Copyright © 2018

by

Canadian Institute of Steel Construction

All rights reserved.
This book or any part thereof must not be reproduced
in any form without the written permission of the publisher.

Third Edition
First Printing, December 2015
Second Revised Printing, January 2017
Third Revised Printing, November 2018

ISBN 978-0-88811-227-9

2.4.1 Fatigue

The calculated fatigue stress range at the detail under consideration, to meet the requirements of Clause 26 of S16-14 and as described in Chapter 3 of this document, will be taken as that due to C_1.

> ***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, for load combinations with crane loads, 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.

2018 09

	Case	Principal Loads	Companion Loads	
1.	$\phi R \geq$	$1.4D$		
2.	$\phi R \geq$	$(1.25D$ or $0.9D) + (1.5C + 1.0L)$	$1.0S$ or $0.4W$	
3.	$\phi R \geq$	$(1.25D$ or $0.9D) + (1.5L + 1.0C)$	$1.0S$ or $0.4W$	
4.	$\phi R \geq$	$(1.25D$ or $0.9D) + 1.5S$	$(1.0C + 0.5L)$	
5.	$\phi R \geq$	$(1.25D$ or $0.9D) + 1.4W$	$(1.0C + 0.5L)$	See Note 8.
6.	$\phi R \geq$	$(1.25D$ or $0.9D) + 1.0C_7$		
7.	$\phi R \geq$	$1.0D + 1.0E$	$1.0C_d + 0.5L + 0.25S$	
8.	$\phi R \geq$	$1.0D + C_1$		

where C is any one of the crane load combinations C_2 to C_6 from Table 2.2.

Loads D, L, S, W and E are loads defined in the National Building Code of Canada (NBCC) issued by the Canadian Commission on Building and Fire Codes with the exception that the load L is all the live loads excluding loads due to cranes. Notes (1) through (9) of table 4.1.3.2.B of the NBCC 2015 shall apply to the factored load combinations.

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.*
7) *Note that the NBCC requires that for storage areas the companion load factor must be increased to 1.0.*
8) *Side thrust due to cranes need not be combined with full wind load.*

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 S16-14 gives the design criterion for load-induced fatigue as follows:

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

where

F_{sr} = fatigue resistance

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

2018 09

$$= \left(\frac{\gamma'}{nN}\right)^{1/5} \leq F_{srt}$$

γ and γ' = fatigue life constants (see Clause 26.3.4)

n = 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 (Clauses 26.3.3 and 26.3.4)

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

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:

2018 09

$$nN = \gamma / F_{sr}^3$$

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 (2010).

3.3.2 Palmgren -Miner Rule

The total or cumulative damage that results from fatigue loading, not applied at constant amplitude, by S16-14 must satisfy the Palmgren-Miner Rule:

$$\sum\left[\frac{(nN)_i}{N_{fi}}\right] \leq 1.0$$

2018 09

where:

$(nN)_i$ = 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

2018 09

α_i = fraction of any particular portion of the stress range to the total number of cycles

$$= \frac{n_i}{\sum n_i}$$

$\Delta\sigma_i$ = 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

$$\left[\left(\frac{208\,000}{312\,000}\right)(188^3) + \left(\frac{104\,000}{312\,000}\right)(219^3)\right]^{1/3} \approx 200 \text{ MPa}$$

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) ·100% = 64%. This is essentially the same result as in 3.3.2 (equivalent stress range was rounded off).

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	1 000	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. See also Comment 14/15.	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 runway 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 4.2 continued

Item	Comment	See Figure
13	Some degree of three-dimensional analysis is required to adequately assess loads in horizontal bracing. Refer to Fisher (2004) 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. The industry practice is to calculate deflections for one crane, not including impact.	-
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. 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 lies between +150°C and -30°C, structural steel meeting the requirements of CSA G40.21 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 limiting deflections and 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 fatigue, large ranges of stress may have to be considered. Knee brace struts should not be used, in particular for class of service C, D, E and F.	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. S16-14 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 AIST (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 not allowed on dynamically loaded structures by some authorities such as AIST (2003) and AWS (2015). The use of these welds should be restricted to applications where fatigue is not a consideration.	-

2018 09

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.

Unless the structure is suitably resistant to change in gauge of crane rails under roof dead load, final alignment of the crane runway beams should be deferred until the full dead load of the roof is in place.

2018 09

Figure 24 shows the requirements of the CMAA. The rate of change should not be applied to a distance less than 6 metres. It is based on requirements for satisfactory crane performance. Other tolerances such as those shown in table 4.1 are related to fabrication and erection tolerances. Both criteria should apply. The fabrication specification should account for required tolerances which may be more severe than the individual standards permit.

In case of conflict with Clause 2.9.7 of S16-14 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.

Referring also to item 5.29, it is recommended that periodic inspection and maintenance be done and a checklist should be prepared for the maintenance personnel.

Fisher (2004), Millman (1991, 1996) and Reemsnyder and Demo (1978) provide additional information.

Calculate Dead Load Supported by the Plate Girder

Section	**Area,** $mm^2 \times 0.785$	$= kg/m \times 0.009\,81$	$= kN/m$
Plate Girder	53.04×10^3	416.3	
50% of Apron Plate	5 175	40.6	
135# Rail		66.96	
Misc. (allowance)		50.0	
Σ		573.8	5.629 kN/m

Calculate the Unfactored Bending Moment M_x Due to Dead Load

$$= 5.629 \times \frac{15.240^2}{8} = 163.4 \text{ kN·m}$$

Calculate the Unfactored Maximum Bending Moment M_x Due to Live Loads with Impact

$$= 2\,751 + 687.8 + 101.7 = 3\,541 \text{ kN·m}$$

Calculate the Unfactored Maximum Bending Moment M_x
Due to Live Loads without Impact

$$= 2751 + 101.7 = 2853 \text{ KN.m}$$

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

$$= 1.094\,6 * \times 221.5 = 242.5 \text{ kN·m}$$

* Amplified due to eccentricity of loads due to side thrust

Calculate M_{fx} with Impact

$M_{fx} = (1.25 \times 163.4) + (1.5 \times 3\,541) = 5\,516 \text{ kN·m}$ (see previous calculations)

Calculate M_{fx} without Impact

$M_{fx} = (1.25 \times 163.4) + (1.5 \times 2853) = 4484 \text{ kN.m}$

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

Calculate M_{fy} at Top

$M_{fy} = 1.5 \times 242.5 = 363.7 \text{ kN·m}$

Check bracing to compression flange by Clause 9.2.7

Maximum factored force in top flange

$$\frac{4\,484 \times 10^6}{28\,270 \times 10^3} \times \frac{16\,369}{10^3} = 2\,596 \text{ kN}$$

Factored UDL due to 5% of this force

$$= \frac{0.05 \times 2596}{15.24} = 8.52 \text{ kN/m}$$

2018 09

Factored bending moment due to this force

$$= \frac{8.52 \times 15.24^2}{8} = 247 \text{ kN} \cdot \text{m} < 363.7 \text{ kN} \cdot \text{m}$$

Does not govern, no need to combine with side thrust.

Calculate M_{fy} at Bottom

$$M_{fy} = 1.5 \times 0.094\,6 \times 221.5 = 31.4 \text{ kN·m}$$

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

Check for live Loads, No Impact

This is the Yielding Limit State (Strength) Check.

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

$$\frac{4484}{8905} + \frac{363.7}{4\,139} = 0.504 + 0.088 = 0.592 \leq 1.0 \quad \text{OK}$$

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

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

$$\frac{363.8}{2\,557.2} = 0.142 < 1.0 \quad \text{OK}$$

Check for M_{fx} and M_{fy} in Bottom Flange, Live loads, no side thrust

$$\frac{5\,516}{8\,574} + \frac{0.0}{591} = 0\ 643 + 0.0 = 0.643 < 1.0 \text{ OK}$$

Check for M_{fx} and M_{fy} in Bottom Flange, Live loads, no impact

$$\frac{4484}{8574} + \frac{31.4}{591} = 0.523 + 0.053 = 0.576 < 1.0 \text{ OK}$$

Calculate Factored Shear in the Vertical Direction, Impact included

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

$$= 53.61 + 1591 = 1\,665 \text{ kN}$$

Check Shear Strength in the Vertical Direction

$$\frac{1\,665}{2\,442} = 0.682 < 1.0 \quad \text{OK}$$

A check for combined bending moment and shear is not required because the section is not transversely stiffened. See S16-14, Clause 14.6.

Check column action

$$A = (2 \times 232 \times 25) + (16 \times 192) = 14\,672 \text{ mm}^2$$

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

$$r = \sqrt{\frac{230.5 \times 10^6}{14\,672}} = 125.3 \text{ mm}$$

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

$$= 0.75 \times 1440 = 1\,080 \text{ mm}$$

$$\frac{KL}{r} = \frac{1 \times 1\,080}{125.3} = 8.61$$

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

$$314 \times \frac{14\,672}{1\,000} = 4\,607 \text{ kN} > 1\,665 \text{ kN} \qquad \text{OK}$$

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

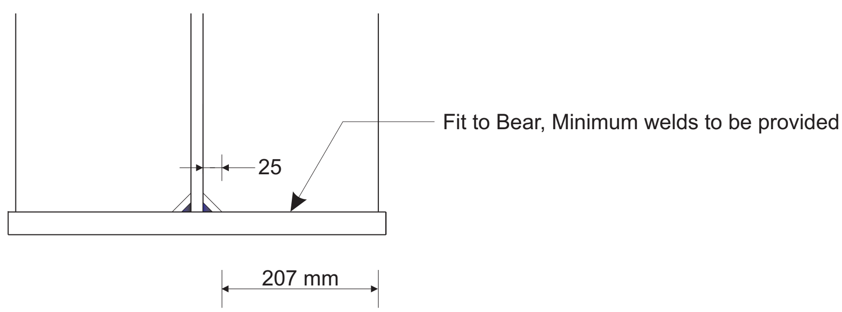

Figure A24
Bearing of Bearing Stiffener

Check one side

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

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 × 207 = 155 mm in contact.

The factored bearing resistance, to clause 13.10

$$= 1.5 \times 0.9 \times 350 \times \frac{155}{1\,000} \times 25 = 1831 \text{ kN} > 832.5 \quad \text{OK}$$

Design welds to web

2018 09

Factored load per weld $= \dfrac{832.5}{2 \times 1350 \text{ (say)}} = 0.308 \text{ kN/mm}$

From Table 3-24, CISC Handbook, need 5 mm for strength; use minimum = 8 mm.