PROBABILISTIC COLUMN-LIKE BUCKLING STRENGTH FOR STIFFENED STEEL PLATES UNDER UNIAXIAL COMPRESSION

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1. INTRODUCTION

Longitudinally stiffened steel plates exhibit two distinct type of buckling behaviours under compression, i.e. a plate-like behaviour and a column-like behaviour. Plate-like buckling refers to the global buckling of the whole panel along with the longitudinal stiffeners, where the stiffened plates possess a significant post-buckling strength reserve due to consideration of the effect of longitudinal edge supports. On the other hand, column-like buckling does not have such post-buckling strength reserve. In case of column-like buckling, the stiffened plate is considered as a series of unconnected compression “struts” where a “strut” consists of a longitudinal stiffener and the associated subpanel width in between two stiffeners. The column behaviour is simply achieved by removing the effect of longitudinal edge supports. Wider stiffened plates with low aspect ratio (≤1.0) exhibits column-like behaviour. By definition, the column-like buckling strength for a stiffened plate is lower than that of plate-like strength. The current Japanese Specification for Highway Bridges (JSHB) neither distinguish between plate-like and column-like buckling, nor considers the column-like buckling. This may lead to overestimation of buckling strengths for stiffened plates exhibiting column-like behaviour.
Ultimate buckling strengths for stiffened plates exhibiting column-like behaviour are investigated in this paper through nonlinear elasto-plastic finite element (FE) analysis. Furthermore, probabilistic information of the buckling strengths are obtained employing Monte Carlo simulation in association with the response surface method. Outcome of the current study can be used as an important baseline for developing a reliability based ultimate strength curve.

2. DETERMINATION OF COLUMN-LIKE BUCKLING STRENGTH

2.1 Stiffened plate model selection

Stiffened plates as shown in the Fig. 1, with 3 equidistant flat plate longitudinal stiffeners and with an aspect ratio α =1 are selected to produce column-like behaviour. Here, *a*, *b* and b_s are the plate length, plate width and subpanel width respectively. Cross-sectional dimensions of the longitudinal stiffeners are determined in such a way that the relative stiffness of a longitudinal stiffener satisfies the required relative stiffness, specified in the JSHB. For parametric study, the reduced slenderness parameter R_R (R_R defined in the JSHB) varied from 0.4 to 1.4 and the plate thickness *t* varied from 10 mm to 90 mm. Steel material grade considered in this study is SM570.

2.2 Nonlinear FE analysis

Due to the symmetricity of geometric and loading conditions, only the shaded part in Fig. 1 was modeled for the FE analysis. Fig. 2 shows the boundary conditions. Both material and geometric nonlinearity were considered for the nonlinear elastoplastic FE analysis. Material nonlinearity was modeled by applying Mises plasticity and isotropic strain hardening theory. As a source of variability of the buckling strengths, initial imperfections of three types i.e. initial out-of-plane deflections with a whole-plate mode, local mode (local deflection in a subpanel) and residual stresses were simulated simultaneously in the FE models. Based on respective mean value and standard deviations, 36 different combination of imperfections were considered for a single stiffened plate model to get the variation in column-buckling strengths for that model. A total of 756 FE analysis were carried out for such 21 different stiffened plate models with varying thickness and R_R values.

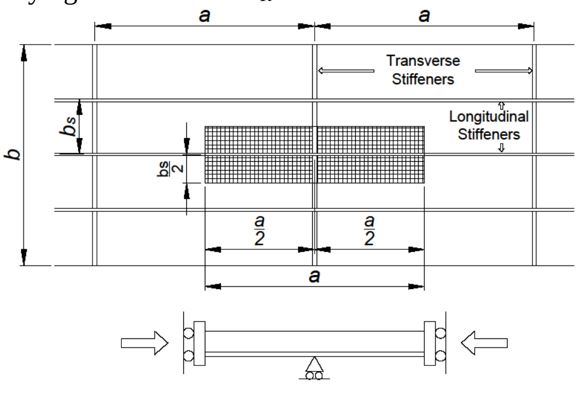

Fig.1 Stiffened plate model for column-like buckling.

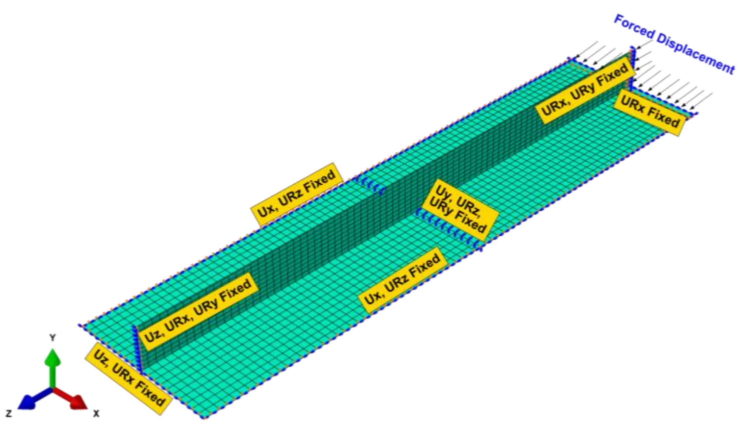

Fig.2 FE model and boundary consitions.

Keywords: Stiffened plates, column-like buckling, initial imperfections, response surface, Monte Carlo simulation.
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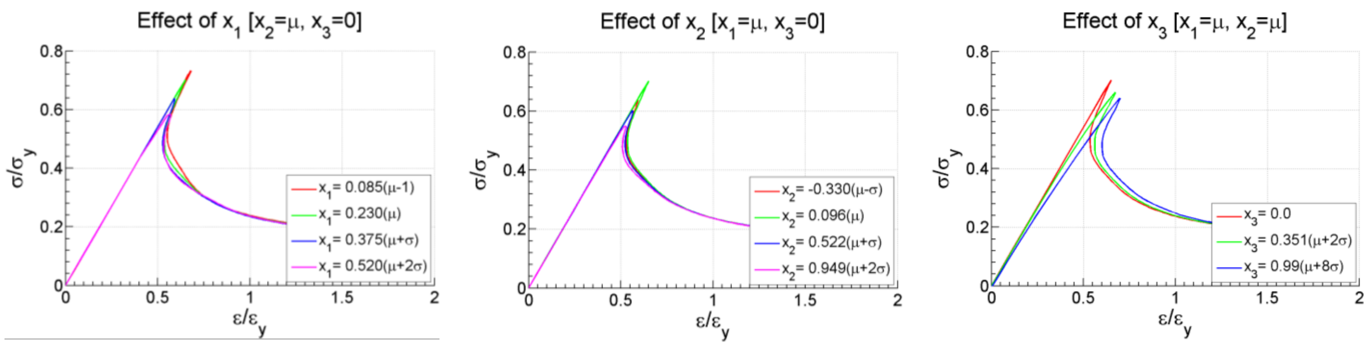
2.3 FE analysis result

Fig. 3 presents the normalized stress (σ/σ_y) – strain ($\varepsilon/\varepsilon_y$) curves obtained from FE analysis. Ultimate buckling strengths are determined from the peak stress values of the stress-strain curves. Each subfigure of Fig.3 shows the effect of variation of one individual imperfection type while the other two are constant, for an example case of R_R = 1.0, t = 30. Here, $x_1 = \sigma_{rc}/\sigma_y$ normalized residual stress, $x_2 = 1000\delta_{01}/a$ normalized initial whole-plate deflection and $x_3 = 150\Delta/b_s$ normalized initial local deflection.



a) Effect of residual stress b) Effect of initial whole-plate deflection c) Effect of initial local deflection

Fig.3 Effect of variation of initial imperfections.

3. PROBABILISTIC ANALYSIS

The probabilistic distributions of column-like buckling strengths were obtained by employing Monte Carlo simulations (MCS) in association with the response surface method. The response surface function was approximated as the following second-order polynomial of three independent variables i.e. x_1, x_2 and x_3:

$$\frac{\sigma_{cr}}{\sigma_y} = \sum p_{ijk} x_1^i x_2^j x_3^k; \quad i = 0\sim2;\ j = 0,2;\ k = 0,2;\ i + j + k \leq 6 \tag{1}$$

where, σ_{cr} is the ultimate strength, σ_y is the yield strength, p_{ijk} are the coefficients of the polynomial, determined by nonlinear multiple regression analysis. Fig. 4 presents the response surface in mesh grid for R_R = 0.8 as an example, showing variation of strengths with respect to variation of x_2 and x_3 when x_1 is constant. The circular dots depict the nonlinear FEA results. In the MCS x_1, x_2 and x_3 were considered as three independent random variables. After randomly generating a set of x_1, x_2 and x_3 values according to their respective probability density functions, the response surface function was used to determine the buckling strength for that set. To obtain converged MCS result, 100,000 iterations were performed. Fig. 5 shows the relative frequency distribution with mean and standard deviation for column-like buckling strengths at R_R = 0.8.

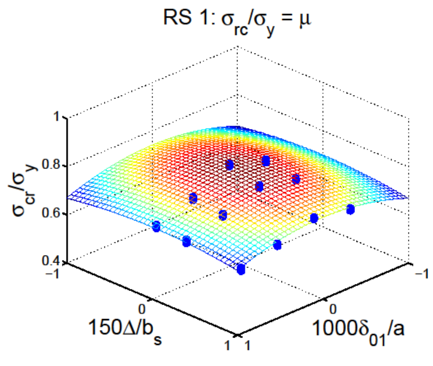


Fig.4 Response surface for R_R = 0.8

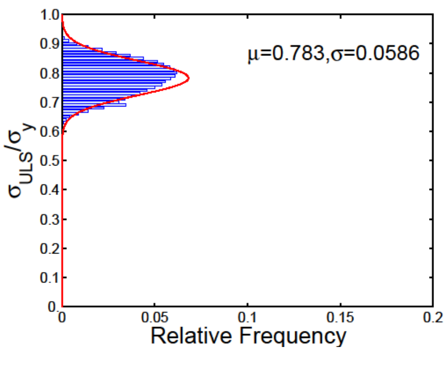


Fig.5 Relative frequency distribution

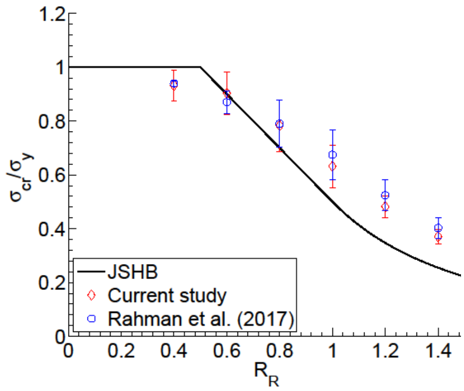


Fig.6 Comparison of probabilistic results

4. RESULT AND DISCUSSION

In Fig.6, the study result was compared with the current JSHB code as well as previous study of the authors (Rahman et al. 2017) where the effect of longitudinal edge support were considered in the column model. Top and bottom error bars represent the 95% and 5% probability of nonexceedance (p_f) while the center mark shows the mean values. It was found that for p_f =5%, current study result is lower than that of Rahman et al. (2017), which is expected. However, the JSHB code is very conservative for $R_R \geq 0.8$.

REFERENCES

Rahman, M., Okui, Y., Shoji, T. and Komuro, M. (2017). "Probabilistic ultimate buckling strength of stiffened plates, considering thick and high-performance steel." *Journal of Constructional Steel Research*, 138, pp. 184-195.