# 40 Bond

**New York, NY**

## Proposal

Samantha D'Agostino

Structural Option

Consultant - Dr. Thomas Boothby

December 12, 2008

# Table of Contents

# Executive Summary

40 Bond is located on a 13,600 $ft^2$ parcel of land located on Bond Street between Lafayette and Bowery Street in New York City. The footprint of the building is 64'-8" by 134'-4" and the building has an overall height of 152'-0" from the cellar to the top of the penthouse structure. There is a 20'-0" setback at the seventh floor with a roof terrace that occupies this space. Typical spans range from 19'-6"×25'-0" to 23'-2 ½"×25'-0" and floor-to-ceiling heights range from 10'-10" to 14'-0". A total of 23 condominium units and 5 townhouses are contained within this building and the plans vary as the type and number of units change throughout. In addition to the building there is also a 140'-0" long, 22'-0" high cast aluminum gate located along Bond Street that was designed to withstand the lateral forces that are present at this site.

After reviewing the existing conditions, examining alternate framing systems and verifying the current lateral system, it is necessary to propose certain changes to 40 Bond that will develop into a study for the remainder of thesis coursework. For the structural depth, the transfer system will be redesigned using Vierendeel trusses in place of the transfer beams at the second, third and seventh floor. The architecture governed the dimensions and resulted in either deep, narrow beams or wide, slender beams; all of which were overly congested with reinforcement and required couplers because there was not a sufficient amount of area for typical splicing to be done. The trusses aim to work more efficiently in transferring loads and allow for an opportunity to incorporate more fully the cast iron architecture prevalent in lower Manhattan. An optimization study will be done on the lateral system and calculations will be done to determine if the existing 30" mat foundation can resist the forces provided by the gravity and lateral systems.

An architectural breadth will be studied due to the introduction of the Vierendeel trusses. Although these proposed structural elements may increase in width in comparison to the existing 10"×10" perimeter columns, the possibilities associated with alternating the architecture at transfer levels can further develop the cast iron typology while still maintaining its modern charm. An additional breadth topic relates to the cladding system. Because of the highly specialized nature of the curved glass and copper mullions, design and detailing must be done to ensure the current cladding can be used when the architecture is altered. Connections to the structural elements must also be considered along with research into the thermal and moisture protection provided by the system.

# Architectural Design Concepts

40 Bond Street was designed by the Swiss firm Herzog & de Meuron in association with New York based Handel Architects. The idea behind this luxury residential building was to reinvent the cast iron building typology that is prevalent in this lower Manhattan neighborhood. The building consists of one below grade level that houses a fitness center, storage space and equipment rooms. The first and second floors contain five through-building, 2-level townhouses. The layout then changes to accommodate four condominium units on each level from the third to the sixth floor. Once again, at the seventh floor the plans change incorporating a 20'-0" setback and reduced number of condominium units including only two per floor from levels 7 to 9. The tenth floor is a full plan condominium with a penthouse structure that rises 20'-0" above the main roof. In the penthouse a direct relation can be made between architectural concepts and structure. A 44'-0" clear span is achieved with two hidden columns and the core shear wall as supports leaving nearly two and a half glass walls.

40 Bond Street is attached on both its east and west faces. The north and south side then display two distinct facades. Along cobble-stoned Bond Street there are 5'-0"×10'-0" tilt-and-turn operable aluminum windows tinted to meet the necessary shading coefficient surrounded by bell-shaped green glass mullions (Figure 1). The Crisunid Cridecor curved glass produced by Cricursa in Barcelona is a combination of 5mm thick green glass laminated to 5mm thick clear glass. The mullions have a gray and green ceramic frit pattern on the edges of the bell to cover the frame but eventually lead to translucent glass at the apex. Below this layer of glass are No. 8 mirror stainless steel plate covers that allow for interesting reflections of the surrounding neighborhood. A rain screen of aluminum frames is also a part of the window assembly. This complicated facade was put in place by ornamental ironworkers and the fine craftsmanship can be seen in the detail. The north face employs the same windows but the material used for the mullions is replaced with pre-patina copper (Figure 2). Over time this material will develop a green patina and be closer in color to its parallel face.


Figure 1 – South Facade


Figure 2 – North Facade

These highly detailed facades imposed some strict tolerances in regards to the structure. Small 10"×10" concrete columns are located behind these mullions and space at 6'-3" on center between the second and tenth floors (Figure 3). The entirely glass south facade limits the variation in columns to less than ½". The variation in layout, fluctuating column dimensions, and necessary setbacks also resulted in different transfer locations that required beams to redirect the loads.


Figure 3 – Interior View of 10"×10" Columns within Glass Mullions

With many buildings located in cities such as New York, there is always an awareness of retail value. The more space there is to offer the more expensive the unit may be. The flat plate concrete system allows for tall floor-to-ceiling heights that remain unobstructed because of the limited number of beams and girders dropping into the space. In order to preserve the architectural design, maximize area and create appealing spaces, the concrete structure deviates from what is typical in the design and construction of a residential building to create an aesthetically pleasing and interesting structure. As a result of these characteristics, however, this 90,000 sf building had a very high cost in comparison to its size which is attributed to such things as formwork required for transfer beams and many slender columns.

# Existing Structural System

## Foundation

The geotechnical engineering study was performed by Langan Engineering & Environmental Services on September 10, 2004. In this study it was found that the water level was approximately 42.8' below the existing ground surface. The cellar extends 12'-8" below grade and therefore there was not a concern in regard to increased uplift pressures at this level. Langan noted that the bearing materials were suitable for a shallow foundation and that the recommended allowable bearing pressure would be 5 kips/ft$^2$. As a result, a 30" reinforced concrete mat foundation was designed with bearing walls and buttresses supported by a strip footing.

The 30" slab is 5 ksi normal weight concrete and increases to a thickness of 48" and 84" within the core shear walls where the elevator pit is located. Reinforcement varies throughout this mat slab. Buttresses ranging in size from 14"×29 ½" to 18"×79" are located around the perimeter. Interior columns ranging in size from 12"×22" to 28"×28" have an increased strength of 8 ksi. Located at columns 3B, 3C and 3F (Figure 4), there are also foundation mat shearheads to resist punching shear due to high loads that continue from the roof down to the foundation.

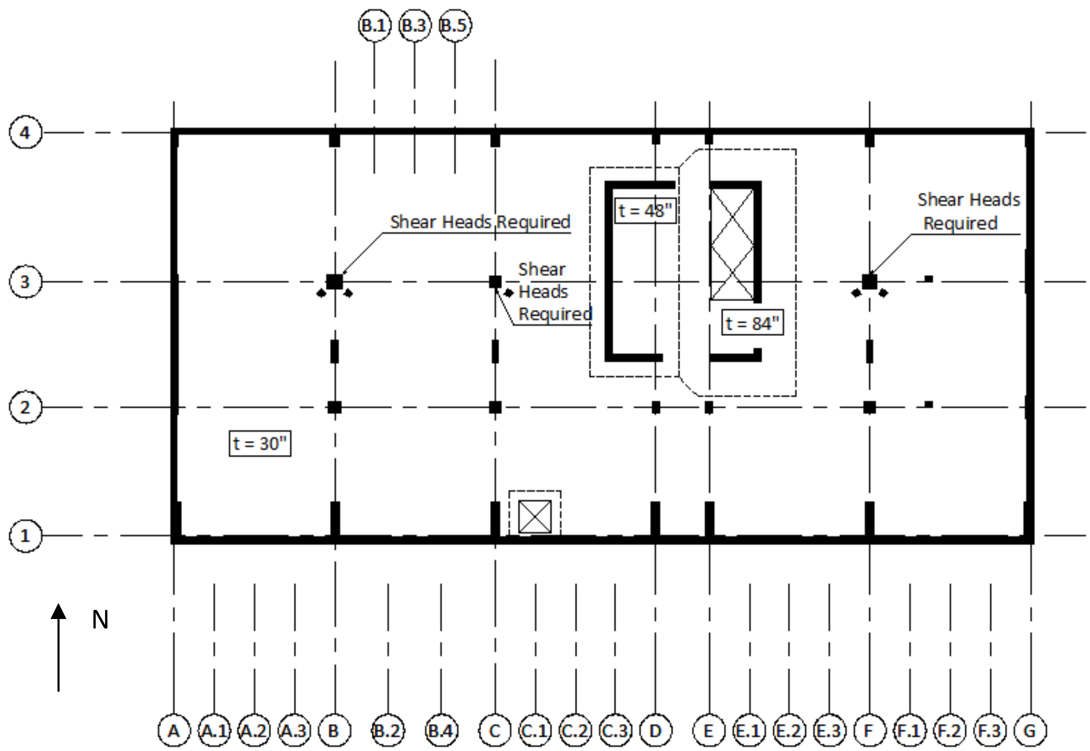



Figure 4 – Foundation Plan with Typical Column Grid and Shearhead Locations Noted

## Superstructure

This building is a reinforced concrete structure that allows for floor-to-ceiling heights up to 14'-0". Each floor consists of a 9" two-way flat plate that has a compressive strength ($f'_c$) of 5.95 ksi from the ground floor to the fourth floor and 5 ksi for the remainder of the upper floors. The typical reinforcement within the slabs is #4@12" top and bottom with various sizes and spacing of bars at column locations. Typical to all floors are small 3" slab depressions at the fireplaces and toilet areas as well as a 14" slabs within the core. The perimeter columns found on the first and second floor range in size from 10"×24" to 16"×58". The interior columns on all floors are a variation of either 12"×22", 22"×22", 22"Ø, 26"×26" or 28"×28". Similar to the slab material properties, the column compressive strength varies from 8 ksi at the foundation, to those columns supporting the fourth floor, and 5 ksi for the remainder of the columns. At the second floor on the north side of the building and at the

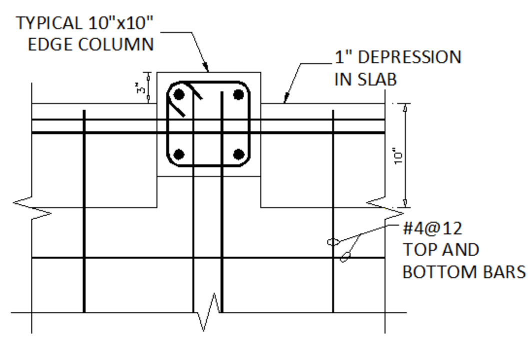



Figure 5 – Typical Perimeter Column Detail

third floor on the south side of the building, there is the introduction of the 10”×10” columns spaced at 6’-3” on center that extend up the remaining height of the building. In order to accommodate the facade connections, these slender columns only have a 7” slab encroachment and a 1” slab depression on either side (Figure 5).

These closely spaced 10”×10” columns transition down to fewer columns below and transfer beams are needed to redirect the load. Extending along the north side of the second floor is a 14”×40” transfer beam that is very much congested with reinforcement (Figure 6). Along the south face at the third floor is a 60”×16” transfer beam, which also requires a large amount of reinforcement (Figure 7). At the seventh floor there is a 20’-0” setback to allow for a roof terrace. As a result of this accessible exterior area, there are increased loads present and the slender columns transfer those loads through a 20”×24” beam (Figure 8). All the transfer beams have a compressive strength of 10 ksi and require rebar couplers for the top and bottom bar splices. Because of the beam dimensions there is not enough room to provide typical bar splicing and the expensive couplers must be applied. Site photographs of this condition as well as elevations and plans denoting the transfer locations may be found in Appendix A. In addition to these beams there is also another transfer at the fourth floor due to the presence of an adjacent chimney and beams around the stair openings in the townhouses.

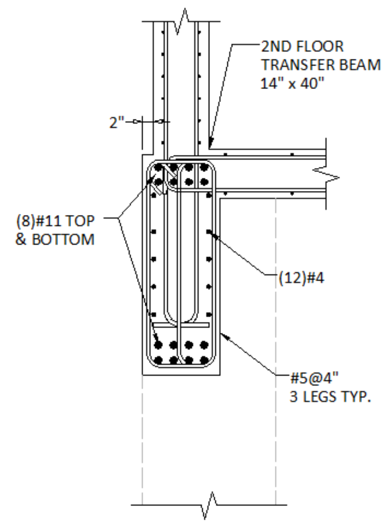



Figure 6 – Second Floor Transfer Beam Detail

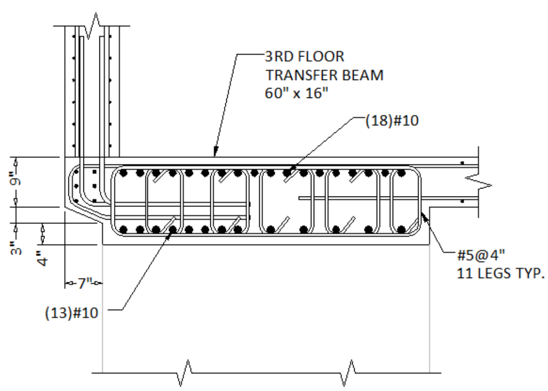



Figure 7 – Third Floor Transfer Beam

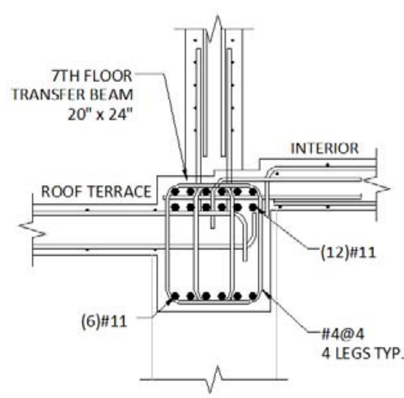



Figure 8 – Seventh Floor Transfer Beam

The penthouse level and its roof are a great example of what can be achieved when using concrete. The dimensions of the penthouse are 23'-4"×44'-6" and it has a thickened 19" slab with #4@12 top bar reinforcement and #5@8 bottom bar reinforcement. A 44'-0" clear span is achieved with the support of the concrete shear walls to the east and two 28"×16" columns to the west. The loads from the two columns need to be transferred and a 32"×24" beam is used to direct these loads to nearby columns, one of which is only 10"×14" (Figure 9). The roof above this long span structure is a combination of upturned beams, inclined piers, and two separate 8" slabs with #5@12 top and bottom spanning between its two supports. Located on the other side of the core is an enclosed elevated mechanical room. Additional loads due to the equipment and its surrounding 8" CMU walls will be applied at this level.

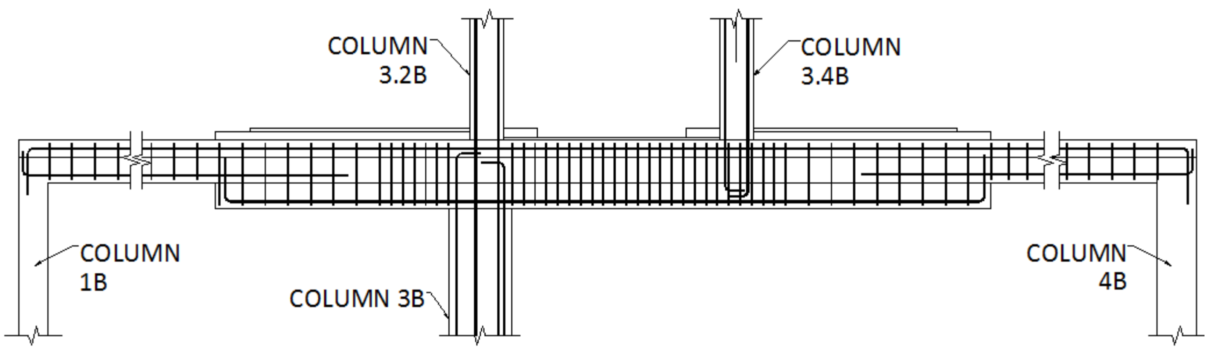



Figure 9 – Transfer Beam at Penthouse Level

## Lateral System

The lateral system is a combination of 12" ordinary reinforced concrete shear walls (Figure 10). Elevations of these walls are located in Appendix B, which clearly defines all openings and the location of coupling beams throughout the height of the building. The typical horizontal reinforcement in these walls is #4@12, while the vertical reinforcement ranges from #4@12 to #8@6, depending on the level they are located on and which portion of the shear wall is being examined. The west shear wall is reinforced with #4@12 as the horizontal reinforcement and a range of vertical reinforcement from #4@12 to #7@12. All shear walls supporting the ground floor to those supporting the fourth floor have concrete with a compressive strength $f'_c = 8$ ksi while those supporting the rest of the building have an $f'_c = 5$ ksi.

The presence of the west shear wall allows for the center of rigidity to move closer towards the middle of the plan. Because the core shear walls are not centralized within the building they draw the rigidity to the east. When the center of rigidity is not in line with the resultant lateral force there is eccentricity and moments due to torsion become a factor.

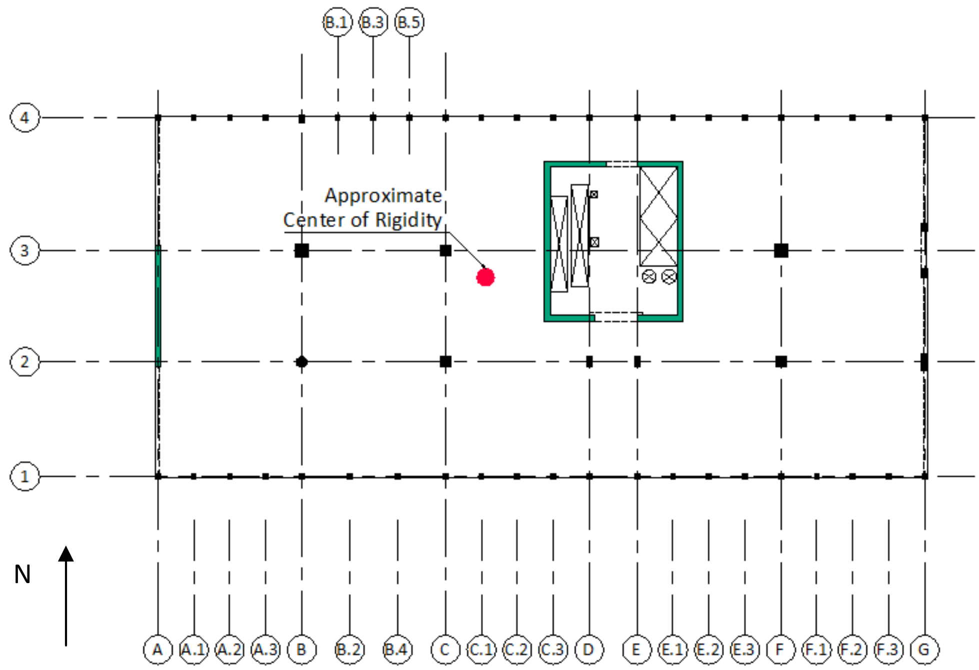



Figure 10 – Typical Plan with Lateral System Highlighted

# Problem Statement

The transfer system within 40 Bond is designed to showcase the architecture with limited obstruction to the openness of both plan and facade. The glass and copper mullions are to be the main focus on the exterior of the building in addition to the 5'-0"×10'-0" operable windows. Therefore, the columns along the north and south face need to have dimensions that will allow them to be enclosed by the mullions. Similarly, providing a grand space within the residences is also a goal, so columns protruding into the area are not favorable either. As a result, 10"×10" concrete columns spaced at 6'-3" on center are able to be contained in the mullion components and stay in line with the window glazing, nearly disappearing into the exterior face.

At the ground floor and the second floor however, longer spans are desired for the townhouse entrances and access to the rear gardens. The column spacing is increased to 25'-0" on center and transfer beams are needed as columns transition from the 6'-3" spacing above. These transfer beam should not compromise the tall floor-to-ceiling heights or project into the program spaces, so the members are kept either slender and deep like the 14"×40" second floor transfer beam or wide and shallow like the 60"x16" third floor transfer beam. In order to carry the load, these beams are heavily reinforced and require rebar couplers because there is not enough space available for splicing. Couplers are also used within the 10"×10" columns for this same reason. The formwork, the couplers, and the installation of these components are very expensive and the beam dimensions are challenging to construct.

After reviewing this information, the goal is to increase the efficiency of the transfer system required by 40 Bond. A study is needed to investigate a more effective way to transfer the loads at the three major interfaces where columns spans vary, while still maintaining the architectural concept. The live loads required by ASCE 7-05 in addition to all dead loads must successfully continue through the superstructure and into the foundation. Lateral loads must also be recalculated based on any changes to the overall building weight due to the application of the proposed study. The existing lateral resisting system is composed of ordinary reinforced concrete shear walls, but the layout of this system should also be examined to ensure that this too is performing efficiently. Finally, because of the possibility of increased loads and any other implications resulting from the implementation of new systems, checks will be done at the foundation to verify whether the existing 30" mat slab is sufficient to withstand the changes or if alterations must be made.

# Proposed Solution

To increase the efficiency of the transfer system present within 40 Bond, Vierendeel trusses will be employed at the existing transfer locations on the second, third and seventh floor. A variety of possible layouts will be analyzed to determine which is best suited for the needs of 40 Bond. The Vierendeel truss is comparable to the grid-like facade created by the slab and 10"×10" columns, because it still maintains open, rectangular panels within the fixed frame it provides (Figure 11). Unlike the current exterior framing system, the trusses will most likely extend into the interior more than the existing columns and the vertical members will at least increase in width at the top joint if not throughout the entire member. Exact member sizes cannot be estimated at this time, but great effort will be put forth to maintain the architectural vision. Deviation from the present layout will be considered in order to lend the opportunity to examine the facade as an architectural statement as well as an independent system. Both reinforced and prestressed as well as cast-in-place and precast concrete trusses will be considered to see which provides the most efficient and economical solution.

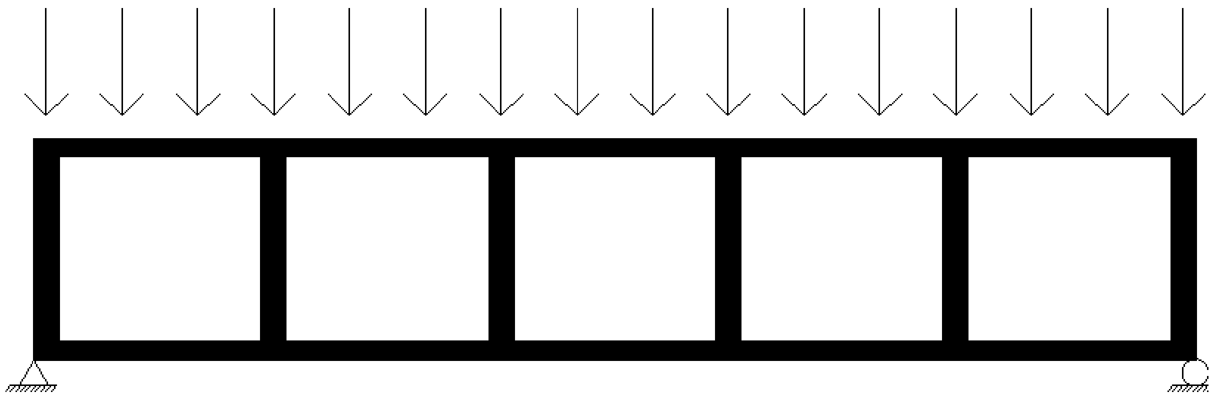


Figure 11 – Schematic Vierendeel Truss

In regards to the requirement for including the lateral resisting system, a new design will not be considered. Rather, an optimization study will be done to ensure that the lateral system is also designed most efficiently. If modifications are required, they will be made and results will be summarized. This optimization study will be done with the updated lateral loads that will be determined once the trusses are designed and the overall building weight is reviewed. Finally, analysis will be done on the foundation to ensure the new loading can be resisted by the existing 30" mat slab or if a re-design is required.

## Solution Method

The gravity loads will be defined according to ASCE 7-05 and applied to the overall structure. Column load takedowns will be required in order to determine the forces that will be applied to each of the Vierendeel trusses. Historical articles and texts will be used to locate the calculations required to analyze the trusses by hand. Time permitting, these calculations will be used to compare to the computer output. From research done prior to this proposal, an ASCE paper from 1936 was found and has a detailed description of the analysis of a Vierendeel truss. Further research will be conducted to see if other methods are also available. Iterative computer analyses of the trusses will be done using RISA 3D to determine the most reasonable arrangement that will work both structurally and architecturally.

Once the trusses are defined for each location, the updated building weight will be calculated. This will then be used in accordance with ASCE 7-05 to determine the lateral loads. Hand calculations will be computed using ACI 318-08 with the assumption that the shear walls withstand 100% of the lateral loads. An ETAB computer model will then be created for use in the optimization study of that lateral force resisting system. Slight alterations to the irregular openings on the lower floors will be made and then compared to the original shear wall layout to determine if the modifications increased the shear capacity of the walls.

After obtaining the values of the loads transmitted through the gravity system and the lateral system, hand calculations will be done to verify if the existing mat foundation is able to withstand the forces due to bearing, overturning, shear and torsion. Supporting calculations and suggestions will be made if a change to the foundation is required.

## Breadth Topic 1 – Facade Architecture

In addition to the depth study, two additional breadth topics are required in an area outside of the structural engineering realm. The first study to be considered is architecture. Located in the NoHo (North of Houston Street) neighborhood of Manhattan, 40 Bond Street is an iconic piece of architecture intertwined with an equally interesting structure. The idea behind this project is a reinvention of the cast iron building typology that is prevalent in this area of New York City and can be seen in buildings located on either side of 40 Bond (Figure 12). With the introduction of Vierendeel trusses, the facade will have to be altered to contain the proposed structure. One possibility is that conical capitals may be needed at the top joints of the truss to provide adequate room for the required reinforcing at that connection. A change like this would directly mimic the columns seen on the storefronts and apartment buildings throughout lower Manhattan. Another possibility is that the spans of the vertical members of the truss will most likely exceed the 6'-3" spacing of the 10"×10" concrete columns. Therefore at the second and third floor where the trusses will be required, a variation in the mullions will be produced similar to the varied column locations seen on the front of Old Stern's Department Store on West 23rd Street in 1878

Figure 12 – 40 Bond and Neighboring Buildings

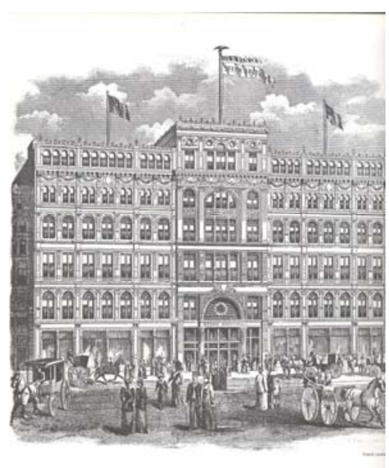


Figure 13 – Old Stern's Department Store 1878
(Credit: *Cast-Iron Architecture in New York*)

(Figure 13). Additional photographs taken from *Cast-Iron Architecture in New York* by Margot Gayle and Edmund V. Gillon Jr. are located in Appendix C.

The aforementioned possibilities are just a few that may result from the structural depth study. When going through the iterative truss analyses, the different options are going to be designed with the intent to not only efficiently transfer loads, but also to provoke architectural interest and complement the remaining facade. Although historic guidelines are not imposed on this building, trying to incorporate it more closely with the surrounding architectural context is one goal for the overall thesis study.

## Breadth Topic 2 –Thermal, Moisture, and Facade Connection Study

The second breadth proposal is related to the building facade as a system. The intricate glass mullions found on the south face along Bond Street had to be installed by ornamental ironworkers (Figure 14). The 5mm thick green glass is laminated to 5mm thick clear glass that encloses the stainless steel plate covers and a rain screen of aluminum frames within the mullion assembly. The proposed study is to examine the facade components, how they interact together, how they are required to connect to the structure, and how the architectural changes made within this thesis study will be constructed. Information is provided by Cricursa, the manufacturer of the curved glass from Barcelona, on additional shapes, forms, allowable radii and installation. The existing structural slab has 1" slab depressions on either side of the 10"×10" perimeter concrete columns, so it is likely that some type of alteration will need to be made to the trusses to provided an adequate connection plane.

Figure 14 – Typical Mullion Intersection

On the north facade copper mullions are used. Similar to the information required to work with the glass mullions, research will be done to determine how to produce the copper shapes necessary to enclose the Vierendeel truss as well as to understand how the mullions work as a system. In addition to creating facade elements to enclose the proposed structural members, research will also be done in regards to proper thermal and moisture protection of this cladding system, including the operable aluminum windows.

# Task and Tools

- **Vierendeel Trusses**

  Task 1: Determine loads to be applied to trusses
  - (a) Determine dead loads and live loads in accordance with ASCE 7-05
  - (b) Apply correct loads to each level and determine distribution to each column (tributary area)
  - (c) Perform a load takedown at all columns along Line 1, Line 2 and Line 4

  Task 2: Research hand calculation methods for analysis and design of Vierendeel trusses

  Task 3: Analyses of Vierendeel Trusses
  - (a) Apply loads determined in Task 1 to the trusses located at the second, third and seventh floors using RISA 3D
  - (b) Repeat analysis at each location with reasonably arranged truss layouts (keeping architecture in mind for both the plan and the facade)
  - (c) Determine the most efficient truss at each location
  - (d) Determine if all three locations can use the same truss (ie. Same spacing of vertical members in order to keep the architecture consistent)
  - (e) Select final trusses to be used
  - (f) Verify computer tabulated results to hand calculations

- **Lateral System**

  Task 4: Determine lateral loads to be applied to lateral force resisting system
  - (a) Recalculate the overall building weight with the addition of the Vierendeel trusses
  - (b) Verify the wind loads determined using ASCE 7-05
  - (c) Recalculate the seismic forces with the updated building weight

  Task 5: Perform optimization study
  - (a) Assuming 100% of lateral load resisted by shear wall, perform hand calculations to analysis shear walls referencing ACI 318-08
  - (b) Use ETABS to create original lateral system model
  - (c) Use ETABS to model the modified shear walls
  - (d) Compare the results to determine if modifications provided significant improvement of the shear resistance of the building

- **Foundation**

  Task 6: Verify existing foundation can adequately carried all loads present in proposed systems

- **Breadth Topic 1 – Facade Architecture Study**

  Task 7: Research existing cast iron architecture in New York City, particularly lower Manhattan

Task 8: Architectural Design
(a) Schematic design of possible concepts
(b) Attempt to mimic original cast iron architecture within the limits of the glass and copper mullions
(c) Decide on final design that still provides an openness with the facade

- **Breadth Topic 2 –Thermal, Moisture, and Facade Connection Study**

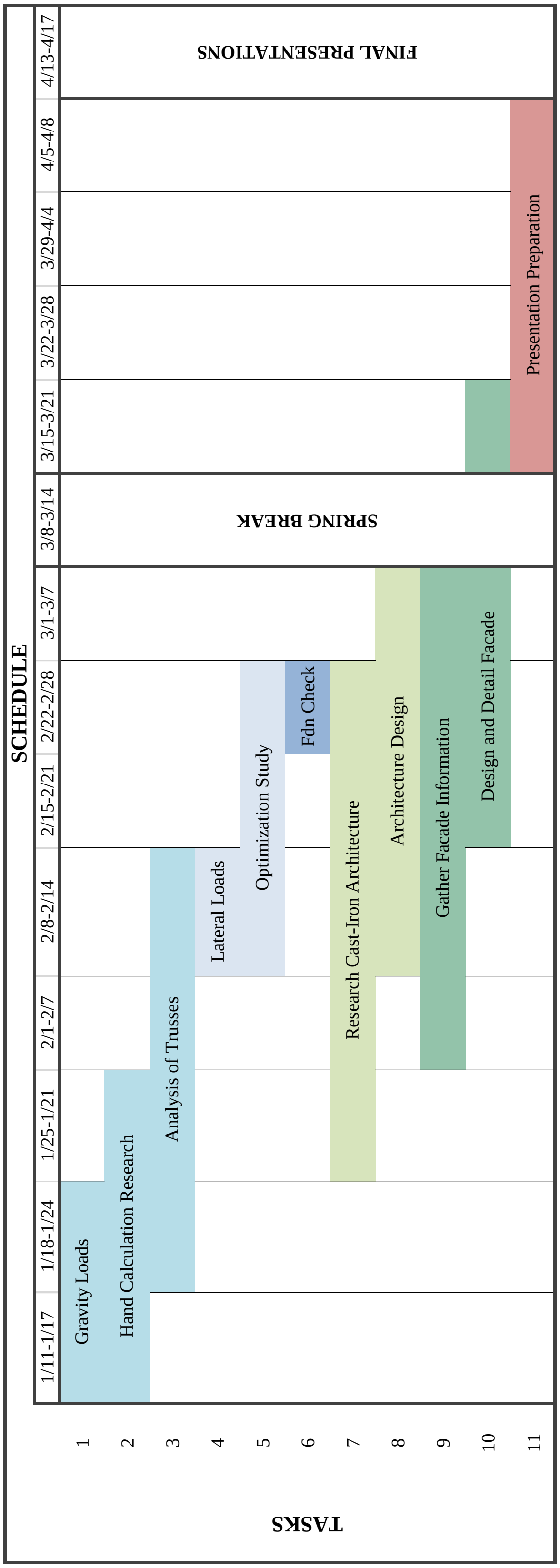
Task 9: Gather information
(a) Research all components in cladding system
(b) Find installation manuals if possible
(c) Research possible concerns with this system
(d) Clarify how the thermal and moisture protection is provided

Task 10: Design
(a) Detail mullions with appropriate radii and curvature (reference Cricursa)
(b) Design proper connection to structural system
(c) Adjust windows if necessary
(d) Include full description and details of proposed facade changes as well as the existing facade

- **Presentation**

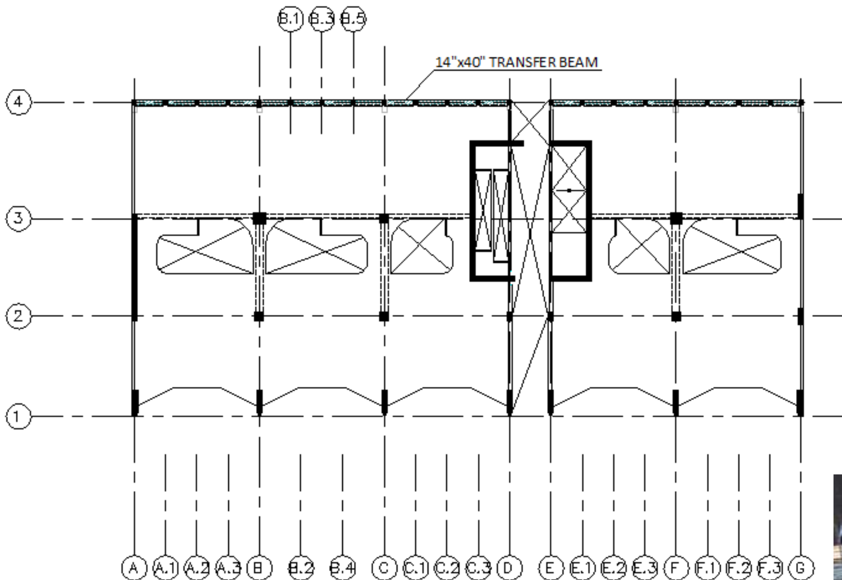
Task 11: Prepare presentation
(a) Powerpoint
(b) Final Report
(c) Updating CPEP with any final information

# Timetable


SCHEDULE
TASKS
1/11-1/17
1/18-1/24
1/25-1/21
2/1-2/7
2/8-2/14
2/15-2/21
2/22-2/28
3/1-3/7
3/8-3/14
3/15-3/21
3/22-3/28
3/29-4/4
4/5-4/8
4/13-4/17
1
2
3
4
5
6
7
8
9
10
11
Gravity Loads
Hand Calculation Research
Analysis of Trusses
Lateral Loads
Optimization Study
Fdn Check
Research Cast-Iron Architecture
Architecture Design
Gather Facade Information
Design and Detail Facade
Presentation Preparation
SPRING BREAK
FINAL PRESENTATIONS

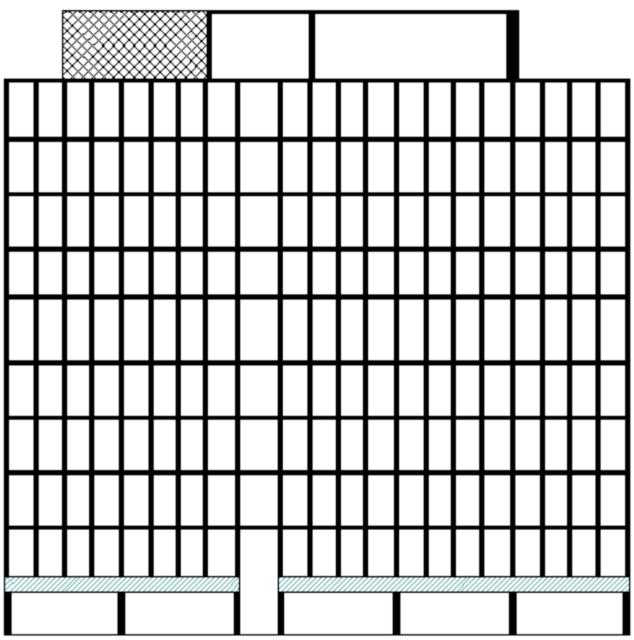
# Appendix A

## Second Floor Transfer

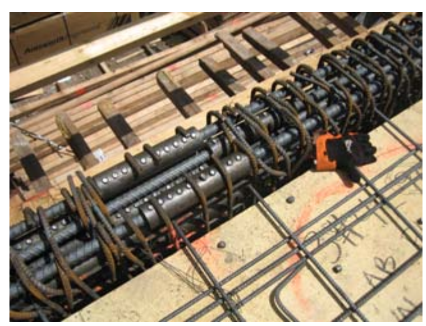



PLAN

TRANSFER BEAM

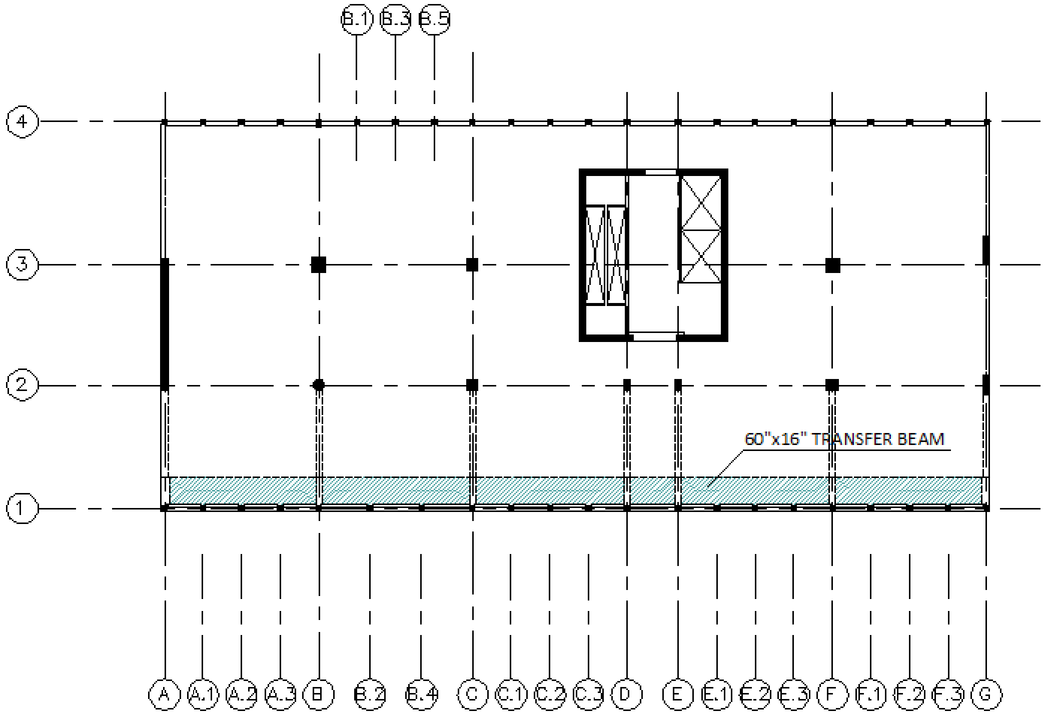


ELEVATION ALONG LINE 4



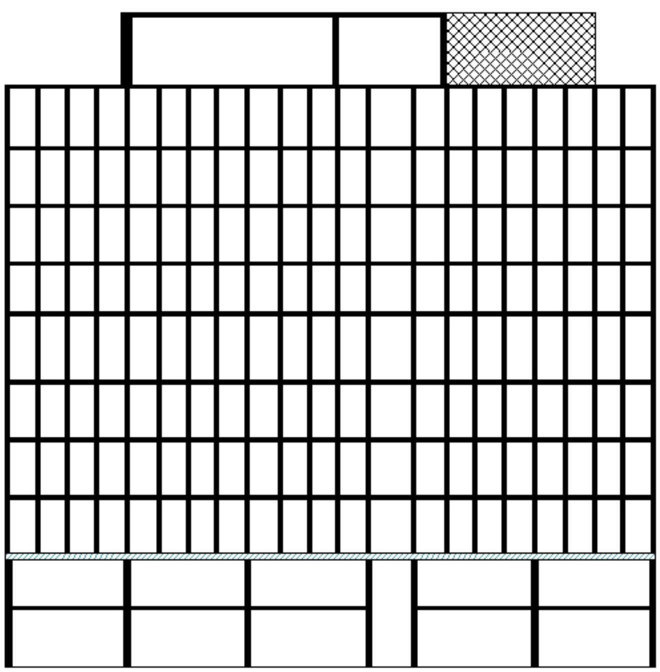
COUPLER DETAIL

# Third Floor Transfer




PLAN

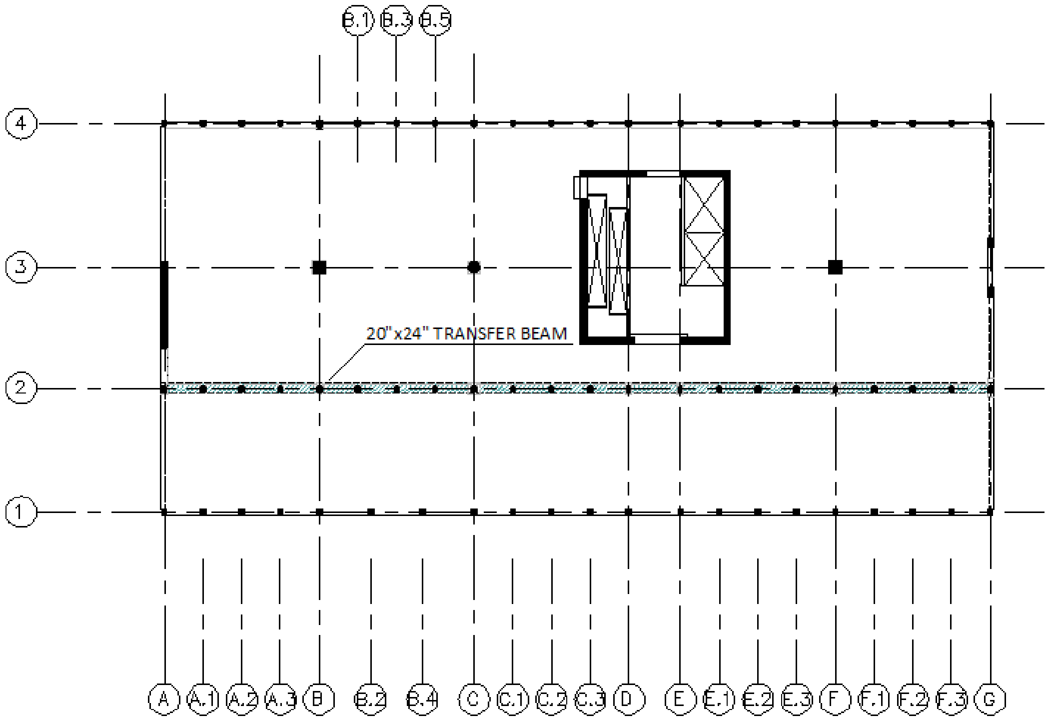
TRANSFER BEAM



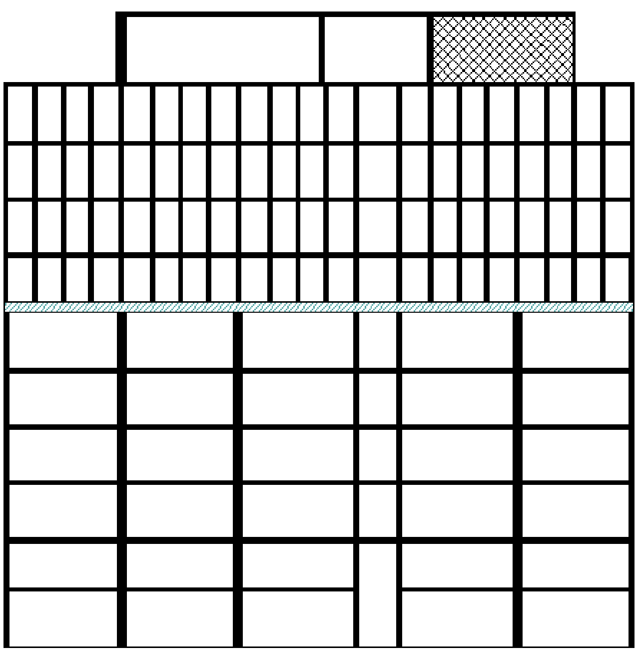
ELEVATION ALONG LINE 1

COUPLER DETAIL

# Seventh Floor Transfer




PLAN



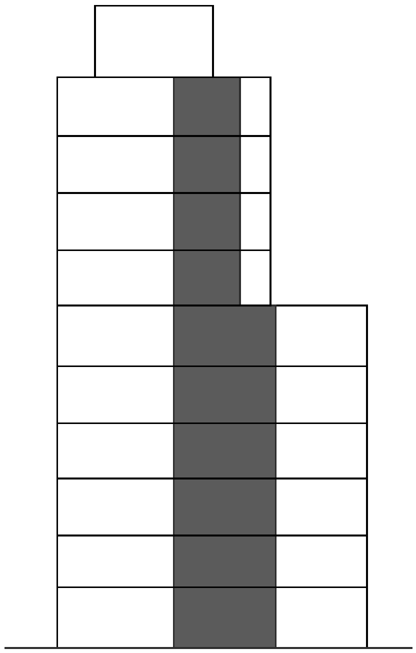
ELEVATION ALONG LINE 2

TRANSFER BEAM

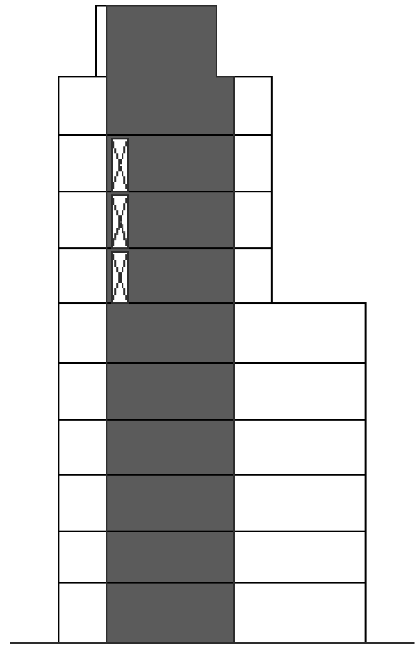
COUPLER DETAIL

# Appendix B

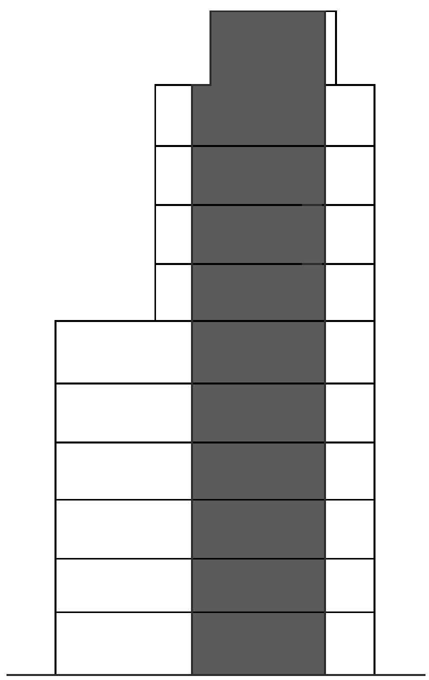
Shear Wall Elevation



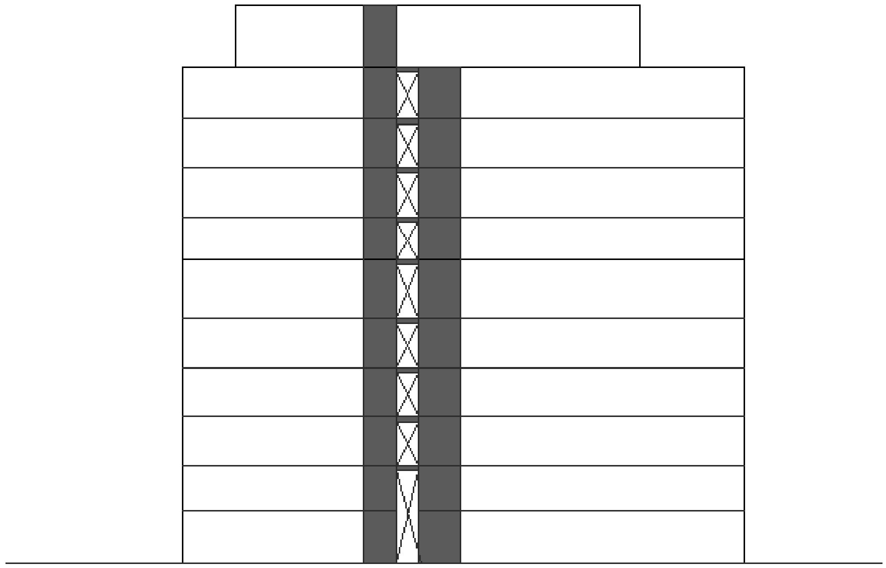
Elevation of Wall 1



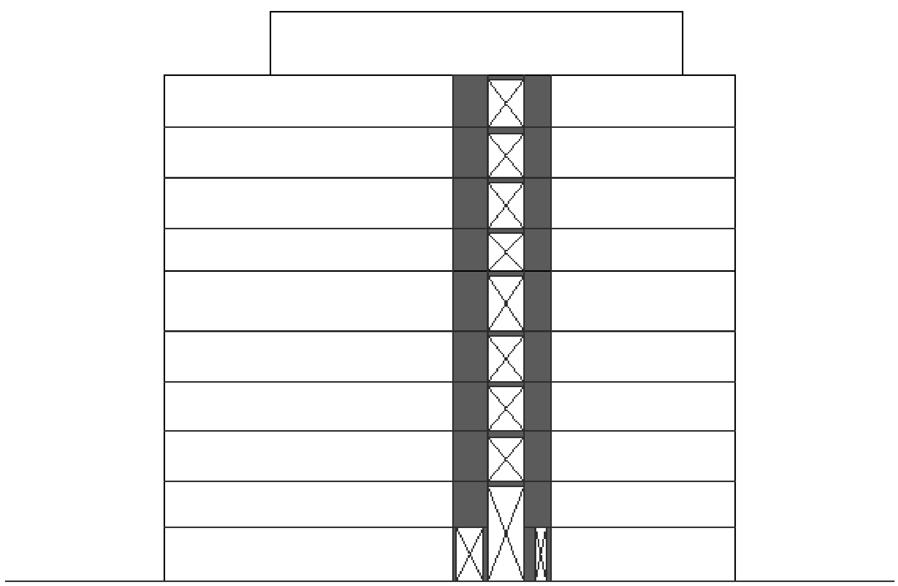
Elevation of Wall 2 (Core)



Elevation of Wall 3 (Core)



Elevation of Walls 4 and 5 (Core)



Elevation of Walls 6 and 7 (Core)

# **Appendix C**

Cast Iron Photographs

1-5 Bond Street (1880)

Old Bond Street Bank, 330 The Bowery (1874)

453-455 Broome Street (1873)

Old McCreery's Dry Good Store, 801 Broadway (1868)

351-353 Canal Street (1871)

116-118 Franklin Street (1869)

Gayle, M. and Gillon, E.V. (1974) *Cast Iron Architecture in New York*, Dover Publications, Inc., New York, NY