ABSTRACT

HALL, BRIAN S., GA-Based Optimization of Steel Moment Frames: A Case Study (Under the direction of Dr. Abhinav Gupta).

Moment resisting frames are utilized in steel construction in order to provide overall stability and to withstand lateral forces related to wind and earthquake loadings. While the industry focuses on producing more economical moment frames by optimizing for the weight of steel, it should be noted that because of present trends the labor cost associated with rigid moment connection fabrication governs the total cost of these frames. Therefore, the objective of this case study is to discover alternative ways of producing more cost-effective moment frames without compromising overall stability. This is achieved by considering both the cost of steel and the cost of connections within the design process.

In order to reduce cost, rigid connections within the frame are replaced with standard pinned connections, and member sizes are increased where needed, a method unlike the current least-weight design approach. Optimization techniques are developed in order to identify the most advantageous locations of the remaining moment connections. A Genetic Algorithm, interfaced with a Java-based frame analysis program, is utilized to produce these optimal solutions.

At the present time, the consideration of connection-related costs is crucial in determining the most cost effective moment frames. By developing an optimization

technique that considers both the number of moment connections and the total weight of the frame, the least weight optimal solution can be improved upon drastically, with an almost 50% reduction in the total cost and an approximately 60% increase in total weight. The unorthodox connection arrangements produced are important, as they provide alternative designs that, while functional, are not currently explored by those in practice.

GA-Based Optimization of Steel Moment Frames: A Case Study

by

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BIOGRAPHY

Brian S. Hall was born on May 1, 1981 in Wilmington, NC, as the son of Tony LeRoy Hall and Sandra Dee Hall. He grew up in nearby Shallotte, NC, and received a diploma from West Brunswick High School in the summer of 1999. He attended North Carolina State University (NCSU) until May, 2003, graduating Summa Cum Laude with a Bachelor's of Science degree in Civil Engineering. Later that same year he enrolled in the graduate program at NCSU to begin work on a Masters of Science degree in Structural Engineering.

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Thanks go out to friends and family members, both in Raleigh and at home, for the help and support they provided during my time at NC State.

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CHAPTER 1: INTRODUCTION

1.1 Background and Motivation

Moment resisting rigid or semi-rigid steel frames are often utilized to provide increased stability for resisting lateral forces such as those due to earthquake and wind loadings. These frames are much heavier relative to braced frames within a structure. Consequently, a majority of steel frame optimization research has focused on developing "least-weight" designs. These include recent studies by Torregosa (2002) and Kim (2004) involving genetic algorithms, search algorithms based on the concepts of natural selection and adaptation (Burns, 2002). In all of these studies, the objective has been to minimize the total weight of the steel frame by selecting members from a database that provide the lightest frame possible subjected to a set of specified strength and serviceability constraints.

In order to provide lateral load resistance, the typical beam-to-column connection within a moment frame is assumed to be rigid or fixed. As an alternative, semi-rigid connections have also been developed. While these connections are not often used in practice, recent studies on several types of moment resisting semi-rigid frames have shown that considering semi-rigid connection behavior in low and mid-rise buildings can result in more economical moment frames. For example, Kameshki (2001) uses genetic algorithm-based optimization for design of nonlinear multistory steel frames with semi-rigid connections. To obtain an optimal design, the algorithm determines the appropriate combination of standard steel sections that would form a least weight design as well as

satisfy serviceability and strength constraints. A nonlinear empirical model is used to include the nonlinear behavior of the moment-rotation relationship of the beam to column connections.

Another study related to moment frames involves design procedures that utilize Ant Colony Optimization (ACO) techniques (Camp, 2004). As in other cases, the objective function considered for this optimization technique is the total weight of the structure subjected to serviceability and strength requirements specified by the AISC Load and Resistance Factor Design (LRFD). Results obtained from ACO are compared with those previously evaluated using genetic algorithms and classical optimization methods.

In another recent study, Sarma (2005) gives comparison between the use of Allowable Stress Design (ASD) methods and LRFD methods in the optimization of high rise buildings. Results are evaluated for two different structures; a 36-story moment frame and a 144-story super high-rise building. This study concludes that the LRFD based design code does not provide any significant "weight savings" when serviceability is the controlling factor. Once again, the study focuses on evaluating the minimum weight design.

While it is obvious that several different optimization techniques have been used to determine the least weight design for steel moment frames, none of the above studies consider a key variable that often dictates the cost of these frames in actual practice: the

labor cost associated with fabrication of the moment connection, which can be excessively high due to in-situ welding. As the industry focuses on producing more economical designs by designing lighter steel frames, it is important to note that the labor costs associated with the fabrication of rigid and semi-rigid moment connections governs the total cost of these frames, not the least weight design. Over the past decade, the labor costs have continued to increase relative to the cost of steel hardware. Therefore, least weight moment resisting frames are not necessarily the most economical. None of the existing steel design software (SAP2000, RAMSteel) can consider these costs directly. Moreover, the existing optimization techniques would need to be modified to achieve such an objective.

1.2 Objective

The objective of this study is to explore the effect of considering the cost of fabricating a connection to produce more economical moment resisting frames without compromising the overall stability of the frame and without focusing primarily on the reduction of frame weight. This is achieved by altering the number of moment connections within the frame to reduce cost; replacing some of the rigid connections within the frame with standard pinned connections, and increasing member sizes wherever needed. For simplicity, this exploratory study focuses solely on rigid connections (and not semi-rigid connections). Optimization techniques are developed in order to determine the optimal number of rigid connections and to identify their optimal locations within a frame. Trade-off curves between the number of connections and total

cost are developed to illustrate the benefit of removing connections while often increasing the frame weight.

As stated previously, each joint or connection is ideally represented as a rigid or fixed connection in moment frame design. To provide this rigidity, several different types of connections can be considered with varying amounts of on-site welding (including the welded flange plate connection discussed in detail in the following chapters). By eliminating some of these connections, a significant reduction can be achieved in the amount of on-site labor involved with connection construction and in the fabrication time leading to an appreciable decrease in the overall cost of these frames. Costs associated with added weight from larger/heavier members is quite minimal because of the increasing discrepancy between the labor costs and the material costs.

In order to determine the total number and configurations of rigid connections within a frame that produces the most economical design, we employ heuristic optimization methods such as Genetic Algorithm which is interfaced with a Java-based frame analysis program for strength and serviceability evaluation. The frame analysis program is validated by comparisons with the corresponding results from SAP2000. While the framework developed in this study can be applied to any typical moment resisting steel frame, this case study considers a 5-story steel frame office building as a test-bed.

Specific tasks needed to achieve the objectives of this study are:

- Select an appropriate building model for the case study.
- Create a hand-designed least-weight moment frame for the selected building model.
- Generate an improved least-weight frame design using optimization in SAP2000.
- Design a Welded-Flange Plate Connection to be used as the standard for cost evaluation.
- Develop a frame analysis program for determining the member-end forces and displacements, and for performing strength and serviceability checks.
- Implement an optimization procedure for evaluating optimal number of rigid connections needed in a frame.
- Develop a strategy for updating members that do not satisfy the strength and serviceability requirements due to connection rearrangement.
- Generate trade-off curves between the total cost and the number of rigid connections.

Chapter 2 of this thesis discusses the office building in greater detail, as well as outlines the load combinations and strength calculations implemented by the optimization programs. Chapter 3 gives a brief introduction to moment connections followed by the detailed design of a welded flange plate connection. Chapter 4 describes the Genetic Algorithm and provides an explanation of the optimization process. In the end, Chapter 5 presents the results and conclusions of this study. Appendix A provides additional solutions which are not discussed in the results section of this paper. Appendix B

includes several tables and figures from the design manuals and building codes referenced in Chapters 2 and 3. A numerical design example for Welded Flange Plate moment connections is presented in Appendix C.

CHAPTER 2: DESCRIPTION & DESIGN OF A TEST-BED BUILDING FRAME

2.1 Building Information

In order to achieve the objectives of this study, a commercial building designed in the structural engineering capstone design course (CE420) at North Carolina State University is considered as a test-bed. The proposed structure is a 100,200 ft^2, 5-story steel building, and is expected to be used for office and/or retail space. The building site is located in the vicinity of Raleigh, NC, placing it in Exposure Category B, defined by the 2002 edition of the North Carolina State Building Code as an urban or suburban area with numerous closely spaced obstructions having the size of single family dwellings or larger. The structure measures 172 ft. in length (in the east-west direction), 116.5 ft. in width (in the north-south direction), and stands a total of 57.5 ft. in height. All floor heights are set at 8 ft. (floor to ceiling) or 11.5 ft (floor to floor). This allows 3.5 ft. for the concrete slab, the decking, the supporting beams and girders, the HVAC space and the suspended acoustical ceiling tile. Included in the design of the building is a 28 ft. (in the east-west direction) by 36.5 ft. (in the north-south direction) central core that houses two stairwells, an elevator, and a lobby. Figures 1 & 2 illustrate these schematics.

The building consists of five bays in the east-west direction as well as five bays in the north-south direction. For the bays in the east-west direction, the distances (in feet) between frames are as follows: 36, 36, 28, 36, and 36. For the bays in the north-south direction, the distances (in feet) are 20, 20, 36.5, 20 and 20. Because of the presence of a central core, lateral bracing is used to resist wind loads in the east-west direction.

However, two rigid moment frames located on opposite ends of the building are used to resist lateral wind forces in the north-south direction, in order to create an open floor plan for office and retail space.

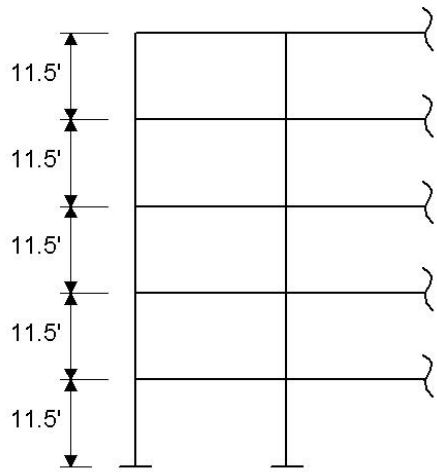


Figure 1: Building Dimensions (Profile View)

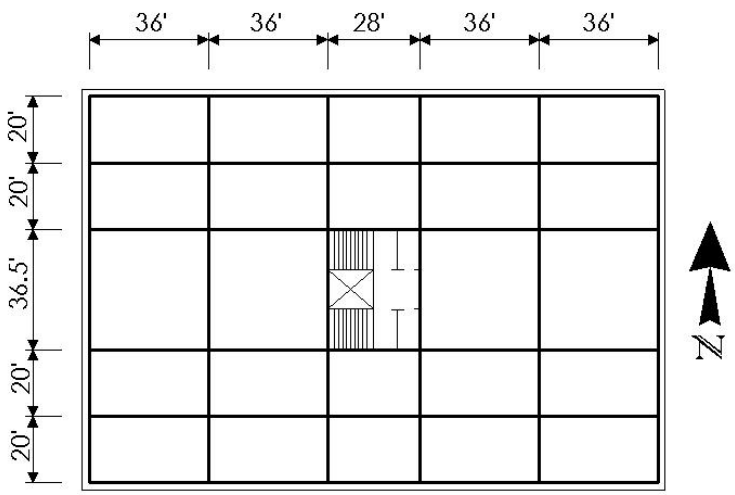


Figure 2: Building Dimensions (Plan View)

Due to the 5x5 bay layout, 50 moment resisting connections are required for each of the moment frames. By placing the two moment frames in the exterior walls of the building, deeper beams can be used due to the lack of HVAC space needed between floors.

The roof of the structure is constructed with 5-ply felt and gravel, 2" rigid insulation, and 1.5" x 22 gage steel roof deck, supported by open web steel joists. The floors are constructed with 2" x 20 gage composite steel floor deck with 3.25" thick lightweight concrete slab. Finished flooring consists of carpeting in most areas, with the exception of restrooms, which are covered in ceramic tile. The ceilings are suspended acoustical tile. Exterior walls are 8" lightweight concrete block covered with 4 feet brick veneer. The exterior walls total 4 feet in height in each story to allow space for windows. The structural steel for the beams and columns is A992 steel, whereas the steel used for connection members is A36 steel. All beams and columns are AISC wide-flanged sections. The welds for the moment resisting connections are E70XX.

2.2 Loads & Load Combinations

In designing the building and the moment frame, load factors and load combinations are taken from the 3rd edition of the AISC Manual of Steel Construction for Load and Resistance Factor Design (LRFD). The Manual of Steel Construction (2001) establishes six load combinations that encompass seven different loads (dead, live, roof live, snow, rain/ice, wind and earthquake loads). These combinations consider that only one of the loads will reach its maximum lifetime value, which is based on a 50-year

recurrence. The other loads that are present in a particular combination will only reach their "arbitrary-point-in-time" values. Since the primary purpose of moment frames is to withstand lateral loads, the strength of the moment frame is tested against the three load combinations listed below, where D, L, W and L_r represent dead, live, wind, and roof live loads, respectively:

$$1.4\,D \tag{1}$$

$$1.2D + 1.6L + 0.5L_r \tag{2}$$

$$1.2D + 1.6W + 0.5L + 0.5L_r \tag{3}$$

Of the gravity loads that are used to verify moment frame strength, the dead loads (*D*) are determined using weight estimations based on material manufacturing data. For this study, these estimations are taken from the Standard Building Code (1994), and are provided in Table B1. A dead load of 25 psf (lb/ft^2) is used for the roof load (which consists of roof covering, insulation, steel decking, roof joists, mechanical and electrical, and ceiling material) and a dead load of 60 psf is used for the floor load (consisting of floor coverings, concrete slab and steel decking, mechanical ducts, electrical and ceiling material). Also, a dead load of 356 lb/ft is applied to each moment frame because of the brick veneer, concrete block and glass on the exterior walls.

The value for the roof live load (L_r) is taken from section 1607.11.2.1 of the North Carolina State Building Code (2002), which states that the live load for flat or pitched roofs should be determined from the following equation:

$$L_r = 20R_1R_2, \quad 12\ psf \leq L_r \leq 20\ psf \tag{4}$$

in which R_1 and R_2 represent reduction factors that are controlled by the tributary area (in ft^2) supported by any structural member and the slope of the roof, respectively. The maximum tributary area supported by any roof member is less than 200 ft^2, resulting in a reduction factor (R_1) equal to 1, and because of the flat roof, the slope is less than 4 inches of rise per foot, providing a reduction factor (R_2) of 1. Consequently, a roof live load of 20 psf is adopted.

The required floor live load (L) is location and use dependent, and is determined using Table 1607.1 of the North Carolina State Building Code (2002). This information is also provided in Table B2. The table states that a minimum of 50 psf should be used for offices, 80 psf for corridors above the first floor, and 100 psf for lobbies and first floor corridors, respectively. The building code also provides a note regarding live loads for movable partitions. Section 1607.5 states that in office buildings where partition locations are subject to change, provisions for partition weight should be made unless the selected live load is greater than 80 psf. If the live load is less than the required 80 psf, a uniformly distributed live load of 20 psf should be added. Based on this information, a minimum of 70 psf could be selected for the live load. However, a conservative value of 100 psf is chosen because the locations of movable partitions and the actual use for the space are unknown.

The North Carolina State Building Code (2002) is also consulted when determining lateral loads for the frame, specifically wind loads (*W*). According to Figure 1609 of this code (also Figure B1 in Appendix B), the basic design wind speed for Wake County is approximately 100 mph. For a building 57.5 feet high located in Exposure Category B, a height and exposure adjustment coefficient of 1.22 is selected from Table B3, which references Table 1609.6.2.1(4) of the North Carolina State Building Code (2002). This office building also has an importance rating of Category I, as defined in Table B4. This rating simply includes all other structures not assigned to Categories II - IV. Category II structures are those that represent a significant hazard to human life in the event of failure including large schools, colleges, health care facilities, jails, and power-generating stations. Category III structures include those designated as essential facilities including fire, rescue and police stations, hospitals that provide emergency treatment and surgery, water treatment facilities, etc. Buildings represented by Category IV are structures that represent a low hazard to human life in the event of a failure such as temporary facilities or small storage facilities (North Carolina State Building Code, 2002).

As a result of the Category I rating, a wind importance factor (I_w) of 1 is selected from Table B4. The height coefficient and the wind importance factor are used to adjust the tabulated values found in Table B5, which lists the main wind force-resisting system loads as seen in Table 1609.6.2.1(1) of the North Carolina State Building Code (2002). With a transverse load direction, a flat roof, and a 100 mph basic design wind speed, the horizontal loads are given as 15.9 psf for the wall end zones and 10.5 for the wall interior

zones. The end zones are calculated to be 23.3 ft in length, creating an interior zone of 125.4 ft, and are defined in Figure B2. The resulting lateral wind load on the exterior of the building in the north-south direction is 11.96 psf (unadjusted). Since the lateral loads are distributed evenly among rigid floor systems, the adjusted concentrated lateral forces applied to the moment frame are illustrated in Figure 3.

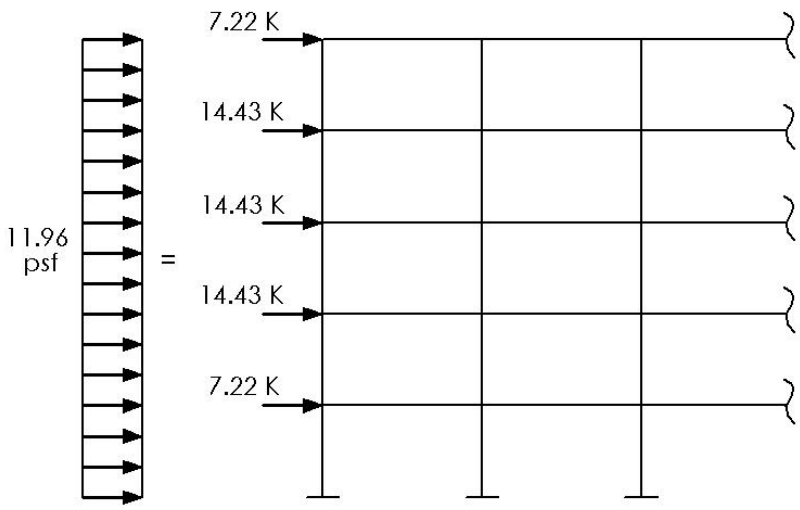


Figure 3: Wind Load Distribution

2.3 Strength & Displacement Calculations

When considering the moment frame, both the beams and the columns experience a combination of axial and bending loads due to the wind loading. Therefore, the strength of each member is evaluated by determining its interaction value, as provided in chapter H of the Manual of Steel Construction (2001):

For $P_u/\Phi P_n \leq 0.2$,

$$P_u / \Phi P_n + (8/9)*[(M_{ux} / \Phi M_{nx}) + (M_{uy} / \Phi M_{ny})] \leq 1.0 \quad (5)$$

For $P_u/\Phi P_n > 0.2$,

$$P_u / 2\Phi P_n + [(M_{ux} / \Phi M_{nx}) + (M_{uy} / \Phi M_{ny})] \leq 1.0 \quad (6)$$

where P_u and P_n represent the required and nominal axial strength, respectively; and M_u and M_n the required and nominal flexural strength, respectively. Also, Φ represents the applicable resistance factor. Due to the fact that this exploratory study considers only a plane frame analysis, the use of the terms M_{uy} and M_{ny} is ignored. When calculating the nominal flexural strength, the modification factor for non-uniform moment is taken conservatively as 1.0, as allowed by section F1.2a in the Manual of Steel Construction (2001). Since the frame analysis program evaluates the strength of the moment frame using the matrix method of analysis and includes second order non-linear analysis for P-Δ effect, the required flexural strength is taken directly from the program, which also calculates the ultimate factored moment along each member of the frame.

The sway of the building is also an important limiting factor for moment frames, and displacement for each story is limited to $L/400$, where L represents the height of the story (in inches). Lastly, the members are evaluated for shear strength using the following equations:

For $h/t_w \leq 2.45 \sqrt{(E/F_{yw})}$,

$$\Phi V_n = \Phi(0.6)F_{yw}A_w \quad (7)$$

For $2.45\sqrt{(E/F_{yw})} < h/t_w \leq 3.07\sqrt{(E/F_{yw})}$,

$$\Phi V_n = \Phi(0.6)F_{yw}A_w*\{[2.45\sqrt{(E/F_{yw})}]/(h/t_w)\} \tag{8}$$

For $3.07\sqrt{(E/F_{yw})} < h/t_w \leq 260$,

$$\Phi V_n = \Phi A_w*[4.52E/(h/t_w)^2] \tag{9}$$

where V_n refers to the nominal shear strength, and $V_u \geq \Phi V_n$. V_u represents the required shear strength, F_{yw} represents the minimum yield stress of the web, A_w is equal to the area of the web (defined as the overall depth of the web multiplied by the web thickness t_w), E is the modulus of elasticity, and h is the clear distance between member flanges less the fillet or corner radius.

CHAPTER 3: MOMENT CONNECTION DESIGN

3.1 Introduction

The Manual of Steel Construction defines moment frames as "frames in which the members and joints are capable of resisting forces by flexure as well as along the axis of the members." These joints, or fully rigid moment connections, are said to possess sufficient rigidity to maintain the angles between intersecting members, or beams and columns. Even for those connections that are characterized as fully rigid connections, it is impractical to achieve zero rotation between members. However, the small amount of flexibility that is often present is neglected, and the connections are idealized as full restraints, allowing no relative rotation (Manual of Steel Construction, 2001). The major functions of these connections are to transfer the beam and floor loads to the columns and to maintain the lateral stability of the frame. The placement of moment frames, as well as the moment connections in the frame, is governed only by the need for symmetry within the structure, to prevent the possibility of torsion that may be present in the absence of symmetry.

In order to create a fully rigid connection, the flanges of the beam as well as the beam web, are welded to the flange of the supporting column, which typically involves on-site labor. It is required that the field welding be performed in the horizontal position and fillet welds are preferred over groove welds (Manual of Steel Construction, 2001). This chapter introduces several conventional moment connection designs, and gives a

detailed design for a Welded Flange Plate Connection, which is selected to develop an estimate of the cost model needed in the proposed optimization study.

3.2 Types of Moment Connections

Although several moment connection designs exist, we consider only four of the most commonly used designs in steel construction. These conventional connections are: (a) Welded Flange, (b) Welded Flange Plate, (c) Bolted Flange Plate and (d) Flange Tee-Stub Bolted. Each of these connection designs are discussed in greater detail below.

3.2.1 Welded Flange Connections

Figure 4 shows a typical Welded Flange connection in which the flange of the beam is welded directly to the outer surface of the supporting column. The beam is coped at the top and bottom of the web to allow for welds to be completed in the field.

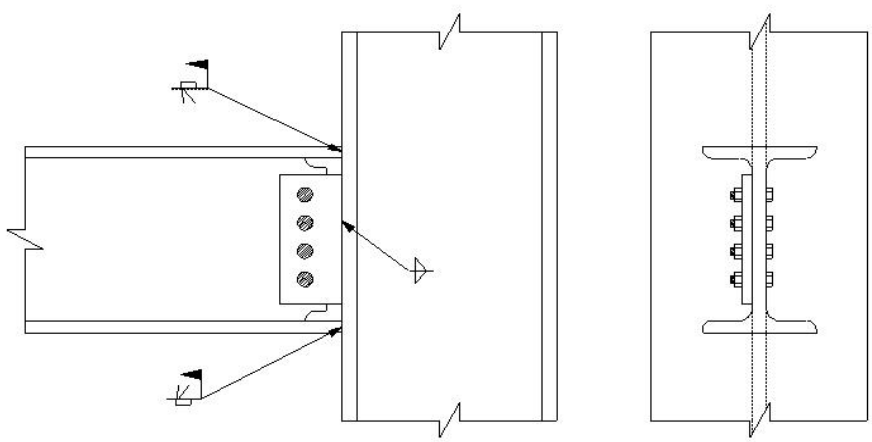


Figure 4: Welded Flange Connection

The web of the beam is bolted to a plate which has been previously welded to the column flange during fabrication. The welds connecting the flanges of the column and beam are full penetration welds, while the weld connecting the web plate to the column flange is a fillet weld.

3.2.2 Welded Flange Plate Connections

A typical Welded Flange Plate connection is shown in Figure 5, where the outer surfaces of the beam flanges are welded to plates, which are then welded to the column flange. The web of the beam is also bolted to a plate that has been previously welded to the column flange during fabrication. It is important to note that the top flange plate is narrower than the beam flange, while the bottom flange plate is wider than the beam flange, which facilitates the use of horizontal fillet welds in the field. The minimum shelf dimensions for fillet welds, which govern the width of the flange plate, are specified in Figure B3, which references Figure 8-12 of the Manual for Steel Construction (2001).

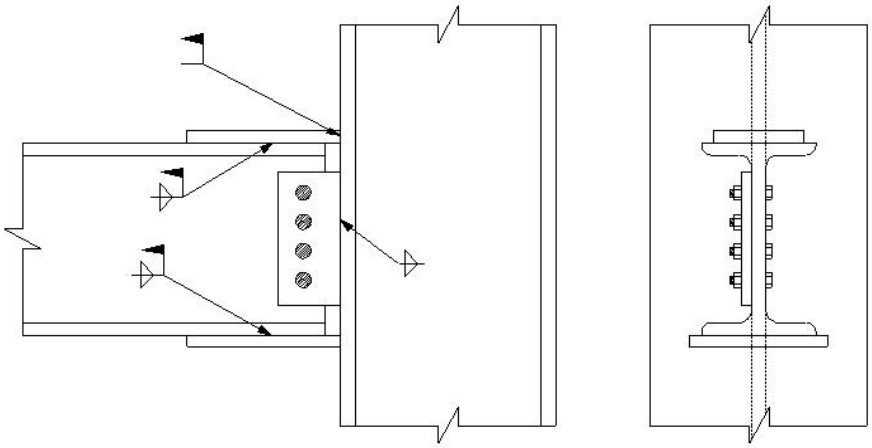


Figure 5: Welded Flange Plate Connection

This minimum shelf dimension is based on the fillet weld size, and must exceed 7/16”. A partial penetration weld is often used to connect the flange plates to the column flange, and a fillet weld is used to join the web plate to the column.

3.2.3 Bolted Flange Plate Connections

Another commonly used moment connection design is the Bolted Flange Plate connection which is shown in Figure 6. This connection is quite similar to the Welded Flange Plate connection and differs in the sense that the flange plates are bolted to the beam flange, but remain welded to the face of the column. The widths of the top and bottom flange plates of this connection are not regulated by minimum shelf dimension requirements. Instead, minimum width requirements are controlled by minimum bolt spacing and edge distances, which are specified in Table B7 and section J3.3 in the Manual of Steel Construction.

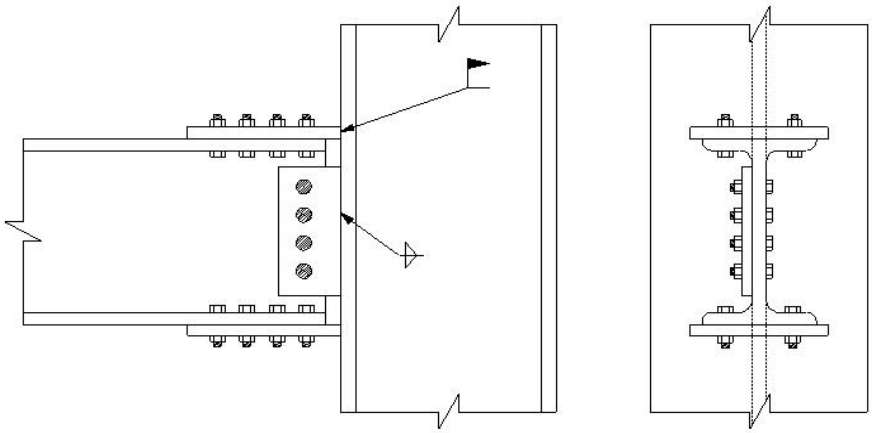


Figure 6: Bolted Flange Plate Connection

3.2.3 Flange Tee-Stub Bolted Connections

An example of the Flange Tee-Stub Bolted Connection is shown in Figure 7. For this connection, the flanges of the beam are connected to the column using tee-stubs such that the beam flanges are bolted to the web of the tee section, and the flange of the tee section is bolted to the column flange. Much like the other three connections discussed earlier in this chapter, the web of the beam is bolted to a plate that has been previously welded to the column flange during fabrication.

Among the four moment connections presented above, fabricators and erectors in North Carolina have conventionally used the Welded Flange and Welded Flange Plate connections. In order to determine an average fabrication and erection cost, the Welded Flange Plate connection is chosen and designed for the particular building considered in this study.

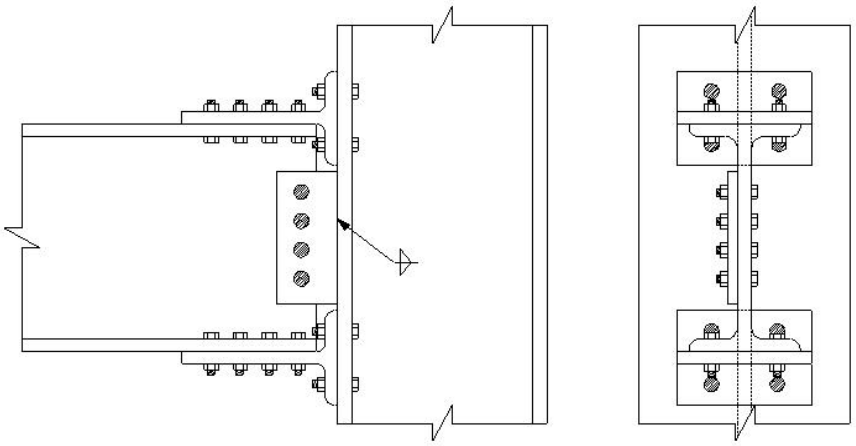


Figure 7: Flange Tee-Stub Bolted Connection

3.3 Design of Welded Flange Plate Connections

The design of fully rigid moment connections is a complex process that incorporates four different areas of consideration: (i) the design of top and bottom beam flange connections, (ii) the design of the beam web connection, (iii) the design of column stiffeners and (iv) the design of panel zone doubler plates. In order to understand the additional costs associated with rigid connection construction, it is necessary to first examine the rigid connection design process and how it varies from simple pin connection fabrication. Therefore, each of the four design aspects listed above are discussed in greater detail below (with respect to the Welded Flange Plate connection described earlier).

3.3.1 Design of Top and Bottom Beam Flange Connections

In beam flange connection design, it is important to identify the required dimensions of the flange plate and the size of the weld needed to join the flange plate to the beam. Once these values are established, each flange plate is evaluated for both the tension as well as the compression in order to account for reversible nature of lateral forces. The final stage of this process involves determining the size of the weld used to join the flange plates to the column. The specific tasks in this procedure can be enumerated as:

- The width of the top flange plate is narrower than the beam flange, and the width of the bottom flange plate is wider than the beam flange.

- Varying the width of the flange plate with respect to the beam flange creates a ledge or "shelf" on each side of the connection, permitting the use of horizontal fillet welds when joining the plate to the beam.
- The minimum shelf dimensions are dependent on the size of the fillet weld used to join the plate to the beam, and are provided in Figure B3.
- Typically, the cross-sectional areas of the top and bottom flange plates are comparable to each other.
- The minimum fillet weld size is provided in Table B8 of Appendix B, which references Table J2.4 of the Manual of Steel Construction (2001).
- The maximum weld size is defined as the thickness of the material (along the edges of material that is less than 1/4" in thickness), or 1/16" less than the thickness of the material (along the edges of material that is 1/4" or greater in thickness).
- The minimum length of weld to be used (ℓ_{min}) is calculated using the following equation:

$$\ell_{min} = P_{uf} / 1.392D \tag{10}$$

in which, P_{uf} represents the flange force and D corresponds to the size of the weld in sixteenths of an inch (for example, a 5/16" weld would result in a D value of 5).

- The flange force can be found using the following equation, where M_u represents ultimate moment capacity and d corresponds to the depth of the beam.

$$P_{uf} = M_u / d \tag{11}$$

- The flange force is also compared to nominal design values in order to ensure that the connection meets the design requirements. All nominal design values should exceed the maximum flange force experienced by the connection during loading.
- When considering tension, the plate should be evaluated for yielding (Equation 12) and rupture (Equation 13), as well as base metal strength (Equation 14). In these equations, F_y and F_u are the minimum yield stress and the minimum tensile strength of the type of steel being used, respectively. U represents the shear lag reduction coefficient used to calculate effective net area, and A_w refers to the area of the weld. A_g is the gross cross-sectional area of the plate.

$$\Phi R_n = \Phi F_y A_g \geq P_{uf} \tag{12}$$

$$\Phi R_n = \Phi F_u U A_g \geq P_{uf} \tag{13}$$

$$\Phi R_n = \Phi 0.6 F_u A_w \geq P_{uf} \tag{14}$$

- Welds should be verified for weld rupture (Equations 15 & 16), where F_{EXX} and A_w represent the electrode strength and area of weld, respectively. θ refers to the angle (in degrees) of loading measured from the weld longitudinal axis.

$$\Phi R_n = \Phi F_w A_w \geq P_{uf} \tag{15}$$

$$F_w = 0.6F_{EXX}(1+0.5\sin^{1.5}\theta) \tag{16}$$

- The beam flange should be checked for block shear (Equation 17). The variables A_{nt} and A_{nv} are net areas for tension and shear, respectively. The areas A_{gt} and A_{gv} are gross cross-sectional areas for tension and shear.

$$\Phi R_n = \Phi\{\max \begin{vmatrix} \text{Ten. Rupure.} + \\ \text{Shear Rupture} \end{vmatrix} \min \begin{vmatrix} \text{Opposite Yield} \\ \text{Opposite Rupture} \end{vmatrix} \} \geq P_{uf} \tag{17}$$

where the above are defined as:

$$\text{Tension Rupture} = F_u A_{nt}$$
$$\text{Shear Rupture} = 0.6F_u A_{nv}$$
$$\text{Tension Yield} = F_y A_{gt}$$
$$\text{Shear Yield} = 0.6F_y A_{gv}$$

- For flange plates under compression, checks are performed for local buckling (both stiffened and unstiffened elements - Equations 18 & 19) and flexural buckling (Equation 20), as well as shear yield (Equation 21) and

rupture (in the plate below or above the welds, depending on the load case – Equation 22).

$$b_f/t_p \leq 1.49\sqrt{(E/F_y)} \quad (18)$$

$$b/t_p \leq 0.56\sqrt{(E/F_y)} \quad (19)$$

$$\Phi R_n = \Phi F_{cr} A_g \geq P_{uf} \quad (20)$$

$$\Phi V_n = \Phi 0.6 F_y A_g \geq P_{uf} \quad (21)$$

$$\Phi V_n = \Phi 0.6 F_u A_g \geq P_{uf} \quad (22)$$

For the stiffened element, b_f is the distance (in inches) equal to the base of the beam flange. For the unstiffened element, b represents the length (in inches) of the flange plate that extends beyond the base of the beam flange. In both instances, t_p refers to the thickness of the plate. In Equation 20, F_{cr} is the critical stress associated with the plate ($f(kl/r)$) and A_g is the gross cross-sectional area of the plate.

- To determine the size of the fillet weld used to join the top and bottom flange plates to the column flange, the following equation is utilized:

$$D_{min} = P_{uf}/(2*1.5*1.392\ell) \quad (23)$$

The variable D_{min} once again refers to the weld thickness measured in sixteenths, P_{uf} corresponds to the flange force and the constants 2 and 1.5

represent a double sided weld and a load perpendicular to the weld, respectively. ℓ represents the length of the weld to be used.

- Due to the orientation of the weld, a maximum size for this weld is not specified. However, a minimum weld size can be found by referencing Table B8.

3.3.2 Design of Beam Web Connection

Design of the flange plates is followed by the development of beam web connection, which consists of a single plate that is welded to the column flange and bolted to the web of the beam. This process is described in greater detail below.

- To determine the geometry of the web plate, it is necessary to first calculate the number of bolts required for shear. The minimum number of bolts (n_{min}) can be found using Equation 24, where the shear experienced by the connection (V_u) is divided by the design strength of the selected bolt (Φr_n).

$$n_{min} = V_u / \Phi r_n \tag{24}$$

- The thickness of the plate is restricted by the diameter of the bolt, and cannot exceed a total thickness of 1/16" plus one half of the bolt diameter.
- The spacing between bolt holes is also dependent upon the diameter of the bolt used, and cannot be less than 2.67 times the nominal diameter of the bolt.

Distances of 3 times the nominal bolt diameter are preferred (Manual of Steel Construction, 2001).

- The minimum edge distances for bolts are listed in Table B7, which is taken from Table J3.4 of the Manual of Steel Construction (2001).
- The total width of the plate should exceed half of the distance between web toes of fillets at the top and bottom of the beam web (*T*), but should not surpass the total distance *T*.
- With dimensions established, the web plate is evaluated for shear yielding and rupture (Equations 21 & 22), block shear (Equation 17), and bearing and tear out (Equations 25, 26, & 27). In these equations, R_n represents the design strength (per bolt hole) for bearing and tear out, and is set equal to the smaller of the two values ($R_n = 1.2L_ctF_u \leq 2.4d_btF_u$). Furthermore, d_b refers to the diameter of the bolt, t represents the thickness of the plate, and L_c is the clear distance between two bolt holes or between one bolt hole and the edge of the plate.

$$2.4d_btF_u \text{ (bearing strength)} \tag{25}$$

$$1.2L_ctF_u \text{ (tear out strength)} \tag{26}$$

$$\Phi\sum Rn \geq P_{uf} \tag{27}$$

- A check is also performed for plate buckling, where the thickness of the plate should exceed 1/64 times the total width of the plate.

- The size of the weld used to join the web plate to the column flange should be greater than 75% of the plate thickness and must be greater than or equal to the minimum weld size given in Table B8.
- It is important to note that for all the bolt hole related limit states, with the exception of bearing and tear out, the effective hole diameter used in calculations is 1/16" greater than the actual hole diameter. For tear out, the actual hole diameter is used in calculations, while the bolt diameter is used for calculating bearing. Table B6 defines the nominal hole dimensions.

3.3.3 Design of Column Stiffeners

The design of actual moment connection is followed by evaluation for several column limit states. These include local flange bending, local web yielding and crippling, and web buckling, which are discussed in further detail in sections K1.2, K1.3, K1.4 and K1.6 of the Manual of Steel Construction. A failure related to these limit states, with the exception of web buckling, results in the use of half depth column stiffeners. A web buckling violation necessitates the use of full depth stiffeners. Next, we provide a detailed description of specific tasks needed in the method for sizing the column stiffeners when installation is required.

- The total stiffener area needed varies with the load type, either tension or compression, and can be calculated using the following equations:

$$A_{st} = (R_u - \Phi R_n)/0.9F_{ys} \text{ (tension)} \quad (28)$$

$$A_{st} = (R_u - \Phi R_n)/0.85F_{ys} \text{ (compression)} \quad (29)$$

in which R_u and R_n are the required and nominal strengths, respectively, F_{ys} is the minimum yield stress of the stiffener material, and Φ is the resistance factor.

- The restrictions for the dimensions of the width or breadth (b_s) and thickness (t_s) of the stiffener plate (Figures 8 & 9), are:

$$b_s + (t_{wc}/2) \geq (b_p/3) \quad (30)$$

$$t_s \geq (t_p/2) \quad (31)$$

$$b_s/t_s \leq 0.56 \sqrt{(E/F_y)} \quad (32)$$

- In Equations 30, t_{wc} refers to the thickness of the column web, and b_p represents the breadth of the plate used in the moment connection. In situations where plates are not utilized, the breadth of the flange (b_f) is used.
- The corners of the stiffener must also be clipped to allow the stiffener to follow the contour of the column, which contains fillet welds where the web and flanges meet. A minimum of 3/4" should be removed in both directions.

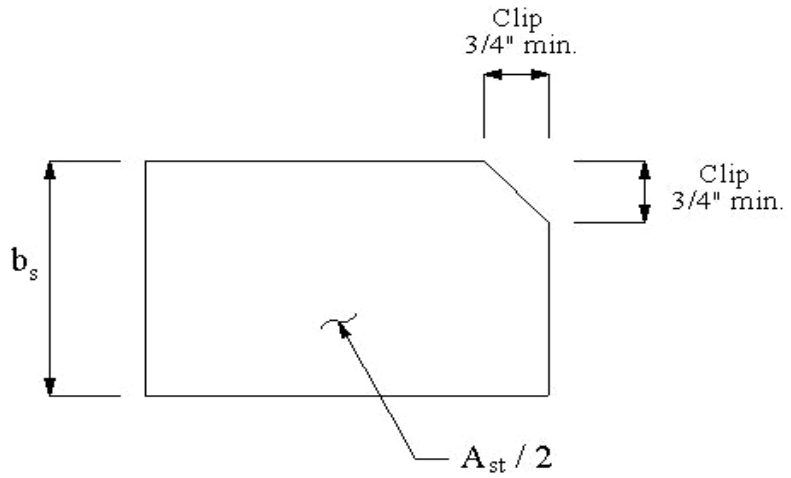


Figure 8: Transverse Column Stiffener Dimensions

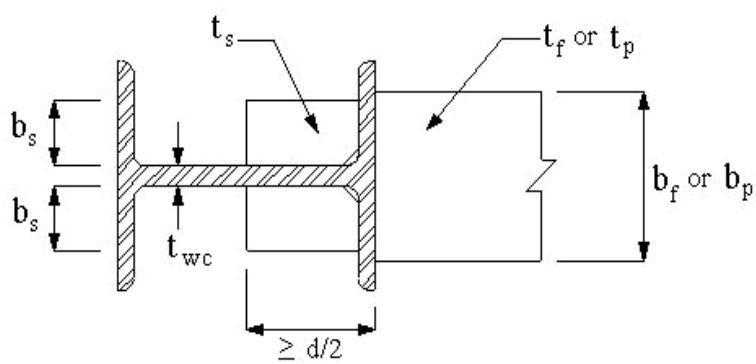


Figure 9: Transverse Column Stiffener

3.3.4 Design of Panel Zone Doubler Plates

One last consideration in the design of moment connections involves checking the column panel zone for yielding (Figure 10).

- The equation used for determining the total force acting on the panel zone is:

$$\sum F_u = [(M_{u1}/d_{m1}) \pm (P_{ub1}/2)] + [(M_{u2}/d_{m2}) \pm (P_{ub2}/2)] - V_{us} \leq 0.9R_v \quad (33)$$

in which The variables M_{u1} and M_{u2} refer to the moments experienced by the beams that are framed into the column at the connection, P_{ub1} and P_{ub2} refer to the beam axial loads, and V_{us} represents the story shear. The lengths d_{m1} and d_{m2} can be found by subtracting the thickness of the beam flange from the corresponding beam depth, which is approximated as 0.95 times the beam depth (Clean Columns, 2005).

- If the sum of forces exceeds the web shear strength (R_v), which is multiplied by the appropriate resistance factor ($\Phi = 0.9$), panel zone doubler plates are required (Figure 10).
- The shear acting on the doubler plate (V_u) then becomes equal to the difference between $\sum F_u$ and ΦR_v.
- A plate thickness (t_p) should be selected so that $Vu \leq 0.9Vn$, where for shear yielding,

$$V_n = 0.6F_{yp}ht_p\,, \;\; h/t_p \leq 2.45\sqrt{(E/F_y)} \quad (34)$$

and for inelastic buckling,

$$V_n = 0.6F_{yp}ht_p*\{[2.45\sqrt{(E/F_y)}]/(h/tp)\}, \;\; h/t_p > 2.45\sqrt{(E/F_y)} \quad (35)$$

in which F_{yp} corresponds to the minimum yield stress of the doubler plate, and F_y represents the minimum yield stress of the column material.

- The dimension h for the doubler plate is found by subtracting a total of twice the column flange thickness from the column depth. If the thickness required is greater than one inch, the use of two doubler plates is necessary (Clean Columns, 2005).

A numerical design example for Welded Flange Plate connections is provided in Appendix C. Understanding the total design of a moment connection is necessary for the evaluating fabrication and in-situ labor costs associated with its construction. These costs are needed in the optimization, which is discussed in the next chapter. Discussions were held with professional fabricators and design consultants on evaluating cost models. Several different options were examined. While an accurate way would be to design each connection in the frame at every stage of the optimization process and use a cost model that would be dependent upon the size of weld, necessity of doubler plate etc. However, such a process would be quite involved and computationally inefficient primarily because several hundreds and thousands of frames are analyzed before arriving at an optimal solution. Given the nature of this exploratory study, it was decided to maintain simplicity of implementation and use a fixed cost of constructing one moment connection. This cost is representative of the average fabrication and erection cost. Based on the input received from steel fabricators and consultants, the average moment connection cost was adopted as $900 per connection. The costs associated with simple pinned or bolted connections

were considered negligible. Furthermore, the cost of steel (for fabrication and materials) was approximated as $600 per ton of steel.

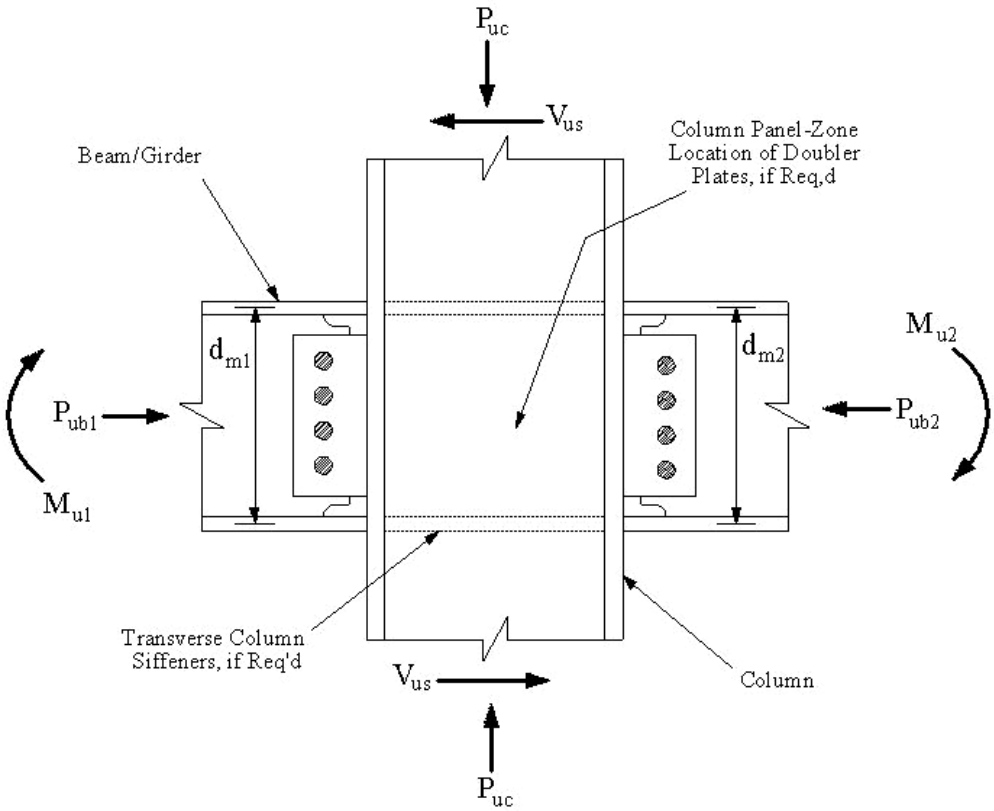


Figure 10: Column Panel Zone

CHAPTER 4: OPTIMIZATION OF MOMENT CONNECTIONS

4.1 Introduction

The majority of optimization studies performed in the past have focused on minimizing the weight of moment resisting steel frames and fail to consider the labor costs associated with fabricating and erecting moment connections. In recent years, the labor cost of in-situ welding and fabrication have contributed significantly to the total frame cost. The purpose of this study is to develop an optimization technique that considers both these costs, i.e. the cost of fabrication and labor associated with rigid connections and the hardware cost of the members. In doing so, the emphasis is not placed on finding lighter members to create a least-weight design, but on identifying the number and locations of moment connections that result in a design with the least overall cost.

In order to determine the number and location of rigid connections that produce the most economical frame design, a Genetic Algorithm-based optimization approach is proposed. During the search process, the Genetic Algorithm generates a number of unorthodox connection arrangements within a given moment frame, and in doing so, can produce locations that are very different from conventional practice. Those beam to column joints that are not assigned moment connections are then assigned to serve as standard pinned connections. Because the number and placement of connections varies between the various solutions generated within the optimization process, each frame is examined using finite element analysis and then evaluated for acceptability with respect

to strength and serviceability constraints. In the event that strength or serviceability violations occur because of a reduction in the total number of moment connections, those beams or columns associated with the violation are replaced with larger/heavier members. A detailed discussion of Genetic Algorithms and the optimization method used in this study is provided in this chapter.

4.2 GA-Based Discrete Optimization

Genetic Algorithms, or GA's, are population-based search algorithms that model the processes of evolution, most notably the concepts of natural selection and adaptation. Because they are population-based algorithms, GA's operate on a collection of potential solutions that are represented using strings, instead of attempting to improve upon a single/initial solution. The strings consist of a sequence of characters, usually numbers, which correspond to the values of distinct design variables of the problem. A string can consist of either binary or integer numbers depending upon the nature of formulation used to represent the optimization problem. It is important to note that a valuable characteristic of the GA is the ability to maintain population diversity during the search process in order to avoid local optimal solutions in the pursuit of finding a global (true) optimal solution.

A measure of the performance of each solution (or each string) is obtained in terms of its *fitness* which is evaluated using a fitness function. The fitness function is a collection of an objective function and some penalty functions (Recent Advances in

Optimal Structural Design, 2002). In structural design, the fitness function typically includes: (a) the total cost of the structure, (b) stress constraints and (c) displacement constraints. Typically, problem-specific constraints are modeled as penalty functions in the fitness function. The total penalty applied to a particular solution is related to the degree by which it violates constraints. The optimization technique used in this study implements a corrective scheme, which does not enforce the application of penalties in the case of a constraint violation. Rather, penalty values are assigned only when the solutions with violations are deemed uncorrectable.

4.2.1 GA-Operators: Populating the GA and Selection

In order to determine an initial population for the GA to start the process, a pseudo random number generator is used to produce a series of solution strings. Then, once the fitness of each string is determined, the GA employs three different genetic operators (selection, crossover and mutation) to spawn the next generation of possible solutions (Recent Advances in Optimal Structural Design, 2002). Due to the fact that GA's apply "survival of the fittest" characteristics when determining an optimal solution, those strings having a higher fitness value receive a greater probability of surviving into the next generation, while those with poor fitness have little chance of being carried through subsequent generations. As later generations improve upon those with high fitness values, and eliminate those with lower fitness values, an optimal solution is eventually discovered. This technique is handled by the selection operation, which allows information stored in solutions with good fitness values to survive into the next

generation. While the selection operation does not change the features of the strings, it provides each string with a varying probability of becoming a parent string. Then, the next generation of solutions is created using selected pairs of parent strings and processing them through the crossover and mutation genetic operators.

4.2.2 GA-Operators: Crossover

The second GA-operator, crossover, allows for a pair of parent strings to spawn two offspring by mixing and matching their desirable traits through a random process. Here, traditional one-point crossover is illustrated as an example. During this method, a single parent string is broken into two segments, and one of these two segments is exchanged with the corresponding segment of the other selected parent string. The location and length of the segments to be switched are chosen at random. A simple example of one-point crossover is provided in Figure 11 in which the division occurs after the fourth bit in a string of length seven bits.

PARENT 1:	0	0	1	1	1	0	1
PARENT 2:	0	1	0	1	0	1	0

OFFSPRING 1:	0	0	1	1	0	1	0
OFFSPRING 2:	0	1	0	1	1	0	1

Figure 11: One-Point Crossover in GA

4.2.3 GA-Operator: Mutation

Mutation is the third genetic operator used by the GA, and allows an occasional feature to be created and passed on to the offspring that may not have existed in either parent string. Introducing these new traits permits the search area to be expanded so that possibly unexplored regions are introduced into the search. The purpose is to maintain diversity for avoiding local optimals. This method mimics natural mutation, reading through the string of binary characters and changing them from 0 to 1 or from 1 to 0 if the random probability test is passed. The probability of mutation is typically very low, but helps to protect against the loss of important characteristics (or diversity) from generation to generation. A simple example of mutation is given in Figure 12. In this case, the probability of mutation is set to 0.003, i.e. only those bits (characters) assigned probability values lower than 0.003 are mutated.

OLD STRING	RANDOM PROBABILITY				NEW STRING
0011	0.571	0.303	0.107	0.001	0010
1011	0.714	0.161	0.002	0.259	1001

Figure 12: Mutation within the GA

4.3 Optimization for Moment Connections

The proposed approach has the following three unique features; (1) it uses a distinctive crossover technique to avoid the limitation in using conventional crossover scheme, (2) imposes symmetry on the moment frames produced and (3) implements corrective algorithms that replace the members which fail with respect to strength and

serviceability constraints. A binary string is used in this study, where a value of 1 represents the existence of a moment connection at that location and a value of 0 represents a pinned connection. Using a "roll-of-the-dice" process to seed the GA results in strings having a random mix of ones and zeros. As stated earlier in this thesis, the focus of our study is on identifying the trade-off between the total cost and the number of moment connections used. Evaluation of the trade-off curves requires generation of the optimal solution for a specified number of connections and repeating the process by varying the specified number of connections. Therefore, each GA run is seeded with all solutions having exactly the same number of moment connections as specified by the user.

Traditional crossover schemes like uniform crossover will have problems maintaining a specified number of ones (rigid connections) in each offspring. Even though the parent strings are assigned the requested number of connections, the offspring are not guaranteed to maintain that amount. Because of this dilemma, a unique uniform crossover scheme developed by Gupta, et. al. (2005), is implemented into the GA model. The new crossover scheme explicitly ensures that the number of connections in each of the offspring is the same as in either of the parent strings. The crossover works by comparing the values b_i that make up the two parent strings, where b_i represents the binary value at location i. At location i for which b_i is equal in both parent strings, each offspring is also assigned that value in the corresponding position i. Then, the remaining numbers of integers within the parent strings are randomly divided equally between the

two offspring (Gupta, 2005). An example of the new crossover technique is illustrated in the following figure.

PARENT 1:	0	1	0	1	1	1	0	0
PARENT 2:	1	0	0	1	0	1	1	0

OFFSPRING 1:	0	0	0	1	1	1	1	0
OFFSPRING 2:	1	1	0	1	0	1	0	0

Figure 13: Unique Crossover Scheme

Imposing symmetry throughout the frame is also an important feature of the GA used in this study. The two dimensional frame solutions are required to be symmetrical in order to provide the same amount of resistance against lateral loading applied in either of the two possible directions. This feature is also preferable in three dimensional models, because it prevents torsion from developing within the structure. Such a restriction would not be necessary if the frame were irregular in shape or if it were exposed to asymmetrical loading. Imposing symmetry also provides a significant reduction in the search space, making it easier for the GA to identify acceptable solutions.

The main purpose of the study is to include the cost of connections in the optimization of moment frames, so the cost function is set as follows:

$$C_T = 900(n_c) + 600(W_s) \tag{36}$$

where C_T, n_c, and W_s represent the total cost of the frame ($), the number of connections, and the weight of the steel (in tons), respectively. The $600 and $900 values are based on industry averages provided by steel fabricators and structural engineering firms located in the building area. After defining the cost function, the GA's optimization scheme is set to minimize the following objective function (Z):

$$Z = C_T + 100000P_I + 100000P_S \tag{37}$$

The variables P_I and P_S correspond to penalties added to the total cost based on violations in strength (interaction values) and serviceability (sway), respectively. These penalties are applicable only to solutions that violate the respective constraints, i.e., $P_I > 0$ only if the solution violates Equations 5 - 9 and $P_S > 0$ only if the sway in any floor of the frame exceeds the limit, $L/400$, where L is the story height. In a feasible solution, each member of a frame must conform to Equations 5 - 9 and also satisfy the story sway requirements. Hence, $P_I = P_S = 0$ for feasible solutions. For an infeasible solution, P_I equals the sum of the left-hand side of Equations 5 - 9 for each of the members of the frame that violate these equations. Similarly, P_S is the sum of the sway violations for each column member in the frame. In Equation 37, the multiplication factors are set very high (= 100,000) to ensure that infeasible solutions are eliminated from the population.

The GA incorporates analysis performed by a Java-based strength and serviceability program when determining the fitness of the solution (Figure 16). When evaluating a particular solution, the string information is processed and a corresponding

frame model is created (Figure 16). A matrix analysis is then performed for the predefined load cases to solve for member end forces and displacements, and shear and moment distributions. P-Δ effects are considered for both the entire frame and for the individual members (by way of the geometric stiffness matrix). The interaction value for each member is computed using the results from matrix method analysis. If violations occur based on this strength condition, the members are replaced with acceptable AISC wide-flanged steel sections from a database. This selection is performed by opting for the column or beam with the next highest moment of inertia in the list. The database for replacement columns is limited to wide-flanged beams 14" in depth. A check is also performed for failure due to shear. This algorithm is illustrated using a flowchart in Figure 14.

After replacing the faulty members, the frame is then re-evaluated, and floor displacements are calculated. When sway violations are present, the program uses the algorithm given in Figure 15 to correct the solution. Suitable members are once again selected from a database of AISC wide-flanged steel sections, which are arranged in order of increasing moment of inertia. Since the floor displacement is proportional to the bending stiffness, all of the members of that specific floor – beams and columns, are replaced with new members such that the moment of inertia for each new member (I_{new}) satisfies the following condition,

$$I_{new} = I_{orig} \, (\delta_{act} / \delta_{allowed}) \tag{38}$$

where δ_{act} is the actual floor displacement, I_{orig} is the moment of inertia for the member being replaced and $\delta_{allowed} = L/400$, the permitted sway given the story height is L.

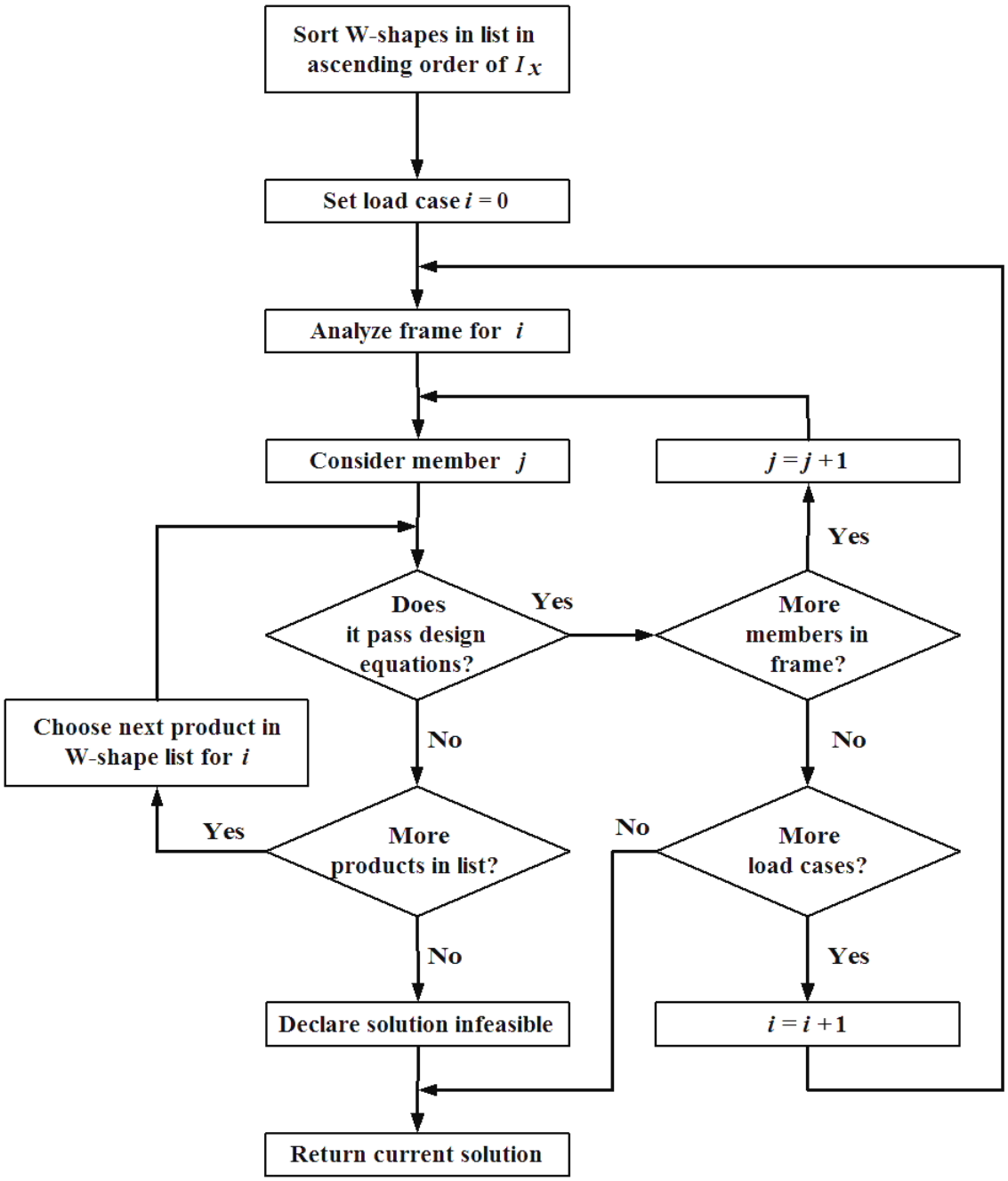


Figure 14: Algorithm for Correcting Solutions that Violate Design Equations

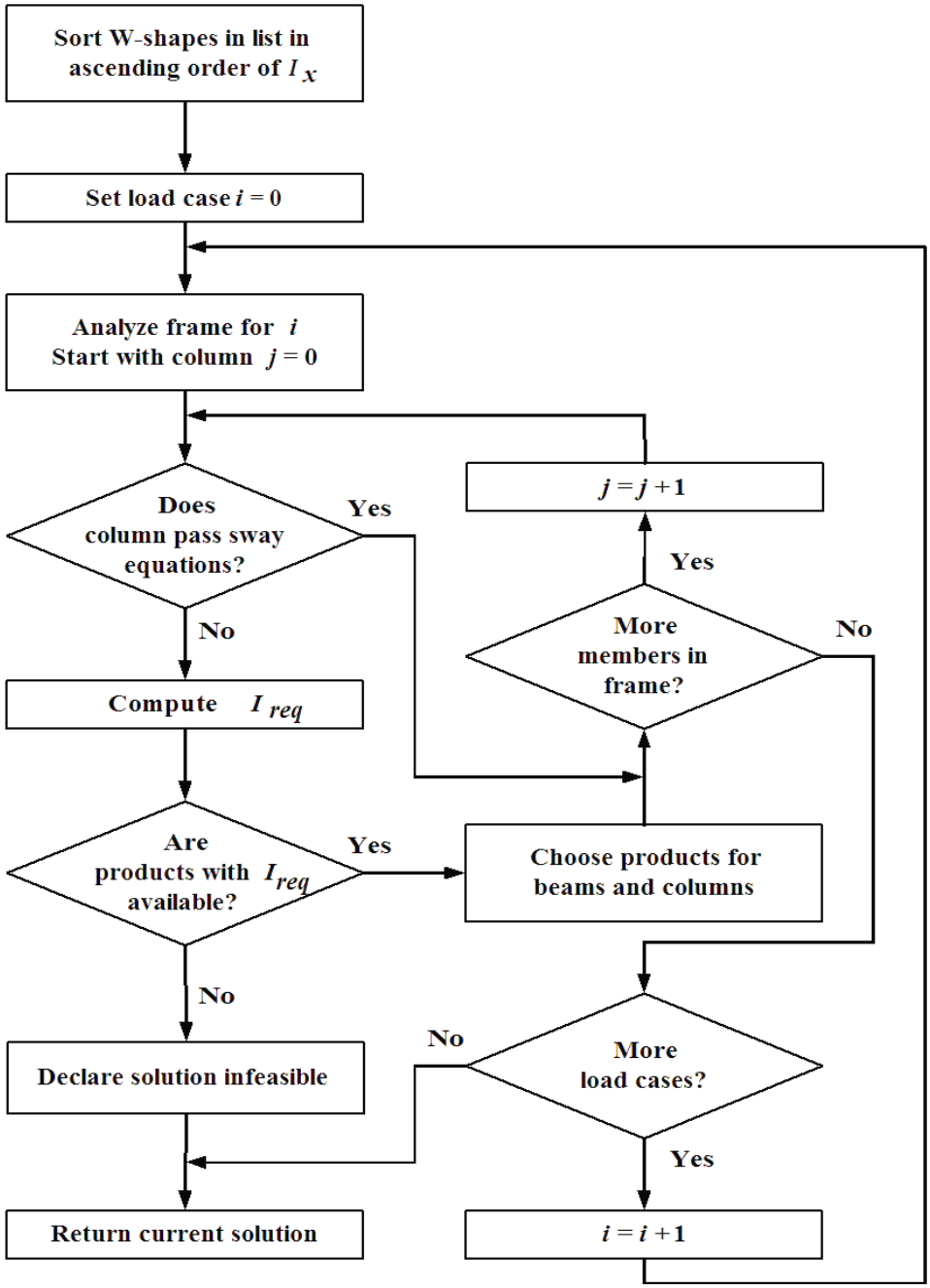


Figure 15: Algorithm for Correcting Solutions that Violate Sway Constraints

In an attempt to increase the total weight by the smallest amount possible, beam replacement members are first grouped according to their moment of inertia, and only the

lightest members are included in the database. The severity of the violation determines the order in which the floors are selected for replacement of members by the program, and the frame is re-examined after each phase. Information for the new solution is then sent back to the GA in order to determine fitness. If a solution can not be found where both of the limit states were met, penalties are applied to the objective function, decreasing the probability that the characteristics of the faulty frame are passed on to future generations. A flowchart of the GA and the fitness evaluation process are provided in the following figure.

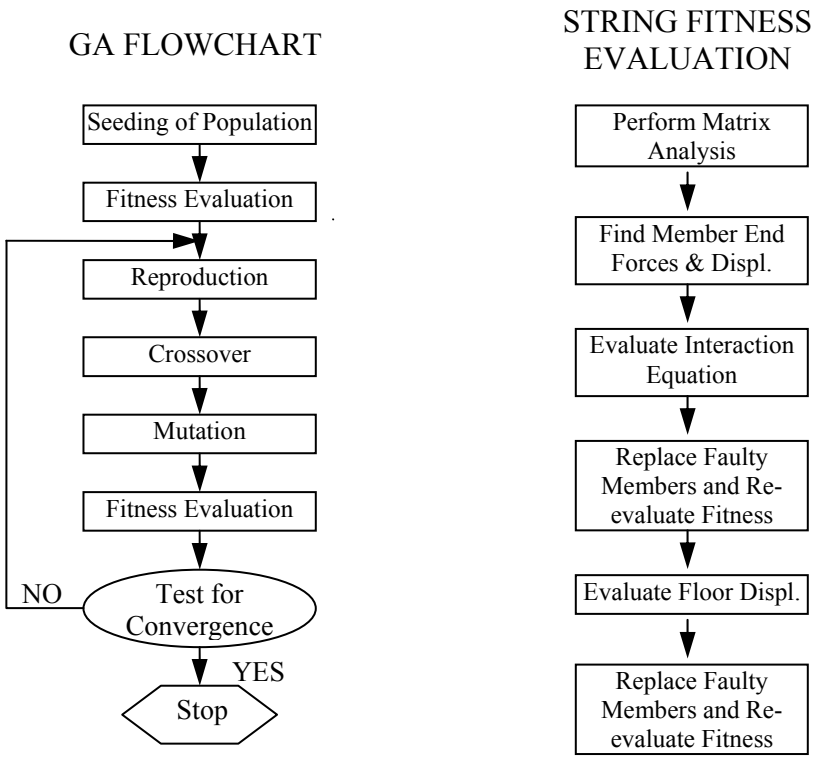


Figure 16: GA Flowchart

4.4 Validation of the Frame Analysis Program

As stated in previous sections, the GA within this optimization method incorporates analysis performed by a Java-based strength and serviceability program when evaluating the effectiveness of new solutions. This analysis program is based on the matrix method of determining member-end forces and displacements. As validation for the accuracy of this program, a comparison of analyses performed by both the Java-based program and SAP2000 is provided in Tables 1 and 2. This comparison includes two separate examples: (1) a simple two member frame provided in Figure 17 and (2) a 5-story moment frame subjected to lateral wind loads. While slight discrepancies exist, the frame analysis program sufficiently models the reaction of the frame to loading.

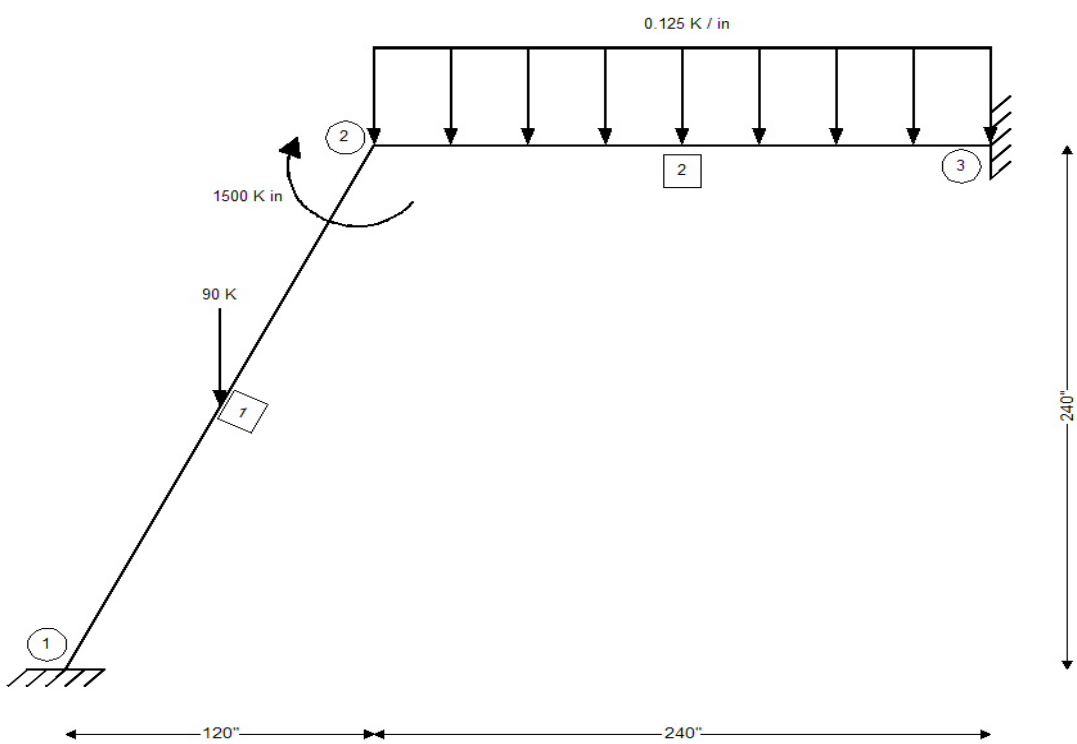


Figure 17: Two Member Frame Example

	NODE 2 OF TWO MEMBER FRAME EXAMPLE	
	FRAME ANALYSIS PROGRAM	**SAP2000**
D_X (in)	0.0213	0.0213
D_Y (in)	-0.0673	-0.0673
R_z (rad)	-0.0025	-0.0026

Table 1: Frame Analysis Program & SAP2000 Comparison

	D_X (in) OF FIVE STORY MOMENT FRAME (LATERAL LOADING ONLY)	
FLOOR	**FRAME ANALYSIS**	**SAP2000**
1	0.1920	0.1920
2	0.4830	0.4870
3	0.7790	0.7950
4	1.019	1.045
5	1.195	1.227

Table 2: Frame Analysis Program & SAP2000 Comparison for 5-Story Moment Frame

CHAPTER 5: RESULTS

5.1 Hand Optimized Moment Frame

The preliminary optimization attempt using a connection-dependent optimization scheme focuses solely on reducing the total number of moment connections, without altering the sizes of the members within the frame. The motivation behind this initial phase is two-fold: improving upon the model of a hand designed least-weight optimal solution and observing patterns associated with connection removal, so that connections more important to overall frame integrity could be identified. The subject frame is designed with moment connections at every beam to column intersection (50 total connections) in an attempt to reduce the total weight of the frame as much as possible. While placing rigid connections at each beam end causes a slight increase in the size of the beam (the beam is required to support greater moments), serviceability constraints related to floor displacements and sway typically control the design of a frame. Therefore, increasing the rigidity of the frame through the use of moment connections ultimately reduces the total weight of the frame. The hand-designed frame has a total weight of approximately 36 tons and costs an estimated $66,568.00 for materials and labor. The largest beam member is a W30x99, while the largest column member is a W14x211.

Results from this initial study prove that significant improvements can be made to the original hand optimized least weight design. Without increasing the size of the members, a nearly 50% reduction in the number of necessary moment connections is achieved, i.e.

the hand designed frame satisfies all the strength and serviceability requirements even if half the number of total possible rigid connections in the frame are eliminated Several solutions with totals ranging between 24 and 28 number of connections are found acceptable, passing both strength and serviceability requirements (Figures 18, 19, & 20). Most, if not all, of the solutions presented by the optimization scheme consisted of unorthodox connection placements, which allows for the discovery of more cost effective designs not considered by current practices. Some of the most notable connection layouts are the 24-connection “inverted arch” and “arch” solutions and the 26-connection “bow-tie” solution. A reduction from 50 to 24 connections results in a difference of $23,400.00, with the 24-connection frame costing an estimated $43,168.00.

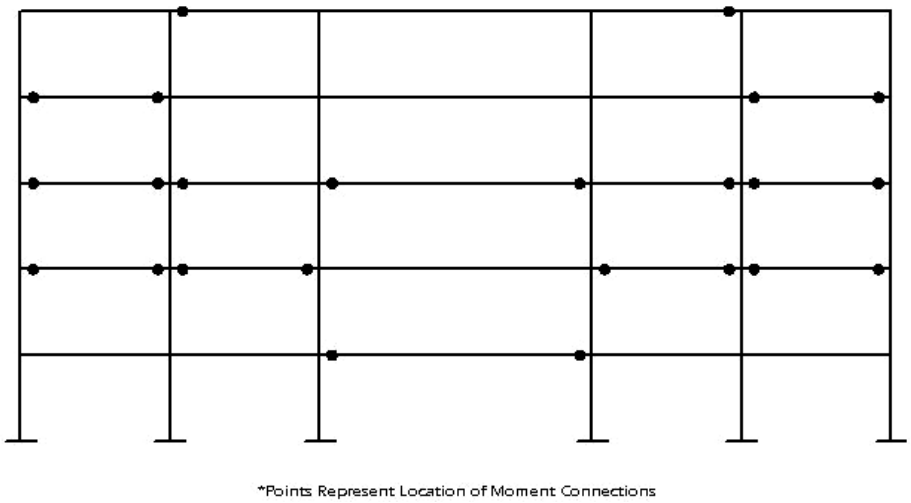


Figure 18: 24-Connection "Inverted Arch" Solution

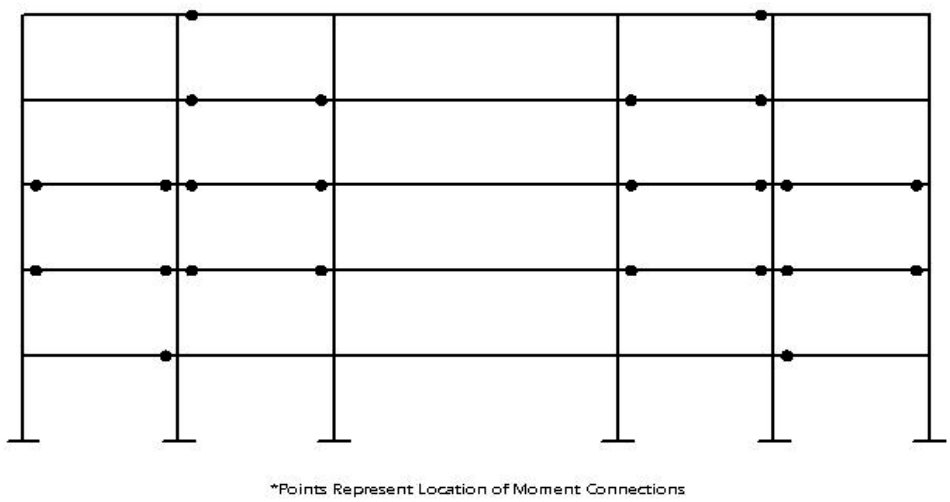


Figure 19: 24-Connection "Arch" Solution

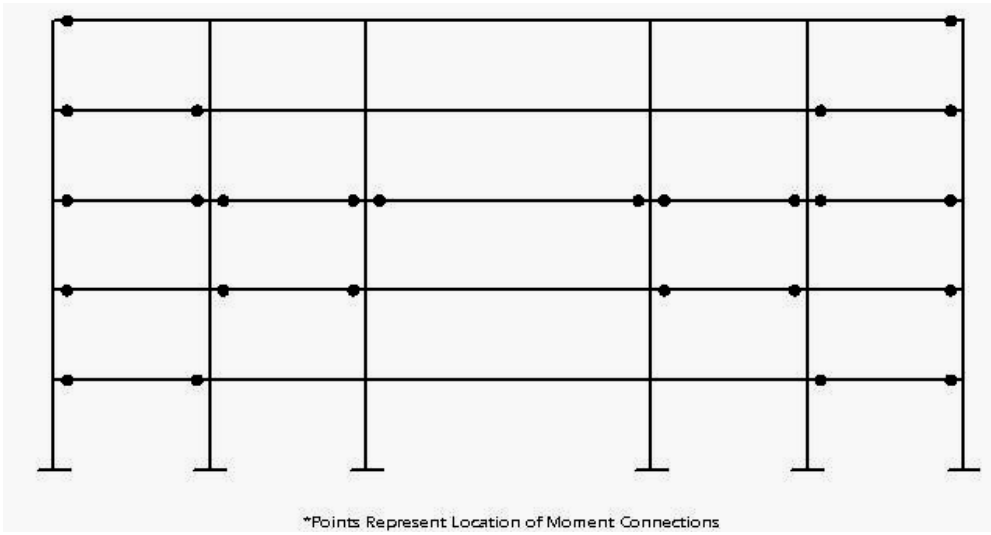


Figure 20: 26-Connection "Bow-Tie" Solution

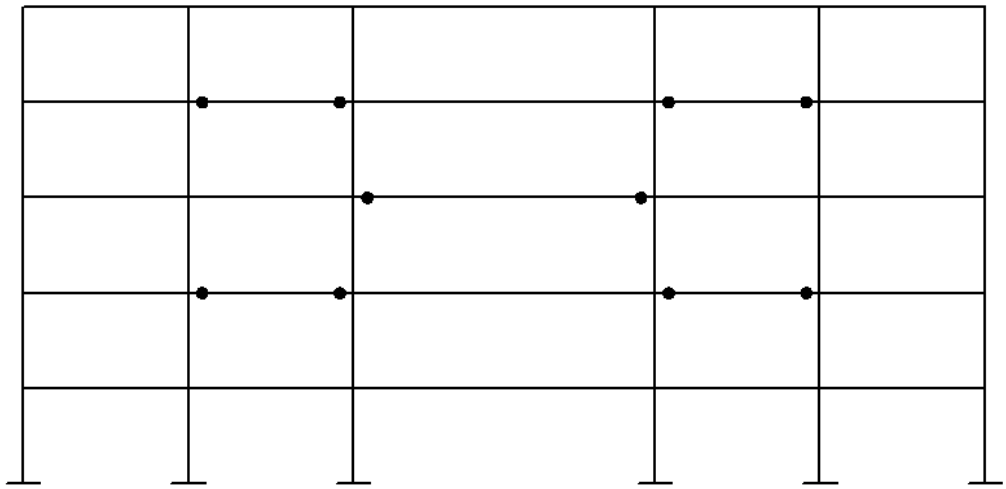
5.2 SAP 2000 Optimized Moment Frame

After evaluating the hand-designed moment frame results, a new emphasis is placed on creating an improved least-weight optimized model and the development of a member

replacement strategy that allows for further reduction in the number of moment connections within the frame. As mentioned previously, the use of moment connections at each beam to column intersection results in the lightest possible moment frame. While the hand-optimized frame provides a good starting point, it is considerably over-designed (as evidenced by the large reduction in moment connections before the occurrence of failure due to sway). Therefore, a least-weight design of this frame with all the connections as rigid connections will provide a more accurate comparison and differences between the least-weight optimization and connection-based optimization. SAP 2000 (version 7.0) is used to generate a new least-weight design using the function Auto Select Frame, a steel optimization method which implements a list of possible frame sections for each member. Unfortunately, this function is limited; only selecting the lightest section in an Auto Select group that satisfies all of the stress requirements.

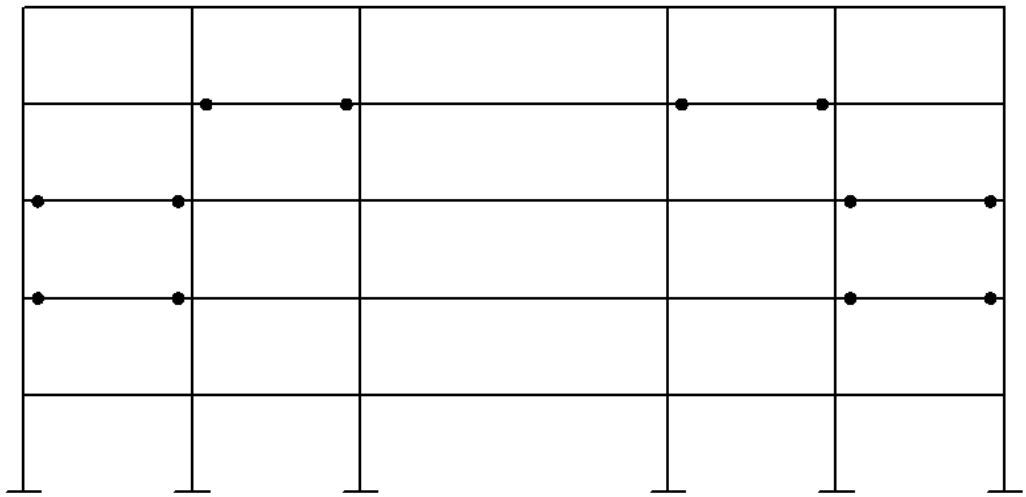
Due to SAP2000's inability to consider sway violations when developing a least-weight design model, hand calculations are performed in order to correct the SAP generated design. Just as the optimization program is prompted to replace entire stories with excessive displacements, each beam and column within a particular floor is substituted with suitable members from a database of AISC wide-flanged steel sections. The moment of inertia for the new replacement member is set to equal the moment of inertia for the failing member multiplied by the ratio between the actual floor displacement and the desired deflection limit. The end result was a much improved model, weighing approximately 22.2 tons and costing an estimated $58,318.65. This is clearly an improvement over the hand-designed frame that we started with initially.

The member replacement approach used to eliminate moment connections from the least-weight design shows that an even greater reduction can be achieved in the number of moment connections and in the total cost. Each possible connection total, between 2 and 48, is processed by the GA on three separate occasions using a population size of 100, producing the graph pictured in Figure 24. On several instances the results are duplicated by the GA, further establishing those values as close to true minimums. Through the exchange of sections, an optimal solution is found to contain a mere 12 rigid connections (Figure 22), weighing 31.679 tons and costing approximately $29,800. Only an additional 9.481 tons of steel (42.7% more than the original least-weight) is required to produce such a drastic reduction in overall cost. These results illustrate our original premise that with currently high costs of fabrication and labor, a majority of the focus should be placed on minimizing the quantity of connections used (or reducing the connection-related costs) instead of on developing methods centered around least-weight designs. Several acceptable alternative designs are found to contain 10 and 14 connections (Figures 21 & 23) with total costs of $30,159 and $31,366, well under the estimated $58,518.65 needed to construct the least-weight frame. These frames weigh 35.625 and 31.277 tons, respectively. Figure 25 further illustrates the ability to produce cost-effective moment frame designs without placing the primary focus on frame weight reduction. Figures 26 and 27 illustrate how the member sizes changed as moment connections were removed from the least-weight optimized frame.



*Points Represent Locations of Moment Connections

Figure 21: 10-Connection Solution



*Points Represent Locations of Moment Connections

Figure 22: 12-Connection Solution

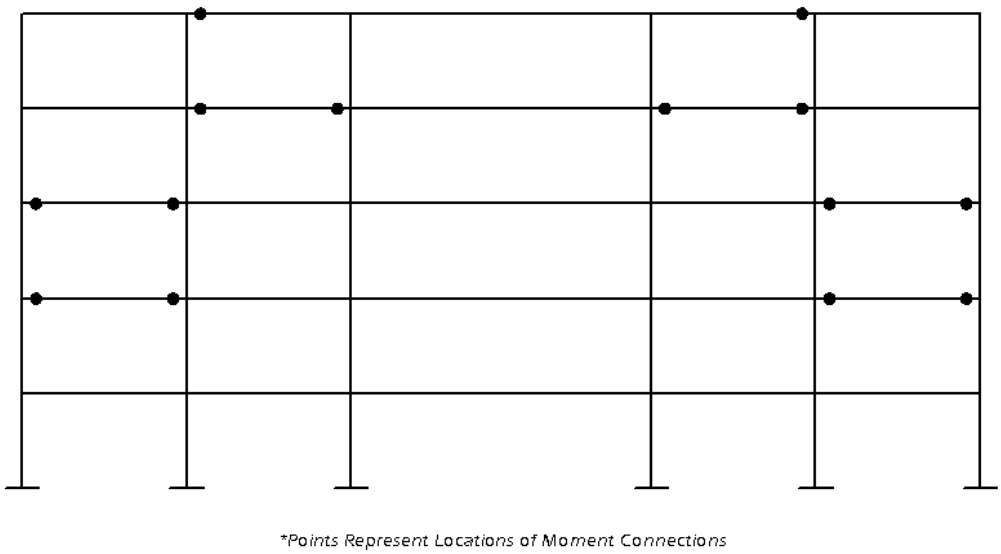


Figure 23: 14-Connection Solution

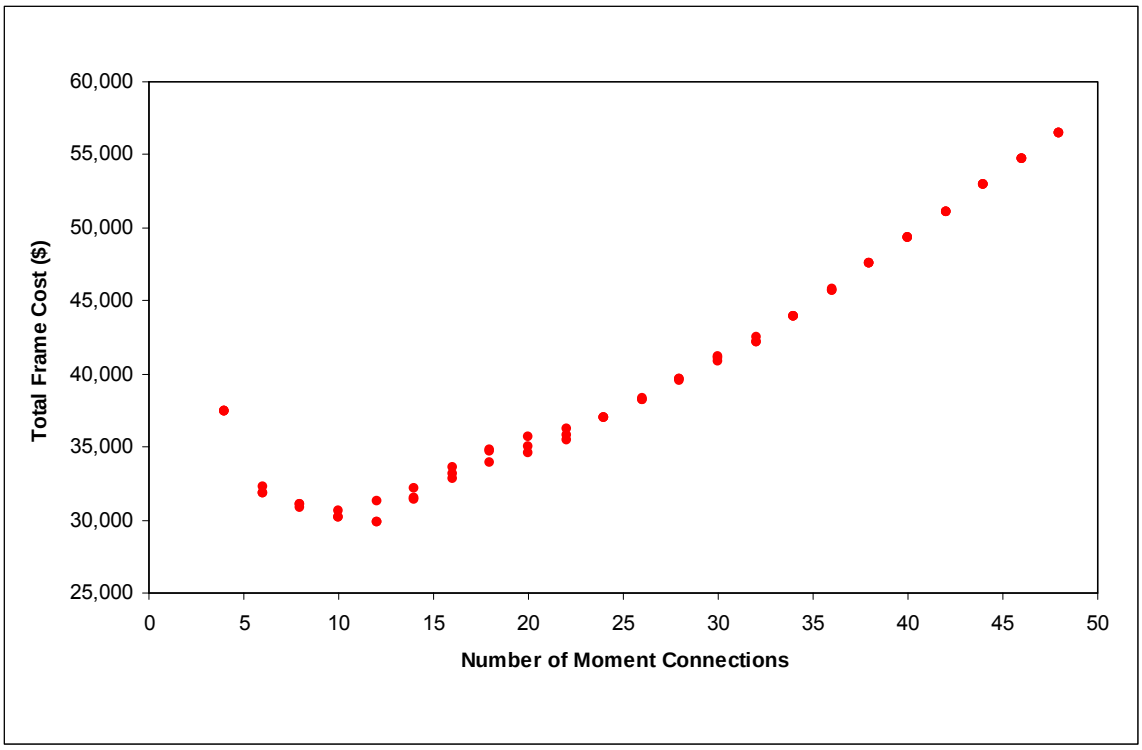


Figure 24: Graph of Connections vs. Total Frame Cost

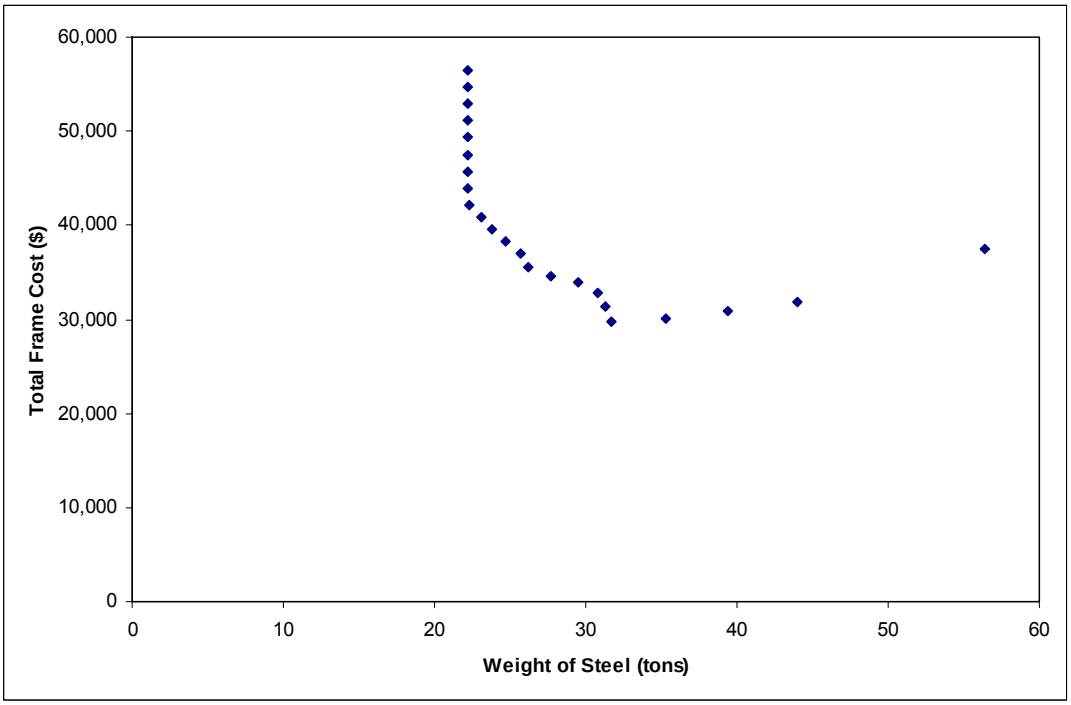


Figure 25: Graph of Total Weight vs. Total Frame Cost

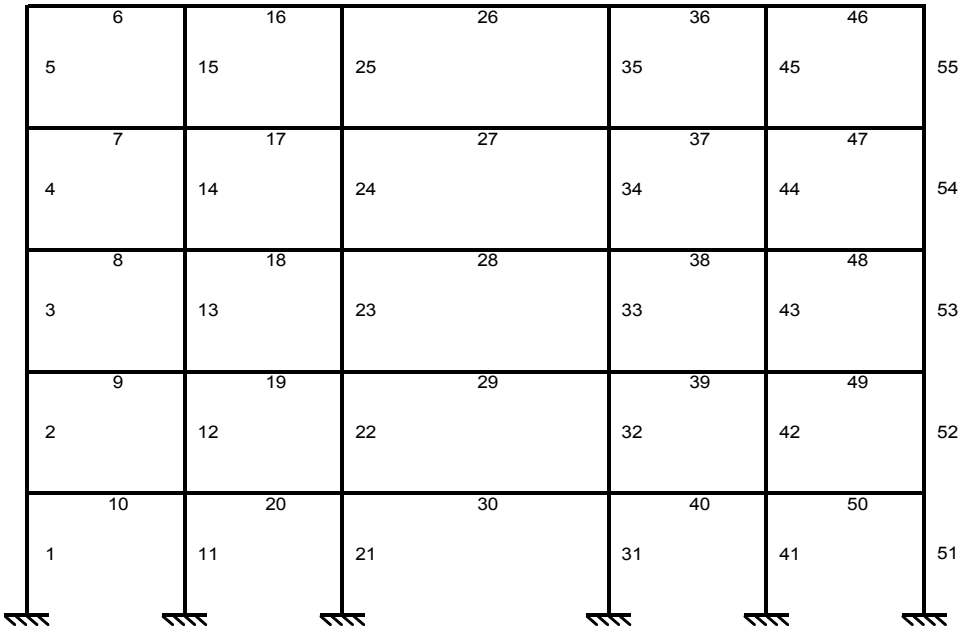


Figure 26: Member Number Designations for Five-Story Frame

Member Number	SAP 2000 Optimized	10 Connector Solution	12 Connector Solution	14 Connector Solution
1	W14x120	W14x132	W14x132	W14x132
2	W14x120	W14x257	W14x233	W14x233
3	W14x53	W14x120	W14x74	W14x74
4	W14x53	W14x74	W14x68	W14x68
5	W14x53	W14x53	W14x68	W14x61
6	W12x14	W12x14	W12x19	W12x16
7	W14x22	W18x35	W18x35	W18x35
8	W18x35	W24x62	W21x44	W21x44
9	W18x35	W24x62	W24x76	W24x76
10	W18x35	W18x40	W16x40	W16x40
11	W14x120	W14x132	W14x132	W14x132
12	W14x120	W14x257	W14x233	W14x233
13	W14x53	W14x120	W14x74	W14x74
14	W14x53	W14x74	W14x68	W14x68
15	W14x53	W14x53	W14x68	W14x61
16	W12x14	W12x14	W12x19	W12x16
17	W14x22	W18x40	W18x35	W16x31
18	W18x35	W24x62	W21x44	W21x44
19	W18x35	W24x84	W24x55	W24x55
20	W18x35	W18x40	W16x40	W16x40
21	W14x120	W14x132	W14x132	W14x132
22	W14x120	W14x257	W14x233	W14x233
23	W14x53	W14x132	W14x74	W14x74
24	W14x53	W14x74	W14x68	W14x68
25	W14x53	W14x53	W14x68	W14x61
26	W14x22	W14x22	W16x26	W14x26
27	W16x26	W18x40	W18x40	W18x40
28	W18x35	W24x62	W21x44	W21x44
29	W18x35	W24x62	W24x55	W24x55
30	W18x35	W18x40	W16x40	W16x40

Member Number	SAP 2000 Optimized	10 Connector Solution	12 Connector Solution	14 Connector Solution
31	W14x120	W14x132	W14x132	W14x132
32	W14x120	W14x257	W14x233	W14x233
33	W14x53	W14x132	W14x74	W14x74
34	W14x53	W14x74	W14x68	W14x68
35	W14x53	W14x53	W14x68	W14x61
36	W12x14	W12x14	W12x19	W12x16
37	W14x22	W18x40	W18x35	W16x31
38	W18x35	W24x62	W21x44	W21x44
39	W18x35	W24x84	W24x55	W24x55
40	W18x35	W18x40	W16x40	W16x40
41	W14x120	W14x132	W14x132	W14x132
42	W14x120	W14x257	W14x233	W14x233
43	W14x53	W14x120	W14x74	W14x74
44	W14x53	W14x74	W14x68	W14x68
45	W14x53	W14x53	W14x68	W14x61
46	W12x14	W12x14	W12x19	W12x16
47	W14x22	W18x35	W18x35	W18x35
48	W18x35	W24x62	W21x44	W21x44
49	W18x35	W24x62	W24x76	W24x76
50	W18x35	W18x40	W16x40	W16x40
51	W14x120	W14x132	W14x132	W14x132
52	W14x120	W14x257	W14x233	W14x233
53	W14x53	W14x120	W14x74	W14x74
54	W14x53	W14x74	W14x68	W14x68
55	W14x53	W14x53	W14x68	W14x61

Figure 27: Member Sizes for Various Solutions

The benefit of evaluating trade-off between the total cost and the number of connections is quite dependent upon the relative costs associated with the labor and the material. Consequently, we decided to conduct a sensitivity study by reevaluating the trade-off curves for different ratios of connection cost (labor and fabrication) to hardware cost (material cost per ton of steel).i.e.

$$CR = \text{Connection Cost} / \text{Hardware Cost} \tag{39}$$

Consequently, the trade-off curve presented earlier in Figure 24 is valid for only a value of CR = $900/$600 = 1.5. To evaluate the sensitivity of trade-off curves to the value of CR, we considered four different values of CR that are equal to 1.0, 1.25, 1.75, and 2.0 by varying the connection cost from $750 to $1000 per connection and by varying the material cost between $500 to $750 per ton of steel. The cost combinations and the corresponding values of cost ratios considered in the study are given in Table 3. Regardless of the cost ratio, the resulting frames all experience a substantial reduction in both the number of moment connections used and in the total cost of the frame. Each optimization series within this range produces an optimal solution with either 10 or 12 moment connections, and total cost savings of at least 38%. Figure 28 gives these trade-off curves.

Cost of Moment Connection ($)	Cost of Steel per Ton ($)	Cost Ratio	Least-Weight Design Cost ($)	Minimized Cost ($)	% Reduction in Total Cost
750.00	750.00	1	54,148.31	33,218.63	38.65
825.00	660.00	1.25	55,900.52	31,367.00	43.89
900.00	600.00	1.5	58,318.65	29,807.25	48.89
945.00	540.00	1.75	59,236.79	28,183.55	52.42
1,000.00	500.00	2	61,098.88	27,409.38	55.14

Table 3: Cost Ratio Comparison

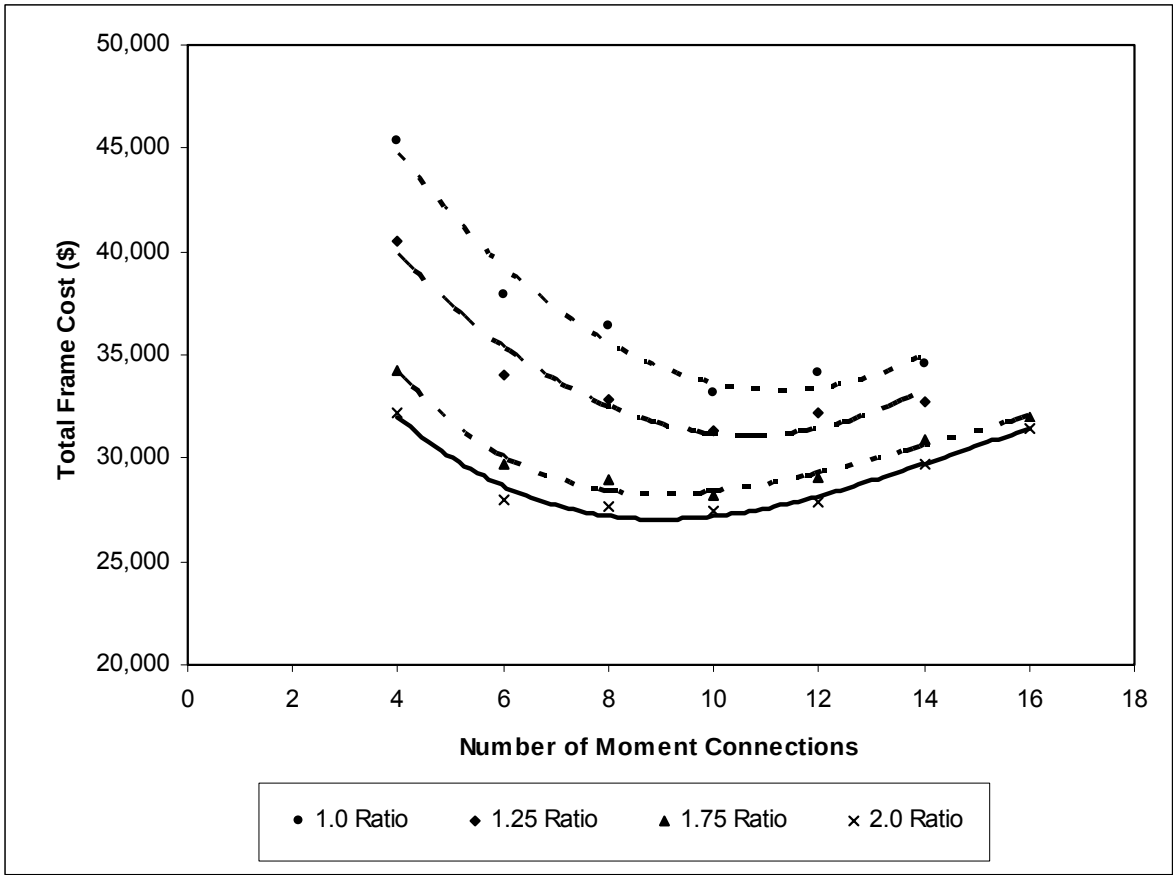


Figure 28: Comparison of Cost Ratios Graph

Upon review of the SAP-optimized frame solutions, an attempt was made to force the genetic algorithm to produce more practical moment frame designs. As observed in Figure 27, a number of the optimal solutions produced by the GA resulted in larger columns being supported by smaller columns. The algorithm was adjusted slightly, with limits set on the size of the supporting column. If a column was modified during the member replacement method described in chapter 5, those immediately beneath it would also be replaced with a member of the same size. With this limit enforced, the results varied slightly, but were still encouraging. For a cost ratio of 1.5 ($600 per ton of steel

and $900 per moment connection) the optimal solution contained only 10 moment connections, weighed approximately 35.710 tons and cost an estimated $30,425. Figure 29 illustrates the same general cost-saving trend as before, but with a small increase in total cost throughout. Figure 30 provides new trade-off cost curves, comparing the loss of moment connections to an increase in total frame weight. Figure 31 lists the member sizes of the optimal 10 connection solution, and also gives an example of how the adjusted GA produces a more realistic moment frame.

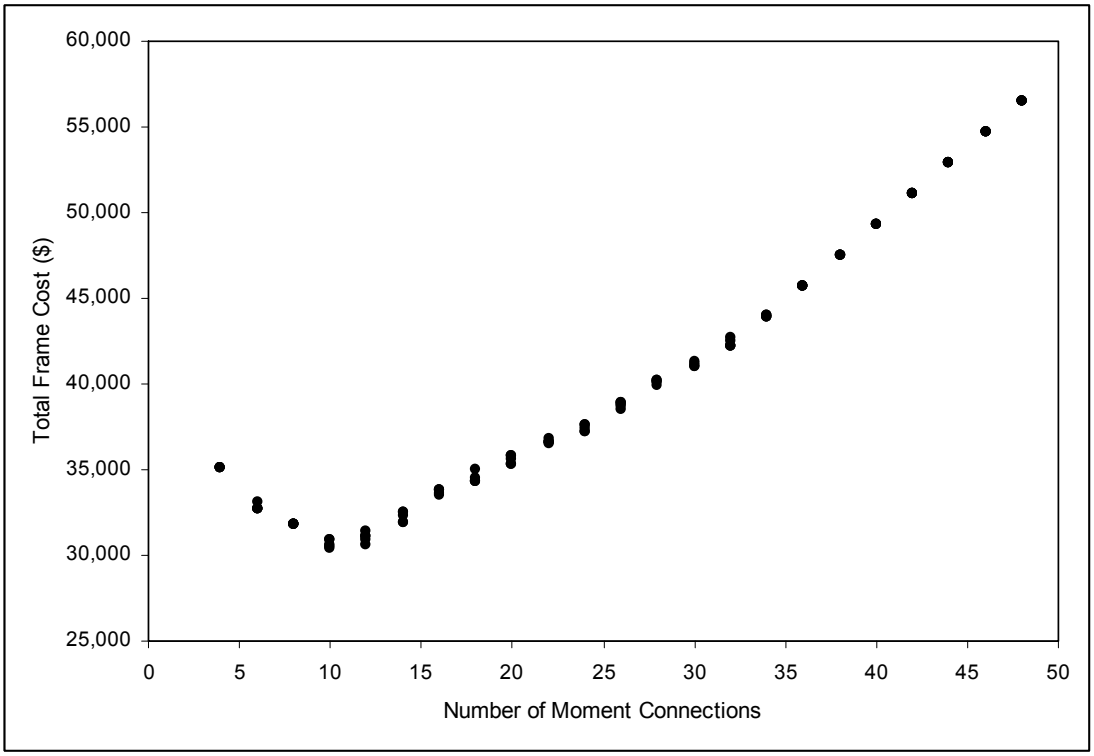


Figure 29: Graph of Connections vs. Total Frame Cost (Lower Column Correction)

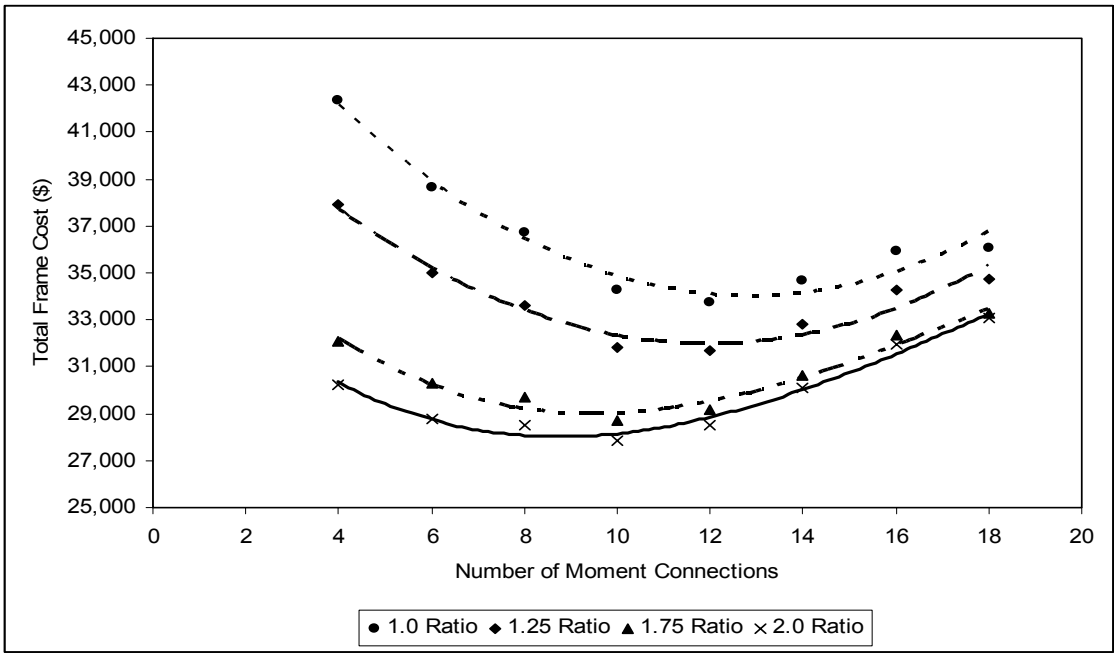


Figure 30: Comparison of Cost Ratios Graph (Lower Column Correction)

Member Number	SAP2000 Optimized	10 Connection Solution
1	W14x120	W14x193
2	W14x120	W14x193
3	W14x53	W14x99
4	W14x53	W14x99
5	W14x53	W14x90
6	W12x14	W12x22
7	W14x22	W21x44
8	W18x35	W21x55
9	W18x35	W21x44
10	W18x35	W18x35
11	W14x120	W14x193
12	W14x120	W14x193
13	W14x53	W14x99
14	W14x53	W14x99
15	W14x53	W14x90
16	W12x14	W12x22
17	W14x22	W18x40
18	W18x35	W21x55
19	W18x35	W24x68
20	W18x35	W18x35
21	W14x120	W14x193
22	W14x120	W14x193
23	W14x53	W14x120
24	W14x53	W14x99
25	W14x53	W14x90
26	W14x22	W16x31
27	W16x26	W21x44
28	W18x35	W21x55

Member Number	SAP2000 Optimized	10 Connection Solution
29	W18x35	W21x44
30	W18x35	W18x35
31	W14x120	W14x193
32	W14x120	W14x193
33	W14x53	W14x120
34	W14x53	W14x99
35	W14x53	W14x90
36	W12x14	W12x22
37	W14x22	W18x40
38	W18x35	W21x55
39	W18x35	W24x68
40	W18x35	W18x35
41	W14x120	W14x193
42	W14x120	W14x193
43	W14x53	W14x99
44	W14x53	W14x99
45	W14x53	W14x90
46	W12x14	W12x22
47	W14x22	W21x44
48	W18x35	W21x55
49	W18x35	W21x44
50	W18x35	W18x35
51	W14x120	W14x193
52	W14x120	W14x193
53	W14x53	W14x99
54	W14x53	W14x99
55	W14x53	W14x90

Figure 31: Optimal Solution Member Sizes

CHAPTER 6: CONCLUSION

6.1 Summary

A majority of optimization studies, past and present, have placed a strong emphasis on reducing the total weight of steel, without formally considering the effect of escalating labor costs associated with fabrication and erection of moment resisting rigid connections. Therefore, the objective of this study is to develop an optimization scheme that focuses on evaluating connection costs in addition to costs associated with frame weight. Instead of determining least-weight designs, optimal solutions are found by altering the number of moment connections within the frame; replacing some of the rigid connections within the frame with standard pinned connections, and increasing member sizes where needed. A Genetic Algorithm is used in order to identify the optimal number and location of connections within a frame, and each frame is evaluated using the matrix method of analysis to ensure that strength and serviceability requirements are met.

6.2 Conclusions

It is found that the consideration of labor costs associated with rigid connection construction is quite important in determining frame designs with least overall cost. In developing an optimization technique that considers both the number of moment connections and the total weight of the frame, an optimal solution can be found with significantly less cost than a least-weight frame with all the connections assumed rigid. For example, the number of moment connections needed in our 5-story frame reduces

considerably, i.e. to only 10 connections instead of 50 connections. In doing so, the weight of the frame increases by 13.512 tons, i.e. from 22.198 to 35.710 tons. The reduction in overall cost is almost 48%. Furthermore, a sensitivity analysis by varying the ratio of connection cost to material cost shows that the results do not change much and the contribution of connection cost remains significantly high.

The unorthodox connection arrangements produced are also important, as they provide alternative designs that, while functional, may never be explored by the designer. Regardless of connection arrangement, a majority of the frames failed due to sway violations rather than strength-related limit states. It is important to note that due to the technique used in the member replacement method, results produced by the GA consist of several different beam and column sizes throughout. In current practice, it is not practical to change the size of the column for each story. Therefore, as with most computer-based optimization or design programs, engineering judgment still plays and important role in developing a final product.

6.3 Future Work

Moment frame designs often include panel zone doubler plates at every beam to column connection. This is a very conservative approach which contributes to a significant increase in the total cost. Future work could include a more complex analysis that would identify the type of connection and bracing required for each joint, thereby creating a more realistic cost model. One approach would be to include four basic classes

of connections with varying cost amounts (instead of applying an average cost of $900 per connection): connections without bracing, connections with column web stiffeners, connections with panel zone stiffener plates, and connections with both forms of bracing. The inclusion of steel erection costs (in addition to the $600 material cost) would also improve the cost model.

Extending the use of this optimization technique to other structures and loading cases could provide more useful data for the eventual development of a new design method. While this study focused on a two-dimensional rectangular structure, this technique could be applied to both irregular structures and three-dimensional symmetrical structures to observe how changes in geometrical characteristics of the moment frame affect the entire structure. These modifications to the moment frame may also affect its ability to resist other lateral loading cases, especially during an earthquake.

Improvements on the current least-weight design model could also be made. By reviewing the information in Figures 24 & 25, one may observe that a total of 16 connections were removed before members were resized due to strength or serviceability failures. While this was a substantial improvement over the hand optimized frame, a structural optimization program (i.e. SAP2000) designed to consider both strength and serviceability requirements when developing least-weight solutions could ultimately provide a superior model for use with the optimization technique discussed in this paper.

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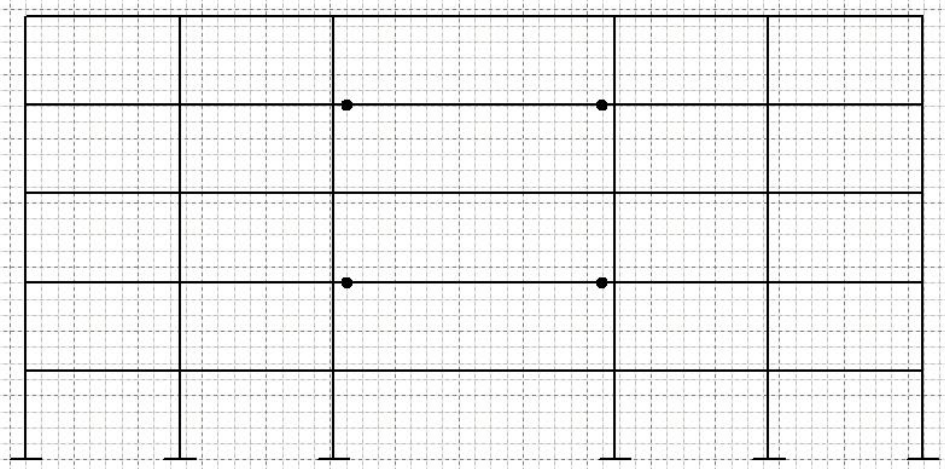
APPENDIX A :

ALTERNATIVE OPTIMAL SOLUTIONS

(Location of Moment Connections)

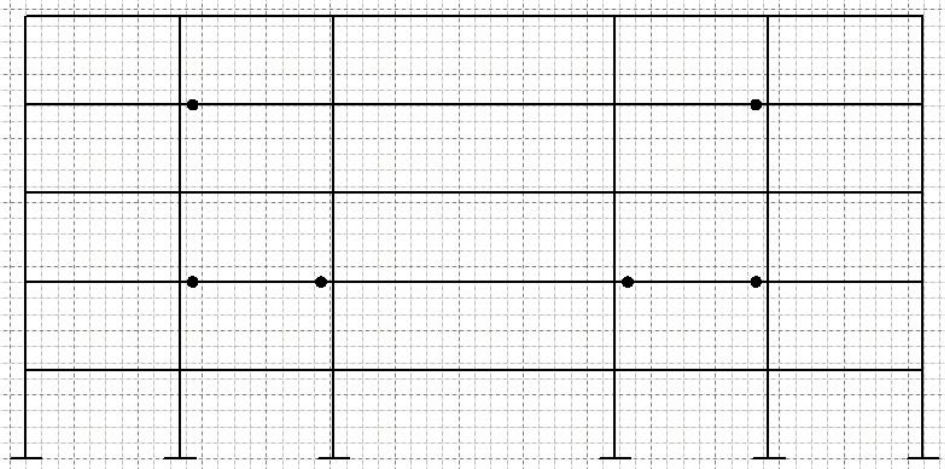
ORGANISM 4-1 (Moment Connection Design - 4 Total)



*Points Represent Locations of Moment Connections

total frame weight = 56.411 tons, total cost = $37,446.30

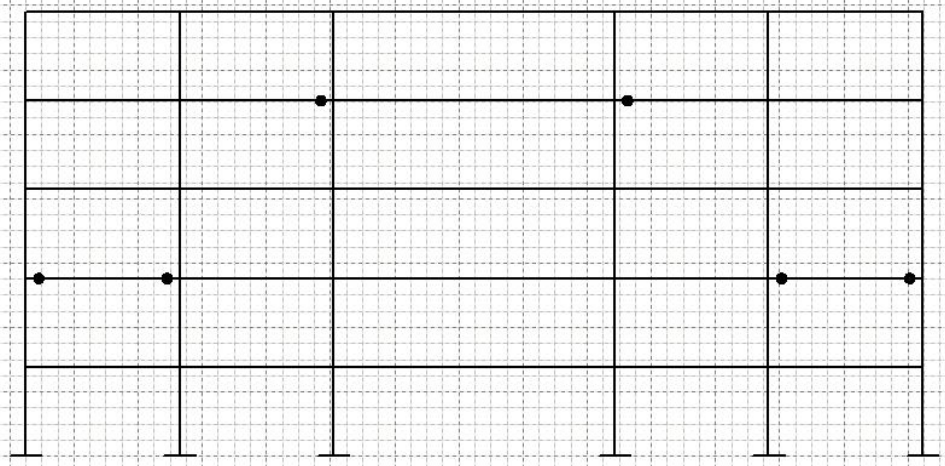
ORGANISM 6-1 (Moment Connection Design - 6 Total)



*Points Represent Locations of Moment Connections

total frame weight = 44.011 tons, total cost = $31,806.75

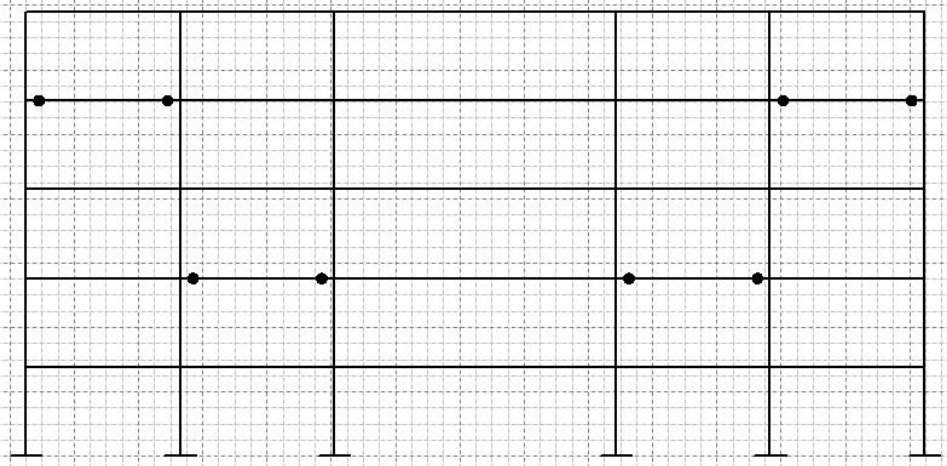
ORGANISM 6-2 (Moment Connection Design - 6 Total)



*Points Represent Locations of Moment Connections

total frame weight = 44.793 tons, total cost = $32,275.50

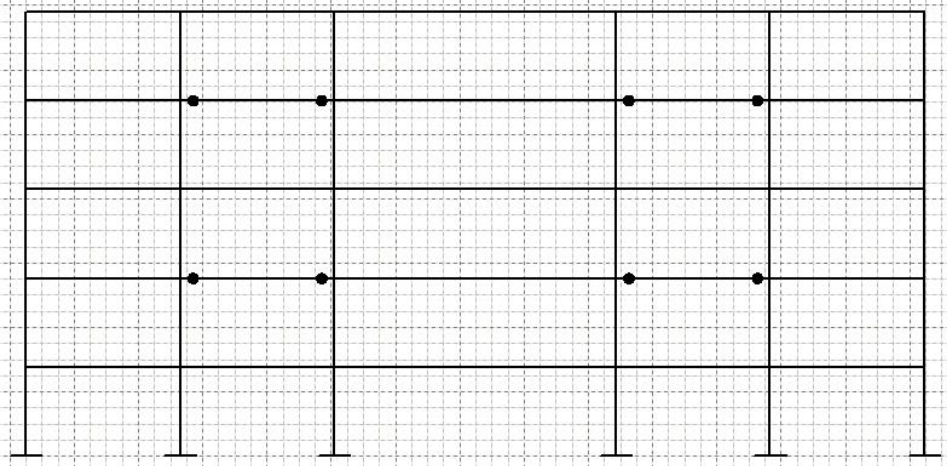
ORGANISM 8-1 (Moment Connection Design - 8 Total)



*Points Represent Locations of Moment Connections

total frame weight = 39.708 tons, total cost = $31,024.50

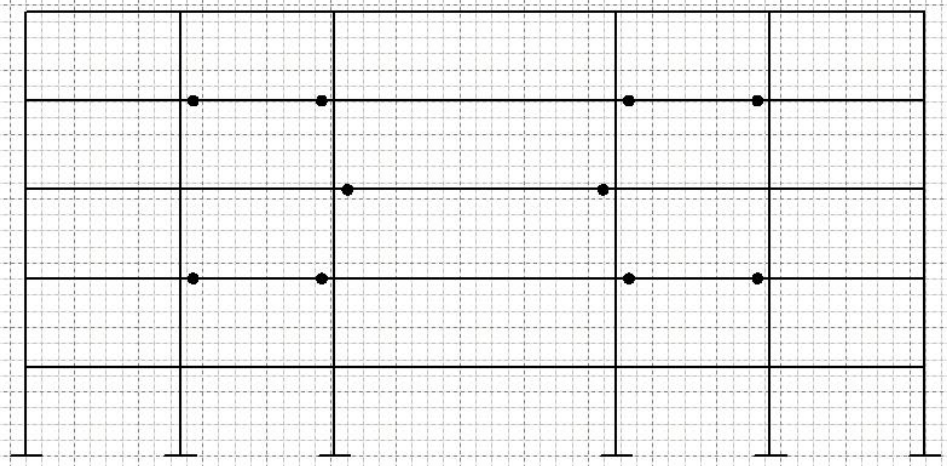
ORGANISM 8-2 (Moment Connection Design - 8 Total)



*Points Represent Locations of Moment Connections

total frame weight = 39.388 tons, total cost = $30,832.65

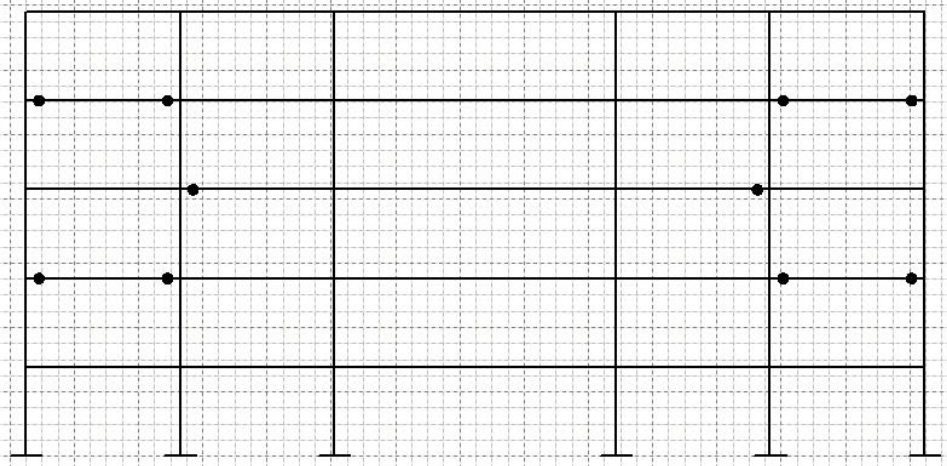
ORGANISM 10-1 (Moment Connection Design - 10 Total)



*Points Represent Locations of Moment Connections

total frame weight = 35.265 tons, total cost = $30,158.70

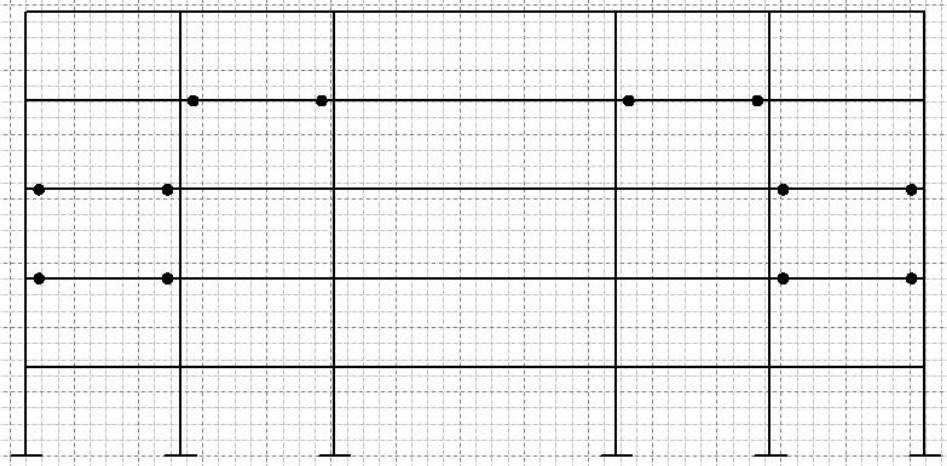
ORGANISM 10-3 (Moment Connection Design - 10 Total)



*Points Represent Locations of Moment Connections

total frame weight = 35.991 tons, total cost = $30,594.45

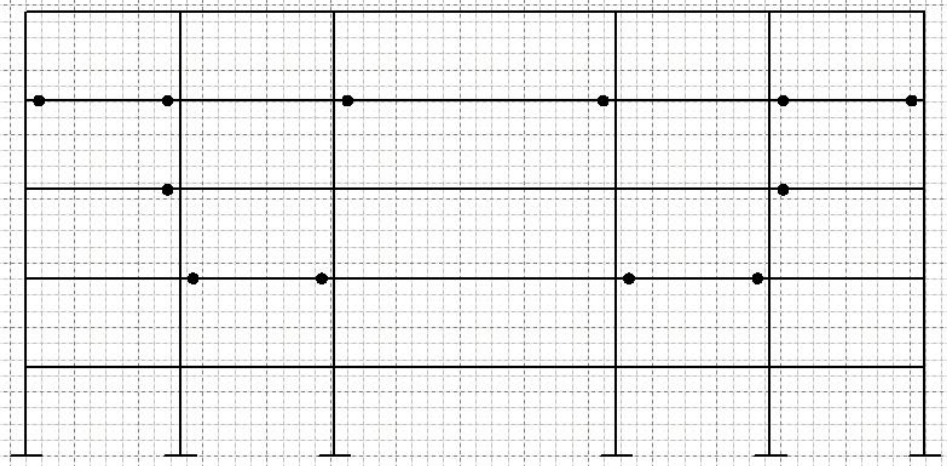
ORGANISM 12-1 (Moment Connection Design - 12 Total)



*Points Represent Locations of Moment Connections

total frame weight = 31.679 tons, total cost = $29,807.25

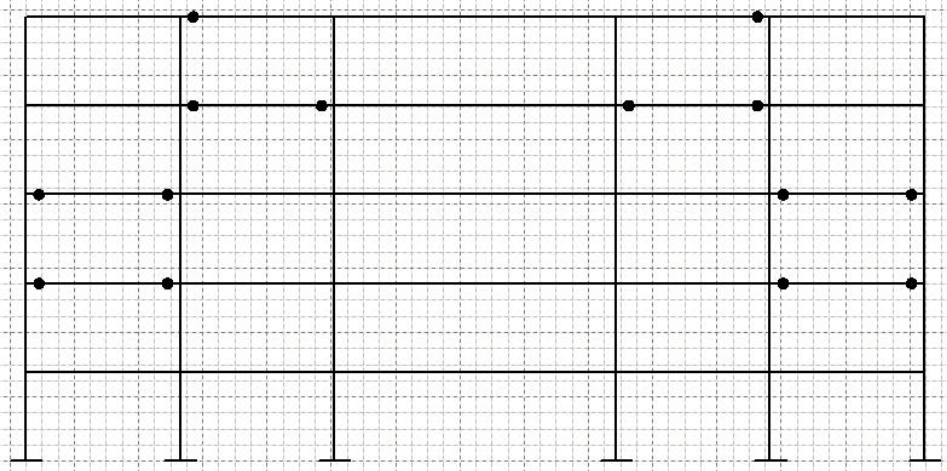
ORGANISM 12-3 (Moment Connection Design - 12 Total)



*Points Represent Locations of Moment Connections

total frame weight = 34.099 tons, total cost = $31,259.40

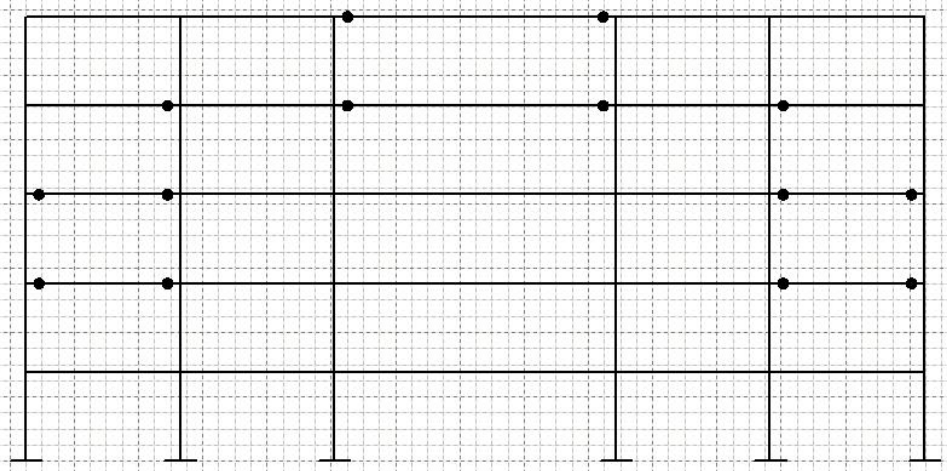
ORGANISM 14-2 (Moment Connection Design - 14 Total)



*Points Represent Locations of Moment Connections

total frame weight = 31.277 tons, total cost = $31,366.35

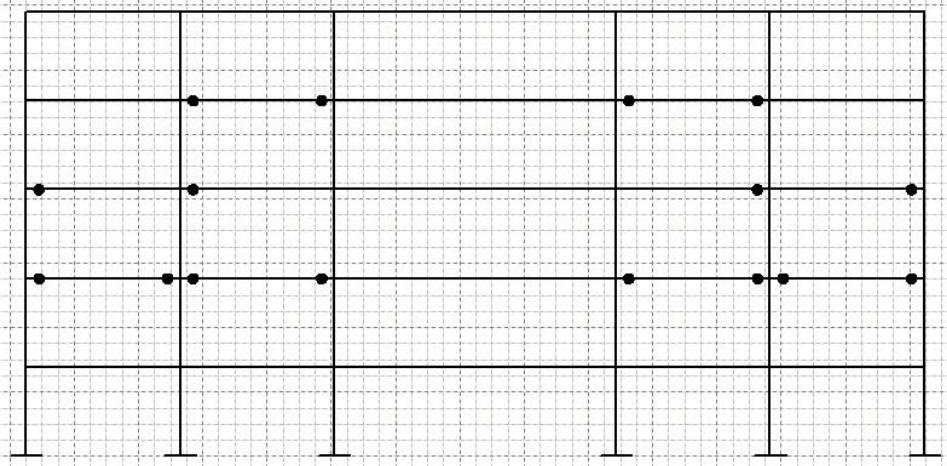
ORGANISM 14-3 (Moment Connection Design - 14 Total)



*Points Represent Locations of Moment Connections

total frame weight = 31.457 tons, total cost = $31,474.35

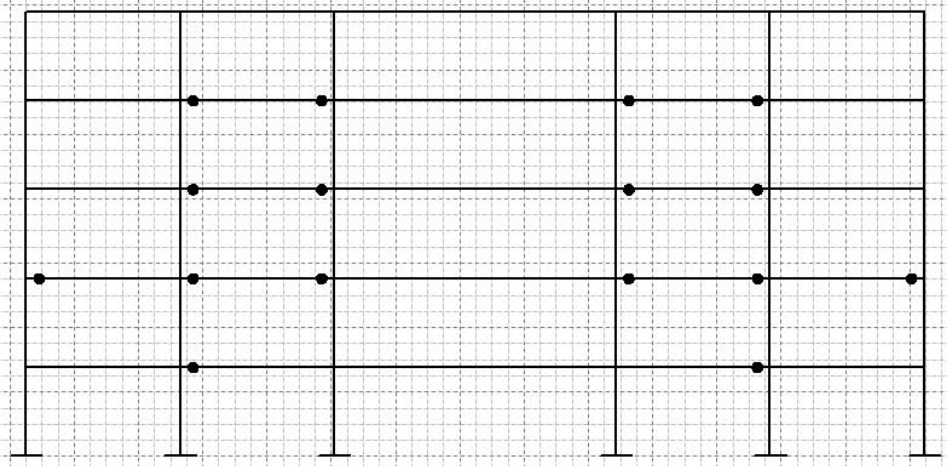
ORGANISM 16-1 (Moment Connection Design - 16 Total)



*Points Represent Locations of Moment Connections

total frame weight = 30.772 tons, total cost = $32,863.20

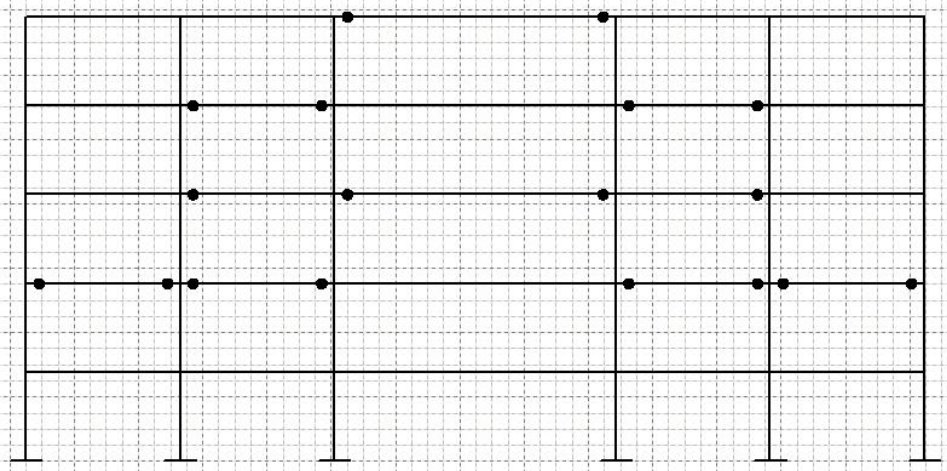
ORGANISM 16-3 (Moment Connection Design - 16 Total)



*Points Represent Locations of Moment Connections

total frame weight = 31.175 tons, total cost = $33,104.70

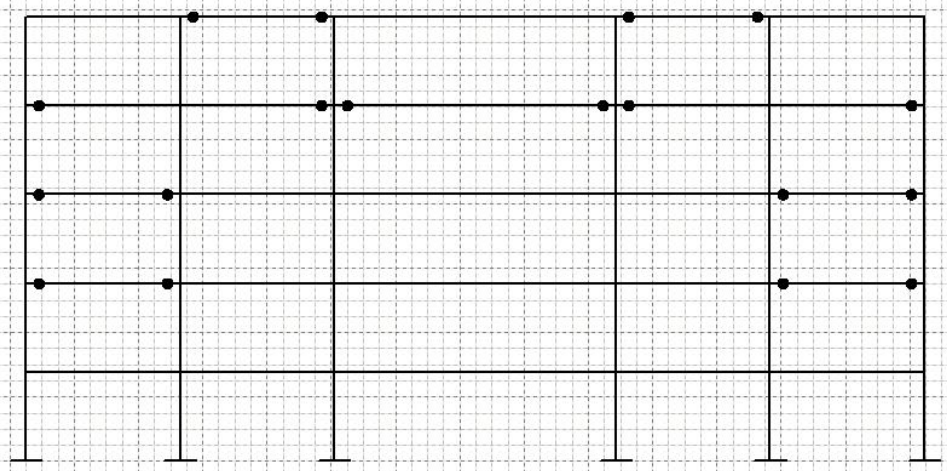
ORGANISM 18-2 (Moment Connection Design - 18 Total)



*Points Represent Locations of Moment Connections

total frame weight = 29.461 tons, total cost = $33,876.60

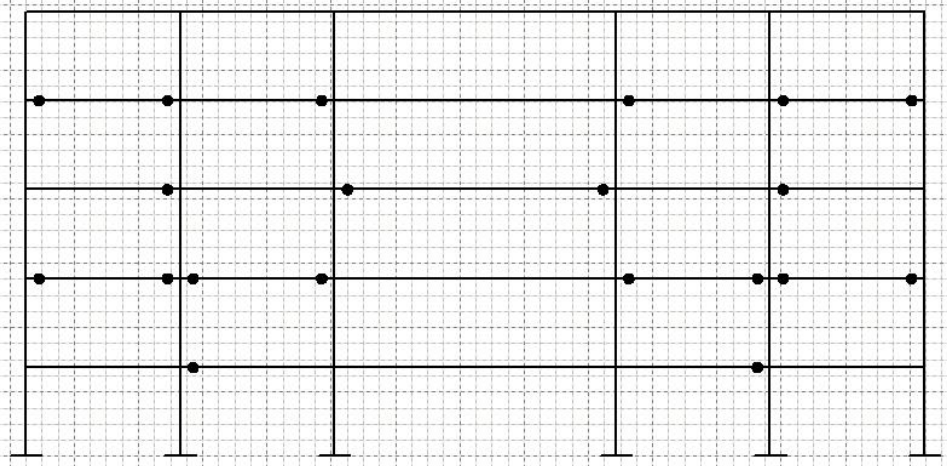
ORGANISM 18-3 (Moment Connection Design - 18 Total)



*Points Represent Locations of Moment Connections

total frame weight = 30.837 tons, total cost = $34,702.20

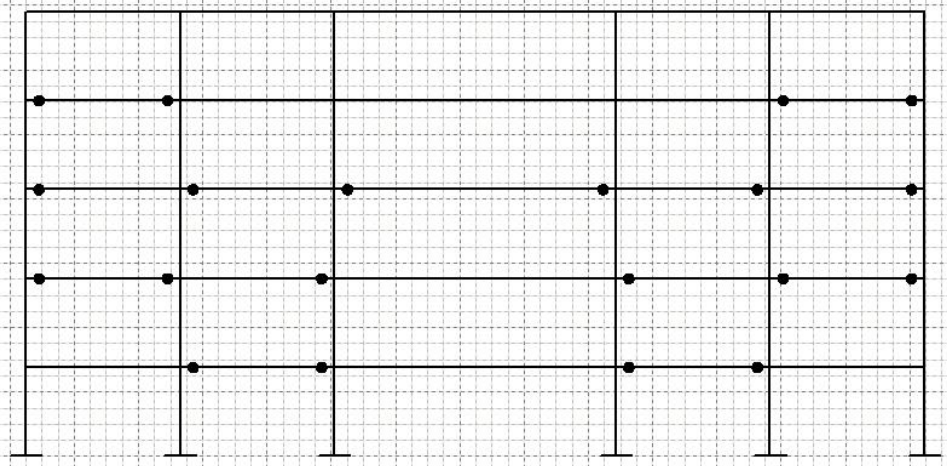
ORGANISM 20-2 (Moment Connection Design - 20 Total)



*Points Represent Locations of Moment Connections

total frame weight = 28.417 tons, total cost = $35,050.05

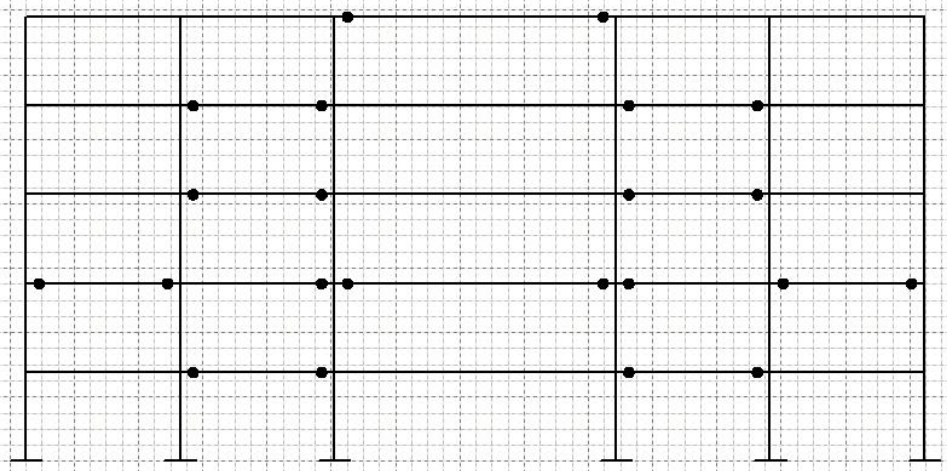
ORGANISM 20-3 (Moment Connection Design - 20 Total)



*Points Represent Locations of Moment Connections

total frame weight = 27.662 tons, total cost = $34,597.05

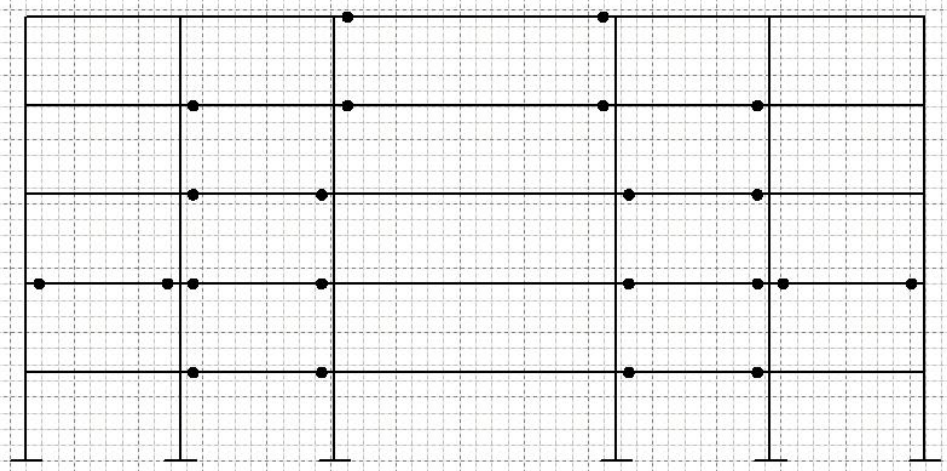
ORGANISM 22-1 (Moment Connection Design - 22 Total)



*Points Represent Locations of Moment Connections

total frame weight = 26.168 tons, total cost = $35,500.95

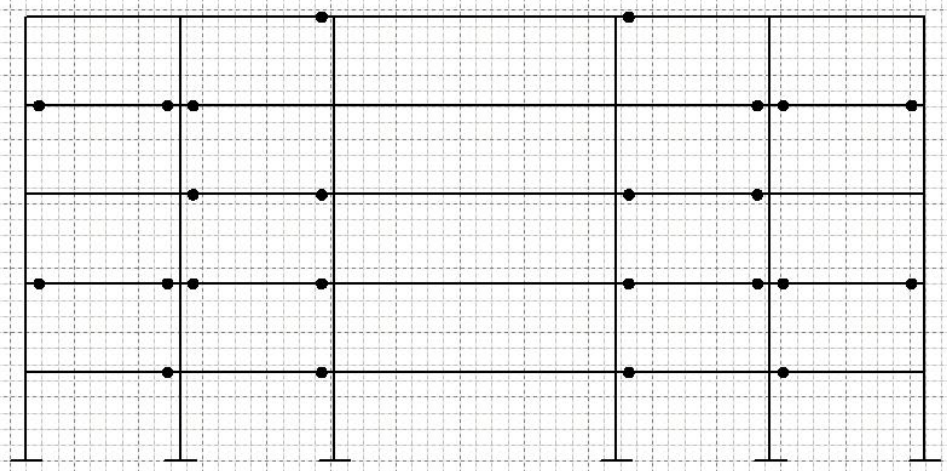
ORGANISM 22-2 (Moment Connection Design - 22 Total)



*Points Represent Locations of Moment Connections

total frame weight = 26.625 tons, total cost = $35,775.00

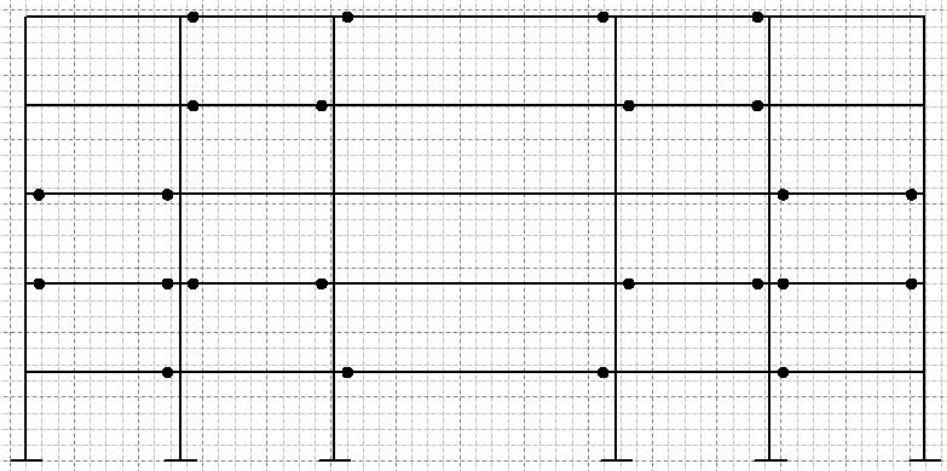
ORGANISM 24-1 (Moment Connection Design - 24 Total)



*Points Represent Locations of Moment Connections

total frame weight = 25.739 tons, total cost = $37,043.55

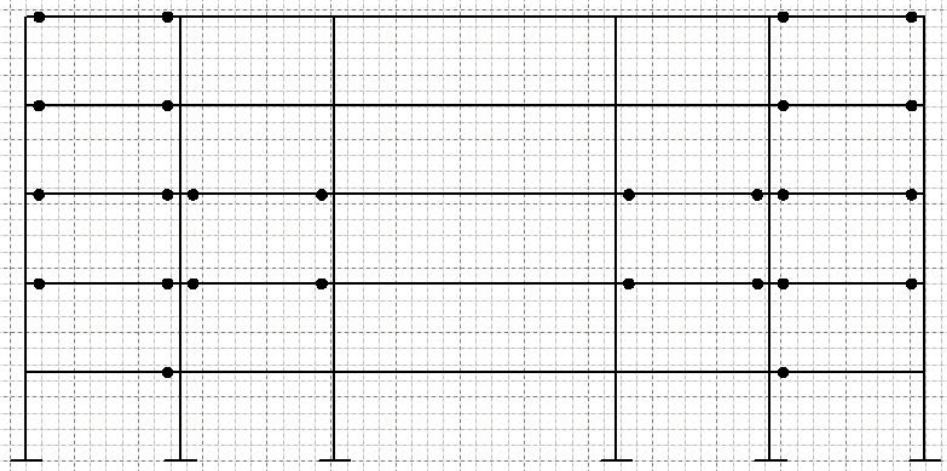
ORGANISM 24-2 (Moment Connection Design - 24 Total)



*Points Represent Locations of Moment Connections

total frame weight = 25.739 tons, total cost = $37,043.55

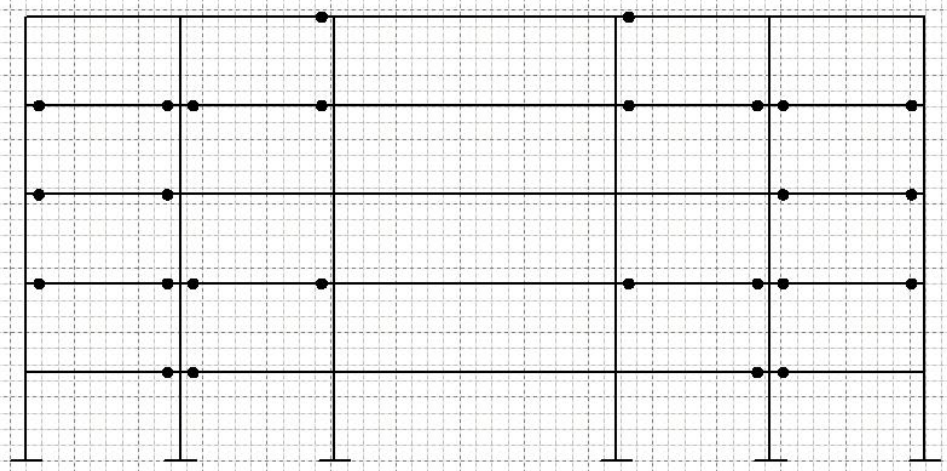
ORGANISM 26-1 (Moment Connection Design - 26 Total)



*Points Represent Locations of Moment Connections

total frame weight = 24.760 tons, total cost = $38,255.85

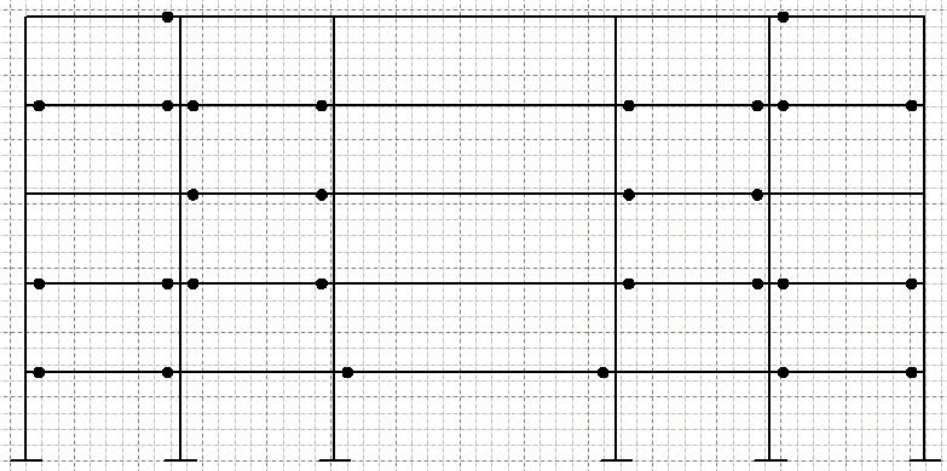
ORGANISM 26-3 (Moment Connection Design - 26 Total)



*Points Represent Locations of Moment Connections

total frame weight = 24.718 tons, total cost = $38,230.50

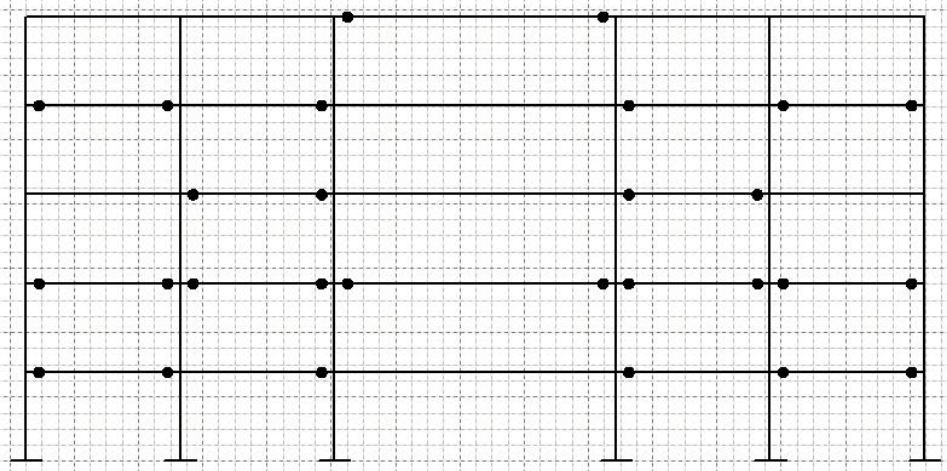
ORGANISM 28-1 (Moment Connection Design - 28 Total)



*Points Represent Locations of Moment Connections

total frame weight = 24.036 tons, total cost = $39,621.60

ORGANISM 28-2 (Moment Connection Design - 28 Total)



*Points Represent Locations of Moment Connections

total frame weight = 23.840 tons, total cost = $39,503.85

APPENDIX B :

REFERENCE TABLES & FIGURES

Dead Load In Pounds Per Square Foot Excerpt from Table A1 of Standard Building Code 1994	
Component	**Load**
Acoustical fiber tile	1
Mechanical duct allowance	4
Suspended steel channel system	2
Five-ply felt and gravel	6
Rigid insulation, 1/2-inch	0.75
Wood or steel stds, 1/2-in gypsum board each side	8

Table B 1: Dead Load Values (lbs)

MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS AND MINIMUM CONCENTRATED LIVE LOADS (Excerpt from Table 1607.1 of the 2000 International Bulding Code)		
OCCUPANCY OR USE	UNIFORM (psf)	CONCENTRATED (lbs.)
25. Office buildings		
File and computer rooms shall be designed for heavier loads based on anticipated occupancy		
Lobbies and first floor corridors	100	2,000
Offices	50	2,000
Corridors above first floor	80	2,000

Table B 2: Distributed Live Load Values (psf)

TABLE 1609.6.2.1(4) HEIGHT AND EXPOSURE ADJUSTMENT COEFFICIENTS[a]			
MEAN ROOF (feet)	EXPOSURE		
	B	C	D
15	1.00	1.21	1.47
20	1.00	1.29	1.55
25	1.00	1.35	1.61
30	1.00	1.40	1.66
35	1.05	1.45	1.70
40	1.09	1.49	1.74
45	1.12	1.53	1.78
50	1.16	1.56	1.81
55	1.19	1.59	1.84
60	1.22	1.62	1.87

For S1: 1 for = 304.8 mm.
a. All table values shall be adjusted for other exposures and heights by multiplying by the above coefficients.

Table B 3: Height & Exposure Adjustment Coeff.

TABLE 1604.5

CLASSIFICATION OF BUILDINGS AND OTHER STRUCTURES FOR IMPORTANCE FACTORS

CATEGORY[a]	NATURE OF OCCUPANCY	SEISMIC FACTOR I_E	SNOW FACTOR I_S	WIND FACTOR I_W
I	Buildings and other structures except those listed in Categories II, III and IV	1.00	1.0	1.00
II	Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: • Buildings and other structures where more than 300 people congregate in one area • Buildings and other structures with elementary school, secondary school or day-care facilities with capacity greater than 250 • Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities • Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities • Jails and detention facilities • Any other occupancy with an occupant load greater than 5,000 • Power-generating stations, water treatment for potable water, waste water treatment facilities and other public utility facilities not included in Category III sufficient quantities of toxic or explosive substances to be dangerous to the public if released	1.25	1.1	1.15
III	Buildings and other structures designated as essential facilities including, but not limited to: • Hospitals and other health care facilities having surgery or emergency treatment facilities • Fire, rescue and police station and emergency vehicle garages • Designated earthquake, hurricane or other emergency shelters • Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response • Power-generating stations and other public utility facilities required as emergency back-up facilities for Category III structures • Structures containing highly toxic materials as defined by Section 307 where the quantity of the material exceeds the exempt amounts of Table 307.7(2) • Aviation control towers, air traffic control centers and emergency aircraft hangars • Buildings and other structures having critical national defense functions • Water treatment facilities required to maintain water pressure for fire suppression	1.50	1.2	1.15
IV	Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: • Agricultural facilities • Certain temporary facilities • Minor storage facilities	1.00	0.8	0.87[b]

a. "Category" is equivalent to "Seismic Use Group" for the purposes of Section 1616.2.
b. In hurricane-prone regions with V>100 miles per hour, I_w shall be 0.77.

Table B 4: Building Classifications & Importance Factors

MAIN WINDFORCE-RESISTING SYSTEM LOADS FOR A BUILDING WITH MEAN ROOF HEIGHT OF 30 FEET LOCATED IN EXPOSURES B[a] (psf) Excerpted from TABLE 1609.2.1(1) from the 2000 INTERNATIONAL BUILDING CODE						
BASIC WIND SPEED V (mph - 3-second gust)	LOAD DIRECTION	ROOF ANGLE	HORIZONTAL LOADS[b]			
			End zone		Interior zone	
			Wall	Roof[c]	Wall	Roof[c]
100	Transverse	0 to 5º	15.9	-8.2	10.5	-4.9
		20º	22.0	-5.8	14.6	-3.2
		30º < angle ≤ 45º	17.8	12.2	14.2	9.8
	Longitudinal	All angles	15.9	-8.2	10.5	-4.9

Table B 5: Main Wind Force-Resisting System Loads

TABLE J3.3 Nominal Hole Dimensions, in.				
	Hole Dimensions			
Bolt Diameter	Standard (Dia.)	Oversize (Dia.)	Short-slot (Width x Length)	Long-slot (Width x Length)
1/2	9/16	8-May	9/16 x 11/16	9/16 x 1 1/4
5/8	11/16	13/16	11/16 x 7/8	11/16 x 1 9/16
3/4	13/16	15/16	13/16 x 1	13/16 x 1 7/8
7/8	15/16	1 1/16	15/16 x 1 1/8	15/15 x 2 3/16
1	1 1/16	1 1/4	1 1/16 x 1 5/16	1 1/16 x 2 1/2
≥ 1 1/8	d + 1/16	d + 5/16	(d + 1/16) x (d + 3/8)	(d + 1/16) x (2.5 x d)

Table B 6: Nominal Hole Dimensions

TABLE J3.4 Minimum Edge Distance, [a] in., From Center of Standard Hole [b] to Edge of Connected Part		
Nominal Rivet or Bold Diameter (in.)	**At Sheared Edges**	**At Rolled Edges of Plates, Shapes or Bars, or Gas Cut Edges [c]**
1/2	7/8	3/4
5/8	1 1/8	7/8
3/4	1 1/4	1
7/8	1 1/2 [d]	1 1/8
1	1 3/4 [d]	1 1/4
1 1/8	2	1 1/2
1 1/4	2 1/4	1 5/8
Over 1 1/4	1 3/4 x Diameter	1 1/4 x Diameter

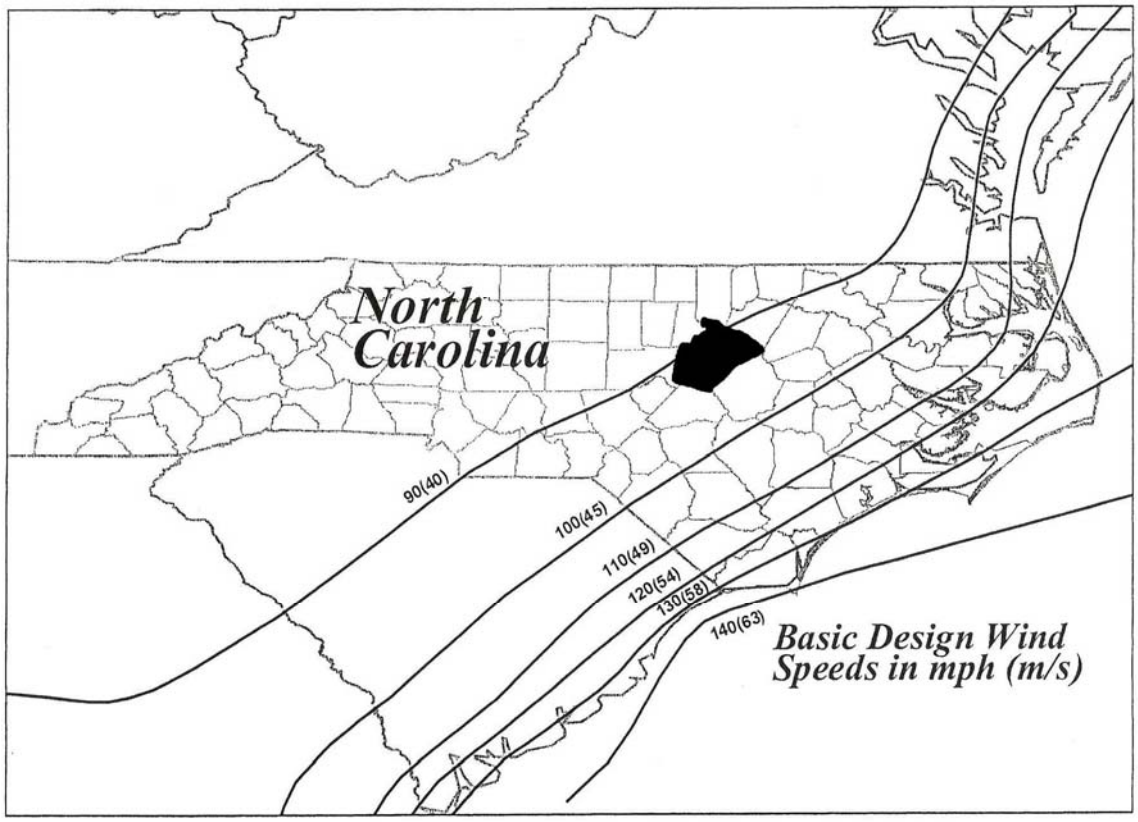
[a] Lesser edge distances are permitted to be used provided Equations from Section J3.10 as appropriated, are satisfied.
[b] For oversized or slotted holes, see Table J3.6.
[c] All edge distances in this column are permitted to be reduced 1/8 - in. when the hole is at a point where stress, does not exceed 25 percent of the maximum design strength in the element.
[d] These are permitted to be 1 1/4 - in. at the ends of beam connection angles and shear end plates.

Table B 7: Minimum Bolt Edge Distances

TABLE J2.4 Minimum Size of Fillet Welds	
Material Thickness of Thicker Part Joined, in. (mm)	**Minimum Size of Fillet Weld[a] in. (mm)**
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
[b] See Section J2 2b for maximum size of fillet welds.

Table B 8: Minimum Size of Fillet Welds



NORTH CAROLINA BASIC WIND SPEEDS

Figure B 1: North Carolina Basic Wind Speeds

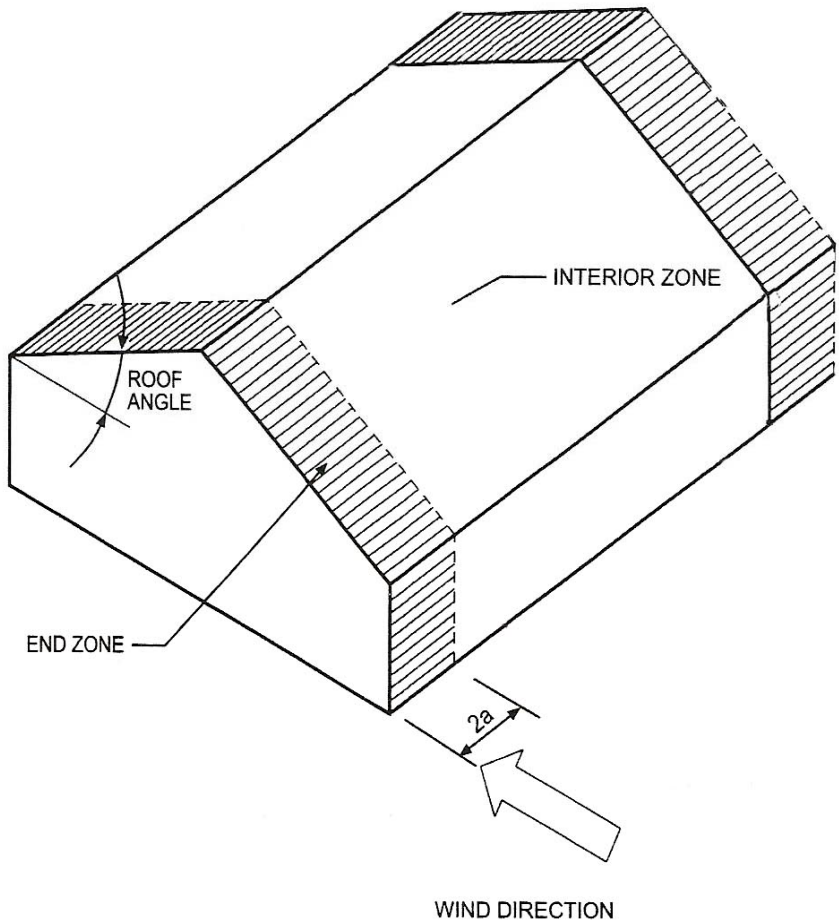


Figure B 2: End & Interior Zones for Wind Force Loading

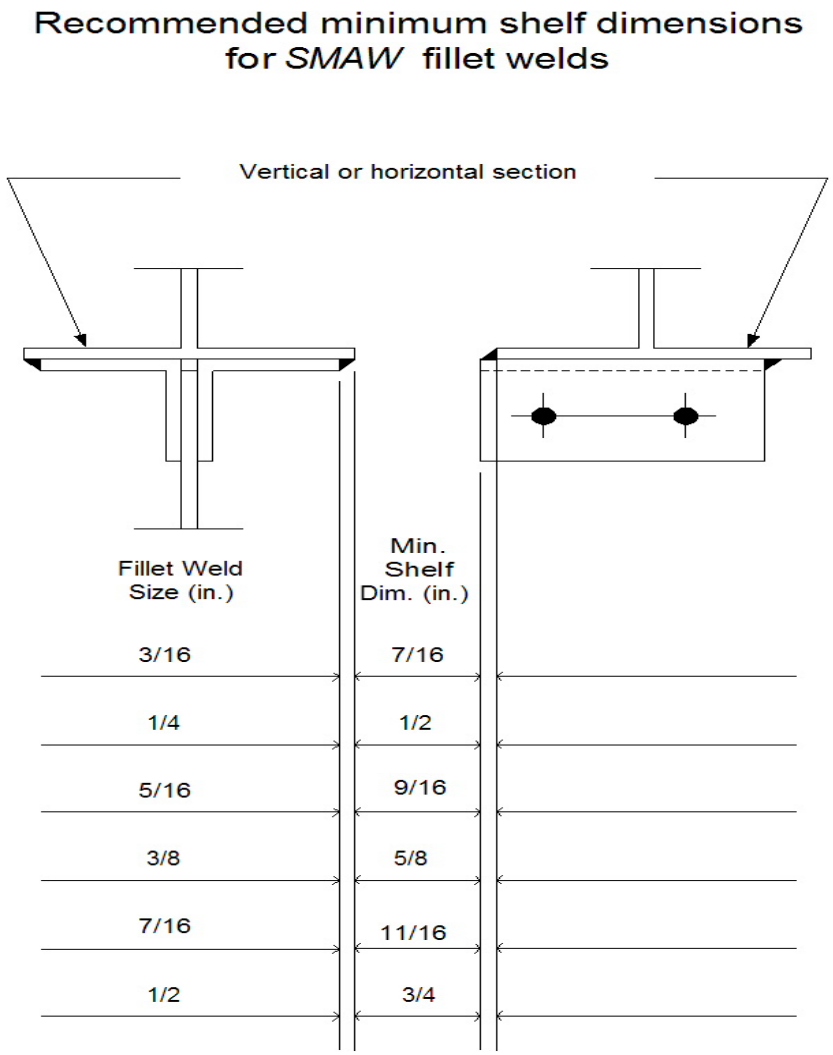


Figure B 3: Shelf Dimensions for Fillet Welds

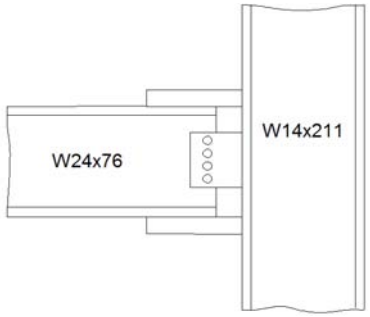
APPENDIX C :

NUMERICAL EXAMPLE FOR WELDED FLANGE PLATE CONNECTION DESIGN

MOMENT CONNECTION DESIGN (Welded Flange Plate / Bolted Web)

A992 Steel (F_y = 50 ksi, F_u = 65 ksi)
A325 Bolts (ASTM)
E70XX Electrodes

* beam continually braced by floor system



From SAP 2000 Analysis:
G + W Factored Load @ Connection
M_u = 281 k · ft
V_u = 40.1 k

Member Info:

W24x76 (beam)	W14x211(column)
d_b = 23.9 in	d_c = 15.7 in
b_f = 8.99 in	b_f = 15.8 in
t_f = 0.680 in	t_f = 1.56 in
t_w = 0.44 in	t_w = 0.98
Z_x = 200 in^3	k = 2.16
	T = 10 in

Check Beam Design Flexural Strength :

$\lambda_{flg} = b_f / 2t_f = 8.99 / (2(0.680)) = 6.61$

$\lambda_p = 0.38 \sqrt{(E/F_y)} = 0.38 \sqrt{(29000/50)} = 9.15 > 6.61$

therefore flange is compact

$\lambda_{web} = h/t_w = 49.0$

$\lambda_p = 3.76 \sqrt{(E/F_y)} = 3.76 \sqrt{(29000/50)} = 90.55 > 49.0$

therefore web is compact

therefore $Z_{req.} = \frac{281(12)}{0.9(50)} = 74.9 \text{ in}^3 < Z_x = 200 \text{ in}^3$ OK

Check Beam Design for Shear :

For W24 x 76 . . . $h / t_w = 49.0$

$2.45 \sqrt{(E/F_y)} = 2.45 \sqrt{(29000/50)} = 59.0 > 49.0$

therefore $\phi_v = 0.9$, $V_n = 0.6\ F_y A_w$

(A_w = Area of web = overall depth (d) multiplied by web thickness (t_w) pg. 16.1-35 Section F2 LRFD)

$A_w = 23.9\ (0.44) = dt_w = \underline{10.516\ in^2}$

$V_n = 0.6\ (50)(10.516) = \underline{315.48\ k}$

$\phi_v V_n = 0.9\ (315.48) = 283.93\ k > 40.1\ k = V_u$ OK

Tension Flange Plate and Connection:

Flange Force = $P_{uf} = M_u / d$ $\quad P_{uf} = \dfrac{281(12)}{23.9} = \underline{141.1\ k}$

* w/min shelf dimension of 5/8", plate width $= b_f - 2(5/8)$

$= 8.99 - 2(5/8)$

$= 7.74$ in

therefore try a 5/8" x 7 3/4" plate

Tension Yielding of Flange Plate:

$\phi R_n = \phi\ F_y A_g = 0.9(50)(5/8)(7\ 3/4) = 217.97\ k > 144.1\ k$ OK

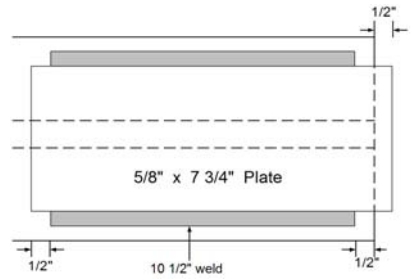
Required Weld Size:

D = size of weld in 16ths (max = 9/16", min = 1/4" = 4/16")

therefore = use 5/16" weld, D = 5

ℓ min = minimum length of weld = P_{uf} / 1.392D

ℓ min = 1.41 / (1.392(5)) = 20.27 in



therefore use 10 1/2 in of weld along each side of the plate (total of 21 in of weld)

<u>Tension Rupture of Flange Plate:</u>

L = 10 1/2 in > W = 7 3/4 in
< 1.5 W = 11 5/8 in } *therefore* U = 0.75

$\phi R_n = \phi F_u U A_g$ = 0.75 (65) (0.75) (5/8) (7 3/4) = 177.1 k > 141.1 k OK

<u>Strength of Welds</u> (Weld Rupture):

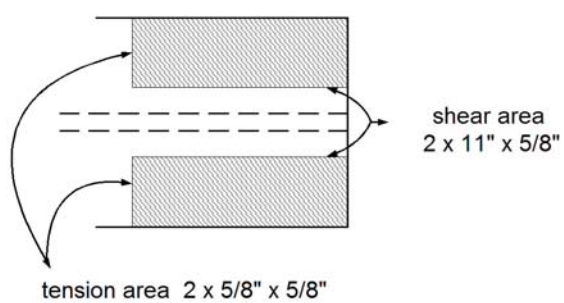
21 (1.392) (5) = 146.2 k > 141.1 k OK

<u>Block Shear for Beam Flange w/ Tension Plate:</u>

R_n = max | F_u (1.25) (5/8) + min | Opp. Yield = max | 50.78 + min | ten. Yield
| 0.6 F_u (22) (5/8) | Opp. Rupture | 536 | 50.8

= 536.25 + min | F_y (1.25) (5/8) = 39.06 → R_n = 536.25 + 39.06
| 50.8

ϕR_n = 0.75 (575.31) = 431.5 k >> 141.1 k OK



Base Metal Strength @ Tension Plate:

$\phi R_n = 0.75\ (0.6)\ F_{up}\ (A_w)$, A_w = Area of Weld

$= 0.75\ (0.6)\ (65)\ (21)\ (5/8)$

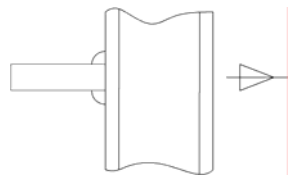
$= 383.9\ k > 141.1\ k = P_{uf}$ OK

Design Fillet Welds from Tension Plate to Column Flange:

$$D\ min = \frac{P_{uf}}{2\ (1.5)(1.392\ell)} = \frac{141.1}{2\ (1.5)(1.392)(7.75)} = 4.37 \rightarrow 5 \text{ sixteenths}$$

load ⊥ to weld

double sided weld



max → N/A

min → 5/16"

therefore use 5/16" fillet welds

Compression Flange Plate & Connection:

* use similar cross-sectional area as tension side . . .

$A = (5/8)\ (7\ 3/4) = 4.84\ in^2$

* with min. shelf dimension of 5/8 in, plate width = $b_f + 2\ (5/8)$

$= 8.99 + 2\ (5/8)$

$= 10.24$ in

therefore try a 1/2" x 10 1/4" plate ($A = 5.125\ in^2$)

Required Weld Size:

* use a total of 21" of 5/16" weld (same as tension plate)

therefore 10 1/2" length on each side of the flange

min → 4/16" or 1/4" (J2.4)

max → 7/16" (t_p - 1/16")

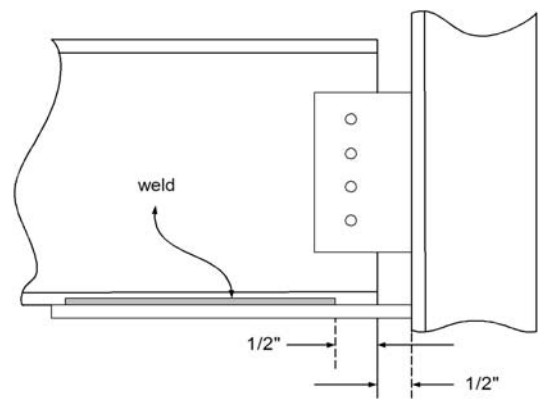
Compression Plate Local Buckling:

Stiffened Element → $\frac{b_f}{t_p} \leq 1.49 \sqrt{\frac{E}{F_y}} = 1.49 \sqrt{\frac{29000}{50}} = 35.88$

$$\frac{8.99}{0.5} = 17.98 < 35.88 \quad \text{OK}$$

Unstiffened Element → $\frac{b}{t_p} \leq 0.56 \sqrt{\frac{E}{F_y}} = 0.56 \sqrt{\frac{29000}{50}} = 13.49$

$$\frac{0.625}{0.5} = 1.25 < 13.49 \quad \text{OK}$$



Compression Plate Flexural Buckling:

k = 0.65, ℓ = 1/2" edge dist. + 1/2" setback = 1"

$$r = \sqrt{\frac{(10.25)(0.5)\ 3/12}{(10.25)(0.5)}} = 0.144$$

$$\frac{k\ell}{r} = \frac{0.65\ (1)}{0.144} = 4.5139$$

therefore $\phi F_{cr} = 42.45$ ksi

(Table 3-50, pg. 16, 1-145)

$\phi R_n = \phi F_{cr} A = 42.45\ (5.125) = 217.56\ k > 141.1\ k$ OK

↑

10 1/4" x 1/2"

Shear Yield (In Compression Plate Below Welds) :

$\phi V_n = 0.9\ (0.6\ F_y)\ A_g,\ A_g = t_p\ (L_{weld})$

$= 0.9\ (0.6)\ (50)\ (1/2)\ (21)$

$= 283.5\ k > 141.1\ k$ OK

Shear Rupture (In Compression Plate Below Welds):

$$\phi V_n = 0.75\ (0.6\ F_u)\ A_g, \quad A_g = t_p\ (L_{weld})$$
$$= 0.75\ (0.6)\ (65)\ (1/2)\ (21)$$
$$= 307.13\ k > 141.1\ k$$

Fillet Welds from Compression Plate to Column Flange:

$$D_{min} = \frac{141.1}{2(1.5)(1.392\ell)} = \frac{141.1}{2(1.5)(1.392)(10.25)} = 3.31$$

Max → N/A *therefore* use 5/16" fillet welds
Min → 5/16"

Check for Flange Plates (Moment Connection, Opposite Column Side):

Tension Plate (In Compression):

Flange Force = P_{uf} = 141.1 k 5/8" x 7 3/4" Plate

* 10 1/2" fillet weld length on each side of plate

Plate Local Buckling:

Stiffened Element → $\frac{b_f}{t_p} \leq 1.49 \sqrt{\frac{E}{F_y}} = 35.88$

$\frac{8.99}{(5/8)} =$ 14.38 < 35.88 OK

Plate Flexural Buckling:

k = 0.65, ℓ = 1" (1/2" edge + 1/2" setback)

$r = \sqrt{\frac{(7.75)(0.625)^3 / 12}{(7.75)(0.625)}} = 0.180 \qquad \frac{k\ell}{r} = \frac{0.65(1)}{0.18} = 3.611$

therefore ϕ For A = 42.5(7.75)(0.625) = 205.9 k > 141.1 k OK

Shear Yield (In Plate Above Weld):

$\phi V_n = 0.9 (0.6 F_y)(A_g)$, $A_g = t_p (L_{weld})$

$\phi V_n = 0.9 (0.6)(50)(0.625)(21) = 354.4$ k > 141.1 OK

Shear Rupture (In Plate Above Weld):

$\phi V_n = 0.75 (0.6 F_u)(A_g)$, $A_g = t_p (L_{weld})$

$\phi V_n = 0.75 (0.6)(65)(0.625)(21) = 354.4$ k > 141.1 OK

= 383.9 k > 141.1 k OK

Compression Plate (In Tension):

P_{uf} = 141.1 k 1/2" x 10 1/4" plate

* 10 1/2" length weld on each side of plate

Tension Yielding (of Plate):

$\phi R_n = \phi F_y A_g$ = 0.9 (50)(0.5)(10.25) = 230.6 k > 141.1 k OK

Tension Rupture (of Plate):

$$\left.\begin{array}{l} L = 10\ 1/2 \text{ in} > W = 10\ 1/4 \text{ in} \\ \qquad\qquad\quad < 1.5\ W = 15\ 3/8 \text{ in} \end{array}\right\} \quad \textit{therefore} \quad U = 0.75$$

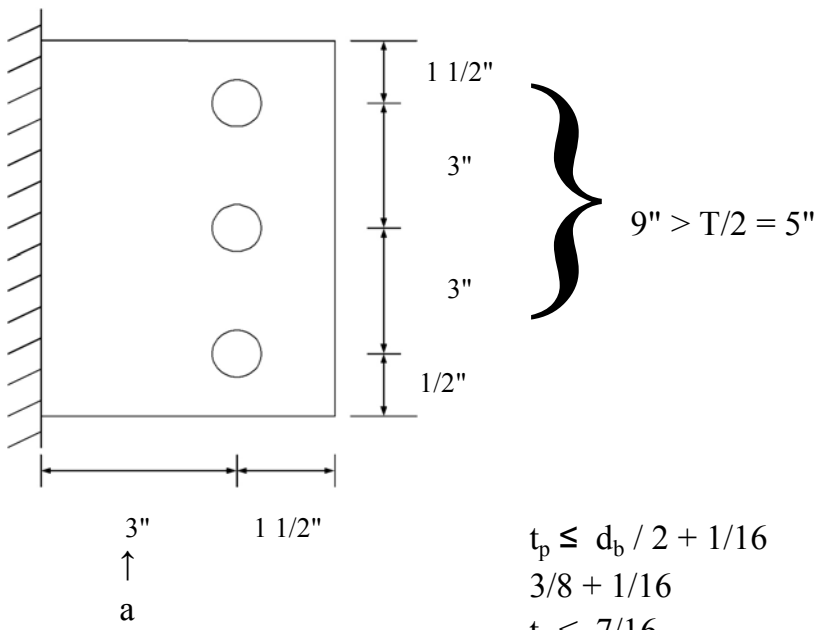
$\phi R_n = \phi F_u U A_g$ = 0.75 (65)(0.75)(0.5)(10.25) = 187.4 k > 141.1 k OK

Web Connections:

* Determine number of 3/4" A325 - N bolts req. for shear

$$n_{min} = \frac{40.1\ k}{15.9\ k\ /\ bolt} = 2.522 \qquad \textit{therefore}\ \text{use 3 bolts}$$

* Determine plate geometry



$t_p \leq d_b / 2 + 1/16$

$3/8 + 1/16$

$t_p \leq 7/16$

therefore try 3/8" x 9" Plate

Plate Buckling for Web Connection:

$t_p \geq L/64 = 9/64$

$t_p = 0.375 > 0.141$ OK

Shear Yielding of Plate:

$\phi V_n = 0.9(0.6\ F_y)(A_g) = 0.9(0.6)(50)(3/8)(9) = 91.1\ k > 40.1\ k$ OK

Shear Rupture of Plate:

$\phi V_n = 0.75\ (0.6F_u)\ A_n = 0.75\ (0.6)(65)(3/8)(9-3(3/4+1/8))$

$= 69.93\ k > 40.1\ k$ OK

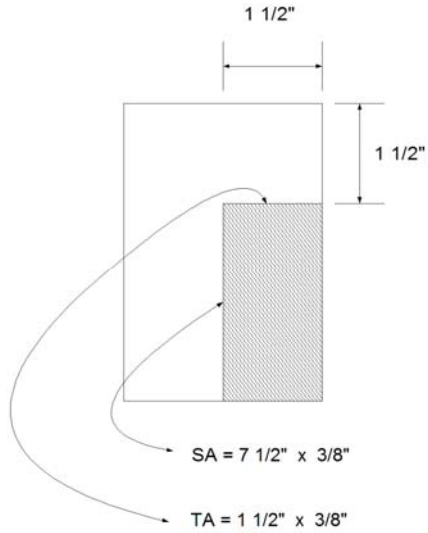
Block Shear:

$\phi R_n = \max \left| \begin{array}{l} 69.93 \text{ k} \\ \text{Tension Rupture} \end{array} \right. + \min \left| \begin{array}{l} \text{Opposite Yield} \\ \text{Opposite Rupture} \end{array} \right.$

Tension Rupture = 0.75 (65)(3/8)1 1/2 - 0.5(3/4 + 1/8))
= 19.42 k

Tension Yield = 0.9 (50)(3/8)(1 1/2) = 25.31 k

therefore ϕR_n = 69.93 + 19.42 = 89.35 k > 40.1 k OK



Bearing / Tear Out:

Bearing → 2.4 (0.75)(3/8)(65) = 43.9 k
$2.4\ d_b t F_u$

Edge Bolts Tear Out → 1.2 (1 1/2 - 1/2 (3/4 + 1/16)) (3/8)(65) = 32.0 k
$1.2\ L_c t F_u$

Other Bolts Tear Outs → 1.2 (3 - (3/4 + 1/16))(3/8)(65) = 64.0 k
$1.2\ L_c t F_u$

ϕR_n = 0.75 ((2*43.9) + (1*32.0)) = 89.9 k > 40.1 k OK

Required Weld Size:

$t_{weld} \geq (3/4)\ t_p$ = (3/4)(3/8) = 9/32"

min weld size = 3/16"
max weld size = N/A *therefore* use t_{weld} = 5/16'

Column Limit States:

Local Flange Bending:

T_u = Flange Force = P_{uf} = 141.1 k

$\phi R_n = 0.9\,(6.25)(t_{fc}^2)(F_{yc})$

$= 0.9\,(6.25)(1.56)^2(50)$

$= 684.5\text{ k} >> T_u = 141.1\text{ k}$ OK

Local Web Yielding:

$\phi R_n = 1.0(F_{yc})(5k + t_p)(t_{wc})$

$= 1.0\,(50)(5(2.16) + 5/8)(0.98)$

$= 559.8\text{ k} >> T_u = 141.1\text{ k}$ OK

Local Web Crippling:

$$R_n = 0.80\,t_w^2 \left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_y\,t_f}{t_w}}$$

$$R_n = 0.80(0.98)^2 \left[1 + 3\left(\frac{0.5}{15.7}\right)\left(\frac{0.98}{1.56}\right)^{1.5}\right]\sqrt{\frac{29{,}000\,(50)(1.56)}{0.98}}$$

$R_n = 0.80\,t_w^2$

$\phi R_n = 0.75\,(1222.8) = 917.1\text{ k} >> Cu = 141.1\text{ k}$ OK

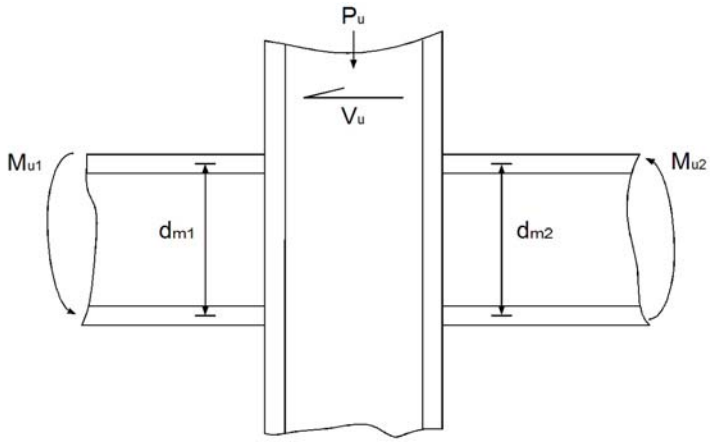
Web Buckling:

$$R_n = 0.9\left(\frac{24\,t_{wc}^3\sqrt{E\,F_{yc}}}{h}\right)$$

h = clear distance between fillets
(get from tabulated h / t_w)

$h / t_w = 11.6 \rightarrow h = 11.6\,(0.98) = 11.368$

$$\phi R_n = \frac{0.9\,(24(0.98)^3\ \sqrt{29{,}000\,(50)}\,)}{11.368} = 2153.4\text{ k} >> 141.1\text{k} \quad \text{OK}$$



Panel Zone:

$$\sum F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u$$

$M_{u1} = 281\text{ k}\cdot\text{ft} = M_{u2} = 3372\text{ k}\cdot\text{in}$

$d_{m1} = d_{m2} = d - t_f = 23.9 - 0.680 = 23.22''$ (same beam)

$V_u = 28$ k

$$\sum F_u = \frac{3372}{23.22} + \frac{3372}{23.22} - 28 = 262.4\text{ k}$$

$$P_u = 248\text{ k} \quad = \quad \frac{(223\text{k} + 273\text{k})}{2}$$

$$0.4\,P_y = 0.4\,F_yA_g = 0.4\,(50)(62) = 1240\text{ k} >> 248\text{ k}$$

therefore $\phi R_v = 0.9\,(0.6\,F_yd_ct_w) = 415.4\text{ k} > 262.4\text{ k}$ OK

If Stiffners are Required: (Tension or Compression)

$$\frac{b_s + t_{wc}}{2} \geq \frac{b_p}{3} \quad \rightarrow \quad \frac{b_s + 0.98}{2} \geq \frac{7.75}{3} \text{ (tension)}$$

$$\frac{10.25}{3} \text{ (compression)}$$

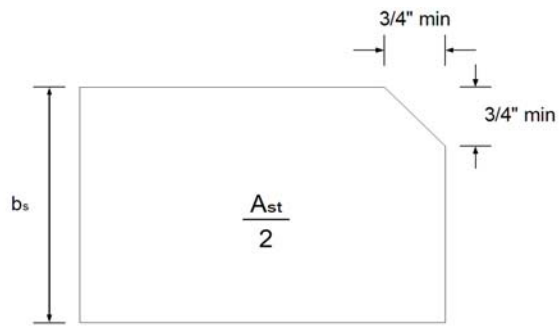
$$t_s \geq \frac{t_p}{2} \rightarrow t_s \geq \frac{(5/8)}{2} \text{ (tension)}$$

$$\frac{(1/2)}{2} \text{ (compression)}$$

$$\frac{b_s}{t_s} \leq 0.56 \sqrt{\frac{E}{F_y}} = 0.56 \sqrt{\frac{29{,}000}{50}} \text{ (compression only)}$$

$$A_{st} = \text{stiffner area (total)} = \frac{R_u - \phi R_n}{0.9\, F_{ys}} \text{ (tension)}$$

$$= \frac{R_u - \phi R_n}{0.85\, F_{ys}} \text{ (compression)}$$



* use full penetration welds for connection to flange and web

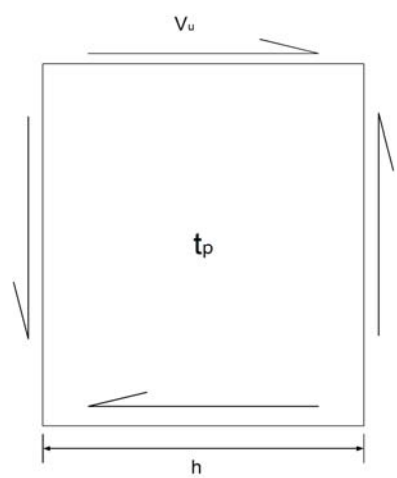
If Panel Zone Doubler Plate is Required:

$$V_u = \sum F_u - \phi R_v \qquad V_u \leq 0.9\, V_n = \phi V_n$$

$$h = d - t_f$$

$$\text{for } \frac{h}{t_p} \leq 2.45\sqrt{\frac{E}{F_y}} \rightarrow V_n = 0.6\, F_{yp} h t_p$$

$$\text{for } \frac{h}{t_p} > 2.45\sqrt{\frac{E}{F_y}} \rightarrow V_n = 0.6\, F_{yp} h t_p \left(\frac{2.45\sqrt{E / F_y}}{h / t_p} \right)$$



* use full penetration welds for connection to column