Design Concepts for Jib Cranes

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Jib cranes are either attached to a building column or cantilever vertically from an independent floor mounted column. Shown in Figure 1 is a representation of a column mounted jib crane. This paper will primarily address jib cranes that are attached to building columns. Essentially a jib crane is a boom with a moveable trolley hoist. The trolley hoist moves along the length of the boom and the boom swivels allowing the lifted load to be maneuvered about in a relatively small semi-circular area.

The hoists and trolleys of jib cranes are usually slow moving and either manually or radio operated. The arc-swing is usually manually accomplished but can be mechanized when required.

There are two different types of column-mounted jib booms normally encountered. The fundamental difference between the two is in the way in which the vertical column force is distributed. The suspended boom as depicted in Figure 2a is analyzed as if it delivers 100 percent of the vertical load to the column at the top hinge. The cantilevered boom (Figure 2b) distributes the vertical load equally between the two hinges.

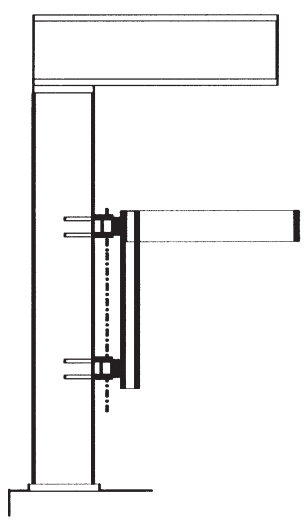

Fig. 1. Column Mounted Jib Crane.

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Column-mounted jib crane forces produce effects on the overall building frame and building bracing systems as well as local effects at the columns to which they are mounted. The effects on the building can be accounted for by placing point loads on the column(s) at the appropriate locations and combining them with the appropriate load combinations as prescribed by the building code. The local effects must be dealt with individually.

GLOBAL JIB CRANE LOADS

General

Jib cranes exert vertical gravity loads and horizontal thrust loads on the supporting column. Hinge forces supplied by the crane manufacturer should be used if available. If unavailable, the loads may be approximated from statics as shown below and in Figures 2a and 2b.

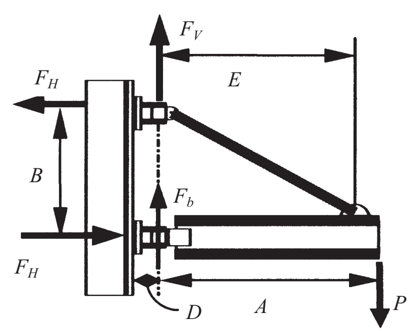


Fig. 2a. Suspended Boom Jib Crane.

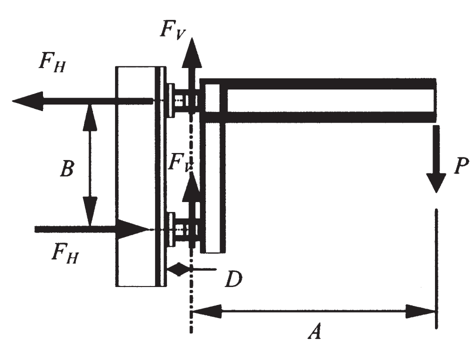


Fig. 2b. Cantilevered Boom Jib Crane.

Suspended Boom

$$F_v = \left[W_{(\mathrm{LIFTED})} + \frac{1}{2} W_{(\mathrm{BOOM})} + W_{(\mathrm{TROLLEY/HOIST})} \right] (A/E)$$

Acting @ top hinge only

$$F_H = \left[W_{(\mathrm{LIFTED})} + \frac{1}{2} W_{(\mathrm{BOOM})} + W_{(\mathrm{TROLLEY/HOIST})} \right] (A/B)$$

Cantilevered Boom

$$F_v = \frac{1}{2} \left[W_{(\mathrm{LIFTED})} + W_{(\mathrm{BOOM})} + W_{(\mathrm{TROLLEY/HOIST})} \right]$$

Acting @ each hinge

$$F_H = \frac{1}{2} \left[W_{(\mathrm{LIFTED})} + \frac{1}{2} W_{(\mathrm{BOOM})} + W_{(\mathrm{TROLLEY/HOIST})} \right] (A/B)$$

If the weight of boom and trolley are unknown it is suggested that a 15 percent factor be applied to the lifted load. Impact factors are required for jib crane column and connection strength design; however, impact need not be included for serviceability checks.

Jib crane loads on columns will result in a horizontal thrust at the top and bottom of the supporting column. The frame action or the building bracing system must resist this horizontal force. The resulting horizontal and vertical column reactions are shown in Figure 3.

For pinned-base columns, the lateral loads at the top of the building columns can be calculated using Equation 2-1.

$$Rh_i = P_i e_i / L_i \qquad (2\text{-}1)$$

At any given column line the horizontal force at the top of a given column (i) may be introduced into the building frames or roof bracing system. The total jib force to the roof bracing along a given column line is:

$$F_{H(total)} = \sum_{i=1}^{n} \left[\frac{P_i e_i}{L_i} \right] \qquad (2\text{-}2)$$

where

n = Total number of jib cranes acting along the column line with due regard to sign convention.

Load Combinations

1. Crane loads in combination with environmental loads:

In the United States model building codes require the following combination of crane loads with environmental loads:

DL + 75% Snow Load + Crane
DL + 50% Wind Load + Crane

2. Combinations with multiple jib cranes:

For the structural analysis of building frames or bracing with multiple jib cranes, it is not necessary to assume that all of the jib cranes are acting in the most severe possible combination simultaneously. It is suggested that unless plant operations dictate otherwise, the structure be designed with the net effect of any two jib cranes acting to cause the most critical effect at a given cross section of the frame and/or the largest column reactions.

3. Combinations with bridge cranes (Building Analysis):

In many cases, buildings with jib cranes will also have overhead bridge cranes in them. When jibs and overhead cranes are in the same building the following combinations need be considered.

A. Effects of net two jib cranes only as described above.
B. Effects of bridge cranes only.
C. Effects of net two jib cranes as described above plus any two-bridge crane. (100% vertical effects w/o impact + 50% lateral for both cranes or 100% from one crane).
D. Other combinations if warranted.

LOCAL JIB CRANE EFFECTS

Column Design Considerations

In addition to overall building considerations, there are localized effects on the columns that must be considered. The vast majority of the building performance problems associated with jib cranes are due to oversights of the local effects.

Since jib cranes are designed to rotate about their support columns, their effects will result in forces acting both in and out of the plane of the building frames. These actions create bi-axial bending and torsion in the support columns. Consider the jib crane support column of Figure 4.

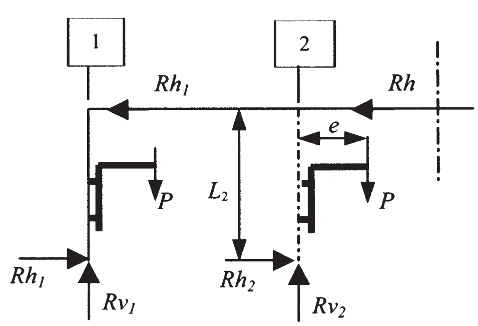


Fig. 3. Free Body Diagram of Cantilevered Boom.

If both jib cranes in Figure 4 are identical then under the loading condition shown there is zero bending moment in the column. However, if one of the jibs is rotated about its hinges major and minor axis moments are produced. If both jib cranes are rotated 90 degrees in the same direction the maximum minor axis moment is produced with zero major axis moment or torsion on the column as shown in Figure 5.

If only one jib crane is rotated 90 degrees as shown in Figure 6 it will produce minor axis bending and torsion in addition to the major axis bending caused by the other jib crane in the plane of the frame.

If both cranes are rotated 90 degrees in opposite directions the net bending moments are zero but a torsional moment is produced as shown in Figure 7a. The amount of torsion in the column depends on the torsional support characteristics at the ends of the column. Figure 7b shows the torsional moment distribution for a column with torsionally simple supports. If either end of the column is free to hor-

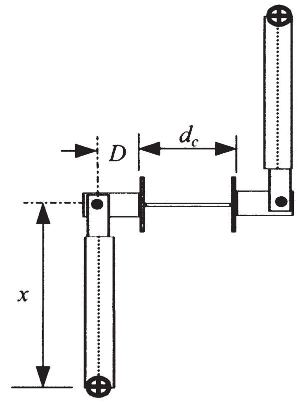


Fig. 7a. Maximum Torsion.

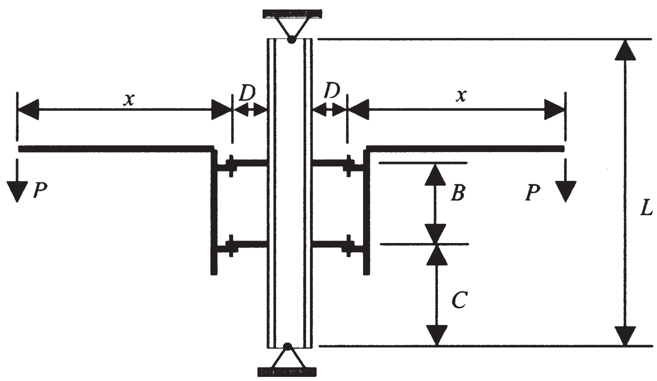


Fig. 4. Jib Crane Support Column.

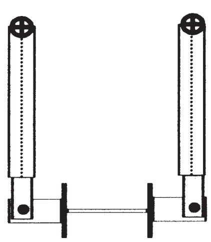


Fig. 5. Maximum Minor Axis Moment.

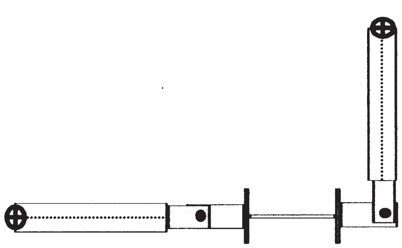


Fig. 6. Maximum Bi-Axial Moment.

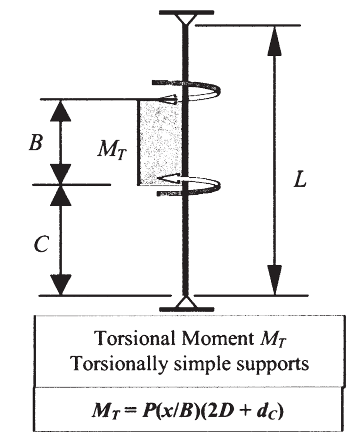


Fig. 7b. Torsion Distribution.

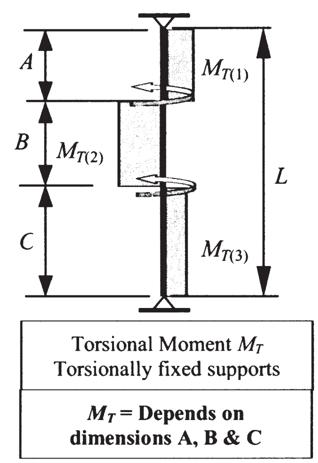


Fig. 7c. Torsion Distribution.

izontally rotate, the column will behave as if both ends are torsionally simple supports (assuming the torsional moments are equal and opposite).

If both ends of the column are torsionally rigid the torsional moments are distributed as shown in Figure 7c. For this case the moments are calculated as follows:

Applied torsion:

$$M_{T(1)} = M_{T(3)} = M_T\left[1 - \frac{1}{\left(1+\left(A+B/C\right)\right)} - \frac{1}{\left(1+\left(B+C/A\right)\right)}\right]$$

$$M_{T(2)} = M_T\left[\frac{1}{\left(1+\left(A+B/C\right)\right)} + \frac{1}{\left(1+\left(B+C/A\right)\right)}\right]$$

$$M_T = P(x/B)(2D + d_C)$$

Given the discussion above it is clear that jib crane support columns must be designed for the most critical combination of major and minor axis bending, torsion, and axial compression.

Local Effects on W-Shaped Columns

If a W-shaped column is used for support of jib cranes the internal column forces caused by torsion will take two forms. One of these is pure torsional shear across the column cross section or Saint-Venant s torsion. The other is a normal flexural stress in the cross section caused by warping of the cross section. The warping component of the torsion is effectively equivalent to a minor axis bending moment (M_Y). Therefore, when minor axis bending and torsion occur together the torsion may be converted to an equivalent minor axis moment and added to the actual minor axis moment. A simple conservative approach to converting the torsion which ignores Saint Venant s torsion is to simply apply an equivalent lateral flange force equal to the torsional moment divided by the center-to-center of flange depth as demonstrated in Figure 8. See Salmon and Johnson (1996).

If this method were applied to the jib cranes shown in Figure 6 the equivalent minor axis lateral loading to be considered in design would be as shown in Figure 9. Notice that the torsional component of P_H has been doubled. This is to account for the fact that it acts on each flange. Therefore, if the full member section properties are to be used in design, the values of M_Y must be doubled.

The equivalent minor axis moments due to torsion are superimposed on the actual minor axis moments caused by the jib crane thrust. Then the column is evaluated per Chapter H of the AISC LRFD Specification (1999) or the AISC ASD Specification (1989) with the critical combination of major and minor axis bending and axial.

Serviceability Considerations with W-shaped Columns

Columns supporting jib cranes must possess sufficient stiffness to prevent excessive deflection at the end of the jib boom when lifting a load. The AISC Steel Design Guide Number 3 (AISC, 1990) recommends a maximum vertical deflection at the end of the jib boom, due to lifted load, of $L/225$ in which L is the total boom span S (see Figure 10). The critical deflection will normally be due to out-of-plane flexural and torsional loads acting on the minor axis of the column as described above. This deflection can be calculated as follows:

1. Calculate the maximum equivalent out-of-plane thrust (P_H) including torsion as described above.
2. Calculate Δ_1 and Δ_2 as follows:

$$\Delta_1 = \left[\frac{P_H}{3EI_Y L}\right]\left[A^2\left(L-A\right)^2 - C\left(\frac{A}{2}\right)\left(L^2 - C^2 - A^2\right)\right]$$

$$\Delta_2 = \left[\frac{P_H}{3EI_Y L}\right]\left[C^2\left(L-C\right)^2 - A\left(\frac{C}{2}\right)\left(L^2 - C^2 - A^2\right)\right]$$

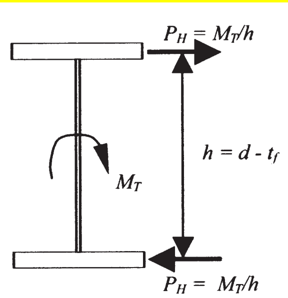


Fig. 8. Flange Forces.

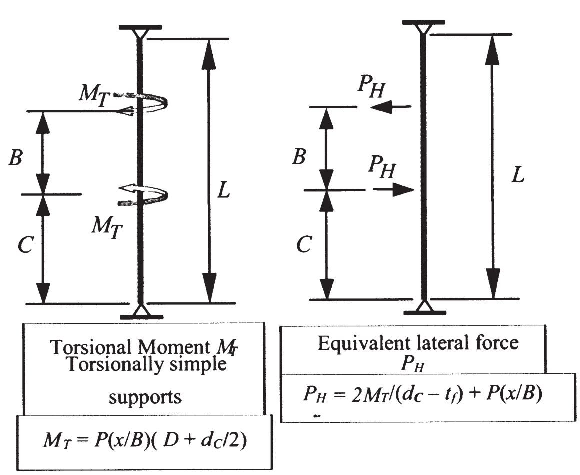


Fig. 9. Column Loads.

3. Calculate Δ_{jib} as follows:

$$\Delta_{jib} = (S/B)(\Delta_1 + \Delta_2)$$

Local Effects on HSS Columns

When closed sections such as HSS are used as support columns for jib cranes the design is somewhat different. When torsion is applied to a closed section there is no tendency for warping. Only pure torsional shear stress is assumed to be produced in the column. Therefore, the column must be checked for the effects of bi-axial bending and axial compression and torsional shear plus horizontal shear.

Axial compression and bi-axial bending is checked based on AISC Chapter H with bending moments calculated as shown above for W-shaped columns except without the torsion-induced flexure.

Shear stress in the HSS wall is calculated as follows:

$$f_V = \text{Max}[f_{VTi} + f_{VHi}]$$

where

f_{VTi} = Torsional shear stress in HSS wall segment i

f_{VHi} = Horizontal shear stress in HSS wall segment i

$$f_{VTi} = M_{Ti} / (2\ [A]\ t_w) \text{ ksi}$$

$$f_{VHi} \cong V_i/(2dt_w) \text{ ksi}$$

where

M_{Ti} = Torsional moment in segment A, B, or C, in.-kip

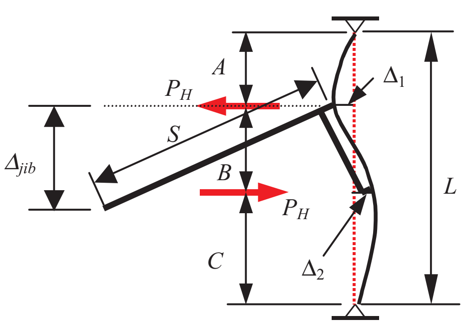


Fig. 10. Jib Crane Deflection.

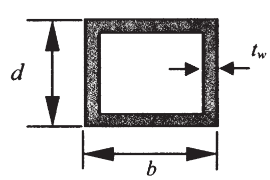


Fig. 11. HSS Cross Section.

$[A]$ = $(d–t_w)(b–t_w)$, in.^{2}

V_i = Horizontal shear in segment A, B or C, kips

Assuming that the walls of the HSS column are thick relative to its width and height the design criteria is:

$$f_V \le 0.4F_y$$

Serviceability Considerations with HSS columns

The deflection at the end of the jib crane boom when mounted from a HSS column is calculated somewhat differently than for a W-shaped column. There are two components of deflection.

1. Flexural deflection:
   Δ_1 and Δ_2 are calculated with the equations given for W-shaped columns using lateral thrust forces P_H with no torsion component.

2. Torsional deflection:
   Incremental displacement Δ_T is calculated by determining the relative torsional rotation between the upper and lower hinges as follows:

$$\theta_R = M_{T(2)}(B)/[4JG] \quad \text{(rad)}$$

where

$M_{T(2)}$ = Torsional moment between jib crane hinges, in.-kip

B = Distance between jib crane hinges, in.

J = HSS polar moment of inertia given in AISC table, in.^{4} *

G = Shear modulus = 11,000 ksi

*NOTE: The section property values for HSS sections given in AISC ASD are based on nominal wall thickness. Actual thickness is approximately 7 percent less for ERW HSS. Therefore, all section properties taken from this document should be multiplied by 0.93. Alternatively the values given in the AISC HSS Manual may be used without modification.

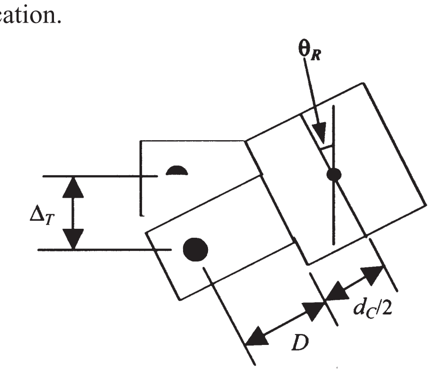


Fig. 12. Translation Due to Rotation.

This rotation is then converted to an effective relative translation at the hinges as follows (refer to Figures 10 and 12):

$$\Delta_T = (D + d_C/2)\sin(180\theta_R/\pi) \text{ in.}$$

The deflection at the end of the boom is:

$$\Delta_{jib} = (S/B)(\Delta_1 + \Delta_2 + \Delta_T)$$

Column End Connections

Horizontal Reactions

Columns supporting jib cranes will exert horizontal forces at both the base and the top of the column. In general, the reactions at the base of the column will not present a problem. The top of the column is of somewhat more concern. If not properly braced, out-of-plane forces applied to the bottom flange of a rafter beam will create minor axis bending in this flange. This will also result in an additional component of deflection at the end of the jib boom that is not considered in the preceding analysis. Therefore, one must brace the rafter at the top of a jib crane in order to transfer the out-of-plane reaction directly to the roof bracing. This can be accomplished in several ways including those shown in Figures 13a and 14a. In some cases a more direct and substantial / stiffer brace detail may be required.

Torsional Reactions

As discussed earlier, jib cranes will produce torsion in their supporting columns. The distribution of torsion along the column length is dependent upon the torsional support conditions. If at least one end of the column is free to rotate then the column will behave as if torsionally simply supported on both ends. If both ends have some degree of torsional restraint then some amount of torsion will be delivered through the column end connections. It is conservative to analyze the column as if torsionally pinned. However, this may not be conservative for the column end connections.

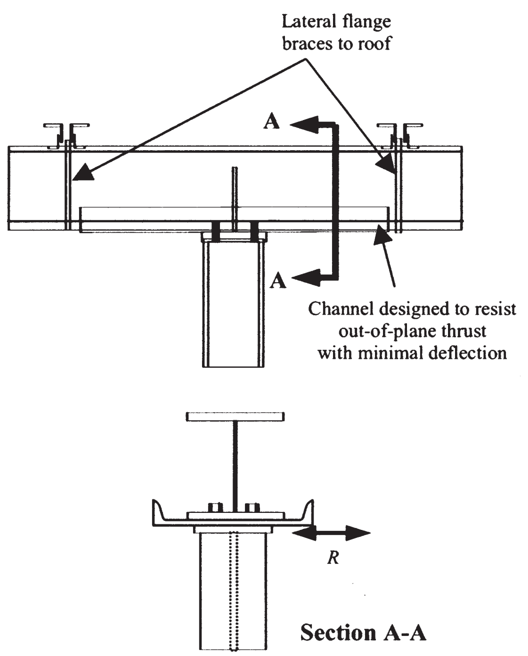


Fig. 13a. Top of Column Connection Detail with Channel.

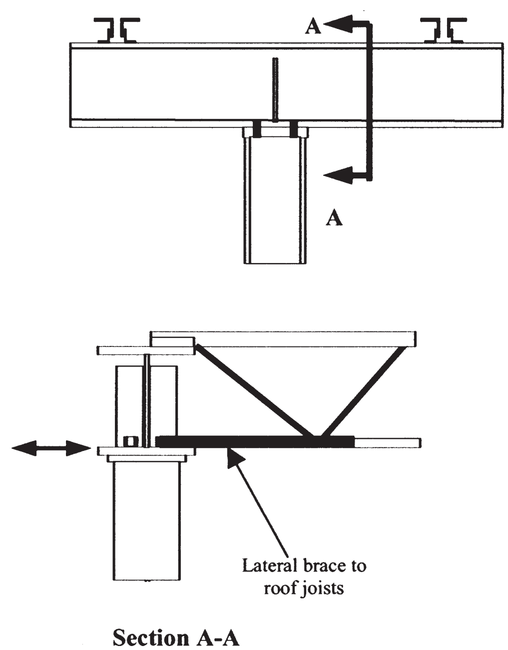


Fig. 14a. Top of Column Connection Detail with Roof Joist.

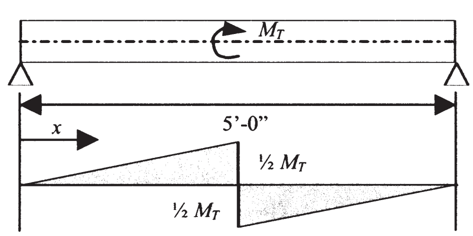


Fig. 13b. Channel Moments.

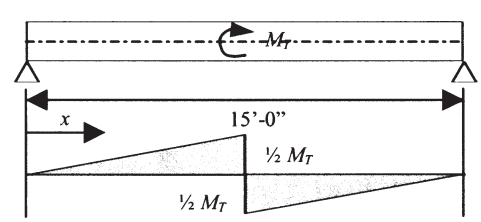


Fig. 14b. Rafter Flange Moments.

The base of column connection will probably behave as if torsionally rigid. The torsional rigidity at the top of the column will depend upon the connection detail. Consider the details in Figures 13a and 14a.

In Figure 13a, a channel is added to the bottom flange and connected at its ends near flange braces that will transfer horizontal reactions from the channel to the purlins. If the purlin spacing is five feet and the column connects at the center of the channel the moment in the channel due to a torsional reaction would be as shown in Figure 13b. (Assuming that the channel acts independent of the rafter flange as a simple beam)

The rotation of the channel at the column connection is:

$$\theta = \left(\frac{2M_T}{3,600EI}\right)\int_0^{30"} x^2 dx$$

Assuming that the channel is a C12x20.7 the torsional stiffness of this connection is:

$$K_T = M_T/\theta = 748,200 \text{ in.-kip/rad}$$

The detail in Figure 14a will provide much less torsional stiffness to the column end connection depending on the size of the rafter flange. If one assumes that the first flange braces are 7.5 feet either side of the column, a model similar to the one in Figure 14b is developed.

The rotation of the channel at the column connection is:

$$\theta = \left(\frac{2M_T}{32,400EI}\right)\int_0^{90"} x^2 dx$$

Assuming that the beam is a W24×55 (7-in. x ‰-in. flanges) torsional stiffness of this connection is:

$$K_T = M_T/\theta = 27,630 \text{ in.-kip/rad}$$

From the above comparison it can be seen that the Figure 13a model is roughly twenty-seven times as rigid as the model for Figure 14a. If flange braces were added to the Figure 14a model similar to the Figure 13a model the torsional stiffness would become:

$$K_T = 82,892 \text{ in.-kip/rad}$$

These models tend to underestimate the true torsional stiffness since the rafters are not actually pinned at the flange brace locations. However, there is also a tendency to overestimate the stiffness since the resistance provided by the flange braces is not truly rigid.

The equations for the torsional moments in the three column segments A, B and C (as shown in Figure 7c) given above are based on two assumptions.

1. The ends of the column are torsionally rigid.
2. The column has uniform torsional properties along its length.

If the end condition does not provide perfect torsional rigidity (as it never will) the equations must be modified. The easiest way to do this is to substitute a modified equivalent length into the equations in place of the segment length that isn t fixed at the end (usually segment A).

The equivalent length is determined by calculating the length of column that would produce an equivalent torsional spring constant as the end support detail and adding this virtual length to the length of segment A. The equivalent length is calculated as follows, where:

L_A = actual segment length, in.
K_T = torsional spring constant of support detail, in.-kip/rad, i.e. 748,200 for the detail in Figure 13b.
G = shear modulus = 11,000 ksi
J = torsional moment of inertia, in.^{4}

The twist of a closed section of length L is equal to:

$$\theta = M_T L/(4GJ)$$

Therefore the equivalent torsional spring constant is $M_T/\theta = 4GJ/L$. By setting this equal to the support spring constant

$$K_T = 4GJ/L$$

Solving for the required virtual column length

$$L_V = 4GJ/K_T$$

Therefore, the equivalent length of segment for use in the equations is:

$$L_e = 4GJ/K_T + L_A$$

and

$$M_{T(1)} = M_{T(3)} = M_T\left[1 - \frac{1}{\left(1+\left(L_e+B\right)/C\right)} - \frac{1}{\left(1+\left(B+C\right)/L_e\right)}\right]$$

$$M_{T(2)} = M_{T(3)} = M_T\left[\frac{1}{\left(1+\left(L_e+B\right)/C\right)} + \frac{1}{\left(1+\left(B+C\right)/L_e\right)}\right]$$

Example

Column = HSS 10×10×½
Top connection detail is same as in Figure 14a
K_T = 27,630 in.-kip/rad
Segment lengths: A = B = C = 8 ft
Applied torsional moments = 300 in.-kip

Calculate torsion in all three column segments.

L_e = effective length of segment A
$= 4GJ/K_T + L_A$
$= [4(11{,}000)439/27{,}630 + 96\ ]/12$
$= 66.26$ ft

$M_{T(1)} = M_{T(3)}$
$= 300[\ 1 \quad 1/(1 + (66.26{+}8)/8) \quad 1/(1 + (8{+}8)/\ 66.26)]$
$= 29.18$ in.-kip

$M_{T(2)} = 300[\ 1/(1 + (66.26\ {+}8)/8) + 1/(1 + (8{+}8)/\ 66.26)]$
$= 271$ in.-kip

Had the stiffness of the top support not been taken into account:

$M_{T(1)} = M_{T(3)} = 100$ in.-kip
$M_{T(2)} = 200$ in.-kip

This indicates that for columns that are very stiff torsionally, very little torsional moment is transferred to the end connections unless the top of column connection detail is also made very torsionally stiff. On the other hand, torsionally flexible columns such as W-shaped columns will transfer more torsion to their connections.

In general W-shaped sections have very low torsional stiffness. Therefore, the assumption of torsionally rigid supports is probably the most accurate for W-shaped columns.

Jib Hinge Connections to Columns

Most jib crane manufacturers furnish bolted hinge connections. Different manufacturers have different hinge types and bolt patterns. Many manufacturers make C -hinges with a four, six or eight bolt pattern. The bolts may range in diameter from ⅝ in. to 1½ in. and have a horizontal gage dimension up to about 7 in. Therefore, if the jib is mounted to a W-shaped column it is important to make the flange wide enough to accommodate the bolts. Unless flange and web are made sufficiently thick, it is necessary to provide bearing stiffeners in order to control web crippling/yielding/buckling and/or local flange bending. Also, the reduction in net flange width due to the holes will need to be considered for load cases causing tension across these sections.

Figure 15 shows various standard hinge configurations offered by one typical jib crane manufacturer. It is very important to ascertain the hinge geometry for the jib crane(s) to be installed. Without this information the columns cannot be designed correctly.

W Sections With Bolted Hinges

Top hinge considerations (Tension)

The design considerations at the top hinge will be web yielding and local flange bending. The stiffeners shown in Figure 16 and their welds are designed to share the thrust force P_H equally. Local flange bending analysis may be based on a tee hanger analogy with bending about the stiffeners only. The preferred stiffener location is between the bolts as shown in Figure 16. This will result in a single stiffener pair for a four-bolt hinge and two pairs of stiffeners for a six and eight-bolt hinge. In some cases the bolt pitch may not be sufficient to allow a stiffener between bolts. When this is the case the stiffeners should be located as shown in Figure 17. For stiffeners located as shown in Figure 17 the required column flange thickness analysis will vary depending on the bolt pattern.

A four-bolt and six-bolt pattern may be analyzed by a rational method such as yield line analysis. For the six-bolt pattern this will cause the outer bolt rows to carry almost all of the horizontal thrust P_H. In addition, since the vertical bracket reaction is eccentric to the bolt group, the upper row will be more highly stressed in tension than the others. In most cases the jib crane manufacturer does not furnish the

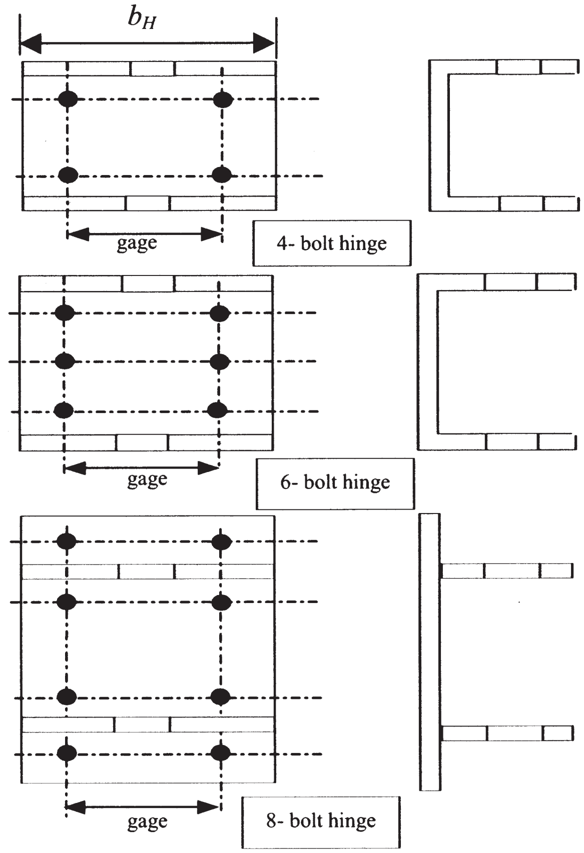


Fig. 15. Hinge Configurations.

bolts for connecting the hinge to the column. Therefore A325 bolts should be specified on the construction drawings for this application. A325 bolts of the diameter corresponding to the holes in the manufacturers hinge bracket should normally work. However, the bolts should be checked for combined shear and tension.

For the eight-bolt pattern shown in Figure 15 the stiffeners are placed between bolt rows. Therefore the flange thickness analysis is based on the tee hanger analogy similar to that shown in Figure 16. The column flange holes are usually field drilled to assure proper location and alignment. If channel caps are used, the local flange bending check must be based on the column flange thickness only unless the channel is welded to the flange in a manner to create a composite section.

Bottom hinge considerations (Compression)

The local column design considerations at the bottom hinge are web crippling, web yielding, and web buckling. Since the horizontal thrust will be concentrated in the hinge flanges, the preferred stiffener location for the bottom hinge will be as shown in Figure 17.

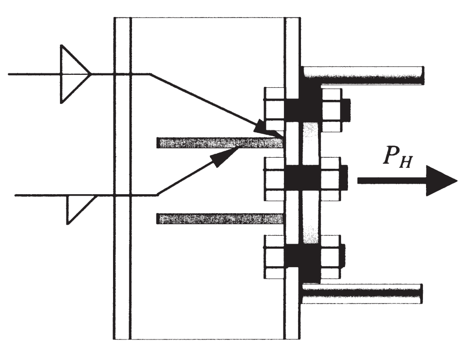


Fig. 16. Reinforced Column.

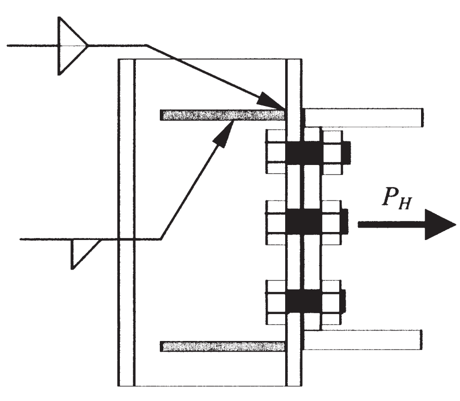
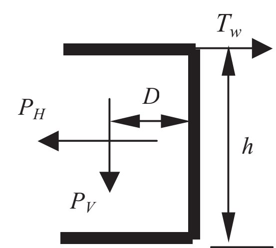


Fig. 17. Reinforced Column.

HSS with Field-Welded Hinges:

If HSS columns are used the jib hinge connections will most likely have to be field welded. Most jib crane manufacturers furnish field bolted hinges. If the hinge assembly is to be field welded the welding requirements must be specified. The hinge welding requirements and HSS geometry are interdependent. Even if the Engineer of Record does not determine the hinge welding requirements, the effects on the HSS column walls must be evaluated. Therefore it is best if the structural engineer determines and shows the welding requirements on the construction drawings.

HSS CHECKS AND DESIGN OF WELDS



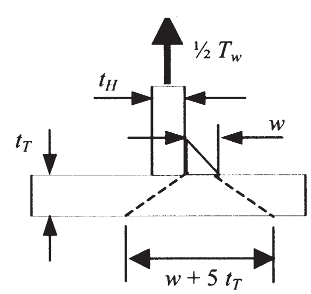
$$T_w = P_H/2 + P_V(D)/h$$

Sidewall Yielding

This criterion may control when the width of the hinge bracket b_H is nearly as wide as the HSS face.

Stress in HSS wall

$$f_t = 0.5T_w/[2(w + 5t_T)] \leq 0.66F_y$$



Effective Width

The out-of-plane stiffness of the HSS wall varies across the width of the HSS. Near the orthogonal HSS walls the stiffness is very high. Near the center of the HSS the stiffness is reduced. The stiffness at the center of the HSS face depends on the width of the HSS and the wall thickness. The outstanding hinge flange spanning laterally across the HSS is stiffer than the HSS wall to which it is attached.

Therefore, the tensile force in the weld and the welded hinge will not be uniform across their entire width. There are higher stresses near the edges and lower stresses near the center of the HSS.

$$b_e = [10 / (b_T/t_T)]\ (F_{yT} / F_{yH})\ (T_T / T_H)\ b_H \le b_H$$

Therefore, the weld and the outstanding leg of the hinge must be checked for an effective unit force of:

$$F_{eff} = T_w / b_e \text{ kip/in.}$$

Punching Shear

The shear in the HSS wall f_{Vtw} due to the out-of-plane force T_w must be checked.

$$f_{Vtw} = T_w / [\ (2b_{ep} + 2w)\ t_T\ ] \le 0.4\ F_{yHSS}$$

$$b_{ep} = [10/(b_T/t_T)]\ b_H \le b_H$$

Horizontal Weld Size

The required horizontal fillet weld size, in sixteenths of an inch:

$$w_h = (T_w / b_e) / (12(0.707)0.0625)$$

Vertical Weld Size

The required vertical fillet weld size, in sixteenths of an inch:

$$w_v = (P_V/2h) / (12(0.707)0.0625)$$

Note: The above analysis is only considered to be applicable to HSS columns with $b_T/t_T \le 30$. (AISC, 1997)

ALTERNATIVE DESIGN APPROACHES

In addition to using W-shaped or HSS columns in torsion as discussed above there are other alternatives that can be employed.

Direct Bracing

Probably the most effective and efficient way to eliminate minor axis bending and torsion in jib crane columns is to place braces at the jib hinge locations that are designed to eliminate the out-of-plane hinge thrust forces. (See Figure 18.)

If jib cranes are located on all column lines, braces such as these need only be placed in every other bay. Serviceability can be an issue with this design. The bracing members must be evaluated to assure that the jib crane boom deflection will not exceed the specified limit. The calculations are similar to those described for flexural deflections except the values of Δ_1 and Δ_2 are based on brace member strains.

These direct braces are often unwanted because they are an obstruction to the internal operations of the building, especially if the jib cranes are mounted relatively low to the ground. Also, braces like this along interior column lines can cause accidental moments in the columns under lateral loads. Lateral loads will also induce forces in the bracing members. However, in general this issue will not be of major significance if the braces are placed as shown in Figure 18 and the main building bracing and diaphragm are sufficiently stiff. Accidental moments can be eliminated if bracing is continued to the floor and the diaphragm and bracing are analyzed accordingly.

Cap Channels

In some cases it is possible to reinforce the column flange by welding a cap channel along its length. One common approach is to simply design the column for 100 percent of the in-plane effects of the jib crane(s) without the cap channel and then design the cap channel for 100 percent of the out-of-plane effects. However, there are a couple of issues that should be considered.

a. Welding a cap channel to the flange of a column modifies both the geometric and stiffness characteristics of the column. If the column is a part of the lateral force resisting system of a building, this may change the distribution of forces in the diaphragms and frames.
b. Welding cap channels to pinned-pinned columns won t effect the distribution of lateral forces. However, the effects of torsion must be properly evaluated (see Figure 19).

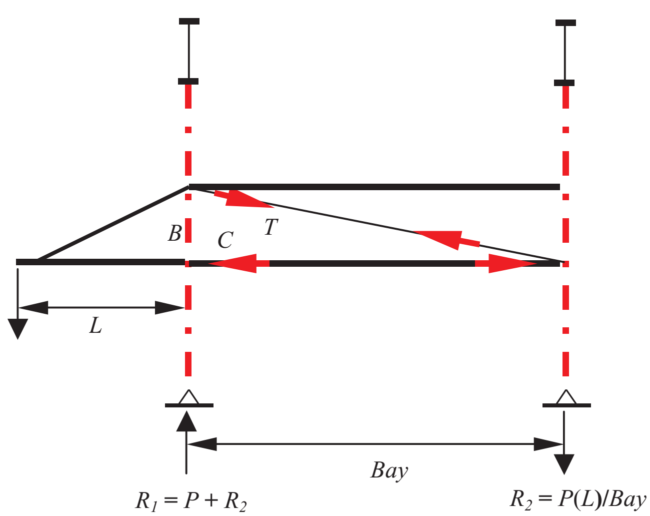


Fig. 18. Direct Bracing.

Auxiliary HSS Columns

If for any reason the jib crane cannot be mounted directly to a building column, an auxiliary HSS column can be placed alongside the building column or between column lines. Beams, struts and/or bracing are used to deliver the horizontal reactions from the auxiliary column back into the building s lateral force resisting systems.

PIVOTING JIB CRANES

Any discussion of jib cranes would be incomplete without the inclusion of the pivoting type jib crane. See Figure 20. This device consists of a vertical column (usually a WF shape) with a thrust bearing at its base and a ring-type roller or ball bearing, or a bushing, at its top, plus one or more horizontal arms (usually a standard beam section so as to accommodate the trolley wheels). Characteristics of the pivot jib are as follows:

1. The components can be assembled in most fabricating shops.
2. The bearings are readily available in a vast range of types and capacities and are inexpensive.
3. There are no hinge assemblies to purchase and install. In the case of a single boom the boom is attached directly to the jib column flange.
4. The bearings generally have less friction and enhance the ease of swinging the jib.
5. The upper bearing/bushing can easily be devised so as to eliminate the application of axial load from the building system into the jib column.
6. In the case of a single boom there are no torsional considerations to be made regarding the jib itself nor to the supporting structure except the miniscule effects from acceleration and braking in the case of a mechanical powered arc swing. (Generally the arc swing is accomplished manually.)
7. A 360-degree arc swing is possible rather than the usual limit of approximately 200-degree in the case of a hinge-mounted jib arm.
8. More than one jib arm can be attached to the jib column in the case where multiple machines are to be serviced by the same jib.
9. The client can locate the jib anywhere in the structure to serve the machinery location rather than to have to locate the machinery to suit the jib location. This may reduce the number of jibs required.
10. A broad range of hoists and trolleys are available.
11. The tops of the jib column can be braced directly back to the building columns without inducing any torsion or axial forces in the building columns.
12. Clients sometimes request some superelevation at the end of the boom to compensate for deflection. This can easily be done in a shop-fabricated piece whereas hinges cannot be made to do this. Calculation of deflection is simpler and more accurate since factors, adjustments, and compensatory equivalents are not required.
13. The pivoting jib can be installed in an existing building with minimal effect on its surroundings. It usually can be moved at will if required.

Many clients feel that the pivoting jib is the ultimate of all jib types because of its many advantages and virtual lack of disadvantages.

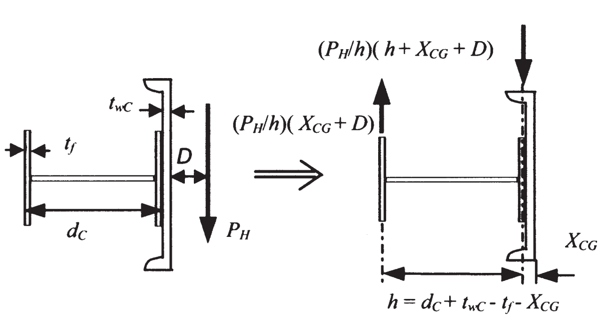


Fig. 19. Force Distribution.

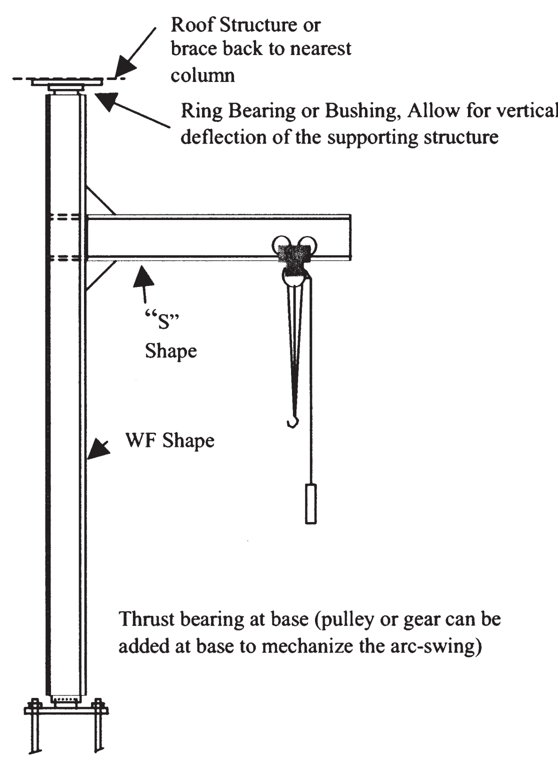


Fig. 20. Pivoting Jib Crane.

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