Mill Building Design Procedure

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The design of an industrial mill building involves myriad assumptions and parameters. In the current state of the art, there is disagreement about the validity of many of these assumptions and parameters. Inconsistencies exist among designers and in textbooks. Most of these inconsistencies concern design loads, combinations of design loads, effective column lengths, location of bracing, and function of the integrated structure.

A limited amount of information exists in two to three dozen publications and textbooks. To the author's knowledge, there are no recent texts which treat mill building design in its entirety. Consequently, an effort was undertaken by the author to consolidate the various publications into a design guide.

DESIGN DATA REQUIRED

To facilitate the design, the following information should be obtained from the client or established by the designer at the outset of the project. This information is presented in outline form so the designer can refer to it as a checklist:

A. Crane Schematic
   1. Horizontal and vertical clearances; span length
   2. Maximum wheel loads and wheel spacing
   3. Life capacity and trolley weight
   4. Type of operation (pendant or cab)

B. Building type
   1. Number of aisles
   2. Side sheds or offices
   3. Building length
   4. Bracing restrictions
   5. Restrictions on column spacing
   6. Size and location of major openings
   7. Construction materials (brick or metal walls, etc.)
   8. Future expansion
   9. Heating and ventilation
   10. Roof vents or skylights
   11. Number of cranes operating on runways
   12. Access to crane cab and roof
   13. Equipment platforms on roof or in truss
   14. Loading docks or rail siding

C. Site information
   1. Location of adjacent buildings, property lines and other information
   2. Soil conditions from borings or existing facilities
   3. Existing pavement and traffic patterns
   4. Drainage and utilities (new and existing)
   5. Rail access
   6. Exposure to wind and temperature variations

DESIGN LOADS

Dead, live, wind, earthquake, and thermal loads must be investigated in mill building design. The various load magnitudes are established in the AISC Manual,[12] regulating building codes, and design handbooks.

With the exception of wind loads, the determination of load magnitudes is straightforward. Unlike the other load types, wind loads depend on the type, shape, and height of the building, design wind velocity, and wind direction and gusting, and should be factored accordingly.[16] The factors applied to wind loads are especially important in light buildings which are partially or fully open.

The effects of thermal loads are often ignored by designers. For this reason, the author issues a word of caution. In other buildings designed by the author, thermal loads frequently have controlled member sizes. Murray and Graham[15] suggest limiting the structure to a maximum length of 400 ft between expansion joints. However, thermal loads and the resultant stresses should still be investigated for buildings below this 400-ft limitation.

Confusion occurs in the selection of load combinations. Most textbooks and publications are very vague in this area and fail to establish load criteria. Designers do not possess the time to perform a probability analysis on various load combinations. Therefore, the author suggests using the load cases below, which combine the load cases established by AASHTO,[11] the recommendations of AISC,[12] and the author's own judgment.

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A 33⅓% increase in unit stress is permitted by AISC[12] for all load cases which include wind. The 0.75 factor (1/1.33), below, reflects this increase. All other factors represent load reductions to account for the improbability of the load combinations in each case.[11]

Case I:	DL + LL
Case II:	Case I + Lateral Crane
Case III:	0.75 (DL + WL)
Case IV:	0.75 (Case II + 0.3WL)
Case V:	0.80 (Case I + T)
Case VI:	0.80 (Case II + T)
Case VII:	0.90 (Case III + T)
Case VIII:	0.90 (Case IV + T)
Case IX:	0.75 (DL + EQ)

The above load cases are applicable for column type **b** (Fig. 1). In this type column, the building column resists roof dead and live loads as well as the lateral crane loads. The craneway column resists vertical and longitudinal crane loads.

For column types **a** and **c** (Fig. 1), the above load cases require modification. The columns in both cases are single members and resist loads in three directions. Therefore, Cases I and II above are modified as follows:

Case I:	DL + LL + Vertical Crane
Case II:	Case I + Lateral Crane + Longitudinal Crane

Usually, all nine load cases are not applicable to a single mill building. Frequently, by rationalization and proper selection of connections, many of the stresses caused by the load cases can be minimized.

TYPE OF CONSTRUCTION

The three basic types of construction, defined in the AISC Manual,[12] are the rigid frame (Type I), the simple framed structure with bracing or wind connections at selected joints (Type II), and the semi-rigid frame (Type III). Traditionally, the most popular type of mill building is the simple framed structure (Type II).

When compared to the other types of construction, the Type II construction usually offers the following advantages:[13]

1. Column size reduction
2. Foundation size reduction
3. Less expensive connections
4. Minimum erection costs
5. Ease of design

In accordance with Type II construction and modern practice, many designers provide bracing in the vertical plane of the long walls to resist longitudinal loads and to stabilize the structure. In the short direction, designers commonly rely upon frame action for wind resistance and column stability.

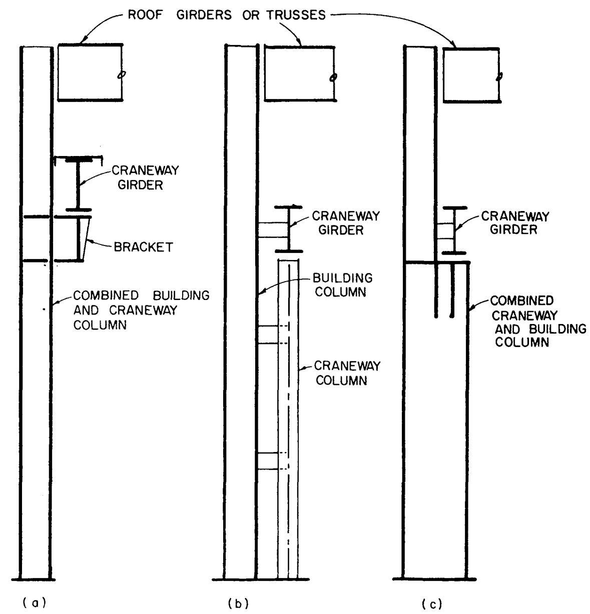

Fig. 1. Crane girder and building columns

The following observations by Murray and Graham[15] are worth noting:

"Unfortunately, many designers, when considering lateral crane loads, still rely upon frame action and assume the building to be a portal. This assumption of plane frame action is not confirmed by any theoretical or experimental evidence. In fact, the testing work that has been done refutes such an assumption."

* * *

"A typical long mill building properly designed on the basis of plane frame action would be very heavy. There are applications for such designs on rare occasions, i.e., a three-bay building where the building is so short that crane loads cannot be successfully distributed to adjacent bents."

* * *

"Two very important considerations determine the basic conditions of design of the mill building frames. The first of these is the fact that it has become uniform practice to provide full-length bottom flange (or chord) bracing (Fig. 2). The second key and related point is the fact that the maximum crane load condition can occur at only one column at a time. Thus, the top of the column is prevented from translating because of the fact that the bottom flange (or chord) bracing transmits horizontal forces to the unloaded columns which resist translation. For these reasons, the structure may be considered to be in effect a space frame."

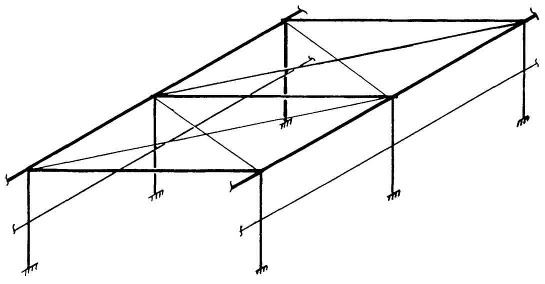

Fig. 2. Common mill building

BRACING

Before the late 50's, a rule of thumb was used for bracing placement. Unfortunately, some designers still follow this rule and apply it arbitrarily.

In the past it was customary to X-brace the roof at the bottom flange (or chord) level in the end bays and every fourth or fifth bay. Bays with roof bracing also had vertical X- or K-bracing in their longitudinal walls. In one structure braced in this way, Murray and Graham[15] found that when the end bay vertical bracing was cut during severe weather conditions the bracing snapped and sprang ⅜-in., indicating that the bracing was overstressed. "Some thought will indicate the fallacy of restraining the structure against thermal displacement. A far better technique would be to brace the center or center two bays and permit the structure to breathe thermally in both directions." [15]

Longitudinal X or K bracing is used often to resist wind and other longitudinal loads, as well as to stabilize the building columns along their weak axes. The use of a X-bracing scheme allows the bracing members to be designed as tensile members in accordance with the AISC limitation, $l/r \leq 300$.[12] On the other hand, bracing members in a K-bracing scheme usually do not qualify for the preceding limitation.

In addition to longitudinal bracing, if possible, it is usually desirable to brace the building columns along their weak axes at their tops and midpoints with struts for lateral support and continuity. Siding and girts may be used in lieu of struts and must be designed accordingly.[1,15]

Lateral bracing is occasionally placed in the end walls of short mill buildings to resist wind loads and other lateral loads. These lateral loads are transmitted into the end wall lateral bracing through a roof truss or roof diaphragm system. The fabrication and erection costs for the aforementioned systems can be expensive.

Along their weak axes, wind columns at the building ends often require lateral bracing for stability. Additional roof bracing is needed in the roof of the end bays for load transfer from the wind columns to the longitudinal wall bracing.

COLUMNS

Craneway and building columns are generally of three types (Fig. 1). The bracket support, type **a**, is used for light crane loads; types **b** and **c** for heavy loads. Type **b** for medium to heavy loads appears to be the most popular in the author's survey. This type of construction affords the economy of rolled sections, a minimum of fabrication cost, and adequate stiffness in both directions.

In column design, the first step is to establish whether or not large vertical crane loads will cause the column top to sway during buckling. Sidesway is unlikely to occur in long mill buildings which possess full lower chord bracing. On the other hand, sidesway would accompany column buckling in a short mill building without roof bracing.[2]

The second step in column design is to establish the column end conditions. For a column braced at its top and prevented from translating, Anderson and Woodward[2] recommend using a pinned top (Fig. 3a). In buildings with roof trusses, the location of the pin is likely to occur midway between the bottom chord and knee brace.[2] A more conservative analysis assumes that the pin occurs at the bottom chord.[2] For a column braced at the top and free to translate, Anderson and Woodward suggest modeling the column top fixity as a slider (Fig. 3b) located midway between the bottom chord and knee brace, or conservatively located at the bottom chord.

The above steps and assumptions are applicable for columns subjected to vertical crane loads in heavy mill buildings with roof trusses. Roof trusses are extremely stiff when compared to the building columns and restrain the column tops from rotating. Mill buildings with roof beams which are stiff in comparison to the building columns can also be designed in accordance with the preceding assumptions (Figs. 3a and 3b).

Columns in unbraced frames subjected to dead and live loads or wind loads are loaded simultaneously. Consequently, the columns must rely on frame action for their stability. When loaded, the columns, depending on the fixity at their tops, begin to rotate and translate at their tops (Fig. 3c). The degree of fixity[12,15] at the column tops is "dependent upon the amount of bending stiffness provided by the other in plane members entering the joint at each end of the unbraced segment." [5]

In compliance with the guidelines of Type II construction, unbraced frames often require connections at selected joints to resist wind loads. DeFalco and Marino[3] present the most common type of connections for a Type II building and a method for determining the effective length of columns in unbraced frames when partially rigid beam to column connections are employed. The design of wind connections is extremely important in single and double aisle mill buildings.

In multi-aisle mill buildings, wind loads become less important. Often, depending on the number of aisles, wind loads are neglected. The designer should not arbitrarily

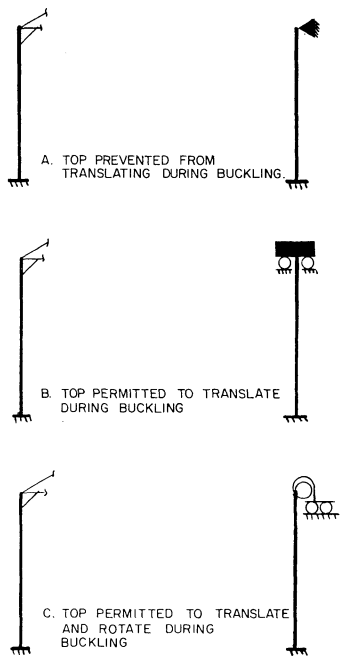

Fig. 3. Fixity at column tops

neglect these loads without an analysis of the structure.

When lateral crane loads or other load combinations including lateral crane loads are applied to a mill building with roof bracing, it is assumed that the loaded columns are prevented from translating by the unloaded columns. Consequently, the column tops are considered pinned. On the other hand, in short mill buildings or long mill buildings without roof bracing, the columns depend entirely upon frame action to resist lateral crane loads. Therefore, the pinned top assumption is not valid for these columns.

Professor Galambos[7] has shown that the amount of diagonal bracing needed to prevent sidesway of a steel building frame under gravity loading is relatively small. Higgins[1] adopted Galambos' approach and applied it to frame stability. The designer should verify that stability is achieved in the individual members as well as the entire structure, in accordance with Higgins' approach.

For column design the author refers the designer to Sandhu's article[6] for the type **a** column; Anderson and Woodward's article[2] for the type **c** column. The design of the remaining column, type **b**, is straightforward. The craneway and building columns are designed independently. In the type **b** column, the crane column along its strong axis is attached to the crane girder with a cap plate. Since one craneway column is vertically loaded at a time, the unloaded columns restrain the loaded columns from translating. Consequently the column top fixity can be idealized as a pin for vertical loads. Along the weak axis, craneway columns are braced by the building columns at intervals selected by the designer.

In the type **b** columns, longitudinal loads are assumed to be distributed into the craneway columns in proportion to their stiffnesses. Conservatively, the columns are assumed to translate at their tops along their strong axes. Because the craneway columns are assumed to translate and are attached to the building columns, twisting can occur in the building columns. Consequently, the crane column to building column connection should be designed to minimize twisting.

CRANE GIRDERS

A mill building crane girder often consists of an inverted channel attached to the top flange of an S or W section. In the past, the channel was designed to resist the lateral crane load; the W or S section was designed for the vertical crane load plus impact. Both members were assumed to laterally support each other. This design procedure neglected the behavior of the combined section and assumed each member acted independently. The result was a conservative section.

Gaylord and Gaylord[5] present a more exact analysis for crane girders. In their analysis, they introduce a more refined procedure. The key items of their design procedure are presented in the following excerpt: "If the span of the beam is such that the allowable stress for bending in the plane of the web must be reduced for lateral buckling, the beam is adequate provided: (1) the stress $F = M_x/S_x$ is equal to or less than the allowable stress for bending in the plane of the web and (2) the combined bending stress $F = M_x/S_x + M_y/S_y$ is equal to or less than the allowable bending stress without regard to lateral buckling."

In addition, Murray and Graham[15] make the following design recommendations:

1. The ratio of a span to depth of girder is usually kept to a maximum of 12.
2. Consideration must be given to end bearing conditions. It is disturbing to find how frequently the end reactions are not properly provided for. The designer should sketch the design of the girder end, showing the method of handling the reaction through stiffeners properly carried into the supporting column material.
3. Precautions must be taken in the design detailing of the end connections to avoid restraining of the girder against end rotation.
4. Adequate provisions for torsional stability should be made. Experience has indicated that any girder having a span over about 30 ft can be torsionally unstable and stiffened at the bottom flange as well as the top.
5. A very desirable, if not essential, practice on girders for outside service is the use of wearing plates.
6. Knee braces attached to the crane girders are not recommended to provide longitudinal bracing for the building.

ROOF GIRDERS OR TRUSSES

Roof girders or trusses in a Type II constructed mill building are designed as simply supported members. Connections or additional members (for trusses) are provided at the member ends for stability and lateral load resistance in unbraced frames.

Roof members in either a Type I or a Type III constructed building are designed as fixed or partially fixed members depending on the degree of fixity at their connections.

CONCLUSIONS

References and assumptions are presented to assist the designer in his attempt to establish an economical mill building design. Some simplified assumptions presented in this paper have been used with success. However, the designer should determine whether these assumptions are valid for his building before he utilizes them.

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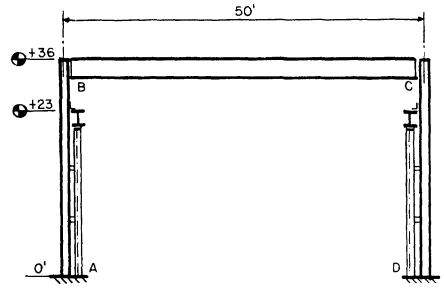

Fig. 4. Building geometry

APPENDIX I—DESIGN EXAMPLE

The following design example (Fig. 4) is based upon an actual case encountered by the author.

The load combinations established in the "Load" section of this paper are used throughout the example problem.

Because column and craneway girder design seem to create the greatest controversy, the author presents a detailed design analysis of these two members. The design of the other mill building components are not presented.

Design Criteria:

Column spacing = 25 ft o.c.
Continuous roof bracing
Long mill building—8 bays
Type II construction
A36 steel
8-in. block exterior wall
Neglect seismic and temperature loads

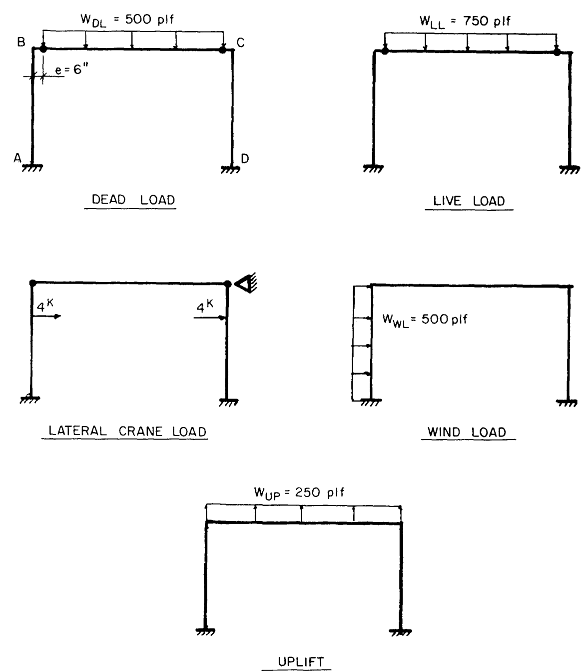

Fig. 5. Load and boundary conditions

Loads (Figs. 5 and 6):

DL = 20 psf
LL = 30 psf
WL = 20 psf
Maximum crane wheel load = 28 kips
Trolley weight = 6 kips
Lift capacity = 40 kips

Building Column Design (Figs. 4 and 5):

Cases I and III:

For load cases I and III, all frames in the building will be equally loaded; therefore, translation of the frames during column buckling can occur. Due to the semi-rigid girder-to-column connection, a reduction in the effective girder stiffness must be made when using the alignment chart in the AISC Manual.[12] See DeFalco and Marino's article[3] for the procedure.

Step 1. Determine effective column length:

Assume Z of girder-to-column connection = 0.0087×10^{-5} radians/kip-inch

$$C_p = \frac{Z \times 10^5 \times I_g}{L_g} = \frac{0.0087\ (2830)}{50.0} = 0.492$$

From Fig. 4 of Ref. 3:

$$C_e = 0.97$$

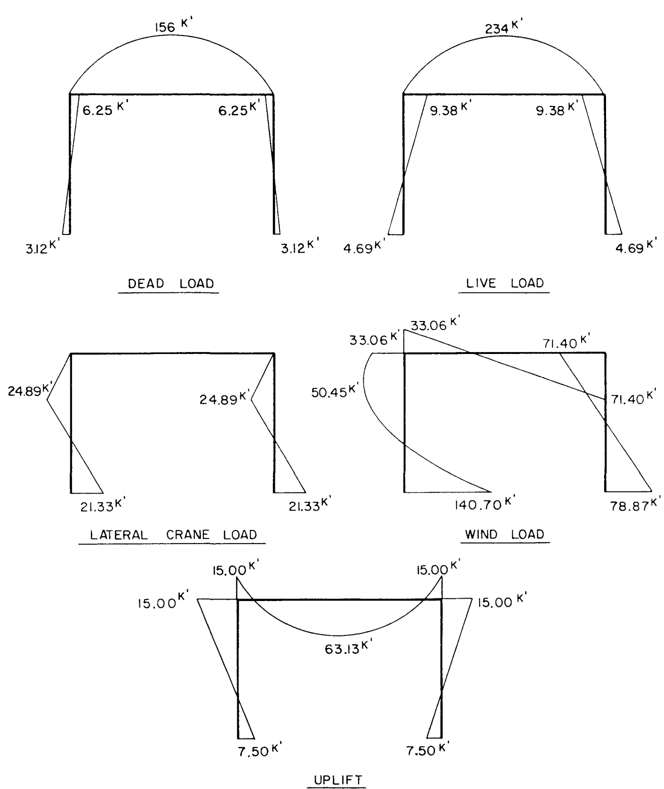

Fig. 6. Moment diagrams

$$G \text{ @ top of column} = \frac{EI_c/L_c}{EC_eI_g/L_g} = \frac{426/36.0}{(0.97)\ 2830/50.0} = 0.22$$

G @ base of column = 1.0

$K_x = 1.19$; $L_x = 36.0$ ft = 432 in.

$K_xL_x = 514$

$K_yL_y = 0$ (assume fully braced)

Step 2. Check combined stress equations for Case I:

Properties of W12 × 53:

$A = 15.6$ in.^{2}; $S_x = 70.7$ in.^{3}; $r_x = 5.23$ in.

$$K_xL_x/r_x = \frac{514}{5.23} = 98.3$$

$F_a = 13.19$ ksi; $F'_{ex} = 15.46$ ksi

At top of columns:

$$f_a = P/A = 31.25/15.6 = 2.00 \text{ ksi}$$

$$f_{bx} = M_x/S_x = 15.63\ (12)/70.3 = 2.65 \text{ ksi}$$

$$\frac{f_a}{F_a} = \frac{2.00 \text{ ksi}}{13.18 \text{ ksi}} = 0.152 \geq 0.150$$

∴ Check AISC Formula (1.6-1a):

$$\frac{f_a}{F_a} + \frac{C_mf_{bx}}{(1 - f_a/F'_{ex})F_b} \leq 1.0$$

$$0.152 + \frac{0.85\ (2.65)}{(1 - 2.00/15.46)\ 24.0} \leq 1.0$$

$0.152 + 0.108 = 0.260 < 1.0$ **o.k.**

By observation, Formula (1.6-lb) need not be checked.

Step 3. Check combined stress equations for Case III at base of windward column:

$$f_a = P/A = \frac{4.69}{15.6} = 0.30 \text{ ksi}$$

$$f_{bx} = M_x/S_x = \frac{109\ (12)}{70.7} = 18.50 \text{ ksi}$$

$$\frac{f_a}{F_a} = \frac{0.30}{13.19} = 0.023 < 0.15$$

∴Check AISC Formula (1.6-2):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \leq 1.0$$

$$0.023 + \frac{18.50}{24.0} \leq 1.0$$

$0.023 + 0.771 = 0.794 \leq 1.0$ **o.k.**

By observation all other points are **o.k.**

Conclusion:
The axial load contribution to the combined stress equations is very small while the bending moment is large, indicating that the column serves mainly as a vertical beam resisting wind loads.

Cases II and IV:
For load Cases II and IV, only one frame will be loaded with the lateral crane load. Since load Case I indicates a large reserve strength for each of the remaining building frames under gravity loads, these frames will tend to prevent the loaded frame from translating; therefore, the pinned top is justified.[15]

Step 1. Determine effective column lengths:

$K_x = 0.8$; $L_x = 36.0$ ft $= 432$ in.; $K_xL_x = 346$ in.; $K_yL_y = 0$

Step 2. Check combined stress equations for Case II:

$K_xL_x/r_x = (346/5.23) = 66.1$

$F_a = 16.83$ ksi; $F'_{ex} = 34.18$ ksi

At base of Column **AB**:

$f_a = P/A = 31.25/15.6 = 2.00$ ksi

$$f_{bx} = M_x/S_x = \frac{29.14(12)}{70.7} = 4.95 \text{ ksi}$$

$$\frac{f_a}{F_a} = \frac{2.00}{16.83} = 0.119 \leq 0.150$$

$\therefore$ Check Formula (1.6-2):

$$\frac{f_a}{F_a} + \frac{f_{bx}}{F_{bx}} \leq 1.0$$

$0.119 + 4.95/24.0 \leq 1.0$

$0.119 + 0.206 = 0.325 \leq 1.0$ **o.k.**

By observation all other points are **o.k.**

Step 3. Check combined stress equations for Case IV at base of leeward column:

$f_a = P/A = 23.44/15.6 = 1.50$ ksi

$$f_{bx} = M_x/S_x = \frac{39.61\ (12)}{70.7} = 6.72 \text{ ksi}$$

$0.089 + 6.72/24.0 \leq 1.0$

$0.098 + 0.280 = 0.369 \leq 1.0$ **o.k.**

By observation all other points are **o.k.**

Craneway girder design (Fig. 7):

Step 1. Moment due to vertical loads:

M_{max}	= 259 kip-ft
Impact 25%	= 65 kip-ft
Girder = $0.1 \times 25^2/8$	= 8 kip-ft
M_x	= 332 kip-ft

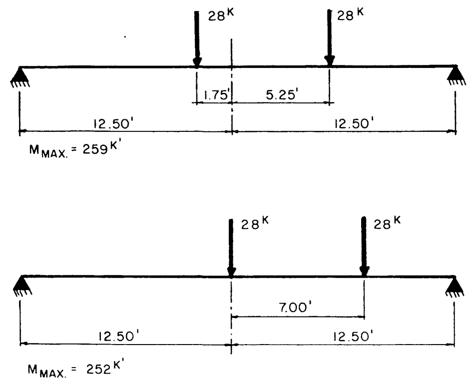

Fig. 7. Crane girder load diagrams

Step 2. Moment from lateral force:

$$M_y = 0.1\left(\frac{28+6}{56}\right)(259) = 15.7 \text{ kip-ft}$$

Step 3. Calculate S_x required:

$S_x = M_x/F_{bx} + 3.5(d/b)(M_y/F_{bx})$

Assume $F_{bx} = 24$ ksi and $d = 24$ in. $= 2$ ft

Note the following relationships:
Shapes 8 to 16 in. deep: $1 \leq d/b \leq 2$
Shapes 16 to 24 in. deep: $1.5 \leq d/b \leq 2.5$
Shapes 24 to 36 in. deep: $2 \leq d/b \leq 3$

Try a W24 with $d/b = 2$:

$$S_x = \frac{322 \times 12}{24} + 3.5(2)\left(\frac{15.7 \times 12}{24}\right) = 221 \text{ in.}^3$$

Step 4. Assume trial section:

Try W24 $\times$ 100:
$S_x = 250$ in.^{3}; $S_y = 37.2$ in.^{3}; $d/A_f = 2.58$; $r_t = 3.18$

Step 5. Calculate combined stresses:

$L_xd/A_f = 774$

$F_{bx} = 12{,}000/774 = 15.5$ ksi

$L_x/r_t = 116$

$F_{bx} = 24 - [(116)^2/1181] = 12.6$ ksi

$f_{bx} = 332(12)/250 = 15.9$ ksi $= 15.5$ ksi (close)

$$f_{by} = 2 \times (118/37.2) = \frac{10.1}{26.0} > 24 \text{ ksi } \textbf{n.g}$$

Step 6. Try W24 $\times$ 110:

$f_{bx} + f_{by} = 14.4 + 9.1 = 23.5$ ksi < 24 ksi **o.k.**

APPENDIX II—NOTATION

A = cross-sectional area
A_f = area of one flange
b = width of beam flange
C_e = efficiency coefficient
C_m = coefficient applied to bending term in stress interaction formula
C_p = flexibility index
d = depth of beam
DL = dead load
Eq = earthquake load
f_a = stress due to axial load
F_a = allowable axial stress
f_b = bending stress due to applied load
F_b = allowable bending stress
F'_e = Euler stress divided by a factor of safety
G = rigidity ratio
I_c = column moment of inertia
I_g = beam moment of inertia
K = effective length factor
L_g = length of beam
LL = live load
M_x = moment about x-axis
M_{max} = maximum moment due to vertical crane load
r_x = radius of gyration about the x-axis
r_y = radius of gyration about the y-axis
r_t = radius of gyration of a tee section about the strong axis
S_x = section modulus about x-axis
S_y = section modulus about y-axis
T = thermal load
UP = subscript designates uplift
W = load per unit length
WL = wind load
Z = moment rotation characteristic of a connection