Practical Approaches in Mill Building Columns Subjected to Heavy Crane Loads

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1. INTRODUCTION

Since the turn of this century, the amount of analytical and experimental research in the area of column design has been extensive. While the information and results of such research is scattered and narrowly focused in very specialized areas, identification of this vast amount of relevant information available is often difficult. There is no legal code governing the design of heavy mill building columns and the designs are based on experience and judgement. Additionally, there are varying types of mill building columns due to assumptions and geometry. This has led to a design practice that varies from one designer to another, based on their experience and knowledge of the mill buildings. For a practicing engineer it is essential to analyze, integrate, and establish a logical and comprehensive design methodology and procedures. It is not possible to establish one generic approach applicable to all types of mill building columns. It is rather easier to deal independently with each type of these complex structural elements in achieving this goal.

This paper presents a design methodology and procedures for "heavy mill building columns" subjected to heavy crane loads. For the purpose of this paper heavy crane loads are defined as loads imparted by cranes with lifting capacity ranges from 100 tons to 500 tons such as hot metal cranes. These columns consist of two or more shafts and are laced together to achieve composite behavior (see Figure 1). The column bases are fully fixed and the roof trusses are simply supported at the top of the building shaft. The bottom chords of the trusses are braced the full length of the building. Other types of mill building columns are not included in the scope of this paper. Therefore, the procedures and methodology presented herein may not be directly applicable to any other type of mill building column.

The performance of any combined column depends upon the proper design of all the elements associated with the column. Important structural aspects affecting the design of individual elements are discussed, and followed by a recommended design procedure. A numerical example is shown to illustrate the procedures discussed.

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2. COLUMNS

Stability and strength of columns are the most important aspects of a structure. Studies on this subject started with Euler in the eighteenth century. His classical study (1744) on elastic critical load (P_{CR}) for a perfectly straight column of an elastic material is still used as a reference point for research today. A multitude of parameters influencing the column strength were studied and the work still continues. Other than the length, some of the primary parameters influencing the column strength are the grade of steel, manufacturing methods, size and shape of cross section, axis of bending, initial eccentricities, and degree of end restraint. This evolutionary progress of the present day column design is enumerated in detail by Bjorhovde.[1]

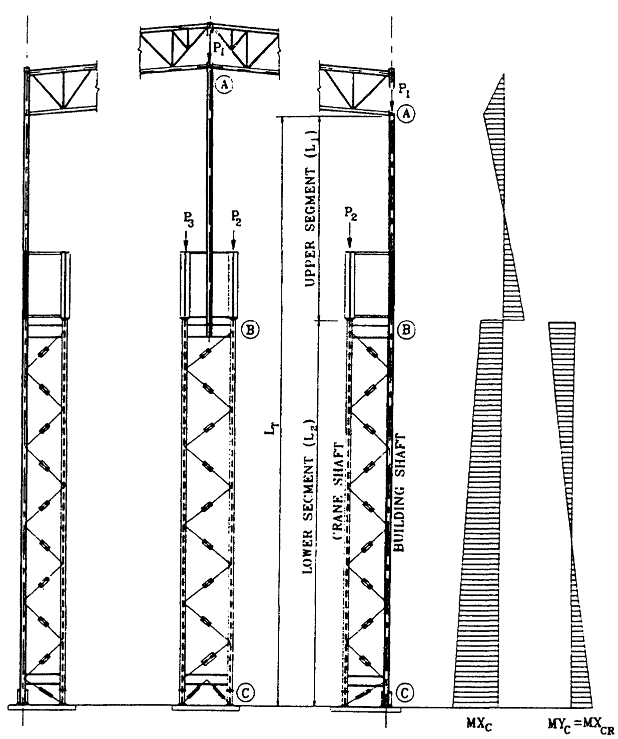

Fig. 1. Heavy mill building columns.

3. MILL BUILDING COLUMNS AND DESIGN CONSIDERATIONS

Mill building columns are different from the most other columns. Design of mill building columns is complex and cumbersome due to the variables such as loading conditions, geometric irregularities, and end restraints that are inherent to the mill building columns. Mill building columns are generally designed for a life span of 50 years. These columns receive a lot of abuse. Users often modify the columns during the life of the structure by upgrading or changing the crane capacities or adding additional aisles or bays to the existing structure, consequently altering the original design parameters. Sometimes the process modifications may result in major changes to column loading.

Structural integrity of heavy mill building columns is greatly influenced by good judgment and prudent application of the important research to all its associated structural elements and their connections. The design of heavy mill building columns consists of two major elements: first, the evaluation of effective column lengths, and second, the evaluation of stability and strength of the column. Additional important aspects are column lacing, bracing requirements, base fixity and rotational restraint, support settlement, lateral drift and column stiffness considerations, connections, and fixed column bases.

3.1. K-Factors and End Restraints

End restraint is one of the most important factor that impacts the column strength. This realization led to the development of the concept of effective length of columns, to determine the strength of a column based on its true end conditions to that of an equivalent pinned end condition. Research has confirmed that the effective length of a column KL is equal to the distance between the inflection points of a perfectly straight column when it is buckled.[2] This in turn facilitates easier determination of column strength. End restrained K factor approach is the most practical approach in determining the column strength. If K factor is incorporated into the Euler's classical equation, the critical load (P_{CR}) can be written as:

$$P_{CR} = \frac{\pi^2 EI}{(KL)^2} \tag{1}$$

Effective length (KL) of any column depends on the end conditions at the ends of its unbraced lengths. It is not realistic to assume a true pinned end condition or true cantilever (fixed-free) condition for any column when calculating the K factor. Columns are connected to structural members, which offer a certain amount of rotational restraint. K value for a cantilever condition is always > 2.0, or for Fixed ends condition K is < 1.0. Usually the true K value is somewhere between "Fixed-Free (K = 2.0)" and "Fixed-Fixed (K = 0.5)" conditions. Any wrong judgment in the end restraint conditions would result in an unacceptable margin of error in the evaluation of column strength. For example, the column strengths could vary from 0.25 ($\pi^2 EI / L^2$) for a cantilever to 4.0 ($\pi^2 EI / L^2$) for a condition of fixed ends, depending upon the assumption. There is a wide range between 0.25($\pi^2 EI / L^2$) and 4.0($\pi^2 EI / L^2$). Therefore, it is extremely important to carefully determine the K factors by applying the realistic end restraint conditions. There is a great deal of research work and lot of refinements in the methods of calculating realistic K values are available.[1,2,3,4]

Determination of effective length for mill building columns is far more difficult due to geometric and loading irregularities with various conditions of loading and end restraints. Therefore, the effective length for the same column is different for different loading combinations and also for different end restraint conditions. These columns are commonly referred to as "Stepped Columns" or "Combined Columns." In this paper these terms are interchangeable without any change in meaning. Extensive literature is available in this area.[5,6,7,8,9,10,11,12,13]

In spite of valuable literature available, confusion still exists in determining the realistic effective length factors. This is primarily due to several variables involved with respect to different segments of combination section, axes, and end restraint conditions, all of which are affected by different loading conditions. Clear understanding of K value parameters is essential for any mill building column design. The Murray-Graham model[14] is the most practical model for stepped columns and determines the basic design conditions of mill building columns. Thus, it will be followed throughout this paper.

The following are the basic concepts of the Murray-Graham model for mill building design:

a. Provide continuous bottom chord bracing.
b. Maximum crane live loads can occur only at one column at a time. Therefore, top of the column is prevented from translation. This is due to the fact that the bottom chord bracing transmits horizontal forces to the unloaded columns, which in turn resist any translation. The structure is considered to be, essentially, a space frame. That is, for crane loading conditions, the column is fixed at the bottom and pinned at the top (side sway prevented). Assume the location of the pin at the bottom chord of the trusses for those columns without knee braces or at midway between the bottom chord and the knee brace for those columns with knee brace.
c. When wind loads are applied to the structure, each frame must resist its own share of the wind attributable to that frame. For this reason, the space frame concept mentioned above is no longer valid for this loading condition. Side sway is therefore assumed to occur at each frame. Also, restraint against rotation at the truss level is assumed. That is, columns subjected to wind loading are assumed to be fixed at the bottom and act as sliders at

the top (translation permitted and rotation restrained). Location of the slider is the same as for pins in the case of crane loading.

Anderson-Woodward[5] developed a computer program for calculating the K values that would permit the use of the Murray-Graham model. Agarwal-Stafiej[6] developed tables with a wide range of values for various end conditions based on the Anderson-Woodward[5] equations. The 1991 edition of AISE[7] provides a wider range of values that can be directly used to calculate K values. The K factors tabulated in AISE are strictly applicable to those columns connected by continuous web plates and are approximately to laced columns. The ratios of lengths and moment of inertias as well as the ratios of applied axial loads at both the segments of the column are required to access these tables. The author recommends the use of either of the above tables to obtain K values since these tables are accurate and convenient to use.

Since the end restraints are different for crane and wind loads, review of these two categories separately and selection of one or two controlling loads for each, based on experience and judgment, will reduce computational time. Where this is not possible, all the load combinations for each category require the same process. For convenience, the author introduced subscripts to commonly used terminology for each column shaft. The subscript for building column is B, for crane column is CR. For example, X_{CR} denotes the X axis of the crane column and Y_C is the Y axis of the combined column and so on (see Figure 2, Figure 3, Figure 4).

The following are the necessary steps involved for obtaining K values of a heavy mill building columns for each controlling loading.

Step 1.

Only the K factors involving the strong axis of the combined column (X_C) are to be calculated. Values for a weak axis (Y_C) will be assumed (see Table 1).

Step 2.

Calculate crane loading case for bottom fixed and top pinned condition.

Step 3.

Calculate wind loading case for bottom fixed and top slider condition.

Step 4.

Two values are needed in the strong axes for each loading condition as shown in Table 1.

Step 5.

KX_{C1} is the value for crane loading for the combined column in its strong axis.

Step 6.

KX_{C2} is the value for wind loading for the combined column in its strong axis.

Step 7.

In the weak axis for lower segment (combined column), building shaft is neglected AISE[7] and crane shaft alone condition will be considered. Therefore, KY_C for lower segment is equal to KX_{CR}. With a fixed base and pinned top the $KY_C = 0.8$.

Step 8.

For upper segment in the weak axis the end conditions are considered to be pinned. Therefore, KY_C for upper shaft is equal to KY_B. For pinned ends of upper shaft $KY_B = 1.0$.

Step 9.

Therefore, in the weak axis of the combined column (Y_C) only two values are required and they need not be calculated since these values are assumed ($KX_{CR} = 0.8$ and $KY_B = 1.0$).

Table 1 is intended as a guide for determining the required K values for heavy mill building stepped columns with their corresponding end restraints. It is also important to under-

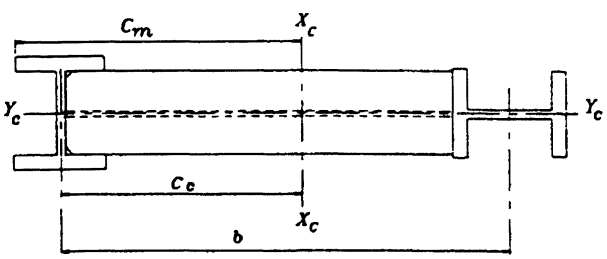

Fig. 2. Section at lower segment (combined column).

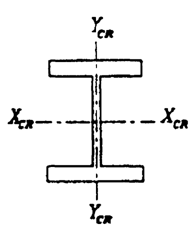

Fig. 3. Section at crane column.

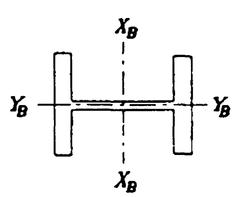

Fig. 4. Section at building column.

Table 1. Effective Length Factors for Combination Columns				
1	2	3	4	5
Axes & End Restraint	Strong Axis X_C		Weak Axis Y_C	
Column Segment	Fix/Pin for Crane Loading	Fix/Slider for Wind Loading	Pin/Pin	Fix/Pin
Upper Segment	—	—	Single Shaft $KY_C = KY_B = 1.0$	—
Lower Segment Combined Section	KX_{C1}	KX_{C2}	—	Neglect Building Shaft $KY_C = KX_{CR} = 0.8$

stand the actual lengths to be used in calculating these K factors:

For the strong axis of the combined column (X_C), use full length of the column, from point A to point C (L_T).

Weak axis of the combined column (Y_C): For lower segment use crane column height from point B to point C (L_2) and for upper segment use unbraced building column height only between points A and B (L_1).

3.2. Strength and Stability Checks

In order to perform strength and stability checks, the structural analyses of columns must be performed and the governing forces and moments must be selected based on various loading conditions. Preliminary design is not only helpful but essential in order to reduce final design hand calculation time.[11] With the advent of computers most of the tedious analysis can be eliminated by utilizing the appropriate computer programs and good preliminary designs can be easily achieved. Long hand frame analysis is rarely performed. The applicable loads and loading combinations are clearly defined in AISE[7] and the application of these loads are presented in various publications.[10,11,15,16,17] The analysis of mill building frames is simple if an appropriate computer analysis program is utilized with proper assumptions. The following are a few points of interest in accomplishing this goal.

a. Computer analysis should be performed for two conditions (a) for crane loading condition, and (b) for wind loading condition. For crane loading condition, the load sharing concept must be included. This can be accomplished by loading the frame with a fictitious restraining force at the bottom chord level for crane loading cases. This restraining force is usually two-thirds the reactive force at the bottom chord level[11] (not the crane horizontal thrust at the crane column level). The author believes that the above value can be conservatively used for almost all the heavy mill building frames. This is particularly valid for buildings with minimum two bays with multiple aisles. Furthermore, the continuous horizontal bottom chord bracing is stiff for heavy mill buildings.

b. For wind loading condition the fictitious restraining force at the bottom chord load must be removed.

c. All the resulting forces should be separated for both the conditions for calculating K factors and performing stress checks.

d. All shafts must be checked for their full locally applied loads.

The strength and stability of the members subjected to combined stresses must be checked by the interaction formulas. AISC[4] interaction formulas H1-1 and H1-2 are modified to reflect the geometric non-linearities and load irregularities that are present in stepped columns. These equations essentially place stress limits on the extreme corner fibers of a column section.[18] AISE[7] modified the interaction equations for the application to mill building columns. AISE equation 14 represents the stability check (increase of maximum bending stresses due to applied axial loads on a deflected column shape). AISE Equation 15 represents the strength check (maximum bending stresses are not affected by the applied axial loads on a deflected column shape).

AISE Equation 14:

$$\frac{f_a}{F_a} + \frac{C_{mx} f_{bx}}{\left[1 - \left(f_a / F'_{ex}\right)\right] F_{bx}} + \frac{C_{my} f_{by}}{\left[1 - \left(f'_a / F'_{ey}\right)\right] F_{by}} \leq 1.0 \tag{2}$$

AISE Equation 15:

$$\frac{f_a}{0.6F_y} + \frac{f_{bx}}{F_{bx}} + \frac{f_{by}}{F_{by}} \leq 1.0 \tag{3}$$

Proper application of the terms in the AISE equations 14 and 15 is complex and cumbersome from a computational standpoint. Lack of clear understanding would result in erroneous results. These terms are applicable only to particular K values. Some of the terms are not applicable to heavy stepped columns consisting of two or more shafts. The confusion is compounded by the fact that the upper segment and the lower

Table 2.
Stability and Strength Checks for Combination Columns and Application of *K* Factors

1		2	3	4	5	6	7
AISE Equations		**AISE—Equation 14**			**AISE—Equation 15**		
Loading and Segments		**Term1**	**Term 2**	**Term 3**	**Term 4**	**Term 5**	**Term 6**
Crane Loading	Upper Segment	$KY_C = KY_B = 1.0 \rightarrow F_a$	$KX_{C1} \rightarrow F_{ex}'$	N/A	—	—	N/A
	Lower Segment	$KX_{C1}, KY_C \rightarrow F_a$	$KY_C \rightarrow F_{bx}$ $KX_{C1} \rightarrow F_{ex}'$	$KY_C = KX_{CR} = 0.8 \rightarrow F_{ey}'$	—	$KY_C = KX_{CR} = 0.8 \rightarrow F_{bx}$	—
Wind Loading	Upper Segment	$KY_C = KY_B = 1.0 \rightarrow F_a$	$KX_{C1} \rightarrow F_{ex}'$	N/A	—	—	N/A
	Lower Segment	$KX_{C2}, KY_C \rightarrow F_a$	$KY_C \rightarrow F_{bx}$ $KX_{C1} \rightarrow F_{ex}'$	$KY_C = KX_{CR} = 0.8 \rightarrow F_{ey}'$	—	$KY_C = KX_{CR} = 0.8 \rightarrow F_{bx}$	—

segment should be checked separately for each governing crane loading condition as well as the governing wind loading condition. It can be seen from Table 2 that it would require calculation of 20 terms in order to check one wind loading condition and one crane loading condition for the same combined column. Therefore, non-governing load combinations should be intuitively eliminated. Elements of each term are explained in AISE.[7]

3.3. Column Design—Recommended Procedure

a. Analyze frame for gravity loads plus crane loads, and wind and seismic loads as previously described. Segregate crane loading combinations from the wind and seismic loading combinations. Limit the governing design loads for each category.
b. Perform preliminary design of upper and lower segments utilizing either a computer program or by using short-cut long-hand methods.
c. Enter Table 1 for each controlling design loading and determine the *K* factors to be calculated and their corresponding end restraint conditions. In the strong axis of the combined column, use length L_T (AC) In the weak axis of the combined column, use length L_2 (BC) for crane column and use length L_1 (AB) for building column.
d. Perform strength and stability checks for upper and lower segments separately for each controlling loading condition. Enter Table 2 to verify appropriate *K* value to be used for each term of the AISE Equations 14 and 15.

4. DESIGN OF COLUMN LACING AND END STAY MEMBERS

In order to achieve the combined behavior and prevent local failure of heavy stepped columns consisting of two or more shafts, a realistic method of lacing design is important. Significant reduction in the buckling capacity can occur due to inadequate provision for horizontal shear flow (VQ/I) between the longitudinal axes of the column shafts. Horizontal shear in a combined column is caused by the application of moments, eccentricities, and transverse forces. Combined column failures prior to 1940 confirm the importance of lacing design.[13] Lack of adequate shear resistance of battens or lacing would result in a spaced column rather than a combined column. Some light to medium duty mill buildings are designed as spaced columns.

In case of the spaced column, the buckling strength is equal to the sum of the critical loads for individual shafts. For spaced columns, combined behavior of the columns cannot be assumed, although a combined column yields more economical design.

Horizontal shear could be resisted by connecting the shafts by means of one of the following:

a. Continuous Column Web Plate
b. Battens
c. Lacing

In the present days of modern steel construction, column web plates are neither economical nor practical. This system also eliminates the usable space between the column shafts, which is commonly used for plant utilities. Battened columns are connected by horizontal batten elements. The shear transfer occurs by their own stiffness and the stiffness of their connections against shear and moment (analogous to a roof truss without diagonal web members). Battened columns possess a greater degree of shear flexibility than the other systems. They result in higher shear distortions and are least resistant to shear forces.[13] Battened columns are not recommended for the use of heavy mill building columns.

The design approach for lacing members and stay members is a matter of experience and the prudent application of a combination of references. The author recognizes AISC,[4] British Standard Code 449 and Galambos[13] as the primary tools in establishing a practical design approach. British Standard 449 recommends that the lacing be designed for the resulting force in the member due to lateral loads or bending in the combined column plus 2.5 percent of the axial loads in the column shafts applied as a transverse load at any point along the length of the combined column. AISC requires that the lacing be proportioned for a shear force equal to 2.0 percent of the axial forces in the columns only. Lacing members shown in Figure 1 are compression members. The lacing members should be as heavy as practical with rigid connections to develop end moments consistent with their shear capacity and to a degree determined by the actual maximum calculated shears.[7] Buckling strength of a combined column is significantly impacted by the stiffness of the end stay members near B and C (see Figure 5). End stay members are important elements of laced column. They distribute applied loads to lacing members and deliver horizontal force to column bases. Design of top stay member supporting the interior building column required careful attention to details and accurate identification of design loads (Figure 5).

4.1. Recommended Design Procedure for Lacing System

a. Analyze the frame for both gravity and lateral forces. Obtain governing joint loads at Joints 1, 2, 3 (see Figure 5), and the forces in the members. All joints for stay members and lacing members are assumed to be fixed.
b. Design of the top stay member involves fixed end moments at joints, 2, 3, loads at joints 1, and compressive force in the member due to applied horizontal forces (i.e., axial load plus governing moment). Moment connections are required at the ends of this member and must be able to resist the larger of M_2 and M_3. The connection must also resist larger of V_2 and V_3. Web stiffeners are frequently required under the location of the building shaft. Gusset plates along the force path of the intersecting lacing member should be considered. It is recommended that a deep and stiff member be chosen. If it is not possible, a built up member may be used. For seismic loads, the out of plane moments at joint 1 may induce torsional forces in this member. A built up closed section will be appropriate for that condition where St. Venant's torsional stresses will govern.
c. Connection at the bottom of building shaft should be designed as an end plate connection for the governing M_1, P_1, and H_1 forces. The force P_1 may not have influence on the connection unless it an uplift force.
d. Column lacing members and bottom stay member will be designed for the governing compressive force in the member plus an additional component of the force equal to 2.5 percent of the summation of maximum column axial loads which is applied as a transverse shear force at any point along the length of column.

The design compressive force in a lacing member or in the bottom stay member = (calculated compressive force due to applied transverse loads and moments) + the component of $(0.025)(P_1 + P_2 + P_3)$. In Figure 5 only one layer of lacing is used, hence the lacing should be designed for the full force shown above. Where multiple planes of lacing are used, the force may be divided equally among all the planes of lacing.
e. AISC recommends that, for lacing stiffness in single system of lacing, L / r should be less than or equal to 140 where L is the distance between welds.
f. Although the end stay members should be as close to the ends of the shafts as possible, the bottom stay member must clear the column base plate and anchor bolt assembly. This may result in unacceptable weak axis bending

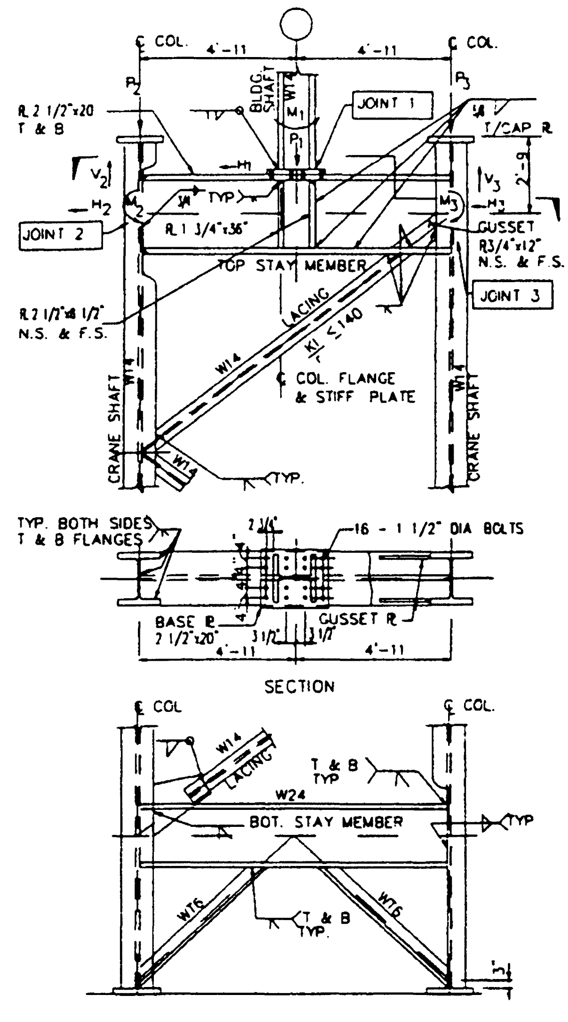

Fig. 5. Column lacing system.

in the column shafts. Additional lacing members as shown below the bottom stay member (see Figure 5) would alleviate this condition.

5. BRACING REQUIREMENTS

Out of plane bracing for the column unbraced lengths is assumed to be at points A, B and C (see Figure 1) from a carrying truss at A, heavy crane girders at B, and from column foundation at C. Additionally, vertical bracing is required in both the planes of crane shafts and the building shafts in the longitudinal direction of the building. Vertical bracing along the crane columns will resist longitudinal forces from crane such as crane starting and stopping forces, the horizontal impact forces at the crane stops, and the seismic forces in the longitudinal direction. Also, it will resist torsional forces between crane and building shafts. It is not practical to design crane column connections to resist torsional effects. Vertical bracing along building shaft will resist wind and seismic forces along the longitudinal direction of the building. Lateral bracing is required at the end walls (in the plane of the end frames) in order to resist the reactions transmitted through the roof diaphragm. This may not be possible due to large opening requirements for mill operations in which case the end frames must be designed to resist such loads. Bottom chords of the trusses need to be braced the full length of the building.[7,14] This will facilitate load sharing capacity between the columns by providing a rigid diaphragm at the truss bottom chord level.[14] Bottom chord bracing also stabilizes the bottom chords when they are in compression due to wind uplift.

6. BASE FIXITY, ROTATIONAL RESTRAINTS, AND SUPPORT SETTLEMENTS

For heavy mill building columns base fixity must be provided. It is reasonable to assume full fixity for all deep foundations with battered piles and caissons, or shallow foundations (spread footings) bearing on rock or similar strata. In order to achieve full fixity, the column bases must be provided with positive anchorage against rotation. For crane loading conditions the base fixity can be conservatively assumed due to load sharing of adjacent columns,[14,11] restraining effects of less strained columns,[13] and transitional nature of crane loading. The maximum crane load can occur only at one column at any given time. Columns with two or more shafts are best suited for providing full base fixity. Base rotations are rarely a concern because of the lever arm generated by stiff building and crane shafts. In heavy mill building columns elongation of anchor bolts affecting rotational restraint is not a realistic issue. Base fixity not only yields an economical structure but also provides better lateral drift control. Columns with fixed bases offer higher stiffnesses due to reduced K values. Positive frame stability can be achieved. On the other hand pinned base columns will result in excessively large moments at the upper portions of the columns.[11]

Problems due to support settlements are also generally rare. The impact of minor settlements, if they do occur, is insignificant in terms of column design. Settlement (Δ) at point C in Figure 1 will cause a fixed end moment at roof level.

$$\text{Fixed end moment} = \frac{6EI\Delta}{L^2} \tag{4}$$

However, the design assumptions for the top end conditions of the column are either a slider or a pin. There is no moment transfer to the column, but the roof truss may undergo minor rotation and may experience minor reversal of stresses in the truss members. When the soil conditions indicate possible settlements the structure could be designed for those anticipated differential settlements. For crane runway system this can usually be corrected by the installation of leveling plates under the crane girder bearing.

Contrary to the effects of support settlement, rotational slip (θ) at the fixed column base will cause a moment release at the column base.

$$\text{Fixed end moment (released)} = \frac{4EI\theta}{L} \tag{5}$$

One half of the above moment will be imposed at the top end of the column. The moment release at the column base and additional imposed moment at the top end of the column will have detrimental effect on lateral drift control for the column. Unless the columns are supported on shallow foundations with poor soil conditions, properly designed fixed column base connections (see Figure 6) can prevent the rotational slip.

7. LATERAL DRIFT AND STIFFNESS CONSIDERATIONS

Lateral drift consideration for the entire height is not as important to a mill building as it is for multistory construction. It is a good design practice to limit the flexibility of the building in order to protect cladding connections, and window glass, etc. Fisher[19] recommends the building drift be limited to $H / 100$ for buildings with pendent operated cranes and $H / 240$ for the buildings with cab operated cranes. For a heavy mill building column this criteria rarely controls. Furthermore, it is important to control the column displacements at the crane level for smooth crane travel and to limit maintenance costs related to the crane runway system and crane equipment. Any limits placed on drift control will increase the cost. Therefore, the building codes do not specify any limits for drift control for mill building columns. Generally, the top of crane runway is less than 80 feet above mill floor for heavy mill buildings. Using a drift limitation of $H / 500$ due to crane loads at the crane level is adequate for heavy mill building columns. This limitation may be relaxed to H / 400 for other transient loads such as wind and seismic forces at the crane level. Lateral sway at the elevation of crane runway should be limited to roughly one inch.[19,20]

8. DESIGN OF FIXED COLUMN BASES

It is recommended that all heavy mill building columns be designed for full base fixity. The design of column bases must be consistent with this assumption. It is important to recognize that the column base fixity can be achieved only when the column flanges are anchored to the foundation. The couple generated by the restraining moment is delivered by the flanges. Therefore, the flanges must be properly connected to the foundations. This means the anchor bolts must have adequate tension capacity to transmit the force to the foundation. Complete design of the column base consists of providing for the transmission of the axial loads and the base shears from the column shafts to the foundation as well. The positive method to achieve this is to provide for a clearly defined path for these forces into the foundation system (see Figure 6). Column axial load requires an adequate bearing area and stiffness in the base plate. Bearing ends of the columns and the top of base plate under the column should be finished to attain uniform contact. Column moments require adequate anchorage and the base shear must be transmitted by the shear lugs at the bottom of base plate. In order to transmit base shear, adequate field welds must be provided at the bottom of the column to the base plate. For heavy mill building columns, anchor bolts should not be designed for the combination of tension and shear. Dependence on the friction between concrete and the base plate to resist base shears is not prudent. Because of long term relaxation of concrete, prestressing of anchor bolts is unreliable and hardly ever justified.[21]

Anchor bolt and base plate design forces for a combined column should be obtained as shown in Figure 7. Forces and moments shown are reversible in direction. Correction due to neutral axis location is not necessary for obtaining anchor bolt forces but any axial tension due to wind or seismic load combinations must be properly accounted for. It should be noted that the design forces for anchor bolts and base plates are often governed by different loading combinations.

Base plate design is relatively simple as compared with a single shaft column subjected to an applied moment at the bottom of the column. As shown in Figure 7, the base plates can be designed for a direct axial loads imparted by the individual column shafts. Area of the plate should be proportioned on the basis of allowable bearing stress (F_p) of steel on concrete as specified in AISC Section J9. On a full area of concrete support $F_p = 0.35f_c'$. This could be increased up to $0.7f_c'$ with certain constraints. Since the base plates will be large and close to each other with space limitation for the concrete surface area, the F_p value of $0.35f_c'$ may used conservatively. Thickness of the base plates should be calculated per the AISC design procedure. Maximum plate thickness should be limited to 8 inches for $F_y = 36$ ksi due to material availabil-

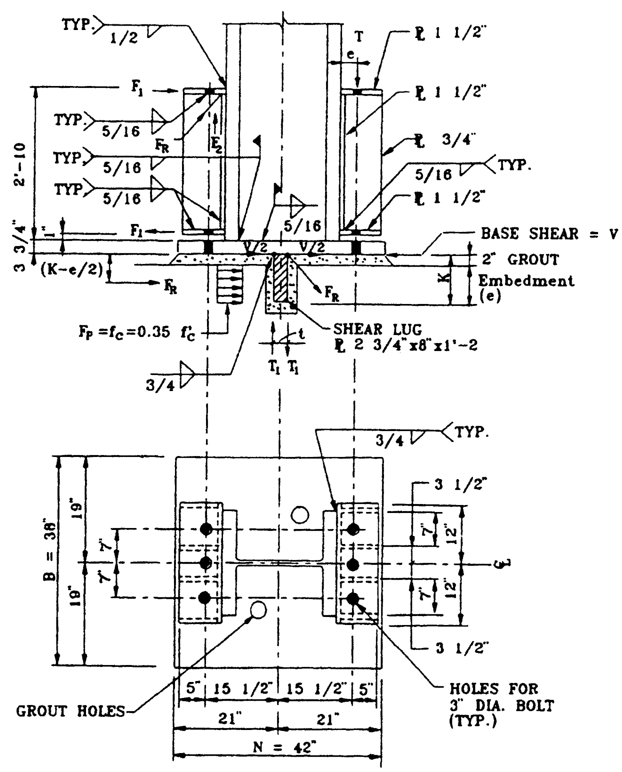

Fig. 6. Base plate.

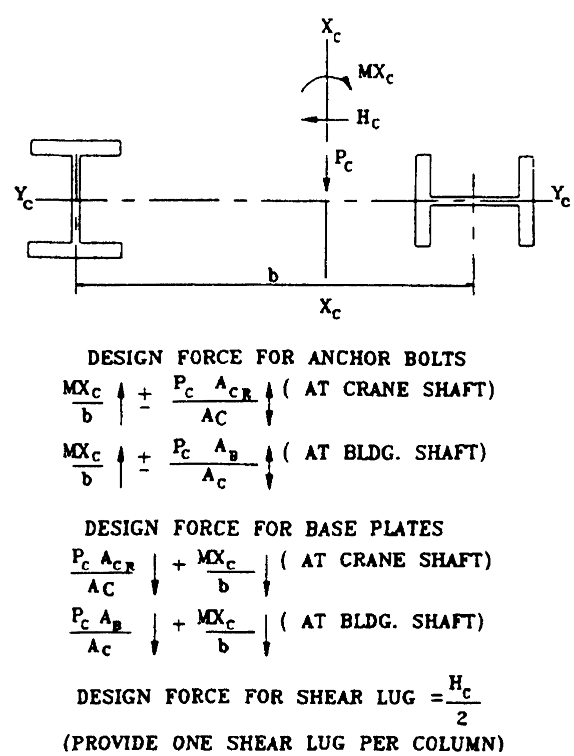

Fig. 7. Design forces for base plates and anchor bolts.

ity. For thickness beyond 8 in. the value of F_y should be reduced to 32 ksi.

Anchor bolt design is also direct since the tensile force is uniformly shared by all the anchor bolts at any given shaft. However, the weldments and the design of stiffener plates for the anchor bolt assembly require special attention. It is recommended that the anchor bolt assembly (bolt cage) be designed for the full capacity of the anchor bolts rather than the actual force being resisted by the anchor bolts. The bolt holes should be oversized per AISC. In order to prevent accidental rotation due to loose bolts and to account for repetitive loading, all column anchor bolts should be provided with double nuts and must be tightened in place after the columns are plumb. In order to develop the tension capacity for large diameter anchor bolts, mechanical anchorage should be used for embedment rather than utilizing bond stress between concrete and the plain bolts.

Shear lug should be designed as a cantilever with the fixed end at the bottom of the base plate. It should be proportioned for an allowable bearing stress (F_p) = $0.35f_c'$ ignoring the grout thickness below the base plate. Weldment between the shear lug and the base plate must be symmetrical about the longitudinal (weak) axis of the plate with end return of the welds for the full thickness of the plate.

9. CONCLUSION

This paper has presented an integrated solution for the mill building columns subjected to heavy crane loads. Several important factors influencing the design of heavy mill building columns have been addressed. Calculation of K factors and the necessary steps involving the determination of K factors for stepped columns are discussed. AISE equations and the application of the terms involved are clarified. Design procedures are recommended for column design, and for the column lacing system. Design of fixed column bases for heavy mill building columns is discussed in detail. A numerical example is presented to illustrate the procedures discussed.

10. NUMERICAL EXAMPLE

10.1. Column Design

Design data from preliminary calcs and frame analysis (exterior column with two shafts)

Lower Shafts (Figure 3 and Figure 4)
Crane column = W14×342: IX_{CR} = 4,900 in.^{4}
IY_{CR} = 1,810 in.^{4}, SX_{CR} = 559 in.^{3}, A_{CR} = 101.0 in.^{2}
rx_{CR} = 6.98 in., ry_{CR} = 4.24 in.
Building column = W14×455
Crane shaft length = L_2 = 68 feet
Crane reaction = P_2 = 510 k

Upper Shaft (Figure 4)
Building column = W14×455: IX_B = 7,190 in.^{4}
IY_B = 2,560 in.^{4}, SX_B = 756 in.^{3}, A_B = 134.0 in.^{2}
rx_B = 7.33 in., ry_B = 4.38 in.
Building reaction = P_1 = 136 k
Building shaft length = L_1 = 27 ft

Properties of combined lower segment and design loads. (See Figure 1 and Figure 2)

Distance between building shaft and crane shaft
b = 4 ft-6 in.
IX_C = 176,940 in.^{4}, SY_C = SX_{CR} = 559 in.^{3},
A_C = 235.0 in.^{2}, rx_C = 27.4 in., ry_C = 5.63 in.,
$L_T = L_1 + L_2$ = 95 ft,
C_m = 39.00 in., C_C = 30.8 in. (see Figure 2).

Governing loading = dead load + live load + crane load
Governing forces and moments:
$P_1 + P_2 = P_C$ = 136k + 510k = 646k
Combined column moment @ B = MX_C @ B = 2,850 k-ft
Combined column moment @ C = MX_C @ C = 3,635 k-ft
Crane column moment due to crane load eccentricity = MY_C = MX_{CR} = 210 k-ft
Building column moment @ B = MX_B @ B = 385 k-ft
Building column moment @ A = MX_B @ A = 536 k-ft

10.1.1. Check Lower Segment BC—AISE Equation 14

A. First term Equation 14 → need f_a and F_a

Step 1. Calculate f_a for combined column.

$$(f_{a_c}) = \frac{P_1 + P_2}{A_C} = \frac{136 + 510}{235} = 2.75 \text{ ksi}$$

Step 2. In order to find F_a for lower segment, first find K factor for lower segment. Since this is a crane loading combination, calculate KX_{C1} and use $KY_C = KX_{CR}$ = 0.8. KX_{C1} is obtained from AISE Table 8 Hinge @ A and Fix @ C condition. Calculate numerical values in order to enter AISE Table 8

$$a = \frac{L_1}{L_T} = \frac{27}{95} = 0.28,\ B = \frac{IX_C}{IX_B} = \frac{176{,}940}{7{,}190} = 24.6$$

$$\frac{P_1}{P_2} = \frac{136}{510} = 0.27$$

Read KX_{C1} = 1.12 from Table 8 AISE (by interpolation)

$$\frac{(KX_{C1})(L_T)}{rx_C} = \frac{1.12 \times 95 \times 12}{27.4} = 46.6$$

$$\frac{(KY_C)(L_2)}{ry_C} = \frac{0.8 \times 68 \times 12}{5.63} = 116$$

Larger value 116 governs

Step 3. Read $(F_a)_C$ = 10.85 ksi, for a value of 116 from AISC Table C-36.

Step 4. Calculate first term

$$\frac{f_a}{F_a} = \frac{2.75}{10.85} = 0.25$$

B. In order to calculate second term the values for C_{mx}, F_{bx}, F_{ex}', f_{bx}, and f_a are required. Assume $(C_{mx})_C = 0.95$ for bending about axis (see AISE[7] for explanation of these terms)

Step 5. Calculate F_{bx}. This requires calculation of F_a for crane column alone condition (MY_C is assumed to be resisted by crane column alone about its strong axis X_{CR}). Assume

$$\frac{(KY_C)(L_2)}{rx_{CR}} = \frac{0.8 \times 68 \times 12}{6.98} = 93.5 \text{ (use 94)}$$

$$(F_a)_{CR} = 13.72 \text{ ksi}$$

$$F_{bx} = F_a\left(\frac{C_m}{C_C}\right) = 13.72\left(\frac{39.00}{30.8}\right) = 17.37 \text{ ksi}$$

$F_{bx} \le 0.6F_y$, $F_{bx} = 17.37$ ksi **governs**

Step 6. Calculate F_{ex}'. The r value in the plane of bending for combined bottom segment about X_C axis is rx_C. Use $KX_{C1} = 1.12$ from Step 2.

$$\frac{(KC_{C1})(L_T)}{rx_C} = \frac{1.12 \times 95 \times 12}{27.4} = 46.6$$

Read $F_{ex}' = 68.79$ ksi from AISC Table 8.

Step 7. Calculate f_{bx}.

$$f_{bx} = \frac{(MX_C)_{\max}(C_m)}{IX_C} = \frac{3{,}635 \times 12 \times 39.00}{176{,}940} = 9.6 \text{ ksi}$$

Step 8. Use $f_a = 2.75$ ksi from Step 1.

Step 9. Now compute second term of AISE Equation 14 for lower segment.

$$\frac{C_{mx} f_{bx}}{[1 - (f_a / F_{ex}')]F_{bx}} = \frac{0.95 \times 9.6}{\left[1 - \left(\dfrac{2.75}{68.79}\right)\right]17.37} = 0.55$$

C. Calculate third term of Equation 14. Requires calculation of C_{my}, f_{by}, f_a', F_{ey}', and F_{by}.

Step 10. For crane shaft with fixed base and pinned top assume $C_{my} = 0.4$.

Step 11. Calculate f_{by}. Moment MY_C axis is assumed to be resisted by crane shaft alone. Any contribution from building shaft is neglected. Therefore, Y axis for combined column is the X axis for crane column ($Y_C = X_{CR}$)

$$f_{by} = \frac{MY_C}{SX_{CR}} = \frac{210 \times 12}{559} = 4.51 \text{ ksi}$$

Step 12. Calculate f_a'. Use average bending moment about X_C axis between points B and C of the combined column. (see Figure 1).

$$MX_{average} = \frac{MX_C \text{ @ } B + MX_C \text{ @ } C}{2}$$

$$= \frac{2{,}850 + 3{,}635}{2} \text{ k-ft,243}$$

$$f_a' = (f_a)_C \pm \frac{(MX_{avg.})(C_C)}{IX_C}$$

$$= 2.75 + \frac{3{,}243 \times 12 \times 30.8}{176{,}940} = 9.5 \text{ ksi}$$

Step 13. Calculate F_{ey}' Use KX_{CR} since this is the axis of bending.

$$\frac{KY_C(L_2)}{rx_{CR}} = 94.0 \text{ (from Step 5)}$$

Read $F_{ey}' = 16.9$ ksi from AISC Table 8 for a value of 94.

Step 14. Since crane column is W14×342 which is fully supported laterally by column lacing and the section meets all AISC requirements (Section F1)

$$F_{by} = (F_{bx})_{CR} = 0.66F_y = 24 \text{ ksi}$$

Step 15. Compute third term of AISE Equation 14 for lower segment.

$$\frac{C_{my} f_{by}}{[1 - (f_a' / F_{ey}')]F_{by}} = \frac{0.4 \times 4.51}{\left[1 - \left(\dfrac{9.5}{16.9}\right)\right]24} = 0.17$$

Step 16. Calculate sum of all terms of the interaction Equation 14.

$0.25 + 0.55 + 0.17 = 0.97$ **o.k.** ≤ 1.0

10.1.2. Check Lower Segment BC for AISE Equation 15.

Since the moments MX and MY are different at locations B and C and there is no provision for MX average as in Equation 14, Equation 15 should be checked at both points B and C. However, intuitive judgment with maximum MY value and the corresponding MX value will reduce the calculation time.

Step 17. Collect previously calculated data

$(f_a)_C = 2.75$ ksi (from Step 1)
$f_{by} = 4.51$ ksi (from Step 11)
$F_{bx} = 17.37$ ksi (from Step 5)
$F_{by} = (F_{bx})_{CR} = 24$ ksi (from Step 14)

Step 18. Calculate $(f_{bx})_C$ @ B

$$(f_{bx})_C = \frac{(MX_C \text{ @ } B)C_m}{IX_C} = \frac{2{,}850 \times 12 \times 39}{176{,}940} = 7.54 \text{ ksi}$$

Step 19. Substitute values from Steps 17 and 18 into the AISE Equation 15

$$\frac{f_a}{0.6F_y}+\frac{f_{bx}}{F_{bx}}+\frac{f_{by}}{F_{by}}=\frac{2.75}{22}+\frac{7.54}{17.37}+\frac{4.51}{24}$$

$$= 0.125 + 0.434 + 0.188 = 0.747 \quad \textbf{o.k.} \le 1.0$$

Step 20. Repeat Steps 17 through 19 for values at location C if necessary.

10.1.3. Check upper segment AB-AISE Equation 14 and 15

The bending in Y axis, Y_B for upper shaft is usually assumed to be zero. From Table 2 it can be seen that only two terms for each equation need to be evaluated

Step 21. Calculate F_a. For upper shaft the unsupported length (AB) is the unbraced length of the building column in Y_B axis.

$$\frac{(KY_B)L_1}{ry_B}=\frac{1.0\times 27\times 12}{4.38}=73.97 \text{ (use 74)}$$

Read $(F_a)_B$ = 16.01 ksi from AISC Table C-36.

Step 22. Calculate f_a

$$f_a=\frac{P_1}{A_B}=\frac{136}{134}=1.02 \text{ ksi}$$

Step 23. Calculate F_{bx} per AISC section F1-3. Note: The value of C_b = 1.0 for crane loading.

$$F_{bx} = 22 \text{ ksi}$$

Step 24. Calculate f_{bx} for building shaft.

$$f_{bx}=\frac{MX_B}{SX_B}=\frac{536\times 12}{756}=8.5 \text{ ksi}$$

Step 25. Assume C_{mx} = 0.4
Use F_{ex}' = 68.79 ksi (from Step 6)

Step 26. Now substitute values from Steps 21 through 25 into Equations 14 and 15.

$$\frac{f_a}{F_a}+\frac{C_{mx}f_{bx}}{(1-f_a/F_{ex}')F_{bx}}=\frac{1.02}{16.01}+\frac{0.4\times 8.5}{\left[1-\left(\frac{1.02}{68.79}\right)\right]22}$$

$$= 0.064 + 0.157 = 0.22 \quad \textbf{o.k.} \le 1.0$$

$$\frac{f_a}{0.6F_y}+\frac{f_{bx}}{F_{bx}}=\frac{1.02}{22}+\frac{8.5}{22}$$

$$= 0.046 + 0.386 = 0.43 \quad \textbf{o.k.} \le 1.0$$

In the above example the upper and lower segments were checked for only one governing load resulting from crane loading. Similarly, other controlling cases must be checked separately.

10.2. Lacing Design

Interior column has three (3) shafts, two (2) crane shafts and one (1) building column shaft (see Figure 5).

Governing loads from different frame analysis

$$P_1 + P_2 + P_3 = 3{,}611 \text{ k}$$

Maximum axial force in the lacing member due to applied lateral forces and moments in the column = 126 k

Angle subtended by the lacing member with the horizontal $\alpha = 38.4°$

Force in the lacing due to 2.5 percent of the maximum vertical load (90 k) applied as a transverse load at any point on the combined column:

$$=\frac{90}{2\cos\alpha}=58 \text{ k}$$

Total design force in the lacing member = 126 + 58 = 184 k (compression). Length of lacing member (see Figure 8):

$$=\sqrt{(9.83)^2+\left(\frac{15.58}{2}\right)^2}=12.5 \text{ ft}$$

From AISC column load tables using KL = 12.5 ft, use W14×43 for lacing member.

Axial capacity = 194.5 k ≥ 184 k **o.k.**

10.3. Design of Column Anchor Bolts and Base Plates

The following data is obtained from a different frame analysis:

MX_C = 3,635 k-ft, P_C = 812 k↓, b = 4 ft-6 in.,

A_C = 235 in.^{2}, A_{CR} = 101 in.^{2}, A_B = 134 in.^{2}

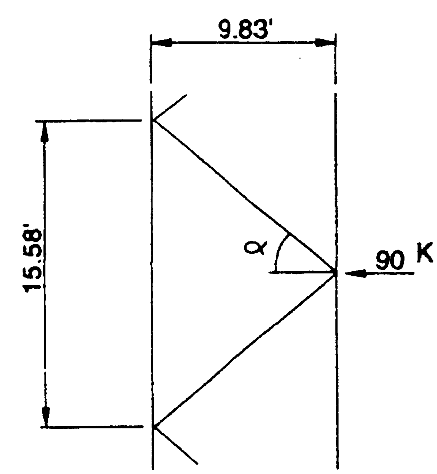

Figure 8.

Calculate anchor bolt and base plate loads for crane shaft W14×342 (see Figure 7)

Force in anchor bolts:

$$= \frac{MX_C}{b}\uparrow - \frac{P_C A_{CR}}{A_C}\downarrow$$

$$= \frac{3{,}635}{4.5} - \frac{(812)(101)}{235} = 459 \text{ k (tension)}$$

But wind loading plus dead weight governs. Tension = 713 k (from frame analysis)

Force in base plate:

$$= \frac{P_C A_{CR}}{A_C}\downarrow + \frac{MX_C}{b}\downarrow$$

$$= \frac{(812)(101)}{235} + \frac{3{,}635}{4.5} = 1{,}157 \text{ k (compression)}$$

Note: Design loads for anchor bolts and base plate are governed by different loading conditions.

10.3.1. Anchor Bolt Design.

Assume six bolts to resist tension.

Tension/Bolt = $\frac{713}{6}$ = 119 k. Use ASTM A36 bolts.

Area of bolt required = $\frac{119}{22}$ = 5.4 in.^{2} Try 3-in. diameter bolts.

Tensile stress area = 5.97 in.^{2} (For 3-in. diameter bolts)

Bolt capacity = $0.6F_y \times 5.97 = 129 \text{ k} \geq 119 \text{ k}$ **o.k.**

Use six 3-in. diameter bolts.

10.3.2. Base Plâte Design (see Figure 6)

Using AISC procedure page 3-106 (9th ed.), for column size W14×342.

Assume $f_c' = 3{,}000$ psi

Allowable bearing stress $F_P = 0.35f_C' = 1.05$ ksi

Area of plate = $\frac{1{,}157}{1.05}$ = 1,102 in.^{2}

In order to accommodate anchor bolt assembly, try plate 38 in.×42 in. (see Figure 6).

Area of plate provided:

$= 38 \times 42 = 1{,}596$ in.^{2} **o.k.** $> 1{,}102$ in.^{2}

f_p = calculated bearing stress = $\frac{1{,}157}{1{,}596}$ = 0.73 ksi

$B = 38$ in., $n = (B - 0.8b_f)0.5 = (38 - 0.8 \times 16.36)0.5 =$ 12.46 in.

$N = 42$ in., $m = (\mathrm{N} - 0.95d)0.5 = (42 - 0.95 \times 17.54)0.5$ = 12.67 in.

$$t_P = 2n\sqrt{\frac{f_P}{F_y}} = 2 \times 12.46\sqrt{\frac{0.73}{36}} = 3.55 \text{ in.}$$

$$t_P = 2m\sqrt{\frac{f_P}{F_y}} = 2 \times 12.67\sqrt{\frac{0.73}{36}} = 3.6 \text{ in.} \quad \textbf{governs}$$

Use plate 3¾-in.×38 in.×3 ft-6 in.

10.3.3. Design of Stiffener Plates (see Figure 6)

Bolt capacity = 129 k = T.

Moment at the face of column flange due to bolt eccentricity = T_e = 129 × 6.73 = 868 k-in. Assume vertical stiffener length = 30 in.

F_1 = Force couple/bolt/stiffener = $\frac{868}{30}$ = 29 k

Since the flange width is smaller than required width for bolt cage, provide a flange plate to extend flange width. Since the last stiffener plate is very close to the edge of the flange and the bolt location is within the flange width, a plate thickness of 1½-in. may be assumed.

A. Thickness of top horizontal plate:

Assume bolt load is uniformly distributed over the plate.

Width of plate = 11¾-in.

Stiffener spacing = 7 in.

Uniform load $w = \frac{129}{11.75 \times 7} = 1.57$ ksi

Assuming (3) span condition, moment in plate:

$$= \frac{wL^2}{10} = \frac{(1.57 \times 11.75)7^2}{10} = 90 \text{ k-in.}$$

$$M = F_b S,\ F_b = 0.75F_y = 27 \text{ ksi}$$

$$90 \text{ k-in.} = 27\left(\frac{11.75t^2}{6}\right),\ t = 1.3 \text{ in.}$$

Use 1½-in.×11¾-in.×2 ft-0 in. for both top and bottom horizontal plates.

B. Vertical stiffener plates:

Load/Stiffener = 129 k (same as bolt capacity)

Size of vertical stiffener plate 10¼-in.×2 ft-6 in.

Thickness of stiffener:

$F_v = 0.4F_y = 14.4$ ksi

Force = $F_v A$

$F_1 = 29 \text{ k} = 14.4(t)(10.25),\ t = 0.2 \text{ in} \rightarrow (a)$

$F_2 = 129 \text{ k} = 14.4(t)(30),\ t = 0.30 \text{ in} \rightarrow (b)$

AISC (Table B5.1) limiting b/t ratio $= \frac{253}{\sqrt{F_y}}$

$10.25/t = \frac{253}{6}$, $t = 0.25$ in. $\rightarrow(c)$

Resultant shear due to forces F_1 and F_2 (see Figure 6):

$$F_R = \left[\left(\frac{29}{10.25}\right)^2 + \left(\frac{129}{30}\right)^2\right]^{1/2} = 5.15 \text{ k / in.}$$

$F_v \times t = F_R$

$t = \frac{5.15}{14.4} = 0.36$ in.$\rightarrow$(d)

Checks (a) through (d) are necessary to estimate the plate thickness.

Check stiffener axial load capacity as a column due to its free edge (approximation).

Try plate ¾-in.×10¼-in.×2 ft-6 in.

$$I_y = \frac{10.25 \times 0.75^3}{12} = 0.36 \text{ in.}, A = 7.7 \text{ in.}^2,$$

$$r_y = \left(\frac{0.36}{7.7}\right)^{1/2} = 0.22 \text{ in.}, \frac{KL}{r} = \frac{0.5 \times 30}{0.22} = 68, F_a = 16.64 \text{ ksi}$$

Allowable axial capacity of stiffener plate: = 7.7 × 16.64 = 128 k **o.k.** (Close to bolt capacity, 129 k). Use plate = ¾-in.×10¼-in.×2 ft-6 in. for vertical stiffener plates.

10.3.4. Design of Weldments.

Force F_2 = bolt capacity = 129 k

A. Vertical weld at both sides of vertical stiffeners.

Minimum fillet weld required per AISC Table J2.4 = 5⁄16-in. Capacity of 5⁄16-in. (minimum fillet weld) for force F_2 both sides of stiffener plate = 278 k **o.k.** > 129 k. Use 5⁄16-in. fillet weld.

B. Weld at top of stiffener plate for force F_1

Capacity of 5⁄16-in. fillet weld 20.5 in. long = 95 k **o.k.** > 29 k.

Use 5⁄16-in. fillet weld both sides at top.

C. Weld between horizontal plates and column flange.

Force = 3 × F_1 = 3 × 29 = 87 k

Available length of weld = 16 in. Using ½-in. fillet weld 16-in. long, the capacity of weld = 118 k **o.k.** > 87 k. Use ½-in. fillet weld between column flange and horizontal plates.

D. Weld at flange plate and column flange:

Moment = 129 k × 6.73 in. × 3 bolts = 2,605 k in.

Shear = V = 129 × 3 = 387 k

Moment of inertia of two welds 30-in. long each:

$$I_{weld} = 2 \times \frac{30^3}{12} = 4{,}500 \text{in.}^4/\text{in.}$$

$$f_t = \frac{MC}{I} = \frac{2{,}605 \times 15}{4{,}500} = 8.7 \text{ k/in.}$$

$$f_v = \frac{V}{L} = \frac{387}{2 \times 30} = 6.45 \text{ k/in.}$$

F_R = resultant force in weld = $(8.7^2 + 6.45^2)^{1/2}$ = 10.8 k/in.

Weld size $= \frac{10.8}{0.707 \times 21} = 0.73$ in.

Use ¾-in. fillet weld between flange plate and column flange.

E. Check flange plate thickness:

$$F_v t = F_R,\ t = \frac{F_R}{F_V},$$

$t = \frac{10.8}{14.4} = 0.75$ in. **o.k.** < 1½-in.

F. Weld between column and base plate.

Horizontal shear 86 k (from frame analysis). Available length = 54 in. Use min. 5⁄16-in. fillet weld × 54 in. long. Capacity = 251 k, **o.k.** > 86k

10.4. Design of Shear lug (see Figure 6)

Horizontal shear at base plate = 86 k (from frame analysis). Try 8-in. deep lug. Set $f_C = F_P = 0.35f_c' = 1.05$ ksi. Embedment e = 8 in. − 2 in. = 6 in.

Required bearing area $= \frac{86}{1.05} = 82$ in.^{2}

b = length of lug $= \frac{82}{6}$ 13.67 in. use 14 in.

Moment at bottom of base plate:

$$= f_c\, be\left(k - \frac{e}{2}\right) = 1.05 \times 14 \times 6(5) = 441 \text{ k-in.}$$

(f_C is close to allowable value. Therefore, the value of 1.05 may be used without further calculations).

$$M = F_b S,\ S = \frac{bt^2}{6},\ F_b = 0.75F_y = 27 \text{ ksi}$$

Substituting, 441 = 27 × $\frac{14t^2}{6}$,

t = 2.65 in., use 2¾-in.×8 in.×1 ft-2 in. shear lug

Design of weldment:

Weldment is required at both sides of lug. Resisting force F_R produces a couple (T_1) at a distance equal to the thickness of the shear lug. (see Figure 6)

$$T_1 = \frac{V\left(K - \frac{e}{2}\right)}{t} = \frac{86 \times 5}{2.75} = 156.4 \text{ k}$$

F_R = resultant force:

$$F_R = \left[\left(\frac{V}{2}\right)^2 + T_1^{\,2}\right]^{1/2} = (43^2 + 156.4^2)^{1/2} = 162.2 \text{ k}$$

This is the resisting force required by each side of the weld (see Figure 6).

Available weld length each side = 14 in. = L

Required weld size:

$$= \frac{F_R}{L \times 0.707 \times 21} = \frac{162.2}{14 \times 0.707 \times 21} = 0.78 \text{ in.}$$

Use ¾-in. fillet weld at both sides of the shear lug with end returns.

Note: In the above numerical examples, the following materials are assumed:

Concrete: $f_c' = 3{,}000$ psi
Steel: $F_y = 36$ ksi
Welding electrodes: E70 series.

NOMENCLATURE

- A Gross cross-sectional area; in.^{2}
- a Ratio of the lower column segment length (L_1). to the total column length (L_T).
- A_B Gross cross-sectional area of the building column, in.^{2}
- A_C Gross cross-sectional area of the combined column, in.^{2}
- A_{CR} Gross cross-sectional area of the crane column, in.^{2}
- α Angle subtended by the lacing member with the horizontal.
- θ Rotational slip at a fixed end connection.
- B Ratio of the moment of inertia about the X axis of the combined column to the moment of inertia about the X axis of the building column, or width of base plate (see AISC p. 3-106 Figure 1).
- b Distance from centerline of building column to centerline of crane column (see Figure 2), ft, or plate dimension normal to the direction of stress, ft.
- b_f Flange width, in.
- C Distance from centroid of member to extreme stress fiber, in.
- C_b Bending coefficient dependent upon moment gradient.
- C_c Distance from centroid of combined column to centroid of crane column (see Figure 2), in.
- C_m Distance from centroid of combined column to extreme stress fiber of crane column flange (see Figure 2), in.
- C_{mx} Coefficient applied to bending term of the interaction equation about the strong axis. Use 0.85 for wind loading condition and 0.95 for crane loading condition.
- C_{my} Coefficient applied to bending term of the interaction equation about the weak axis. For crane shaft with fixed base and pinned top use 0.4.
- d Depth of a member, in.
- Δ Settlement of column base, in.
- E Modulus of elasticity, ksi.
- e Distance from edge of column flange to centerline of anchor bolt (see Figure 6), or depth of embedment of the shear lug into the concrete (see Figure 6), in.
- F_a Allowable axial stress in a centrally loaded column. (Subscripts C, CR, and B denote the values for combined column, crane column, and building column respectively), ksi.
- f_a Average axial stress, ksi.
- f_a' Average axial stress in the crane shaft alone for the combined column for checking the bending of lower shaft. Average bending stress is the value obtained from the universal interaction equation $P / A \pm M_C / I$ where M is the average X-axis moment between points B and C (see Figure 1)

$$f_a' = \frac{(P_1 + P_2)}{A_C} \pm \frac{M_{avg} C_C}{I_C}$$

- F_b Allowable bending stress, ksi.
- F_{bx} Permissible extreme fiber stress due to bending about X axis. This requires calculation of F_a for crane column alone condition. F_a = F_{bx} and must be multiplied by ratio C_m / C_C but not greater than $0.6F_y$ Use:

$$F_{bx} = F_a\left(\frac{C_m}{C_C}\right) \leq 0.6F_y$$

Use $KY_C = KX_{CR} = 0.8$ in calculating F_a for crane column alone.

- f_{bx} Maximum calculated stress due to bending about X axis. For upper shaft use M_B / S_B and for lower shaft use $M_C\, C_m / I_C$.
- F_{by} Permissible bending stress due to bending about Y axis. F_{by} should be calculated per AISC requirements Section F1 (9th Edition).

$$F_{by} = (F_{bx})_{CR} \leq 0.66F_y$$

- f_{by} Maximum calculated stress due to bending about Y axis. For upper shaft usually M_y is zero. For lower shaft all M_y is resisted by crane column shaft. Use

$$f_{by} = My_{CR} / Sx_{CR}$$

- f_c Allowable stress in concrete, ksi.
- f_c' Specified compressive strength of concrete, ksi.
- F_{ex}' Bending moment magnification factor in the plane of bending about X_C axis based on KX_{C1} or KX_{C2} (see Table 1).
- F_{ey}' Bending moment magnification factor in the plane of

bending about Y_c axis based on KY_c / rx_{CR} with $K = 0.8$ for crane column alone (Length = BC).

F_P Allowable bearing stress, ksi.

f_p Computed bearing stress, ksi.

F_R Resultant force or reactive force.

f_t Computed tensile stress, ksi.

F_V Allowable shear stress, ksi.

f_v Computed shear stress, ksi.

F_y Specified minimum yield stress; ksi.

F_1 Force couple in horizontal plates (see Figure 6), kips.

F_2 Vertical force in stiffener plates (see Figure 6), kips.

H Building height, in.

H_C Horizontal force at combined column (see Figure 7), kips.

H_i Horizontal force at the i-th joint (see Figure 5), kips.

I Moment of inertia, in.^{4}

I_C Moment of inertia of the combined column, in.^{4}

I_{weld} Moment of inertia of the weld, in.^{4}/in.

IY Moment of inertia about the Y axis, in.^{4}

IX_B Moment of inertia about the X axis for the building column, in.^{4}

IX_C Moment of inertia about the X axis for the combined column, in.^{4}

IX_{cr} Moment of inertia about the X axis for the crane column, in.^{4}

IY_B Moment of inertia about the Y axis for the building column, in.^{4}

IY_{CR} Moment of inertia about the Y axis for the crane column, in.^{4}

K Effective length factor, or width of shear lug, (see Figure 6), in.

KX_C Effective length factor for the combined column about the X axis.

KX_{C1} Effective length factor for the combined column about the X axis for crane loading.

KX_{C2} Effective length factor of the combined column about the X axis for wind loading.

KX_{CR} Effective length factor for the crane column about the X axis.

KY_B Effective length factor for the crane column about the Y axis.

KY_C Effective length factor for the combined column about the Y axis.

L Member length, ft.

L_T Total length of upper and lower column segments (see Figure 1), ft.

L_1 Length of upper column segment (see Figure 1), ft.

L_2 Length of lower column segment (see Figure 1), ft.

M Maximum bending moment, ft-k.

m Distance from effective column edge to edge of base plate (see AISC p. 3-106 Figure 1), in.

M_i Maximum bending moment at the i-th joint, ft-k.

MX Maximum bending moment about the X axis, ft-k.

MX_{avg} Average bending moment about the X axis between points B and C of the combined column (see Figure 1), ft-k.

MX_B Maximum bending moment about the X axis of the building column, ft-k.

MX_C Maximum bending moment about the X axis of the combined column, ft-k.

MX_{CR} Maximum bending moment about the X axis of the crane column, ft-k.

MY Maximum bending moment about the Y axis at B, ft-k.

MY_C Maximum bending moment about the Y axis of the combined column, ft-k.

N Length of base plate (see AISC p. 3-106, Figure 1), in.

n Distance from effective column edge to edge of base plate (see AISC p. 3-106, Figure 1), in.

P Vertical load, kips.

P_C Vertical load in the combined column, kips.

P_{CR} Euler's elastic critical buckling load.

P_i Vertical load at the i-th joint (see Figure 5), kips.

Q The first moment of inertia of an area from the point where the stress is measured to the extreme stress fiber about the neutral axis, in.^{3}

r Radius of gyration, in.

rx_B Radius of gyration about the X axis of the building column, in.

rx_C Radius of gyration about the X axis of the combined column, in.

rx_{CR} Radius of gyration about the X axis of the crane column, in.

ry Radius of gyration about the Y axis, in.

ry_B Radius of gyration about the Y axis of the building column, in.

ry_C Radius of gyration about the Y axis of the combined column, in.

ry_{CR} Radius of gyration about the Y axis of the crane column, in.

S Section modulus, in.^{3}

SX_B Section modulus about the X axis of the building column, in.^{3}

SX_C Section modulus about the X axis of the combined column, in.^{3}

SX_{CR} Section modulus about the X axis of the crane column, in.^{3}

SY_C Section modulus about the Y axis of the combined column, in.^{3}

T Tensile force in the anchor bolts (see Figure 6), kips.

T_1 Force couple in shear lug (see Figure 6), kips.

t Thickness of shear lug (see Figure 6), in., or thickness of vertical stiffener plates (see Figure 6), in., or thickness of flange plate (see Figure 6), in.

t_p thickness of base plate, in.

V Horizontal shear, kips.

V_i Vertical reaction at the i-th joint (see Figure 5), kips.

w Applied uniform load, kip-in.

X_B X axis of building column.

X_c	X axis of combined column.
X_{CR}	X axis of crane column.
Y_B	Y axis of building column.
Y_C	Y axis of combined column.
Y_{CR}	Y axis of crane column.

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