Steel and Composite Structures, Vol. 10, No. 6 (2010) 475-487 475
DOI: http://dx.doi.org/10.12989/scs.2010.10.6.475

Buckling capacity of uniformly corroded steel members in terms of exposure time

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(Received June 15, 2009, Accepted October 22, 2010)

Abstract. Most of steel structures in various industries are subjected to corrosion due to environmental exposure. Corrosion damage is a serious problem for these structures which may reduce their carrying capacity. These aging structures require maintenance and in many cases, replacement. The goal of this research is to consider the effects of corrosion by developing a model that estimates corrosion loss as a function of exposure time. The model is formulated based on average measured thickness data collected from three severely corroded I-beams (nearly 30 years old). Since corrosion is a time-dependent parameter. Analyses were performed to calculate the lateral buckling capacity of steel beam in terms of exposure time. Minimum curves have been developed for assessment of the remaining lateral buckling capacity of ordinary I-beams based on the loss of thicknesses in terms of exposure time. These minimum curves can be used by practicing engineers for better estimates on the service life of corrosion damaged steel beams.

Keywords: steel structures; exposure time; corroded beams; lateral buckling.

1. Introduction

Corrosion occurs wherever water can leak onto an area of a superstructure where water and debris can accumulate (Kulicki *et al.* 1990). Corrosion may have many deleterious effects such as appearance, maintenance and operating cost, plant shutdown (for repair), and product contamination, loss of valuable products, product liability, safety and reliability. The effects of corrosion on steel structures vary from need of maintenance of non-structural element to catastrophic failure. The conditions created by corrosion can reduce the static, fatigue, fracture, or buckling strength of steel members, and reduce the overall carrying capacity of structural elements.

Steel structures can be affected by corrosion in many ways. The main effects can be loss of material from the surface which leads to thinner sections. The loss of material due to uniform corrosion may take place over a large area which results in the reduction of section properties for the member, such as area, moment of inertia, radius of gyration, and etc. These changes will occur in a non-linear manner because the geometric properties are related to square or cube of the dimensions. Furthermore, the position of the neutral axis may change. The reduction in member section dimensions leads to lower

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bending, shear, and axial capacity. Buckling capacity of members will be critically affected by the reduction in section modulus and slenderness ratio (Czarnecki and Nowak 2008). Also it has been pointed out by Gallon (1993) that the modes of failure of a member may be changed from one mechanism to another depending on the relative thickness loss in its various parts.

It has been pointed out by Smith (1993), corrosion may reduce the material strength and section size of a member. This can leads to a reduction in member stiffness thus causing excessive distortions. Effect of corrosion on the ultimate strength of steel plates subjected to in-plane compression and bending has been investigated by Nakai *et al.* (2006), a series of nonlinear finite element analysis have been performed for corroded plates subjected to in-plane compressive loads and bending moments. Rahgozar (2009) has proposed a series of minimum curves for universal I-beams which can be used along with the information on percentage loss of thickness in order to estimate the percentage remaining capacity of corrosion damaged beams.

Recently, formulae have been developed to relate the percentage of remaining shear capacity to percentage thickness loss of the corrosion damaged I-beam by Sharifi and Rahgozar (2010a). In another study by Sharifi and Rahgozar (2010b), two methods were proposed for assessment of remaining moment capacity in corrosion damaged I-beams based on thickness loss. Main objectives of this paper are: (1) to develop a corroded decay model for I-beams based on the average measured thickness of corroded beams compiled from three severely corroded I-beams (nearly 30 years old), (2) to develop a set of minimum curves for assessing the residual buckling capacity of corroded I-beams based on thickness loss in terms of exposure time.

2. Corrosion modeling

There are many factors that can influence the corrosion rate of steel and the available statistical data is not sufficient to formulate the analytical models. The rate of corrosion of steel in different environments has been evaluated using a large amount of data that has been collected on the loss of material of metal specimens in several studies (Townsened and Zoccola 1982, Rahgozar and Smith 1997, Rahgozar 1998). It has been observed that the corrosion rate is highly variable, depending on the local environment such as industrial, marine, or rural. The assessment of existing structures requires only the information concerning the magnitude of the loss of material at the time of assessment.

Therefore, in this paper analysis of corrosion effects is carried out using the thickness loss data presented in Table 1 were compiled from three samples of corrosion damaged universal I-beams, that were removed from a petro-chemical industry.

The corrosion decay model developed for the uniform thickness loss model sections is shown in Fig. 1 based on the thickness measurement of three universal I-beams as listed in Table 1.

$$\text{Thickness of the top flange} = T_N\,(1-\eta) \tag{1}$$

$$\text{Thickness of the bottom flange} = T_N\,(1-\eta) \tag{2}$$

$$\text{Average thickness of the flanges, } T_C = T_N\,(1-\eta) \tag{3}$$

$$\text{Thickness of the web, } t_c = t_N\,(1-\eta) \tag{4}$$

Table 1 Average measured thicknesses of corroded beams (Rahgozar 2009)

Element	As new	Beam 1	Beam 2	Beam 3
Average thickness of top flange	10.20	7.45	7.81	7.23
Average thickness of bottom flange	10.20	5.62	5.85	4.84
Average thickness of top and bottom flanges	10.20	6.54	6.83	6.04
Average thickness loss of top and bottom flanges	0.00	3.66	3.37	4.16
Percentage average thickness loss of top and bottom flanges	0.00	35.9%	33.3%	40.8%
Average thickness of upper part of web (0.75 h_w)	6.10	5.63	5.74	5.45
Average thickness of lower part of web (0.25 h_w)	6.10	3.16	4.32	3.18
Average thickness of web	6.10	5.01	5.39	4.88
Average thickness loss of web (mm)	0.00	1.09	0.71	1.22
Percentage average thickness loss of web	0.00	17.8%	11.7%	20.0%

Note: h_w = depth of web, all measurements are in millimeters

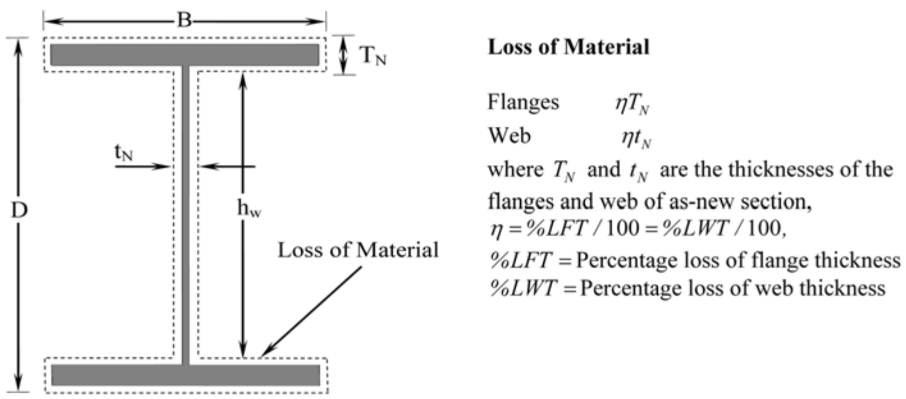


Fig. 1 Corrosion decay model for uniform thickness loss

where

$$\eta = \eta_F = \eta_w$$

$$\eta_F = \% LFT/100 \tag{5}$$

$$\eta_w = \%LWT/100 \tag{6}$$

In this case, the average thickness is used to calculate the percentage loss of both flanges and web thickness. For corrosion damaged I-beams of the same section size, the overall dimensions B, D and h_w can be considered as constant 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

$$S_{xC} = BT_C(D - T_C) + t_C\frac{h_W^2}{4} \text{ as corroded sections} \tag{7}$$

$$S_{xN} = BT_N(D - T_N) + t_N\frac{h_W^2}{4} \text{ as new sections} \tag{8}$$

Substituting for T_C and t_C (Eqs. (3) and (4)) into Eq. (7), gives the plastic modulus of corrosion damaged I-sections as

$$S_{xC} = \left(BT_N(D - T_N) + t_N\frac{h_W^2}{4}\right) - \eta\left(BT_N(D - 2T_N) + t_N\frac{h_W^2}{4}\right) - BT_N^2\eta^2 \tag{9}$$

3. Corrosion rate modeling

To estimate the loss of beam thickness, probabilistic corrosion rate modeling needs to be carried out in advance. Such modeling has to include time as a basic parameter and other random variables that describe the environmental effects on the corrosion rate. Kayser (1988) has collected data on the corrosion performance of actual steel bridges. It has been pointed out by Komp (1987) and Paik *et al.* (1998) that average corrosion penetration can be modeled with a good degree of approximation using the following power law function.

$$C(t) = At^B \tag{10}$$

where $C(t)$ is the average corrosion penetration in micrometers (10^{-3} mm); t is the time in years; A and B are parameters to be determined from regression analysis of the experimental data.

In Table 2 the mean values, coefficients of variation, and coefficients of correlation for A and B have been listed, and corrosion penetration versus exposure time is plotted in Fig. 2 (Kayser 1988). It should be noted that the determination of A and B involves a considerable degree of uncertainty. These parameters demonstrate the effects of different environmental conditions on the corrosion loss. In this study the marine environment parameter has been employed in order to estimate the propagation of

Table 2 Statistical parameters for A and B (Kayser 1988)

Parameters	Carbon steel		Weathering steel	
	A ($\times 10^{-3}$ mm)	B	A ($\times 10^{-3}$ mm)	B
	Rural Environment			
Mean value, μ	34.0	0.65	33.3	0.498
Coefficient of variation, σ/μ	0.09	0.10	0.34	0.09
Coefficient of correlation, ρ_{AB}	Not available		-0.05	
	Urban Environment			
Mean value, μ	80.2	0.593	50.7	0.567
Coefficient of variation, σ/μ	0.42	0.40	0.30	0.37
Coefficient of correlation, ρ_{AB}	0.68		0.19	
	Marine Environment			
Mean value, μ	70.6	0.789	40.2	0.557
Coefficient of variation, σ/μ	0.66	0.49	0.22	0.10
Coefficient of correlation, ρ_{AB}	−0.31		−0.45	

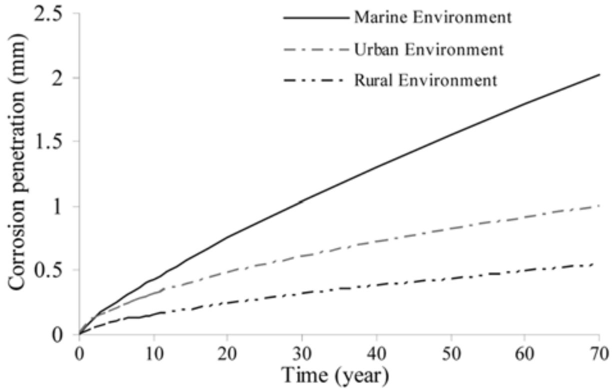


Fig. 2 Corrosion penetration versus exposure time for carbon steel in various environments

corrosion damage loss in steel beams. Table 2 presents the rate probabilistic parameters for two types of steel. Using aforementioned random variable parameters the probabilistic analysis has been employed to calculation the load-carrying capacity of corrosion damaged beams as it ages.

4. Lateral buckling capacity of I-beams

It is well known in the case of an idealized perfectly straight beam, there are no deformations 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 are interdependent, when the beam deflects laterally; the applied moment exerts a component torque about the deflected longitudinal axis which causes the beam to twist. This behavior, which is important for long unrestrained I-beams, is called lateral buckling (Trahair *et al.* 2001).

For a simply supported beam where lateral deflection and twist rotation are prevented at its ends and the flange ends are free to rotate in horizontal planes, the solution for elastic critical moment was given by Timoshenko and Gere (1961) as

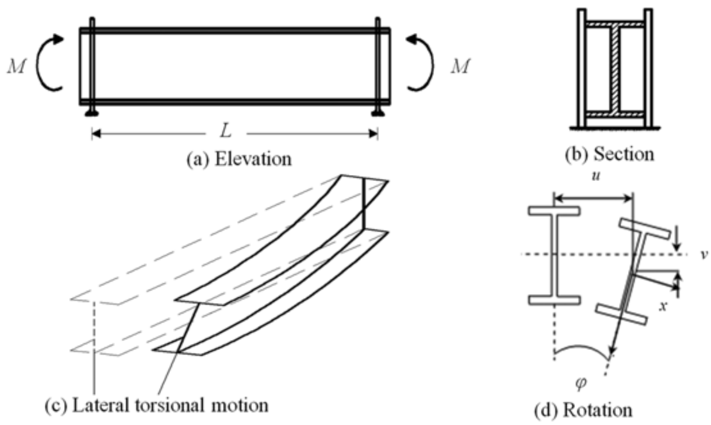


Fig. 3 Lateral buckling of a simply supported I-beam

$$M_E = \frac{\pi}{L}\sqrt{(EI_y GJ)\left(1 + \frac{\pi^2 EI_w}{L^2 GJ}\right)} \tag{11}$$

where, M_E is elastic critical buckling moment, EI_y is the minor axis flexural rigidity, GJ is the torsional rigidity, EI_w is the warping rigidity of the beam, and L is the beam length, Eq. (11) shows that the resistance to buckling depends on the geometric mean of the flexural resistance ($\pi^2 EI_y/L^2$) and the torsional resistance ($GJ + \pi^2 EI_w/L^2$).

The magnitude of the critical moment given by Eq. (11) 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 and the equation for the elastic critical buckling moment, given by Martin and Purkiss (1992) as follows

$$M_E = \frac{\pi}{L}\sqrt{\left(\frac{EI_y GJ}{\gamma}\right)\left(1 + \frac{\pi^2 EI_w}{L^2 GJ}\right)} \tag{12}$$

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

$$\gamma = 1 - \frac{I_y}{I_x} \tag{13}$$

The beam is assumed to be geometrically perfect, in reality, beams have initial curvature, twist, residual stresses, and the loads are applied eccentrically. The theory set out here requires modification to account for actual behavior. At intermediate slenderness ratios the behavior is dependent on the buckling moment and the plastic moment of resistance. The equivalent slenderness for beam buckling is defined by Trahair *et al.* (2001)

$$\lambda_{LT} = \sqrt{\left(\frac{\pi^2 E}{p_y}\right)\left(\frac{M_p}{M_E}\right)} \tag{14}$$

where, M_P is full plastic moment and p_y is the yield stress. Eq. (14) is not convenient for design purposes as the calculation of M_E is cumbersome. To simplify the problem, two further section properties are required and are defined by the following Eqs. (15-16) for sections that are symmetric about both axes. For sections symmetric about the major axis, the buckled shape (u) and the slenderness factor (v) for the I-beam with equal flanges are defined by the following equations (Trahair *et al.* 2001)

$$u = \sqrt[4]{\left(\frac{4\gamma(M_p/p_y)^2}{A^2 h_s^2}\right)} \tag{15}$$

$$v = \sqrt[-4]{\left(1 + 0.156\frac{\mathrm{J}}{A h_s^2}\left(\frac{L_E}{r_y}\right)^2\right)} \tag{16}$$

$$r_y^2 = \frac{I_y}{A} \tag{17}$$

where 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, J is the polar moment of inertia, A is the cross section area and x is the torsional index. The equation for elastic critical buckling moment can be given as

$$M_E = \frac{\pi^2 E S_x}{(\lambda u v)^2} \tag{18}$$

Eqs. (14) and (18) can be shown by,

$$\lambda_{LT} = \lambda u v \tag{19}$$

The intermediate slenderness cases between elastic buckling (M_E) and full plastic moment (M_P), a 'Perry-Roberston' approach is used in BS 5950 and the buckling resistance moment, M_b, is given as the least square root of the following equation

$$(M_P - M_b)(M_E - M_b) = \xi_{LT} M_E M_b \tag{20}$$

where η_{LT} is a coefficient to allow for initial imperfections and residual stresses
The least square root of Eq. (20), M_b, is given by

$$M_b = \Phi_B + \sqrt{\Phi_B^2 - M_E M_P} \tag{21}$$

where,

$$\Phi_B = \frac{M_P + (\xi_{LT} + 1) M_E}{2} \tag{22}$$

It was noticed from tests that buckling does not occur at values of $(M_P / M_E)^{1/2}$ of less than 0.4, thus a limiting equivalent slenderness ratio, λ_{LO}, is defined by (Trahair *et al.* 2001):

$$\lambda_{LO} = 0.4 \sqrt{\left(\frac{\pi^2 E}{p_y}\right)} \tag{23}$$

The imperfection coefficient, ξ_{LT} is defined by (Trahair *et al.* 2001):

$$\xi_{LT} = 0.007(\lambda_{LT} - \lambda_{LO}) \qquad \text{for rolled sections} \tag{24}$$

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 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.

5. Buckling capacity evaluation method

Lateral 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 buckling capacity of beams. In this paper, the corroded model in Section 2 and the theory given in Section 4 were used for the evaluation of remaining lateral buckling capacity of corrosion damaged ordinary beams. Eq. (21) has been used with a few approximations to derive Eq. (26) which is in terms of the equivalent slenderness, λ_{LT}, material properties, p_y and modulus of elasticity, E.

$$\frac{M_b}{S_x} = P_b \approx \frac{1.022\pi^2 E\left(\lambda_{LT}^2 + \frac{0.007\pi^2 E}{p_y}\lambda_{LT} + \frac{\pi^2 E}{p_y}\left(1 - 0.0028\left(\frac{\pi^2 E}{p_y}\right)^{\frac{1}{2}}\right)\right)}{\left(\left(\lambda_{LT}^2 + \frac{0.007\pi^2 E}{p_y}\lambda_{LT} + \frac{\pi^2 E}{p_y}\left(1 - 0.0028\left(\frac{\pi^2 E}{p_y}\right)^{\frac{1}{2}}\right)\right)^2 - \frac{\pi^2 E}{p_y}\lambda_{LT}^2\right)} \tag{25}$$

where

$$\lambda_{LT} = uv\left(\frac{L_E}{r_y}\right) \tag{26}$$

Eq. (26) shows that the design strength is an important factor on the lateral buckling capacity of beams. In order to verify the effect of design strength on the remaining buckling capacity of corrosion damaged beams, a universal beam, *UB60*, with the model developed in Section 2 was analyzed. The probabilistic analysis performed in order to calculate the load-carrying capacity of the corroded beam as it ages. The beam was analyzed for different design strength as shown in Fig. 4. It should be noted, the length and the resistance conditions were assumed to be the same in all cases.

The buckling capacity decreases at a considerable rate as a beam gets older. The buckling is the critical failure mode at the early stages of corrosion for the decay modeling beams. Also it is evident from Fig. 4 that the variation in these curves for different values of design strength is negligible. When the above analyses were carried out 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 lateral buckling moment of a beam as it was damaged by corrosion need not be considered when developing minimum curves. Any value for the design strength may be used when developing minimum curves.

Since the effective length is an important parameter for developing the time dependent load carrying-capacity of a beam, it should be investigated in advance. Therefore, short and long span beam is considered here. In this case, a family of sections with varying thickness loss was analyzed to study the lateral buckling capacity of beams as they get old in short span beams. Effective length of the beams

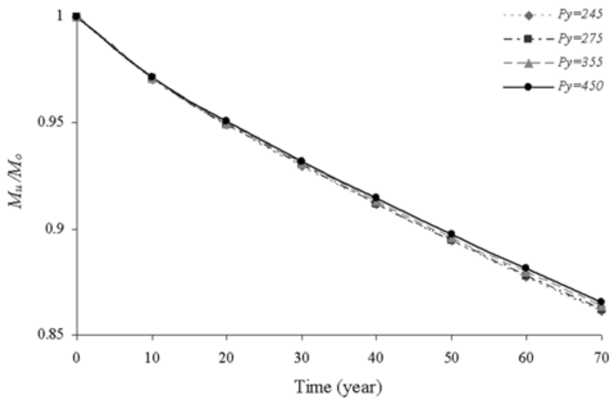


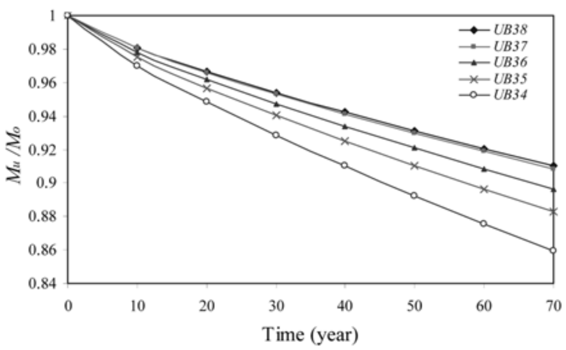
Fig. 4 Effect of design strength on the remaining lateral buckling moment of a corroded beam with respect to exposure time

was taken as the critical effective length. The remaining lateral buckling capacity 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. 5.

For a family of sections with critical effective length, the beam with the lowest value of torsional index, x, gives the minimum curve for the family (Fig. 5). For beams with critical effective length, the bending strength is equal to design strength of the sections and this corresponds to the case of plastic moment capacity of beams. Using this information, all the beams with lowest value of x from each family were analyzed to obtain a minimum curve for the assessment of lateral buckling capacity with age of short span beams with critical effective length. The results for five beams and details of sections are given in Fig. 6.

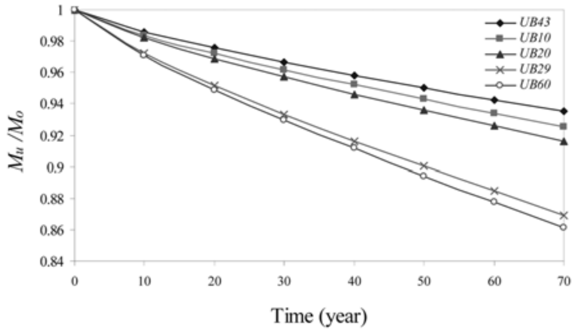
As shown in Fig. 6, the beam with lowest value of torsional index, x, gives the minimum curve for the lateral torsion buckling moment in a whole range of beams with critical effective length.

In order to obtain minimum curves for the remaining lateral buckling moment 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 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 these long span beams with L_E/D = 30. Details of the sections and the corresponding results are given in Fig. 7.



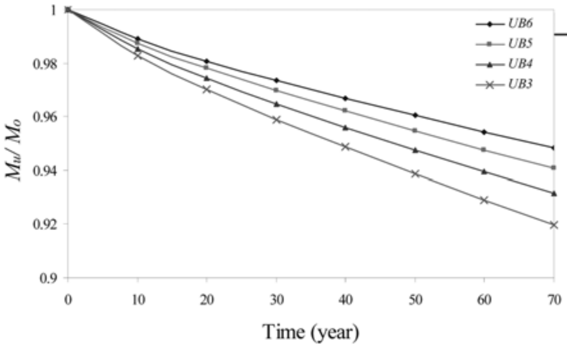
No.	Section	x
UB38	457×152 *UB* 52	43.9
UB37	457×152 *UB* 60	37.5
UB36	457×152 *UB* 67	33.6
UB35	457×152 *UB* 74	30.0
UB34	457×152 *UB* 82	27.3

Fig. 5 Variation of lateral buckling capacity for a family of short span beams with respect to exposure time



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

Fig. 6 Variation of lateral buckling capacity for five family of short span beams with respect to exposure time



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

Fig. 7 Variation of lateral buckling capacity of a family of long span beams with respect to exposure time for ($L_E/D = 30$)

As shown in Fig. 7 the rate of reduction in the lateral buckling moment of long span beams increases with decreasing *b/T* ratio. The beam with lowest value of *b/T* (*UB3*) gives the minimum curve for the family. 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 long span beams with $L_E / D = 30$. The results for five beams and the details of the sections are given in Fig. 8.

It is evident from Fig. 8 that the beam that has the lowest value of *b/T* ratio (*UB34*) gives the minimum curve for the whole range for the long span beams with $L_E/D=30$.

The effective length is the major factor that governs the bending strength of ordinary beams; minimum curves were developed for assessment of the remaining lateral buckling capacity of ordinary beams in terms of their length. Using the results, minimum curves were obtained and are given in Fig. 9 for different cases:

Minimum curves of short and long span beams can be used to estimate the remaining lateral buckling capacity of intermediate span beams by using interpolation.

6. Conclusions

The varying thickness loss of corrosion damaged model, which was developed based on the detailed

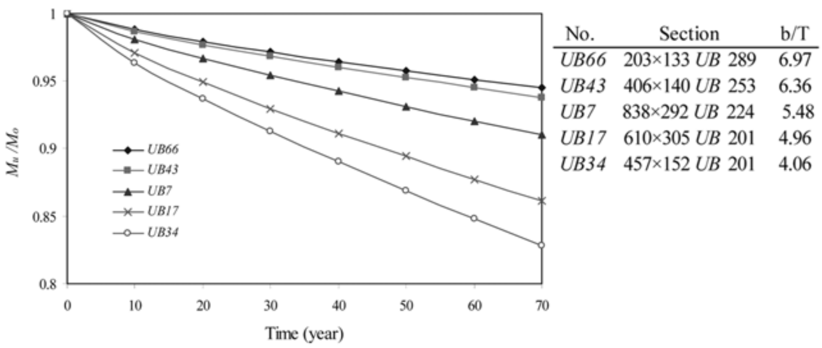


Fig. 8 Variation of lateral buckling capacity of five family of long span beams with respect to exposure time

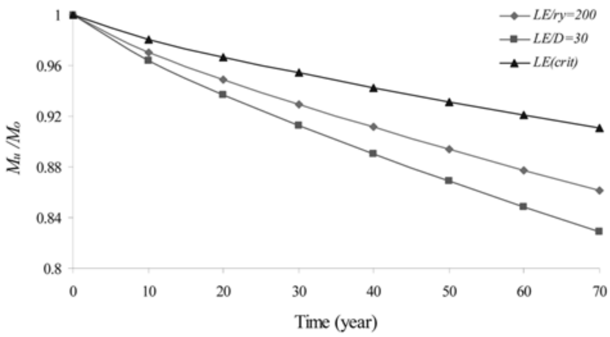


Fig. 9 Minimum curves for estimating the lateral buckling capacity of corrosion damaged beams with respect to exposure time

measurements of three samples of corrosion damaged beams, is very useful in analyzing the effect of corrosion in steel beams. A family of minimum curves was obtained for all sections that have the lowest value of b/T and torsional index (x), for short and long span beam to estimate the residual lateral buckling capacity of corroded steel I-beam which manufactured in the UK. For all practical purposes, the proposed minimum curves can be used along with the information on the material loss (percentage loss of thickness) to estimate the percentage remaining lateral torsional capacity of corroded damaged members in terms of exposure time.

Symbols

A corrosion parameter or cross-sectional area
B corrosion parameter
C average corrosion penetration
b width of compression flange
E modulus of elasticity
EI_x flexural rigidity of the beam about x axial

EI_y	flexural rigidity of the beam about y axial
EI_w	warping rigidity of the beam
GJ	torsional rigidity
h_s	distance between flange shear centers
I_x	moment of inertia about major axis
I_y	moment of inertia about minor axis
J	polar moment of inertia
L	length of beam
L_E	effective length
$L_{E(crit)}$	critical effective length
M_b	buckling resistance moment
M_E	elastic critical buckling moment
M_P	full plastic moment
p_b	buckling strength
p_y	yield stress
r_y	radius of gyration about minor axis
S_X	plastic section modulus of the beam
T	flange thickness
u	buckled shape
v	slenderness factor
x	torsional index
γ	correction factor
α_b	constant
λ_{LT}	equivalent slenderness
λ_{LO}	limiting equivalent slenderness
ξ_{LT}	imperfection coefficient (Perry coefficient)
M_u, M_o	random variable representing the lateral buckling moment of a corroded or uncorroded beam

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