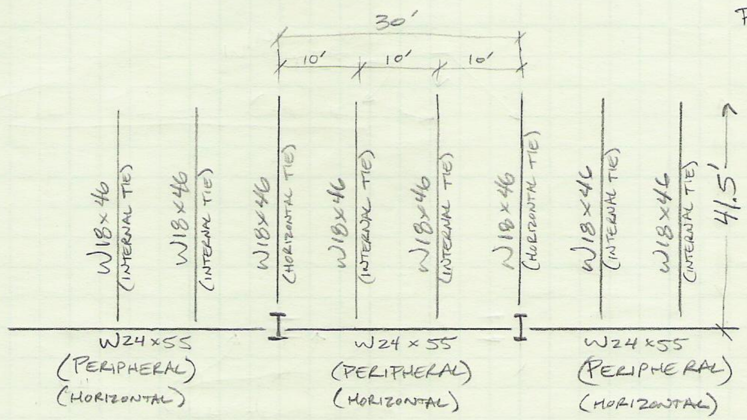
APPENDIX C

PROGRESSIVE COLLAPSE DESIGN

PROGRESSIVE COLLAPSE DESIGN - HLOP (HYPOTHETICALLY)

TIE FORCE REQUIREMENTS (INDIRECT METHOD)

- INTERNAL TIES = $.5(1.2D + 1.6L)s_T L_1$ BUT NOT LESS THAN 16.9^k
- PERIPHERAL TIES = $.25(1.2D + 1.6L)s_T L_1$ BUT NOT LESS THAN 8.4^k
- HORIZONTAL TIES TO COLUMNS = LARGER $\begin{cases} .01(4)(A_{TRIB})(1.2D+1.6L) \\ \text{INTERNAL TIE FORCE} \end{cases}$
- VERTICAL TIES TO COLUMNS = $(A_{TRIB})(1.2D + 1.6L)$, MUST BE CONTINUOUS THROUGH EACH BM-TO-BM CONN. MUST APPLY ALT. PATH METHOD OTHERWISE



L_1 (SPAN): SPANDREL GIRDER = 30 ft
INTERIOR SPAN = 41.5 ft

S_T (SPACING): INTERNAL TIES = 10 ft

DEAD LOAD: 79 PSF

LIVE LOAD: 100 PSF => REDUCED $\begin{cases} \text{BEAM LL} = 68 \text{ PSF} \\ \text{COLUMN LL} = 55 \text{ PSF} \end{cases}$

- INTERNAL TIE FORCE REQUIRED

$$.5(1.2(.079) + 1.6(.068))(10)(41.5) = \underline{42.2^K}$$

- PERIPHERAL TIE FORCE REQUIRED

$$.25\,(1.2(.079) + 1.6(.068))(10)(30) = \underline{15.3^K}$$

- HORIZONTAL TIES TO COLUMNS

$$\text{LARGER}\begin{cases} .01(4)(623)(1.2(.079) + 1.6(.068)) = 5.1^K \\ \text{INTERNAL TIE FORCE} = \underline{42.2^K} \end{cases}$$

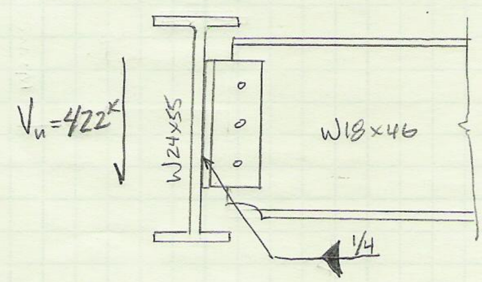
- VERTICAL TIE FORCE TO COLUMNS

$$(623)(1.2(.079) + 1.6(.055)) = \underline{113.9^K}$$

- <u>MINIMUM INTERNAL TIE</u> TO BEAM CONNECTION

* SUBJECT TO CHANGE TO MEET ALT. LOAD PATH REQM'TS

INTERNAL TF REQUIRED = 42.2^K



- BOLTED/WELDED DOUBLE ANGLE CONN.

3-A325 $\frac{1}{4}\phi$ BOLTS

$\frac{1}{2}$" ANGLE, $\frac{1}{4}$" WELD

$$\phi R_n = 48.1^K > 42.2^K \therefore \text{OK}$$

* NOTE: SHEAR IS SAME VALUE AS INTERNAL TIE FORCE REQUIREMENT, THEREFORE ORIGINAL CONNECTION IS SUFFICIENT AND THERE IS <u>NO COST INCREASE</u>.

MINIMUM PERIPHERAL TIE TO COLUMN CONNECTION

* SUBJECT TO CHANGE TO MEET ALT. LOAD PATH REQMT'S

PERIPHERAL (HORIZONTAL) TF REQUIRED = 42.2K

* DUE TO FACT THAT THESE HORIZONTAL MEMBERS ARE EXTERIOR GRAVITY MEMBERS AND THE REACTION (50K) IS GREATER THAN THE REQUIRED TIE FORCE THE ORIGINAL CONNECTION IS ADEQUATE. HOWEVER TO INCREASE DUCTILITY AND REDUNDACY THROUGHOUT STRUCTURE ADEQUATE MOMENT CONNECTIONS COULD BE ADDED. THIS WILL CAUSE A COST INCREASE WHICH CAN BE SEEN IN THE COST ANALYSIS.

MINIMUM TIE REQUIREMENTS

* ALL CONNECTIONS THROUGH BEAMS ARE CONTINUOUS AND ORIGINAL SPLICE CONNECTIONS SATISFY VERTICAL TIE FORCE REQUIREMENTS.

∴ NO COST INCREASE

CONCLUSION: ALL TIE FORCE REQUIREMENTS WERE MET. HOWEVER, ONCE THE MEMBERS ARE ANALYZED FOR ALTERNATIVE LOAD PATH THE CONNECTIONS ARE SUBJECT TO CHANGE.

GSA design criteria for progressive collapse

The step-by-step procedure for conducting the linear elastic, static analysis follows.

Step 1. Remove a vertical support from the location being considered and conduct a linear-static analysis of the structure as indicated in Section 5.1.2.2. Load the model with 2(DL + 0.25LL).

Step 2. Determine which members and connections have DCR values that exceed the acceptance criteria provided in Table 5.1. <u>If the DCR for any member end or connection is exceeded based upon shear force, the member is to be considered a failed member. In addition, if the flexural DCR values for both ends of a member or its connections, as well as the span itself, are exceeded (creating a three hinged failure mechanism – Figure 2.2), the member is to be considered a failed member.</u> Failed members should be removed from the model, and all dead and live loads associated with failed members should be redistributed to other members in adjacent bays.

Step 3. For a member or connection whose Q_{UD}/Q_{CE} ratio exceeds the applicable flexural DCR values, place a hinge at the member end or connection to release the moment. This hinge should be located at the center of flexural yielding for the member or connection. Use rigid offsets and/or stub members from the connecting member as needed to model the hinge in the proper location. For yielding at the end of a member the center of flexural yielding should not be taken to be more than ½ the depth of the member from the face of the intersecting member, which is usually a column (Figure 5.6).

Step 4. At each inserted hinge, apply equal-but-opposite moments to the stub/offset and member end to each side of the hinge. The magnitude of the moments should equal the expected flexural strength of the moment or connection, and the direction of the moments should be consistent with direction of the moments in the analysis performed in Step 1.

Step 5. Re-run the analysis and repeat Steps 1 through 4. Continue this process until no DCR values are exceeded. <u>**If moments have been re-distributed throughout the entire building and DCR values are still exceeded in areas outside of the allowable collapse region, the structure will be considered to have a high potential for progressive collapse.**</u>

Component/Action	Values for Linear Procedures DCR
Beams – flexure	
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{418}{\sqrt{F_{ye}}}$	3
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{640}{\sqrt{F_{ye}}}$	2
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.
Columns – flexure	
For $0 < P/P_{CL} < 0.5$	
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{300}{\sqrt{F_{ye}}}$	2
b. $\frac{b_f}{2t_f} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{460}{\sqrt{F_{ye}}}$	1.25
c. Other	Linear interpolation between the values on lines a and b for both flange slenderness (first term) and web slenderness (second term) shall be performed, and the lowest resulting value shall be used.

Component/Action	Values for Linear Procedures *DCR*
Columns – flexure	
For $P/P_{CL} > 0.5$	
a. $\frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{ye}}}$ and $\frac{h}{t_w} \le \frac{260}{\sqrt{F_{ye}}}$	1
b. $\frac{b_f}{t_w} \ge \frac{65}{\sqrt{F_{ye}}}$ or $\frac{h}{t_w} \ge \frac{400}{\sqrt{F_{ye}}}$	1
Columns Panel Zone – Shear	2
Column Core – Concentrated Forces[2]	1.5
Fully Restrained Moment Connections	
Pre-Northridge (Pre 1995)	
Welded unreinforced flange (WUF)	2
Welded flange plate (WFP)	2
Welded cover plated flanges	2
Bolted flange plate (BFP)	2
Post-Northridge (FEMA 350) Public Domain	
Improved WUF-bolted web	2
Improved WUF-welded web	2
Free flange	2
Welded top and bottom haunches	2
Reduced beam section	2
Post-Northridge (FEMA 350) Proprietary[3]	
Proprietary System	≤ 3 (See Footnote 3)

Component/Action	Values for Linear Procedures DCR
Partially Restrained Moment Connection	
Top and bottom clip angle	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of horizontal leg of angle	1.5
c. Tension failure of rivets or bolts	1.5
d. Flexural Failure of angle	3
Double split tee	
a. Shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Tension failure of rivets or bolts	1.5
c. Tension failure of split tee stem	1.5
d. Flexural Failure of split tee	3
Bolted flange plate	
a. Failure in net section of flange plate or shear failure of rivets or bolts	3 (rivets); 1.5 (high strength bolts)
b. Weld failure or tension failure on gross section of plate	1.5
Bolted end plate	
a. Yield of end plate	3
b. Yield of rivets or bolts	2 (rivets); 1.5 (high strength bolts)
c. Failure of weld	1.5
Composite top and clip angle bottom	
a. Failure of deck reinforcement	2
b. Local flange yielding and web crippling of column	3
c. Yield of bottom flange angle	3
d. Tensile yield of rivets or bolts at column flange	1.5 (rivets); 1 (high strength bolts)
e. Shear yield of beam flange connections	2
Shear connection with or without slab	2

1. Notation for Table 5.1:

bf = Width of the compression flange
F_{ye} = Expected yield strength
h = Distance from inside of compression flange to inside of tension flange
t_w = Web thickness
P_{CL} = Lower bound compression strength of the column
P = Axial force in member taken as Q_{uf}
t_f = Flange thickness
d = Beam depth
d_{bg} = Depth of the bolt group

PROGRESSIVE COLLAPSE ANALYSIS - GSA REQUIREMENTS

LOAD COMBO: $2(DL + .25LL)$ (DIRECT OR SPECIFIC LOAD PATH)

DEAD LOAD$_{FLOOR}$ = .079 KSF
ROOF: .025 KSF

LIVE LOAD$_{FLOOR}$ = .100 KSF
ROOF = .025 KSF

EXTERIOR CONSIDERATION (ALLOWABLE EXTENTS)

- ALL ADJACENT BAYS OR 1800 ft² (SMALLER)

$41.5' \times 30(2) = 2460 \text{ ft}^2 > 1800 \text{ ft}^2 \therefore$ USE 1800 ft²

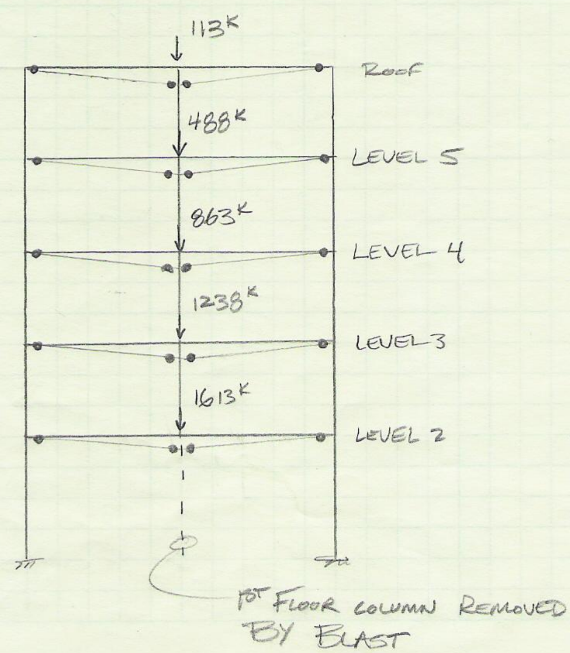
LOADS: (AS POINT LOAD)

TYPICAL FLOOR:

$$2(.079 + (.25)(.100)) = .208 \text{ KSF } (1800 \text{ ft}^2) = 375^k$$

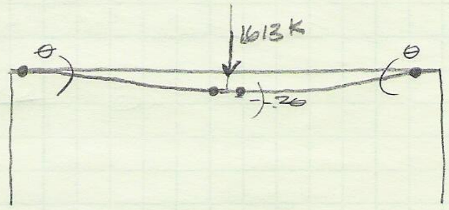
ROOF:

$$2(.025 + (.25).025) = .0625 \text{ KSF } (1800 \text{ ft}^2) = 113^k$$



PROGRESSIVE COLLAPSE ANALYSIS - GSA REQUIREMENTS

LEVEL 2 - PLASTIC ANALYSIS (VIRTUAL WORK



$$M_p\theta + M_p\theta + M_p 2\theta + M_p 2\theta = P(30\theta)$$

$$6M_p\cancel{\theta} = 30P\cancel{\theta}$$

$$M_p = 5(1613^k) = 8065'^k$$

ACCEPTANCE CRITERIA

$$DCR = \frac{Q_{UD}\ \text{(ACTING FORCE)}}{Q_{CE}\ \text{(EXPECTED ULTIMATE)}} = 3 \quad \text{(BEAM IN FLEXURE)}$$

$$Q_{CE} = \frac{8065}{3} = 2688.3'^k \Rightarrow W36\times182\ (\phi M_p = 2690'^k)$$

W36×182 ∴ $b_f = 12.1''$, $t_f = 1.18''$, $h = 33.94''$, $t_w = .725''$

(FLANGE SLENDERNESS) $\frac{12.1}{2(1.18)} \leq \frac{52}{\sqrt{50}} \Rightarrow 5.13 \leq 7.35$ ∴ ✓ OK

(WEB SLENDER.) $\frac{33.94}{.725} \leq \frac{418}{\sqrt{50}} \Rightarrow 46.8 \leq 59$ ∴ ✓ OK

<u>W36×182</u> IS ADEQUATE

CONNECTION TYPE → FULLY RIGID MOMENT CONNECTION

DCR = 2 (FR MOM. CONN.)

$$\frac{8065}{2} = 4033'^k = Q_{CE} \Rightarrow$$ MOMENT @ CONNECTION

SEE CALCS FOR PANEL ZONE & STIFFNER DESIGN

PROGRESSIVE COLLAPSE ANALYSIS - GSA REQUIREMENTS

LEVEL 3

$M_p = 5(1238) = 6190\,'^k$

ACCEPTANCE CRITERIA

$DCR = \frac{Q_{UD}}{Q_{CE}} = 3$

$Q_{CE} = \frac{6190}{3} = 2064\,'^k \Rightarrow W36 \times 150 \; (\phi M_p = 2180)$

$W36 \times 150: \; b_f = 12.0'', \; t_f = .94'', \; h = 34'', \; t_w = .625''$

(FLANGE SLENDERNESS) $\frac{12}{2(.94)} \leq \frac{52}{\sqrt{50}}$ $(6.38 \leq 7.35 \; \therefore \text{OK} \checkmark)$

(WEB SLENDERNESS) $\frac{34}{.625} \leq \frac{418}{\sqrt{50}}$ $(54.4 \leq 59 \; \therefore \text{OK} \checkmark)$

W36 x 150 IS ADEQUATE

LEVEL 4

$M_p = 5(863) = 4315\,'^k$

ACCEPTANCE CRITERIA $\quad DCR = 3$

$Q_{CE} = \frac{4315}{3} = 1439\,'^k \Rightarrow W33 \times 130 \; (\phi M_p = 1750)$

$W33 \times 118: \; b_f = 11.5'' \; t_f = .855'' \; h = 31.39'' \; t_w = .58''$

(FLANGE SLENDERNESS) $\frac{11.5''}{2(.855'')} = 6.72 < \frac{52}{\sqrt{50}} = 7.35 \; \therefore \text{OK} \checkmark$

(WEB SLENDERNESS) $\frac{31.39}{.58} < \frac{418}{\sqrt{50}}$ $(54.1 \leq 59 \; \therefore \text{OK} \checkmark)$

W33 x 130 IS ADEQUATE

PROGRESSIVE COLLAPSE ANALYSIS - GSA REQUIREMENTS

LEVEL 5

$M_p = 5(488) = 2440'^k$

ACCEPTANCE CRITERIA $\underline{DCR = 3}$

$Q_{CE} = \frac{2440}{3} = 814 \Rightarrow W24\times84 \quad (\phi M_p = 840'^k)$

$W24\times84: \; b_f = 9.02'', \; t_f = .77'', \; h = 22.56, \; t_w = .470''$

(FLANGE SLENDERNESS) $\frac{9.02}{2(.77)} < \frac{52}{\sqrt{50}}$ (5.86 < 7.35 ∴ OK ✓)

(WEB SLENDERNESS) $\frac{22.56}{.47} < \frac{418}{\sqrt{50}}$ (48 < 59 ∴ OK ✓)

$\underline{W24\times84}$ IS ADEQUATE

ROOF

$M_p = 5(113) = 565'^k$

ACCEPTANCE CRITERIA (DCR = 3)

$Q_{CE} = \frac{565}{3} = 189'^k \Rightarrow W21\times48 \quad (\phi M_p = 398'^k)$

$W21\times48: \; b_f = 8.14'', \; t_f = .43'' \; h = 19.74'' \; t_w = .35''$

(FLANGE SLENDERNESS) $\frac{8.14}{2(.43)} < \frac{52}{\sqrt{50}}$ (9.46 > 7.35 ∴ NO GOOD)

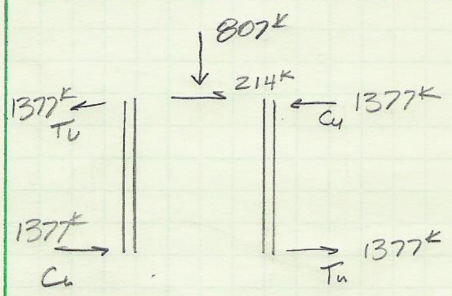
∴ $\underline{DCR = 2}$

$\Rightarrow Q_{CE} = \frac{565}{2} = 283'^k \Rightarrow W21\times48$ WORKS

∴ $\underline{W21\times48}$ IS ADEQUATE

(ORIGINAL DESIGN)

PROGRESSIVE COLLAPSE ANALYSIS – GSA REQUIREMENTS



$M_p : 4033'^k$

$\frac{4033(12)}{36.34 - 1.18} = \underline{1377^k}$

COLUMN FLANGE BENDING

$\phi R_n = .9(6.25) t_f^2 F_y$

$1377 = .9(6.25) t_f^2 (50)$

$t_f = 2.21 \Rightarrow$ TRY W14×257 w/ STIFFENERS

$\hookrightarrow \phi R_n = 1004^k$

COLUMN WEB YIELDING

$\phi R_n = 1.0(5 k_{DET} + t_{f\,BEAM}) F_{yc} t_{wc}$

$= 1.0(5(3.1875) + 1.18)\, 50 (1.18)$

$= 1010^k < 1377^k$ ∴ STIFFENERS REQ'D (FULL DEPTH)

GSA REQUIREMENTS (FOR W14×257)

$P = 807$
$P_c = 2960$ $\quad P/P_c < .5$

$\therefore \quad \frac{b_f}{2t_f} \le \frac{52}{\sqrt{F_{yc}}} \Rightarrow \frac{16.0}{2(1.84)} \le \frac{52}{\sqrt{50}}$

$4.35 \le 7.35$ ∴ OK ✓

$\frac{h}{t_w} \le \frac{300}{\sqrt{F_{yc}}} \quad \frac{12.62}{1.18} \le \frac{300}{\sqrt{50}}$

$6.67 \le 42.42$ ∴ OK ✓

DCR = 2.0 (COLUMN IN FLEXURE)

DCR = 2.0 (PANEL ZONE SHEAR – COLUMN)

INTERACTION DIAGRAM

$P_{AXIAL} = 807^k$

$M_u = 1377 \Rightarrow \frac{1377}{2\ (DCK)} = \underline{684^{'k}}$

SEE PRINTOUT

$H1\text{-}1a = \underline{.92 < 1}$ ∴ OK ✓ $\underline{W14 \times 257}$ OK

STIFFENER DESIGN

$C_u \,\&\, T_u = 1377^k - 1004^k = 373^k$

$A_{ST} = (b_s - CLIP)\,t_s$

$A_{ST} = \frac{373/2}{.9(36)} = 5.75\,in^2$

TRY 6.5" × 1" PL

$(6.5 - 3/4)(1) = 5.75$ ✓ OK

THICKNESS LIMITATIONS

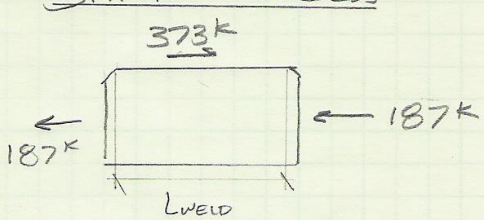
$6.5 + \frac{1.18}{2} \geq \frac{12.1}{3}$ ($7.49 > 4$ ∴ OK ✓)

$1.0 \geq \frac{1.18}{2}$ ($1.0 > .59$ ∴ OK ✓)

$1.0 > 1.79(6.5)\sqrt{36/29000}$

$1.0 > .410$ ∴ OK ✓

STIFFENER WELDS

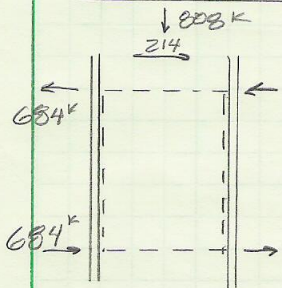


$L_{WELD} = T_{COL} = 10"$

$D_{FLANGE} = \frac{.9(10)(36)}{1.5(1.392)(2)} = 5.2 \Rightarrow 3/8"$ FILLET

$D_{WEB} = \frac{373}{(10)(2)(1.392)} = 13.4 \Rightarrow 7/8"$ FILLET

PANEL ZONE SHEAR



$P_u = 808^K$

$P_y = 75.6\,in^2(50)(.4)$
$= 1512^K > 808^K$

$\therefore R_v = .6 F_y d_c t_{wc}$
$= .6(50)(16.4)(1.18)$
$= 580.56^K < 1582^K$

$V_n = 684 + 684 - 214$
$V_n = 1582^K$

$\therefore$ DOUBLER PL's REQ'D

DESIGN DOUBLER PLATES

ASSUME $\frac{h}{t_p} \leq 1.10\sqrt{\frac{K_v E}{F_y}}$

$t_p = \frac{1582 - 581}{(1.0)(.6)(36)(16.4)} = 2.82 > 1''$

USE (2) DOUBLER PLATES
$t_p = 1.5''$

CHECK PL BUCKLING

ASSUME $h/t_p \leq 1.1\sqrt{\frac{K_v E}{F_y}}$

$\frac{16.5 - 2(1.89)}{1.5} = 8.48 < 1.1\sqrt{\frac{5(29000)}{36}} = 69.8$ $\therefore$ ASSUMPTION CORRECT

PANEL ZONE WELDS:

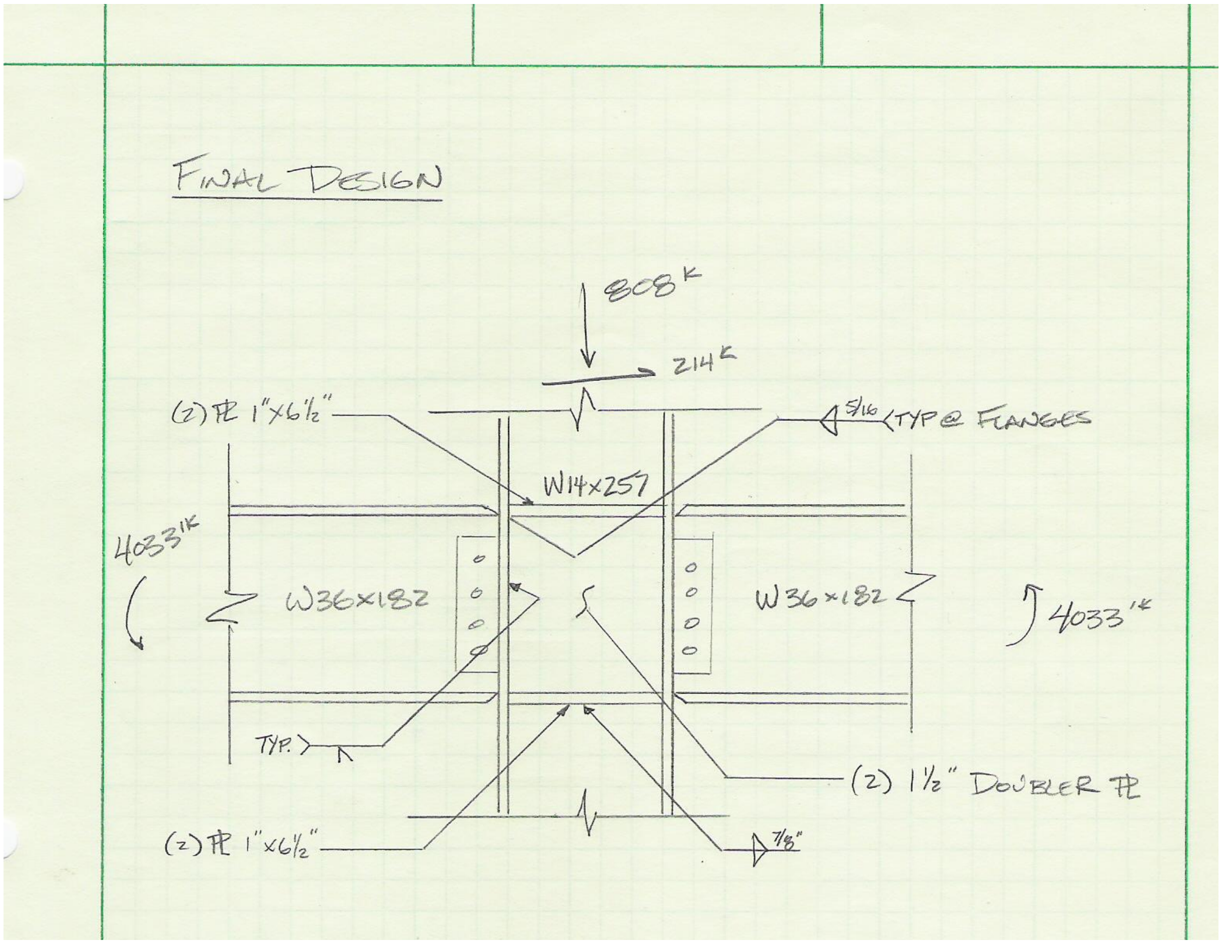
A. LONG SIDE

$V_u = \frac{1582 - 581}{2} = 501$

$D_{req} = \frac{501}{(35.16 \cdot 1.392)} = 14.85/16'' \Rightarrow 1''$ OR FULL PENETRATION

B. SHORT SIDE

MINIMUM REQ'D



This was the original design, however after comparing it to SidePlate connection system SidePlate was chosen as the connection for the design.

BOLTED END PLATE (BEP) VS. SIDEPLATE®

5-Story Office Building in high seismic

LATERAL SYSTEM: STEEL MOMENT FRAMES
AREA: 106,600 S.F.

	Bolted End Plate WEIGHT	SidePlate® system WEIGHT
Lateral Columns	W14x211	W21x93
	W14x233	W21x122
	W14x455	W21x132
		W21x166
		W21x182
	5.91 psf	**2.05 psf**
Lateral Beams	W21x50	W16x31
	W24x76	W21x50
	W24x103	W24x55
		W24x62
		W24x94
		W24x103
	2.43 psf	**1.41 psf**
Gravity Columns	0.08 psf	0.45 psf
Gravity Beams	4.34 psf	4.79 psf
Connection Plates	0.09 psf	0.81 psf
Misc Steel	1.65 psf	1.65 psf
Total Steel Weight	**14.5 psf (1732 tons)[1]**	**11.2 psf (1512 tons)[1]**
Estimated Fabricated & Erected Steel Costs	$2,675,000 ($3,460/T)	$2,118,000 ($3,560/T)
SidePlate® Services & License Fee[2]	N/A	$42,000
Total Estimated Costs	**$2,675,000**	**$2,160,000**
Estimated Savings w/ SidePlate® Moment Frames	**$2,675,000**	**$2,160,000**

(1) Based on data obtained from ETABS Model and SidePlate Systems, Inc. for connection weight
(2) See Attached List for Services Included with selection of SidePlate® Connection Technology

Recommendation:

Use SidePlate® connection technology and *save the owner*:

- $515,000 in steel fabrication & erection costs ($4.83/sf) PLUS
- 24% (50) fewer moment connections to install in the field resulting in faster construction schedule

APPENDIX D – ARCHITECTURE BREADTH

ARCHITECTURE BREADTH

DETERMINATION OF FRANGIBLE / NON-FRANGIBLE FACADE

STRUCTURAL BAY AREA = 14.33 ft × 30 ft = 430 ft²

WINDOWS = 26 ft × 10 = 260 ft² (~60%)

METAL PANELS/PRECAST P. = 430 − 260 = 170 ft² (~40%)

PRECAST PANELS

ASSUMING PRECAST PANELS PROPERTIES:

$f_y = 60$ ksi $d = 4.5$ (6" PANEL)

$f'_c = 4$ ksi $L = 50\frac{1}{2}$"

$\rho = .0025$

$$\text{CAPACITY} = \frac{8 M_u}{L^2}$$

$$M_u = \rho b d^2 \left(1 - \frac{\rho f_y}{1.7 f'_c}\right) = .0025(1)(4.5)^2\left(1 - \frac{.0025(60)}{1.7(4)}\right)$$

$$M_u = 2.97 \text{ k}\cdot\text{in/in}$$

$$\text{CAPACITY} = \frac{8(2.97 \cdot 1000)}{(50.5)^2} = 9.31 \text{ psi} > 1.0 \text{ psi}$$

∴ PRECAST PANELS ARE CONSIDERED "NON-FRANGIBLE"

WINDOWS

TYPICAL OPENING = 4.5' × 6.5' (54" × 78")

STANDOFF DISTANCE (FRONT) : 232' (SITE REDESIGN)

ASSUMED 500 lb-TNT EQUIVALENT ('HIGH' RISK)

FROM GRAPH ⇒ 71 PSF (3-SECOND DURATION EQU. DESIGN LOAD)
(ASTM F 2248)

71 PSF & 30 ft² ⇒ ½" NOMINAL THICKNESS (ANNEALED MONO.)

⇒ ¼" NOMINAL THICKNESS (HS-LG)

ARCHITECTURE BREADTH

REQUIRED GLAZING

1/4" HEAT STRENGTHENED LAMINATED GLASS
(1/8" GLASS PLY, .03" PVB LAYER, 1/8" GLASS PLY)

* NOTE: ALSO MEETS DOD REQUIREMENTS

71 PSF = .5 psi < 1 psi ∴ SURFACE CONSIDERED "FRANGIBLE"

PRECAST PANELS STRONGER (9.31 psi > .5 psi), THEREFORE SURFACE CONSIDERED "NON-FRANGIBLE"

* ACCORDING TO GSA & ISC THE REQUIRED LEVEL OF PROTECTION IS 130'

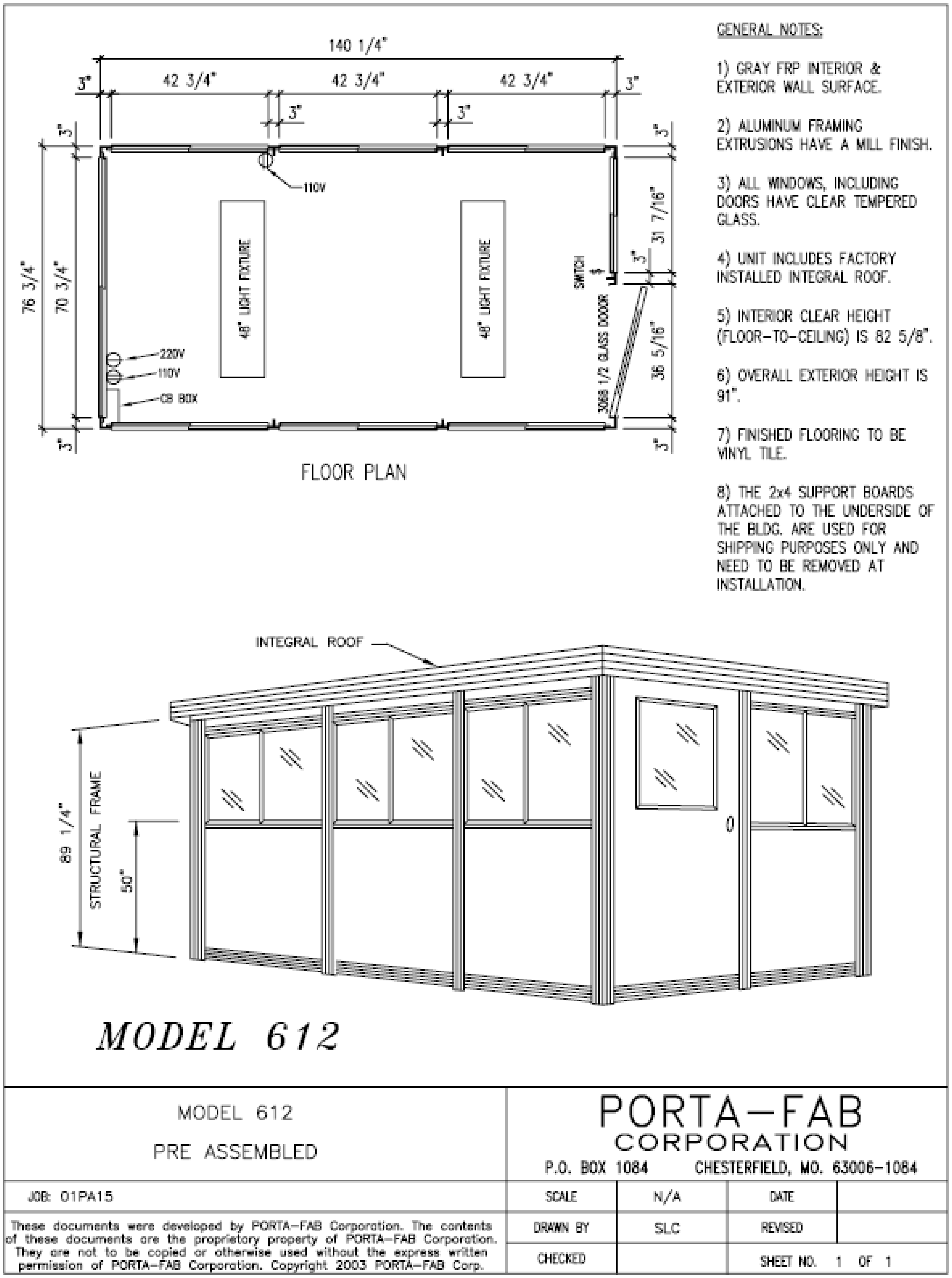
FRONT STANDOFF = 230' > 130' ∴ OK ✓
BACK STANDOFF: 135' > 130' ∴ OK ✓

```
RAM SColumn V1.01                                    Column Design Results
                                      B~^è3
Job Name:                                                3/31/09   18:53
Comments:
Steel Code:  ASD 9th Ed.

    Fy (ksi) =  50.00                        Column Size = W14X257

INPUT DESIGN PARAMETERS:                             X-Axis    Y-Axis
    Lu (ft) ..............................   14.25      0.00
    K ....................................    1.00      1.00
    Braced Against Joint Translation.......    No         No
    Loaded Between Joints..................    No         No
    Ends Fixed Against Rotation............    No         No

COLUMN LOADS:                   Design
    Axial   (kip) .......        808.0
    Top Mx (kip-ft)......        684.0
        My (kip-ft)......          0.0
    Bot Mx (kip-ft)......        684.0
        My (kip-ft)......          0.0

CALCULATED PARAMETERS:
    Allowable Stress Increase Factor.. 1.00
    fa  (ksi) =  10.69   Fa  (ksi) =  27.69
    fbx (ksi) =  19.78   Fbx (ksi) =  33.00
                        Fbx1 (ksi) =  33.00
    fby (ksi) =   0.00   Fby (ksi) =  37.50

    Single curvature about X-Axis
    Single curvature about Y-Axis

    Cb    =   1.00
    KL/Rx =  25.50    KL/Ry =   0.00
    F'ex  = 229.68    F'ey  =   0.00
    Cmx   =   0.85    Cmy   =   0.85

INTERACTION EQUATION:
    fa/Fa  = 0.39
    Eq H1-1: 0.39 + 0.53 + 0.00 = 0.92
    Eq H1-2: 0.36 + 0.60 + 0.00 = 0.96
```

Summary of Security Zones

ZONES	ELEMENTS/ACTIONS
ZONE 1 Neighborhood	
This can be an area of one or more blocks surrounding a facility, depending on how the site is used. It may include streetscape, public spaces, parking lots, and other facilities that visitors frequent. **Opportunities:** Site treatments include architectural, visual, and public-use cues. Neighborhood-based solutions, such as operational security and traffic guidance/control countermeasures, are also effective.	1. Coordinate with existing and proposed development plans, guidelines, and programs 2. Collaborate with other neighborhood security operations 3. Modify traffic conditions 4. Consider including public right-of-way in the standoff zone 5. Consider closing part or all of an existing street if necessary 6. Install temporary barriers for heightened levels of alert 7. Develop and coordinate personal safety programs
ZONE 2 Standoff Perimeter	
A security perimeter keeps vehicle-borne explosives at a distance, thus reducing potential destruction and harm. Depending on the risk anaysis, the perimeter may require secured or unsecured standoff. **Opportunities:** Enhancements to the functionality and aesthetics of the site for the public, employees, and visitors are possible, while satisfying standoff needs.	1. Determine the level of protection needed, based on accepted risk 2. Ascertain the standoff zone location and dimensions 3. Establish a hardened perimeter where warranted, using ■ Bollards ■ Sculptural or seating barriers ■ Walls ■ Hardened street furniture ■ Fences ■ Topography ■ Dry moats ■ Collapsible surfaces ■ Water ■ Landscaping and plantings
ZONE 3 Site Access and Parking	
Various elements and services provide and control access to a facility. This zone can include the inspection of both vehicles and visitors. **Opportunities:** Satisfying security requirements can also promote effective access, natural surveillance, and increased convenience for those who use the facility.	1. Delineate drop-off and pick-up areas 2. Control site access by incorporating ■ Inspection areas ■ Retractable bollards ■ Gates ■ Guard booths ■ Sally ports 3. Monitor loading and service areas 4. Maintain clear access routes for first responders 5. Establish clear pedestrian circulation routes 6. Establish secure parking areas inside and outside the standoff perimeter ■ Garage parking ■ Surface parking ■ Wayfinding, lighting, and signage

ZONES	ELEMENTS/ACTIONS
ZONE 4 Site	
Once within the security perimeter, the site zone may provide an additional layer of elements, or hardening, to assist in deterring or preventing the destruction of or harm to a facility. With a sufficiently hardened perimeter, the site zone's primary role would be to serve more as a welcoming public space, with amenities, programs, and activities that serve building tenants, visitors, and the larger community. **Opportunities:** Site features, such as reflecting pools, benches, and security pavilions on the site and inside the standoff zone perimeter, may offer enhanced security, safety, and amenities.	1. Design site amenities, such as furnishings, planters, water features, lighting, and vegetation, to serve multiple purposes 2. Create usable space 3. Designate weather-protected space for queuing at entries 4. Design security pavilions and other freestanding buildings to blend with the site's architectural character
ZONE 5 Building Envelope	
Control of heating, ventilation, and air-conditioning (HVAC) vents/air intakes; location and operation of entry and egress points; additional surveillance by security personnel or cameras; and lighting occur at the interface between site and building. **Opportunities:** Security improvement may also increase everyday safety of the site.	1. Prevent access to vents/air intakes 2. Design emergency egress to allow easy evacuation from a facility 3. Place cameras and light fixtures to maximize visibility 4. Harden the building structure and envelope 5. Design orientation and massing of building to lessen impact of explosion
ZONE 6 Management and Building Operations	
Building programs and layout can be modified to increase security, such as moving high-risk tenants to the interior of the facility. Additional security personnel can also be added to increase surveillance. **Opportunities:** Modifications to space planning and building operations can reduce some risk, without changing the site itself.	1. Design for flexibility in building programming and space planning 2. Consider guards and alternative security operations when faced with site and cost constraints 3. Choose no mitigation and accept risk when it is neither practical nor plausible to harden site elements or the exterior of a facility


140 1/4"
3"
42 3/4"
42 3/4"
42 3/4"
3"
3"
3"
3"
3"
76 3/4"
70 3/4"
110V
48" LIGHT FIXTURE
48" LIGHT FIXTURE
SWITCH
3"
31 7/16"
3068 1/2 GLASS DOOR
36 5/16"
220V
110V
CB BOX
3"
3"
FLOOR PLAN
GENERAL NOTES:
1) GRAY FRP INTERIOR & EXTERIOR WALL SURFACE.
2) ALUMINUM FRAMING EXTRUSIONS HAVE A MILL FINISH.
3) ALL WINDOWS, INCLUDING DOORS HAVE CLEAR TEMPERED GLASS.
4) UNIT INCLUDES FACTORY INSTALLED INTEGRAL ROOF.
5) INTERIOR CLEAR HEIGHT (FLOOR-TO-CEILING) IS 82 5/8".
6) OVERALL EXTERIOR HEIGHT IS 91".
7) FINISHED FLOORING TO BE VINYL TILE.
8) THE 2x4 SUPPORT BOARDS ATTACHED TO THE UNDERSIDE OF THE BLDG. ARE USED FOR SHIPPING PURPOSES ONLY AND NEED TO BE REMOVED AT INSTALLATION.
INTEGRAL ROOF
89 1/4"
STRUCTURAL FRAME
50"
MODEL 612
MODEL 612
PRE ASSEMBLED
PORTA-FAB
CORPORATION
P.O. BOX 1084 CHESTERFIELD, MO. 63006-1084
JOB: 01PA15
SCALE N/A
DATE
DRAWN BY SLC
REVISED
CHECKED
SHEET NO. 1 OF 1
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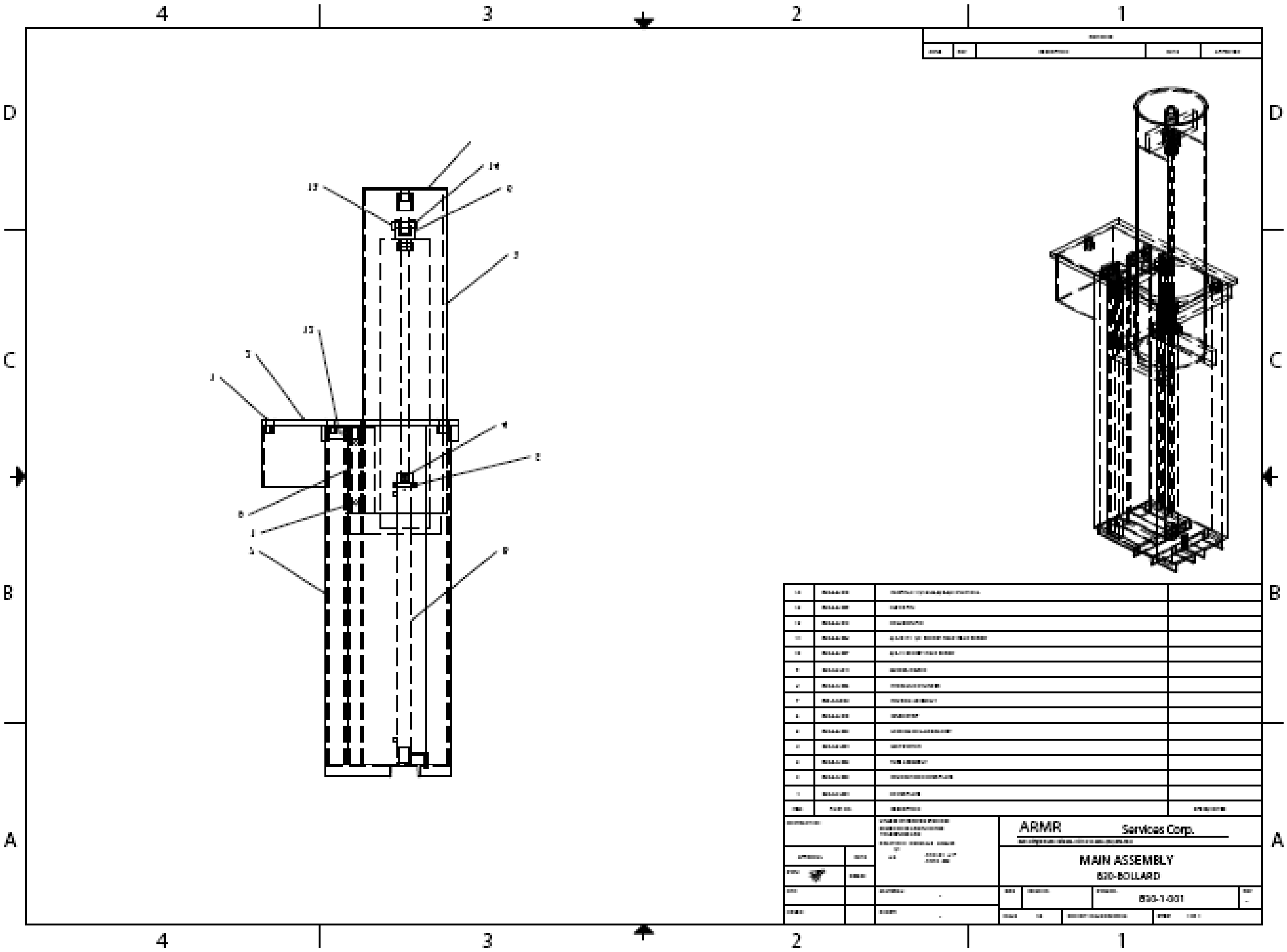
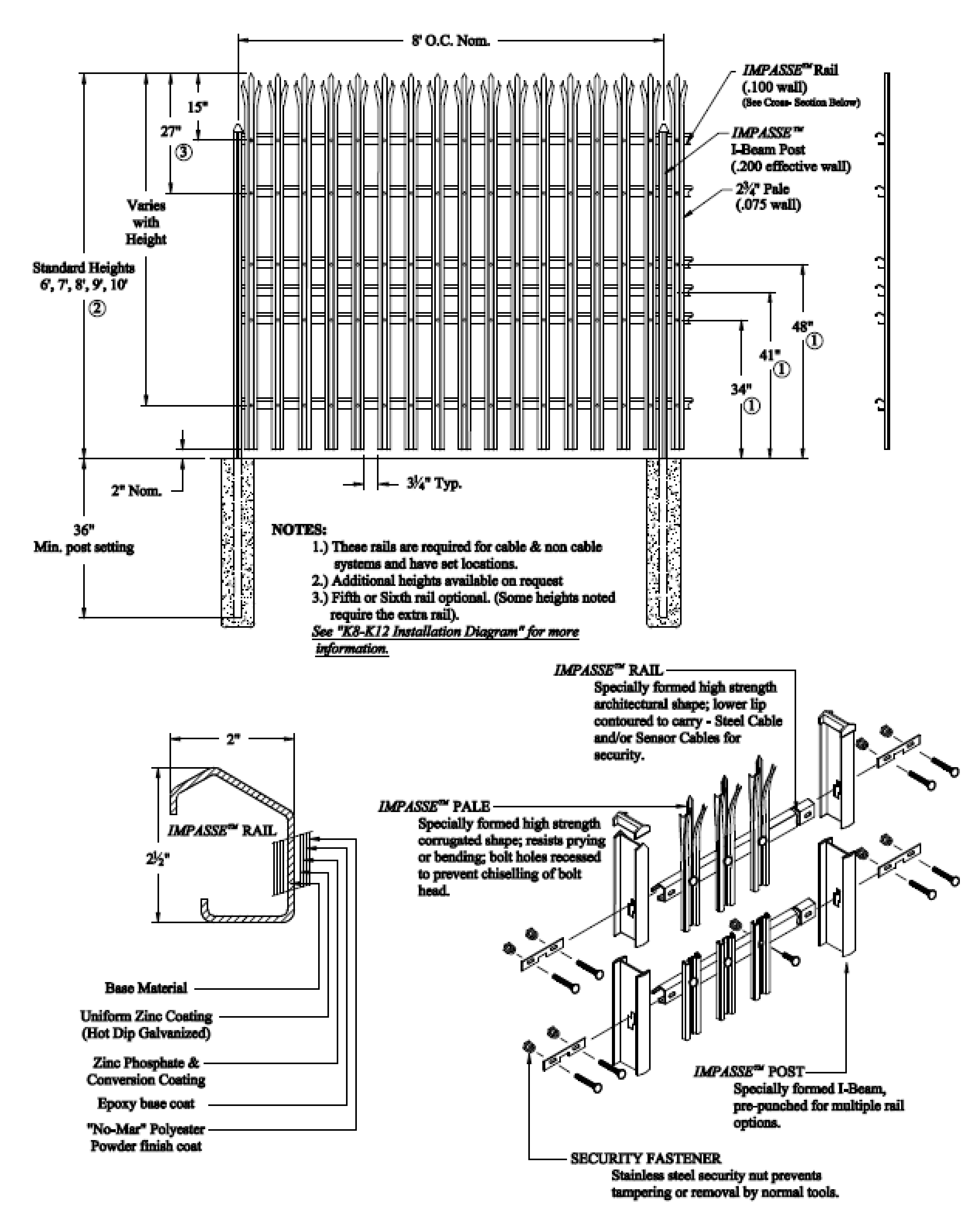

ARMR
Services Corp.
MAIN ASSEMBLY
B30-BOLLARD
B30-1-001



HIGH SECURITY STEEL FENCE

3K8-12TESO

IMPASSE TRIDENT 4/5/6-RAIL			AMERISTAR® 1555 N. Mingo Tulsa, OK 74116 1-888-333-3422 www.ameristarfence.com
R: RTM	SH . 1of 1	SCALE: DO NOT SCALE	
C: MM	Date 5/15/06	REV: a	

Laminated Glass

Blast-Resistant Glass

Introduction

In recent years, the bomb has become the weapon of choice for many terrorist attacks. The high-explosive detonation, with its associated property damage, injury, flames and noise, draws immediate attention and instills fear beyond that of armed attacks.

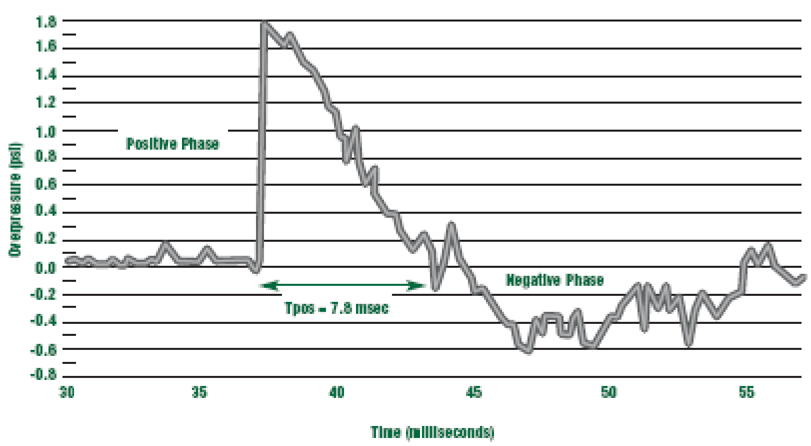
Extensive research has been carried out following terrorist bombing events in New York, Oklahoma, London, Israel, and many other locations. It has been documented that the blast energy causes collateral damage to many surrounding structures, not just the intended target. Glass fragmentation hazards have been identified as a main cause of injury in the targeted site, as well as the peripheral sites. Because collateral damage often extends several blocks from the site of the bomb, it can affect hundreds, possibly thousands, of people, especially in urban areas.

Description

Laminated glass is an excellent glazing choice in all types of buildings that may be subjected to bomb blasts. The tough plastic interlayer holds the glass together after an impact, and with the proper framing systems, the glazing will be retained in the opening. Thus, the amount of flying glass, as well as the consequential injuries, can be dramatically reduced.

The pressure from a bomb typically consists of a wave that rises almost instantaneously to a very high peak pressure that falls back to zero in a very short duration, as measured in milliseconds. For example, a 27 lb. bomb detonated from a stand-off distance of 48 ft. produces a peak pressure of 10 psi (1,440 psf) for 3.3 milliseconds. The area under the pressure time graph is called the impulse and is measured in psi-ms. Blast wave energy decreases very rapidly with distance so that the most effective protection is to increase this "stand-off" distance. However, this is not always a viable or economic option.

Typical Blast Wave–Incident (Side-on) Overpressure



8 lbs of C-4 explosive (TNT equivalent of 10 lbs) detonated 57 feet from target; atmospheric pressure of 12.9 psi.

(continued on back)

Laminated Glass

Blast-Resistant Glass

Description (continued)

The General Services Administration (GSA), which is responsible for all US nonmilitary federal buildings, developed an approach for blast resistance. This approach has been included in the Interagency Security Committee (ISC) document that is now being used to evaluate vulnerability and provide design guidelines for government-owned and leased buildings.

The building type is defined in Table 1, and the protection level is defined in Table 2, taking into account the sensitivity of the area behind the glazing.

Table 1

GSA Building Classification	Examples	Max Overpressure	Max Impulse
A	No protection	0	0
B	No protection	0	0
C	Fed courts, fed buildings, etc.	4 psi	28 psi ms
D	High-level military, e.g., Pentagon	10 psi	89 psi ms
E	White House	Classified	Classified

Table 2

Hazard 1	Hazard 2	Hazard 3	Hazard 3B	Hazard 4	Hazard 5
No glass breakage	Minimal spall	Spall up to **3ft** (1m)	Spall up to **10ft** (3m)	Hits back wall up to **2ft** high	Hits back wall ≥ **2ft** high

Hazard 1 allows no breakage at all. This is required in locations where complete vision must be maintained after the event and where personnel would be situated immediately behind the glazing. Control points and lookout positions would fall into this category. Hazards 2-3 and 3B allow increasing amounts of limited spalling, very small chips of glass, so the immediate injuries would be minor. The glazing in these locations would remain in the frame, providing protection from additional outside debris or the weather. Hazards 4 and 5 occur when larger amounts of glass, or other debris, fly off with considerable energy and can cause serious injury to the occupants of the building. The glazing would not always be retained in the frame. Hazards 4 and 5 would only be specified for very low occupancy buildings and/or storage areas.

ASTM F1642 *Standard Test Method for Glazing and Glazing Systems Subject to Airblast Loadings* details a test method for this type of glazing. The newest version of this standard has six hazard criteria similar to the GSA recommendations. However, the detailed definitions vary slightly. The frame is an integral part of the blast mitigation glazing system. The blast pressure applies a load to the glass and will be transmitted to the frame through the fasteners, and on to the structure of the building. If the glazing is made very stiff, this entire load will be transmitted to the building, which can damage the structural integrity of the building. In the case where the glazing is very thick and stiff the structure of the building has to be significantly modified and strengthened to accept this additional load.

Oldcastle-Arpal offers *Blast-Tec™* blast mitigation, energy-absorbing aluminum framing systems which, together with the laminated glass, absorb much of the blast pressure, allowing only a minimal transfer of energy to the surrounding walls. Thus,

(continued on next page)

Laminated Glass

Blast-Resistant Glass

Description (continued)

the *Blast-Tec*™ glazing systems offer design alternatives that result in a cost-effective way to resist a bomb blast without having to reinforce the structure of the building. The *Blast-Tec*™ series includes curtain wall systems, fixed and operable windows, internal blast shields for historical preservation and doors for all levels of blast threats.

For full details, please see the Green Blast Mitigation Oldcastle-Arpal, LLC. Tab or log on to www.oldcastlearpal.com.

Capabilities

The following constructions of laminated glass are most commonly specified for bomb-blast resistance. As with all laminated glazing, the glass can be supplied as tinted or reflective for light and solar control purposes. The lites of glass can be either annealed or heat-strengthened. Oldcastle Glass® does not recommend tempered laminated glass in this type of application. When insulating glass units are required for thermal performance, Oldcastle Glass® recommends that both lites of the IG unit be laminated in order to provide maximum protection for those both inside and outside the building. If only one lite in the IG unit is to be laminated, it must be the interior lite so as to protect the occupants of the building.

Product #	Construction	Thickness		Weight	
	Glass-PVB-Glass: inches	inches	mm	lbs/ft²	kg/m²
110100	1/8–0.060–1/8	5/16	8	3.58	17.5
110110	3/16–0.060–3/16	7/16	11	5.21	25.4
110120	1/4–0.060–1/4	9/16	14	6.83	33.3

Additional Important Information

Design Criteria

Details on the following important topics can be found in the Black Design Criteria Tab: Glazing Instructions, Thermal Stress, Deflection, Glass Design Loads, Glass Thickness Selection, Spontaneous Breakage of Tempered Glass, Roller Wave Distortion in Heat-treated Glass, Mock-ups and Warranties.

Specifications

A sample Section 08800 Specification for North America can be found in the Black Specifications Tab. Information specific to two-ply (two lites of glass) laminated glass can be found in Part 2 Products, 2.02 Materials.

For specifications on other laminated glass makeups, call 1-866-OLDCASTLE(653-2278) or log on to www.oldcastleglass.com and click on "Need Assistance with a Project," click on "General Inquiry" and enter your request.

Contact Us

For any additional information, including details, technical data, specifications, technical assistance and samples, or to speak with an architectural specialist, call 1-866-OLDCASTLE(653-2278).

Visit Us on the Web

Log on to www.oldcastleglass.com for project photos, product colors, general inquiries and project assistance.

To view performance data on a wide range of glass makeups, or to build your own product specification, log on to www.oldcastleglass.com and choose GlasSelect.™

Oldcastle Glass® *Where glass becomes architecture*™

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