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SILVER SPRING GATEWAY
Senior Thesis Final Report

1133 East-West Highway
Silver Spring, Maryland

The Pennsylvania State University
Department of Architectural Engineering
Senior Thesis 2007-2008

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SILVER SPRING GATEWAY

SILVER SPRING, MARYLAND

GENERAL BUILDING DATA:

LOCATION: 1133 East West Highway Silver Spring, Maryland

OCCUPANCY: Retail, Condominiums, and Apartments

SIZE: 766,459 Square Feet

HEIGHT: 15 stories with a maximum height of 143 feet

CONSTRUCTION DATES: July 06 – July 08

BID COST: $89 million

DELIVERY METHOD: Design-Bid-Build under a GMP contract

ARCHITECTURE:

- Mixed Use Development including 14,080 sq. ft. of retail space, 100,215 sq. ft. of parking, 395,439 sq. ft. of residential space, 1,000 sq. ft. roof top swimming pool

- Building Envelop consists of brick cavity walls and aluminum Centria storefront curtain walls

STRUCTURAL SYSTEM:

- Two-way flat plate post-tensioned slabs supported by 176 reinforced concrete columns

- Full penetration welded steel trusses span 36 feet for the pedestrian bridge over the garage entrance

- Supported on Caissons and Transfer Beams to bedrock below

PROJECT TEAM:

- **OWNER:** The JBG Companies
- **ARCHITECT:** WDG Architecture
- **STRUCTURAL ENGINEER:** Tadjer , Cohen, Edelson Assoc., Inc.
- **MECHANICAL ENGINEER:** Atlas Air Conditioning Co.
- **ELECTRICAL ENGINEER:** Power Design, Inc.
- **CONSTRUCTION MANAGER:** Turner Construction co.

M.E.P. SYSTEMS:

- Each unit controls a split system heat pump and gas-fired hot water heater
- No Boilers or Chillers
- 120/208V 3 Phase 4 wire system
- 277/480V 3 phase 4 wire system
- Diesel fuel powered Fire Pumps and Emergency Generator

DAVID S. FINLEY

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EXECUTIVE SUMMARY:

Building Description

The Silver Spring Gateway is a mixed-use high rise development including 14,080 square feet of retail space, 100,215 square feet of parking, 395,439 square feet of residential space, and a 1,000 square feet roof top swimming pool. The building envelope consists of brick cavity walls and aluminum Centria storefront curtain walls. The main structural system consists of two-way flat plate post-tensioned slabs supported by 176 reinforced concrete columns without a typical bay grid. Every column transfers its load into transfer beams or directly into caissons carrying the load to the bedrock below. The lateral loads are resisted by three twelve inches thick reinforced concrete shear walls in the East-West direction and concrete moment frames in the North-South direction. The Silver Spring Gateway also contains a steel truss bridge spanning thirty-six feet over the garage entrance to connect the two portions of the residential space.

Structural Proposal

For the purposes of this thesis, JBG, hypothetically, acquired a lot in downtown Washington D.C. JBG will petition to alter the C-3-C zone to a C-4 zone to gain twenty to forty more feet in building height thus matching the surrounding buildings. Altogether, this site can accommodate the same architectural layout as the Silver Spring Gateway; however, due to the high profile aspect of the clientele, such as foreign diplomats and national delegates, and proximity to government buildings, the structural design may need altered to resist possible terrorist attacks. First, the locations exuding the most vulnerability to an attack need determined, such as, parking garage, entrance tunnel, exterior façade, etc. Since the current design lends well to several unique scenarios, the structural elements within the existing system will be analyzed per each scenario and redesigned, if necessary, to mitigate the effects of an explosion and to prevent a progressive collapse in case a localized failure occurs.

Breadth Proposal

Site and Landscape Architecture: The site design will need redesigned to prevent the attack scenarios, discussed within the structural study, as much as possible. The new site is larger than the existing site; therefore, more plaza area and landscaping can be accomplished to the accent the building. These designs must keep the tenants safe and comfortable.

Façade Redesign: In addition, the façade may need further attention due to possible historic guidelines and to resisting a street side explosion. The façade change requires focus on blast resistance, architectural and historic concerns, and thermal and moisture protection. Any fenestration changes may also affect mechanical loads and day lighting effects.

Results

Since the explosive weight utilized in an attack is arbitrary and difficult to predict with certainty, the main premise of this thesis is to compromise certain structural members and design to mitigate a progressive collapse. The design methods for alleviating progressive collapse mechanisms are defined in *ASCE 7-05* as a Direct Method (load redistribution) and an Indirect Method (catenary action through tie forces). The existing structure under both methods failed the criteria set by the General Services Administration (GSA) and the Department of Defense (DoD). First, structure was redesigned to pass the criteria, then the cost was analyzed to determine that the Indirect Method is the most efficient and economical method for this building.

In addition to redesigning the structure to withstand abnormal loads from terrorist attacks, the site architecture layout was updated to increase security through passive countermeasures such as bollards,

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public and private spaces to increase the stand-off range, crash rated planters, and perimeter walls. The site access points were reconfigured to limit the number of entrances and incorporate guard houses.

With the stand-off range being over 100 feet, a street side blast has a small effect on the façade glazing which currently is specified as the most optimal choice for blast resistance. With most of the new developments in the area utilizing the same materials for façade wall assemblies, the existing materials also did not need to be redesigned for local considerations; however, the wall assemblies did need to add a vapor barrier to eliminate the potential for condensation and biological growth within the framing and insulation.

Overall, this thesis shows that the building of a residential high rise for foreign dignitaries and domestic diplomats would be relatively feasible especially with a large site. It is recommended that JBG pursue this development in the near future.

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I would like to thank the following people for their assistance on this thesis as well as on my academic development over the past five years:

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INTRODUCTION

With the completion of the Silver Spring Gateway still several months away, the owner, JBG Companies, (JBG) hypothetically has already sold all commercial and residential leases for the property. Due to this simulated success and for the purposes of this thesis, JBG, theoretically, has decided to research possibilities of constructing a high security, luxury development for international diplomats and national delegates while maintaining similar qualities of square footage, layout, style, moisture and noise control and implementing more stringent security protocols and design. The scope of this thesis will include a threat analysis to determine specific locations of higher probability for progressive collapse due to an abnormal loading, such as the malignant use of explosives. With a firm understanding of blast loading theory, an analysis of the existing structure using these prescribed loads will determine deficiencies that need attention. The structure will be redesigned to mitigate any progressive collapse response to an explosion. In addition to the structure, other factors such as site layout and façade treatments will be redesigned to further mitigate the effects of the attack and transform the building to resemble its surrounding environment.

BACKGROUND

The Silver Spring Gateway (Cover and Figure 1 in Appendix A) is located at 1133 East-West Highway in Silver Spring, Maryland. The existing tight, flat urban brownfield site, surrounded by Blair Mill Road to the Northwest, East-West Highway to the South, and CSX Transportation, Inc. Railway to the Northeast was used primarily as a parking lot (Figure 2 in Appendix A). The Silver Spring Gateway site currently abandons a section of Blair Mill Road, transforming the original trapezoidal shaped site to a more useable, rectangular shaped site (Figure 3 in Appendix A). Construction of the fifteen-story, 766,459 square feet building was started in July 2006 and is scheduled to be completed in July 2008 with an estimated bid cost of $89 million. The mixed-use, primarily residential, building owned by The JBG Companies was designed by Weihe Design Group (WDG) of Washington, D.C., and is being constructed under a gross mean price, design-build contract by multiple prime contractors, including general contractor and construction manager Turner Construction Company (Turner) of Washington, D.C. Tadjer, Cohen, Edelson Associates, Inc. (TCE) of Silver Spring, Maryland served as the structural engineering firm (See Appendix C for Project Team Directory).

According to the Urban Land Institute, a development containing "three or more significant revenue producing uses, significant functional and physical integration of the different uses, and conforms to a coherent plan" is defined as a mixed use development. The Silver Spring Gateway certainly exudes this quality as it contains 14,080 square feet of retail space located on the Ground Floor, 100,215 square feet of parking extending from the Basement Level (B1) to the Seventh Floor, and 395,439 square feet of residential space (condominiums and apartments) dispersed among the Second Floor through the Fifteenth Floor (Figure 5 in Appendix A). The Basement Level is a rectangular space below grade completely dedicated to parking. The parking garage is sited in the rear of the building or northeast section and continues with the same shape and overall size for eight floors. The Ground Floor is "L"

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shaped with the long leg parallel to and the short leg pointing toward the East-West Highway and accommodates the lobby, fitness center, and common spaces for the residents; as well as, the retail portion of the building (Figure 6 in Appendix A). The retail space is located in the front of the building or south and southwest section along the East-West Highway and is divided by an internal street located at the southeast corner leading to the parking garage entrance. The service corridor and loading dock for the retail space acts as a buffer between the residential public and retail spaces and the parking garage. The service corridor, loading dock, and portions of the internal street utilize a heated ceiling system.

The second floor contains a portion of the residential space located toward the front of the building and a section of the parking garage located in the rear of the building. With a shape similar to the Ground Floor, the second floor also helps reconnect the portion of the building separated by the internal street with an enclosed pedestrian bridge spanning approximately 36 feet. Floors three through six follow the same layout and shape as the second floor except for the bridge area, which contains residential space. The Seventh Floor also maintains the same layout and shape as floors three through six; however, the floor initiates a shape and layout change through the parking garage section. The center portion of the last parking garage level will be open from above and will be surrounded on three sides by the remaining floors (Photo 2).

The remaining eight floors are strictly for residential use and organized in a "figure four". The corridor running through the center of the layout is doubly loaded, that is, habitable rooms on both sides of the corridor. Starting on the Twelfth Floor, the southern tip of the building shortens and creates a restricted access roof for the remaining four floors. The penthouse roof maintains the "figure four" layout from below and contains several mechanical and electrical rooms, picnic areas, and a 1,000 square foot residential swimming pool with related functional amenities to complete the fifteen story mixed use development (Figure 7 in Appendix A).

The exterior façade of the Silver Spring Gateway is comprised of several different systems. The primary system is a Norwegian and Engineer brick masonry veneer with cold formed light gauge steel back-up framing. The Ground Floor utilizes a similar system, however, is expressed differently with prairie stone along with an aluminum storefront curtain wall system for retail areas. Small portions of the building also exhibit Centria aluminum faced composite panels and metal screen walls near the penthouse level and on the parking garage elevation for acoustical concerns. The owner has also opted to incorporate a moisture control initiative with extensive flashing details and unorthodox elevation construction.

EXISTING STRUCTURAL SYSTEM DISCUSSION

With the Silver Spring Gateway located approximately seven miles from Washington, D.C., it comes as no surprise that the primary structural material is concrete. Per the geotechnical report published by GCE, the foundation system utilizes caissons ranging from 30 inches to 66 inches in diameter with a minimum depth of 10'-0" below grade. Exterior grade and transfer girder beams ranging in size from 12 inches by 30 inches to 54 inches by 66 inches were needed to avoid the 72 inches in diameter storm line that travels through the site. A four inches thick slab on grade and spread footings were also employed where appropriate.

While the basement level and ground floor systems are 8 inches or 12 inches thick normal weight cast in place reinforced concrete, the remaining floors utilize a 7 to 9 inches thick two-way flat plate post tensioned concrete system with one-way banded tendon distribution over column lines perpendicular to uniformly distributed tendons (Figure 4 in Appendix A). One hundred and seventy-six reinforced

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concrete columns, ranging in concrete compressive strength from 4,000 pounds per square inch to 8,000 pounds per square inch, support the selected floor systems. Only the lower level columns have 10 feet by 10 feet by 5 ½ inches thick drop panels. Several columns are sloped to realign the upper floor grid with the lower floor grid. While the bay dimensions are not consistent throughout the building with rotated columns and radial column lines, the longest span of the two-way flat plate post tensioned floor slab is approximately 27 to 30 feet. The building envelop is supported by continuous 3/8 inches thick bent plates with ¾-inch diameter wedges at two feet on center. The lateral load resistance of the Silver Spring Gateway relies on a combined system of shear walls and concrete moment frames. Lateral loads acting in East-West direction are resisted by three 12 inches thick concrete shear walls, located in the north, east, and south corners of the building, reinforced with #6 bars at six inches on center below the Second Floor and #5 bars at eight inches on center above the Second Floor. In the North-South direction, the concrete moment frames along each column line resist the lateral loading.

Although most of the Silver Spring Gateway structure is cast in place reinforced or post tensioned concrete, the enclosed pedestrian bridge and canopy structures are exposed structural steel. The bridge system in particular is constructed of a 6 ½ inches thick composite concrete slab on six steel trusses composed of W14x114 and W12x210 chords, W12x190s, and W10x45 web members spanning approximately 36 feet (Photo 7 in Appendix B). Several W16, W14, and W12 composite infill beams, along with the steel trusses, are moment connected utilizing full penetration welds (Photo 8 in Appendix B). Composite W14x257 steel columns encased in a two feet by two feet concrete column supports the entire bridge structure. The canopy members and wall panel supports are typically tube shaped steel members.

PROBLEM STATEMENT

For the purposes of this thesis, JBG, hypothetically, acquired the lot surrounded by the Northwest sections of 9th Street to the East, 11th Street to the West, New York Ave to the North, and H Street to the South (Figure 8 in Appendix A). JBG will petition to alter the C-3-C zone to a C-4 zone in order to gain twenty to forty more feet in building height thus matching the surrounding buildings. Altogether, this site can accommodate the same architectural layout as the Silver Spring Gateway; however, due to high profile clientele and proximity to government buildings, the security of the building's site and design against enemy attacks is paramount for the area and its residents, now more than ever. According to Eve Hinman, "hardening structures against weapon effects has been, until recently, of concern almost exclusively of the military. However, with the increase of terrorist activities directed against civilian targets, there is a growing interest in applying these principles to the design of non-military structures" (qtd. in Bounds 1-17). Evidence shows that among terrorist attacks the use of explosive devices is most common as it requires few skills and limited to zero socio-political faction aid (National Research Council 9-10). As a part of the National Research Council's study, they found the following statistics for non-military, domestic structures:

- Residential + Commercial = 74 + 13 = 87 killed (55% of total killed in all incidents)
- Residential + Commercial = 473 + 1290 = 1763 injured (71% of total injured in all incidents)
- Residential + Commercial = 2553 + 1468 = 4021 incidents (37% of total incidents)

With these figures and lingering horrific images of other high profile buildings, such as the Alfred P. Murrah Building in Oklahoma City in 1995 and the World Trade Center in New York City in 1993 and 2001, it is clear that the structural capacity to withstand an explosive attack without an ensuing progressive collapse needs to be addressed in the overall design of the building.

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PROPOSED SOLUTION

"Designing structures to withstand the effects of a deliberately placed explosive device can entail many types of protective measures. Some will increase the difficulty of placing a bomb close enough to a structure to damage it; others will physically strengthen all or parts of the structure while still others will aim to ensure the survival and rescue of the occupants in the event of a bomb explosion" (National Research Council *ix*). Eve Hinman simplifies and categorizes these measures stated in order of declining efficacy as "'deterrence', 'keep-out', 'deception', and 'hardening'" (1-17). Public safety remains the paramount goal of all ethical engineers; therefore, the scope of this thesis will focus on redesigning the site layout to maintain a level of prevention and deny entry to possible attackers and on redesigning the façade and structural elements to respond safely in case the attacker manages to succeed.

Structural Breadth

Before any structural redesign can commence, a firm understanding of possible threats to the entire building as well as the structure, of probable locations of high failure, and of blast load theory is needed. With this knowledge, the existing structure will be modeled to predict the blast load effect at specific areas of the structure. Since a structural member will mostly likely be compromised due to the blast load, the remaining structure will need to span a greater distance than was originally designed; therefore, causing over-stressed structural elements, thus leading to a progressive collapse. The ensuing redesign to strengthen the structure's response to the explosive will take the form of one of two possible concepts compared based on cost, architectural feasibility, and strength efficiency.

The first design will utilize either one or two, two-story Vierendeel trusses located at the Fourth and Tenth Floor. The truss action will redistribute the loads from the structure above the increased span to undamaged structural elements below. The truss will also provide support to any remaining structural elements below by allowing them to hang from the bottom chord. A Vierendeel truss will be utilized instead of a roof-top mega-truss or other floor level trusses with diagonal web members to avoid defiling the architectural layout.

The second design will incorporate the use of beams at each floor to span the targeted area. The beams will not be designed with the capacity to carry the redistributed loads, merely yield without ultimate failure to enable safe evacuation. This efficient design will rely on catenary action of the reinforcement to maintain some building integrity.

Site Architectural Layout Breadth

While the structural breadth will enhance the capacity of the structure to mitigate a progressive collapse cause by local failure near the epicenter of the blast, the primary goal is preventing the attack from occurring altogether. The new site is large enough to entertain several design concepts, such as barriers, increased stand-off distances, controlled access, etc., to reduce the possibility and ability of an attack. However, the design should maintain a balance between security and a pleasant inhabitable space surrounding the exterior of the building.

Façade Breadth

Most of the structural redesigns will try to have little to no adverse effects to the façade of the high-rise; however, the façade will need altered in the sense of materials to match the local architectural language of

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the area. The effects of the new materials on the structural supporting system, heating and cooling, moisture and thermal resistance, and artificial lighting of the building will be closely monitored to maintain as much of the original design as possible.

BUILDING THREAT ANALYSIS

While this thesis focuses on the threats related to the structural system, any building undergoing a security analysis would be required to address all possible threats posed on the building. The following is a good representative list compiled by James A. Shipe:

1. Structural System
   a. Stationary Blasts/Explosions
      i. Interior
      ii. Exterior
   b. Mobile Attack
      i. Airplane
      ii. Missile
      iii. Truck or Car
2. Mechanical System
   a. Chemical Weapons
   b. Biological Weapons
   c. Other forms of intentional contamination
3. Electrical System
   a. Alarm system tampering
   b. Power disruption
   c. Critical systems interference
4. Fire Protection
   a. Arson
   b. Accidental
5. Utilities
   a. Potable water protection
   b. Electrical supply protection
   c. Data cable protection
6. Site Access
   a. Dumping on site
   b. Unlawful residents
7. Building Access
   a. Aggressive/combative person(s)
   b. Robbery
   c. Protection of sensitive information
   d. Computer network infiltration
8. Other
   a. Mail
   b. Hostage

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BLAST LOAD THEORY

Currently, with many political and religious factions pledging to destroy the fabric of Western Civilization, more emphasis in blast resistant design for our domestic structures has occurred. In order to accomplish this ideal, the dynamic properties of explosions and their effective loads on buildings must be researched and fully comprehended to enable successful, safe design.

Explosive load effects are directly proportional to stress-wave propagation resulting in a dynamic pressure on the structure. For buildings, the two primary types of explosions are either close open-air blasts (exterior) or confined explosions (interior). For a close explosion, the air blast creates an overpressure on all sides of the building due to the impinging shock front and its reflections. The incident dynamic pressure from the spherical shock wave can be related to the following equation:

$$P=W/R^3$$

Where P is the incident pressure on the building, R is the radial distance between the epicenter of the blast and the target or stand-off distance, and W is the equivalent explosive weight, normally expressed in TNT. The graph below shows the magnitude of the incident pressure compared to the stand-off distance for a given explosive weight:

It is clear that a greater distance between the explosive device and the structure results in a lower intensity of the blast pressure (National Research Council 29-30).

While open-air blasts are more defined and researched, confined explosions are more uncertain. While all blasts follow the above properties, a confined explosion results in increased shock reflections, possible internal explosions, and a rapid increase in gas pressure. The shock reflections and gas pressures need to decay over time in a controlled manner to produce similar qualities as an open-air blast; therefore, the variability in these factors depends on the ventilation of the structure. The complexity of this type of explosion requires the use of semi-empirical Computational Fluid Dynamics (CFD) and Solid Mechanics (CSM) modeling programs to better predict the blast loading effect on the structure (National Research Council 35-36).

Along with the determination of the equivalent load and properties of the blast, the understanding of the structural response to the shock wave is critical for proper application of the loads. The building responds to the explosive load in a sequence of three separate stages (Figure 1). First the shock wave reflects off the nearest surface, or exterior façade, causing the glazing to shatter and the exterior walls and edge columns to deflect under the reflected incident pressure. Additionally, the shock waves will expand and diffract to allow the overpressure to act on the remaining sides (Hinman 7).

Secondly, the reflected pressure that penetrates through the openings created from the shattered glazing and other infringements of the facade becomes an internal pressure. This pressure will exert both a downward and upward pressure on the floor slabs. The upward pressure is critical during this
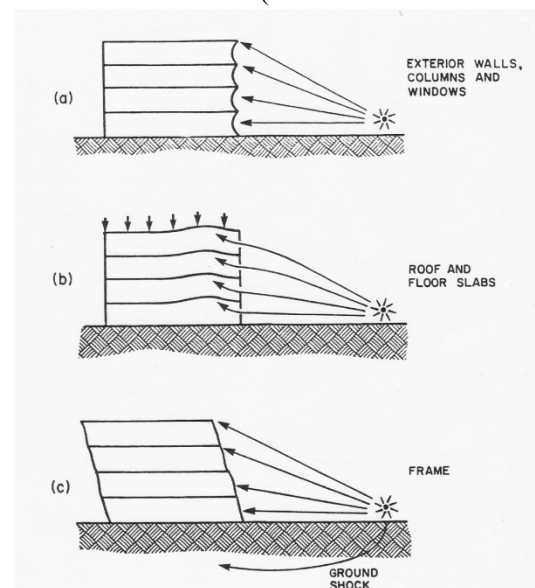

Figure 1: Blast Load Phases (Hinman 7)

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stage as the columns and slabs are not designed for load reversals. It should also be noted that during this stage of internal pressures, occupants could experience injury to their lungs and blood vessels (Hinman 7).

Lastly, the structural frame reacts to the ground transmitted shock from the blast loads as the mass or inertia of the building drifts. The side sway exhibited by the structural frame is equivalent to the motion due to a short duration, high intensity earthquake (Hinman 7).

The properties of explosions and their effective loads on a structure are critical to the successful design of a safe, blast resistant building. Due to the nature and location of the explosive placements in this thesis, the explosion would be considered confined. Since this is a relatively difficult load to determine and the actual explosive weight used would be merely a prediction, it will be assumed that the primary targets will be damaged by the blast and the analysis of the remaining structure will focus on progressive collapse mitigation.

BLAST THREAT SCENARIOS

Using the data from past attacks, it is safe to predict that any attack on the structural system will use an explosive device. With the time allotted for this thesis it will not be feasible to assess every possible location of localized failure due to a blast; however, it will focus on two locations exhibiting the most likely bomb placement and the most probable location for a progressive collapse. The first area of interest is the transfer girder truss acting as a bridge to connect two portions of the building (Figure 2). There are four trusses connecting the building sections and creating an entry point for the parking garage located within the building. Each truss supports one 16x28 inch column at mid-span. General Services Administration (GSA) discourages the use of transfer girders in potential target structures due to the lack of redundancy and the lack of alternative load paths since it is defined as a single point failure mechanism (GSA 2-4). Evidence of the possible result of losing a transfer girder due to a blast load can be seen in images of the Alfred P. Murrah Building in Oklahoma City post detonation (Figure 3).
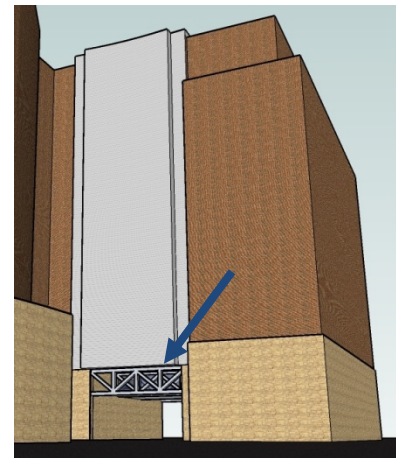

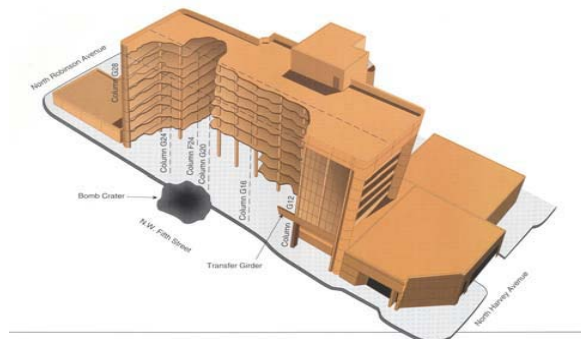Figure 2: Mass model showing location of transfer girder (*blue arrow*)

Figure 3: Failure Boundaries (*left*); North side of building after explosion (*right*) (NIST 181)

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With four trusses each supporting a column, the potential for a car bomb to take out one or more transfer girder trusses is relatively high. Thus leaving an approximately 60 foot span unsupported which could trigger a progressive collapse of the structure. As mentioned previously, the actual blast load will not be calculated due to the setting of the explosion. Since this entrance is acting as a tunnel, the explosion will be partially vented and confined thus creating uncertainty in the intensity of the reflected shock waves. With uncertainty in the load effect and in the variable bomb weight used, it will be assumed that only one truss and its adjacent columns will be hypothetically removed due to a blast load. However, the retrofit design for this span will be reflected in the other three spans.

The second area will be within the parking garage itself. Here, a column and a slab section will be removed in locations that warrant the greatest risk of triggering a progressive collapse. Firstly, the existing post-tensioned slab will need to be redesigned as a reinforced concrete slab. Limited research has been conducted in the energy absorption properties and overall blast resistance of a post-tensioned slab; however, it is inherent that a slab containing unbonded tendons subjected to a blast load could potentially destroy the tendon anchorage zones and the tendon strands, thus lowering the capacity of the slab along that tendon line throughout the entire length of tendon line.

With the new reinforced concrete slab, a similar procedure as the transfer girder will take place at a column within the garage. Again, due to the close proximity of the blast epicenter from the column and the confined, partially vented space within the garage the blast load is not well-defined without advanced computer modeling; therefore, one scenario will assume that one column will be completely destroyed causing a potential 56 foot slab span.

PROGRESSIVE COLLAPSE MITIGATION THEORY

Since the assumption, previously mentioned, to compromise the target structural element due to the blast load complexity causes a localized failure, it is prudent to understand and mitigate the effects of a progressive collapse. *ASCE Standard 7-05* defines a progressive collapse as "the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it" (ASCE 249). With the potential for the assumed local failure to develop into a more serious global failure, a proper discussion on progressive collapse mitigation prior to retrofitting the highlighted areas of interest is warranted, since each redesign for the target locations will utilize the same load combinations and similar concepts to increase the ductility and redundancy of the structure to better withstand the abnormal loading.

Since *ASCE Standard 7-05* does not explicitly define a load combination for extreme or abnormal loading conditions, the load combination presented in a report published by the National Institute for Standards and Technology (NIST) will be used:

$$(0.9 \text{ or } 1.2) \, D + 0.5 \, L + (0.2 \, W \text{ or } 0.2 \, S)$$

Most of the load factors are less than unity due to probability and damage. The probability of a full design snow or wind load at the time of the blast is very high; in fact, the chance of exceeding this annual probability is about 0.05. The damage to the structure and its contents following an explosion allows for the live load factor reduction. A similar concept could be utilized on the dead load of the structure; however, due to uncertainty in the actual dead load and suspicion in the dependability of a damaged structure, the load factor could be taken as 1.0 or higher, especially as the number of unaffected stories increases (NIST 22).

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The design methodologies of mitigating a progressive collapse under the load specified by this load combination are defined in *ASCE Standard 7-05* within the commentary. On page 250, the "design alternatives" are prescribed as follows:

Direct Design: Explicit consideration of resistance to progressive collapse during the design process through either:

Alternative Path Method: A method that allows local failure to occur, but seeks to provide alternate load paths so that the damage is absorbed and major collapse is averted.

Specific Local Resistance Method: A method that seeks to provide sufficient strength to resist failure from accidents or misuse.

Indirect Design: Implicit consideration of resistance to progressive collapse during the design process through the provision of minimum levels of strength, continuity, and ductility.

Since the assumption has been made to allow local failure to occur, the Specific Local Resistance Method will not be utilized. It is worth noting, that this method requires a medium level of complexity in analysis and generally produces more expensive structural sections. The other direct design method of developing an alternative load path can use either a secondary transfer system or using non-linear inelastic analysis methods. This method and any indirect approaches are relevant to the assumptions made for this thesis as they address a proposed failure condition without consideration of the explosion that caused it. "This threat independent approach is specified by U.S. Government agencies including the U.S. General Services Administration and the Department of Defense" (NIST 31).

STRUCTURAL REDESIGN DISCUSSION

Using the guidance of collective U.S. Government agency publications and *ASCE Standard 7-05*, the blast threat scenarios described earlier will be redesigned to perform safely in the event of a terrorist attack. Each location will be redesigned using the direct and indirect method. The resulting structural elements will be compared based on effectiveness and overall additional cost to JBG.

Transfer Girder Truss

Indirect Method

In order to enhance the continuity and ductility of a reinforced concrete frame one must provide tie reinforcement. These ties are a convenient method since the flexural reinforcement can be employed to resist the tie forces due to abnormal loadings; however, the detailing of the tie is critical to ensure it acts continuously to resist the catenary action mechanism. Tie reinforcement comes in many forms: internal, vertical, horizontal to column, peripheral and corner column ties (Figure 4).
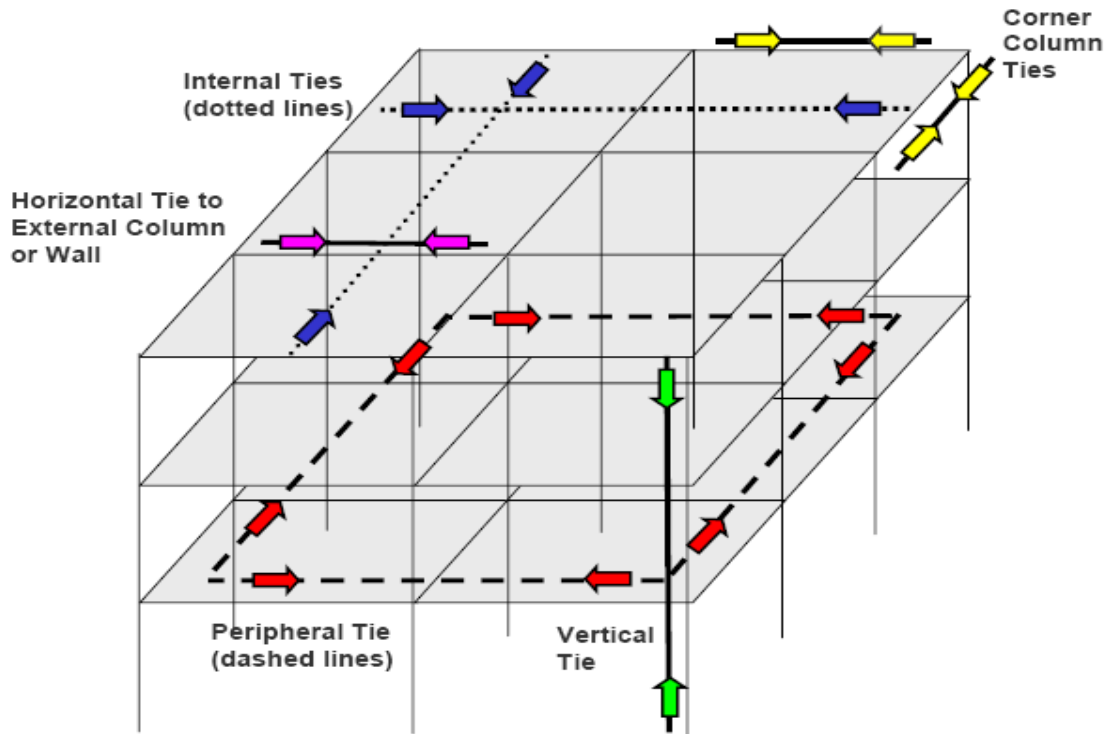
Figure 4: Diagram of tie forces taken from Design of Buildings to Resist Progressive Collapse, UFC 4-023-03.

For this particular location, internal, vertical, horizontal to column and peripheral ties will be needed. The procedure for tie design is as follows:

- Determine tie forces
- Determine required steel area to resist tie force
- Check provided steel area
- Add steel and detail connections if needed

The Department of Defense (DoD) has developed equations for each of the individual tie forces. For internal ties the tie force is the smaller of:

$$\frac{D+L}{156.6}\frac{l_r}{16.4}\frac{F_t}{3.3} \text{ or } \frac{F_t}{3.3} \text{ (kip/ft)}$$

where l_r equals the greater of distances between the centers of the columns supporting any two adjacent floor spaces in the direction of the tie and F_t is the lesser of:

$$4.5 + 0.9n_o \text{ or } 13.5 \text{ (kip/ft)}$$

where n_o is the number of stories (4-3). Vertical ties within the columns have a tie force equal to the tensile force created from the maximum factored single story load received by the column in kips. The horizontal to column tie force is defined as the greater of:

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$$0.03[4(D + L)]A_{trib} \text{ (kips) or the lesser of}$$

$$2.0 \, F_t \text{ or } \frac{l_s}{8.2} \, F_t \text{ (kips)}$$

where l_s equals the floor-to-floor height (4-4). Peripheral ties have a design strength of $1.0F_t$ in kips (4-3).

According to the DoD, the reinforcement must provide a design strength greater than the required tie strength multiplied by a strength reduction factor, Φ, and an over-strength factor, Ω_o. The strength reduction factor is prescribed by the American Concrete Institute (ACI) for steel reinforcement in tension as 0.75 (ACI 109). The over-strength factor is given as 1.25 for both materials in Table 4-1 (DoD 4-1). The over-strength factor considers the extra "hidden" capacity within the materials. That is, when designed most structural sections are larger than required, the design capacity is reduced by its appropriate factor, and the material strength itself is a conservative statistical average such that most elements are higher than the design strength. All of these reasons allow for an over-strength factor for this application.

The following table shows the results of the tie forces and the steel areas (See Appendix E for sample calculations):

Tie Type	Tie Force	Required A_s	Provided A_s
Peripheral	13.5 kips	0.24 in^2	3.95 in^2
Internal (N-S)	7049 lb/ft-width	0.125 in^2/ft	0.61 in^2/ft
Internal (E-W)	6579 lb/ft-width	0.117 in^2/ft	0.402 in^2/ft
Horizontal	14.95 kips	0.266 in^2	1.24 in^2
Vertical	149.3 kips	2.654 in^2	5.08 in^2

The original steel provided within the columns and with the slab are adequate to resist the prescribed tie forces; however, the existing detail of the rebar within the slab lacks continuity and will not properly react to the tie forces (Figure 5). Therefore, the new details to ensure proper load transfer and enhance ductility show continuity and proper anchorage (Figures 6 and 6a).

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Figure 5: Existing detail follows typical detail within ACI 318-05 with (a) equaling the greater of $0.22L_n$ (Middle Strip), $0.30L_n$ (Column Strip), or the cantilever length and (b) equaling $0.22L_n$ (Middle Strip), $0.30L_n$ (Column Strip).
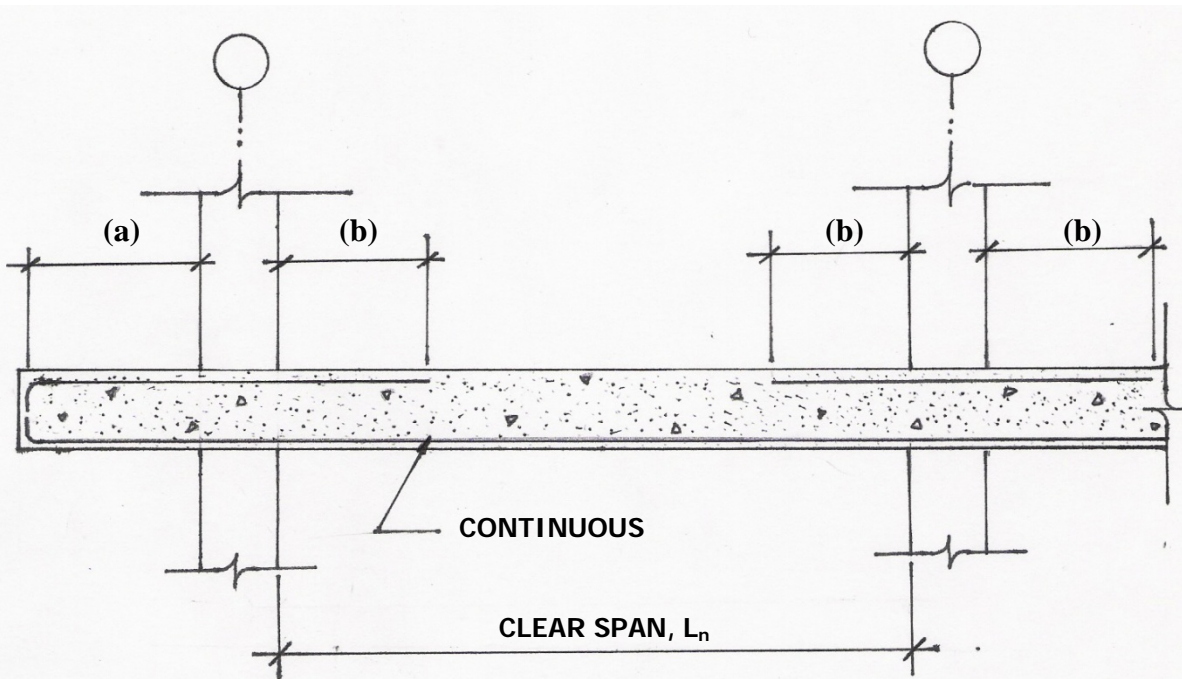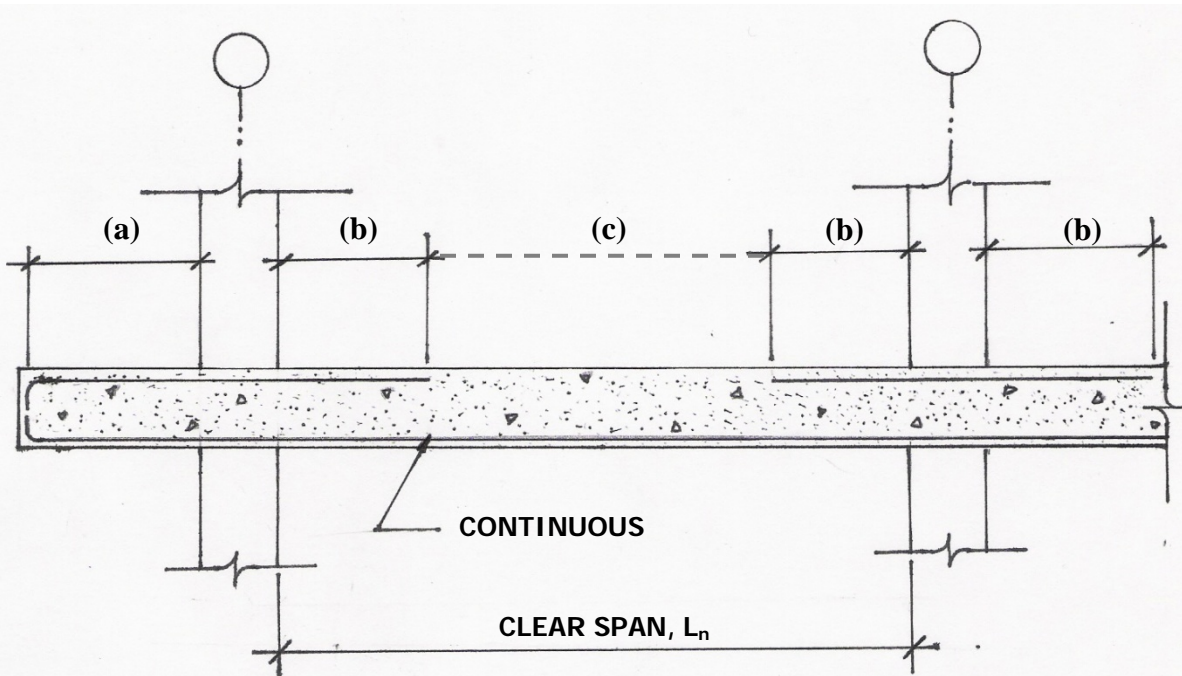

Figure 6: New detail follows typical detail within ACI 318-05 with (a) equaling the greater of $0.22L_n$ (Middle Strip), $0.30L_n$ (Column Strip), or the cantilever length, (b) equaling $0.22L_n$ (Middle Strip), $0.30L_n$ (Column Strip) and (c) continuous portion of top reinforcement to satisfy the tie force strength.

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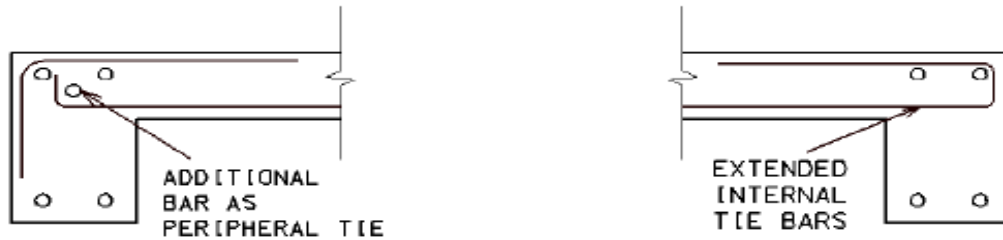
Figure 6a: Typical anchorage and placement details provided by Design of Buildings to Resist Progressive Collapse, UFC 4-023-03.

The indirect method proves to be an efficient design to mitigate a progressive collapse as it uses existing steel; however, the extra detailing will increase the amount of rebar consequently inflating the total cost. A summary of the construction costs and lease premiums can be found in the *Conclusion* section on page 18.

Direct Method

As previously mentioned, the assumptions made for this thesis rules out the Specific Local Resistance Method as a viable design solution; therefore, the Alternative Load Path approach will be analyzed. An alternative load path can be established in a variety of ways. NIST documents the following possibilities (68-69):

- Two-way action
- Secondary trusses
- Vierendeel action
- Strong floors

The low floor-to-floor height will not make the placement of strong floors easy. In order not to defile the rentable space and reduce the lease premium on specific floors, this option will not be analyzed. Secondary trusses are rather effective in reducing the displacement from the loss of a critical member; however, a secondary truss would need to be designed for each of the four columns. Truss placement within the building is discouraged for this thesis, as it would disrupt the architectural layout. Therefore, it would need to be placed on the roof. While this could be integrated into the architecture, it would add too much redundancy to the structure because the building would now maintain two identical truss systems at the Roof and 2nd Floor levels. This option also will not be analyzed due to its inefficiency and added expense.

Since the structure already utilizes a moment frame to resist lateral loads in the direction of the transfer girder truss span, vierendeel action can be a great unobtrusive solution if the connections can be enhanced to support the abnormal loading. With further investigation, the chords of a vierendeel truss could have a maximum depth of thirteen inches thus making the width impractical. If a vierendeel truss were designed for each floor above the transfer girder trusses, the chord width would still be approximately four feet wide which would have a negative impact on the rentable space. While the existing conditions presented a possible solution, it is certainly not the best visually for the occupant. (See Appendix E for calculations regarding this system)

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The post-tensioned slab utilizes two-way action increasing the robustness of the structure; therefore, this action will be exploited to allow plastic hinges and moment redistribution as proscribed by GSA to perform agreeably under the constraints of this thesis.

The linear elastic analysis prescribed by GSA allows the existing design to remain with slight alterations to slab depth and/or reinforcement sizes. The acceptance criteria established by GSA determines the distribution of demand loads due to the devastating event. The distribution of primary and secondary structural elements will be shown by "Demand Capacity Ratios (*DCR*):

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

Where, Q_{UD} is the acting force within the structural element and Q_{CE} is the unfactored ultimate capacity of that structural member. If the ratio is exceeded based on the following allowable values, then a plastic hinge will develop at that location:

- *DCR* ≤ 1.5 for atypical structural configurations (transfer girder)
- *DCR* ≤ 2.0 for typical structural configurations (parking garage)

The following step-by-step procedure on page 40 of *Progressive Collapse Analysis and Design Guidelines* published by GSA was followed utilizing Staad.Pro for member forces, moment distribution, and hinge development:

Step 1. Remove a vertical support from the location being considered and conduct a linear-static analysis of the structure.

Step 2. Determine which members and connections have *DCR* values that exceed the acceptance criteria. If the *DCR* for any member end 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), the member is to be considered a failed member. 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.

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. **If moments have been re-distributed throughout the entire**

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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.

The analysis considered the effects of gravity loads only and took several iterations before the redistribution ceased causing further failure. The following Staad.pro outputs show the existing frame with the transfer girder and supporting column removed (Figure 7), the existing frame failure after complete hinge development leaving a potential instability in the slender portion to resist lateral load potentially causing further collapse (Figure 8).
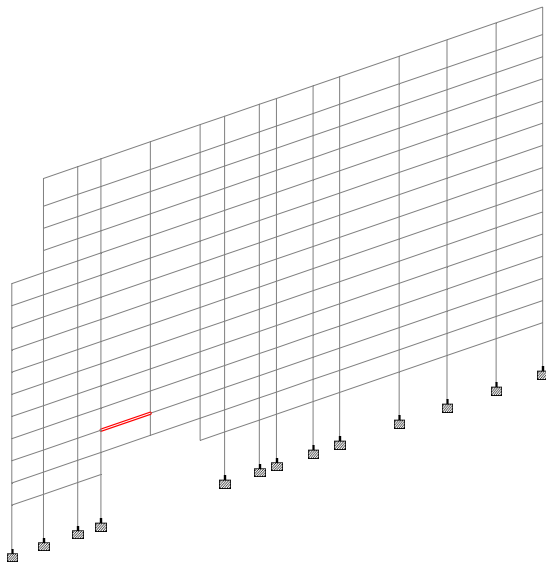
Figure 7: Original frame on Col. Line 11 (See Figure 9 in Appendix A).
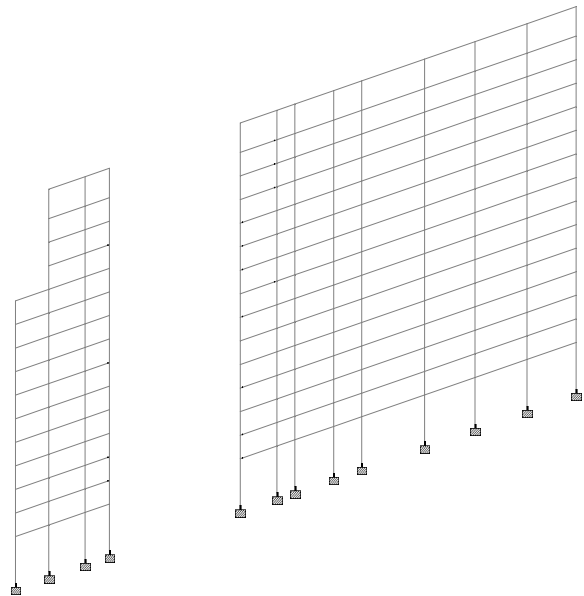
Figure 8: Partial frame collapse after explosion.

After the first iteration, the mild-steel reinforcing was redesigned to increase the ultimate moment capacity of the post-tensioned slab. Over several runs, the steel area was increased causing more retention of the structure; however, the failure extent still exceeded the provisions of GSA, which allows for local failure of members or bays connected to the compromised member(s), and the required steel area became impractical. The final iteration utilized an inch and a half thicker slab and increased steel area to produce an acceptable failure mode (Figures 9 and 10). (See Appendix E for calculations).

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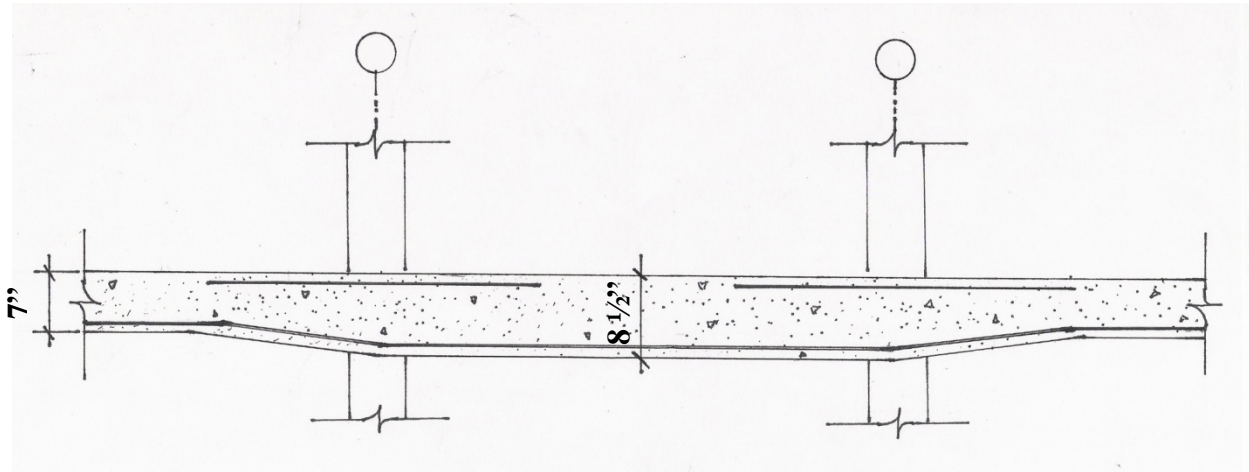
Figure 9: Typical detail of increased slab and rebar at transfer girder locations (See Appendix E for Calculations).
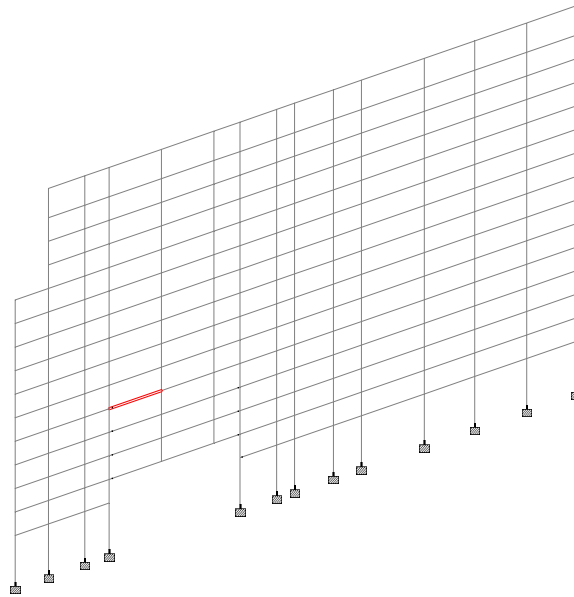
Figure 10: Extent of failure after redesign entails a portion of the column
and slab connected to the transfer girder and column below which passes
the GSA criterion.

This direct method of linear elastic analysis proves to be an efficient design to mitigate a progressive collapse as it uses existing structural sections; however, the extra steel and concrete will increase the total cost. A summary of the construction costs and lease premiums can be found in the *Conclusion* section on page 18.

Parking Garage

Since there is a lack of research on the energy absorption properties and overall blast resistance of a post-tensioned slab and the vulnerability of tendon strands and tendon anchorage zones within the slab, the

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existing post-tensioned slab was redesigned with two options using hand calculation, PCASlab, and SAP 2000 to verify the design. The first option will utilize a ten inches thick two-way flat plate reinforced concrete slab and will be analyzed per the indirect method. The second option for the direct method analysis will be a two-way reinforced concrete slab with beams (See Appendix E for calculations and computer output and Figure 9 in Appendix A for location).

Indirect Method

Using the same procedure and theory presented by the Department of Defense previously utilized for the transfer girder, the tie forces and required steel areas (shown below) were found for the parking garage slab (See Appendix E for sample calculations):

Tie Type	Tie Force	Required A_s	Provided A_s
Peripheral	11.7 kips	0.22 in^2	1.24 in^2
Internal (N-S)	10531 lb/ft-width	0.1877 in^2/ft	0.62 in^2/ft
Internal (E-W)	9593 lb/ft-width	0.1705 in^2/ft	0.402 in^2/ft
Horizontal	14.95 kips	0.266 in^2	0.93 in^2
Vertical	149.3 kips	2.654 in^2	5.08 in^2

The redesigned slab reinforcement is adequate to resist the prescribed tie forces; however, the existing detail of the rebar within the slab lack continuity and will not properly react to the tie forces (Figure 5). Therefore, the new details, to ensure proper load transfer and enhance ductility, show continuity and proper anchorage (Figure 6 and 6a).

The indirect method proves to be an efficient design to mitigate a progressive collapse as it uses existing steel; however, the extra detailing will increase the amount the rebar and inflate the total cost. A summary of the construction costs and lease premiums can be found in the *Conclusion* section on page 18.

Direct Method

Using the same procedure and theory presented by GSA previously utilized for the transfer girder, the parking garage slab will undergo a linear elastic analysis to determine hinge development and load redistribution. For this analysis, SAP 2000 was utilized to determine the member forces and hinge locations. The deflected structure once removing the column at the critical bay thus creating the maximum span for the beam is shown in Figure 11.

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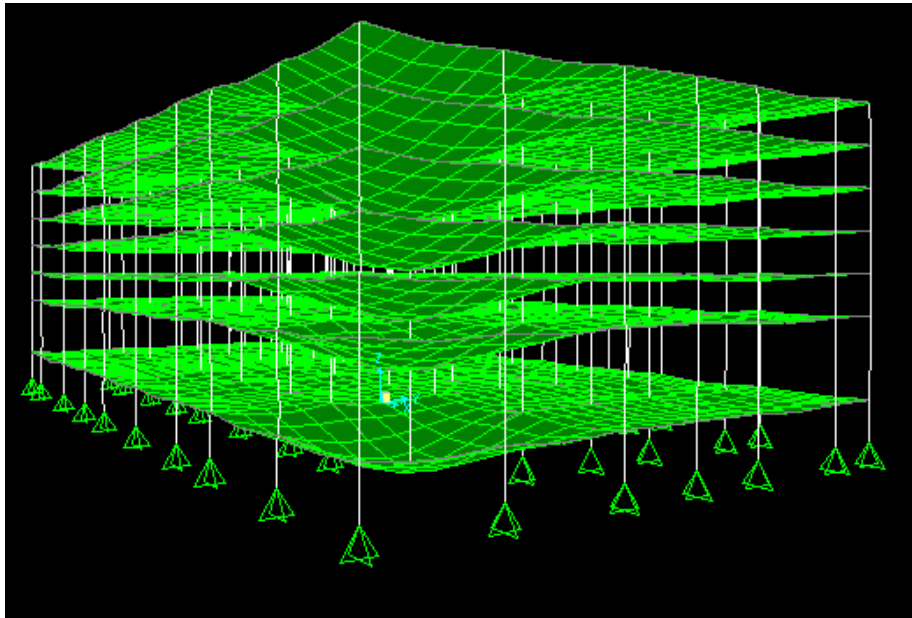
Figure 11: SAP 2000 3D view of deformed shape with target column removed.

The following table summarizes the required steel area provided in the redesign and in the linear elastic analysis for the failure location (Due to the size of the spreadsheet containing the required steel for the entire garage, just the blast location is presented; however, the spreadsheet is available upon request):

Frame	Span	Location	$A_{sDESIGN}$ (in^2)	$A_{sCOLLASPE}$ (in^2)	$A_{sADD.}$ (in^2)	% Increase
10	H-I	Top	0.692	1.007	0.315	45.5
10	H-I	Bottom	0.658	0.658	0	0
10	I-J	Top	0.732	0.821	0.089	12.2
10	I-J	Bottom	0.658	0.658	0	0
I	8-9	Top	1.338	4.020	2.682	200
I	8-9	Bottom	1.125	1.836	0.711	63.2
I	9-10	Top	1.107	4.594	3.487	315
I	9-10	Bottom	0.936	2.120	1.184	126

This direct method of linear elastic analysis proves to be an efficient design to mitigate a progressive collapse as it uses existing structural sections; however, the extra steel area will increase the total cost. Given the time for this thesis, every column in the entire garage cannot be done; therefore, the percentage increase of the steel will be utilized for every bay to determine an estimated cost for additional reinforcement. A summary of the construction costs and lease premiums can be found in the *Conclusion* section.

Conclusion and Cost

Since both methods of progressive collapse design succeed in maintaining most, if not all, of the original architectural layout and mitigating a catastrophic collapse, the most economical design method will be chosen overall. The estimated cost of the redesign will be based on the changes and will assume all other original construction costs fixed.

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Indirect Method

For the transfer girder redesign, only rebar detailing changed overall; therefore, the cost premium will be assessed based on the cost due to the additional length of rebar necessary to provide the proper tie forces calculated prior. First, the existing cost of the rebar will be determined and compared with the cost of the redesign. RSMeans sets the price of rebar for elevated slabs at $900 per ton. The cost analysis can be found in the table below:

Frame	Span (ft)	Existing Reinforcement		Existing Cost	Redesign Reinforcement		Redesign Cost
		Top (lb)	Bottom (lb)		Top (lb)	Bottom (lb)	
11AB	15.45	143.93	0.00	$64.77	143.93	32.23	$79.27
11BC	16.75	93.87	0.00	$42.24	105.86	34.94	$63.36
11CD	11.28	68.84	0.00	$30.98	69.42	23.53	$41.83
11DE	24.14	87.61	0.00	$39.43	112.94	50.36	$73.48
11EF	24.30	81.35	0.00	$36.61	109.10	50.69	$71.90
11FG	12.00	68.84	0.00	$30.98	70.92	25.03	$43.18
11GH	21.08	93.87	0.00	$42.24	106.55	43.97	$67.74
11HI	22.10	262.84	138.30	$180.51	262.84	138.30	$180.51
		TOTAL per Floor and Frame		$467.75	TOTAL per Floor and Frame		$621.28
		TOTAL per Frame		$7,016.31	TOTAL per Frame		$9,319.18
		TOTAL (N-S)		$28,065.25	TOTAL (N-S)		$37,276.72
A1111A	26.00	150.19	83.44	$105.13	179.40	95.96	$123.91
A11A12	23.00	116.82	83.44	$90.12	141.85	89.70	$104.20
A1213	14.00	79.27	58.41	$61.95	85.53	58.41	$64.77
		TOTAL per Floor and Frame		$257.20	TOTAL per Floor and Frame		$292.87
		TOTAL per Frame		$3,858.06	TOTAL per Frame		$4,393.12
		TOTAL (E-W)		$42,438.63	TOTAL (E-W)		$48,324.28
		TOTAL		$70,504	TOTAL		$85,601

The difference between the existing design and the redesign is approximately $15,700 adjusted for the Washington D.C. cost index. According to the National Research Council, for every one percent of cost premium, the lease rate increases by 0.385 percent (86). With a total building cost of $89 million, the blast hardening produces a negligible change in the lease premium rate.

For the parking garage, a change in the floor system was necessary to increase ductility and resistance to abnormal loadings. In this thesis, the existing two-way flat plate post-tensioned concrete slab was redesigned as a two-way flat plate reinforced concrete slab. For the D.C. area, the cost per square foot for a post-tensioned slab is roughly $1.50 more than a conventionally reinforced slab. Similar to the transfer girder, the rebar detailing increases the code mandated length of negative reinforcement, thus increasing the cost of the slab. With the same prices used for the transfer girder, the following table shows the cost analysis for this system:

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Frame	Existing Reinforcement		Existing Cost	Redesign Reinforcement		Redesign Cost
	Top (lb)	Bottom (lb)		Top (lb)	Bottom (lb)	
1	356.97	534.36	$401.10	605.76	534.36	$513.05
2	785.33	1068.73	$834.32	1034.12	1068.73	$946.28
3	856.72	1202.32	$926.57	1105.51	1202.32	$1,038.52
4	785.33	1068.73	$834.32	1034.12	1068.73	$946.28
5	785.33	1202.32	$894.44	1034.12	1202.32	$1,006.40
6	928.11	1335.91	$1,018.81	1176.90	1335.91	$1,130.77
7	785.33	1202.32	$894.44	1034.12	1202.32	$1,006.40
8	785.33	1202.32	$894.44	1034.12	1202.32	$1,006.40
9	1070.90	1469.50	$1,143.18	1319.69	1469.50	$1,255.14
10	571.15	801.55	$617.71	819.94	801.55	$729.67
	TOTAL per Floor		$8,459.34	TOTAL per Floor		$9,578.89
	TOTAL (N-S)		$59,215.36	TOTAL (N-S)		$67,052.26
H	1282.37	1349.12	$1,184.17	1668.83	1349.12	$1,358.08
I	2180.03	2473.39	$2,094.04	2566.49	2473.39	$2,267.95
J	1923.55	2248.53	$1,877.44	2310.02	2248.53	$2,051.35
K	1538.84	1798.83	$1,501.95	1925.31	1798.83	$1,675.86
L	1795.32	2023.68	$1,718.55	2181.78	2023.68	$1,892.46
M	1667.08	1798.83	$1,559.66	2053.55	1798.83	$1,733.57
N	641.18	674.56	$592.09	1027.65	674.56	$765.99
	TOTAL per Floor		$10,527.89	TOTAL per Floor		$11,745.26
	TOTAL (E-W)		$73,695.22	TOTAL (E-W)		$82,216.80
	TOTAL		$132,911	TOTAL		$149,269

The difference between the existing design and the redesign is approximately $17,000 adjusted for the Washington D.C. cost index. However, the overall cost savings for the floor system accounting for the approximate square footage costs is $270,000. According to the National Research Council, for every one percent of cost premium, the lease rate increases by 0.385 percent (86). With a total building cost of $89 million, the blast hardening produces a 0.12% reduction in the lease premium rate.

Direct Method

At the transfer girder, the slab depth and rebar area had to be increased to develop enough ductility to redistribute the loads appropriately to mitigate a progressive collapse. Using costs published in RSMeans for rebar and concrete, $900 per ton and $109 per cubic yard respectively, the following tables display the cost analysis:

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Frame	Span (ft)	Existing Reinforcement		Concrete (CY)	Existing Cost
		Top (lb)	Bottom (lb)		
11AB	15.45	143.93	0.00	5.24	$636.10
11BC	16.75	93.87	0.00	5.68	$661.64
11CD	11.28	68.84	0.00	3.83	$448.10
11DE	24.14	87.61	0.00	8.19	$932.10
11EF	24.30	81.35	0.00	8.24	$935.20
11FG	12.00	68.84	0.00	4.07	$474.73
11GH	21.08	93.87	0.00	7.15	$821.76
11HI	22.10	262.84	138.30	7.50	$997.75
		TOTAL per Floor and Frame			$5,907.38
		TOTAL per Frame			$88,610.74
		TOTAL			$354,443

Frame	Span (ft)	Redesign Reinforcement		Concrete (CY)	Redesign Cost
		Top (lb)	Bottom (lb)		
11AB	15.45	143.93	0.00	6.36	$758.52
11BC	16.75	374.00	569.50	6.90	$1,176.70
11CD	11.28	374.00	383.52	4.65	$847.39
11DE	24.14	408.00	820.76	9.94	$1,636.91
11EF	24.30	374.00	826.20	10.01	$1,631.24
11FG	12.00	340.00	408.00	4.94	$875.44
11GH	21.08	510.00	716.72	8.68	$1,498.58
11HI	22.10	262.84	138.30	9.10	$1,172.87
		TOTAL per Floor and Frame			$9,597.66
		TOTAL per Frame			$143,964.89
		TOTAL			$575,860

The difference between the existing design and the redesign is approximately $230,000 adjusted for the Washington D.C. cost index. Using the comparison given by the National Research council, the blast hardening produces a 0.099% increase in the lease premium rate.

Under this method, the parking garage floor system changed from a two-way flat plate post-tensioned concrete slab to a two-way concrete slab with beams. Since the cost of rebar for elevated slabs is constant for #4 through #7 rebar sizes the estimated cost difference will be based on the square footage cost. For a post-tensioned slab the cost per square foot is approximately $15.35 which is $3.95 less than the concrete slab with beams. The overall effect of this design change causes a 0.33% increase in the lease premium.

In summary, the indirect method proves to be the less expensive redesign for both areas of interest; therefore, it is recommended that the extra rebar detailing to increase ductility be utilized to increase the

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structure's resistance to progressive collapse mechanisms. The following table summarizes the changes in cost and lease premiums per each method:

Method	Transfer Girder Premium		Parking Garage Premium		Total Premium	
	Cost	Lease	Cost	Lease	Cost	Lease
Indirect	$15,686	0.007%	-$273,574	-0.117%	-$257,888	-0.11%
Direct	$230,052	0.099%	$763,489	0.33%	$993,541	0.43%

SITE ARCHITECTURE REDESIGN DISCUSSION

With the structure redesigned to perform better under a terrorist attack, the best design approach to deter any attack is within the site architecture. Here, the design completely mitigates the attack, so that the structure will never be asked to bear the consequences. According to Eve Hinman, controlling site access to vehicular and pedestrian traffic and securing the perimeter are the most important facets to good site design. These components can be obtained through intrusion detection alarms, fences or walls, bollards, chicanes, and guard houses (Bounds 1-17).

The new site is much larger than the original site allowing for 100 plus feet of stand-off distance, visitor parking, and a plaza to create an inviting space for the tenants to enjoy without fear. With four major avenues and streets surrounding the site, access to resident parking and loading docks can easily be provided and controlled. Figure 12 shows the existing site design and Figure 13 reveals the redesign for site security.
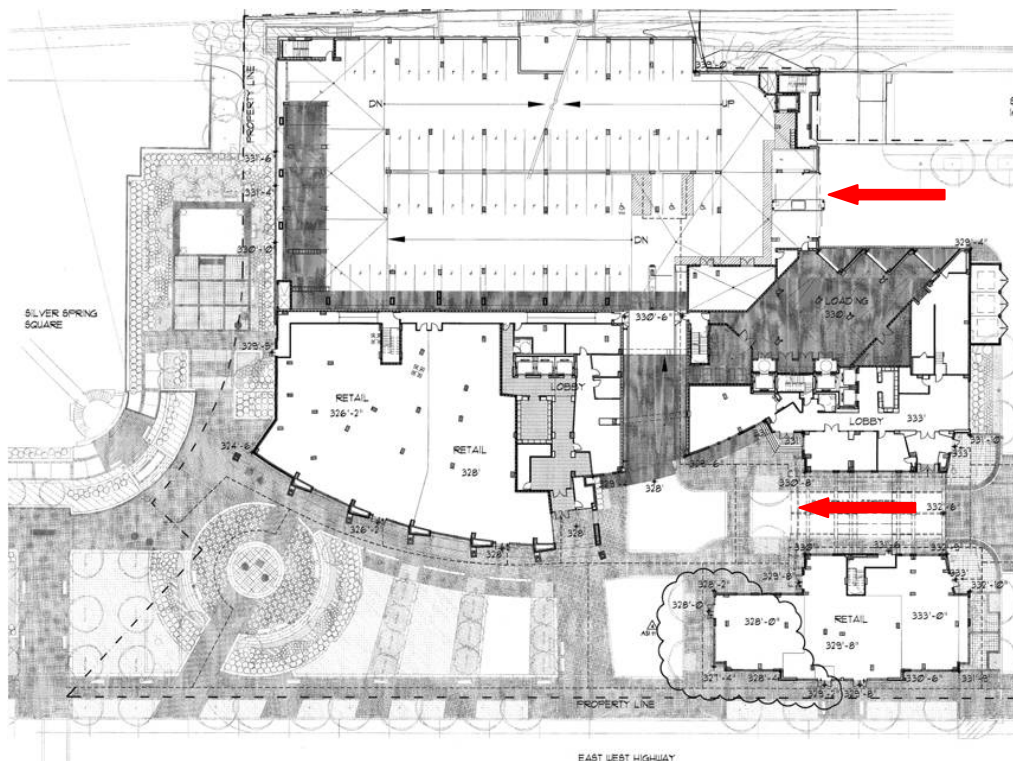
Figure 12: Existing site design layout for Silver Spring Gateway. Note dual access points for parking garage (*red arrow*).

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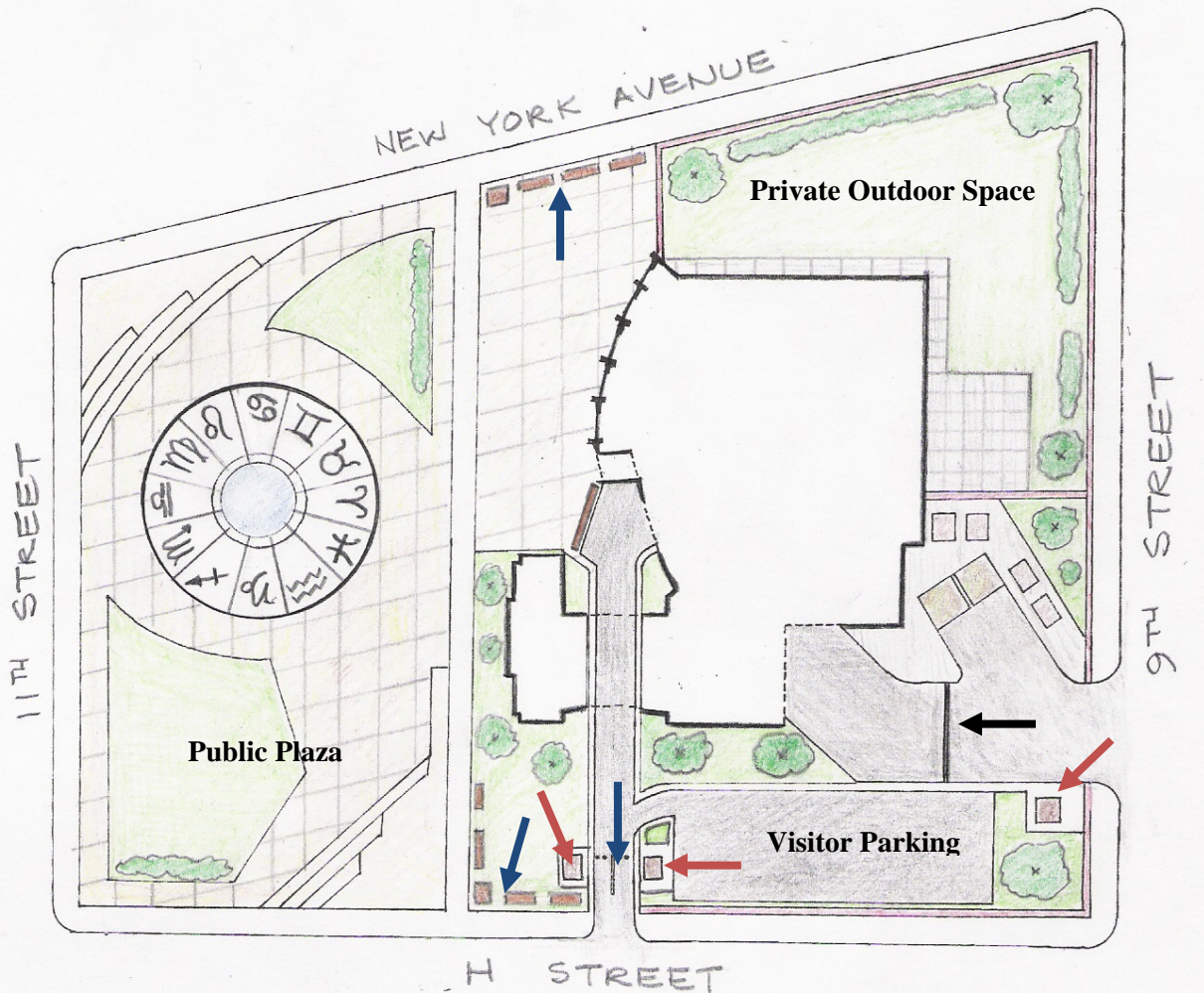
Figure 13: Hand rendered site layout redesign. Guard Houses (*red arrow*), bollards and planters (*blue arrow*), motorized gate (*black arrow*).

Vehicular Access

The new site design controls vehicular access through several means. First, the number of entrances and exits for resident and visitor parking and service areas were reduced to one each. This allows for concentrated access which functions well for the guard houses located at both of these entrances. Both access points also utilize a motorized gate for the locking dock and hydraulic bollards (Figure 14) for the resident parking garage to restrict unpermitted entrance. These manually operated barriers to site intrusion maintain security immediately outside and inside the building. The visitor parking keeps only vehicles belonging to the tenants within the parking garage to mitigate potential security breaches. The second step to protect the building from rogue vehicles is the six foot high perimeter wall, three foot high knee wall, and crash rated planters (Figure 15) along portions of New York Avenue and H Street and entirely along 9th Street. Also, the raised plaza in front of the building creates a buffer between vehicular traffic from 11th Street and the building. Between the walls and planters and overall size of the site allow for greater standoff ranges and protection from vehicular attacks.

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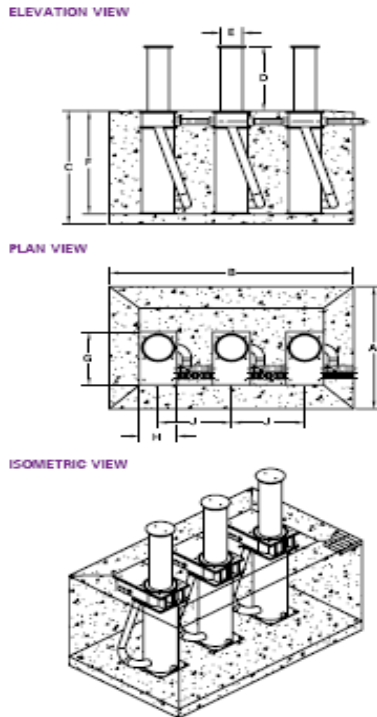
Figure 14: Hydraulic Bollard Detail (Boon Edam Tomsed)
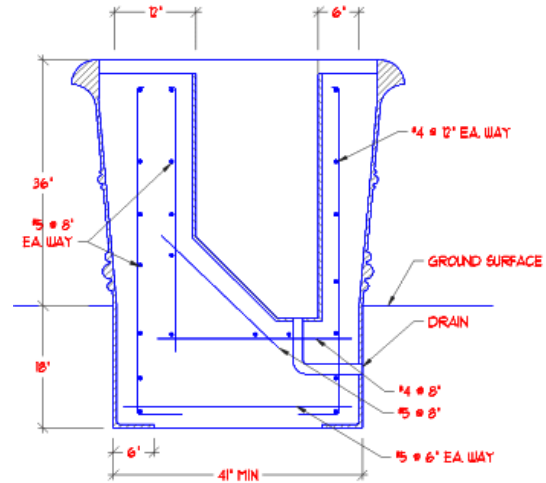
Figure 15: Planter Barrier Detail (Magnum FP by Stonewear)

Pedestrian Access and Public and Private Space

While vehicular access is rather simple to control, controlling migrant, sadistic pedestrians while maintaining a secure public space is a challenge. Pedestrians much like vehicles cannot be granted access into the building were one could inflict critical damage to the structure and its inhabitants. However, due to the retail areas on the bottom floor pedestrians must be allowed to shop without hindrance. Therefore, a site landscaping planter will be utilized to keep pedestrians from the garage entrance beyond the internal street within the building which keeps pedestrians from accessing the residential portions of the building. Also, the main entrance for residents will be card access to increase the security of the building and its tenants.

The public space encompasses the building plaza linking the retail shops and the street plaza for the general public. The existing building plaza design will be maintained just resized for this site; however, due to the extra portion past the current 10th Street pedestrian way, this space will act as a pleasant and secure buffer for the building. The plaza will be raised two feet acting as a barrier and a great place to sit and relax. The plaza will also contain two green areas, benches, and a fountain surrounded by a zodiac relief. Since this building is located close to the famous mall triangle originally planned with astrological and Masonic significance and located within a historic section of Washington D.C. which displays 23 zodiacs on or within public and government buildings (Ovason), the inclusion of a zodiac reference is certainly not unusual; rather, expected by the original planners of the urban space surrounding the nation's capital.

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The space surrounding the building on the New York Avenue and 9th Street side will be walled to create a private outdoor patio and garden space for the tenants. This amenity much like the public areas acts as a green buffer adding security and beauty to the site.

Conclusion

Overall, the site size allowed for generous stand off zones and public and private space buffers that enable security demands to be met without sacrificing the atmosphere of the surrounding public plazas. Added gates, guards, bollards, and crash-rated planters also help increase the overall security of the grounds. The larger site and additional security devices will significantly affect the cost of the building and inherently the lease premium for the tenants; however, it is the goal for the tenants to feel secure in their living space which may outweigh this cost increase.

FACADE REDESIGN DISCUSSION

Since the owner is looking at a new location for development, the architectural language of the surrounding area must be researched to design the façade to blend into its new environment. Along with this, the façade must also be able to withstand a blast loading and maintain a continuous barrier to the weather. All of these criteria will be assessed to properly redesign the façade of this high rise development.

Local Architectural Materials

Luckily, the historic Mount Vernon Square and the Penn Quarter Neighborhoods are currently updating their surroundings with more modern residential high rises. For instance, Ten Ten Mass located at 1010 NW Massachusetts Avenue, City Vista located on the corner of 5th and K Streets, and the Atlantic Building located on F Street are just a few blocks away and exhibit similar architectural languages and façade materials as the Silver Spring Gateway currently utilizes (Figures 16, 17, and 18 respectively).

Figure 16: Elevation Rendering   Figure 17: Elevation Rendering   Figure 18: Photo by J. Otavio Thompson

Since most of the new construction within these districts uses brick along with metal panels, glazing, and storefront assemblies as their main facade materials, the existing façade materials will not be altered as they currently match other surrounding developments.

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Blast Resistance

According to *FEMA 427: Primer for the Design of Commercial Buildings to Mitigate Terrorist Attacks*, exterior blasts against the glazing are more hazardous if the glass has a higher capacity than the frame that supports it, as a larger projectile will cause more damage than glass fragments designed to minimize injury (6-28). Therefore, the glazing system will be designed to fail such that it does not inflict damage on the building or its inhabitants. Within the existing specifications, the window and storefront glazing will mostly be tempered which happens to be the optimal choice for blast loadings as it breaks into cube like particles. For tempered glass, the typical breakage pressure for a 60 second loading is approximately 24,000 psi. While this does not give great insight into its performance for a blast loading which is several times faster, the stand off distance provided by the new site layout will produce a 20 psi incident pressure at 100 feet as shown in Figure 19.
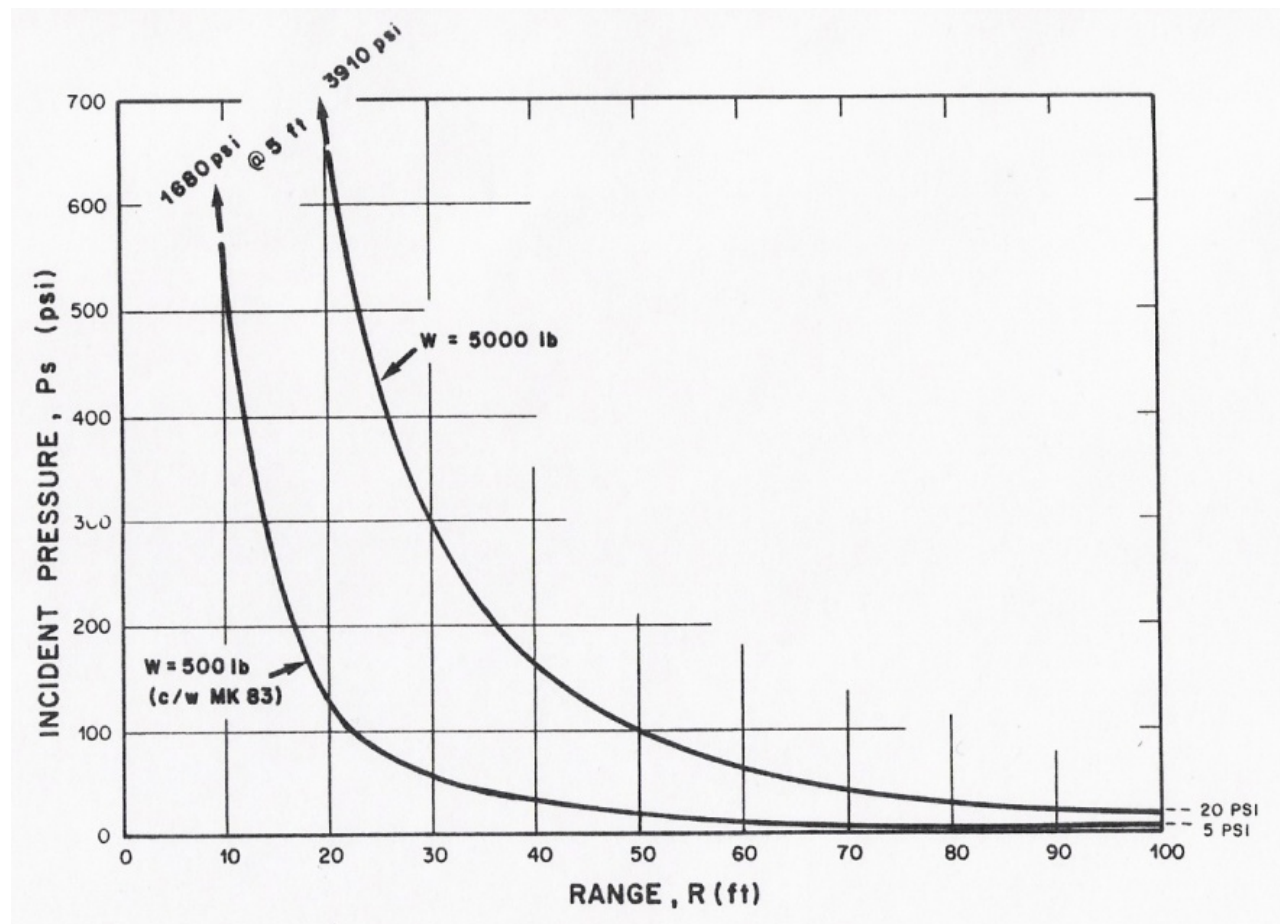
Figure 19: Plots showing pressure decay with distance

Since this pressure is fairly low compared to the ultimate strength of the glass under a 60 second loading and the detonation pressure, it is assumed that the glass will break under conditions one or two in accordance to the criteria set by FEMA on page 6-30 (Figure 20); therefore, the existing glazing specified will be adequate to resist street-side blasts.

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Performance Condition	Protection Level	Hazard Level	Description of Window Glazing
1	Safe	None	Glazing does not break. No visible damage to glazing or frame.
2	Very High	None	Glazing cracks but is retained by the frame. Dusting or very small fragments near sill or on floor acceptable.
3a	High	Very Low	Glass cracks. Fragments enter space and land on floor no further than 1 meter (3.3 feet) from window.
3b	High	Low	Glazing cracks. Fragments enter space and land on floor no further than 3 meters (10 feet) from the window.
4	Medium	Medium	Glazing cracks. Fragments enter space and land on floor and impact a vertical witness panel at a distance of no more than 3 m (10 feet) from the window at a height no greater than 2 feet above the floor.
5	Low	High	Glazing cracks and window system fails catastrophically. Fragments enter space impacting a vertical witness panel at a distance of no more than 3 meters (10 feet) from the window at a height greater than 0.6 meters (2 feet) above the floor.

Figure 20: Performance Conditions for Windows

Thermal and Moisture Performance

Since the existing façade materials will be used, the last step is to ensure that the wall system provides adequate thermal and moisture protection to keep the tenants comfortable and healthy. The façade utilizes two types of systems: an uninsulated metal panel and a brick cavity wall (Figures 21 and 22).

Figure 21: Existing Metal Panel Wall Section

Figure 22: Existing Brick Cavity Wall Section

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Both systems have the same interior components consisting of 1/2" thick gypsum wall board, metal studs with batt insulation, Georgia Pacific Densglass Gold sheathing, and a Tyvek Commercial Wrap air barrier. Each system will be assessed based on their thermal resistance and vapor permeability for proper performance using the following assumptions and averages:

Average Winter Temperature	36° F
Average Summer Temperature	77° F
Assumed Indoor Temperature	70° F
Assumed Outdoor Relative Humidity	80%
Assumed Indoor Relative Humidity	50%

The uninsulated metal panel and brick cavity wall both exhibit a potential problem as the thermal gradient within the wall passes through the dew point of 51° F within the framing and insulation portion of the wall section (Figures 23 and 24). If the vapor pressure is high enough, condensation could occur on the cold side of the insulation and framing causing damage and possible unhygienic biological growth; therefore the vapor permeability of the wall section will play a role in the performance of these wall systems.
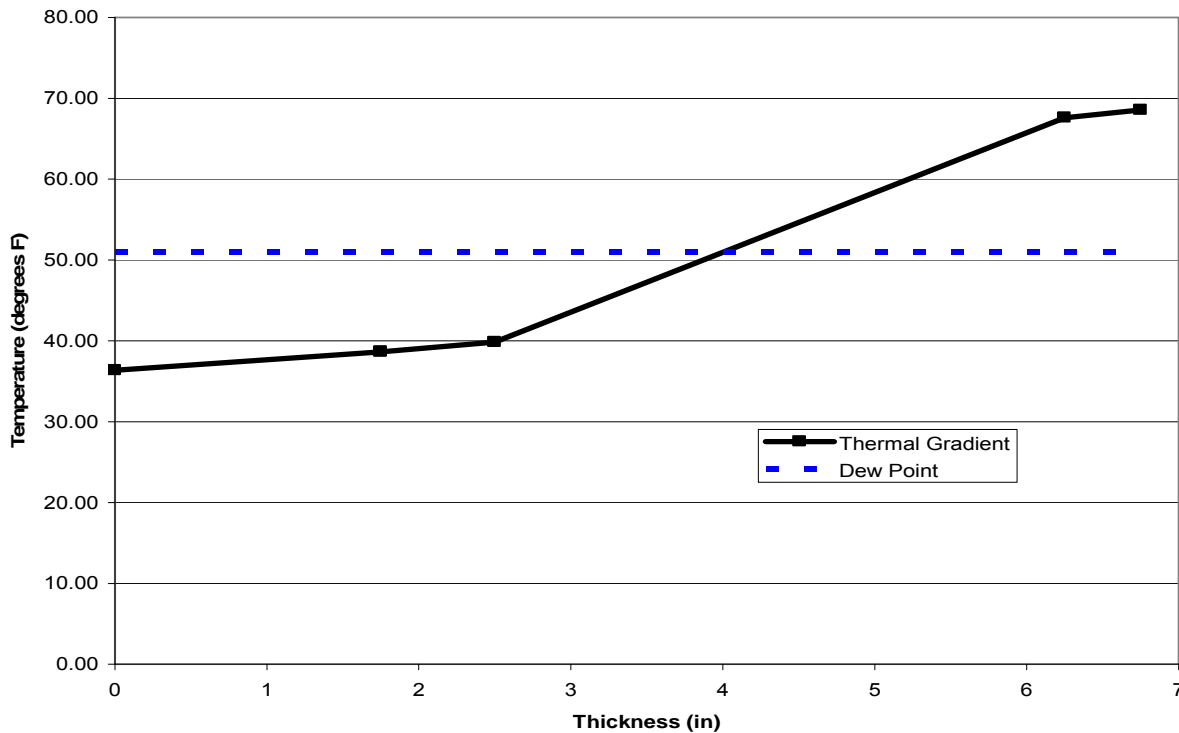
Figure 23: Thermal gradient for metal panel wall assembly (Each square dot represents a material interface).

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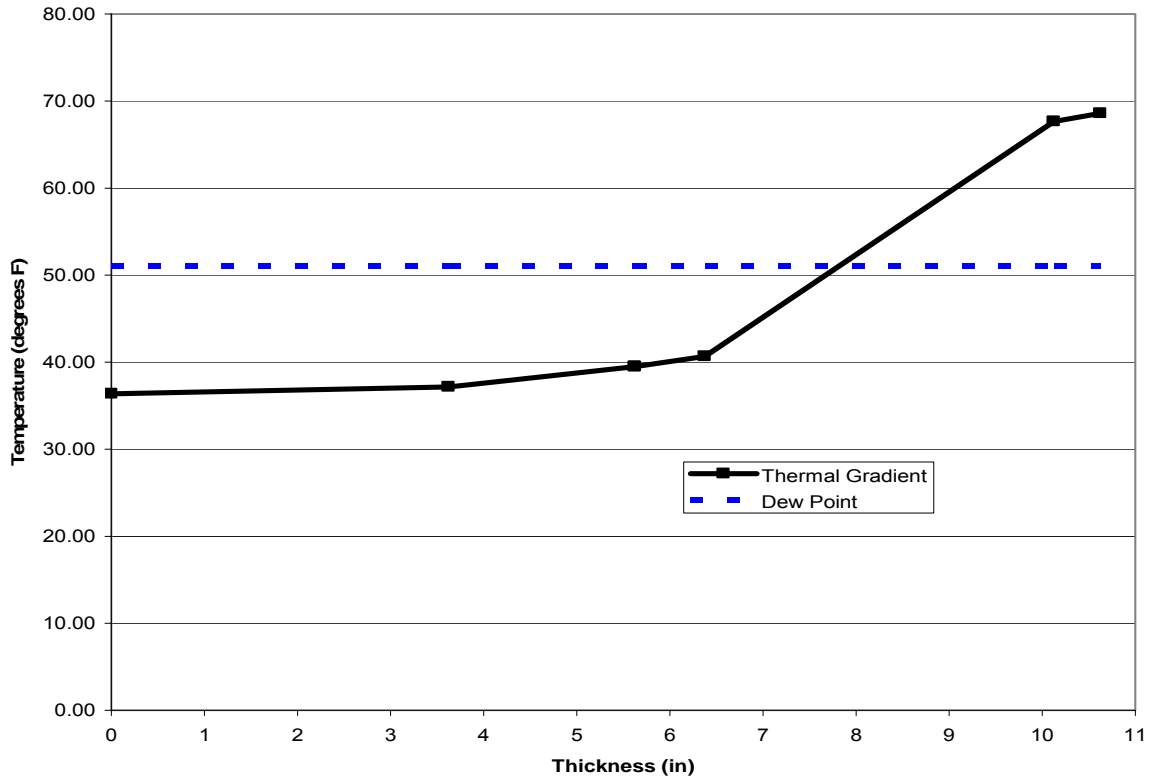
Figure 24: Thermal gradient for brick veneer cavity wall assembly (Each square dot represents a material interface).

The vapor pressure within both wall systems exceeds the saturation pressure which causes condensation at the sheathing-insulation interface which is unsatisfactory; therefore, the wall will need to be redesigned. The first trial added rigid insulation to the air cavity in the brick veneer system and an insulated metal panel to change the thermal gradient and saturation pressure; however, it did not succeed in curbing the condensation. The second trial removed the added insulation and added interior plywood sheathing and a Grace vapor barrier membrane inboard of the framing. This significantly dropped the vapor pressure within the wall systems and removed the possibility of condensation; therefore, the new facade will be constructed based on the following wall sections (See Appendix F for Calculations):

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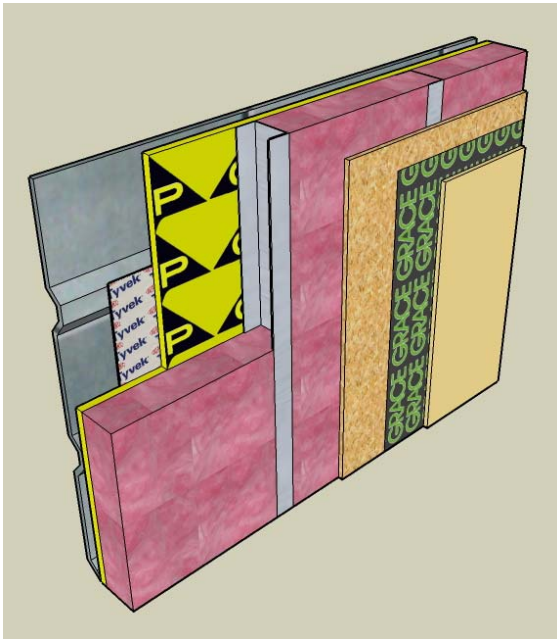
Figure 25: New Metal Panel Wall Section
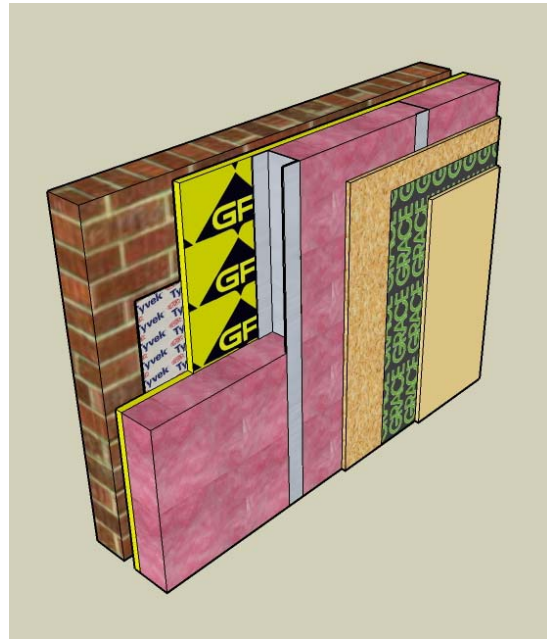
Figure 26: New Brick Cavity Wall Section

Conclusion

While the surrounding architecture and site layout did not have an impact on the design of the building façade, thermal and moisture properties of the wall required the wall sections to be modified slightly. The overall thickness of the metal panel wall system increased by a 1/2 inch and the brick veneer wall system remained the same thickness because the air cavity absorbed the extra 1/2 inch due to the added sheathing and vapor barrier. The total cost of the wall will increase with the inclusion of the sheathing and vapor barrier; however, with respect to the grand total it should not affect the overall project cost significantly.

CONCLUSION AND RECOMMENDATIONS

This thesis has shown that most of the existing conditions of the Silver Spring Gateway are adequate for reuse for another similar, more secure high rise. Minor changes in the structure, such as rebar detailing, can significantly mitigate a progressive collapse without increasing the cost drastically. The current façade design already blends well with the surrounding architectural language and materials and the specified glazing is the most optimal choice; however, a minor change to eliminate susceptibility to condensation within the wall assembly will increase the functionality of the building. The most significant reason for feasibility is the site selection. The size of the site allows for great passive countermeasures to deter attacks due to stand-off ranges and access control while also maintaining a habitable space for consumers and tenants alike. It is recommended that JBG pursue the development of this site for its desired clientele; however, if a different site is chosen due to land costs, it is suggested that thorough attention be given to the layout of the building to maintain the performance suggested in this thesis.

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Structural: S1.01-S4.05 dated 31 August 2006 by TCE and WDG

Architectural: A1.01-A12.41 dated 17 November 2006 by WDG

Rebar Shop Drawings: F1.01-F15.02, B1.01-B1.03, C.01-C.39, L1.01-L1.08, PH.01-PH.07,W.01-W.05 dated 17 March 2006 by Harris Rebar

Tendon Shop Drawings: PT-0.00-PT-1.61 dated 12 August 2006 by Suncoast Post-Tension L.P.

Specifications: Sections 00001-14560 dated 6 April 2007 by WDG

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APPENDIX A – FIGURES

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Figure 1: Architectural Rendering of Silver Spring Gateway from the corner of East-West Highway and Blair Mill Road.

Figure 2: Original site (red hatch) and surrounding streets, railway, and buildings.

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Figure 3: Current site (red hatch) abandons a portion of Blair Mill Road.
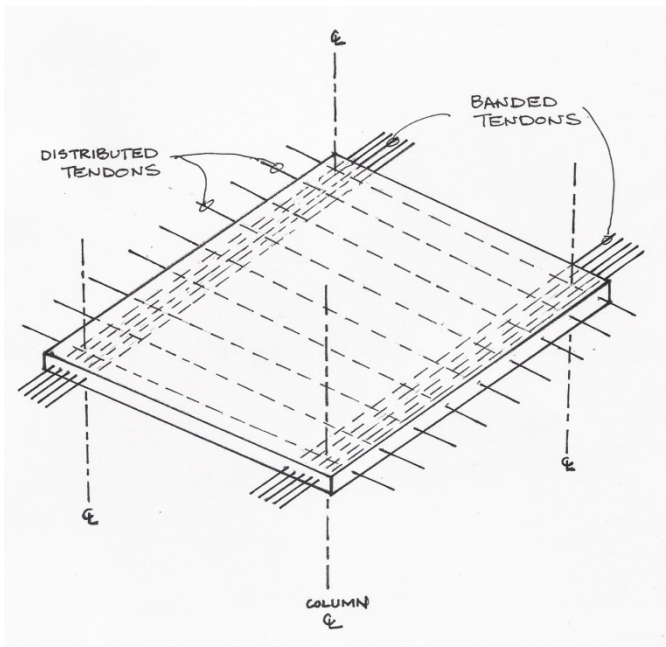
Figure 4: Typical post-tensioning tendon layout.

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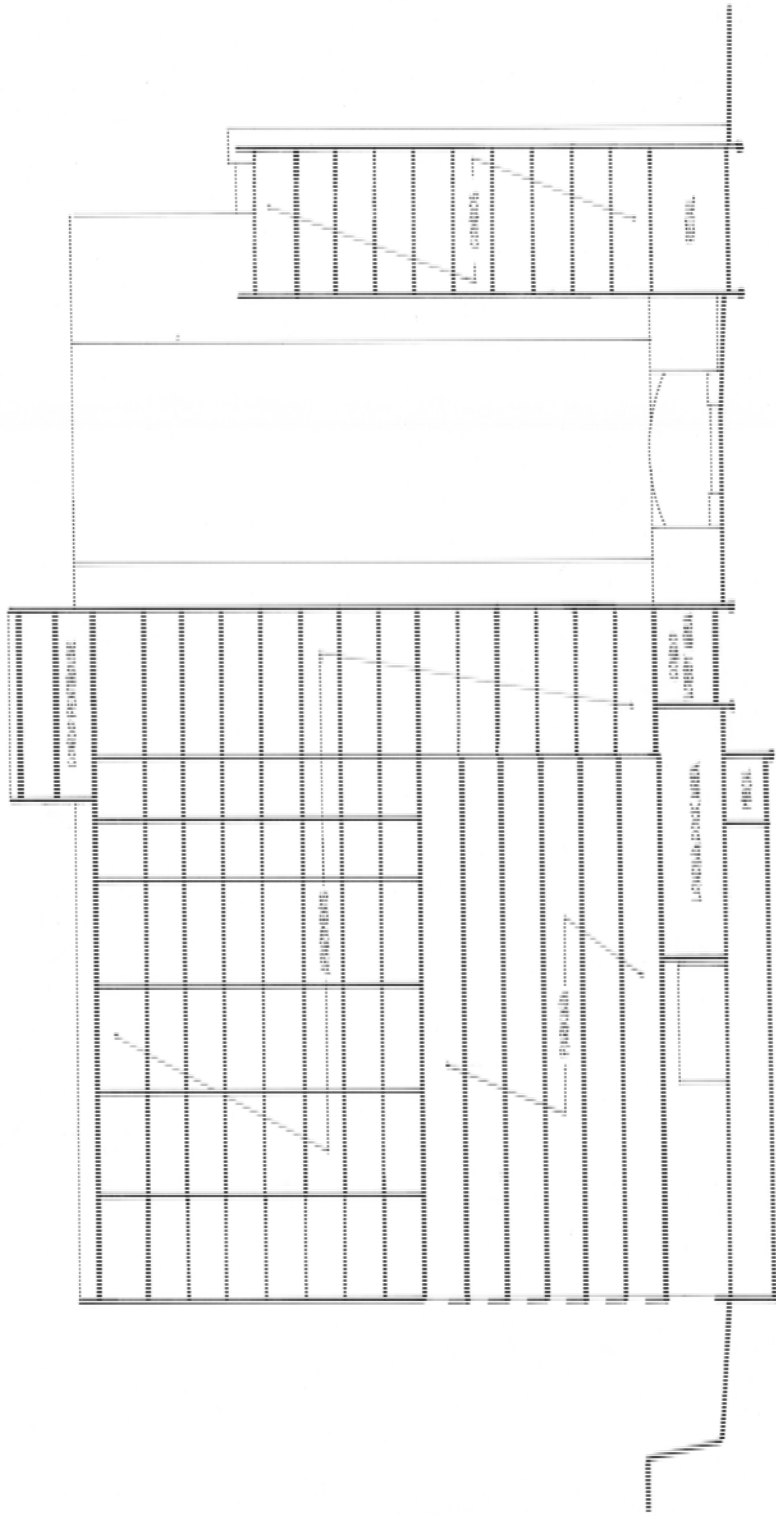
Figure 5: Building section showing occupancies per floor.

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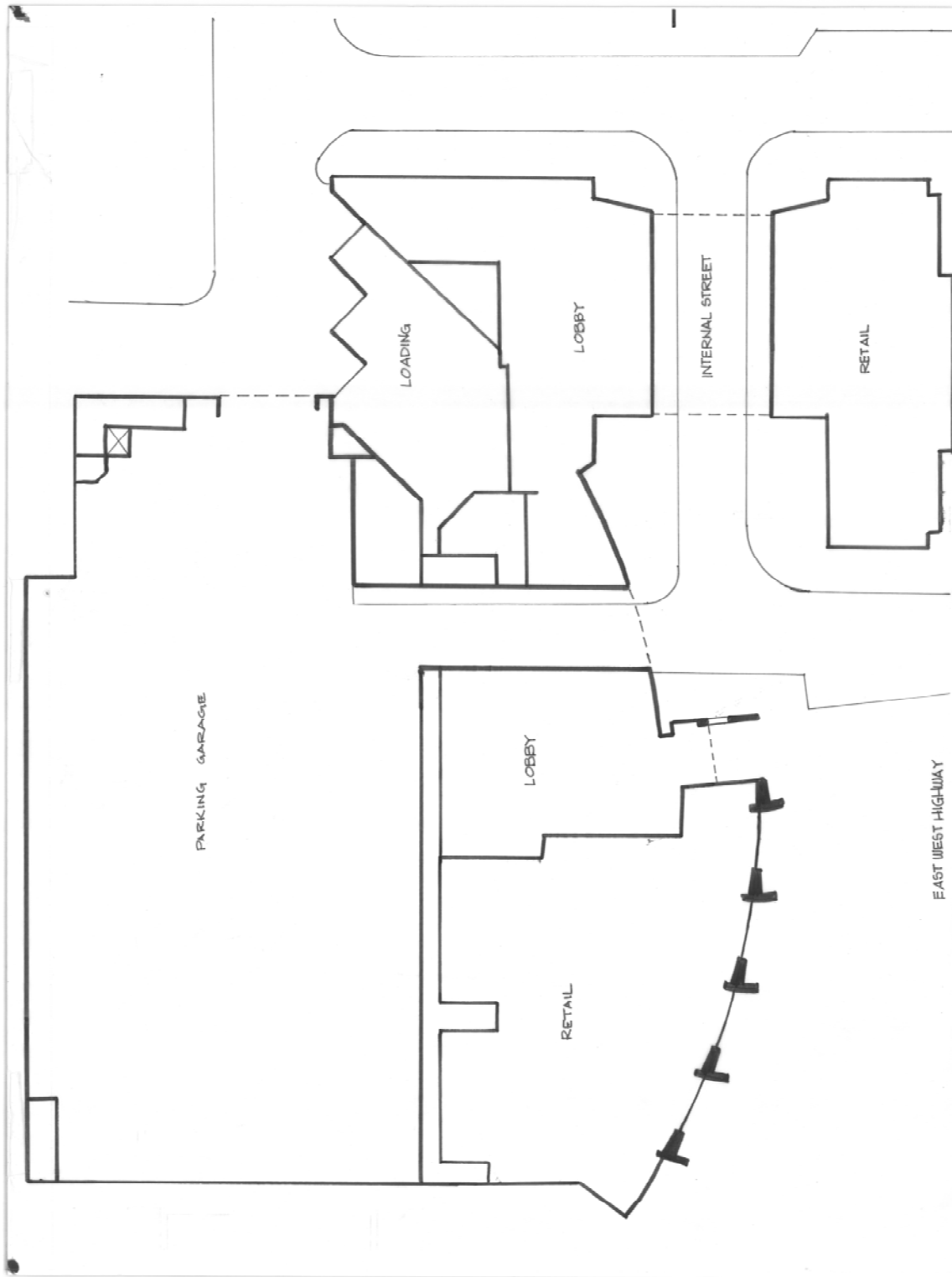
Figure 6: First Floor and Overall Site Plan showing overall shape for lower floors.

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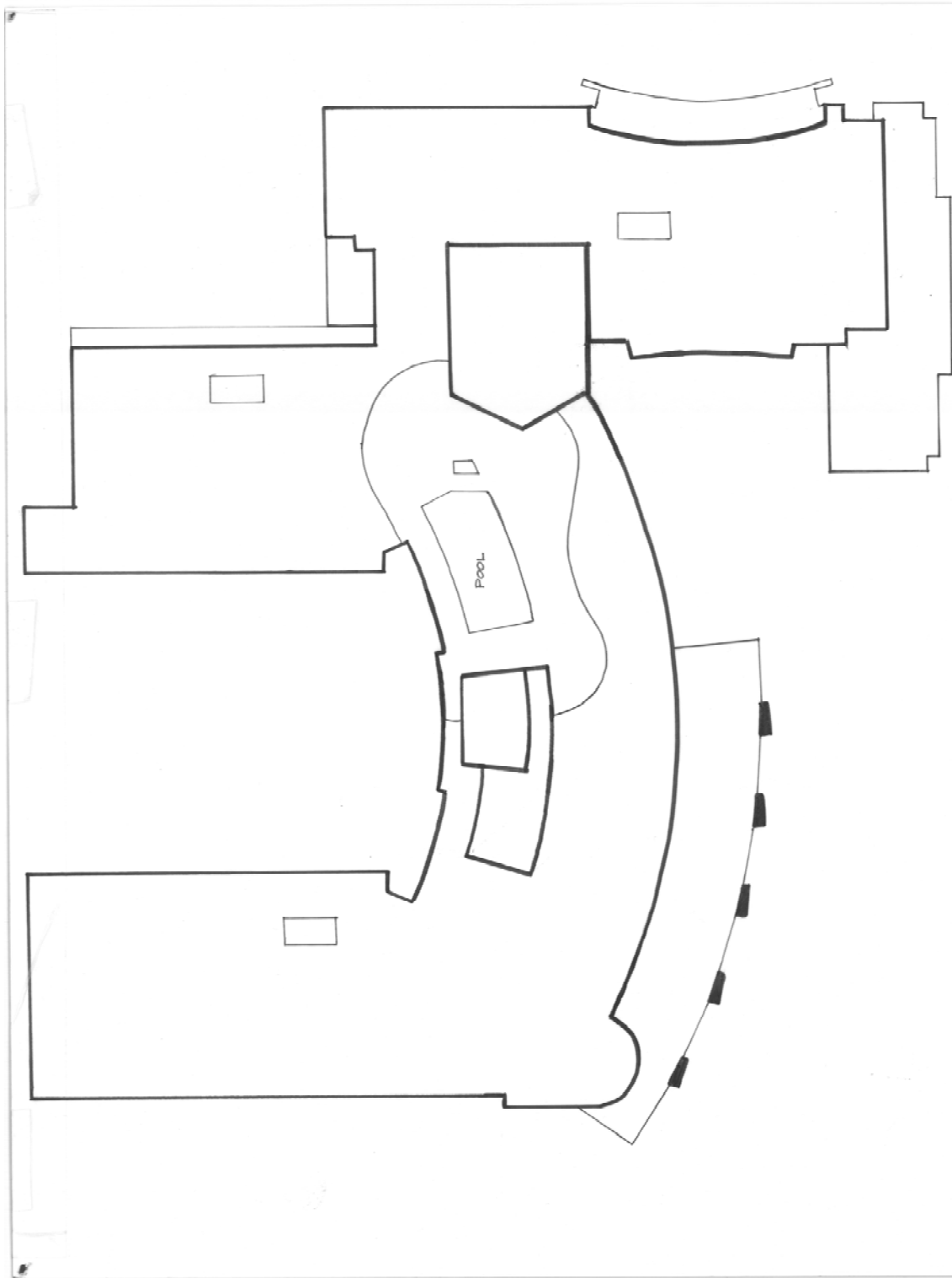
Figure 7: Penthouse Roof Plan showing overall shape of the upper floors and location of penthouse amenities.

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Figure 8: Location of new development site in Washington D.C.

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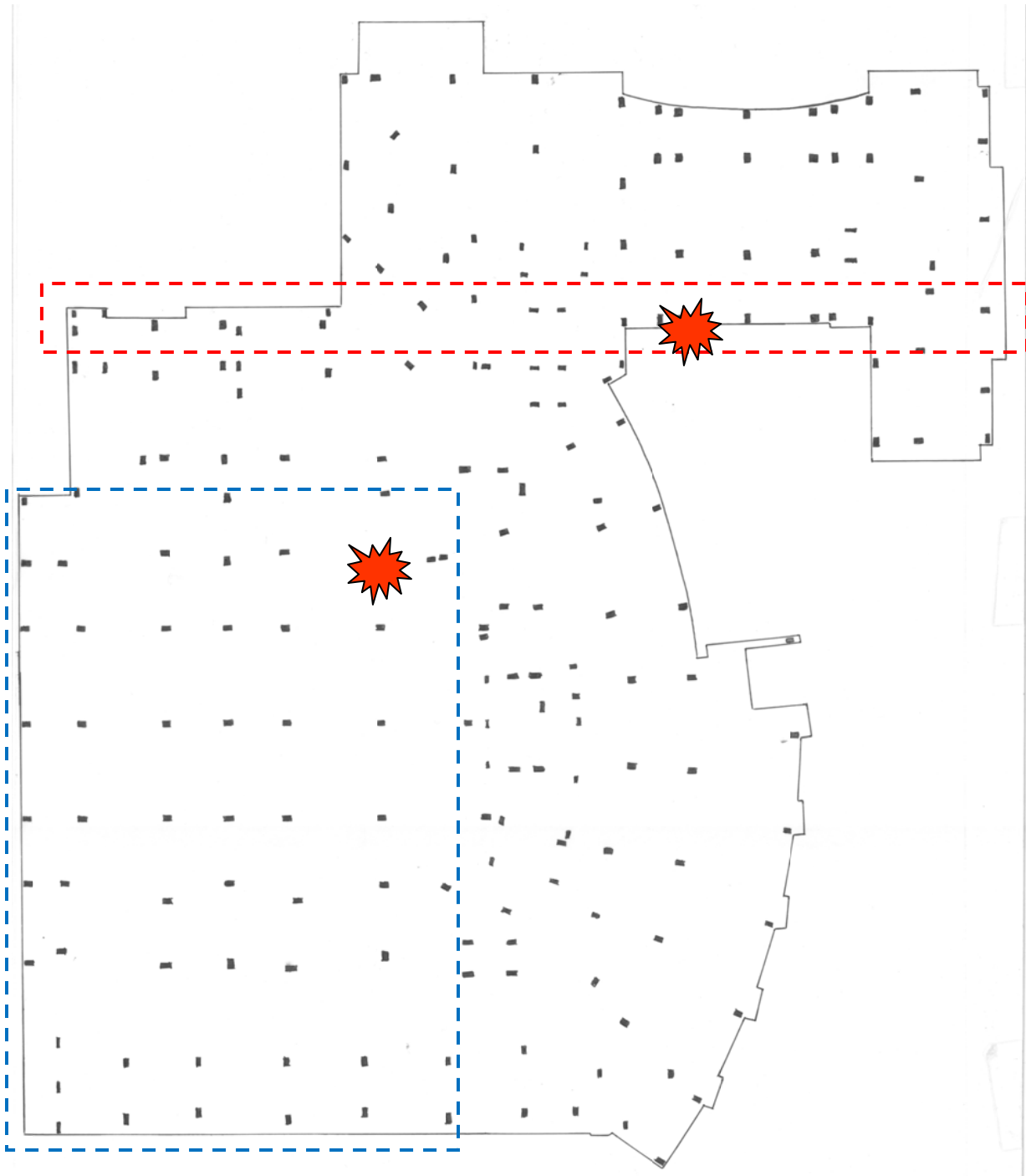
Figure 9: Column Layout with Transfer Girder Frame (red) and Parking Garage Slab (blue) location. (Blast locations are also annotated).

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APPENDIX B – PHOTOS

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Photo 3: Overall view of Southeast elevation of Silver Spring Gateway.

Photo 4: Partial view of courtyard from the top level of the parking garage.

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Photo 5: Partial view of the Southwest elevation.

Photo 6: Partial view of the inside corner between the Southwest elevation and a small portion of the West elevation.

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Photo 7: Partial view of lower floor construction on East-West Highway elevation (Southwest).

Photo 8: Interior View of the parking garage.

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Photo 9: Interior view of pedestrian bridge steel structure.

Photo 10: Typical full penetration welded connection of the bridge truss structure.

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Photo 11: Interior view of a typical residential corridor.

Photo 12: View of post-tensioning cables prior to jacking force application.

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APPENDIX C – PROJECT TEAM DIRECTORY

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Role	Firm	Website
Owner	The JBG Companies 4445 Willard Ave., Suite 400 Chevy Chase, MD 20815	www.jbg.com
Architect	WDG Architecture 1025 Connecticut Ave., Suite 300 Washington, DC 20036	www.wdgarch.com
Civil Engineer	Loiederman Soltesz Associates, Inc. 1390 Piccard Drive, Suite 100 Rockville, MD 20850	www.LSAssociates.net
Structural Engineer	Tajder-Cohen-Edelson Associates, Inc. 1109 Spring Street Silver Spring, MD 20910	www.tadjerco.com
Landscape Architect	Hord Coplan Macht 750 E. Pratt Street, Suite 1100 Baltimore, MD 21202	www.hcm2.com
Interior Designer	Carlyn and Company 746 Walker Road, Suite 22 Great Falls, VA 22066	www.carlynco.com
Environmental Consultant	Environmental Resolutions, Inc. 14609 Jaystone Drive, Suite 100 Silver Spring, MD 20905	
Geotechnical Consultant	GeoConcepts Engineering, Inc. 19955 Highland Vista Drive, Suite 170 Ashburn, VA 20147	www.geoconcepts-eng.com
Acoustics Engineer	Cerami & Associates, Inc. 1250 Connecticut Ave., N.W. Washington, DC 20036	www.ceramiassociates.com
Mechanical Engineer	Atlas Air Conditioning Company 10693 Wakeman Ct. Manassas, VA 20110	www.atlascsusa.com
Electrical Engineer	Power Design, Inc. 11207 S. Danka Blvd., Suite A St. Petersburg, FL 33716	www.powerdesigninc.us
Construction Manager	Turner Construction Company 10400 Little Patuxent Pkwy., Suite 200 Columbia, MD 21044	www.tcco.com
Seismic Monitoring	Seismic Surveys P.O. Box 1185 Frederick, MD 21702	www.seismicsurveys.net

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APPENDIX D – BUILDING LOADS

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In order to analyze the Silver Spring Gateway, the gravity loads acting on the building must be determined. The construction documents, including drawings and specifications, AISC 13th ed., and ASCE 7 provide insight to code compliant loadings and material specifications and weights. The following table lists the appropriate gravity loads classified by type and system:

Floor System Loads			
Load Type	Material / Occupancy	Load	Reference
Dead Load	Normal Weight Concrete	150 pcf	ACI 318
	Steel	Per shape	AISC 13th ed.
	Steel Deck	2 psf	USD
	Partitions	15 psf	ASCE 7
	Brick Masonry	40 psf	AISC 13th ed.
	Miscellaneous	10 psf	
Live Load	Lobby and Common Spaces	100 psf	ASCE 7
	Corridors	100 psf	ASCE 7
	Apartments and Condominiums	40 psf	ASCE 7
	Corridors servicing Residential Spaces	40 psf	ASCE 7
	Balconies	60 psf	ASCE 7
	Parking Garage	40 psf	ASCE 7
	Retail Spaces	100 psf	ASCE 7

Roof and Terrace System Loads			
Load Type	Material / Occupancy	Load	Reference
Dead Load	Normal Weight Concrete	150 pcf	ACI 318
	Water (Swimming Pool)	62.4 pcf	AISC 13th ed.
	Green Roof	70 pcf	AISC 13th ed.
	Ballast, insulation, and waterproofing membrane	8 psf	AISC 13th ed.
	Brick Masonry	40 psf	AISC 13th ed.
	Miscellaneous	10 psf	
Live Load	Assembly and Pool Space	100 psf	ASCE 7
	Roof	20 psf	ASCE 7
Snow Load	Ground Snow Load	30 psf	Montgomery County
	Terrain Category	B	ASCE 7
	C_e Exposure Factor	1.0	ASCE 7
	C_t Thermal Factor	1.0	ASCE 7
	Importance Factor	1.0	ASCE 7
	Flat Roof Load	21 psf	ASCE 7

The miscellaneous gravity load will include building components such as ductwork, lighting, telecommunications, drop ceilings, etc. Snow drift loads will accumulate around the penthouses increasing the dead load on the roof; however, the magnitude of this loading was not determined for this report.

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APPENDIX E – STRUCTURAL CALCULATIONS

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PROGRESSIVE COLLAPSE   INDIRECT METHOD   TIE FORCES   1

THE FOLLOWING HAND CALCULATION WILL SHOWN THE
DETERMINATION OF THE REQUIRED TIE FORCES AT
THE TRANSFER GIRDER TRUSS LOCATION:

PERIPHERAL TIES

REQUIRED TIE FORCE = $1.0 F_t$

$F_t = \min \begin{vmatrix} 4.5 + 0.9 n_o \, ; \, n_o = \text{\# OF STORIES} = 18^k \\ 13.5^k \checkmark \end{vmatrix}$

DESIGN STRENGTH = $\phi \Omega f_y A_s \, ; \, \phi = 0.75 , \, \Omega = 1.25$

$A_{s, REQ'D} = \dfrac{13.5}{(0.75)(1.25)(60)} = 0.24 \, IN^2$

$A_{s, PROV'D} = 3.95 \, IN^2$ ADDITIONAL $A_s = 0.00 \, IN^2$

$\therefore$ PROVIDED REINFORCEMENT IS ADEQUATE

INTERNAL TIES

REQUIRED TIE FORCE = $\begin{vmatrix} \dfrac{(D+L)}{156.6} \cdot \dfrac{L_r}{16.4} \cdot \dfrac{F_t}{3.3} \\[2mm] \text{MAX} \quad \dfrac{F_t}{3.3} \end{vmatrix}$

WHERE: $D = SW + SDL = 87.5 + 20 = 107.5 \, PSF$
$\quad L = 40 \, PSF$
$\quad L_r = $ GREATEST SPAN LENGTH IN DIRECTION
$\qquad$ OF TIE,

$\quad F_t = 13.5^k$ (CALCULATED ABOVE)

$R_{u_{N-S}} = \begin{vmatrix} \left(\dfrac{147.5}{156.6}\right)\left(\dfrac{30}{16.4}\right)\left(\dfrac{13.5}{3.3}\right) = 7049 \, LB/FT \, WIDTH \checkmark \\[3mm] \text{MAX} \quad \left(\dfrac{13.5}{3.3}\right) = 4091 \, LB/FT \, WIDTH \end{vmatrix}$

$R_n = \phi \Omega f_y A_s = 56250 \, A_s$

$A_{s, REQ'D} = \dfrac{7049}{56250} = 0.1253 \, IN^2/FT \cdot WIDTH \, (N-S)$

$A_{s, PROV'D} = 0.61 \, IN^2/FT$ ADDITIONAL $A_s = 0.00 \, IN^2/FT$

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PROGRESSIVE COLLAPSE   INDIRECT METHOD   TIE FORCES   2

$$R_{u_{E-W}} = \left. \begin{array}{l} \left(\dfrac{147.5}{156.6}\right)\left(\dfrac{28}{16.4}\right)\left(\dfrac{13.5}{3.3}\right) = 6579 \text{ LB/FT-WIDTH} \checkmark \\[4mm] \left(\dfrac{13.5}{3.3}\right) = 4091 \text{ LB/FT-WIDTH} \end{array} \right|_{MAX}$$

$$A_{s,REQ'D} = \frac{6579}{56250} = 0.117 \text{ IN}^2/\text{FT} \cdot \text{WIDTH} \quad (E-W)$$

$A_{s,PROV'D} = 0.402 \text{ IN}^2/\text{FT}$ ADDITIONAL $A_s = 0.00 \text{ IN}^2$

∴ (N-S): PROVIDED REINFORCEMENT IS ADEQUATE
  (E-W): PROVIDED REINFORCEMENT IS ADEQUATE

HORIZONTAL TIES TO COLUMNS

$$R_u = \left. \begin{array}{l} 0.03 A_T (4')(D+L) = 0.03(9)(70)(4)(147.5) = 11.15^k \\[3mm] \left. \begin{array}{l} 2.0 F_t = 2(13.5) = 27^k \\[3mm] \dfrac{F_t L_s}{8.2} = \dfrac{13.5(9.0833)}{8.2} = 14.95^k \checkmark\checkmark \end{array} \right|_{MIN} \end{array} \right|_{MAX}$$

$$A_{s,REQ'D} = \frac{14.95}{56.25} = 0.266 \text{ IN}^2$$

$A_{s,PROV'D} = 1.24 \text{ IN}^2$ ADDITIONAL $A_s = 0.00 \text{ IN}^2$

∴ PROVIDED REINFORCEMENT IS ADEQUATE

VERTICAL COLUMN TIES

$$R_u = A_T(D+L) = (9+35)(9+14)(147.5) = 149.3^k$$

$$A_{s,REQ} = \frac{149270}{56250} = 2.654 \text{ IN}^2$$

$A_{s,PROV'D} = (4)\#10 = 5.08 \text{ IN}^2$

ADDITIONAL $A_s = 0.00 \text{ IN}^2$

∴ DESIGNED COLUMN REINFORCEMENT WILL PROVIDE
  ENOUGH TIE STRENGTH

THIS ANALYSIS SHOWS THAT THE PROVIDED AMOUNT OF
REBAR SATISFIES THE STRENGTH REQUIREMENT;
HOWEVER, CURRENTLY ARE NOT DETAILED PROPERLY
TO DEVELOP THE STRENGTH.

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PROGRESSIVE COLLAPSE | INDIRECT METHOD | TIE FORCES · 3

THE FOLLOWING HAND CALCULATION WILL SHOWN THE
DETERMINATION OF THE REQUIRED TIE FORCES
WITHIN THE PARKING GARAGE:

PERIPHERAL TIES

REQUIRED TIE FORCE = 1.0 F_t

$$F_t = \left. \begin{array}{|l} 4.5 + 0.9\, n_0 = 11.7^k \quad \checkmark \\ 13.5^k \end{array} \right. _{MIN}$$

DESIGN STRENGTH = $\phi \Omega f_y A_s$; $\phi = 0.75$, $\Omega = 1.25$

$$A_{s,REQ} = \frac{11.7}{56.25} = 0.208 \text{ IN}^2$$

$$A_{s,PROV} = 1.24 \text{ IN}^2$$

∴ PROVIDED REINFORCEMENT IS ADEQUATE

INTERNAL TIES

REQUIRED TIE FORCE, $R_{u_{E-W}} = \left| \begin{array}{l} \left(\frac{255}{156.6}\right)\left(\frac{30}{16.4}\right)\left(\frac{11.7}{3.3}\right) = 10561 \dfrac{LB}{FT. WIDTH} \quad \checkmark \\[2mm] \left(\frac{11.7}{3.3}\right) = 3545 \text{ LB/FT-WIDTH} \end{array} \right._{MAX}$

$R_n = \phi \Omega f_y A_s = 56250 A_s$

$$A_{s,REQ'D} = \frac{10561}{56250} = 0.1877 \text{ IN}^2 / FT-WIDTH$$

$A_{s,PROV} =$

$R_{u_{N-S}} = \left| \begin{array}{l} \left(\frac{255}{156.6}\right)\left(\frac{27.25}{16.4}\right)\left(\frac{11.7}{3.3}\right) = 9593 \text{ LB/FT. WIDTH} \quad \checkmark \\[2mm] \end{array} \right.$

$\left. \right|_{MAX} \; 3545 \text{ LB/FT. WIDTH}$

$$A_{s,REQ'D} = \frac{9593}{56250} = 0.1705 \text{ IN}^2 / FT-WIDTH$$

$A_{s,PROV} = 0.62 \text{ IN}^2 \text{ (N-S)} \quad AND \quad 0.93 \text{ IN}^2 \text{ (E-W)}$

∴ PROVIDED REINFORCEMENT IS ADEQUATE

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PROGRESSIVE COLLAPSE | INDIRECT METHOD | TIE FORCES · 4

HORIZONTAL TIES TO COLUMN

$$R_u = \begin{array}{c|l} & 0.03(30)(27.25)(255)(4) = 25.02^k \checkmark \\[6pt] & 2(11.7) = 23.4^k \\[6pt] & \dfrac{(11.7)(9.0933)}{8.2} = 12.96^k \checkmark \\[4pt] \text{MAX} \;|\; \text{MIN} \end{array}$$

$A_{S, REQ'D} = \dfrac{25.02}{56.25} = 0.445 \text{ IN}^2$

$A_{S, PROV} = 0.93 \text{ IN}^2$

∴ PROVIDED REINFORCEMENT IS ADEQUATE

VERTICAL COLUMN TIES

$R_u = (27.25)(30)(255) = 208.5^k$

$A_{S, REQ} = \dfrac{208500}{56250} = 3.71 \text{ IN}^2$

$A_{S, PROV} = (4) \,\#10 = 5.08 \text{ IN}^2$

∴ DESIGNED COLUMN REINFORCEMENT WILL
   PROVIDE ENOUGH TIE STRENGTH

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PROGRESSIVE COLLAPSE — DIRECT DESIGN   VIERENDEEL TRUSS   1

① CHECK THE FEASIBILITY OF ONE TRUSS AT THE 6TH (TOP CHORD) AND 4TH (BOTTOM CHORD) FLOORS

② CHECK THE FEASIBILITY OF TWO TRUSSES AT THE 6TH AND 4TH AND 15TH AND 13TH FLOORS.

CHORD BEAM DEPTH $\leq 13''$ TO MAINTAIN A CEILING-TO-FLOOR HEIGHT OF $8'-0''$

①
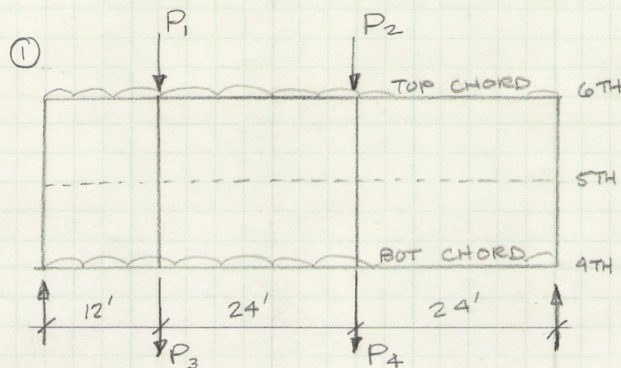
$f'_c = 5000$ PSI
$f_y = 60$ KSI

$h \leq 13''$

COL: 16×28 TYP.

$D + 0.5L + 0.2S$

LOADS:

$DL = $ SLAB WT $= 7(150)/12 = 88$ PSF
COL WT $= 16(28)(93.083)/144 = 43439$ LB

$LL = 40$ PSF $+ 20$ PSF $= 60$ PSF
$L_r = 30$ PSF

$P_1 = (88 + 60(.5))(20)(18)(9) + 43.4 + .2(30)(20)(18)$

$= 428^k + 4.24^k = 432^k$

$P_2 = (88 + .5(60))(24)(20)(9) + 43.4 + .2(30)(20)(24)$

$= 556^k + 4.24^k = 560^k$

$P_3 = 8478 + (88 + 60(.5))(20)(18) + (88 + 60(.5)(20)(6)$

$= 65^k + 4.24^k + (88 + 60(.5)(20)(18) = 112^k$

$P_4 = 8478 + 2(88 + 60(.5)(20)(24) = 122^k$

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PROGRESSIVE COLLAPSE DIRECT METHOD VIERENDEEL TRUSS 2

22229"k   15855"k   37914"k

-2346"k   -19332" -38788"k   STAAD PRO
                                     OUTPUT

47706"k 13229"k   44047"k

-30952"k   -19142"k -40734"k

TOP CHORD DESIGN
_____

MAX POSITIVE MOMENT: $40734^{"k}$ ($3395^{'k}$)

ESTIMATE SIZE: $bd^2 = 20 M_u$

$d = 13" - 2.5" = 10.5"$ $b = \dfrac{20(3395)}{(10.5)^2} = 616"$

REQUIRED STEEL AREA:

$A_s = \dfrac{M_u}{4d} = \dfrac{3395}{4(10.5)} = 80.82 \text{ IN}^2$

BOTH ESTIMATED REQUIRED WIDTH AND STEEL AREA
UNPRACTICAL. TRY TWO TRUSSES.

② P_1 P_2 w

Ⓐ

        P_3 P_4

        P_5 P_6

Ⓑ

        P_7 P_8

    12'   24'   24'

$f'_c = 5000 \text{ PSI}$
$f_y = 60000 \text{ PSI}$

$h \le 13"$
COL: 16 × 28, TYP

D + 0.5L + 0.2S

$P_7 = P_3$ (PREVIOUS)

$P_8 = P_4$ (PREVIOUS)

LOADS: SLAB = 88 PSF
        COL = 4239 #/COL

        LL = 60 PSF
        SL = 30 PSF

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PROGRESSIVE COLLAPSE | DIRECT METHOD | VIERENDEEL TRUSS 3

$w = 88 + 0.5(60) = 118 (20) = 2.36$ KLF

$P_1 = 8478 + [(88 + 0.2(30))(20)(18)] = 42.3^k$

$P_2 = 8478 + [(88 + 0.2(30))(20)(24)] = 53.6^k$

$P_3 = 4.5(4239) + 4(118)(20)(18) = 189^k$

$P_4 = 4.5(4239) + 4(118)(20)(24) = 246^k$

$P_5 = 4.5(4239) + 3(118)(20)(18) = 147^k$

$P_6 = 4.5(4239) + 3(119)(20)(24) = 189^k$

MAX POSITIVE
CHORD MOMENT = 21872 "k

MAX POSITIVE
CHORD MOMENT = 20771 "k

BOTTOM CHORD Ⓐ DESIGN

ESTIMATE SIZE: $bd^2 = 20 M_u$

$b = \dfrac{20(1823)}{(10.5)^2} = 331"$

REQUIRED STEEL AREA $= \dfrac{M_u}{4d} = \dfrac{1823}{4(10.5)} = 43.4 \text{ in}^2$

AGAIN, SIZE AND STEEL REQUIREMENTS ARE UNPRACTICAL

EACH TRUSS APPEARS TO HALVE THE MOMENT, SIZE, AND STEEL AREA, SO HOW MANY TRUSSES ARE NEEDED.

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PROGRESSIVE COLLAPSE   DIRECT METHOD   VIERENDEEL TRUSS   4

$P_{d1} = P_3$ (ONE TRUSS) = 112^k
$P_{12} = P_4$ (ONE TRUSS) = 122^k

$W_F = 2.36$ KLF $W_R = 1.88$ KLF.
COL = $4239 \, \#/$COL

$D_s = 88$ $L = 60$

$P_1 = 4239 = 4.24^k = P_2$

$P_3 = 2(118)(18)(20) + 2.5(4239) = P_7$
$= 95.6^k$

$P_4 = 2(118)(24)(20) + 2.5(4239) = 124^k = P_8$

$P_5 = 2.5(4239) + 118(18)(20) = 53.1^k = P_9$

$P_6 = 2.5(4239) + 118(24)(20) = 67.2^k = P_{10}$

ESTIMATE BEAM SIZE:

$M_{u, MAX} = 15304^{"k}$ $b = \dfrac{20(1275)}{(10.5)^2} = 231^{"}$

$A_s = 1275 / 42 = 30.4 \, IN^2$

AFTER THREE ITERATIONS, IT IS EVIDENT THAT EACH ADDITIONAL TRUSS REDUCES THE SIZE BY THE NUMBER OF TOTAL TRUSSES, THEREFORE THE SMALLEST BEAM WIDTH WOULD APPROX. EQUAL:

$$\frac{616^{"}}{13} = 47.9^{"} \quad OR \quad 4'$$

WITH 13" OF POSSIBLE DEPTH VIERENDEEL ACTION IS NOT THE BEST APPROACH.

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PROGRESSIVE COLLAPSE | MOMENT CAPACITY | TWO-WAY PT SLAB | 1

EXISTING SLAB PROPERTIES

$A_{ps} = 0.153(12) = 1.836 \text{ in}^2$ # OF TENDONS = 12

$f'_c = 5000 \text{ PSI}$ $d = 5\frac{1}{4} + \frac{1}{4} = 5\frac{1}{2}''$

$f_y = 60000 \text{ PSI}$ $t = 7''$

$b = 10(12) = 120''$ $A_s = 2.48 \text{ in}^2 \; ; \; 1.24 \text{ in}^2$

$f_{ps} = 184000 + [(5000)(120)(5.5) / (300 \times 1.836)] = 190000 \text{ PSI}$

ULTIMATE MOMENT CAPACITY

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$

$a = (A_s f_y + A_{ps} f_{ps}) / (0.85 f'_c b)$

$= (2.48(60) + 1.836(190)) / (0.85(5)(120))$

$= 0.976''$

$M_n = [2.48(60) + 1.836(190)](5.5 - \dfrac{0.976}{2})$

$= 498(5.012) = 2494^{''k} = 208^{'k}$ (NEG. MOMENT)

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$

$a = (A_s f_y + A_{ps} f_{ps}) / 0.85 f'_c b$

$= (1.24(60) + 1.836(190)) / (0.85(5)(120))$

$= 0.8299''$

$M_n = [1.24(60) + 1.836(190)](5.5 - \dfrac{0.8299}{2})$

$= 423.24(5.0851) = 2152^{''k} = 179^{'k}$
(POS. MOMENT)

GSA ALLOWABLE

$D_{CR} = Q_{UD} / Q_{CE}$

$D_{CR} = 1.5$

$Q_{UD} = $ ACTING FORCE

$Q_{CE} = $ UNFACTORED EXPECTED
ULTIMATE CAPACITY

$Q_{UD}^{-} = 3741^{''k}$

$Q_{UD}^{+} = 3228^{''k}$

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PROGRESSIVE COLLAPSE MOMENT CAPACITY | TWO-WAY PT SLAB · 2

ITERATION #1

MAX FAILING MOMENT : $6360^{\text{"k}}$

$D_{CR} = \dfrac{Q_{UD}}{Q_{CE}}$

$Q_{CE} = Q_{UD} / D_{CR} = 6360 / 1.5 = 4240$

$M_n^+ = 4240 = \left[A_S(60) + 348.84 \right]\left(5.5 - \left[\dfrac{A_S(60) + 348.84}{0.85(5)(120)} \right] \right)$

$4240 = (60 A_S + 348.84)(5.5 - (.117647 A_S^2 + .684))$

$= (60 A_S + 348.84)(5.158 - 0.058824 A_S)$

$A_S = 9.56 \text{ IN}^2 \quad (10 \ \# 9)$

ITERATION #2

$\overline{Q_{UD}} = 5000^{\text{"k}}$

$Q_{CE} = 5000 / 1.5 = 3333^{\text{"k}}$

$\overline{M_n} = 3333 = (60 A_S + 348.84)(5.158 - 0.059 A_S)$

$A_S = 5.71 \text{ IN}^2 \quad (6 \ \# 9)$

ITERATION #3

$\overline{Q_{UD}} = Q_{UD}^+ = 6360$

$A_S = 9.56 \text{ IN}^2 \quad (10 \ \# 9)$

ITERATION #4

$Q_{UD}^+ = 7500^{\text{"k}} \qquad Q_{CE} = 5000^{\text{"k}}$

$A_S = 13.21 \text{ IN}^2 \quad (11 \ \# 10)$

STEEL AREA IS GETTING EXCESSIVE; HOWEVER
FROM PAST ITERATIONS A Q_{CE}^+ OF $9000^{\text{"k}}$
IS IDEAL INCREASE SLAB THICKNESS TO GAIN
MORE CAPACITY

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| PROGRESSIVE COLLAPSE | MOMENT CAPACITY | TWO-WAY PT SLAB | 3 |

ITERATION #5

$A_s = 10.00 \text{ IN}^2$ (10 #9)

$M_n = 6000 = 948.84 (x - 0.93024)$ $x = 7.25''$

$7.25'' + 1.0'' + .25 = 8.5''$

$Q_{UD} = 6000 (1.5) = 9000$ ''k ✓

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PROGRESSIVE COLLAPSE | FLOOR SYSTEM | RC TWO-WAY SLAB W/BEAMS - 1

$f'_c = 5000$ PSI
$f_y = 60000$ PSI

$LL = 50(1.6) = 80$ PSF
$DL = 5(1.2) = 6$ PSF

BEAM SIZE = 12 X 18

TO VERIFY THE PCA SLAB OUTPUT, THE INTERIOR BAY SHOWN WILL BE DESIGNED FOR COMPARISON

ASSUME $t = 9"$

$b_{eff} = b_w + 2h_w \le b_w + 8t$
$= 12 + 18 = 30 \le 12 + 72 \checkmark$

$\beta = \dfrac{l_n}{s_n} = \dfrac{336}{296} =$

$b_e/b_w = 2.5$; $t/h = 0.5$ $I_b = 1.5\left(\dfrac{12(18)^3}{12}\right) = 8748$ IN^4

$I_s = (29.33)(12)(9)^3/12 = 21384$ IN^4

$\alpha_m = 0.41$: $t_{min} = \dfrac{336(0.8 + 0.3)}{36 + 5(1.14)(0.21)} = 9.94" \Rightarrow 10"$

SELF WT: $\dfrac{10(150)}{12} = 125 (1.2) = 150$ PSF

TOTAL LOAD = 236 PSF

TOTAL STATIC MOMENT

$M_{0_{N-S}} = \dfrac{0.236(29.33)(27 - 28/12)^2}{8} = 527^{1k}$

$M_{0_{E-W}} = \dfrac{0.236(27)(29.33 - 16/12)^2}{8} = 624^{1k}$

INTERIOR SPAN MOMENT DISTRIBUTION

$\alpha = 0.41$ $l_1 = 29.33$ $l_2 = 27$

N-S: $\dfrac{\alpha l_2}{l_1} = 0.45$ E-W: $\dfrac{\alpha l_2}{l_1} = 0.38$

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PROGRESSIVE COLLAPSE   FLOOR SYSTEM   RC TWO-WAY SLAB W/BEAMS 2

$Mo^+_{N-S} = 0.35 (527) = 184^{1k}$

$Mo^-_{N-S} = 0.65 (527) = 343^{1k}$

$Mo^+_{E-W} = 0.35 (624) = 218^{1k}$

$Mo^-_{E-W} = 0.65 (624) = 406^{1k}$

COLUMN STRIP	N-S M^-	N-S M^+	E-W M^-	E-W M^+
TOTAL STATIC MOMENT, 1k	343	184	406	218
COLUMN STRIP MOMENT, 1k	261	123	309	146
SLAB MOMENT, 1k	177	84	210	99
BEAM MOMENT, 1k	84	39	99	47
STRIP WIDTH, IN	162	162	162	162
EFFECTIVE DEPTH, IN	8.625	8.625	8.625	8.625
Mn, 1k	197	93	233	110
R, PSI	196	93	232	110
ρ	.0033	.0016	.00397	.0019
As, IN²	4.61	2.24	5.55	2.65
As, MIN, IN²	3.24	3.24	3.24	3.24
N, # OF BARS	15	11	18	11
N, MIN, # OF BARS	9	9	9	9

MIDDLE STRIP	N-S M^-	N-S M^+	E-W M^-	E-W M^+
Mo, 1k	343	184	406	218
MIDDLE STRIP MOMENT, 1k	82	61	97	72
STRIP WIDTH, IN	162	162	162	162
EFFECTIVE DEPTH, IN	8.625	8.625	8.625	8.625
Mn, 1k	91	68	108	80
R, PSI	91	68	108	80
ρ	.0015	.0011	.0018	.0013
As, IN²	2.1	1.54	2.52	1.82
As, MIN, IN²	3.24	3.24	3.24	3.24
N, # OF BARS	11	11	11	11
N, MIN, # OF BARS	9	9	9	9

BEAM CAPACITY

$M^-_{MAX} = 99^{1k}$ $M^+_{MAX} = 47^{1k}$

$$\frac{\phi f_y^2}{1.7 f'_c b} As^2 - \phi f_y d As = 99^{1k} \text{ (NEGATIVE)}$$
$$47^{1k} \text{ (POSITIVE)}$$

$As^- = 1.34 \text{ IN}^2 \Rightarrow 4 \# 6$

$As^+ = 0.65 \text{ IN}^2 \Rightarrow 3 \# 5$

SHEAR NOT CHECKED @ THIS TIME

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PROGRESSIVE COLLAPSE | FLOOR SYSTEM | REIN. CONC. SLAB DESIGN | 1
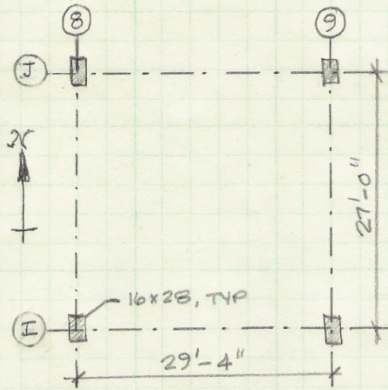
TWO - WAY FLATE PLATE SLAB

$f'_c = 5000$ PSI $f_y = 60000$ PSI

LL = 50 PSF (1.6) = 80 PSF
DL = 5 PSF (1.2) = 6 PSF
TL = 86 PSF + SELF WT.

$$t = \frac{ln}{30} = \frac{352 - 16}{30} = 11.2'' \ (11\frac{1}{2}'')$$

TO VERIFY THE PCA SLAB OUTPUT, THE INTERIOR BAY
SHOWN WILL BE DESIGNED FOR COMPARISON

TOTAL STATIC MOMENT

$$Mo_{N-S} = \frac{0.259 (29.33)(27 - 28/12)^2}{8} = 578^{1k}$$

$$Mo_{E-W} = \frac{0.259 (27)(29.33 - 16/12)^2}{8} = 711^{1k}$$

INTERIOR SPAN MOMENT DISTRIBUTION

$$Mo_{N-S}^+ = 0.35 (578) = 202^{1k}$$

$$Mo_{N-S}^- = 0.65 (578) = 376^{1k}$$

$$Mo_{E-W}^+ = 0.35 (711) = 249^{1k}$$

$$Mo_{E-W}^- = 0.65 (711) = 462^{1k}$$

COLUMN STRIP	N-S		E-W	
	M⁻	M⁺	M⁻	M⁺
TOTAL STATIC MOMENT, (1k)	376	202	462	249
COLUMN STRIP MOMENT, (1k)	282	121	347	149
STRIP WIDTH, b, (IN)	162	162	162	162
EFFECTIVE DEPTH, d, (IN)	10.125	10.125	10.125	10.125
M_n, (1k)	313	134	386	166
R, (PSI)	226	96.8	279	120
ρ	.0039	.0016	.0048	.0020
A_s, (IN²)	6.35	2.63	7.90	3.33
$A_{s.MIN}$, (IN²)	3.73	3.73	3.73	3.73
N, (# OF BARS)	21	13	26	13
N_{MIN}, (# OF BARS)	8	8	8	8

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PROGRESSIVE COLLAPSE | FLOOR SYSTEM | REIN. CONC. SLAB DESIGN | 2

MIDDLE STRIP	N-S M^-	M^+	E-W M^-	M^+
TOTAL STATIC MOMENT, ('k)	376	202	462	249
MIDDLE STRIP MOMENT, ('k)	94	81	115	100
STRIP WIDTH, b, (IN)	162	162	162	162
EFFECTIVE DEPTH, d, (IN)	10.125	10.125	10.125	10.125
M_n, ('k)	104	90	128	111
R, (PSI)	75.1	65	92.5	80.2
ρ	.0013	.0011	.0016	.0014
A_s, (IN²)	2.13	1.80	2.68	2.30
$A_{s\,MIN}$, (IN²)	3.73	3.73	3.73	3.73
N, (# OF BARS)	13	13	13	13
N_{MIN}, (# OF BARS)	8	8	8	8

BEAM ACTION SHEAR

$$V_u = \omega A = 0.259(355) = 92^k$$

$$\phi V_c = 0.75(2)\sqrt{5000}\,(10.125)(27)(12) = 348^k \quad V_u \leq \phi V_c \checkmark$$

PUNCHING SHEAR

$$\alpha_c = 40 \qquad \beta_c = 28/16 = 1.75 \qquad b_0 = 128.5 \text{ IN}$$

$$V_u = \omega A = 0.259(792 - (996/144)) = 203^k$$

$$\min \begin{vmatrix} 4 \checkmark \\ 2 + 4/1.75 = 4.29 \\ 2 + 40(10.125)/128.5 = 5.15 \end{vmatrix}$$

$$\phi V_c = 0.75(4)\sqrt{5000}\,(128.5)(10.125) = 276^k$$

$$V_u \leq \phi V_c \checkmark$$

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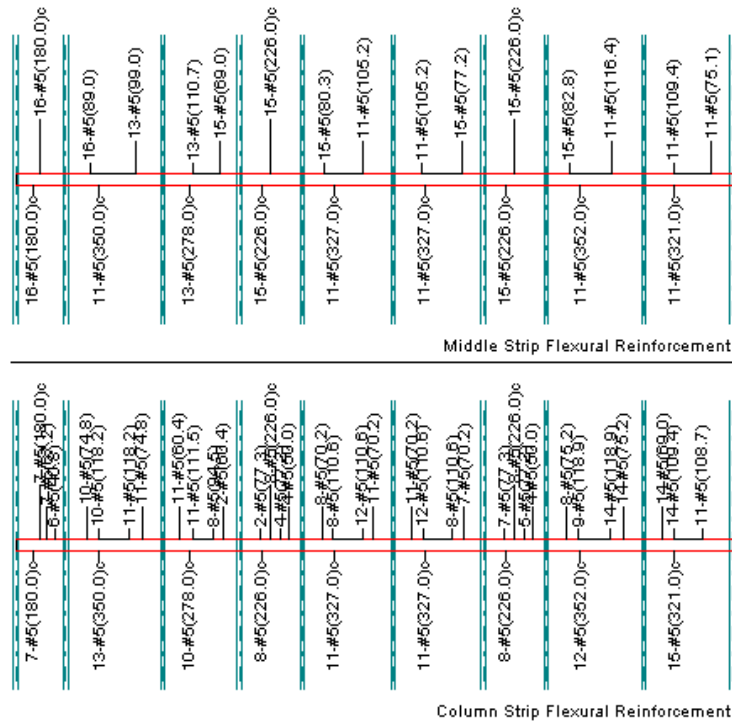
PCA Slab Output of reinforcement for flat plate reinforced concrete slab in E-W Direction.
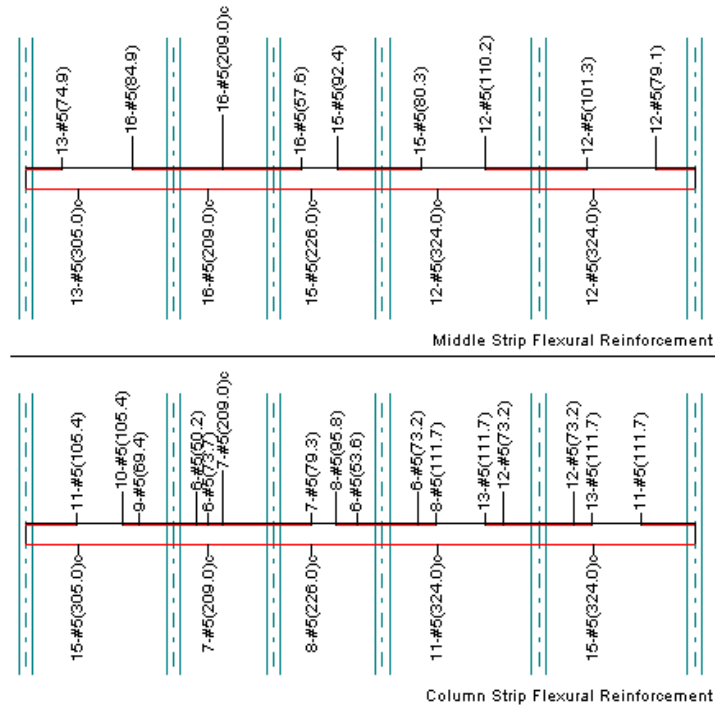
PCA Slab Output of reinforcement for flat plate reinforced concrete slab in N-S Direction.

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SAP 2000 Ouput of reinforcement for reinforced concrete slab with beams in E-W Direction (Location of interest on following page).

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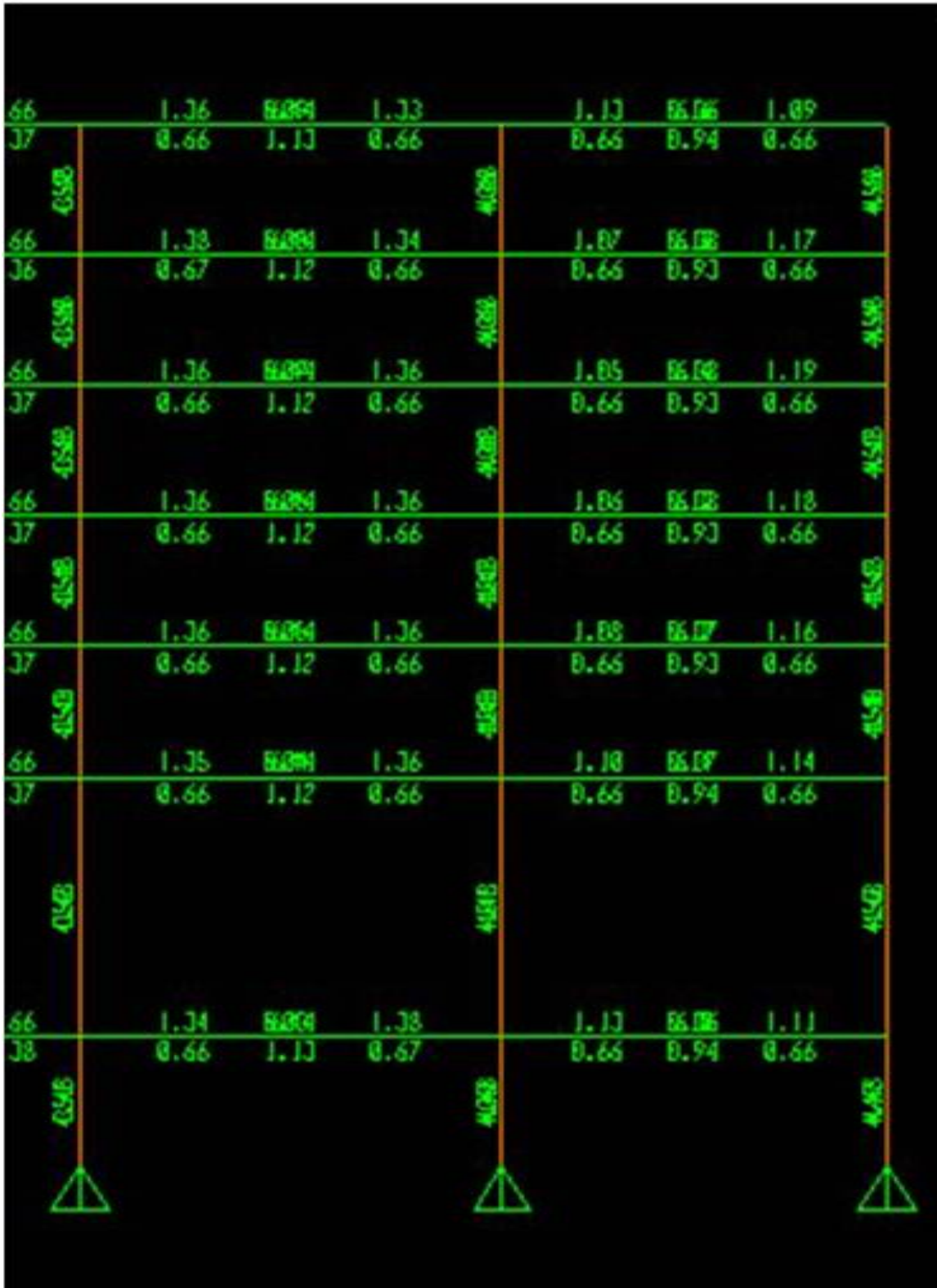
Enlarged view of SAP 2000 Output of rreinforcement for reinforced concrete slab with beams in E-W Direction.

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APPENDIX F – THERMAL AND MOISTURE CALCULATIONS

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Thermal Gradient and Vapor Pressure Data for Existing Wall Assembly:

Metal Panel Thermal Gradient							
Surface	Thickness (in)	Stud			Insulation		
		R	T_{xhigh} (°F)	T_{xlow} (°F)	R	T_{xhigh} (°F)	T_{xlow} (°F)
Outside	0	0.17	76.87	36.65	0.17	76.93	36.36
Metal Panel	1.75	1.07	76.03	40.71	1.07	76.46	38.64
Air Barrier	1.75	0.00	76.03	40.71	0.00	76.46	38.64
Sheathing	2.5	0.56	75.59	42.85	0.56	76.21	39.84
Stud w/ Insulation	6.25	6.00	70.89	65.70	13.00	70.50	67.59
GWB	6.75	0.45	70.53	67.41	0.45	70.30	68.55
Inside	6.75	0.68	70.00	70.00	0.68	70.00	70.00
	ΣR	8.93			ΣR	15.93	
	U-Value	0.11			U-Value	0.06	

Metal Panel Vapor Pressure							
Surface	Thickness (mm)	Stud			Insulation		
		R_v	P (Pa)	P_{sat} (Pa)	R_v	P (Pa)	P_{sat} (Pa)
Outside	0	0.000013	580.75	739.58	0.000013	580.14	731.31
Metal Panel	44.45	0.0000	580.75	867.10	0.0000	580.14	799.89
Air Barrier	44.45	0.00083	839.50	867.10	0.00083	799.88	799.89
Sheathing	63.5	0.00077	1078.54	941.47	0.00077	1002.88	838.10
Stud w/ Insulation	158.75	0.00000	1078.54	2168.51	0.00039	1105.48	2315.05
GWB	171.45	0.00051	1236.40	2300.93	0.00051	1239.54	2392.73
Inside	171.45	0.000067	1257.22	2514.45	0.000067	1257.22	2514.45
	ΣRv	0.00219			ΣRv	0.00258	

Brick Veneer Thermal Gradient							
Surface	Thickness (in)	Stud			Insulation		
		R	T_{xhigh} (°F)	T_{xlow} (°F)	R	T_{xhigh} (°F)	T_{xlow} (°F)
Outside	0	0.17	76.87	36.62	0.17	76.93	36.35
4" Face Brick	3.625	0.385	76.59	38.01	0.385	76.76	37.15
2" Air Space	5.625	1.13	75.74	42.10	1.13	76.28	39.49
Air Barrier	5.625	0.00	75.74	42.10	0.00	76.28	39.49
Sheathing	6.375	0.56	75.33	44.13	0.56	76.04	40.65
Stud w/ Insulation	10.125	6.00	70.84	65.90	13.00	70.48	67.65
GWB	10.625	0.45	70.51	67.53	0.45	70.29	68.59
Inside	10.625	0.68	70.00	70.00	0.68	70.00	70.00
	ΣR	9.37			ΣR	16.37	
	U-Value	0.11			U-Value	0.06	

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Brick Veneer Vapor Pressure							
Surface	Thickness (mm)	Stud			Insulation		
		R_v	P (Pa)	P_{sat} (Pa)	R_v	P (Pa)	P_{sat} (Pa)
Outside	0	0.000013	577.49	738.69	0.000013	577.47	731.03
4" Face Brick	92.075	0.009	1121.92	780.45	0.009	1103.71	754.47
2" Air Space	142.875	0.00027	1138.27	914.71	0.00027	1119.51	826.82
Air Barrier	142.875	0.00062	1175.91	914.71	0.00062	1155.89	826.82
Sheathing	161.925	0.00077	1222.44	988.85	0.00077	1200.87	865.07
Stud w/ Insulation	257.175	0.00000	1222.44	2183.86	0.00039	1223.60	2320.25
GWB	269.875	0.00051	1253.17	2310.65	0.00051	1253.31	2395.95
Inside	269.875	0.000067	1257.22	2514.45	0.000067	1257.22	2514.45
	ΣR_v	0.01125			ΣR_v	0.011638	

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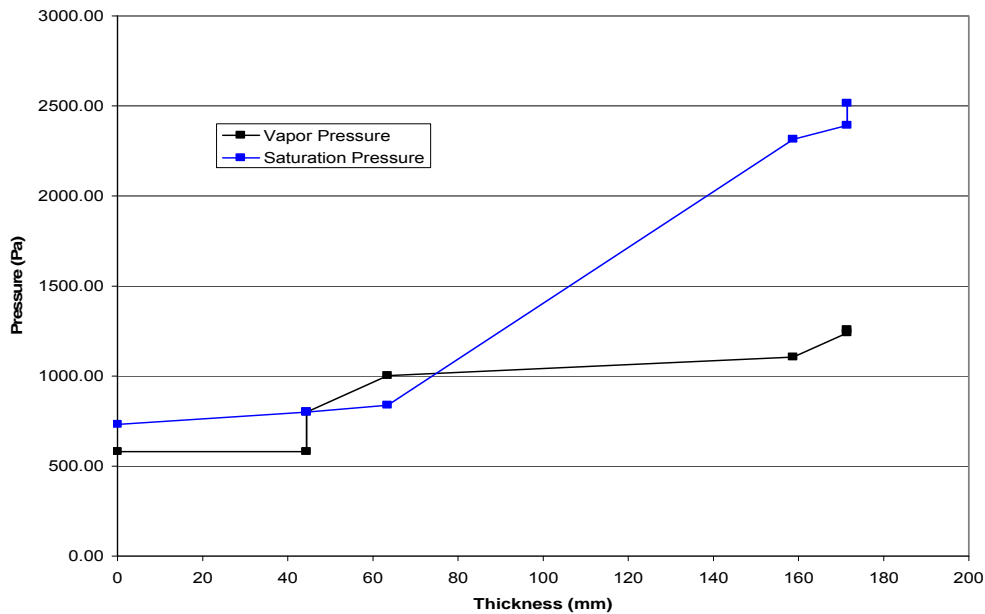
Vapor Pressure for Existing Brick Veneer Cavity Wall Assembly
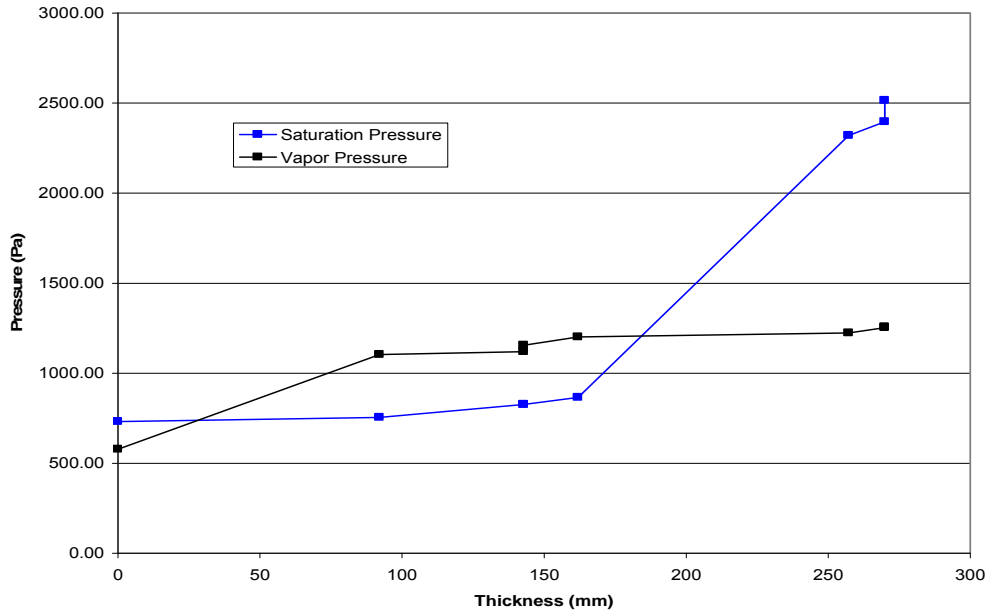
Thermal Gradient and Vapor Pressure Data for New Wall Assembly:

Metal Panel							
Surface	Thickness (in)	Stud			Insulation		
		R	T_{xhigh} (°F)	T_{xlow} (°F)	R	T_{xhigh} (°F)	T_{xlow} (°F)
Outside	0	0.17	76.88	36.59	0.17	76.93	36.34
Metal Panel	1.75	1.07	76.11	40.30	1.07	76.48	38.51
Air Barrier	1.75	0.00	76.11	40.30	0.00	76.48	38.51
Sheathing	2.5	0.56	75.71	42.25	0.56	76.25	39.64
Stud w/ Insulation	6.25	6.00	71.42	63.11	13.00	70.83	65.99
Sheathing	6.75	0.85	70.81	66.07	0.85	70.47	67.71
Vapor Barrier	6.75	0.00	70.81	66.07	0.00	70.47	67.71
GWB	7.25	0.45	70.49	67.64	0.45	70.28	68.62
Inside	7.25	0.68	70.00	70.00	0.68	70.00	70.00
	ΣR	9.78			ΣR	16.78	
	U-Value	0.10			U-Value	0.06	

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Metal Panel							
Surface	Thickness (mm)	Stud			Insulation		
		R_v	P (Pa)	P_{sat} (Pa)	R_v	P (Pa)	P_{sat} (Pa)
Outside	0	0.000013	576.88	737.94	0.000013	576.87	730.78
Metal Panel	44.45	0.0000	576.88	853.43	0.0000	576.87	795.71
Air Barrier	44.45	0.00083	587.65	853.43	0.00083	587.57	795.71
Sheathing	63.5	0.00077	597.61	920.17	0.00077	597.45	831.78
Stud w/ Insulation	158.75	0.00000	597.61	1981.63	0.00039	602.45	2190.52
Sheathing	171.45	0.00040	602.74	2196.84	0.00040	607.54	2324.81
Vapor Barrier	171.45	0.0500	1249.78	2196.84	0.0500	1249.84	2324.81
GWB	184.15	0.00051	1256.36	2318.84	0.00051	1256.36	2398.77
Inside	184.15	0.000067	1257.22	2514.45	0.000067	1257.22	2514.45
	ΣR_v	0.05259			ΣR_v	0.05298	

Brick Veneer							
Surface	Thickness (in)	Stud			Insulation		
		R	T_{xhigh} (°F)	T_{xlow} (°F)	R	T_{xhigh} (°F)	T_{xlow} (°F)
Outside	0	0.17	76.88	36.40	0.17	76.93	36.34
4" Face Brick	3.625	0.385	76.62	37.31	0.385	76.77	37.10
1.5" Air Space	5.125	1.12	75.86	41.56	1.12	76.32	39.30
Air Barrier	5.125	0.00	75.86	41.56	0.00	76.32	39.30
Sheathing	5.875	0.56	75.47	43.43	0.56	76.09	40.41
Stud w/ Insulation	9.625	6.00	71.36	63.41	13.00	70.81	66.09
Sheathing	10.125	0.85	70.77	66.24	0.85	70.46	67.77
Vapor Barrier	10.125	0.00	70.77	66.24	0.00	70.46	67.77
GWB	10.625	0.45	70.47	67.74	0.45	70.28	68.66
Inside	10.625	0.68	70.00	70.00	0.68	70.00	70.00
	ΣR	10.21			ΣR	17.21	
	U-Value	0.10			U-Value	0.06	

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Brick Veneer							
Surface	Thickness (mm)	Stud			Insulation		
		R_v	P (Pa)	P_{sat} (Pa)	R_v	P (Pa)	P_{sat} (Pa)
Outside	0	0.000013	576.85	732.46	0.000013	576.85	730.53
4" Face Brick	92.075	0.009	676.31	759.28	0.009	675.68	752.80
1.5" Air Space	130.175	0.00021	678.58	896.03	0.00021	677.94	820.71
Air Barrier	130.175	0.00062	685.46	896.03	0.00062	684.78	820.71
Sheathing	149.225	0.00077	693.96	962.66	0.00077	693.22	856.82
Stud w/ Insulation	244.475	0.00000	693.96	2002.01	0.00039	697.49	2198.19
Sheathing	257.175	0.00040	698.34	2209.54	0.00040	701.85	2329.41
Vapor Barrier	257.175	0.0500	1250.87	2209.54	0.0500	1250.91	2329.41
GWB	269.875	0.00051	1256.48	2326.85	0.00051	1256.49	2401.62
Inside	269.875	0.000067	1257.22	2514.45	0.000067	1257.22	2514.45
	ΣR_v	0.061582			ΣR_v	0.061971	

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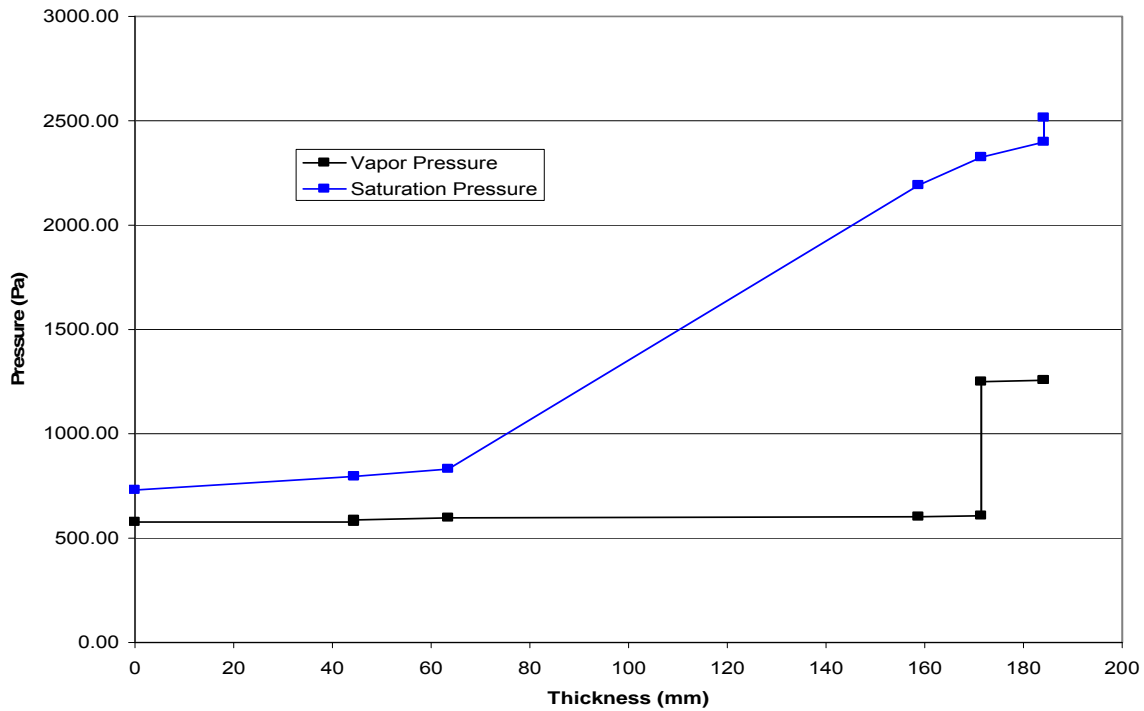
Vapor Pressure for New Brick Veneer Cavity Wall Assembly