# FEASIBILITY STUDY OF PARTIALLY RESTRAINED CONNECTIONS

by

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(ABSTRACT)

In recent years the idea of using partially restrained connections in building structures has become more practical and economical. Partially restrained connections resist moment and also allow rotation, therefore distributing the moments and stresses more evenly throughout the element. Combining this idea with steel joists, which are also quite common in construction, makes for shallower story heights and lower steel weights. This initial study analyzes partially restrained connections for both hot rolled shapes and steel joists using non-composite and composite construction. The designs are than compared with respect to complexity, practicality, serviceability and economics. The results of this study show that partially restrained joist connections are economically superior to comparable hot rolled member designs.

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# Chapter 1

# INTRODUCTION

There are many factors in building design that must be considered to produce efficient and practical structures. These factors include purpose, cost, materials, serviceability and aesthetics. In structural design, the factor that is under the direct control of the engineer is materials. For the structural designer to be cost efficient and profitable, he must consider all materials and their use within the structure.

For structures of three stories or more, the choice of material types typically comes down to using steel or concrete. A design firm will produce a typical design using both materials along with a cost estimate for each. They will then present these options to the owner. In this study, the main goal is to compare building designs using two basic types of structural steel to see which is most cost efficient. The two types of structural steel used are hot rolled steel sections and steel joists.

These two types of steel members can be used in several different types of construction. The two types that will be presented here are non-composite and composite construction. The first represents the oldest and most widely used type of construction in building design. In non-composite construction, the applied loads are supported by the steel beams and girders with no interaction with the accompanying concrete slab. The structural steel resists all the forces that come from dead, live, snow, wind, and seismic loads that are typical on buildings. The concrete slab is assumed to provide no strength or resistance to the building structure. The slab, in non-composite construction design, is used only to transfer the floor loads to the beams and girders.

In composite construction, the slab is used additionally to provide strength and resistance to the applied loads through composite action with the beam or joist. The slab is anchored to the steel beams and girders in such a way as to provide a greater resistance to bending in the floor members. Composite construction has many benefits such as smaller floor deflections and shallower steel sections. By using the concrete slab the steel section can be reduced and live load deflections are minimized.

Once the type of construction has been chosen, the next step is to choose the type of steel section to use. The choices are hot rolled steel sections or steel joists. The hot rolled sections are designed based on the LRFD Specifications and can be picked from available databases (AISC 1994). Hot rolled section behavior has been thoroughly researched and these sections are easily obtained from many different manufacturers. A disadvantage to using hot rolled shapes is the complexity of the connection analysis and many limit state checks. The reasons to select steel joists include their low weights, greater stiffness per unit weight and ease of construction. Also, because they are essentially trusses, the duct work can be placed between the web members thereby potentially reducing floor-to-floor heights. The disadvantage to using steel joists are there poor vibration characteristics and the need for bridging. Hot rolled shapes and steel joists can both be designed with either non-composite or composite construction.

The final decision to be made in the design is what type of connections to use in the structure for stability. In common practice, the decision has been to use either moment resisting

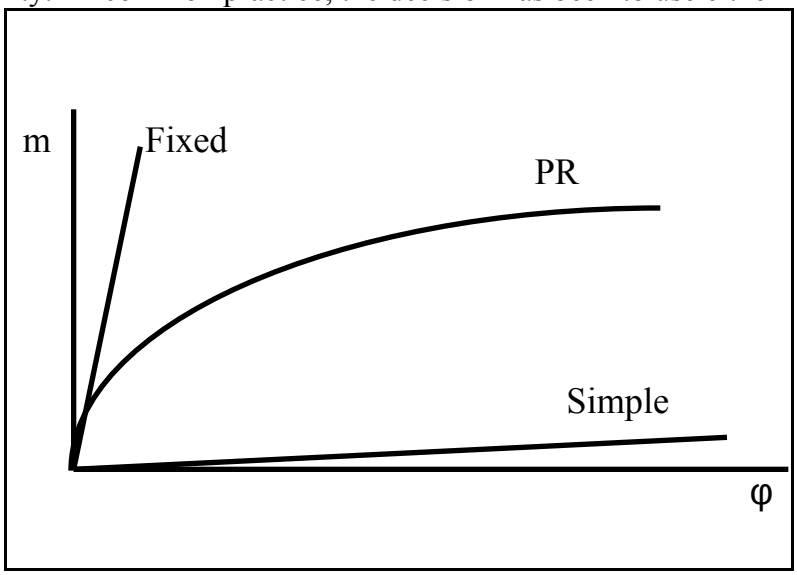


Figure 1.1 Moment Rotation Curve

frames or bracing. These two types have both benefits and drawbacks in both construction and practicality. In the case of moment resisting frames, the cost of fabricating and erecting is quite high relative to braced frames. Moment connections require more expensive details in terms of fabrication and significant field welding which can increase construction time and cost. The other alternative is the use of braced frames. Bracing consists of structural members that are placed diagonally in a bay, or in other arrangements, to resist drift. These braces tend to interfere with the architectural plans and limit the layout of the building.

In recent years a new type of lateral resisting frame, referred to as partially restrained, is receiving a great deal of attention. These frames obtain their strength from partially restrained connections. In the AISC-LRFD Specifications, two types of construction are described (AISC 1994). The first being Type FR which uses a fully restrained connection, which resists all moment and allows little rotation. The second being Type PR which uses pinned and partially restrained connections. These connections resist some moment and allow some rotation. The moment rotation curves for various connections are shown in Figure 1.1. In practice, it is not common to use a partially restrained frame to resist lateral loads such as wind or seismic, but in actuality, these frames are more ductile. They tend to be more flexible in cases of extreme events, dissipating energy and preventing abrupt failures due to fracture. The reason they are not used in practice is because of the limited code specifications on this type of design and the relative complexity of the modeling process.

The primary objective of this report is to conduct an economic study using a typical four story steel building. Comparisons are made between non-composite and composite construction, hot rolled sections and joists, and between partially restrained, braced and fully restrained frames.

The layout of the building, loads applied, and material properties are presented in Appendix A. The footprint of the building is 100 ft by 150 ft and it is 60 ft tall. A penthouse is located on the roof that is 28 ft by 50 ft and it is 10 ft tall. It is assumed that all column to footing connections are fixed since this provides more overall frame stiffness compared to pinned footing connections. A list of the eight framing designs is presented in Table 1.1.

The first set of designs compares the cost of using non-composite hot rolled sections versus steel joists. Braced frames are used for the lateral resisting system in the north-south direction and a moment resisting frame is used at varying frame lines in the east-west direction.

The next set of designs compares the use of partially restrained connections in hot rolled sections versus steel joists. The partially restrained connections are used in both directions at all frame lines to resist the effects of wind. The construction is non-composite.

The third set of designs compares composite hot rolled sections and composite steel joists. Once again, as in the second set, partially restrained connections are used at all frame lines.

The last set of designs are similar to the previous set, except that all beam to girder connections are also partially restrained composite construction.

The types of connections used and typical details of the connections are presented in Chapter 6. Each of the designs, modeling, connection details, and analysis are explained in the pages to come. The following specifications and codes are used to complete the designs: AISC-LRFD Specifications (AISC 1994) and ASCE 7-95 Minimum Design Loads for Buildings and Other Structures (ASCE 1995). The Means Building Construction Cost Data (Means 1994) is used for the cost estimates. The design procedures and joist specifications are described in subsequent chapters.

| Design | N-S Frame Construction | E-W Frame Construction | Girders/Joist Girders | Girder/Joist Girder Connections | Beams/Steel Joists | Beams/Steel Joists Column Connections | Beams/Steel Joists Girder Connections |
|---|---|---|---|---|---|---|---|
| I | Braced | Moment | Hot-Rolled | FR | Hot-Rolled | Simple | Simple |
| II | Braced | Moment | Joist Girders | FR | Steel Joists | Simple | Simple |
| III | PR | PR | Hot-Rolled | PR-Steel | Hot-Rolled | PR-Steel | Simple |
| IV | PR | PR | Joist Girders | PR-Steel | Steel Joists | PR-Steel | Simple |
| V | PR | PR | Hot-Rolled | PR-Composite | Hot-Rolled | PR-Composite | Simple-Composite |
| VI | PR | PR | Joist Girders | PR-Composite | Steel Joists | PR-Composite | Simple-Composite |
| VII | PR | PR | Hot-Rolled | PR-Composite | Hot-Rolled | PR-Composite | PR-Composite |
| VIII | PR | PR | Joist Girders | PR-Composite | Steel Joists | PR-Composite | PR-Composite |

Table 1.1 Framing Designations

# Chapter 2

# LITERATURE REVIEW

There has been much research conducted in the past several years on the analysis of partially restrained connections in building design. The research includes the analysis of several types of partially restrained connections to determine their moment rotation behavior, and the development of design procedures for partially restrained frames using these connections with hot rolled structural steel members. The research that has been done in this area is quite extensive so only research pertaining to the type of connections and the types of partially restrained frames used in this work will be discussed here.

The most important part of any partially restrained connection analysis is to obtain an accurate and easy way to determine the moment rotation curve. To develop these curves many different mathematical models have been developed such as linear, bilinear, mulitlinear, polynomial, cubic b spline, power, and exponential (Wolfram 1996). In many cases the power model has been chosen for its ease of use and its accurate representation of the moment rotation curve.

The power model is expressed in the following form for partially restrained connections:

$$m = \frac{\theta}{(1+\theta^{n})^{1/n}} \qquad (2.1)$$

where $m = M/M_u$ or the connection moment M, divided by the ultimate connection moment $M_u$, $\theta = \theta_r/\theta_o$, or the relative rotation between beam and column $\theta_r$, divided by the relative plastic rotation, $\theta_o = M_u/R_{ki}$ which is the ultimate connection moment divided by the initial connection stiffness $R_{ki}$, and n is power that changes the shape of the curve. The larger the value of n, the steeper the transition of the moment rotation curve from the yield point to ultimate. Some of the effects that are neglected in this model are those due to torsion, lateral bending, shear, and strain. These effects tend to be minimal in normal connections and symmetric buildings so these omissions can be justified.

The power model has been shown to be accurate in predicting connection behavior (Liew, et al 1993). Generalized connection parameters and equations have been developed for framing angle connections, such as single and double angle, top and seat angle, and top and seat angle with double web angle connections, to aid in the analysis. These connections are illustrated in Figure 2.1. These parameters and the analysis of partially restrained top and seat angle, and top and seat angle with double web angle connections are given in Appendix B. The top and seat angles make up the moment couple in the connection while the double web angles resist shear. In the design of the members with partially restrained connections, the first step is to design the member as if fully restrained at its ends. Using that member, the partially restrained connection can be

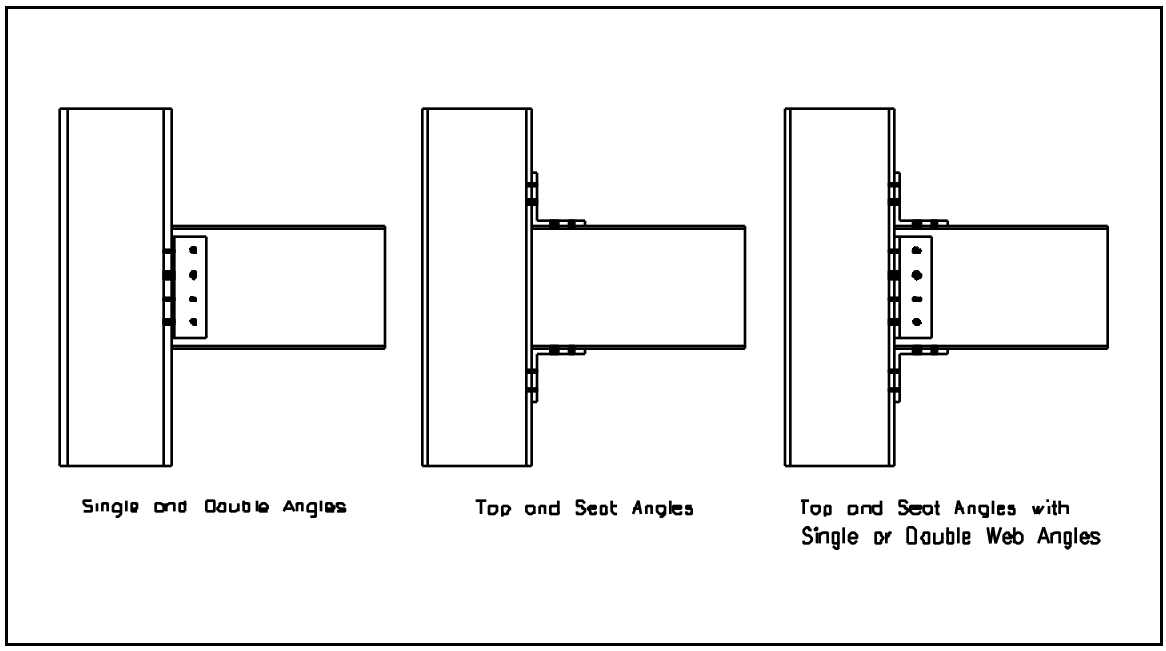


Figure 2.1 Partially Restrained Connections

modeled and the frame analyzed with the selected members. This design is than altered based on the moments calculated using a second order frame analysis. Several iterations may be needed to produce the most efficient design. The ideal result in any partially restrained frame is to balance the end and mid span moments. Doing such will produce the most efficient frame design. The top and seat angle with double web angle connection is used exclusively in the non-composite, hot rolled member construction designs. This is done because this connection proves to be stiffer, thus reducing lateral deflections.

Another type of connection used in this study is partially restrained composite connections. This connection is detailed in Figure 2.2. It consists of a bottom angle and slab reinforcement which form the moment couple. The double angles are assumed to carry the shear in the connection and are designed as such. The behavior of this connection has also been studied (Leon 1994) and general connection specifications have been developed (Leon, et al 1994).

This connection utilizes the additional strength and stiffness provided by the floor slab which is developed by adding shear studs and slab reinforcement in the negative moment regions (tension on top). The advantage of this connection is that it can be easily detailed to limit strength by the amount of top reinforcement with the double web angles providing adequate stiffness. The addition of steel reinforcement and composite action provide added ductility and capacity. The connection also avoids problem areas such as local buckling, shear yielding at the panel zone, and the formation of weak column and strong beam mechanisms. The use of slab steel will result in a tension yield, unlike angle

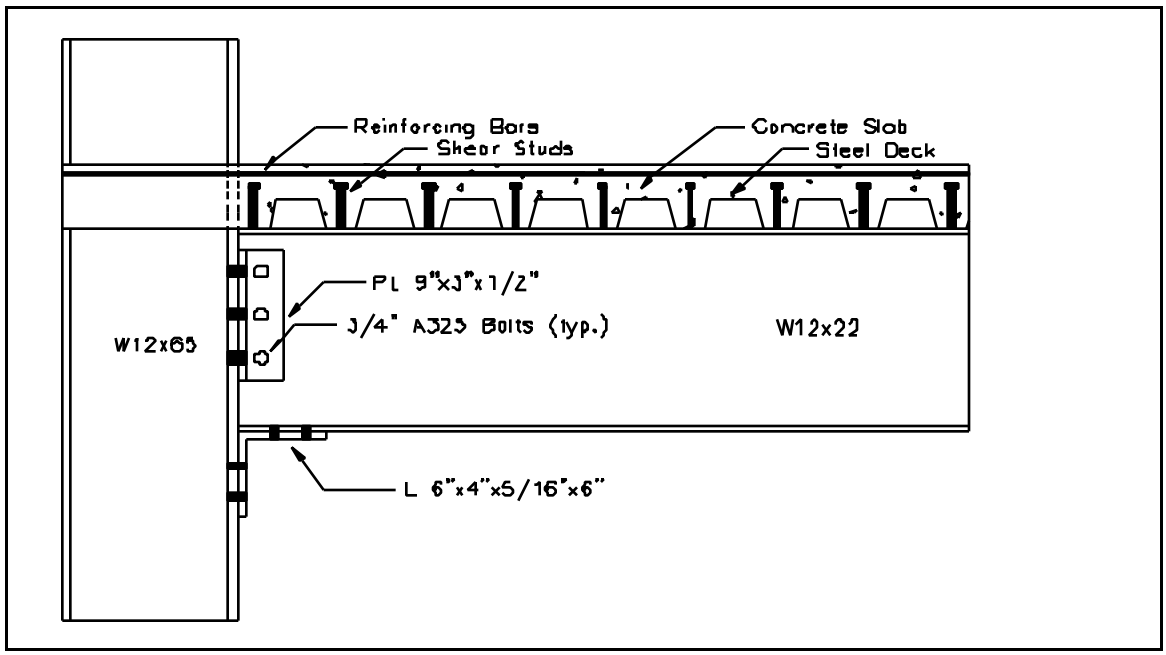


Figure 2.2 Partially Restrained Composite Connection

connections which yield both axially and in bending. The design and analysis of the connection can be found in Appendix B.

The design procedure for partially restrained connections is as follows (Leon 1996):

1. Determine the beam size based on construction loads assuming pinned end conditions.
2. Size the composite section based on factored gravity loads also using pinned end conditions.
3. Determine column sizes based on a rigid frame analysis and use the moment of inertia found from the following equations:
   $I_{eq} = 0.6I_{LB} + 0.4I_n$ (for partially restrained connections on both ends) (2.2)
   $I_{eq} = 0.75I_{LB} + 0.25I_n$ (for simple connection on one end) (2.3)
   were $I_{eq}$ is the resulting moment of inertia, $I_{LB}$ is the composite moment of inertia and $I_n$ is the moment of inertia at the point of connection.
4. Perform a lateral load analysis using software that can represent the non-linear behavior of the connections and the frame.
5. Check all members for strength based on the resulting model forces. Check all connections to assure they are designed for the calculated forces.
6. Detail the connections.

For service loads, the connections act similar to rigid connections, while at ultimate they provide excellent ductility and energy-dissipation capacity. The end result is a decrease

in drift and second order p-delta effects, as compared to non-composite construction, while providing redundancy at little additional expense.

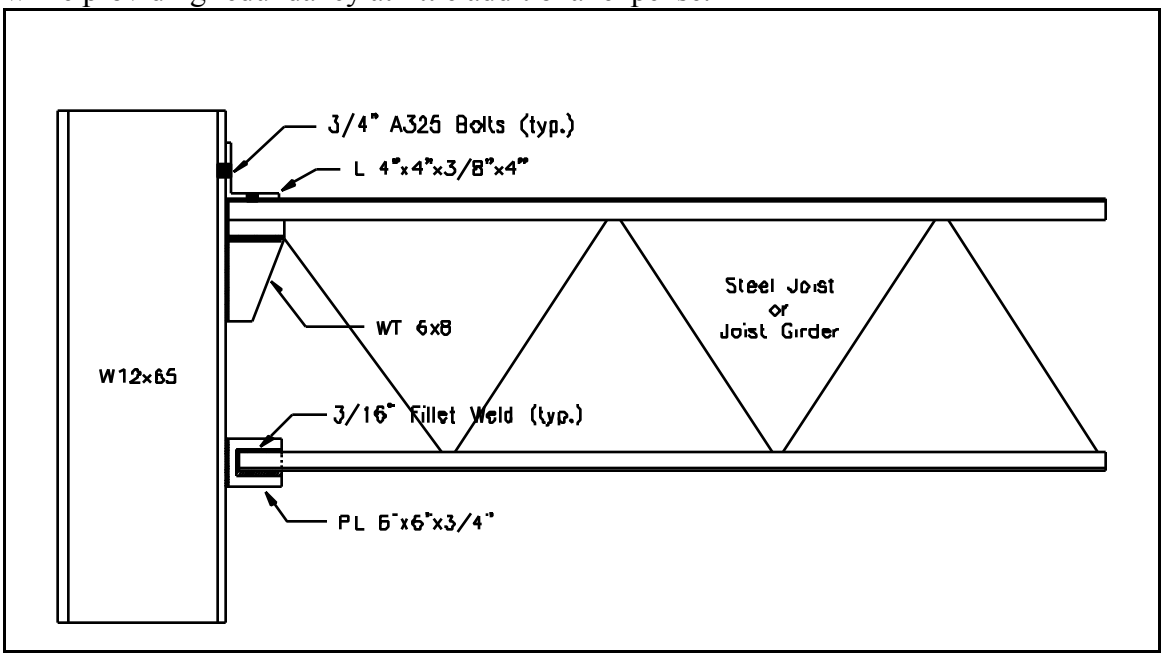


Figure 2.3 Partially Restrained Non-Composite Steel Joist

The use of steel joists in design and construction has been on the increase in recent years. This is due to the practicality and ease of construction of the members. To date, most designs have called for joists to be erected as simple members with no end moment resistance. The use of steel joists in rigid frames can be easily used with some slight modifications. By extending the bottom chord of the joist and welding it to the columns, the joist becomes fixed thus reducing the steel weight needed and the depth of the member (Fisher, et al 1991). This method can be used in both non-composite and composite construction. In non-composite construction, the joist bottom chord can be welded to the column at any time during the construction process. The most economical time is to weld it after the construction load stage, to resist superimposed loads. The reason is that rigidly supported members have larger end moments then midspan moments while simply supported members have only midspan moment. Therefore, the construction load stage produces the majority of the midspan moment while the superimposed loads produces all of the end moment. This two phase analysis balances out the moments over the member length efficiently utilizing the members section properties at its ends and midspan. For composite construction the same process takes place where the joist is simply supported at the construction stage but becomes fixed when the concrete hardens. Composite construction utilizes the slab and steel reinforcement as added strength for the joist. Schematics of these two systems are shown in Figures 2.3 and 2.4. The design of a non-composite and composite joist are similar.

The factors to be considered are the magnitude of continuity, wind and seismic forces, design of the bottom chord for proper forces and the bottom chord connection. The bottom chord force can be found by finding the maximum moment and dividing it by the joist depth. The chord can be connected to the column using a simple plate welded to the

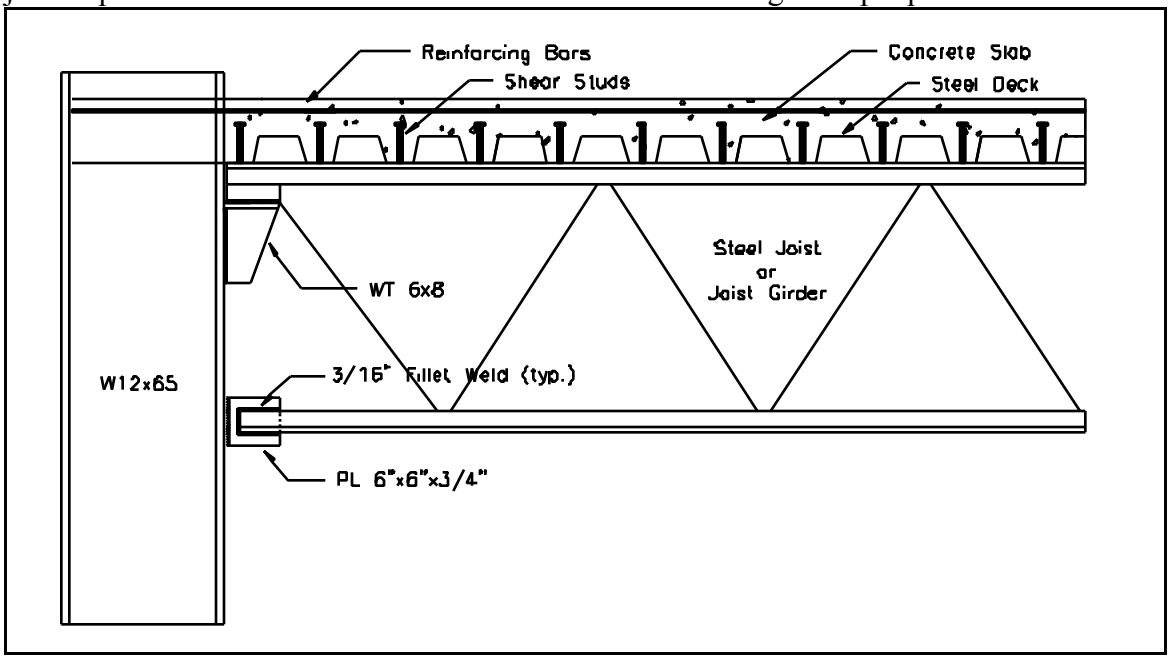


Figure 2.4 Partially Restrained Composite Steel Joist

column and bottom chord. The connection and joist designs are discussed in Chapter 5 along with actual sample designs in Appendix C.

There have been many papers written on the most practical and efficient way to design partially restrained frames. The design procedure that is used in this work is presented in Chapter 3. The guidelines found in research papers revolve around one essential point which is whether or not access to analysis software that includes non-linear effects. The moment rotation behavior of the connections and second order effects is what produces non-linear model behavior. In this case study, a finite element program (ANSYS) is used to analyze the non-linear effects. In many instances, this type of software is too expensive and not cost effective for a small consulting office. Therefore, two procedures that analyze a partially restrained frame are presented along with a third procedure that utilizes a non-linear analysis software package.

The first design procedure is based on the absence of a program capable of accounting for non-linear effects. The procedure is outlined below (King, et al 1993):

Method One:

Rigid Frame Analysis:

1. Perform a first order elastic rigid frame analysis using a structural analysis software package.

2. Use the equations found in King, et al (1993) to calculate second order lateral deflection and the notional loads. The notional loads are represented by the following equation:

$$\Sigma H' = \Sigma H + \Sigma P_U \Delta / L \qquad (2.4)$$

where ΣH` is the notional load, ΣH is the total sum of all the horizontal forces, $\Sigma P_u$ is the sum of all axial loads on a story, Δ is the second-order lateral deflection due to P-Δ effect and L is the story height. The notional load includes the horizontal force plus second order effects.

3. Use the notional loads and gravity loads to perform first order elastic analysis, second order effects are included in this step.
4. Calculate $B_1$ factor using an effective length factor equal to one and multiply times corresponding moments. The $B_1$ factor represents the moment modification factor due to P-δ effects and is given in the LRFD specifications (AISC 1994).
5. Check the interaction equations in Chapter H of the LRFD specifications.

Partially restrained frame analysis:

1. Select connections based on maximum beam-column joint moments in rigid frame analysis.
2. Determine the initial connection stiffness, $R_{ki}$ based on connection type and member size. The initial connection stiffness is calculated in Appendix B for a sample partially restrained connection.
3. Substitute _ $R_{ki}$ for the partially restrained connection.
4. Use _ $R_{ki}$ and the notional loads to carry out a first order elastic analysis.
5. Calculate $B_1$ factor using an effective length factor equal to one and multiply times corresponding moments.
6. Check interaction equations and adjust the G factor for columns using the modified moment of inertia in Appendix D.

This procedure uses a simple tangent connection stiffness as a representation of the partially restrained connection which proves to be quite accurate and is a good estimate of moment rotation behavior. This method gives similar results compared to a non-linear analysis and gives a good estimate of column and beam moments. Finally, the use of notional lateral loads is simple and avoids tedious k factor determination.

The second procedure also does not include a second order analysis and is based solely on the LRFD procedures for design (Barakat, et al 1990). The proposed method is shown below. This method uses a procedure for frame design similar to the LRFD specifications with some simplified modifications, which includes two linearized moment rotation relations and a modified relative stiffness factor G`. The whole procedure is summarized below and all equations are found in Appendix D.

Method Two

1. Determine basic rigid frame parameters for a second order analysis. These parameters are discussed in Appendix D.

2. Determine modification factors for flexible frames as described in Appendix D
3. Using a basic spreadsheet program the parameters can be calculated easily, and the resulting members can be designed

This method follows the design procedures of LRFD specifications more closely.

The final procedure assumes a second order analysis software package is available which incorporates non-linear springs to represent the connection behavior and has the capability to include p-delta effects. The procedure here is as follows (Liew, et al 1993):

Method Three

1. Perform a preliminary analysis using rigid frame action. The magnitude of end moments is based on the gravity load case. This analysis should account for second order effects.
2. Select the type of partially restrained connections to be used. Design the connections based on moments from step one.
3. Determine the ultimate moment capacity and initial connection stiffness using methods proposed by Liew, Chen and White (Liew, et al 1993).
4. Check the limit states of all connections.
5. Perform second order analysis including partially restrained connections and second order effects.
6. Check strength limit states of members and connections.
7. Check serviceability and drift.
8. Preliminary design is based on rigid-frame action so design may be conservative. A more cost effective design may be found.

This procedure is more practical since the number of calculations and iterations has been reduced. The only requirement is access to a non-linear software package.

A major concern in partially restrained frames is lateral drift because the connections are ductile. The flexibility of the frames produce relatively large drift under wind or seismic forces. It is common to limit drift to H/400, where H is the building height and h/250, where h is the inter-story heights. This is not a code requirement but is used by many designers as a serviceability check. These requirements may be difficult to meet so the limit on building height, using partially restrained connections, is usually eight to nine stories (Leon 1990). When considering drift, the loads to be applied during the analysis of lateral deflection are in the range of 1.0D + 0.2L + 1.0W (Liew, et al 1993) to 1.0D + 0.5L + 1.0W (Swensson, et al 1995). For an estimate on drift the following equations may be used (Chen 1991):

Top and seat angles:

$$\frac{\Delta}{H} = \frac{W}{90 + 160\ \frac{B}{H}} \tag{2.5}$$

Flange plates:

$$\frac{\Delta}{H} = \frac{W}{130 + 160\ \frac{B}{H}} \tag{2.6}$$

In these equations $\Delta$ represents the lateral deflection, H is the building height, W is the lateral load intensity (in kips/ft of vertical height) and B is the building width.

The applicability of partially restrained connections has been under scrutiny for many years. One of the leading structural engineering design firms in the United States published a paper to discuss the use and applicability of partially restrained connections in steel frames (Swensson, et al 1995). The major concerns of the paper are design guidelines, connection behavior, and connection applicability.

The first concern is the lack of design guidelines for partially restrained frames. The LRFD specification permits the use of partially restrained connections, to resist lateral loads in unbraced frames, with no explanation on their design or analysis.

The second concern is of connection applicability. The connections should be of similar size and type to those elements tested. Also, most connection models have a maximum moment or rotation beyond which mathematical models no longer apply. The reason is a connection will reach ultimate capacity and then yield. A mathematical model has no cutoff and assumes the connection can continuously rotate or carry moment without failing. This factor is of great importance in design and must be considered when using partially restrained connections in frames. Another consideration in frame analysis is beam stiffness. For frames with relatively flexible connections, the negative moments are small and the beam will act compositely over the majority of its length. The beam is mostly in positive bending and the slab will be in compression throughout the member length. Therefore, the beam is sized for gravity loads and the bare steel or composite properties are used in the frame analysis. For frames with relatively stiff connections, the negative moments are of the same order as the midspan moments, leading to the use of an effective moment of inertia in the frame analysis. The moment of inertia is a weighted average of the steel and composite sections. This will greatly add to the stiffness of the frames.

When considering beam and column moments in frames, the analysis becomes more complicated. Since the frames are non-linear in nature, superposition is an invalid assumption and a non-linear analysis must be utilized to account for both the connections and second order effects. Due to the use of a second order analysis the $B_1$ and $B_2$ factors for LRFD design need not be used. In the case of partially restrained composite connections, a two phase analysis is suggested. For lack of any other analysis method, the frame analysis may be superimposed onto the non-composite dead load condition. The ultimate moment capacity of the connection must be greater than the factored loads.

In the case of columns, the effective length factors for columns need to be modified based on connection stiffness. The use of modified G factor alignment charts are

of limited use due to the connection being loaded into the inelastic region while the adjacent connection unloads. This area still needs more research. There are some basic equations derived to account for connection stiffness. The following equation is used to modify the beam or girder stiffness (King, et al 1993):

$$\alpha = \frac{1}{1 + \frac{2EI}{R_{ki} L}} \tag{2.7}$$

It has been shown that cost savings, about 20%, when comparing partially restrained frames with rigid frames is possible (Bjorhovde, et al 1991). The goal of this study is to show that partially restrained frames are more economical then their rigid or braced frame counterparts.

# Chapter 3

# DESIGN GUIDELINES

In this study, a typical four story office building is designed with story heights of 10 ft to 14 ft and a footprint of 100 ft x 150 ft. The same basic materials are used throughout the design: steel framing with a steel deck and concrete topping. Thus, this building is considered as typical construction.

In designing the eight different framing systems, a simple procedure was followed to promote efficiency and a well thought out design. The eight framing systems are detailed in Table 1.1. This procedure is shown as a simple step by step procedure. It is assumed that the reader has a great deal of knowledge in the design of steel structures and its components so the basic steps will be outlined, not the actual member design processes or the frame analysis.

The design guidelines for steel buildings is presented here:

1. Framing Members (hot rolled or steel joists) - If steel is the material of choice the first step is to choose the type of member to be used, hot rolled steel sections or steel joists.
2. Construction Method (Non-Composite or Composite) - There are two choices for construction, non-composite and composite. Non-composite pertains to the steel members providing the strength and composite meaning that the steel and concrete work together to provide strength.
3. Non-Frame Line Member Design - This step refers to the design of all members not in the frames lines such as the decking, intermediate beams or joists, spandrel beams, etc.
4. Frame Type (Braced or Unbraced) - This refers to the type of system used to resist lateral loads such as wind or seismic. This may be influenced by the height of the structure, aesthetics, size of the lateral forces, etc. The unbraced or sway frames may consist of rigid or partially restrained connections to resist lateral forces.
5. Frame Analysis - Here, a rigid frame analysis is conducted based on the factored gravity loads for sway frames and a braced frame analysis for non-sway frames. Sway frames are frames that resist lateral loads based on the stiffness of the connections and members in the frame, whereas, non-sway frames resist lateral loads based on bracing. This analysis should include second order effects.
6. Moment Resisting Connection Types - This step refers to the type of connection to be used, for example top and seat angles or extended end plates. The type of connection chosen should be based on the stiffness properties and the ultimate capacity of that connection. For taller buildings using sway frames, stiffer connections should be used to reduce drift produced by lateral loads. The connections are designed based on the frame analysis performed in the previous step.
7. Moment Rotation Curves - For connections that are classified as partially restrained a moment rotation curve needs to be defined. This is possible by using some of the newer

methods developed by Chen (1991), Kishi (1993), Liew (1993), and Leon (1996) which are described in Appendix B.

8. Connection Ductility - The connections need to be checked for ductility, meaning the connection will not fail by yielding or fracture. All connections need to yield according to the basic steel deformation curve.
9. Partially Restrained Frame Analysis - If partially restrained connections are used in the frame, a second order non-linear analysis must be performed using the non-linear connection characteristics. This can be done in any program which uses non-linear elements. The members used in the rigid frame analysis and the factored gravity loads are used in this model.
10. Member Strength Requirements - Many of the members will probably need to be redesigned based on the partially restrained analysis. The optimum frame is one in which the end moments of the beams and girders are similar to their respective mid- span moments. This yields the most efficient use of the steel framing members. All members in the frame must meet the respective code requirements. For partially restrained frames the stiffness of the connections must be considered in the column design and the appropriate modifications performed (Barakat, et al 1990)
11. Drift & Serviceability - The following requirements are typical when considering building drift and floor serviceability:
    - H/400 for building drift
    - h/250 for inter-story drift
    - L/240 for roof member live load deflection
    - L/360 for floor member live load deflection

    where H is the building height, h is the story height, and L stands for span length. In determining drift requirements it is common to use the loading combination 1.0D + 0.5L + 1.0W (Swensson, et al 1995). In some instances the building design will depend largely on drift requirements, especially in the case of partially restrained sway frames.
12. Final Design - The design may need to be altered based on the results of the drift and serviceability requirements. All connections should be detailed and member designs checked. Floor vibration due to human activity should be checked but was not in this study..

These design guidelines are used throughout this study and are easily applied to typical building structures.

# Chapter 4

# MODELING

In partially restrained frames one of the most critical analysis steps is the modeling process. The modeling of any structure begins with an accurate representation of its members and components. To be able to do such, allows for a realistic, economic and safe design.

The most difficult part of structural analysis is developing an accurate model that will correctly represent the structural system. In many cases it is impossible to represent any building exactly with a model without making some general assumptions. For instance, structural materials are assumed to deform according to basic mechanics of materials. This assumption is reasonable for modeling purposes but in actuality may deviate due to weather conditions, construction and the actual consistency of the material. In developing a model there are different levels of precision that can be achieved. This usually depends largely on the complexity of the structure, time allocated for design, cost of engineering and the uniqueness of the geometry or loads.

In the designs developed in this project, the structure is of ordinary geometry and loading. The only uniqueness is in the presence of partially restrained connections. This means that many of the modeling methods used on typical building frames will not work. Some of the basic frame analysis methods such as slope deflection, moment distribution, stiffness and flexibility methods can be modified to work with partially restrained connections but tend to be very tedious and complicated. Because most companies use computers in the analysis of frames, there are many software packages designed to analyze structures. Some such programs are RISA, SAP90, and STAAD. The problem is that they can not represent partially restrained connection behavior. So for this reason a general purpose finite element package, ANSYS 5.3, is used to analyze the partially restrained frames.

The first step in a building analysis is to reduce it into basic components. These components include columns, beams and girders, flooring, foundations, and bracing. The next step is to develop a model of these components and their interaction with each other. This is are the frame analysis becomes important. There are two types of frames, non-sway and sway. A sway frame consists of columns with beams and girders that are stiff enough to resist lateral loads. A non-sway frame has all of those components, but uses bracing to resist lateral movement. These types of frames can be analyzed in similar ways using the techniques described above, such as slope deflection and moment distribution. A sway and a non-sway frame are illustrated in Figure 4.1.

In Design's I and II both sway and non-sway frames are used. The other designs use only sway frames. Because a frame analysis depends largely on the properties of the frame components, there must be a basic elastic analysis performed to develop a preliminary model. The components can be designed based on simple structural analysis methods. The preliminary member designs, and their member properties, can than be input into the frame model.

The need for a frame analysis is to understand how the components interact. The individual member properties will determine the stiffness and strength of the frame. The member

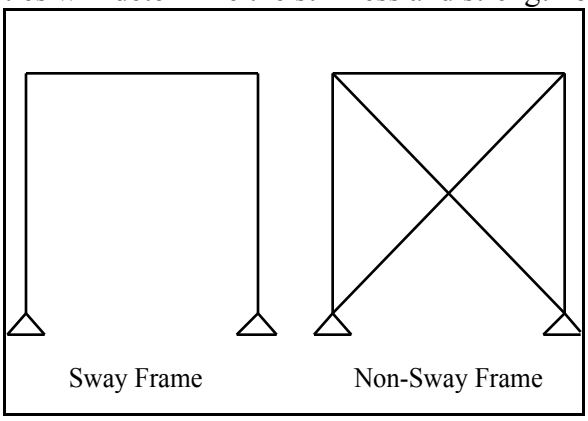


Figure 4.1 Frame Types

properties of the components reflect how the frame deforms and the distribution of loads. As a rule, more force will go to the stiffer element. Therefore, the frame analysis may show that the preliminary designs are not adequate and another iteration is needed.

In the designs done in this study, there is an important uniqueness. Because the frames are partially restrained, special analysis procedures are required. This procedure included the use of a general purpose finite-element program (ANSYS 5.3). The reason for using ANSYS is that it accurately represents the partially restrained connections with a non-linear spring element. A basic ANSYS input file is shown in Appendix E. The other important reason for using ANSYS is that the second-order behavior is evaluated accurately for partially restrained frames.

The non-linear behavior of the partially restrained connection, namely the moment rotation curve, is represented by the non-linear spring. By inputting the curve as data points to define the spring, the connection is represented. If the connection is very stiff the spring acts as a rigid joint while if the connection is very flexible the spring acts as a pin. The use of a spring in this situation allows for the representation of any connection more accurately then a typical pinned or rigid joint. The accuracy in modeling a spring connection can be checked using slope deflection by using the given connection rotation and calculating the moment resistance.

Second order effects in partially restrained design is very important because of the flexible connections. Second order effects are the modification of the column moments due to eccentricity of the axial load caused by deformation of the column. Second order effects in partially restrained connections are less severe because the moments at the ends of the beams and girders are smaller. Nonetheless, these effects need to be considered. Using ANSYS these effects are considered using several iterations until the model results converge to a specified value.

In Design's I and II, the analysis is performed using unbraced and braced frames. Because all the connections are either rigid or pinned, RISA-3D is used for structural analysis. The building is broken up into five different frames, Lines A-D, B-C, 1-6, 2-5, and 3-4 as illustrated in Figure A.1 of Appendix A. Each frame is input into RISA-3D as a two dimensional frame. This

is done because two dimensional frames are easily modeled since three dimensional models tend to be hard to evaluate properly. The components are first designed as individual members and their respective member properties are entered into the model such as area, moment of inertia and elastic modulus. The frame is evaluated using the loads in Appendix A and redesigned if necessary. The final designs and cost estimates can be found in Appendix F. For these designs, bracing is used in the north-south directions and rigid frames are used in the east-west direction. The braces are placed in the interior of bays 2 and 5. The rigid joints are placed in frame lines A and D in the exterior bays and the center bay. The placing of bracing and rigid joints represent the optimum locations in the building to satisfy drift requirements and to minimize inconvenience to the residents.

In Designs III,V and VII, it is essential to use ANSYS for the frame analysis. This is due to the presence of partially restrained connections. In Design III, ANSYS is used to model all the frames using their non-composite components. In Designs V and VII a two phase process is performed. The first phase assumed the connections to be pinned, only the construction dead loads are applied and the non-composite section properties used. The second phase employed ANSYS to analyze the partially restrained behavior of the composite connections. The designs are analyzed as described in Chapter 3. In these designs partially restrained connections are used at all beam-or girder-to-column connections. The cost estimates for the designs are summarized in Appendix F.

In Designs IV,VI and VIII, RISA-3D is used to model the frames containing steel joists and joist girders. This is done in a two stage process. The initial frame analysis assumed the joists to be pinned for construction dead loads. Therefore, the dead loads are applied to a frame with pinned joints at every connection yielding zero moments at beam-or girder-to-column connections. During this phase the joists and joist girders are designed using the non-composite section properties. The second phase fixed all the connections, by connecting the bottom chord, and the factored superimposed and wind loads are applied. In this phase, composite section properties are used for Designs VI and VIII and non-composite for Design IV. The load effects from these two stages are then superimposed to design the members. Samples of the joist and joist girder designs are described in Appendix C. During these two phases an initial section is needed to perform the frame analysis. The initial joist and joist girder designs are derived in the same manner as for the hot rolled sections. The exception is that the loading stages are considered separately and the resulting member forces are summed together to design each joist member.

To summarize the frame analysis process for each of the designs:

Designs I and II:

1. The building is divided into five frames, Lines A-D, B-C, 1-6, 2-5, and 3-4
2. The beam or girder members are designed based on simple or fixed end conditions.
3. The member characteristics are entered into a two-dimensional frame model with the appropriate end conditions.

4. The factored gravity load case is evaluated and the members are redesigned if necessary.
5. The factored lateral load case is evaluated and again the members are redesigned if inadequate.
6. The final check is for service load drift requirement of H/400 using a loading combination of 1.0D + 0.5L + 1.0W.

Designs III, V and VII:

1. The building is divided into five frames, Lines A-D, B-C, 1-6, 2-5, and 3-4
2. The beam and girder members are designed using pinned end conditions for factored dead loads and fixed end conditions for superimposed loads (composite load case only).
3. Moment rotation curves are than derived using the member designs in step two and the type of connection chosen. For Design III it is top and seat angles with double web angles and for Designs V and VII it is composite partially restrained connections.
4. The member characteristics are entered into a two-dimensional frame model using ANSYS with the appropriate partially restrained connection characteristics. For Designs V and VII, the composite designs, a two phase analysis is performed. The first phase used RISA-3D with pinned connections and factored dead loads. The second phase used ANSYS with partially restrained connections and the factored superimposed loads. Here the factored gravity load is checked and the members are redesigned.
5. The factored lateral load case is then evaluated and again the members are redesigned if inadequate. Similar to step four a two phase analysis is used for composite sections, one with the factored dead loads and the second with the factored superimposed and wind loads.
6. The final check is for service load drift requirement of H/400 using a loading combination of 1.0D + 0.5L + 1.0W. This analysis is again like step five. In some instances this is the controlling requirement therefore the beam, girder or column sizes are increased to provide more frame stiffness.

Designs IV, VI and VIII:

1. The building is divided into five frames, Lines A-D, B-C, 1-6, 2-5, and 3-4.
2. The joist and joist girder members are designed using pinned end conditions for factored dead loads and fixed end conditions for superimposed loads.
3. The member characteristics are entered into a two-dimensional frame modeling program, RISA-3D. For all three designs, a two phase analysis is performed. The first phase used pinned connections and factored dead loads. The second phase used fixed connections and the factored superimposed loads. Here the factored gravity loads are checked and the members are redesigned. In both phases, 85% of the joist chord moment of inertia is used.

4. The factored lateral load case is then evaluated and again the members are redesigned if inadequate. Similar to step four a two phase analysis is done, one with the factored dead loads and the second with the factored superimposed and wind loads.
5. The final check is for service load drift requirement of H/400 using a loading combination of 1.0D + 0.5L + 1.0W. This analysis is again like step five. This check is not the controlling requirement for joists and joist girders due to the larger stiffness of the members when compared to beams and girders.

The modeling process for all of the partially restrained frames contain various exceptions to normal design. The modeling process for hot rolled partially restrained frames becomes complicated without the use of a non-linear modeling program. However, the modeling process of steel joists and joist girders is similar to a simple or rigid frame analysis. As a result, the joist frame designs yield less engineering time and are stiffer than their hot rolled counterparts. All of the member designs are explained in greater detail in Chapter 5.

# Chapter 5

## FRAMING MEMBER DESIGN

In all of the framing designs discussed in this project, each one had unique requirements. The main types of members in any building, such as columns, beams, girders, bracing and flooring, all have specific design provisions. Throughout the design process the most important and complicated members to design are the beams, girders and columns.

The flooring consisted of steel deck with a normal weight concrete topping. The deck was chosen from the Vulcraft Steel Deck Catalog (Nucor 1996). For non-composite floors the deck chosen is 3C18 with a 2 in concrete topping and a span of 10 ft. For composite floors the deck chosen is 2VLI with a 2 in concrete topping. The roof decking is 3N22 and has an approved roofing material. In the cases where bracing is used, Designs I and II, double angles are used and are designed based on their tension carrying capacity.

In the design of the beams, the first step is to determine the loading conditions. The loads used are presented in Appendix A. The second step is to determine the type of member, hot rolled or joist, and the end condition. The final step is to decide whether the member is to be non-composite or composite.

For the members located in the frame lines the following procedure is used. First, the member is designed with fixed end connections based on its plastic moment capacity. Using that member size a moment rotation curve is developed using the methods discussed in Chapter 2. In all of the non-composite designs the connection used is the top and seat angles with double web angles.

For composite hot rolled sections, a different approach is taken. First the non-composite member is designed based on construction loads and simple supports. Using the member size obtained from the construction load stage the member is designed with 25% composite action. This is done because the effects of the partially restrained connection will redistribute the moments along the section. This will reduce the positive moment and therefore require less strength at midspan. The idea behind the use of a partially restrained connection is to balance the negative and positive moments to fully develop the member cross section.

The member is then placed in the frame model with its respective moment rotation curve. The moment rotation curves for hot rolled composite and non-composite sections will be discussed in Chapter 6. For non-composite sections, the load combinations are based on the LRFD specifications (AISC 1994). For composite sections there is a two stage loading process as described in the previous chapter.

In ANSYS, the member is input as an elastic beam element and the moment rotation curve is input as a non-linear spring. This is typical for both the non-composite and composite cases. The information that is needed in the frame analysis is the area, moment of inertia and elastic modulus of each member, along with the data points for the moment rotation curve. Additional input such as shear and stress characteristics may also be input if this type of output is required.

For composite sections a weighted average of the positive and negative moments of inertia is used due to the presence of the partially restrained composite connection.

Once the frame is analyzed, the resulting positive and negative moments for each member are checked against all strength requirements. For the non-composite case, the member is chosen based on its plastic moment capacity. In many cases the member is over designed. The reason is that the partially restrained connection provided enough stiffness to balance the negative and positive moments throughout the section. In the case of composite design, the midspan moments or positive moment is summed from both analysis phases and checked versus the composite strength of the member.

The joist and joist girders Designs IV,VI and VIII followed a similar two phase approach. In Appendix C there are sample joist and joist girder designs for both non-composite and composite members. The first step in the design process is the determination of the loads acting on the member. For all the designs the loading process is the same. The joist is initially modeled with dead load only. This phase consisted of the member, steel deck, and concrete topping and the joist is considered to be simply supported. In the second phase of modeling, the superimposed loads or service loads are applied. The end connection during this phase is assumed to be fixed. To obtain fixity in a steel joist the bottom chord needs to be secured to the column. This is modeled by assuming the bottom chord of the joist would be welded or bolted to the column.

In Appendix C, spreadsheets are used to efficiently analyze and design partially restrained non-composite and composite joists and joist girders. These spreadsheets can be found in Appendix C. A brief discussion of the joist member loads and the uniqueness of their designs will be explained here.

Joist and joist girders are essentially trusses. The most critical step in the design is a correct determination of the chord and web member forces. In the two phase loading process the joist and joist girders had both simple and fixed supports respectively. Using basic structural analysis, the member moments can be calculated assuming the member acts as an elastic beam. Initially with simple supports and dead load, secondly with fixed supports and superimposed loads. These moments are than summed. The moment resultants are than divided by the joist depth which produces axial forces in each chord. The web member loads can be determined based on the shear diagram. Figure 5.1 shows the forces as they act on the joist and joist girder.

Using the loads determined for each member, the sections are designed as truss links. This means that only axial loads are checked. The member is either designed for compression or tension. The bottom chord-to-column connection, in some instances, required a plate to develop enough compressive resistance. The plate is welded to the underside of the bottom chord angles. This is done so the bottom chord angles would not need to be increased over the entire joist length, because the extra capacity is only needed at the bottom chord extensions.

For the composite joist and joist girder designs, additional analysis is needed to incorporate the slab into the design. The first step is to determine the effective width of the slab. This can be found in Chapter I3.1 of the specifications for composite design (AISC 1994). The

next step is to calculate the number of shear connectors needed to develop both the positive and negative moment regions of the member. The number of studs for the positive moment region is based on the area of the bottom chord. The number of shear connectors in the negative region is based on the area of reinforcement. In both instances, a ductility factor of 1.3 is used to ensure that in event of a failure the bottom chord would yield before the studs failed. The final step is

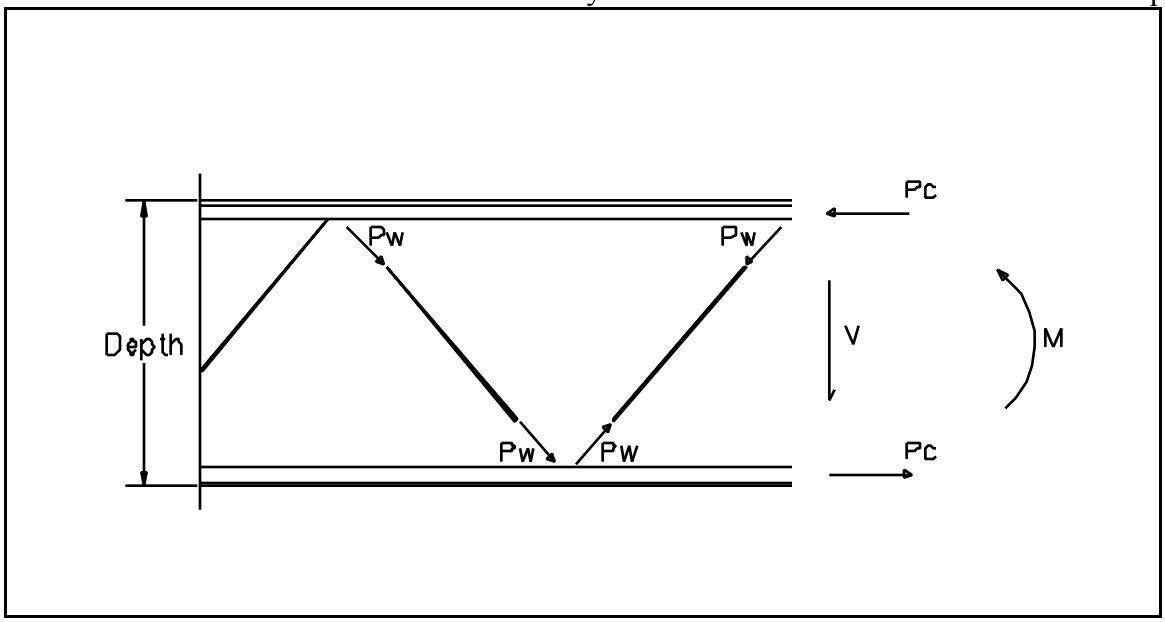


Figure 5.1 Joist Member Loading Diagram

to determine the area of reinforcement needed in the negative moment region. The design of the reinforcement is based on the same concept explained in Figure 5.1. The reinforcement needs to provide enough strength, in tension, to resist the tensile part of the moment couple.

In the case of the joist girder designs, there are many requirements that needed to be met besides those already described. The first being that the top chord of all joist girders must be a minimum of 2L 2.5 in x 2.5 in x 0.212 in for interior girders and 2L 1.5 in x 1.5 in x 0.113 in for exterior girders. This is to ensure adequate bearing for the steel joists framing into the girder along with adequate base thickness for attaching the shear studs. The next requirement is that the minimum bottom chord is 2L 2 in x 2 in x 0.163 in and this is for ductility purposes. The final requirement is that the web member be a minimum of 2L 1.5 in x 1.5 in x 0.155 in to ensure adequate shear resistance.

Once the joist and joist girders are designed they are input into RISA-3D to analyze the frames. To do this, the area and the moment of inertia of each member is required. The moment of inertia for the joist and joist girders is determined in two different ways for the non-composite and composite cross-sections. For the non-composite sections, only the top chord and bottom chord are considered in the moment of inertia calculation. The composite section moment of inertia is based on the bottom chord and the slab. The top chord is not considered because its location is close to the neutral axis thus providing little contribution. This value is then

multiplied by 0.85 for use in the frame lines and deflection calculations. This is done because the calculated moment of inertia is not the effective moment of inertia of the joist. The coefficient, 0.85 is chosen by modeling a joist in RISA-3D and applying a load to it. The analyzed deflection, load, elastic modulus and span are than used to calculate the moment of inertia. In most instances it is determined that the moment of inertia calculated using the process described above is to large. Therefore, as a conservative estimate, 85% of the calculated moment of inertia is used.

The final members to be designed are the columns. The column orientation is as shown in Appendix A with the strong side in the north-south frame lines. This is done to provide adequate stiffness against sway in the smaller frames. The columns are designed based on the AISC LRFD Specifications. Some unique steps in the design are the use of partially restrained connections in the calculation of the G factor. The summing of loads from more than one analysis and the use of a spreadsheet to help in the design and analysis of all the basic equations.

When modifying the G factor what needs to be considered is the flexibility of the connection. The G factor equation, as described below,

$$G = \frac{\Sigma\left(\frac{I_C}{L_C}\right)}{\Sigma\left(\frac{\alpha I_G}{L_G}\right)} \qquad (5.1)$$

is modified using a factor, $\alpha$, less than 1.0 for all types of connections. This factor relates to the connection flexibility at the beam-or girder-to-column joints. For flexible connections the factor is described below:

$$\alpha = \frac{1}{1 + \frac{2EI}{R_{ki} L}} \qquad (5.2)$$

where $R_{ki}$ is the initial connection stiffness, E is elastic modulus, I is the moment of inertia of the beam/girder and L is the span. This factor is then used to modify the G factor and the columns respective effective length factor (King 1993).

The process of summing the loads for both phases is done in a spreadsheet. Designs I,II and III are non-composite, thus the following procedure does not apply. For all other designs the first phase of loading, the initial dead loads, accounted for axial loading only because all the connections are simple. The second phase of loading produced both axial load and moment. The two phases are summed and the column is then designed accordingly. The spreadsheets for column design are shown in Appendix C.

Sample member designs are shown in Appendix C.

# Chapter 6

# CONNECTION ANALYSIS AND DESIGN

In partially restrained designs, the most critical aspect in the design is an accurate representation of the partially restrained connection. There has been much research done on the analysis and design of partially restrained connections. The largest and most practical being the work done by W.F. Chen, W.S. King, J.Y. Liew and D.W. White (Liew, et al 1990 and King, et al 1993). Their work on angle connections in non-composite hot rolled steel is very thorough and is easily applied to the analysis and design of steel frames. For composite hot rolled sections, the work by R.T. Leon has been documented and published in Steel Design Guide Series 8 (Leon, et al 1996). Each of the aforementioned connections will be discussed further in this chapter.

The first type of connection that will be explained is the partially restrained connection used in the non-composite steel designs such as Design III and the roofs of Designs V and VII. The connection is shown in Figure 6.1. The connection itself consists of top and seat angles with double web angles. This connection proved to be very stiff and limited the amount of drift in the frames. The concept behind this connection is that the double web angles resist the shear in the connection while the top and seat angle resist the moment couple produced in the member.

Using the power model as shown below a moment rotation curve can be derived:

$$m = \frac{\theta}{(1+\theta^{n})^{1/n}} \qquad (6.1)$$

where $m = M/M_u$, or the connection moment M, divided by the ultimate connection moment $M_u$, $\theta = \theta_r/\theta_o$, or the relative rotation between beam and column $\theta_r$, divided by the relative plastic rotation $\theta_o$, $\theta_o = M_u/R_{ki}$, or the ultimate connection moment $M_u$, divided by the initial connection stiffness $R_{ki}$, and n is the shape factor. The larger the value of n, the steeper the transition of the moment rotation curve from the yield point to ultimate.

In reference to Appendix B, the connection behavior is as follows. The connection is assumed to rotate about the bottom angle heel. Other assumptions are that out of plane movement and torsion of the connection will not occur.

The partially restrained connections are all analyzed in a similar fashion. The first step is to develop standardized connection parameters relating the connection geometry (Liew, et al 1993). The second step is to determine the initial connection stiffness $R_{ki}$ represented by the initial slope of the moment rotation curve. This value is important because it is utilized in the partially restrained frame analysis. The next variable derived is the ultimate moment capacity of the connection $M_u$. This is important because it is the limiting point on the moment rotation curve. The final value calculated is the shape parameter n, this value represents the sharpness of the curve. These values have all been calculated in a sample spreadsheet in Appendix B. The moment rotation curve can now be developed using the above power model equation.

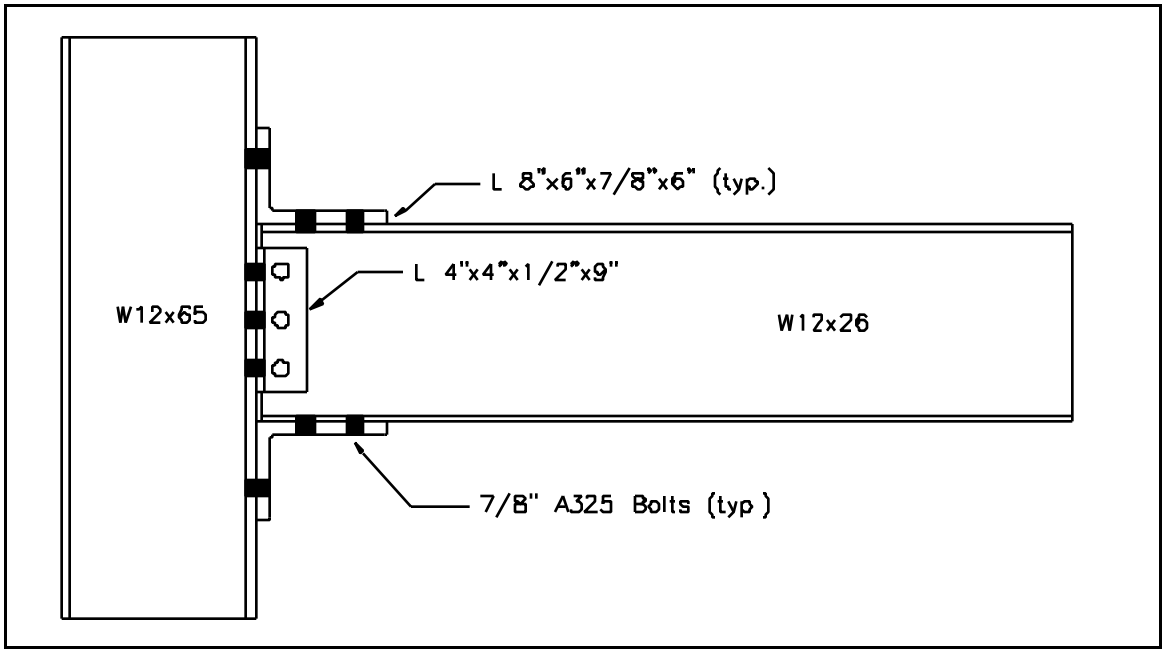


Figure 6.1 Top and Seat Angles with Double Web Angles

The moment rotation behavior of the connection is largely dependent on the type of top and seat angles used and the depth of the member. The most efficient members are of 12 in. to 16 in. deep. This is because when deeper members are tried ,with similar connection characteristics, the moment rotation stiffness did not increase proportionally to the member depth.

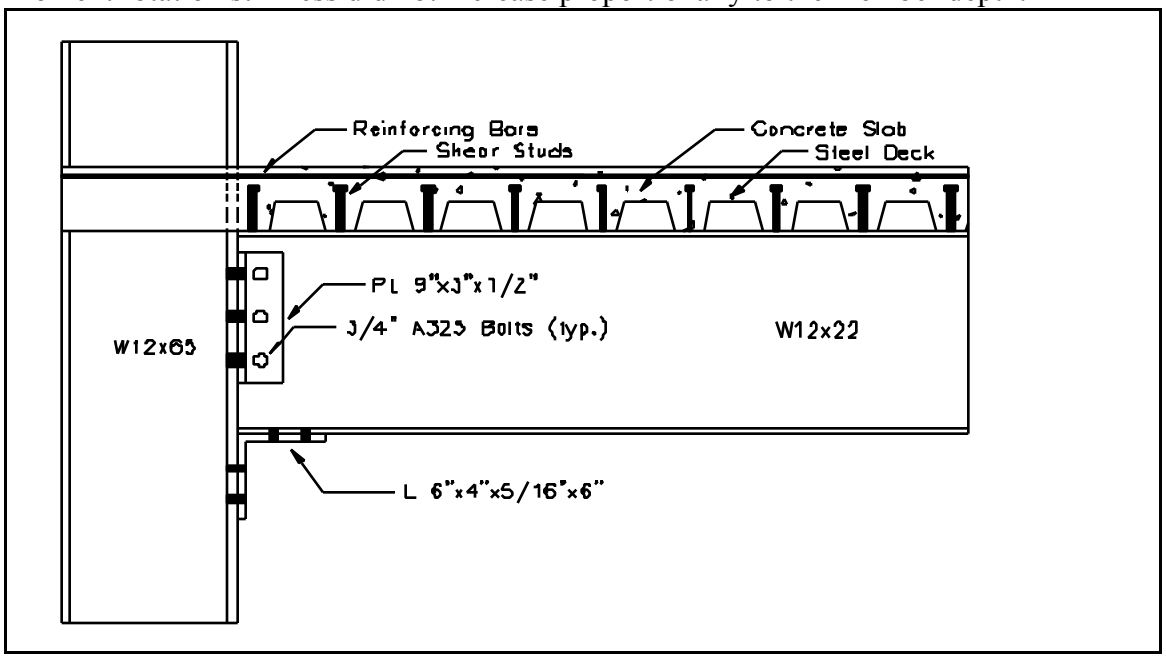


Figure 6.2 Partially Restrained Composite Connection (PRCC)

In the partially restrained frames with composite construction, another type of connection is used, called PRCC or Partially Restrained Composite Connections (Leon, et al 1996). These connections provide a great deal of economy and strength. They are used in Designs V and VII. The connection is shown in Figure 6.2.

Some of the important advantages to using this type of connection are as follows. The amount of moment capacity at the ends of the member can be controlled by the amount of steel in the slab. The use of PRCC connections will reduce the beam size due in part to the balancing of the midspan and end moments and also reduce deflection. One limitation is that PRCC's should only be used in buildings of ten stories or less.

The analysis and design of this connection is shown in Appendix B. The curve is based on the connection component characteristics and the slab properties. The curve is graphed in both the positive and negative moment regions. This accounts for the possibility of moment reversal. It can be seen from Figure 6.3 that the connection is stiffer in the negative moment region. This is due to the reinforcement steel being very

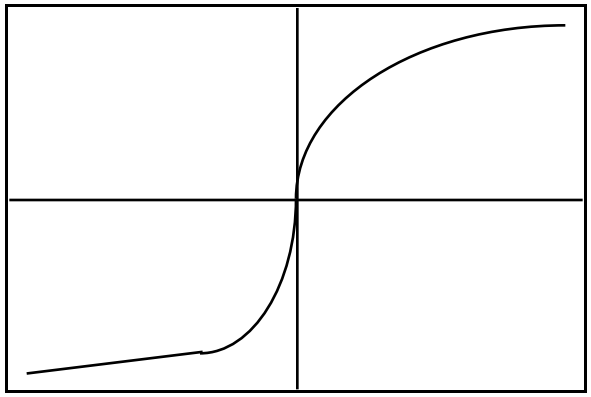

Figure 6.3 Moment rotation Curve for a PRCC

ductile and the bottom angle being stiff. The initial connection stiffness is less in the positive region but the ultimate moment capacity is the same for both regions.

The analysis and design of the PRCC is rather simple but its consideration in the frame is very important. Frame analysis using this type of connection is discussed in Chapter 4. It is important to understand that for the connection to act as analyzed the connection must be detailed properly. This means the number of reinforcing bars used, placement of the bars and adequate development lengths must be specified. The Steel Design Guide Series 8 also includes tables for prequalified connections, adjusted moments of inertia as explained in Chapter 4 and other connection characteristics (Leon, et al 1996).

The final connection designs are for joists and joist girders, both non-composite and composite. An example of non-composite and composite joist connections can be found in Figures 6.4 and 6.5 respectively. A two phase design is considered in all joist connections. The first is based on a simple connection. This connection consists of the top chord of the joist

welded or bolted to the column using an angle or a plate. This step is for the initial dead load case only. The second phase considers the bottom chord of the joist or joist girder welded or

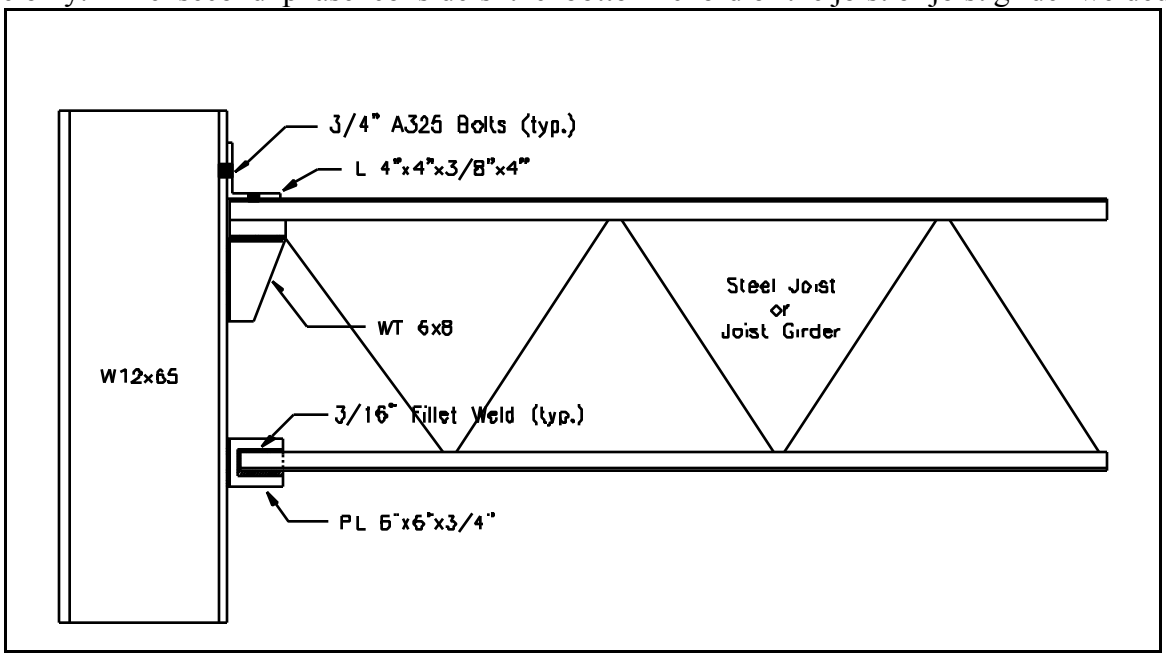


Figure 6.4 Partially Restrained Non-Composite Steel Joist Connection

bolted to the column flange. This is done before the addition of superimposed loads. Both the top chord and bottom chord connection are similar in detail.

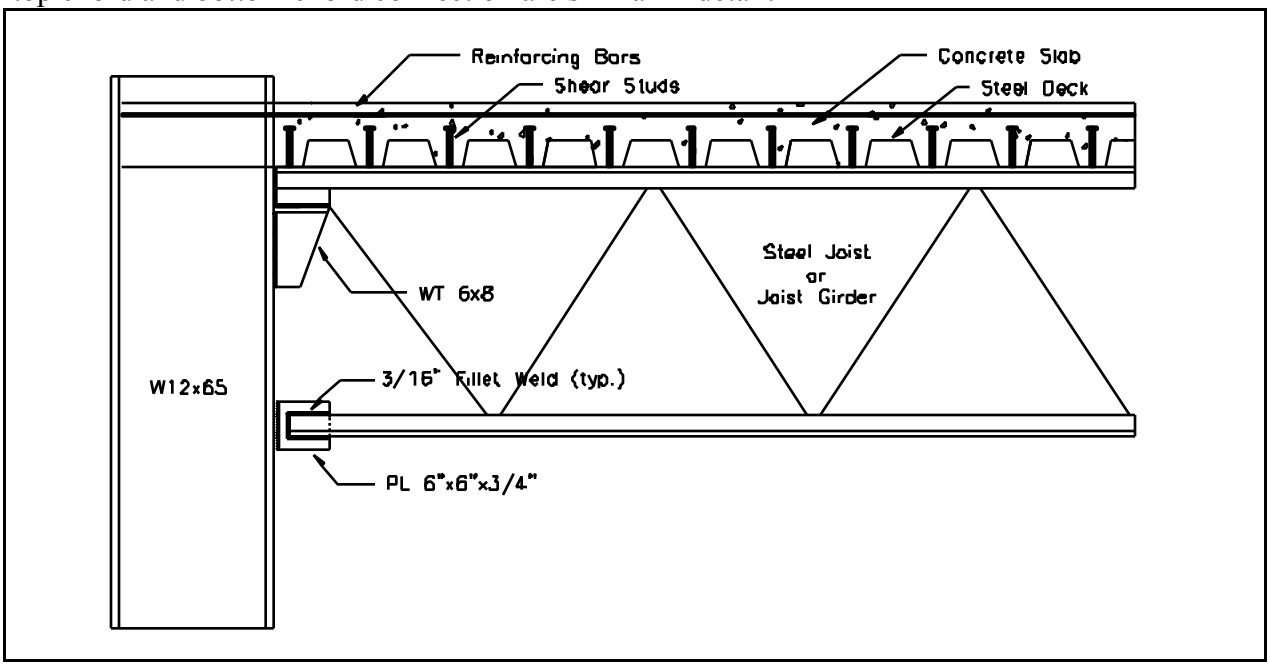


Figure 6.5 Partially Restrained Composite Steel Joist Connection

When connecting the bottom chord, the compressive force in the extension becomes the controlling design factor. Instead of increasing the entire bottom chord size to account for this extra force, additional steel can be added at this critical location. This can be done by welding plates to the bottom chord angles of the joist. This stabilizes the bottom chord at the extensions and provides adequate strength.

In the case of composite connections, the joist top chord is assumed to resist only the initial dead load case. The reinforcement is assumed to couple with the bottom chord to resist the negative moment when the service loads are applied. The use of the steel in the negative moment region reduces the force in the bottom chord because it increases the moment arm by three to four inches. The use of steel reinforcement in the negative moment region requires an added number of studs to successfully transfer the load between the joist and slab.

An example of the hot rolled connection analysis is found in Appendix B and that for joist and joist girder connections in Appendix C. There are limitations to the use of the methods and equations described in this section. The first being that there has been little research or experimentation on the effects of partially restrained connections to the weak axis of columns. This has been neglected in this study since the goal is to determine cost savings. It has also been assumed that all of the hot rolled partially restrained connections will act as analyzed. Finally, it has been assumed that the joists and joist girders can be produced as designed and their behavior as partially restrained connections will act according to analysis. This assumption needs to be tested and verified before either of the non-composite or composite designs can be used in construction.

# Chapter 7

# DRIFT ANALYSIS

In partially restrained frames, lateral drift is an important consideration. The amount of drift allowed in the building of this study is H/400. The inter-story drift is limited to h/250. H is the building height and h stands for the story height respectively.

When considering partially restrained hot rolled frames, drift can be the most important factor in the member design. In the design of the hot rolled frames of this study, lateral drift is not a factor in the member design. For the joist and joist girder designs, drift is also not a problem. This is due to the large amount of stiffness provided by the members and connections. The building in this study had a drift requirement of 1.8 in. The drifts for each design are summarized at the end of this chapter in Table 7.1.

In analyzing the drift for each frame, a loading combination of 1.0D + 0.5L + 1.0W is used (Swensson, et al 1996). This loading combination is common in practice and is used by many engineers to determine service load drift. Some assumptions that are made is that each frame acts separately and does not interact with adjacent frames. This is done for ease of modeling and yields a conservative estimate of actual building drift. Another assumption is that the flooring did not contribute any stiffness to the building. This can only be done in a three dimensional model. The use of such a model can prove to be beneficial if drift is a consideration because the interaction of all the frames reduces the drift of the entire structure.

From the data in Table 7.1, drift is not a consideration in the design of the partially restrained frames of this study. Drift only becomes a factor for taller buildings with more flexible connections. The drift of each frame and their individual stories is summarized for each design in Table 7.1

When comparing the drift of hot rolled sections and joist frames, it is easily seen that the joist and joist girder frames produce the least amount of drift. This is because of the large amount of stiffness the joists provide. Therefore in sway frames it is beneficial to use joists and joist girders because they reduce lateral drift.

Total Drift Limitation = H/400 Story 1 is 14' Stories 2-4 are 12' Penthouse story is 10'
Inter-Story Drift = h/250

| Frame | Story | Drift Limitation | Design 1 | Design II | Design III | Design IV | Design V & VII | Design VI & VIII |
|---|---|---|---|---|---|---|---|---|
| Line 1-6 | 1st | 0.67 | NA | NA | 0.17 | 0.14 | 0.23 | 0.14 |
| | 2nd | 0.58 | NA | NA | 0.19 | 0.14 | 0.27 | 0.15 |
| | 3rd | 0.58 | NA | NA | 0.17 | 0.09 | 0.22 | 0.10 |
| | 4th | 0.58 | NA | NA | 0.08 | 0.07 | 0.13 | 0.08 |
| | Total | 1.50 | NA | NA | 0.61 | 0.44 | 0.85 | 0.47 |
| Line 2-5 | 1st | 0.67 | 0.42 | 0.40 | 0.22 | 0.17 | 0.29 | 0.16 |
| | 2nd | 0.58 | 0.50 | 0.44 | 0.23 | 0.16 | 0.36 | 0.15 |
| | 3rd | 0.58 | 0.34 | 0.28 | 0.20 | 0.12 | 0.30 | 0.12 |
| | 4th | 0.58 | 0.24 | 0.14 | 0.10 | 0.10 | 0.17 | 0.09 |
| | Total | 1.50 | 1.50 | 1.26 | 0.75 | 0.55 | 1.12 | 0.52 |
| Line 3-4 | 1st | 0.67 | NA | NA | 0.30 | 0.19 | 0.32 | 0.18 |
| | 2nd | 0.58 | NA | NA | 0.40 | 0.18 | 0.39 | 0.18 |
| | 3rd | 0.58 | NA | NA | 0.42 | 0.17 | 0.36 | 0.17 |
| | 4th | 0.58 | NA | NA | 0.29 | 0.15 | 0.26 | 0.14 |
| | Penthouse | 0.48 | 0.16 | 0.13 | 0.22 | 0.03 | 0.22 | 0.03 |
| | Total | 1.80 | 0.16 | 0.13 | 1.63 | 0.72 | 1.55 | 0.70 |
| Line A-D | 1st | 0.67 | 0.50 | 0.50 | 0.20 | 0.23 | 0.38 | 0.22 |
| | 2nd | 0.58 | 0.47 | 0.45 | 0.11 | 0.15 | 0.34 | 0.12 |
| | 3rd | 0.58 | 0.36 | 0.35 | 0.13 | 0.16 | 0.23 | 0.15 |
| | 4th | 0.58 | 0.17 | 0.16 | 0.02 | 0.07 | 0.08 | 0.06 |
| | Total | 1.50 | 1.50 | 1.46 | 0.46 | 0.61 | 1.03 | 0.55 |
| Line B-C | 1st | 0.67 | NA | NA | 0.20 | 0.23 | 0.35 | 0.23 |
| | 2nd | 0.58 | NA | NA | 0.12 | 0.15 | 0.31 | 0.12 |
| | 3rd | 0.58 | NA | NA | 0.22 | 0.21 | 0.30 | 0.26 |
| | 4th | 0.58 | NA | NA | 0.09 | 0.12 | 0.13 | 0.14 |
| | Penthouse | 0.48 | 0.24 | 0.15 | 0.07 | 0.01 | 0.14 | 0.02 |
| | Total | 1.80 | 0.24 | 0.15 | 0.70 | 0.72 | 1.23 | 0.77 |

** Even numbered designs are with joists
** Odd numbered designs are with hot-rolled sections

Table 7.1 Building Design Drifts

# Chapter 8

## MATERIAL TAKEOFFS & COST ESTIMATES

In most building projects, the final decision as to which material type and construction method to is based on economics. The cost of a project is evaluated by different firms based on their own in-house cost estimating procedures, their source of materials, the contractors they will hire and the amount of profit desired. One common method of estimating a project is the use of the Means Building Construction Cost Data (Means 1994). This was the estimating procedure chosen to provide realistic building costs for the eight designs evaluated in this study.

To introduce the Means cost estimating procedure, a brief explanation of the manual and its components will be presented. In this manual, all aspects of the project are priced based on the cost of material, labor, equipment, overhead, and profit. The manual is broken down into sections such as site work, concrete, masonry, and metals. Within each section are subsections, such as pre-cast or cast-in-place for the concrete section. Finally, the items are priced out individually based on a unit cost. Under each item the crew needed to do the work is described, their daily output, and the number of man hours it takes to do one unit of work. Also described is the type of unit, such as linear feet, the cost of materials, labor, equipment and overhead and profit per unit. For a more detailed explanation of the use of the Means Building Construction Cost Data refer to the second page of the manual (Means 1994).

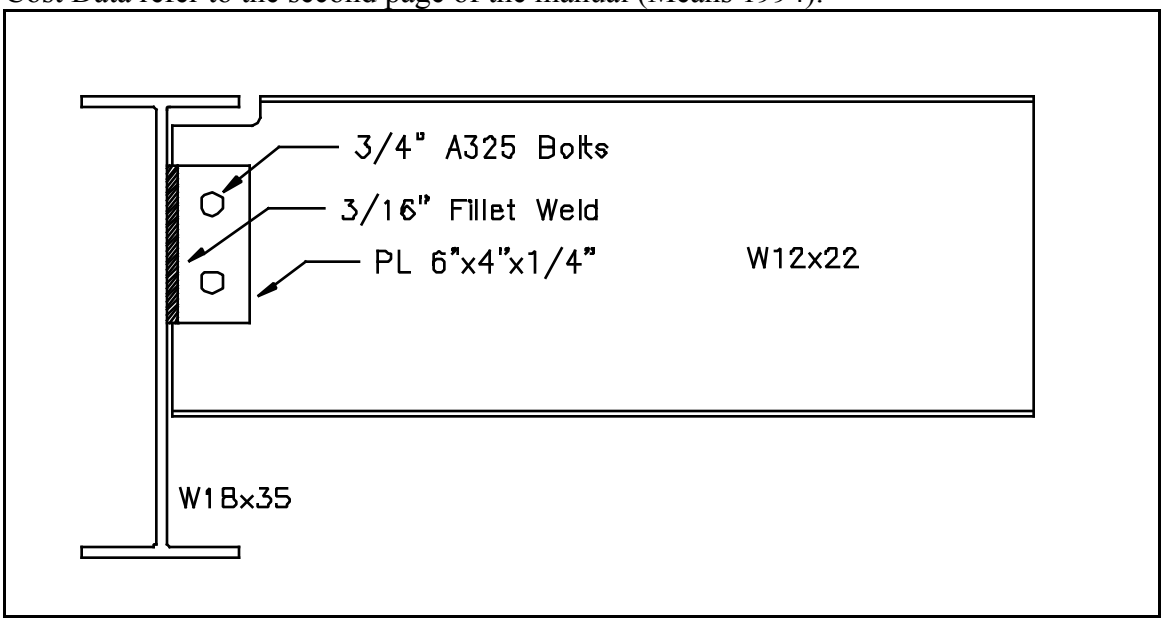


Figure 8.1 Simple Shear Tab Connection

Using Means the projects were estimated. To simplify the estimating process of the connections, a rounded figure for the cost of each type of connection was calculated. The

connection costs are summarized in the following section along with a brief description and illustration of the connection.

A simple shear connection is illustrated in Figure 8.1. This was used in all of the hot rolled designs, mostly at beam to girder connections. The estimated cost constructing and erecting this connection is $50.

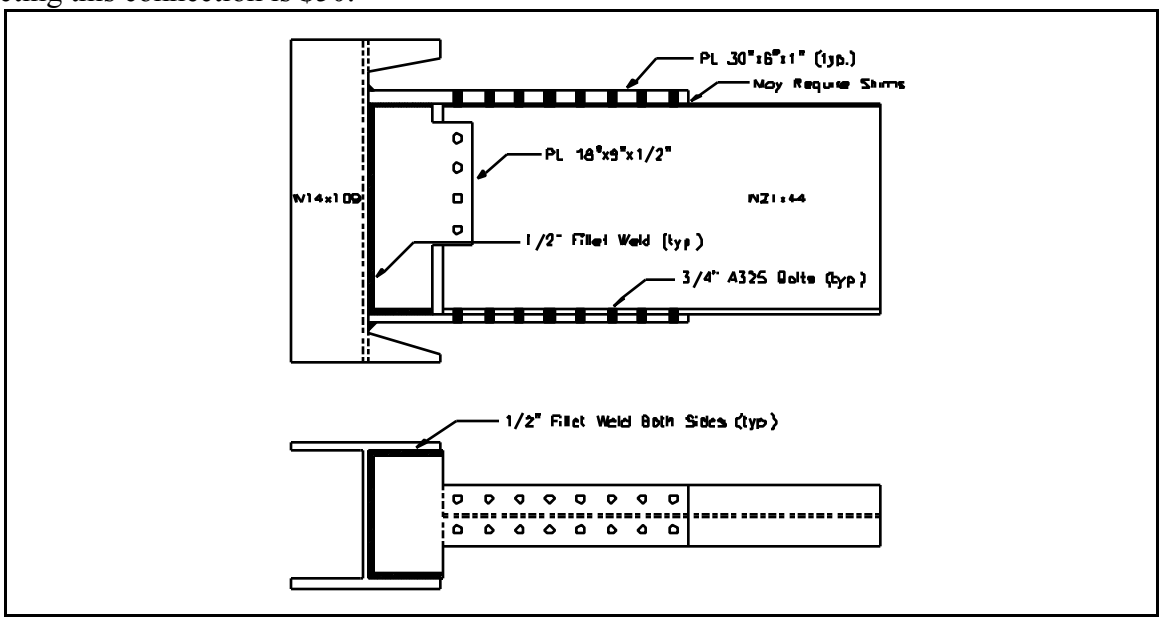


Figure 8.2 Flange Plate Moment Connection

In Figure 8.2 a typical fully restrained or rigid connection is shown. This connection is very complicated and requires shims and a large number of bolts to be installed correctly. This is the most expensive connection used in this study and is only used in Design I sparingly. The purpose of this connection in design one is to provide lateral stability against wind loads. The cost of this connection is $500.

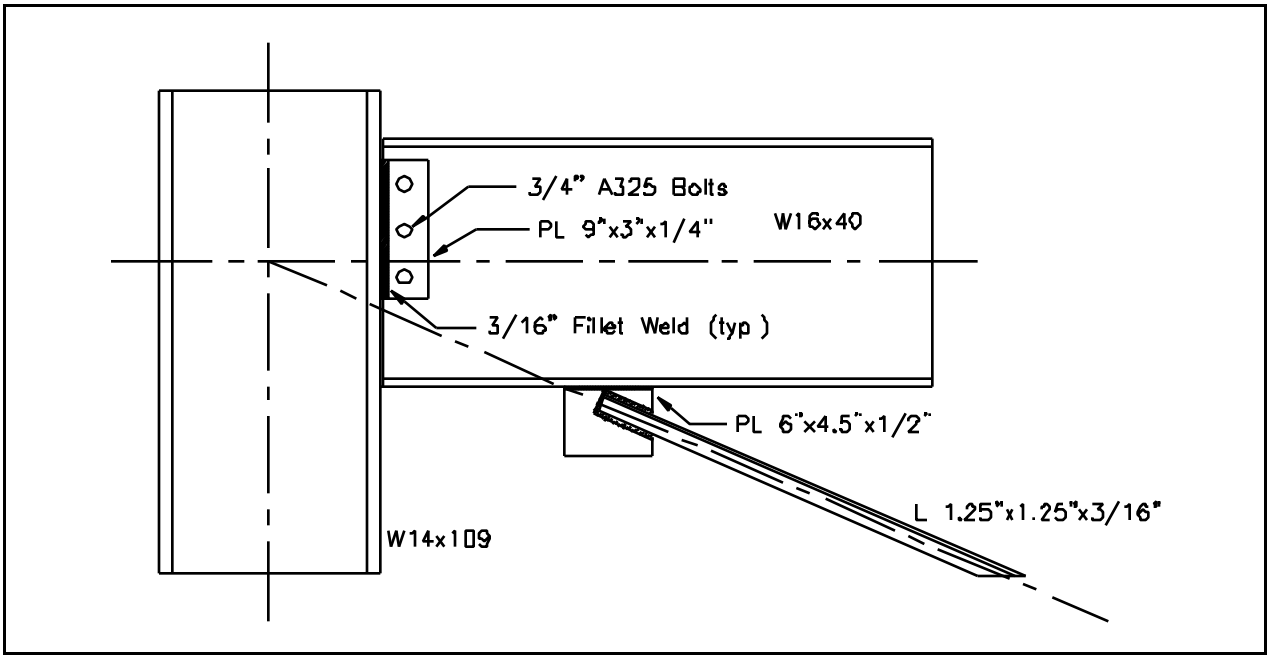

Figure 8.3 Bracing Connection

In Figure 8.3 a bracing connection is detailed. This connection is designed to resist lateral loads and is only used in the braced frames of Designs I and II. This connection is quite easy to design but requires field welding to install the bracing angles. The average cost for this connection is $100.

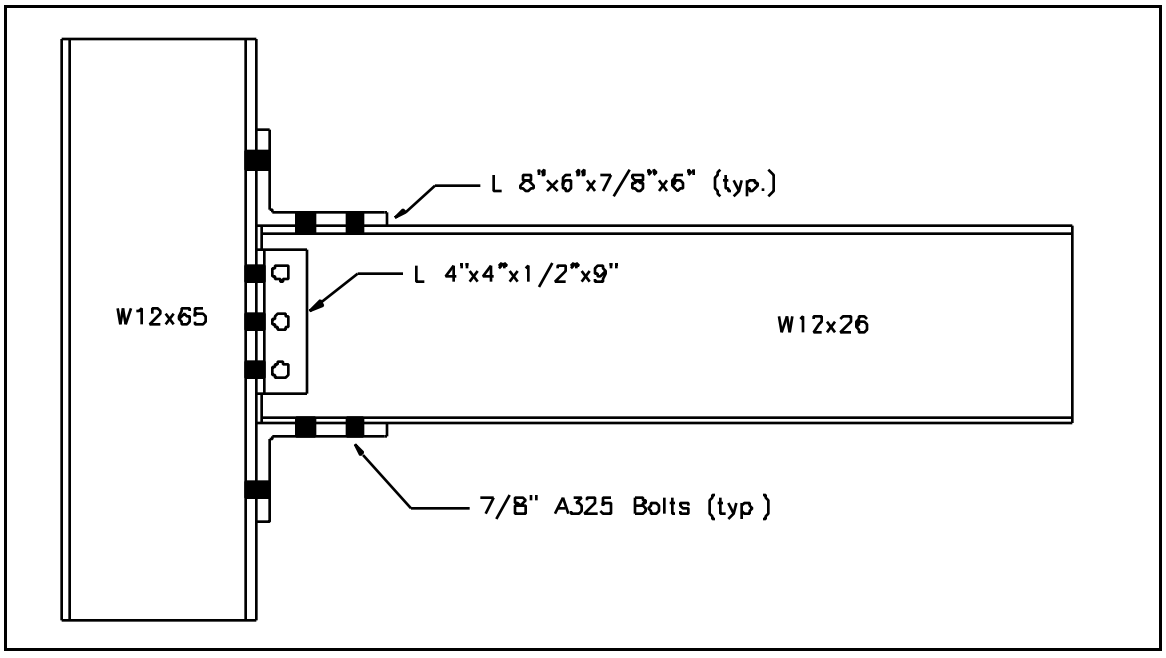

Figure 8.4 Partially Restrained Connection

In Figure 8.4 is a typical partially restrained non-composite hot rolled connection used in Designs III,V, and VII. The average cost of this connections is $200. The same connection can also be made to the weak axis of the column by connected the angles to the web instead of the flange. In some cases where there would not be a similar connection on the opposite side of the column, plates may be needed to stiffen the column web opposite the connection.

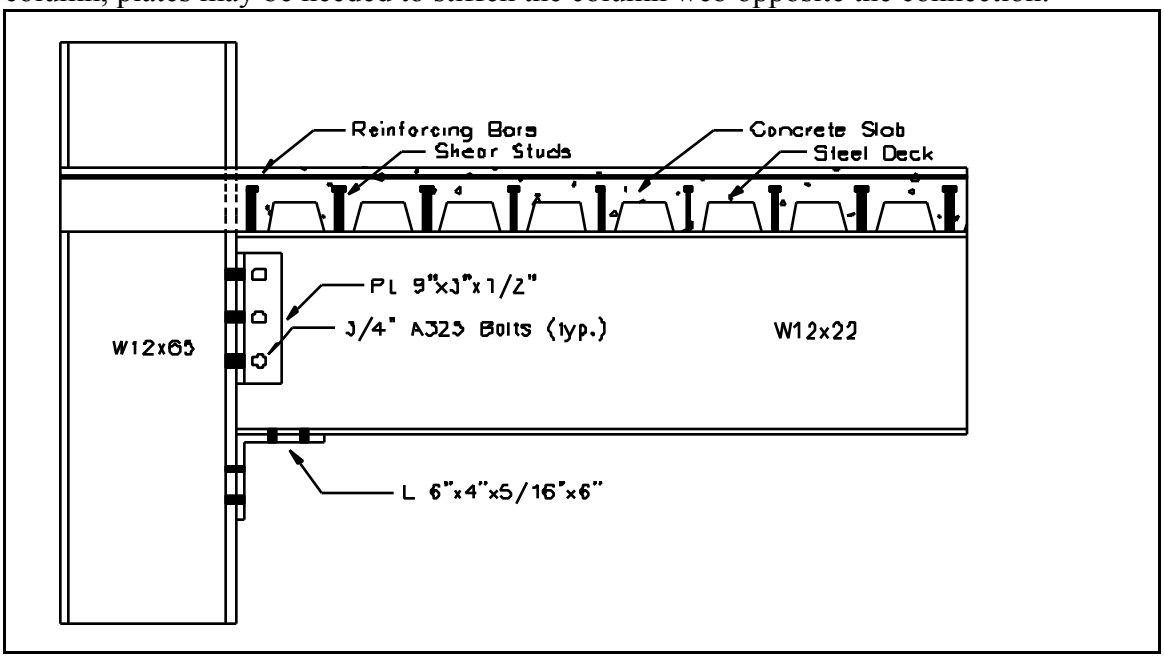


Figure 8.5 Partially Restrained Composite Connection

In Figure 8.5 a partially restrained composite connection is illustrated. This connection was used in Designs V and VII exclusively. In Design VII this connection is also used at all beam to girder connection locations. The average cost of this connection is $150. This connection may be used at column weak axis locations with plates to stiffen the seat angle.

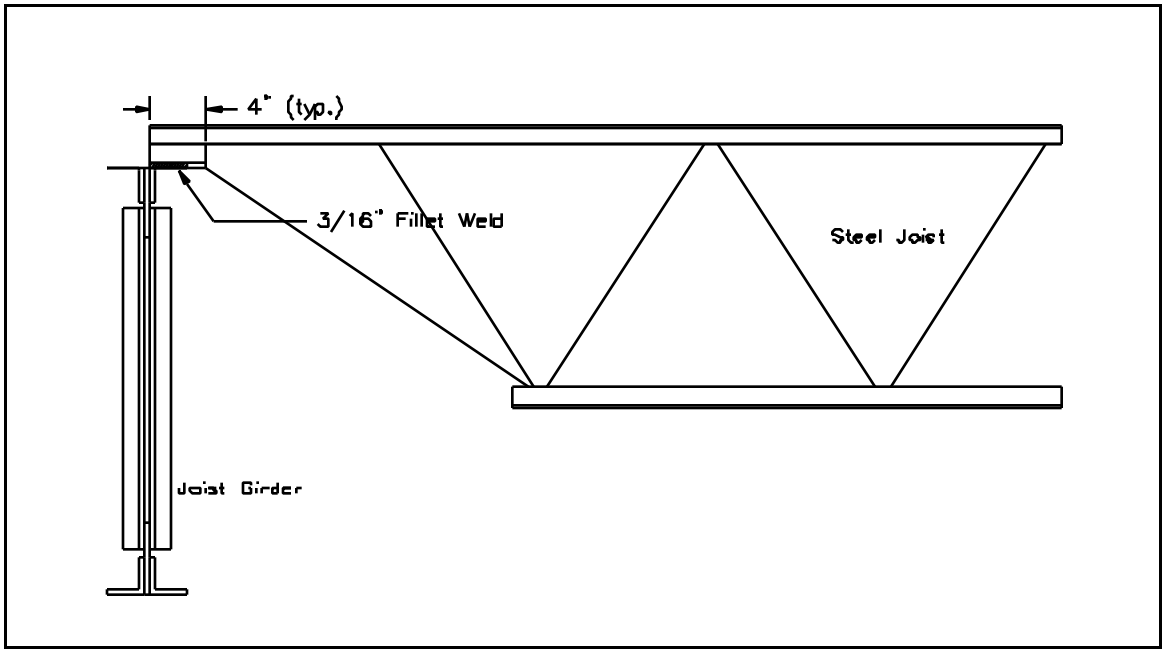


Figure 8.6 Joist to Joist Girder Connections

In Figure 8.6 a joist to joist girder connection is pictured. This connection is used in all of the joist designs including Designs II, IV, VI, and VIII. It is the least expensive connection since only requires a field weld for installation and bridging throughout its length for stability. The cost of this connection is $50.

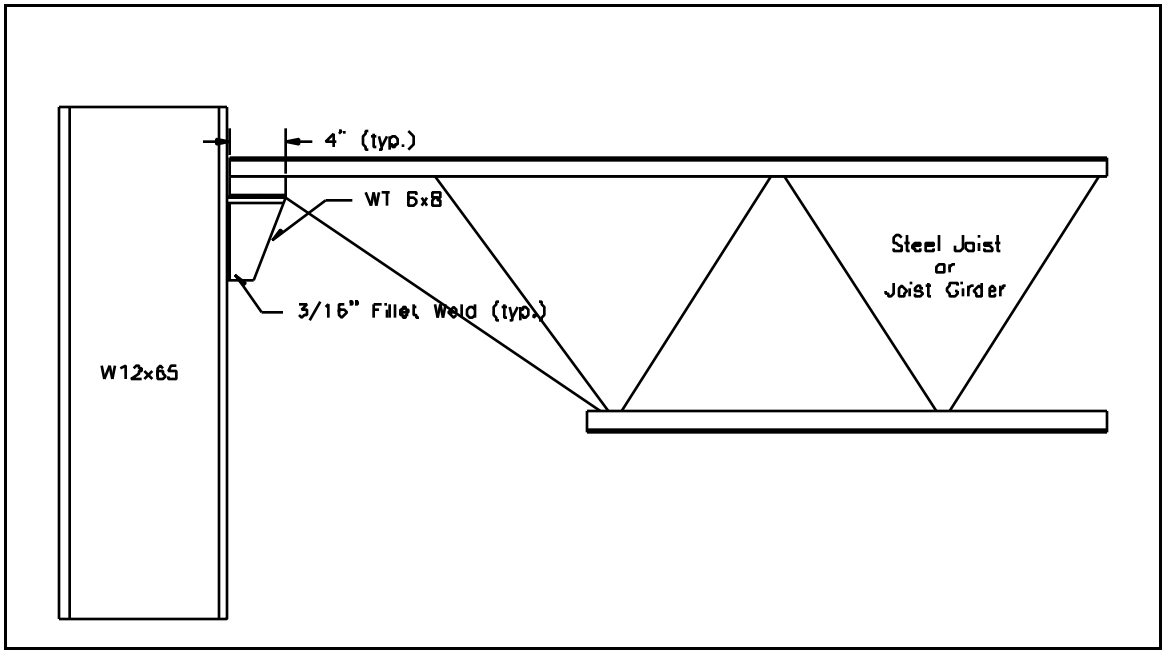


Figure 8.7 Joist or Joist Girder to Column Connection

The connection illustrated in Figure 8.7 is similar to the previous connection except that it uses a top angle to connect the joist or joist girder to the column. This connection is more expensive than the previous connection because it requires positioning while being bolted into place. The cost of this connection is $50.

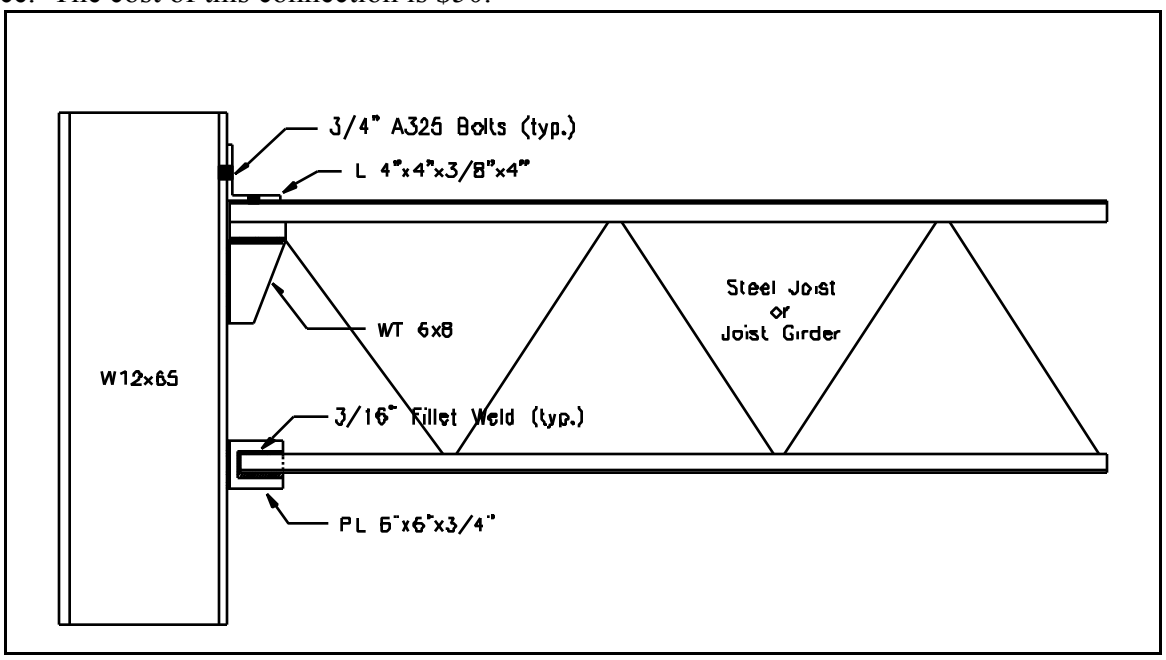


Figure 8.8 Partially Restrained Joists to Column Connection

In Figure 8.8 a typical non-composite partially restrained joist connection is shown. This connection is the same as the previous connection with the bottom chord welded to a stabilizer plate. This makes the joist fixed for all loads added after a designated load stage. The optimum time in which to weld the bottom chord is after the initial dead loads have been applied, this includes the slab weight, deck, bridging and other dead loads. The joist would than become fixed for service and wind loads. The price of this connection is $125.

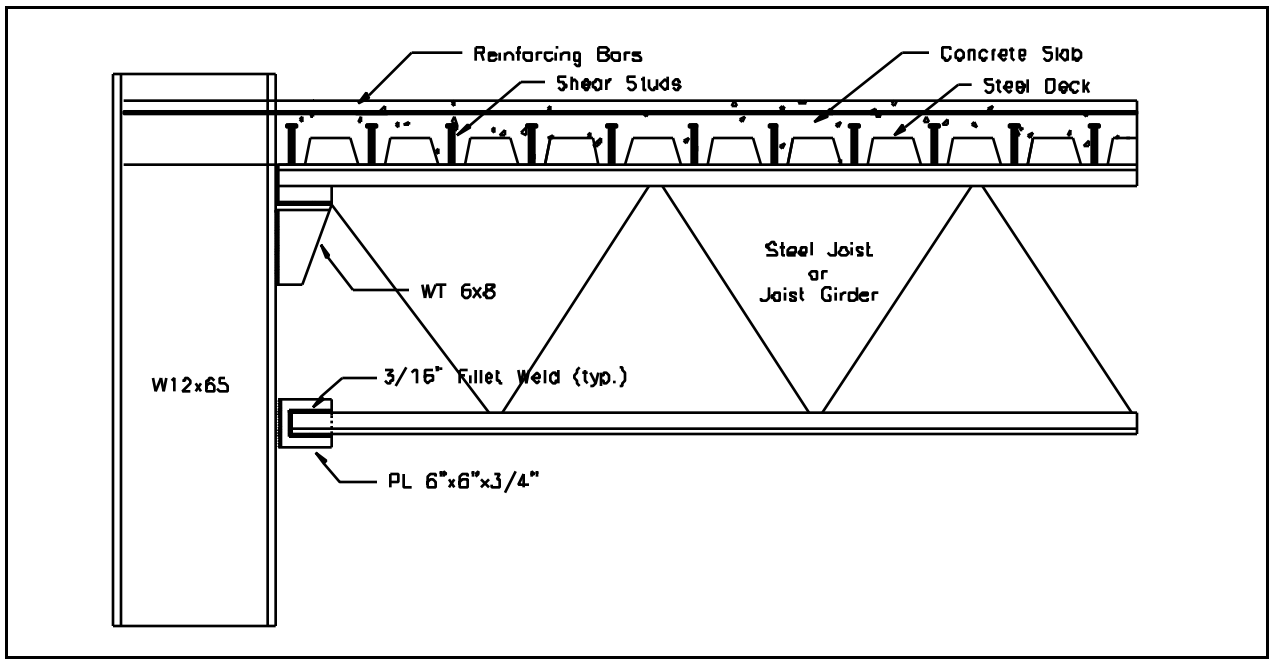


Figure 8.9 Partially Restrained Composite Joists to Column Connection

In Figure 8.9 a partially restrained composite joist connection is detailed. This connection is similar to design and methodology as the previous connection. The welding of the bottom chord occurs at the same load step. The addition of reinforcement and composite action reduces the steel joist or joist girder weight and also provides ductility in the connection with the reinforcement. The average cost of this connection is $100.

Using the average costs of these basic connections, a cost estimate was produced for each of the eight designs. Table 8.1 shows the cost of each design along with material takeoffs. The material takeoff is a summary of steel weight, concrete, reinforcement and shear studs. These particular items were chosen because they had the greatest effect on the cost. The cost estimates only include the superstructure because this is the only part of the design that varied.

| Design | Material | Construction | Lateral System | Steel (tons) | Rein. (tons) | Studs (#) | Cost |
|---|---|---|---|---|---|---|---|
| I | Hot rolled | Non-Composite | E/W Unbraced N/S Braced | 187.4 | 11.2 | - | $ 539,500 |
| II | Joists | Non-Composite | E/W Unbraced N/S Braced | 138.0 | 11.2 | - | $ 407,000 |
| III | Hot rolled | Non-Composite | Partially restrained | 169.8 | 11.2 | - | $ 529,500 |
| IV | Joists | Non-Composite | Partially restrained | 107.9 | 11.2 | - | $ 404,000 |
| V | Hot rolled | Composite | Partially restrained | 141.9 | 13.7 | 2660 | $ 509,000 |
| VI | Joists | Composite | Partially restrained | 99.0 | 12.5 | 5200 | $ 389,000 |
| VII | Hot rolled | Composite | Partially restrained | 140.3 | 15.5 | 1756 | $ 525,000 |
| VIII | Joists | Composite | Partially restrained | 86.1 | 13.7 | 5744 | $ 402,000 |

Table 8.1 Material Breakdown & Cost Estimate

A detailed cost estimate of each design is shown in Appendix F. In reference to Table 1.1 Designs V and VII, and VI and VIII, the only difference is that all beam-to-girder and joist-to-joist girder connections are partially restrained in the later designs.

As can be seen from Table 8.1 the joist and joist girder designs were the most economical. This is due to the smaller amount of steel needed, less erection time, simpler connections and an overall savings in labor and material handling. The least expensive design was the composite joists and joist girders (Design VI). This design used composite construction with partially restrained connections at all of the frame lines. The non-composite joists and joist girder designs were a close second when comparing savings.

The overall savings of steel and money is obvious with steel joists. What is also deduced from this study is the savings when partially restrained connections are used. The savings, when using partially restrained connections, is seen in both the hot rolled steel shapes and steel joists. An overall savings in steel is in the range of 30% to 40% for steel joists versus hot rolled shapes with PR connections. When considering composite versus non-composite construction, steel savings is in the range of 10% to 20% for both hot rolled shapes and steel joists with PR connections. The cost savings of using steel joists versus hot rolled sections is in the range of 20% to 30% and finally when comparing composite versus non-composite the savings is 5% to 10% for both hot rolled shapes and steel joists with PR connections. Table 8.1 also shows that

the partially restrained beam-to-girder and joist-to-joist girder designs (Designs VII and VIII) are more expensive than there simple connections counterparts (Designs V and VI). This is due to an increased amount of reinforcement needed at these locations. Therefore, the most economical designs were the composite designs with partially restrained connections at frame line locations only.

# Chapter 9

# CONCLUSIONS

The objective of this study was to provide cost estimates of eight different framing designs to see which is the most economical. Each of the eight designs were unique in their material choice, construction and lateral system. The two material choices were hot rolled steel shapes and steel joists and joist girders. The construction methods were either non-composite or composite. The lateral systems included rigid frames, braced frames and partially restrained frames.

The use of partially restrained design and construction has yielded both benefits and disadvantages. The benefit being the savings in steel but the disadvantage is the amount of engineering time utilized in the design process. The applicability and modeling complexity in partially restrained design may justify the use of moment frames or braced frames. The decision is up to the designers and their knowledge of partially restrained construction.

As described in Chapter 8 there is a noticeable savings in steel and cost when using steel joists coupled with partially restrained connections. What goes unnoticed is the actual design time that was used in hot rolled or steel joist partially restrained design. The hot rolled designs took a considerable amount of time, maybe twice as long, to implement when compared to the steel joists. The extra design time is understood because the hot rolled shapes required a non-linear analysis when considering connections while the steel joists did not. This aspect of the hot rolled design actuality took less time than if done at a design firm due to the modeling capability of the analysis tool (ANSYS). General purpose finite element programs are not routinely used in design firms because of the significant cost. The design of steel joists uses a two phased approach, as described in the chapter on modeling. The first being a simple pinned model for the construction load phase and, the second being a rigid fixed model for the service and wind load stage. Thus limiting the design time needed and produced the most economical designs.

The use of composite construction in the case of partially restrained design is also beneficial in many instances. This is understood because the general reinforcement in the slab, required for temperature and shrinkage, contributes to connection strength. This contributes largely to why the hot rolled designs decreased significantly in cost. In the steel joist designs there was limited savings for composite construction. This is because there are many design requirements that limit how small the joist members can become (Nucor 1996). Therefore the results show that non-composite and composite steel joist construction are similar in cost when using partially restrained connections.

There are limitations to the applicability of partially restrained construction for hot rolled shapes. The first being that buildings are limited to ten stories, mostly due to drift considerations. The second being the behavior of partially restrained connections to the weak axis of columns. No research was identified on this type of connection. Also there has been much confusion on a standard code of practice on the design of both partially restrained hot

rolled connections and their respective frame analysis. There has been much research done in this area but with limited acceptance by the structural engineering industry. This is due largely in part to the vast amount of information on the subject with little standardization.

Similar to hot rolled shapes, the ten story limitation and the column weak axis connection are also applicable to steel joists. There is however, a need for future research into the areas of partially restrained steel joist design. The first being the actual response of partially restrained composite steel joists under loading. The question asked is whether the reinforcement provides enough strength, or pullout strength, to resist the tension part of the moment couple. The second question is whether it is possible to non-compositely or compositely create a partially restrained connection between the steel joist and joist girder. The problem with this connection being the resistance to the compressive part of the moment couple by the joist girder web. Other possible research topics may be in the vibration and ductility aspects of partially restrained joist construction.

The results of this study are encouraging and the benefits of partially restrained construction for both hot rolled shapes and steel joists are obvious. With some additional research and a better understanding of partially restrained behavior the savings of material, cost and time can be passed on to the design engineer and finally to the public.

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# Appendix A

# APPLIED LOADS, MATERIAL PROPERTIES AND FRAMING LAYOUTS

## A.1 Load Combinations

Using ASCE 7-95 the minimum design loads for the office building are calculated. Loads considered were dead, live, snow and wind loads. Snow drift for the roof was also considered. Snow drift was calculated according to the provisions of Section 7 of ASCE 7-95. The snow drift profiles are shown in Figure A.5. To calculate member forces the LRFD load combinations are used. The loading combinations are:

- 1.4D
- 1.2D + 1.6L +0.5($L_r$ or S)
- 1.2D + 1.6($L_r$ or S) + (0.5L+0.8W)
- 1.2D+1.3W+0.5L+0.5 $L_r$
- 0.9D ± 1.3W

The largest calculated using these combinations are then used to design the members.

## A.2 Applied Loads

Roof Loads:

- Dead Loads

| | |
|---|---|
| Insulation | 2 psf |
| Mechanical | 4 psf |
| Ceiling | 3 psf |
| Roofing | 14 psf |

- Snow Loads

| | |
|---|---|
| Uniform | 18 psf |
| Drift | See accompanying diagram |

Floor Loads:

- Dead Loads

| | |
|---|---|
| Slab | 40 psf |
| Mechanical | 4 psf |
| Ceiling | 2 psf |
| Curtain Walls | 20 psf |

- Live Loads

| | |
|---|---|
| Office | 50 psf |
| Lobby | 100 psf |
| Penthouse | 100 psf |
| Partitions | 20 psf |

## A.3 Material Properties

The following material properties were used throughout the project.

Steel Properties:

Yield Stress : A572 50 ksi

Elastic Modulus : 29,000 ksi

Bolts : A490

Concrete Properties (For use with hot rolled sections):

Compressive Stress : 4 ksi

Elastic Modulus : 3,600 ksi

Normal Weight : 145 pcf

Concrete Properties (For use with steel joists):

Compressive Stress : 3 ksi

Young's Modulus : 3,200 ksi

Normal Weight : 145 pcf

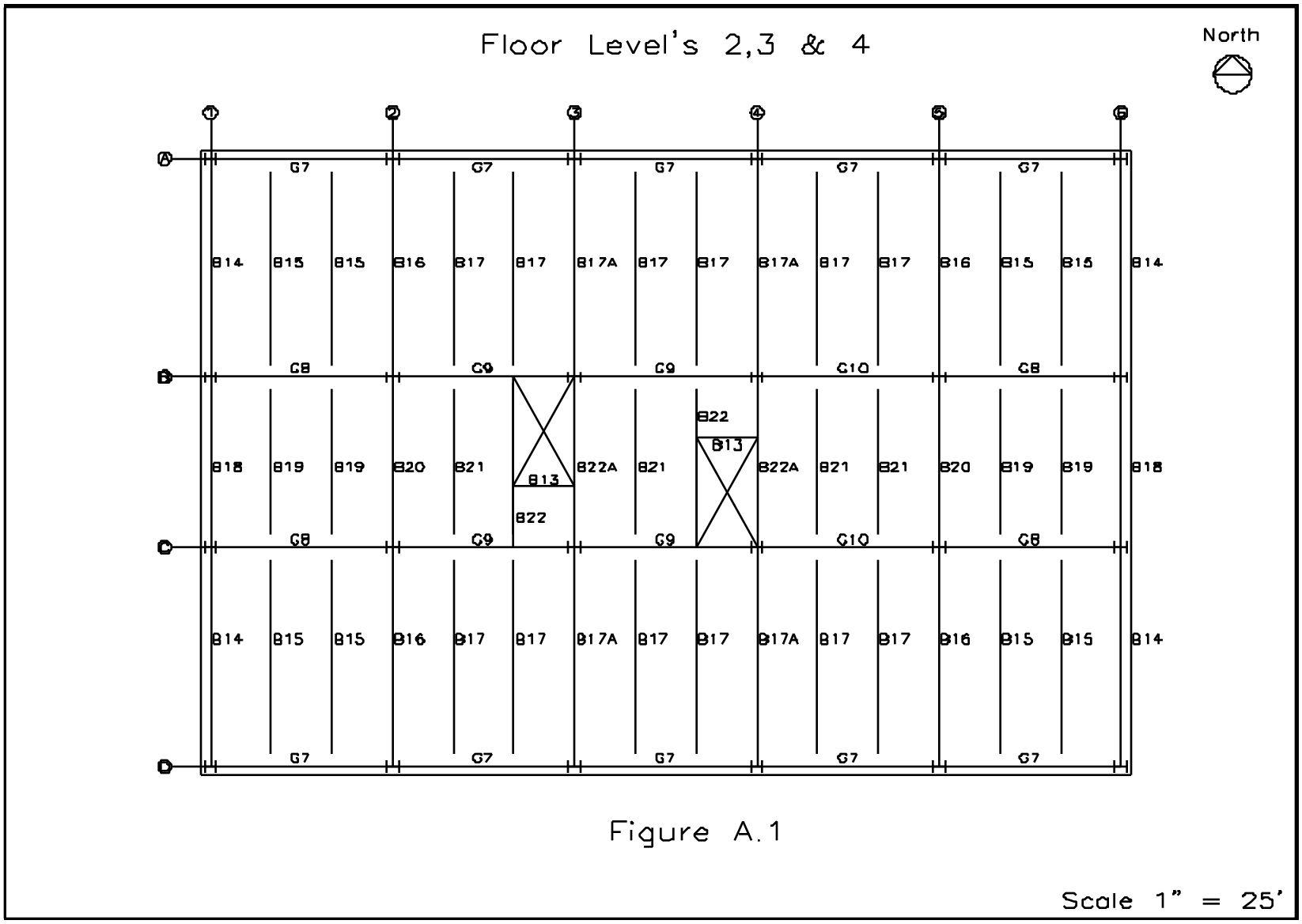



Figure A.1

Scale 1" = 25'

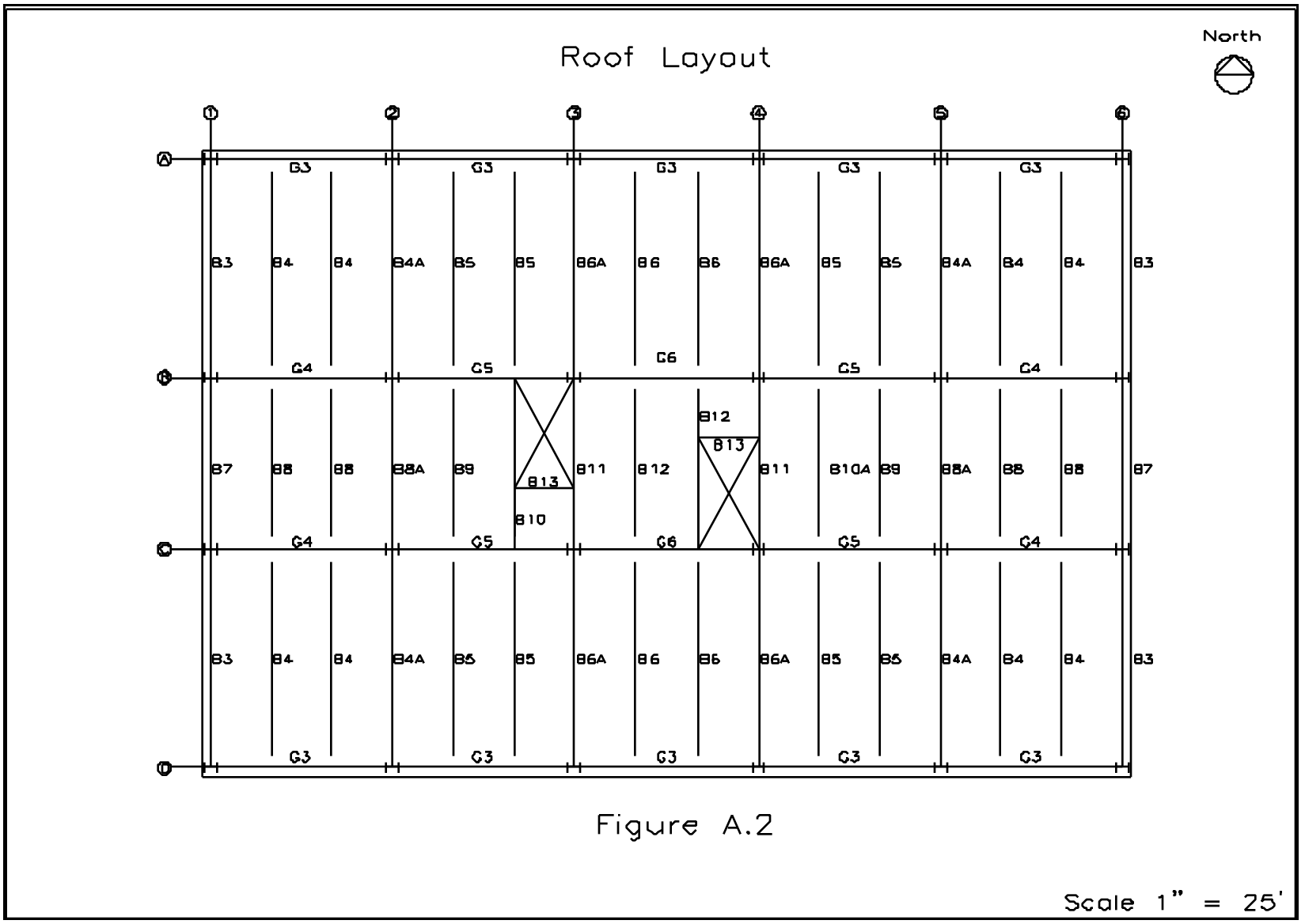

Roof Layout
North
1
2
3
4
5
6
A
B
C
D
G3
G4
G5
G6
B3
B4
B4A
B5
B6
B6A
B7
B8
B8A
B9
B10
B10A
B11
B12
B13
Scale 1" = 25'

Figure A.2

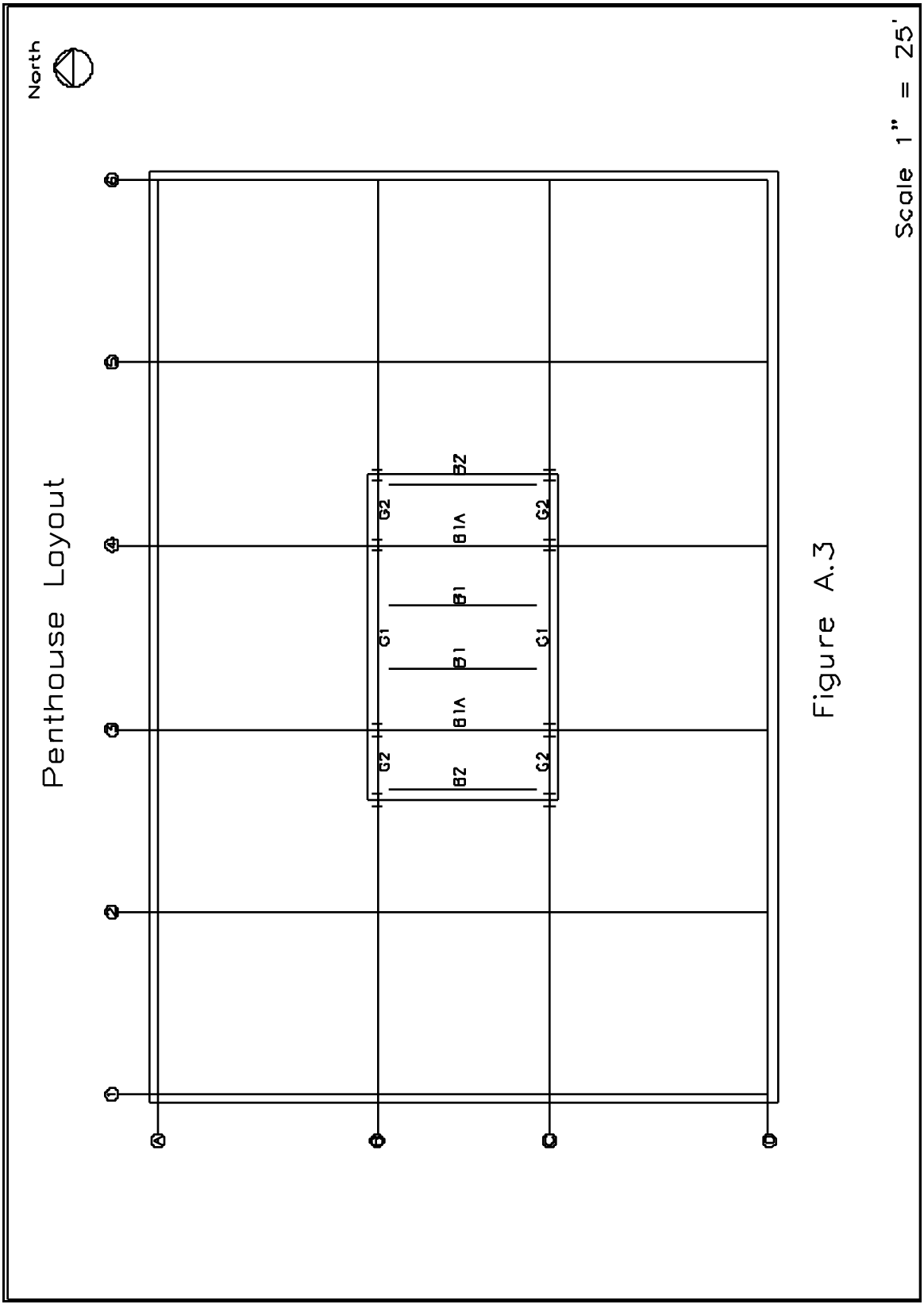



Figure A.3

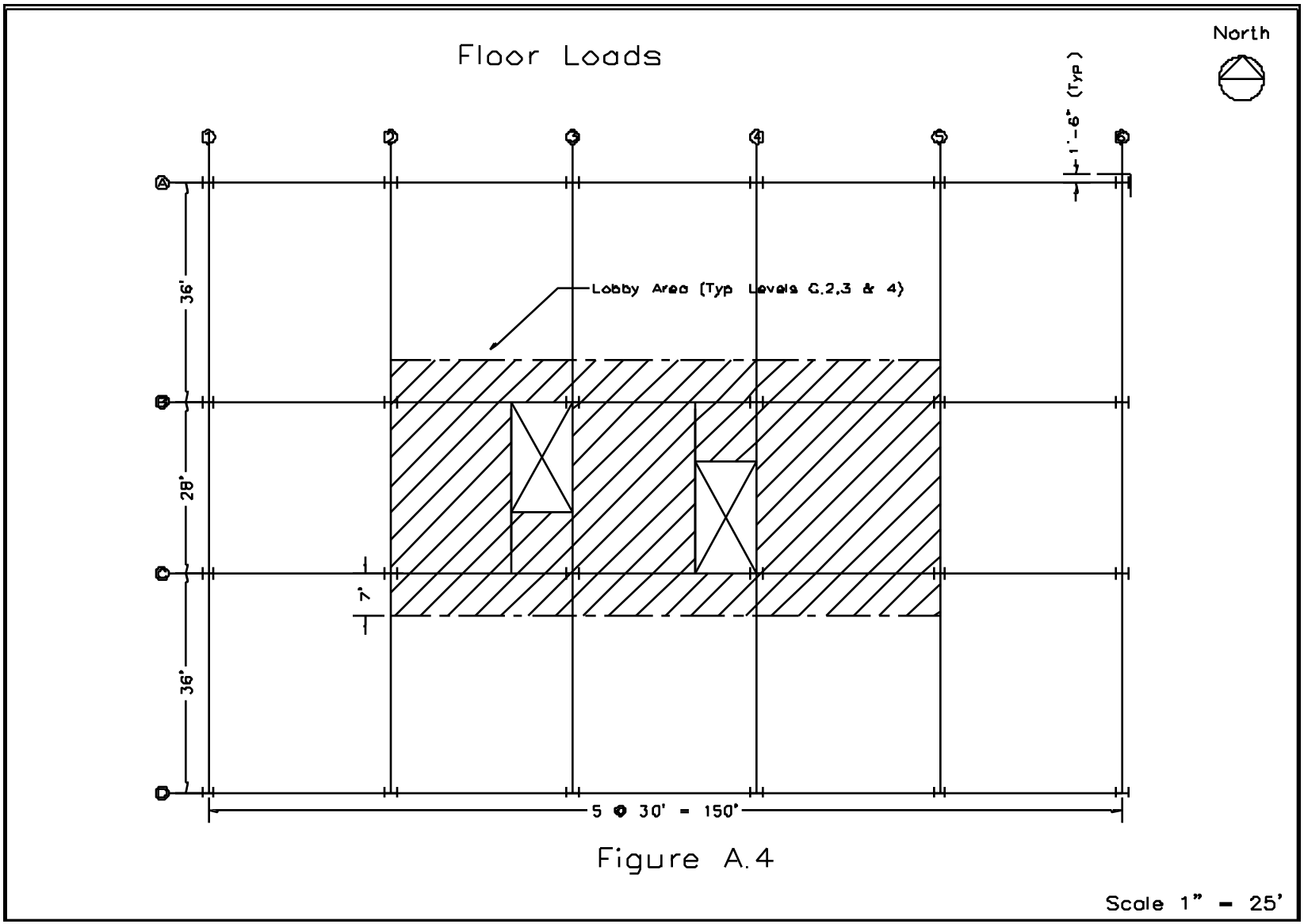

Floor Loads
North
1'-6" (Typ.)
Lobby Area (Typ Levels G,2,3 & 4)
36'
28'
36'
7'
5 @ 30' = 150'
Scale 1" = 25'

Figure A.4

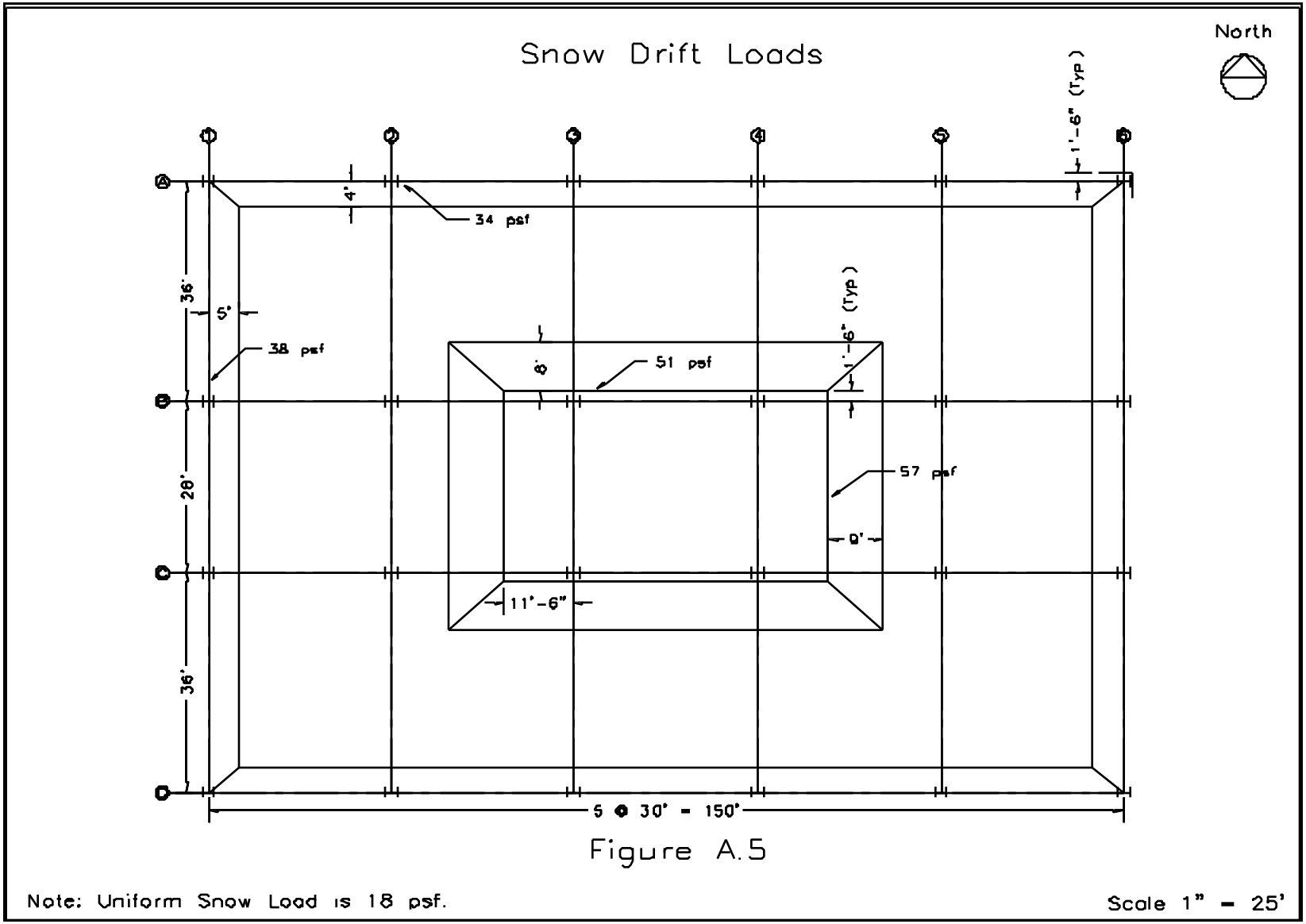



Figure A.5

Note: Uniform Snow Load is 18 psf.

Scale 1" = 25'

East—West Elevation

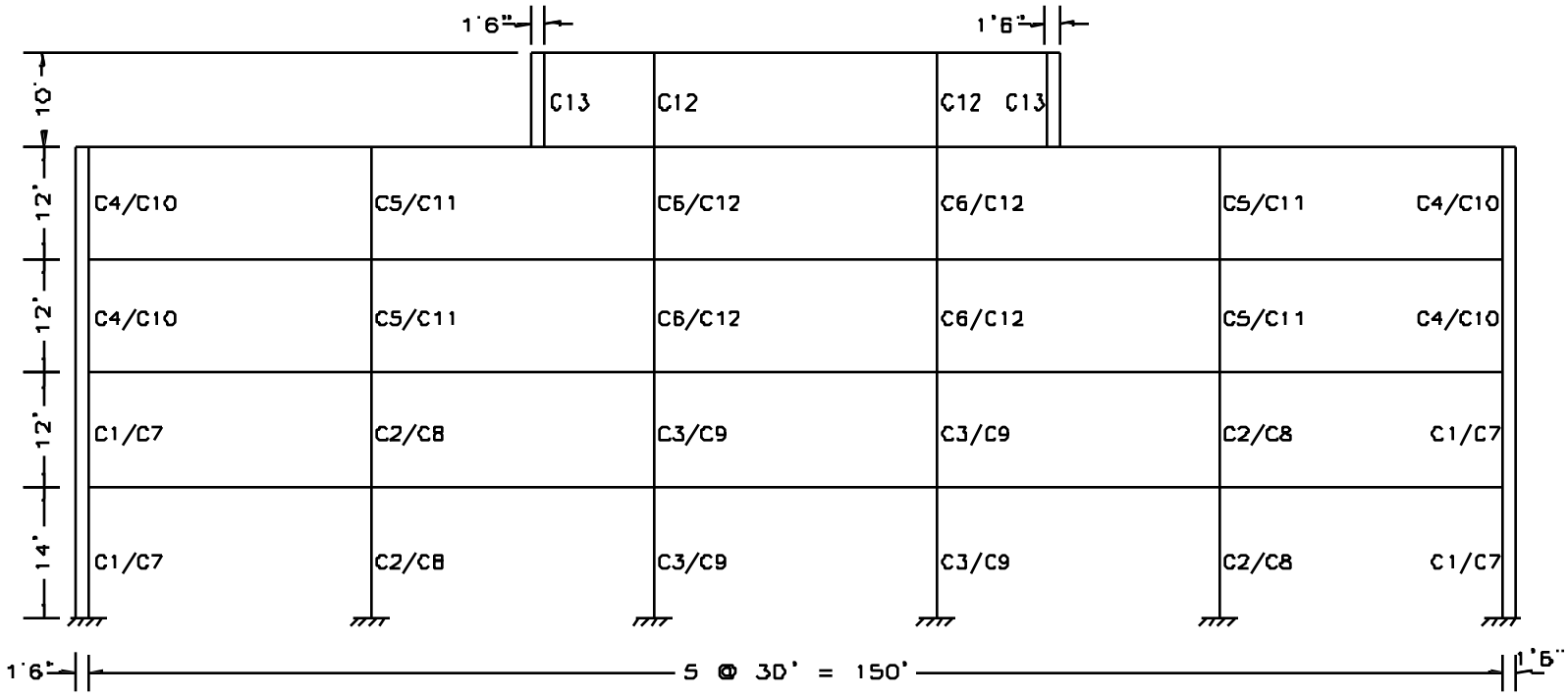



Figure A.6

NOTES 1) Column labels represent exterior frames and interior frames respectively

Scale 1" = 20'

North–South Elevation

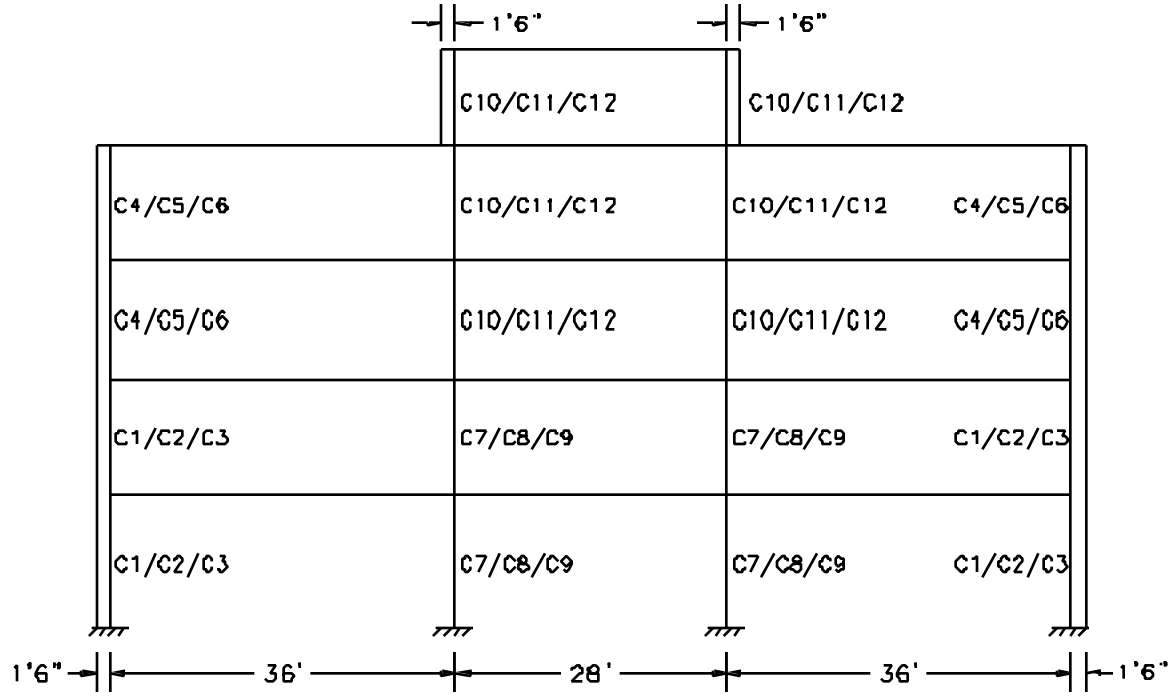



Figure A.7

NOTES 1) Column labels represent exterior frames and interior frames respectively

Scale 1" = 20'

Wind Load Acting in N-S Direction
(150 ft normal to wind)

Exterior Bay (ft) 30
Interior Bay (ft) 30

| Floor Level | Height (ft) | Story (ft) | Pressure (psf) | 1 & 6 Trib. Area (in2) | 2 & 5 Trib. Area (in2) | 3 & 4 Trib. Area (in2) | 1 & 6 Lateral Force (K) | 2 & 5 Lateral Force (K) | 3 & 4 Lateral Force (K) | Story Shear (K) |
|---|---|---|---|---|---|---|---|---|---|---|
| Penthouse Roof | 60 | 10 | 10 | - | - | 132.5 | - | - | 1.33 | |
| | | | | | | | | | | 2.65 |
| Roof | 50 | 12 | 10 | 99.0 | 180.0 | 312.5 | 0.99 | 1.80 | 3.13 | |
| | | | | | | | | | | 14.48 |
| 4 | 38 | 12 | 10 | 198.0 | 360.0 | 360.0 | 1.98 | 3.60 | 3.60 | |
| | | | | | | | | | | 32.84 |
| 3 | 26 | 12 | 10 | 198.0 | 360.0 | 360.0 | 1.98 | 3.60 | 3.60 | |
| | | | | | | | | | | 51.20 |
| 2 | 14 | 14 | 10 | 214.5 | 390.0 | 390.0 | 2.15 | 3.90 | 3.90 | |
| | | | | | | | | | | 71.09 |

Wind Load Acting in E-W Direction
(100 ft normal to wind)

Exterior Bay (ft) 36
Interior Bay (ft) 28

| Floor Level | Height (ft) | Story (ft) | Pressure (psf) | A & D Trib. Area (in2) | B & C Trib. Area (in2) | A & D Lateral Force (K) | B & C Lateral Force (K) | Story Shear (K) |
|---|---|---|---|---|---|---|---|---|
| Penthouse Roof | 60 | 10 | 10 | - | 77.5 | - | 0.78 | |
| | | | | | | | | 1.55 |
| Roof | 50 | 12 | 10 | 117.0 | 269.5 | 1.17 | 2.70 | |
| | | | | | | | | 9.28 |
| 4 | 38 | 12 | 10 | 234.0 | 384.0 | 2.34 | 3.84 | |
| | | | | | | | | 21.64 |
| 3 | 26 | 12 | 10 | 234.0 | 384.0 | 2.34 | 3.84 | |
| | | | | | | | | 34.00 |
| 2 | 14 | 14 | 10 | 253.5 | 416.0 | 2.54 | 4.16 | |
| | | | | | | | | 47.39 |

**Note: Minimum desing wind pressure of 10 psf used according to ASCE 7-95 Section 6.4.2.1.

Table A.2 Wind Load Summary

# Appendix B

# CONNECTION DESIGN AND ANALYSIS

## B.1 Top and Seat Angles with Double Web Angles Connection

| | | | |
|---|---|---|---|
| | | Elastic Modulus | $E$ (ksi) = 29000 |
| | | Yield Stress | $F_y$ (ksi) = 50 |

| Beam Properties | W12x22 | Bolt Properties | A490 |
|---|---|---|---|
| Beam depth | $d$ (in) = 12.31 | Bolt diameter | $d_b$ (in) = 0.875 |
| Beam length | $L$ (ft) = 28.0 | Nut width | $w$ (in) = 1.4375 |

| Top Angle | 8x6x7/8 | Seat Angle | 8x6x7/8 |
|---|---|---|---|
| Leg thickness | $t_t$ (in) = 0.875 | | $t_s$ (in) = 0.875 |
| Leg length | $l_t$ (in) = 4 | | $l_s$ (in) = 4 |
| Distance to bolt hole | $g_c$ (in) = 3.0 | | $g_c$ (in) = 3.0 |
| Bolt diameter | $D$ (in) = 0.875 | | $D$ (in) = 0.875 |
| Distance from heel to toe of fillet | $k_t$ (in) = 1.375 | | $k_t$ (in) = 1.375 |

| Web Angles | 4x4x1/4 |
|---|---|
| Leg thickness | $t_t$ (in) = 0.25 |
| Leg length | $l_t$ (in) = 10 |
| Distance to bolt hole | $g_c$ (in) = 3.0 |
| Bolt diameter | $D$ (in) = 0.875 |
| Distance from heel to toe of fillet | $k_t$ (in) = 0.625 |

Standardized Parameters

Top Angle

| | | | |
|---|---|---|---|
| Beta | $B_t$ = | $g_c / l$ = | 0.75 |
| Gamma | $\gamma_t$ = | $l / t$ = | 4.57 |
| Delta | $\delta_t$ = | $d / t$ = | 14.07 |
| Kappa | $\kappa_t$ = | $k / t$ = | 1.57 |
| Omega | $\omega_t$ = | $w / t$ = | 1.64 |

Web Angle

| | | | |
|---|---|---|---|
| Beta | $B_w$ = | $g_c / l$ = | 0.30 |
| Gamma | $\gamma_w$ = | $l / t$ = | 40.00 |
| Delta | $\delta_w$ = | $d / t$ = | 49.24 |
| Kappa | $\kappa_w$ = | $k / t$ = | 2.50 |
| Omega | $\omega_w$ = | $w / t$ = | 5.75 |
| Rho | $\rho_o$ = | $t_w/t_t$ = | 0.29 |

Initial Connection Stiffness

| | | |
|---|---|---|
| Beta prime of top angle | $B_t' = B_t - 1/2\gamma_t(1+\omega_t)$ = | 0.4609 |
| Beta prime of web angle | $B_w' = B_w - 1/2\gamma_w(1+\omega_w)$ = | 0.2156 |
| Normalized connection stiffness for top angle | $D_{ts} = 3/(B_t'(\gamma_t^2 B_t'^2+0.78))$ | |
| | $D_{ts}$ = 1.2468 | |
| Normalized connection stiffness for web angle | $D_w = 3/(2B_w'(\gamma_w^2 B_w'^2+0.78))$ | |
| | $D_w$ = 0.0925 | |
| Inertia of top angle | $I_{ot}$ = $t_t^3/12$ = | 0.05583 |
| Bending stiffness | $EI_{ot}$ = 1619 | |

Total initial connection stiffness $R_{ki}$ = 468062

Ultimate Moment Capacity

Beta star for the top angles $B_t^* = B_t'^*\gamma_t - \kappa_t$ = 0.5357

Zeta $0 = \zeta_t^4 + B_t^*\zeta_t - 1$ $\zeta_t$ = 0.857

= 0.00

Beta star for the top angles $B_w^* = B_w'^*\gamma_w - \kappa_w$ = 6.1250

Zeta $0 = \zeta_w^4 + B_w^*\zeta_w - 1$ $\zeta_w$ = 0.163

= 0.00

Initial plastic moment $M_{ot} = F_y^*t_t^2/4$ = 9.57

Top angle moment capacity $M_{uts} = M_{ot}^*t_t^*\gamma_t^*\{1+\zeta_t[1 + B_t^* + 2(\kappa_t + \delta_t)]\}$ = 92.9

Web angle moment capacity $M_{uw} = 2^*M_{ot}^*t_t^*\gamma_w^*(1+\zeta_w)^*\rho^{3*}\{[\gamma_w(\zeta_w-1)]/[3^*(\zeta_{w+}1)]+\delta_w+1/\rho)$ = 65.3

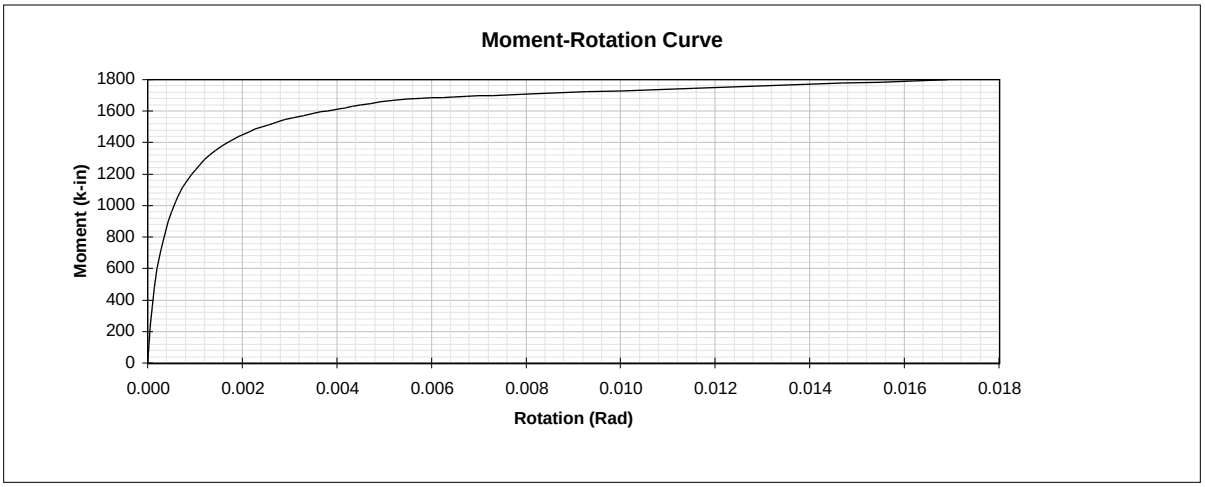
Ultimate moment capacity $M_u = M_{ot}^*t_t^*[M_{uts}/M_{ot}^*t_t+M_{uw}/M_{ot}^*t_t]$ = 158.2

Reference rotation $\theta_o$ $\theta_o = M_u/R_{ki}$ = 0.000338

Shape Parameter $n = 5.483^*\log_{10}\theta_o+14.745$ 0.800 0.800

| Moment (k-ft) | Moment (k-in) | Rotation (rad) | Instant Stiffness (k-ft/rad) |
|---|---|---|---|
| 0 | 0 | 0.000000 | #DIV/0! |
| 10 | 120 | 0.000025 | 404735.32 |
| 20 | 240 | 0.000056 | 359036.81 |
| 30 | 360 | 0.000094 | 318869.46 |
| 40 | 480 | 0.000142 | 282238.08 |
| 50 | 600 | 0.000201 | 248273.63 |
| 60 | 720 | 0.000277 | 216503.20 |
| 70 | 840 | 0.000375 | 186642.41 |
| 80 | 960 | 0.000505 | 158514.55 |
| 90 | 1080 | 0.000682 | 132014.09 |
| 95 | 1140 | 0.000796 | 119356.33 |
| 100 | 1200 | 0.000934 | 107090.22 |
| 105 | 1260 | 0.001103 | 95217.18 |
| 110 | 1320 | 0.001314 | 83741.96 |
| 115 | 1380 | 0.001582 | 72673.15 |
| 120 | 1440 | 0.001935 | 62023.95 |
| 125 | 1500 | 0.002413 | 51813.36 |
| 130 | 1560 | 0.003090 | 42068.24 |
| 135 | 1620 | 0.004112 | 32826.76 |
| 140 | 1680 | 0.005798 | 24144.67 |
| 150 | 1800 | 0.016916 | 8867.35 |

| Beam/Girder | W12x22 |
|---|---|
| Length (ft) | 28 |
| Top Angle | 8x6x7/8 |
| Seat Angle | 8x6x7/8 |
| Web Angle | 4x4x1/4 |
| A572 Grade 50 | |
| A490 7/8" Diameter Bolts | |



## B.2 Partially Restrained Hot Rolled Steel Composite Connection

Use prequalified connections from Partially Restrained Composite Connections Steel Design Guide Series 8 (Leon 1996).

Connection characteristics

| | | | |
|---|---|---|---|
| Type of bars = | #4 | Bottom angle = | L6x4x5/16x6 |
| Number of bars = | 6 | $A_l$ ($in^2$) = | 1.875 |
| $A_s$ ($in^2$) = | 1.20 | | |
| | | Web Angles = | 2L4x4x1/4x9 |
| Section = | W12x14 | $A_{wl}$ ($in^2$) = | 3.88 |
| Beam depth (in) = | 11.91 | | |
| $F_y$ (ksi) = | 50.0 | $F_{yrb}$ (ksi) = | 36.0 |

Y3 (in) = distance from top flange of the girder to the centroid of the reinforcement = 3.0

Negative Bending Moment Curve $M_n^- = C1(1-e^{-C2\theta})+C3\theta$

| | | |
|---|---|---|
| C1 = | $0.18*(4*A_sF_{yrb}+0.857A_lF_y)(d+Y3)$ = | 679.4 |
| C2 = | 0.775 | |
| C3 = | $0.007(A_l+A_{wl})F_y(d+Y3)$ = | 30.03 |

Positive Bending Moment Curve $M_n^+ = C1(1-e^{-C2\theta})+(C3+C4)\theta$

| | | |
|---|---|---|
| C1 = | $0.2400*[(0.48*A_{wl})+A_l]*(d+Y3)*F_y$ = | 668.7 |
| C2 = | $0.0210*(D+Y3/2)$ = | 0.2816 |
| C3 = | $0.0100*(A_{wl}+A_l)*(d+Y3)*F_y$ = | 42.90 |
| C4 = | $0.0065*A_{wl}*(d+Y3)*F_y$ = | 18.80 |

Equivalent Moment of Inertia PRCC at both ends or one end: Both

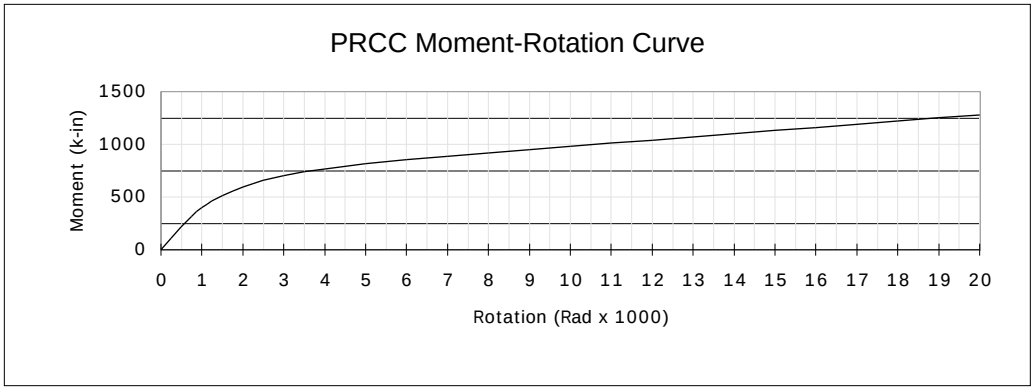
| | |
|---|---|
| $I_{LB}$ ($in^4$) = | 168.3 |
| $I_n$ ($in^4$) = | 163.0 |
| $I_{eq}$ ($in^4$) = | 166.2 |

Curves

| Rotation (radx1000) | Moment (kip-in) |
|---|---|
| 0 | 0.0 |
| 1 | 396.4 |
| 2 | 595.3 |
| 3 | 703.1 |
| 4 | 768.9 |
| 5 | 815.4 |
| 6 | 853.1 |
| 7 | 886.6 |
| 8 | 918.3 |
| 9 | 949.0 |
| 10 | 979.4 |
| 11 | 1009.6 |
| 12 | 1039.7 |
| 13 | 1069.8 |
| 14 | 1099.8 |
| 15 | 1129.9 |
| 16 | 1159.9 |
| 17 | 1189.9 |
| 18 | 1220.0 |
| 19 | 1250.0 |
| 20 | 1280.0 |

Connection Summary

| | | | |
|---|---|---|---|
| Type of bars = | #4 | Bottom angle = | L6x4x5/16x6 |
| Number of bars = | 6 | $A_l$ (in$^2$) = | 1.875 |
| $A_s$ (in$^2$) = | 1.20 | | |
| | | Web Angles = | 2L4x4x1/4x9 |
| Section = | W12x14 | $A_{wl}$ (in$^2$) = | 3.88 |
| Beam depth (in) = | 11.91 | | |
| $F_y$ (ksi) = | 50.0 | $F_{yrb}$ (ksi) = | 36.0 |
| $I_{eq}$ (in$^4$) = | 166.2 | | |



# Appendix C

# MEMBER DESIGNS

## C.1 Hot Rolled Steel Section Composite Design

**Bay Properties**

| | |
|---|---|
| Beam spacing on left (ft) = | 0.0 |
| Beam spacing on right (ft) = | 10.0 |
| Beam/Girder type (exterior/interior) = | exterior |

**Material Properties**

| | |
|---|---|
| Yield Stress (ksi) = | 50.0 |
| Concrete Strength (ksi) = | 4.0 |
| Concrete Weight (pcf) = | 145.0 |

**Beam & Concrete Properties**

| | |
|---|---|
| Span (ft) = | 28.0 |
| Approximate flange width (in) = | 8.0 |
| Effective Width (in) = | 46.0 |

**Deck & Stud Properties**

| | |
|---|---|
| Slab Thickness (in) = | 2.0 |
| Deck Thickness (in) = | 2.0 |
| Stud Height (in) = | 3.5 |
| Rib Width (in) = | 5.0 |
| Number of studs/rib = | 1.0 |
| SRF = | 1.0 |

A. Load Tabulation

| | Service Load (klf) | LF | Factored Load (klf) |
|---|---|---|---|
| $w_{LC}$ | 0.000 | 1.6 | 0.000 |
| $w_{DC}$ | 0.000 | 1.2 | 0.000 |
| $w_D$ | 0.000 | 1.2 | 0.000 |
| $w_L$ | 0.000 | 1.6 | 0.000 |
| Total | 0.000 | | 0.000 |

B. Beam Moments & Shear

| | Uniform | Non-Uniform |
|---|---|---|
| $M_u$ (k-ft) = | 0.0 | 139.3 |
| $M_{CL}$ (k-ft) = | 0.0 | 51.3 |
| Service $M_{CL}$ (k-ft) = | 0.0 | 38.6 |
| Service $M_{LL}$ (k-ft) = | 0.0 | 62.7 |

| | Uniform |
|---|---|
| $V_u$ (k) = | 0.0 |

| | Non-Uniform |
|---|---|
| $V_u$ (k) = | 24.8 |

C. Select section & determine properties

| | |
|---|---|
| Assume a (in) = | 0.33 |
| Y2 (in) = | 3.84 |

From composite tables possible sections are

| | Trial 1 | Trial 2 | Trial 3 |
|---|---|---|---|
| Section | W12x14 | | |
| $F_y$ (ksi) = | 50.0 | | |
| Y2 (in) = | 3.84 | | |
| Y1 (in) = | 3.36 | | |
| $\Sigma Q_n$ (K) = | 52.0 | | |
| $\phi M_n$ (k-ft) = | 93.1 | | |
| $\phi M_p$ (k-ft) = | 65.2 | | |
| $I_x$ ($in^4$) = | 88.6 | | |
| $I_{LB}$ ($in^4$) = | 168.3 | | |
| $A_w$ ($in^2$) = | 2.38 | | |

Compute Y2 for $\Sigma Q_n$

| | | |
|---|---|---|
| $a = \Sigma Q_n/(0.85f'_c b)$ = | | 0.33 |
| Y2 = | 3.83 | |

D. Compute number of studs required

| | | |
|---|---|---|
| $Q_n$ (K) = | 26.1 | Table 5.1 |
| Number of studs = | 4 | |

E. Construction phase strength check

| | | | |
|---|---|---|---|
| $M_u$ (k-ft) = | 51.3 | O.K. | |

F. Service load deflections

| | | | |
|---|---|---|---|
| $\Delta_{CL}$ (in) = | 2.12 | | |
| Camber (in) = | 2.00 | ≤ | 2.50 |
| $I_{x.Req'd}$ ($in^4$) = | 75.2 | | |
| $\Delta_{LL}$ (in) = | 1.81 | L/360 = | 0.93 |
| $I_{LB.Req'd}$ ($in^4$) = | 327 | | |

| | |
|---|---|
| Service construction load deflection | O.K. |
| Service live load deflection | N.G. |

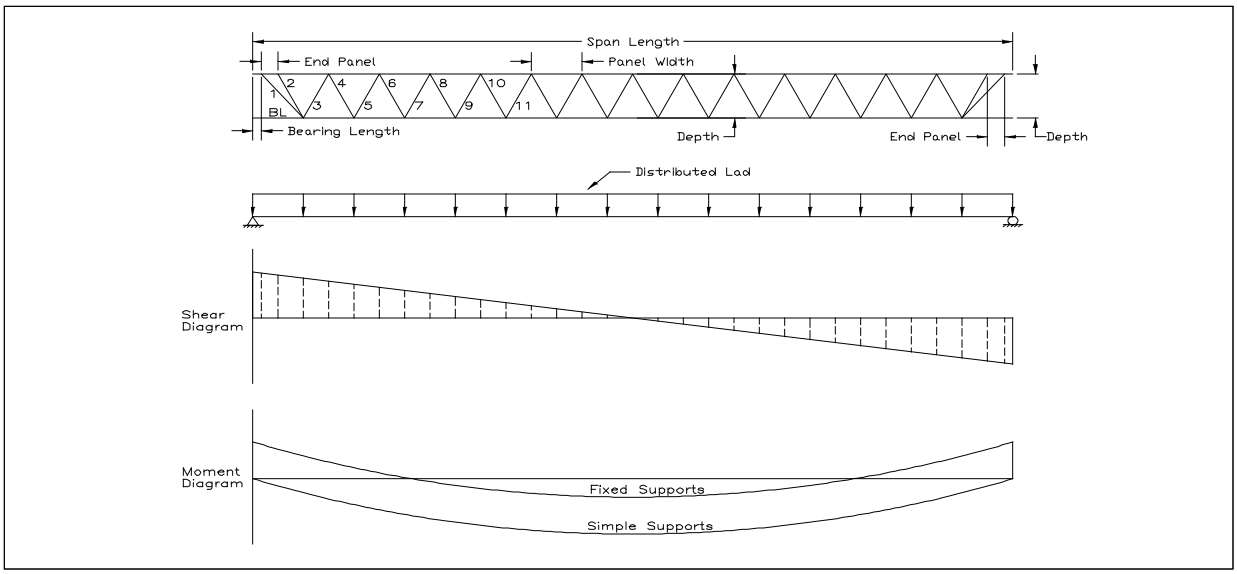
G. Check shear

| | | |
|---|---|---|
| $V_u$ (k) = | 24.8 | |
| $\varphi V$ = | $\varphi(0.6)F_{yw}A_w$ = | 64 |
| | Shear check | O.K. |

H. Final section

| | |
|---|---|
| Use: | W12x14 |
| $I_{LB}$ ($in^4$) = | 168 |
| $F_v$ (ksi) = | 50 |
| Camber (in) = | 2.00 |
| Studs = | 4 |
| Stud diameter (in) = | 3/4 |
| PRCC Needed = | Yes |

## C.2 Non-Composite Partially Restrained Steel Joist Design




Steel Joist Properties

| | | | |
|---|---|---|---|
| Span (ft) = | 28.0 | Yield Stress (ksi) = | 50.0 |
| Number of Panel Widths = | 13 | Ultimate Strength (ksi) = | 65.0 |
| Panel Width (in) = | 24.0 | Young's Modulus (ksi) = | 29000.0 |
| End Panel (in) = | 8.0 | | |
| Bearing Length (in) = | 4.0 | | |
| Depth (in) = | 28.0 | | |
| Type of Joist (floor or roof) = | FLOOR | | |

**Load Step 1 :** Initial Dead Load Only — Type of End Connection (Simple or Fixed) = Simple

| | |
|---|---|
| Dead Load (plf) = | 315.0 |
| Live Load (plf) = | 0.0 |
| Factored Load (plf) = | 378.0 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = wl^2/8 = | 37.0 |
| End Moment (k-ft) = | 0.0 | 0.0 |
| Reaction (k) = | 5.3 | |

**Load Step 2 :** Superimposed Loads — Type of End Connection (Simple or Fixed) = Fixed

| | |
|---|---|
| Dead Load (plf) = | 75.0 |
| Live Load (plf) = | 700.0 |
| Factored Load (plf) = | 1210.0 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = wl^2/24 = | 39.5 |
| End Moment (k-ft) = | Mu = wl^2/12 = | 79.1 |
| Reaction (k) = | 16.9 | |

**Top Chord Design (Compression)**

$P_u$ (k) = $M_u$/(depth -1) = 34.0

Effective Length k = 0.8
Unbraced Length $L_b$ (in) = 24.0
Estimated radius of gyration r (in) = 0.459

Using LRFD Specifications Chapter E

For $\lambda_c$ $Q \leq 1.5$ $F_{cr} = Q(0.658^{Q\lambda c^2})F_y$ (A-B5-15)
For $\lambda_c$ $Q > 1.5$ $F_{cr} = (0.877/\lambda c^2)F_y$ (A-B5-16)

$\lambda_c = (kl/r\pi)\ (F_y/E)$ $\lambda_c$ = 0.553 (E2-4)
$F_{cr}$ = 44.00

$\phi P_n = \phi A_g F_{cr}$ (E2-1)

$A_g$ = 0.91

Trial Size = 1.5 x 1.5 x 0.1700 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 0.553 | |
| $r_x$ (in) = | 0.459 | $F_{cr}$ (ksi) = | 44.0 | |
| $r_z$ (in) = | 0.294 | $P_u$ (k) = | 36.0 | OK |
| $A_g$ ($in^2$) = | 0.962 | I ($in^4$) = | 0.203 | |
| | | Wt (plf) = | 3.27 | |

Filler spacing

$kl / r_x$ = 41.8

$s_{req'd}$ = 12.3 Therefore use 1 filler

Slenderness Requirements

l/r = 81.6 ≤ 90 OK

**Bottom Chord Design (Tension)**

$P_u$ (k) = $M_u$/(depth -1) = 34.0

Using LRFD Specifications Chapter D

$$\phi P_n = \phi A_g F_y \qquad \text{(D1-1)}$$

$A_g$ = 0.76

Trial Size = 1.75 x 1.75 x 0.1550 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.541 | L (in) = | 24.0 |
| $r_z$ (in) = | 0.345 | I ($in^4$) = | 0.304 |
| $A_g$ ($in^2$) = | 1.036 | Wt (plf) = | 3.53 |

Slenderness Requirements

l/r = 69.6 ≤ 240 OK

Calculate bottom chord yield force

$T_s = A_g * F_v$ = 51.8

Check that $A_g * F_v < 0.8 * A_e * F_u$ 53.9 OK

**Web Member Design**

Member 1 (Tension):

$P_u$ (k) = 21.7

$A_g$ (Req'd) = 0.48 (D1-1)

Trial Size = 1.75 x 1.75 x 0.1700

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.539 | L (in) = | 34.4 |
| $r_z$ (in) = | 0.344 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.1646 |
| $A_g$ ($in^2$) = | 0.566 | Wt (plf) = | 1.93 |

Slenderness Requirements

l/r = 99.9 ≤ 240 OK

Member 2 & 3 (Compression):

$P_u$ (k) = 20.6

| | | |
|---|---|---|
| Effective Length | k = | 0.8 |
| Unbraced Length | $L_b$ (in) = | 30.46 |
| Estimated radius of gyration | r (in) = | 0.539 |

| | | |
|---|---|---|
| $\lambda_c$ = | 0.597 | (E2-4) |
| $F_{cr}$ = | 43.07 | (A-B5-15 or 16) |
| $A_g$ = | 0.56 | (E2-1) |

Trial Size = 1.75 x 1.75 x 0.1700

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 0.597 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 43.1 | |
| $r_x$ (in) = | 0.539 | $P_u$ (k) = | 20.7 | OK |
| $r_z$ (in) = | 0.344 | I ($in^4$) = | 0.165 | |
| $A_g$ ($in^2$) = | 0.566 | Wt (plf) = | 1.93 | |

Slenderness Requirements

l/r = 88.5 ≤ 200 OK

Member 4 & 6 (Tension):

$P_u$ (k) = 17.5

$A_g$ (Req'd) = 0.39 (D1-1)

Trial Size = 1.5 x 1.5 x 0.1700

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.459 | L (in) = | 30.5 |
| $r_z$ (in) = | 0.294 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.1013 |
| $A_g$ ($in^2$) = | 0.481 | Wt (plf) = | 1.64 |

Slenderness Requirements

l/r = 103.6 ≤ 240 OK

**Web Member Design (Continued)**

Member 5 & 7 (Compression):

$P_u$ (k) = 15.9

| | | | |
|---|---|---|---|
| Effective Length | k = | 0.8 | |
| Unbraced Length | $L_b$ (in) = | 30.46 | |
| Estimated radius of gyration | r (in) = | 0.461 | |
| | $\lambda_c$ = | 0.699 | (E2-4) |
| | $F_{cr}$ = | 40.8 | (A-B5-15 or 16) |
| | $A_g$ = | 0.46 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1700

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 0.702 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 40.7 | |
| $r_x$ (in) = | 0.459 | $P_u$ (k) = | 16.6 | OK |
| $r_z$ (in) = | 0.294 | I ($in^4$) = | 0.101 | |
| $A_g$ ($in^2$) = | 0.481 | Wt (plf) = | 1.64 | |

Slenderness Requirements

l/r = 103.6 ≤ 200 OK

Member 8,10 & Others (Tension):

$P_u$ (k) = 14.3

$A_g$ (Req'd) = 0.32 (D1-1)

Trial Size = 1.5 x 1.5 x 0.1380

| | | | | |
|---|---|---|---|---|
| $r_x$ (in) = | 0.463 | L (in) = | 30.5 | |
| $r_z$ (in) = | 0.296 | | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.0848 | |
| $A_g$ ($in^2$) = | 0.395 | Wt (plf) = | 1.34 | |

Slenderness Requirements

l/r = 103.1 ≤ 240 OK

Member 9,11 & Others (Compression):

$P_u$ (k) = 12.7

| | | | |
|---|---|---|---|
| Effective Length | k = | 0.8 | |
| Unbraced Length | $L_b$ (in) = | 30.46 | |
| Estimated radius of gyration | r (in) = | 0.466 | |
| | $\lambda_c$ = | 0.692 | (E2-4) |
| | $F_{cr}$ = | 40.9 | (A-B5-15 or 16) |
| | $A_g$ = | 0.37 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1380

| | | | | |
|---|---|---|---|---|
| Q = | 0.996 | $\lambda_c$ = | 0.695 | |
| $r_x$ (in) = | 0.463 | $F_{cr}$ (ksi) = | 40.7 | |
| $r_z$ (in) = | 0.296 | $P_u$ (k) = | 13.7 | OK |
| $A_g$ ($in^2$) = | 0.395 | I ($in^4$) = | 0.0848 | |
| | | Wt (plf) = | 1.34 | |

Slenderness Requirements

| | | | |
|---|---|---|---|
| l/r = | 103.1 | ≤ 200 | OK |

**Bottom Chord Extension Design**

| | | | |
|---|---|---|---|
| $P_u$ (k) = | Mu/(depth -1) = | 35.1 | |
| Effective Length | k = | 0.8 | |
| Unbraced Length | $L_b$ (in) = | 30.46 | |
| Estimated radius of gyration | r (in) = | 0.460 | |
| | $\lambda_c$ = | 0.700 | (E2-4) |
| | $F_{cr}$ = | 40.72 | (A-B5-15 or 16) |
| | $A_g$ = | 1.02 | (E2-1) |

Use Plate (Yes/No) No

Trial Size = -

Plate Properties

| | | | |
|---|---|---|---|
| $L_b$ (in) = | 30.0 | I ($in^4$) = | - |
| w (in) = | 2.0 | $A_g$ ($in^2$) = | - |
| t (in) = | 0.125 | $r_x$ (in) = | - |

Trial Size = 1.75 x 1.75 x 0.1550 Eq. Legs with plate -

Angle Properties

| | | | |
|---|---|---|---|
| Q = | 0.983 | $r_x$ (in) = | 0.541 |
| y (in) = | 0.495 | $A_g$ ($in^2$) = | 1.036 |
| $r_z$ (in) = | 0.345 | I ($in^4$) = | 0.304 |

Hybrid Properties

| | | | | |
|---|---|---|---|---|
| $y_{cent}$ (in) = | - | $\lambda_c$ = | 0.595 | |
| $I_h$ ($in^4$) = | - | $F_{cr}$ (ksi) = | 42.49 | |
| $A_{eh}$ ($in^2$) = | - | $P_u$ (k) = | 37.4 | OK |
| $r_x$ (in) = | - | Wt (plf) = | 3.53 | |

Filler spacing

| | | | |
|---|---|---|---|
| kl / $r_x$ = | 45.0 | | |
| $s_{req'd}$ = | 15.5 | Therefore use | 1 filler |

Slenderness Requirements

| | | | |
|---|---|---|---|
| l/r = | 88.3 | ≤ 120 | OK |

**Deflection Checks**

Assumptions: 1) Joist centroid at depth/2.
2) Distances from angle centroids to joist centroids are approximately (d -1)/2.

| | | |
|---|---|---|
| Approximate equivalent moment of inertia $I_x$ ($in^4$) = | 364.6 | |
| 85% of approximate equivalent moment of inertia $I_x$ ($in^4$) = | 309.9 | |
| Dead load deflection (in) = | 0.582 | |
| Camber (in) = | 0.500 | |
| Service load deflection (in) = | 0.215 | |
| Allowable service load deflection (in) = | 0.933 | OK |

**Material Takeoff**

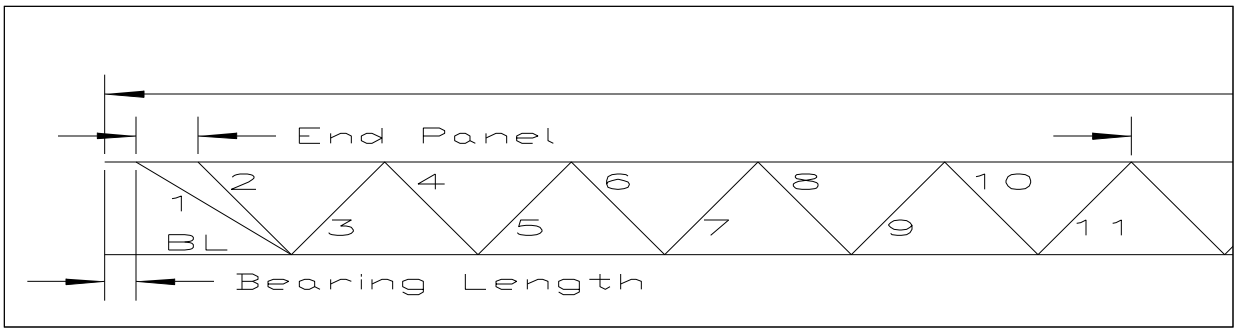
| Member | Section | Length (ft) | Weight (lb's) |
|---|---|---|---|
| Top Chord | 1.5 x 1.5 x 0.1700 Eq. Legs | 28.0 | 91.7 |
| Bottom Chord | 1.75 x 1.75 x 0.1550 Eq. Legs | 28.0 | 98.7 |
| Web Member 1 | 1.75 x 1.75 x 0.1700 | 5.7 | 11.0 |
| Web Member 2 & 3 | 1.75 x 1.75 x 0.1700 | 10.2 | 19.6 |
| Web Member 4 & 6 | 1.5 x 1.5 x 0.1700 | 10.2 | 16.6 |
| Web Member 5 & 7 | 1.5 x 1.5 x 0.1700 | 10.2 | 16.6 |
| Web Member 8,10 & Others | 1.5 x 1.5 x 0.1380 | 17.8 | 23.9 |
| Web Member 9,11 & Others | 1.5 x 1.5 x 0.1380 | 17.8 | 23.9 |
| Buckling Link | 1.75 x 1.75 x 0.1550 Eq. Legs | 5.1 | 17.9 |
| | - | | |
| | | Total | 319.9 |

**Design Summary**

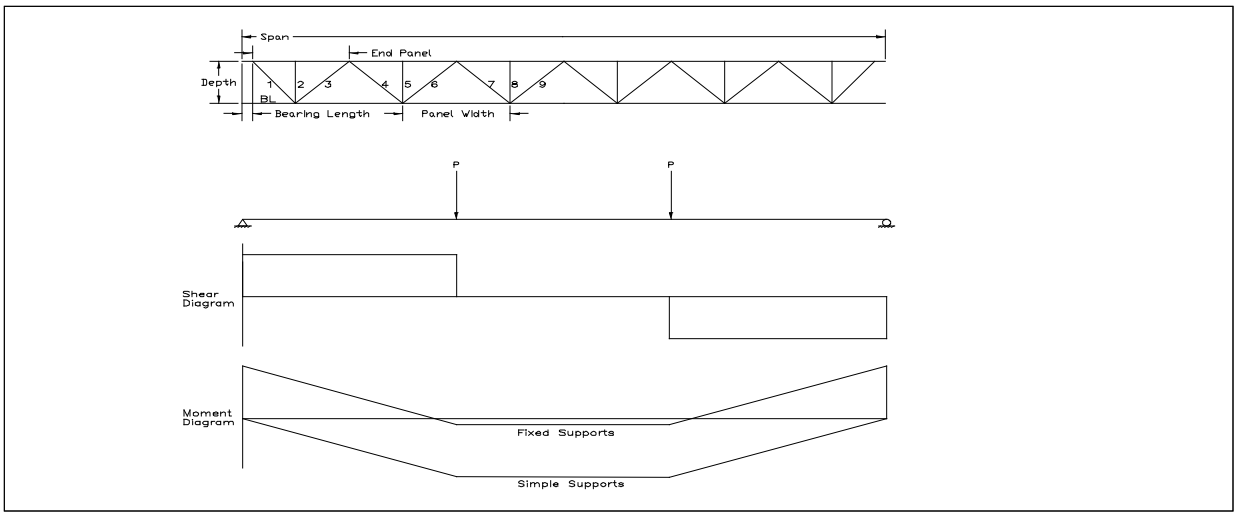
This joist design is for simply supported initial dead load with a buckling link added after dead load is applied to help resist the live load.

| | |
|---|---|
| Type of Joist (Floor or Roof) = | FLOOR |
| Type of End Connection (Simple, PR or Fixed) = | PR |

| | | | |
|---|---|---|---|
| Span (ft) = | 28.0 | Dead Load (plf) = | 315.0 |
| Number of Panel Widths = | 13 | Live Load (plf) = | 700.0 |
| Panel Width (in) = | 24.0 | Factored Load (plf) = | 1498.0 |
| End Panel (in) = | 8.0 | | |
| Bearing Length (in) = | 4.0 | Yield Strength (ksi) = | 50.0 |
| Depth (in) = | 28.0 | Young's Modulus (ksi) = | 29000.0 |
| Approximate Moment of Inertia ($in^4$) = | 365 | Joist Area ($in^2$) = | 3.36 |
| Effective Moment of Inertia 85% ($in^4$) = | 310 | | |
| | | **Total Joist Weight (plf) =** | **11.42** |



## C.3 Non-Composite Partially Restrained Steel Joist Girder Design




Joist Girder Properties

| | | | |
|---|---|---|---|
| Span (ft) = | 30.0 | Yield Stress (ksi) = | 50.0 |
| Number of Panel Widths = | 4 | Young's Modulus (ksi) = | 29000.0 |
| Panel Width (in) = | 60.0 | | |
| End Panel (in) = | 56.0 | | |
| Bearing Length (in) = | 4.0 | | |
| Depth (in) = | 30.0 | | |
| Type of joist girder (floor or roof) = | Roof | | |

**Load Step 1 :** Initial Dead Load Only — Type of End Connection (Simple or Fixed) = SIMPLE

| | |
|---|---|
| Dead Load (k) = | 0.6 |
| Live Load (k) = | 0.0 |
| Factored Load (k) = | 0.7 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = PL/3 = | 7.2 |
| End Moment (k-ft) = | 0.0 | 0.0 |
| Reaction (k) = | 0.7 | |

**Load Step 2 :** Superimposed Loads — Type of End Connection (Simple or Fixed) = SIMPLE

| | |
|---|---|
| Dead Load (k) = | 4.1 |
| Live Load (k) = | 3.9 |
| Factored Load (k) = | 11.2 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = PL/3 = | 111.6 |
| End Moment (k-ft) = | 0.0 | 0.0 |
| Reaction (k) = | 11.2 | |

**Top Chord Design (Compression)**

$P_u$ (k) = $M_u$/(depth -1) = 49.2

| | | |
|---|---|---|
| Effective Length | k = | 1.0 |
| Unbraced Length | $L_b$ (in) = | 30.0 |
| Estimated radius of gyration | r (in) = | 0.600 |
| Angle Effectiveness | Q = | 1.0 |

Using LRFD Specifications Chapter E

For $\lambda_c$ $Q \le 1.5$ $F_{cr} = Q(0.658^{Q\lambda c2})F_y$ (A-B5-15)

For $\lambda_c$ $Q > 1.5$ $F_{cr} = (0.877/\lambda_c^2)F_y$ (A-B5-16)

$\lambda_c=(kl/r\pi)\ (F_y/E)$ $\lambda_c$ = 0.661 (E2-4)

$F_{cr}$ = 41.65

$\phi P_n = \phi A_g F_{cr}$ (E2-1)

$A_g$ = 1.39

Trial Size = 2 x 2 x 0.1870 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 0.642 | |
| $r_x$ (in) = | 0.617 | $F_{cr}$ (ksi) = | 42.1 | |
| $r_z$ (in) = | 0.394 | $P_u$ (k) = | 51.0 | OK |
| $A_g$ ($in^2$) = | 1.426 | I ($in^4$) = | 0.544 | |
| | | Wt (plf) = | 4.85 | |

Filler spacing

kl / $r_x$ = 48.6

$s_{req'd}$ = 19.1 Therefore use 1 filler

Slenderness Requirements

l/r = 48.6 ≤ 90 OK

**Bottom Chord Design (Tension)**

$P_u$ (k) = Mu/(depth -1) = 49.2

Using LRFD Specifications Chapter D

$\phi P_n = \phi A_g F_y$ (D1-1)

$A_g$ = 1.09

Trial Size = 2 x 2 x 0.1630 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.621 | L (in) = | 60.0 |
| $r_z$ (in) = | 0.395 | I ($in^4$) = | 0.482 |
| $A_g$ ($in^2$) = | 1.250 | Wt (plf) = | 4.25 |

Slenderness Requirements

l/r = 151.9 ≤ 240 OK

**Web Member Design**

Member 1 & 4 (Tension):

$P_u$ (k) = 16.8

(D1-1)

$A_g$ (Req'd) = 0.37

Trial Size = 1.5 x 1.5 x 0.1550 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.461 | L (in) = | 42.4 |
| $r_z$ (in) = | 0.461 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.1874 |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 |

Slenderness Requirements

l/r = 92.0 ≤ 240 OK

Member 3 & 6 (Compression):

$P_u$ (k) = 16.8

| | | |
|---|---|---|
| Effective Length | k = | 1.0 |
| Unbraced Length | $L_b$ (in) = | 42.43 |
| Estimated radius of gyration | r (in) = | 0.463 |

| | | |
|---|---|---|
| $\lambda_c$ = | 1.212 | (E2-4) |
| $F_{cr}$ = | 27.04 | (A-B5-15 or 16) |
| $A_g$ = | 0.73 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1550 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 1.216 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 26.9 | |
| $r_x$ (in) = | 0.461 | $P_u$ (k) = | 20.2 | OK |
| $r_z$ (in) = | 0.295 | I ($in^4$) = | 0.187 | |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 | |

Filler spacing

kl / $r_x$ = 92.0

$s_{req'd}$ = 27.1 Therefore use 1 filler

Slenderness Requirements

l/r = 144.0 ≤ 200 OK

**Web Member Design (Continued)**

Member 7 (Tension):

$P_u$ (k) = 3.0

$A_g$ (Req'd) = 0.07 (D1-1)

Trial Size = 1.5 x 1.5 x 0.1550 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.461 | L (in) = | 42.4 |
| $r_z$ (in) = | 0.295 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.0765 |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 |

Slenderness Requirements

l/r = 144.0 ≤ 240 OK

Member 9 (Compression):

$P_u$ (k) = 3.0

| | | | |
|---|---|---|---|
| Effective Length | k = | 1.0 | |
| Unbraced Length | $L_b$ (in) = | 42.43 | |
| Estimated radius of gyration | r (in) = | 0.463 | |
| | $\lambda_c$ = | 1.212 | (E2-4) |
| | $F_{cr}$ = | 27.04 | (A-B5-15 or 16) |
| | $A_g$ = | 0.13 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1550 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 1.216 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 26.9 | |
| $r_x$ (in) = | 0.461 | $P_u$ (k) = | 20.2 | OK |
| $r_z$ (in) = | 0.295 | I ($in^4$) = | 0.187 | |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 | |

Filler spacing

kl / $r_x$ = 92.0

$s_{req'd}$ = 27.1 Therefore use 1 filler

Slenderness Requirements

l/r = 144.0 ≤ 200 OK

**Web Member Design (Continued)**

Member 2,5 & 8 (Zero Force Members): $P_u$ (k) = 1.0

| | | | |
|---|---|---|---|
| Effective Length | k = | 1.0 | |
| Unbraced Length | $L_b$ (in) = | 30.00 | |
| Estimated radius of gyration | r (in) = | 0.463 | |
| | $\lambda_c$ = | 0.857 | (E2-4) |
| | $F_{cr}$ = | 36.77 | (A-B5-15 or 16) |
| | $A_g$ = | 0.03 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1550 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 0.860 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 36.7 | |
| $r_x$ (in) = | 0.461 | $P_u$ (k) = | 27.5 | OK |
| $r_z$ (in) = | 0.295 | I ($in^4$) = | 0.187 | |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 | |

Filler spacing

| | | | |
|---|---|---|---|
| $kl / r_x$ = | 65.1 | | |
| $s_{req'd}$ = | 19.2 | Therefore use | 1 filler |

Slenderness Requirements

| | | | |
|---|---|---|---|
| l/r = | 101.8 | ≤ 200 | OK |

**Bottom Chord Extension Design**

| | | | |
|---|---|---|---|
| $P_u$ (k) = | Mu/(depth -1) = | 0.0 | |
| Effective Length | k = | 1.0 | |
| Unbraced Length | $L_b$ (in) = | 30.00 | |
| Estimated radius of gyration | r (in) = | 0.467 | |
| | $\lambda_c$ = | 0.849 | (E2-4) |
| | $F_{cr}$ = | 36.97 | (A-B5-15 or 16) |
| | $A_g$ = | 0.00 | (E2-1) |

Use Plate (Yes/No) No

Trial Size = -

Plate Properties

| | | | |
|---|---|---|---|
| $L_b$ (in) = | - | I ($in^4$) = | - |
| w (in) = | - | $A_g$ ($in^2$) = | - |
| t (in) = | - | $r_x$ (in) = | - |

Use Link (Yes/No) No

Trial Size = - with plate -

Angle Properties

| | | | |
|---|---|---|---|
| Q = | - | $r_x$ (in) = | - |
| y (in) = | - | $A_g$ ($in^2$) = | - |
| $r_z$ (in) = | - | I ($in^4$) = | - |

Hybrid Properties

| | | | | |
|---|---|---|---|---|
| $y_{cent}$ (in) = | - | $\lambda_c$ = | - | |
| $I_h$ ($in^4$) = | - | $F_{cr}$ (ksi) = | - | |
| $A_{gh}$ ($in^2$) = | - | $P_u$ (k) = | - | - |
| $r_x$ (in) = | - | Wt (plf) = | - | |

Filler spacing

kl / $r_x$ = -

$s_{req'd}$ = - Therefore use 1 filler

Slenderness Requirements

| | | | |
|---|---|---|---|
| l/r = | - | ≤ 120 | - |

**Deflection Checks**

Assumptions: 1) Joist girder centroid at depth/2.
2) Distances from angle centroids to joist girder centroids are approximately (d -1)/2.

| | | |
|---|---|---|
| Approximate equivalent moment of inertia $I_x$ ($in^4$) = | 563.7 | |
| 85% of approximate equivalent moment of inertia $I_x$ ($in^4$) = | 479.1 | |
| Dead load deflection (in) = | 0.086 | |
| Camber (in) = | 0.000 | |
| Service load deflection (in) = | 0.465 | |
| Allowable service load deflection (in) = | 1.500 | OK |

**Material Takeoff**

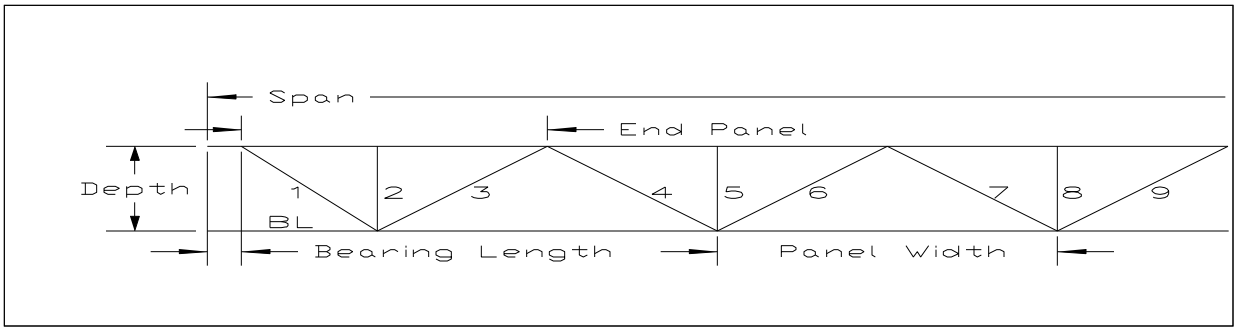
| Member | Section | Length (ft) | Weight (lb's) |
|---|---|---|---|
| Top Chord | 2 x 2 x 0.1870 Eq. Legs | 30.0 | 145.6 |
| Bottom Chord | 2 x 2 x 0.1630 Eq. Legs | 25.0 | 106.3 |
| Web Member 1 & 4 | 1.5 x 1.5 x 0.1550 Eq. Legs | 13.7 | 41.1 |
| Web Member 3 & 6 | 1.5 x 1.5 x 0.1550 Eq. Legs | 14.1 | 42.4 |
| Web Member 7 | 1.5 x 1.5 x 0.1550 Eq. Legs | 7.1 | 21.2 |
| Web Member 9 | 1.5 x 1.5 x 0.1550 Eq. Legs | 7.1 | 21.2 |
| Web Member 2,5 & 8 | 1.5 x 1.5 x 0.1550 Eq. Legs | 15.0 | 45.0 |
| Buckling Link | - | 5.0 | 0.0 |
| | - | | |
| | | Total | 422.9 |

**Design Summary**

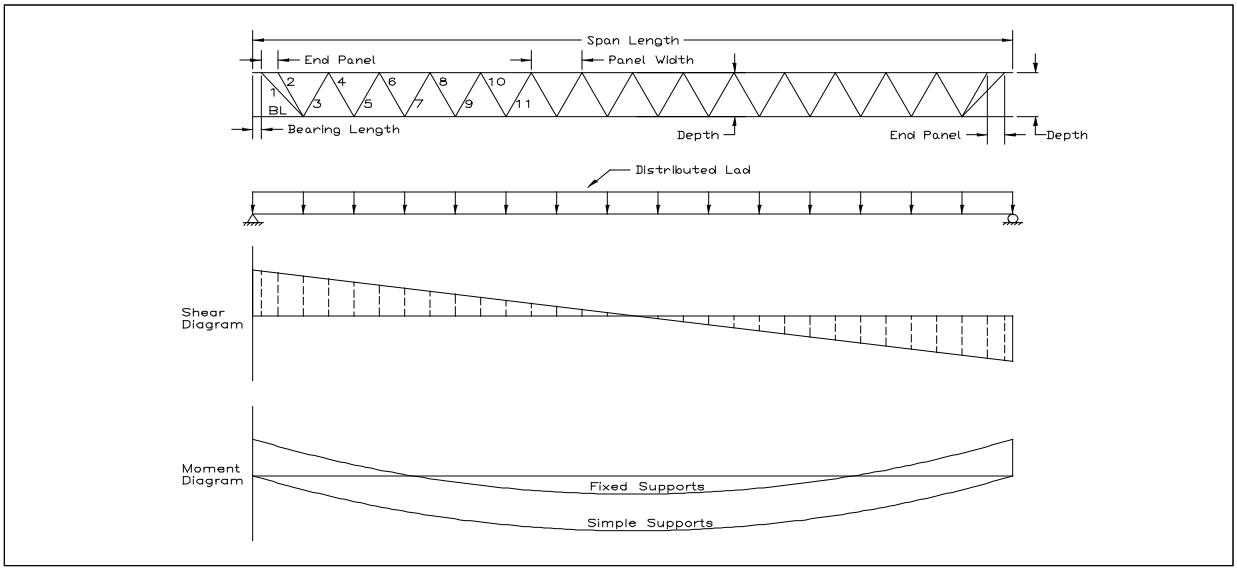
This joist girder design is for simply supported initial dead load with a buckling link added after dead load is applied to help resist the live load.

| | |
|---|---|
| Type of Joist Girder (Floor or Roof) = | Roof |
| Type of End Connection (Simple, PR or Fixed) = | PR |

| | | | |
|---|---|---|---|
| Span (ft) = | 30.0 | Dead Load (plf) = | 0.6 |
| Number of Panel Widths = | 4 | Live Load (plf) = | 3.9 |
| Panel Width (in) = | 60.0 | Factored Load (plf) = | 7.0 |
| End Panel (in) = | 56.0 | | |
| Bearing Length (in) = | 4.0 | Yield Strength (ksi) = | 50.0 |
| Depth (in) = | 30.0 | Young's Modulus (ksi) = | 29000.0 |
| Approximate Moment of Inertia ($in^4$) = | 564 | Joist Area ($in^2$) = | 4.14 |
| Effective Moment of Inertia 85% ($in^4$) = | 479 | | |
| | | **Total Joist Weight (plf) =** | **14.10** |



C.4 Composite Partially Restrained Steel Joist Design




**Load Step 1 :** Initial Dead Load Only

Type of End Connection (Simple or Fixed) = Simple

| | |
|---|---|
| Dead Load (plf) = | 300.0 |
| Live Load (plf) = | 0.0 |
| Factored Load (plf) = | 360.0 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = wl^2/8 = | 35.3 |
| End Moment (k-ft) = | 0.0 | 0.0 |
| Reaction (k) = | 5.0 | |

**Load Step 2 :** Superimposed Loads

Type of End Connection (Simple or Fixed) = Fixed

| | |
|---|---|
| Dead Load (plf) = | 75.0 |
| Live Load (plf) = | 680.0 |
| Factored Load (plf) = | 1178.0 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = wl^2/24 = | 38.5 |
| End Moment (k-ft) = | Mu = wl^2/12 = | 77.0 |
| Reaction (k) = | 16.5 | |

| Joist Properties | |
|---|---|
| Yield Stress (ksi) = | 50.0 |
| Ultimate Strength (ksi) = | 65.0 |
| Young's Modulus (ksi) = | 29000.0 |
| Span (ft) = | 28.0 |
| Number of Panel Widths = | 13 |
| Panel Width (in) = | 24.0 |
| End Panel (in) = | 8.0 |
| Bearing Length (in) = | 4.0 |
| Depth (in) = | 28.0 |
| Joist (floor or roof) = | FLOOR |

| Reinforcement Properties | |
|---|---|
| Yield Strength (ksi) = | 60.0 |

| Bay Properties | |
|---|---|
| Joist spacing on left (ft) = | 0.0 |
| Joist spacing on right (ft) = | 10.0 |
| Joist (exterior/interior) = | exterior |

| Slab Properties | |
|---|---|
| Concrete Strength (ksi) = | 3.0 |
| Unit weight of concrete (pcf) = | 145 |
| Elastic Modulus (ksi) = | 3024 |
| Reinforcement Concrete Cover (in) = | 1.0 |
| Slab Thickness Above Deck (in) = | 2.5 |
| Rib Height (in) = | 1.5 |
| Rib Width (in) = | 5.0 |

| Stud Properties | |
|---|---|
| Stud Diameter (in) = | 0.500 |
| Stud Area ($in^2$) = | 0.196 |
| Stud Height (in) = | 3.5 |
| Number of studs/rib = | 1.0 |

| Effective Width | |
|---|---|
| Approximate flange width (in) = | 0.0 |
| Effective Width (in) = | 42.0 |

**Bottom Chord Design (Tension)**

$P_u$ (k) = $M_u$/(depth -1) = 32.8

Using LRFD Specifications Chapter D

$\phi P_n = \phi A_g F_y$ (D1-1)

$A_g$ = 0.73

Trial Size = 1.5 x 1.5 x 0.1380 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.463 | L (in) = | 24.0 |
| $r_z$ (in) = | 0.296 | | |
| $y_{bc}$ (in) = | 0.426 | I ($in^4$) = | 0.170 |
| $A_g$ ($in^2$) = | 0.790 | Wt (plf) = | 2.69 |

Slenderness Requirements

l/r = 81.2 ≤ 240 OK

Calculate bottom chord yield force

$T_s = A_g*F_y$ = 39.5

Check that $A_g*F_y < 0.8*A_e*F_u$ 41.1 OK

Calculate the depth of the compressive stress block

$a = T_s/(0.85*f`_c*b_{eff})$ = 0.369

Calculate member depth

d = joist depth + slab depth - a/2 - $y_{bc}$ = 31.4

Calculate nominal moment capacity

$\phi M_n = \phi*T_s*d$ = 93.0 > $M_u$ = 73.8 OK

**Top Chord Design (Compression)**

$P_u$ (k) = $M_u$/(depth -1) = 15.7

| | | |
|---|---|---|
| Effective Length | k = | 0.8 |
| Unbraced Length | $L_b$ (in) = | 24.0 |
| Estimated radius of gyration | r (in) = | 0.467 |

Using LRFD Specifications Chapter E

For $\lambda_c$ $Q \le 1.5$ $F_{cr} = Q(0.658^{Q\lambda c^2})F_y$ (A-B5-15)

For $\lambda_c$ $Q > 1.5$ $F_{cr} = (0.877/\lambda c^2)F_y$ (A-B5-16)

$\lambda_c=(kl/r\pi)\ (F_y/E)$ $\lambda_c$ = 0.544 (E2-4)

$F_{cr}$ = 44.18

$\phi P_n = \phi A_g F_{cr}$ (E2-1)

$t_{s,req'd} = \phi_s/3.0$ = 0.167 $A_g$ = 0.42

Trial Size = 1.75 x 1.75 x 0.1700 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 0.471 | |
| $r_x$ (in) = | 0.539 | $F_{cr}$ (ksi) = | 45.6 | |
| $r_z$ (in) = | 0.344 | $P_u$ (k) = | 43.9 | OK |
| $A_g$ ($in^2$) = | 1.132 | I ($in^4$) = | 0.329 | |
| | | Wt (plf) = | 3.85 | |

Filler spacing

kl / $r_x$ = 35.6

$s_{req'd}$ = 12.3 Therefore use 1 filler

Slenderness Requirements

l/r = 69.7 ≤ 90 OK

**Web Member Design**

Member 1 (Tension): $P_u$ (k) = 21.0

$A_g$ (Req'd) = 0.47 (D1-1)

Trial Size = 1.5 x 1.5 x 0.1130 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.467 | L (in) = | 34.4 |
| $r_z$ (in) = | 0.297 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.142 |
| $A_g$ ($in^2$) = | 0.652 | Wt (plf) = | 2.22 |

Slenderness Requirements

l/r = 115.9 ≤ 240 OK

Member 2 & 3 (Compression): $P_u$ (k) = 20.0

| | | |
|---|---|---|
| Effective Length | k = | 0.8 |
| Unbraced Length | $L_b$ (in) = | 30.46 |
| Estimated radius of gyration | r (in) = | 0.467 |

| | | |
|---|---|---|
| $\lambda_c$ = | 0.690 | (E2-4) |
| $F_{cr}$ = | 40.97 | (A-B5-15 or 16) |
| $A_g$ = | 0.57 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1130 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 0.920 | $\lambda_c$ = | 0.690 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 38.3 | |
| $r_x$ (in) = | 0.467 | $P_u$ (k) = | 21.2 | OK |
| $r_z$ (in) = | 0.297 | I ($in^4$) = | 0.142 | |
| $A_g$ ($in^2$) = | 0.652 | Wt (plf) = | 2.22 | |

Filler spacing

kl / $r_x$ = 52.2

$s_{req'd}$ = 15.5 Therefore use 1 filler

Slenderness Requirements

l/r = 102.6 ≤ 200 OK

Member 4 & 6 (Tension): $P_u$ (k) = 16.9

$A_g$ (Req'd) = 0.38 (D1-1)

Trial Size = 1.25 x 1.25 x 0.1090 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.387 | L (in) = | 30.5 |
| $r_z$ (in) = | 0.247 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.0782 |
| $A_g$ ($in^2$) = | 0.522 | Wt (plf) = | 1.78 |

Slenderness Requirements

l/r = 123.5 ≤ 240 OK

**Web Member Design (Continued)**

Member 5 & 7 (Compression): $P_u$ (k) = 15.4

| | | | |
|---|---|---|---|
| Effective Length | k = | 0.8 | |
| Unbraced Length | $L_b$ (in) = | 30.46 | |
| Estimated radius of gyration | r (in) = | 0.387 | |
| | $\lambda_c$ = | 0.832 | (E2-4) |
| | $F_{cr}$ = | 37.4 | (A-B5-15 or 16) |
| | $A_g$ = | 0.48 | (E2-1) |

Trial Size = 1.25 x 1.25 x 0.1090 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 0.978 | $\lambda_c$ = | 0.832 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 36.8 | |
| $r_x$ (in) = | 0.387 | $P_u$ (k) = | 16.3 | OK |
| $r_z$ (in) = | 0.247 | I ($in^4$) = | 0.078 | |
| $A_g$ ($in^2$) = | 0.522 | Wt (plf) = | 1.78 | |

Filler spacing

| | | | | |
|---|---|---|---|---|
| kl / $r_x$ = | 63.0 | | | |
| $s_{req'd}$ = | 15.5 | | Therefore use | 1 filler |

Slenderness Requirements

| | | | |
|---|---|---|---|
| l/r = | 123.5 | ≤ 200 | OK |

Member 8,10 & Others (Tension): $P_u$ (k) = 13.8

| | | | |
|---|---|---|---|
| | $A_g$ (Req'd) = | 0.31 | (D1-1) |

Trial Size = 1.25 x 1.25 x 0.1090 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.387 | L (in) = | 30.5 |
| $r_z$ (in) = | 0.247 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.0782 |
| $A_g$ ($in^2$) = | 0.522 | Wt (plf) = | 1.78 |

Slenderness Requirements

| | | | |
|---|---|---|---|
| l/r = | 123.5 | ≤ 240 | OK |

Member 9,11 & Others (Compression): $P_u$ (k) = 12.3

| | | | |
|---|---|---|---|
| Effective Length | k = | 0.8 | |
| Unbraced Length | $L_b$ (in) = | 30.46 | |
| Estimated radius of gyration | r (in) = | 0.387 | |
| | $\lambda_c$ = | 0.832 | (E2-4) |
| | $F_{cr}$ = | 37.4 | (A-B5-15 or 16) |
| | $A_g$ = | 0.39 | (E2-1) |

Trial Size = 1.25 x 1.25 x 0.1090 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 0.978 | $\lambda_c$ = | 0.832 | |
| $r_x$ (in) = | 0.387 | $F_{cr}$ (ksi) = | 36.8 | |
| $r_z$ (in) = | 0.247 | $P_u$ (k) = | 16.3 | OK |
| $A_g$ ($in^2$) = | 0.522 | I ($in^4$) = | 0.0782 | |
| | | Wt (plf) = | 1.78 | |

Filler spacing

$kl / r_x$ = 63.0

$s_{req'd}$ = 15.5 Therefore use 1 filler

Slenderness Requirements

l/r = 123.5 ≤ 120 OK

**Shear Connectors**

SRF = $0.85/\ N_r[w_r/h_r(H_s/h_r-1.0)] \le 1.0$ = 1.0

$Q_n = 0.5*A_s*\ (f'_c*E_c) \le A_s*F_u$ = 9.4

Number of studs required for Positive Moment Region

$N = 1.3*A_s*F_v/Q_n$ = 5.5

Total number of studs = 12

Number of studs required for Negative Moment Region

$N = 1.3*A_r*F_{vr}/Q_n$ = 5.0

Total number of studs = 10

**Bottom Chord Extension Design**

$P_u$ (k) = $M_u$/(depth -1) = 34.2

| | | | |
|---|---|---|---|
| Effective Length | k = | 1.0 | |
| Unbraced Length | $L_b$ (in) = | 30.46 | |
| Estimated radius of gyration | r (in) = | 0.539 | |
| | $\lambda_c$ = | 0.747 | (E2-4) |
| | $F_{cr}$ = | 39.60 | (A-B5-15 or 16) |
| | $A_g$ = | 1.02 | (E2-1) |

Use Plate (Yes/No) Yes

Trial Size = PL 30 x 3.5 x 0.25

Plate Properties

| | | | |
|---|---|---|---|
| $L_b$ (in) = | 30.0 | I ($in^4$) = | 0.000 |
| w (in) = | 3.0 | $A_g$ ($in^2$) = | 0.38 |
| t (in) = | 0.125 | $r_x$ (in) = | 0.036 |

Trial Size = 1.5 x 1.5 x 0.1380 Eq. Legs with plate PL 30 x 3.5 x 0.25

Angle Properties

| | | | |
|---|---|---|---|
| Q = | 0.996 | $r_x$ (in) = | 0.463 |
| y (in) = | 0.426 | $A_g$ ($in^2$) = | 0.790 |
| $r_z$ (in) = | 0.296 | I ($in^4$) = | 0.170 |

Hybrid Properties

| | | | | |
|---|---|---|---|---|
| $y_{cent}$ (in) = | 0.309 | $\lambda_c$ = | 0.889 | |
| $I_h$ ($in^4$) = | 0.239 | $F_{cr}$ (ksi) = | 35.83 | |
| $A_{eh}$ ($in^2$) = | 1.165 | $P_u$ (k) = | 35.5 | OK |
| $r_x$ (in) = | 0.453 | Wt (plf) = | 3.96 | |

Filler spacing

kl / $r_x$ = 67.2

$s_{req'd}$ = 19.9 Therefore use 1 filler

Slenderness Requirements

l/r = 103.1 ≤ 120 OK

**Reinforcement Design**

$A_r$ is the area of negative reinforcement
$F_{yr}$ is the negative reinforcement steel strength

Calculate moment arm

d = joist depth + slab depth - $y_{bc}$ - reinforcement cover = 30.6

reinforcement force = $M_u$/d = $T_{s.req'd}$ = 30.2

$A_{req'd} = T_{s,req'd}/\phi F_{yr}$ = 0.56

Therefore use 3 #4 Bars

$A_g$ ($in^2$) = 0.60

**Deflection Checks**

| | | |
|---|---|---|
| Joist centroid (in) = | 2.91 | |
| 85% Non-Composite moment of inertia $I_x$ ($in^4$) = | 298.2 | |
| 85% Composite moment of inertia $I_x$ ($in^4$) = | 611.6 | |
| Dead load deflection (in) = | 0.576 | |
| Camber (in) = | 0.500 | |
| Service load deflection (in) = | 0.106 | |
| Allowable service load deflection (in) = | 0.933 | OK |

**Material Takeoff**

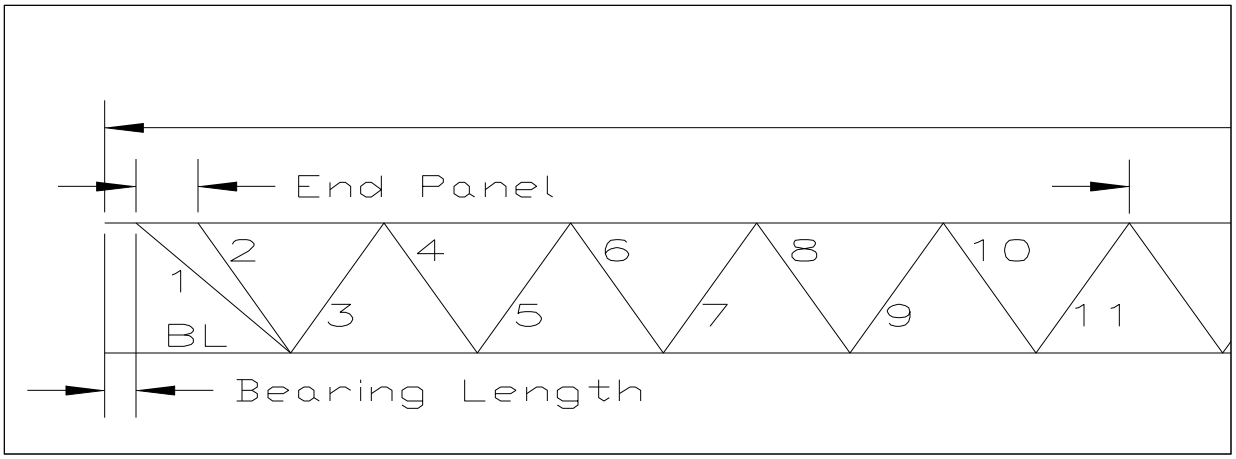
| Member | Section | Length (ft) | Weight (lb's) |
|---|---|---|---|
| Top Chord | 1.75 x 1.75 x 0.1700 Eq. Legs | 28.0 | 107.9 |
| Bottom Chord | 1.5 x 1.5 x 0.1380 Eq. Legs | 22.9 | 61.6 |
| Web Member 1 | 1.5 x 1.5 x 0.1130 Eq. Legs | 5.7 | 12.7 |
| Web Member 2 & 3 | 1.5 x 1.5 x 0.1130 Eq. Legs | 10.2 | 22.5 |
| Web Member 4 & 6 | 1.25 x 1.25 x 0.1090 Eq. Legs | 10.2 | 18.0 |
| Web Member 5 & 7 | 1.25 x 1.25 x 0.1090 Eq. Legs | 10.2 | 18.0 |
| Web Member 8,10 & Others | 1.25 x 1.25 x 0.1090 Eq. Legs | 17.8 | 31.6 |
| Web Member 9,11 & Others | 1.25 x 1.25 x 0.1090 Eq. Legs | 17.8 | 31.6 |
| Buckling Link | 1.5 x 1.5 x 0.1380 Eq. Legs | 5.1 | 20.1 |
| | with PL 30 x 3.5 x 0.25 | | |
| | | Total | 324.1 |
| Reinforcement | 3 #4 Bars | 7.0 | 14.3 |

**Design Summary**

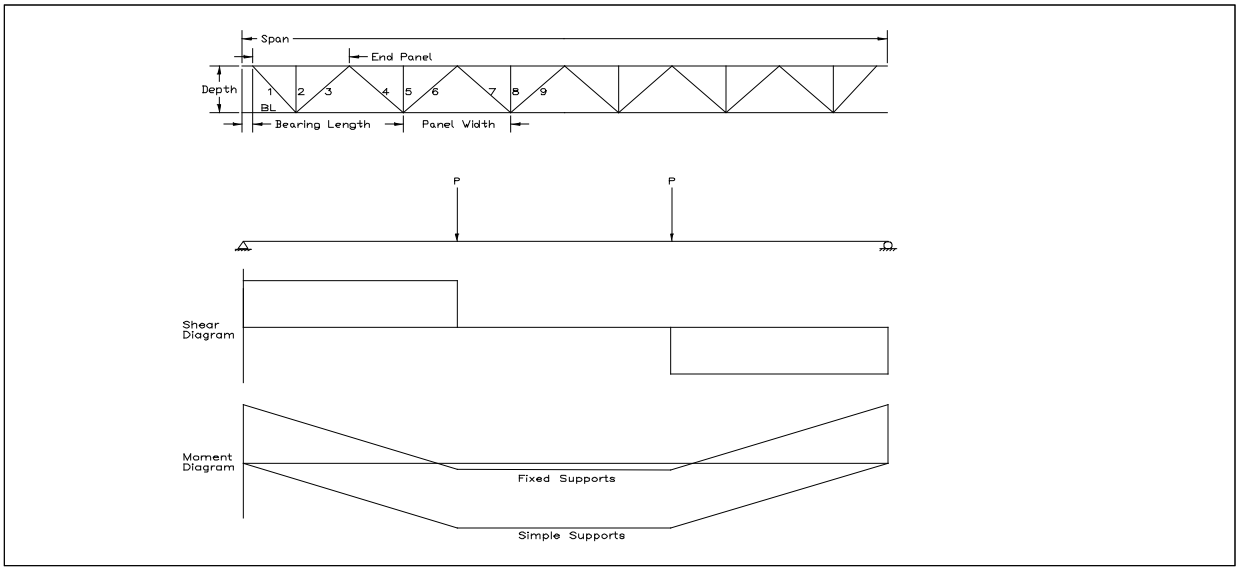
This joist design is for simply supported initial dead load with a buckling link added after dead load is applied to help resist the live load.

| | |
|---|---|
| Type of Joist (Floor or Roof) = | FLOOR |
| Type of End Connection (Simple, PR or Fixed) = | PR |

| | | | |
|---|---|---|---|
| Span (ft) = | 28.0 | Non-Composite Load (plf) = | 300.0 |
| Number of Panel Widths = | 13 | Composite Load (plf) = | 755.0 |
| Panel Width (in) = | 24.0 | Total Factored Load (plf) = | 1538.0 |
| End Panel (in) = | 8.0 | Type of Studs (in) = | 0.500 |
| Bearing Length (in) = | 4.0 | Number of Studs = | 22 |
| Depth (in) = | 28.0 | Reinforcement Type = | 3 #4 Bars |
| $F_y$ of steel (ksi) = | 50.0 | Reinforcement Area ($in^2$) = | 0.60 |
| $F_{yr}$ of reinforcement (ksi) = | 60.0 | Joist Area ($in^2$) = | 3.40 |
| | | **Total Joist Weight (plf) =** | **11.57** |



## C.5 Composite Partially Restrained Steeel Joist Girder Design




Joist Girder Properties

**Load Step 1 :** Initial Dead Load Only — Type of End Connection (Simple or Fixed) = Simple

| | |
|---|---|
| Dead Load (k) = | 6.5 |
| Live Load (k) = | 0.0 |
| Factored Load (k) = | 7.8 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = PL/3 = | 78.0 |
| End Moment (k-ft) = | 0.0 | 0.0 |
| Reaction (k) = | 7.8 | |

**Load Step 2 :** Superimposed Loads — Type of End Connection (Simple or Fixed) = Fixed

| | |
|---|---|
| Dead Load (k) = | 6.0 |
| Live Load (k) = | 19.3 |
| Factored Load (k) = | 38.1 |

Moments based on type of end connection

| | | |
|---|---|---|
| Midspan Moment (k-ft) = | Mu = PL/9 = | 127.0 |
| End Moment (k-ft) = | Mu = 2PL/9 = | 254.0 |
| Reaction (k) = | 38.1 | |

| Joist Girder Properties | |
|---|---|
| Yield Stress (ksi) = | 50.0 |
| Ultimate Strength (ksi) = | 65.0 |
| Elastic Modulus (ksi) = | 29000.0 |
| Span (ft) = | 30.0 |
| Number of Panel Widths = | 4 |
| Panel Width (in) = | 60.0 |
| End Panel (in) = | 56.0 |
| Bearing Length (in) = | 4.0 |
| Depth (in) = | 30.0 |
| Joist Girder (floor or roof) = | Floor |

| Reinforcement Properties | |
|---|---|
| Yield Strength (ksi) = | 60.0 |

| Bay Properties | |
|---|---|
| Joist Girder spacing on left (ft) = | 0.0 |
| Joist Girder spacing on right (ft) = | 28.0 |
| Joist Girder (exterior/interior) = | exterior |

| Slab Properties | |
|---|---|
| Concrete Strength (ksi) = | 3.0 |
| Unit weight of concrete (pcf) = | 145 |
| Elastic Modulus (ksi) = | 3024 |
| Reinforcement Concrete Cover (in) = | 1.0 |
| Slab Thickness Above Deck (in) = | 2.5 |
| Rib Height (in) = | 1.5 |
| Rib Width (in) = | 5.0 |

| Stud Properties | |
|---|---|
| Stud Diameter (in) = | 0.500 |
| Stud Area ($in^2$) = | 0.196 |
| Stud Height (in) = | 3.5 |
| Number of studs/rib = | 1.0 |

| Effective Width | |
|---|---|
| Apprx. flange width (in) = | 0.0 |
| Effective Width (in) = | 45.0 |

**Bottom Chord Design (Tension)**

$P_u$ (k) = $M_u/(depth - 1)$ = 84.8

Using LRFD Specifications Chapter D

$$\phi P_n = \phi A_g F_{cr} \quad \text{(D1-1)}$$

$A_g$ = 1.88

Trial Size = 2.5 x 2.5 x 0.2120 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.775 | L (in) = | 60.0 |
| $r_z$ (in) = | 0.493 | | |
| $y_{bc}$ (in) = | 0.703 | I ($in^4$) = | 1.219 |
| $A_g$ ($in^2$) = | 2.030 | Wt (plf) = | 6.91 |

Slenderness Requirements

l/r = 121.6 ≤ 240 OK

Calculate bottom chord yield force

$T_s = A_g * F_v$ = 101.5

Check that $A_g * F_v < 0.8 * A_e * F_u$ 105.6 OK

Calculate the depth of the compressive stress block

$a = T_s/(0.85 * f`_c * b_{eff})$ = 0.885

Calculate member depth

d = joist depth + slab depth - a/2 - $y_{bc}$ = 32.9

Calculate nominal moment capacity

$\phi M_n = \phi * T_s * d$ = 250.1 > $M_u$ = 205.0 OK

**Top Chord Design (Compression)**

$P_u$ (k) = $M_u$/(depth -1) = 32.3

| | | |
|---|---|---|
| Effective Length | k = | 1.0 |
| Unbraced Length | $L_b$ (in) = | 30.0 |
| Estimated radius of gyration | r (in) = | 0.621 |

Using LRFD Specifications Chapter E

For $\lambda_c$ $Q \leq 1.5$ $F_{cr} = Q(0.658^{Q\lambda c^2})F_y$ (A-B5-15)

For $\lambda_c$ $Q > 1.5$ $F_{cr} = (0.877/\lambda c^2)F_y$ (A-B5-16)

$\lambda_c = (kl/r\pi)\ (F_y/E)$ $\lambda_c$ = 0.639 (E2-4)

$F_{cr}$ = 42.15

$\phi P_n = \phi A_g F_{cr}$ (E2-1)

$t_{s,req'd} = \phi_s/3.0$ = 0.167 $A_g$ = 0.90

Trial Size = 2.5 x 2.5 x 0.2120 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 0.967 | $\lambda_c$ = | 0.512 | |
| $r_x$ (in) = | 0.775 | $F_{cr}$ (ksi) = | 43.5 | |
| $r_z$ (in) = | 0.493 | $P_u$ (k) = | 75.0 | OK |
| $A_g$ ($in^2$) = | 2.030 | I ($in^4$) = | 1.219 | |
| | | Wt (plf) = | 6.91 | |

Slenderness Requirements

l/r = 60.8 ≤ 90 OK

Filler spacing

kl / $r_x$ = 38.7

$s_{req'd}$ = 19.1 Therefore use 1 filler

**Web Member Design**

Member 1 & 4 (Tension):

$P_u$ (k) = 64.9

$A_g$ (Req'd) = 1.44 (D1-1)

Trial Size = 2 x 2 x 0.2050 Eq. Legs

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.615 | L (in) = | 42.4 |
| $r_z$ (in) = | 0.393 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.5883 |
| $A_g$ ($in^2$) = | 1.556 | Wt (plf) = | 5.29 |

Slenderness Requirements

l/r = 108.0 ≤ 240 OK

Member 3 & 6 (Compression):

$P_u$ (k) = 64.9

| | | |
|---|---|---|
| Effective Length | k = | 1.0 |
| Unbraced Length | $L_b$ (in) = | 42.43 |
| Estimated radius of gyration | r (in) = | 0.775 |

$\lambda_c$ = 0.724 (E2-4)

$F_{cr}$ = 40.16 (A-B5-15 or 16)

$A_g$ = 1.90 (E2-1)

Trial Size = 2.5 x 2.5 x 0.2120 Eq. Legs

| | | | | |
|---|---|---|---|---|
| Q = | 0.967 | $\lambda_c$ = | 0.724 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 39.1 | |
| $r_x$ (in) = | 0.775 | $P_u$ (k) = | 67.5 | OK |
| $r_z$ (in) = | 0.493 | I ($in^4$) = | 1.219 | |
| $A_g$ ($in^2$) = | 2.030 | Wt (plf) = | 6.91 | |

Filler spacing

kl / $r_x$ = 54.8

$s_{req'd}$ = 27.0 Therefore use 1 filler

Slenderness Requirements

l/r = 86.0 ≤ 200 OK

**Web Member Design (Continued)**

Member 7 (Tension):

$P_u$ (k) = 11.5

$A_g$ (Req'd) = 0.25 (D1-1)

Trial Size = 1.5 x 1.5 x 0.1550

| | | | |
|---|---|---|---|
| $r_x$ (in) = | 0.461 | L (in) = | 42.4 |
| $r_z$ (in) = | 0.295 | | |
| d (in) = | 0.000 | I ($in^4$) = | 0.0765 |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 |

Slenderness Requirements

l/r = 144.0 ≤ 240 OK

Member 9 (Compression):

$P_u$ (k) = 11.5

| | | | |
|---|---|---|---|
| Effective Length | k = | 1.0 | |
| Unbraced Length | $L_b$ (in) = | 42.43 | |
| Estimated radius of gyration | r (in) = | 0.463 | |
| | $\lambda_c$ = | 1.212 | (E2-4) |
| | $F_{cr}$ = | 27.04 | (A-B5-15 or 16) |
| | $A_g$ = | 0.50 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1550

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 1.216 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 26.9 | |
| $r_x$ (in) = | 0.461 | $P_u$ (k) = | 20.2 | OK |
| $r_z$ (in) = | 0.295 | I ($in^4$) = | 0.187 | |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 | |

Filler spacing

kl / $r_x$ = 92.0

$s_{req'd}$ = 27.1 Therefore use 1 filler

Slenderness Requirements

l/r = 144.0 ≤ 200 OK

**Web Member Design (Continued)**

Member 2,5 & 8 (Zero Force Members):

$P_u$ (k) = 0.6

| | | | |
|---|---|---|---|
| Effective Length | k = | 1.0 | |
| Unbraced Length | $L_b$ (in) = | 30.00 | |
| Estimated radius of gyration | r (in) = | 0.463 | |
| | $\lambda_c$ = | 0.857 | (E2-4) |
| | $F_{cr}$ = | 36.77 | (A-B5-15 or 16) |
| | $A_g$ = | 0.02 | (E2-1) |

Trial Size = 1.5 x 1.5 x 0.1550

| | | | | |
|---|---|---|---|---|
| Q = | 1.000 | $\lambda_c$ = | 0.860 | |
| d (in) = | 0.000 | $F_{cr}$ (ksi) = | 36.7 | |
| $r_x$ (in) = | 0.461 | $P_u$ (k) = | 27.5 | OK |
| $r_z$ (in) = | 0.295 | I ($in^4$) = | 0.187 | |
| $A_g$ ($in^2$) = | 0.882 | Wt (plf) = | 3.00 | |

Filler spacing

$kl / r_x$ = 65.1

$s_{req'd}$ = 19.2 Therefore use 1 filler

Slenderness Requirements

l/r = 101.8 ≤ 200 OK

**Shear Connectors**

$SRF = 0.6*[w_r/h_r(H_s/h_r-1.0)] \le 1.0$ = 1.0

$Q_n = 0.5*A_s*(f`_c*E_c) \le A_s*F_u$ = 9.4

Number of studs required for Positive Moment Region

$N = 1.3*A_s*F_v/Q_n$ = 14.1

Total number of studs = 30

Number of studs required for Negative Moment Region

$N = 1.3*A_r*F_{vr}/Q_n$ = 15.5

Total number of studs = 32

**Bottom Chord Extension Design**

$P_u$ (k) = $M_u$/(depth -1) = 105.1

| | | | |
|---|---|---|---|
| Effective Length | k = | 1.0 | |
| Unbraced Length | $L_b$ (in) = | 30.0 | |
| Estimated radius of gyration | r (in) = | 0.900 | |
| | $\lambda_c$ = | 0.441 | (E2-4) |
| | $F_{cr}$ = | 46.10 | (A-B5-15 or 16) |
| | $A_g$ = | 2.68 | (E2-1) |

Use Plate (Yes/No) Yes

Trial Size = PL 30 x 3.5 x 0.25

Plate Properties

| | | | |
|---|---|---|---|
| $L_b$ (in) = | 30.0 | I ($in^4$) = | 0.005 |
| w (in) = | 3.5 | $A_g$ ($in^2$) = | 0.88 |
| t (in) = | 0.25 | $r_x$ (in) = | 0.072 |

Trial Size = 2.5 x 2.5 x 0.2120 Eq. Legs with plate PL 30 x 3.5 x 0.25

Angle Properties

| | | | |
|---|---|---|---|
| Q = | 0.967 | $r_x$ (in) = | 0.775 |
| y (in) = | 0.703 | $A_g$ ($in^2$) = | 2.030 |
| $r_z$ (in) = | 0.493 | I ($in^4$) = | 1.219 |

Hybrid Properties

| | | | | |
|---|---|---|---|---|
| $y_{cent}$ (in) = | 0.529 | $\lambda_c$ = | 0.514 | |
| $I_h$ ($in^4$) = | 1.731 | $F_{cr}$ (ksi) = | 43.45 | |
| $A_{eh}$ ($in^2$) = | 2.905 | $P_u$ (k) = | 107.3 | OK |
| $r_x$ (in) = | 0.772 | Wt (plf) = | 9.89 | |

Filler spacing

kl / $r_x$ = 38.9

$s_{req'd}$ = 19.2 Therefore use 1 filler

Slenderness Requirements

l/r = 60.8 ≤ 120 OK

**Reinforcement Design**

$A_r$ is the area of negative reinforcement
$F_{yr}$ is the negative reinforcement steel strength

Calculate moment arm

d = joist depth + slab depth - $y_{bc}$ - reinforcement cover = 32.3

reinforcement force = $M_u$/d = $T_{s.req'd}$ = 94.4

$A_{req'd}$ = $T_{s,req'd}$/$\phi F_{yr}$ = 1.75

Therefore use 6 #5 Bars

$A_g$ ($in^2$) = 1.86

**Deflection Checks**

| | | |
|---|---|---|
| Joist Girder centroid (in) = | 5.19 | |
| 85% Non-Composite moment of inertia $I_x$ ($in^4$) = | 728 | |
| 85% Composite moment of inertia $I_x$ ($in^4$) = | 1616 | |
| Dead load deflection (in) = | 0.735 | |
| Camber (in) = | 0.750 | |
| Service load deflection (in) = | 0.303 | |
| Allowable service load deflection (in) = | 1.000 | OK |

**Material Takeoff**

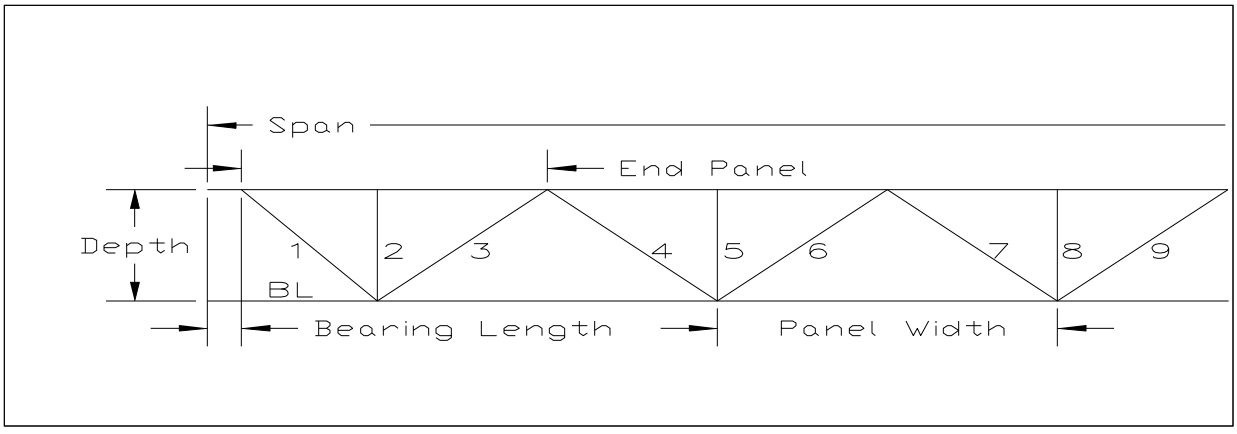
| Member | Section | Length (ft) | Weight (lb's) |
|---|---|---|---|
| Top Chord | 2.5 x 2.5 x 0.2120 Eq. Legs | 30.0 | 207.2 |
| Bottom Chord | 2.5 x 2.5 x 0.2120 Eq. Legs | 25.0 | 172.7 |
| Web Member 1 & 4 | 2 x 2 x 0.2050 Eq. Legs | 13.7 | 72.5 |
| Web Member 3 & 6 | 2.5 x 2.5 x 0.2120 Eq. Legs | 14.1 | 97.7 |
| Web Member 7 | 1.5 x 1.5 x 0.1550 | 7.1 | 21.2 |
| Web Member 9 | 1.5 x 1.5 x 0.1550 | 7.1 | 21.2 |
| Web Member 2,5 & 8 | 1.5 x 1.5 x 0.1550 | 15.0 | 45.0 |
| Buckling Link | 2.5 x 2.5 x 0.2120 Eq. Legs with PL 30 x 3.5 x 0.25 | 5.0 | 49.4 |
| | | Total | 687.0 |
| Reinforcement | 6 #5 Bars | 7.5 | 47.5 |

**Design Summary**

This joist design is for simply supported initial dead load with a buckling link added after dead load is applied to help resist the live load.

| | |
|---|---|
| Type of Joist (Floor or Roof) = | Floor |
| Type of End Connection (Simple, PR or Fixed) = | PR |

| | | | |
|---|---|---|---|
| Span (ft) = | 30.0 | Non-Composite Load (k) = | 6.5 |
| Number of Panel Widths = | 4.0 | Composite Load (k) = | 25.3 |
| Panel Width (in) = | 60.0 | Total Factored Load (k) = | 45.9 |
| End Panel (in) = | 56.0 | Type of Studs = | 0.500 |
| Bearing Length (in) = | 4.0 | Number of Studs = | 62 |
| Depth (in) = | 30.0 | Reinforcement Type = | 6 #5 Bars |
| Fy of steel (ksi) = | 50.0 | Reinforcement Area ($in^2$) = | 1.86 |
| Fyr of reinforcement (ksi) = | 60.0 | Joist Area ($in^2$) = | 6.73 |
| | | **Total Joist Weight (plf) =** | **22.90** |



COLUMN LOADS AND DESIGN SUMMARY (SAMPLE OF DESIGN V)

| | | Axial Load | Strong Axis Moment | Weak Axis Moment | Strong Axis (North/South) | | | Weak Axis (East/West) | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Column | Length | ($P_u$,k) | ($M_{ux}$.k-ft) | ($M_{uy}$.k-ft) | $G_{Bot}$ | $G_{Top}$ | $k_x$ | $G_{Bot}$ | $G_{Top}$ | $k_y$ |
| 1-C1 | 14 | 178.8 | 18.0 | 37.4 | 1.0 | 6.3 | 1.75 | 1.0 | 1.0 | 1.30 |
| 1-C2 | 14 | 317.4 | 21.2 | 0.4 | 1.0 | 3.2 | 1.55 | 1.0 | 0.5 | 1.20 |
| 1-C3 | 14 | 317.0 | 24.1 | 0.4 | 1.0 | 3.2 | 1.55 | 1.0 | 0.5 | 1.20 |
| 2-C1 | 12 | 126.1 | 44.7 | 59.2 | 6.3 | 6.5 | 2.50 | 1.0 | 1.0 | 1.30 |
| 2-C2 | 12 | 224.2 | 55.8 | 4.0 | 3.2 | 2.8 | 1.80 | 0.5 | 0.3 | 1.10 |
| 2-C3 | 12 | 223.5 | 58.9 | 0.4 | 3.2 | 2.8 | 1.80 | 0.5 | 0.3 | 1.10 |
| 3-C4 | 12 | 73.5 | 31.8 | 41.7 | 6.5 | 6.1 | 2.50 | 1.0 | 1.0 | 1.30 |
| 3-C5 | 12 | 131.2 | 37.8 | 0.7 | 2.8 | 2.2 | 1.65 | 0.3 | 0.2 | 1.10 |
| 3-C6 | 12 | 130.1 | 41.3 | 0.3 | 2.8 | 2.1 | 1.60 | 0.3 | 0.2 | 1.10 |
| 4-C4 | 12 | 21.5 | 42.4 | 53.5 | 6.1 | 11.9 | 2.80 | 1.0 | 1.8 | 1.40 |
| 4-C5 | 12 | 38.2 | 49.4 | 6.3 | 2.2 | 5.0 | 1.85 | 0.2 | 0.4 | 1.10 |
| 4-C6 | 12 | 37.2 | 49.4 | 0.3 | 2.1 | 5.0 | 1.85 | 0.2 | 0.4 | 1.10 |
| 1-C7 | 14 | 300.6 | 3.6 | 45.0 | 1.0 | 3.9 | 1.60 | 1.0 | 0.6 | 1.25 |
| 1-C8 | 14 | 598.7 | 2.2 | 11.0 | 1.0 | 4.5 | 1.65 | 1.0 | 0.5 | 1.25 |
| 1-C9 | 14 | 700.2 | 4.3 | 0.6 | 1.0 | 6.4 | 1.75 | 1.0 | 0.5 | 1.25 |
| 2-C7 | 12 | 211.4 | 13.8 | 77.6 | 3.9 | 3.8 | 1.90 | 0.6 | 0.6 | 1.15 |
| 2-C8 | 12 | 420.7 | 13.5 | 18.3 | 4.5 | 3.4 | 1.95 | 0.5 | 0.4 | 1.15 |
| 2-C9 | 12 | 509.1 | 16.5 | 1.1 | 6.4 | 4.7 | 2.30 | 0.5 | 0.3 | 1.10 |
| 3-C10 | 12 | 122.3 | 7.8 | 50.6 | 3.8 | 3.4 | 1.90 | 0.6 | 0.5 | 1.20 |
| 3-C11 | 12 | 242.9 | 4.6 | 10.1 | 3.4 | 2.0 | 1.60 | 0.4 | 0.2 | 1.05 |
| 3-C12 | 12 | 318.2 | 10.9 | 0.8 | 4.7 | 2.0 | 1.85 | 0.3 | 0.2 | 1.10 |
| 4-C10 | 12 | 34.2 | 14.9 | 55.4 | 3.4 | 5.9 | 1.80 | 0.5 | 1.2 | 1.15 |
| 4-C11 | 12 | 64.9 | 15.5 | 6.5 | 2.0 | 4.8 | 1.70 | 0.2 | 0.6 | 1.05 |
| 4-C12 | 12 | 127.8 | 3.2 | 4.4 | 2.0 | 5.7 | 1.90 | 0.2 | 0.6 | 1.15 |
| 5-C12 | 10 | 24.2 | 15.4 | 4.7 | 5.7 | 14.2 | 2.50 | 0.6 | 1.6 | 1.35 |
| 5-C13 | 10 | 6.1 | 0.0 | 4.7 | 1.0 | 2.0 | 1.45 | 1.0 | 0.1 | 1.15 |

Table C.1

| | | | | 1st appx | 1st appx | 1st appx | | 2nd appx | 2nd appx | 2nd appx | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $k_x l$ (ft) | $k_y l$ (ft) | $r_x/r_y$ | max kl (ft) | m | u | Pueq (K) | Trial Sect. | m | u | Pueq (K) | Trial Sect. | Column | $\lambda_c$ | $F_{cr}$ (ksi) |
| 24.5 | 18.2 | 1.71 | 18.2 | 1.40 | 2 | 309 | W10x54 | 1.50 | 1.87 | 311 | W10x54 | 1-C1 | 1.13 | 29.37 |
| 21.7 | 16.8 | 1.71 | 16.8 | 1.50 | 2 | 350 | W10x49 | 1.50 | 1.83 | 350 | W10x49 | 1-C2 | 1.05 | 31.54 |
| 21.7 | 16.8 | 1.71 | 16.8 | 1.50 | 2 | 354 | W10x49 | 1.50 | 1.83 | 354 | W10x49 | 1-C3 | 1.05 | 31.54 |
| 30.0 | 15.6 | 1.71 | 17.6 | 1.40 | 2 | 355 | W10x54 | 1.50 | 1.87 | 359 | W10x54 | 2-C1 | 1.09 | 30.44 |
| 21.6 | 13.2 | 1.71 | 13.2 | 1.75 | 2 | 336 | W10x49 | 1.80 | 1.83 | 338 | W10x49 | 2-C2 | 0.82 | 37.62 |
| 21.6 | 13.2 | 1.71 | 13.2 | 1.75 | 2 | 328 | W10x49 | 1.80 | 1.83 | 331 | W10x49 | 2-C3 | 0.82 | 37.62 |
| 30.0 | 15.6 | 1.71 | 17.5 | 1.45 | 2 | 241 | W10x49 | 1.50 | 1.83 | 236 | W10x49 | 3-C4 | 1.09 | 30.30 |
| 19.8 | 13.2 | 2.16 | 13.2 | 1.75 | 2 | 200 | W10x33 | 1.85 | 1.87 | 203 | W10x33 | 3-C5 | 1.08 | 30.71 |
| 19.2 | 13.2 | 2.16 | 13.2 | 1.75 | 2 | 204 | W10x33 | 1.85 | 1.87 | 208 | W10x33 | 3-C6 | 1.08 | 30.71 |
| 33.6 | 16.8 | 1.71 | 19.6 | 1.30 | 2 | 216 | W10x49 | 1.40 | 1.83 | 218 | W10x49 | 4-C4 | 1.23 | 26.68 |
| 22.2 | 13.2 | 2.16 | 13.2 | 1.75 | 2 | 147 | W10x33 | 1.80 | 1.87 | 148 | W10x33 | 4-C5 | 1.08 | 30.71 |
| 22.2 | 13.2 | 2.16 | 13.2 | 1.75 | 2 | 125 | W10x33 | 1.80 | 1.87 | 127 | W10x33 | 4-C6 | 1.08 | 30.71 |
| 22.4 | 17.5 | 1.71 | 17.5 | 1.45 | 2 | 436 | W10x60 | 1.50 | 1.90 | 434 | W10x60 | 1-C7 | 1.08 | 30.69 |
| 23.1 | 17.5 | 1.75 | 17.5 | 1.45 | 2 | 634 | W12x79 | 1.40 | 2.01 | 633 | W12x79 | 1-C8 | 0.91 | 35.35 |
| 24.5 | 17.5 | 1.75 | 17.5 | 1.45 | 2 | 708 | W12x87 | 1.40 | 2.02 | 708 | W12x87 | 1-C9 | 0.90 | 35.51 |
| 22.8 | 13.8 | 1.71 | 13.8 | 1.70 | 2 | 499 | W10x60 | 1.80 | 1.90 | 502 | W10x60 | 2-C7 | 0.85 | 36.91 |
| 23.4 | 13.8 | 1.75 | 13.8 | 1.70 | 2 | 506 | W12x79 | 1.55 | 2.01 | 499 | W12x79 | 2-C8 | 0.72 | 40.31 |
| 27.6 | 13.2 | 1.75 | 15.7 | 1.60 | 2 | 539 | W12x87 | 1.40 | 2.02 | 535 | W12x87 | 2-C9 | 0.81 | 37.90 |
| 22.8 | 14.4 | 1.71 | 14.4 | 1.75 | 2 | 313 | W10x49 | 1.80 | 1.83 | 303 | W10x49 | 3-C10 | 0.90 | 35.65 |
| 19.2 | 12.6 | 1.71 | 12.6 | 1.75 | 2 | 286 | W10x49 | 1.75 | 1.83 | 283 | W10x49 | 3-C11 | 0.79 | 38.59 |
| 22.2 | 13.2 | 1.71 | 13.2 | 1.75 | 2 | 340 | W10x49 | 1.75 | 1.83 | 340 | W10x49 | 3-C12 | 0.82 | 37.62 |
| 21.6 | 13.8 | 1.71 | 13.8 | 1.70 | 2 | 248 | W10x49 | 1.70 | 1.83 | 232 | W10x49 | 4-C10 | 0.86 | 36.64 |
| 20.4 | 12.6 | 1.71 | 12.6 | 1.75 | 2 | 115 | W10x49 | 1.80 | 1.83 | 114 | W10x49 | 4-C11 | 0.79 | 38.59 |
| 22.8 | 13.8 | 1.71 | 13.8 | 1.70 | 2 | 148 | W10x49 | 1.70 | 1.83 | 147 | W10x49 | 4-C12 | 0.86 | 36.64 |
| 25.0 | 13.5 | 1.71 | 14.6 | 1.70 | 2 | 66 | W10x49 | 1.65 | 1.83 | 64 | W10x49 | 5-C12 | 0.91 | 35.31 |
| 14.5 | 11.5 | 2.71 | 11.5 | 1.80 | 2 | 23 | W6x12 | 2.80 | 2.13 | 34 | W6x12 | 5-C13 | 1.99 | 11.11 |

Table C.1 (continued)

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $\phi P_n$ (K) | $L_b$ (ft) | $C_b$ | $L_r$ (ft) | $L_p$ (ft) | $M_{px}$ (k-in) | $M_{rx}$ (k-in) | $\phi_b M_{nx}$ (k-in) | $M_{py}$ (k-in) | $M_{ry}$ (k-in) | $\phi_b M_{ny}$ (k-in) | Interaction | Note |
| 394.4 | 14.0 | 1.0 | 28.3 | 9.1 | 277.5 | 250.0 | 243.4 | 130.4 | 85.6 | 107.0 | 0.83 | OK |
| 386.1 | 14.0 | 1.0 | 32.6 | 9.0 | 251.7 | 227.5 | 221.9 | 117.9 | 77.8 | 98.5 | 0.91 | OK |
| 386.1 | 14.0 | 1.0 | 32.6 | 9.0 | 251.7 | 227.5 | 221.9 | 117.9 | 77.8 | 98.5 | 0.92 | OK |
| 408.8 | 12.0 | 1.0 | 28.3 | 9.1 | 277.5 | 250.0 | 246.0 | 130.4 | 85.6 | 111.2 | 0.94 | OK |
| 460.5 | 12.0 | 1.0 | 32.6 | 9.0 | 251.7 | 227.5 | 223.7 | 117.9 | 77.8 | 101.5 | 0.74 | OK |
| 460.5 | 12.0 | 1.0 | 32.6 | 9.0 | 251.7 | 227.5 | 223.7 | 117.9 | 77.8 | 101.5 | 0.72 | OK |
| 370.9 | 12.0 | 1.0 | 28.3 | 9.0 | 251.7 | 227.5 | 223.1 | 117.9 | 77.8 | 100.5 | 0.66 | OK |
| 253.5 | 12.0 | 1.0 | 21.8 | 6.9 | 161.7 | 145.8 | 140.6 | 58.3 | 38.3 | 46.3 | 0.77 | OK |
| 253.5 | 12.0 | 1.0 | 21.8 | 6.9 | 161.7 | 145.8 | 140.6 | 58.3 | 38.3 | 46.3 | 0.78 | OK |
| 326.5 | 12.0 | 1.0 | 28.3 | 9.0 | 251.7 | 227.5 | 223.1 | 117.9 | 77.8 | 100.5 | 0.76 | OK |
| 253.5 | 12.0 | 1.0 | 21.8 | 6.9 | 161.7 | 145.8 | 140.6 | 58.3 | 38.3 | 46.3 | 0.56 | OK |
| 253.5 | 12.0 | 1.0 | 21.8 | 6.9 | 161.7 | 145.8 | 140.6 | 58.3 | 38.3 | 46.3 | 0.43 | OK |
| 459.1 | 14.0 | 1.0 | 30.2 | 9.1 | 310.8 | 277.9 | 272.9 | 145.8 | 95.9 | 120.8 | 1.00 | OK |
| 697.2 | 14.0 | 1.0 | 35.7 | 10.8 | 495.8 | 445.8 | 440.4 | 226.3 | 149.0 | 194.7 | 0.91 | OK |
| 772.8 | 14.0 | 1.0 | 38.4 | 10.9 | 550.0 | 491.7 | 489.0 | 251.7 | 165.6 | 217.7 | 0.92 | OK |
| 552.2 | 12.0 | 1.0 | 30.2 | 9.1 | 310.8 | 277.9 | 275.7 | 145.8 | 95.9 | 125.0 | 0.98 | OK |
| 794.8 | 12.0 | 1.0 | 35.7 | 10.8 | 495.8 | 445.8 | 444.1 | 226.3 | 149.0 | 200.2 | 0.64 | OK |
| 824.7 | 12.0 | 1.0 | 38.4 | 10.9 | 550.0 | 491.7 | 492.8 | 251.7 | 165.6 | 223.3 | 0.65 | OK |
| 436.3 | 12.0 | 1.0 | 19.7 | 9.0 | 251.7 | 227.5 | 220.4 | 117.9 | 77.8 | 96.0 | 0.78 | OK |
| 472.3 | 12.0 | 1.0 | 21.8 | 9.0 | 251.7 | 227.5 | 221.4 | 117.9 | 77.8 | 97.6 | 0.62 | OK |
| 460.5 | 12.0 | 1.0 | 28.3 | 9.0 | 251.7 | 227.5 | 223.1 | 117.9 | 77.8 | 100.5 | 0.74 | OK |
| 448.5 | 12.0 | 1.0 | 19.7 | 9.0 | 251.7 | 227.5 | 220.4 | 117.9 | 77.8 | 96.0 | 0.68 | OK |
| 472.3 | 12.0 | 1.0 | 21.8 | 9.0 | 251.7 | 227.5 | 221.4 | 117.9 | 77.8 | 97.6 | 0.20 | OK |
| 448.5 | 12.0 | 1.0 | 28.3 | 9.0 | 251.7 | 227.5 | 223.1 | 117.9 | 77.8 | 100.5 | 0.34 | OK |
| 432.2 | 10.0 | 1.0 | 28.3 | 9.0 | 251.7 | 227.5 | 225.4 | 117.9 | 77.8 | 104.2 | 0.14 | OK |
| 33.5 | 10.0 | 1.0 | 10.2 | 3.2 | 34.6 | 30.5 | 27.5 | 9.7 | 6.2 | 5.7 | 0.91 | OK |

Table C.1 (continued)

# Appendix D

# LITERATURE REVIEW EQUATIONS

D.1 LRFD Analysis for Semi-Rigid Frame Design

(King, et al 1993)

ΣH = Sum of all story lateral loads
ΣH' = Notional lateral loads
$\Delta_o$ = First-order translational deflection of the story
Δ = Second-order translational deflection of the story
$P_u$ = Axial Design Force
L = Story height
$B_2$ = P - Δ moment amplification factor
I = Moment of Inertia
I' = Adjusted moment of inertia for semi-rigid connections
$K_i$ = Initial semi-rigid connection stiffness

$$S_F = \frac{\sum H}{\Delta_o} = \frac{\sum H + \sum P_u / L}{\Delta} = \frac{\sum H'}{\Delta} \tag{19}$$

$$\Delta = (\sum H + \sum P_u / L)\ \frac{\Delta_o}{\sum H} = (1 + \frac{\sum P_u \Delta}{\sum HL})\Delta_o \tag{20}$$

$$\Delta(1 - \frac{\sum P_u \Delta_o}{\sum HL}) = \Delta_o \tag{21}$$

$$\Delta = \frac{\Delta_o}{(1 - \frac{\sum P_u \Delta_o}{\sum HL})} = B_2 \Delta_o \tag{22}$$

$$\sum H' = \sum H + \sum P_u \Delta / L \tag{23}$$

$$I' = \frac{I}{1 + \frac{2EI}{K_i L}} \tag{24}$$

D.2 Design Analysis of Semi-Rigid: Evaluation and Implementation

(Barakat, et al 1991)

The basic LRFD equations for frame analysis can be found in the LRFD Manual of Steel Construction Specifications section C. Frames and Other Structures and will not be rewritten here. The following equations are used to modify these equations for semi-rigid action.

$R_{ko}$ = Modified initial stiffness for each connection
$R_{kb}$ = Secant connection stiffness determined by the beam line method
$I`_o$ = Modified moment of inertia of beam which accounts for connection stiffness $R_{ko}$ at its end
$I`_b$ = Modified moment of inertia of beam which accounts for connection stiffness $R_{kb}$ at its end

$\bar{\alpha}_{nt} = \frac{\sum I_b}{\sum \left(I'_{nt}\right)_b}$, scaling factor for nonsway relative stiffness factor G

$\bar{\alpha}_{lt} = \frac{\sum I_b}{\sum \left(I'_{lt}\right)_b}$, scaling factor for sway relative stiffness factor G

$G`_a = \bar{\alpha} G_a$, modified relative stiffness factor at column end A

$G`_b = \bar{\alpha} G_b$, modified relative stiffness factor at column end B

# Appendix E

# ANSYS PARTIALLY RESTRAINED FRAME MODEL INPUT

## E.1 Sample ANSYS 5.3 input for frame line 1-6

```
!***This run was for load combination:
!***1.2D+1.6L

/PREP7                    *Start the preprocessor,,,,,,

!*** ELEMENT TYPES ***
ET,1,BEAM3,,,,,,1
KEYOPT,1,9,3,,
ET,2,COMBIN39,0,0,6,0,,0,

!*** REAL CONSTANTS ***
!*** COLUMNS ***
R,1,19.1,533,1,1.2        *W12x65-C1
R,2,17.6,341,1,1.2        *W10x60-C2
R,3,17.6,341,1,1.2        *W10x60-C3
R,4,14.4,272,1,1.2        *W10x49-C4
R,5,13.3,248,1,1.2        *W10x45-C5
R,6,11.5,209,1,1.2        *W10x39-C6
R,7,17.6,341,1,1.2        *W10x60-C7
R,8,23.2,662,1,1.2        *W12x79-C8
R,9,26.5,999,1,1.2        *W14x90-C9
R,10,13.3,248,1,1.2       *W10x45-C10
R,11,13.3,248,1,1.2       *W10x45-C11
R,12,14.4,272,1,1.2       *W10x49-C12
R,13,3.55,22.1,1,1.2      *W6x12- C13

!*** GIRDERS ***
R,21,4.16,88.6,1,1.2      *W12x14-G1
R,22,4.16,88.6,1,1.2      *W12x14-G2
R,23,6.48,156,1,1.2       *W12x22-G3
R,24,9.12,375,1,1.2       *W16x31-G4
R,25,9.12,375,1,1.2       *W16x31-G5
R,26,10.3,510,1,1.2       *W18x35-G6
R,27,10.3,510,1,1.2       *W18x35-G7
R,28,14.7,984,1,1.2       *W21x50-G8
R,29,16.2,1350,1,1.2      *W24x55-G9
```

```
!*** BEAMS ***
R,31,3.54,53.8,1,1.2          *W10x12-B1A
R,33,4.16,88.6,1,1.2          *W12x14-B3
R,34,4.71,103,1,1.2           *W12x16-B4A
R,36,6.48,156,1,1.2           *W12x22-B6A
R,37,3.54,53.8,1,1.2          *W10x12-B7
R,38,3.54,53.8,1,1.2          *W10x12-B8A
R,41,6.48,156,1,1.2           *W12x22-B11
R,44,7.65,204,1,1.2           *W12x26-B14
R,46,9.12,375,1,1.2           *W16x31-B16
R,47,9.12,375,1,1.2           *W16x31-B17A
R,49,5.57,130,1,1.2           *W12x19-B18
R,51,7.65,204,1,1.2           *W12x26-B20
R,53,6.48,156,1,1.2           *W12x22-B22A

!*** REAL CONSTANTS ***
!*** NON-LINEAR SPRING CONSTANTS ***
R,61,0,0,0.00001,60,0.00002,120                *W12x26-B14
RMORE,0.00004,240,0.00006,360,0.00009,480
RMORE,0.00012,600,0.00016,720,0.0002,840
RMORE,0.00026,960,0.00033,1080,0.00041,1200
RMORE,0.00052,1320,0.00066,1440,0.00085,1560
RMORE,0.00111,1680,0.00148,1800,0.00206,1920
RMORE,0.00307,2040,0.00511,2160,0.01094,2280
R,62,0,0,0.00001,60,0.00003,120                *W12x19-B18
RMORE,0.00004,180,0.00006,240,0.00008,300
RMORE,0.0001,360,0.00012,420,0.00015,480
RMORE,0.00017,540,0.00021,600,0.00024,660
RMORE,0.00029,720,0.00039,840,0.00052,960
RMORE,0.00071,1080,0.00098,1200,0.00139,1320
RMORE,0.00208,1440,0.0034,1560,0.00669,1680
R,63,0,0,0.00001,60,0.00003,120                *W12x14-B3
RMORE,0.00004,180,0.00006,240,0.00008,300
RMORE,0.0001,360,0.00013,420,0.00015,480
RMORE,0.00018,540,0.00022,600,0.00026,660
RMORE,0.0003,720,0.00041,840,0.00056,960
RMORE,0.00077,1080,0.00107,1200,0.00154,1320
RMORE,0.00236,1440,0.00403,1560,0.00882,1680
R,64,0,0,0.00001,30,0.00002,60                 *W10x12-B7
RMORE,0.00003,90,0.00004,120,0.00006,180
```

```
RMORE,0.00009,240,0.00012,300,0.00016,360
RMORE,0.0002,420,0.00025,480,0.00031,540
RMORE,0.00038,600,0.00046,660,0.00056,720
RMORE,0.00083,840,0.00127,960,0.00207,1080
RMORE,0.00383,1200,0.00989,1320,0.02351,1380

!*** MATERIAL BEHAVIOR ***
MP,EX,1,29000,,,

!*** NODES ***
!*** GROUND FLOOR ***
N,1,0,0,0
N,2,432,0,0
N,3,768,0,0
N,4,1200,0,0

!*** 1st FLOOR ***
N,101,0,168,0
N,102,432,168,0
N,103,768,168,0
N,104,1200,168,0
!*** SPRING NODES ***
N,151,0,168,0
N,152,432,168,0
N,153,432,168,0
N,154,768,168,0
N,155,768,168,0
N,156,1200,168,0

!*** 2nd FLOOR ***
N,201,0,312,0
N,202,432,312,0
N,203,768,312,0
N,204,1200,312,0
!*** SPRING NODES ***
N,251,0,312,0
N,252,432,312,0
N,253,432,312,0
N,254,768,312,0
N,255,768,312,0
```

```
N,256,1200,312,0

!*** 3rd FLOOR ***
N,301,0,456,0
N,302,432,456,0
N,303,768,456,0
N,304,1200,456,0
!*** SPRING NODES ***
N,351,0,456,0
N,352,432,456,0
N,353,432,456,0
N,354,768,456,0
N,355,768,456,0
N,356,1200,456,0

!*** ROOF FLOOR ***
N,401,0,600,0
N,402,432,600,0
N,403,768,600,0
N,404,1200,600,0
!*** SPRING NODES ***
N,451,0,600,0
N,452,432,600,0
N,453,432,600,0
N,454,768,600,0
N,455,768,600,0
N,456,1200,600,0

!*** ELEMENTS ***
!*** GROUND FLOOR COLUMNS ***
MAT,1,,,
TYPE,1,,,
REAL,1,,,
EN,1,1,101
EN,4,4,104
REAL,7,,,
EN,2,2,102
EN,3,3,103

!*** 1st FLOOR COLUMNS ***
```

```
MAT,1,,,
TYPE,1,,,
REAL,1,,,
EN,21,101,201
EN,24,104,204
REAL,7,,,
EN,22,102,202
EN,23,103,203

!*** 2nd FLOOR COLUMNS ***
MAT,1,,,
TYPE,1,,,
REAL,4,,,
EN,31,201,301
EN,34,204,304
REAL,10,,,
EN,32,202,302
EN,33,203,303

!*** 3rd FLOOR COLUMNS ***
MAT,1,,,
TYPE,1,,,
REAL,4,,,
EN,41,301,401
EN,44,304,404
REAL,10,,,
EN,42,302,402
EN,43,303,403

!*** 1st FLOOR BEAMS ***
MAT,1,,,
TYPE,1,,,
REAL,44,,,
EN,101,151,152
EN,103,155,156
REAL,49,,,
EN,102,153,154
!*** NON-LINEAR SPRINGS ***
MAT,1,,,
TYPE,2,,,
```

```
REAL,61,,,
EN,151,101,151
EN,152,152,102
EN,155,103,155
EN,156,156,104
REAL,62,,,
EN,153,102,153
EN,154,154,103

!*** 2nd FLOOR BEAMS ***
MAT,1,,,
TYPE,1,,,
REAL,44,,,
EN,201,251,252
EN,203,255,256
REAL,49,,,
EN,202,253,254
!*** NON-LINEAR SPRINGS ***
MAT,1,,,
TYPE,2,,,
REAL,61,,,
EN,251,201,251
EN,252,252,202
EN,255,203,255
EN,256,256,204
REAL,62,,,
EN,253,202,253
EN,254,254,203

!*** 3rd FLOOR BEAMS ***
MAT,1,,,
TYPE,1,,,
REAL,44,,,
EN,301,351,352
EN,303,355,356
REAL,49,,,
EN,302,353,354
!*** NON-LINEAR SPRINGS ***
MAT,1,,,
TYPE,2,,,
```

```
REAL,61,,,
EN,351,301,351
EN,352,352,302
EN,355,303,355
EN,356,356,304
REAL,62,,,
EN,353,302,353
EN,354,354,303

!*** ROOF BEAMS ***
MAT,1,,,
TYPE,1,,,
REAL,33,,,
EN,401,451,452
EN,403,455,456
REAL,37,,,
EN,402,453,454
!*** NON-LINEAR SPRINGS ***
MAT,1,,,
TYPE,2,,,
REAL,63,,,
EN,451,401,451
EN,452,452,402
EN,455,403,455
EN,456,456,404
REAL,64,,,
EN,453,402,453
EN,454,454,403

!*** CONSTRAINTS ***
CP,101,UX,101,151
CP,102,UX,102,152,153
CP,103,UX,103,154,155
CP,104,UX,104,156
CP,201,UX,201,251
CP,202,UX,202,252,253
CP,203,UX,203,254,255
CP,204,UX,204,256
CP,301,UX,301,351
CP,302,UX,302,352,353
```

```
CP,303,UX,303,354,355
CP,304,UX,304,356
CP,401,UX,401,451
CP,402,UX,402,452,453
CP,403,UX,403,454,455
CP,404,UX,404,456
CP,151,UY,101,151
CP,152,UY,102,152,153
CP,153,UY,103,154,155
CP,154,UY,104,156
CP,251,UY,201,251
CP,252,UY,202,252,253
CP,253,UY,203,254,255
CP,254,UY,204,256
CP,351,UY,301,351
CP,352,UY,302,352,353
CP,353,UY,303,354,355
CP,354,UY,304,356
CP,451,UY,401,451
CP,452,UY,402,452,453
CP,453,UY,403,454,455
CP,454,UY,404,456

!*** Boundary Conditions ***,,,,,,
D,1,UX,0,,4,1
D,1,UY,0,,4,1
D,1,ROTZ,0,,4,1

/SOLU          *Start Solution Processor,,,,,,

!*** DEFINE ANALSYS ***,,,,,,
ANTYPE,STATIC,NEW,,,,
AUTOTS,ON          *Turn Automatic Time Stepping On,,,,,
NLGEOM,            *Include Large Deformation Effects,,,,,
NROPT,FULL,,On     *Use Full Newton Raphson Method With Adaptive Descent,,,
CNVTOL,F,,0.10,,,
CNVTOL,M,,0.10,,,
NEQIT,200          *Allow Up to 200 Equilibrium Iterations,,,,,
OUTRES,ALL,LAST*Write all solution items to DB at the last substep,,,,
```

```
SAVE

!*** DISTRIBUTED LOADS ***
!*** MEMBER LOADS + SELF-WEIGHTS + DEAD & LIVE LOAD ***
!*** 1st FLOOR BEAMS ***
SFBEAM,101,1,PRES,0.099
SFBEAM,102,1,PRES,0.098
SFBEAM,103,1,PRES,0.099

!*** 2nd FLOOR BEAMS ***
SFBEAM,201,1,PRES,0.099
SFBEAM,202,1,PRES,0.098
SFBEAM,203,1,PRES,0.099

!*** 3rd FLOOR BEAMS ***
SFBEAM,301,1,PRES,0.099
SFBEAM,302,1,PRES,0.098
SFBEAM,303,1,PRES,0.099

!*** ROOF BEAMS ***
SFBEAM,401,1,PRES,0.046
SFBEAM,402,1,PRES,0.046
SFBEAM,403,1,PRES,0.046

SAVE
SOLVE

/POST1                          *Enters the General Postprocessor

/OUTPUT,Aline-16,out,,,,
SET,LAST
NSEL,S,NODE,,1,4,1
PRRSOL,
NSEL,S,NODE,,104,404,100
PRNSOL,UX
ESEL,S,TYPE,,1
PRESOL,FORC
ETABLE,INT,SMISC,18
PRETAB,INT
```

## Partially Restrained Frame Nodes

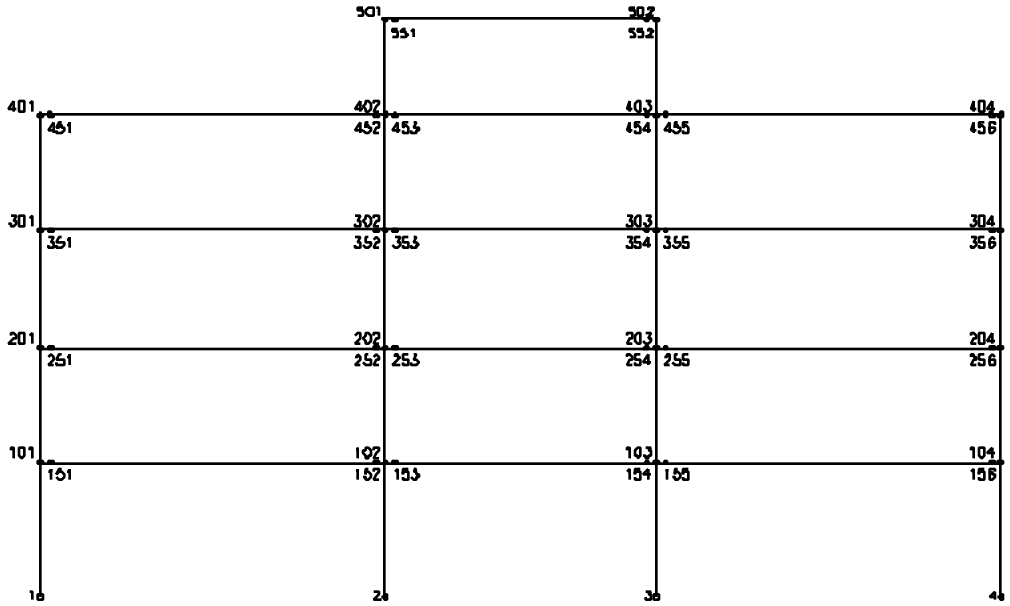



Figure E.1

Notes: Labels stand for node numbers

Scale 1" = 20'

Partially Restrained Frame Elements

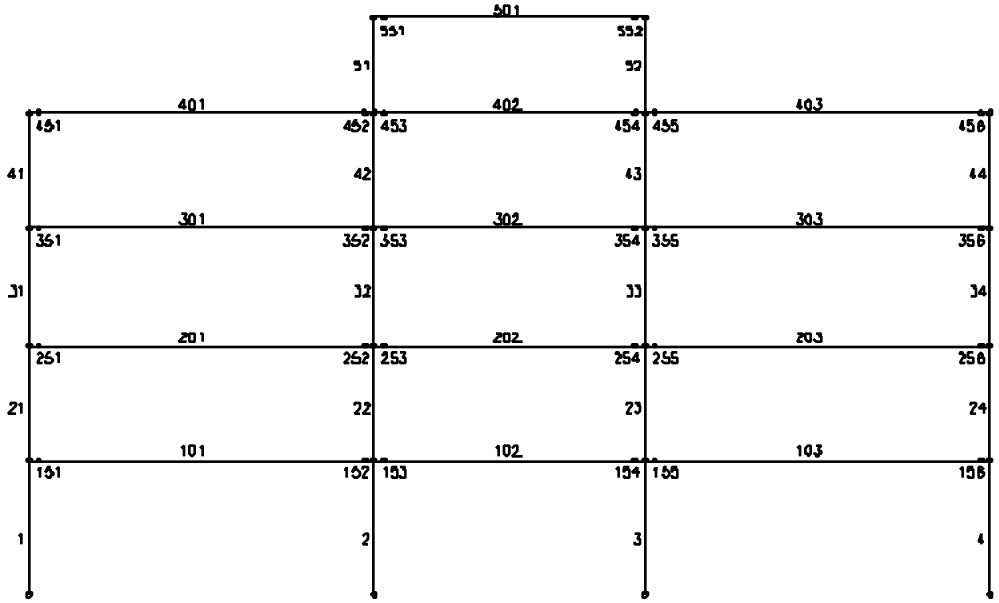



Figure E.2

Notes: Labels stand for element numbers

Scale 1" = 20'

# Appendix F

# MATERIAL TAKEOFFS AND COST ESTIMATES

## F.1 Explanation of Cost Estimating Process

To estimate the cost of the eight building designs the Means Building Construction Cost Data book was used (Means 1994). The book contains large amounts of information on material costs, labor costs, equipment costs, the type of crews required for construction, etc. The book is broken down into sixteen sections such as concrete, metals, site work, electrical, mechanical, etc. Each section than has subsections. The subsections than contain the cost data for items used in the construction process. To understand how to use this manual a brief example will be shown. To use this reference without error it is suggested that the introduction be read thoroughly.

For this example I will show how the cost of the W8x10 beam was chosen in Table F.1. The first step is to locate the section in the Means book. Structural steel sections are located in Section 5 Metals. The subsection is 051 250. These are structural steel members, common WF sizes, spans 10' to 45', including bolted connections and labor. A W8x10 section is listed with a unit price of $11.45 per linear foot. This price includes material, labor, equipment, overhead and a 10% profit. Under the crew listing is E-2 which has a structural steel foremen, a structural steel worker, a welder, one gas welding machine, and one torch. This information tells you what type of labor and equipment is needed to erect the W8x10 section.

The rest of the information found in these cost estimates were derived at in the same manner.

Design I-Penthouse Roof

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | W12x14 | B1 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | Temp. & Shrink. @12" | B1A | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | W8x10 | B2 | LF | 28.0 | $ 4.95 | $ 2.41 | $ 1.62 | $ 11.45 | $ 641.20 |
| 2 | W12x14 | G1 | LF | 30.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 702.00 |
| 4 | W12x14 | G2 | LF | 10.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 468.00 |
| 4 | W10x54 | C12 | LF | 10.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,220.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 4 | L 1.25 x1.25 x 3/16 | BR1 | lb | 43.9 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 237.06 |
| 16 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 800.00 |
| 8 | Flange Plate Connection | Moment | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 4,000.00 |
| 8 | Bracing Connection | Bracing | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 800.00 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ -

Design I-Roof

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 3C18 | Mech. Floor | SF | 1400.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 1,876.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | W12x14 | B3 | LF | 36.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,684.80 |
| 8 | W12x22 | B4 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x22 | B4A | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,318.40 |
| 8 | W12x22 | B5 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x22 | B6 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,318.40 |
| 4 | W12x22 | B6A | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,318.40 |
| 2 | W10x12 | B7 | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 4 | W12x14 | B8 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,310.40 |
| 2 | W12x14 | B8A | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | W12x16 | B9 | LF | 28.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 728.00 |
| 1 | W12x22 | B10 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 450.80 |
| 1 | W12x22 | B10A | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 450.80 |
| 2 | W12x26 | B11 | LF | 28.0 | $ 12.85 | $ 1.64 | $ 1.10 | $ 18.25 | $ 1,022.00 |
| 2 | W18x35 | B12 | LF | 28.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 1,344.00 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 4 | W12x26 | G3 | LF | 30.0 | $ 12.85 | $ 1.64 | $ 1.10 | $ 18.25 | $ 2,190.00 |
| 6 | W12x26 | G3F | LF | 30.0 | $ 12.85 | $ 1.64 | $ 1.10 | $ 18.25 | $ 3,285.00 |
| 4 | W18x35 | G4 | LF | 30.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 2,880.00 |
| 4 | W21x44 | G5 | LF | 30.0 | $ 21.00 | $ 1.97 | $ 1.05 | $ 27.50 | $ 3,300.00 |
| 2 | W21x50 | G6 | LF | 30.0 | $ 23.50 | $ 1.97 | $ 1.05 | $ 30.50 | $ 1,830.00 |
| 4 | W14x48 | C4 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | W14x48 | C5 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | W14x48 | C6 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | W10x33 | C10 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x33 | C11 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x54 | C12 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | L 1.25 x1.25 x 3/16 | BR1 | lb | 44.6 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 240.99 |
| 124 | Shear Connections | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 6,200.00 |
| 12 | Flange Plate Connection | Moment | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 6,000.00 |
| 8 | Bracing Connection | Bracing | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 800.00 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | Page Subtotal | $ 83,862.59 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.1 Material List and Cost Estimate (continued)

Design I-Floors

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 3C18 | Floor | SF | 15000.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 60,300.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | Temp. & Shrink. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | W16x31 | B14 | LF | 36.0 | $ 15.35 | $ 1.61 | $ 1.08 | $ 21.00 | $ 9,072.00 |
| 24 | W18x40 | B15 | LF | 36.0 | $ 19.80 | $ 2.19 | $ 1.16 | $ 27.00 | $ 23,328.00 |
| 12 | W16x40 | B16 | LF | 36.0 | $ 19.80 | $ 1.81 | $ 1.21 | $ 26.50 | $ 11,448.00 |
| 24 | W18x46 | B17 | LF | 36.0 | $ 23.00 | $ 2.19 | $ 1.16 | $ 30.00 | $ 25,920.00 |
| 12 | W18x46 | B17A | LF | 36.0 | $ 23.00 | $ 2.19 | $ 1.16 | $ 30.00 | $ 12,960.00 |
| 6 | W14x22 | B18 | LF | 28.0 | $ 12.85 | $ 1.46 | $ 0.98 | $ 17.80 | $ 2,990.40 |
| 12 | W16x26 | B19 | LF | 28.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 5,964.00 |
| 6 | W16x31 | B20 | LF | 28.0 | $ 15.35 | $ 1.61 | $ 1.08 | $ 21.00 | $ 3,528.00 |
| 12 | W18x35 | B21 | LF | 28.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 8,064.00 |
| 6 | W14x26 | B22 | LF | 28.0 | $ 12.85 | $ 1.46 | $ 0.98 | $ 17.80 | $ 2,990.40 |
| 6 | W14x26 | B22A | LF | 28.0 | $ 12.85 | $ 1.46 | $ 0.98 | $ 17.80 | $ 2,990.40 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 18 | W21x44 | G7 | LF | 30.0 | $ 21.00 | $ 1.97 | $ 1.05 | $ 27.50 | $ 14,850.00 |
| 12 | W21x44 | G7F | LF | 30.0 | $ 21.00 | $ 1.97 | $ 1.05 | $ 27.50 | $ 9,900.00 |
| 12 | W24x55 | G8 | LF | 30.0 | $ 26.00 | $ 1.89 | $ 1.00 | $ 33.00 | $ 11,880.00 |
| 18 | W24x68 | G9 | LF | 30.0 | $ 32.00 | $ 1.89 | $ 1.00 | $ 40.00 | $ 21,600.00 |
| 4 | W14x48 | C4 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | W14x48 | C5 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | W14x48 | C6 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | W10x33 | C10 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x33 | C11 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x54 | C12 | LF | 12.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 1,464.00 |
| 4 | W14x109 | C1 | LF | 26.0 | $ 55.50 | $ 1.51 | $ 1.01 | $ 65.00 | $ 6,760.00 |
| 4 | W14x109 | C2 | LF | 26.0 | $ 55.50 | $ 1.51 | $ 1.01 | $ 65.00 | $ 6,760.00 |
| 4 | W14x109 | C3 | LF | 26.0 | $ 55.50 | $ 1.51 | $ 1.01 | $ 65.00 | $ 6,760.00 |
| 4 | W10x33 | C7 | LF | 26.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 2,080.00 |
| 4 | W10x54 | C8 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x54 | C9 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 8 | L 1.25 x1.25 x 3/16 | BR1 | lb | 45.0 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 486.46 |
| 4 | L 1.25 x1.25 x 3/16 | BR2 | lb | 46.2 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 249.61 |
| 372 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 18,600.00 |
| 36 | Flange Plate Connection | Moment | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 18,000.00 |
| 24 | Bracing Connection | Bracing | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 2,400.00 |
| 24 | Column Splices | Splice | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,606.33 |
| 1 | High Strength Steel | | ton | 187.4 | $ 50.00 | $ - | $ - | $ 55.00 | $ 10,304.46 |
| | | | | | | | | | $ - |
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| | | | | | | | | | $ - |
| | | | | | | | | Page Subtotal | $ 442,616.06 |
| | | | | | | | | Design Tot | $ 539,301.31 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.1 Material List and Cost Estimate (continued)

Design II-Penthouse Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | 20K7 | B1 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 336.00 |
| 2 | 20K7 | B1A | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 336.00 |
| 2 | 20K3 | B2 | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | Joist Girder | G1 | ton | 0.24 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 532.40 |
| 4 | Joist Girder | G2 | ton | 0.07 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 312.40 |
| 4 | Bottom Chord Extension | G2 | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 70.40 |
| 4 | W8x35 | C12 | LF | 10.0 | $ 17.00 | $ 1.34 | $ 0.90 | $ 22.00 | $ 880.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 4 | L 1.25 x1.25 x 3/16 | BR1 | lb | 43.9 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 237.06 |
| 4 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 200.00 |
| 12 | Joist to Column | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 600.00 |
| 8 | Joist to Column | Moment | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 1,000.00 |
| 8 | Bracing Connection | Bracing | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 800.00 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ -

Design II-Roof

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 3C18 | Mech. Floor | SF | 1400.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 1,876.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | 26K9 | B3 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 1,036.80 |
| 8 | 26KCS5 | B4 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | 26KCS5 | B4A | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 1,036.80 |
| 8 | 26KCS5 | B5 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | 26KCS5 | B6 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 1,036.80 |
| 4 | 26KCS5 | B6A | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 1,036.80 |
| 2 | 20K6 | B7 | LF | 28.0 | $ 2.80 | $ 1.05 | $ 0.56 | $ 5.50 | $ 308.00 |
| 4 | 20K7 | B8 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 672.00 |
| 2 | 20K7 | B8A | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 336.00 |
| 2 | 20K9 | B9 | LF | 28.0 | $ 3.51 | $ 1.05 | $ 0.56 | $ 6.35 | $ 355.60 |
| 1 | 20LH07SP | B10 | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 1 | 20LH07SP | B10A | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 2 | 28VLH 1100/500 | B11 | LF | 28.0 | $ 5.20 | $ 1.17 | $ 0.62 | $ 8.45 | $ 473.20 |
| 2 | 28VLH 1524/1000 | B12 | LF | 28.0 | $ 7.50 | $ 1.17 | $ 0.62 | $ 11.00 | $ 616.00 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 4 | Joist Girder | G3 | ton | 0.21 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 937.20 |
| 6 | Joist Girder | G3F | ton | 0.21 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,405.80 |
| 4 | Joist Girder | G4 | ton | 0.28 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,249.60 |
| 4 | Joist Girder | G5 | ton | 0.34 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,504.80 |
| 2 | Joist Girder | G6 | ton | 0.48 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,062.60 |
| 4 | Bottom Chord Extension | G3F | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 70.40 |
| 4 | W10x45 | C4 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x49 | C5 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C6 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W8x24 | C10 | LF | 12.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 787.20 |
| 4 | W8x31 | C11 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x35 | C12 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | L 1.25 x1.25 x 3/16 | BR1 | lb | 44.6 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 240.99 |
| 60 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 3,000.00 |
| 64 | Joist to Column | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 3,200.00 |
| 12 | Joist to Column | Moment | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 1,500.00 |
| 8 | Bracing Connection | Bracing | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 800.00 |
| | | | | | | | | | $ - |
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| | | | | | | | | Page Subtotal | $ 58,357.19 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.2 Material List and Cost Estimate (continued)

Design II-Floors

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 3 | 3C18 | Floor | SF | 15000.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 60,300.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | Temp. & Shrink. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | 28VLH 852/350 | B14 | LF | 36.0 | $ 7.50 | $ 1.17 | $ 0.62 | $ 11.00 | $ 4,752.00 |
| 24 | 28VLH 1091/567 | B15 | LF | 36.0 | $ 8.10 | $ 1.17 | $ 0.62 | $ 11.70 | $ 10,108.80 |
| 12 | 28VLH 1200/682 | B16 | LF | 36.0 | $ 8.50 | $ 1.17 | $ 0.62 | $ 12.00 | $ 5,184.00 |
| 24 | 28VLH 1524/1000 | B17 | LF | 36.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 11,232.00 |
| 12 | 28VLH 1524/1000 | B17A | LF | 36.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 5,616.00 |
| 6 | 28VLH 852/350 | B18 | LF | 28.0 | $ 7.50 | $ 1.17 | $ 0.62 | $ 11.00 | $ 1,848.00 |
| 12 | 28VLH 1143/619 | B19 | LF | 28.0 | $ 6.00 | $ 1.17 | $ 0.62 | $ 9.00 | $ 3,024.00 |
| 6 | 28VLH 1374/1024 | B20 | LF | 28.0 | $ 7.50 | $ 1.17 | $ 0.62 | $ 11.00 | $ 1,848.00 |
| 12 | 28VLH 1524/1000 | B21 | LF | 28.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 4,368.00 |
| 6 | 28VLH 1524/1000 | B22 | LF | 28.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 2,184.00 |
| 6 | 28VLH 1524/1000 | B22A | LF | 28.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 2,184.00 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 12 | Joist Girders | G7 | ton | 0.41 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 5,412.00 |
| 18 | Joist Girders | G7F | ton | 0.25 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 4,969.80 |
| 12 | Joist Girders | G8 | ton | 0.64 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 8,461.20 |
| 18 | Joist Girders | G9 | ton | 0.78 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 15,444.00 |
| 18 | Bottom Chord Extension | G7F | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 316.80 |
| 4 | W10x45 | C4 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x60 | C5 | LF | 12.0 | $ 27.00 | $ 1.40 | $ 0.94 | $ 36.00 | $ 1,728.00 |
| 4 | W12x72 | C6 | LF | 12.0 | $ 40.00 | $ 1.47 | $ 0.99 | $ 41.00 | $ 1,968.00 |
| 4 | W8x24 | C10 | LF | 12.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 787.20 |
| 4 | W8x31 | C11 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x35 | C12 | LF | 12.0 | $ 17.00 | $ 1.34 | $ 0.90 | $ 22.00 | $ 1,056.00 |
| 4 | W12x65 | C1 | LF | 26.0 | $ 32.00 | $ 1.47 | $ 0.94 | $ 40.00 | $ 4,160.00 |
| 4 | W12x65 | C2 | LF | 26.0 | $ 32.00 | $ 1.47 | $ 0.94 | $ 40.00 | $ 4,160.00 |
| 4 | W12x65 | C3 | LF | 26.0 | $ 32.00 | $ 1.47 | $ 0.94 | $ 40.00 | $ 4,160.00 |
| 4 | W10x45 | C7 | LF | 26.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 2,912.00 |
| 4 | W10x60 | C8 | LF | 26.0 | $ 27.00 | $ 1.40 | $ 0.94 | $ 36.00 | $ 3,744.00 |
| 4 | W12x72 | C9 | LF | 26.0 | $ 40.00 | $ 1.47 | $ 0.99 | $ 41.00 | $ 4,264.00 |
| 8 | L 1.25 x1.25 x 3/16 | BR1 | lb | 45.0 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 486.46 |
| 4 | L 1.25 x1.25 x 3/16 | BR2 | lb | 46.2 | $ 0.72 | $ 0.29 | $ 0.03 | $ 1.35 | $ 249.61 |
| 180 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 9,000.00 |
| 192 | Joist to column | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 9,600.00 |
| 36 | Joist to column | Moment | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 4,500.00 |
| 24 | Braing Connection | Bracing | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 2,400.00 |
| 24 | Column Splice | Splice | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,606.33 |
| 1 | High Strength Steel | | ton | 138.0 | $ 50.00 | $ - | $ - | $ 55.00 | $ 7,590.85 |
| | | | | | | | | | $ - |
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| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | Page Subtotal | $ 340,633.05 |
| | | | | | | | | Design Total | $ 407,218.50 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.2 Material List and Cost Estimate (continued)

Design III-Penthouse Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | W12x14 | B1 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | W10x12 | B1A | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 2 | W8x10 | B2 | LF | 28.0 | $ 4.95 | $ 2.41 | $ 1.62 | $ 11.45 | $ 641.20 |
| 2 | W12x14 | G1 | LF | 30.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 702.00 |
| 4 | W12x14 | G2 | LF | 10.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 468.00 |
| 4 | W10x49 | C12 | LF | 10.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,200.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 16 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 800.00 |
| 8 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 1,600.00 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ -

Design III-Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 3C18 | Mech. Floor | SF | 1400.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 1,876.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | W12x14 | B3 | LF | 36.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,684.80 |
| 8 | W12x22 | B4 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x16 | B4A | LF | 36.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 1,872.00 |
| 8 | W12x22 | B5 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x22 | B6 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,318.40 |
| 4 | W12x16 | B6A | LF | 36.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 1,872.00 |
| 2 | W10x12 | B7 | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 4 | W12x14 | B8 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,310.40 |
| 2 | W10x12 | B8A | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 2 | W12x16 | B9 | LF | 28.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 728.00 |
| 1 | W12x22 | B10 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 450.80 |
| 1 | W12x22 | B10A | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 450.80 |
| 2 | W12x22 | B11 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 901.60 |
| 2 | W18x35 | B12 | LF | 28.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 1,344.00 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 10 | W12x22 | G3 | LF | 30.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,830.00 |
| 4 | W16x31 | G4 | LF | 30.0 | $ 15.35 | $ 1.61 | $ 1.08 | $ 21.00 | $ 2,520.00 |
| 4 | W18x35 | G5 | LF | 30.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 2,880.00 |
| 2 | W18x35 | G6 | LF | 30.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 1,440.00 |
| 4 | W10x49 | C4 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x45 | C5 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x39 | C6 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x45 | C10 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x45 | C11 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 60 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 3,000.00 |
| 36 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 7,200.00 |
| 40 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 8,000.00 |
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| | | | | | | | | Page Subtotal | $ 86,425.00 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.3 Material List and Cost Estimates (continued)

Design III-Floors

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 3C18 | Floor | SF | 15000.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 60,300.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | Temp. & Shrink. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | W12x26 | B14 | LF | 36.0 | $ 12.85 | $ 1.64 | $ 1.10 | $ 18.25 | $ 7,884.00 |
| 24 | W18x40 | B15 | LF | 36.0 | $ 19.80 | $ 2.19 | $ 1.16 | $ 27.00 | $ 23,328.00 |
| 12 | W16x31 | B16 | LF | 36.0 | $ 15.35 | $ 1.61 | $ 1.08 | $ 21.00 | $ 9,072.00 |
| 24 | W18x46 | B17 | LF | 36.0 | $ 23.00 | $ 2.19 | $ 1.16 | $ 30.00 | $ 25,920.00 |
| 12 | W16x26 | B17A | LF | 36.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 7,668.00 |
| 6 | W12x19 | B18 | LF | 28.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 17.00 | $ 2,856.00 |
| 12 | W16x26 | B19 | LF | 28.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 5,964.00 |
| 6 | W12x26 | B20 | LF | 28.0 | $ 12.85 | $ 1.64 | $ 1.10 | $ 18.25 | $ 3,066.00 |
| 12 | W18x35 | B21 | LF | 28.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 8,064.00 |
| 6 | W12x22 | B22 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,704.80 |
| 6 | W12x22 | B22A | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,704.80 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 30 | W18x35 | G7 | LF | 30.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 21,600.00 |
| 12 | W21x44 | G8 | LF | 30.0 | $ 21.00 | $ 1.97 | $ 1.05 | $ 27.50 | $ 9,900.00 |
| 18 | W21x50 | G9 | LF | 30.0 | $ 23.50 | $ 1.97 | $ 1.05 | $ 30.50 | $ 16,470.00 |
| 4 | W10x49 | C4 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x45 | C5 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x39 | C6 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x45 | C10 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x45 | C11 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W12x65 | C1 | LF | 26.0 | $ 36.00 | $ 1.44 | $ 0.97 | $ 40.00 | $ 4,160.00 |
| 4 | W10x60 | C2 | LF | 26.0 | $ 27.00 | $ 1.44 | $ 0.97 | $ 35.00 | $ 3,640.00 |
| 4 | W10x60 | C3 | LF | 26.0 | $ 27.00 | $ 1.44 | $ 0.97 | $ 35.00 | $ 3,640.00 |
| 4 | W10x60 | C7 | LF | 26.0 | $ 27.00 | $ 1.44 | $ 0.97 | $ 35.00 | $ 3,640.00 |
| 4 | W12x79 | C8 | LF | 26.0 | $ 41.00 | $ 1.47 | $ 0.99 | $ 49.00 | $ 5,096.00 |
| 4 | W14x90 | C9 | LF | 26.0 | $ 48.00 | $ 1.49 | $ 1.00 | $ 55.00 | $ 5,720.00 |
| 180 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 9,000.00 |
| 108 | PR Connection | Semi-Rigid | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 21,600.00 |
| 120 | PR Connection | Semi-Rigid | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 24,000.00 |
| 24 | Column Splice | Splice | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,605.66 |
| 1 | High Strength Steel | | ton | 169.8 | $ 50.00 | $ - | $ - | $ 55.00 | $ 9,340.87 |
| | | | | | | | | | $ - |
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| | | | | | | | | Page Subtotal | $ 433,808.13 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Design Total $ 529,646.33

Table F.3 Material List and Cost Estimates (continued)

Design IV-Penthouse Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | 20K7 | B1 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 336.00 |
| 2 | PR-Steel Joist | B1A | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | 20K3 | B2 | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | Joist Girder | G1 | ton | 0.25 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 550.00 |
| 4 | Joist Girder | G2 | ton | 0.07 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 308.00 |
| 6 | Bottom Chord Extension | | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 105.60 |
| 4 | W8x35 | C12 | LF | 10.0 | $ 17.00 | $ 1.34 | $ 0.90 | $ 22.00 | $ 880.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 4 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 200.00 |
| 4 | Joist to Column | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 200.00 |
| 4 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 500.00 |
| 12 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 1,500.00 |
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| | | | | | | | | Page Subtotal | $ 7,783.60 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.4 Material List and Cost Estimates

Design IV-Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 3C18 | Mech. Floor | SF | 1400.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 1,876.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | PR-Steel Joist | B3 | LF | 36.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 720.00 |
| 8 | 26KCS5 | B4 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | PR-Steel Joist | B4A | LF | 36.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 864.00 |
| 8 | 26KCS5 | B5 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | 26KCS5 | B6 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 1,036.80 |
| 4 | PR-Steel Joist | B6A | LF | 36.0 | $ 2.40 | $ 1.05 | $ 0.56 | $ 5.00 | $ 720.00 |
| 2 | PR-Steel Joist | B7 | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 4 | 20K7 | B8 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 672.00 |
| 2 | PR-Steel Joist | B8A | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | 20K9 | B9 | LF | 28.0 | $ 3.51 | $ 1.05 | $ 0.56 | $ 6.35 | $ 355.60 |
| 1 | 20LH07SP | B10 | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 1 | 20LH07SP | B10A | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 2 | PR-Steel Joist | B11 | LF | 28.0 | $ 5.00 | $ 1.17 | $ 0.62 | $ 8.00 | $ 448.00 |
| 2 | 28VLH 1524/1000 | B12 | LF | 28.0 | $ 7.50 | $ 1.17 | $ 0.62 | $ 11.00 | $ 616.00 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 10 | Joist Girder | G3 | ton | 0.21 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 2,343.00 |
| 4 | Joist Girder | G4 | ton | 0.26 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,130.80 |
| 4 | Joist Girder | G5 | ton | 0.28 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,214.40 |
| 2 | Joist Girder | G6 | ton | 0.35 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 759.00 |
| 20 | Bottom Chord Extensions | | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 352.00 |
| 4 | W10x39 | C4 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x33 | C5 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x33 | C6 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x39 | C10 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x39 | C11 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 29.00 | $ 1,392.00 |
| 60 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 3,000.00 |
| 36 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 4,500.00 |
| 40 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 5,000.00 |
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| | | | | | | | | Page Subtotal | $ 60,942.20 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.4 Material List and Cost Estimate (continued)

Design IV-Floors

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 3C18 | Floor | SF | 15000.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 60,300.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | emp. & Shrink. Reinf. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | PR-Steel Joist | B14 | LF | 36.0 | $ 5.00 | $ 1.17 | $ 0.62 | $ 8.00 | $ 3,456.00 |
| 24 | 28VLH 1091/567 | B15 | LF | 36.0 | $ 8.10 | $ 1.17 | $ 0.62 | $ 11.70 | $ 10,108.80 |
| 12 | PR-Steel Joist | B16 | LF | 36.0 | $ 6.00 | $ 1.17 | $ 0.62 | $ 9.00 | $ 3,888.00 |
| 24 | 28VLH 1524/1000 | B17 | LF | 36.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 11,232.00 |
| 12 | PR-Steel Joist | B17A | LF | 36.0 | $ 6.70 | $ 1.17 | $ 0.62 | $ 10.00 | $ 4,320.00 |
| 6 | PR-Steel Joist | B18 | LF | 28.0 | $ 3.00 | $ 1.50 | $ 0.80 | $ 6.50 | $ 1,092.00 |
| 12 | 28VLH 1143/619 | B19 | LF | 28.0 | $ 6.00 | $ 1.17 | $ 0.62 | $ 9.00 | $ 3,024.00 |
| 6 | PR-Steel Joist | B20 | LF | 28.0 | $ 5.00 | $ 1.17 | $ 0.62 | $ 8.00 | $ 1,344.00 |
| 12 | 28VLH 1524/1000 | B21 | LF | 28.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 4,368.00 |
| 6 | 28VLH 1524/1000 | B22 | LF | 28.0 | $ 9.00 | $ 1.17 | $ 0.62 | $ 13.00 | $ 2,184.00 |
| 6 | PR-Steel Joist | B22A | LF | 28.0 | $ 5.00 | $ 1.17 | $ 0.62 | $ 8.00 | $ 1,344.00 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 30 | Joist Girders | G7 | ton | 0.32 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 10,395.00 |
| 12 | Joist Girders | G8 | ton | 0.44 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 5,742.00 |
| 18 | Joist Girders | G9 | ton | 0.51 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 10,038.60 |
| 60 | Bottom Chord Extension | G7F | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 1,056.00 |
| 4 | W10x39 | C4 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x33 | C5 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x33 | C6 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x39 | C10 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x39 | C11 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 29.00 | $ 1,392.00 |
| 4 | W10x49 | C1 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 29.00 | $ 3,016.00 |
| 4 | W10x54 | C2 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x54 | C3 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x54 | C7 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W12x79 | C8 | LF | 26.0 | $ 36.50 | $ 1.47 | $ 0.99 | $ 44.00 | $ 4,576.00 |
| 4 | W12x87 | C9 | LF | 26.0 | $ 43.00 | $ 1.47 | $ 0.99 | $ 51.00 | $ 5,304.00 |
| 180 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 9,000.00 |
| 108 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 13,500.00 |
| 120 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 15,000.00 |
| 24 | Column Splice | Splice | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,606.33 |
| 1 | High Strength Steel | | ton | 107.9 | $ 50.00 | $ - | $ - | $ 55.00 | $ 5,936.56 |
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| | | | | | | | | Page Subtotal | $ 335,203.29 |
| * Note: All estimates come from the Means Building Construction Cost Data 1994 Manual. | | | | | | | | Design Total | $ 403,929.09 |

Table F.4 Material List and Cost Estimate (continued)

Design V-Penthouse Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | W12x14 | B1 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | W10x12 | B1A | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 2 | W8x10 | B2 | LF | 28.0 | $ 4.95 | $ 2.41 | $ 1.62 | $ 11.45 | $ 641.20 |
| 2 | W12x14 | G1 | LF | 30.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 702.00 |
| 4 | W12x14 | G2 | LF | 10.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 468.00 |
| 4 | W10x49 | C12 | LF | 10.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,200.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 8 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 400.00 |
| 16 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 3,200.00 |
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| | | | | | | | | Page Subtotal | $ 10,613.20 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.5 Material List and Cost Estimate

Design V-Roof

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 2VLI | Mech. Floor | SF | 1400.0 | $ 1.14 | $ 0.26 | $ 0.02 | $ 1.75 | $ 2,450.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | W12x14 | B3 | LF | 36.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,684.80 |
| 8 | W12x22 | B4 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x16 | B4A | LF | 36.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 1,872.00 |
| 8 | W12x22 | B5 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x22 | B6 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,318.40 |
| 4 | W12x16 | B6A | LF | 36.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 1,872.00 |
| 2 | W10x12 | B7 | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 4 | W12x14 | B8 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,310.40 |
| 2 | W10x12 | B8A | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 2 | W12x19 | B9 | LF | 28.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 840.00 |
| 1 | W12x19 | B10 | LF | 28.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 420.00 |
| 1 | W12x19 | B10A | LF | 28.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 420.00 |
| 2 | W12x14 | B11 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | W14x22 | B12 | LF | 28.0 | $ 10.90 | $ 1.46 | $ 0.98 | $ 15.75 | $ 882.00 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 10 | W12x19 | G3 | LF | 30.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 4,500.00 |
| 4 | W14x22 | G4 | LF | 30.0 | $ 10.90 | $ 1.46 | $ 0.98 | $ 15.75 | $ 1,890.00 |
| 4 | W14x22 | G5 | LF | 30.0 | $ 10.90 | $ 1.46 | $ 0.98 | $ 15.75 | $ 1,890.00 |
| 2 | W18x35 | G6 | LF | 30.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 1,440.00 |
| 4 | W10x49 | C4 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x33 | C5 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x33 | C6 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x49 | C10 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C11 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 72 | 3/4" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 120.96 |
| 4 | #4 Reinforcing Steel | | LF | 64.0 | $ - | $ - | $ - | $ 0.39 | $ 100.86 |
| 60 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 3,000.00 |
| 32 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 6,400.00 |
| 36 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 7,200.00 |
| 8 | PRC Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 150.00 | $ 1,200.00 |
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| | | | | | | | | Page Subtotal | $ 83,828.82 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.5 Material List and Cost Estimate (continued)

Design V-Floors

| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Total Incl. O & P | Total |
|---|---|---|---|---|---|---|---|---|---|
| 3 | 2VLI | Floor | SF | 15000.0 | $ 1.14 | $ 0.26 | $ 0.02 | $ 1.75 | $ 78,750.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | Temp. & Shrink. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | W12x22 | B14 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 6,955.20 |
| 24 | W14x30 | B15 | LF | 36.0 | $ 14.85 | $ 1.61 | $ 1.08 | $ 20.50 | $ 17,712.00 |
| 12 | W16x26 | B16 | LF | 36.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 7,668.00 |
| 24 | W14x30 | B17 | LF | 36.0 | $ 14.85 | $ 1.61 | $ 1.08 | $ 20.50 | $ 17,712.00 |
| 12 | W16x26 | B17A | LF | 36.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 7,668.00 |
| 6 | W12x14 | B18 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,965.60 |
| 12 | W12x22 | B19 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 5,409.60 |
| 6 | W12x22 | B20 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,704.80 |
| 12 | W14x22 | B21 | LF | 28.0 | $ 10.90 | $ 1.46 | $ 0.98 | $ 16.10 | $ 5,409.60 |
| 6 | W12x19 | B22 | LF | 28.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.50 | $ 2,268.00 |
| 6 | W12x14 | B22A | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,965.60 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 30 | W16x26 | G7 | LF | 30.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 15,975.00 |
| 12 | W18x40 | G8 | LF | 30.0 | $ 19.80 | $ 2.19 | $ 1.16 | $ 27.00 | $ 9,720.00 |
| 18 | W21x44 | G9 | LF | 30.0 | $ 21.00 | $ 1.97 | $ 1.05 | $ 27.50 | $ 14,850.00 |
| 4 | W10x49 | C4 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x33 | C5 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x33 | C6 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x49 | C10 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C11 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x54 | C1 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x49 | C2 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 3,120.00 |
| 4 | W10x49 | C3 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 3,120.00 |
| 4 | W10x60 | C7 | LF | 26.0 | $ 27.00 | $ 1.44 | $ 0.97 | $ 35.00 | $ 3,640.00 |
| 4 | W12x79 | C8 | LF | 26.0 | $ 41.00 | $ 1.47 | $ 0.99 | $ 49.00 | $ 5,096.00 |
| 4 | W12x87 | C9 | LF | 26.0 | $ 43.00 | $ 1.47 | $ 0.99 | $ 51.00 | $ 5,304.00 |
| 2544 | 3/4" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 4,273.92 |
| 114 | #4 Reinforcing Steel | | LF | 64.0 | $ - | $ - | $ - | $ 0.39 | $ 2,874.62 |
| 180 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 9,000.00 |
| 108 | PRC Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 150.00 | $ 16,200.00 |
| 120 | PRC Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 150.00 | $ 18,000.00 |
| 24 | Column Splice | Splice | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,606.33 |
| 1 | High Strength Steel | | ton | 141.9 | $ 50.00 | $ - | $ - | $ 55.00 | $ 7,806.33 |
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| | | | | | | | | Page Subtotal | $ 414,426.60 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Design Total $ 508,868.62

Table F.5 Material List and Cost Estimate (continued)

Design VI-Penthouse Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | 20K7 | B1 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 336.00 |
| 2 | PR-Steel Joist | B1A | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | 20K3 | B2 | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | Joist Girder | G1 | ton | 0.25 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 550.00 |
| 4 | Joist Girder | G2 | ton | 0.07 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 308.00 |
| 6 | Bottom Chord Extension | | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 105.60 |
| 4 | W8x35 | C12 | LF | 10.0 | $ 17.00 | $ 1.34 | $ 0.90 | $ 22.00 | $ 880.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 4 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 200.00 |
| 4 | Joist to Column | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 200.00 |
| 4 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 400.00 |
| 12 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 1,200.00 |
| | | | | | | | | | $ - |
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| | | | | | | | | | $ -

Design VI-Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 3C18 | Mech. Floor | SF | 1400.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 1,876.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | PR-Steel Joist | B3 | LF | 36.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 720.00 |
| 8 | 26KCS5 | B4 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | PR-Steel Joist | B4A | LF | 36.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 864.00 |
| 8 | 26KCS5 | B5 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | 26KCS5 | B6 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 1,036.80 |
| 4 | PR-Steel Joist | B6A | LF | 36.0 | $ 2.40 | $ 1.05 | $ 0.56 | $ 5.00 | $ 720.00 |
| 2 | PR-Steel Joist | B7 | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 4 | 20K7 | B8 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 672.00 |
| 2 | PR-Steel Joist | B8A | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | 20K9 | B9 | LF | 28.0 | $ 3.51 | $ 1.05 | $ 0.56 | $ 6.35 | $ 355.60 |
| 1 | 20LH07SP | B10 | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 1 | 20LH07SP | B10A | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 2 | PR-Steel Joist | B11 | LF | 28.0 | $ 3.90 | $ 1.17 | $ 0.62 | $ 6.50 | $ 364.00 |
| 2 | PR-Steel Joist | B12 | LF | 28.0 | $ 5.20 | $ 1.17 | $ 0.62 | $ 8.45 | $ 473.20 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 10 | Joist Girder | G3 | ton | 0.21 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 2,343.00 |
| 4 | Joist Girder | G4 | ton | 0.26 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,130.80 |
| 4 | Joist Girder | G5 | ton | 0.28 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,214.40 |
| 2 | Joist Girder | G6 | ton | 0.35 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 759.00 |
| 20 | Bottom Chord Extensions | | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 352.00 |
| 4 | W8x31 | C4 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C5 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C6 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C10 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x39 | C11 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x45 | C12 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 92 | 1/2" Shear Studs | | ea | 1.0 | $ 0.25 | $ 0.43 | $ 0.45 | $ 1.68 | $ 154.56 |
| 80 | 5/8" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 134.40 |
| 4 | #4 Reinforcing Steel | | LF | 32.0 | $ - | $ - | $ - | $ 0.39 | $ 50.43 |
| 60 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 3,000.00 |
| 36 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 3,600.00 |
| 40 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 4,000.00 |
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| | | | | | | | | Page Subtotal | $ 58,434.79 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.6 Material List and Cost Estimate (continued)

Design VI-Floors

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 3 | 3C18 | Floor | SF | 15000.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 60,300.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | Temp. & Shrink. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | PR-Steel Joist | B14 | LF | 36.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 2,808.00 |
| 24 | PR-Steel Joist | B15 | LF | 36.0 | $ 5.50 | $ 1.17 | $ 0.62 | $ 9.00 | $ 7,776.00 |
| 12 | PR-Steel Joist | B16 | LF | 36.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 2,808.00 |
| 24 | PR-Steel Joist | B17 | LF | 36.0 | $ 6.75 | $ 1.17 | $ 0.62 | $ 10.00 | $ 8,640.00 |
| 12 | PR-Steel Joist | B17A | LF | 36.0 | $ 3.75 | $ 1.17 | $ 0.62 | $ 7.50 | $ 3,240.00 |
| 6 | PR-Steel Joist | B18 | LF | 28.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 1,092.00 |
| 12 | PR-Steel Joist | B19 | LF | 28.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 2,184.00 |
| 6 | PR-Steel Joist | B20 | LF | 28.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 1,092.00 |
| 12 | PR-Steel Joist | B21 | LF | 28.0 | $ 5.20 | $ 1.17 | $ 0.62 | $ 8.45 | $ 2,839.20 |
| 6 | PR-Steel Joist | B22 | LF | 28.0 | $ 5.20 | $ 1.17 | $ 0.62 | $ 8.45 | $ 1,419.60 |
| 6 | PR-Steel Joist | B22A | LF | 28.0 | $ 5.00 | $ 1.17 | $ 0.62 | $ 8.00 | $ 1,344.00 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 30 | Joist Girders | G7 | ton | 0.29 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 9,570.00 |
| 12 | Joist Girders | G8 | ton | 0.38 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 4,950.00 |
| 18 | Joist Girders | G9 | ton | 0.42 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 8,316.00 |
| 60 | Bottom Chord Extension | G7F | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 1,056.00 |
| 4 | W8x31 | C4 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C5 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C6 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C10 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x39 | C11 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x45 | C12 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x49 | C1 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 29.00 | $ 3,016.00 |
| 4 | W10x54 | C2 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x54 | C3 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x49 | C7 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 29.00 | $ 3,016.00 |
| 4 | W12x72 | C8 | LF | 26.0 | $ 33.50 | $ 1.47 | $ 0.99 | $ 40.50 | $ 4,212.00 |
| 4 | W12x87 | C9 | LF | 26.0 | $ 43.00 | $ 1.47 | $ 0.99 | $ 51.00 | $ 5,304.00 |
| 1860 | 1/2" Shear Studs | | ea | 1.0 | $ 0.25 | $ 0.43 | $ 0.45 | $ 1.68 | $ 3,124.80 |
| 3120 | 5/8" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 5,241.60 |
| 96 | #4 Reinforcing Steel | | LF | 32.0 | $ - | $ - | $ - | $ 0.39 | $ 1,210.37 |
| 18 | #5 Reinforcing Steel | | LF | 48.0 | $ - | $ - | $ - | $ 0.61 | $ 527.04 |
| 180 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 9,000.00 |
| 108 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 10,800.00 |
| 120 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 12,000.00 |
| 24 | Column Splice | Splice | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,606.33 |
| 1 | High Strength Steel | | ton | 99.0 | $ 50.00 | $ - | $ - | $ 55.00 | $ 5,446.74 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | Page Subtotal | $ 323,419.67 |
| * Note: All estimates come from the Means Building Construction Cost Data 1994 Manual. | | | | | | | | Design Total | $ 389,238.06 |

Table F.6 Material List and Cost Estimate (continued)

Design VII-Penthouse Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | W12x14 | B1 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | W10x12 | B1A | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 2 | W8x10 | B2 | LF | 28.0 | $ 4.95 | $ 2.41 | $ 1.62 | $ 11.45 | $ 641.20 |
| 2 | W12x14 | G1 | LF | 30.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 702.00 |
| 4 | W12x14 | G2 | LF | 10.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 468.00 |
| 4 | W10x49 | C12 | LF | 10.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,200.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 8 | Shear Connection | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 400.00 |
| 16 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 3,200.00 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ -

Design VII-Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 2VLI | Mech. Floor | SF | 1400.0 | $ 1.14 | $ 0.26 | $ 0.02 | $ 1.75 | $ 2,450.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | W12x14 | B3 | LF | 36.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,684.80 |
| 8 | W12x22 | B4 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x16 | B4A | LF | 36.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 1,872.00 |
| 8 | W12x22 | B5 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 4,636.80 |
| 4 | W12x22 | B6 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,318.40 |
| 4 | W12x16 | B6A | LF | 36.0 | $ 8.00 | $ 1.64 | $ 1.10 | $ 13.00 | $ 1,872.00 |
| 2 | W10x12 | B7 | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 4 | W12x14 | B8 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,310.40 |
| 2 | W10x12 | B8A | LF | 28.0 | $ 5.95 | $ 2.41 | $ 1.62 | $ 12.55 | $ 702.80 |
| 2 | W12x19 | B9 | LF | 28.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 840.00 |
| 1 | W12x19 | B10 | LF | 28.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 420.00 |
| 1 | W12x19 | B10A | LF | 28.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 420.00 |
| 2 | W12x14 | B11 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 655.20 |
| 2 | W12x19 | B12 | LF | 28.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 840.00 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 10 | W12x19 | G3 | LF | 30.0 | $ 9.50 | $ 1.64 | $ 1.10 | $ 15.00 | $ 4,500.00 |
| 4 | W14x22 | G4 | LF | 30.0 | $ 10.90 | $ 1.46 | $ 0.98 | $ 15.75 | $ 1,890.00 |
| 4 | W14x22 | G5 | LF | 30.0 | $ 10.90 | $ 1.46 | $ 0.98 | $ 15.75 | $ 1,890.00 |
| 2 | W18x35 | G6 | LF | 30.0 | $ 17.35 | $ 2.19 | $ 1.16 | $ 24.00 | $ 1,440.00 |
| 4 | W10x49 | C4 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x33 | C5 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x33 | C6 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x49 | C10 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C11 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 40 | 3/4" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 67.20 |
| 6 | #4 Reinforcing Steel | | LF | 64.0 | $ - | $ - | $ - | $ 0.39 | $ 151.30 |
| 56 | Shear Conneciton | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 2,800.00 |
| 32 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 6,400.00 |
| 36 | PR Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 200.00 | $ 7,200.00 |
| 12 | PRC Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 150.00 | $ 1,800.00 |
| | | | | | | | | | $ - |
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| | | | | | | | | Page Subtotal | $ 84,183.50 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.7 Material List and Cost Estimate (continued)

Design VII-Floors

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 3 | 2VLI | Floor | SF | 15000.0 | $ 1.14 | $ 0.26 | $ 0.02 | $ 1.75 | $ 78,750.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | Temp. & Shrink. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | W12x22 | B14 | LF | 36.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 6,955.20 |
| 24 | W14x30 | B15 | LF | 36.0 | $ 14.85 | $ 1.61 | $ 1.08 | $ 20.50 | $ 17,712.00 |
| 12 | W16x26 | B16 | LF | 36.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 7,668.00 |
| 24 | W16x26 | B17 | LF | 36.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 15,336.00 |
| 12 | W16x26 | B17A | LF | 36.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 7,668.00 |
| 6 | W12x14 | B18 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,965.60 |
| 12 | W12x22 | B19 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 5,409.60 |
| 6 | W12x22 | B20 | LF | 28.0 | $ 10.90 | $ 1.64 | $ 1.10 | $ 16.10 | $ 2,704.80 |
| 12 | W14x22 | B21 | LF | 28.0 | $ 10.90 | $ 1.46 | $ 0.98 | $ 16.10 | $ 5,409.60 |
| 6 | W12x14 | B22 | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,965.60 |
| 6 | W12x14 | B22A | LF | 28.0 | $ 6.95 | $ 1.64 | $ 1.10 | $ 11.70 | $ 1,965.60 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 30 | W16x26 | G7 | LF | 30.0 | $ 12.85 | $ 1.45 | $ 0.97 | $ 17.75 | $ 15,975.00 |
| 12 | W18x40 | G8 | LF | 30.0 | $ 19.80 | $ 2.19 | $ 1.16 | $ 27.00 | $ 9,720.00 |
| 18 | W21x44 | G9 | LF | 30.0 | $ 21.00 | $ 1.97 | $ 1.05 | $ 27.50 | $ 14,850.00 |
| 4 | W10x49 | C4 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x33 | C5 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x33 | C6 | LF | 12.0 | $ 16.00 | $ 1.34 | $ 0.90 | $ 21.00 | $ 1,008.00 |
| 4 | W10x49 | C10 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C11 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x49 | C12 | LF | 12.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 1,440.00 |
| 4 | W10x54 | C1 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x49 | C2 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 3,120.00 |
| 4 | W10x49 | C3 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 30.00 | $ 3,120.00 |
| 4 | W10x60 | C7 | LF | 26.0 | $ 27.00 | $ 1.44 | $ 0.97 | $ 35.00 | $ 3,640.00 |
| 4 | W12x79 | C8 | LF | 26.0 | $ 41.00 | $ 1.47 | $ 0.99 | $ 49.00 | $ 5,096.00 |
| 4 | W12x87 | C9 | LF | 26.0 | $ 43.00 | $ 1.47 | $ 0.99 | $ 51.00 | $ 5,304.00 |
| 1668 | 3/4" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 2,802.24 |
| 192 | #4 Reinforcing Steel | | LF | 64.0 | $ - | $ - | $ - | $ 0.39 | $ 4,841.47 |
| 288 | PRC Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 150.00 | $ 43,200.00 |
| 120 | PRC Connection | PR | ea | 1.0 | $ - | $ - | $ - | $ 150.00 | $ 18,000.00 |
| 24 | Column Splice | Splice | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,606.33 |
| 1 | High Strength Steel | | ton | 140.3 | $ 50.00 | $ - | $ - | $ 55.00 | $ 7,713.97 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
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| | | | | | | | | | $ - |
| | | | | | | | | Page Subtotal | $ 430,151.01 |
| | | | | | | | | Design Total | $ 524,947.70 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.7 Material List and Cost Estimate (continued)

Design VIII-Penthouse Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Pent. Roof | SF | 1400.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 1,988.00 |
| 2 | 20K7 | B1 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 336.00 |
| 2 | PR-Steel Joist | B1A | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | 20K3 | B2 | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | Joist Girder | G1 | ton | 0.25 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 550.00 |
| 4 | Joist Girder | G2 | ton | 0.07 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 308.00 |
| 6 | Bottom Chord Extension | | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 105.60 |
| 4 | W8x35 | C12 | LF | 10.0 | $ 17.00 | $ 1.34 | $ 0.90 | $ 22.00 | $ 880.00 |
| 4 | W6x12 | C13 | LF | 10.0 | $ 11.90 | $ 1.34 | $ 0.90 | $ 16.40 | $ 656.00 |
| 4 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 200.00 |
| 4 | Joist to Column | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 200.00 |
| 4 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 500.00 |
| 12 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 1,500.00 |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ -

Design VIII-Roof

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 1 | 3N22 | Roof | SF | 13600.0 | $ 0.89 | $ 0.23 | $ 0.02 | $ 1.42 | $ 19,312.00 |
| 1 | 3C18 | Mech. Floor | SF | 1400.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 1,876.00 |
| 1 | Concrete Topping | Mech. Floor | SF | 1400.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 2,394.00 |
| 1 | Temp. & Shrink. @12" | | LF | 2800.0 | $ - | $ - | $ - | $ 0.39 | $ 1,092.00 |
| 4 | PR-Steel Joist | B3 | LF | 36.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 720.00 |
| 8 | 26KCS5 | B4 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | PR-Steel Joist | B4A | LF | 36.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 864.00 |
| 8 | 26KCS5 | B5 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 2,073.60 |
| 4 | 26KCS5 | B6 | LF | 36.0 | $ 4.48 | $ 0.95 | $ 0.51 | $ 7.20 | $ 1,036.80 |
| 4 | PR-Steel Joist | B6A | LF | 36.0 | $ 2.40 | $ 1.05 | $ 0.56 | $ 5.00 | $ 720.00 |
| 2 | PR-Steel Joist | B7 | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 4 | 20K7 | B8 | LF | 28.0 | $ 3.00 | $ 1.05 | $ 0.56 | $ 6.00 | $ 672.00 |
| 2 | PR-Steel Joist | B8A | LF | 28.0 | $ 2.15 | $ 1.05 | $ 0.56 | $ 5.00 | $ 280.00 |
| 2 | 20K9 | B9 | LF | 28.0 | $ 3.51 | $ 1.05 | $ 0.56 | $ 6.35 | $ 355.60 |
| 1 | 20LH07SP | B10 | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 1 | 20LH07SP | B10A | LF | 28.0 | $ 6.00 | $ 1.50 | $ 0.80 | $ 10.15 | $ 284.20 |
| 2 | PR-Steel Joist | B11 | LF | 28.0 | $ 3.90 | $ 1.17 | $ 0.62 | $ 6.50 | $ 364.00 |
| 2 | PR-Steel Joist | B12 | LF | 28.0 | $ 5.00 | $ 1.17 | $ 0.62 | $ 8.00 | $ 448.00 |
| 1 | M8x6.5 | B13 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 109.00 |
| 10 | Joist Girder | G3 | ton | 0.21 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 2,343.00 |
| 4 | Joist Girder | G4 | ton | 0.26 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,130.80 |
| 4 | Joist Girder | G5 | ton | 0.28 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 1,214.40 |
| 2 | Joist Girder | G6 | ton | 0.35 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 759.00 |
| 20 | Bottom Chord Extensions | | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 352.00 |
| 4 | W8x31 | C4 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C5 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C6 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C10 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x39 | C11 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x45 | C12 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 112 | 1/2" Shear Studs | | ea | 1.0 | $ 0.25 | $ 0.43 | $ 0.45 | $ 1.68 | $ 188.16 |
| 80 | 5/8" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 134.40 |
| 4 | #4 Reinforcing Steel | | LF | 272.0 | $ - | $ - | $ - | $ 0.39 | $ 428.67 |
| 56 | Joist to Joist Girder | Simple | ea | 1.0 | $ - | $ - | $ - | $ 50.00 | $ 2,800.00 |
| 12 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 1,200.00 |
| 32 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 4,000.00 |
| 36 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 125.00 | $ 4,500.00 |
| | | | | | | | | | $ - |
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| | | | | | | | | | $ - |
| | | | | | | | | Page Subtotal | $ 60,721.43 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.8 Material List and Cost Estimate (continued)

Design VIII-Floors

| | | | | | | | | Total | |
|---|---|---|---|---|---|---|---|---|---|
| Quantity | Piece | Designation | Unit | Units | Material | Labor | Equipment | Incl. O & P | Total |
| 3 | 3C18 | Floor | SF | 15000.0 | $ 0.81 | $ 0.23 | $ 0.02 | $ 1.34 | $ 60,300.00 |
| 3 | Concrete Topping | Floor | SF | 15000.0 | $ 0.63 | $ 0.55 | $ 0.16 | $ 1.71 | $ 76,950.00 |
| 3 | Temp. & Shrink. @12" | | LF | 30000.0 | $ - | $ - | $ - | $ 0.39 | $ 35,100.00 |
| 12 | PR-Steel Joist | B14 | LF | 36.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 2,808.00 |
| 24 | PR-Steel Joist | B15 | LF | 36.0 | $ 5.50 | $ 1.17 | $ 0.62 | $ 9.00 | $ 7,776.00 |
| 12 | PR-Steel Joist | B16 | LF | 36.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 2,808.00 |
| 24 | PR-Steel Joist | B17 | LF | 36.0 | $ 6.75 | $ 1.17 | $ 0.62 | $ 10.00 | $ 8,640.00 |
| 12 | PR-Steel Joist | B17A | LF | 36.0 | $ 3.75 | $ 1.17 | $ 0.62 | $ 7.50 | $ 3,240.00 |
| 6 | PR-Steel Joist | B18 | LF | 28.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 1,092.00 |
| 12 | PR-Steel Joist | B19 | LF | 28.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 2,184.00 |
| 6 | PR-Steel Joist | B20 | LF | 28.0 | $ 3.50 | $ 1.17 | $ 0.62 | $ 6.50 | $ 1,092.00 |
| 12 | PR-Steel Joist | B21 | LF | 28.0 | $ 5.20 | $ 1.17 | $ 0.62 | $ 8.45 | $ 2,839.20 |
| 6 | PR-Steel Joist | B22 | LF | 28.0 | $ 5.20 | $ 1.17 | $ 0.62 | $ 8.45 | $ 1,419.60 |
| 6 | PR-Steel Joist | B22A | LF | 28.0 | $ 5.00 | $ 1.17 | $ 0.62 | $ 8.00 | $ 1,344.00 |
| 6 | M8x6.5 | B23 | LF | 10.0 | $ 4.45 | $ 2.41 | $ 1.62 | $ 10.90 | $ 654.00 |
| 30 | Joist Girders | G7 | ton | 0.29 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 9,570.00 |
| 12 | Joist Girders | G8 | ton | 0.38 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 4,950.00 |
| 18 | Joist Girders | G9 | ton | 0.42 | $ 700.00 | $ 140.00 | $ 74.00 | $ 1,100.00 | $ 8,316.00 |
| 60 | Bottom Chord Extension | | ea | 2.0 | $ 8.00 | $ - | $ - | $ 8.80 | $ 1,056.00 |
| 4 | W8x31 | C4 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C5 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C6 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W8x31 | C10 | LF | 12.0 | $ 15.35 | $ 1.34 | $ 0.90 | $ 20.00 | $ 960.00 |
| 4 | W10x39 | C11 | LF | 12.0 | $ 20.00 | $ 1.40 | $ 0.94 | $ 26.00 | $ 1,248.00 |
| 4 | W10x45 | C12 | LF | 12.0 | $ 22.50 | $ 1.40 | $ 0.94 | $ 28.00 | $ 1,344.00 |
| 4 | W10x49 | C1 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 29.00 | $ 3,016.00 |
| 4 | W10x54 | C2 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x54 | C3 | LF | 26.0 | $ 25.00 | $ 1.40 | $ 0.94 | $ 30.50 | $ 3,172.00 |
| 4 | W10x49 | C7 | LF | 26.0 | $ 24.00 | $ 1.40 | $ 0.94 | $ 29.00 | $ 3,016.00 |
| 4 | W12x72 | C8 | LF | 26.0 | $ 33.50 | $ 1.47 | $ 0.99 | $ 40.50 | $ 4,212.00 |
| 4 | W12x87 | C9 | LF | 26.0 | $ 43.00 | $ 1.47 | $ 0.99 | $ 51.00 | $ 5,304.00 |
| 3228 | 1/2" Shear Studs | | ea | 1.0 | $ 0.25 | $ 0.43 | $ 0.45 | $ 1.68 | $ 5,423.04 |
| 2256 | 5/8" Shear Studs | | ea | 1.0 | $ 0.37 | $ 0.43 | $ 0.45 | $ 1.68 | $ 3,790.08 |
| 174 | #4 Reinforcing Steel | | LF | 32.0 | $ - | $ - | $ - | $ 0.39 | $ 2,193.79 |
| 18 | #5 Reinforcing Steel | | LF | 48.0 | $ - | $ - | $ - | $ 0.61 | $ 527.04 |
| 180 | Joist to Joist Girder | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 18,000.00 |
| 108 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 10,800.00 |
| 120 | Joist to Column | PR | ea | 1.0 | $ - | $ - | $ - | $ 100.00 | $ 12,000.00 |
| 24 | Column Splices | Splices | ea | 1.0 | $ - | $ - | $ - | $ 500.00 | $ 12,000.00 |
| 24 | PL 2' x 2' x 1" | Base Plates | lb | 163.3 | $ 0.48 | $ 0.21 | $ - | $ 0.92 | $ 3,606.33 |
| 1 | High Strength Steel | | ton | 86.1 | $ 50.00 | $ - | $ - | $ 55.00 | $ 4,735.07 |
| | | | | | | | | | |
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| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | | $ - |
| | | | | | | | | Page Subtotal | $ 333,538.15 |
| | | | | | | | | Design Total | $ 402,043.18 |

* Note: All estimates come from the Means Building Construction Cost Data 1994 Manual.

Table F.8 Material List and Cost Estimate (continued)

# VITA

Joseph P. Migliozzi was born in the Bronx, New York on September 21st, 1973. When he was four his family moved to Peekskill, New York. He is the only son of Joseph and Angelica Migliozzi. He lived in Peekskill, later renamed Cortlandt Manor, where he went to Walter Panas High School. In high school he concentrated his efforts in mathematics and science. He continued his studies, after high school, at Manhattan College which is located in Riverdale, New York. Here, he received a Presidential Scholarship which paid for half of his tuition. While living at home, he worked at a Digital Oscilloscope Manufacturer as a computer hardware and software assistant. In college, Joseph earned a degree in Civil Engineering with a specialization in Structures. He graduated Magna Cum Laude with a 3.8 GPA overall and an in major GPA of 4.0 in the Spring of 1995. After working that summer for Thornton-Tomassetti Structural Engineers he left New York to continue his education. In the fall of 1995 he entered Virginia Polytechnic Institute and State University's structural engineering department. Here he received a teaching assistantship and research assistantship. In the winter of 1997 Joseph completed his Master of Science Degree in Civil Engineering. He is now working in Baltimore, Maryland as a bridge engineer with Whitman Requardt Associates.