Visteon Village Corporate Center

Van Buren Township, MI

Thesis Final Report
Jamison David Morse
Structural Option
Advisor: Dr. Andres Lepage

Visteon Corporate Village Center

One Village Center Dr
Van Buren, MI 48111

General Information:

Function: Office and laboratory work areas, gathering spaces, and cafeteria
Size: 130,000 gsf
Height: 64' above grade
Overall Project Cost: $85 Million
Construction Dates: 2002- December 2004
Delivery Method: Design-Build

Project Team:

Owner: Visteon Corporation
General Contractor/CM: Walbridge Aldinger
Architects/Engineers/Master Planning: SmithGroup
Geotechnical Consultants: Somat Engineering
Testing and Inspection: SME Consultants

Architecture:

- Pressure-bar type glass curtain wall system
- FBX face brick or split face CMU exterior masonry walls
- Factory foamed insulated metal panelling system constructed in a board and batten style
- Extruded aluminum window and door framing
- UL listed class A continuous panel metal roofing system

Structural System:

- Two-way reinforced slab on grade
- Structural steel framing
  - Wide flange columns
  - Wide flange girders and beams
  - Typical bay sizes of 20'x20' and 40'x20'
  - Steel moment frames to resist lateral loads
- Composite slab floor and roof system
- Pre-engineered steel structure at penthouse level

Mechanical Systems:

- Under-floor air distribution system (UFADS)
- Dedicated VAV/reheat laboratory exhaust system for Chemistry wet labs
- Natural gas fired 4-pass fire tube boilers
- Water-cooled electric centrifugal water chillers with plate and frame heat exchangers

Lighting/Electrical System:

- Single 13,800 volt, 60 Hertz primary feeder
- 1000kw and 350 kw 480y/277 volt emergency backup generators
- General office areas, conference rooms and laboratories consist of indirect/direct pendant luminaires with T5 HO fluorescent lamps

Jamison D. Morse :: Structural Option www.engr.psu.edu/ae/thesis/portfolios/jdm5017

Table of Contents

Acknowledgments

The author would like to extend special thanks and recognition to the following individuals, faculty, and members of the professional community for their generous support and aid in the completion of this thesis:

SmithGroup

Star Seismic

Schuff Steel

Thornton Tomasetti

Nintendo of America

The Pennsylvania State University
The entire Architectural Engineering faculty and staff, particularly Prof M. Kevin Parfitt, Prof Robert Holland, and especially my advisor Dr Andres Lepage for all his help and guidance

My entire family for their continual support, especially my mother, father, sister and grandmother

A special thanks to all of my friends, particularly my roommates Stephen Lumpp and David Sivin for enduring the process with me and keeping me sane

Executive Summary

This thesis evaluates the feasibility of constructing the Visteon Village Corporate Center in the suburban area of Orinda, California outside of San Francisco, rather than its current location outside of metro Detroit in Van Buren, Michigan. Due to the much higher seismic activity in the west coast area, the location change calls for a redesign of the structural system with special attention to the lateral force resisting system. The depth of this thesis centers around integrating a seismically sufficient structural system into the building that will adequately handle the design loading scenarios, with a focus on the seismically detailed connections. The breadth studies will cover the architectural changes that the project will incur upon installation of the new system, and the changes to the fabrication schedule and cost of the construction process based upon the system selection.

The current lateral load resisting system of the Visteon Village Corporate Center is composed of special steel moment frames with spans up to 40 feet. These frames utilize large steel sections and expensive connections to resist the critical design wind loading of the Detroit area. The critical load case in Orinda, California is due to seismic forces which were calculated to be much larger than the wind loads the frames were designed for making the current system inadequate and thus in need of a redesign. Due to efficiency and economy, the column grid was changed to provide shorter spans, and a buckling restrained braced frame (BRBF) system was integrated into the building. While the bracing elements themselves were more expensive, this was offset by the cheaper connections, quicker fabrication and erection times, and potential to minimize damage in the event of an earthquake.

The architecture breadth focuses on providing a functional plan layout when the BRBF system is integrated into the building. The change to the column grid the restriction of the clear spans in the braced bays needed to be addressed to ensure the flow of building would not suffer. This study shows that with minimal architectural change a feasible layout can be achieved. The construction process breadth study looks at the fabrication and costs associated with different framing types and connections. Since the project is design-build, the fabrication schedule of the structural elements can cause a huge impact on the construction process as adequate lead time has to be provided between the completion of the construction documents and the planned date of steel erection.

As a whole, this thesis aims to create the most efficient structural system to handle the critical loading, while minimizing the cost associated with the project. This report shows that the proposed system has met these goals, and is a feasible design if the project was ever constructed in an area with high seismic activity.

Introduction: Visteon Corporate Village Center

The Visteon Corporate Village Center is located in the Detroit metro area of Van Burin, MI. The facility is one of many office and laboratory buildings present on the corporate campus of the global automotive supplier. The campus is laid out and styled to provide a village type of atmosphere, complete with sidewalks and streetlights. All master planning, architecture and engineering of the campus and its various buildings was completed by the Detroit office of the SmithGroup.

The Visteon Corporate Village Center is five stories high, with the fifth story penthouse reaching a height of 72'-9" above grade, and has an overall size of 130,000 gross square feet. The building is a steel framed structure consisting of a composite steel decking system resisting gravity loading and a special steel moment frame system for lateral support. The majority of the building consists of 40'-0" x 20'-0" bays providing a large amount of floor area that is uninterrupted by column placement. Included in the Village Center building is a large cafeteria space and multiple public presentation spaces as well as a large amount of office areas on the upper floors.

Site and General Architecture

The Visteon Village Corporate Campus is currently located on a man-made lake in Van Buren, Michigan. The building being analyzed in this thesis is the Visteon Village Corporate Center, the center building of the complex. As previously mentioned, the buildings are laid out in a village format, with lit walkways and greenery in between.

Design Guides and Criteria

During the analysis of the lateral system used by the Visteon Corporate Village Center, the following design aids were used:

The 2006 International Building Code (IBC 2006)

Building Code Requirements for Structural Concrete 2008, American Concrete Institute (ACI 318-08)

Steel Construction Manual, 13th Edition, American Institute of Steel Construction (AISC)

Minimum Design Loads for Buildings and Other Structures 2005, American Society of Civil Engineers (ASCE 7-05)

Drift Criteria per the 2006 International Building Code

$D_{seismic} = 0.25h_{sx}$ (Allowable Story Drift)

The load cases used during this analysis were taken from section 1605 of the 2006 International Building Code. They included:

1.4D
1.2D + 1.6L + 0.5Lr
1.2D + 1.6Lr + (1.0L or 0.8W)
1.2D + 1.6W + 1.0L + 0.5Lr
1.2D + 1.0E +1.0L
0.9D + 1.6W
0.9D + 1.0E

These combinations were analyzed in different directions and applied to various eccentricities during the computer analysis. There were 122 LRFD load combinations that were generated and analyzed. Due to time constraints and simplicity, snow loading was not included in this analysis.

	Dead	Live
Roof	30 psf	30 psf
Fifth/PH	94 psf	150 psf
Fourth	92 psf	100 psf
Third	92 psf	100 psf
Second	92 psf	100 psf
First	92 psf	100 psf

Existing Framing System

Foundation:

All of the foundation systems for the Visteon Village Corporate Center were designed based upon the findings of a geotechnical investigation performed by Somat Engineering on October 14, 2002**.** There is a deep foundation system to support all building columns, walls, grade beams and other foundation elements. The deep foundation elements are comprised of friction steel H-piles in native medium compact to compact sand. All H-piles consist of 75 foot long HP12x84 sections with concrete pile caps and are of ASTM A992 steel (Fy = 50 ksi). The number of piles for each foundation element range from 1 to 7 providing capacities of 100 kips to 1050 kips respectively. The concrete pile caps are of reinforced concrete construction with their top elevation at a minimum depth of 3'-6" below finished grade as to prevent frost heave. The dimensions of the caps range from 3'x3' for a single H-pile element up to 13'x11'-8" for a 7 H-pile element. All concrete used in the foundation systems has a minimum compressive strength of 3000 psi.

Columns:

All of the columns of the building are composed of structural steel. The main column system is made up of ASTM A992 wide flange shapes ranging in size from W14x43 to W14x311. Typically, these columns rest upon the deep foundation system and extend 72 feet to the penthouse level with a column splice at an elevation of 52 feet (falling within the third story). These multistory columns are also part of the special moment frame system that resists lateral loading.

Floor and Roof Framing System:

The typical framing system for the Visteon Village Corporate Center is composed of structural steel composite beams and girders. The supported floor consists of 40 foot long ASTM A992 wide flange shapes spanning a column free space. The typical bay for each floor is 40'x20' with wide flange beams spaced at 10' on center supporting 3" composite metal floor deck with 3-1/4" light weight concrete fill providing a total slab depth of 6-1/4".

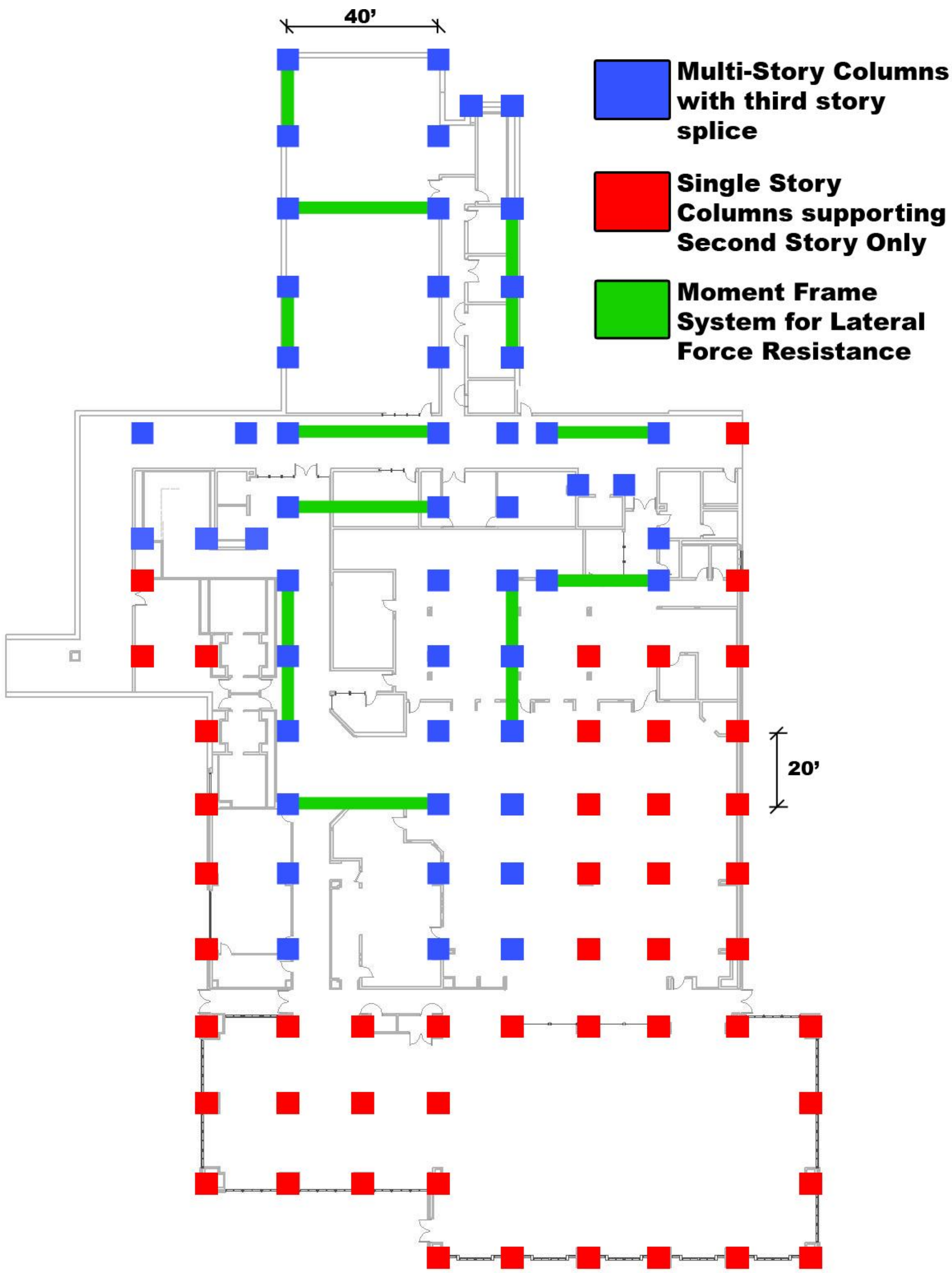


Figure 1 – Layout of Existing Structural System

Lateral System:

All lateral loads caused by wind and seismic forces are resisted by special steel moment frames. There are five moment frames running in the North/South direction of analysis and six moment frames running in the East/West direction of analysis. Each moment frame consists of multistory wide flange columns and wide flange beams. The columns are spliced at the third story, with the top three stories consisting of a W14x211 section being supported by a W14x311 extending through the lower two stories.

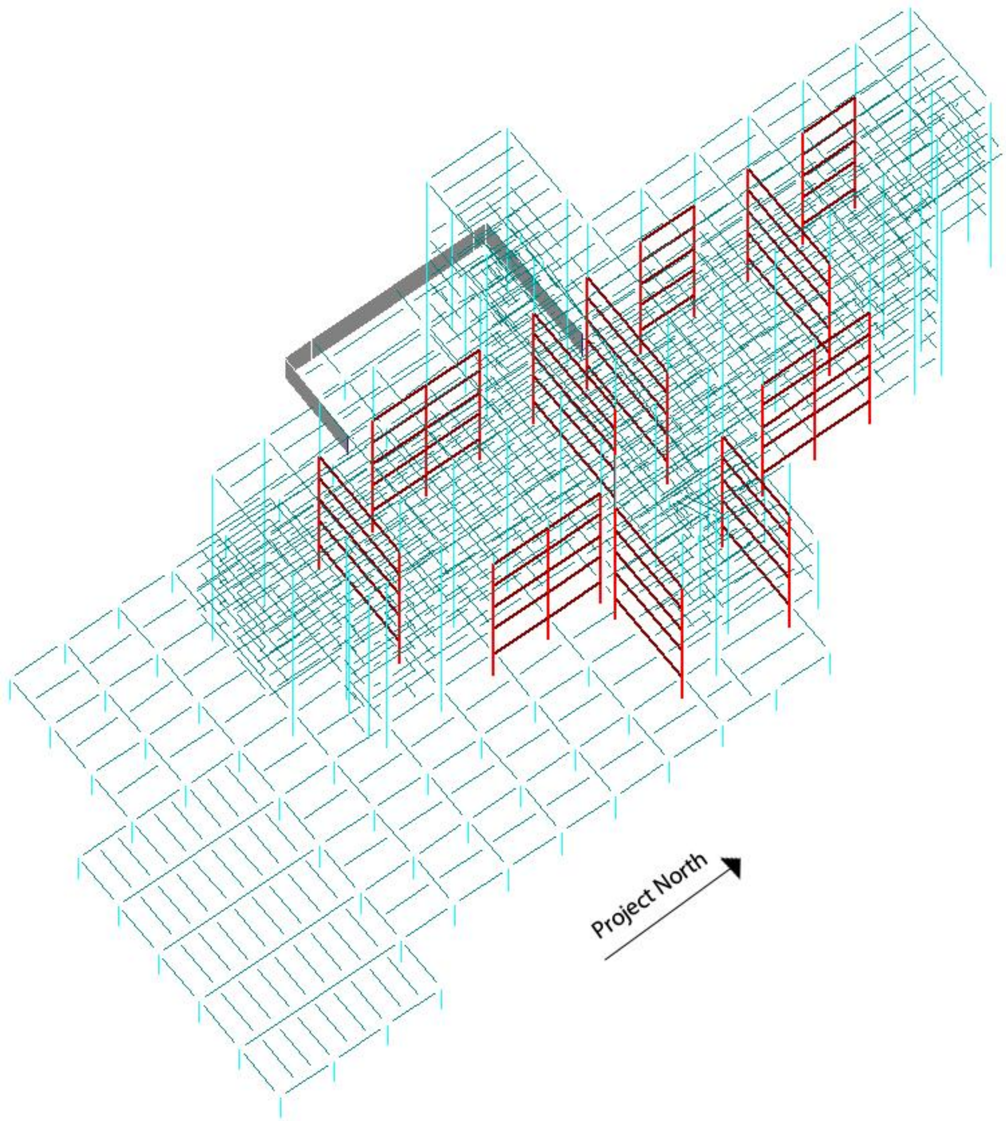


Figure 2 – Locations of Existing Special Moment Frames

Thesis Overview

Problem Statement

The current design of the Visteon Village Corporate Center's lateral load resisting system is comprised of special steel moment frames. While it is important to note that during the third technical report this system was deemed adequate, the additional lateral loading due to the relocation to a high seismic region makes the system not only inefficient but very uneconomical. The new loading caused extremely large story drifts, some of which were not allowable by code and the connections and steel sections that were necessary to ensure proper transfer and handling of the large moments and forces were extremely expensive in terms of both materials and fabrication. With these topics in mind, there was a need for a new lateral framing system that would optimize performance using economical steel sections and construction techniques, while maximizing the capacity of the lateral load resistance of the structure. This system must also be feasibly integrated into the architecture of the building to ensure the layout still has its intended functionality.

Problem Solution

The issues of economical design and drift optimization were the main areas that needed to be addressed. This lead to the notion of a complete redesign of the lateral load resisting system of the Visteon Village Corporate Center using braced frame systems. Multiple concentric and eccentric braced framing schemes were assessed for their feasibility of application to the project. Using a braced framing system to handle the lateral loading provided a substantially more efficient way to keep story drifts under control, specifically on a floor by floor basis. These connections also required significantly less field welding, which will save on labor and material costs during the construction phase. The design of proper connections was looked at closely as part of the depth of this thesis. The advantages and disadvantages the framing changes have on the current construction process were thoroughly investigated as a breadth study of this thesis. By changing the current moment frame system to braced frames, the open layout of the floor plan was slightly compromised as the frames interfered with the previously open spans. Movement of the frames to accommodate the architectural flow of the building was performed and assessed structurally, and a redesign of the floor plan layout to accommodate the framing system was performed. These studies encompass an architectural breadth to ensure unity between the structural and functional design. Once all analyses of the proposed and current systems were completed, a comparison was performed to determine the system's feasibility, cost, and efficiency.

Breadth Studies

In addition to the main structural redesign of the lateral framing system of the Visteon Village Corporate Center, two breadth studies were performed. The first study analyzes the effects that the framing changes have on the construction process. The second study focuses on the architectural accommodations that will be required to integrate the new framing system into the building.

The construction study focuses on the benefits and drawbacks the different types of framing systems provide to the construction process in comparison with the existing moment frame system as well as with each other. This study covers the topics of cost, installation, estimating, and scheduling issues. As this project is of design-build nature, the fabrication stages of the connections were looked at extensively as this period of time provides the greatest impact on the construction process.

The architecture study focuses on the design issues caused by the implementation of the new braced frame system. Keeping the lateral resisting frames in their current locations would mean that multiple spans along the column grid which are currently open would have some sort of bracing interfering with the layout in the new design. A redesign of the floor plan was in order to show that a feasible and functional change of the architectural layout can be achieved. Another area of this study was changing the locations of the lateral resisting members to optimize the functionality and convenience of the architectural plan, which required additional structural analysis.

Thesis Depth Study:
Seismically Detailed Lateral System and Connections

The main concentration of this thesis revolves around the complete redesign of the lateral load resisting system. As discussed in the previous technical reports, the existing system is comprised of eleven special steel moment frames: six in the east-west direction and five in the north-south direction. This framing method was selected for the original design because of the versatility it provided for the architectural layout with its large uninterrupted column to column spaces. Four of the six frames in the East-West orientation spanned a length of forty feet, minimizing restrictions of the architectural flow of the building. As investigated in the third technical report of this thesis, the framing system called for very large steel sections and expensive connections, but adequately handled the lateral forces applied by the critical wind loading scenario.

When the project was relocated to Orinda, California, a suburb of San Francisco, the loading and framing system had to be reanalyzed to ensure its adequacy in handling the new critical loading. According to ASCE 7-05, the basic wind speed for this area to be used in wind load calculation was V=85 mph. As the current system was designed for V=90 mph, the code value for Van Buren, Michigan, it could be easily seen that the existing special moment frame system would be able to handle the critical wind loading of Orinda, California. The dramatic changes in loading occurred when the seismic case was analyzed, as the selected site in California is a region with high seismic activity.

The area code of 94563 for Orinda, CA was plugged into the USGS provided “Seismic Hazard Curves and Uniform Hazardous Response Spectra” program to obtain S_s and S_1 values. A seismic analysis was then performed using criteria from Chapter 12 of ASCE 7-05. The results of this analysis are included in the appendix of this report. As the new critical lateral seismic load case was nearly eight times larger than the critical wind loading of the original Van Buren location, it was easily proven that the current system was inadequate to handle the loading and a redesign was in order.

The first step in this process was to assess whether the current system could be modified in an efficient and economical way to resist the critical seismic loading. Several iterations were performed using RAM Frame and SAP2000 software to determine the steel frame sections required. The resulting system used incredibly large and highly economical steel members, which still could not satisfy the seismic drift criteria set forth by ASCE 7-05. It was concluded that to provide the most efficient and economical lateral system, the layout had to be modified.

Under the loading conditions, the column spacing of forty feet along the north-south length of the building (see Fig. 1) was looked into as a potential area to be modified. Special steel moment frames spanning that length were previously deemed extremely difficult and uneconomical, and the geometry made it difficult for braced frames to work efficiently as well. Concentric braced framing would cause the brace element to resist much larger than 70% of the total horizontal force, which made unable to meet the requirements of Seismic Provisions Section 13.2c as set forth by the AISC Seismic Design Manual. Eccentric braced frames were analyzed next, but required very large steel sections to accommodate the large span and were also deemed inefficient and uneconomical. At this point the decision was made to modify the column grid of the building along the north-south axis, providing more manageable spans to integrate the lateral bracing system.

The long axis (north-south) of the building is comprised of three column gridlines spaced forty feet and twenty feet apart respectively. The center gridline of the three was moved ten feet to the west, providing two equal spans of thirty feet each. Due to the shift of the column grid the gravity framing had to be assessed and redesigned. Due to the symmetry that the new layout provided along the north-south axis and the fact that the loading was uniform across both bays, the gravity framing was identical for both bays. This provided a potentially more efficient and economical gravity load resisting system than the previous design as the repetitive nature could reduce erection times and lower fabrication costs. The existing gravity framing designs as well as the modified gravity framing designs can be found in the appendix of this report.

Many lateral framing systems were debated upon for integration into the Visteon Village Corporate Center and preliminary analyses were performed. The large story forces from the critical seismic load case made story drifts a problematic issue. It was quickly seen that even with the reduced span of thirty feet, a moment frame system would not be an efficient choice. To satisfy the drift criteria as set forth by ASCE 7-05, the steel sections used for the framing had to be very large, or a large number of moment frames had to be used. While increasing the number of moment frames made the required steel sections smaller, this also increased the number of connections required which effectively cancelled out any cost savings of the lesser sections. The moment connections required multiple doubler plates at the panel zones making them an expensive option. These trial analyses lead to the conclusion that using some form of braced framing would be the best option to satisfy all criteria of the lateral framing system.

Upon deciding which form of bracing to use special concentrically braced frames (SCBFs) and eccentrically braced frames (EBFs) were compared. While EBFs provided more leeway for architectural design, they are not as efficient as a SCBF system for a few reasons. The symmetry of concentrically braced frames provided a more efficient system as identical members and connections were used for both sides of the frame. Also, the geometry of the bracing provided the most efficient load transfer to the braces, and minimized any extra loading the beam members had to endure due to the lateral forces. For these reasons, it was initially decided that SCBFs were to be used for the new design of the lateral load resisting system.

Since the new system was to be designed with a modified column grid and braced frames, the feasibility of integration without compromising the architecture had to be assessed. This topic is covered extensively in the architecture breadth study section of this report. The study was completed concurrently with the preliminary SCBF system design to ensure that the design would work with the project prior to further analysis being performed. As shown in the study, the SCBF system can be successfully integrated in the building while maintaining architectural functionality, as long as the flow restrictions caused by the bracing was minimized. For this reason the system was designed with three frames oriented in the east-west direction, and four frames in the north-south direction. The frames were all positioned to maximize their efficiency while minimizing required architectural changes. The layout of the new framing system can be seen below (Fig. 3).

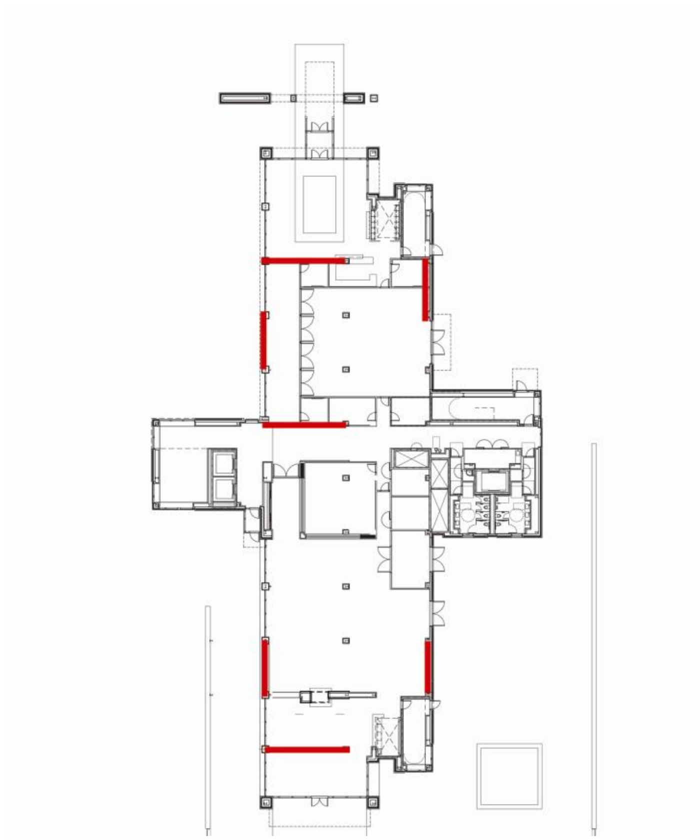


Figure 3 – Locations of Braced Frames

Once the layout of the framing system was established, the designing of the framing elements began. First, the critical seismic load case was established, and the base shear was calculated using an R=6 (SCBF) as dictated by ASCE 7-05. A summary of the story forces can be seen in (Fig. 4) below, and a summary of how the story forces were determined can be found in the appendix.

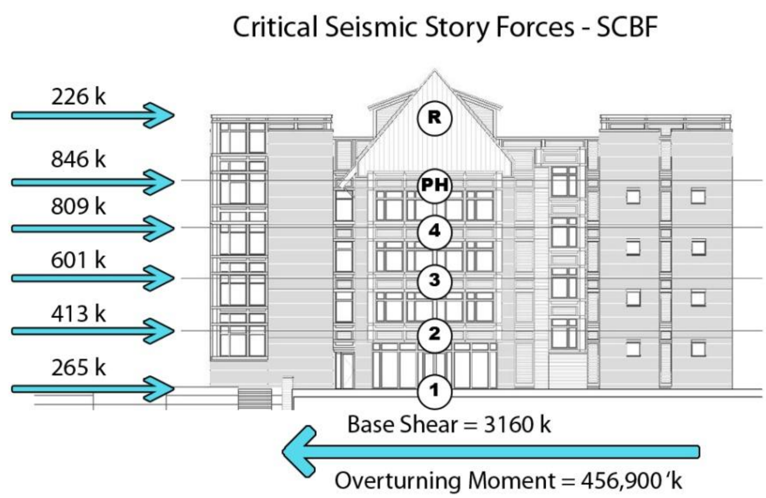


Figure 5 – SCBF Story Forces

A two story cross-bracing scheme was chosen to minimize the uneven loading on the beams by transferring all horizontal forces into the braces. Since the lateral framing system is not an even number of stories tall at five, the top brace was modeled as a chevron brace and the beam was designed to handle the uneven loading. All of the supporting calculations for member selection can be found in the appendix. The bracing scheme and the steel members that were selected can be seen in the image on the following page (Fig. 5).

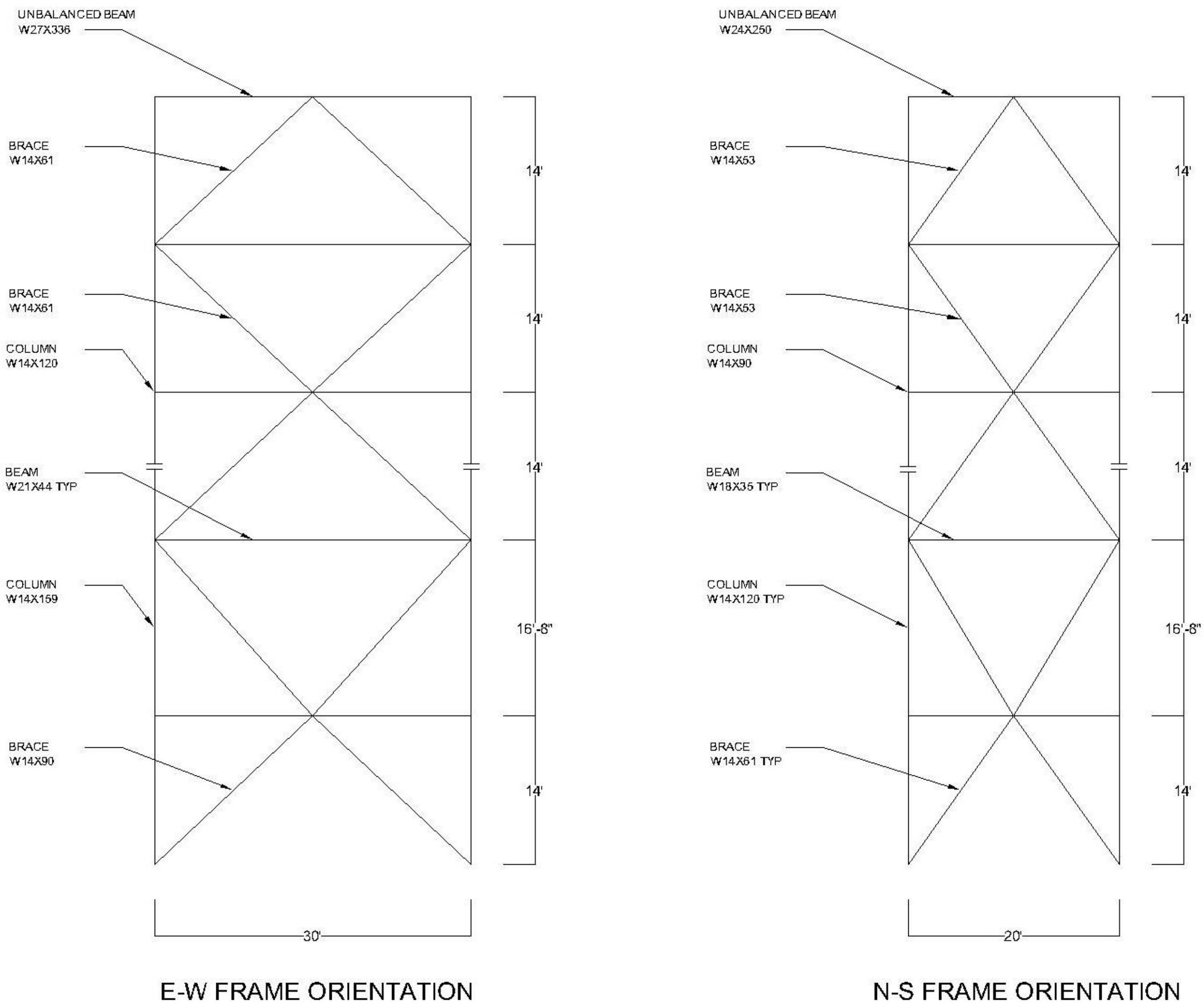


Figure 5 – SCBF Design

Once the SCBF framing and bracing was determined, a seismically detailed connection design was undertaken as specified by the AISC Seismic Design Manual to ensure that the SCBF connection could adequately handle the loading while maintaining R=6. All of the supporting calculations for the connection can be found in the appendix of this report. A detail of the final connection design can be seen below (Fig. 6).

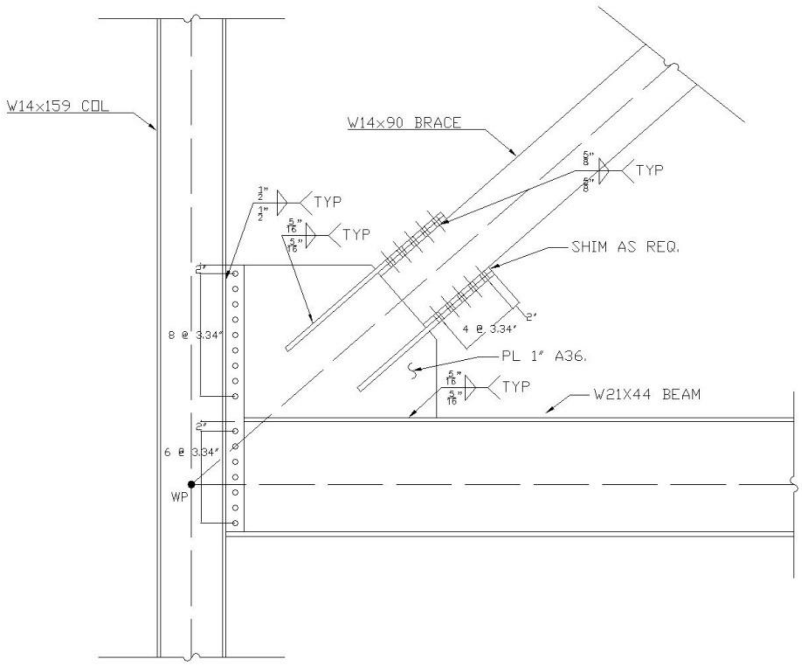


Figure 6 - SCBF Connection Design

The foundation system of the Visteon Village Corporate Center also had to be reassessed to ensure its adequacy in handling the loads caused by the overturning moment from the heavy seismic loading. Looking at soil property maps provided by the United States Geological Survey it was determined that large deposits of limestone were prevalent in the building's new region. Using this information, a deep foundation system was designed using HP12x84 steel piles and concrete pile caps. The calculations for this system can be found in the appendix, and the detail can be seen on the following page. (Fig. 7)

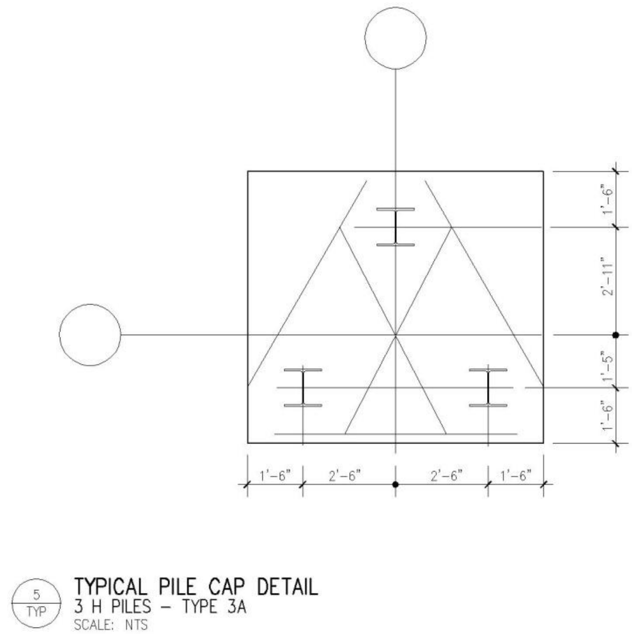


Figure 7 – Foundation Design

A direct shear, torsional shear, and drift analysis was performed on the bracing scheme using Microsoft Excel and SAP2000 software. The results show that the system was adequate in all of these categories, and the summary of these tests can be viewed in the appendix of this report. Now that an efficient and economical lateral system had been initially established and designed, similar systems were researched to see if any improvements in performance and costs could be obtained. The product of this research showed that this building could benefit by using a buckling restrained braced frame (BRBF) system in lieu of the designed SCBF system in place in both performance and economy.

The unique characteristic of a BRBF system is that the bracing elements yield inelastically in both compression and tension. This is accomplished by encasing the steel core within the bracing element, in turn creating a limit of the core's buckling. Axial loads are handled by the steel core while the casing acts as a buckling restraining mechanism by resisting overall brace buckling and restraining high-mode steel core buckling or rippling. There are a few manufacturers that produce these systems, but the PowerCat design by Star Seismic was chosen for use in this thesis.

Using this system, the braces were configured in the same way as the previously designed SCBF system. The base shear of the critical load case was decreased by using this system, as ASCE 7-05 provides that an R=8 can be used which increased from the previous system's R=6 value. While the special bracing members are definitely more expensive than a standard wide-flange shape that the SCBF design required, the savings in materials in both the columns and connections more than make up for the bracing expense. The total amount of steel used by this system decreased by about eighteen tons when compared to a SCBF system, equating to a cost savings of roughly $65,000. The following figures show the story forces and brace designs using a BRBF system. (Fig 8 and 9)

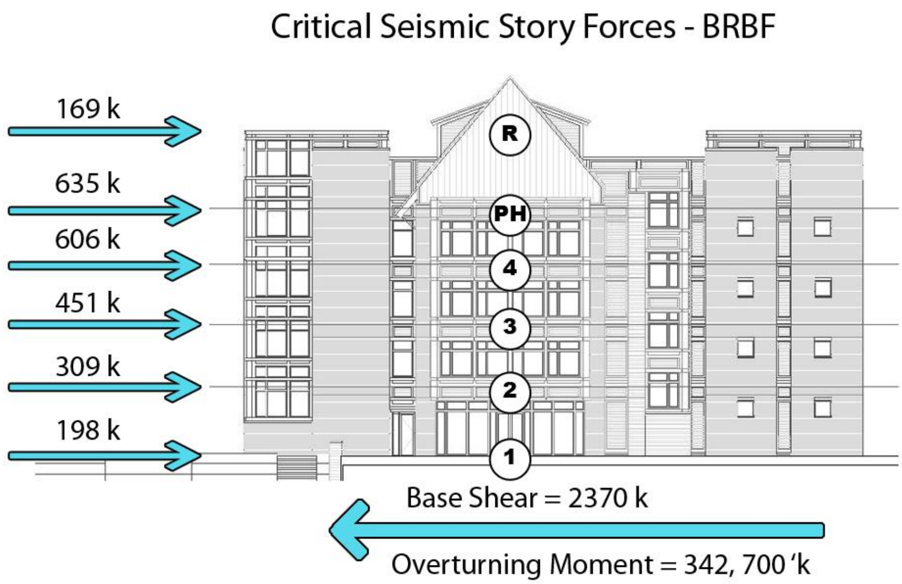


Figure 8 – BRBF Story Forces

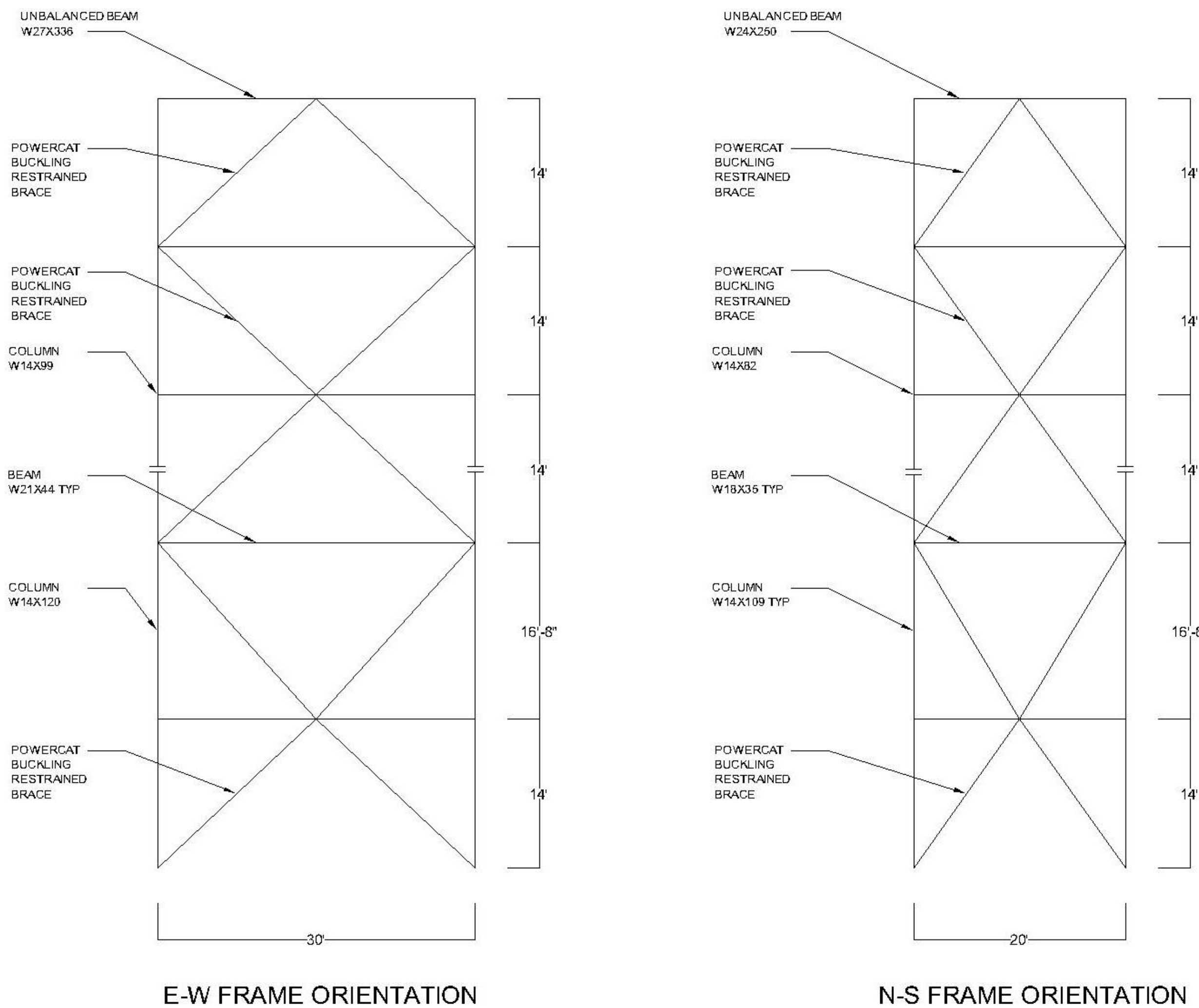


Figure 9 – BRBF Design

An independent study on BRBF systems was performed by Dasse Design Inc, now a part of Thornton Tomasetti, on March 7, 2007. This study encompassed a cost comparison between a SCBF system and a BRBF system. This study concludes that in areas of high seismic activity, BRBFs become a significantly more economical choice in buildings of three stories or higher, providing greater savings as the number of floors increases. Dasse Design Inc provided a graph to illustrating the general relationship between the number of stories a building has and the total cost of the lateral force resisting system which can be seen on the following page. (Fig. 10)

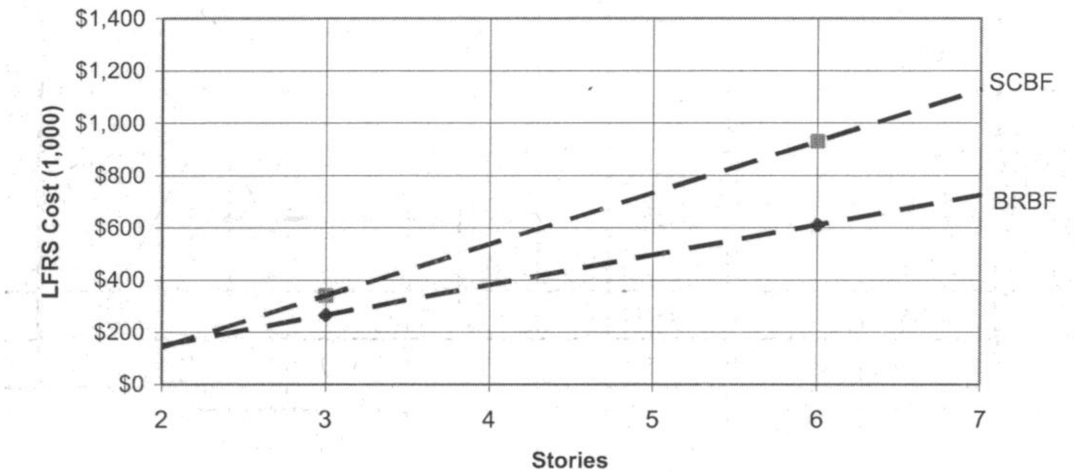


Figure 10 – Cost Relative to Building Height

Noted in the study is the substantial savings on seismically detailed connections the BRBF system provides over an R=6 bolted connection that was detailed previously. Specifically looking at the PowerCat series of braced frames by Seismic Star, the connection consists of gusset plates of standard orthogonal shapes. No stiffeners are required for this system as they have a patented design that builds the stability of the brace into the brace itself. The braces are connected by one specially designed pin at each end which saves time in the erection process. The amount of welding in the connection is much smaller than a typical SCBF connection and only simple fillet welds are necessary. Standard details of the PowerCat's connections as provided by Seismic Star can be seen below. (Fig. 11 and 12)

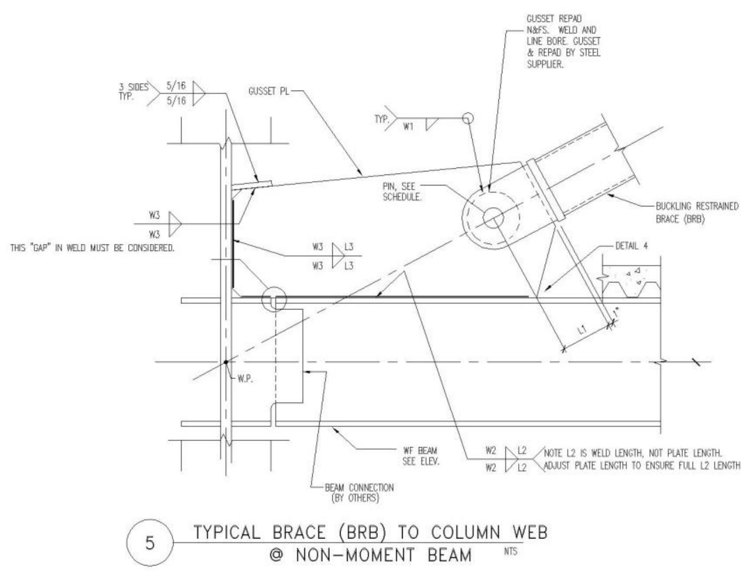


Figure 11 – PowerCat Brace to Column Web Detail

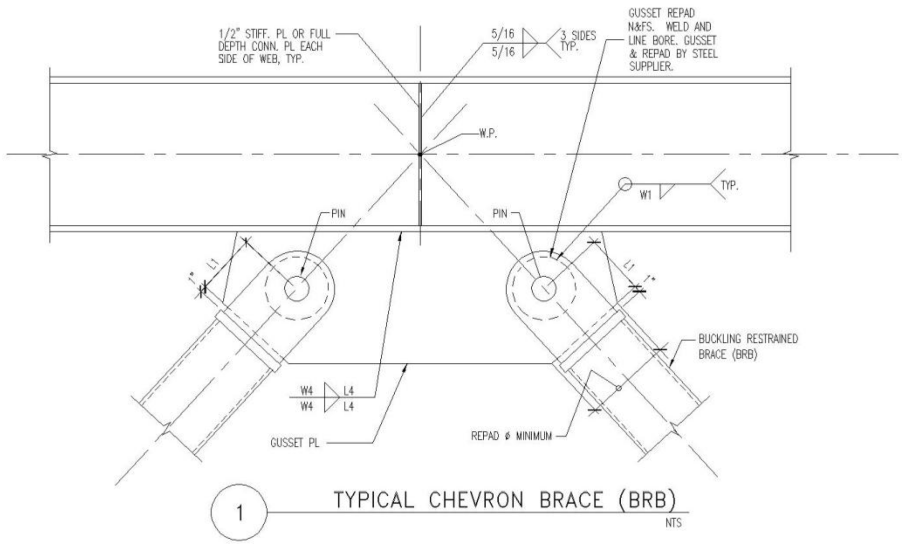


Figure 12 – PowerCat Brace to Beam Detail

Another advantage to using the PowerCat BRBF system is the savings that can be had after the occurrence of an earthquake. The seismic energy is almost completely dissipated in the braces themselves causing the beams and columns not to deform. This can minimize the overall damage of the building as beams and columns can be very expensive and tedious to replace. This property of BRBFs could also potentially minimize the damage done to non-structural elements of the building such as mechanical systems, partitions, and walls. The PowerCat's unique pin connection design also allows for the braces to be removed, analyzed, and replaced if necessary. This process is much less difficult than replacing the beams and columns, and essentially having to retrofit an entirely new lateral system after a seismic event.

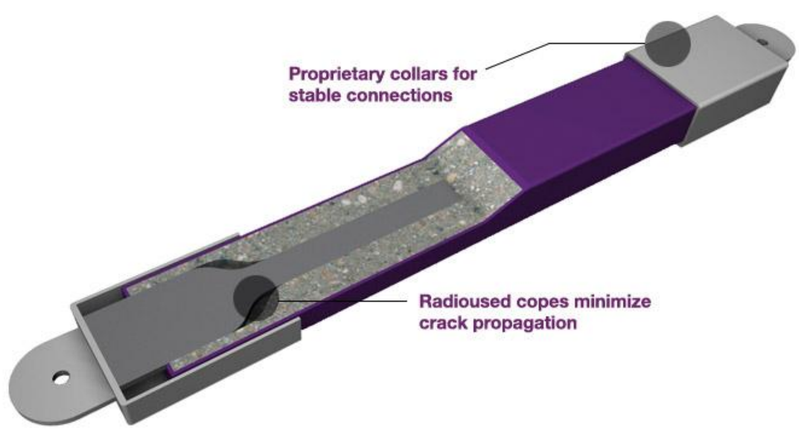


Figure 13 – PowerCat Brace

Depth Study Conclusion

It was determined that the Visteon Village Corporate Center's lateral force resisting system should be composed of a series of buckling restrained concentrically braced frames. The process involved optimizing the column grid placement and the preliminary analyses of a variety of different systems. The BRBF system provides substantial performance under the determined critical seismic loading case in Orinda, California. This scheme also provides savings at both the fabrication and erection stages of the construction process, as well as potential savings post installation in the event that a damaging earthquake was to occur. Overall, it is believed that this is the best system to be used for the Visteon Village Corporate Center as it satisfies all of the criteria posed by the project relocation as efficiently and economically as possible.

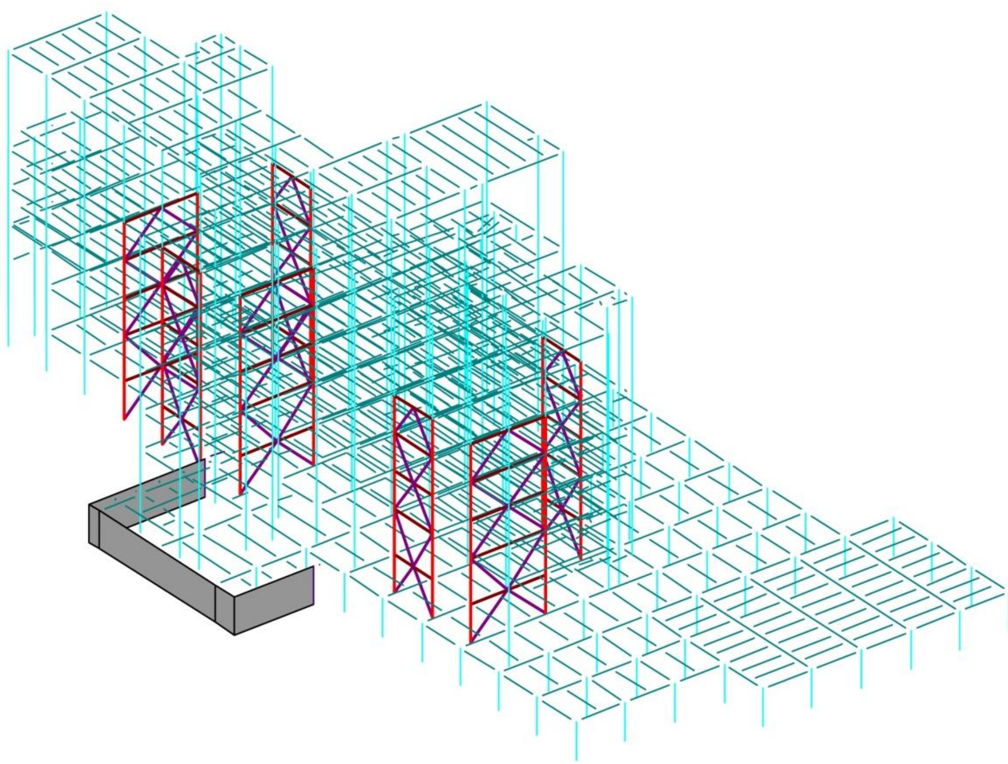


Figure 14 – Locations of BRBF System

Architecture Breadth Study

The existing structural system of the Visteon Village Corporate Center allows for a very open architectural floor plan. This is accomplished by the wide forty foot column grid spacing along the north-south axis of the building and the fact that the large bays are not interrupted by bracing as a special moment frame system was used to handle all of the lateral loading. In the depth study area of this report, it was deemed necessary to adjust the column grid spacing from forty feet down to thirty feet to allow a more efficient lateral system to be designed. Obviously, this change would have an impact on the architectural layout of the building, and had to be assessed to ensure the plan's functionality was not compromised. Once the most efficient lateral bracing system was decided to be concentrically braced frames, it also had to be proven that this system could successfully be integrated into the building without disrupting the architectural flow. The minimal design changes that were necessary are shown and explained below.

Here the configuration of the buckling restrained braced frame system is shown in the E-W direction. As the braces are restrictive in comparison to a moment frame system, it had to be assessed whether large enough openings could be achieved through the braced areas to satisfy IBC 2006. The smaller opening shown is 6' wide by 8' tall. This is the rough opening dimension for the double egress doors that are already being used as standard on this project. The large opening under the chevron braced floors is 12' wide by 8' tall, and can be used as a wide passageway where applicable.

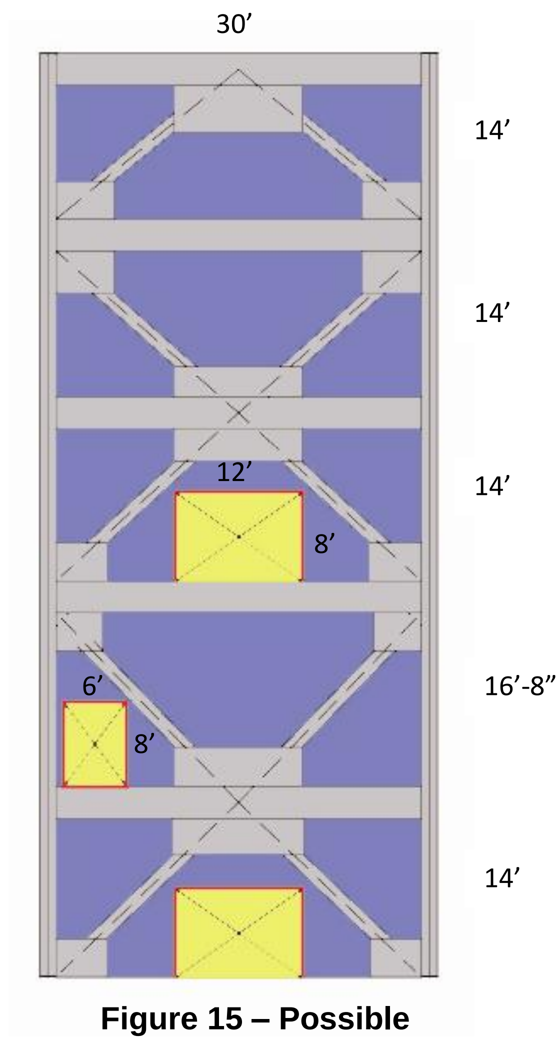
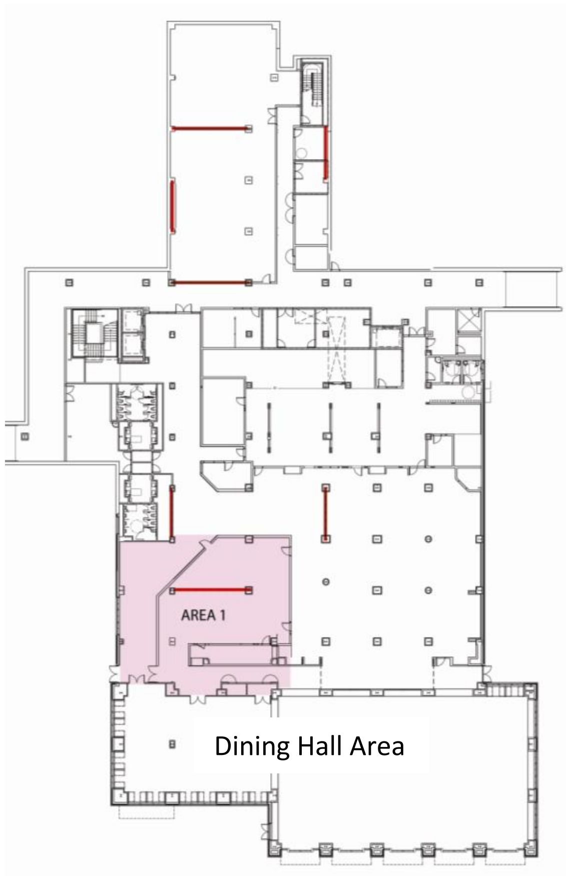
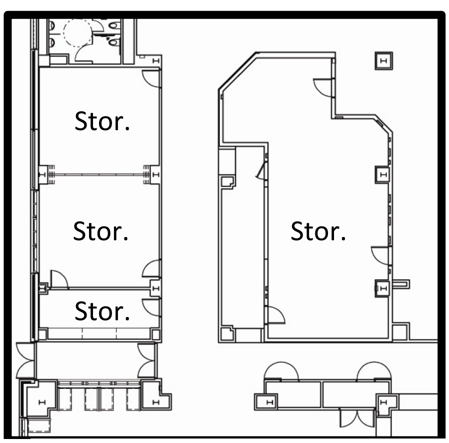


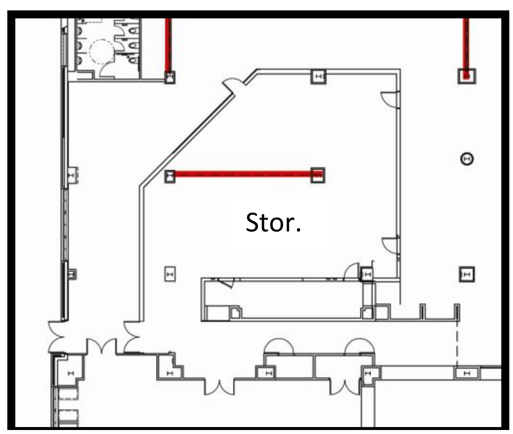
Figure 15 – Possible Openings in Braced Frame



Ground Floor Modified Plan

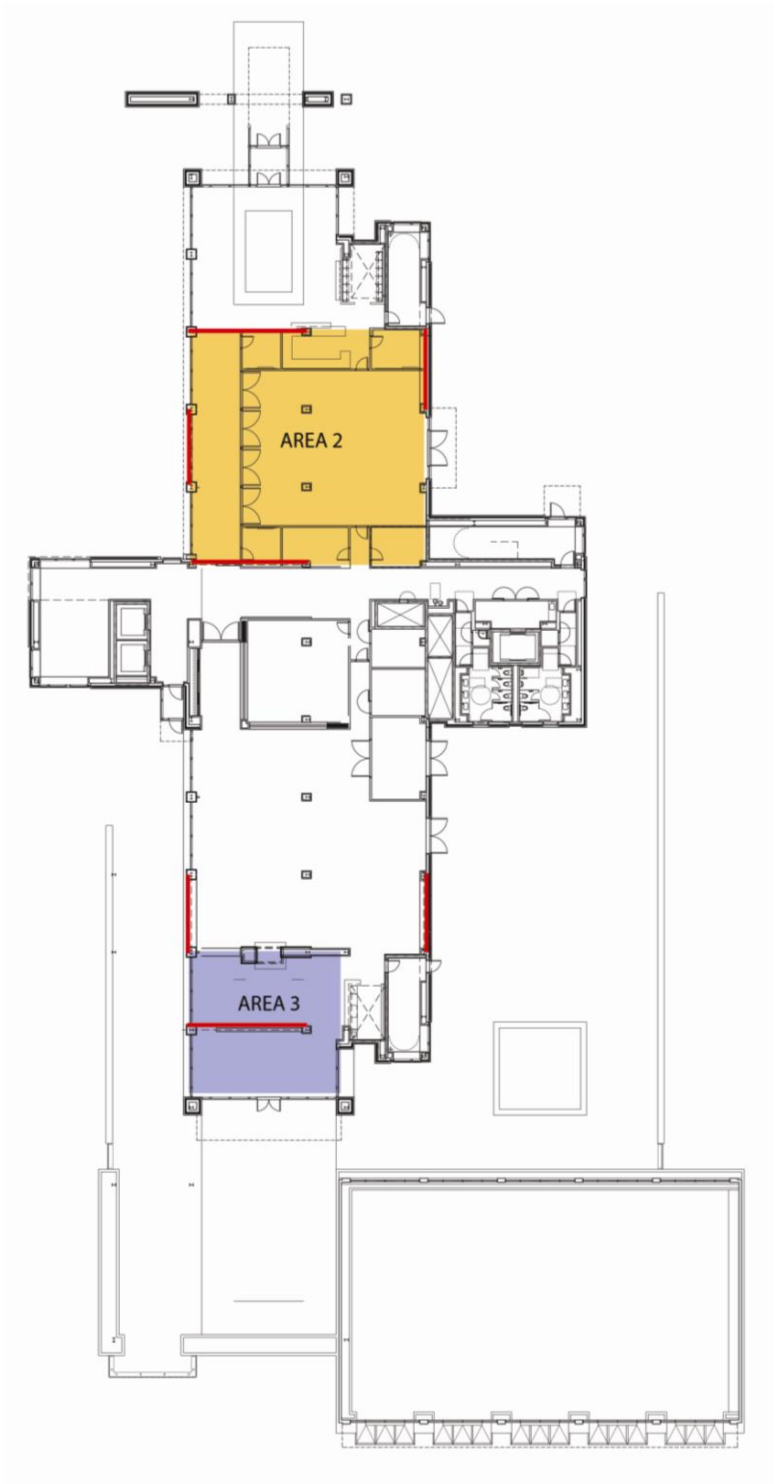


Area 1 – Original Design

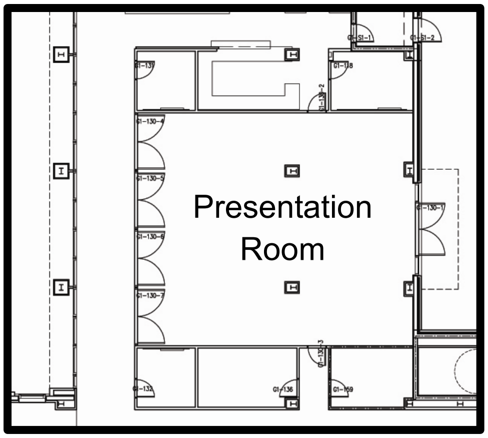


Area 1 – New Design

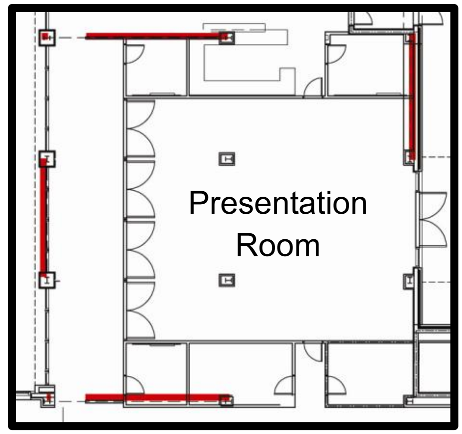
The most significant difference to the ground floor plan is near the entrance to the dining hall area. The original design showed storage areas on both sides of the hallway. The new design rerouted the hall to the exterior of the building and combined the storage spaces so the brace would not interfere with the flow of the building. The original design had 750 ft^2 of storage, while the new design has slightly less at 710 ft^2.



First Floor Modified Plan

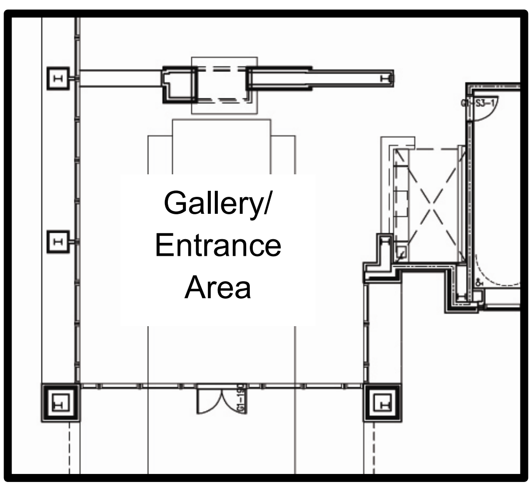


Area 2 – Original Design

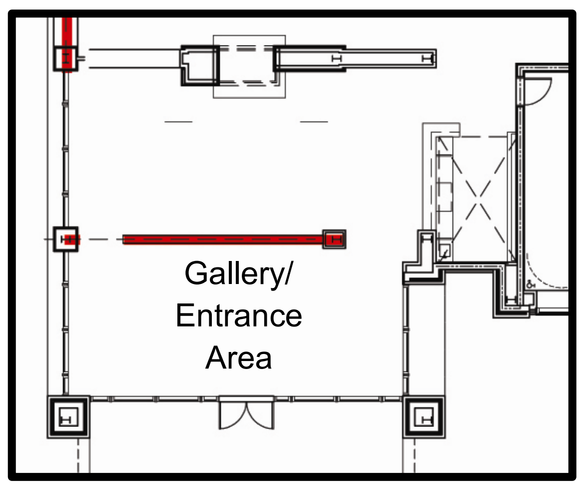


Area 2 – New Design

As the braced frames crossed the hallway on the first floor outside of the presentation room, this area had to be looked at to make sure that the primary flow path through the building was not compromised. As seen in Figure 15 earlier in this section, a 6' wide opening can be created on each side of the 12' wide hallway. This may even be beneficial to the flow as it naturally draws passers-by to the exterior side of the hallway, as not to interfere with people entering and exiting the presentation area. The movement of the column placement inside the presentation room caused no significant changes to the architectural design.

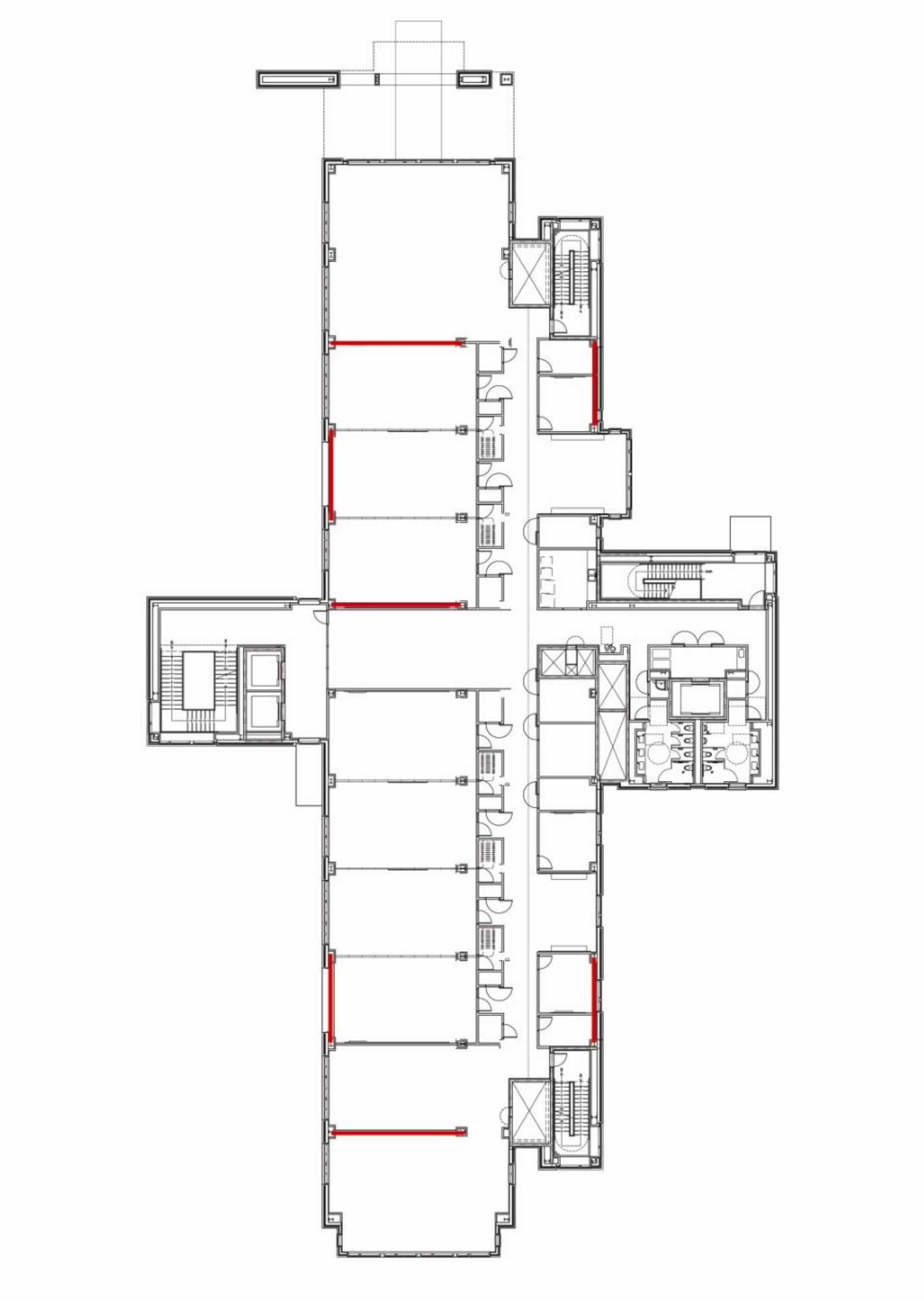


Area 3 – Original Design



Area 3 – New Design

The gallery/entrance space on floor one is one of the few areas where a brace is placed in a previously completely open area, rather than in a wall. The addition of a wall containing the brace can potentially enhance this space because it adds more wall area in which to display projects. It also helps to control the flow in this area in case security becomes an issue.



Office Floor Modified Plan

The second through fourth floors of the Visteon Village Corporate Center consist of offices and are of identical construction, simplifying the design. All of the brace locations corresponded to an existing wall location, so no architectural changes were needed to integrate the system.

West Elevation

The shaded areas represent the fenestration on the façade that will be affected by the braced frame locations. Since braces are located along the exterior wall in these areas and hidden inside a wall, these windows cannot be used. The elimination of the window elements causes the façade to look unbalanced and detracts from the aesthetics. For this reason a spandrel glass faux window system is being proposed. This ensures that the overall architectural style of the exterior is not compromised as no visual change will be noticeable from the outside.

Construction Process Breadth Study

Since there were various systems being considered for the lateral force resisting system of the Visteon Village Corporate center, it was important to keep in mind how each option would affect the schedule and cost of the construction process. A generalized comparison study was completed focusing on various framing systems and their connections and what their impacts were in terms of time and cost of their fabrication and erection.

The costs and times of fabrication usually depend heavily on the equipment and expertise of the steel fabricator. Due to this fact, precise numbers were not used and a relative study was performed between the connections. A steel fabricator was chosen that was local to the Orinda, CA area so that the information collected would be a realistic representation of construction data in the chosen region of the project's relocation. Schuff Steel was selected, as they are a nationwide steel fabrication company with many locations local to the proposed site. The chart below shows a comparison of the connection types and their relative times to fabricate and erect, as well as relative overall cost. (1=Most, 5=Least)

	Fabrication Cost ($)	Erection Cost ($)	Overall Cost ($)	R
Intermediate Moment Frame (Bolted End Plate)	3	2	1	4.5
Special Moment Frames (Reduced Beam Section)	4	1	2	8
Special Concentric Braced Frame (Wide Flange)	1	3	3	6
Special Concentric Braced Frame (HSS)	2	4	4	6
Buckling Restrained Braced Frame	5	5	5	8

The two moment frame connections were found to be the most expensive connections. This is mostly due to the amount of field welding required in the erection process. A moment frame system was found to be an inefficient option for the lateral force resisting system of the Visteon Village Corporate Center, and this data shows that it is an uneconomical choice as well.

The special concentric braced frame connection was looked at using two different bracing elements: a wide flange and a square HSS. The wide flange bracing scheme came out to be higher for a few reasons. First, the WF connection would require gusset stiffeners which the HSS connection would not need. Also, the amount of material needed and bolts required would go up, causing the overall fabrication time to increase. A rough estimation by Schuff Steel indicated that designing this connection for an HSS shape rather that a wide flange shape would save about $250 dollars and about three hours of fabrication time per brace end. The drawback of using an HSS system is that the steel is generally 42 ksi instead of the 50 ksi steel used to fabricate wide flange members. The reduction in strength would cause the overall brace and framing members to be composed of heavier shapes than a wide flange braced system in order to handle the loading.

The buckling restrained braced frame connections came out to be the most economical. The material required is minimized as all of the stability is provided by the brace itself. Gusset stiffeners are not required with this connection and the amount of welding necessary also decreases greatly when compared to a special concentric braced frame system. The erection of this system is also very quick and cheap as only one specially designed pin needs to be installed at each end of the brace upon installation. It must be noted that the bracing members used in this system are much more expensive than a wide flange member used in a SCBF system. While this may be the case, the savings on connection time and cost, as well as the significant reduction of column sizes required make this system an economical choice.

It was concluded that the different connection fabrication and erection times do not make a huge difference on the overall construction schedule. Most of the decisions are solely based upon economy and structural adequacy. While the BRBF system uses much more expensive braces than a typical SCBF system, its savings in connection and fabrication costs of connections as well as the overall reduction of steel for the framing system make this system an economical choice.

Conclusion

The overall goal of this thesis was to evaluate the feasibility of constructing the Visteon Village Corporate Center in the suburban area of Orinda, California outside of San Francisco, rather than its current location outside of metro Detroit in Van Buren, Michigan. The structural system of the building, specifically the lateral force resisting system, had to be redesigned to handle the large loads caused by the high seismic activity in this area. In doing so, it was determined that a concentrically braced frame system was the most efficient and economical choice of the options assessed. Upon further research, a buckling restrained braced frame system was used instead of a special concentric braced frame for reasons of economy and efficiency.

The architecture breadth ensured that the integration of the redesigned lateral system would not compromise the integrity of the architectural functionality. It was shown that through moderate reconfigurations of the plan layout and exterior, it was feasible to mesh the BRBF system into the design of the building's architecture. This assessment leads to the recommendation of the BRBF system for use as the building's lateral force resisting system.

The construction process breadth study focused on the effects that different framing connections would have on the overall construction cost and schedule. It was concluded that the fabrication and erection times did not cause a significant impact on the overall construction schedule. The connections for the BRBF came out to be the most economical as expected, as this was one of the advantages to this system in comparison to a SCBF system. This study reinforces the fact that a BRBF system is the best choice for this project.

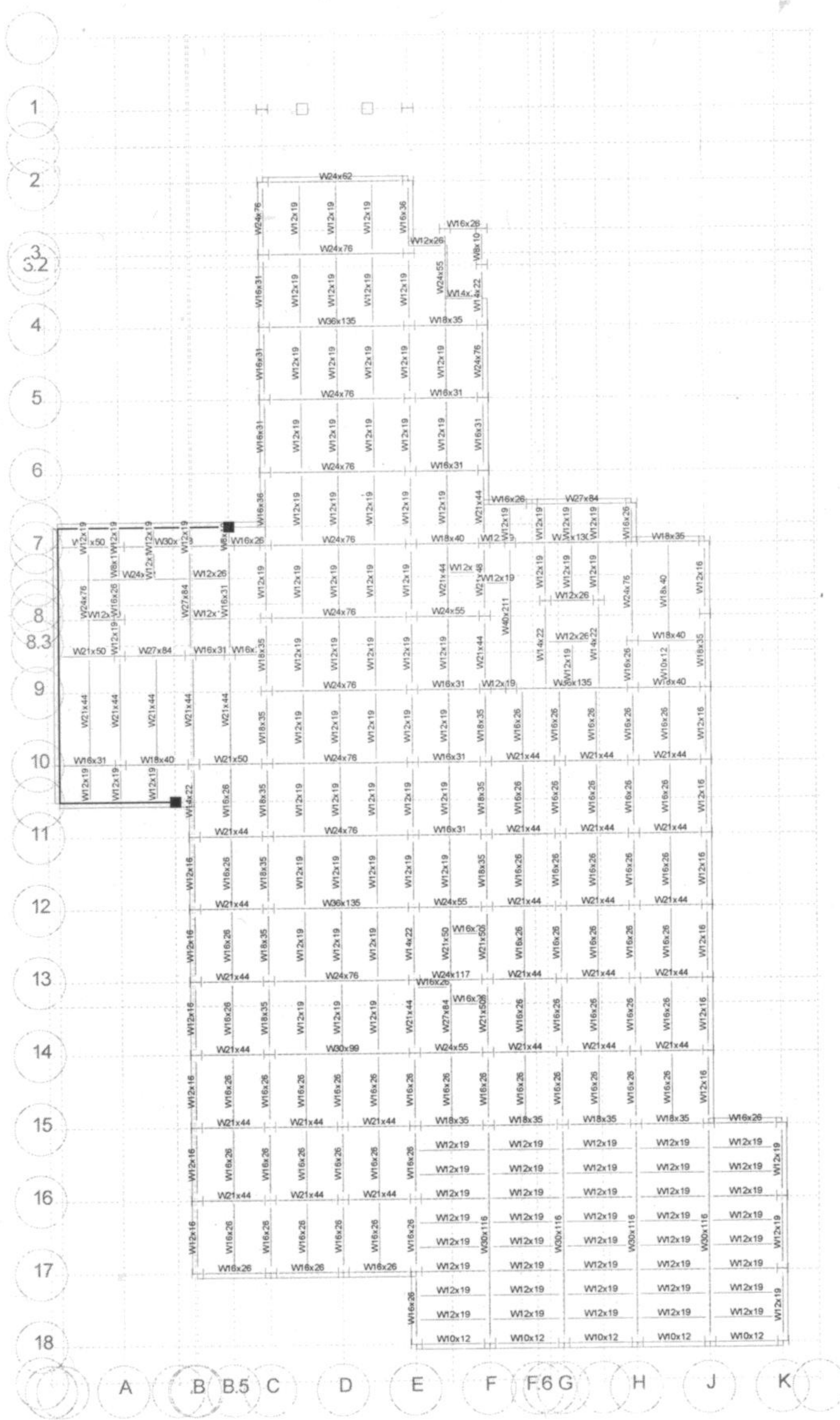
Overall, all of the studies done lead to the conclusion that a BRBF system is the best choice for use as the lateral force resisting system of the Visteon Village Corporate center. The system is structurally adequate, economical, and can feasibly be integrated into the architectural design with moderate alterations. This meets the goal of feasibly designing the structural system of this building to handle the increased loading due to its relocation to Orinda, California.

All design values and processes were in accordance to the current applicable codes as listed in the "Design Guides and Criteria" section of this report. Any comments, questions, or concerns can be directed to Jamison D. Morse at JamisonMorse@gmail.com.

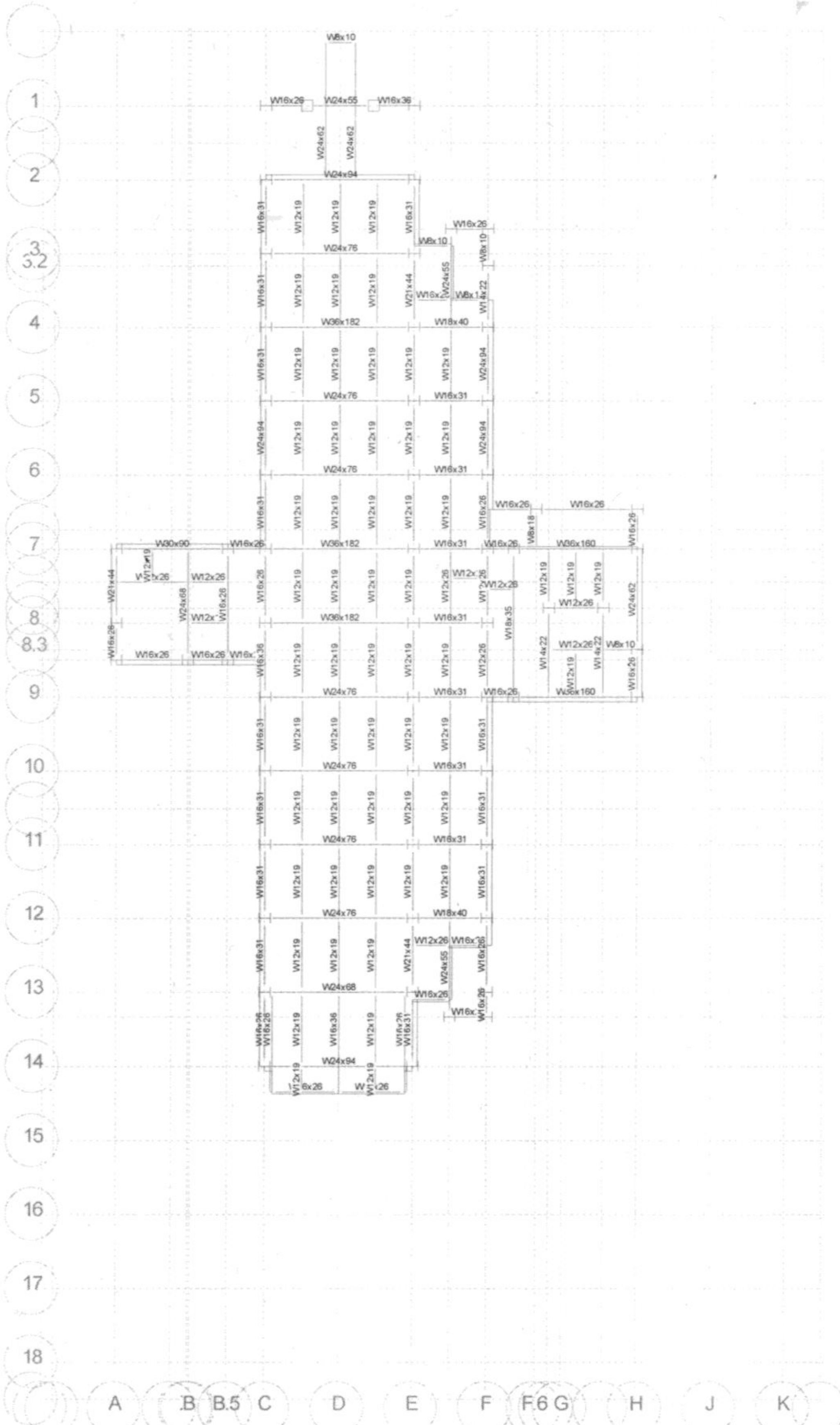
Appendix

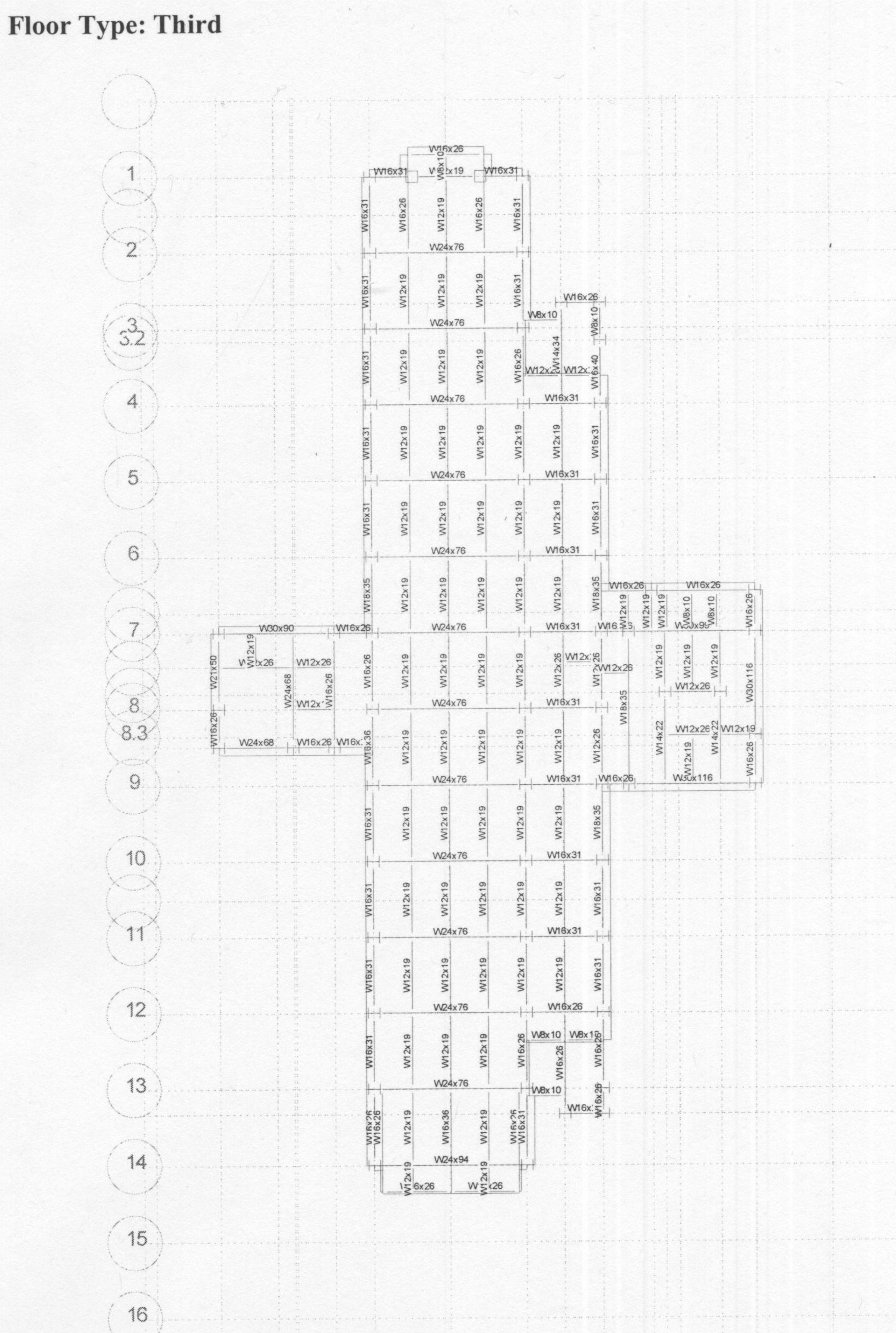
Gravity Framing System (Original Column Grid)

Floor Type: First

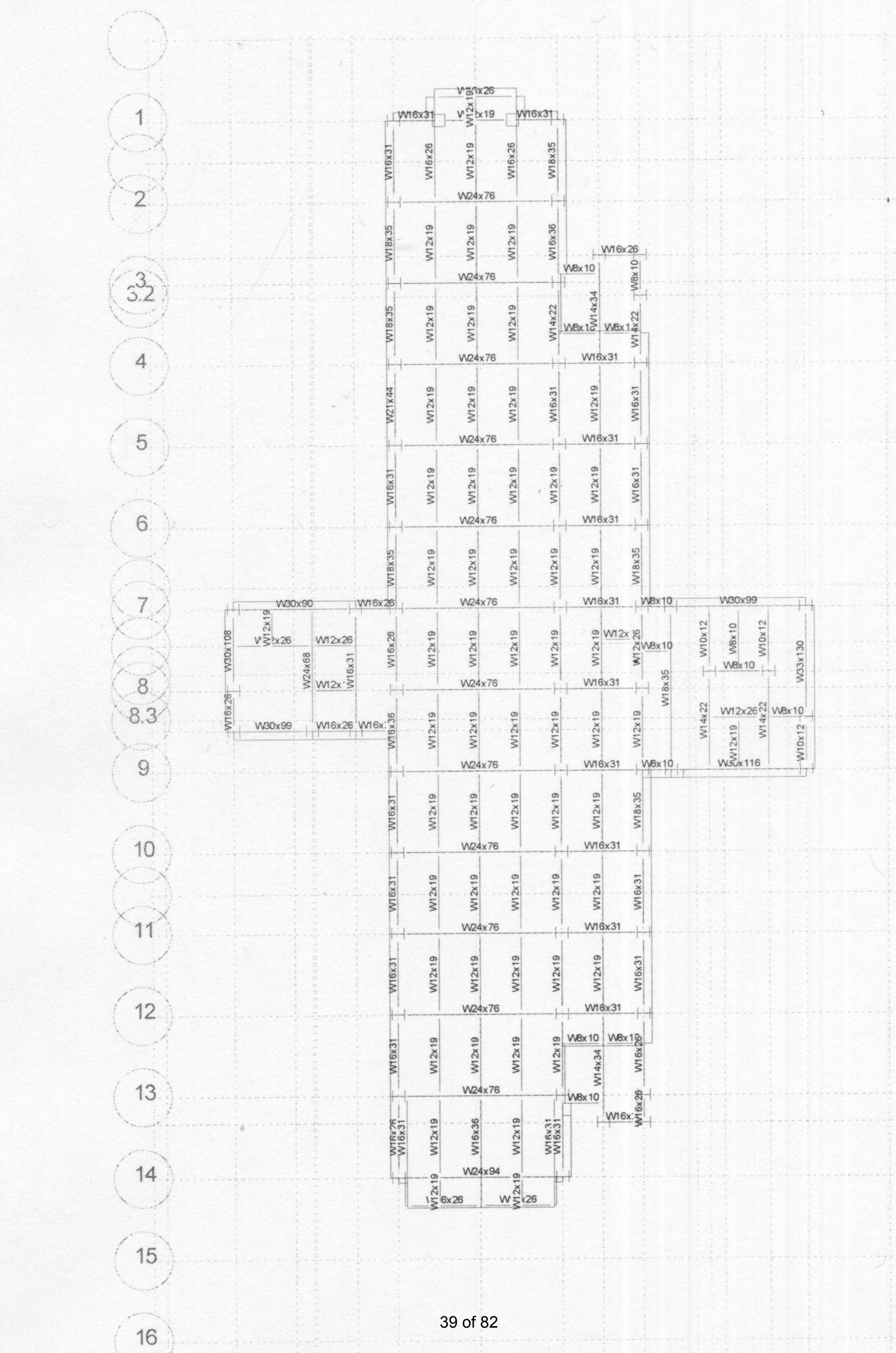


Floor Type: Second

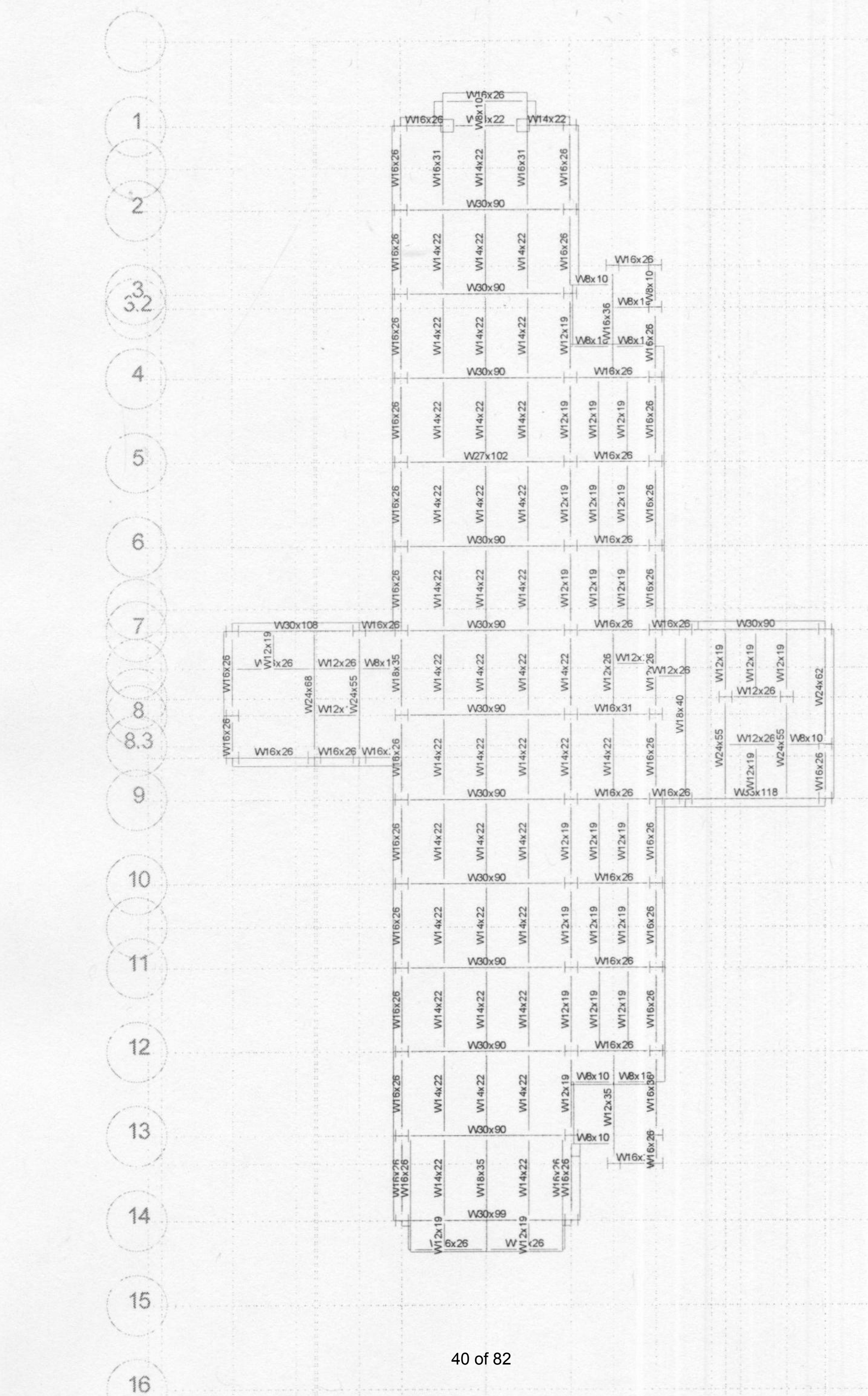



Floor Type: Third
1
2
3
3.2
4
5
6
7
8
8.3
9
10
11
12
13
14
15
16
W16x26
W16x31
W12x19
W24x76
W16x26
W8x10
W14x34
W16x40
W16x31
W18x35
W30x90
W21x50
W24x68
W16x26
W30x116
W12x26
W14x22
W16x36
W24x94

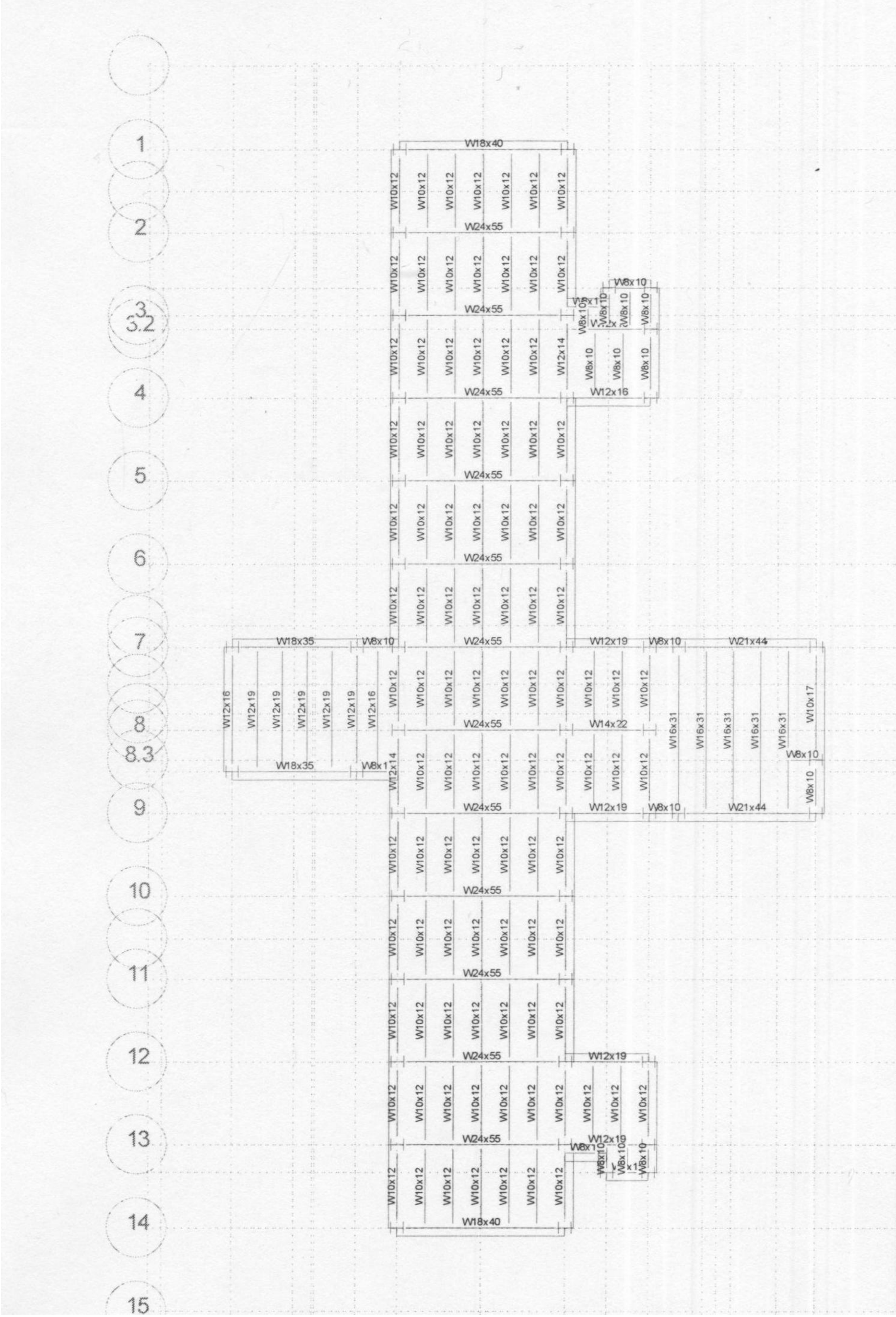
Floor Type: Fourth



Floor Type: Fifth-PH floor

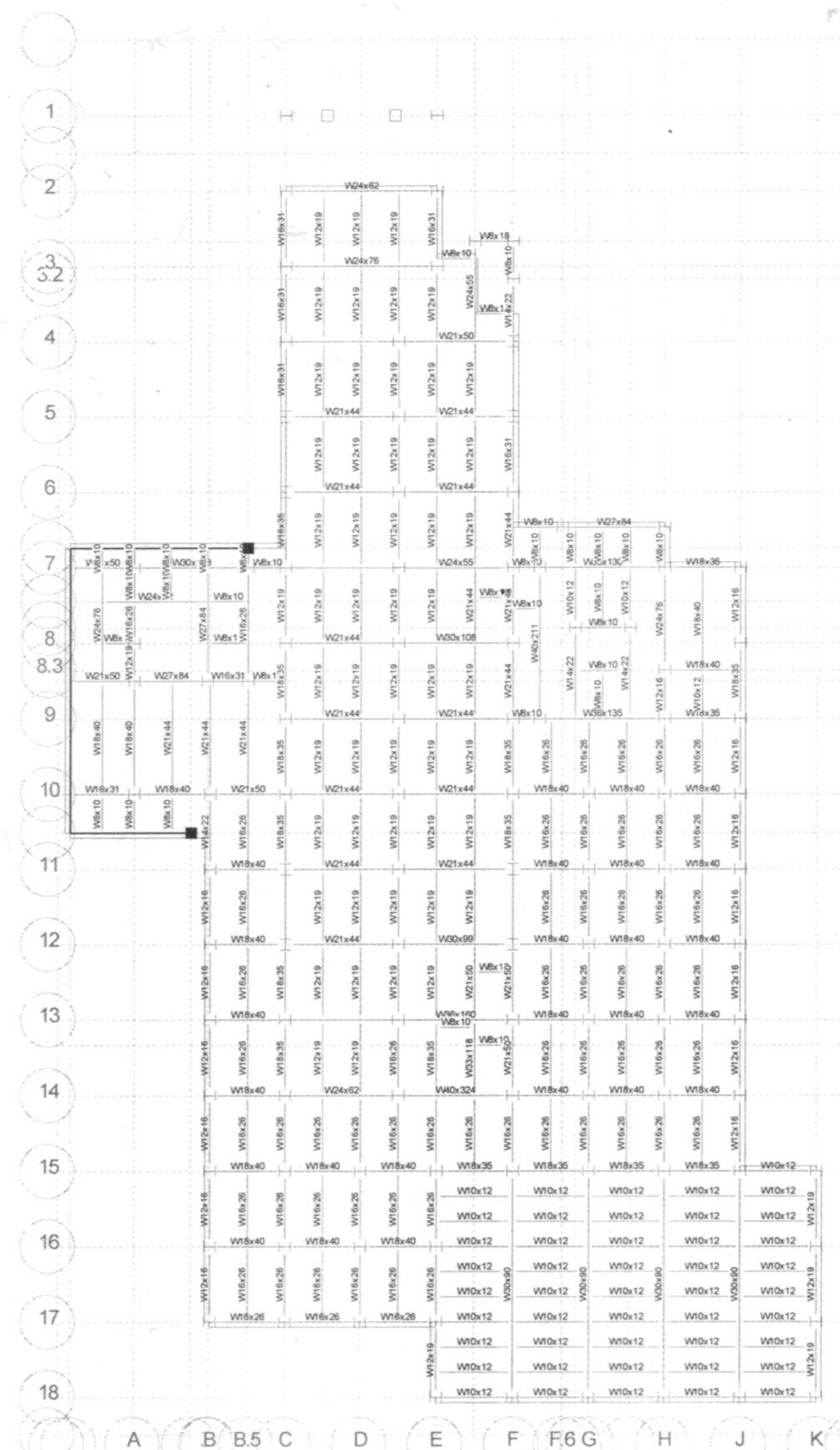


Floor Type: Sixth-PH roof

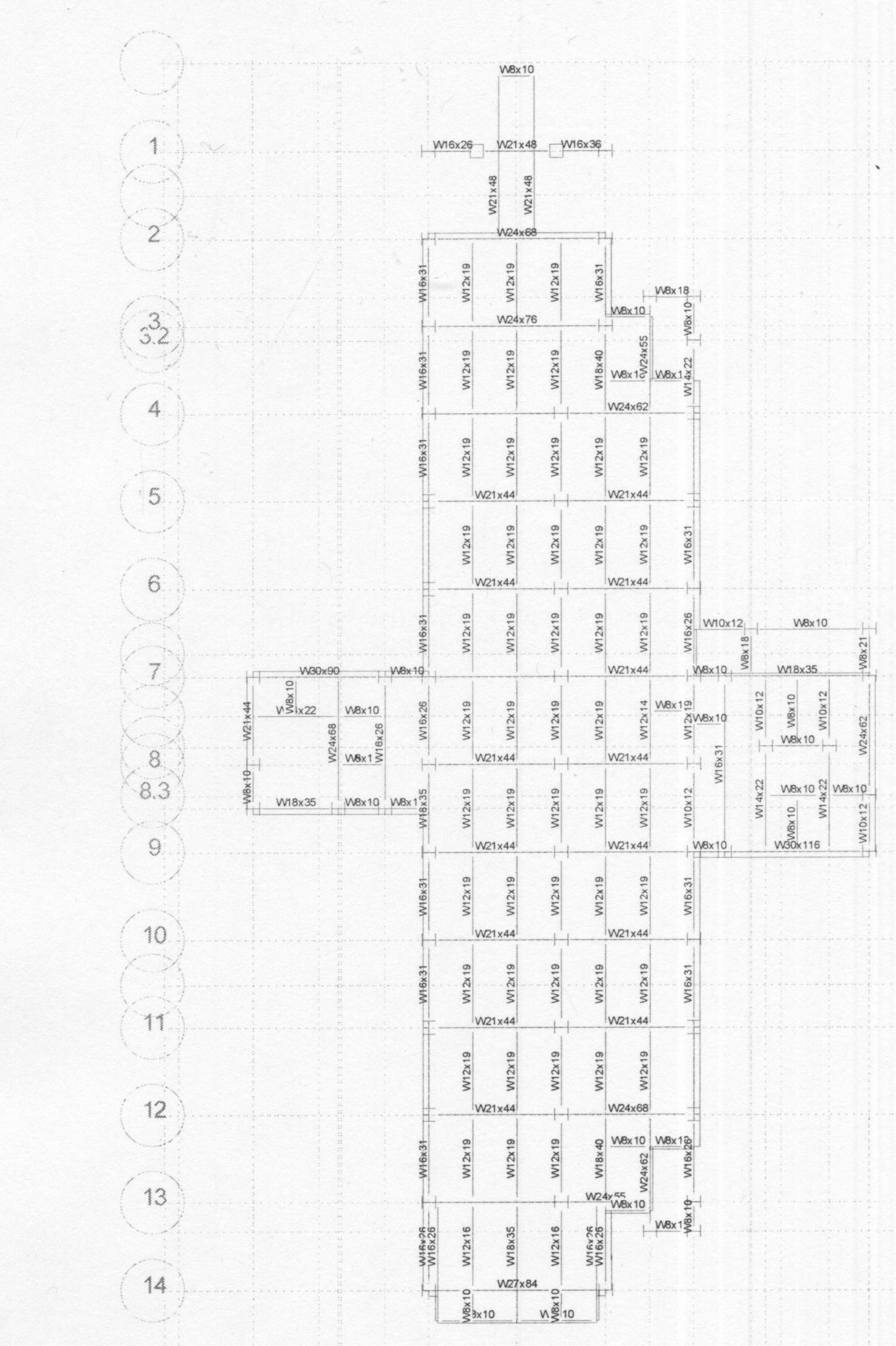


Gravity Framing System (Modified Column Grid)

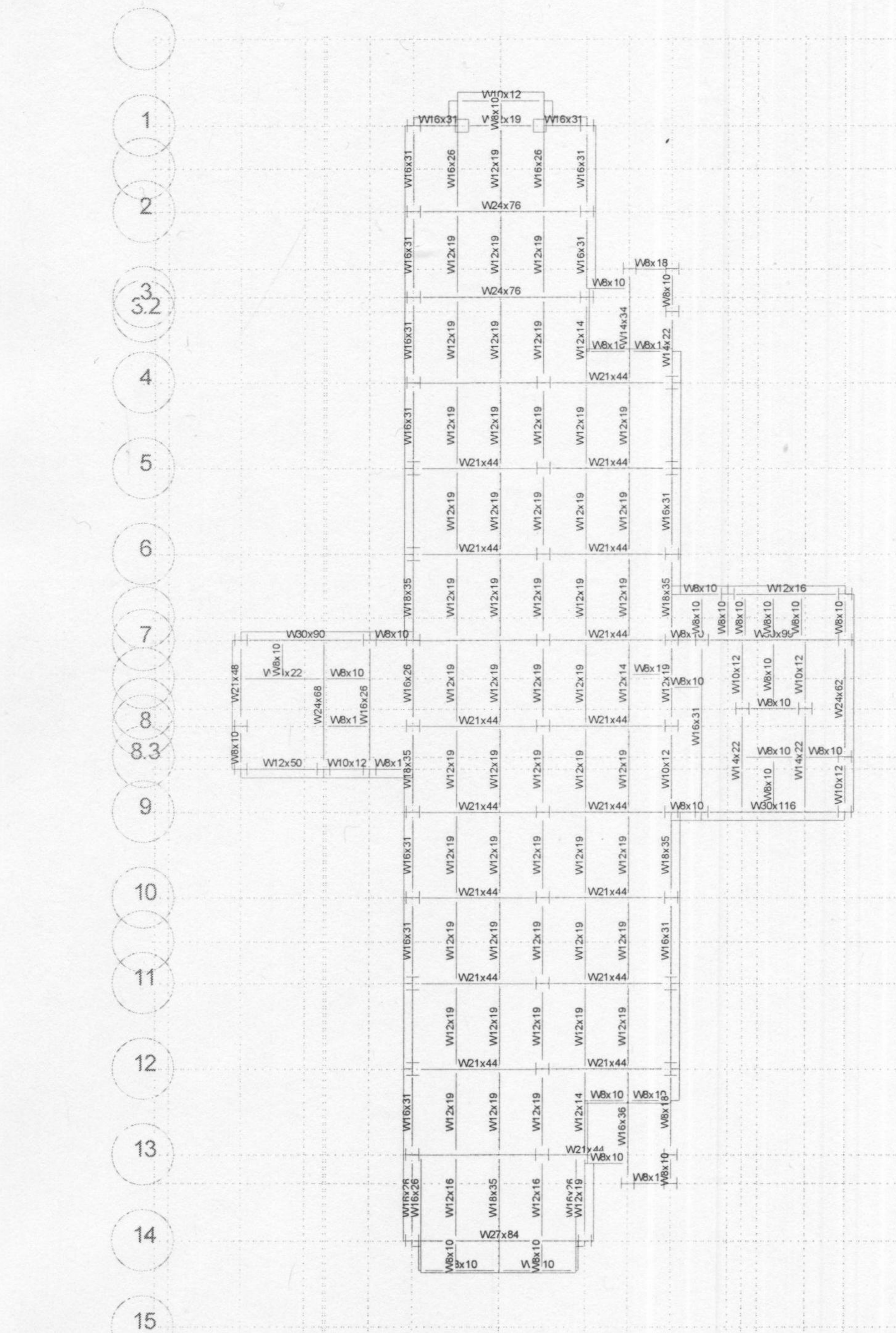
Floor Type: First



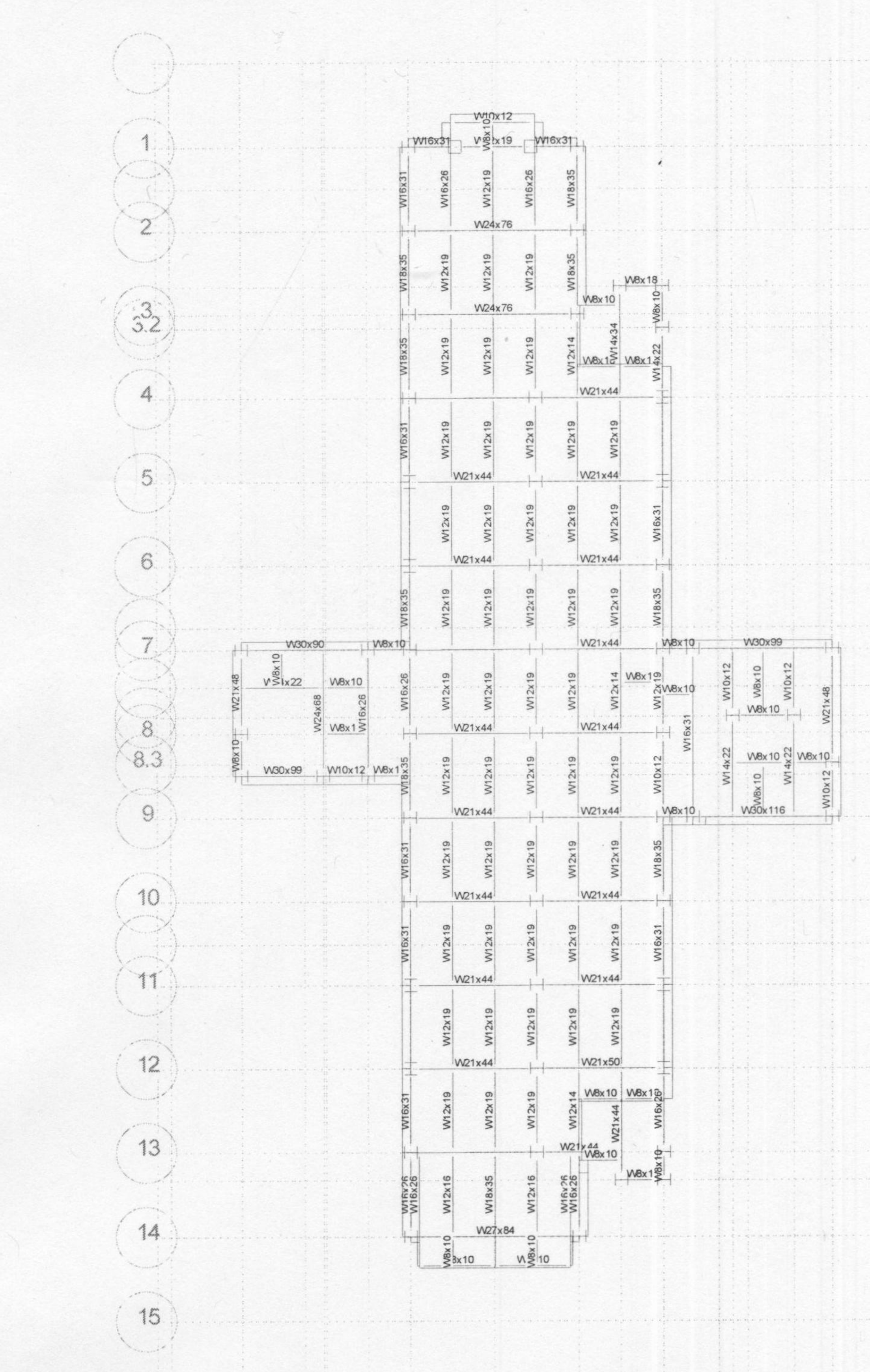
Floor Type: Second



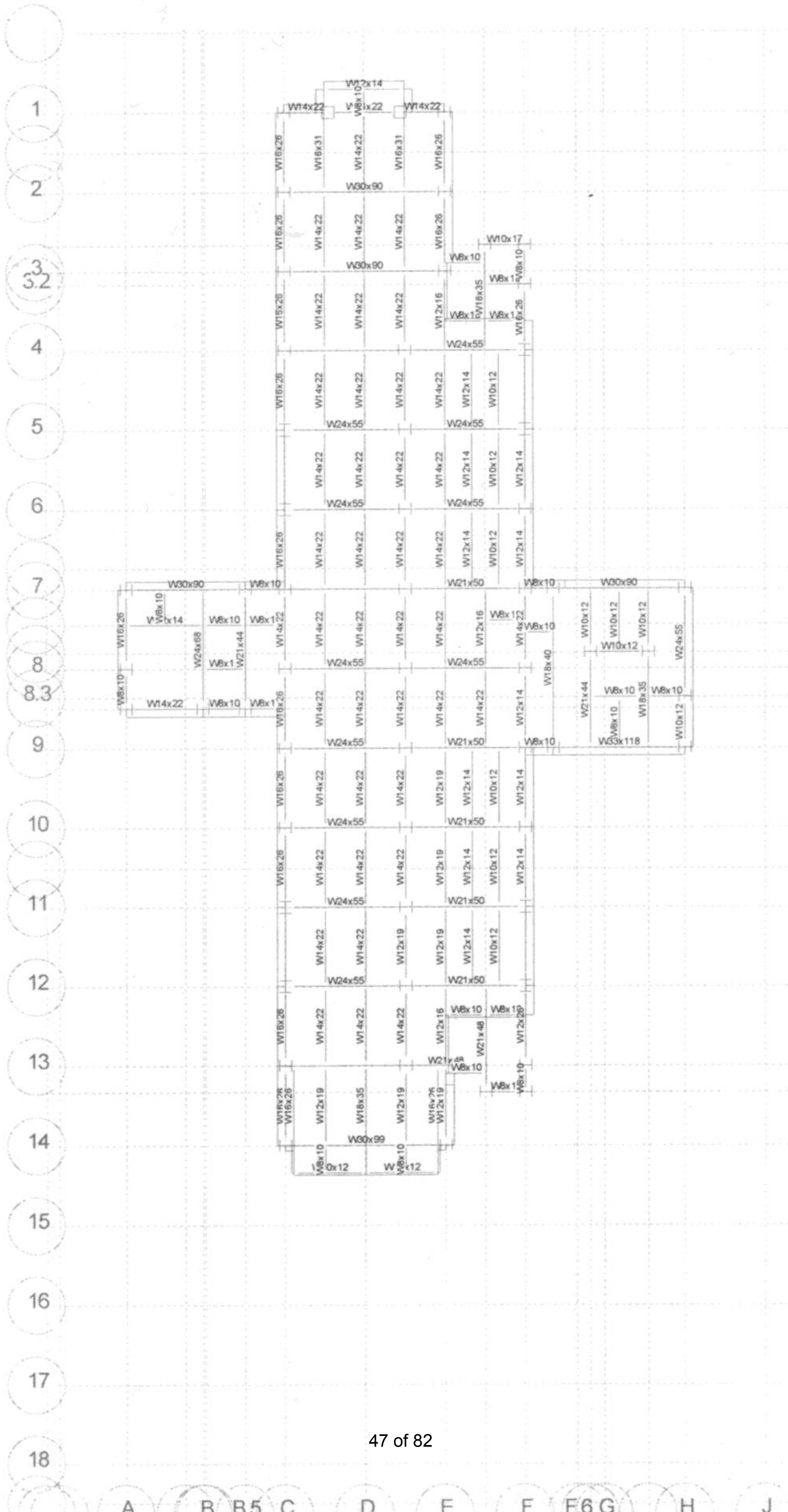
Floor Type: Third



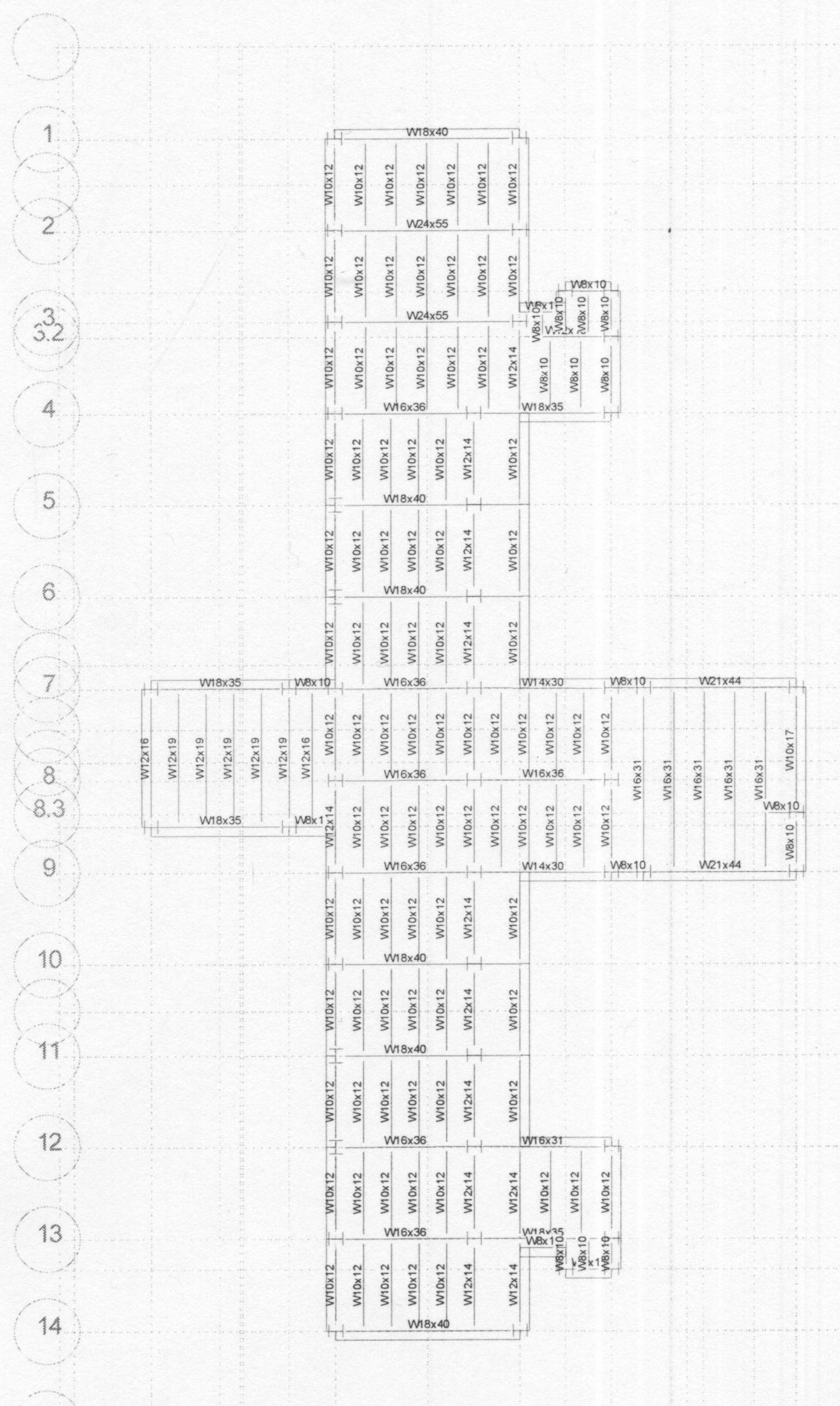
Floor Type: Fourth



Floor Type: Fifth-PH floor



Floor Type: Sixth-PH roof



Special Concentric Braced Frame Design

SmithGroup General Work Sheet

THESIS RESEARCH

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

SEISMIC AREA: ORINDA, CA 94563

$S_s = 1.500$ $S_{ms} = (1.5)(1.500) = 1.500$
$S_1 = 0.600$ $S_{m1} = (1.5)(0.600) = 0.900$ (USGS)

$S_{DS} = \frac{2}{3}(1.500) = 1.00$ CATEGORY III $F_a = 1.0$
$S_{D1} = \frac{2}{3}(0.900) = 0.600$ SITE CLASS D $F_v = 1.5$
$C_t = 0.02$ (CONCENTRICALLY BRACED STL FRAMES) (ALL OTHERS) 12.8-2 TBL
$h_n = 108$
$x = 0.75$

0.02

$T_a = C_t h_n^x = 0.02(108)^{(0.75)} = 0.670$
$T_s = S_{D1}/S_{DS} = 0.600/1.00 = 0.600$
SDC = D
R = 6 (TBL 12.2-1 ASCE 7-05) (SPECIAL STEEL CONCENTRICALLY BRACED FRAMES)
I = 1.25
$T_L = 12$

C_s

T

$$C_s = \frac{S_{DS}}{(R/I)} = \frac{1.00}{(6/1.25)} = 0.208$$

RANGE

MIN
$C_s = 0.01$

MAX
$$C_s = \frac{S_{D1}}{T(R/I)} = \frac{0.600}{(0.670)(6/1.25)} = 0.187$$
USING T_a

USE $C_s = 0.187$
TOTAL WT = 16935 K
$V = C_s W = (0.187)(16935^k) = 3160$ K

0.5	1
0.67	1.09
2.5	2

K = 1.09

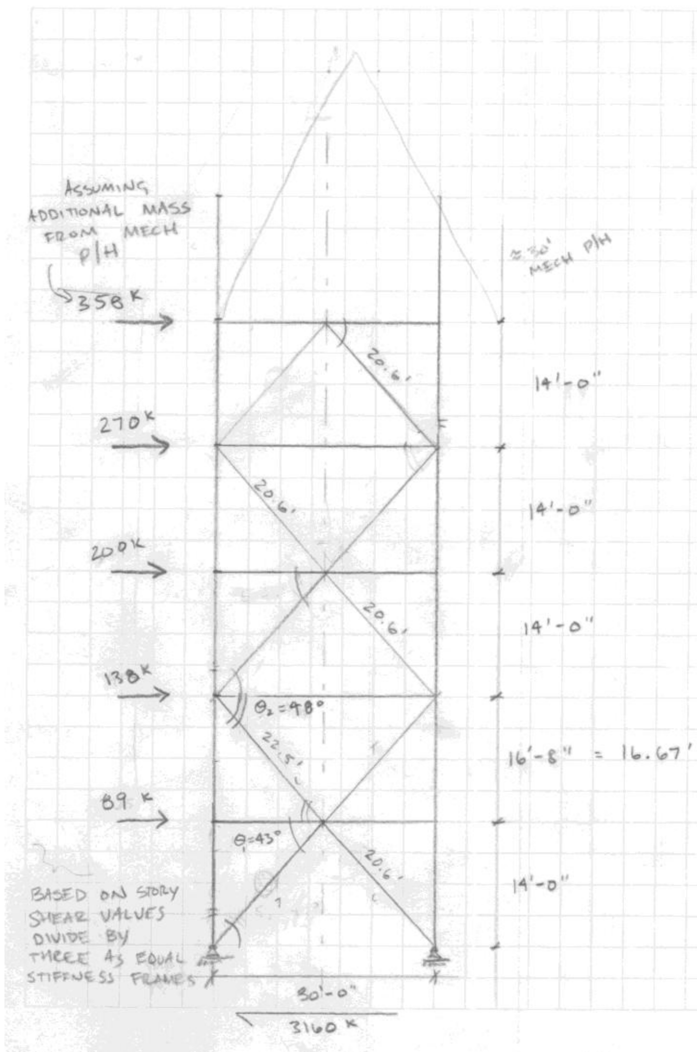
- FOUNDATIONS ADEQUATE? ✓
- BASEMENT WALLS
- HSS CONNECTION ✓
- WF CONNECTION ✓ - MF

SCBF k= **1.09** V= **3160**

lvl	ht	wx	x	y	wx*ht^k	Cvx	Fx	Vx	
Fifth	108.48	435	66.86667	207.3333	71948.28	0.071509	225.9669	225.9669	24512.89
Fourth	72.67	2525	63.90972	206.2212	269860	0.268211	847.5453	1073.512	78012.13
Third	58.67	3042	70.1224	206.7881	257473.6	0.2559	808.6436	1882.156	110426.1
Second	44.67	3042	70.1224	206.7881	191283.1	0.190114	600.76	2482.916	110911.8
First	30.67	3147	70.12623	206.8157	131345.4	0.130543	412.5145	2895.43	88802.85
Ground	14	4745	86.45129	139.1551	84239.5	0.083725	264.5697	3160	44240
				Total=	1006150			Overturning Moment=	456905.8

BRBF V= **2370**

lvl	ht	wx	x	y	wx*ht^k	Cvx	Fx	Vx	
Fifth	108.48	435	66.86667	207.3333	71948.28	0.071509	169.4752	169.4752	18384.67
Fourth	72.67	2525	63.90972	206.2212	269860	0.268211	635.659	805.1341	58509.1
Third	58.67	3042	70.1224	206.7881	257473.6	0.2559	606.4827	1411.617	82819.56
Second	44.67	3042	70.1224	206.7881	191283.1	0.190114	450.57	1862.187	83183.89
First	30.67	3147	70.12623	206.8157	131345.4	0.130543	309.3858	2171.573	66602.13
Ground	14	4745	86.45129	139.1551	84239.5	0.083725	198.4273	2370	33180
				Total=	1006150			Overturning Moment=	342679.3



SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

STORY 2 BRACE

$P_u = 1424^k$

$T_u = -1353^k$

$KL = 22.5 \approx 18'$ UNBRACED W 14 × 145 @ 18' COMP $\phi P_n = 1550^k$ (4-1) > 1424^k

TEN $\phi P_n = 1920^k > 1353^k$

USE W 14 × 145 FOR ALL BRACES

W14 × 145 $A = 42.7\ in^2$

$d = 14.8\ in$

COLUMN

$P_D = 4(27) + \left(\frac{94}{92}\right)(27) + \left(30/92\right)(27) = 145^k$

$P_L = 4(21.3) + (150/100)(21.3) + (30/100)(21.3) = 170^k$

$P_{QE} = 1055^k$

$P_S = 0$

$P_u = (1.2 + 0.2)\ 145^k + (1.3)(1055^k) + 0.5(170^k) + 0$

$P_u = 1660\ k$

$T_u = (0.9 - 0.2)\ 145^k + 1.3(-1055) + 1.6(0)$

$T_u = -1270^k$

W 14×159 UNBRACED = 17' MAX $\phi P_n = 1740$

$A_g = 46.7\ in^2$ $b_f = 15.6''$ $d = 15\ in$

$r_x = 6.38\ in$ $t_f = 1.19''$ $t_w = 0.745''$

$r_y = 4.00\ in$

$\lambda_f = \frac{15.6}{2(1.19)} = 6.55$ $\lambda_{rs} = 0.30\sqrt{\frac{E}{F_y}} = 7.22$

$6.55 < 7.22$ ∴ ✓

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

ACCORDING TO 1-2 SDM W14 x 159 SATISFIES
LOCAL BUCKLING CONDITIONS ∴ ✓

COL COMPRESSIVE STRENGTH

$$\frac{KL}{r} = \frac{1.0\,(16.67')\,(12\,''/ft)}{4.00\ in} = 50.0$$

$$4.71\sqrt{E/F_y} = 113$$

$$F_e = \frac{\pi^2 E}{(50)^2} = 114\ ksi$$

$KL/r < 113$ 50

$$F_{cr} = \left[0.658^{(50/114)}\right]50 = 41.6\ ksi$$

$$P_n = F_{cr} A_g = 41.6\ ksi\ (46.7) = 1943\ k$$

$$\phi P_n = 0.9\,(1943) = 1749^k > 1660^k \quad \therefore ✓$$

TENSION

$$\phi P_n = 0.9\,(50)\,(46.7) = 2101^k > 1270^k \quad \therefore ✓$$

AXIAL LOAD RATIOS

$$\frac{T_u}{\phi P_n} = \frac{1270}{1749} = 0.73$$

$$\frac{P_u}{\phi P_n} = 0.95 \qquad (8.3)$$

USE W14 x 159 ✓

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No. N-S

Drawn By

COLUMN

$P_D = 4(20^k) + \left(\frac{94}{92}\right)(20^k) + (30/92)(20^k) = 107^k$

$P_L = 4(25^k) + (150/100)(25^k) + (80/100)(25^k) = 145^k$

$P_{QE} = 792^k$

$P_S = 0$

$P_u = (1.2 + 0.2)\,107^k + (1.3)(792^k) + 0.5(145^k) = 1252^k$

$T_u = (0.9 - 0.2)\,107^k + 1.3(-792^k) + 1.6(0) = -955^k$

$UBL \approx 17'$

W14x120

@ $UBL = 17'$ $\phi P_n = 1280^k > 1252^k$ ∴ ✓

E-W BRACE

$P_u = (1.2 + 0.2)\,27^k + 1.0\,\frac{(528)}{\cos(43)} + 0.5(29.3^k) = 842^k$

$T_u = (0.9 - 0.2)(27^k) + 1.0\,\frac{(528)}{\cos(43)} + 1.6(0) = 807^k$

$UBL \approx 18'$

USE W14x90

@ UBL of 18' $\phi P_n = 928^k > 842^k$ ✓

N-S BRACE

$P_u = (1.2 + 0.2)\,20^k + \frac{(396^k)}{\cos(35°)} + 0.5(25^k) = 491^k$

$T_u = (0.9 - 0.2)(20^k) + \frac{396^k}{\cos(35)} + 1.6(0) = 497^k$

$UBL \approx 15'$

USE W14x61

@ $UBL = 15'$ $\phi P_n = 543^k > 497^k$ ✓

Special Concentric Braced Frame Connection Design

SmithGroup
SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

General Work Sheet

Subject: BRACE CONNECTION DESIGN (W-SHAPE BRACE)

Date

Project No.

Drawn By

SDM 3-79 Ex 3.12

BRACE TO BEAM

BEAM = W27×336		BRACE = W14×90		COLUMN = W14×159	
$d = 30''$	$t_w = 1.26''$	$d = 14.0''$	$A_g = 26.5\ in^2$	$d = 15.0''$	$A_g = 46.7\ in^2$
$b_f = 14.6''$	$t_f = 2.28''$	$b_f = 14.5''$	$t_w = 0.440''$	$b_f = 15.6''$	$t_w = 0.745''$
		$k_{des} = 1.31''$	$t_f = 0.710''$	$k_{des} = 1.79''$	$t_f = 1.19''$

- EXPECTED TENSILE DESIGN OF BRACE

$$T_u = R_y F_y A_g = 1.1(50)(26.5) = 1460^k > 850^k$$

- $1.1 R_y$ TIMES NOMINAL COMP STRENGTH

UBL = 18'

$$1.1 R_y P_n = \frac{1.1(1.1)(621^k)}{0.9} = 930^k$$

- WELDS CONNECTING CONNECTION PLATES TO BRACES

	$\bar{x}$	A	$\bar{x}A$
FLANGE	3.63"	5.15 in^2	18.69
WEB	7.03	2.77 in^2	19.47
	Σ	7.92 in^2	38.16

$$\bar{x} = \frac{\Sigma \bar{x}A}{\Sigma A} = \frac{38.16\ in^3}{7.92\ in^2} = 4.82''$$

- U USING 18" CONNECTION LENGTH

$$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{4.82}{18} = 0.732$$

$$A_n = A_g = 24\ in^2$$

$$A_e = UA_n = (0.732)(24\ in^2) = 19.41\ in^2$$

- STRENGTH REQUIRED PER INCH OF WELD

$$\frac{850^k}{4(18'')} = 11.81^k/in \qquad \frac{11.81^k}{1.392^k/in} = 8.48 \text{ SIXTEENTHS} = 5/8'' \text{ FILLET WELD}$$

1 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

$t_{min} = \dfrac{(3.09)\,D}{F_u \text{ brace}} = \dfrac{(3.09)(10)}{65} = 0.475''$

$t_f = 0.710'' > 0.475'' \therefore$ ✓

USE 18 INCH LONG 5/8" FILLET WELDS ALONG EACH FLANGE TO CONNECT THE CONNECTION PLATE

• TRIAL CONNECTION PL THICKNESS

CONSERV → 1" > 0.475" ✓

$b = 14 + 2(1'') = 16''$ PL → 1" THICK PL

• CONNECTION PL DESIGN FORCE

(TEN) $R_{ut} = 1/2(850) = 425^k$

(COMP) $R_{u_c} = 930^k/2 = 465^k$

• SHEAR YIELDING STRENGTH OF CONNECTION PLATES

$A_g = 2(18)(1'') = 36\ in^2$

$\phi R_n = \phi(0.6F_y)A_g = 1.0(0.6)(50\ ksi)(36\ in^2) = 1080^k$

$1080^k > 465^k \therefore$ ✓

$> 425^k \therefore$

• BOLTS BETWEEN CONNECTION PL AND GUSSET FLANGE PL

TRY 1-1/4" A490 N $\phi_v r_n = 55.2\ ^k/\text{BOLT}$

$N_b = \dfrac{930^k}{2(55.2)} = 8.43 \approx 10$ BOLTS IN 2 ROWS

STD HOLES IN EACH CONNECTION PLATE-TO-FLANGE PLATE CONNECTION

2 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

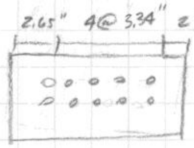
Date

Project No.

Drawn By

- BEARING STRENGTH OF CONNECTION PL w/ $2\,2/3\ d_b$ SPACING

$\phi r_n = 118(1'') = 118\ ^k/\text{BOLT}$



$\phi r_n = 78.6(1'') = 78.6\ ^k/\text{BOLT}$

$\underset{PL}{78.6\ ^k/\text{BOLT}} > \underset{\text{BOLT}}{55.2\ ^k/\text{BOLT}}$

PL IS ADEQUATE.

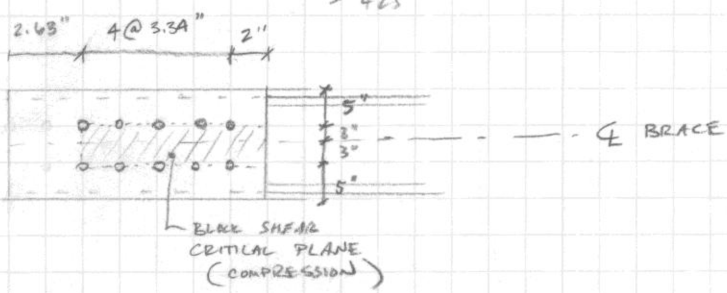
- SHEAR RUPTURE STRENGTH OF CONNECTION PLATES

$A_n = 36\ \text{in}^2 - 2(5)(1\,^1\!/_4 + {}^1\!/_{16} + {}^1\!/_{16})(1'') = 22.25\ \text{in}^2$

$\phi R_n = 0.75(0.6)(65\ \text{ksi})(22.25\ \text{in}^2) = 650^k$

$650^k > 465^k$

$> 425^k \quad \therefore$ ✓



$A_{gv} = 2(2 + 4(3.34))(1'') = 30.72\ \text{in}^2$

$A_{nv} = 30.72\ \text{in}^2 - 2(4.5)(1\,^1\!/_4 + {}^1\!/_8)(1) = 18.35\ \text{in}^2$

$A_{gt} = 6''(1'') = 6\ \text{in}^2$

$A_{nt} = 6 - (1)(1\,^1\!/_4 + {}^1\!/_8)(1) = 4.63\ \text{in}^2$

$F_u A_{nt} = 65(4.63) = 300^k$

$0.6 F_u A_{nv} = 0.6(65)(18.35) = 715^k$

$0.6 F_y A_{gv} = 0.6(50)(30.72) = 921^k$

3 OF

$R_n = 0.6 F_u A_{nv} + U_{bs} F_u A_{nt} \le 0.6 F_y A_{gv} + U_{bs} F_u A_{nt}$

$\phi R_n = 0.75[715^k + 1.0(300^k)] \le 0.75[921^k + 1.0(300^k)]$

$\underline{761^k} \le 915^k$

$761^k > 465^k \therefore$ ✓

$> 425^k$

USE 1" x 16" ASTM A572 GRADE 50 CONNECTION PL's

- DETERMINE TRIAL THICKNESS FOR FLANGE PLs

$b_{top} = 16 - 2(1) = 14.0"$

$b_{bot} = 16 + 2(1) = 18.0"$ TRY $1\frac{1}{2}"$ PL ASTM A36

- BEARING STRENGTH FOR FLANGE PLs

(1 1/4" A325X) $\phi r_n = 105(1.5) = 157.5^k$/BOLT

$\phi r_n = 70.1(1.5) = 105.1^k$/BOLT (2" EDGE DIST)

$105.1 > 53.2^k \therefore$ ✓

- TENSION YIELDING OF FLANGE PLATE

CRITICAL OVER TEN COM

$\phi R_n = \phi F_y A_g = 0.9(36)(21 in^2) = 680^k > 465^k$ ✓

- TENSION RUPTURE OF FLANGE PLATES

(UP AS MORE CRITICAL)

$A_n = 21 in^2 - 2(1\,1/4 + 1/8)(1.5) = 16.88 in^2$

$0.85 A_g = 0.85(21) = 17.85 > 16.88 \therefore$ OK USE

$\phi R_n = \phi F_u A_e = \phi F_u (UA_n) = 0.75(58 ksi)(1.0)(16.88)$

$\phi R_n = 734^k > 465^k \therefore$ ✓

4 OF

SmithGroup **General Work Sheet**

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

• BLOCK SHEAR RUPTURE STRENGTH OF FLANGE PLs

$A_{gv} = 2(2" + 4(3.34))(1.5) = 46 \text{ in}^2$

$A_{nv} = 46 \text{ in}^2 - 2(4.5)(1\,1/4" + 1/8")(1.5") = 27.44 \text{ in}^2$

$A_{gt} = 6(1.5) = 9 \text{ in}^2$

$A_{nt} = 9 - (1)(1\,1/4 + 1/8)(1.5") = 6.94 \text{ in}^2$

$F_u A_{nt} = 58(6.94) = 402^k$

$0.6 F_u A_{nv} = 0.6(58)(27.44) = 955^k$

$0.6 F_y A_{gv} = 0.6(36)(46) = 993^k$

$\phi R_n = 0.75\left[955 + (1.0)(402)\right] \le 0.75\left[993 + 1.0(402)\right]$

$\underline{1017^k} < 1046^k$

$1017^k >> 465^k \therefore$ ✓

• SHEAR LAG OF FLANGE PLATES AT WELDED CONNECTION TO GUSSET PL

ASSUMPTIONS: $\bar{x} \approx b/4 =$

$U = 1 - \frac{\bar{x}}{\ell} = 1 - \frac{b/4}{b} = 0.75$

BOT FLANGE PL

$A_n = (18 - 1\,1/4)(1.5) = 25.1 \text{ in}^2$

$A_e = UA_n = 0.75(25.1) = 18.8 \text{ in}^2$

$\phi P_n = \phi F_u A_e = 0.75(58)(18.8) = 817^k > 465^k \therefore$ ✓

TOP FLANGE PL

$U = 1 - \frac{14}{4(18)} = 0.806$

$A_n = (14 - 1\,1/4")(1.5") = 19.1 \text{ in}^2$

$A_e = UA_n = 0.806(19.1) = 15.4 \text{ in}^2$

$\phi P_n = \phi F_u A_e = 0.75(58)(15.4) = 670^k > 465^k \therefore$ ✓

5 OF

• CHECK COMPRESSION BUCKLING OF FLANGE PLS
ASSUMING 1/2" SETBACK UBL = 2.75 + 1/2 = 3.25" AVG

$$r = \frac{t}{\sqrt{12}} = \frac{1.5}{\sqrt{12}} = 0.433''$$

$$\frac{KL}{r} = \frac{0.65(3.25'')}{0.433} = 4.87 \;\checkmark$$

$$A_g = 1.5(14+18) = 48\ IN^2$$

$$\phi P_n = \phi F_y A_g = 0.90(36)(48) = 1555.2 \gg 465^K \therefore \checkmark$$

• USE 1 1/2" THICK ASTM A36 FLANGE PLATES
USE 14" WIDTH FOR TOP FLANGE PLATE AND
18" WIDTH FOR BOTTOM FLANGE PLATE
USE STD HOLES
EACH BOLT GROUP WILL HAVE A 3.34" BOLT SPACING ($2\,^2/_3\, d_b$)
6" GAGE AND 2" EDGE DIST @ BRACE END
OF CONNECTION AND FLANGE PLATES

• DESIGN WELDS CONNECTING FLANGE PLATES TO GUSSET

$$\phi R_n = 1.392\, D\, l_w \qquad MIN = 5/16$$

$$1.392\,(4\ \text{WELDS})(5)(17'') = 473^K > 465^K \therefore \checkmark$$

• CHECK GUSSET PL FRACTURE AT FLANGE PL WELDS
TRY 1" A36 GUSSET PL

$$t_{min} = \frac{6.19\,D}{F_u} = \frac{6.19\,(5)}{58} = 0.534''$$

$$1'' > 0.534'' \therefore \checkmark$$

• USE DOUBLE SIDED 5/16" FILLET WELDS ON EA SIDE OF GUSSET PL
TO CONNECT FLANGE PL TO THE GUSSET PL

6 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

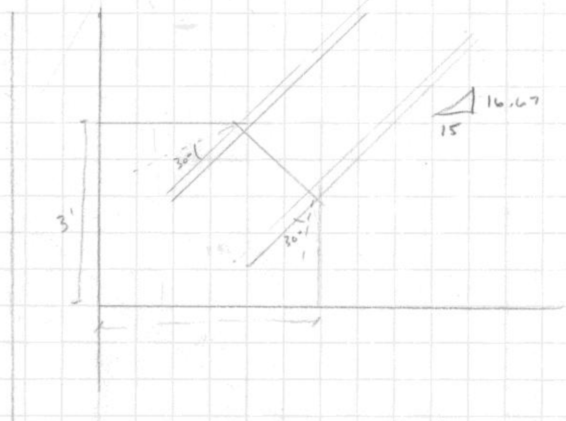
Subject

Date　　　　Project No.

Drawn By

• CHECK TEN YIELDING STRENGTH OF GUSSET PL

2-½" OFFSET　$l_w = 14.5'' + 2(1'' + 1.5'') + 2(17)\tan 30° = 39.2 \text{ IN}^2$



$\phi P_n = \phi F_y A_g = 0.9(36)(39.2)(1'') = 1270^k > 850^k \therefore$ ✓

• COMPRESSION BUCKLING STRENGTH OF GUSSET PL

$r = \frac{t}{\sqrt{12}} = \frac{1''}{\sqrt{12}} = 0.288$

AVG BUCKLING LENGTH

$\frac{KL}{r} = \frac{0.65(7.56'')}{0.288} = 17.06$

$\phi R_n = \phi F_y A_g = 0.9(36)(39.2)(1'') = 1270 > 930^k \therefore$ ✓

• CONNECTION DESIGN FORCES

TEN = 850^k
COMP = 930^k

TEN　$R_{ub} = 0.7(P_D) + 0... = 0.7(13.8) = 9.7^k$

COMP - $R_{ub} = 1.4(P_D) + 0.5(P_L) + 0... = 1.4(13.8) + 0.5(15) = 26.8^k$

$e_b = \frac{30''}{2} = 15''$　　$\theta = 48°$

$e_c = \frac{15''}{2} = 7.50''$　　$\beta = 18''$

7 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date Project No.

Drawn By

SM 13-5 SDM 3-93

$$\alpha = (15 + 18)\tan 48 - 7.50'' = 33.2''$$

EDGE DIM:

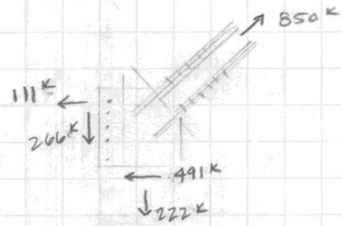
$$2(33.2'' - 5'' - 1/2) + 5 + 1/2 = 60.9'' \quad \text{USE } 5'\text{-}0''$$

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

$$r = \sqrt{(33.2 + 7.5)^2 + (18 + 15)^2} = 57.41'' \approx 57.5''$$

• CONNECTION INTERFACE FORCES FOR EXPECTED TENSILE STRENGTH

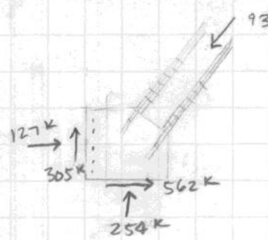
$$V_{uc} = \frac{\beta}{r} P_u = \frac{18}{57.5}(850^k) = 266^k$$
$$H_{uc} = \frac{e_c}{r} P_u = \frac{7.5}{57.5}(850^k) = 111^k$$
$$V_{ub} = e_b/r\, P_u = 15/57.5\,(850^k) = 222^k$$
$$H_{ub} = \alpha/r\, P_u = 33.2/57.5\,(850^k) = 491^k$$



• CONNECTION INTERFACE FORCE FOR EXPECTED COMPRESSIVE STRENGTH

930^K

$$V_{uc} = \frac{\beta}{r} P_u = \frac{18}{57.5}(930^k) = 305^k$$
$$H_{uc} = e_c/r\, P_u = 7.5/57.5\,(930^k) = 127^k$$
$$V_{ub} = e_b/r\, P_u = 15/57.5\,(930^k) = 254^k$$
$$H_{ub} = \alpha/r\, P_u = 33.2/57.5\,(930^k) = 562^k$$



• DESIGN GUSSET TO BEAM CONNECTION

RESULTANT FORCE ON THE CONNECTION

$$R_{ub} = \sqrt{(254)^2 + (562)^2} = 617^k$$

LENGTH OF WELD ALONG GUSSET-TO-BEAM INTERFACE:

$$l_{wb} = 2(33.2'' - 5'' - 1/2'') = 55.4$$

NO MOMENT ∴ PEAK STRESS = AVG STRESS SO 1.25 FACTOR MUST BE USED

$$f_r = \frac{1.25(617)}{55.4} = 13.92 \text{ KIPS/IN}$$

* USE DOUBLE-SIDED, 1/2" FILLET WELDS TO ATTACH GUSSET PLATE TO THE BEAM

18 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

- GUSSET PLATE RUPTURE AT BEAM WELDS

MIN GUSSET PLATE THICKNESS $t_{min} = \dfrac{(6.19\ ^{kips}/_{in})(8/1.25)}{58\ ksi} = 0.684'' < 0.75''$ ✓

- CHECK LOCAL YIELDING OF BEAM WEB FOR TENSION AND COMPRESSION

SM TBL 9-4 FOR W27X336

$\phi R_1 = 483\ ^k \qquad \phi R_2 = 63\ ^k/_{in}$

$\phi R_n = \phi R_1 + N\phi R_2 = 483^k + 55.4''(63\ ^k/_{in}) = 3973\ ^k >> V_{ub} = 254\ ^k$ ✓

- CHECK LOCAL WEB CRIPPLING OF BEAM FOR COMPRESSION

$\phi R_5 = 708\ ^k \qquad \phi R_6 = 42.3\ ^k/_{in}$

$\phi R_n = \phi R_5 + N\phi R_6 = 708\ ^k + 55.4''(42.3\ ^k/_{in}) = 3051\ ^k >> 227\ ^k$ ✓

- DETERMINE LOADS AT GUSSET-TO-SINGLE-PLATE CONNECTION

$H_{um} = \dfrac{(V_{uc} + V_{ub} - R_{ub})e}{e_b + \beta}$ ← 2.5" (1/2 OF WIDTH)

$= \dfrac{(305\ ^k + 254\ ^k - 26.8\ ^k)(2.5'')}{(15'' + 18'')} = 40.3\ ^k$

FORCE COMPONENTS AT CONNECTION: (CONTROLLING IS COMP)

$V_u = V_{uc} = 305\ ^k$

$H_u = H_{uc} + H_{um} = 127\ ^k + 40.3\ ^k = 167.3\ ^k$

- RESULTANT LOAD AT CONNECTION

$R_u = \sqrt{V_u^2 + H_u^2} = \sqrt{305^2 + 167^2} = 348\ ^k$

- DESIGN BOLTS AT GUSSET-TO-SINGLE-PLATE CONNECTION

TRY 1 1/4" A-490 N 55.2 k/BOLT

$\dfrac{451^k}{55.2\ ^k/_{bolt}} = 8.17 \Rightarrow 9$ BOLTS ✓ 30.7" REQ'D HAVE 36" ∴ ✓

- CHECK SINGLE PLATE BEARING

TRY 5/8" THICK PL TBL 7-5

SPACING = 3.34" $(2\ ^2/_3\ d_B)$ $\phi r_n = 118(0.625) = 73.7\ ^k$

EDGE (2") $\phi r_n = 78.6(0.625) = 49.1\ ^k$

$8(73.7\ ^k) + 49.1\ ^k = 633\ ^k > 348\ ^k$ ∴ ✓

9 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

- CHECK GUSSET BEARING

  GUSSET HAS SAME BEARING AS SINGLE ℞ AND IS THICKER SO INTUITIVELY IT IS OK

- GUSSET YIELDING

$\phi R_n = \phi\,(0.6\,F_y)\,A_g = 1.0(0.6)(36\text{ ksi})(36")(3/4") = 583^k > 348^k \therefore$ ✓

BLOCK SHEAR RUPTURE OF GUSSET - TO - SINGLE PLATE CONNECTION

$A_{gv} = [2" + 8(3.34")](0.625) = 17.9\text{ in}^2$

$A_{nv} = 17.9\text{ in}^2 - 8.5(1\,1/4 + 1/8)(0.625) = 10.6\text{ in}^2$

$A_{gt} = 2.5"(0.625") = 1.56\text{ in}^2$

$A_{nt} = 1.56\text{ in}^2 - 1/2\,(1\,1/4 + 1/8)(0.625) = 1.13\text{ in}^2$

$F_u A_{nt} = 58\text{ ksi}\,(1.13\text{ in}^2) = 65.5^k$

$0.6\,F_u A_{nv} = 0.6(58)(10.6) = 368^k$

$0.6\,F_y A_{gv} = 0.6(36)(17.9) = 386^k$

$\phi R_n = 0.75[368^k + (1.0)(65.5)] \leq 0.75[386 + 65.5]$

$325 \leq 338.8$

$338.8 < 348^k \therefore$ NO GOOD

TRY 1" ℞ w/ 3.34" (22/3d_b) SPACING

$A_{gv} = [2" + 8(3.34")][1] = 28.72\text{ in}^2$

$A_{nv} = 28.72 - 8.5(1\,1/4 + 1/8) = 17.03\text{ in}^2$

$A_{gt} = 2.5\text{ in}^2$

$A_{nt} = 2.5\text{ in}^2 - 1/2\,(1\,1/4 + 1/8) = 1.81\text{ in}^2$

$F_u A_{nt} = 58\text{ ksi}\,(2.5\text{ in}^2) = 145^k$

$0.6\,F_u A_{nv} = 0.6(58)(17.03) = 592^k$

$0.6\,F_u A_{gv} = 0.6(36)(28.72) = 620^k$

$\phi R_n = 0.75[592 + 145] \leq 0.75[620^k + 145^k]$

$552 \leq 573^k$

$573^k > 348^k \therefore$ ✓

10 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date

Project No.

Drawn By

- DETERMINE LOADS AT BEAM-TO-SINGLE-PLATE CONNECTION

$V_u = V_{ub} - R_{ub} = 254^k - 26.8^k = 227.2^k$

$H_u = H_{uc} + H_{um} + \Omega_o P_{oE} = 127^k + 40.3^k + 100^k = 267.3^k$

$R_u = \sqrt{227.2^2 + 267.3^2} = 351^k$

- REQUIRED NUMBER OF BOLTS

$N_b = \dfrac{351^k}{55.2^k/_{BOLT}} = 6.35 = 7$ BOLTS

* USE 7 1 1/4" A490N BOLTS IN STD HOLES

LOCATE BOLTS 2 1/2" FROM COLUMN FACE

- SINGLE PLATE BEARING

$\phi R_n = 7\,(105\ \text{kips/in})(1") = 735^k > 351^k \therefore$ ✓

- BEAM WEB BEARING

BEAM WEB IS THICKER THAN PLATE WITH SAME BEARING ∴ ✓

- BEAM WEB YIELDING

$\phi R_n = \phi(0.6F_y)\,d\,t_w$

$= 1.0(0.6)(50\text{ksi})(30)(1.26) = 1{,}130^k \gg 351^k$

- BLOCK SHEAR RUPTURE OF BEAM-TO-SINGLE-PLATE CONNECTION

$A_{gv} = [1.5 + 7(3.34)] = 35\ \text{in}^2$

$A_{nv} = 35\ \text{in}^2 - 6.5(1\ 1/4 + 1/8)(1) = 26\ \text{in}^2$

$A_{gt} = 2.5\ (1) = 2.5\ \text{in}^2$

$A_{nt} = 2.5 - 1/2(1\ 1/4 + 1/8)(1) = 1.81\ \text{in}^2$

$F_u A_{nt} = 1.81(58\text{ksi}) = 105^k$

$0.6 F_u A_{nv} = 905^k$

$0.6 F_y A_{gv} = 756^k$

$\phi R_n = 0.75\,(756^k + 105^k) \le 0.75\,(905^k + 105^k)$

$645^k \le 757^k$

$645^k > 351^k \therefore$ ✓

11 OF

SmithGroup General Work Sheet

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject

Date — Project No.

Drawn By

- SHEAR YIELDING OF SINGLE PLATE

$L = 36" + \frac{1}{2}(30") + 3(3") + 1.5" = 61.5"$

COMP CONTROLS

$V_u = V_{uc} + V_{u6} - R_{u6} = 305 + 254 - 26.8 = 532.2^K$

SHEAR YIELDING

$\hookrightarrow \phi R_n = \phi(0.6F_y)A_g = 1.0(0.6)(36)(61.5)(1) = 1328^K$

$1328^K \gg 532K \therefore \checkmark$

- SHEAR RUPTURE OF SINGLE PLATE

$A_n = [61.5 - 13(1\,1/4 + 1/8)](1") = 43.6\ IN^2$

$\phi R_n = 0.75(0.6)(58)(43.6) = 1137^K \gg 532^K \therefore \checkmark$

* 1" PLATE ADEQUATE

- DESIGN SINGLE-PLATE-TO-COLUMN CONNECTION

$V_u = V_{u6} - R_{u6} = 227^K$

$H_u = H_{uc} + \Omega_o P_{QE} = 127^K + 100^K = 227^K$

$R_u = \sqrt{227^2 + 227^2} = 321^K$

CONNECTION LENGTH

$6(3") + 2(1\,1/2") = 21"$

$S_r = \frac{321}{21} = 15.3\ K/IN$

LOAD ANGLE $\theta = TAN^{-1}\left(\frac{H_u}{V_u}\right) = TAN^{-1}\left(\frac{227}{227}\right) = 45°$

FILLET WELD STRENGTH

$\phi r_n = 1.392\left(1.0 + 0.5\sin^{1.5}(45)\right) = 1.80\ K/IN$

$D_{min} = \frac{15.3}{2(1.8\ K/IN)} = 4.23 \rightarrow$ USE $5/16"$

* USE DOUBLE SIDED 5/16" FILLET WELDS

12 of

SmithGroup **General Work Sheet**

SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

Subject
Date Project No.
Drawn By

• CHECK SINGLE-PLATE FRACTURE AT SINGLE-PLATE-TO-COL WELD

$t_{min} = \frac{6.19(5)}{58} = 0.534'' < 1'' \therefore ✓$

* USE 1" THICK SINGLE PLATE CONNECTION BETWEEN BEAM/GUSSET AND COL

• CHECK LOCAL YIELDING AT COLUMN WEB

SM 16.1-116 $\phi R_n = \phi(5k + N) F_{yw} t_w$ (H_{uc})

$= 1.0(5(1.79) + 24)(50)(0.44'') = 724^k >> 127^k \therefore ✓$

• CHECK LOCAL WEB CRIPPLING OF COLUMN

TBL 9-4 W14 x 159 $\phi R_3 = 253^k$ $\phi R_4 = 25.1$ k/in $(H_{uc} + \Omega_o P_{QE})$

$\phi R_n = 2[253 + 24(25.1)] = 1710^k >> 227^k$ ✓

• COLUMN WEB SHEAR

$\phi R_n = 1.0(0.6)(50)(15)(0.745) = 335^k > 227^k \therefore ✓$

• VERIFY FLEXURAL STRENGTH OF CONNECTION

W14x90 $1.1 R_y M_p = 1.1 R_y F_y Z_y = 1.1(1.1)(50)(75.6) = 4573^{k\text{-}in}$

- DIST BETWEEN CENTERLINES OF CONNECTION PLATES:

$d_{cp} = b_f + t_{cp} = 14.5 + 1 = 15.5''$

$T_{cp} = C_{cp} = \frac{4573^{k\text{-}in}}{15.5''} = 295^k$

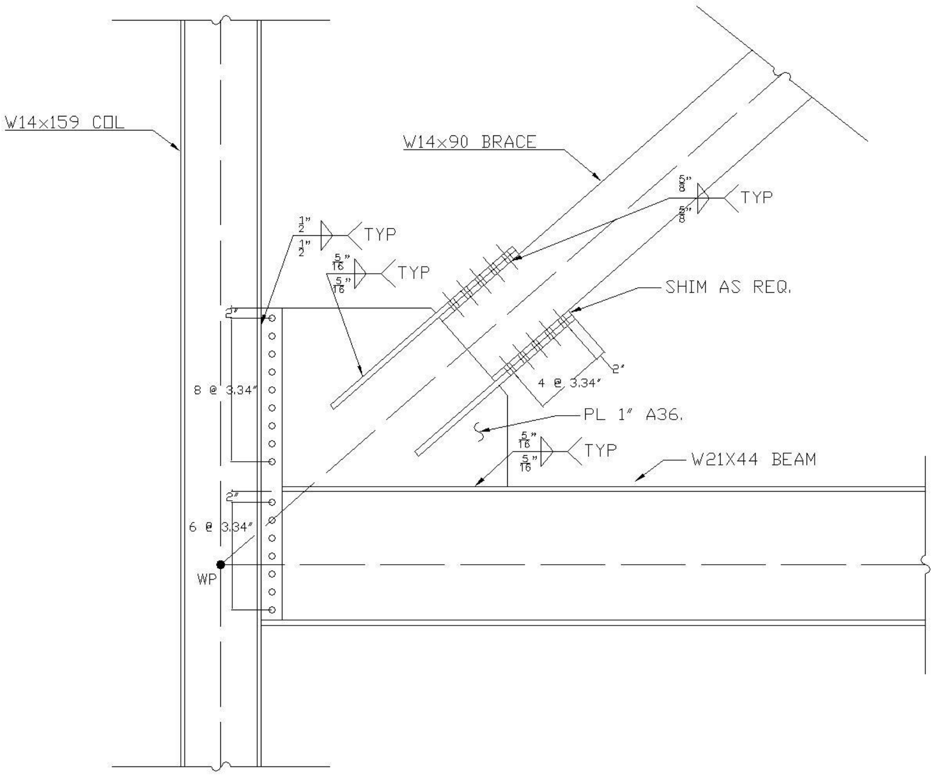
$H_{uc} = \frac{1.1 R_y M_p}{e_b + \beta} = \frac{4573^{k\text{-}in}}{15'' + 18''} = 139^k$

$H_{ub} = -H_{uc} = -139^k$

$V_{ub} = V_{uc} = 0$

$M_{ub} = 1.1 R_y M_p - H_{uc}\beta = 4573^{k\text{-}in} - 139^k(18'') = 2071^{k\text{-}in}$

13 OF



Final Connection Design

Torsion Analysis

Center of Rigidity

RAM Frame v12.1
DataBase: New Column Layout With Braces 03/26/09 00:37:41

CRITICIA:

Rigid End Zones: Ignore Effects
Member Force Output: At Face of Joint
P-Delta: Yes Scale Factor: 1.00
Ground Level: Base
Wall Mesh Criteria :
Max. Allowed Distance between Nodes (ft) : 8.00

		Centers of Rigidity		Centers of Mass	
Level	**Diaph. #**	**Xr**	**Yr**	**Xm**	**Ym**
		ft	**ft**	**ft**	**ft**
Fifth-PH Floor	1	70.01	199.96	67.20	208.57
Fourth	1	70.00	199.96	62.57	205.28
Third	1	69.98	199.98	70.46	207.84
Second	1	69.94	200.01	69.63	198.58
First	1	69.71	200.08	76.74	147.73

	COM		COR		Eccentricity		5% (Used by RAM)	
	x	y	x	y	x	y	x	y
5PH	67.2	208.57	70.01	199.96	2.81	-8.61	7.2	13.76
4	62.57	205.28	70	199.96	7.43	-5.32	7.2	13.76
3	70.46	207.84	69.98	199.98	-0.48	-7.86	7.2	13.76
2	69.63	198.58	69.94	200.01	0.31	1.43	7.13	12.44
1	76.74	147.73	69.71	200.08	-7.03	52.35	9.97	16.13

	Frame 1		Frame 2		Frame 3		Frame 4	
N-S	x	y	x	y	x	y	x	y
Coord	40	250	40	130	100	270	100	130
5PH	30.01	-50.04	30.01	69.96	-29.99	-70.04	-29.99	69.96
4	30	-50.04	30	69.96	-30	-70.04	-30	69.96
3	29.98	-50.02	29.98	69.98	-30.02	-70.02	-30.02	69.98
2	29.94	-49.99	29.94	70.01	-30.06	-69.99	-30.06	70.01
1	29.71	-49.92	29.71	70.08	-30.29	-69.92	-30.29	70.08

	Frame 5		Frame 6		Frame 7			
E-W	x	y	x	y	x	y		
Coord	55	280	55	220	55	100		
5PH	15.01	-80.04	15.01	-20.04	15.01	99.96		
4	15	-80.04	15	-20.04	15	99.96		
3	14.98	-80.02	14.98	-20.02	14.98	99.98		
2	14.94	-79.99	14.94	-19.99	14.94	100.01		
1	14.71	-79.92	14.71	-19.92	14.71	100.08		

TORSION

x	Vtot	M	1	2	3	4		5	6	7
5ph	1073	7725.6	8.994615	8.994615	-8.98862	-8.98862		-31.6663	-7.92845	39.54729
4	1882	13550.4	15.77095	15.77095	-15.7709	-15.7709		-55.5415	-13.9062	69.3644
3	2482	17870.4	20.78501	20.78501	-20.8127	-20.8127		-73.2304	-18.3213	91.49677
2	2895	20641.35	23.97586	23.97586	-24.072	-24.072		-84.5536	-21.1305	105.7157
1	3160	31505.2	36.3131	36.3131	-37.022	-37.022		-128.941	-32.1384	161.4663

y	Vtot	M	1	2	3	4		5	6	7
5ph	1073	14764.48	17.18971	17.18971	-17.1783	-17.1783		-60.5179	-15.1521	75.57927
4	1882	25896.32	30.14003	30.14003	-30.14	-30.14		-106.146	-26.5763	132.5631
3	2482	34152.32	39.72247	39.72247	-39.7755	-39.7755		-139.951	-35.0141	174.8605
2	2895	36013.8	41.83165	41.83165	-41.9993	-41.9993		-147.524	-36.8672	184.4465
1	3160	50970.8	58.74927	58.74927	-59.8962	-59.8962		-208.607	-51.9952	261.2288

DIRECT SHEAR

x	Vtot	1	2	3	4
5ph	1073	268.25	268.25	268.25	268.25
4	1882	470.5	470.5	470.5	470.5
3	2482	620.5	620.5	620.5	620.5
2	2895	723.75	723.75	723.75	723.75
1	3160	790	790	790	790

y	Vtot	5	6	7	
5ph	1073	354.09	354.09	354.09	
4	1882	621.06	621.06	621.06	
3	2482	819.06	819.06	819.06	
2	2895	955.35	955.35	955.35	
1	3160	1042.8	1042.8	1042.8	

TOTAL
SHEAR

x	1	2	3	4
5ph	277.2446	277.2446	259.2614	259.2614
4	486.2709	486.2709	454.7291	454.7291
3	641.285	641.285	599.6873	599.6873
2	747.7259	747.7259	699.678	699.678
1	826.3131	826.3131	752.978	752.978

y	5	6	7	
5ph	293.5721	338.9379	429.6693	
4	514.914	594.4837	753.6231	
3	679.1086	784.0459	993.9205	
2	807.826	918.4828	1139.797	
1	834.1928	990.8048	1304.029	

Drift Analysis using SAP2000

E-W

Story	Height	Story Drift	.025hsx	Acceptable	Total Drift	.025hsx	Acceptable
5ph	72.67	0.51	4.2	Yes	2.81	21.801	Yes
4	58.67	0.6	4.2	Yes	2.3	17.601	Yes
3	44.67	0.61	4.2	Yes	1.7	13.401	Yes
2	30.67	0.77	5.001	Yes	1.09	9.201	Yes
1	14	0.32	4.2	Yes	0.32	4.2	Yes

N-S

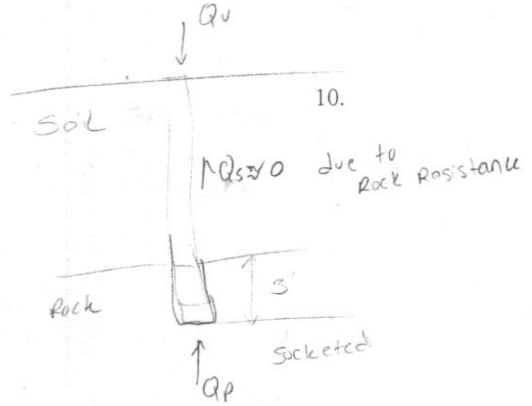
Story	Height	Story Drift	.025hsx	Acceptable	Total Drift	.025hsx	Acceptable
5ph	72.67	1.2	4.2	Yes	5.57	21.801	Yes
4	58.67	1.25	4.2	Yes	4.37	17.601	Yes
3	44.67	1.16	4.2	Yes	3.12	13.401	Yes
2	30.67	1.48	5.001	Yes	1.96	9.201	Yes
1	14	0.48	4.2	Yes	0.48	4.2	Yes

Foundation Design

CE 513 - ADVANCED FOUNDATION DESIGN

Dr. WANG

10.



Piles on Rock –

Point Bearing Capacity –

$$q_p = q_u (N_\varphi + 1)$$

in which

$$N_\varphi = \tan^2(45 + \phi'/2)$$

φ' = drained internal friction angle of rock

q_u = unconfined compressive strength of rock

$q_u = q_{u(lab)}/5$

1) Rock is not uniform
2) there might be fracture in field
3) progressive

Table 11.8 Typical Unconfined Compressive Strength of Rocks (lab test data)

Type of rock	q_u MN/m²	q_u lb/in²
Sandstone	70–140	10,000–20,000
Limestone	105–210	15,000–30,000
Shale	35–70	5000–10,000
Granite	140–210	20,000–30,000
Marble	60–70	8500–10,000

Table 11.9 Typical Values of Angle of Friction ϕ' of Rocks

Type of rock	Angle of friction, ϕ' (deg)
Sandstone	27–45
Limestone	30–40
Shale	10–20
Granite	40–50
Marble	25–30

$q_a = \frac{q_p}{FS}$ $FS = 3.0$ min

$Q_a = q_a A_p$

SmithGroup
SmithGroup Incorporated
110 Miller Avenue
Ann Arbor, Michigan 48104
t: 734 769 9444
f: 734 769 0999
www.smithgroup.com

General Work Sheet

Subject

Date

Drawn By

Project No. FOUNDATION DESIGN

HP 12 x 84

$A_s = 24.6 \text{ IN}^2$

3 - PILE FOUNDATION

$A_s = 3(24.6) = 73.8 \text{ IN}^2$

USING LIMESTONE AS IDENTIFIED BY USGS AS PREVALENT IN THE AREA

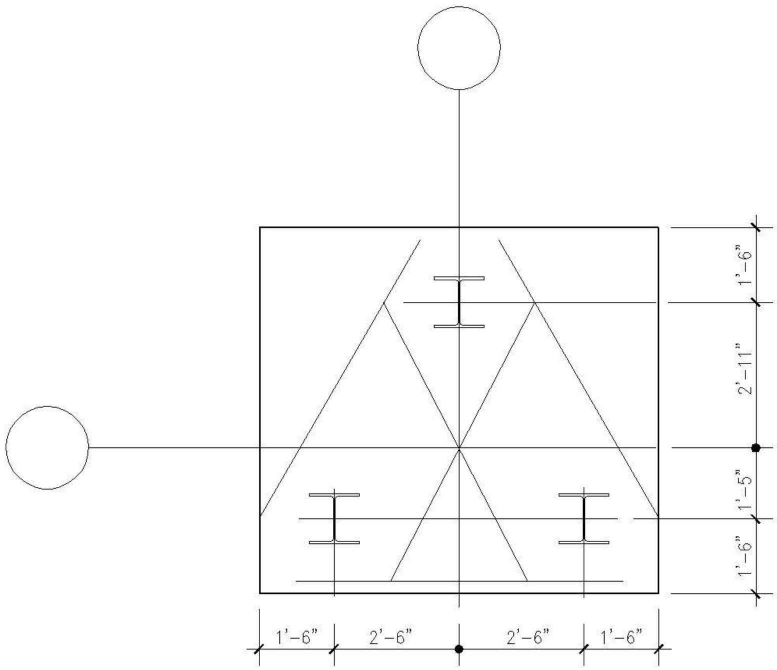
ANGLE OF FRICTION: $\phi = 35°$

STRENGTH $q_u = (25{,}000 \text{ PSI})(73.8 \text{ IN}^2) = 1845^K$

$N_\phi = \tan^2(45 + \phi/2) = \tan(45 + 35/2) = 1.92$

$q_r = q_u(N_\phi + 1) = 1845^K * (1.92 + 1) = 5390^K$ ✓

5390^K MUCH GREATER THAN CRITICAL FOUNDATION FORCES.



5
TYP
TYPICAL PILE CAP DETAIL
3 H PILES – TYPE 3A
SCALE: NTS

Sample BRBF vs SCBF Cost Analysis

DASSE

Dasse Design Inc.
Structural Engineers

Table 3: LFRS Material Quantities and Costs

	Item	BRBF		SCBF		BRBF Savings	
6 STORY	Columns	52 Tons	$139,730	72 Tons	$213,200	20 Tons	$73,470
	Braces	N/A	$242,345	34 Tons	$168,249	N/A	-$74,096
	Connections	N/A	$128,546	N/A	$290,584	N/A	$162,038
	Frame Beams	Typical Beams Adequate	$0	13 Tons	$27,500	13 Tons	$27,500
	Piles & Pile Caps	16 Piles 48 yd³	$75,200 $25,440	40 Piles 83 yd³	$188,000 $43,990	24 Piles 35 yd³	$112,800 $18,550
	Spread Footings	145 yd³	$76,850	413 yd³	$218,890	268 yd³	$142,040
	Total Cost - Pile Foundation		$611,261		$931,523		**$320,262**
	Total Cost - Spread Footings		$587,471		$918,423		**$330,952**
3 STORY	Columns	15 Tons	$39,000	23 Tons	$58,800	8 Tons	$19,800
	Braces	N/A	$120,430	17 Tons	$60,740	N/A	-$59,690
	Connections	N/A	$62,230	N/A	$102,230	N/A	$40,000
	Frame Beams	Typical Beams Adequate	$0	13 Tons	$30,000	13 Tons	$30,000
	Piles & Pile Caps	8 Piles 12 yd³	$37,600 $6,360	16 Piles 23 yd³	$75,200 $12,190	8 Piles 9 yd³	$37,600 $5,830
	Spread Footings	50 yd³	$26,500	110 yd³	$58,300	268 yd³	$31,800
	Total Cost - Pile Foundation		$265,620		$339,160		**$73,540**
	Total Cost - Spread Footings		$248,160		$310,070		**$61,910**

The cost savings generated by the BRBF systems is more significant at taller buildings, as the greater quantities of material utilized offsets the premium paid for the BRBF members. In addition, the period and base shear advantage of BRBF buildings to SCBF buildings increases with building height. Figure 4 demonstrates LFRS cost relative to building height for each of the model buildings.