SEISMIC BEHAVIOR OF THIN-WALLED CORRUGATED BRIDGE-PIERS

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ABSTRACT

This paper presents a new innovative design concept, "thin-walled corrugated steel columns", primarily to be used in highway bridges as an alternative to thin-walled steel columns. Compared with the traditional design concepts, this new innovative concept might results in cost-effective designs, with high stiffness and improved buckling behavior under combined axial and flexural loading, caused by earthquake excitations. Using a finite element model, that takes the non-linear material properties into consideration, it is shown that the corrugations will act like longitudinal stiffeners that are supporting each other, thus improving the buckling behavior and allowing for reduction of the overall wall thickness of the column.

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INTRODUCTION

Thin walled steel columns, either circular or box shaped, are commonly used as piers in elevated highway bridges, because of their high stiffness-cross sectional area ratios. Especially in areas where construction space is limited, thin walled steel columns are preferable because they occupy less space when compared with reinforced concrete columns. Recent earthquakes, e.g. Hyogoken-Nanbu 1995, on the other hand have shown that these structures are vulnerable to damage when subjected to cyclic or earthquake loading. The observed damage during the Hyogoken-Nanbu earthquake was mainly due to local buckling of the columns. The damage was such that it took several years to restore the transportation systems, which caused extreme hardship to the public and had a negative economic impact to the region. After Hyogoken-Nanbu earthquake several analytical and experimental studies were conducted to determine the lateral load capacity and ductility of circular and box shaped (with or without stiffeners) column sections, and to develop a reliable design methodology to prevent these structures from failure under cyclic loading (Usami et al., 1992; Jiang et al. 2000; Goto et al., 1998; Gao et al., 1998a and 1998b).

Migita et al. (1992) proposed thin walled polygonal section steel columns as an alternative to box-cross sections. In this study a total of 23 specimens with box, pentagonal, hexagonal and octagonal cross-sections were tested under axial compression load. Although it is reported that the ultimate strength of polygonal thin-walled steel columns are higher than box-sections, to the author's knowledge there are no applications of polygonal shaped thin walled steel bridge piers.

Structural members consisting of corrugated components are being used in aerospace and mechanical engineering fields, since 1950's, to increase the out-of-plane stiffness and buckling strength of the components, without using vertical stiffeners. More recently corrugated members are used as webs for girders in civil engineering applications, buildings and bridges. Studies conducted have shown that using members consisting of corrugated components will result in better performance characteristics for both buckling and post-yielding behavior while weight is being reduced by 15%-20%.

This paper presents an innovative concept, "thin-walled corrugated steel columns", primarily to be used in highway bridges as an alternative to thin-walled steel columns with circular or rectangular cross-sectional areas. Using a finite element model, that takes into consideration the non-linear material properties, it is shown that the corrugations will act like longitudinal stiffeners that are supporting each other, improving the buckling behavior under combined axial and flexural loading. To show the applicability of the proposed concept it is compared with the cyclic load test results of 1/3-scale steel pier specimens conducted by the Public Works Research Institute of Japan (Nishikawa et. al., 1996; Goto et al., 1998).

THIN-WALLED STEEL COLUMNS

Thin-Walled Circular and Box/Rectangular Cross Sections

The lateral load capacity and ductility of thin walled columns are influenced greatly by the occurrence of local buckling. This phenomenon is a function of the geometry, the boundary conditions, the material properties, and the axial load of the column. One of the most important indicators for the occurrence of local buckling and its severity is the ratio of σ_y/σ_{cr}. Where σ_y is the yield stress of the material and σ_{cr} is the critical buckling stress assuming elastic buckling. This ratio for columns with circular and rectangular sections is given by the following two equations respectively.

$$\frac{\sigma_y}{\sigma_{cr}} = \frac{\sigma_y}{E}\frac{r}{t}\sqrt{3(1-\nu^2)} \text{ (for circular sections)} \quad (1)$$

$$\frac{\sigma_y}{\sigma_{cr}} = \frac{\sigma_y}{E}\frac{b^2}{t^2}\frac{12(1-\nu^2)}{k\pi^2} \text{ ; } k = 4 \text{ (for rectangular sections)} \quad (2)$$

where E and ν are the Young's modulus and Poisson's ratio respectively; r = radius of the circular column; b = breath of the rectangular column; t = plate thickness.

It has been shown both analytically and experimentally that local buckling occurs for columns with values of the σ_y/σ_{cr} ratio higher than 0.03. The higher the value of this ratio the more severe the local buckling in the column is. Gao et al. (1998a) present a thorough analytical investigation on the factors that affect ductility and strength of steel cylinders under compressive and bending actions. Jiang et al. (2000) reports that circular columns with $\sigma_y/\sigma_{cr} < 0.07$ under lateral cyclic load exhibit behavior similar to the one presented in Figure 1(a). This is a desired behavior for earthquake engineering applications since the lateral load vs deformation loops are stable showing constant strength and increasing energy dissipation capability as the plastic deformations increase. This stable behavior can be attributed to limited/slight local buckling in the elastic range concentrated close to the base of the column. The area that the material has been undergoing buckling with each cycle remains confined and buckling does not propagate to the rest of the column. Failure occurs due to excessive plastic deformations when strain reaches an ultimate value. However, columns with $\sigma_y/\sigma_{cr} <0.07$ are not considered economically viable solutions to be used in practice.

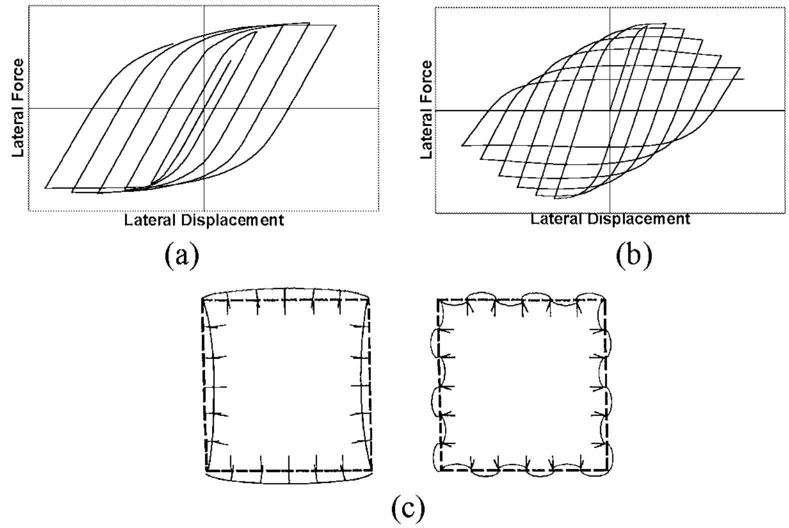


Figure 1. (a) Failure due to excessive deformation (limited local buckling in the elastic range), (b) Failure due to severe local buckling in the plastic range, and (c) overall wall panel buckling and local panel buckling (from MacRae and Kawashima 1992)

Circular thin-walled columns with σ_y/σ_{cr} >0.07 exhibit hysteretic behavior under lateral loads similar to the one presented in Figure 1(b). It can be observed that the strength degrades for increased plastic deformations. This type of behavior is typical of excessive local buckling in the plastic range and failure occurs due to the interaction between local and overall buckling of the column. Columns with σ_y/σ_{cr} >0.17 are not likely to be used in practice due to their limited ductility capacity.

Usami et al. (1992) contacted a series of tests on thin-walled steel box columns. In these tests seven specimens were used to investigate the influence of longitudinal stiffeners (ribs) on lateral load capacity and ductility of the column. In addition two specimens were tested to study the effect of concrete fill on the same performance parameters. In all the tests the ratio σ_y/σ_{cr} was kept constant to approximately 0.2 (see Equation 2). For the stiffened columns the ratio of the relative flexural rigidity to the optimal flexural rigidity (γ/γ^*) of the stiffeners used was equal to 3, 5 and 13.6. (For more information on evaluation of the stiffener's relative and optimal flexural rigidity the interested reader may referred to Zheng et al. 1995) The researchers concluded that as the flexural rigidity of the stiffeners increased from 3 to 13.6 no noticeable improvement was obtained in the strength, ductility, and energy absorption capacity of the columns. However, some improvement on the ductility and energy absorption capacity was obtained when higher strength steel was used in the stiffeners. That the presence of stiffeners with increased rigidity in rectangular thin-walled columns does not contribute much to their strength and energy absorption capacity can be attributed to the fact that the stiffeners have a free side and even though they reduce the buckling width of the plate panels for local panel buckling, contribute very little to the reduction of the overall wall panel buckling as shown in Figure 1(c).

Thin-Walled Corrugated Cross Sections

To overcome the shortcomings of the circular and the box sections as were described above a corrugated thin-walled cross-section is introduced. A corrugated column consists of either cold-formed corrugated plates or inclined plates that are welded to each other as depicted in Figure 2. In a corrugated column, each plate (or fold in case of cold forming) will act like a stiffener that supports the adjacent plate (or fold) in the vertical direction. At the first glance the corrugated column may look similar to a rectangular shaped column with vertical stiffeners. However since each fold is supported on both vertical edges, the buckling properties of corrugated columns are more desirable than the stiffened rectangular sections.

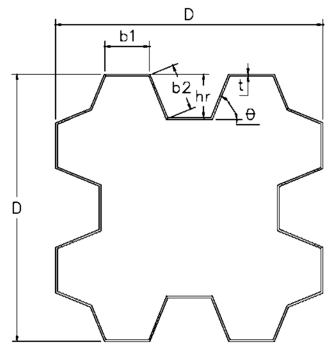


Figure 2 Cross Section of Corrugated Column

The lateral strength of a corrugated column depends on the critical buckling stress of each corrugation. Each panel will act like a vertical stiffener that is supported on both sides. Since the panel width, which is subjected to buckling, is much smaller than the panel width in rectangular or circular sections, higher critical buckling stresses can be achieved with thinner plates. Moreover local buckling of a single plate will not cause failure because the sharp edges between the folds will prevent buckling from propagation. As a result the section can undergo large plastic deformations without significant, if any, strength reduction, hence dissipate energy under cyclic loading or earthquake excitations. Furthermore by changing the corrugation density the engineer can control the critical buckling stress. With denser corrugations higher critical buckling stresses can be achieved since as b decreases in equation 2, where b is the larger value of b_1 or b_2, the critical buckling stress will increase. However if the corrugations are too dense failure will occur due to global buckling of the section, similar to orthotropic plates, which is not desirable in our case.

PROTOTYPE DESIGN

Nishikawa et al. (1996) tested 1/3-scale thin walled columns, with circular and rectangular cross-sections (see Figure 3), under cyclic loading, at the Public Research Work Institute of Japan. The prototype design work is based on the geometric and material properties that were reported in the above-mentioned study and in Goto et al. (1998). Two of the tested columns (one circular and one rectangular section) are chosen, and alternative corrugated column designs for each section are developed. FEM analysis is carried out to facilitate comparison between the columns. The analyzed specimen properties are given in Table 1. In Table 1, specimen No-8 is the tested circular column by Nishikawa et al. (1996) as reported by Goto et al. (1998) where AL-06 is its corrugated column replacement and specimen No-2 the tested stiffened rectangular column, where AL-08 is its corrugated column replacement.

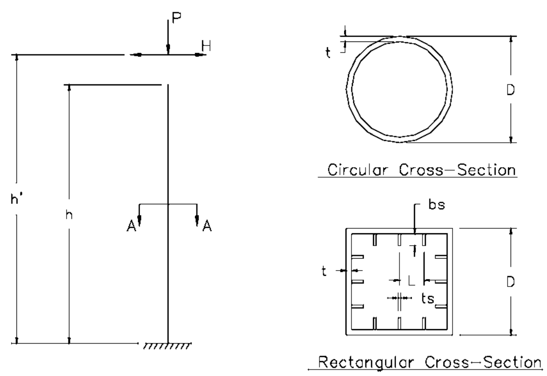


Figure 3 Cross Sections of the Tested Specimens by Nishikawa et al. (1996)

Table 1 Geometric Properties of the Tested Specimens by Nishikawa et al. (1996) and their Corresponding Corrugated Sections

Parameter / Specimen	h' (mm)	h (mm)	d (mm)	t_s (mm)	t_s (mm)	b_1 (mm)	b_2 (mm)	h_r (mm)	θ (deg)	b_s (mm)	l (mm)
No-8	3403	3173	900	9	-	-	-	-	-	-	-
AL-6	-	3173	900	6	-	150	158	147	68	-	-
No-2	3403	3173	900	9	6	-	-	-	-	80	225
AL-9	-	3173	900	9	-	150	158	147	68	-	-

FEM-Modeling and Analysis

To determine the hysteretic behavior FEM analyses is carried out, that takes both material and geometric non-linearities into consideration. For this purpose the commercial multi purpose finite element package ANSYS Ver.8.1 is used. The specimens are modeled using non-linear Shell 181 elements that are available in ANSYS element library. Using symmetric boundary conditions only half of each column is discretized. At the base, fixed boundary conditions are applied. The corrugated columns are meshed so that each fold is divided into three shell elements. For the circular and rectangular columns the mesh density is determined using trial-error method, and the discretization is done so that the errors between FEM analysis and test results were minimized.

The material properties of the specimens are based on the tensile coupon test results that were reported in Goto et al. (1998). The material non-linearity is utilized with the superposition of non-linear kinematic hardening rule and isotropic hardening rule. The material constants for the constitutive model are determined by curve fitting using the reported stress-stain relationship given in Figure 4.

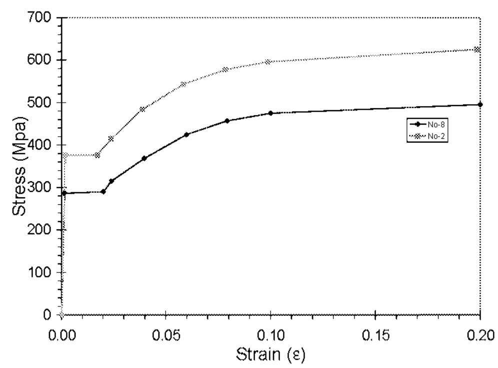


Figure 4 Uni-axial Stress-Strain Relation of the Materials in the No-8 and No-2 columns from Nishikawa et al. (1996).

To facilitate comparison, the corrugated columns were analyzed using the same axial load and displacement time history that Nishikawa et al. (1996) used. In the experiments the axial load (P) was kept constant and the horizontal load (H) was applied using displacement control method according to the cyclic loading protocol given in Figure 5.

For the analyses the effects of residual stresses and initial deflections of the column walls are neglected for all of the sections. Although these effects will decrease the initial buckling, they will not affect much the cyclic behavior of the specimens (Banno et al. 1998).

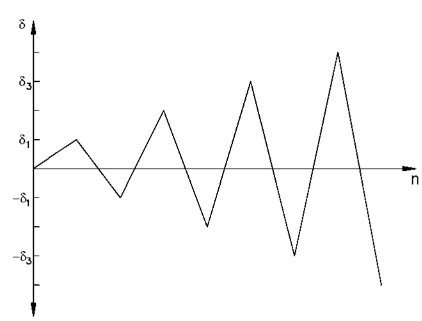
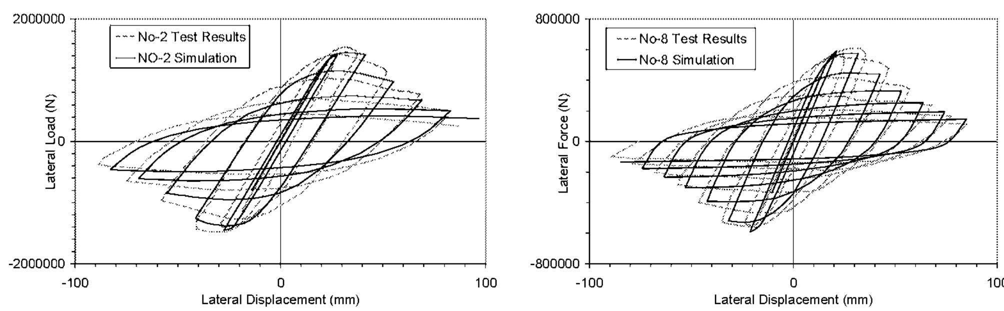


Figure 5 Specimen Loading Pattern

ANALYSES RESULTS AND DISCUSSION

Analysis vs. Test Results for Circular and Box Cross-Sections

Figure 6 compares FEM analysis results for the box and circular cross section columns with the experimental results for the same sections reported by Goto et al. (1998). Both models showed good agreement with the experimental results. The initial stiffness and the ultimate strength obtained from the analyses agreed well with the ones from the experiments. The model was also able predict the strength degradation, caused by cyclic loading as well as the ductility of the specimens. The stiffness degradation predicted by the model differed from the stiffness degradation observed in the experiment; this is caused because the constitutive model assumes perfectly elastic material during the unloading phase. Also differences are observed in the reduction rate of the kinematic yield surface. The reduction in the yield surface predicted by the model was faster than the actual case. This is caused by the parameters of the constitutive model. These parameters were calibrated using the coupon tests, since no additional data was available. In reality data from stabilized tension-compression hysteretic curves at different strain amplitudes is required to calibrate these parameters.



(a) (b)

Figure 6. Comparison between analytical and experimental results (a) box section; (b) circular section

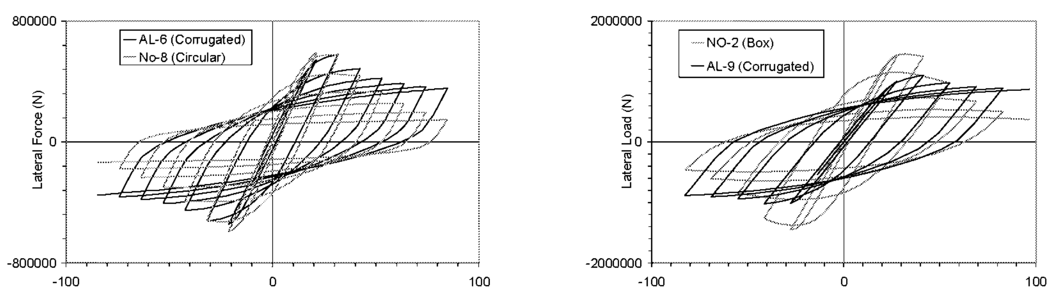
Conventional Cross-Sections vs. Corrugated Sections

The conventional and corrugated column pairs, where analyzed under same loading conditions, meaning under a constant axial load and cyclic horizontal displacement, where the yield displacement parameter that controls the horizontal loading (δ in Figure 5) is the yield displacement of the circular or the rectangular column as calculated from a methodology proposed by Usami et al (1992). The interested reader can also find the equations used to determine δ_y in Gao et al. (1998b). Although the circular column and its corrugated counterpart, AL-06, have similar cross sectional areas and moment of inertias, it must be noted that the moment of inertia of the rectangular/box column is approximately 25% more than its corrugated counterpart, AL-09. Table 2 depicts the cross-sectional area (A), moments of inertia (I_{zz}), vertical load (P), the displacement amplitude (δ_y) that controls the alternating horizontal load, and the ratio of the displacement amplitude to the yield displacement (δ/δ_y) of the column.

Table 2. Specimen Loading Properties

Parameter / Specimen	A (mm^2)	I_{zz} ($10^9 mm^4$)	P (kN)	$P/(\sigma_y A)$	δ_y (mm)	δ/δ_y
No-8 (circular section)	25192	2.5	904	0.124	10.6	1
AL-06	25846	2.5		0.124		1
No-2 (box section)	37836	4.8	1,700	0.122	13.8	1
AL-09	36633	3.5		0.122		1.37

In Figure 7(a) the comparison of FEM analysis results of the circular column No-8 and the corrugated column AL-06 is given. The analysis, on which the comparison is based on, is carried out until the column top displacement amplitude reached 84.8mm. This amplitude is 8 times the yield displacement of the column No-8. From the hysteretic curve it can be observed that the first local buckling in the circular column initiates at around $3\delta_y$. After this the strength rapidly decreases as the displacement amplitude increases. Finally at $8\delta_y$ the column strength drops approximately to 35% of its initial strength. The observed buckling pattern has an elephant foot shape (see Figure 8(a)), which matches the buckling pattern that was observed during the actual tests by Nishikawa et al. (1996). It is also worth mentioning that the piers failed during the Hyogoken-Nanbu earthquake had the same elephant-foot buckling pattern. On the other hand the first local buckling in the corrugated column initiated at around $3\delta_y$, after this amplitude the decrease in the strength was very slow. At $8\delta_y$ the corrugated column had retained 60% of its initial strength. The deformed shape of the corrugated column at $6\delta_y$ is given in Figure 8(b). From this figure it can be observed that buckling is localized and does not affect the global stability of the system. Figure 9 compares absorbed energy of the two systems. The energy absorption capacity of the circular column drops after yielding, while in case of corrugated column local buckling slightly affects the amount of the absorbed energy.



(a) (b)

Figure 7 Analytical Hysteretic Curves of Models Columns obtained using ANSYS (a) Circular vs. corrugated column; (b) box vs. corrugated column

In Figure 7(b) the hysteretic curves for the rectangular column No-2 and the corrugated column AL-9 are depicted. In this case the analysis is carried out until the top displacement reached $6\delta_y$ of the rectangular column. As expected the initial stiffness and the ultimate strength of the rectangular column are higher than the corrugated column's, because of the difference in the moment of inertia of the specimens. However this is not the case when we compare their ductility and energy absorption capacities. From the deformed shape (Figure 8(c)) it can be observed that the rectangular column web and the flanges suffered from severe global buckling, which the stiffeners were unable to prevent. The stiffeners have also buckled following the global buckling mode of the column. From the hysteretic curves it can be observed that the first buckling initiated at around $2\delta_y$. With the increase of the displacement amplitude, its strength decreased rapidly. The local buckling in the corrugated column on the other hand occurred at around $3\delta_y$. After this point the decrease in the strength is minimal. Figure 8(d) shows the deformed shape of the corrugated column at $8\delta_y$. The effect of buckling mode and hysteretic behavior on the energy absorption capacity is shown in Figure 9. Although the energy absorbed by the rectangular column seems to be higher at low displacement amplitudes (due to the difference in the strength between the two), the rate of energy absorption of the rectangular column drops where the rate of energy absorption for the corrugated pier remains constant and higher than the rectangular pier.

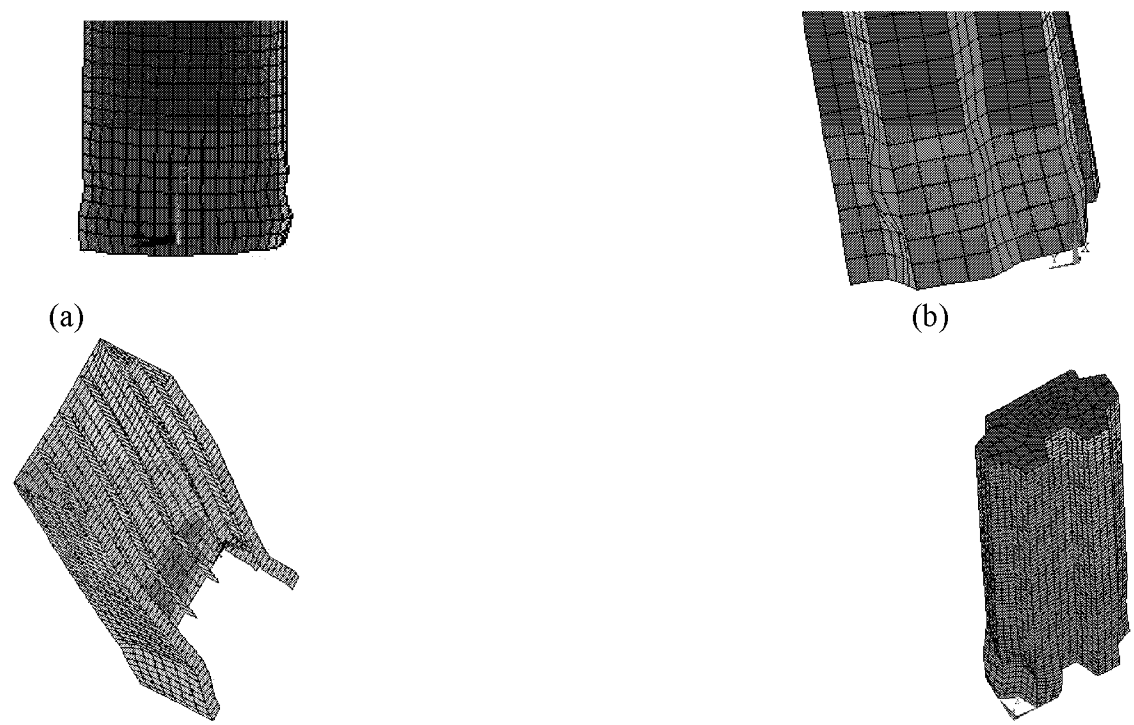


Figure 8 (a). Deformed shape of No-8 at $6\delta_y$; (b) Deformed shape of AL-06 at $6\delta_y$;(c) Deformed shape of No-2 at $5\delta_y$; (d) Deformed shape of AL-09 at $8\delta_y$

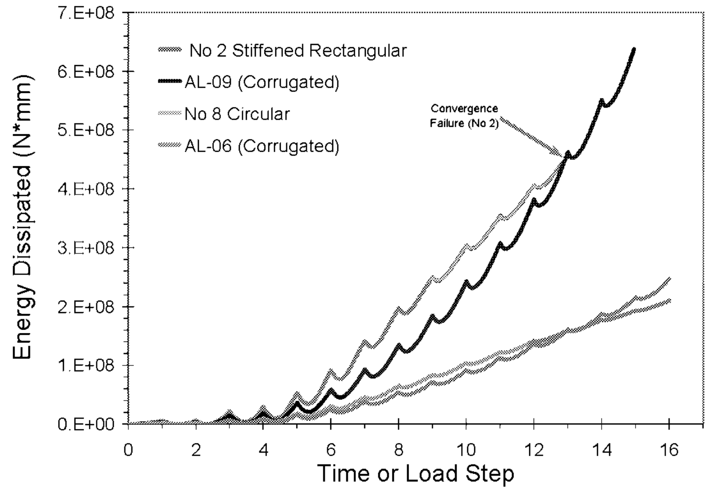


Figure 9 Cumulative Energy Dissipated by the Studied Columns

CONCLUSIONS

In the corrugated column every panel acts as a vertical stiffener that is supported on both sides in contrary to the stiffeners used in thin-walled rectangular columns. The panel width, which is subjected to buckling, is much smaller in the corrugated columns, thus higher critical buckling stresses can be achieved with thinner plates. The local buckling of a single plate will not cause failure because the edges between the folds will prevent buckling from propagating. As a result the section can undergo large plastic deformations without significant, strength reduction, hence dissipate energy under cyclic loading.

It has been shown that the thin-walled corrugated columns can be designed to meet higher performance criteria than the thin-walled circular or box cross-sectional columns. They show stable hysteretic behavior with constant strength for large plastic deformations, which result in increased ductility and energy dissipation capacity. Their behavior is suitable for earthquake type of excitations

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