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Evaluation of the remaining lateral torsional buckling capacity in corroded steel members

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Abstract: Corrosion is one of the main causes of deterioration in steel structures. Loss of thickness in flanges and web of corroded steel beams leads to reduction in section properties which can reduce the lateral torsional buckling capacity of the member. In this paper, thickness loss data were compiled from four samples of corrosion damaged I-beams removed from a petro-chemical plant. Visual examination of the four corroded beams showed that they were corroded uniformly. To improve the accuracy of the results, a large number of measurements for surface roughness were taken for each beam, totally 770 values to obtain the average thickness of flanges and web of each beam. The data was used to develop a corrosion decay model in order to calculate the percentage remaining lateral torsional buckling capacity of long and short span beams which are laterally unrestrained. To estimate the percentage of remaining lateral torsional buckling capacity in the corroded damaged I-beams, the readily available minimum curves for different types of universal beams in conjunction with information on the thickness loss were used. The results can be used by practicing engineers for better estimation on the service life of deteriorated steel structures.

Key words: Corrosion, Steel structures, Lateral torsional buckling, Beams
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1 Introduction

Corrosion in steel structures appears to be the most important degradation mechanism in determining the remaining life of these structures. The infrastructures in various parts of the world, particularly in Asia, are getting older. Many steel structures have exceeded 50 years of service life and are often in a severely deteriorated condition. In most countries, the petro-chemical industries use steel extensively as the primary structural material for pipe bridges, support frames in vessels and process equipment. The most common problem for all of these steel structures is deterioration due to corrosion, which is more rapid in the aggressive environments of chemical plants. In addition, the exposed coastal areas in which chemical plants are often located tend to exacerbate the problem (Gallon, 1993).

Deterioration of a steel structure due to corrosion can alter its stiffness and behavior. Therefore, the analysis of corrosion damaged structures may differ from the analysis of a structure under design. Corrosion can not only cause a fracture to occur, but also yielding or buckling of members. This can result in an increase in stress, change in geometric properties, buildup products, and the reduction in member cross-section properties, such as section modulus or slenderness ratio (Czarnecki and Nowak, 2008). Also the class of a section (plastic, compact, semi-compact, or slender) may be changed from one to another due to loss of thickness in compression flanges and web caused by corrosion. For example, a section that is plastic or compact may become semi-compact due to loss of thickness, and local buckling may prevent the development of full plastic moment (BS 5950, 1985) in such cases. Also the modes of failure of a member may be changed from one mechanism to another

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depending on the relative thickness loss in its various parts. It was shown in (Gallon *et al*., 1995) that the bending failure mode governs a member in its early stages and after several years of corrosion the shear failure mechanism becomes critical. The shear mode also becomes critical when holes created by corrosion are present in the web.

Significant developments have been made for a better understanding of the causes of corrosion, in enhancement of corrosion-resistance in new steel structures, and in structural evaluation methods that assess the safe strength in existing corroded structures. Indeed, many of recently adapted codes specifically require corrosion protection in steel structures, especially those structures located in severe environments. Such protection is provided by means of suitable alloying elements in the steel, protection coatings, provision of extra thickness as a corrosion allowance, and other approved means. The type and degree of corrosion protection to be provided should be indicated on the structural drawing (Bruneau and Zahrai, 1997).

A number of studies were carried out to investigate the ultimate strength characteristics of plate elements with pit corrosion wastage under axial compressive (Paik, 2003; Paik *et al*., 2003) and shear loads (Paik *et al*., 2004). A series of ANSYS nonlinear finite element (FE) analyses for steel plates under axial compressive and shear loads was performed, varying the degree of pit corrosion intensity and plate geometric properties. It has been discovered that for a corroded plate subjected to compressive loads, the smallest cross-sectional area is the dominant parameter to represent the ultimate strength reduction characteristics due to pitting corrosion, whereas the ultimate strength of a damaged localized corrosion plate with shear loads is governed by the degree of pit corrosion intensity.

Nakai *et al*. (2004; 2005; 2006a) employed a series of actual tests and FE analyses to consider the effect of pitting corrosion on the lateral distortional buckling behavior of hold frame of bulk carrier. It was found that the residual ultimate structural strength with pit corrosion is almost the same as that of the structural models whose web has the equivalent uniform corrosion thickness loss. In (Ok *et al*., 2007) the effect of pit corrosion on the ultimate compressive strength of plate elements was investigated. A series of FE nonlinear analyses with various locations and sizes of pitting corrosion was performed. They found that the length, width and depth of pit corrosion have remarkable effects on decreasing the ultimate compressive strength of the plate elements. Despite of the significant effect of pitting locations on the ultimate strength of plate elements it was found that plate slenderness has a scant effect on strength reduction. The effect of corrosion on the ultimate strength of steel plates subjected to in-plane compression and bending was investigated by Nakai *et al*. (2006b), a series of nonlinear FE analyses was performed for corroded plates subjected to in-plane compressive load and bending moments.

The nature of corrosion in various forms was described in detail along with how corrosion affects steel structures by Rahgozar (1998) and Kulicki *et al*. (1990). Uniform corrosion is the most common form of corrosion, which leads to a gradual thinning of members (Fontana, 1987). Kayser (1988) also mentioned that uniform corrosion is the most serious form of corrosion observed on steel bridges. In the present study, uniform corrosion which mostly appears in rolled beams of steel structures subjected to static loading was considered. It is assumed that it results in a loss of metal section leading to a reduction of the load carrying capacity, and consequently a reduction of the structural reliability.

Smith (1993) indicated that the simplest consequence of corrosion is a reduction in material strength and section size due to loss of material. This in turn leads to a reduction in the carrying capacity of the structure and in its member stiffness thus causing excessive distortion in the members. The effects of corrosion on the remaining moment and shear capacities of steel beams using minimum curves were considered by Rahgozar (2009). He proposed a series of minimum curves for universal I-beams which can be used together with the information on the percentage loss of thickness to estimate the percentage remaining capacities of corrosion damaged beams. Nethercot (2001) mentioned that when corroded beam strength is considered, lateral distortional buckling strength and local buckling strength are important factors. The main objective of this paper is to use the measurement data and the model developed, to calculate the percentage of remaining lateral torsional buckling capacity of long span and short span beams that are laterally unrestrained.

2 Corrosion decay models

Corrosion of steel occurs when electrolytes are present on the surface, particularly in places where water and contaminants can accumulate. The places most commonly found with corrosion are the top surface of the bottom flange where water is collected from dew or splash and on the web near the abutments and joints. Kayser and Nowak (1989) pointed out that the type of corrosion most likely to occur will be section loss on the top surface of the bottom flange and on the lower portion of the web, as shown in (Fig. 1).

Fig. 1 Typical locations where corrosion can occur on a steel girder bridge (Kayser and Nowak, 1989)

Corrosion decay model usually requires the information on locations where corrosion normally occurs and the types of corrosion damage in steel members. Czarnecki and Nowak (2008) pointed out that an important part of a corrosion model is the corrosion pattern and the areas with concentration of high corrosion rate within the structure. In this study, the thickness loss data were compiled from four samples of corrosion damaged universal I-beams, which were compiled from a petro-chemical industry. Thickness measurement of corroded damaged beams showed that there is a large variation in the measurement (Rahgozar, 1998; Sarveswaran *et al*., 1998). To improve the accuracy of the results, measurements were taken at various locations, 770 points from flanges and web of each beam to obtain an average value for the thickness. Using these detailed measurements, the distribution parameters such as mean value and coefficient of variation (COV) of the corroded beam were calculated (Table 1). It was found that variation in thickness measurements is directly related to the degree of corrosion.

The analysis of corrosion effects was carried out using the corrosion decay model described by Rahgozar (1998) and Sarveswaran *et al.* (1998). Based on the percentage thickness loss of the beams as shown in Table 1, a corrosion decay model has been developed (Fig. 2). This model may be used for reliability assessment of corrosion damaged steel beam based on visual assessments of thickness loss.

The following relationships can be used for a corrosion decay model as shown in Fig. 2.

$$T_c=T_N(1-\mu), \tag{1a}$$
$$t_c=t_N(1-0.5\mu), \tag{1b}$$

where T_c and t_c is average thickness of the flanges and web, respectively, T_N and t_N is the flanges and web thickness of intact cross-sectional, respectively, and h_w is the height of the beam's web. *B* and *D* are the width and overall depth (Fig. 2).

Table 1 Distribution parameters of the corroded thickness of sample beams

Element		As-new	Beam 1	Beam 2	Beam 3	Beam 4
Top flange (mm)	Mean	10.20	7.15	7.39	7.01	7.29
	COV		0.46	0.24	0.28	0.25
Bottom flange (mm)	Mean	10.20	5.13	5.21	4.20	7.09
	COV		0.53	0.40	0.63	0.28
Average flange thickness (mm)		10.20	6.14	6.30	5.60	7.19
Average thickness loss of flange (%)		0.00	39.80	38.20	45.00	29.50
Upper part of web ($0.75h_w$) (mm)	Mean	6.10	5.33	5.43	5.14	5.41
	COV		0.03	0.03	0.04	0.02
Lower part of web ($0.25h_w$) (mm)	Mean	6.10	3.02	4.11	3.01	4.34
	COV		0.71	0.28	0.72	0.34
Average web thickness (mm)		6.10	4.18	4.77	4.08	4.88
Average thickness loss of web (%)		0.00	31.50	21.80	33.10	20.00

COV: coefficient of variation; h_w: height of web for a new beam

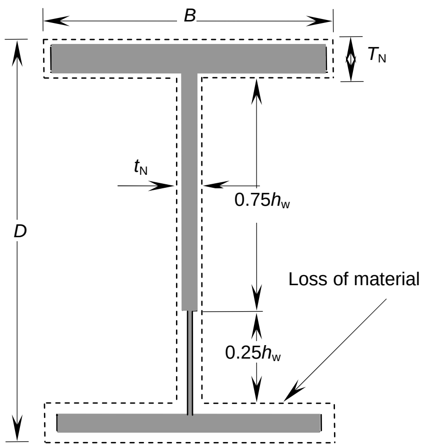


Fig. 2 Varying thickness loss model

The thickness loss of top flange, upper part of the web ($0.75h_w$), lower part of the web ($0.25h_w$), and bottom flange is $0.7\mu T_N$, $0.25\mu t_N$, $0.25\mu t_N$, and $1.3\mu T_N$, respectively, μ is the percentage loss of thickness, μ_F and μ_w is the percentage loss of flange thickness and web thickness, respectively, and $\mu=\mu_F=2\mu_w$

For corrosion damaged I-beams of the same section size, the overall dimensions B, D and h_w can be considered as constants throughout their service life, although there will be a small reduction due to corrosion. Therefore, the plastic modulus of I-sections with equal flanges about its major axis may be given by

for corrosion damaged sections

$$S_{xC} = BT_C(D - T_C) + t_C \frac{h_w^2}{4}, \tag{1c}$$

for as-new sections

$$S_{xN} = BT_N(D - T_N) + t_N \frac{h_w^2}{4}. \tag{1d}$$

Substituting Eqs. (1a) and (1b) into Eq. (1c), we can obtain the plastic modulus of a corrosion damaged model about an asymmetrical axis:

$$S_{xC} = \left(BT_N(D - T_N) + t_N \frac{h_w^2}{4} \right) - \mu \left(BT_N(D - 2T_N) + t_N \frac{h_w^2}{4} \right) + \mu t_N \frac{h_w^2}{8} - BT_N^2 \mu^2. \tag{1e}$$

If Eq. (1d) is substituted into Eq. (1e) and the μ^2 term is neglected, the following relationship is obtained for the plastic modulus of corrosion damaged sections:

$$S_{xC} \approx S_{xN} - \mu \left(S_{xN} - t_N \frac{h_w^2}{8} \right), \tag{1f}$$

where the term $t_N(h_w^2/8)$ is the plastic modulus of the half of the shear area.

3 Lateral torsional buckling capacity of I-beams

The compression flange of an I-beam acts like a column and will buckle sideways if the beam is not sufficiently stiff or the flange is not restrained laterally. The load at which the beam buckles can be much less than that causing the full moment capacity to develop. For an idealized perfectly straight beam, there is no deformation normal to the loading plane until the applied moment reaches a critical value M_E, less than the moment capacity. At this point the beam buckles by deflecting laterally and twists as shown in Fig. 3. These two deformations u and v (horizontal and vertical deformations) are interdependent, when the beam deflects laterally, the applied moment exerts a component torque with a rotation (φ) about the deflected longitudinal axis which causes the beam to twist. This behavior, which is important for long unrestrained I-beams, is called lateral torsional buckling.

A perfectly straight beam which is loaded by equal and opposite end moments is shown in Fig. 3. The beam is simply supported at its ends so that lateral deflection and twist rotation are prevented,

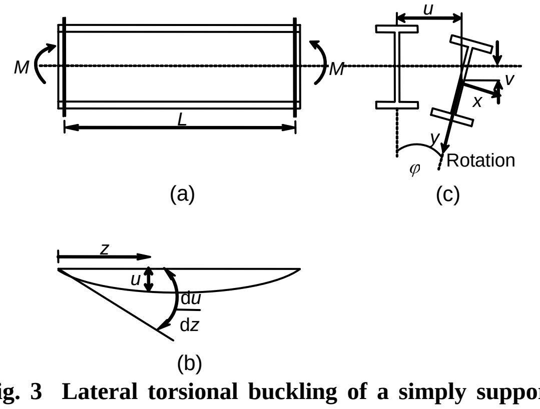


Fig. 3 Lateral torsional buckling of a simply supported I-beam. (a) Elevation; (b) Plan on the longitudinal axis; (c) Section

while the flange ends are free to rotate in horizontal planes. The elastic theory is used to set up equilibrium equations that equate external disturbances to the lateral bending and torsional resistance of the beam. The solution of these equations for the elastic critical moment was given by Timoshenko and Gere (1961):

$$M_{\mathrm{E}}=\frac{\pi}{L}(EI_yGJ)^{1/2}\left(1+\frac{\pi^2EI_{\mathrm{w}}}{L^2GJ}\right)^{1/2}, \tag{2}$$

where M_{E} is the elastic critical buckling moment, E is the modulus of elasticity, I_y is the minor axis moment of inertia, EI_y is the minor axis flexural rigidity, G is the shear modulus of elasticity, J is the polar moment of inertia, GJ is the torsional rigidity, EI_{w} is the warping rigidity of the beam, and L is the beam length. Eq. (2) shows that the resistance to buckling depends on the geometric mean of the flexural resistance (π^2EI_y/L^2) and the torsional resistance ($GJ+\pi^2EI_{\mathrm{w}}/L^2$).

The magnitude of the critical moment given by Eq. (2) does not depend on the major axis flexural rigidity EI_x of the beam in the vertical plane. This conclusion is obtained as a result of the assumption that the deflections in the vertical plane are small as shown in Fig. 3, which is justifiable since the flexural rigidity EI_x is much greater than the rigidities EI_{w} and EI_y. If the rigidities are of the same order of magnitude, the effects of bending in the vertical plane should be considered (Timoshenko and Gere, 1961). The equation for the elastic critical buckling moment, which includes the effect of major axis bending, is given by Martin and Purkiss (1992):

$$M_{\mathrm{E}}=\frac{\pi}{L}\left(\frac{EI_yGJ}{\gamma}\right)^{1/2}\left(1+\frac{\pi^2EI_{\mathrm{w}}}{L^2GJ}\right)^{1/2}, \tag{3}$$

where γ is the correction factor, which is just less than the unity for most beam sections, and is given by

$$\gamma=1-\frac{I_y}{I_x}. \tag{4}$$

In theoretical analysis, the beam was assumed to be geometrically perfect, i.e., had no imperfections due to lack of straightness, and no residual stresses due to the manufacturing process. In reality, beams have initial curvature, twist, and residual stresses, and the loads are applied eccentrically. The theory set out here requires modification to account for actual behavior. Theoretical studies and tests by Nethercot (1974) show that at low slenderness ratios the beam achieves its full plastic moment capacity, whereas at high slenderness ratios the behavior closely approximates to that predicated by Eq. (3). At intermediate slenderness ratios the behavior is dependent on the buckling moment and the plastic moment of resistance. This lateral torsional buckling behavior of a beam as a function of slenderness is shown in Fig. 4.

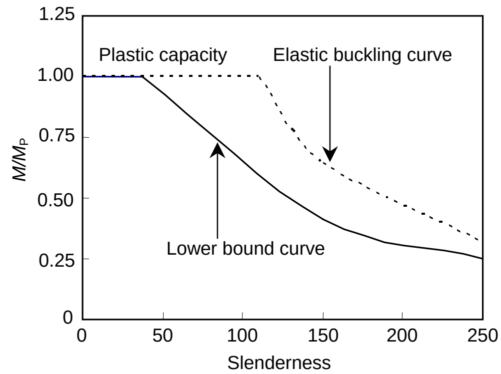


Fig. 4 Lateral torsional buckling behavior of a beam

The equivalent slenderness for beam buckling, λ_{LT}, is defined as

$$\lambda_{\mathrm{LT}}=\left(\frac{\pi^2E}{p_{\mathrm{y}}}\right)^{1/2}\left(\frac{M_{\mathrm{P}}}{M_{\mathrm{E}}}\right)^{1/2}, \tag{5}$$

where M_{P} is the full plastic moment and p_{y} is the yield stress.

The equation for λ_{LT} is not convenient for design purposes as the calculation of M_{E} is cumbersome. For sections that are symmetric about the major axis the following parameters for u, v and λ for the I-beam are defined as follows (Trahair *et al*., 2001):

$$u=\left(\frac{4S_x^2\gamma}{A^2h_{\mathrm{s}}^2}\right)^{1/4}, \tag{6a}$$

$$v=\left(1+\frac{1}{20}\left(\frac{\lambda}{x}\right)^2\right)^{-1/4}, \tag{6b}$$

$$\lambda=L_{\mathrm{E}}/r_y, \tag{6c}$$

$$S_x=\frac{M_{\mathrm{P}}}{p_{\mathrm{y}}}, \tag{6d}$$

$$x = 0.566 h_{\mathrm{s}} \left(\frac{A}{J} \right)^{1/2}, \tag{6e}$$

$$r_y^2 = I_y / A, \tag{6f}$$

where u is the buckled shape, v is the slenderness factor, h_{s} is the distance between flange shear centers, L_{E} is the effective length, r_y is the minor axis radius of gyration, S_x is the plastic section modulus, A is the cross-section area, and x is the torsional index. The equation for elastic critical buckling moment can be given as

$$M_{\mathrm{E}} = \frac{\pi^2 E S_x}{(\lambda u v)^2}. \tag{7}$$

Using Eqs. (5) and (7) we can obtain:

$$\lambda_{\mathrm{LT}} = u v \lambda. \tag{8}$$

To deal with the intermediate slenderness cases between elastic buckling (M_{E}) and full plastic moment (M_{P}), a 'Perry-Robertson' approach is used in BS 5950 (1985) and the buckling resistance moment, M_{b}, is given as the least square root of the following equation:

$$(M_{\mathrm{P}} - M_{\mathrm{b}})(M_{\mathrm{E}} - M_{\mathrm{b}}) = \eta_{\mathrm{LT}} M_{\mathrm{E}} M_{\mathrm{b}}, \tag{9}$$

where η_{LT} is a coefficient to allow for initial imperfections and residual stresses.

The least square root of Eq. (9), M_{b}, is given by

$$M_{\mathrm{b}} = \Phi_{\mathrm{B}} \pm (\Phi_{\mathrm{B}}^2 - M_{\mathrm{E}} M_{\mathrm{P}})^{1/2}, \tag{10}$$

where

$$\Phi_{\mathrm{B}} = \frac{M_{\mathrm{P}} + (\eta_{\mathrm{LT}} + 1) M_{\mathrm{E}}}{2}. \tag{11}$$

Note that buckling does not occur at values of $(M_{\mathrm{P}}/M_{\mathrm{E}})^{1/2}$ of less than 0.4, thus a limiting equivalent slenderness ratio, λ_{LO}, is defined by Trahair *et al*. (2001):

$$\lambda_{\mathrm{LO}} = 0.4 \left(\frac{\pi^2 E}{p_{\mathrm{y}}} \right)^{1/2}. \tag{12}$$

The imperfection coefficient, η_{LT}, called the Perry coefficient and is defined by

$$\eta_{\mathrm{LT}} = 0.007(\lambda_{\mathrm{LT}} - \lambda_{\mathrm{LO}}), \quad \text{for rolled sections}, \tag{13}$$

$$\eta_{\mathrm{LT}} = 2\alpha_{\mathrm{b}} \lambda_{\mathrm{LO}}, \quad \text{for welded sections}, \tag{14}$$

where α_{b} is a constant.

The theoretical solution applies to a beam subjected to a uniform moment. In other cases where the moment varies, the tendency to buckling is reduced. If the load is applied to the top flange and can move sideways, it is destabilizing, and buckling occurs at lower loads than that if the load were applied at the centroid or to the bottom flange. It is therefore necessary to modify the above approach to allow for loading along the span of the beam either in the form of distributed loading or in the form of point loading; and to allow for the effects of support conditions where, for example, twisting may occur. The problems of non-uniform moments and varying end conditions can be solved using BS 5950 (1985).

4 Assessment methods for lateral torsional buckling capacity

Loss of thickness in the flanges and web leads to a reduction in section properties of steel beams which in turn leads to a reduction in lateral torsional buckling capacity. Lateral torsional buckling is a critical failure mode mainly for long and short span beams that are laterally unrestrained. Several geometric parameters, such as the beam length, end conditions, plastic modulus, lateral stiffness, torsional properties, and warping resistance of the section influence the lateral torsional buckling capacity of beams. In this study, the corroded model proposed by Sarveswaran *et al*. (1998), and the theory given in the previous section were used for the evaluation of remaining lateral torsional buckling capacity of corrosion damaged ordinary beams.

4.1 Bending strength of a beam

A simplified equation for bending strength, p_{b}, is derived to verify the influence of the factors such as the equivalent slenderness, λ_{LT}, material properties, p_{y}, and modulus of elasticity, E. The following approximations given by Trahair *et al*. (2001) were used together with Eq. (10) to obtain Eq. (15):

$$p_{\mathrm{b}}=\frac{M_{\mathrm{b}}}{S_x}\approx\frac{1.022\pi^2E\left(\lambda_{\mathrm{LT}}^2+\dfrac{0.007\pi^2E}{p_{\mathrm{y}}}\lambda_{\mathrm{LT}}+\dfrac{\pi^2E}{p_{\mathrm{y}}}\left(1-0.0028\left(\dfrac{\pi^2E}{p_{\mathrm{y}}}\right)^{1/2}\right)\right)}{\left(\lambda_{\mathrm{LT}}^2+\dfrac{0.007\pi^2E}{p_{\mathrm{y}}}\lambda_{\mathrm{LT}}+\dfrac{\pi^2E}{p_{\mathrm{y}}}\left(1-0.0028\left(\dfrac{\pi^2E}{p_{\mathrm{y}}}\right)^{1/2}\right)\right)^2-\dfrac{\pi^2E}{p_{\mathrm{y}}}\lambda_{\mathrm{LT}}^2}, \tag{15}$$

$$I_{\mathrm{w}}=I_y h^2/4,\quad J=0.25AT^2,\quad h=0.9D,$$

$$\lambda_{\mathrm{LT}}=uv\left(\frac{L_{\mathrm{E}}}{r_y}\right), \tag{16}$$

$$v=\left(1+\frac{1}{20}\left(\frac{L_{\mathrm{E}}}{r_y x}\right)^2\right)^{-1/4}, \tag{17}$$

where T is the flange thickness.

The validity of Eq. (15) is evaluated for various values of λ_{LT} and compared with data given in BS 5950 (1985) as shown in Fig. 5 for different values of design strength, such as 275 and 355 N/mm^2. It was found that Eq. (15) differs from BS 5950 (1985) by only −5.0% to +6.5%.

4.1.1 Critical effective length for the maximum bending strength

One of the important factors that govern the lateral torsional buckling capacity of a beam is the effective length of the beam. Therefore, this must be taken into account when developing minimum curves for the assessment of remaining lateral torsional buckling capacity of corroded members. A critical effective length ($L_{\mathrm{E(crit)}}$) for the maximum bending strength can be found for any beam when its bending strength is equal to its design strength, by equating the equivalent slenderness, λ_{LT} to the limiting equivalent slenderness, λ_{LO}. Using Eqs. (12) and (16), a simplified relation is obtained for the critical effective length:

$$L_{\mathrm{E(crit)}}=0.408\left(\frac{\pi^2E}{p_{\mathrm{y}}}\right)^{1/2}\left(\frac{r_y}{u}\right). \tag{18}$$

The above equation may be rewritten as

$$\frac{L_{\mathrm{E(crit)}}u}{r_y}=0.408\left(\frac{\pi^2E}{p_{\mathrm{y}}}\right)^{1/2}=k_1, \tag{19}$$

where k_1 is a constant for a given p_{y}.

It follows from Eq. (19) that

$$\lambda=\frac{L_{\mathrm{E(crit)}}}{r_y}=\frac{k_1}{u}. \tag{20}$$

Using Eqs. (17) and (20) together with Eq. (16), the following expression is obtained for the equivalent slenderness:

$$\lambda_{\mathrm{LT}}=k_1\left(1+\frac{1}{20}\left(\frac{k_1}{ux}\right)^2\right)^{-1/4}. \tag{21}$$

Using Eq. (21), it can be obtained that the factors that influence the equivalent slenderness, λ_{LT} in this case are the torsional index, x and the buckling parameter, u. Eq. (15) shows that for a given p_{y}, the equivalent slenderness is the critical factor that governs the bending strength, p_{b}, of beams. In summary, x and u are the critical factors that govern the bending strength of beams with critical effective length, $L_{\mathrm{E(crit)}}$ and constant design strength p_{y}.

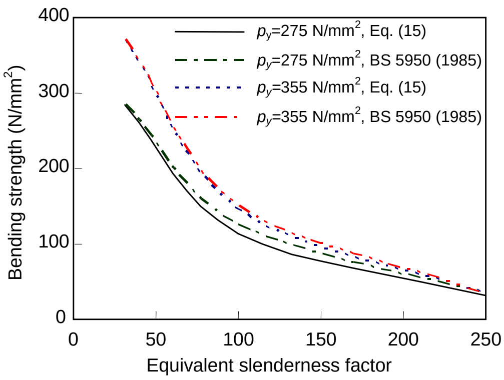


Fig. 5 Validity of Eq. (15)

4.1.2 Constant slenderness factor λ

In a case where the slenderness factor, λ, and the design strength of the beams are constant:

$$\lambda = \frac{L_{\mathrm{E}}}{r_y} = k_2, \tag{22}$$

where k_2 is a constant. Combining Eqs. (17) and (22) together with Eq. (16) gives

$$\lambda_{\mathrm{LT}} = uk_2 \left(1 + \frac{1}{20}\left(\frac{k_2}{x}\right)^2\right)^{-1/4}. \tag{23}$$

Therefore, for beams which have constant slenderness factor λ and design strength, using Eqs. (15) and (23) it can be said that the torsional index, x, and the buckling parameter, u, are the critical factors that govern the bending strength of such beams. Note that the critical parameters are identical to those govern the bending strength of beams with critical effective length.

4.1.3 Constant L_{E}/D

$$\frac{L_{\mathrm{E}}}{D} = k_3, \tag{24}$$

where k_3 is a constant.

Eq. (16) can be modified using the following approximations:

$$x \approx D/T, \text{ and } B = 4.4r_y,$$

then

$$\lambda_{\mathrm{LT}} = u\frac{4.4k_3}{(B/D)}\left(1 + \frac{1}{20}\left(\frac{4.4k_3}{2(b/T)}\right)^2\right)^{-1/4}. \tag{25}$$

Using Eqs. (15) and (25), it can be obtained that the buckling parameter, u, and the ratios of b/T and B/D are the critical factors that govern the buckling strength of beams with constant L_{E}/E and p_{y}.

4.2 Effect of design strength on the remaining lateral torsional buckling capacity

The simplified equation obtained for the bending strength of a beam (Eq. (15)) shows that the design strength is an important factor on the lateral torsional buckling capacity of beams. To verify the effect of design strength on the remaining lateral torsional buckling capacity (RLTBC) of corrosion damaged beams, a universal beam which is classified in BS standard steel profiles based on the serial sizes and mass per meter from UB1 until UB67, was used. UB60 (254×146×43, where 254×146 is the serial sizes in mm and 43 is the mass per meter in kg) was selected to verify the effect of design strength on the remaining lateral buckling capacity using the aforementioned proposed corroded model. The beam was analyzed for four values of design strength (245, 275, 355 and 450.0 N/mm^2). The length and the resistance conditions were assumed to be the same in all cases. Results of the analyses are given in Fig. 6.

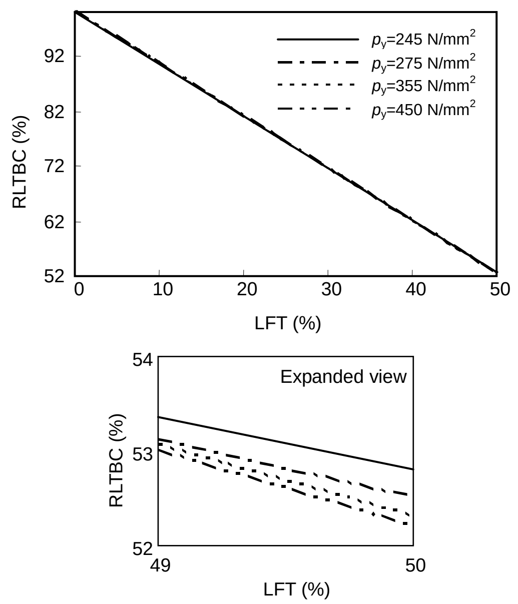


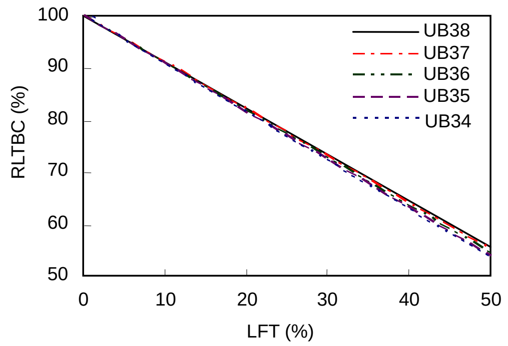
Fig. 6 Effect of design strength on the RLTBC of a corrosion damaged beam (UB60)

It can be seen that the remaining lateral torsional capacity of a member is reduced due to the loss in thickness. The lateral torsional buckling capacity decreases at a considerable rate with the loss of flange thickness. The reduction in lateral torsional capacity of the beam is almost linearly proportional to the section loss in this case. Therefore, the later torsional buckling is the critical failure mode at the early stages of corrosion for the decay modeling beams.

Also it is evident from Fig. 6 that the variation in the RTLBC curves for different values of design strength is negligible (less than 0.5% when the flange thickness loss is 50%). When the above analyses were undertaken using various span beams, it was found that the above results are true for beams of any span. Therefore, the effect of design strength on the RLTBC not need to be considered when developing minimum curves for the RLTBC of corrosion damaged beams. Any value for the design strength may be used when developing minimum curves.

4.3 Short span beams with critical effective length

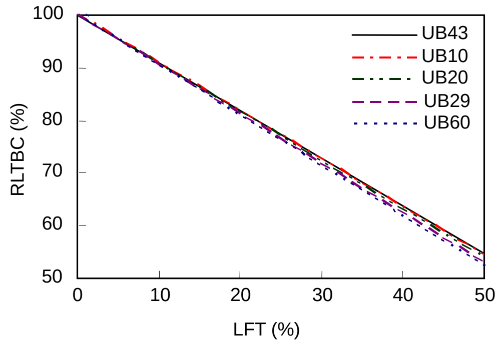
A family of sections with varying thickness loss was analyzed to study the RLTBC of corrosion damaged short span beams. The effective length of the beams was taken as the critical effective length. RLTBC was calculated based on the buckling resistance moment (M_b) of the beams. The details of the sections and results of the analyses are given in Fig. 7.



No.	Section	*x*
UB38	457×152×52	43.9
UB37	457×152×60	37.5
UB36	457×152×67	33.6
UB35	457×152×74	30.0
UB34	457×152×82	27.3

Fig. 7 Behavior of RLTBC of a family of short span beams

Fig. 7 shows that, for a family of sections with critical effective length, the beam with the lowest value of torsional index, *x* (UB34), gives the minimum curve for the family. The RLTBC curves of these beams are straight lines. For beams with critical effective length, the bending strength is equal to the design strength of the sections and this corresponds to the case of plastic moment capacity of beams. Using this information, all the beams with the lowest value of *x* from each family were analyzed to obtain a minimum curve for the RLTBC of short span beams with critical effective length. The results for five beams and details of sections are given in Fig. 8.



No.	Section	*x*
UB43	406×140×46	38.8
UB10	762×267×197	33.2
UB20	610×229×140	30.5
UB29	457×191×98	25.8
UB60	254×146×43	21.1

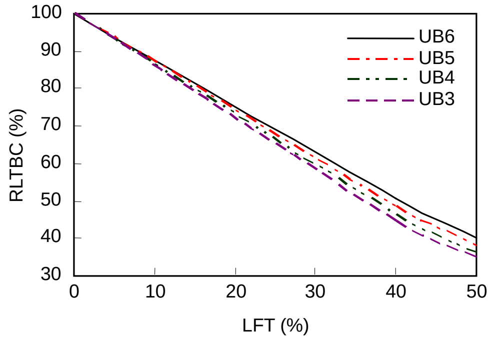
Fig. 8 Behavior of RLTBC of short span beams from five families

It can be seen from Fig. 8 that the beam with the lowest value of torsional index, *x* (UB60) gives the minimum curve for the RLTBC in a whole range of beams with critical effective length. The variation in the RLTBC curves of beams with the maximum and minimum values of *x* is quite small (less than 3% when the thickness loss is 50%).

4.4 Long span beams

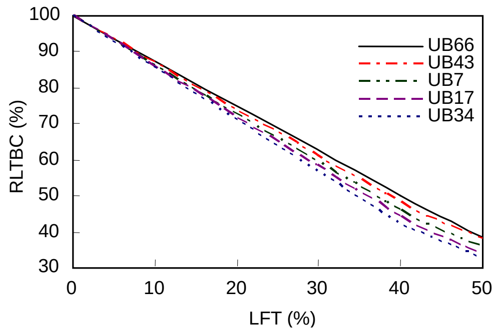
To obtain minimum curves for the RLTBC of long span beams, it is necessary to define the span that is long and widely used. Two sets of long spans in terms of the slenderness of beams, λ and the ratio of L_E/D are considered for the development of RLTBC minimum curves. These analyses were conducted for the case of long span beams with L_E/D=30. A family of sections with varying thickness loss were analyzed first to study the behavior of RLTBC long span beams with L_E/D=30. Details of the sections and the corresponding results are given in Fig. 9.

It is evident from Fig. 9 that the rate of reduction in the RLTBC of long span beams increases with decreasing half of the beam width (b=B/2) to flange thickness ratio, b/T. The beam with the lowest value of b/T (UB3) gives the minimum curve for the family. The variation in the RLTBC curves is small (less than 7% when the thickness loss is 50%). Based on the above findings, all sections that have the lowest value of b/T from each of the families were analyzed to obtain a minimum curve for the RLTBC of long span beams with L_E/D=30. The results for five beams and the details of the sections are given in Fig. 10.



No.	Section	b/T
UB6	914×305×201	7.51
UB5	914×305×224	6.36
UB4	914×305×253	5.47
UB3	914×305×289	4.81

Fig. 9 Behavior of RLTBC of a family of long span beams with L_E/D=30



No.	Section	b/T
UB66	203×133×289	6.97
UB43	406×140×253	6.36
UB7	838×292×224	5.48
UB17	610×305×201	4.96
UB34	457×152×201	4.06

Fig. 10 Behavior of RLTBC of long span beams from five families

It is evident from Fig. 10 that the beam that has the lowest value of b/T ratio (UB34) gives the minimum curve for the RLTBC for the whole range of the long span beams with L_E/D=30. It can also be seen that the variation in the percentage remaining capacities of beams with maximum and minimum values of b/T ratio is small (less than 7% when the thickness loss is 50%).

5 Minimum curves

As the effective length is the major factor that governs the bending strength of ordinary beams, minimum curves were developed for the RLTBC of ordinary beams in terms of their length. Using the results, minimum curves were obtained and are given in Fig. 11 for the following cases: (1) Short span beams with $L_{E(crit)}$; (2) Long span beams with L_E/r_y=200; (3) Long span beams with L_E/D=30.

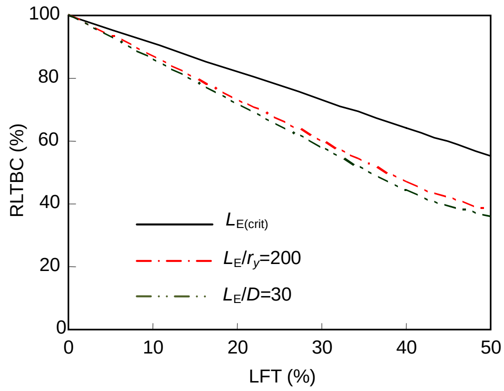


Fig. 11 Minimum curves for estimating the RLTBC of corrosion damaged beams

The minimum curves obtained for the short and long span beams may be used to estimate the RLTBC of intermediate span beams by interpolation. The variation in the minimum curves for the RLTBC of long span ordinary beams is approximately 12% when the loss of thickness in the flange is 50% (Fig. 11).

6 Conclusions

Assessment of the remaining capacity of corrosion damaged steel structures such as the lateral torsional buckling requires only information on the thickness loss of the elements. Therefore, analysis of corrosion effects can be undertaken using percentage loss of thickness in the elements. The varying thickness loss of corrosion damaged model, which was developed, based on the details of a large number of measurements (770 points on each beam) is sufficiently comprehensive to be used in analyzing the effect of corrosion in steel beams.

The proposed assessment method, which gives the remaining lateral torsional capacity of any I-beam manufactured in the UK can be used for an assessment of corrosion damaged steel beams. The proposed assessment method can help to make a reasonably accurate decision regarding the future of corrosion damaged members and will avoid inappropriate actions being taken (e.g., premature plant closures). It is believed that this method together with improvements to visual assessment procedures will be beneficial in terms of cost and safety.

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