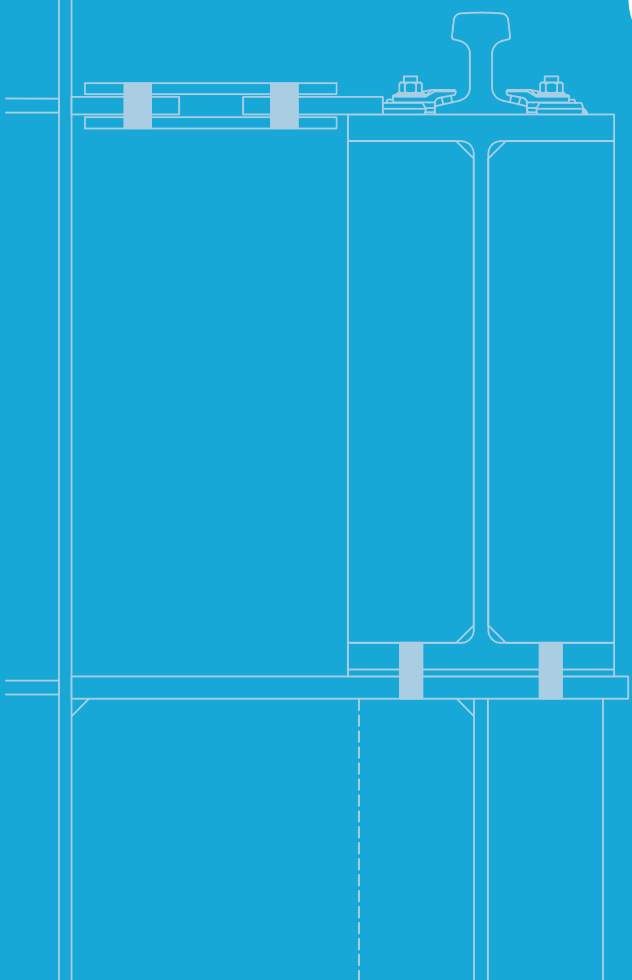
Crane-Supporting Steel Structures

Design Guide



R.A. MacCrimmon
Acres International
Niagara Falls, Ontario

GUIDE FOR THE DESIGN OF CRANE-SUPPORTING STEEL STRUCTURES

R.A. MACCRIMMON

ACRES INTERNATIONAL LIMITED

NIAGARA FALLS, ONTARIO

CANADIAN INSTITUTE OF STEEL CONSTRUCTION
INSTITUT CANADIEN DE LA CONSTRUCTION EN ACIER
201 CONSUMERS ROAD, SUITE 300
WILLOWDALE, ONTARIO M2J 4G8

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FOREWORD

The Canadian Institute of Steel Construction is a national industry organization representing the structural steel, open-web steel joist and steel plate fabricating industries in Canada. Formed in 1930 and granted a Federal charter in 1942, the CISC functions as a nonprofit organization promoting the efficient and economic use of fabricated steel in construction.

As a member of the Canadian Steel Construction Council, the Institute has a general interest in all uses of steel in construction. CISC works in close co-operation with the Steel Structures Education Foundation (SSEF) to develop educational courses and programmes related to the design and construction of steel structures. The CISC supports and actively participates in the work of the Standards Council of Canada, the Canadian Standards Association, the Canadian Commission on Building and Fire Codes and numerous other organizations, in Canada and other countries, involved in research work and the preparation of codes and standards.

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This booklet has been prepared and published by the Canadian Institute of Steel Construction. It is an important part of a continuing effort to provide current, practical, information to assist educators, designers, fabricators, and others interested in the use of steel in construction.

Although no effort has been spared in an attempt to ensure that all data in this book is factual and that the numerical values are accurate to a degree consistent with current structural design practice, the Canadian Institute of Steel Construction, the author and his employer, Acres International, do not assume responsibility for errors or oversights resulting from the use of the information contained herein. Anyone making use of the contents of this book assumes all liability arising from such use. All suggestions for improvement of this publication will receive full consideration for future printings.

CISC is located at

201 Consumers Road, Suite 300
Willowdale, Ontario, M2J 4G8

and may also be contacted via one or more of the following:

Telephone: (416) 491-4552
Fax: (416) 491-6461
Email: info@cisc-icca.ca
Website: www.cisc-icca.ca

Revisions

This Edition of the Design Guide supercedes all previous versions posted on the CISC website: *www.cisc-icca.ca*. Future revisions to this Design Guide will be posted on this website. Users are encouraged to visit this website periodically for updates.

CHAPTER 1 - INTRODUCTION

This guide fills a long-standing need for technical information for the design and construction of crane-supporting steel structures that is compatible with Canadian codes and standards written in Limit States format. It is intended to be used in conjunction with the National Building Code of Canada, 2005 (NBCC 2005), and CSA Standard S16-01, Limit States Design of Steel Structures (S16-01). Previous editions of these documents have not covered many loading and design issues of crane-supporting steel structures in sufficient detail.

While many references are available as given herein, they do not cover loads and load combinations for limit states design nor are they well correlated to the class of cranes being supported. Classes of cranes are defined in CSA Standard B167 or in specifications of the Crane Manufacturers Association of America (CMAA). This guide provides information on how to apply the current Canadian Codes and Standards to aspects of design of crane-supporting structures such as loads, load combinations, repeated loads, notional loads, monosymmetrical sections, analysis for torsion, stepped columns, and distortion induced fatigue.

The purpose of this design guide is twofold:

1. To provide the owner and the designer with a practical set of guidelines, design aids, and references that can be applied when designing or assessing the condition of crane-supporting steel structures.
2. To provide examples of design of key components of crane-supporting structures in accordance with:
   (a) loads and load combinations that have proven to be reliable and are generally accepted by the industry,
   (b) the recommendations contained herein, including NBCC 2005 limit states load combinations,
   (c) the provisions of the latest edition of S16-01, and,
   (d) duty cycle analysis.

The scope of this design guide includes crane-supporting steel structures regardless of the type of crane. The interaction of the crane and its supporting structure is addressed. The design of the crane itself, including jib cranes, gantry cranes, ore bridges, and the like, is beyond the scope of this Guide and is covered by specifications such as those published by the CMAA.

Design and construction of foundations is beyond the scope of this document but loads, load combinations, tolerances and deflections should be in accordance with the recommendations contained herein. For additional information see Fisher (1993).

In the use of this guide, light duty overhead cranes are defined as CMAA Classes A and B and in some cases, C. See Table 3.1. Design for fatigue is often not required for Classes A and B but is not excluded from consideration.

The symbols and notations of S16-01 are followed unless otherwise noted. Welding symbols are generally in accordance with CSA W59-03.

The recommendations of this guide may not cover all design measures. It is the responsibility of the designer of the crane-supporting structure to consider such measures. Comments for future editions are welcomed.

The author wishes to acknowledge the help and advice of; Acres International, for corporate support and individual assistance of colleagues too numerous to mention individually, all those who have offered suggestions, and special thanks to Gary Hodgson, Mike Gilmor and Laurie Kennedy for their encouragement and contributions.

CHAPTER 2 - LOADS

2.1 General

Because crane loads dominate the design of many structural elements in crane-supporting structures, this guide specifies and expands the loads and combinations that must be considered over those given in the NBCC 2005. The crane loads are considered as separate loads from the other live loads due to use and occupancy and environmental effects such as rain, snow, wind, earthquakes, lateral loads due to pressure of soil and water, and temperature effects because they are independent from them.

Of all building structures, fatigue considerations are most important for those supporting cranes. Be that as it may, designers generally design first for the ultimate limit states of strength and stability that are likely to control and then check for the fatigue and serviceability limit states. For the ultimate limit states, the factored resistance may allow yielding over portions of the cross section depending on the class of the cross-section as given in Clause 13 of S16-01. As given in Clause 26 of S16-01, the fatigue limit state is considered at the specified load level - the load that is likely to be applied repeatedly. The fatigue resistance depends very much on the particular detail as Clause 26 shows. However, the detail can be modified, relocated or even avoided such that fatigue does not control. Serviceability criteria such as deflections are also satisfied at the specified load level.

Crane loads have many unique characteristics that lead to the following considerations:

(a) An impact factor, applied to vertical wheel loads to account for the dynamic effects as the crane moves and for other effects such as snatching of the load from the floor and from braking of the hoist mechanism.

(b) For single cranes, the improbability of some loads, some of short duration, of acting simultaneously is considered.

(c) For multiple cranes in one aisle or cranes in several aisles, load combinations are restricted to those with a reasonable probability of occurrence.

(d) Lateral loads are applied to the crane rail to account for such effects as acceleration and braking forces of the trolley and lifted load, skewing of the travelling crane, rail misalignment, and not picking the load up vertically.

(e) Longitudinal forces due to acceleration and braking of the crane bridge and not picking the load up vertically are considered.

(f) Crane runway end stops are designed for possible accidental impact at full bridge speed.

(g) Certain specialized classes of cranes such as magnet cranes, clamshell bucket cranes, cranes with rigid masts (such as under hung stacker cranes) require special consideration.

This guide generally follows accepted North American practice that has evolved from years of experience in the design and construction of light to moderate service and up to and including steel mill buildings that support overhead travelling cranes (AISE 2003, Fisher 1993, Griggs and Innis 1978, Griggs 1976). Similar practices, widely used for other types of crane services, such as underslung cranes and monorails, have served well (MBMA 2002). The companion action approach for load combinations as used in the NBCC 2005, and similar to that in ASCE (2002) is followed.

2.2 Symbols and Notation

The following symbols and nomenclature, based on accepted practice are expanded to cover loads not given in Part 4 of the NBCC 2005. The symbol, L, is restricted to live loads due only to use and occupancy and to dust buildup. The symbol C means a crane load.

C_{vs} - vertical load due to a single crane

C_{vm} - vertical load due to multiple cranes

C_{ss} - side thrust due to a single crane

C_{sm} - side thrust due to multiple cranes

C_{is} - impact due to a single crane

C_{im} - impact due to multiple cranes

C_{ls} - longitudinal traction due to a single crane in one aisle only

C_{lm} - longitudinal traction due to multiple cranes

C_{bs} - bumper impact due to a single crane

C_d - dead load of all cranes, positioned for maximum seismic effects

D - dead load

E - earthquake load (see Part 4, NBCC 2005)

H - load due to lateral pressure of soil and water in soil

L - live load due to use and occupancy, including dust buildup (excludes crane loads defined above)

S - snow load (see Part 4, NBCC 2005)

T - See Part 4, NBCC 2005, but may also include forces induced by operating temperatures

W - wind load (see Part 4, NBCC 2005)

Additional information on loads follows in Section 2.3.

2.3 Loads Specific to Crane-Supporting Structures

2.3.1 General

The following load and load combinations are, in general, for structures that support electrically powered, top running overhead travelling cranes, underslung cranes, and monorails. For examples of several different types of cranes and their supporting structures, see Weaver (1985) and MBMA (2002).

Lateral forces due to cranes are highly variable. The crane duty cycle may be a well-defined series of operations such as the pick up of a maximum load near one end of the bridge, traversing to the centre of the bridge while travelling along the length of the runway, releasing most of the load and travelling back for another load. This is sometimes the case in steel mills and foundries. On the other hand, the operation may be random as in warehousing operations. Weaver (1985) provides examples of duty cycle analyses albeit more appropriate for crane selection than for the supporting structure.

Crane supporting structures are not usually designed for a specific routine but use recommended factors for crane loading as shown in Table 2.1. These are based on North American practice (Fisher 1993, Griggs and Innis 1978, Rowswell 1987). Other jurisdictions, e.g., Eurocodes, have similar but different factors. In addition to these, load factors for the ultimate limit states as given in Section 2.4 are applied. A statistically significant number of field observations are needed to refine these factors.

AISE (2003) notes that some of the recommended crane runway loadings may be somewhat conservative. This is deemed appropriate for new mill type building design where the cost of conservatism should be relatively low. However when assessing existing structures as covered in Chapter 6 engineering judgment should be applied judiciously as renovation costs are generally higher. See AISE (2003), CMAA (2004), Griggs (1976), Millman (1991) and Weaver (1985) for more information.

2.3.2 Vertical Loads

Impact, or dynamic load allowance, is applied only to crane vertical wheel loads, and is only considered in the design of runway beams and their connections. Impact is factored as a live load. AISE Report No. 13 recommends that impact be included in design for fatigue, as it is directed to the design of mill buildings. For most applications, this is thought to be a conservative approach. Following Rowswell (1978) and Millman (1996) impact is not included in design for fatigue.

For certain applications such as lifting of hydraulic gates, the lifted load can jamb and without load limiting devices, the line pull can approach the stalling torque of the motor, which may be two to three times the nominal crane lifting capacity. This possibility should be made known to the designer of the structure.

Table 2.1
Crane Vertical Load, Side Thrust and Tractive Force as Percentages of Respective Loads

Crane Type[a]	Vertical Load Including Impact	Total Side Thrust (two sides)-Greatest of:			Tractive Force
	Maximum Wheel Load[b]	Lifted Load[c]	Combined Weight of Lifted Load[c] and Trolley	Combined Weight of Lifted Load[c] and Crane Weight	Maximum Load on Driven Wheels
Cab Operated or Radio Controlled	125	40[d]	20[e]	10[d]	20
Clamshell Bucket and Magnet Cranes[f]	125	100	20	10	20
Guided Arm Cranes, Stacker Cranes	125	200	40[g]	15	20
Maintenance Cranes	120	30[d]	20	10[d]	20
Pendant Controlled Cranes	110		20	10	20
Chain Operated Cranes[h]	105		10		10
Monorails	115		10		10

Notes:

(a) Crane service as distinct from crane type is shown in Section 3.4.2.

(b) Occurs with trolley hard over to one end of bridge.

(c) Lifted load includes the total weight lifted by the hoist mechanism but unless otherwise noted, not including the column, ram, or other material handling device which is rigidly guided in a vertical direction during hoisting.

(d) Steel mill crane service (AISE 2003).

(e) This criterion has provided satisfactory service for light (see Table 3.1) to moderate duty applications and is consistent with the minimum requirements of the NBCC 2005.

(f) Severe service as in scrap yards and does not include magnet cranes lifting products such as coils and plate in a warehousing type operation.

(g) Lifted load includes rigid arm.

(h) Because of the slow nature of the operation, dynamic forces are less than for a pendant controlled cranes.

In determining crane vertical loads, the dead weight of the unloaded crane components by definition is a dead load. Historically, information provided on weights of crane components, particularly trolleys, has been rather unreliable and therefore is not necessarily covered by the commonly used dead load factor. Caution should be exercised and if deemed necessary, the weight should be verified by weighing.

Crane manufacturers provide information on maximum wheel loads. These loads may differ from wheel to wheel, depending on the relative positions of the crane components and the lifted load. The designer usually has to determine the concurrent wheel loads on the opposite rail from statics, knowing the masses of the unloaded crane, the trolley, the lifted load, and the range of the hook(s) (often called hook approach) from side to side. See Figure 4. Note that minimum wheel loads combined with other loads such as side thrust may govern certain aspects of design. Foundation stability should be checked under these conditions.

2.3.3 Side Thrust

Crane side thrust is a horizontal force of short duration applied transversely by the crane wheels to the rails. For top running cranes the thrust is applied at the top of the runway rails, usually by double flanged wheels. If the wheels are not double flanged, special provisions, not covered by this document, are required to ensure satisfactory service and safety. For more information see CMAA (2004) and Weaver (1985). For underslung cranes the load is applied at top of the bottom flange. Side thrust arises from one or more of

- acceleration or braking of the crane trolley(s)
- trolley impact with the end stop
- non-vertical hoisting action
- skewing or “crabbing” of the crane as it moves along the runway
- misaligned crane rails or bridge end trucks

The effect of the side thrust forces are combined with other design loads as presented subsequently. Side thrust is distributed to each side of the runway in accordance with the relative lateral stiffness of the supporting structures. For new construction it is assumed that the cranes and supporting structures are within tolerances. Severe misalignment, as one may find in older or poorly maintained structures, can lead to unaccounted for forces and consequential serious damage.

Side thrust from monorails is due only to non-vertical hoisting action and swinging, therefore, the values in Table 2.1 are less then those for bridge cranes.

The number of cycles of side thrust is taken as one-half the number of vertical load cycles because the thrust can be in two opposite directions.

More information can be found in AISE (2003), CMAA (2004), Fisher (1993), Griggs and Innis (1978), Griggs (1976), Millman (1996), Rowswell (1987), and Tremblay and Legault (1996)

2.3.4 Traction Load

Longitudinal crane tractive force is of short duration, caused by crane bridge acceleration or braking. If the number of driven wheels is unknown, take the tractive force as 10% of the total wheel loads.

2.3.5 Bumper Impact

This is a longitudinal force exerted on the crane runway by a moving crane bridge striking the end stop. The NBCC 2005 does not specifically cover this load case. Provincial regulations, including for industrial establishments, should be reviewed by the structure designer. Following AISE (2003), it is recommended that it be based on the full rated speed of the bridge, power off. Because it is an accidental event, the load factor is taken as 1.0.

2.3.6 Vibrations

Although rarely a problem, resonance should be avoided. An imperfection in a trolley or bridge wheel could set up undesirable forcing frequencies.

From Rowswell (1987), the probable amplification of stress that may occur is given by the following magnification factor:

$$Magnification\ Factor = \frac{1}{1 - \left[\frac{forcing\ \ frequency}{natural\ \ frequency}\right]^2}$$

2.4 Load Combinations Specific to Crane-Supporting Structures

The structure must also be designed for load combinations without cranes, in accordance with the NBCC 2005. Load combinations comprising fewer loads than those shown below may govern.

Where multiple cranes or multiple aisles are involved, only load combinations that have a significant possibility of occurring need to be considered. Load combinations as given in the NBCC 2005, but including crane loads, are presented here.

Crane load combinations C1 to C7 shown in Table 2.2 are combinations of the crane loads given in Section 2.2 that are used in the industry. For more information see AISE (2003), Fisher (1993), and MBMA (2002).

For load combinations involving column-mounted jib cranes, see Fisher and Thomas (2002).

Table 2.2
Crane Load Combinations

C1	$C_{vs} + 0.5C_{ss}$	Fatigue.
C2	$C_{vs} + C_{is} + C_{ss} + C_{ls}$	Single crane in a single aisle.
C3	$C_{vm} + C_{ss} + C_{ls}$	Any number of cranes in single or multiple aisles.
C4	$C_{vm} + 0.5C_{sm} + 0.9C_{lm}$	Two cranes in tandem in one aisle only. No more than two need be considered except in extraordinary circumstances.
C5	$C_{vm} + 0.5C_{sm} + C_{im} + 0.5C_{lm}$	One crane in each adjacent aisle.
C6	$C_{vm} + 0.5C_{sm}$	Maximum of two cranes in each adjacent aisle, side thrust from two cranes in one aisle only. No more than two need be considered except in extraordinary circumstances
C7	$C_{vs} + C_{is} + C_{bs}$	Bumper impact.

2.4.1 Fatigue

The calculated fatigue stress range at the detail under consideration, to meet the requirements of Clause 26 of S16-01 and as described in Chapter 3 of this document, will be taken as that due to C1.

Note: *Dead load is a steady state and does not contribute to the stress range. However, the dead load stress may cause the stress range to be entirely in compression and therefore favourable or wholly or partly in tension and therefore unfavourable.*

2.4.2 Ultimate Limit States of Strength and Stability

In each of the following inequalities, the factored resistance, ϕR, and the effect of factored loads such as $0.9D$, are expressed in consistent units of axial force, shear force or moment acting on the member or element of concern. The most unfavourable combination governs. In any combination, the dead load and the first transient load are the principal loads and the second transient load is the companion load. Except in inequalities Nos 4, 6 and 7, the crane load combination C is any one of the combinations C2 to C6.

1. $\phi R \geq 1.4D$
2. $\phi R \geq 1.25D + 1.5C + 0.5S \text{ or } 0.4W \text{ or } 0.5L$
3. $\phi R \geq 1.25D + 1.5S \text{ or } 1.4W \text{ or } 1.5L + 0.5C$ *
4. $\phi R \geq 1.25D + 1.0C7$
5. $\phi R + 0.9D \geq 1.4W \text{ or } 1.5L \text{ or } 1.5C \text{ or } 1.5S$
6. $\phi R \geq 1.0\left[D + C_d\right] + 1.0E + 0.25S$
7. $\phi R + 1.0\left[D + C_d\right] \geq 1.0E$

* The companion load factor 0.5 on the crane load C in inequality No. 3 is considered appropriate for structures supporting Crane Service Classifications A, B, and C. For Crane Service Classifications D, E, and F a companion load factor of up to 1.0 should be considered

Notes:

1) *The combinations above cover the whole steel structure. For design of the crane runway beams in an enclosed structure for instance, S and W would not normally apply.*
2) *Crane runway columns and occasionally crane runway beams support other areas with live loads.*
3) *The effects of factored imposed deformation, 1.25T, lateral earth pressure, 1.5H, factored pre-stress, 1.0P, shall be considered where they affect structural safety.*
4) *The earthquake load, E, includes earthquake-induced horizontal earth pressures.*
5) *Crane wheel loads are positioned for the maximum effect on the element of the structure being considered.*
6) *The basic NBCC load factors shown above are in accordance with information available at the time of publication of this document. The designer should check for updates.*

CHAPTER 3 - DESIGN FOR REPEATED LOADS

3.1 General

The most significant difference between ordinary industrial buildings and those structures that support cranes is the repetitive loading caused by cranes. Steel structures that support cranes and hoists require special attention to the design and the details of construction in order to provide safe and serviceable structures, particularly as related to fatigue. The fatigue life of a structure can be described as the number of cycles of loading required to initiate and propagate a fatigue crack to final fracture. For more detailed information, see Demo and Fisher (1976), Fisher, Kulak and Grondin (2002), Kulak and Smith (1997), Fisher and Van de Pas (2002), Millman (1996), Reemsnyder and Demo (1998) and Ricker (1982).

The vast majority of crane runway beam problems, whether welded or bolted, are caused by fatigue cracking of welds, bolts and parent metal. Problems have not been restricted to the crane runway beams, however. For example, trusses or joists that are not designed for repeated loads from monorails or underslung cranes have failed due to unaccounted for fatigue loading. For all crane service classifications, the designer must examine the structural components and details that are subjected to repeated loads to ensure the structure has adequate fatigue resistance. Members to be checked for fatigue are members whose loss due to fatigue damage would adversely affect the integrity of the structural system.

As given in S16-01, Clause 26, the principal factors affecting the fatigue performance of a structural detail are considered to be the nature of the detail, the range of stress to which the detail is subjected, and the number of cycles of a load. The susceptibility of details to fatigue varies and, for convenience, Clause 26, in common with fatigue requirements in standards world-wide, specifies a limited number of detail categories. For each category the relationship between the allowable fatigue stress range of constant amplitude and the number of cycles of loading is given. These are the S-N (stress vs. number of cycles) curves.

Two methods of assessing crane-supporting structures for fatigue have developed. Historically, at least for structures with relatively heavy crane service, the first of these was to classify the structure by "loading condition"as related to the crane service. Section 3.4.1 covers this. While this has worked reasonably well, this approach has two shortcomings. First, the number of cycles, by "pigeon-holing" the structure, may be set somewhat too high as related to the service life of the structure in question, and second, only the maximum stress range is considered. The second, more recent, approach is to assess the various ranges of stress and corresponding numbers of cycles to which the detail is subjected and to try to determine the cumulative effect using the Palmgren-Miner rule as given in Section 3.3.2. This can be advantageous, especially in examining existing structures.

The assessment of the number of cycles nN requires care as an element of the structure may be exposed to fewer or more repetitions than the number of crane lifts or traverses along the runway. For example, if out-of-plane bending is exerted on a crane runway beam web at its junction with the top flange by a rail which is off-centre, a significant repetitive load occurs at every wheel passage and the number of cycles is "n" times the number of crane passages "N" where "n" is the number of wheels on the rail, per crane. Also, for short span crane runway beams depending on the distances between the crane wheels, one pass of the crane can result in more than one loading cycle on the beam, particularly if cantilevers are involved. On the other hand, when the crane lifts and traverses are distributed among several bays, a particular runway beam will have fewer repetitions that the number of lifts. For additional discussion of crane-structure interaction, see Section 5.2.

The provisions here apply to structures supporting electrically operated, top running, overhead travelling cranes (commonly referred to as EOT's), underslung cranes, and monorails. Light duty crane support structures, where components are subjected to not more than 20 000 cycles of repeated load and where high ranges of stress in fatigue susceptible details, are not present need not be designed for fatigue.

It is necessary to evaluate the effect of repeated crane loadings before concluding that fewer than 20 000 cycles of loading will occur. Referring to Table 3.3 and 3.4, and Section 3.4.3, even supporting structures for Crane Service Classification A could require consideration of somewhat more than 20 000 full cycles of repeated load.

3.2 Exclusion for Limited Number of cycles

Clause 26.3.5 of S16-01 presents the situation when the number of stress range cycles of loading is limited and fatigue is therefore not likely to be a problem. First, fatigue-sensitive details with high stress ranges, likely with stress reversals, are excluded from these provisions and should be investigated for fatigue in any case. Second, the requirements of Clause 26.1 that the member and connection be designed, detailed, and fabricated to minimize stress concentrations and abrupt changes in cross section are to be met. Only then, if the number of cycles is less than the greater of two criteria, 20 000 or γ / f_{sr}^3 is no fatigue check required. The detail category may determine the limit.

For example, for detail category E, from Table 10, the fatigue life constant, $\gamma = 361 \times 10^9$ MPa and, say, calculations give a fatigue stress range, $f_{sr} = 210\ MPa$. Hence the second criterion yields a limit of 39 000 cycles. Therefore, the limit of 39 000 cycles controls and if the detail is subject to fewer than 39 000 cycles, no fatigue check is necessary.

3.3 Detailed Load-Induced Fatigue Assessment

3.3.1 General

Clause 26.3.2 of S16-01 gives the design criterion for load-induced fatigue as follows:

$$F_{sr} \geq f_{sr}$$

where

f_{sr} = calculated stress range at the detail due to passage of the fatigue load

F_{sr} = fatigue resistance

$$= \left(\frac{\gamma}{\eta N} \right)^{1/3} \geq F_{srt}$$

γ = fatigue life constant, see Clause 26.3.4

η = number of stress range cycles at given detail for each application of load

N = number of applications of load

F_{srt} = constant amplitude threshold stress range, see Clauses 26.3.3 and 26.3.4.

Above the constant amplitude fatigue threshold stress range, the fatigue resistance (in terms of stress range) is considered to vary inversely as the number of stress range cycles to the 1/3 power. Rearranging the expression for the fatigue resistance, the number of cycles to failure is:

$$\eta N = \gamma / F_{sr}^3$$

Accordingly the number of cycles to failure varies inversely as the stress range to the third power. Below the constant amplitude fatigue threshold stress range, the number of cycles to failure varies inversely as the stress range to the fifth power.

The effect of low stress range cycles will usually be small on crane supporting structures but should be investigated nonetheless. It requires the addition of a second term to the equivalent stress range (see Section 3.3.3) where the value of *m* is 5 for the relevant low stress range cycles.

As stated in Section 2.4, a dead load is a steady state and does not contribute to stress range. However, the dead load stress may cause the stress range to be entirely in compression and therefore favourable or wholly or partly in tension and therefore unfavourable. In this regard, web members of trusses subjected to live load compressive stresses may cycle in tension when the dead load stress is tensile. This condition may also apply to cantilever and continuous beams. On the other hand, the compressive stresses due to dead load in columns may override the tensile stresses due to bending moments.

For additional information on analysis of stress histories where complex stress variations are involved, see Fisher, Kulak and Smith (1997), and Kulak and Grondin (2002).

3.3.2 Palmgren - Miner Rule

The total or cumulative damage that results from fatigue loading, not applied at constant amplitude, by S16-01 must satisfy the Palmgren-Miner Rule:

$$\sum\left[\frac{(\eta N)_i}{N_{fi}}\right] \le 1.0$$

where:

$(\eta N)_i$ = number of expected stress range cycles at stress range level I.

N_{fi} = number of cycles that would cause failure at stress range I.

In a typical example, the number of cycles at load level 1 is 208 000 and the number of cycles to cause failure at load level 1 is 591 000. The number of cycles at load level 2 is 104 000 and the number of cycles to cause failure at load level 2 is 372 000. The total effect or "damage" of the two different stress ranges is

$$\frac{208\,000}{591\,000} + \frac{104\,000}{372\,000} = 0.63 < 1.0 \quad \text{OK}$$

3.3.3 Equivalent Stress Range

The Palmgren-Miner rule may also be expressed as an equivalent stress range.

$$\Delta\sigma_e = \left[\sum \alpha_i \Delta\sigma_i^m\right]^{1/m}$$

where:

$\Delta\sigma_e$ = the equivalent stress range

α_i = $\dfrac{(\eta N)_i}{N_{fi}}$

$\Delta\sigma_\iota$ = the stress range level I.

m = 3 for stress ranges at or above the constant amplitude threshold stress range. For stress ranges below the threshold, m = 5.

For example, if the stress range at level 1 in the above example is 188 MPa and the stress range at level 2 is 219 MPa, then the equivalent stress range is

$$\left[\left(\frac{208000}{312000}\right)\left(188^3\right) + \left(\frac{104000}{312000}\right)\left(219^3\right)\right]^{1/3} \approx 200 \quad MPa$$

A calculation of the number of cycles to failure (see Section 3.3.1) and where $\gamma = 3\,930 \times 10^9$ gives 491 000 cycles. Since the actual number of cycles is 312 000, the percentage of life expended (damage) is $(312\,000/491\,000) \cdot 100\% = 64\%$. This is essentially the same result as in 3.3.2 (equivalent stress range was rounded off).

3.3.4 Equivalent Number of Cycles

For a particular detail on a specific crane runway beam, the cumulative fatigue damage ratio can be assessed considering that:

(1) the detail has a unique fatigue life constant as listed in Table 10 of S16-01,

(2) the stress range is proportional to the load,

(3) the number of cycles at the detail, nN, is proportional to the number of cycles of load on the crane runway beam, N,

(4) above and below the constant amplitude fatigue threshold stress range the number of cycles to failure varies inversely as the stress range to the 3rd and 5th power respectively.

The equivalent number of cycles at the highest stress range level, N_e, where N_m is the number at the highest stress range level, for cycles above the constant amplitude fatigue threshold stress range, is

$$N_m + \sum\left[N_i\left(C_i/C_m\right)^3\right]$$

where C_m and C_i are the respective proportional constants of the stress ranges at the maximum stress range level and the stress range level respectively to the crane-induced load. For cycles below the constant amplitude fatigue threshold stress range, similar terms are developed based on the flatter, 1/5 slope of the S-N diagram. Many cycles below the constant amplitude fatigue threshold stress range do cause fatigue damage, albeit at a reduced rate.

For the example in Section 3.3.3, the equivalent number of cycles at the highest stress range level is

$$104\,000 + 208\,000\left(188/219\right)^3 = 104\,000 + 131\,584 = 235\,584 \quad \text{cycles}$$

A calculation of the number of cycles to failure (see Section3.3.1) and where $\gamma = 3\,930 \times 10^9$ gives $374\,160$ cycles. The percentage of life expended (damage) is $\left(235\,584/374\,160\right) \cdot 100\% = 63\%$. This is the same result as in Section 3.3.2.

This approach is useful for relating duty cycle information to class of service and can be used to simplify calculations as shown in Section 3.5 and Appendix A, Design Example 2.

3.3.5 Fatigue Design Procedure

The recommended procedure for design for fatigue is as follows:

- Choose details that are not susceptible to fatigue.
- Avoid unaccounted for restraints.
- Avoid abrupt changes in cross section.
- Minimize range of stress where practicable.
- Account for eccentricities of loads such as misalignment of crane rails.
- Examine components and determine fatigue categories.
- Calculate stress ranges for each detail.
- Calculate fatigue lives for each detail.
- Compare the fatigue life of the details to the results obtained from the detailed load induced fatigue assessment.
- Adjust the design as necessary to provide adequate resistance to fatigue.

3.4 Classification of Structure

3.4.1 General

To provide an appropriate design of the crane supporting structure, the Owner must provide sufficiently detailed information, usually in the form of a duty cycle analysis or results thereof. While the structure designer may provide input to a duty cycle analysis, the basic time and motion analysis should be done by plant operations personnel. A duty cycle analysis of interest to the structure designer should yield the spectrum of loading cycles for the structure taking into account such items as:

- numbers of cranes, including future use,
- total number of cycles for each crane, by load level,
- the distribution of the above cycles for each crane over the length of the runway and along the length of the bridge of the crane(s).

The number of cycles of loading, by load level, can therefore be determined for the critical location and for all other elements of the structure.

In the past it was somewhat common for designers to classify the structure based on ranges of number of cycles at full load. In some references (Fisher 1993, AISE 2003, CMAA 2004, MBMA 2002) this was associated with a "loading condition." Some of these references (Fisher 1993, Fisher and Van de Pas 2002, and MBMA 2002) provide information on relating the loading condition to class of crane service. A duty cycle analysis was done to the extent required to assess which of several loading conditions was most suitable.

New fatigue provisions are based on working with actual numbers of cycles and require consideration of cumulative fatigue damage. Therefore the loading condition concept is no longer recommended, and is used only for reference.

In order that the designer can determine ηN for all structural elements subject to fatigue assessment, the design criteria should contain a statement to the effect that cycles refers to crane loading cycles N.

Unless otherwise specified by the owner, Clause 26.1 of S16-01 gives a life of 50 years. It is now common for owners to specify a service life span of less than 50 years.

This section of the guide provides methods of classifying the crane-supporting structure, describes preparation of the structure design criteria for fatigue, and describes fatigue design procedure.

3.4.2 Crane Service Classification

Crane service classifications as given in CSA B167-96 closely resemble the same classifications of the Crane Manufacturer's Association of America (CMAA). Lifting capacity is not restricted in any classification and there is a wide variation in duty cycles within each classification. For instance, number of lifts per hour does not necessarily suggest continuous duty and may be more relevant to rating of electrical gear than to structural design. Weaver (1985) provides additional information on the operation of several types of crane service and notes that the service classification may differ for the different components of a crane. The main hoist, auxiliary hoist, and bridge may have three different classifications.

Bridge speeds vary from 0.2 m/sec (usually massive cranes in powerhouses) to 2 m/sec (usually lower capacity cab operated industrial cranes), to as much or more than 5 m/sec in some automated installations.

There are many more cranes of Classes A and B, used for lighter duty, than heavy duty cranes of Classes D, E and F. Class C cranes of moderate service may in some cases be included in this lighter duty category. For additional information, see Table 3.1.

Lighter duty cranes may be pendant, cab, or radio controlled. While fatigue must be considered, many of the problems associated with their supporting structures are due to poor design details, loose construction tolerances and unaccounted for forces and deflections. Examples of poor details are welding runway beams to columns and brackets and inappropriate use of standard beam connections. Refer to the figures for other examples. Regarding Table 2.1, the designer must decide, after assessing the design criteria (see Chapter 7), which of the three lighter duty crane types should apply.

For chain operated cranes, because of the slow (usually less than 1 m/sec hoisting, trolley and bridge speed) nature of the operation the number of cycles expected are not sufficient to warrant design for fatigue.

Portions of the classifications relevant to the supporting structure are given here. The service classification is based on the frequency of use of the crane and the percentage of the lifts at or near rated capacity.

- **Class A (Standby or Infrequent Service)**

  This covers cranes used in installations such as powerhouses, public utilities, turbine rooms, motor rooms, and transformer stations, where precise handling of equipment at slow speeds with long, idle periods between lifts is required. Hoisting at the rated capacity may be done for initial installation of equipment and for infrequent maintenance.

- **Class B (Light Service)**

  This covers cranes used in repair shops, light assembly operations, service buildings, light warehousing, or similar duty, where service requirements are light and the speed is slow. Loads may vary from no load to occasional full-rated loads, with 2 - 5 lifts per hour.

- **Class C (Moderate Service)**

  This covers cranes used in machine shops or paper mill machine rooms, or similar duty, where service requirements are moderate. The cranes will handle loads that average 50% of the rated capacity, with 5 - 10 lifts/hour, with not over 50% of the lifts at rated capacity.

- **Class D (Heavy Service)**

  This covers cranes that may be used in heavy machine shops, foundries, fabricating plants, steel warehouses, container yards, lumber mills, or similar duty, and standard duty bucket and magnet operations where heavy-duty production is required. Loads approaching 50% of the rated capacity are handled constantly during the working period. High speeds are desirable for this type of service, with 10 - 20 lifts/hour, with not over 65% of the lifts at rated capacity.

- **Class E (Severe Service)**

  This requires cranes capable of handling loads approaching the rated capacity throughout their life. Applications may include magnet, bucket, and magnet/bucket combination cranes for scrap yards, cement mills, lumber mills, fertilizer plants, container handling, or similar duty, with 20 or more lifts/hour at or near the rated capacity.

- **Class F (Continuous Severe Service)**

  This requires cranes capable of handling loads approaching rated capacity continuously under severe service conditions throughout their life. Applications may include custom-designed specialty cranes essential to performing the critical work tasks affecting the total production facility. These cranes must provide the highest reliability, with special attention to ease-of-maintenance features.

The load spectrum, reflecting the actual or anticipated crane service conditions as closely as possible, may be used to establish the crane service classification. The load spectrum (CMAA 2004) leads to a mean effective load factor applied to the equipment at a specified frequency. Properly sized crane components are selected based on the mean effective load factor and use as given in Table 3.1 adapted from CMAA (2004).

From the load spectrum (CMAA 2004), the mean effective load factor is:

$$k = \sqrt[3]{\sum W_i^{\,3} P_i}$$

where:

k = Mean effective load factor (used to establish crane service class only).

W_i = Load magnitude; expressed as a ratio of the lift load to the rated capacity. Lifts of the hoisting gear without the lifted load must be included.

P_i = The ratio of cycles under the 1ift load magnitude condition to the total number of cycles. $\sum P_i = 1.0$

For example, if from 100 000 lifts, 10 000 are at full capacity, 70 000 are at 30% of capacity, and 20 000 are at 10% of capacity, then:

$$k = \sqrt[3]{1.0^3 \times 0.1 + 0.3^3 \times 0.7 + 0.1^3 \times 0.2} = 0.492$$

Table 3.1 shows a definition of Crane Service Class in terms of Load Class and use. Note that this table does not necessarily describe the crane carrying structure.

Table 3.1
Crane Service Classification based on k.

***k* = Mean Effective Load Factor**	**Use**			
	Irregular occasional use followed by long idle periods	**Regular use of intermittent operation**	**Regular use in continuous operation**	**Regular use in severe continuous operation**
≤ 0.53	A *	B *	C	D
$0.531 < k \leq 0.67$	B *	C *	D	E
$0.671 < k \leq 0.85$	C	D	E	F
$0.85 < k \leq 1.00$	D	E	F	F
* Generally fits the light duty category of service.				

3.4.3 Number of Full Load Cycles Based on Class of Crane

The number of full load cycles from the CMAA fatigue criteria for crane design is listed in Table 3.2.

These criteria cannot be applied directly to a supporting structure. Issues that must be considered are:

(a) span lengths of the supporting structure compared to the crane wheel spacing.

(b) the number of spans over which the crane operates. For instance, if the crane operates randomly over “x” spans, the equivalent number of full load cycles for each span might be more like the number of cycles above, divided by “x”. On the other hand, in a production type operation, each span on one side of the runway may be subjected to almost the same number of full load cycles as the crane is designed for if the crane travels the length of the runway fully loaded each time.

(c) the number of cranes.

(d) over or under utilization of the crane with respect to its class.

For Class of Crane Service A, B, or C where the lifting operation is randomly distributed along the length of the runway beams and across the crane bridge, it is suggested that the number of cycles of loading of varying amplitude for components of the crane supporting structure can be estimated as the number of full load cycles for the class of crane divided by the number of spans and multiplied by the number of cranes further provided that the life of the runway is the same as the life of the crane.

Table 3.2
CMAA Number of Full Load Cycles by Class of Crane

Class of Crane	Number of Thousands of Full Load Cycles
A	100
B	200
C	500
D	800
E	2 000
F	> 2 000

Table 3.3
Ranges of Existing Suggestions for Cycles for Design of Crane-supporting Structures

Class of Crane	Number of Thousands of Full Load Cycles
A	0 to 100
B	20 to 100
C	20 to 500
D	100 to 2000
E	500 to 2000
F	Greater than 2000

The basis of selecting these numbers is not explained nor is it evident whether these are the total number of cycles or the equivalent number of full cycles (see Section 3.3.3).

For instance, the runway for a new Class C crane, 5 spans, would be designed for 100 000 cycles.

The suggested number of cycles for the design of the crane supporting structure as a function of the class of crane vary widely among the sources. Fisher (1993), Fisher and Van de Pas (2001), and MBMA (2002) give the values shown in Table 3.3.

Table 3.4 presents the recommended number of cycles for the design of the crane supporting structure based on the structural class of service, itself derived from the crane service classification. The numbers were determined by duty cycle analyses as presented in Section 3.4.4. Examples of the analyses are given in Section 3.5. "N" is defined as full load cycles. Each full load cycle can exert nN cycles on the supporting structure. To differentiate from the crane, the class of service for the crane-supporting structure will be prefixed with S.

By comparing the recommended number of cycles in Table 3.4 to the number of cycles for the crane in Table 3.2, it appears that for this approach to structural classification, the structural class of service should be 20% of the full load cycles for crane Classes A, B and C, and 50% for crane Classes D, E and F.

The information in Table 3.4 is not meant to take the place of a duty cycle analysis for the installation being investigated.

3.4.4 Fatigue Loading Criteria Based on Duty Cycle Analysis

As discussed in Sections 3.4.1 and 3.4.3, a duty cycle analysis for one or more cranes will yield the spectrum of loading cycles for the crane-supporting structure. Note that only the results of the duty cycle analysis that are of interest to the structure designer are shown herein. To determine the location of the critical element of the structure and its loading spectrum requires a time and motion study beyond the scope of this document. Weaver (1985) and Millman (1996) provide examples of duty cycle analyses.

Table 3.4
Recommended Number of Cycles for Design of the Crane-supporting Structure

Structural Class of Service	Recommended [a] Number of Thousands of Full Load Cycles, N
SA	20
SB	40
SC	100
SD	400
SE	1000
SF	Greater than 2000 [b]

a Used as a calibration of the supporting structure (Structural Class of Service) to class of crane service in Chapter 4. As is the case for the crane, the supporting structure will withstand many more cycles of varying amplitude loading.

b Due to the unlimited fatigue life of the crane, a duty cycle and analysis is required to define the fatigue design criteria.

After identifying the critical component of the structure and the equivalent number of full loading cycles, the fatigue design criteria for the structure can be prepared.

This is the most accurate and is the preferred method of determining the fatigue design criteria.

3.4.5 Preparation of Design Criteria Documentation

The structural class of service for entry into Checklist Table 4.1 is determined from the duty cycle information or from previous procedures related to crane service class.

Refer also to Chapter 7 for other information that should be obtained for preparation of the design criteria.

3.4.5.1 Fatigue Criteria Documentation Based on Duty Cycle Analysis

Compute N, the equivalent number of full loading cycles for the location deemed most critical. This is the lower limit of N to be used in Table 4.1. For example, if N is calculated to be 500 000 cycles, go to Structural Class of Service SD. Use the actual numbers of cycles of loading from that point on. The spectrum of loading cycles for the critical elements of the structure should be included in the design criteria.

The design criteria statement for fatigue design might appear as follows:

The supporting structure will be designed for cyclic loading due to cranes for the loads as follows:

Load Level, % of Maximum Wheel Loads	***Number of Thousands of Cycles, N****
100	*10*
75	*50*
52	*100*
25	*200*

** Means number of passes of cranes.*

Design for cyclic side thrust loading will be for 50% of each number of cycles above with the corresponding percentage of side thrust for cyclic loading.

3.4.5.2 Criteria Documentation Based on Class of Crane Service (Abbreviated Procedure)

The design criteria statement for fatigue design might appear as follows:

The supporting structure will be designed for cyclic loading due to cranes for the following loads.

Load Level, % of Maximum Wheel Loads	*Number of Cycles, N**
100	*40,000*
** Means number of passes of cranes* *Design for cyclic side thrust loading will be for 50% of the number of cycles above with the corresponding percentage of side thrust for cyclic loading.*	

3.5 Examples of Duty Cycle Analyses

3.5.1 Crane Carrying Steel Structures Structural Class Of Service SA, SB, SC

A Class C crane operates over several spans (say 5 or 6). In accordance with the CMAA standards, the crane is designed for 500 000 cycles of full load, but only 50% of the lifts are at full capacity. The lifts are evenly distributed across the span of the crane bridge. The operation along the length of the runway has been studied and the conclusion is that no one span of the supporting structure is subjected to more than 250 000 cycles of a crane with load and 250 000 cycles of an unloaded crane. The loading spectrum for the critical member of the supporting structure is shown in Table 3.5.

Table 3.5 - Example Loading Spectrum for Class SA, SB & SC

Percent of Maximum Wheel Loads	Number of Cycles, N	Description
100	62 500	Fully loaded crane
80	62 500	*
60	62 500	*
40	62 500	*
30	250 000	Unloaded crane
* Loads and trolley positions vary.		

The equivalent number of cycles at full wheel loads is calculated as follows:

$$N = 62500 + 62500\left(0.8^3 + 0.6^3 + 0.4^3\right) + 250000 \times 0.3^3$$
$$= 62500 + 49500 + 6750 = 118750 \quad \text{cycles}$$

The supporting structure should be designed for, say, 120 000 full cycles.

118 750 cycles is 24% of the number of cycles that the crane is designed for.

The above duty cycle is probably more severe than most for these classes of cranes and this type of operation, so use 20% as the criterion. This should serve as a conservative assessment for most applications.

3.5.2 Crane Carrying Steel Structures Structural Class of Service SD, SE, SF

A Class D or E crane operates in a well defined production mode over several spans. The crane is designed for 2 000 000 cycles of full load. In addition to the loaded cycles, the supporting structure will be subjected to an equal number of unloaded cycles. The operation has been studied, the critical member is identified, and the conclusion is that the loading spectrum for the critical member of the supporting structure is as follows:

The equivalent number of cycles at full wheel loads is calculated as shown in Table 3.6.

$$N = 500\,000 + 500\,000\left(0.8^3 + 0.6^3 + 0.4^3\right) + 2\,000\,000 \times 0.3^3$$
$$= 500\,000 + 396\,000 + 54\,000 = 950\,000 \quad \text{cycles}$$

The supporting structure should be designed for, say, 1 000 000 full cycles.

950 000 cycles is 48 % of the number of cycles that the crane is designed for.

The above duty cycle is probably more severe than most for these classes of cranes and this type of operation. Use 50 % as the criterion. This should serve as a conservative assessment for most applications.

Table 3.6 - Example Loading Spectrum for Class SD, SE & SF

Percent of Maximum Wheel Loads	Number of Cycles, N	Description
100	500 000	Fully loaded crane
80	500 000	*
60	500 000	*
40	500 000	*
30	2 000 000	Unloaded crane

* Loads and trolley positions vary.

CHAPTER 4 - DESIGN AND CONSTRUCTION MEASURES CHECK LIST

4.1 General

The check list in Table 4.1, calibrated to structural class of service (see Section 3.4.3), has been prepared as a guide for the design criteria and construction specifications. Other sections of this design guide provide additional recommendations. "Runway beam" refers to the runway beam or girder. Items that are fatigue related, and therefore not necessarily part of the design of structures subjected to less than 20 000 cycles, are designated (f). Items designated "∗ "are not usually required. Those designated "•"are recommended. Those designated "r" are required in order to provide a structure that can reasonably be expected to perform in a satisfactory manner. A check list prepared by other engineers experienced in the design of crane-supporting structures may differ.

Parallelling the requirements of Clause 4 of S16-01, it is suggested that before final design, a design criteria document should be prepared by the designer of the structure for approval by the owner. As a minimum, this document should define the codes and standards, the materials of construction, the expected life of the structure, crane service classifications, loads and load combinations, criteria for design for fatigue, and a record of the design and construction measures selected. Foundation conditions and limitations should also be included.

Table 4.1
Design Check List for Crane Supporting Steel Structures

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit 'N'					
	20	40	100	400	1000	Not Defined
Items 1 to 41 , are generally related, but not limited to, analysis and design						
1. Design drawings should show crane load criteria including numbers, relative positions, lifting capacity, dead load of bridge, trolley and lifting devices, maximum wheel loads, bridge speed, bumper impact loads at the ends of the runway, and fatigue loading criteria for vertical and horizontal crane-induced loads by the criteria determined in accordance with Sections 3.4.5.1 or 3.4.5.2.	r	r	r	r	r	r
2. Use of continuous runway beams is not recommended without careful evaluation of possible problems due to uneven settlement of supports, uplift, fatigue, and difficulty in reinforcing or replacing.	∗	∗	•	•	•	•

Table 4.1 continued

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit 'N'					
	20	40	100	400	1000	Not Defined
3. Brackets should not be used to support crane beams with unfactored reactions greater than about 250 kN.	r	r	r	r	r	r
4. Building and crane support columns made up of two or more column sections should be tied together to act integrally. (f)	*	*	*	r	r	r
5. Where crane columns and building support columns are not tied rigidly, the axial shortening of the crane carrying columns should be accounted for. (f)	*	*	•	r	r	r
6. Where building bents share crane- induced lateral loads, a continuous horizontal bracing system should be provided at or above the crane runway level. (f)	r	r	r	r	r	r
7. Use of diaphragm action of roof deck for crane-induced load sharing between bents not advisable. (f)	r	r	r	r	r	r
8. Use of girts for lateral support for crane carrying columns not advisable unless designed for cyclic loading. For Classes A, B and C, this provision need not apply to the building column if there is a separate crane carrying column attached to the building column. (f)	*	*	•	r	r	r
9. Crane bridge tractive and bumper impact forces should be accounted for by the use of vertical bracing directly under the runway beams or by suitable horizontal bracing or diaphragm action to the adjacent building frame. The effects of torsion about the vertical axis of rigid frame members should be resisted by bracing.	•	•	•	r	r	r
10. Use of tension field analysis for runway beam webs not advisable unless service loads can be accommodated without such action. (f)	•	•	r	r	r	r
11. Eccentricities of crane-induced loads such as rails not centred within specified tolerance over beams below and weak axis bending on columns should be accounted for.	r	r	r	r	r	r

Table 4.1 continued

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit 'N'					
	20	40	100	400	1000	Not Defined
12. Side thrust from cranes should be distributed in proportion to the relative lateral stiffness of the structures supporting the rails.	r	r	r	r	r	r
13. Structural analysis should account for three-dimensional effects such as distribution of crane induced lateral loads between building bents.	r	r	r	r	r	r
14. Vertical deflection of runway beams under specified crane loads, one crane only, not including impact, should not exceed the indicated ratios of the span.	r $\frac{1}{600}$	r $\frac{1}{600}$	r $\frac{1}{600}$	r $\frac{1}{800}$	r $\frac{1}{1\,000}$	r $\frac{1}{1\,000}$
15. Horizontal deflection of runway beams under specified crane loads should not exceed the indicated ratios of the span.	r $\frac{1}{400}$	r $\frac{1}{400}$	r $\frac{1}{400}$	r $\frac{1}{400}$	r $\frac{1}{400}$	r $\frac{1}{400}$
16. Building frame lateral deflection at run-way beam level from unfactored crane loads or from the unfactored 1-in-10-yr wind load should not exceed the specified fractions of the height from column base plate or 50 mm, whichever is less.	r $\frac{1}{240}$	r $\frac{1}{240}$	r $\frac{1}{240}$	r $\frac{1}{400}$	r $\frac{1}{400}$	r $\frac{1}{400}$
Exceptions for pendant-operated cranes are noted:	The lesser of 1/100 or 50 mm					
17. Relative lateral deflection (change in gauge) of runway rails due to gravity loads should not exceed 25 mm.	r	r	r	r	r	r
18. Effect of temperatures above +150°C and below -30°C should be investigated.	•	•	•	r	r	r
19. Ends of simply supported ends of runway beams should be free of restraint to rotation in the plane of the web and free from prying action on hold down bolts. (f)	•	•	r	r	r	r

Table 4.1 continued

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit 'N'					
	20	40	100	400	1000	Not Defined
20. Where lateral restraint to runway beams is provided, the relative movements between the beam and the supporting structure should be accounted for. (f)	•	•	r	r	r	r
21. Complete joint penetration welds with reinforcing should be provided at runway plate girder web-to-top-flange connection. (f)	*	*	•	r	r	r
22. Web and flange splice welds subjected to cyclic loads should be complete joint penetration. (f)	r	r	r	r	r	r
23. Electro-slag and electro-gas welding not recommended for splices subjected to cyclic tensile loads. (f)	•	•	•	r	r	r
24. Use of intermittent fillet welds not advisable for cover plates or cap channels, even though always in compression. (f)	*	*	r	r	r	r
25. Runway plate girder web-to-top-flange weld should be capable of supporting all of the crane wheel load, distributed through the rail and top flange.	r	r	r	r	r	r
26. Column cap plates supporting crane runway beams and similar details should have complete joint penetration welds unless contact bearing as defined by "at least 70% of the surfaces specified to be in contact shall have the contact surfaces within 0.2 mm of each other and no remaining portion of the surfaces specified to be in bearing contact shall have a separation exceeding 0.8 mm". Shimming should not be permitted. Alternatively, the welds should be designed to withstand all imposed static and cyclic loads. (f)	*	*	•	r	r	r
27. Runway beam stiffeners should be adequately coped. Provide complete joint penetration weld for stiffener to beam top flange. Continuously weld or bolt stiffener to the web. (f)	*	*	•	r	r	r

Table 4.1 continued

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit 'N'					
	20	40	100	400	1000	Not Defined
28. Intermediate stiffeners should be applied in pairs. (f)	•	•	•	r	r	r
29. Detailing and installation of crane rails should be in accordance with generally accepted practice to limit wear and tear on the runway and cranes:	r	r	r	r	r	r
– rails rigidly attached to flanges beneath not advisable. (f)	•	•	r	r	r	r
– where the rail is installed by others after the runway beams are in place, the final installation should be in accordance with the recommendations included herein unless previously agreed to the contrary. The runway should be inspected and accepted by the rail installer prior to installing rails.	r	r	r	r	r	r
– rail clips should provide lateral restraint to the rail.	•	•	•	r	r	r
30. Impact factors are applied to crane vertical wheel loads and should be applied to runway beams and their connections and connecting elements, including brackets, but excluding columns and foundations.	r	r	r	r	r	r
31. Design of runway beams should account for gravity loads applied above the shear centre.	r	r	r	r	r	r
32. Use of slip critical bolted connections for connections subjected to repeated loads or vibrations required. (f)	r	r	r	r	r	r
33. Use of fully pretensioned high strength bolts in all bracing and roof members advisable. (f)	*	*	•	r	r	r
34. Use of snug tight bolted connections for secondary members acceptable. (f)	yes	yes	yes	no	no	no
35. Use of elastomeric rail pad advisable. (f)	*	*	*	•	•	•

Table 4.1 continued

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit ‘N’					
	20	40	100	400	1000	Not Defined
36. Ratio of depth-to-web thickness of crane beam webs should not exceed: (f) $h/t \leq \dfrac{1900}{\sqrt{M_f/\phi S}}$	•	•	•	r	r	r
37. Where web crippling may occur, web bearing stresses should be below yield and avoid the possibility of web buckling.	*	*	•	r	r	r
38. Use of rubber noses on rail clips advisable. (f)	*	*	•	r	r	r
39. Use of welded rail splices advisable. (f)	*	*	•	r	r	r
40. Out-of-plane flexing of crane beam webs at terminations of stiffeners, diaphragm connections and the like, should be accounted for in the design. (f)	*	*	•	r	r	r
41 Struts to columns located below the crane runway beams should be designed for fatigue loads due to effects of flexure in the bottom flanges of the runway beams. (f)	•	•	•	r	r	r
Items 42 to 53 cover, generally, but are not limited to, inspection and construction						
42. Removal of shims before grouting base plates recommended.	*	*	*	•	•	•
43. Web and flange splice welds subjected to cyclic loads should be ground flush, grinding direction parallel to direction of axial or bending stress. (f)	*	*	•	r	r	r

Table 4.1 continued

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit 'N'					
	20	40	100	400	1000	Not Defined
44. Crane beams or trusses of spans greater than 20 m should be cambered for dead load plus 50% of live load deflection, without impact.	r	r	r	r	r	r
45. If top cover plates or cap channels are used, contact should be maintained across the section after welding. (f)	*	*	-	r	r	r
46. Standards for Inspection and Quality of Welding for runway beams and their connections to the supporting structure should be as follows:						
– splices in tension areas of web plates and flanges should be radiographically or ultrasonically inspected to the degree shown in percent. (f)	15	25	50	100	100	100
– web and flange splices in compression areas should be radiographically or ultrasonically inspected to the degree shown in percent.	*	*	*	25	25	50
– complete joint penetration web-to-flange welds should be ultrasonically inspected to the degree shown in percent. (f)	*	*	50	100	100	100
– fillet welded web-to-flange welds should be 100% inspected by liquid penetrant or magnetic particle inspection. (f)	•	•	r	r	r	r
47. Special procedures for maintaining tolerances for shop fabrication and field erection of columns and crane beams recommended.	*	*	•	r	r	r
48. Accumulated fabrication and erection tolerance for centring of crane rail over supporting beam should be such that the crane rail eccentricity shall not exceed three-fourths of the beam web thickness.	•	•	r	r	r	r

Table 4.1 continued

Description	Structural Class of Service One Crane Only					
	SA	SB	SC	SD	SE	SF
	Thousands of Full Loading Cycles Lower Limit ‘N’					
	20	40	100	400	1000	Not Defined
49. Unless otherwise agreed with the crane manufacturer, centre-to-centre distance of crane rails as constructed should not exceed the indicated distance in mm from the theoretical dimension at 20°C.	r ±10	r ±10	r ±10	r ±8	r ±6	r ±6
50. Tops of adjacent runway beam ends should be level to within the distances shown in mm.	3	3	3	2	2	2
51. Crane runway beam bearings should be detailed, fabricated and assembled so that even bearing is achieved after final alignment. No gap should exceed 1.0 mm. Any proposal for shimming should achieve the required tolerances and should be submitted to the designer for review. (f)	*	*	•	r	r	r
52. Flanges of crane runway beams, for a distance of 500 mm from their ends, should not be curved as viewed in cross section, and should be normal to the webs to within 1 mm in 300 mm.	•	•	•	r	r	r
53. Ends of rails at splices should be hardened and milled. (f)	*	*	•	r	r	r

4.2 Comments on the Checklist

Comments on the check list in Table 4.1 are given in Table 4.2 on an item-by-item basis. Background information on most of the measures can be found in the references. The recommendations for Crane Service Classifications A and B take into consideration at least 20 000 cycles of loading but because they are defined as infrequent or light service cranes, they are generally less stringent than for Classes C to F. There is a wide range of duty cycles for Class C but because severe problems have not been widespread historically, the recommendations are somewhat less severe than for Class D.

When measures are correlated to crane service classification, it should be noted that the suggested measures have been calibrated to a concept of a crane runway of several spans and with one crane on each runway. See Section 3.4.3 for details.

For all classes, the designer of the structure should advise the owner prior to completing the design if recommended measures are not intended to be implemented, along with reasons. For design-build projects, it is recommended that the owner's specification requires that the same information is included in the proposal.

Table 4.2
Comments of Check List for Design of Crane Supporting Steel Structures

Item	Comment	See Figure
2	Occasionally runway beams are designed as simple span but supplied in lengths that provide continuity over supports. Fisher (1993) and Rowswell (1987) provide information on this topic. The structure designer should consider the effect of settlement of supports, particularly for underslung cranes. The owner should be made aware of any proposal to provide continuity, and the implications thereof.	-
4	The crane runway support is sometimes designed as a separate set of columns, beams, and longitudinal bracing, attached to the adjacent building support columns for lateral support of the runway and to reduce the unsupported length of crane runway carrying columns. This is acceptable if properly executed, taking into account movements such as shown in Figure 8. However, the interconnecting elements are occasionally subjected to unaccounted for repeated forces and distortion induced fatigue. Flexible connections are undesirable for the more severe classifications of services.	1 8
5	Refer also to Item 4. The interconnecting elements and connections may be subject to distortion-induced fatigue.	8
6	For light capacity cranes where building framing is relatively rugged, sharing of loads between building bents may not be required. Unless it can be shown that without help from roof diaphragm action, horizontal differential movement of adjacent columns due to crane side thrust or crane gravity loads is less than column spacing divided by 2 000, it is recommended that continuous horizontal bracing should be provided at roof level. In this way, the roofing material will not be subject to repeated severe diaphragm action.	3
7	This item does not preclude the use of metal deck to provide lateral support to compression flanges of purlins and top chords of joists or for diaphragm action provided that an effective horizontal bracing system for crane loads is in place.	-
8	This recommendation need not apply on light duty structures.	-
9	Refer to Fisher (1993) for additional information.	-
10	Clause 26.4.2 of S16-01 places a restriction on the h/w ratio under fatigue conditions. Tension field analysis is a post-buckling analysis and is not desirable under buckling distortion fatigue conditions.	-
11	Several eccentricities should be considered.	5

Table 4.2 continued

Item	Comment	See Figure
13	Some degree of three-dimensional analysis is required to adequately assess loads in horizontal bracing. Refer to Fisher (1993) and Griggs (1976) for additional information.	3
14/15	Recommended deflection limits for Items 14 and 15 are consistent with the recommendations of the CMAA. Deflections are elastic beam deflections. Differential settlement of foundations can cause serious problems and should be limited to 12 mm unless special measures are incorporated.	-
17	Excessively flexible columns and roof framing members can result in undesirable changes in rail-to-rail distance, even under crane-induced gravity loads that cause sway of the structure. These movements can create crane operational problems and unaccounted for lateral and torsional loads on the crane runway beams and their supports. Under some circumstances, final runway alignment should be left until after the full dead load of the roof is in place.	1
18	For applications where the ambient temperature range less between +150°C and -30°C, structural steel meeting the requirements of CSA G40.1 grade 350W can be expected to perform adequately. For service at elevated temperatures, changes in properties of the steel may warrant adjustment of design parameters. While notch toughness at low temperatures is often required by bridge codes, this is not usually a requirement for crane runway beams, one reason being the relatively small cost of replacement compared to a bridge beam.	-
19	Limiting restraint to rotation and prying action on bolts can often be accommodated by moving the hold-down bolts from between the column flanges to outside as shown in Figures 14 and 18. The cap plate thickness should be limited or use of finger tight bolts is recommended to minimize prying action on the bolts. Note that the eccentricity of vertical loads shown in Figure 18 may cause a state of tension in the column flanges. For design for fatique, large ranges of stress may have to be considered.	9 14 15 18
20	Where lateral restraint is not provided, the runway beams should be designed for bending about both the strong and weak axes. See AISC (1993), Rowswell and Packer (1989), and Rowswell (1987).The use of details that are rigid in out-of-plane directions should be avoided. S16-01 requires consideration of the effects of distortion induced fatigue.	13 14 15 16
21	The web-to-flange weld can be subjected to torsional forces due to lateral loads applied at the top of the rail and rail to flange contact surface not centred over the web beneath, for instance. There is no directly applicable fatigue category. Refer to AISE (2003) for additional information.	5 10
24	Use of intermittent fillet welds on tension areas of built up runway beams is prohibited by CSA W59. Intermittent fillet welds have shown poor resistance to fatigue and are categorically not allowed on dynamically loaded structures by some authorities such as AISE (2003) and AWS (1999). The use of these welds should be restricted to applications where fatigue is not a consideration.	-

Table 4.2 continued

Item	Comment	See Figure
26	The recommendations for contact bearing are similar to railroad bridge standards and are more stringent than for statically loaded structures.	9 18
27	Refer to Fisher (1993). Refer to figures for details at bottom flange.	16 18 19
29	Square bars welded to the crane runway beam beneath has been used successfully for less severe applications. Welds fastening rail bars should be properly sized to resist vertical loads, shear flow loads, and fatigue. The effects of induced continuity in otherwise simple spans should be accounted for. Intermittent fillet welds are not allowed in tension areas as would occur on continuous beams. A method to allow realignment of the rail and supporting beam should be provided. Railway type, ASCE, or other rails of hardened material should not be welded to the supporting structure under any circumstance. Bolted splices should be staggered. Rail splices should not occur over ends of beams. See Fisher (1993) and AISE (2003) for more information on detailing practices. A gap should be provided between the end of the rail and the end stop to allow for thermal movement of the rail.	13 14 15 16 17 18
30	The designer should review the complete connection that supports the runway beam for fatigue. Impact factors should be applied to cantilever brackets and for underslung cranes and monorails, to adjacent truss members and connections.	-
31	Refer to Section 5.9 and the CISC commentary on S16-01.	5 6
32	S16-01, Clause 22.2.2 provides requirements for use of pretensioned bolts and slip-critical connections. Some judgement on the part of the designer is required to determine whether the fatigue loads warrant slip critical connections for all main and secondary members, particularly where structural integrity would not be compromised. Slip critical connections for wind loads or reversals due to wind loads are not normally required. Use of finger tight bolts with burred threads or welded nuts is not recommended for connections subject to fatigue but may be considered, however, for lighter duty structures such as shown in Figures 14 and 15.	-
33	Bolts have come loose due to vibration and dropped, causing not only weakened connections, but also a safety hazard.	-
34	Snug tight bolts are acceptable in light duty applications for roof members, girts , and the like.	-
35	Elastomeric bearing pads have been shown to reduce noise, increase rail life, and reduce stresses at the web-to-flange junction of the crane runway beam beneath.	19
36	From AISE (2003) and S16-01.	-
37	See Item 9.	-

Table 4.2 continued

Item	Comment	See Figure
38	Rubber nosings have been shown to reduce failures of rail clips due to uplift from "bow wave" effect while at the same time resisting uplift. Rubber nosings should be used with elastomeric rail pads.	19
39	Welded rail splices should be used with elastomeric rail pads.	-
40	Many failures have occurred due to out-of-plane flexing.	10 19
41	As the bottom flange of the crane runway beam elongates due to flexure, repeated loads are imposed on struts beneath it.	9
42	There is no general agreement, but shims left in place are reported to have caused splitting of the concrete beneath. Levelling screws are recommended for large loose base plates. The usual method of removing shims is to leave edges exposed and pull them after the grout has sufficiently cured.	-
43	Only experienced operators should do this work and caution must be exercised to avoid notching the parent metal, particularly at tapers at changes in plate thickness.	25
45	Welding of cap channels to top flanges often results in a gap between the channel web and the flange beneath the crane rail, subjecting the welds to undesirable and unaccounted for forces that can cause premature cracking. The criteria for contact should be considered similar to that contained in Clause 28.5 of S16-01.	-
46	This item should be read in conjunction with requirements for welding details. A discontinuity in a continuous fillet weld in areas of tension or reversal can lead to a fatigue induced crack in the parent metal. Failure of any NDT test in a tension zone should lead to 100% testing of all tension area welds. Failure of the test in a compressive zone should result in testing double the recommended percentage.	25
47	See Section 5.27, Fisher (1993), ASCE (2002), and AISE (2003) for additional information.	24
48	The effect of rail eccentricity from the centre line of the runway beam web beneath under repeated loads can lead to premature failure due to unaccounted for torsional loads. Refer to Item 21, Section 5.28 and the references for more information.	5
49	This tolerance is subject to review by the crane manufacturer and the structure designer and may be increased, depending on the rail-to-rail distance and the crane wheel design.	24
51	See Item 26.	6 16 18
52	To provide proper bearing and to keep webs vertical and in line.	-

CHAPTER 5 - OTHER TOPICS

5.1 General

This chapter presents a number of topics briefly. More detailed information may be found in the references cited.

5.2 Crane-Structure Interaction in Mill or Similar Buildings

Obviously the crane itself and the supporting structure interact. The extent to which the structural designer takes this into account is a matter of judgement. That the crane bridge ties the two crane rails together is acknowledged when the transverse lateral forces due to trolley accelerations or to picking the load up non-vertically are distributed to the two crane rails in proportion to the lateral stiffness of the supporting structure. It is only necessary that friction or the double-flanged wheels transfer these forces to the rails. It follows that the crane could be considered a part of the structure under other load combinations provided only that the frictional force exceeds the appropriate specified or factored transverse lateral forces depending on the limit state being investigated.

A second factor to consider is that the dead weight of the crane may not be distributed symmetrically either transversely or longitudinally resulting in heavier wheel loads on one rail than the other or loads distributed non-uniformly along one rail from front to back. Be that as it may, pairs of crane wheels are usually articulated such that the vertical loads within the pair on a side are equal while multiple articulations increase the number of wheels with nominally equal loads.

Beyond this, however, the transverse stiffness of the crane end truck assemblies can affect the distribution of the lateral forces to the rails. Keep in mind that the function of the truck assemblies is to distribute the load to the wheels. In buildings such as mill buildings, heavy-duty cranes with several sets of wheels may have a wheelbase longer than the bay spacing. The crane does not simply impose a set of independent wheel loads on the structure because the end assembly may have a lateral stiffness comparable to that of the crane runway beam. It is not a question of a wind or other such load, with no structure behind it, which follows the structure as it deforms. But as the crane runway beam deflects the end truck assembly tends to span between the wheels that are acting against the hard spots. While common practice has been historically not to take this into account, the assessment of crane-structure interaction particularly when examining existing structures may be beneficial. For example the end truck assembly may in fact supply some continuity from span to span for transverse loads even when the lateral stiffening trusses are not continuous.

Note: *The argument presented above applies to side thrusts where friction or flanged wheels may generate the shear forces necessary for the two elements being bent to act together.*

5.3 Clearances

Every crane requires operating space that must be kept free of obstructions. The layout of an industrial building with overhead cranes must be developed in conjunction with this envelope. AISE (2003), CMAA (2004), MBMA (2002) and Weaver (1985) provide blank clearance diagrams. Problem areas that have been encountered are:

- cranes fouling with building frame knee braces,
- insufficient clearance allowed to the underside of the roof structure above, sometimes due to deflections and structural connections not shown on the design drawings,
- insufficient clearance under crane runway beams,
- insufficient clearance to face of columns . Weaver (1985) suggests that if personnel are allowed on the runway, then there should be about 450 mm clearance to face of columns, as little as 25 mm if not. Refer also to owner's safety standards,
- insufficient clearance to the building end wall, resulting in reduced operating space or costly "doghouse" extensions to the ends of the runways.

See Figure 4 for important clearance considerations. The references cited above give several other possible clearance considerations.

5.4 Methods of Analysis

At the very least second order elastic methods of analysis should be used for structures covered by this design guide in keeping with the philosophy of S16-01. Plastic design methods are not recommended except perhaps for rehabilitation studies where aspects such as deflection and fatigue may not control.

Use of computerized structural modelling with proven software to account for sway effects, P-Δ, instead of the more approximate methods of Clause 8.7.1 of S16-01 are recommended. Commonly used computer software is easily capable of not only doing second order elastic analysis, but by adding joints along the length of compression members subject to bending, the P-δ effects (Clause 13.8.4 of S16-01) are generated along with the P-Δ effects. Consideration of these effects can be simplified by judicious structural Modelling. The experienced designer should be able to isolate critical load combinations and thus reduce the number of load combinations that require a second order analysis.

5.5 Notional Loads

S16-01 requires use of "notional loads" to assess stability effects (Clause 8.7.2). This approach is somewhat different from AISE and AISC ASD, WSD and LRFD methods where effective lengths using the well known but approximate elastic factor "K" are used. Notional loads are used in Europe, Australia and South Africa and are recognized by US researchers. Their use avoids weak beams. Notional loads are fictitious or pseudo-lateral loads, taken in S16-01 as a small percentage (0.5%) of the factored gravity loads at each "storey" of the structure. The translational load effects thus generated (otherwise there might be no lateral load) transform the sway buckling or bifurcation problem to an in-plane strength problem. There is no need to consider "effective" length factors greater than one.

The use of notional loads applied to a crane supporting structure requires considerations beyond those usually encountered in residential or commercial construction because lateral loads are applied at the crane runway beam level. The definition of a "storey" for an industrial building may be open to interpretation and the concepts of "effective" and "equivalent" lengths as applied to stepped columns requires steps in the analysis and design that are not well covered in commonly used design aids.

MacCrimmon and Kennedy 1997 provide more detailed information and a worked example is presented. See also, Section 5.6.

5.6 Stepped Columns

Several different column configurations can be used for crane carrying structures (see Fisher 1993 and Galambos 1998). Where stepped columns are used and where the components of built-up sections are connected so that they act integrally, the concept of "equivalent lengths" of the column segments may be applied and a buckling analysis may be required. Galambos (1998) and MacCrimmon and Kennedy (1997) provide the designer with information on limit states analysis and design methods. Fisher (1993) and AISE (2003) contain design aids.

Section 5.5 refers to aspects of notional loads that require consideration. Schmidt (2001) provides an alternative method of analysis of stepped columns using notional loads.

5.7 Building Longitudinal Bracing

For lighter crane duty service, a properly designed single plane of bracing at the columns should provide satisfactory service. A decision whether to add another plane of bracing, under the runway beams, should be taken considering the magnitude of the longitudinal forces and the effects of eccentricity in plan. It is suggested that when the magnitude of longitudinal forces due to traction or end stop collision exceed a (specified) load of 100 kN, that a second plane of bracing should be introduced. For large forces, and for Crane Service Classifications C and up, bracing also in the plane of the crane runway beams similar to that shown in Figure 9 is recommended.

Compared to ordinary industrial buildings, it is even more important in crane carrying structures subjected to repeated loads that the longitudinal bracing be located as close as possible to the mid point between expansion joints or ends of the building.

The interaction of continuous crane rails that are allowed to "float" along the length of the runway and a long building with expansion joints is complex. Experience has shown that these installations usually perform well when temperature fluctuations are not too extreme as is the usual case indoors. The rail might tend to migrate along the length of the runway, and adjustments may be necessary.

For more information, see Fisher (1993).

5.8 Building Expansion Joints

Distance between expansion joints, in general, should not exceed 150 m. Use of double columns is recommended over sliding joints, particularly where design for fatigue is required and for Crane Service Classifications C and up. For more information, see Fisher (1993).

Expansion joints are not usually provided in crane rails. The rail is allowed to “float” over the joint.

5.9 Mono-symmetric Crane Runway Beams, Lateral Torsional Buckling

Mono-symmetric sections such as shown in Figure 5 for crane runway beams are used not only for top running cranes, but for monorails and underslung cranes as well. These sections often have long laterally unsupported spans.

CSA Standard S16-01 does not cover their analysis. Clause 13.6(e) refers to a “rational method of analysis”such as the SSRC guide (Galambos 1998). The AISC LRFD specification contains provisions for lateral-torsional buckling of Mono-symmetric three-plate sections but the common section of a wide flange with a cap channel is not well covered.

Practical and theoretical aspects of crane runway beams that are Mono-symmetric (I-shaped beams with channel caps, for instance) are addressed by Ellifrit and Lue 1998, Galambos 1998, Laman 1996, Lue and Ellifrit 1993, Salmon and Johnson 1996, and Tremblay and Legault 1996.

Top running cranes apply loads to the crane runway beams above the shear centre (see Figure 5), thereby reducing resistance to lateral torsional buckling. Additionally, side thrust is applied at or above the top flange level, generating a torsional moment on the section.

A problem of concern is that torsional effects due to accidental eccentricities as shown in Figure 5 (see also 5.11) are not well defined and experience must be relied upon. To account for the above, designers use a procedure known as the flexure analogy (see Figure 6) whereby the top flange is designed to resist all lateral loads and the bottom flange assists in resisting torsional loads. The compressive stress due to the warping component is the most important quantity and the shear stress contributions are not of much significance. The influences of the warping section constant C_w, the St-Venant torsion constant J, also the influence of welding details, are addressed by Tremblay and Legault 1996.

Regardless of the degree of investigation of the effects of torsion, lateral-torsional buckling must be considered. The procedure for doubly symmetrical I shaped sections is given in S16-01 and Kulak and Grondin (2002). These calculations involve the quantities C_w, J, and the coefficient of mono-symmetry β. Fisher 1993 and Ellifrit and Lue 1998 provide useful recommendations and examples. Values for C_w for beams with cap channels can be found in Fisher (1993), and Lue and Ellifrit (1993). Approximations of built-up beam moment resistances for varying unbraced lengths are provided by Laman (1996) along with a Fortran program to generate moment capacities for these sections.

Sections other than beams with cap channels are often used. To aid in calculating section properties, the CISC has made available on its website a design aid for determining torsional section properties of steel shapes. Calculations for a W section with a continuously welded cap plate instead of a cap channel are similar to those for a single flange plate of similar area and moment of inertia in the y-y axis.

A rational design method follows and is used in Appendix A, Design Example 1.

5.9.1 Design Method

A rational method for calculating the factored moment resistance of a laterally unsupported beam, similar to the method proposed by Ellifritt and Lue (1998) is as follows:

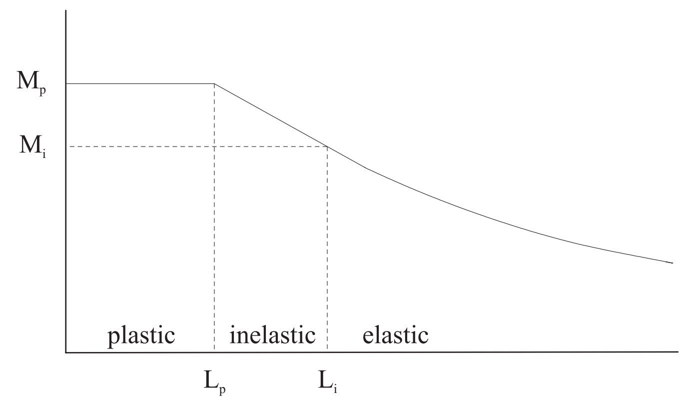
Referring to the typical moment resistance diagram above for unbraced lengths, the portion of the curve for the intermediate or inelastic range is reasonably close to a straight line. The AISC LRFD specification uses a straight line transition from the elastic buckling curve at $M_u = M_i$, $L = L_i$ to $M_u = M_p$, $L = L_p$.

Establish the class of section in bending and determine if the limiting strength may be governed by the yield stress or by local flange or web buckling.

For $L \leq L_p$, $M_r/\phi = M_p$

For $L_p < L \leq L_i$

The unfactored moment resistance for simply supported beams under uniform moment, loaded at the shear centre, can be determined by the following formula:



Moment resistance for unbraced length L

$$\frac{M_r}{\phi} = M_p - \left(M_p - M_i\right)\left[\frac{L - L_p}{L_i - L_p}\right] \leq M_p$$

For $L > L_i$, $M_r/\phi = M_u$

The general formula for M_u, the critical elastic moment of the unbraced Mono-symmetric beam, by Galambos (1998), is expressed by the following equations:

$$M_u = \frac{\pi C_b}{KL}\left[\sqrt{EI_y GJ}\left(B_1 + \sqrt{1 + B_2 + B_1^2}\right)\right]$$

where

$$B_1 = \frac{\pi\beta_x}{2KL}\sqrt{\frac{EI_y}{GJ}}$$

$$B_2 = \frac{\pi^2 EC_w}{(KL)^2 GJ}$$

β_x, C_w and J can be calculated using information in Part 7 of the CISC Handbook of Steel Construction for calculating torsional sectional properties.

$$M_i = F_L S_{xc} \text{ or } F_y S_{xt}\text{, whichever is less}$$

$$L_p = 1.76 r_{yc} \sqrt{\frac{E}{F_y}}$$

L_i can not be calculated directly and must be solved by a trial and error iteration until the unbraced length used in the formula for M_u produces a moment $M_u = M_i$. That length is then L_i.

The symbols from the reference documents are not necessarily covered by S16-01. Symbols different to or in addition to those in S16-01, for these calculations only, are as follows:

C_b = Moment diagram modifier, dependent on moment gradient, usually taken as 1.0

F_L = $F_y - F_r$

F_r = Compressive residual stress in the flange
= 69 MPa for rolled shapes
= 114 MPa for (continuously) welded shapes

K = Coefficient to account for increased moment resistance of a laterally unsupported beam segment when subject to a moment gradient (ω_2 in S16-01), usually taken as 1.0

L_i = Limiting laterally unbraced length for inelastic lateral-torsional buckling

L_p = Limiting laterally unbraced length for full plastic bending capacity, uniform moment case

r_{yc} = Radius of gyration of the compression flange about the beam axis of symmetry

S_{xc} = Section modulus referred to the compression flange

S_{xt} = Section modulus referred to the tension flange

5.10 Biaxial Bending

Crane runway beams subject to biaxial bending are proportioned in accordance with Clause 13.8.3 of S16-01, which when the axial compression is zero, gives

$$\left[M_{fx}/M_{rx}\right] + \left[M_{fy}/M_{ry}\right] \le 1.0$$

The capacity of the member is examined for

(a) overall member strength, and

(b) lateral torsional buckling strength.

It is noted that this formulation requires lateral torsional buckling about the strong axis to be considered as appropriate and allows inelastic action to be considered provided that the width-thickness ratios of the elements are sufficiently stocky.

See Appendix A, Design Examples 1 and 2.

5.11 Heavy Construction

A commonly encountered detail involving what is commonly referred to as an apron plate is shown in Figure 7, along with recommendations based on S16-01. The designer should refer also to Clause 11.2 and Table 2 of S16-01 for criteria for maximum width-to-thickness ratios.

The design of such members for horizontal strength is usually done by rational analysis if the section that resists lateral forces is of reasonable depth (say about span/15 minimum) and can function as a web-horizontal beam. See Design Example 2 in Appendix A.

Web crippling and yielding under concentrated wheel loads is covered in S16-01, Clause 14.3.2. In accordance with AISE 2003 and CMAA 2004, the concentrated wheel load is distributed at 1:1 from the top of the rail to the contact surface at the top of the beam.

Referring to Figure 5, crane load eccentricities can cause local out-of-plane bending in the web. An exact analysis is complex. DIN, Australian Standards, and work by Cornell University address this topic. Rosewell 1987 notes that AISE does not take into account the wheel load acting off the centre of the web, or the tilt of the beam section, accounting for these and other eccentricities by use of the flexure analogy. Experience in the industry is that this procedure generally provides satisfactory results for commonly used rolled beam sections.

Problems, usually cracked welds, have occurred in plate girder sections, particularly where webs are fillet welded to top flanges and where fatigue becomes a factor. Good results have been achieved using complete joint penetration welds with reinforcing for the web to flange connection.

It is recommended that local torsional effects be examined for welded sections. See Appendix A, Design Example 2 for typical calculations.

For crane runway beams, including welded sections, it is not common practice to check interaction of out of plane effects and principal stresses in the local web to flange region, perhaps because of the complex distribution of forces and because of experience in the industry. More research on this topic is needed.

Additional recommendations for large, heavy duty crane runway girders with apron plates as one would encounter in steel making facilities are given by Fisher (1993), AISE (2003) and Rowswell (1987).

Some references show a calculation of local wheel support stresses based on older editions of AISE Technical Report No.13. This is no longer recommended and is not included in AISE (2003).

A bearing detail that has been used successfully is shown in Figure 20. This detail can reduce eccentricities, facilitates achieving tolerances in squareness and elevation, and reduces restraints at beam bearings.

As noted in Chapter 4, special measures are usually implemented to control shop and erection tolerances.

5.12 Intermediate Web Stiffeners

For longer spans and heavier installations, a decision often must be made whether to use intermediate web stiffeners (see Figure 19) or to use a thicker web and avoid the use of these stiffeners.

Structures of each type have been providing satisfactory service. If weight is not the governing factor, many experienced designers would agree that a thicker web without intermediate stiffeners is the better solution because of simplicity, more rugged web-to-flange connection, and elimination of details subject to fatigue in the tension zone of the web.

Use of horizontal web stiffeners as for highway bridges is not common and is not recommended for new construction for the same reasons as noted above. These stiffeners may be part of a solution for upgrading, however. Caution must be exercised in zones of tension, particularly at splices in the stiffeners. If the stiffeners are not butt welded full strength and ground in the direction of stress, a fatigue crack might propagate into the web.

5.13 Links to Crane Runway Beams

To accommodate differential longitudinal and vertical movements between the crane runway beam and supporting structure, but at the same time to provide lateral restraint to the beam, articulated links are often provided for Crane Service Classifications C and up (see Items 19 and 20 in Section 4.2). For more information, see Griggs (1976), Rowswell and Packer (1989), Rowswell (1987), and Figures 16 and 17.

These links often are a proprietary design with hardened spherical bearings. Manufacturer's literature is usually referenced during preparation of the design and specifications.

With due regard to considerations such as patents and class of service, these links are sometimes designed by the structure designer.

5.14 Bottom Flange Bracing

AISE (2003) recommends that lateral bracing (see Figure 19) be provided to bottom flanges of crane runway beams that span more than about 11 m. Canadian Standards do not require such measures.

There are many successful installations with spans up to 20 m that do not include bottom flange lateral restraints. It is suggested that, for the usual crane runway beam proportions and for structures built and maintained within the specified tolerances, the need for bottom flange bracing should be at the discretion of the owner and the structure designer.

5.15 Attachments

The design drawings should state that no attachments should be made to the crane runway beams without authorization of the designer.

Attachments for the collector rails to power the cranes should be located above the neutral axis of the beams and should be bolted if attached directly to the web. See Figure 19.

5.16 End Stops

End stops on crane runways may or may not have an energy dissipating device to reduce the impact on the end stop. Devices such as rubber, springs, or hydraulic bumpers may be mounted on the end stops or on the cranes. For light duty applications, rubber bumpers are often used. For Crane Service Classifications C and up, hydraulic bumpers are usually specified. For more information see Fisher (1993), AISE (2003), CMAA (2004), Rowswell (1987) and Tremblay and Legault (1996).

Design of end stops should include an assessment of the maximum factored load that the end stop and the rest of the supporting structure can reasonably resist, and this force should be made known to the crane and/or bumper designer.

Bumper specifications are usually prepared with the aid of manufacturer's literature.

5.17 Unequal Depth Beams

Where unequal depth beams meet at a support, several different details have been used. For heavier duty cycle applications, Crane Service Classification C and up, suggested details are shown in Figures 21 and 22.

"Stools" on the bottom of the shallow beam are generally not recommended because of the magnified longitudinal movement due to rotation of the end of the beam and bottom flange elongation. Details that involve one beam bearing on the end of the other (see Figure 21) present implications of sequence of erection and difficulties in replacement of the deeper beam.

5.18 Underslung Cranes and Monorails

These installations are somewhat different from other overhead cranes because the loads from the crane runway beams usually are not transferred directly to the columns. See Section 2.3.3 for additional information. Locations and loads are often subject to changes to suit plant operations. Installations are often proposed for structures not designed for this duty. Runway beams are often supplied by the crane supplier. Quite often, the runway beams will be suspended from roof beams or open web joists.

Important considerations are as follows:

- Compression flanges are generally unsupported laterally.
- Runway beams are usually continuous. The design should account for such things as vertical flexibility of supports and for differential settlement.
- Runway beams are subjected to secondary stresses at trolley wheels.
- The building designer must coordinate support locations with the crane beam supplier and the open web joist supplier. The degree of flexibility of support locations should be considered.
- At hanger locations, methods of vertical and lateral adjustment should be incorporated in the hanger design so that the crane beams can be aligned and so that loads will be distributed evenly and in accordance with the design assumptions.
- Requirements for design for fatigue should be made known to the open web joist designer by showing requirements on the structure design drawings.
- Anti sway braces should be provided at hanger locations, otherwise premature failure due to fatigue may occur (see Figure 23).
- Longitudinal sway braces should be provided at regular intervals (say 10 m).
- Splices in runway beams require special attention to allow a smooth running crane. Typical splices are shown by Fisher (1993) and Goldman (1990).
- Specialized and hybrid beams such as WT or WWT top with ST or special bottom flange are used and may fabricated from a mix of different steels. Information for design of these beams is provided by CMAA (2004), Galambos (1998), Goldman (1990), and Weaver (1985). Certain configurations of hybrid beams are manufactured as proprietary items. Manufacturers' literature, including design aids, is available.
- Fisher (1993) recommends that the deflection due to wheel loads should be limited to span/450.

Curved monorail beams should be analysed as horizontally unsupported curved beams. The Australian Standard for crane runways and monorails AS1418.18-2001 includes a provision that if the horizontal radius is larger than twice the distance between supports and provided that there is continuity by at least one span on each side of the section being considered, the effect of curvature can be neglected.

For more information on these structures, see Fisher (1993), Goldman (1990), and Weaver (1985).

5.19 Jib Cranes

Jib cranes usually have a rotating boom attached to a mast which is held in a vertical position by floor and ceiling mounting or by column or wall mounting. A floor mounted variation is sometimes called a pillar crane. The hoist is usually mounted on the boom as a monorail.

Jib cranes are often an add-on to facilitate material handling. Unaccounted-for forces can cause several problems including column distortion, column failures, crane runway misalignments, and excessive column base shear. Fisher and Thomas (2002) provide recommendations. Excessive deflection of the boom can lead to a downhill loss of control of the hoist. Fisher (1993) recommends that deflection should be limited to boom length divided by 225.

5.20 Truss Type Crane Runway Supports

Long spans may require the use of primary trusses instead of crane runway beams. The design of these trusses is similar in many ways to railroad design but in this case the rail is usually supported directly on the top chord. The structural analysis should be done with the aid of computers and must account for secondary stresses due to the usual fixity of the joints. See Clause 15.1.2 of S16-01, detailed method of truss design. The joints and members, particularly at the top chord, should account for torsional forces as from side thrust and eccentricity of rail placement. See Figure 5.

For these structures, careful attention to design for fatigue is necessary.

For more information, see Fisher (1993).

5.21 Column Bases and Anchor Rods

Design of crane carrying columns sometimes requires the use of shear keys. Anchor rods may be subjected to repeated upward loads. Fisher (1993) and Cannon and Godfrey (1981) provide useful information.

Strict tolerances on anchor rod placement are often specified so that the crane runway beams can be erected within the required tolerances.

5.22 Dissimilar Materials

Special consideration should be given to the interaction of crane carrying steel structures subject to movement and vibration with other materials that are often more rigid and brittle such as masonry.

Columns are sometimes tied to the wall system. Some flexibility should be provided at the connection (see Figure 2). Fisher (1993) provides recommendations.

For Crane Service Classifications C and up, the steel structure should be isolated from masonry if distress in the masonry is to be avoided.

5.23 Rails

Rails are usually selected by the crane manufacturer. CMAA (2004), Goldman (1990)and Weaver (1985) provide additional information.

There is no published criteria for crane rail replacement due to wear and tear. The decision to replace due to wear and tear is not usually based on structural considerations unless adverse effects on the structure are noted.

Refer also to Chapter 4, Item 29.

5.24 Rail Attachments

For lighter duty applications, hook bolts or non-patented rail clips are sometimes specified. The designer should be aware that hook bolts (sometimes called J bolts) do not provide lateral restraint to the rail. Unaccounted for rail eccentricities and rail misalignment may occur.

Rail clips for Crane Service Classifications C and up are usually two-plate rail clamps or patented, manufactured clips. For the patented clip, the designer usually refers to manufacturer's literature when specifying the type of clip, spacing, and attachment of the clips to the support.

The clips shown in the Figures 8 to 22 are a manufactured type. Other types may be suitable.

For more information, see Fisher (1993), Ricker (1982), Rowswell (1987) and Weaver (1985).

5.25 Outdoor Crane Runways

Outdoor runways require special attention to the following:

- Because there is usually no tie across the top, the distance between the rails (gauge) is vulnerable to change due to foundation conditions.
- In extremely cold climates, consideration should be given to use of brittle fracture resistant steel (WT or AT) for crane service Classes D and up.
- Distance between expansion joints should be carefully evaluated, considering ranges in temperature.
- Other environmental effects such as from wind, snow and ice should be considered.

Fisher (1992), Rowswell (1987) and Tremblay and Legault (1996) provide more information.

5.26 Seismic Design

AISE (2003) and Weaver (1985) provide information on measures sometimes used where there is danger of displacement of wheels from rails.

Current seismic provisions contain recommendations for anchoring of architectural, electrical and mechanical components of structures, but do not deal in depth with travelling cranes.

AISE (2003) and MBMA (2002) suggest that the designer consider the dead load of cranes parked for maximum effect.

In case of seismic activity, the mass of the crane will interact with the mass of the supporting structure, acting as a tie between rails, whether the crane and supporting structure were so designed or not.

Suggestions are:

- The designer should check to ensure that the effect of the lateral load due to "E" does not govern over side thrust on the crane runway beams.
- For zones of higher seismic activity where it can be shown that there is a significant risk of displacement, hold down devices should be considered.
- In zones where seismic design provisions may be more severe than for wind, the designer should consider use of a dynamic structural analysis, considering the crane(s) as a tie between rails. The resulting forces on the crane should be made known to the crane manufacturer.

The special seismic design provisions of the NBCC 2005 and Clause 27 of S16-01 "Seismic Design Requirements" (Clause 27) are most appropriate for building structures characterized by residential and commercial occupancies. Several reference papers appear in the Canadian Journal of Civil Engineering.

Other steel structures supporting cranes such as certain manufacturing facilities and steel mills, may be generalized as one storey building structures.

The response of these structures to seismic activity may be such that several of the recommendations in the NBCC 2005 and Clause 27 for the class of structures envisaged would not applicable in whole or in part. Some of the recommendations would be impractical for customary structural arrangements. The inherent ruggedness and redundancy of these structures may provide resistance to strong motions otherwise unaccounted for in the above referenced provisions of Clause 27.

Recognizing the above, a footnote (3) to Table 4.1.8.9, Conventional Construction, in the NBCC 2005 removes the height restrictions for single-storey buildings such as steel mills, powerhouses, and aircraft hangers.

5.27 Standards for Welding for Structures Subjected to Fatigue

The construction specification should define which portions of the structure will be subject to the more stringent requirements for cyclically loaded structures. See Figure 25 for typical requirements. Usually the critical elements would be the crane runway beams and their attachments to the supports, but it is the responsibility of the structure designer to determine if any other components (open web joists supporting monorails, for example) need be included in this category.

Flange plates used in the design of heavy duty runways should be inspected for the absence of lamellar inclusions in accordance with the provisions of W59. Material not meeting the standards should be rejected.

See also Chapter 4, Item 26.

5.28 Erection Tolerances

Where possible, bearings and lateral restraints should permit lateral adjustment of the crane runway beams to maintain alignment with the crane rail. This is often accomplished by use of slotted or oversize holes, and shims. See Figures 13 to 17. Alignment procedures should be reviewed by the designer of the structure. For instance, an incorrect alignment sequence could result in uneven bearing and eccentricities such as E6 on Figure 5.

Anchor bolt locations should be carefully checked before erection of structural steel. Base plates must be accurately located so that required tolerances in crane runway beams can be achieved.

Erection tolerances of crane runway rails should be compatible with minimization of eccentricities on the supporting structure and within tolerances set by the crane manufacturers. Allowable sweep of crane runway beams should be consistent with design assumptions for rail eccentricity, rail clip adjustment tolerances and rail alignment tolerances.

Figure 24 shows the requirements of the CMAA.

In case of conflict with Clause 2.9.7 of S16-01 and recommendations contained elsewhere in this design guide, the more stringent requirements should govern.

Checking of erection tolerances should be by independent survey. Where the specified tolerances are exceeded, the designer should be notified. After assessment, the designer should specify remedial measures as may be required.

5.29 Standards for Inspection

Refer also to Sections 5.27 and 5.28.

Figure 25 shows commonly used standards for welding and inspection of crane runway beams.

See W59 for more information.

Referring to CSA Standard W59, Welding inspection organizations and individual inspectors must be certified to CSA Standards W178 and W178.2 respectively. For inspection of other aspects of fabrication and erection, no standard for certification exists. Inspectors should be completely familiar with the requirements of the design drawings and project specifications including all specified standards and codes, including requirements for dynamically loaded structures as may be applicable.

CSA Standard B167-96 specifies the minimum requirements for inspection, testing, and maintenance of cranes and includes supporting structures. Section 4.4.5.2 specifies that a Professional Engineer must certify the supporting structure. The user is advised to consult with the jurisdiction having authority regarding adoption of this Standard, and whether there may be exemptions or additions.

5.30 Maintenance and Repair

Crane carrying structures subjected to fatigue, in combination with:

- age,
- unintended use (often called abuse),
- inadequate design,
- imperfections in materials,
- substandard fabrication,
- substandard erection methods, and
- building component movements, such as foundations,

require maintenance and repair. Repair procedures should incorporate the recommendations of an experienced structure designer, or the repair can create effects that are more serious that the original imperfection.

Fisher (1993), Millman (1991,1996) and Reemsnyder and Demo (1978) provide additional information.

CHAPTER 6 - REHABILITATION AND UPGRADING OF EXISTING CRANE CARRYING STEEL STRUCTURES

6.1 General

Designers may be asked to assess and report on the condition of a crane carrying steel structure for different reasons such as:

- concern about the condition of the structure,
- due diligence brought on by a change in ownership,
- to extend the useful life under the same operating conditions,
- to increase production by adding cranes or other equipment, and
- to modify processes and add new and possibly heavier cranes or other equipment.

The structure may be several decades old, materials of construction are not clear, drawings and calculations are non existent, and past crane duty cycles unknown.

The local building code authority may be unprepared to accept measures which might be interpreted as contrary to the provisions of the local building code.

Little guidance is available that is directly related to crane carrying structures in Canada.

AISE (2003) and Millman (1991) provide guidance and are the basis of several of the recommendations contained herein. AISE (2003) provides an appendix that addresses recommended practices for inspecting and upgrading of existing mill building structures.

6.2 Inspections, Condition Surveys, Reporting

An inspection plan should be prepared that is based on the following as a minimum:

- site visits,
- review of existing drawings, specifications, calculations, site reports, photographs,
- available records of modifications to the structure and equipment,
- interviews with plant personnel, to gain insight into the operation, past and present, and
- review of the applicable codes and standards.

The field inspection may involve use of a professional inspection and testing agency and may include the following:

- visual inspection noting defects such as corrosion, cracks, missing components, reduction of area, detrimental effects of welding, and physical damage,
- visual inspection of crane rails and their connections,
- visual inspection of connections,
- recording of field alterations not noted on available drawings,
- comments on misalignments and settlement, including need for an alignment survey, and
- special investigations such as identifying older steel, weldability, nondestructive testing, measurements of actual crane wheel loads, strain gauging, impact measurements, deflection under live load measurements, and thermal loads.

A common problem when evaluating older structures is to identify older steel. S16-01 covers this in Clause 5.2.

The report of the field inspection should be tailored to the ultimate purpose of the inspection. Suggested contents, as a minimum, are as follows:

- background, including purpose of the inspection,
- scope,
- available records, records of discussions,

- general description of the structure,
- field conditions,
- history of the use of the structure, including crane duty cycles,
- history of performance and maintenance of the structure,
- description of defects,
- description of modifications,
- photographs, results of testing,
- special investigations, and
- need for further work.

6.3 Loads, Load Combinations

The loads and load combinations given in Chapter 2 of this guide have proven satisfactory for the design of new facilities. It is recognized (AISE 2003) that some of the loads are conservative, particularly those generated by crane or trolley motion. A study of overload conditions may reveal a very low probability of occurrence and/or short duration such that, with the owner's approval, these overloads can be eliminated from further consideration or used with reduced load combination factors. For instance, the probability of simultaneous occurrence of maximum vertical loads from more than two cranes along with impact will likely be low enough that a reduced load combination factor can be used. For more information, see Millman (1991).

A history of satisfactory performance over many years combined with a knowledge of operating conditions may provide the necessary degree of confidence so that loads, load combination and fatigue design criteria can be realistically assigned for the particular operations.

Millman (1991) recommends exclusion of "Any combination of instantaneous dynamic crane loads which originate from different functional processes." The following examples are provided:

- hoist operation and trolley travel,
- crane and trolley travel,
- hoist operation and crane travel, and
- trolley bumper collision and hoist operation.

Impact factors can be reassessed based on studies and field measurements. See Millman (1991) for more information.

Side thrust loads can be studied analytically and can be assessed in the field using strain gauges under the most severe operating conditions. Many experienced designers would agree that for side thrust, providing that loads on the trolley end stop do not govern and that the runway is not badly out of alignment, side thrust should not be expected to exceed the lateral loads generated by friction due to locked trolley wheels. Thus, side thrust values may not be expected to exceed those for normal radio controlled cranes as shown in Table 2.1, unless unusual conditions exist at the trolley end stops.

Regarding fatigue, the simultaneous occurrence of maximum vertical wheel loads with side thrust can sometimes be eliminated from consideration.

The weight of cranes can be considered to be dead load (see Section 2.3.2). If weighing in place is required, this can be done using load cells.

Duty cycle analyses can be done to study the effects of fatigue.

Environmental loads are based on probability of occurrence during the life of the structure. If the expected remaining life of the structure is somewhat less than for a new structure, the probabilities of exceedance could be examined and then the parameters might be adjusted accordingly. This examination and resulting recommendations should be undertaken by qualified people.

6.4 Structural Modelling

Modern methods of analysis using three-dimensional computerized models will provide the most accurate information on how loads will be distributed throughout the structure, including the foundations, and may result in substantial cost savings. Cranes often act as links between two sides of a runway (see Section 5.1). This action should not be assumed for new designs unless the cranes are designed to act this way, but if it can be shown that this is happening without ill effect, it may be included in the assessment.

Where lateral torsional buckling is a critical consideration, Ellifritt and Lue (1998) question whether lateral torsional buckling can occur, given that the crane acts as a link. The linked beam may have a reserve of lateral strength to prevent lateral torsional buckling, being more lightly loaded than the beam under investigation. Refer also to Section 5.2.

6.5 Reinforcing, Replacement

When these conditions are encountered, an inspection plan should be drawn up in accordance with guidelines presented above.

Methods of repairs and replacements are varied and are a challenge to the ingenuity of the designer. Considerations may include, but are not limited to:

- degree and nature of physical damage,
- degree of deterioration, from corrosion, for instance,
- materials of construction,
- weldability,
- existing details,
- fatigue life remaining,
- ease of construction and replacement,
- expected future service conditions, and
- past performance under similar conditions.

Acceptance criteria for older buildings where tolerances are outside those recommended for new construction should be established by an experienced designer after careful study, on an individual basis.

6.5.1 Reinforcing an Existing Runway Beam

Solutions that have been applied to this common problem are:

- add vertical stiffeners,
- add horizontal stiffeners,
- add lateral support,
- weld a tee on the bottom,
- weld angles to the top flange,
- reconfigure the runway beam as a truss, and
- install new columns.

6.5.2 Reinforcing an Existing Column

Solutions include adding metal by welding and also adding a new column under the existing brackets.

Structures have been stiffened by adding horizontal bracing to improve load sharing between frames.

6.5.3 Welding to Existing Structures

Remember that loads may not be shared uniformly between existing and new material, particularly if there is significant load in the existing member.

Confirm weldability, particularly for older structures.

Develop welding techniques that will not compromise the strength of the existing load carrying member due to excessive heat input.

Practical hints can be found in the July 2002 issue of “Modern Steel Construction”, published by the AISC.

CHAPTER 7 - SUGGESTED PROCEDURE FOR DESIGN OF CRANE RUNWAY BEAMS

7.1 General

Two examples are provided in Appendix A to illustrate design of top running crane runway beams. Fisher (1993), Fisher and Van de Pas (2002), Kulak and Grondin (2002), and Salmon and Johnson (1996) also provide examples, including monorails, to limit states design principles.

An outline of the general procedure for design of top running crane runway beams is presented in Table 7.1. The order is somewhat flexible. Procedures are similar for other types of runways.

7.2 Design Criteria

Establish, with the owner's approval, the design criteria. A checklist of items to consider should be prepared and should include some or all of the data in Table 7.1. Refer also to Section 4.1.

Table 7.1
Design Criteria for Crane-Supporting Steel Structure

Design Criteria	Value/Units
Codes and Standards	
Importance (see NBCC 2005)	
Life of the Structure	years
Materials (Plates, Shapes, Fasteners, etc.)	
Span	mm
Provision for Future Expansion?	
Simple Span?	
Lateral Support for Top Flange?	
Top of Rail Elevation, or Height from Main Floor	m
Required Clearance to U/S Beam	mm
Side Thrust Equally Distributed Both Sides of Runway?	
Number of Cranes, Each Runway	
Collector Rail Mounting Details	

Table 7.1 continued

Design Criteria		Value/Units
Design for Future Additional Cranes		
Jib Cranes, or Provision for Jib Cranes		
Design for Future Upgrades		
Class of Cranes		CMAA Class
Service (Description)		
Type of Duty (see Table 2.1 and Section 3.4.2)		
Crane Hook Capacity	# hook(s) each	
	Capacity each hook	kg
Weight of Crane Bridge		kg*
Weight of Crane Trolley		kg*
Bridge Wheels per Rail	Total Number Driven	
Bridge Wheel Spacing		mm
Minimum Distance Between Wheels of Cranes in Tandem		mm
Maximum Wheel Load, Each Crane (not including impact)		kN
Minimum Wheel Load, Each Crane (not including impact)		kN
Crane Rail	Description	
	Self load	kN/m
Rail Joints (bolted or welded)		
Resilient Pad Under Rail?		
Bridge Speed		m/sec
Type of Bumpers		

Table 7.1 continued

Design Criteria	Value/Units
Bumpers Supplied with Crane?	
Bumper Force on Runway end Stop (Ultimate Load)	kN
Fatigue Criteria: Vertical - Equivalent passes on one crane, maximum wheel loads Horizontal - Equivalent cycles of side thrust at 50% of maximum side thrust	 # of passes # of cycles
Deflection Criteria: Vertical Limit (one crane, not including impact) Horizontal Limit	 Span/ Span/
Impact Criteria: Percentage of maximum wheel loads, one crane only	 %
Foundation Conditions, Limitations	
Other Consideration	
*Weight Certified?	

7.3 Design Procedure

- Calculate Side Thrust

  Using the side thrust criteria from Table 7.1 and Table 2.1, calculate the side thrust force C_s from each crane to each side of the runway and distribute to the wheels, usually equally. Calculate the side thrust to each wheel as a percentage of the maximum vertical load to each wheel.

- Select a Preliminary Section

  Using the wheel loads, deflection criteria and approximate methods, choose a section that, after further analysis, could provide the required moments of inertia about each axis.

- Moving Load Analysis

  From manual calculations (for assistance, see Beam Diagrams and Formulae in the CISC Handbook), or using a computer, compute the governing deflections, bending moments, shears and reactions for the wheel loads for a single crane and for multiple cranes as may be required. Effects of impact should not be included at this time.

Review the section properties required for deflection and adjust and recalculate if necessary.

- Refine the Trial Section

  Determine class and member properties.

- Calculate Other Forces in the Vertical Plane

  Calculate loads due to Dead Loads, Axial Loads, Tractive Loads, Temperature, Bracing, etc.

- Calculate effects of Torsional Loads
- Re Evaluate Deflections
- Calculate Factored Loads
- Calculate Factored Resistance and Compare to Factored Loads
- Check Local Wheel Support
- Iterate as Necessary
- Design Stiffeners
- Design Bearings and Lateral Restraints
- Design Element Welds and/or Bolts for Factored Loads
- Check for Fatigue Resistance

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FIGURES

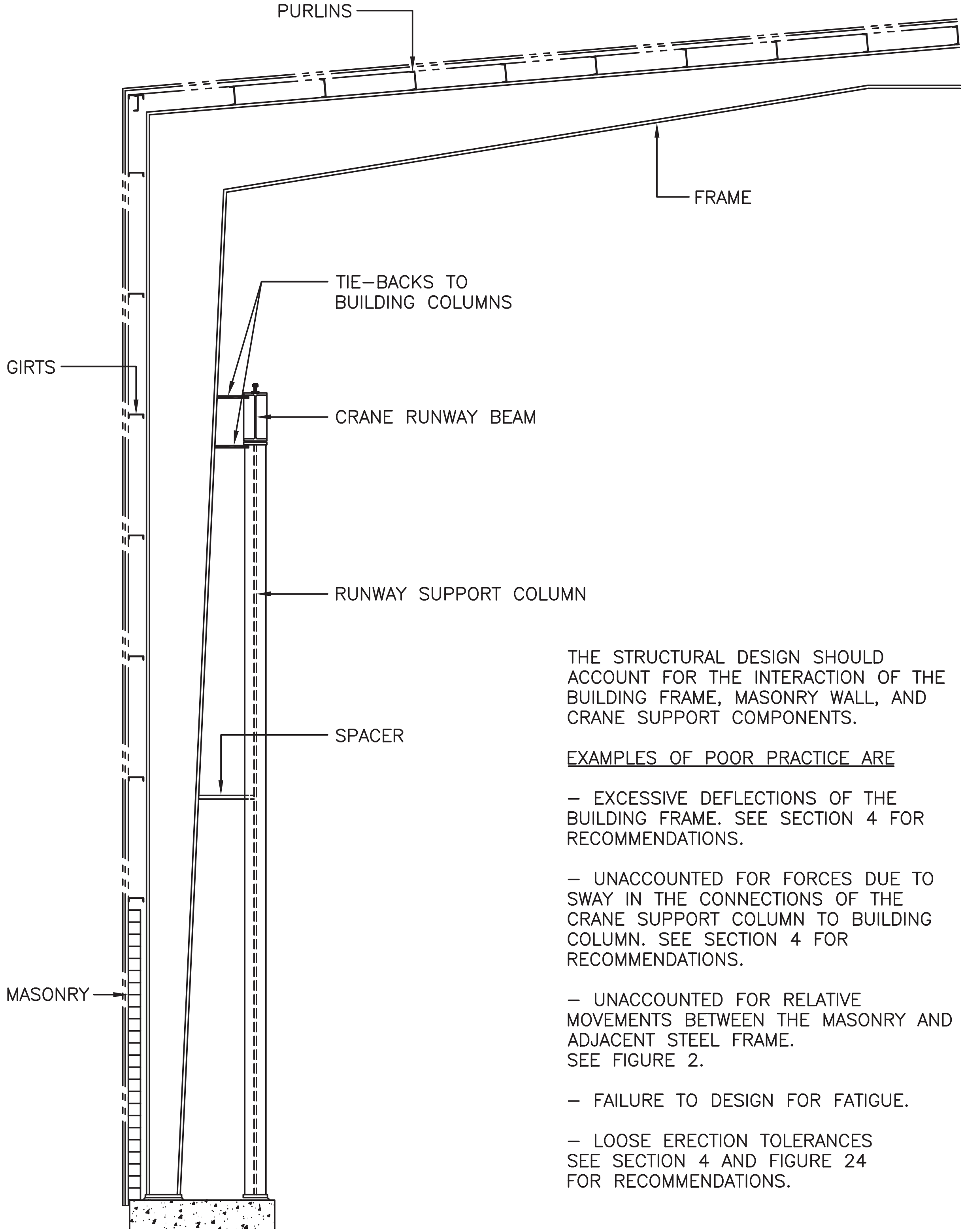


Figure 1
A Common Example of a Crane Supporting Structure

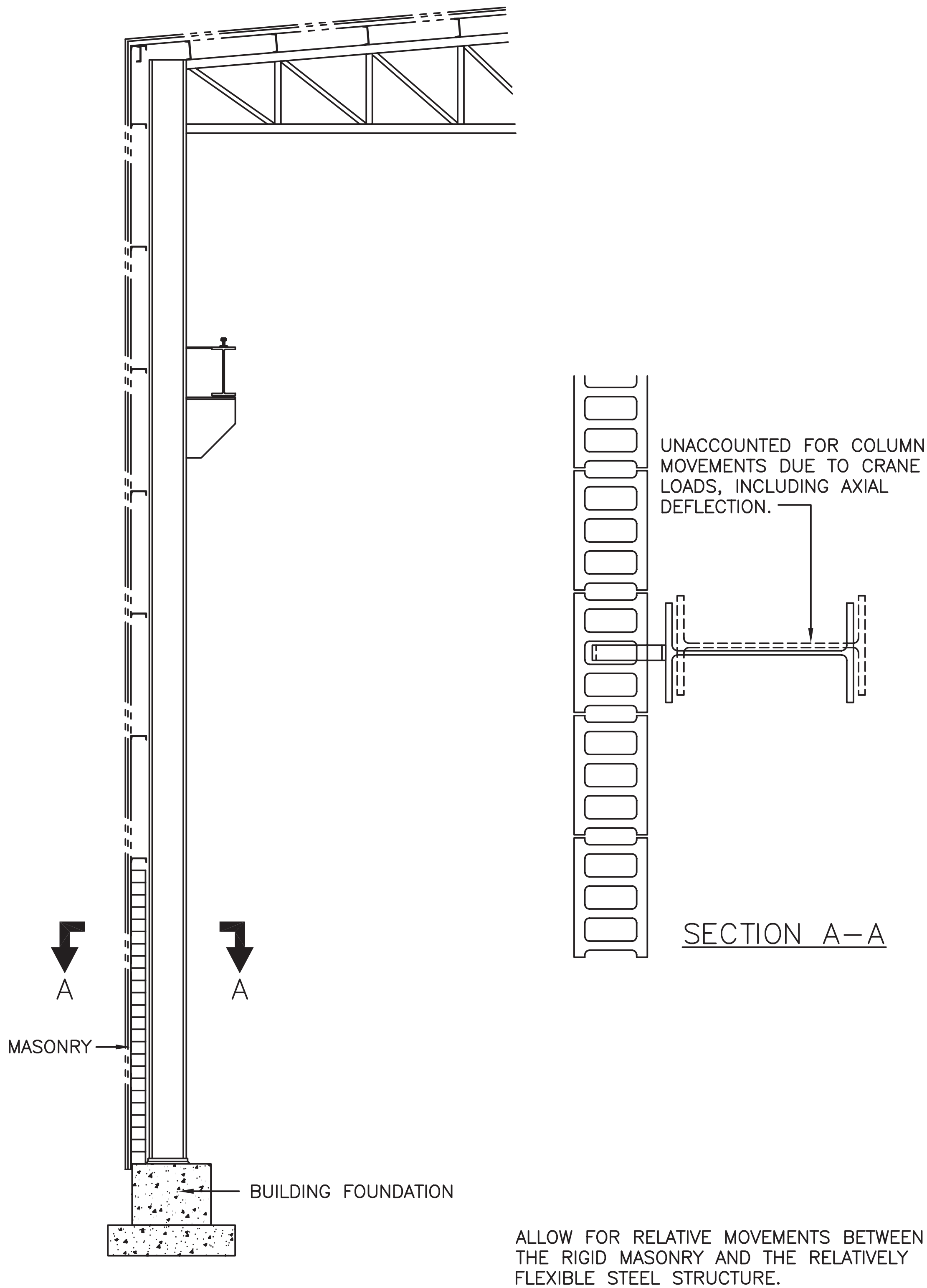


Figure 2
Illustration of Interaction of Dissimilar Materials

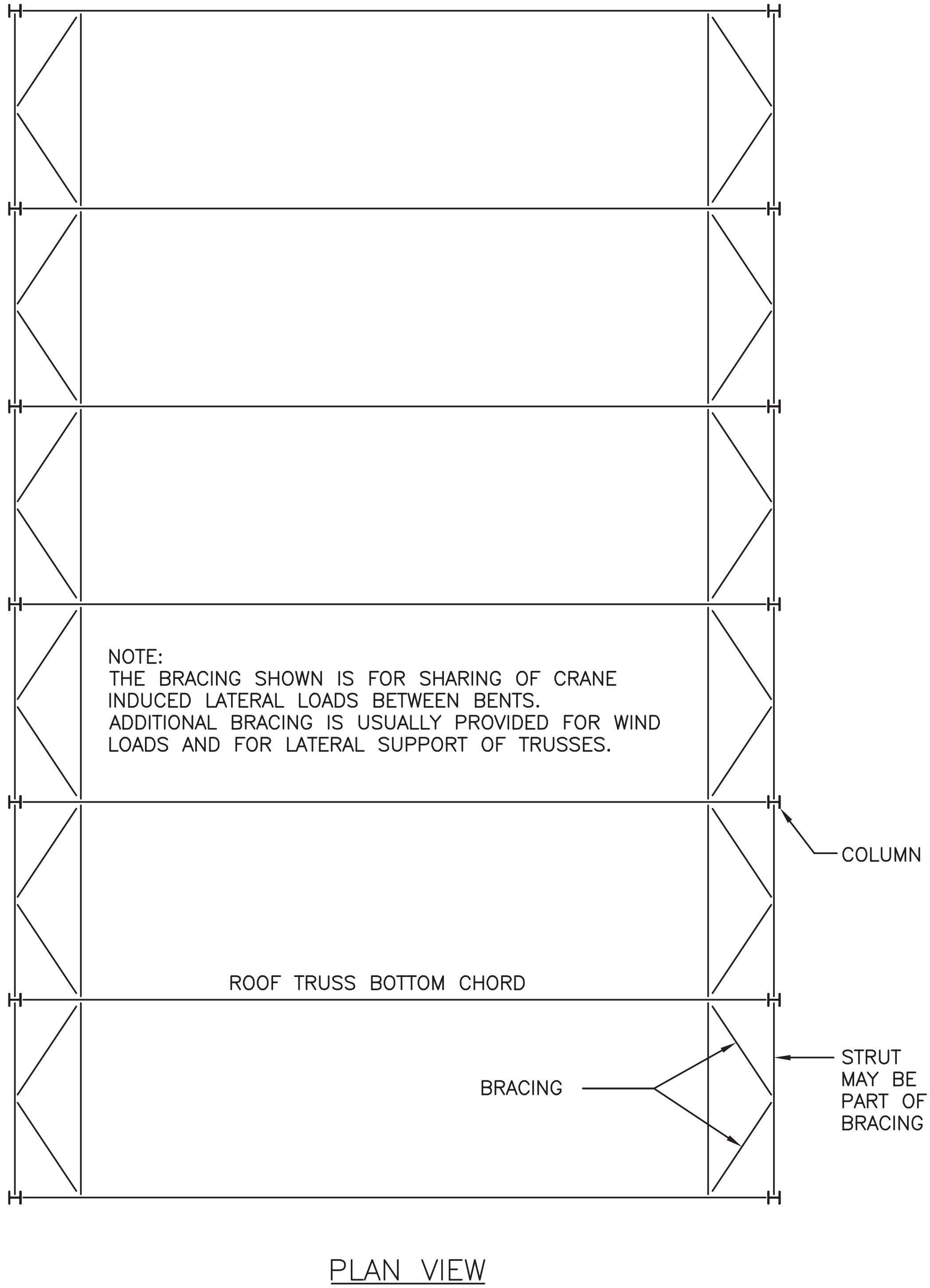
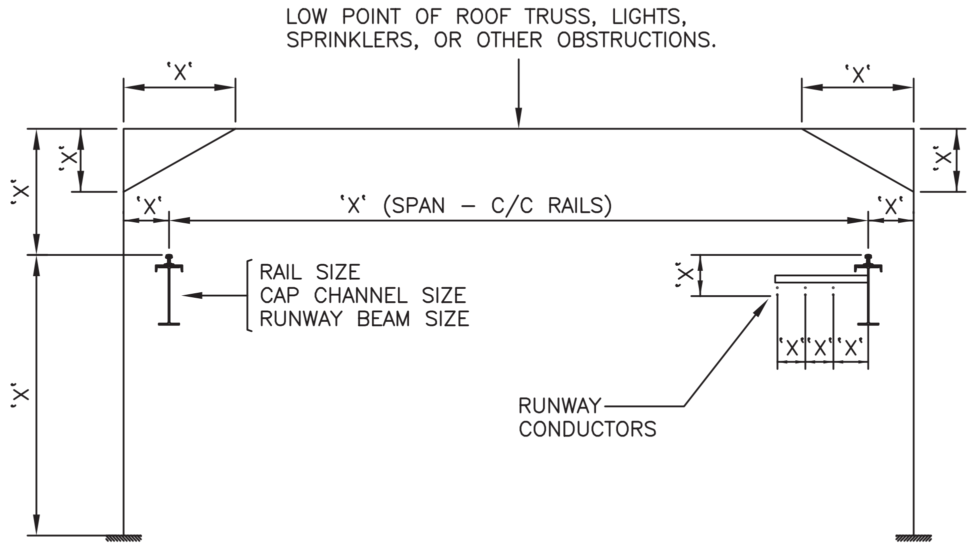
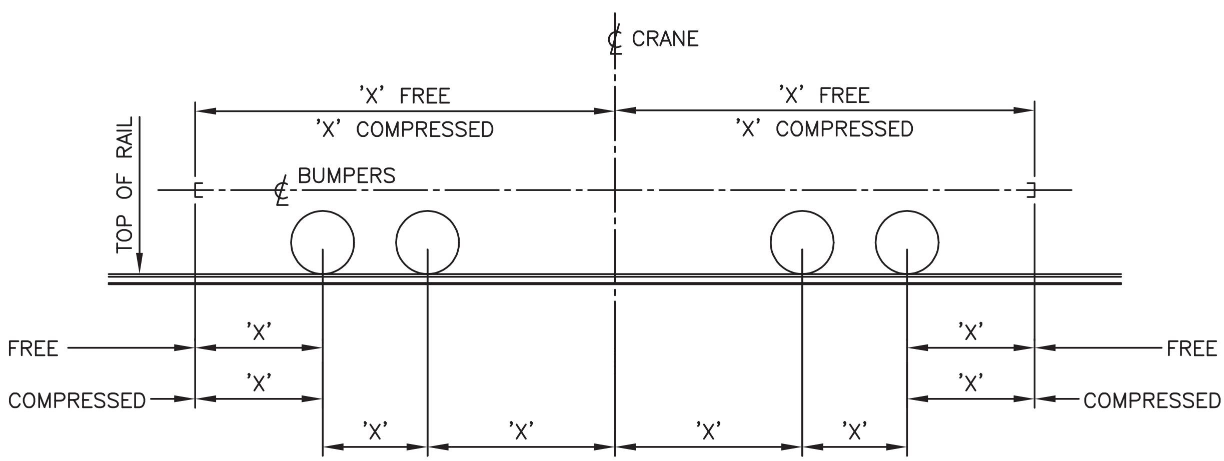


Figure 3
Typical Horizontal Roof Bracing at Lower Chords of Roof Trusses



CRANE RUNWAY CROSS–SECTION

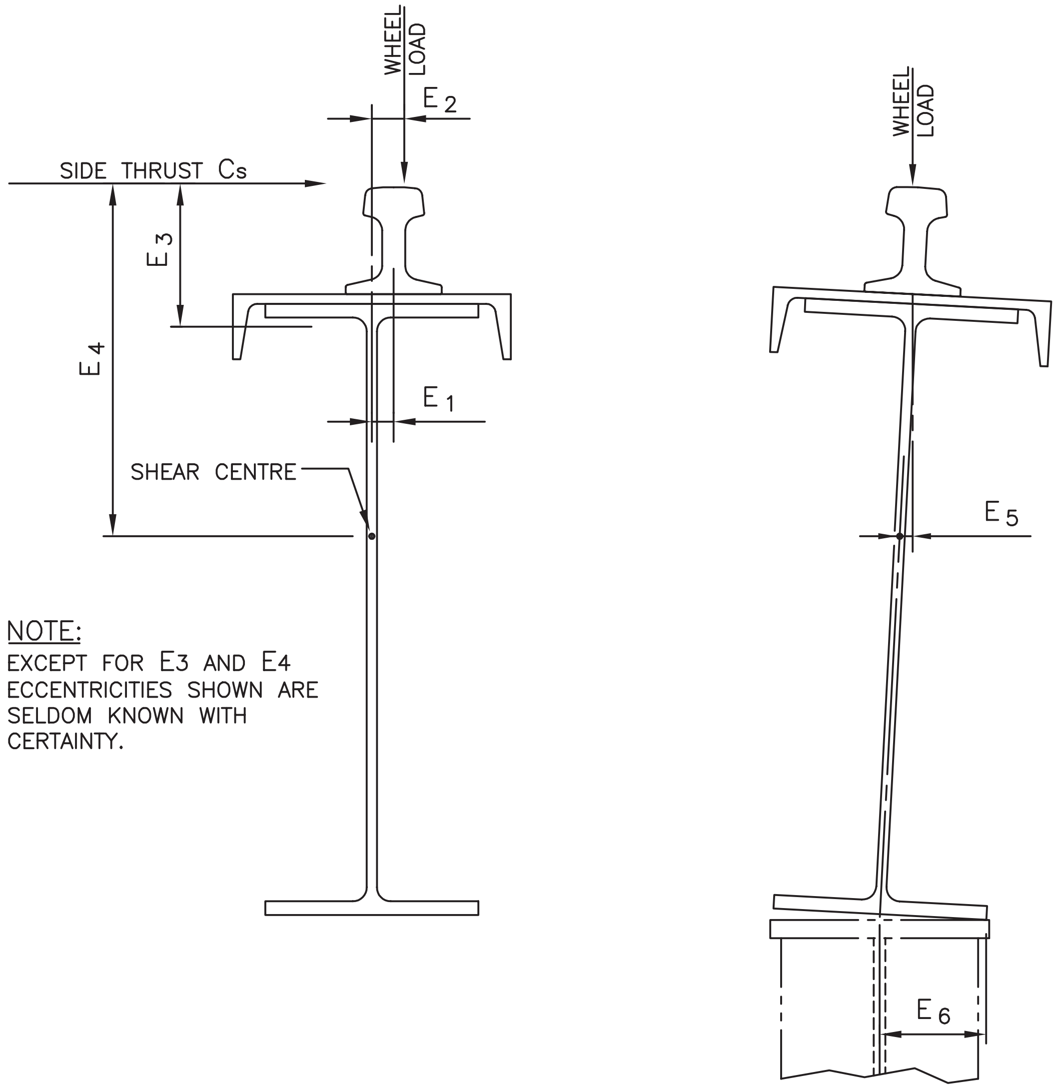


Max. bridge wheel load = 'X'
Min. bridge wheel load* = 'X'
Trolley weight = 'X'
Total crane weight = 'X'

*Bridge wheel load which occurs simultaneously with maximum wheel load on opposite on opposite side of bridge.

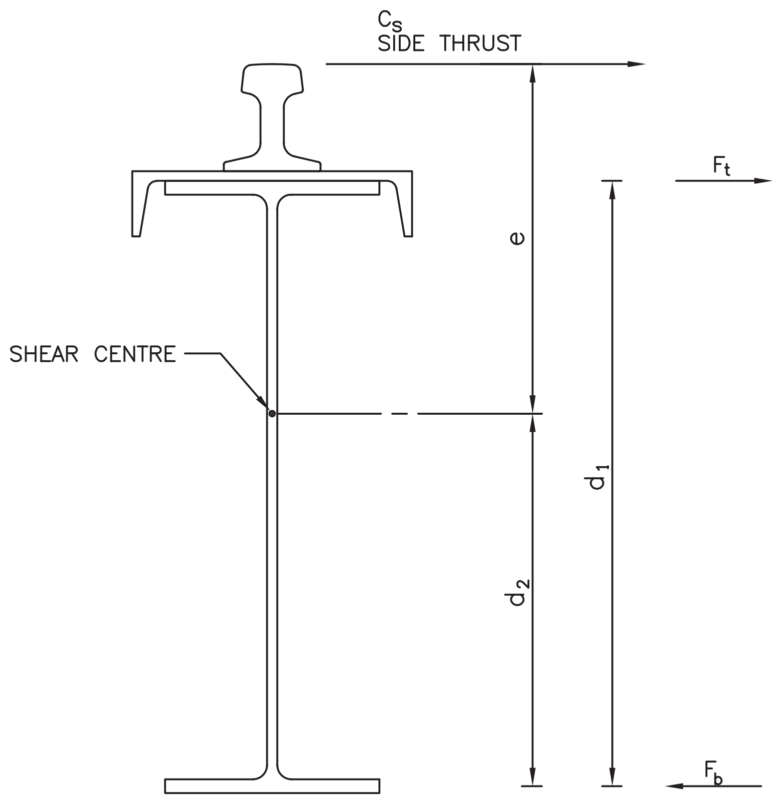
CRANE BRIDGE WHEEL LOAD DIAGRAM

Figure 4
Typical Clearance and Wheel Load Diagram



E_1: rail not being centered over web beneath
E_2: wheel load not centered on rail
E_3: side thrust, at the flange–to–web intersection
E_4: side thrust not applied at the shear centre
E_5: line of action of wheel load not through shear centre, beam not plumb
E_6: uneven bearing at beam support

Figure 5
Typical Crane Load Eccentricities



$$\sum M_b = 0 = C_s(e + d_2) - F_t(d_1)$$

$$\therefore F_t = C_s\left[\frac{e}{d_1} + \frac{d_2}{d_1}\right]$$

$$\sum F_x = 0 \therefore F_b + C_s = F_t$$

$$F_b = C_s\left[\frac{e}{d_1} + \frac{d_2}{d_1} - 1\right]$$

For many cases, $e \approx \frac{d_1}{2}$, $d_2 \approx \frac{d_1}{2}$, and satisfactory results are obtained by applying all the sidethrust to the top flange.

Figure 6
Flexure Analogy

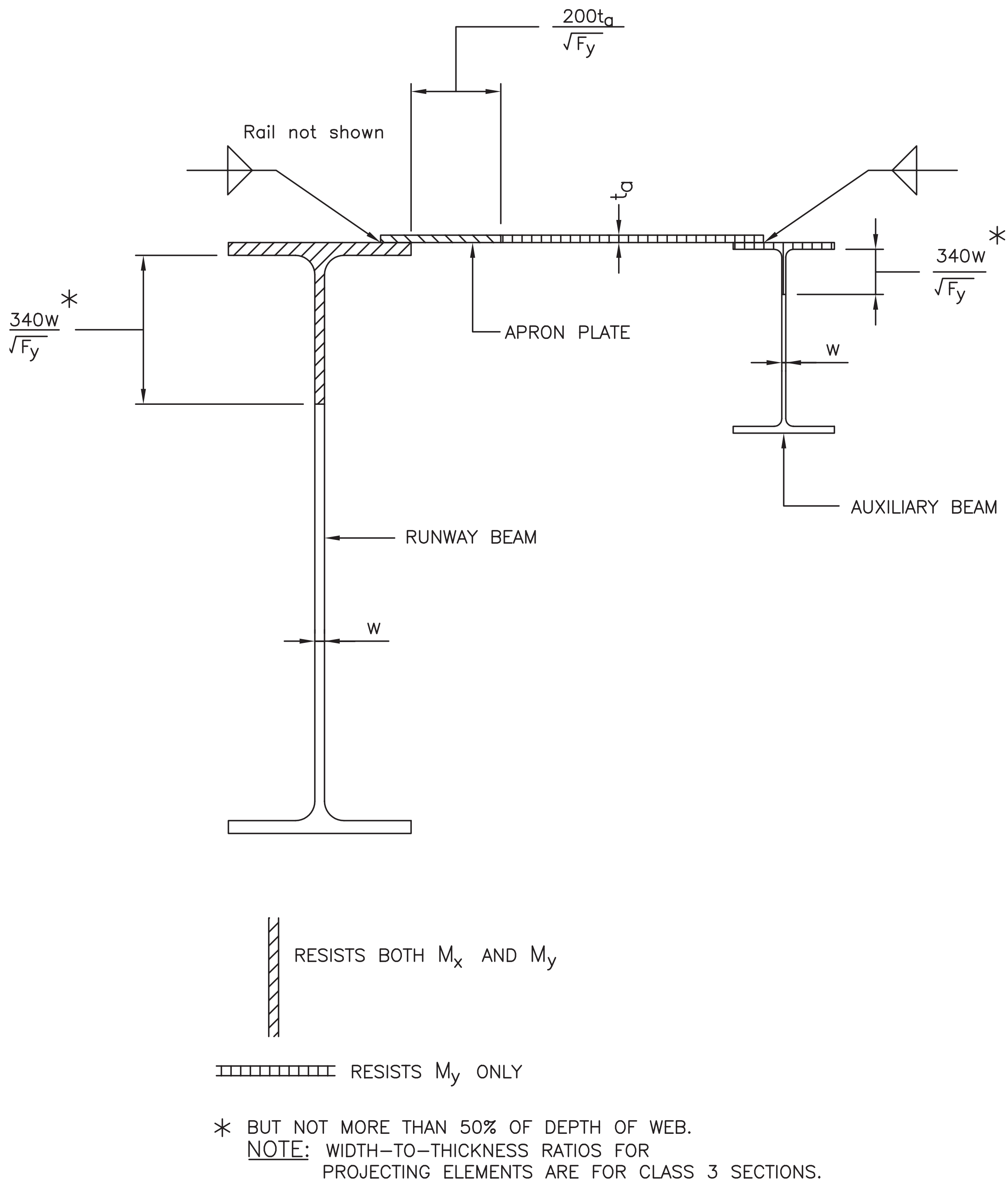


Figure 7
Runway Beam with Apron Plate

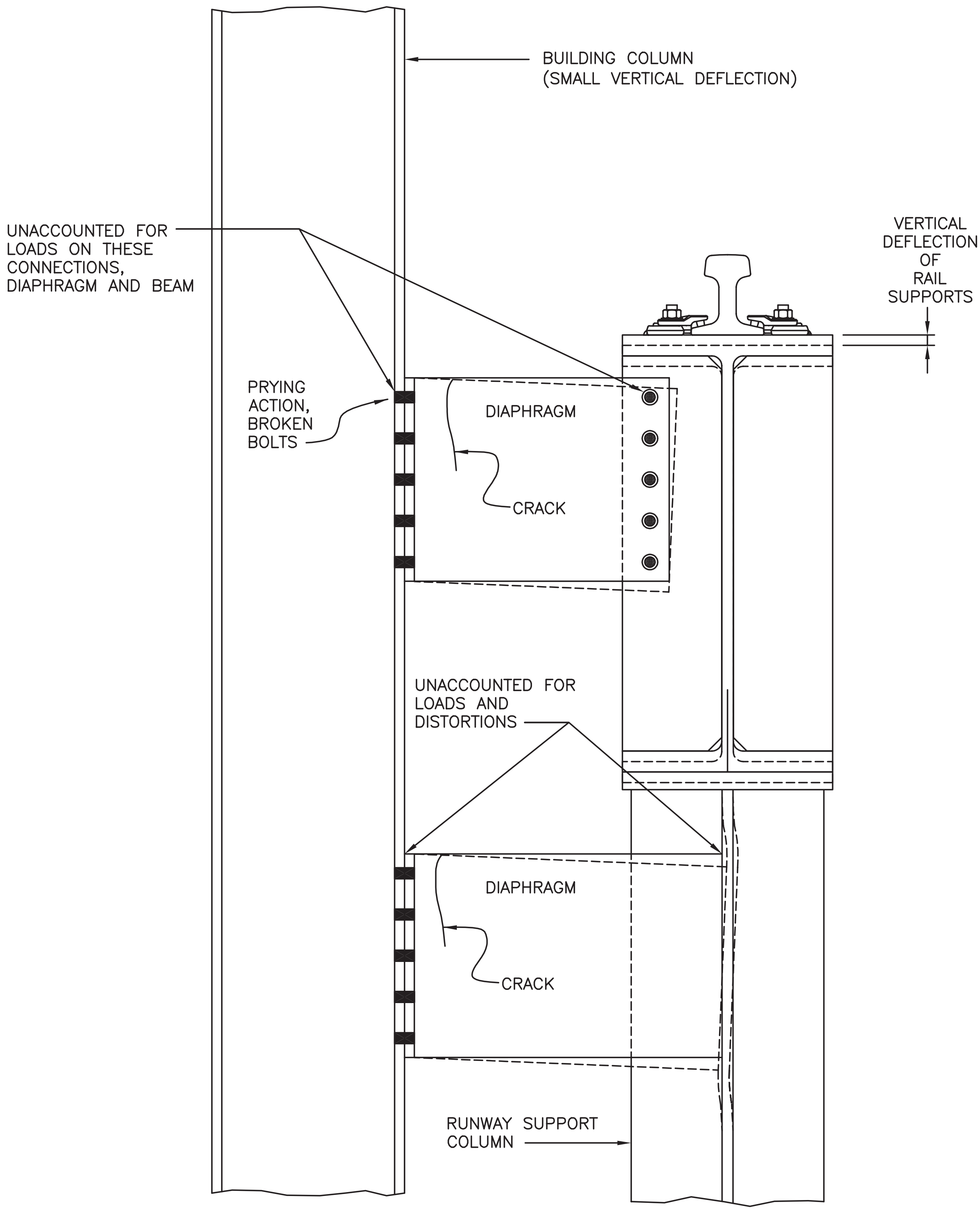


Figure 8
Typical Damage Near Columns Due to Fatigue and Unaccounted For Forces

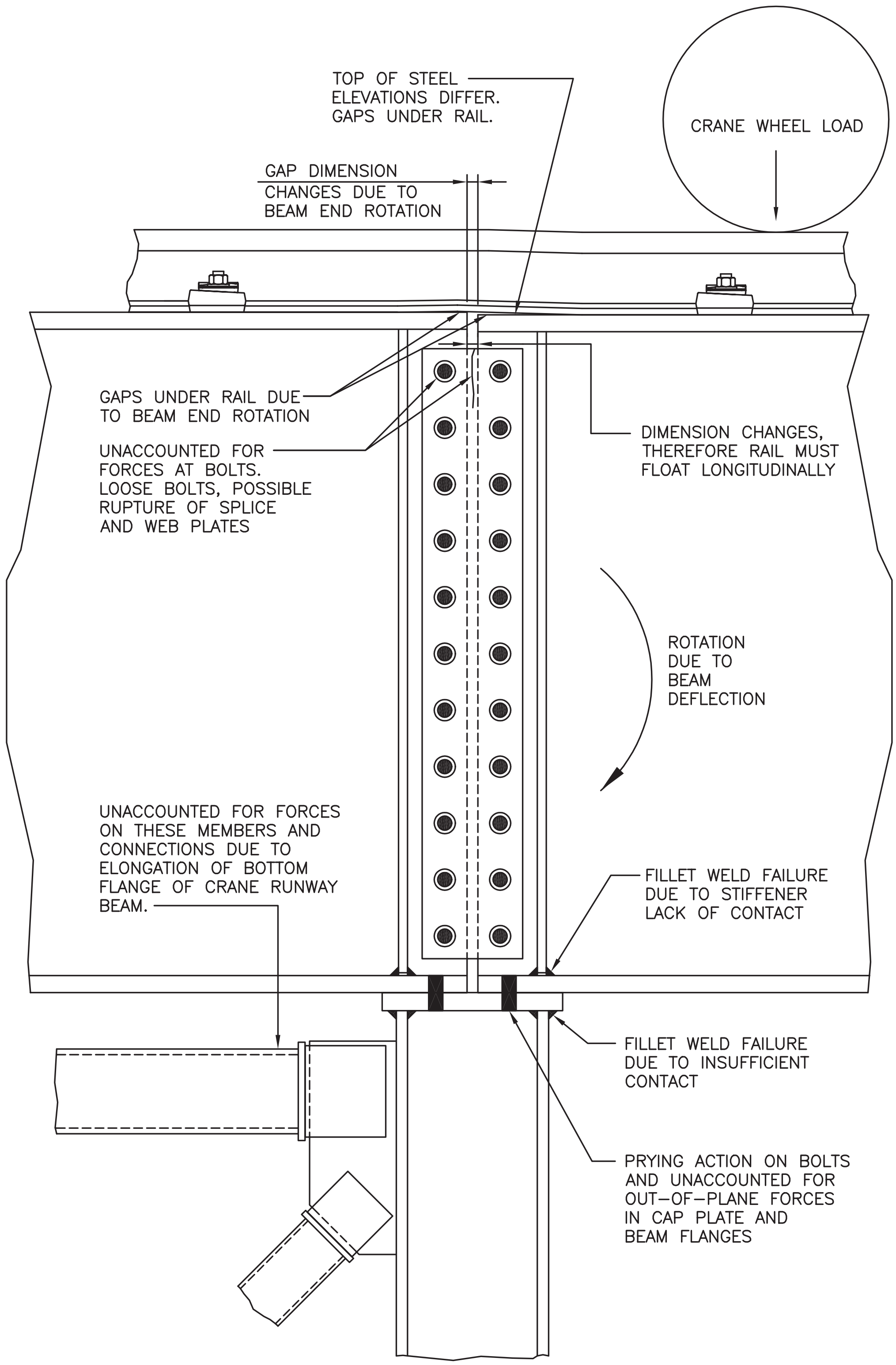


Figure 9
Examples of Unaccounted For Forces and Fatigue Damage at Beam Supports

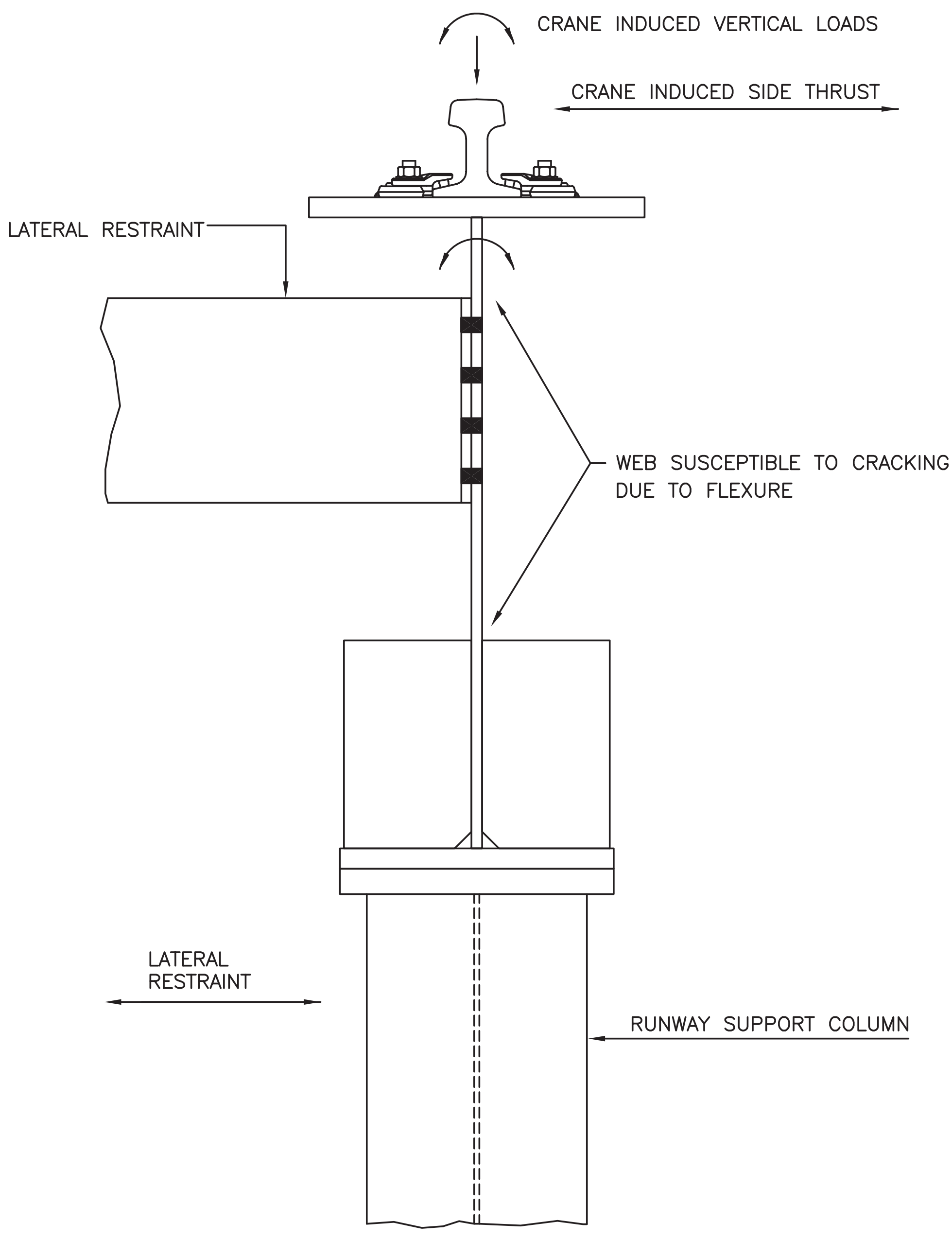


Figure 10
Example of Unaccounted For Forces and Fatigue at Beam Lateral Restraints

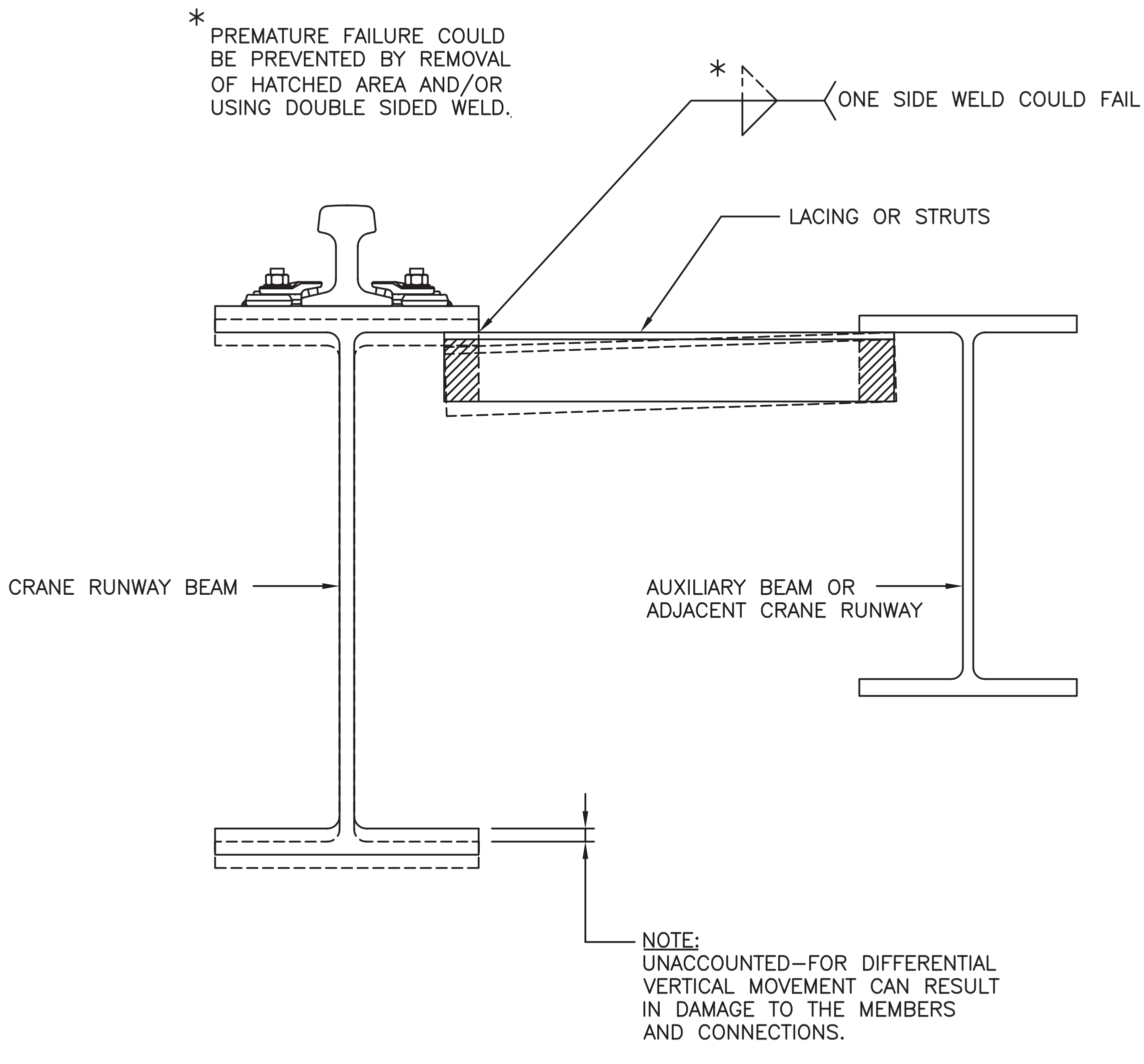


Figure 11
Example of Unaccounted For Differential Movements

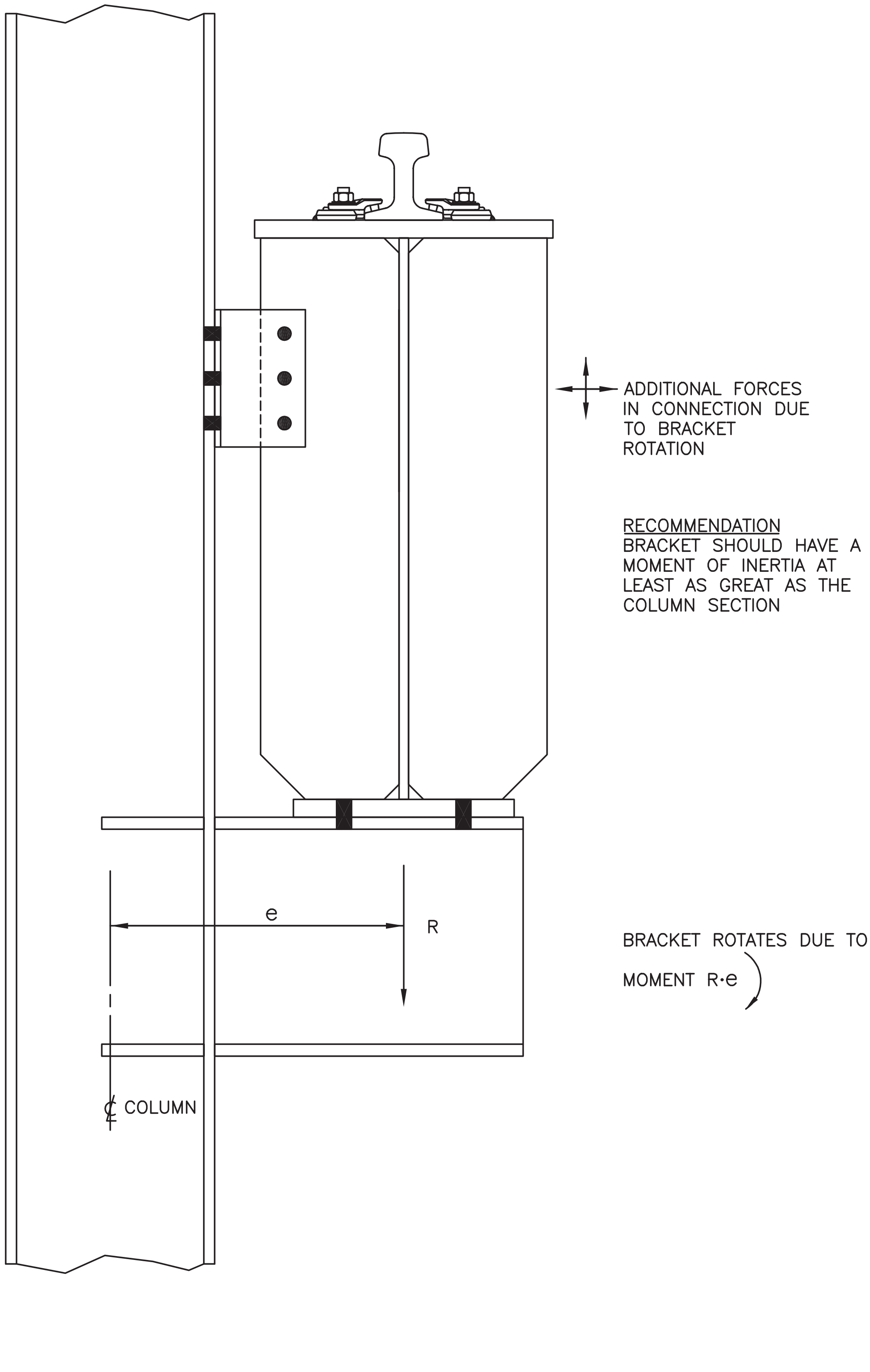


Figure 12
Compatible Deformation Forces Due to Deflection of Bracket

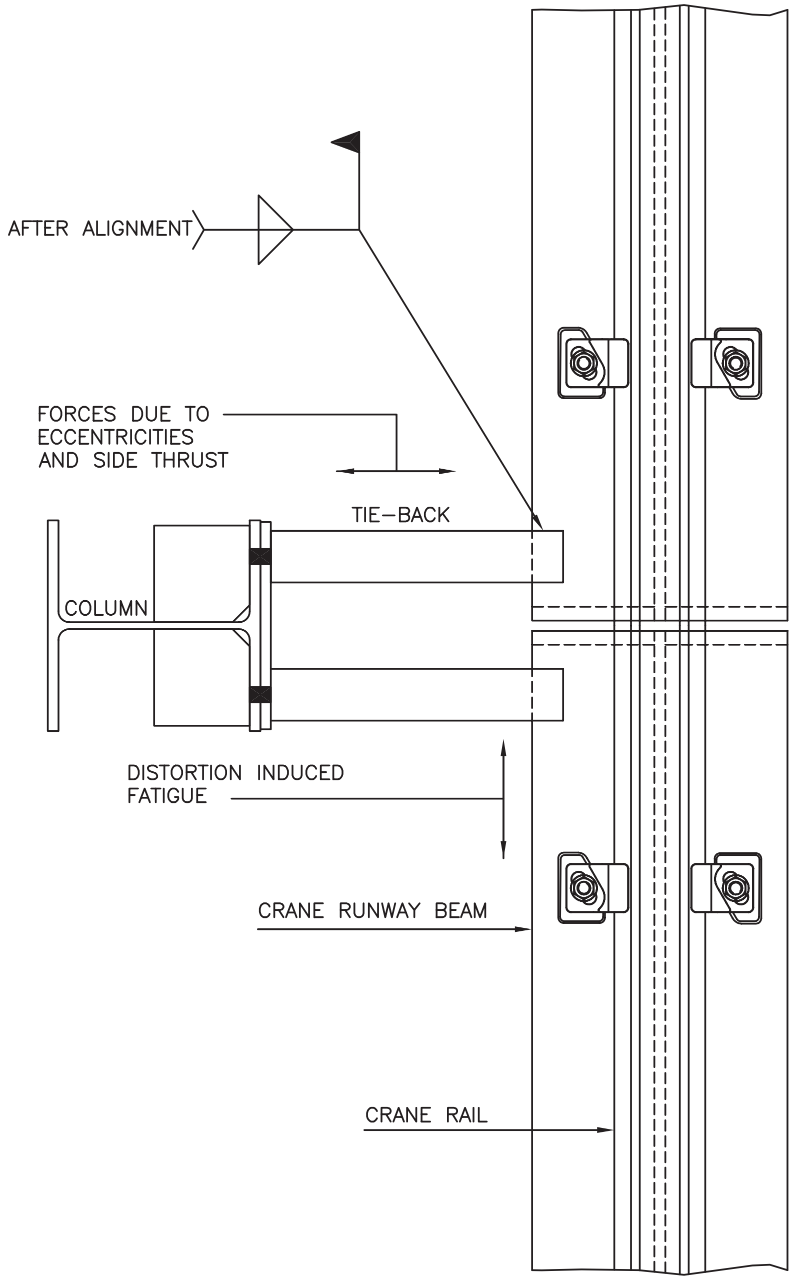


Figure 13
Example of a Light Duty Tie-Back

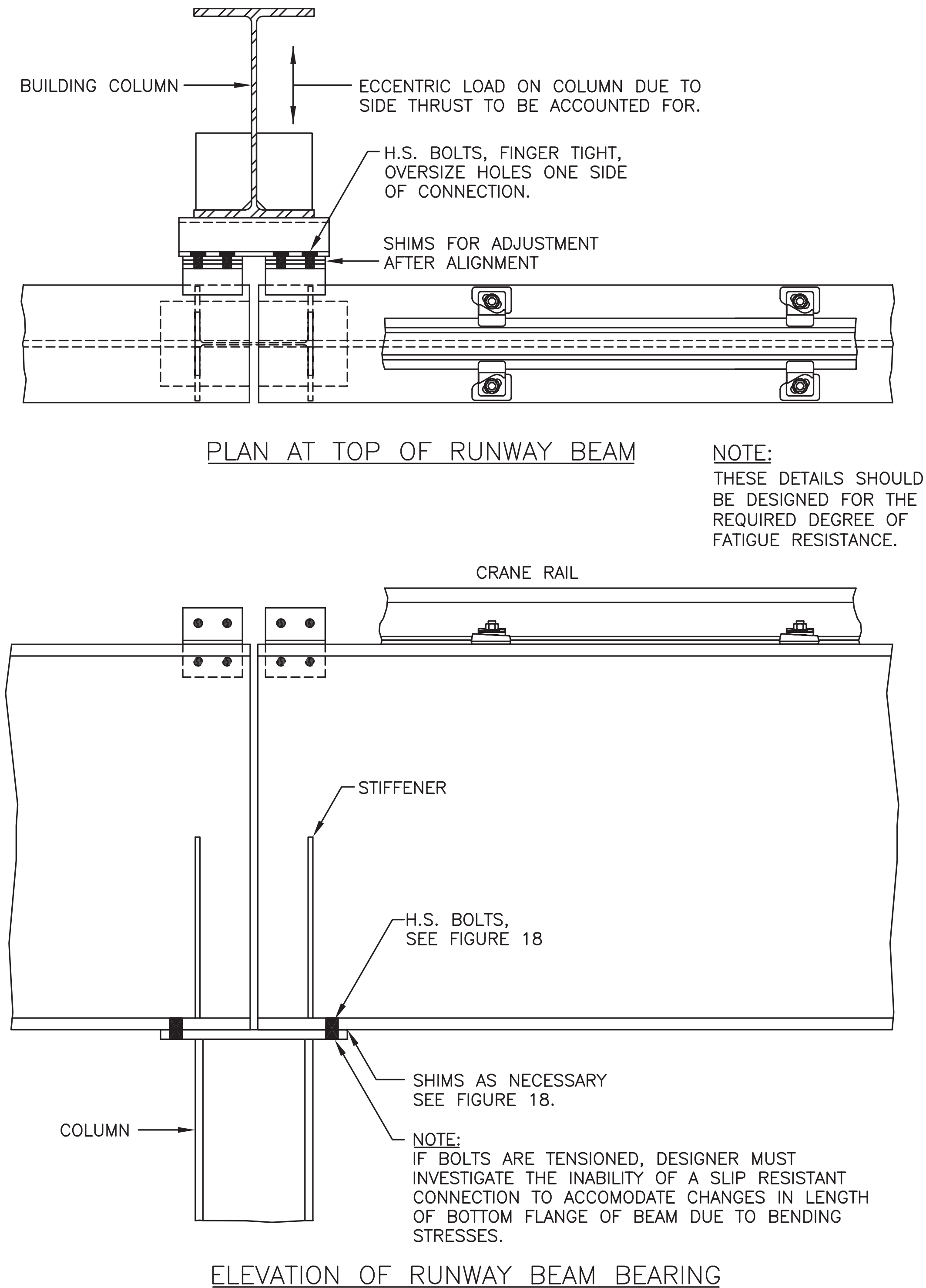
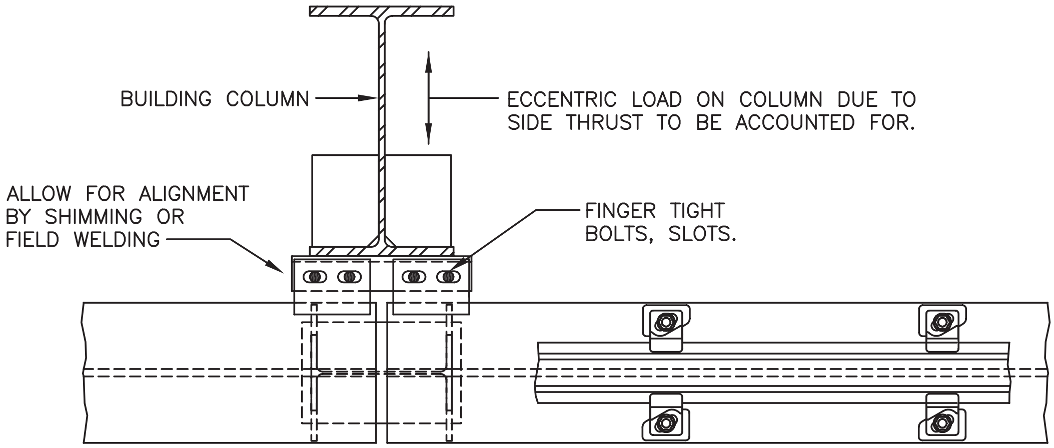
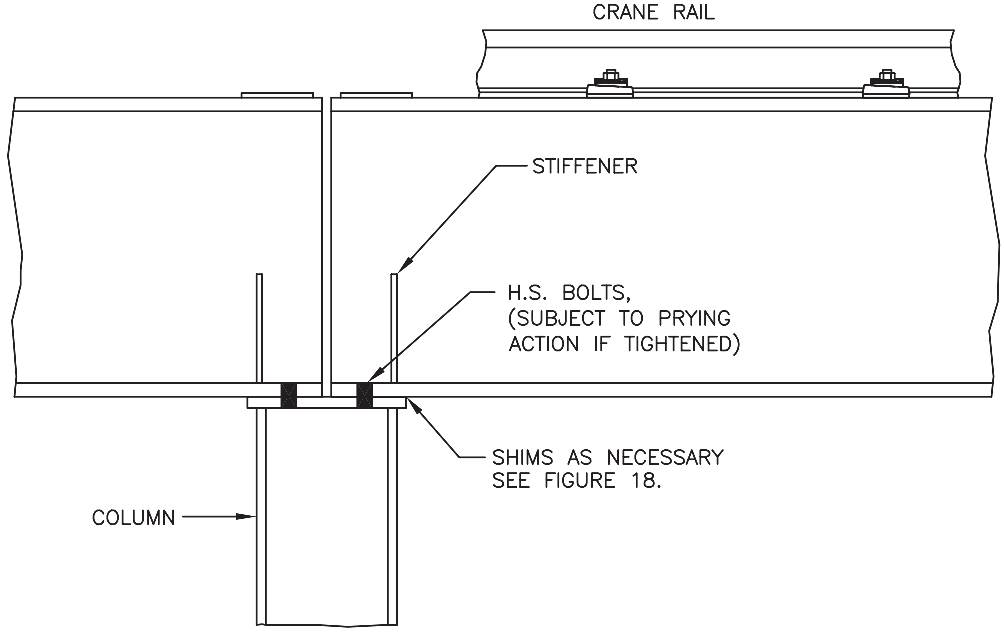


Figure 14
Details Suitable for Many Class SA, SB And SC Services



PLAN AT TOP OF RUNWAY BEAM

NOTE:
THESE DETAILS ARE
NOT FATIGUE RESISTANT



ELEVATION OF RUNWAY BEAM BEARING

Figure 15
Details Suitable for Light Duty Where Fatigue is Not a Consideration

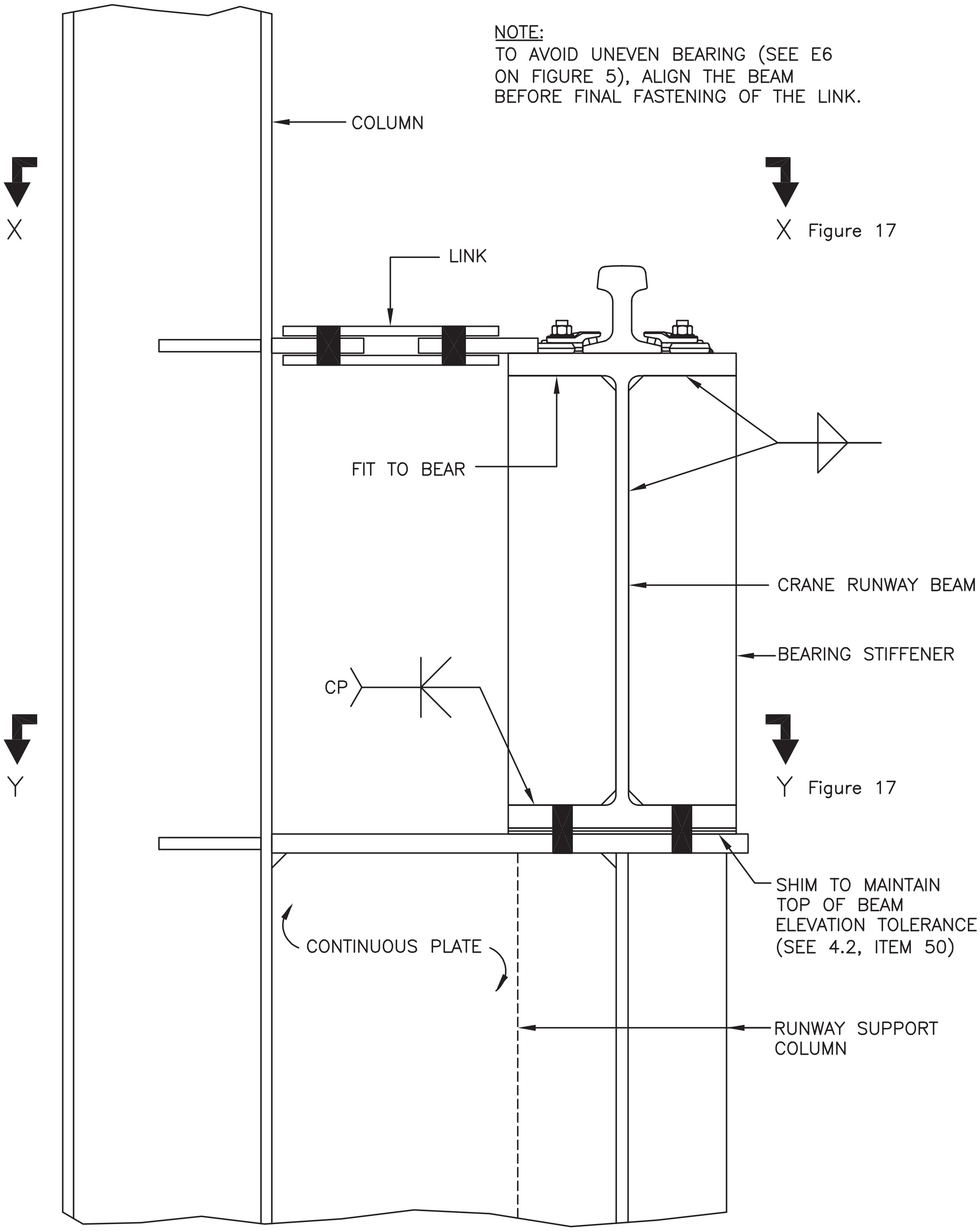
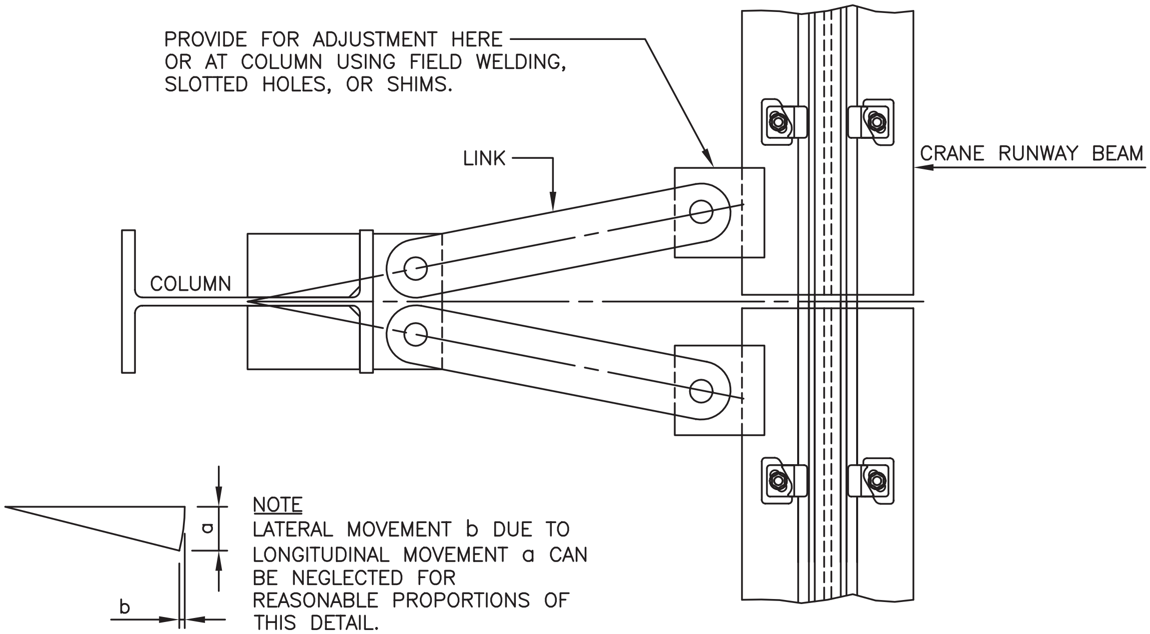
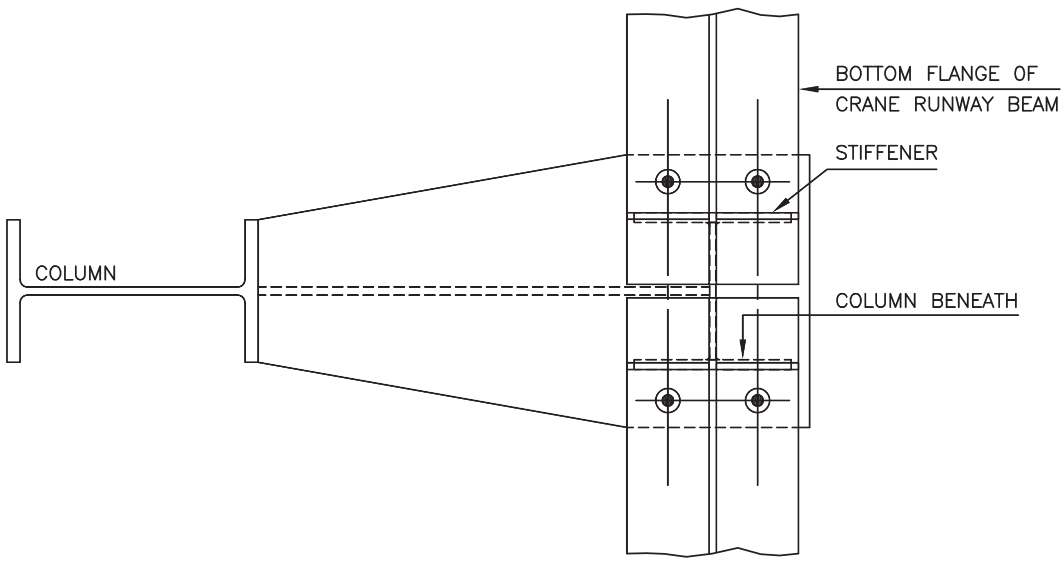


Figure 16
Example of a Fatigue Resistant Beam Support



PLAN X–X (TIE–BACK)



PLAN Y–Y (BEARING)

Figure 17
Details from Figure 16

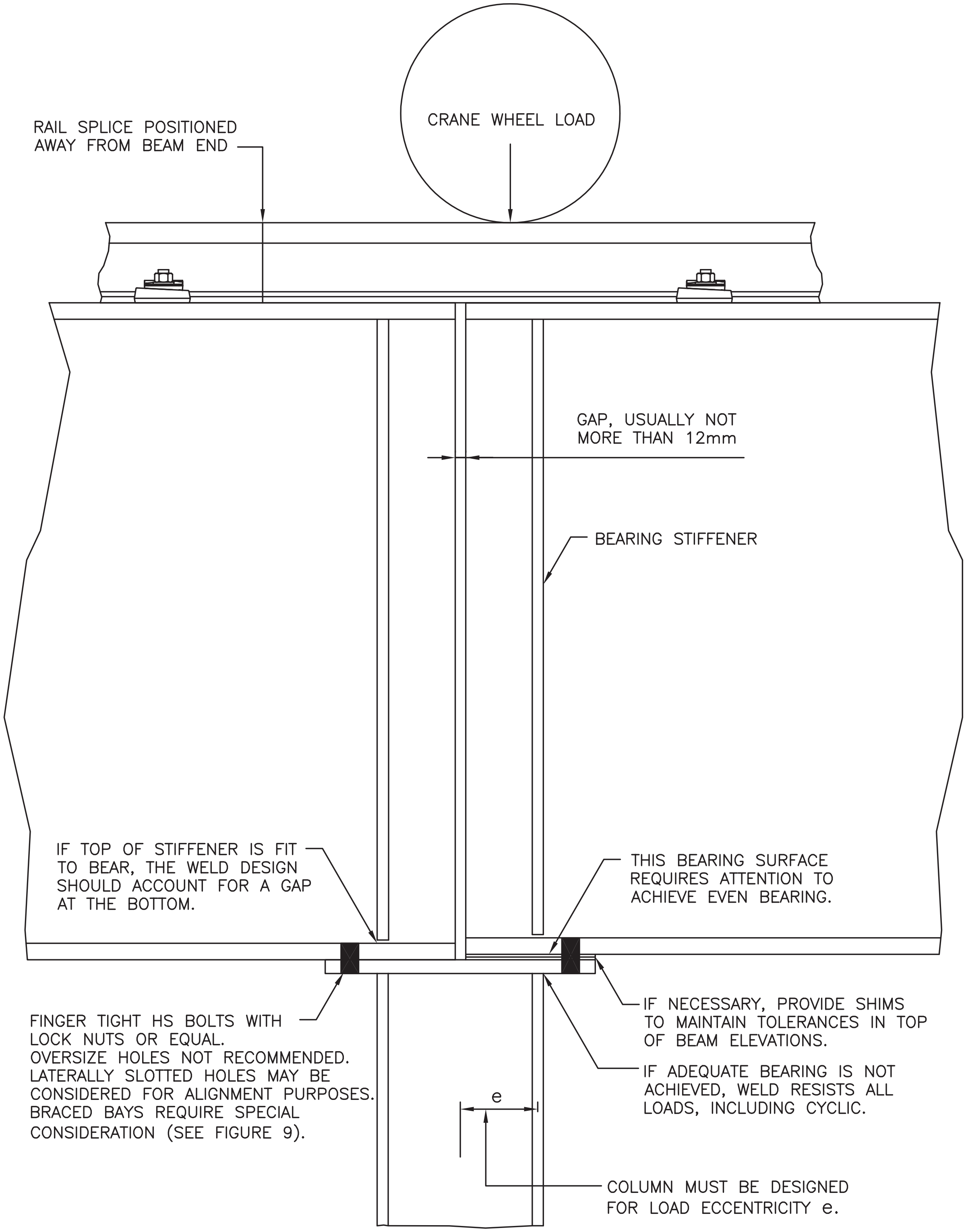


Figure 18
Bearing Detail Suitable for all Classes of Service

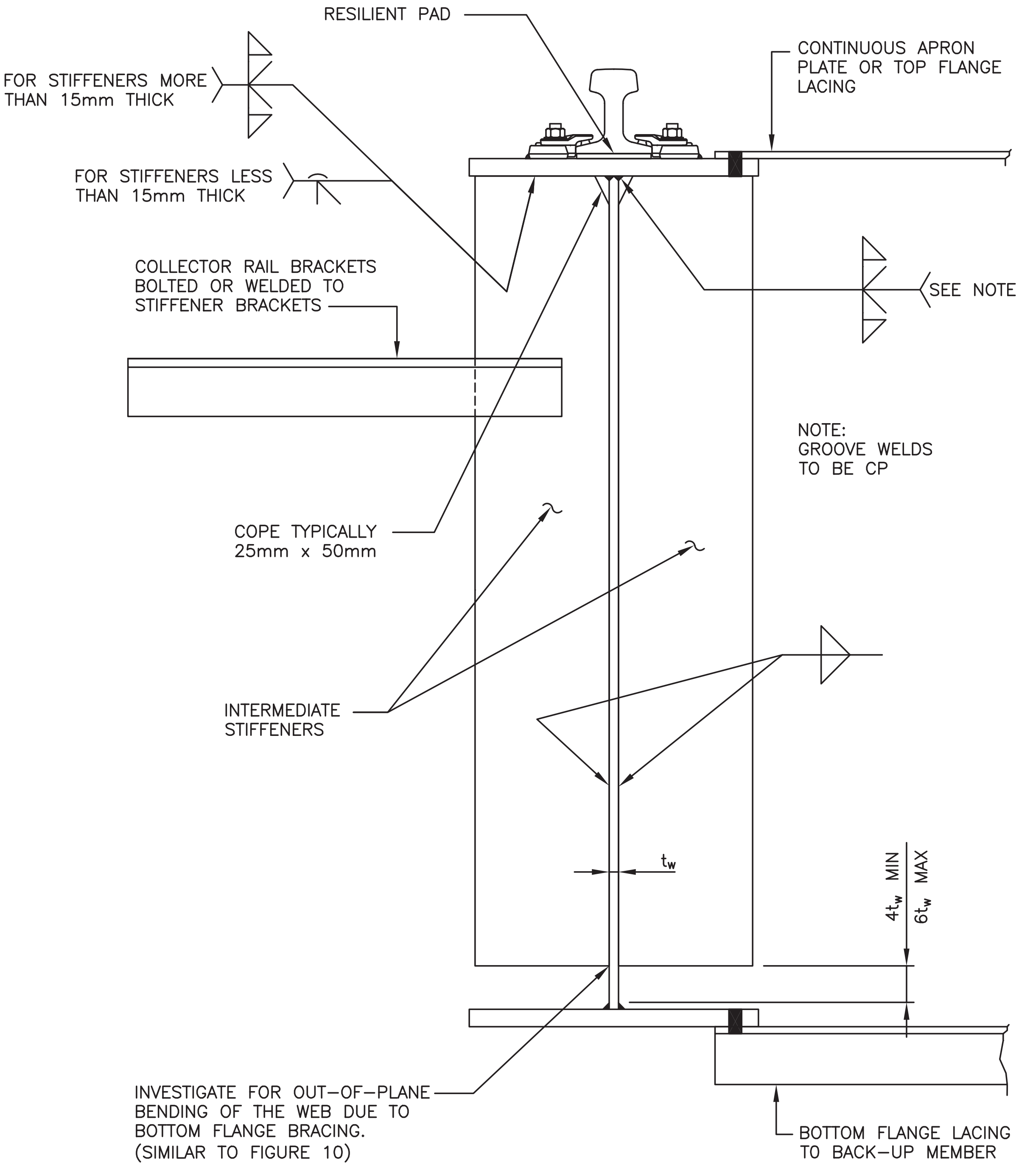


Figure 19
Typical Heavy Duty Crane Runway Beam

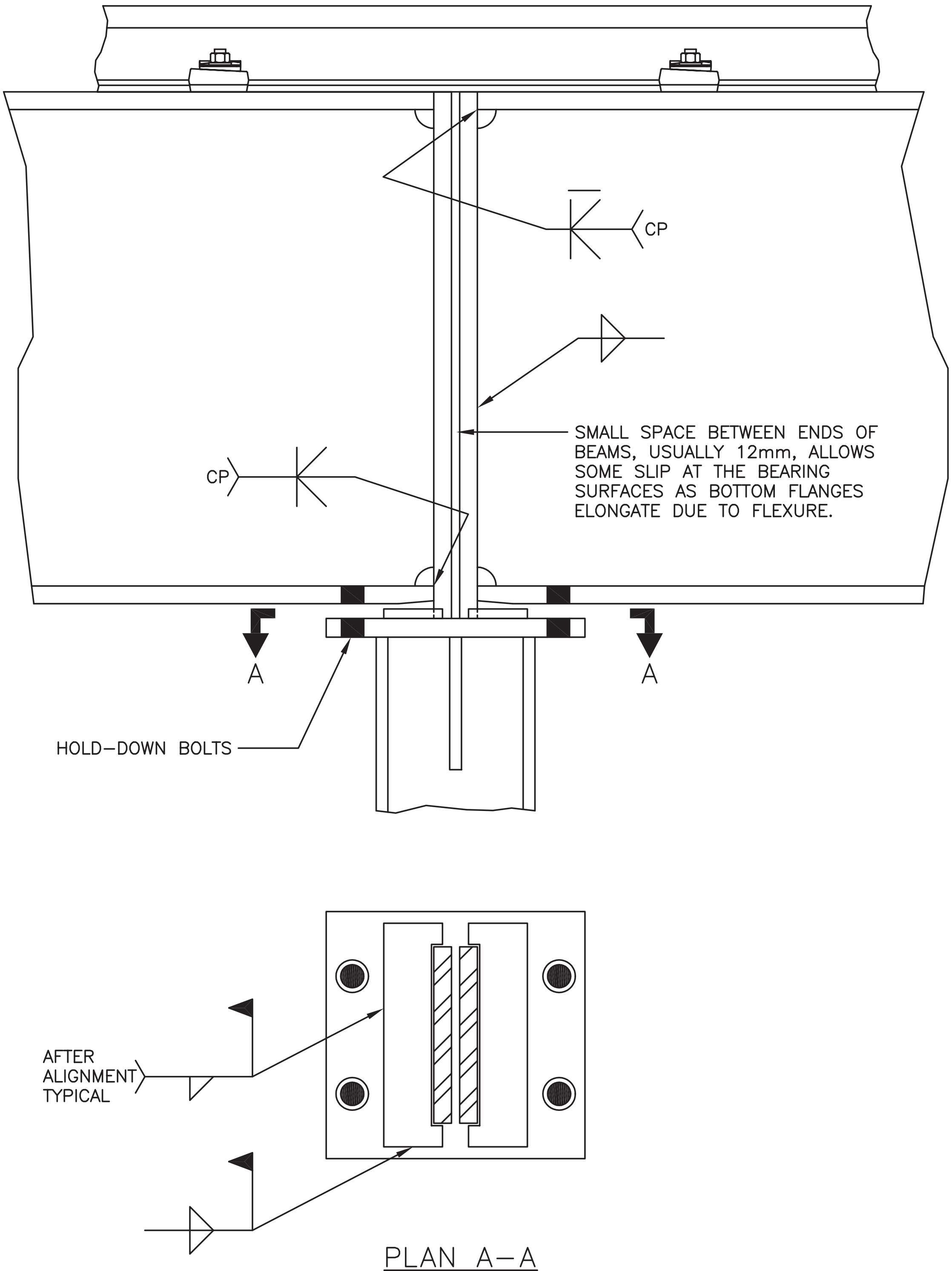
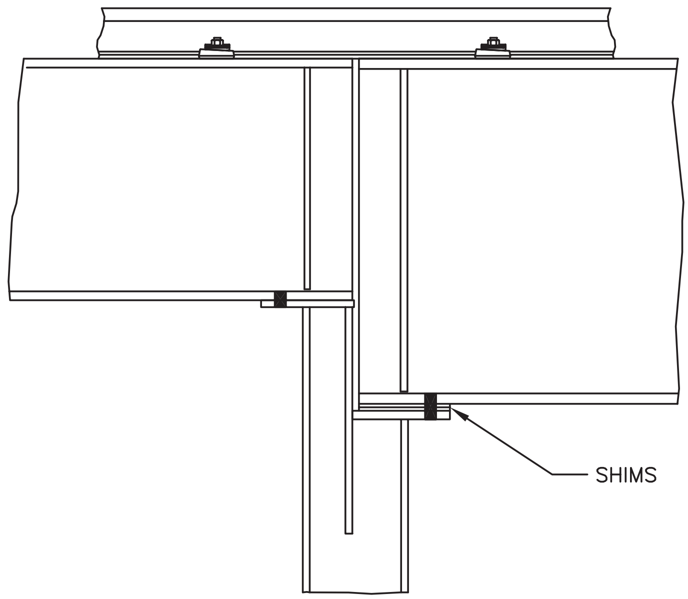
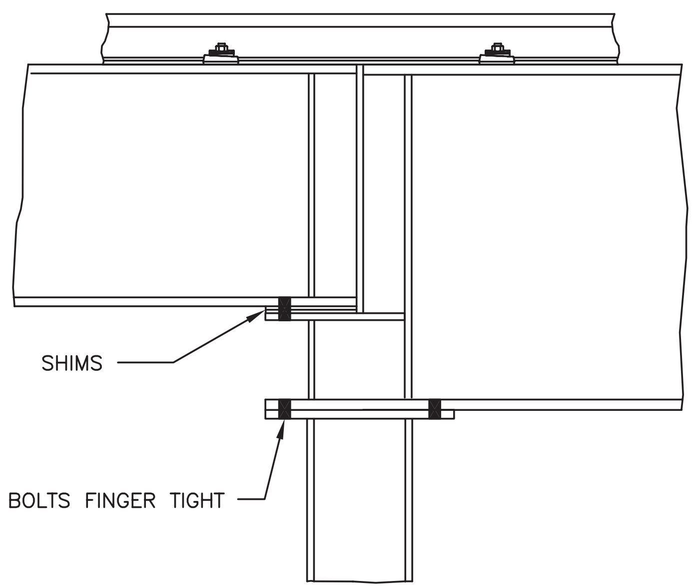


Figure 20
Example of a Heavy Duty Bearing Detail With Minimal End Restraint and Eccentricity



RECOMMENDED DETAIL FOR BEAM DEPTH CHANGE



THIS DETAIL MORE SUSCEPTIBLE TO FATIGUE
AND REPLACEMENT OF THE DEEPER BEAM IS DIFFICULT

Figure 21
Details for Beam Change in Depth

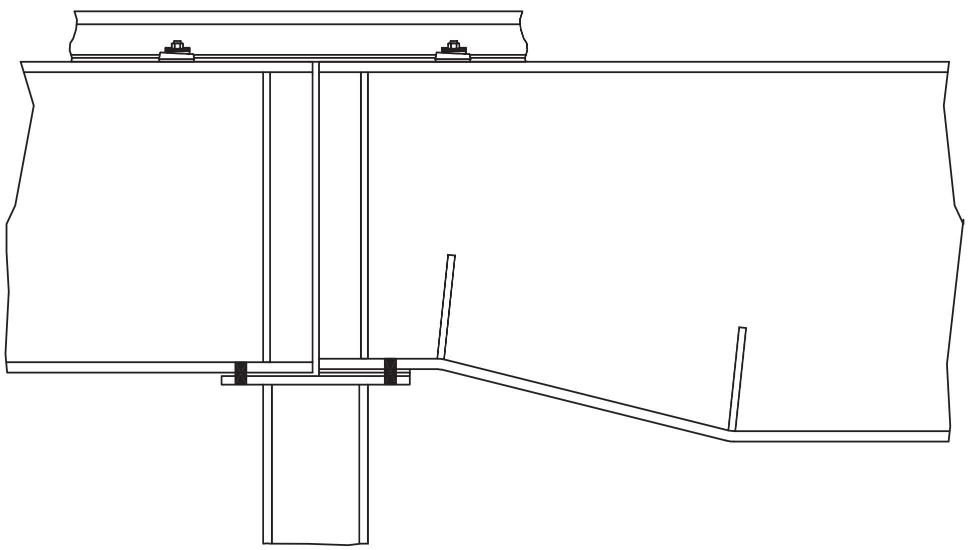


Figure 22
Alternative Detail for Change in Depth

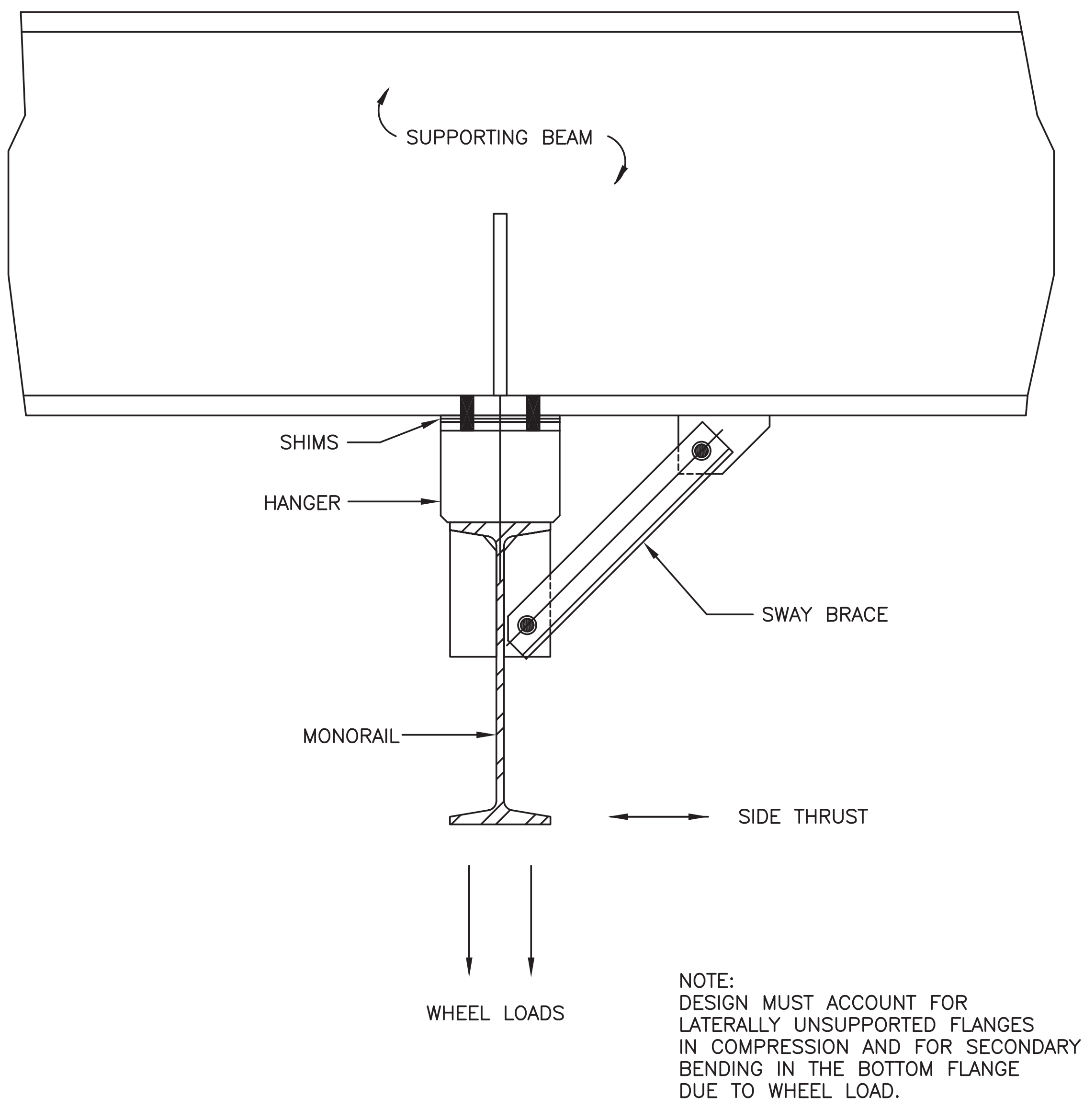


Figure 23
Details for Support of Underslung Cranes

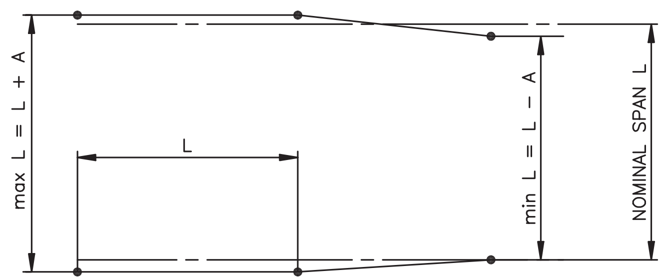
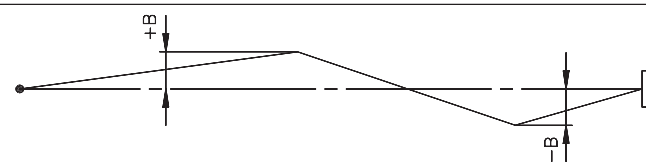
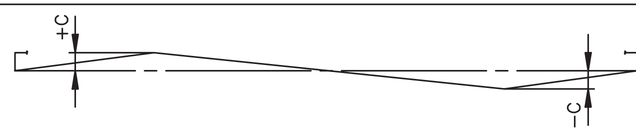
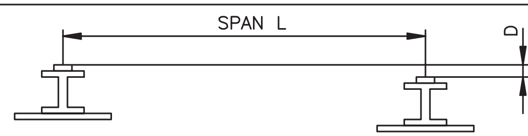
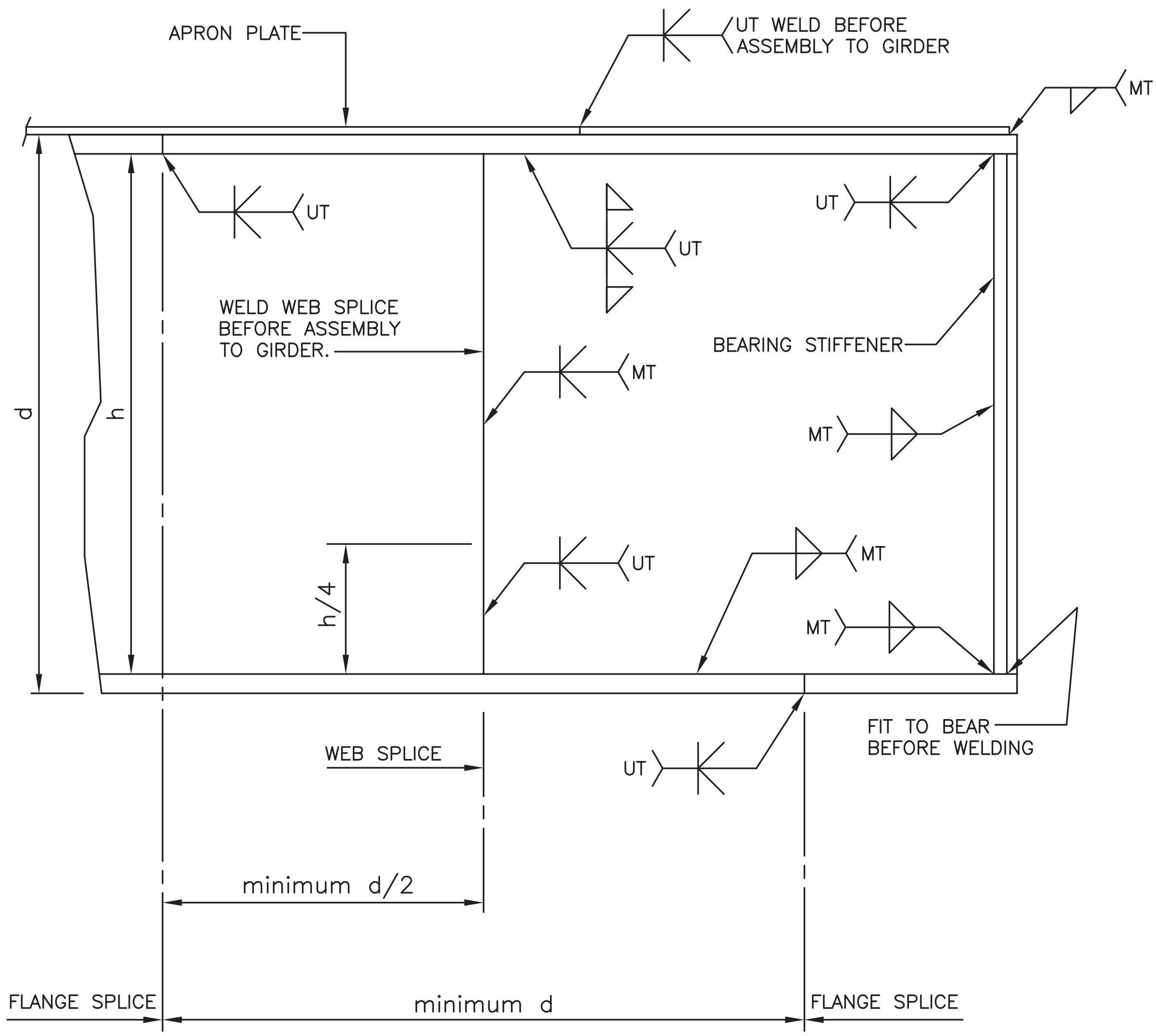
ITEM	FIGURE	OVERALL TOLERANCE	MAXIMUM RATE OF CHANGE
CRANE SPAN (L)		L<=15m A=5mm L>15m=<30m A=6mm L>30M A=10mm	1 IN 1000
STRAIGHTNESS (B)		B = 10mm	1 IN 1000
ELEVATION (C)		C = 10mm	1 IN 1000
RAIL−TO−RAIL ELEVATION (D)		L<=15m D=+/−5mm L>15m=<30m D=+/−6mm L>30m D=+/−10mm	1 IN 1000

Figure 24
Crane Runway Beam Erection Tolerances



NOTES:

1. CONSTRUCTION AND INSPECTION TO BE IN ACCORDANCE WITH CSA W59.
2. ALL GROOVE WELDS TO BE COMPLETE PENETRATION, GROUND FLUSH IN THE DIRECTION OF THE STRESS.
3. SPLICE LOCATIONS AND NUMBERS TO HAVE PRIOR APPROVAL BY THE STRUCTURE DESIGNER.

Figure 25
Typical Welding and Inspection Practice for Heavy Duty Beams

APPENDIX A

DESIGN EXAMPLES

Design Example 1
Illustration of Design of a Mono-symmetric Section Crane Runway Beam

(Note: Design is for bending strength only and is not a complete design)

Design Criteria		Value/Units
Codes and Standards		CSA S16-01
Importance (see NBCC 2005)		N.A.
Life of the Structure		N.A.
Materials (Plates, Shapes, Fasteners, etc.)		CSA G40.21 Grade 350W
Span		10 670 *mm*
Provision for Future Expansion?		N.A.
Simple Span?		Yes
Lateral Support for Top Flange?		No
Top of Rail Elevation, or Height from Main Floor		N.A.
Required Clearances to U/S Beam		N.A.
Side Thrust Equally Distributed Both Sides of Runway?		Yes
Number of Cranes, Each Runway		1
Collector Rail Mounting Details		N.A.
Design for Future Additional Cranes		No
Jib Cranes, or Provision for Jib Cranes		No
Design for Future Upgrades		No
Class of Cranes		CMAA Class A
Service (Description)		N.A.
Type of Duty (see table 2.1)		Light
Crane Hook Capacity	# hook(s) each	1
	Capacity each hook	22.68 tonnes, incl lifting gear
Weight of Crane Bridge		N.A.

Design Criteria	Value/Units
Weight of Crane Trolley	2 721 *kg*
Bridge Wheels per Rail — Total Number Driven	Two One
Bridge Wheel Spacing	3 050 *mm*
Minimum Distance Between Wheels of Cranes in Tandem	N.A.
Maximum Wheel Load, Each Crane (not including impact)	169.0 *kN*
Crane Rail — Description Self Load	ASCE 40, 89 mm height 19.8 kg/m
Rail Joints (bolted or welded)	N.A.
Resilient Pad Under Rail?	N.A.
Bridge Speed	N.A.
Type of Bumpers	N.A.
Bumpers Supplied with Crane?	N.A.
Bumper Force on Runway End Stop (Ultimate Load)	N.A.
Fatigue Criteria: Vertical - Equivalent passes of one crane, maximum wheel loads Horizontal - Equivalent cycles of side thrust at 50% of maximum side thrust	N.A.
Deflection Criteria: Vertical Limit (one crane, not including impact) Horizontal Limit	 Span/600 Span/400
Impact Criteria: Percentage of maximum wheel loads, one crane only	25%
Other Considerations	N.A.
*Weight Certified?	N.A.

Design Data

$$\text{Lifted Load} = \frac{22\,680\ (kg) \times 9.81\ (m/s^2)}{1000} = 222.4\ \ kN$$

$$\text{Trolley Load} = \frac{2\,721\ \ (kg) \times 9.81\ \ (m/s^2)}{1000} = 26.69\ \ kN$$

Crane Runway Beam Span $= 10\,670\ \ mm$

Crane Wheel Base $= 3\,050\ \ mm$

Maximum Wheel Loads $= 169\ \ kN$, not including impact

1) Calculate M_x

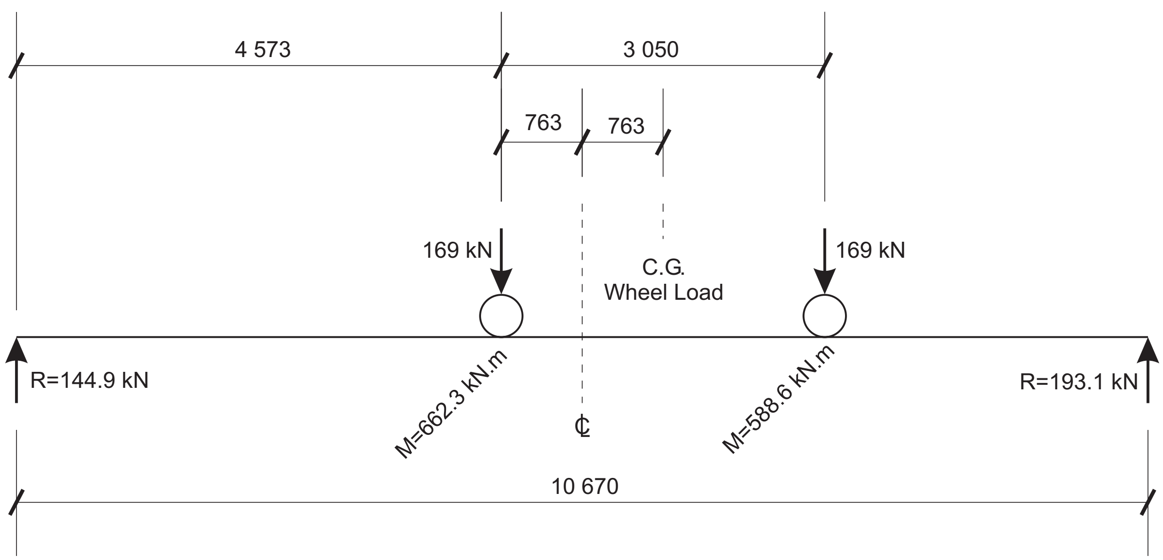


Figure A1
Wheel Loads

Point of maximum bending moment is at $0.5\left(10\,670 - \dfrac{3\,050}{2}\right) = 4\,573\ \ mm$

M_{LL} under wheel load closest to mid-span $= 144.9 \times 4.573 = 662.6\ \ kN \cdot m$

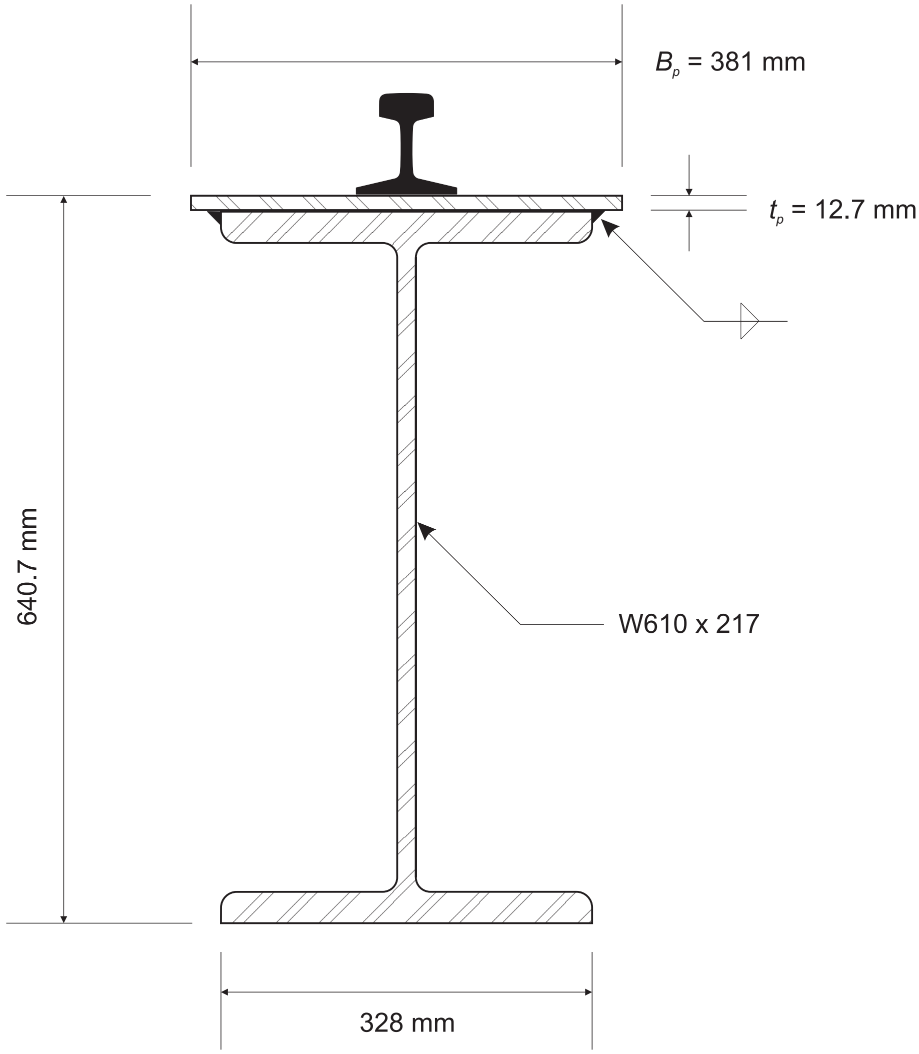
If necessary, the left reaction (144.9) or the right reaction (193.1) can be calculated as follows:

$$R_l = 169\frac{\left[10.67 - \left(4.573 + 3.050\right)\right]}{10.67}\ \ (right\ wheel) + 169\frac{\left[10.67 - 4.573\right]}{10.67}\ \ (left\ wheel)$$

$$= 48.3 + 96.6 = 144.9\ \ kN$$

$$R_r = 169\frac{\left[4.573 + 3.050\right]}{10.67}\ \ (right\ wheel) + 169\frac{4.573}{10.67}\ \ (left\ wheel)$$

$$= 120.7 + 72.4 = 193.1\ \ kN$$



<u>W610 × 217</u>
(All units in mm)

$A = 27\,800$	$d = 628$	$r_x = 262$
$I_x = 1\,910 \times 10^6$	$b = 328$	$Z_x = 6\,850$
$S_x = 6\,070 \times 10^3$	$t = 27.7$	$r_y = 76.7$
$I_y = 163 \times 10^6$	$w = 16.5$	$Z_y = 1\,530$
$S_y = 995 \times 10^3$	$J = 5\,600 \times 10^3$	$C_w = 14\,700 \times 10^9$

Rail $40^{\#}$, $d = 89$ mm

Figure A2
Runway Beam Section

M due to impact $= 0.25 \times 662.3 = 165.6 \;\; kN \cdot m$

Estimated dead load, including rail and conductors is $2.64 \;\; kN/m$

$$M_{DL} = 2.64 \times \frac{10.67^2}{8} = 37.57 \;\; kN \cdot m$$

Factored Moment $\quad M_{fx} = 1.25(37.57) + 1.5(662.3 + 165.6)$

$$= 47 + 1242 = 1289 \;\; kN \cdot m$$

2) Determine Side Thrust

Use 20% of the sum of the lifted load and the trolley (see Table 2.1), equally distributed to each side.

Side Thrust $= 0.2(222.4 + 26.69) = 49.82 \;\; kN = 12.45 \;\; kN$ / wheel

Ratio of side thrust to maximum wheel load $= \dfrac{12.45}{169} = 0.073\,67$

Specified moment M_H due to side thrust

$$M_H = 0.073\,67 \times 662.3 = 48.79 \;\; kN \cdot m$$

Factored moment due to side thrust

$$M_{HF} = 1.5 \times 48.79 = 73.19 \;\; kN \cdot m$$

3) Select a trial section

For vertical deflection, a preliminary analysis shows that a section with $I_x = 2.0 \times 10^9 \;\; mm^4$ will deflect 18.5 mm maximum.

Using $\dfrac{l}{600}$ as the criterion, the maximum allowable vertical deflection $= \dfrac{10\,670}{600} = 17.78 \;\; mm$

therefore I_x should be at least $\dfrac{18.5}{17.78} \times 2.0 \times 10^9 = 2.081 \times 10^9 \;\; mm^4$

Considering that horizontal deflection $< \dfrac{l}{400}$, then $\Delta_{\max} = \dfrac{10\,670}{400} = 26.7 \;\; mm$, and

$$I_{y(top\ flange)} \geq \frac{18.5}{26.7} \times 0.073\,67 \times 2.0 \times 10^9 = 102.1 \times 10^6 \;\; mm^4$$

After some preliminary calculations, the cover plated $W610 \times 217$ section in Figure A2 is chosen for analysis.

4) Determine Class of Section

Check for Class 2 (Compact) (S16-01, Clause 11.2)

For flanges and projecting elements $\qquad \dfrac{b}{t} \leq \dfrac{170}{\sqrt{F_y}} = 9.09$

Cover plates between lines of welds $\qquad \dfrac{b}{t} \leq \dfrac{525}{\sqrt{F_y}} = 28.06$

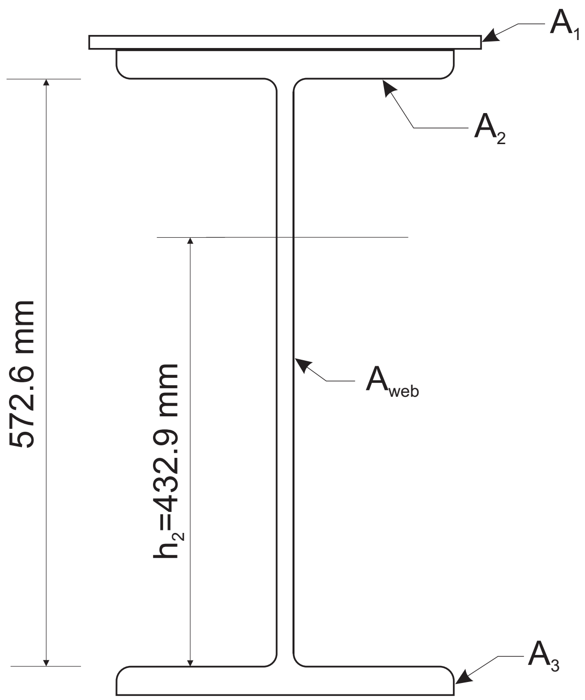
$W610 \times 217$ - Class1 for bending (Table 5.1 in CISC Handbook)

Cover Plate 381×12.7 *mm*, projection $= (381 - 328)/2 = 26.5 \;\; mm$

$$\frac{b}{t} \text{ of projecting element} = \frac{26.5}{12.7} = 2.09 \quad < \quad 9.09 \quad \text{OK}$$

$$\frac{b}{t} \text{ between welds} = \frac{328}{12.7} = 25.82 \quad < \quad 28.06 \quad \text{OK}$$

Section qualifies as Class 2 in bending (actually Class 1)



$$A_1 = 12.7 \times 381 = 4839 \;\; mm^2$$
$$A_2 = 27.7 \times 328 = 9086 \;\; mm^2$$
$$A_{web} = 16.5 \times 572.6 = 9448 \;\; mm^2$$
$$A_3 = 27.7 \times 328 = 9086 \;\; mm^2$$

Figure A3
Section Areas

5) Calculate M_p and Z for both axes

Calculate plastic neutral axis of the section (See Figure A3)

$$16.5h_2 = 16.5\left(572.6 - h_2\right) + 4839$$
$$33h_2 = 9448 + 4839$$
$$h_2 = 432.9 \;\; mm$$

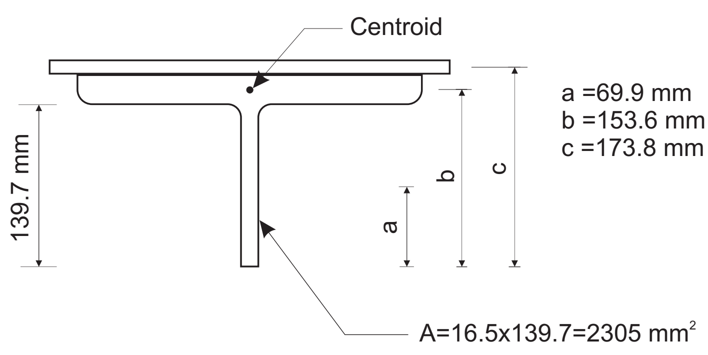


Figure A4
Centroid of Section Above Neutral Axis

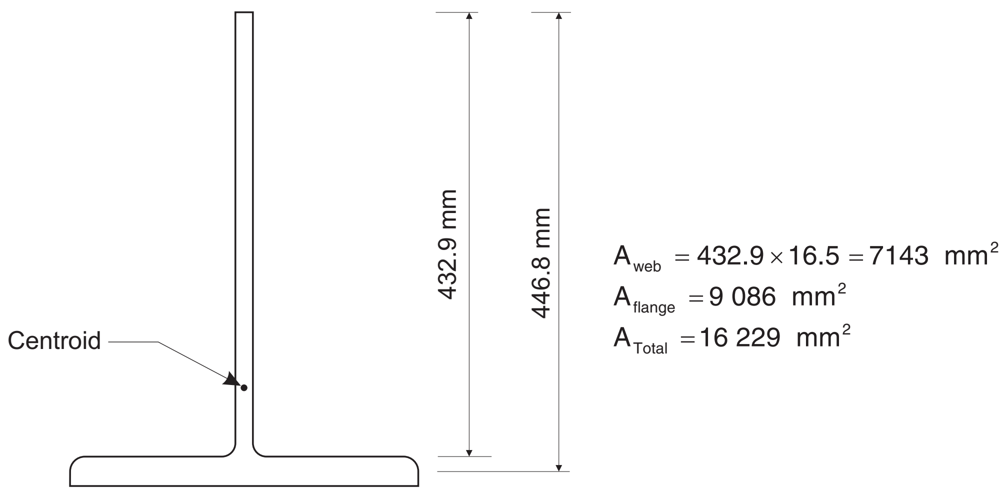


Figure A5
Centroid of Section Below Neutral Axis

Calculate Centroids of Top and Bottom

$$\text{Centroid Top} = \frac{(4839 \times 173.8) + (9086 \times 153.6) + (2305 \times 69.9)}{4839 + 9086 + 2305} = 147.7\ mm$$

$$\text{Centroid Bottom} = \frac{(9086 \times 446.8) + (7143 \times 216.5)}{9086 + 7143} = 345.4\ mm$$

Distance centroid to centroid $= 147.7 + 345.4 = 493.1\ mm$

$$M_p = \frac{350 \times 16\,229 \times 493.1}{10^6} = 2800\ kN.m$$

$$Z = \frac{2800 \times 10^6}{350} = 8.0 \times 10^6\ mm^3$$

For the weak axis, top flange only

$$Z = 12.7 \times \frac{381^2}{4} + 27.7 \times \frac{328^2}{4} = 1.206 \times 10^6\ mm^3$$

$$M_p = 350 \times 1.206 \times \frac{10^6}{10^6} = 422\ kN.m$$

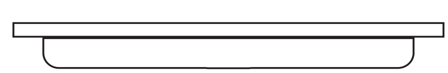


Figure A6
Top Flange Only

6) Calculate Elastic Section Properties x-x (for the built-up section)

Material	A (mm^2)	y_b (mm)	Ay_b $(10^3 \ mm^3)$	Ay_b^2 $(10^6 \ mm^4)$	I_0 $(10^6 \ mm^4)$
W	27 800	314	8 730	2 740	1 910
Plate	4 839	634.4	3 070	1 948	0.065
$\sum$	32 639		11 800	4 688	1 910

$$y_B = \frac{\sum Ay_b}{\sum A} = \frac{11\,800 \times 10^3}{32\,639} = 361.5 \ mm \quad \text{and} \quad y_T = 640.7 - 361.5 = 279.2 \ mm$$

$$I_{xx} = \sum I_0 + \sum Ay_b^2 - y_B^2 \sum A$$

$$= 1910 \times 10^6 + 4\,688 \times 10^6 - 32\,639 \left(361.5\right)^2 = 2\,332 \times 10^6 \ mm^4$$

$$S_B = \frac{I_{xx}}{y_B} = \frac{2\,332 \times 10^6}{361.5} = 6\,451 \times 10^3 \ mm^3$$

$$S_T = \frac{I_{xx}}{y_T} = \frac{2\,332 \times 10^6}{279.2} = 8\,352 \times 10^3 \ mm^3$$

7) Calculate Elastic Section Properties y-y

$$I_{yy} \ top \ flange \qquad = \left(27.7 \times \frac{328^3}{12}\right) + \left(12.7 \times \frac{381^3}{12}\right)$$

$$= 81.46 \times 10^6 + 58.53 \times 10^6 = 140 \times 10^6 \ mm^4$$

$$I_{yy} \ web \qquad = 572.6 \times \frac{16.5^3}{12} = 0.2143 \times 10^6 \ mm^4$$

$$I_{yy} \ bottom \ flange \qquad = 81.46 \times 10^6 \ mm^4$$

$$\sum I_{yy} \qquad = 221.7 \times 10^6 \ \text{mm}^4$$

$$S_{yy} \ top \ flange \qquad = \frac{140 \times 10^6}{190.5} = 0.7349 \times 10^6 \ mm^3$$

$$S_{yy} \ bottom \ flange \qquad = \frac{81.46 \times 10^6}{164} = 0.4967 \times 10^6 \ mm^3$$

8) Calculate " Equivalent" top flange

$$A = \left(27.7 \times 328\right) + \left(12.7 \times 381\right) = 13.92 \times 10^3 \ mm^2$$

$$I = 140 \times 10^6 \ mm^4$$

$$\frac{tw^3}{12} = 140 \times 10^6 \ mm^4$$

Note: Two parallel plates must be continuously welded and the projecting element must be relatively small. For more information, refer to Tremblay and Legault 1996.

$$tw = 13.92 \times 10^3 \quad mm^2$$

$$\frac{13.92 \times 10^3}{w} \times \frac{w^3}{12} = 140 \times 10^6$$

$$w = \sqrt{\frac{12 \times 140 \times 10^6}{13.92 \times 10^3}} = 347.4$$

$$t = \frac{13.92 \times 10^3}{347.4} = 40.07$$

Use equivalent top flange for purposes of analysing the Mono-symmetric section

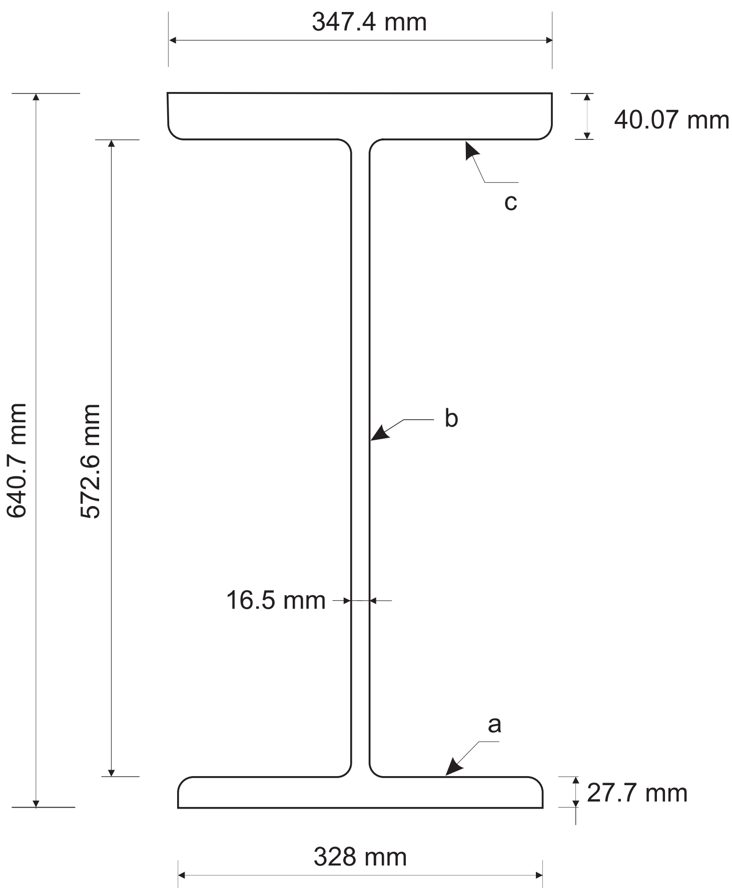


Figure A7
Section for Purposes of Monosymetric Section Analysis

9) Calculate Section properties x-x

Material	A (mm^2)	y_b (mm)	Ay_b $(10^3 \ mm^3)$	Ay_b^2 $(10^6 \ mm^4)$	I_0 $(10^6 \ mm^4)$
a	9 086	13.85	125.8	1.743	0.5809
b	9 448	314	2 967	931.5	258.1
c	13 920	620.7	8 640	5 363	1.863
$\sum$	32 450		11 730	6 296	260.5

$$y_B = \frac{\sum Ay_b}{\sum A} = \frac{11\,730 \times 10^3}{32\,450} = 361.5 \ mm \quad \text{and} \quad y_T = 640.7 - 361.5 = 279.2 \ mm$$

$$I_{xx} = \sum I_0 + \sum Ay_b^2 - y_B^2 \sum A$$

$$= 260.5 \times 10^6 + 6\,292 \times 10^6 - 32\,450(361.5)^2$$

$$= 2\,317 \times 10^6 \ mm^4$$

$$S_{xB} = \frac{I_{xx}}{y_B} = \frac{2\,317 \times 10^6}{361.5} = 6\,409 \times 10^3 \ mm^3$$

$$S_{xT} = \frac{I_{xx}}{y_T} = \frac{2\,317 \times 10^6}{279.2} = 8\,300 \times 10^3 \ mm^3$$

10) Calculate Section Properties y-y

I_{yy} *top flange* $= 140 \times 10^6 \ mm^4$

I_{yy} *web* $= 572.6 \times \dfrac{16.5^3}{12} = 0.2143 \times 10^6 \ mm^4$

I_{yy} *bottom flange* $= 81.46 \times 10^6 \ mm^4$

$\sum I_{yy} = 221.7 \times 10^6 \ mm^4$

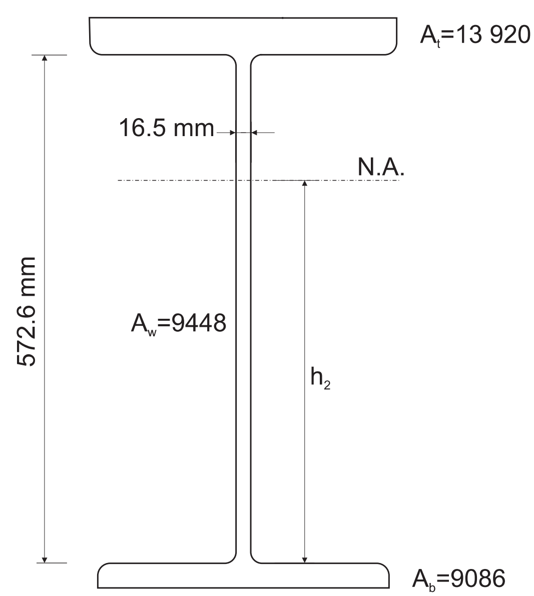


Figure A8
Neutral axis of Equivalent Section

11) Calculate M_p and Z

Determine distance to neutral axis.

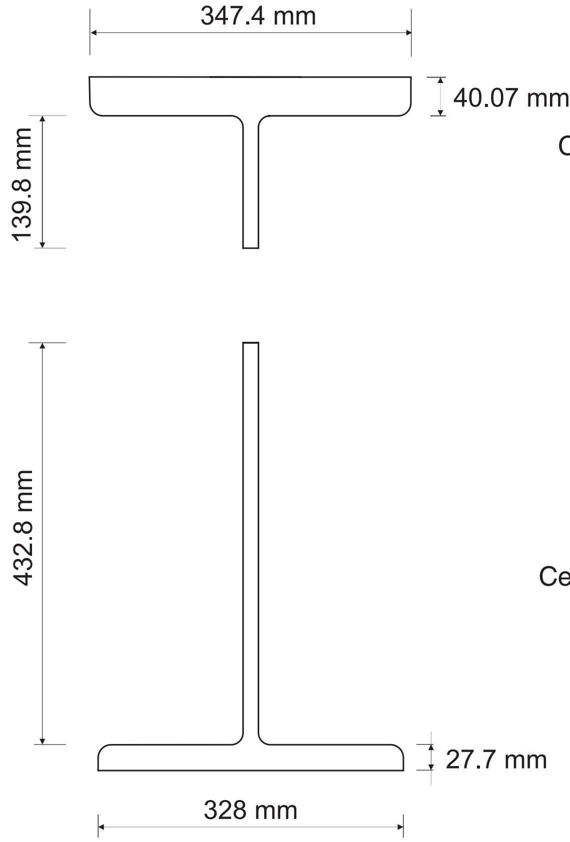
$$13\,920 + 16.5(572.6 - h_2) = 9086 + 16.5h_2$$

$$13\,920 + 9448 - 16.5h_2 = 9086 + 16.5h_2$$

$$h_2 = \frac{23\,368 - 9086}{33} = 432.8 \ mm$$

then $M_p = 350 \times 16\,227 \times \dfrac{(345.4 + 147.0)}{10^6} = 2797 \ kN.m$

$$Z = \frac{2797 \times 10^6}{350} = 7.99 \times 10^6 \ mm^3$$



$$\text{Centroid Top} = \frac{\dfrac{16.5 \times 139.8^2}{2} + 13\,920 \times 159.8}{16\,227} = 147.0 \;\; \text{mm}$$

$$\text{Centroid Bottom} = \frac{\dfrac{16.5 \times 432.8^2}{2} + 9086 \times 446.7}{16\,227} = 345.4 \;\; \text{mm}$$

Figure A9
Centroid of Top and Bottom Flange

12) Calculate Section Properties for Mono-symmetric Analysis

Refer to Galambos (1998), or visit http://www.cisc-icca.ca/structural.html

$$\alpha = \frac{1}{1 + \left[\left(\dfrac{347.4}{328} \right)^3 \times \left(\dfrac{40.07}{27.7} \right) \right]} = 0.3678$$

$$d' = 640.7 - \frac{(40.07 + 27.7)}{2} = 606.8 \quad mm$$

$$J = \frac{(347.4 \times 40.07^3) + (328 \times 27.7^3) + (606.8 \times 16.5^3)}{3} = 10.69 \times 10^6 \quad mm^4$$

$$C_w = \frac{606.8^2 \times 347.4^3 \times 40.07 \times 0.3678}{12} = 19.0 \times 10^{12} \quad mm^6$$

Shear Centre Location

$$y_0 = 279.2 - \frac{40.07}{2} - (0.3678 \times 606.8) = +35.98 \;\; mm$$, therefore above the centroid

$$\rho = \frac{140 \times 10^6}{221.7 \times 10^6} = 0.6315$$

$$\frac{I_y}{I_x} = \frac{221.7 \times 10^6}{2317 \times 10^6} = 0.0957 \quad < \quad 0.5$$

therefore

$$\beta_x = (+1) \times 0.9\left((2 \times 0.6315) - 1\right) \times 606.8\left(1 - 0.0957^2\right) = 142.3 \;\; mm$$

13) Investigate strength of the section in bending

The span is $10.67\ m$ and unbraced laterally. Use the procedure from Reference 8 and Section 5.9, based on SSRC and AISC procedures.

$$M_u = \frac{\pi C_b}{KL}\left[\sqrt{EI_y GJ}\left(B_1 + \sqrt{1 + B_2 + B_1^2}\right)\right]$$

M_u is the unfactored strength for elastic buckling

$$B_1 = \frac{\pi \beta_x}{2KL}\sqrt{\frac{EI_y}{GJ}} \qquad , \qquad B_2 = \frac{\pi^2 EC_w}{(KL)^2 GJ}$$

$K = C_b = 1.0$ (conservative for this application)

$$B_1 = \frac{\pi \times 142.3}{2 \times 1 \times 10\,670}\sqrt{\frac{200\,000 \times 221.7 \times 10^6}{77\,000 \times 10.69 \times 10^6}} = 0.1538 = \frac{1641}{L}$$

$$B_2 = \frac{\pi^2 \times 200\,000 \times 19.0 \times 10^{12}}{10\,670^2 \times 77\,000 \times 10.69 \times 10^6} = 0.4002 = \frac{45.56 \times 10^6}{L^2}$$

$$M_u = \frac{\pi \times 1}{10\,670}\left[\sqrt{200\,000 \times 221.7 \times 10^6 \times 77\,000 \times 10.69 \times 10^6}\left(0.1538 + \sqrt{1 + 0.4002 + 0.1538^2}\right)\right]$$

$$= 2.395 \times 10^9 \;\; N.mm = 2\,395 \;\; kN.m$$

14) Calculate M_i

$M_i = (F_y - F_r)S_{xB}$, where $F_r = 16.5\ ksi = 113\ MPa$ or , $F_y S_{xT}$, which ever is smaller.

$$= \frac{(350 - 113) \times 8.30 \times 10^6}{10^6} = 1967 \;\; kN.m \;\; \text{Governs}$$

or $$\frac{350 \times 6.409 \times 10^6}{10^6} = 2243 \;\; kN.m$$

$$M_i = 1967 \;\; kN.m$$

15) Calculate L_p, the limiting lateral unbraced length for plastic bending capacity

$$L_p = 1.76 r_{yc}\sqrt{\frac{E}{F_y}}$$

$$r_{yc} = \sqrt{\frac{140 \times 10^6}{13920}} = 100.3\ mm$$

$$L_p = 1.76 \times 100.3\sqrt{\frac{200\,000}{350}} = 4220\ mm \quad < \quad 10670$$

16) Determine L_i, the Limiting lateral unbraced length for inelastic buckling

(i.e, iterate until $M_u = M_i$, and that length in M_u is then L_i.)

$$M_u = M_i = 1967 \times 10^6 = \frac{\pi}{L}\left[6.041 \times 10^{12}\left(\frac{1641}{L} + \sqrt{1 + \frac{45.32 \times 10^6}{L^2} + \frac{2.693 \times 10^6}{L^2}}\right)\right]$$

L	M_u (kN.m)
12 500	1935
12 000	2042
12 300	1977 ≈ 1967

Then use $L_i = 12\,300\ mm \quad > \quad 10\,670\ mm$

17) Calculate $\frac{M_r}{\phi}$

$$\frac{M_r}{\phi} = M_p - \left(M_p - M_i\right)\left[\frac{L - L_p}{L_i - L_p}\right]$$

$$= 2797 - \left(2797 - 1967\right)\left[\frac{10\,670 - 4220}{12\,300 - 4220}\right] = 2134\ \ kN.m$$

In accordance with Canadian nomenclature

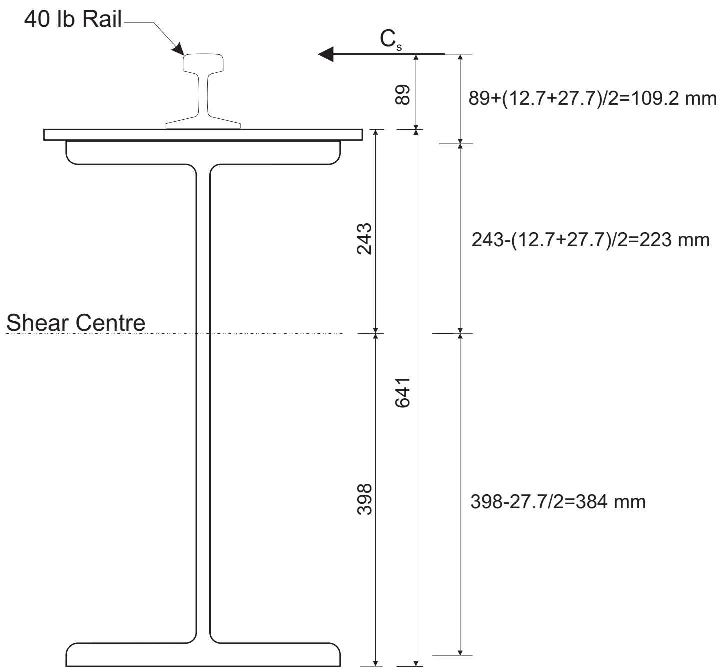
$$M_r = 0.9 \times 2134 = 1921\ \ kN.m$$

18) Calculate distribution of the sidethrust C_s by flexural analogy

See Figures A10 and A11.

Moment at Shear Centre $= C_s\left(243 + 89\right) = 332\,C_s$

Couple, applied to each flange $= \dfrac{332\,C_s}{\left(223 + 384\right)} = 0.5470\,C_s$



Dimensions rounded to the nearest mm

Figure A10
Distribution of Sidethrust

Distribution of horizontal load applied at shear centre, as a simple beam analogy

- to top flange $\dfrac{C_s \times 384}{(223+384)} = 0.6326\,C_s$

- to bottom flange $= 0.3674\,C_s$

M_{fyt}(top flange) $= 1.18 \times 73.19 = 86.36 \;\; kN.m$

M_{fyb}(bottom flange) $= 0.1796 \times 73.19 = 13.15 \;\; kN.m$

$M_{fx} = 1289 \;\; kN.m$

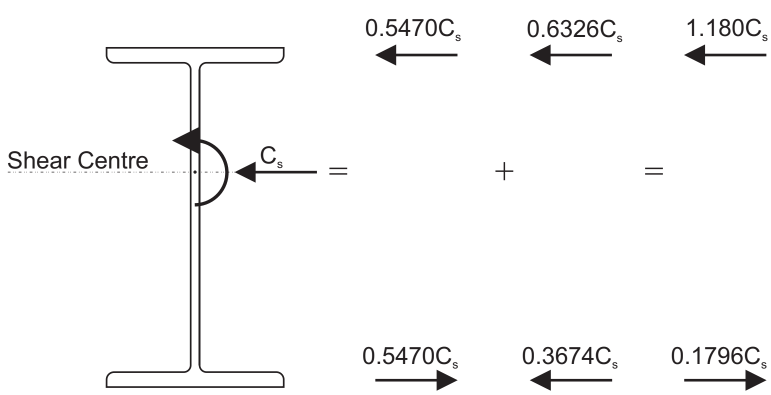


Figure A11
Moments about Shear Centre

19) Check overall member strength

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \quad \leq \quad 1.0$$

$$\frac{1289}{0.9 \times 2800} + \frac{86.36}{0.9 \times 422} = 0.511 + 0.227 = 0.739 \quad < \quad 1.0 \quad \text{OK}$$

20) Check stability (Lateral torsional buckling)

$$\frac{1289}{0.9 \times 1967} + \frac{86.36}{0.9 \times 422} = 0.728 + 0.227 = 0.955 \quad < \quad 1.0 \quad \text{OK}$$

No further checks are required (see Section 5.10)

Conclusion: Section is adequate in bending

Design Example 2
Illustration of Design of a Heavy Duty Plate Girder Type Crane Runway Beam

Design Criteria	Value/Units
Codes and Standards	CSA S16-01
Importance (see NBCC 2005)	N.A.
Life of the Structure	N.A.
Materials (Plates, Shapes, Fasteners, etc.)	CSA G40.21 Grade 350W
Span	15 240 *mm*
Provision for Future Expansion?	N.A.
Simple Span?	Yes
Lateral Support for Top Flange?	Yes
Top of Rail Elevation, or height from main Floor	N.A.
Required Clearances to U/S Beam	N.A.
Side Thrust Equally Distributed Both Sides of Runway?	Yes
Number of Cranes, Each Runway	2 identical cranes
Collector Rail Mounting Details	N.A.
Design for Future Additional Cranes	N.A.
Jib Cranes, or Provision for Jib Cranes	No
Design for Future Upgrades	N.A.
Class of Cranes	CMAA Class D
Service (Description)	Heavy
Type of Duty (see table 2.1)	Steel Mill, cab operated or radio controlled
Crane Hook Capacity # hook(s) each Capacity each hook	1 45 tonnes
Weight of Crane Bridge	106 600 *kg**

Design Criteria		Value/Units
Weight of Crane Trolley		29 500 *kg**
Bridge Wheels per Rail	Total Number Driven	4 2
Bridge Wheel Spacing		See Figure A12
Minimum Distance Between Wheels of Cranes in Tandem		3 658 *mm*
Maximum Wheel Load, Each Crane (not including impact)		276 *kN*
Crane Rail	Description Self Load	Bethlehem 135 *lb/yd* 0.657 *kN/m*
Rail Joints (bolted or welded)		Yes
Resilient Pad Under Rail?		Yes
Bridge Speed		1.5 *m/sec*
Type of Bumpers		N.A.
Bumpers Supplied with Crane?		N.A.
Bumper Forced on Runway end Stop (Ultimate Load)		N.A.
Fatigue Criteria: Vertical - Equivalent passes on one crane, maximum wheel loads Horizontal - Equivalent cycles of side thrust at 50% of maximum side thrust		1 000 000 500 000
Deflection Criteria: Vertical Limit (one crane, not including impact) Horizontal Limit		Span/800 Span/400
Impact Criteria: Percentage of maximum wheel loads, one crane only		25%
Other Considerations		Use elastomeric pad under rail. First two axles of each crane are driven.
*Weight Certified?		No

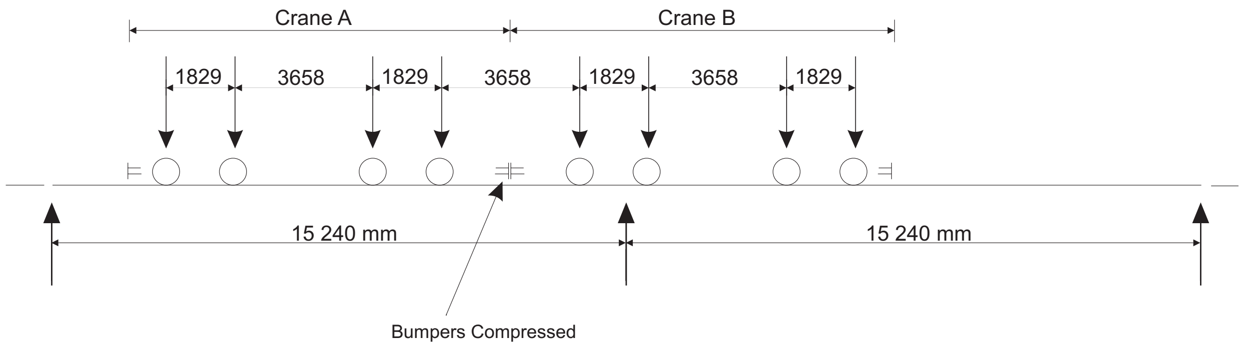


Figure A12
Wheel Configuration - Two Cranes

Design Data

A preliminary analysis shows that a moment of inertia in the strong axis of approximately 15×10^9 mm^4 will be required. A computerized moving load analysis for one and two cranes using $I = 14.5 \times 10^9$ mm^4 yields the following results:

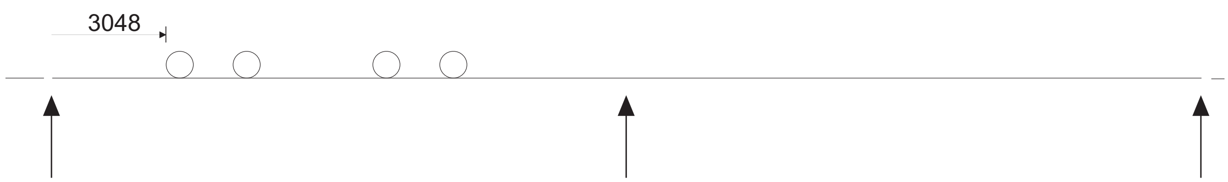


Figure A13
Wheel Location - One Crane

M_{max}, 1 Crane, no Impact
M_{LL}, specified $= 2751\, kN.m$
V_{LL}, specified $= 839.0\, kN$

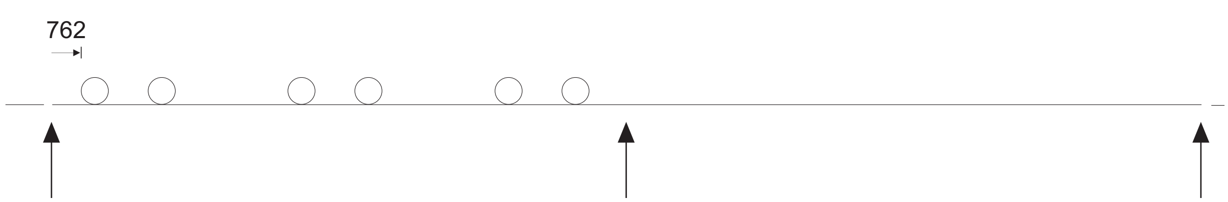


Figure A14
Wheel Location - Two Cranes

M_{max}, 2 Crane, no Impact
M_{LL}, specified $= 3051\, kN.m$
V_{LL}, specified $= 960.5\, kN$

From the crane data provided, moments and shears for one crane without impact are as follows.

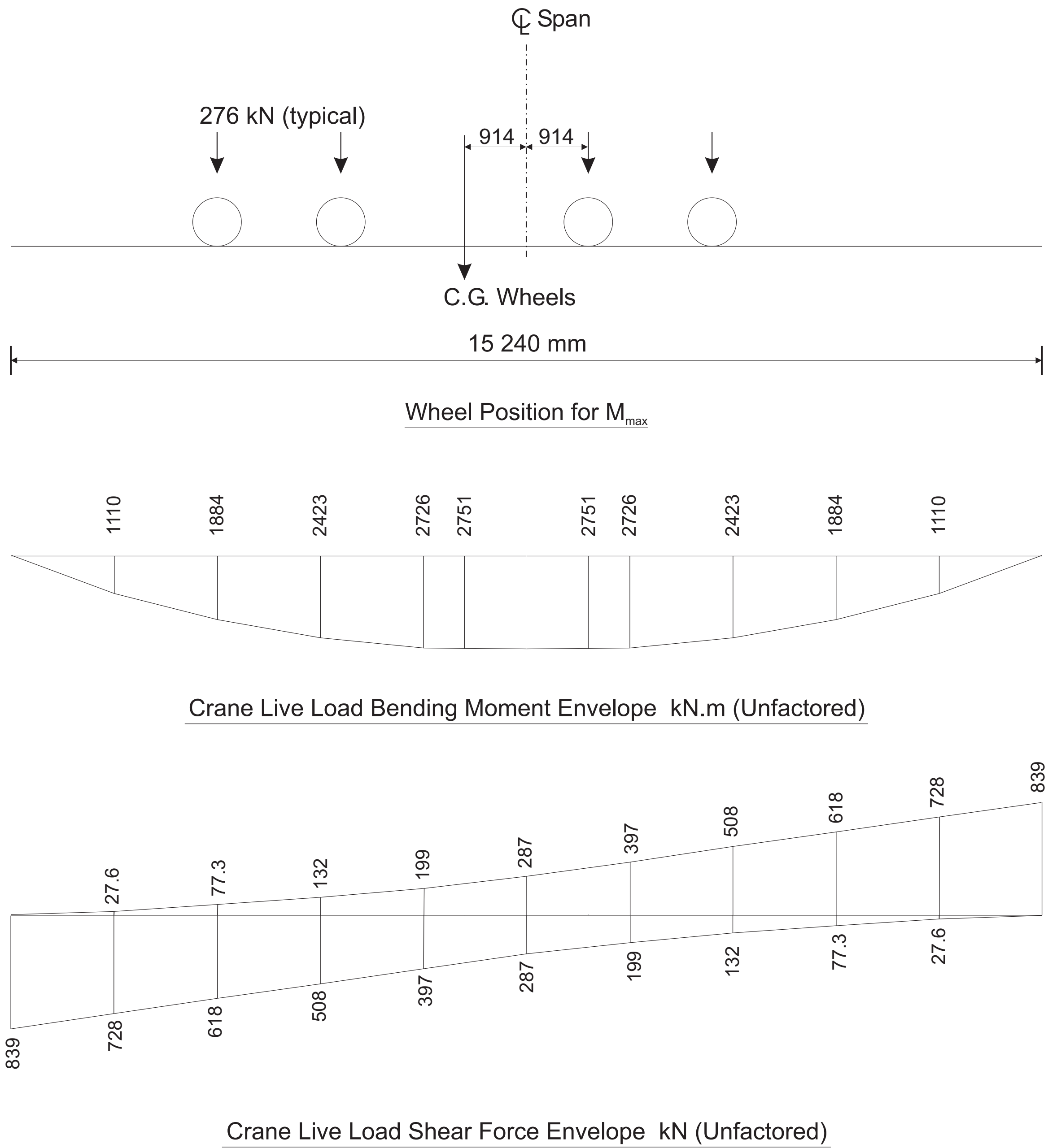


Figure A15
Bending Moments and Shears - One Crane

Moments and shears for two cranes in tandem, bumpers compressed, without impact, are as follows.

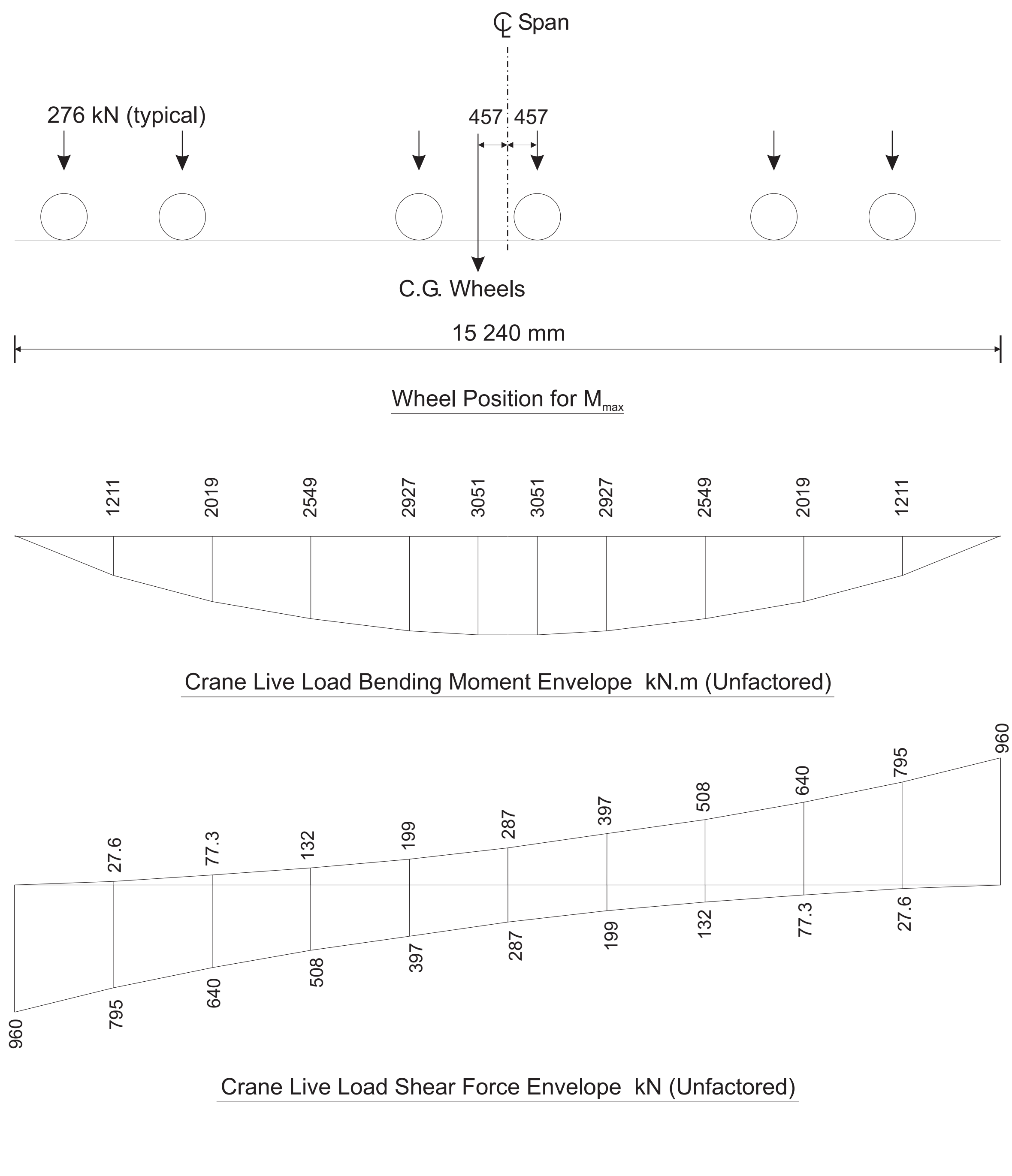


Figure A16
Bending Moments and Shears - Two Cranes

Consider the Forces from Traction, C_{ls} (One crane only)

Wheels are positioned for $M_{\max}$.

Criterion for C_{ls} is 20% of the load in the driven wheels. For worst case, assume all horizontal load resisted at RHS (RH)

C_{ls}, specified load $= 0.2 \times 2 \times 276 = 110.4\ kN$

$$R_R = R_L = \frac{110.4 \times 1.646}{15.24} = 11.92\ kN$$

The maximum (+) moment M_r will occur under the same wheel as for gravity loads $= 11.92 \times 8.534 = 101.7\ kN$

Note: Axial load is not significant for this section and will not be considered further in this example.

1) Calculate Side Thrust

Refer to Section 2.3.1 and Table 2.1 for cranes of type "Cab Operated or Radio Controlled". Total Side Thrust for one crane is the greatest of:

- 40% of lifted load $0.4 \times 45 \times 9.81 = 176.6\ kN$
- 20% of (lifted load + trolley) $0.2 \times (45 + 29.5) \times 9.81 = 146.2\ kN$
- 10% of (lifted load +crane weight) $0.1 \times (45 + 136.1) \times 9.81 = 177.7\ kN$ Governs

Stiffness in the direction of sidethrust is the same on both sides of the runway, therefor the maximum value, 177.7 kN will be distributed equally to each side,

$$\frac{177.7}{2 \times 4} = 22.21\ kN \text{ per wheel}$$

Therefore moments and shears due to sidethrust will be $\frac{22.21}{276} = 0.0805$ times the vertical wheel load moments and shears.

Summary Table - Unfactored Live Load Bending Moment and Shear Summary

		Moments ($kN.m$)	Shears (kN)					
			at End	*at 1524*	*at 3048*	*at 4572*	*at 6096*	*at 7620*
One Crane	*Live Load*	2751	839.0	728.0	618.0	508.0	397.0	287.0
	Impact	687.8	209.8	182.0	154.5	127.0	99.25	71.75
	Sidethrust	221.5	67.54					
	Traction	101.7	11.92					
Two Cranes	*Live Load*	3051	960.0	795.0	640.0	508.0	397.0	287.0
	Impact	–	–	–	–	–	–	–
	Sidethrust	221.5	67.54					
	Traction	101.7	11.92					

Note that in the above summary for two cranes, the values for sidethrust will be slightly conservative because the maximum values for a single crane positioned for maximum effects were used. If a rigorous approach is used, the designer may be faced with a formidable number of possibilities for the critical combination. From the summary table, one crane will govern for strength calculations.

Investigate Deflection due to Live Load

From a computerized moving load analysis using $I_{xx} = 14.5 \times 10^9$ mm^4, maximum deflection due to live load not including impact, is as follows. Deflection for two cranes is shown for information only.

	Maximum Deflection, *mm*	Span/800, *mm*
One crane	23.6	19.1
Two cranes	25.8	19.1

Therefore, for deflection $\leq$ 19.1 mm,

$$I_{xx} \text{ minimum} = \frac{23.6}{19.1} \times 14.5 \times 10^9 = 17.9 \times 10^9 \quad mm^4$$

Pick Trial Section

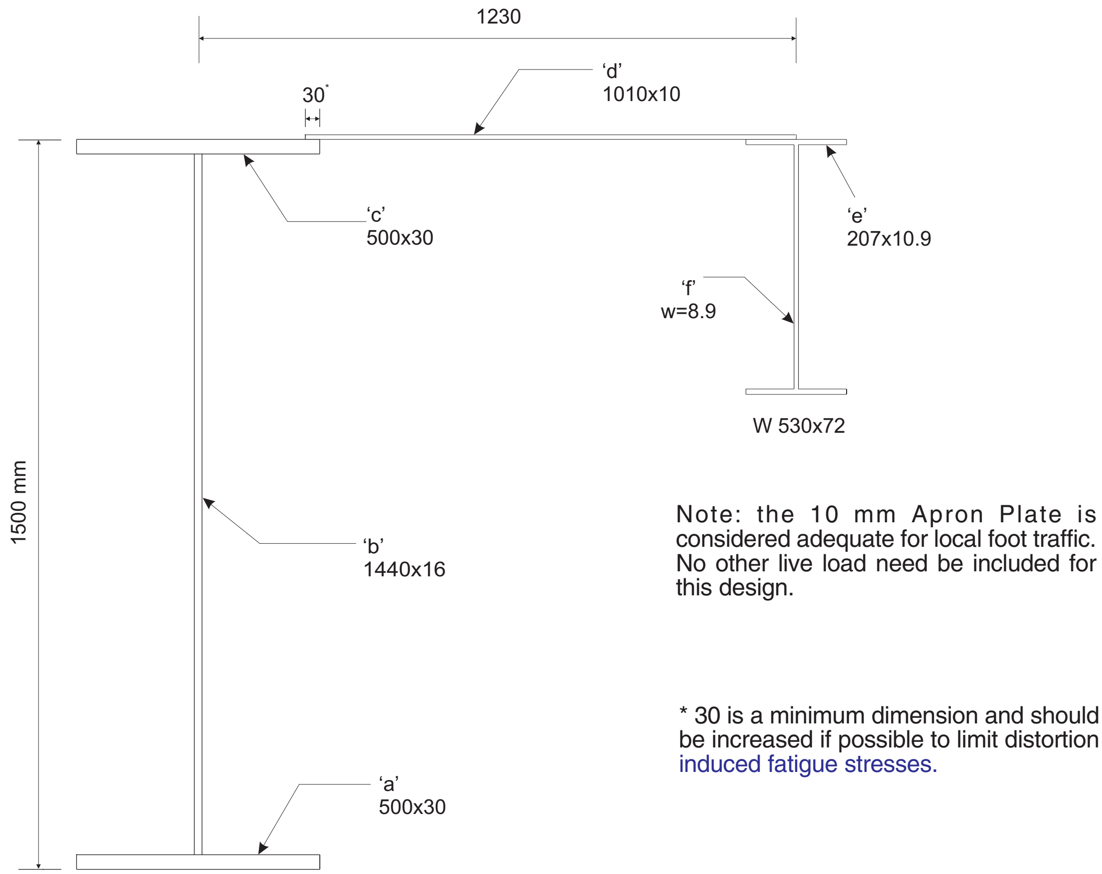


Figure A17
Trial Section

Class of Section for Bending x-x, Grade of Steel 350*W*

Flanges	$\frac{b}{t} = \frac{250}{30} = 8.33 \quad > \quad \frac{145}{\sqrt{F_y}}$	Class2
Web	$\frac{h}{w} = \frac{1440}{16} = 90 \quad > \quad \frac{1700}{\sqrt{F_y}}$ $< \quad \frac{83\,000}{F_y}$	Class2 OK (14.3.1)

However, since the composite section, including a portion of the apron plate, will not have an axis of symmetry in the plane of bending (see S16-01, Clause 11.1.3), the section will be considered Class 3.

Therefore, in accordance with S16-01, Clause 11.2, Table 2;

Projecting flanges	$\frac{b}{t} \leq \frac{200}{\sqrt{F_y}}$	$= 10.69$
Stems of Tee Sections	$\frac{b}{t} \leq \frac{340}{\sqrt{F_y}}$	$= 18.17$
Webs	$\frac{h}{w} \leq \frac{1900}{\sqrt{F_y}}$	$= 101.6$

The maximum slenderness ratio of a web $\not> \frac{83\,000}{F_y} = 237.1$ (Clause 14.3.1)

If the web slenderness ratio $\frac{h}{w} > \frac{1900}{\sqrt{\frac{M_f}{\phi S}}}$,

then the moment resistance must be required in accordance with clause 14.3.4

Consider Eccentricity of Loads Due to Sidethrust (C_{ss}) in the Horizontal Direction

$$H_T = C_{ss} \times \frac{15146}{15005} = 1.0094 C_{ss}$$

$$H_B = 0.0094 C_{ss}$$

Referring to the moments due to sidethrust, increase the bending moments in the horizontal beam by a factor of 1.0094, and apply a bending moment to the bottom flange of the plate girder $= 0.0094$ times the calculated lateral moment due to sidethrust. The bending moment y-y in the bottom flange is very small and will be neglected for this sample.

Note: Resilient pad not included above. Effect is small and can be neglected.

Calculate Section Properties x-x

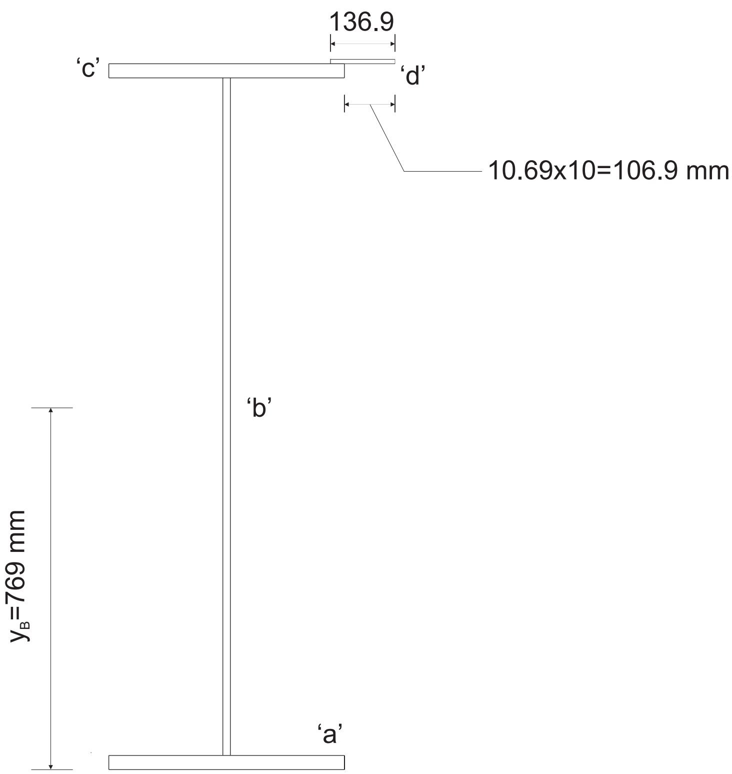


Figure A18
Section Properties of Girder with Apron Plate, About x-x Axis

Element	A (mm^2)	y_b (mm)	Ay_b $(10^3\ mm^3)$	Ay_b^2 $(10^6\ mm^4)$	I_0 $(10^6\ mm^4)$
a	15 000	15	225	3.4	-
b	23 040	750	17 280	12 960	3 981
c	15 000	1 485	22 280	33 080	-
d	1 369	1 505	2 060	3 101	-
$\sum$	54 410		41 850	49 140	3 981

$$y_B = \frac{\sum Ay_b}{\sum A} = \frac{41850 \times 10^3}{54\,410} = 769.2\ mm \quad \text{and} \quad y_T = 1510 - 769.2 = 740.8\ mm$$

$$I_{xx} = \sum I_0 + \sum A y_b^2 - y_B^2 \sum A$$
$$= 3\,981 \times 10^6 + 49\,140 \times 10^6 - 54\,410 (769.2)^2$$
$$= 20\,940 \times 10^6 \quad mm^4 \quad > \quad 17\,900 \times 10^6$$

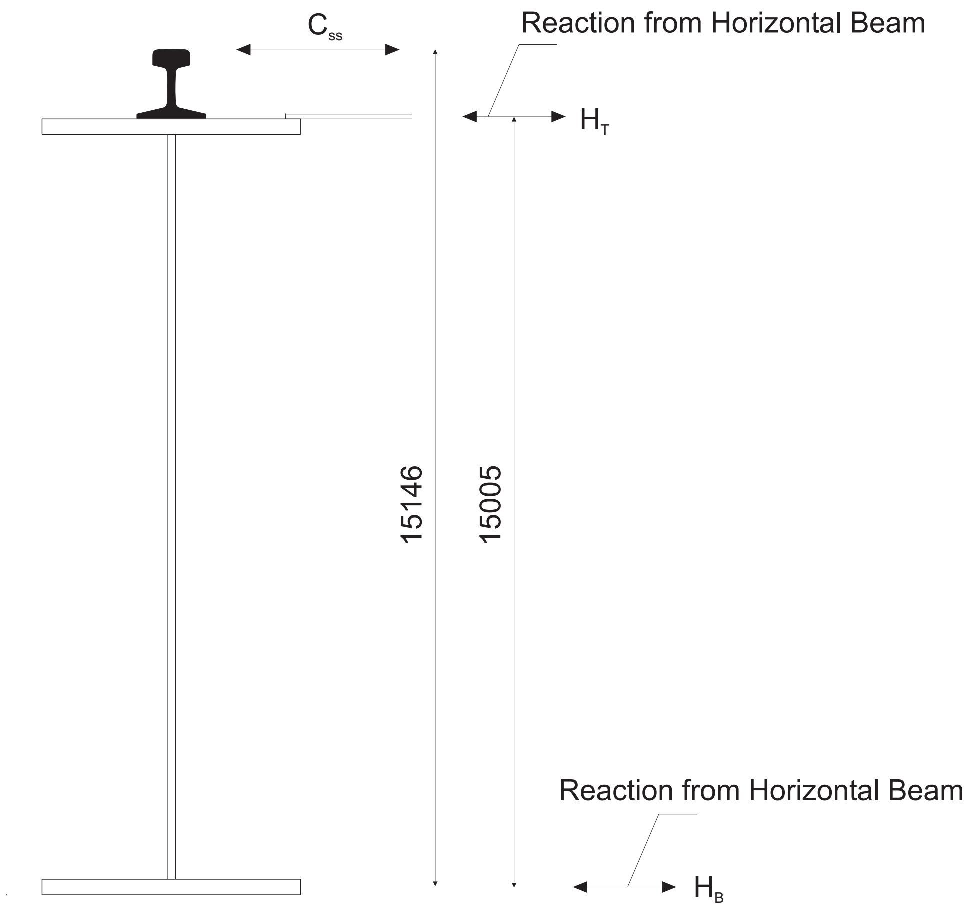


Figure A19
Sidethrust

Therefore vertical deflection due to crane load will be less than $\frac{\text{span}}{800}$ and will be $\frac{1.79}{2.094} \times 19.1 = 16.3\ mm$

$$S_{x_{Bottom}} = \frac{I_{xx}}{y_B} = \frac{20\,940 \times 10^6}{769.2} = 27\,220 \times 10^3 \quad mm^3$$

$$S_{x_{Top}} = \frac{I_{xx}}{y_T} = \frac{20\,940 \times 10^6}{740.8} = 28\,270 \times 10^3 \quad mm^3$$

Calculate Section Properties y-y

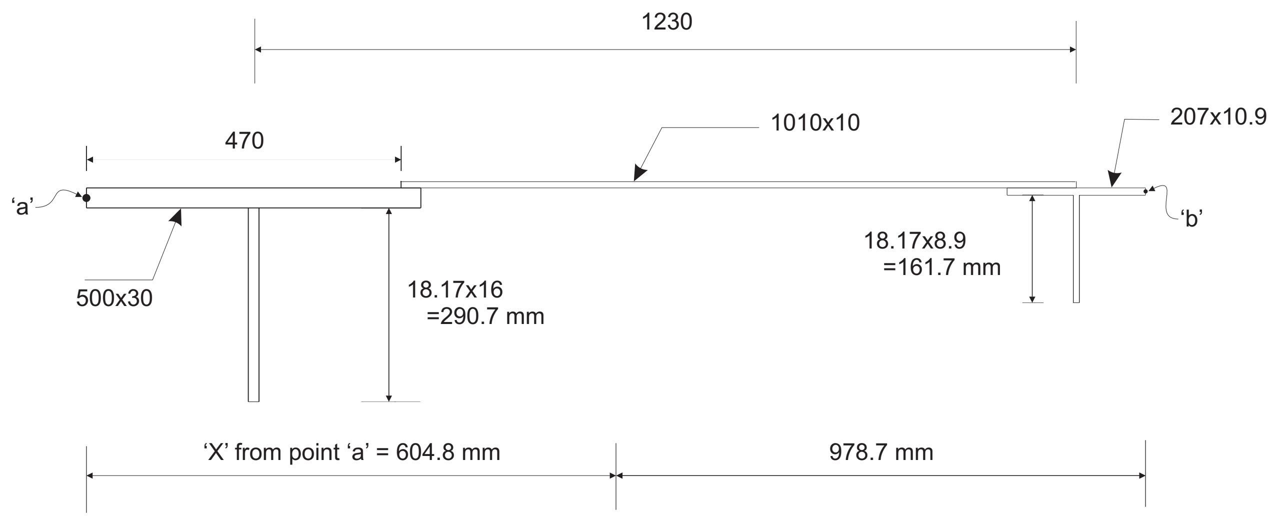


Figure A20
Section Properties of Girder with Apron Plate, About y-y Axis

Element	A (mm^2)	$x_{'a'}$ (mm)	$Ax_{'a'}$ $(10^3\ mm^3)$	$Ax^2_{'a'}$ $(10^6\ mm^4)$	I_0 $(10^6\ mm^4)$
500 × 30	15 000	250	3 750	938	313
290.7 × 16	4 651	250	1 163	290.8	-
1 010 × 10	10 100	975	9 848	9 602	859
207 × 10.9	2 256	1 480	3 339	4 942	81
161.7 × 8.9	1 439	1 480	2 130	3 152	-
$\sum$	33 450		20 230	18 920	1 253

$$x = \frac{\sum Ax_{'a'}}{\sum A} = \frac{20\,230 \times 10^3}{33\,450} = 604.8\ mm \quad \text{and} \quad x' = 1583.5 - 604.8 = 978.7\ mm$$

$$I_{yy} = \sum I_0 + \sum Ax^2_{'a'} - x^2 \sum A$$

$$= 1253 \times 10^6 + 18\,920 \times 10^6 - 33\,450 \left(769.2\right)^2$$

$$= 7\,945 \times 10^6\ mm^4$$

$$S_{y'a'} = \frac{I_{yy}}{x} = \frac{7\,945 \times 10^6}{604.8} = 13\,140 \times 10^3\ mm^3$$

$$S_{y'b'} = \frac{I_{yy}}{x'} = \frac{7\,945 \times 10^6}{978.7} = 8118 \times 10^3 \quad mm^3$$

Calculate Lateral Deflection due to Sidethrust

$$= \frac{0.0805 \times 20.94 \times 10^9 \times 16.3}{7.945 \times 10^9} = 3.5 \quad mm$$

$$\frac{Span}{400} = \frac{15\,240}{400} = 38.1\, mm \quad > \quad 3.5\, mm \quad \text{OK}$$

Calculate Factored Moment Resistance $M_{rx} = \phi S_x F_y$ (Clause 13.5)

at top flange $= 0.9 \times 28.27 \times 10^6 \times 350 \times 10^{-6} = 8905 \quad kN.m$

at bottom flange $= \dfrac{2.722}{2.783} \times 8905 = 8710 \quad kN.m$

Calculate Factored Moment Resistance $M_{ry} = \phi S_y F_y$ (Clause 13.5)

at rail side $= 0.9 \times 13.14 \times 10^6 \times 350 \times 10^{-6} = 4139 \quad kN.m$

at back side $= \dfrac{8.118}{1.314} \times 4139 = 25\,572 \quad kN.m$

Check for Reduction in Moment Resistance M_{rx} due to a slender web (14.3.4)

Factored Moment M_{fx} is approximately $(1.2 \times 200) + (1.5 \times 3500) = 5490 \quad kN.m$

then $\dfrac{1900}{\sqrt{\dfrac{M_{fx}}{\phi S_x}}}\ (\text{min}) = \dfrac{1900}{\sqrt{\dfrac{5490 \times 10^6}{0.9 \times 27.22 \times 10^6}}} = 126.9 \quad > \quad 87.5 \quad \text{OK}$

Check for Reduction in Moment Resistance M_{ry} due to a slender web

Factored Moment M_{fy} is approximately $1.5 \times 221.5 = 332 \quad kN.m$

then $\dfrac{1900}{\sqrt{\dfrac{M_{fy}}{\phi S_y}}}\ (\text{min}) = \dfrac{1900}{\sqrt{\dfrac{332 \times 10^6}{0.9 \times 8.118 \times 10^6}}} = 281 \quad > \quad \dfrac{876}{10} \quad \text{OK}$

Calculate Shear Capacity of the Unstiffened Plate Girder Web (Clause 13.4)

$$V_{rf} = \phi A_w F_s$$

F_s is calculated in accordance with the web slenderness ratio $\dfrac{h}{w}$

Go to the CISC Handbook of Steel Construction, where the factored ultimate shear stress ϕF_s is given for girder webs.

For Grade 350, $\dfrac{h}{w} = 90$, no intermediate stiffeners $\phi F_s = 106 \quad MPa$

then $V_{rf} = \dfrac{106 \times 1440 \times 16}{1000} = 2442 \quad kN$

Check for Possibility of a Thinner Web

Capacity seems to be more than adequate, try 12 mm plate (flanges will have to be increased to maintain $I_{x\,\min}$), $\frac{h}{w} = \frac{1440}{12} = 120 \quad > \quad \frac{1900}{\sqrt{F_y}}$, therefore bending strength is calculated (S16-01, Clause 14).

From CISC Handbook $\phi F_s = 60 \;\; MPa,$ therefore $V_{rf} = \frac{60 \times 1440 \times 12}{1000} = 1036 \;\; kN$

Factored Shear Force $\approx 1.5\left(839 + 209 + 11.9\right) = 1590 \;\; kN$ plus Dead load > 1036, therefore stiffeners would be required.

Calculate Dead Load Supported by the Plate Girder

Section	Area, $mm^2 \times 0.785$	$= kg/m \times 0.00981$	$= kN/m$
Plate Girder	53.04×10^3	416.3	
50% of Apron Plate	5175	40.6	
$135^{\#}$ Rail		66.96	
Misc. (allowance)		50.0	
$\sum$		573.8	$5.629 \;\; kN/m$

Calculate the Unfactored Bending Moment M_x due to Dead Load

$$= 5.629 \times \frac{15.240^2}{8} = 163.4 \;\; kN \cdot m$$

Calculate the Unfactored Maximum Bending Moment M_x due to Live Loads

$$= 2751 + 687.8 + 101.7 = 3541 \;\; kN.m$$

Calculate the Unfactored Maximum Bending Moment M_y due to Live Loads (sidethrust)

$$= 1.0094^{*} \times 221.5 = 223.6 \;\; kN.m$$

* Amplified due to eccentricity in the vertical direction

Calculate M_{fx}

$M_{fx} = \left(1.25 \times 163.4\right) + \left(1.5 \times 3541\right) = 5\,516 \;\; kN.m$ (see previous calculations)

If the unloaded crane has been weighed (C_{DL}) and knowing the lifted load (C_{LL}), the factored vertical crane load would be $1.25C_{DL} + 1.5C_{LL}$.

Calculate M_{fy} at Top

$$M_{fy} = 1.5 \times 223.6 = 335.4 \;\; kN.m$$

Calculate M_{fy} at Bottom

$$M_{fy} = 1.5 \times 0.0094 \times 221.5 = 3.13 \ \ kN.m$$

Check Trial Section for Biaxial Bending, Top corner, Rail Side.

This is the Yielding Limit State (Strength) Check.

$$\frac{M_{fx}}{M_{rx}} + \frac{M_{fy}}{M_{ry}} \quad \leq \quad 1.0$$

$$\frac{5516}{8905} + \frac{335.4}{4139} = 0.691 + 0.081 = 0.722 \quad < 1.0 \quad \text{OK}$$

Check for Lateral Torsional Buckling

Limit State (Stability) is not required because the section is laterally supported by the horizontal beam.

Check for Bending Strength Top Corner, Back Side

$$\frac{M_{fy}}{M_{ry}} \quad \leq \quad 1.0$$

$$\frac{335.4}{2\,557.2} = 0.131 \quad < 1.0 \quad \text{OK}$$

Check for M_{fx} and M_{fy} in Bottom Flange

OK by inspection

Calculate Factored Shear in the Vertical Direction

$$= \left(1.25 \times 5.629 \times \frac{15.24}{2}\right) + 1.5\left(839.0 + 209.8 + 11.92\right)$$

$$= 53.61 + 1591 = 1665 \ \ kN$$

Check Shear Strength in the Vertical Direction

$$\frac{1665}{2442} = 0.682 \quad < \quad 1.0 \qquad \text{OK}$$

A check for combined bending moment and shear is not required because the section is not transversely stiffened.
See S16-01, Clause 14.6.

Check Local Wheel Support

(a) Check Web Crippling and Yielding (Clause 14.3.2)

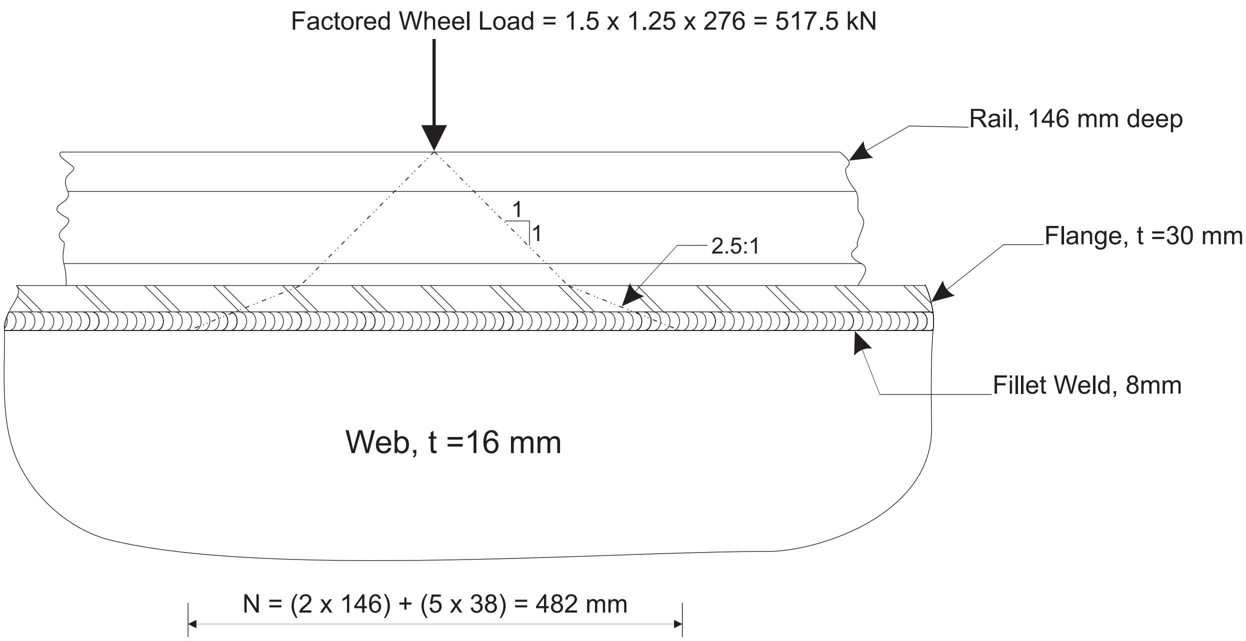


Figure A21
Web Crippling Under Crane Wheel

Check Interior

$$(i) \quad B_r = 0.8 \times 16 \left(482 + 300\right) \frac{350}{1000} = 3\,503 \ kN \qquad 14.3.2(I)$$

$$(ii) \quad B_r = 1.45 \times \frac{0.8 \times 16^2}{1000} \sqrt{350 \times 200000} = 2485 \ kN \qquad 14.3.2(ii) \qquad \text{Governs}$$

the factored resistance of $2485 \ kN \ > \ 517.5 \ kN$ OK

A check at the ends is not necessary because bearing stiffeners will be used.

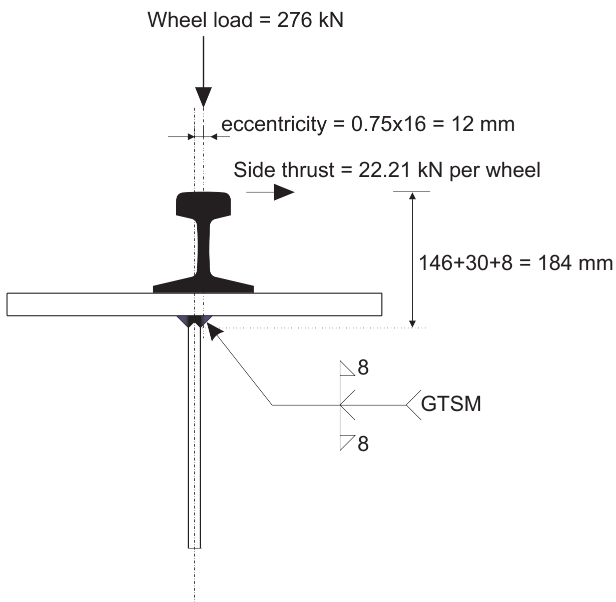
(b) Check torsional effects on web under a wheel load including for rail eccentricity and sidethrust.

Factored Vertical Load $= 1.5 \times 1.25 \times 276 = 517.5 \ kN$, including impact

Factored moment due to eccentricity $= 1.5 \times 1.25 \times 276 \times \frac{12}{1000} = 6.21 \ kN.m$

Factored moment due to sidethrust $= 1.5 \times 22.21 \times \frac{184}{1000} = 6.13 \ kN.m$

$$M_f = 6.21 + 6.13 = 12.34 \ kN.m$$



Note: The procedure below is conservative, neglecting torsional restraint provided by the rail and flange. Refer to Reference 1 for information on a more exact method established by Cornel University. Australian Standard A5 1418.18-2001 also includes a procedure using limit states methods.

Figure A22
Stability and Strength of Web Under Combined Loads

For length of web $= 482\ mm$, as previously calculated

$$Z = \frac{bd^2}{4} = \frac{482 \times 16^2}{4} = 30\,848\ mm^3$$

$$M_r = 0.9 \times 30\,848 \times 350 \times 10^{-6} = 9.717\ kN \quad < \quad 12.34\ kN.m, \text{ No Good}$$

Since the torsional resistance of the rail and flange was not included in the above approximation, check using a more exacting method such as the Australian Standard A5 1418.18. Using this method:

$$\text{Factored bending moment} = 15\,000\quad N \cdot mm/mm\ length\ of\ weld$$

$$\text{Factored resistance} = 0.9 \times \frac{16^2}{4} \times 350 = 20160\quad N \cdot mm/mm\ length\ of\ weld \qquad \therefore \text{OK}$$

No need to check at ends because bearing stiffeners have been used.

Design Bearing Stiffeners

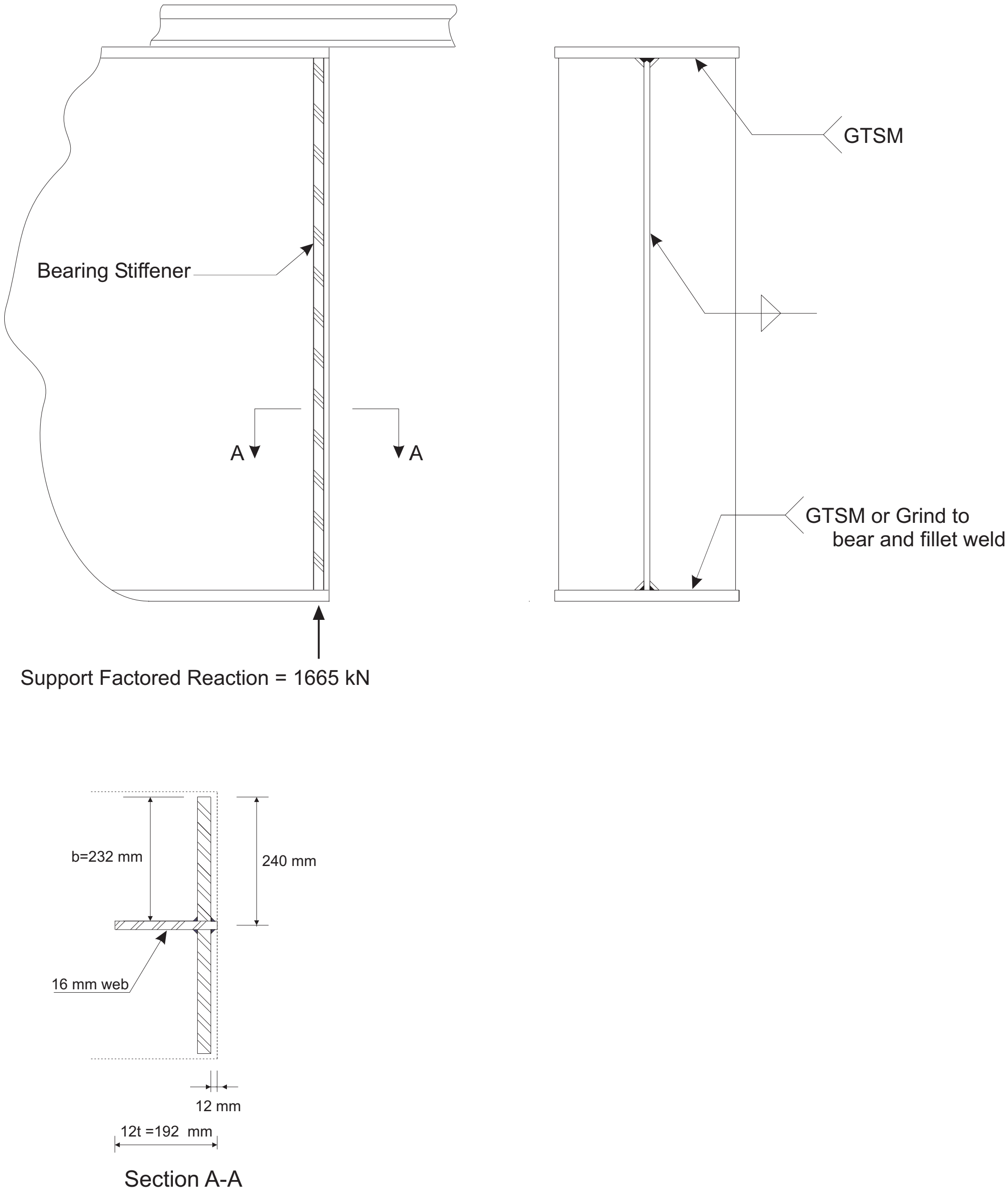


Figure A23
Bearing Stiffeners

For stiffeners, $\frac{b}{t} \quad \leq \quad \frac{200}{\sqrt{F_y}} = 10.69$ Clause 11.2

therefore minimum $t = \frac{232}{10.69} = 21.7 \quad mm$

Try 25 *mm* thick stiffeners

Check column action

$$A = \left(2 \times 232 \times 25\right) + \left(16 \times 192\right) = 14672 \ \ mm^2$$

$$I = 25 \times \frac{480^3}{12} + \frac{\left(192 - 25\right) \times 16^3}{12} = 230.5 \times 10^6 \ \ mm^4$$

$$r = \sqrt{\frac{230.5 \times 10^6}{14672}} = 125.3 \ \ mm$$

$L = \dfrac{3}{4}$ of the length of the stiffeners

$$= 0.75 \times 1440 = 1080 \ \ mm$$

$$\frac{KL}{r} = \frac{1 \times 1080}{125.3} = 8.61$$

Using Table 4-4 of the CISC Handbook, the factored resistance for 350 MPa stiffeners is

$$314 \times \frac{14672}{1000} = 4607 \ \ kN > 1665 \ \ kN \qquad \text{OK}$$

***Check Bearing* (Clause 13.10)**

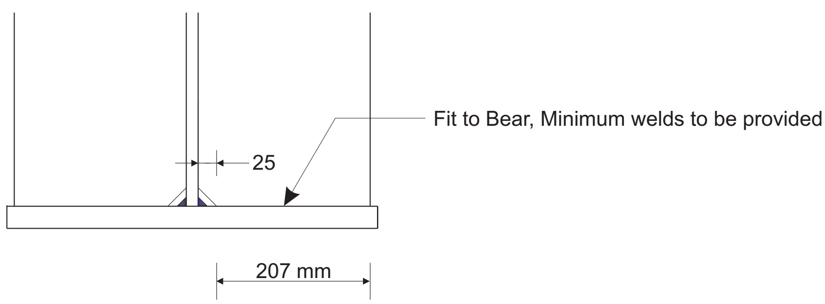


Figure A24
Bearing of Bearing Stiffener

Check one side

Factored load $= \dfrac{1665}{2} = 832.5 \ \ kN$

Clause 28.5 states that at least 75% of the area must be in contact. To guard against fillet welds supporting the load, check for $0.75 \times 207 = 155 \ \ mm$ in contact.

The factored bearing resistance, to clause 13.10

$$= 1.5 \times 0.9 \times 350 \times \frac{155}{1000} \times 25 = 1831 \, kN \quad > \quad 832.5 \qquad \text{OK}$$

Design welds to web

Factored load per weld $= \dfrac{1665}{2 \times 1350\,(say)} = 0.617 \ \ kN/mm$

From Table 3-24, CISC Handbook, need 5 mm for strength, use minimum $= 8 \ mm$ (50% loaded)

Design Bottom Flange Filled Welds For Strength

Maximum Factored Shear $V_{fx} = 1665\ kN$

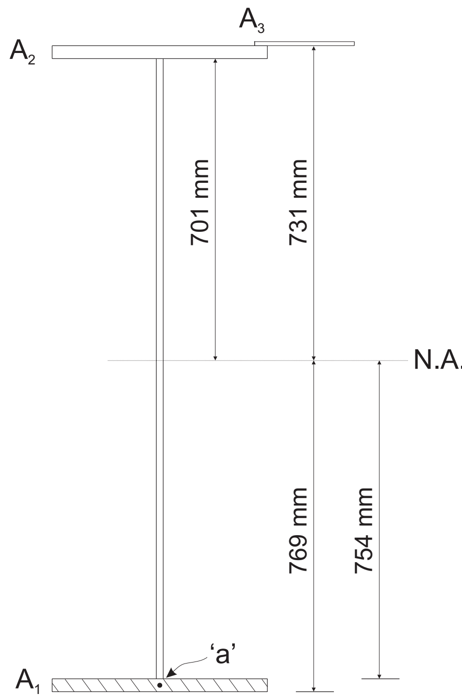


Figure A25
Factored Shear Flow at Web-to-Flange Junction 'a'

Calculate Shear Flow $\frac{VA\bar{y}}{I}$

Factored shear flow at web-to-flange junction 'a'

$$= \frac{1665 \times 10^3 \times 11.31 \times 10^6}{20.94 \times 10^9} = 899.3\ N/mm$$

The minimum fillet weld is $8\ mm$ (Page 6-172 of the CISC Handbook). Using an E49XX electrode and Table 3-24 in the CISC Handbook, the factored shear resistance for a pair of $8\ mm$ fillet welds is

$$2 \times 1.24 = 2.48\ kN/mm \quad > \quad 0.8993 \quad \text{OK}$$

Continuous welds would be used to $\frac{0.899}{2.48} = 0.36$ capacity

Design Upper welds for Strength

Maximum Factored Shear $V_{fy} = 1.5 \times 67.54 = 101.3\ kN$

To design these welds, shear flow from sidethrust is also included

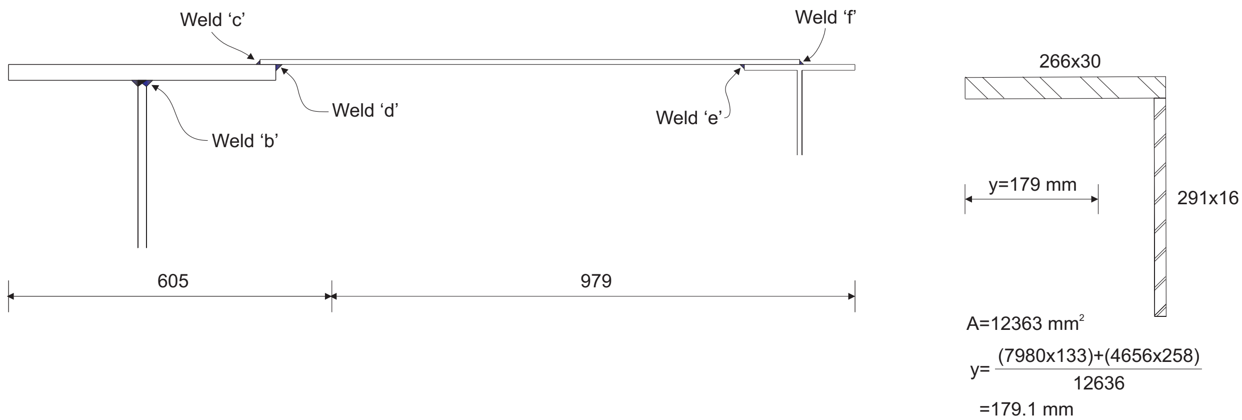


Figure A27
Upper Welds

For weld 'b', $A\bar{y} = 12\,636 \times (605 - 179) = 5.383 \times 10^{6} \quad mm^{3}$

For welds 'e', and 'd' (Calculate 'c', use for both)

$$A\bar{y} = 19\,651 \times (605 - 250) = 6.976 \times 10^{6} \quad mm^{3}$$

For welds 'e' $\quad A = (103.5 \times 10.9) + (161.7 \times 8.9) + (103.5 \times 10) = 1128 + 1439 + 1035 = 3602 \quad mm^{2}$

$$N.A. = \frac{(2567 \times 103.5) + (1035 \times 51.8)}{3602} = 88.6\, mm \quad \text{(118.4 from RHS)}$$

$$A\bar{y} = 3602 \times (979 - 118.4) = 3.10 \times 10^{6} \quad mm^{3}$$

For weld 'f' $\quad A = (103.5 \times 10.9) + (161.7 \times 8.9) = 1128 + 1439 = 2567 \quad mm^{2}$

$$N.A. = \frac{(1128 \times 51.8) + (1439 \times 103.5)}{2567} = 80.8\, mm \quad \text{(from RHS)}$$

$$A\bar{y} = 2567 \times (979 - 80.8) = 2.306 \times 10^{6} \quad mm^{3}$$

For weld 'b'

$$A = 1500 + 1369 = 2869 \quad mm^{2}$$

$$N.A. = \frac{(1500 \times 15) + (1369 \times 35)}{2869} = 24.5\, mm$$

(725.5 from NA of entire section)

$$A\bar{y} = 2869 \times 725.5 = 2.081 \times 10^{6} \quad mm^{3}$$

For weld 'c'

$$A = 1369 \quad mm^{2}$$

$$\bar{y} = 741 - 5 = 736 \quad mm$$

(from NA of entire section)

$$A\bar{y} = 1369 \times 736 = 1.008 \times 10^{6} \quad mm^{3}$$

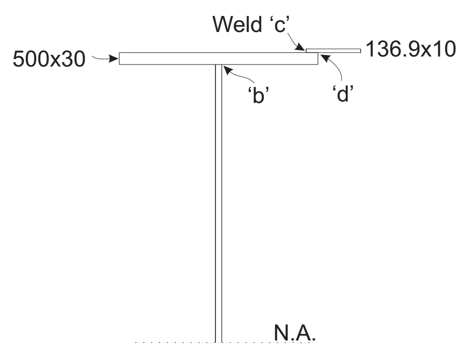


Figure A26
Welds 'b', 'c' and 'd'

Calculate Factored Shear Flows

$$\text{weld 'b'} = \frac{1665\times10^3 \times 2.081\times10^6}{20.94\times10^9} + \frac{101.3\times10^3 \times 5.383\times10^6}{7.945\times10^9}$$

$$= 165.5 + 68.6 = 234.1 \ {}^{N}\!/_{mm} \quad \text{(2 welds)}$$

$$\text{weld 'c' and 'd'} = \frac{1665\times10^3 \times 1.008\times10^6}{20.94\times10^9} + \frac{101.3\times10^3 \times 6.976\times10^6}{7.945\times10^9}$$

$$= 80.1 + 88.9 = 169.0 \ {}^{N}\!/_{mm} \quad \text{(2 welds)}$$

$$\text{weld 'e'} = \frac{101.3\times10^3 \times 3.10\times10^6}{7.945\times10^9} = 39.5 \ {}^{N}\!/_{mm}$$

$$\text{weld 'f'} = \frac{101.3\times10^3 \times 2.306\times10^6}{7.945\times10^9} = 29.4 \ {}^{N}\!/_{mm}$$

For fillet welds, refer to the CISC Handbook, Table 3-24, and Page 6-172.

Weld	Factored Shear Flow, $^{N}\!/_{mm}$			Minimum Fillet, mm	
	x-x	y-y	Combined	Strength	Thickness
a	449.7	–		5 (58%)	8 (36%)
b	82.8	34.3	117.1	5	8
c	40.1	44.5	84.6	5	8 (7%)
d	40.1	44.5	84.6	5	8 (7%)
e	*	39.5	39.5	5 (3%)	5
f	*	29.5	29.4	5 (2%)	5

* No significant gravity loads for purpose of this example. (%) means % of capacity required.

Regarding weld 'a', a complete joint penetration groove weld with reinforcing will be provided. No further evaluation.

Simplify Fatigue Loading

The criterion for vertical loading is 1 000 000 passes of a crane, maximum wheel loads.

The criterion for sidethrust is 500 000 cycles of loading at 50% sidethrust.

Find the level of sidethrust that for 1 000 000 cycles, will cause the same damage.

Fatigue life is inversely proportional to the value of the stress range for values above constant amplitude threshold.* Stress range is proportional to load.

* Does not include consideration of low stress cycles, not significant for these calculations.

$$\frac{life1}{life2} = \left(\frac{load\ range2}{load\ range1}\right)^3 = Load\ Ratio^3$$

then $Load\ Ratio = \sqrt[3]{0.5} = 0.794$

i.e. use $0.794 \times 50\% = 39.7\%$ of specified sidethrust in calculations for strength.

Calculate Fatigue Loads and Stress Ranges

For M_x, criterion is $1\,000\,000$ crane passages, maximum wheel load without impact.

$$M_{x_{specified}} = 2751\ kN.m, \text{ no reversal}$$

$$V_{x_{specified}} = 839\ kN$$

For M_y, criterion is $1\,000\,000$ cycles of sidethrust, including reversal, at $0.397 \times$ *full load*

$$M^{top}_{y_{specified}} = \pm 0.397 \times 223.6 = 87.20\ kN.m$$

$$M^{bottom}_{y_{specified}} = \pm 0.397 \times \frac{3.13}{1.5} = 0.0828\ kN.m$$

$$V_{y_{specified}} = \pm 0.397 \times 67.54 = 26.81\ kN$$

At welded rail clips, check if net tension exists under minimum wheel loads (trolley st other side) and 50% sidethrust. Wheel loads $\approx \frac{106\,600}{8} + \left(\frac{0.1 \times 29\,500}{4}\right) + \left(\frac{0.1 \times 45\,000}{4}\right) = 15187\ kg \quad = 149.0\ kN$

$$f_{sv} = + \frac{\frac{149}{276} \times 2751 \times 10^6}{28.27 \times 10^6} = 52.53\ MPa$$

$$f_{sh} = \pm \frac{87.20 \times 10^6 \times (605 - 100)}{7.945 \times 10^9} = 5.54\ MPa \quad < \quad 52.53 \qquad \underline{\text{No Tension}}, \quad \text{OK}$$

Before proceeding further with a check on base metal, weld details need to be addressed. Referring to strength calculations, intermittent fillet welds would be adequate at welds a, c, d, e and f.

Use of intermittent fillet welds in tension areas is not advisable. These welds should be continuous fillets. Bolted connections would be considered for the apron plate, but welds will be used for purposes of this example. Evaluation for continuous fillet welds of the same size at a, b, c, d, e, f and g.

Calculate Stress Ranges in Base Metal

(+) means tension

base metal at bottom flange $\quad f_{sr} = + \frac{2751 \times 10^6}{27.22 \times 10^6} = +101.1\ MPa$

at 'a' $\quad f_{sr} = + \frac{2751 \times 10^6 \times 739}{20.94 \times 10^9} = +97.09\ MPa$

$$= -0.0$$

at 'b' $\quad f_{sr} = -\left(\frac{2751 \times 10^6 \times 731}{20.94 \times 10^9}\right) \pm \left(\frac{87.20 \times 10^6 \times 355}{7.945 \times 10^9}\right)$

$$= -96.04 \pm 3.90 = -99.94$$

$$= +0.0 \quad (\textit{No Tension})$$

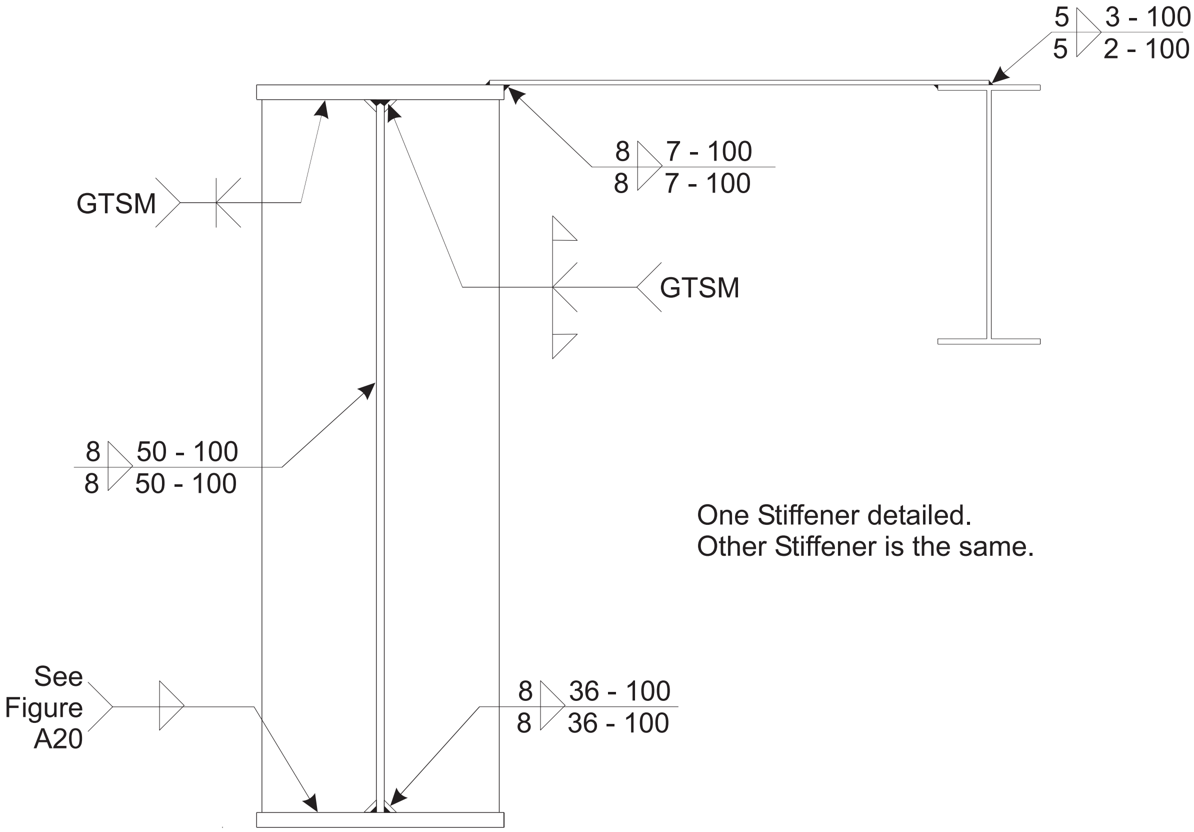


Figure A28
Minimum Welds Required for Factored Loads (Except GTSM weld) Mimimum Effective Welds and Fatigue Considerations not included

at 'c' and 'd'

$$f_{sr} = -\left(\frac{2751\times10^{6}\times731}{20.94\times10^{9}}\right)\pm\left(\frac{87.20\times10^{6}\times175}{7.945\times10^{9}}\right)$$

$$= -96.22 \pm 1.92 = -98.14\ MPa$$

$$= +0.0\ \left(No\ Tension\right)$$

at 'e'

$$f_{sr} = \pm\frac{87.20\times10^{6}\times772}{7.945\times10^{9}} = \pm8.47\ MPa = 16.94\ MPa \qquad \text{(Reversal)}$$

at 'f'

$$f_{sr} = \pm\frac{87.20\times10^{6}\times875}{7.945\times10^{9}} = \pm9.60\ MPa = 19.20\ MPa \qquad \text{(Reversal)}$$

Calculate Ranges of Shear Flow in Weld Metal

at 'a'

$$V_r = \frac{839\times10^{3}\times11.31\times10^{6}}{20.94\times10^{9}} = 453.1\ N/mm$$

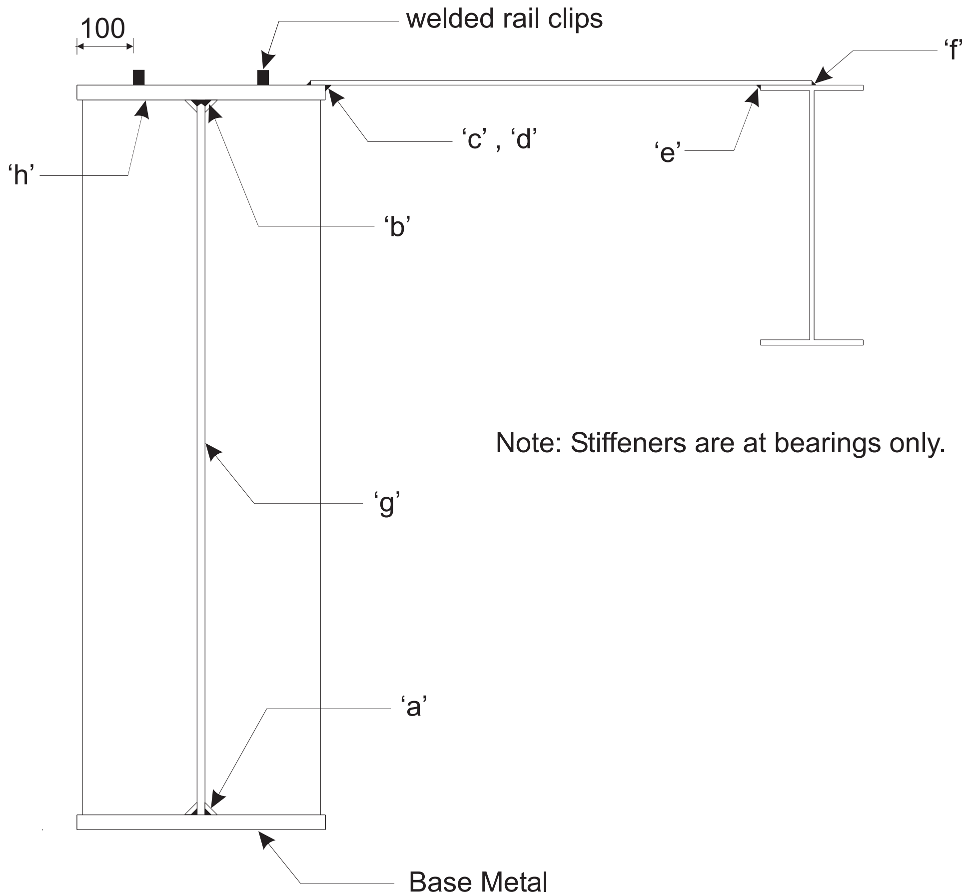


Figure A29
Locations of Fatigue Checks on Cross Section

at 'b'

$$V_r = +\frac{839\times10^3\times2.081\times10^6}{20.94\times10^9} \pm \frac{26.81\times10^3\times5.383\times10^6}{20.18\times10^9}$$

$$= +833.8 \pm 6.94 = +840.74 \ {N}/{mm}$$

$$= -0.0$$

at 'c', 'd'

$$V_r = +\frac{839\times10^3\times1.008\times10^6}{20.94\times10^9} \pm \frac{26.81\times10^3\times6.976\times10^6}{20.18\times10^9}$$

$$= +40.39 \pm 9.27 = +49.66 \ {N}/{mm}$$

$$= -0.0$$

at 'e'

$$V_r = \pm\frac{26.81\times10^3\times3.10\times10^6}{20.18\times10^9} = \pm3.99 = 7.98 \ {N}/{mm}$$

at 'f'

$$V_r = \pm\frac{26.81\times10^3\times2.306\times10^6}{20.18\times10^9} = \pm3.06 = 6.12 \ {N}/{mm}$$

Examine Base Metal

Refer to CSA S16-01, Clause 26, and Tables 9 and 10

Location	**Stress Range** f_{sr} MPa	**Category**	γ MPa	F_{rst} MPa	**Fatigue Life cycles** $nN = \gamma \times f_{sr}^3$	**Comment**
Base metal bottom flange	101.1	A	8190×10^9	165	$> \ 1 \times 10^6$	OK
a	97.1	B	3930×10^9	110	$> \ 1 \times 10^6$	OK
b	no tension	Special Case *				OK
c, d	no tension					OK
e	16.9	B	3930×10^9	110	$> \ 1 \times 10^6$	OK
f	19.2	B	3930×10^9	110	$> \ 1 \times 10^6$	OK

* Detail is subject to 8 repetitions of load with each crane passage ($nN \approx 8\ 000\ 000$ cycles). There is no category but this type of weld detail is known to provide satisfactorily service.

Examine Weld Metal

Location	**Weld Size** mm	**Throat Area** mm^2/mm	**Stress Range** f_{sr} MPa	**Category**	γ MPa	F_{rst} MPa	$nN = \gamma \times f_{sr}^3$	**Comment**
a	8	5.656	$453.1 \div 5.656 \div 2 = 40.05\ MPa$	E	361×10^9	31	5.619×10^6	$> \ 1 \times 10^6$ OK
b	Full Strength Groove Weld			B	3930×10^9	110	See Note	$> \ 1 \times 10^6$ OK
c, d	8	5.659	$49.66 \div 5.659 \div 2 = 4.39\ MPa$	E	361×10^9	31	"	$> \ 1 \times 10^6$ OK
e	5	3.535	$7.98 \div 3.535 \div 2 = 2.26\ MPa$	E	361×10^9	31	"	$> \ 1 \times 10^6$ OK
f	5	3.535	$6.12 \div 3.535 \div 2 = 1.73\ MPa$	E	361×10^9	31	"	$> \ 1 \times 10^6$ OK

Note: an examination of f_{sr} compared with F_{srt} and clause 26.3.4, Figure 1 shows that fatigue life is well above the requirement of 1 000 000 cycles.

Consider Distortion Induced Fatigue

The area of most vulnerability is at welds 'c' and 'd' where differential vertical deflection between the runway beam and the W530 beam at the back of the apron plate may cause premature failure of these welds. In addition, the fabricator/erector may prefer a bolted connected for ease of fabrication, shipping, and erection.

Provide a bolted connection, slip critical, class A surfaces, 22mm diameter A325 bolts. Table 3-11 of the CISC Handbook provides a value $V_s = 45.2\ kN$ per bolt in single shear for slip resistance. Table 3-4 of the Handbook provides a value of $88.9\ kN$ factored shear resistance, threads included. OK for 10 mm plate.

$$\text{Unfactored Shear Flow} = \frac{1061 \times 10^3 \times 1.008 \times 10^6}{20.94 \times 10^9} + \frac{67.54 \times 10^3 \times 6.976 \times 10^6}{7.945 \times 10^9} = 51.1 + 59.3 = 110.4\ N/mm$$

$$\text{Factored Shear Flow} \quad = 169.0\ N/mm$$

Calculate minimum bolt spacing for shear flows

$$= \frac{45.2 \times 10^3}{110.4} = 409\ mm \qquad \text{(Slip)} \qquad \text{Governs}$$

$$\text{or} \quad = \frac{88.9 \times 10^3}{169.0} = 526\ mm \qquad \text{(Strength)}$$

Determine minimum bolt spacing for built-up members in accordance with S16-01, Clause 19. Spacing for bolts, not staggered, should not exceed $\dfrac{330t}{\sqrt{F_y}} = \dfrac{330 \times 10}{\sqrt{350}} = 176\ mm \ \not> \ 300\ mm$

Since this provision governs over slip resistance, a smaller bolt diameter will do. M20 bolts provide $37.4\ kN$ slip resistance, therefore OK by inspection.

Check Fatigue at Stiffener Welds

Specified Shear $839.0 + 209.8 + 11.92 = 1061\ kN$

f_{sr} in 8 mm fillets

$$= 1061 \times \frac{10^3}{4} \times 8 \times 0.707 \times 1350 \ (4\ welds)$$

$$= 34.7\ MPa$$

For category E, $\gamma = 361 \times 10^9\ MPa$, $F_{srt} = 31\ MPa$

$$nN = \frac{\gamma}{f_{sr}^3} = \frac{361 \times 10^9}{34.7^3} = 8.64 \times 10^6\ cycles \quad > \quad 1.0 \times 10^6 \quad \text{OK}$$

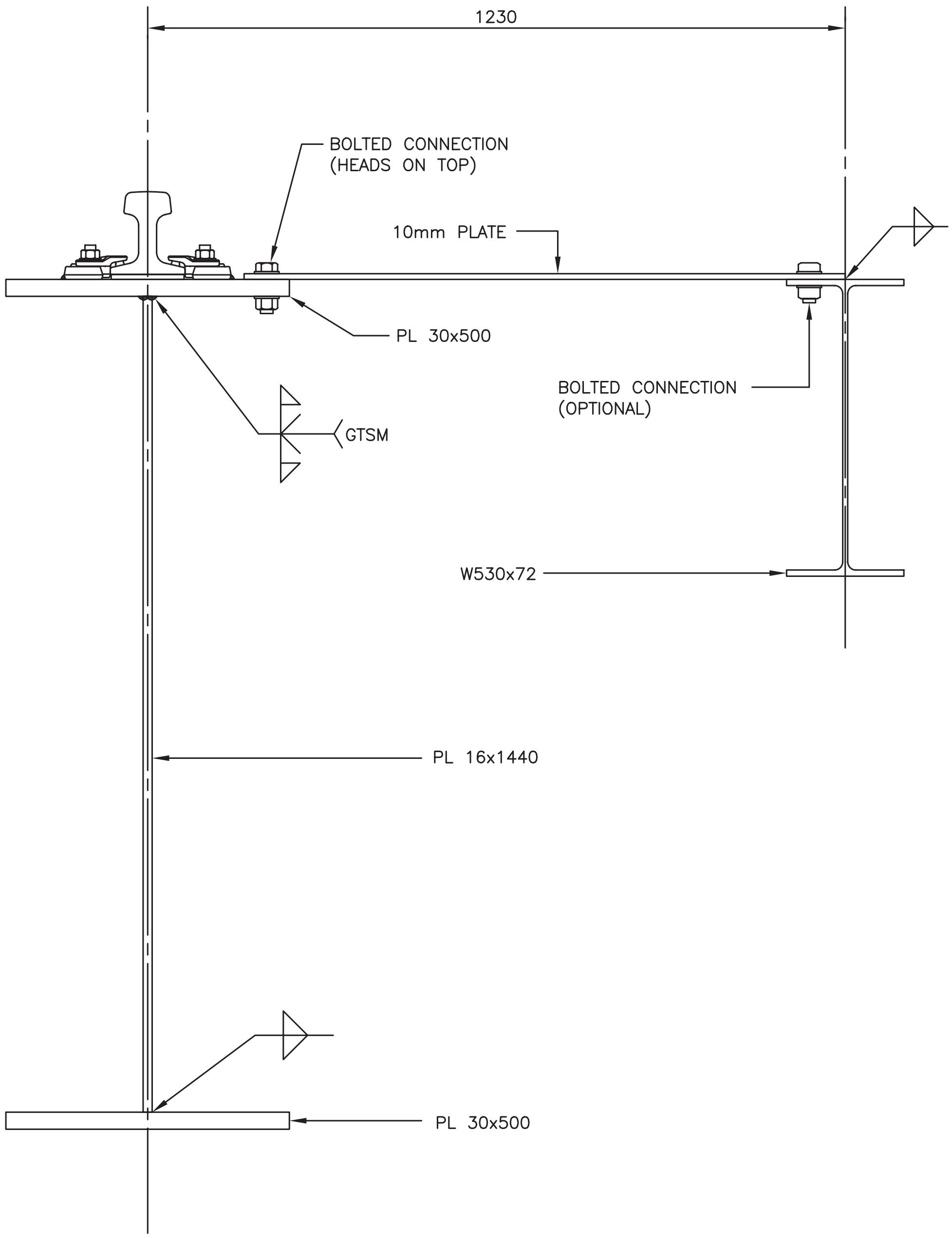
Examine Weld to Top Flange

No calculation is necessary here. CJP welds with reinforcing are recommended to reduce possibility of cracking due to repeated stress due to loads from the crane rail. nN could be as high as 4×10^6 for this detail.

Conclusion

Crane runway beam design shown below is OK.

Could investigate use of a lighter section and alternative grade of steel.


1230
BOLTED CONNECTION
(HEADS ON TOP)
10mm PLATE
PL 30x500
BOLTED CONNECTION
(OPTIONAL)
GTSM
W530x72
PL 16x1440
PL 30x500

Canadian Institute of Steel Construction
300-201 Consumers Road
Willowdale, ON M2J 4G8
Tel 416-491-4552   Fax 416-491-6461
www.cisc-icca.ca

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