Proceedings of the 9th International Conference on Civil Structural and Transportation Engineering (ICCSTE 2024)
Chestnut Conference Centre - University of Toronto, Toronto, Canada – June 13-15, 2024
Paper No. 250
DOI: 10.11159/iccste24.250

Hysteresis Behavior of Thin-Walled Steel Tubular Columns under Cyclic Loading

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Abstract - This paper investigates the stability of thin-walled steel tubular columns when subjected to cyclic bidirectional loading. It compares their behavior under conventional cyclic unidirectional loads, examining the decline in strength and ductility due to severe cyclic bidirectional loads using nonlinear finite element analysis and experimental data. Pseudo-dynamic bidirectional tests from existing literature validate the finite element analysis. The results underscore the significance of understanding steel column behavior under bidirectional loading. Both bidirectional tests and finite element analysis reveal the complex behavior of tubular columns, characterized by a circular trajectory post-local buckling. This trajectory leads to a monotonic development of the local buckling bulge and larger residual deformations. In contrast, unidirectional loading tests and analyses tend to underestimate earthquake-induced damage and residual displacements. Thus, it's crucial to consider bidirectional loading effects in evaluating ductility and designing for seismic resistance in steel structures.

***Keywords*:** Cyclic loading, Thin-walled, Steel, Hysteresis behavior, Local Buckling, Bidirectional lateral loading, Columns

1. Introduction

Thin-walled steel tubular columns, widely employed as steel bridge piers in Japan, offer distinct advantages over concrete counterparts due to their lightweight and ductile nature. They are particularly favored in urban areas like Tokyo, and Los Angeles, where space constraints demand efficient utilization. Additionally, they are preferred in areas with soft grounds and reclaimed lands.

Typically designed as single cantilever columns or multi-story frames, these steel bridge piers consist of relatively thin-walled closed-section members, often box or circular, for enhanced strength and torsional rigidity [1]. However, their susceptibility to damage from coupled instability, resulting from the interaction of local and overall buckling, becomes evident during major seismic events. For instance, severe local buckling damage near the base of hollow box section piers was observed during the Kobe earthquake, as depicted in Fig. 1.

Thin-walled steel columns are favored in Japan for elevated bridge piers in urban areas due to their documented advantages and high earthquake resistance. Following insights gained from the 1995 Kobe earthquake, as outlined by the JSCE Earthquake Engineering Committee in 2000, the Japanese seismic design code for highway bridges, established by the Japan Road Association in 2002, introduced a new design concept. This concept aims to control column damage, ensuring residual deformations remain within acceptable limits after exposure to earthquake waves.

Given this context, accurate prediction of the ultimate behavior of thin-walled steel columns subjected to severe earthquakes becomes imperative. The local buckling of thin-walled steel plate elements within these structural members significantly influences their strength and ductility. Given that earthquake waves have three-dimensional components, the coupling of horizontal components exacerbates the challenges faced by these columns. Understanding and accurately forecasting the response of these columns under extreme seismic events are critical for ensuring the structural integrity and safety of elevated bridges in urban areas. Hence, it is imperative to investigate the ultimate behavior of thin-walled steel bridge piers under cyclic bidirectional lateral loading.

Fig. 1: Local buckling of steel bridge pier, Kobe Earthquake, January 1995.

2. Characteristics of Thin-Walled Steel Tubular Columns

Thin-walled steel tubular columns represent a significant structural element in various engineering applications, particularly in bridge construction. Their unique characteristics, such as lightweight design, ductility, and adaptability to constrained urban environments, make them a preferred choice over conventional concrete structures. This paper delves into the distinctive features and behavior of thin-walled steel tubular columns, highlighting their vulnerability to coupled instability phenomena, notably the interaction between local and overall buckling. Drawing upon case studies and experimental data, the paper investigates the impact of cyclic bidirectional loading on the ultimate behavior of these columns, shedding light on crucial design considerations for seismic resistance and structural performance. Through a comprehensive analysis of geometric and material nonlinearities, this study aims to provide valuable insights into optimizing the design and performance of thin-walled steel tubular columns in engineering practice.

Steel tubular columns utilized in highway bridge systems typically consist of relatively thin-walled members with closed cross-sections, which are predominantly box or circular in shape. This choice is driven by their superior strength and torsional rigidity, as depicted in Figs. 1 and 4. These structural configurations differ significantly from columns found in buildings. Unlike their building counterparts, highway bridge columns are prone to failure due to local buckling in the thin-walled members. The irregular distribution of story mass and stiffness, along with the phenomenon of strong beams and weak columns, characterizes highway bridge systems with low-rise structures (1-3 stories). Additionally, there's a critical need to assess residual displacement. These factors render the columns susceptible to damage from the interaction of local and overall buckling during severe earthquakes.

In the practical design and ductility evaluation of thin-walled steel hollow box sections, key parameters include the width-to-thickness ratio of the flange plate for box sections, the radius-to-thickness ratio of circular sections, and the slenderness ratio of the column [2]. While the width-to-thickness ratio and radius-to-thickness ratio influence local buckling, the slenderness ratio governs global stability. These parameters are expressed as:

$$R_f = \frac{b}{t}\frac{1}{n\pi}\sqrt{3(1-\nu^2)\frac{\sigma_y}{E}} \quad (\text{for box section}) \tag{1}$$

$$R_t = \frac{r}{t}\sqrt{3(1-\nu^2)}\,\frac{\sigma_y}{E} \quad \text{(for circular section)} \tag{2}$$

$$\lambda = \frac{2h}{r_g}\frac{1}{\pi}\sqrt{\frac{\sigma_y}{E}} \tag{3}$$

in which, $b =$ flange width; $t =$ plate thickness; $\sigma_y =$ yield stress; $E =$ Young's modulus; $\nu =$ Poisson's ratio; $n =$ the number of subpanels divided by the longitudinal stiffeners in each plate panel ($n = 1$ for unstiffened sections); r = the radius of the circular section; h = column height; r_g = the radius of gyration of the cross-section.

The elastic strength and deformation capacity of the column are expressed by the yield strength H_{y0}, and the yield deformation δ_{y0} (neglecting shear deformations), respectively, corresponding to zero axial load. They are given by:

$$H_{y0} = \frac{M_y}{h} \tag{4}$$

$$\delta_{y0} = \frac{H_{y0}h^3}{3EI} \tag{5}$$

where $M_y =$ yield moment and $I =$ moment of inertia of the cross-section. Under the combined action of buckling under constant axial and monotonically increasing lateral loads, the yield strength is reduced to a value denoted by H_y. The corresponding yield deformation is denoted by δ_y. The value H_y is the minimum of yield, local buckling, and instability loads evaluated by the following equations:

$$\frac{P}{P_u} + \frac{0.85H_y h}{M_y(1-P/P_E)} = 1 \tag{6}$$

$$\frac{P}{P_u} + \frac{H_y h}{M_y} = 1 \tag{7}$$

in which $P =$ the axial load; $P_y =$ the yield load; $P_u =$ the ultimate load; and $P_E =$ the Euler load.

3. Cyclic Loading

The objective of this study is to assess the potential maximum deterioration in strength and ductility of thin-walled circular steel columns under constant axial load and cyclic bidirectional lateral loads, in comparison with conventional cyclic unidirectional loads. To achieve this, cyclic bidirectional loading programs must be selected to enable direct comparison with conventional displacement-controlled cyclic unidirectional loading programs. Additionally, these chosen bidirectional loading programs should represent the most severe conditions resulting in maximum deterioration of strength and ductility in steel columns, ensuring their safety.

Common practical cyclic bidirectional loading programs include diagonal (biaxial linear), rectangular, diamond, circular loading patterns (refer to Figure 2) [3,4]. Circular columns demonstrate isotropic behavior concerning the x and axes (as depicted in Figure 2a). Cyclic circular loads are considered among the most severe bidirectional cyclic loads, comparable to conventional cyclic unidirectional loads [5]. This study examines the behavior of thin-walled steel tubular columns under cyclic circular bidirectional loads while maintaining constant vertical compressive load, as depicted in Figures 2 and 3.

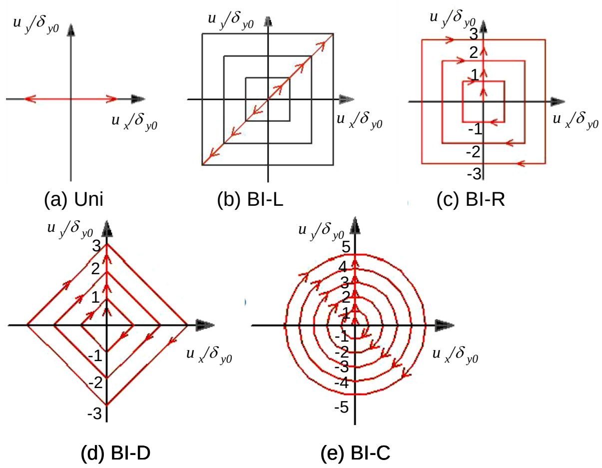


Fig. 2: Cyclic loading patterns: (a) Unidirectional, (b) Bidirectional-linear, (c) Bidirectional rectangular, (d) Bidirectional-diamond, and (e) Bidirectional-circular.

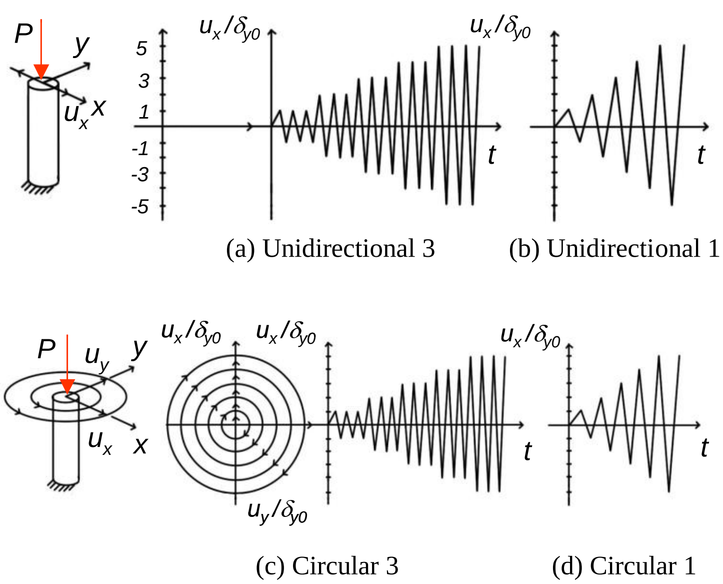
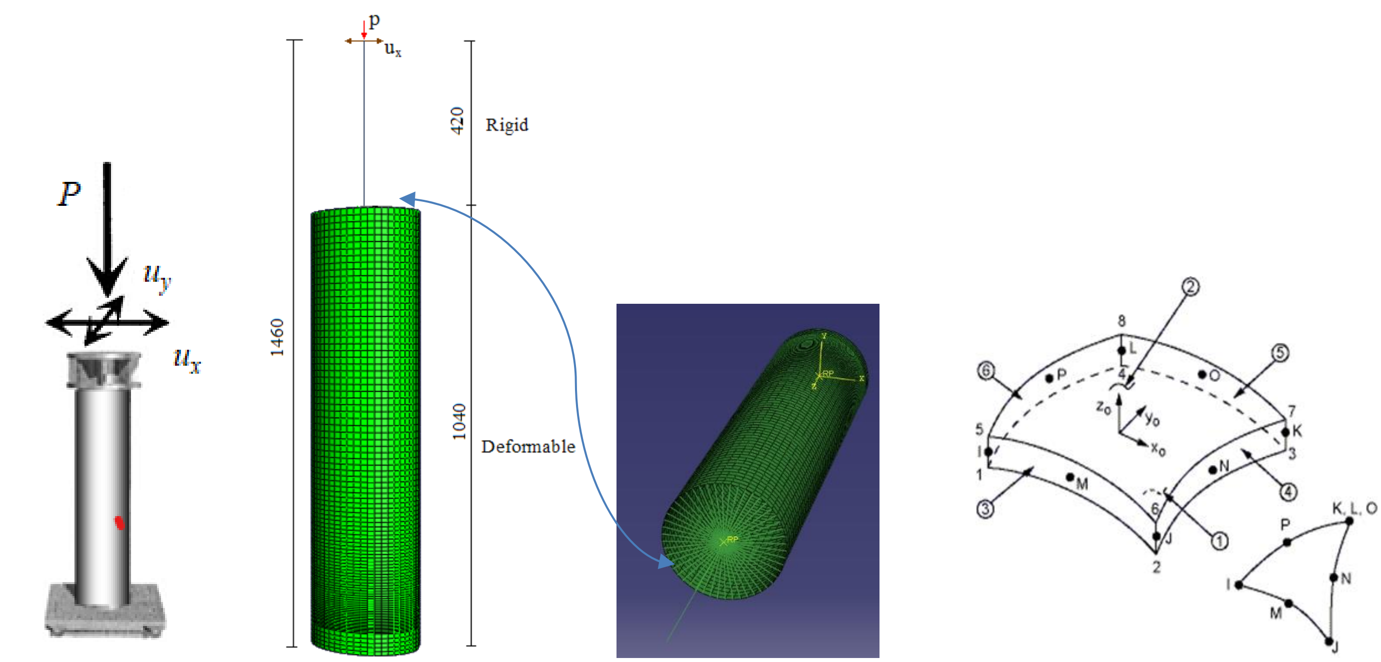


Fig. 3: Loading programs.

While maintaining a constant vertical compressive load, the displacement amplitude corresponding to the radius of the loading circle is incrementally increased by integer multiples of the initial yield displacement (denoted as δ_{y0}). Under the same amplitude, column specimens are subjected to either one loading cycle or three loading cycles. Although most cyclic unidirectional loading tests for thin-walled columns involve only one loading cycle, ECCS (1986) recommends three loading cycles for comprehensive evaluation [6].

4. Finite Element Analysis

Numerical studies on the cyclic behavior of thin-walled steel tubular columns, Fig. 4(a), were conducted using the computer program ABAQUS [7]. Although a comprehensive analysis of the entire column was conducted to ensure accurate results, different mesh sizes were employed to reduce computation time. Figs. 4(b) and 4(c) illustrate the meshing of the column.



(a) Steel Tubular Column (b) Meshing details of column (c) Rigid Diaphragm (d) Shell Element (S4R)

Fig. 4: Finite Element Modeling of the column

The column was modeled using the shell element S4R, a 4-node reduced integration shell element. Plasticity was assumed to be distributed along the cross-section of the shell element and the length of the column. To distribute the plasticity across the cross-section, 5 Gaussian integration points were utilized. Fig. 4(b) presents a schematic diagram of the S4R element [7].

4. Numerical Method

A series of numerical studies on the cyclic behavior of thin-walled steel tubular columns were carried out using the commercial computer program ABAQUS [7]. These results were then compared with experiments conducted in Japan by Obata and Goto [5].

Model PT4.5-1 in this study was subjected to unidirectional loading with three cycles, as detailed in Table 1. Other specimens were subjected to displacement-controlled cyclic circular loads, with either one loading cycle or three loading cycles, as illustrated in Fig. 3. These cyclic loading conditions allow for comparison with conventional cyclic loads (Table 1).

Table 1 Geometry and boundary condition parameters

Specimen	h (mm)	$D=2R$ (mm)	t (mm)	α	H_0 (kN)	δ_0 (mm)	Loading program
PT4.5-1	1,460	251.0	4.698	0.129	44.0	5.34	Unidirectional 3
PT4.5-2	1,460	259.2	4.634	0.126	46.6	5.24	Circular 3
PT4.5-3	1,460	259.2	4.470	0.137	43.9	5.07	Circular 1
PT3.5-1	1,460	258.2	3.500	0.136	34.5	4.92	Circular 1

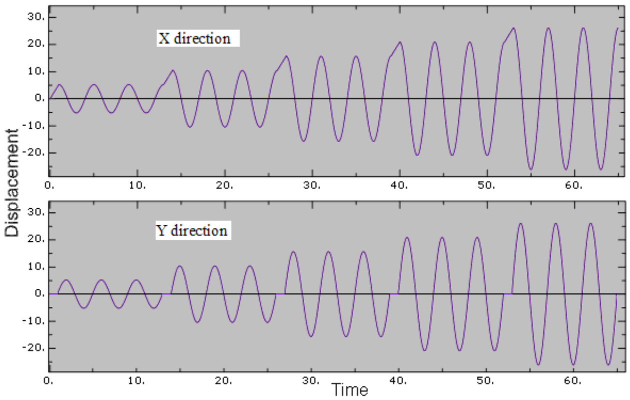
$\alpha = P/(\sigma_y A)$; $H_0 = (\sigma_y - P/A)Z/h$; $\delta_0 = H_0 h^3/(3EI_{av})$

While maintaining a constant vertical compressive load, the displacement amplitude coinciding with the radius of the loading circle was incrementally increased by integer multiples of the initial yield displacement denoted by δ_y, where H_y is the horizontal force causing the initial yield of a column. Young's modulus E, Poisson's ratio ν, yield stress σ_y, and ultimate tensile stress σ_u for the specimens are provided in Table 2.

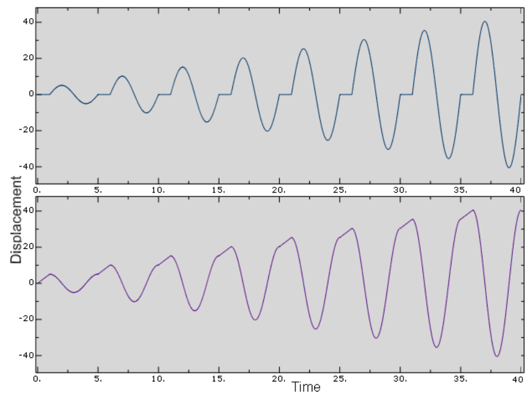
Table 2 material properties

Specimen	E (GPa)	ν	σ_y (MPa)	σ_u (MPa)
PT4.5-1,PT4.5-2	206	0.3	324.6	592.4
PT4.5-3,PT3.5-1	206	0.3	320.5	578.5

The vertical force (P) was calculated using the formula (α) provided in Table 1. Lateral loads were applied as displacements in the x-direction, following the patterns illustrated in Figures 2 and 3. The initial yield displacement (δ_y) for all specimens is also documented in Table 1. Displacements and rotations were constrained at the base circle nodes. The lateral load was applied at the top node of the rigid part, as depicted in Figure 5. Circular loading was simulated by applying cyclic loading in both the x and y directions, as illustrated in Figures 5(a) and 5(b).



(a) Circular-3



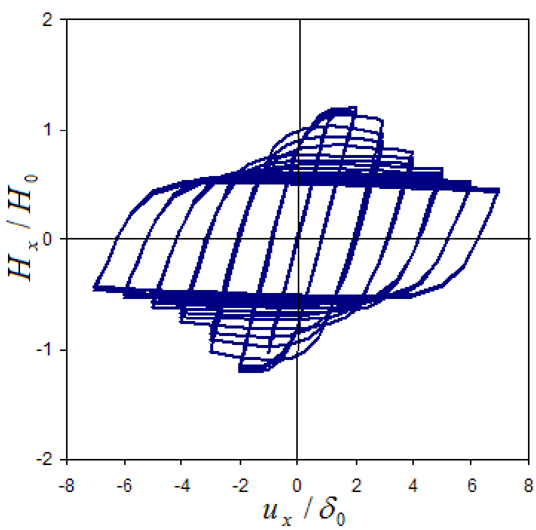
(b) Circular-1

Fig. 5: Applied lateral load

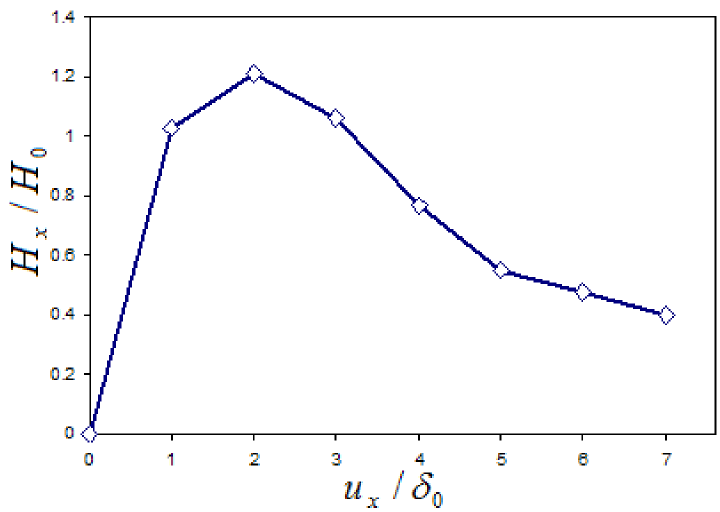
4. Numerical Results

4.1-Specimen PT4.5-1

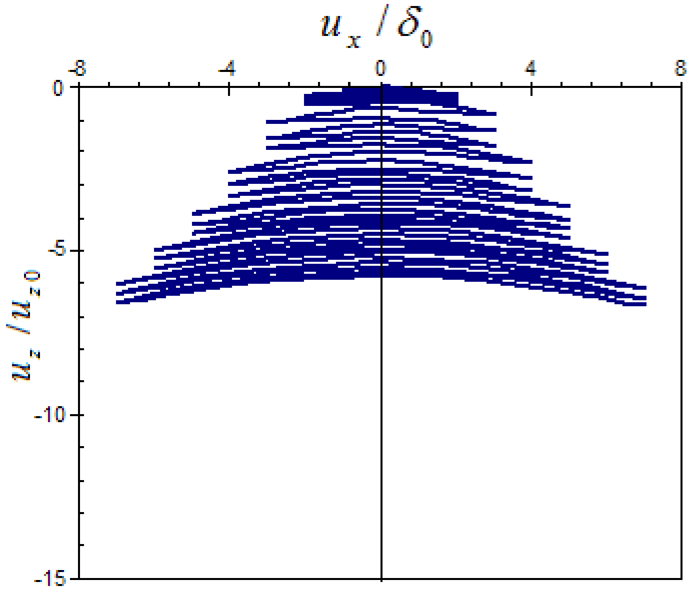
The result of the cyclic unidirectional loading test on the column specimen [5] is summarized in Fig. 6. Local buckling and plastic deformation in the lower part of the cylindrical plate in the last step is shown in Fig. 6(d). As shown in Fig. 6(c) relative displacements in the z-direction are less than what is from the experiment [5]. In addition, the pattern of behavior is different. As shown in Fig. 6(c), the increase of the longitudinal z displacement is almost the same during each load cycle in the loading process but in the test results [5] the z displacement accelerated through the load cycles.



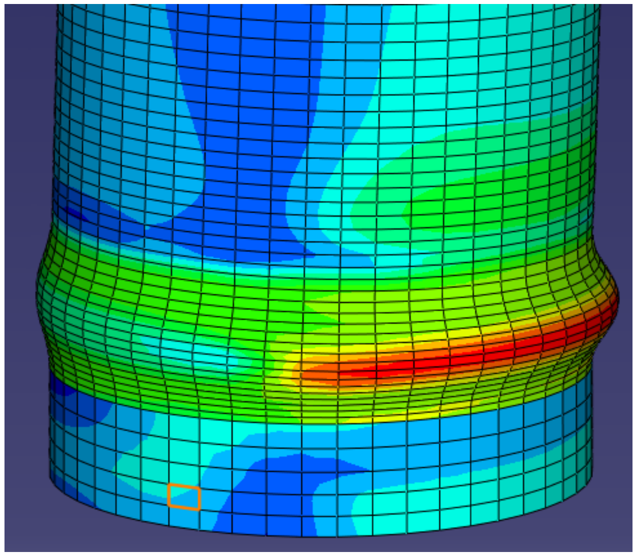
(a) Hysteresis curve (analysis)



(b) Envelope (analysis)



(c) Vertical shortening (analysis)



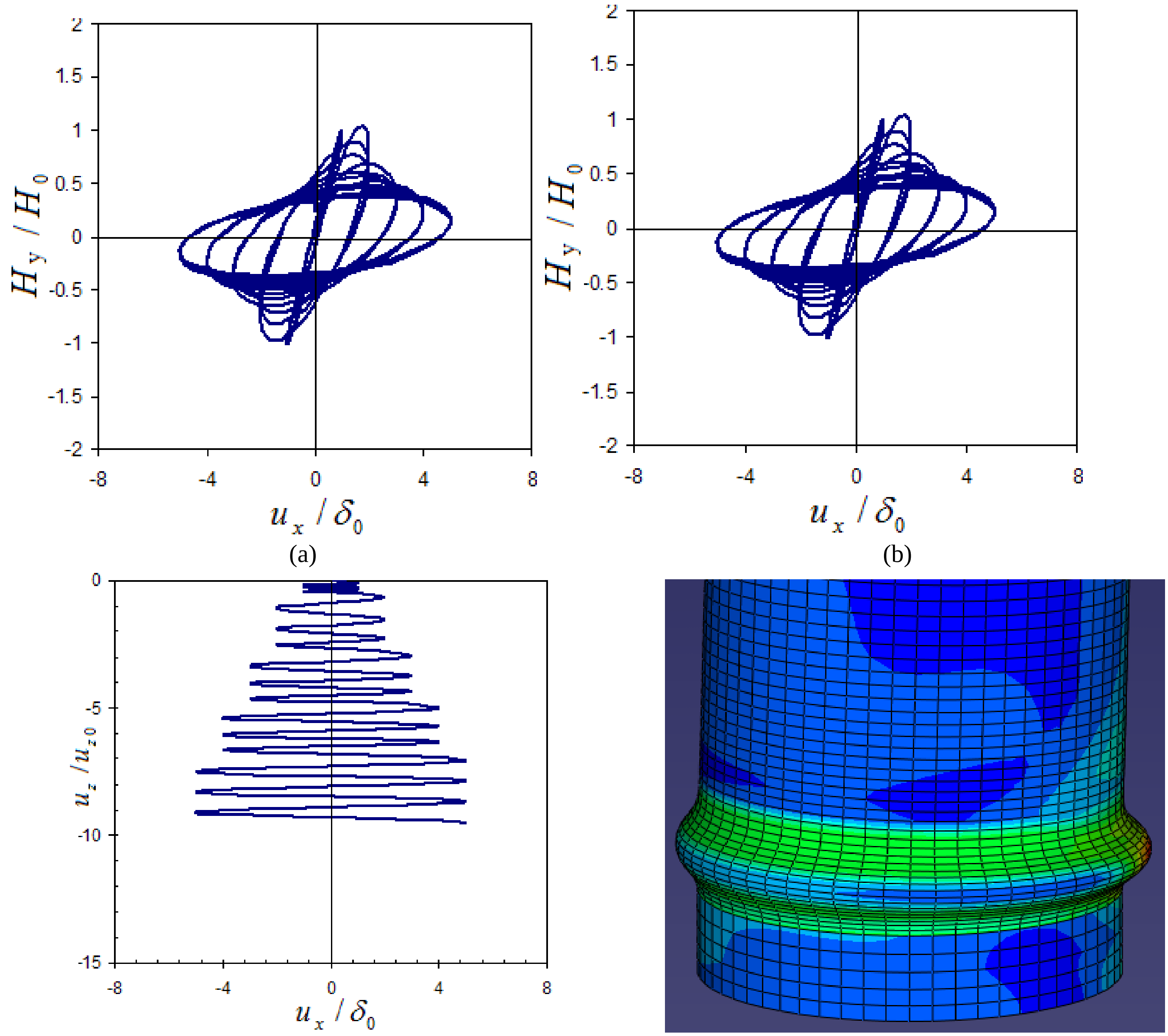
(d) Local Buckling (analysis)

Fig. 6: Analysed Specimen PT4.5-1, (a) Hysteretic behavior of the column, (b) Envelope curve,
(c) Relative displacements in the z-direction, (d) Buckling near the base of the column

Figs. 6(b) shows the envelope curve of the horizontal load-displacement relations shown in Fig. 6(a). The envelope curve under the cyclic circular-1 load is obtained from the hysteresis curves in the x direction.

4.2-Specimen PT4.5-2

The result of the circular loading with three loading cycles on the specimen PT4.5-2 in [5] is summarized in Fig. 7. The results of the FE analysis and test results for the remaining specimens will be presented at the conference.



(a) (b)

(c) (d)

Fig. 7: Analysed Specimen PT4.5-2, (a) Hysteretic behavior of the column in *x* direction, (b) Hysteretic behavior of the column in *y* direction, (c) Relative displacements in z direction, (d) Buckling near the base of the column

5. Conclusion

The paper investigates the stability of thin-walled steel tubular columns under cyclic bidirectional loading, comparing their response to conventional cyclic unidirectional loading. Through a combination of nonlinear finite element analysis and experimental data, it examines the decline in strength and ductility caused by severe cyclic bidirectional loads. Validation of the finite element analysis is performed using pseudo-dynamic bidirectional tests from existing literature. The Following conclusion can be drawn from this study:

- The findings highlight the importance of understanding steel column behavior under bidirectional loading. Both experimental tests and finite element analysis demonstrate the complex behavior of tubular columns, characterized by a circular trajectory post-local buckling. This trajectory results in a monotonic development of the local buckling bulge and larger residual deformations compared to unidirectional loading scenarios.

- In contrast, unidirectional loading tests and analyses tend to underestimate earthquake-induced damage and residual displacements. Consequently, it is essential to account for bidirectional loading effects when assessing ductility and designing for seismic resistance in steel structures. This study provides valuable insights for improving structural design practices to better withstand bidirectional loading conditions and enhance overall seismic performance.

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