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Numerical Evaluation of Steel Columns Buckling under Cyclic Loading

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ABSTRACT

This paper aims to exhibit the use of finite element analysis as a numerical method to validate the experimental results of steel column stability under cyclic loading. Finite element package ANSYS was utilized for this purpose. The structural behavior of hollow box steel columns under the merged action of a constant axial load and cyclic lateral loads was investigated. Hot rolled unstiffened steel box column sections were simulated as cantilever-type columns as in common usage in the bridge piers. A nonlinear buckling finite element analysis was carried out in which both material and geometric nonlinearities were taken into account. The results show a reasonable agreement with Load and Resistance Factor Design (LRFD) formula. In addition, transient dynamic analysis was executed to determine the dynamic response of the specimens under the action of two loading tests: monotonic test and three cycle test. Isotropic strain-hardening and kinematic strain-hardening were included in simulating plastic deformation. It is shown that the non-linear finite element analysis shows good capabilities in simulating the buckling behavior of a steel column under cyclic loading.

KEYWORDS: Finite element method, Cyclic loading, Bridge piers, Non-linear buckling of steel columns, Transient dynamic analysis.

INTRODUCTION

Hollow box sections are extensively used in steel bridge piers worldwide. Unlike the columns in structures, these piers are usually under low axial force to squash load ratio, and therefore are formed with relatively high width-thickness ratios of component plates. This makes them subject to damage by local buckling throughout a severe seismic event (Kumar and Usami, 1996). It is clear that from a point of view of earthquake resistant design, cantilever column type of piers is very critical. The most common shapes of steel bridge piers are usually thin walled box sections or tube sections (Usami et al., 1992). In recent years, the use of finite element analysis in studying complicated steel structural components has increased due to the advancement in the knowledge and capabilities of computer software and hardware. Buckling of structural steel columns under cyclic loading has a great space in literature. The aim here is to establish a better understanding and improve the techniques in the evaluation of buckling of steel structural columns subjected to repeated loading. Kumar and Usami (1996) presented a damage model which is developed and established for cyclic loading tests of hollow box columns forming bridge piers. The box columns were checked under a firm axial load and repeated lateral loads. It was found that the degree and kind of damage allowed depend on parameters of the structure and the loading history. Usami et al. (1992)

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tested nine cantilever thin-walled steel box columns modeling steel bridge piers under constant compressive axial loads and cyclic lateral loads. The test specimens were made of stiffened box sections (box sections stiffened by longitudinal ribs and shutters). They inspected the probability of increasing the column ductility by using two concrete-filled columns and one hybrid stiffened column in which a higher grade of steel is used for the stiffeners than for the plate panels. It has been shown that the concrete-filled columns and the hybrid stiffened column increased both flexibility and energy-absorption capacity meaningfully. Fukumoto and Kusama (1985) presented an experimental study of the inflexible cyclic load-deformation behavior of welded built-up short columns of square box–section located below the cyclic axial loading. The tests revealed that the axial stiffness and greatest capacity of plate elements became worse for compression loading with each cycle. Kawashima et al. (1992) tested twenty-two model stiffened box steel bridge piers under applied lateral loading and shaking table testing in order to assess their strength and ductility. The specimens were hollow and partially filled-concrete. The strength of samples with concrete infill was increased; however their deformation capacity was decreased. The samples with greater section slenderness usually had lower ductility capacities. Ge and Usami (1992) presented an experimental study on the strength and distortion of concrete-filled square box stub columns. The study revealed that high strength and high flexibility can be assumed from the concrete-filled composite columns. The fracture of the concrete-filled columns depended to a large extent on the fracture of the filled concrete part and thus special attention must be given to the shaping of the concrete. Usami and Ge (1994) tested eleven cantilever thin-walled steel box column specimens under constant axial loads and cyclic lateral loads. The test samples were of un-stiffened and stiffened box sections. It has been shown that the concrete-filled columns increase both flexibility and energy-preoccupation capacity significantly. Otsuka et al. (1998) demonstrated an experiment using non-concrete-filled and concrete-filled pier specimens that were subjected to three cycles of loading with a certain magnitude of lateral displacement. It was found that in specimens filled with concrete, unlike non-filled specimens, local buckling hardly occurs at the stiffened plate and both strength and ductility are significantly higher. Ge et al. (2000) studied the cyclic inelastic behavior of stiffened steel box columns weakened by local and global instability under constant compressive axial load and repeated lateral loading. In the analysis, a modified two-surface plasticity model was operated to model material non-linearity. It was shown that the modified two-surface model is a sufficient model for predicting the repeating hysteretic behavior of both thin- and thick-walled steel box columns. Usami et al. (2000) evaluated the ultimate strength and flexibility capacity of stiffened steel box columns which failed by local and global interaction instability under a constant axial force and repeated lateral loading. The experiment results denoted that the two-side repeated loading case is the most serious case, because the strength decline due to repeating loading is very large. Zheng et al. (2000) tested thin-walled steel box columns below the combined compression and bending loads to simulate the loading conditions under horizontal earthquake conditions. The numerical analyses presented that the key parameters affecting the flexibility in terms of failure strains are the magnitude of the axial force, flange width-thickness ratio and stiffener`s slenderness ratio. Dicleli and Mehta (2007) simulated the cyclic axial force-distortion behavior of steel braces including buckling using nonlinear finite element software. The nonlinear repeating axial force-distortion simulation was done for braces with box sections. It was found that the exactness of the shapes of the analytical hysteresis loops and the energy dissipated relative to the experimental ones is sufficient for analysis and design purposes in customary action. Pavlovčič et al. (2010) presented tests on slender thin-walled box columns, vulnerable to changeability of both types: to global Euler buckling as well as to local buckling of steel plates. The results pointed out that, for precise numerical simulation of elements in compression, it is not avoidable to carry out various initial imperfections very cautiously based on real data provided from various accompanying trials and measurements. Talikoti and Bajoria (2005) described a method which can be adopted to enhance the torsional and also the distortion durability of skinny-walled cold-formed steel columns used in pallet racking systems. The column sections were constructed distortionally stronger by adding simple spacers. It was found that the use of spacers at suitable intervals helps not only in enlarging load carrying capacity but also in varying the mode of failure due to improvement in the torsional hardness of the sections. Lue et al. (2009) investigated the compressive strength of slender C-shaped cold-formed steel parts with web openings. It was found that the reduction in the compression durability of the samples with web openings appears to be negligible. Furthermore, it seemed that the finite element analysis was able to predict the greatest loads and failure modes of the samples. In the absence of test data and for the purpose of initial design, the finite element appears to provide an encouraging soothsaying skill. Good agreement between simulations of Finite Element Models (FEMs) and experimental observations confirms that FEMs are suitable for predicting the buckling behavior of structural steel columns under cyclic loading.

FINITE ELEMENT ANALYSIS (FEA)

In this study, the computer software ANSYS is used for performing finite element analysis to simulate the cyclic behavior of experimental results of hot rolled steel columns executed by Kumar and Usami (1996). Two hot-rolled box column finite element models made of SS400 steel (equivalent to American Society for Testing and Materials (ASTM) A36) are formulated and built up as cantilever-type columns imitating fixed conditions at the foundation and free at the top as in common practice in bridge piers. The two un-stiffened steel box columns (coded as U70-40 and U45-40) have a width-to-thickness ratio parameter R_f = 0.70 and R_f = 0.45, respectively, while the column slenderness ratio parameter is λ=0.40 for both columns. The geometry of the two columns is illustrated in Fig. 1 and Table 1. According to the American Iron and Steel Construction (AISC) code, both columns U70-40 and U45-40 lie in the inelastic range. The U70-40 column is non-compact while the U45-40 column is compact. The modulus of elasticity of steel E_s = 217,000 MPa, Poisson`s ratio for steel ν = 0.27, and the density ρ = 78.6 kN/m^3 for SS400 steel. Eight node solid finite element with three structural degrees of freedom per node will be utilized in the analysis. The behavior of steel is assumed to have bilinear stress-strain relationship as shown in Fig. 2.

Table 1. Geometric properties of test sections (Kumar and Usami, 1996)

Specimen	h (mm)	b (mm)	d (mm)	t (mm)	I_x (mm^4)	r_x (mm)	A (mm^2)
U70-40	1,217.00	219.00	162.00	5.91	22,126,718.37	69.09	4,646.3238
U45-40	779.00	140.00	104.00	5.91	5,984,664.46	44.47	3,026.98

Two approaches are used for simulating the steel columns: non-linear buckling analysis and transient dynamic analysis. Nonlinear buckling analysis is commonly the most effective approach and is therefore advised for the design and evaluation of real structure behavior. It employs a nonlinear static analysis with progressively increasing loads to try to obtain the load level at which the structure becomes unstable. The nonlinear buckling analysis takes into account the geometric and material nonlinearities. However, linear eigen buckling analysis is first conducted to obtain the hypothetical buckling pressure of the ideal linear elastic structure and the buckled mode shapes used to find imperfections that are incorporated in the nonlinear analysis. It is also an effective way to verify the completeness and correctness of the numerical model. If a structure is faultlessly symmetric (that is, its mesh and load geometry are both symmetric), non-symmetric buckling does not take place numerically, and a nonlinear buckling analysis does not work because non-symmetric buckling responses cannot be initiated. Although small disturbance loads can also be introduced to attend the same purpose, it is not a perfect method because it is difficult to calculate how large the loads should be and where to apply them. Also, larger perturbation load can change the problem completely.

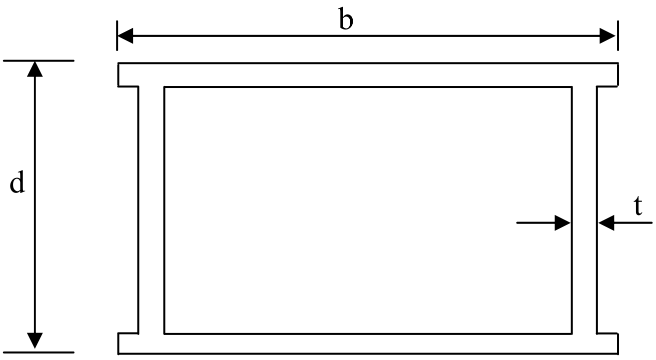


Figure (1): General layout of the column sections

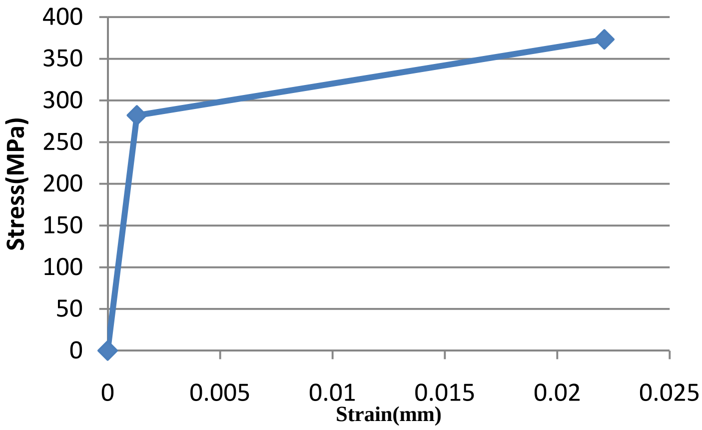


Figure (2): Bilinear stress-strain behavior

In this study, small geometric imperfections (comparable with those caused by manufacturing a real structure) were established to cause and activate the buckling responses. The imperfection magnitudes are mostly dependent on the geometry and should be in the same range as the manufacturing tolerance (typically less than one percent of the section dimensions), so that they do not change the problem during the analysis.

The load must be permitted to increase using automatic time stepping increments, so that the anticipated critical buckling load can be predicted precisely. It typically includes many load steps with sub-steps to simulate the cyclic load behavior. The equilibrium and convergence are checked through Newton-Raphson equilibrium algorithm at each sub-step, which provides convergence at the end of each load increment within endurance limits equal to 0.001. Failure for each model was recognized when the solution for 0.001 MPa load increment was not converging, which means that unstable condition is reached. The automatic time stepping predicts and controls the time step size for all sub-steps in a load step.

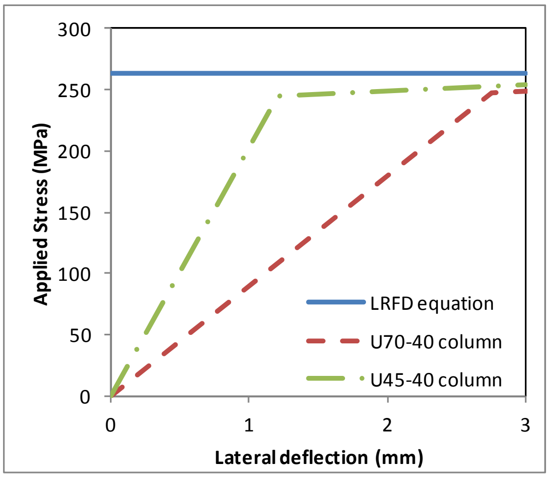


Figure (3): NLFEM applied stress *versus* deflection curve for column sections under axial load

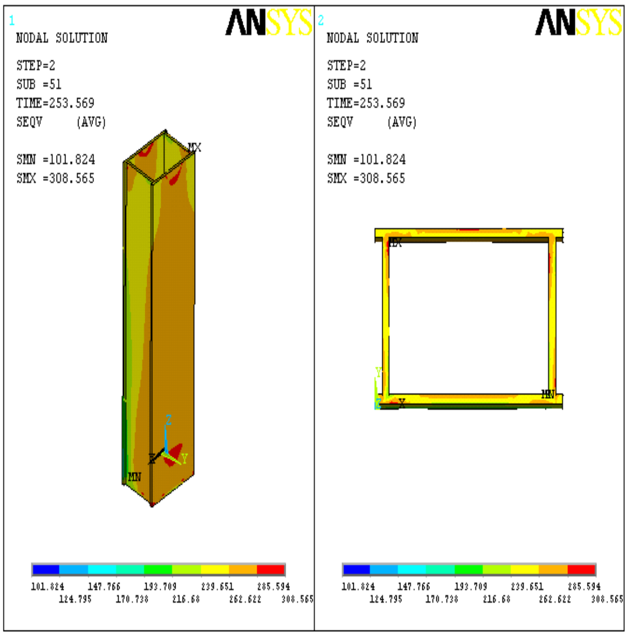
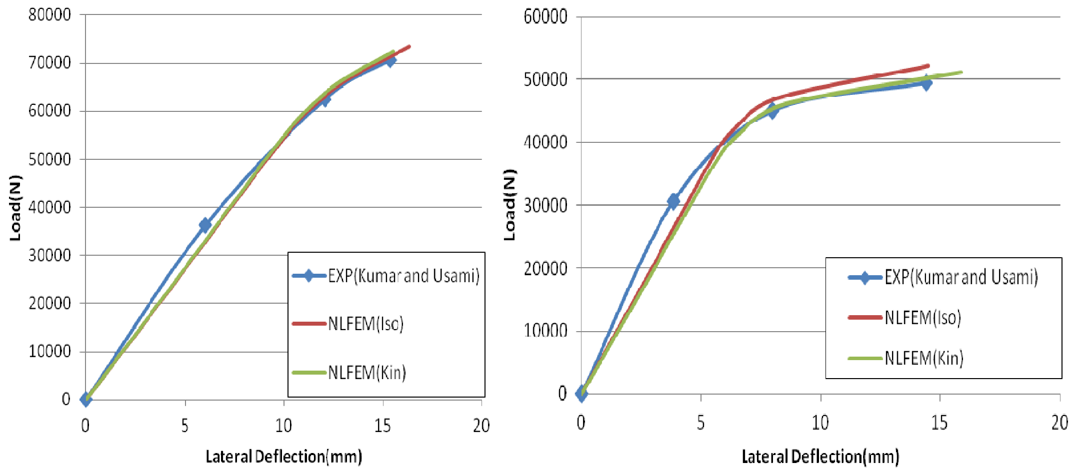
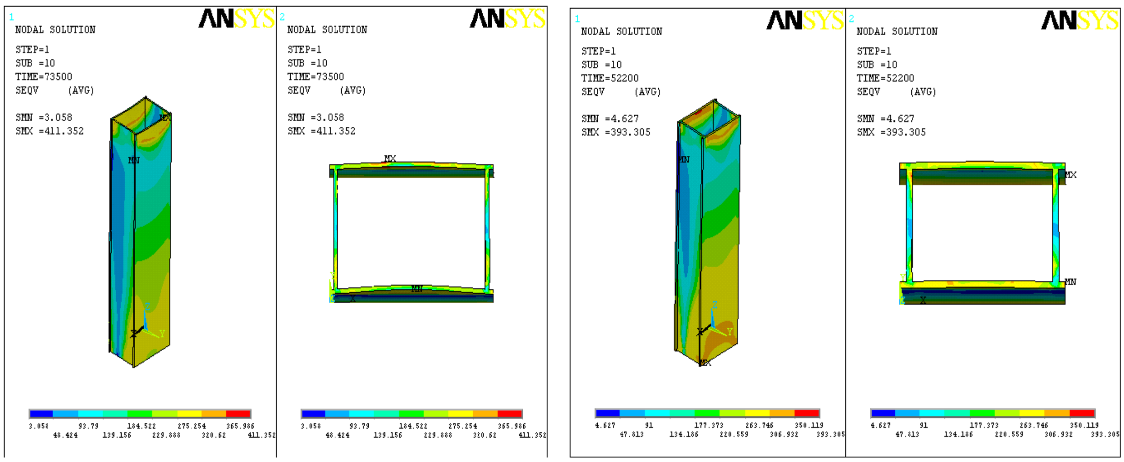


Figure (4): NLFEM Von-Misses stress distribution for section U45-40



(a) (b)

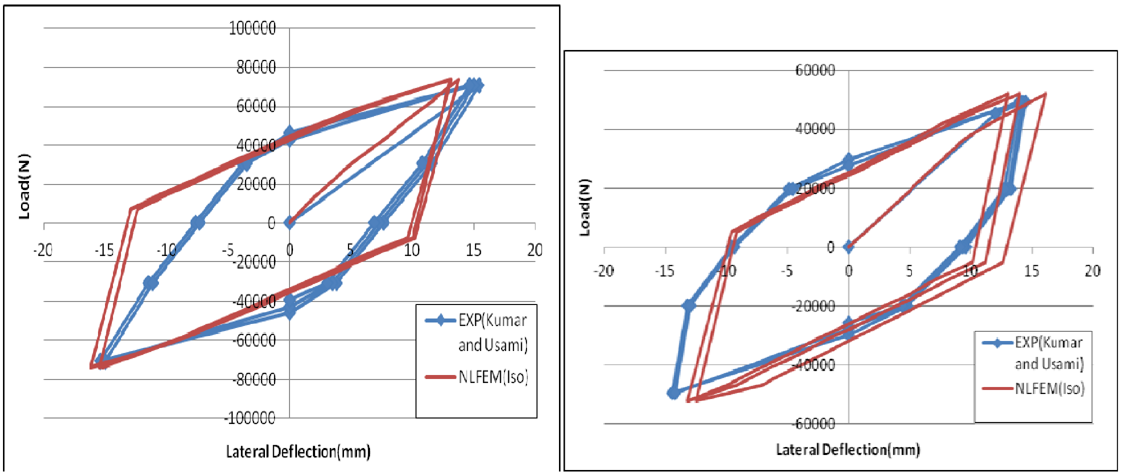
Figure (5): NLFEM load-deflection curve for column section (a) U70-40, and (b) U45-40



(a) (b)

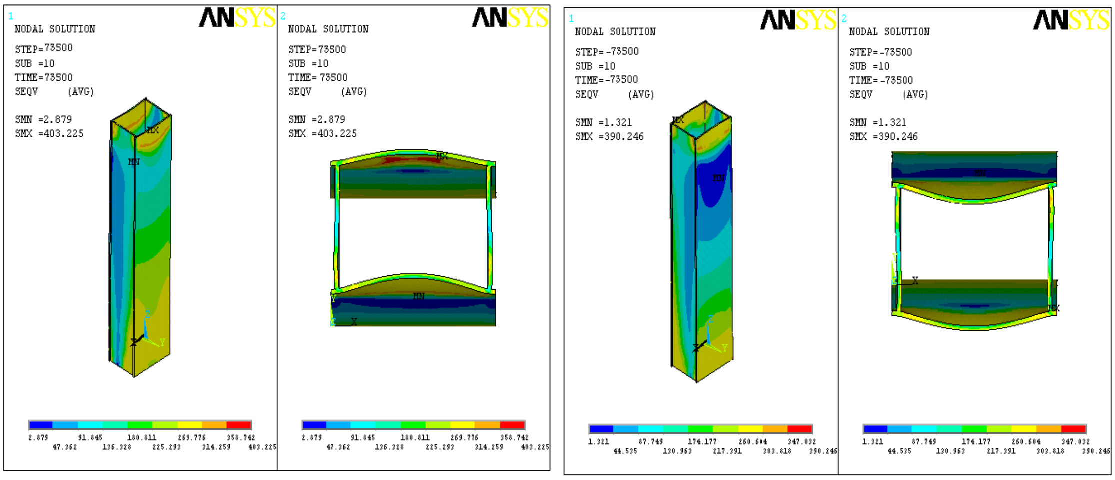
Figure (6): NLFEM Von-Misses stress distribution with isotropic hardening for section (a) U70-40, and (b) U45-40

Transient dynamic analysis, occasionally called "time history analysis", is used to settle the time-varying displacements, stresses, strains and forces as it responds to any joining of static, transient and harmonic loads. The time-scale of loading is such that inertial or damping effects are regarded to be important. A transient analysis includes loads which are functions of time. For the transient analysis, a static analysis is first conducted to generate initial conditions, after which a dynamic transient analysis will be carried out. Effect of isotropic and kinematic strain-hardening is implemented and investigated in capturing material response under cyclic loading. Two loading tests for each column will be carried out: monotonic test and three cycle test. All these simulations are carried out under a constant axial applied load P of about 20% of the axial yield load P_y of the column. The lateral load history consists of three constant fully reversed load cycles. The horizontal load (H) reaches 70.70 kN and 49.45 kN for the U70-40 and the U45-40 column, respectively.



(a) (b)

Figure (7): NLFEM load-deflection curve with isotropic hardening for column sections (a) U70-40, and (b) U45-40

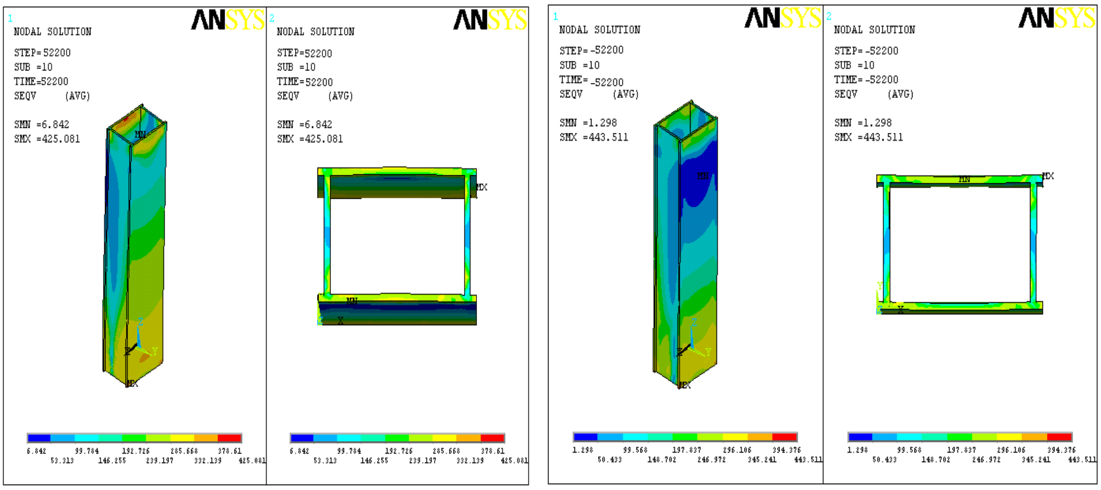


(a) loading stage (b) reverse loading stage

Figure (8): NLFEM Von-Misses stress distribution with isotropic hardening for section U70-40 (1st cycle)

RESULTS AND DISCUSSION

First, nonlinear buckling analysis of the two columns is carried out. Lateral deflection is measured with increasing the applied stress on the column. Fig. 3 shows the non-linear finite element model (NLFEM) applied stress *versus* lateral deflection curve for sections U70-40 and U45-40. The columns have reached an ultimate compressive stress (buckling stress) equal to 252.6 MPa and 263.5 MPa, respectively, approaching LRFD value of 263.5 MPa. However, column U70-40 showed almost twice the lateral deformation before buckling compared to column U45-40, since it has almost twice the width-to-thickness ratio. Material is assumed to have isotropic hardening in this stage, but it has no effect on the results since the test is monotonic loading. It is clear that finite element results underestimate column strength by about 4%. The reason for the small discrepancies might be due to the material bilinear stress-strain assumption, the boundary conditions assumption or the assumptions inherited in LRFD formula itself. However, this can be considered as accepted accuracy in finite element method. Deformed shape at failure (buckling) is illustrated in Fig. 4 along with Von-Misses stress distribution contours, where one can see that the buckling mode is of global type.



(a) loading stage (b) reverse loading stage

Figure (9): NLFEM Von-Misses stress distribution with isotropic hardening for section U45-40 (1st cycle)

Fig. 5 shows the NLFE model load-deflection curve for sections U70-40 and U45-40, which shows that the specimen has reached an ultimate load around 73 kN and 52 kN, respectively. The finite element results coincide well with the experimental results by Kumar and Usami (1996) for both isotropic and kinematic hardening. It is seen that both isotropic and kinematic strain-hardening give good simulation results that are close to the experimental results. Section U70-40 shows an almost linear relation between load and displacement, while section U45-40 shows some nonlinearity in load-displacement curve due to the lower width-to-thickness ratio. The fully plastic stress distribution of section U45-40 is attributed to the compactness of the section, while section U70-40 is not capable of reaching a fully plastic stress distribution before buckling, since it is a non-compact section. Von-Misses stress distribution with isotropic hardening is shown in Fig. 6. Buckling mode is noticed to be a global buckling, while stresses concentrate at the upper and lower ends of the columns.

Fig. 7 shows the NLFE model load-deflection curve for sections U70-40 and U45-40 with isotropic hardening. The specimen was subjected to three constant fully reversed load cycles equal to the buckling load. The finite element is seen to capture the general behavior of the cyclic loading of the steel columns, but with some divergence. It is clear that the finite element method coincides well with the experimental results in the loading stage, but diverges in the unloading phase. Finite element underestimates the displacement of the column section U70-40 by around 11% in the loading phase, while it predicts the displacement well in the reverse loading phase. However, the residual displacement was highly overestimated. On the other hand, the displacement of the section U45-40 was underestimated in the reverse loading more than in the loading stage. The residual displacement was well predicted here despite that the full general behavior could not be fully simulated.

Von-Misses stress distributions under isotropic hardening in the loading and reverse loading stages of the first cycle are shown in Fig. 8 for section U70-40 and in Fig.9 for section U45-40. It is clear that both sections suffered global buckling in the flanges; one flange buckles first during the loading stage in one direction, and then the other flange buckles next when reverse loading is applied in the opposite direction. The buckling starts at the top free edge of the column and propagates to the bottom. The buckling deformation is clearer in section U70-40 than in section U45-40, since it has smaller wall thickness. The column webs are noticed to have no buckling signs.

CONCLUSIONS

In this research, the use of finite element method in simulating cyclic buckling of hollow section steel columns has been studied. The monotonic nonlinear buckling analysis was seen to give results that were in very good agreement with the corresponding experimental tests, and slightly different than the LRFD formula results. The inclusion of both material and geometrical nonlinearities is very important for obtaining accurate results. The cyclic behavior was harder to be fully captured using the nonlinear transient dynamic analysis, where the accuracy of the results degraded slightly compared to monotonic simulation. Both isotropic and kinematic strain-hardening rules gave good approximate outputs compared with experimental results. Overall, the nonlinear finite element shows good capabilities in simulating the buckling behavior of a steel column under cyclic loading.

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