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SEISMIC DESIGN AND RETROFIT OF THIN-WALLED STEEL TUBULAR COLUMNS

Iraj H.P. MAMAGHANI[1]

SUMMARY

This paper deals with seismic design and retrofit of thin-walled steel tubular columns supporting highway bridge superstructures. The basic characteristics of the thin-walled steel tubular columns are noted and the importance of various retrofit techniques in improving strength and ductility capacity of such structures is explained. A seismic design method for ultimate strength and ductility evaluation of the retrofitted, thin-walled, steel tubular beam-columns is presented. The application of the method is demonstrated by comparing the computed strength and ductility of some cantilever columns with test results. The method is applicable for both the design of new and retrofitting of existing thin-walled steel tubular columns. The effects of some important parameters such as width-to-thickness ratio, column slenderness ratio, height of infill concrete, residual stress, arrangement of additional longitudinal stiffeners and energy absorption segment on the ultimate strength and ductility of thin-walled steel tubular columns are presented and discussed.

INTRODUCTION

Steel columns in highway bridge systems are commonly composed of relatively thin-walled members of closed cross-sections, either box or circular in shape because of their high strength and torsional rigidity. Such structures are considerably different from columns in buildings. The former are characterized by: failure attributed to local buckling in the thin-walled members; irregular distribution of the storey mass and stiffness; strong beams and weak columns; low rise (one to three-storey frames); and a need for the evaluation of the residual displacement. These make them vulnerable to damage caused by local and overall interaction buckling in the event of a severe earthquake.

The seismic design and retrofitting methods of new and existing steel bridge piers, with hollow and composite (concrete-filled steel tube) sections, has been the subject of extensive research in Japan following Kobe earthquake in 1995, among others, Usami [1,2], Kitada [3,4] and Mamaghani [5-8]. Outside Japan, research on the strength and ductility of hollow and composite beam columns under cyclic loading has been carried out, among others, by Hajjar [9, 10] and Varma [11], where their investigations limited to column members in steel buildings having small cross sections as compared with those used in

[1]Assistant Professor, Department of Civil Engineering, University of North Dakota, Grand Forks, USA.
Email: iraj.mamaghani@mail.und.nodak.edu

steel bridge piers. Based on the damage sustained by steel bridge piers in Kobe earthquake and extensive test results, one of the important considerations on the seismic design and retrofit of steel bridge piers is to increase the ductility of existing steel bridge piers while keeping their ultimate strength almost unchanged. Therefore a sound understanding of the inelastic behavior of thin-walled steel tubular columns is important in developing a rational seismic design methodology and ductility evaluation of such structures. Also seismic retrofits of existing steel tubular bridge piers that do not satisfy the new seismic design method regulations is needed in order to enhance their strength and ductility capacity.

In this paper, the basic characteristics of the thin-walled steel tubular columns are noted and the importance of various retrofit methods in improving strength and ductility capacity of such structures is explained. The basic seismic retrofitting techniques of the tubular columns, such as, partially filling the column with concrete; and retrofitting the component stiffened plates of the column member by using additional steel members, such as transverse stiffeners, small longitudinal stiffeners, flange plates, are outlined. Adoption of each technique is examined and discussed. A seismic design method for ultimate strength and ductility evaluation of retrofitted hollow and concrete-filled thin-walled steel tubular columns is presented. The method involves an elastoplastic pushover analysis and definition of failure criterion taking into account local buckling and residual stresses due to welding. The application of the method is demonstrated by comparing the computed strength and ductility of some cantilever columns with test results. The effects of some important parameters such as width-to-thickness ratio, column slenderness ratio, height of infill concrete, residual stress, arrangement of additional longitudinal stiffeners and energy absorption segment on the ultimate strength and ductility of thin-walled steel tubular beam-columns are presented.

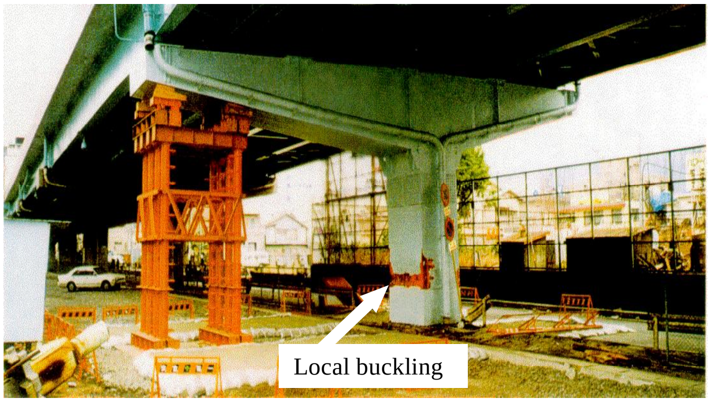
STEEL BRIDGE PIERS OF TUBULAR SECTIONS

Thin-walled steel tubular columns used as steel bridge piers have found wide application in highway bridge systems in Japan compared with other countries, where such structures are much less adopted. Steel tubular bridge piers, compared with concrete ones, are light and ductile. They can be built under severe constructional restrictions, such as in limited spaces at urban areas like New York and Tokyo, where the effective use of the limited spaces are desired strictly. They are also applied to locations where heavy superstructures are unfavorable, such as on soft ground, reclaimed land and bay areas.

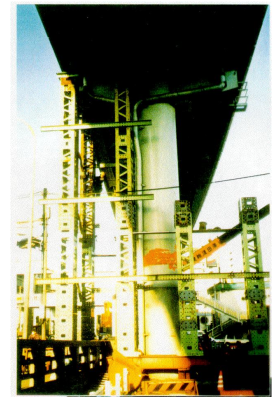
In general, because of these restrictions, steel bridge piers are designed as single columns of the cantilever type, or one to three-storey frames, and they are commonly composed of relatively thin-walled members of closed cross-sections, either box or circular in shape because of their high strength and torsional rigidity. These make them vulnerable to damage caused by local and overall interaction buckling in the event of a severe earthquake. For example, Figure 1 shows two bridge piers of rectangular box section and circular section, which suffered severe local buckling damage in the Kobe earthquake. The piers were partially filled with concrete and local buckling occurred at the hollow portion just above the concrete-filled section. In this section some important parameters affecting the seismic performance, such as strength and ductility capacity, of steel bridge piers are addressed and discussed.

Characteristics of the Cross-section

The cross-section of columns used in bridge piers greatly differs from those of buildings, as shown in Figure 2. The main characteristics of the cross-section of bridge pier columns compared with those in buildings are:



(a) Rectangular section



(b) Circular section

Figure 1. Partially concrete-filled bridge piers, which suffered severe local buckling damage in the Kobe Earthquake, 1995 (truss towers used as temporary supports).

1. the cross-section is large (about 3-4 m in bridge piers compared with 0.6-1 m in tall buildings);
2. stiffened plates are used;
3. ratio of applied axial compressive load to the squash load of the cross-section is small (less than about 0.15). This ratio is as high as 0.6-0.7 for the leeward columns of a tall building;
4. for concrete-filled sections, the ratio of the squash load of the outer steel cross-section (hollow section) to the squash load of the whole (steel plus concrete) section, $\overline{\gamma}$, is small. ($\overline{\gamma}$ is about 0.2 for bridge piers and is about 0.75 for columns in buildings.)

The ductility behavior of steel bridge piers of welded box and circular sections is mainly governed by local component members such as plates, with or without stiffeners.

Design Parameters

The most important parameters considered in the practical design and ductility evaluation of thin-walled steel hollow box sections are the width-to-thickness ratio parameter of the flange plate R_f , and the slenderness ratio parameter of the column $\overline{\lambda}$ (Mamaghani [6,7]). While the former influences local buckling of the flange, the latter controls the global stability. They are given by:

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

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

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

in which, b = flange width; t = plate thickness; σ_y = yield stress; E = Young's modulus; ν = Poisson's ratio; n = number of subpanels divided by longtudinal stiffeners in each plate panel ($n=1$ for unstiffened sections, Fig. 2a); d = diameter of the circular section; h = column height; r = radius of gyration of the

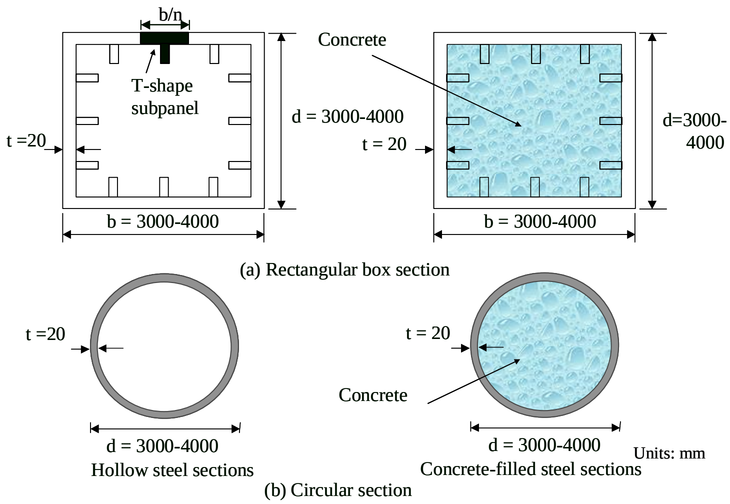


Figure 2. Column cross-sections in bridge piers.

cross section. The stiffener's equivalent slenderness ratio $\bar{\lambda}_s$, magnitude of axial load P/P_y and type of stiffener material are other important parameters considered in a practical design. The parameter $\bar{\lambda}_s$ controls the deformation capacity of the stiffeners and local buckling mode, and is given by:

$$\bar{\lambda}_s = \frac{1}{\sqrt{Q}} \frac{a}{r_s} \frac{1}{\pi} \sqrt{\frac{\sigma_y}{E}} \tag{4}$$

$$Q = \frac{1}{2R_f}[\beta - \sqrt{\beta^2 - 4R_f}\,] \leq 1.0 \tag{5}$$

$$\beta = 1.33R_f + 0.868 \tag{6}$$

where r_s = radius of gyration of a T-shape cross-section consisting of one longitudinal stiffener and the adjacent subpanel of width (b/n) (Figure 2a); a = distance between two adjacent diaphragms; and Q = local buckling strength of the sub-panel plate (Usami [2]). An alternative parameter reflecting the characteristics of the stiffener plate is the stiffener's relative flexural rigidity, γ, which is interdependent on $\bar{\lambda}_s$ and obtained from elastic buckling theory, DIN-4114 [12]. Thus only $\bar{\lambda}_s$ is considered in the ductility equations. The elastic strength and deformation capacity of the column are expressed by the yield strength H_{y0}, and the yield deformation (neglecting shear deformations) δ_{y0}, respectively, corresponding to zero axial load. They are given by:

$$H_{y0} = \frac{M_y}{h} \tag{7}$$

$$\delta_{y0} = \frac{H_{y0}h^3}{3EI} \tag{8}$$

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 gets reduced from H_{y0} 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:

$$\frac{P}{P_u} + \frac{0.85 H_y h}{M_y (1 - P/P_E)} = 1 \qquad (9)$$

$$\frac{P}{P_y} + \frac{H_y h}{M_y} = 1 \qquad (10)$$

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

Loads

In bridge piers the primary loadings may be considered to be axial compressive force (to support superstructures) with little variations, and alternating bending in response to seismic loads and wind loads. Bending force is the primary section force, appearing as a stress resultant which produces repetitive axial forces in opposite, outer, thin plate elements of the cross section. These thin elements are subjected to cyclic axial compression and tension due to alternating bending moment, resulting in successive buckling until the occurance of complete sectional collapse.

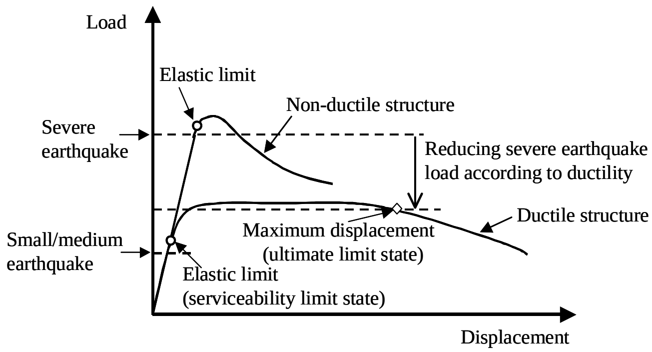
RETROFITTING TECHNIQUES

It has been widely realized in modern seismic design philosophy that a rational seismic design method should be able to explicitly evaluate the structural nonlinear performance that is expected to occur under the design seismic load. Recently, for reasons of economy, the design of structural restraint to severe earthquakes has focused on the idea of ductility-based design, a shift away from strength-based design. This requires attention to both strength and ductility. Figure 3a illustrates a schematic load-displacement response of ductile and non-ductile steel bridge piers. As shown in this figure, design loads for severe earthquakes can be considerably reduced according to the ductility of the structure. Since bridge piers are either statically determinate or indeterminate to a low degree, an increase in strength for large displacements cannot be expected, even if local instability is disregarded. Hence a large degradation in strength, due to local buckling at least, needs to be prevented.

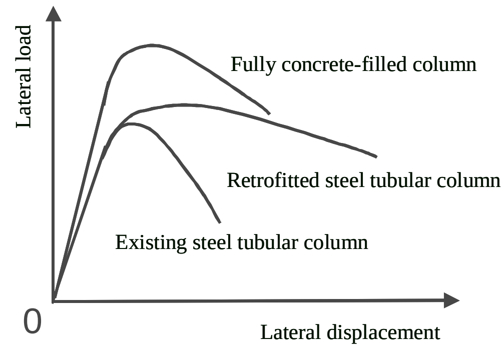
Necessity for Retrofitting

Steel bridge piers, which are elastically designed for low and moderate earthquakes, on the basis of a seismic horizontal acceleration of up to 200 gals, should never collapse under strong earthquakes, that rarely occur during their design life, although they may lose some of their serviceability functions. One of the main reasons for this requirement is the collapse of bridges will have large impact on the post-earthquake disaster preventions. Therefore seismic retrofit adopted for improving the seismic performances of a bridge pier needs to satisfy the following requirements:

1. The retrofit technique should be easy, practical and economic.
2. Damage can be easily detected and repaired.
3. The retrofitted steel bridge piers should endure strong earthquakes, such as Kobe earthquake, without suffering serious damage.
4. The retrofitted steel bridge piers should provide the intended serviceability functions without any restriction after the earthquake.



(a) Concept of seismic design



(b) Concept of retrofitting

Figure 3. Concepts of seismic design and retrofitting.

Based on the survey of damage to steel bridge piers in Kobe earthquake and the findings from analyses and experiments three fundamental concepts of the seismic design and retrofit of steel bridge piers are:

1. Avoidance of brittle failure mode
2. Improvement of ductility
3. Restriction of residual displacement.

Avoidance of brittle failure mode:

Brittle failure modes of steel bridge pier with rectangular section may take place when vertical cracks along the welded corners of stiffened steel plate panels occurred after the commence of serious local buckling of stiffened steel plate panels. For bridge piers of circular section, local buckling causes concentrated large deformation at a damaged zone, resulting in cracking around the column's circumference due to the increased deformation. As shown in Figure 3b for fully concrete-filled steel tubular columns, both the ductility and load carrying capacity can be enhanced significantly because the encased concrete can prevent the stiffened plates of the steel column member from buckling. If concrete is encased throughout the column height, cracks at the corners of the steel box cross sections or fracture of anchor bolts supporting the column member at its bottom are possible failure modes. Brittle failure mode causes loss of the compression strength against the dead load, weight of the superstructure, resulting in full collapse of the bridge. Such non-ductile failures are undesirable not only because detection and inspection are difficult, but also because the repair, replacement or retrofitting of the foundation is expensive and intricate. Therefore retrofitting the existing bridge piers not satisfying the current design requirements must prevent brittle failure mode.

Improvement of ductility

The ductility design method applied to steel bridge piers since damages caused by the recent earthquakes suggest the importance of ensuring sufficient ductility of the structural systems. Especially for the seismic retrofitting of existing steel bridge piers, it is important to select an adequate retrofit technique to ensure the proper seismic performance by considering the design condition and the foundation of the pier. That is, the design concept based on a ductility capacity design method will be a key factor for a sound retrofit. As the load carrying capacity of the foundation for an existing bridge pier is limited, a retrofit technique to enhance the ductility capacity without significant increase in the ultimate strength capacity should be adopted, because it is very uneconomical to strengthen existing foundation structures.

Restriction of residual displacement
Seismic design based on the ductility requires restricting the residual displacement to a repairable limit. This is because the excessive ductility in ductile segment of steel bridge piers causes large deformation accompanied with large residual displacements, which makes the restoration work more difficult.

Techniques to Improve Strength and Ductility
The techniques to improve ductility capacity without significant increase in strength of steel bridge piers of hollow tubular sections can be summarized as follows:

1. limiting the witdth-to-thickness ratio of component plates;
2. limiting the slenderness of columns;
3. filling the tube with concrete (composite construction);
4. use of the inelastic characteristics of high performance structural steel; and
5. use of a ductile cross-sectional shape.

Among these, in practice, limiting the width-to-thickness ratio of component plates and the slenderness of the column are the most suitable methods for enhancing the strength and ductility. The current Japanese design guideline limits the width-to-thickness ratio parameter to $R_f \leq 0.4$ (Kitada [3]), the rigidity of longitudinal stiffeners to $\gamma / \gamma^* \geq 3.0$ (γ^* = optimum value of γ), the column slenderness ratio to $\bar{\lambda} \leq 0.60$, and the axial compressive force ratio to $P/P_y \leq 0.20$ (Fukumoto [13], HEPC [14]). These quite conservative limitations ensure reasonable safety for steel bridge piers, even in the event of a strong earthquake (Usami [2]).

In practice, one or a combination of the following techniques can be adopted for retrofitting of the steel tubular columns.

1. Stiffeners strengthening technique;
2. Partially concrete-filled columns technique;
3. Energy absorption segment technique.

Stiffeners strengthening technique
The stiffeners strengthening technique for existing steel bridge piers can be adopted when the concrete filling technique is not adoptable. The following retrofits are applied to existing stiffened plate panels and longitudinal stiffeners (Figures 4 and 5) to satisfy the above-mentioned restrictive conditions on the buckling parameters:

1. reinforcing with additional longitudinal stiffeners;
2. increasing the rigidity of longitudinal stiffeners;
3. adding transverse stiffeners between diaphragms;
4. adding corner reinforcement for box sections; and
5. providing stepped steel cross-section by adding steel jacket plates.

The plate panels between the existing longitudinal stiffeners are stiffened with the additional small longitudinal stiffeners. The existing longitudinal stiffeners are further stiffened with the additional flange plates to meet the required buckling parameter of the stiffened plate panels and longitudinal stiffeners. Moreover, small gaps are introduced at both ends of the retrofitting longitudinal stiffeners between stiffeners and the diaphragms. The segments of column at these gaps are expected to undergo large plastic deformations prior to the deformation of the other parts of the column members. This allows the retrofit technique to improve the ductility capacity, through preventing buckling of the column member

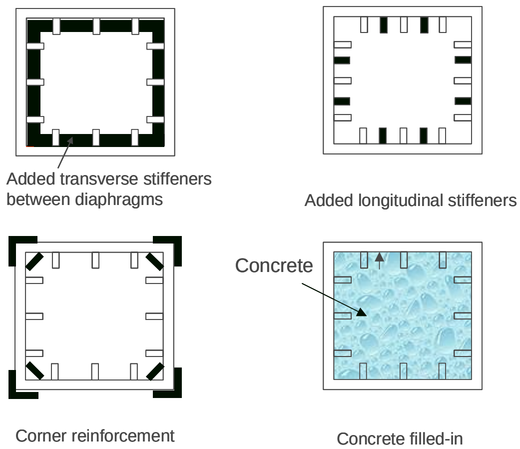


Figure 4. Retrofitting technique for rectangular box columns.

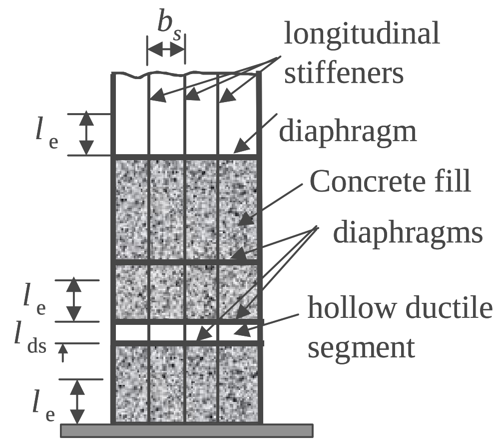


Figure 5. Outline of concrete-filled steel bridge pier column with energy absorption segment.

consisting of the stiffened plate panels, without increasing the load carrying capacity significantly.

Partially concrete-filled columns technique

Partially filling the hollow steel tube with concrete (and thereby using composite construction) is one of the most suitable available methods for improving strength and ductility because the concrete core inhibits local buckling of the thin steel tube and, at the same time, the steel tube provides a confining stress to the concrete core (Figure 6). In this technique existing steel tubular columns are retrofitted by:

1. Filling with concrete inside steel sections from the base, up to an optimal level;
2. Providing inner steel tube and filling the gap between tubes, up to an optimal level; and
3. Placing a diaphragm over the filled-in concrete.

The main ideas underlying partially concrete-filled bridge piers are:

1. to increase the ductility for resisting strong earthquakes, without substantially increasing the stiffness and/or strength, compared with steel bridge piers of hollow tubular sections of the same size (see Figure 3b for the retrofitted steel tubular column curve);
2. to reduce the self weight compared with fully concrete-filled steel bridge piers.

This is advantageous because:

1. fully concrete-filled steel bridge piers can substantially increase stiffness, and/or strength, but they behave in a relatively brittle (nonductile) manner and do not provide sufficient ductility to resist very strong earthquakes (see Figure 3b for the fully concrete-filled column curve);
2. foundation structures supporting bridge piers become large and expensive if the self-weight and strength of the piers are increased.

Energy absorption segment technique

The energy absorption segment technique utilizes installing a short energy absorption (hollow ductile) segment in the column member, as illustrated in Figure 5. The energy absorption segment deforms plastically prior to the other parts of the column member to enhance the ductility capacity and to control the ultimate strength capacity

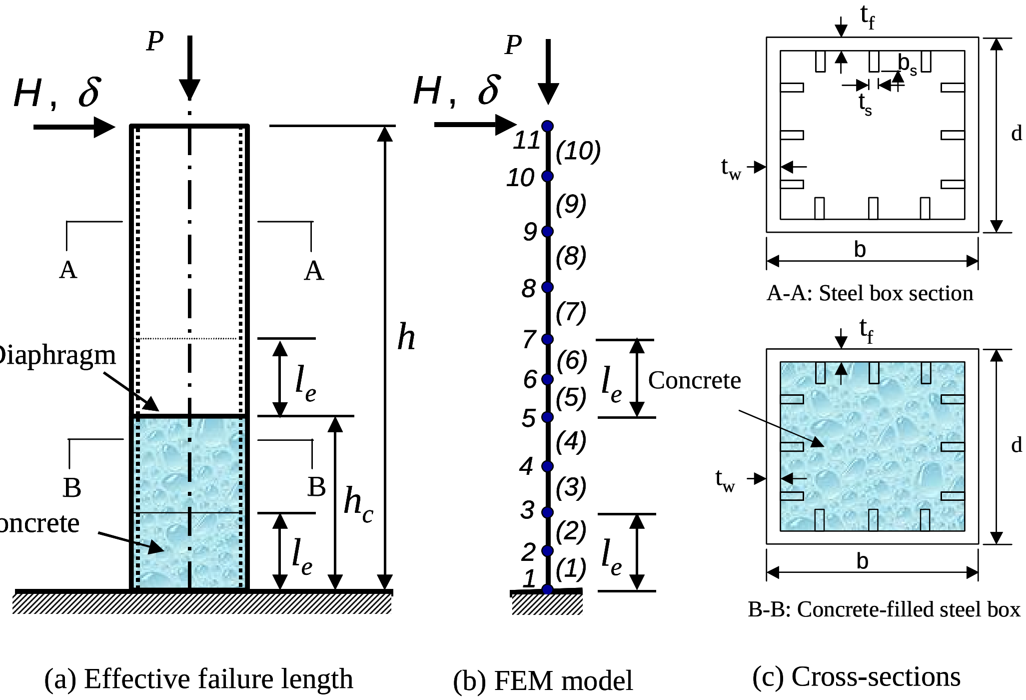


Figure 6. Analytical modeling of partially concrete-filled steel box columns.

of the column member not to exceed that of the foundation. The length of the energy absorption segment should be small enough so as not to cause the stiffened plate to buckle in the segment, which is expected to undergo large deformations and to extenuate the increasing of the ultimate strength. This retrofitting technique provides large energy dissipation capacity. This technique provides an economic solution while it is practically easy to perform (Kitada [3]).

The behavior of hollow and concrete-filled steel tubular bridge pier columns has been studied by several researchers, among others, by Mamaghani [15-17] and Usami [1,2]. The results of conducted tests and observed damage sustained by bridge piers in the Kobe earthquake indicate that local buckling may occur at the hollow portion just above the concrete-filled section (Figure 1), when the height of concrete fill is small compared to the height of the column. Using the above retrofitting techniques it is possible to enhance the seismic performance of steel tubular columns substantially. In what follows, an analytical procedure for ultimate strength and ductility evaluation of the retrofitted steel tubular columns is presented and discussed.

PROCEDURE OF STRENGTH AND DUCTILITY EVALUATION

The strength and ductility evaluation of the retrofitted, thin-walled steel tubular columns of the cantilever type, modeling steel bridge piers, consists of the following steps:

1. Initially determine the general layout of the existing structure to be retrofitted. Determine the section type and size, the height of column, material, height of concrete fill, and estimate the superstructure weight and lateral loads via an elastic seismic design method.
2. Carry out a pushover analysis to obtain the load-displacement relationship. The basic procedures in the pushover analysis utilized in this study are summarized as follows:
   (a) Based on the general layout and loading condition of the structure, establish the analytical model as shown in Figure 6 by using beam-column elements.

(b) Set the height of concrete fill h_c (initially h_c / h may be set to 0.25 for partially concrete-filled columns).

(c) Choose failure criteria for both steel and concrete. The ultimate limit state corresponding to the failure criterion is used as the termination of the analysis. The adopted stress-strain relation for steel and concrete and the effective failure length and failure criteria are explained later.

(d) Apply the constant vertical load (including weight of the superstructure) and laterally push the structure until the ultimate limit state is reached.

(e) Based on the base shear H versus top lateral displacement δ curve from the analysis, determine the ultimate strength H_u and ductility δ_u of the structure.

3. Check for seismic performance of the structure. If the capacity cannot meet the seismic demand, change the height of concrete fill and/or retrofitting stiffeners and repeat the procedure starting from step 2 until the damage indices for both steel and concrete simultaneously approach unity (corresponding to optimum height of concrete fill). In designing new structures, if the capacity still cannot meet the seismic demand, change the general layout of the structure and repeat the procedure from step 1 until the capacity meets the seismic demand. The seismic verification procedure is presented later.

The above procedure is easily applicable for the steel hollow sections and all retrofitting techniques discussed above. It can be implemented for both the design of new steel bridge piers and the retrofit of existing bridge piers composed of thin-walled members filled with concrete.

Failure Strain for Steel Plate Members

Local buckling controls the capacity of bridge piers composed of thin-walled steel members. A number of experimental and numerical analyses have shown that the critical local buckling of thin-walled steel structures always appears in the compressive flange plates within an effective failure range (Usami [1]). The ductility of isolated plates with and without longitudinal stiffeners has been investigated through extensive parametric analyses (Usami [1], Banno [18]). The normalized failure strain $\varepsilon_{f,s} / \varepsilon_y$ of a plate under pure compression is given by:

for an unstiffened plate:

$$\frac{\varepsilon_{f,s}}{\varepsilon_y} = \frac{0.07}{(R_f - 0.2)^{2.53}} + 1.85 \le 20.0 \qquad (11)$$

for a stiffened plate:

$$\frac{\varepsilon_{f,s}}{\varepsilon_y} = \frac{0.145}{(\bar{\lambda}_s - 0.2)^{1.11}} + 1.19 \le 20.0 \qquad (12)$$

in which, $\varepsilon_{f,s}$ = failure strain corresponding to 95% of maximum strength after peak load. The occurrence of local buckling can be neglected when $\bar{\lambda}_s$ is smaller than 0.2.

Failure Criteria

In a thin-walled steel bridge pier, excessive deformation tends to develop in a local part and consequently the redistribution of stress becomes unexpected. The test results indicate that the local buckling occurs near the column base (for a composite section) or in the hollow section just above the concrete fill in the range of about 0. 7b (b = breadth of the flange) or between the transverse diaphragms, if any, depending on the height of the concrete (Usami [2]). These critical local parts are defined by an effective failure length l_e, as marked in Figures 5 and 6. The energy absorption segment is also taken as an effective length, $l_e = l_{ds}$, when this technique is adopted as shown in Figure 5. To establish the ultimate state, the average strain in the outer fibers of the concrete and steel segments along the length l_e, at composite and hollow parts, is recorded. The failure criterion can be described by damage indices for steel D_s and concrete D_c, which are defined by:

$$D_s = \frac{\varepsilon_{ave,s}}{\varepsilon_{f,s}} \qquad (13)$$

$$D_c = \frac{\varepsilon_{ave,c}}{\varepsilon_{f,c}} \qquad (14)$$

The ultimate limit state of the structure is considered to be attained when any one of these indices first reaches unity. Here, $\varepsilon_{ave,s}$ represents the average strain of the compressive flange for the box section and $\varepsilon_{ave,c}$ is the average strain of the most compressive outer fibre of the concrete core over corresponding effective failure lengths. $\varepsilon_{f,c}$ denotes the failure strain of concrete and is defined to be $\varepsilon_{f,c} = 0.011$. $\varepsilon_{f,s}$ denotes the failure strain of steel plate members given by Equations 11 and12.

NUMERICAL ANALYSIS

The pushover analysis, discussed above, is applied to simulate the load-displacement response of the tested columns with and without retrofitting (Usami [1]). An example of the analytical model is shown in Figure 6. The partially concrete-filled steel bridge pier is modeled by using beam-column elements. An elastoplastic finite element formulation for a beam-column, considering geometrical and material nonlinearities, was developed and implemented in the computer program FEAP (Zienkiewicz [19]), which was used in the analysis (Mamaghani [5,8]).

Uniaxial stress-strain relations for steel and concrete are assigned to beam-column elements, which are employed to model the steel and concrete segments. The monotonic stress-strain curve of the modified two-surface plasticity model (2SM), developed by the author and his co-workers at Nagoya University (Mamaghani [20], Shen [21]), is employed in the analysis. The stress-strain model for the concrete material employed in the analysis accounts for the confinement effect provided by the surrounding steel plates (Uasmi [1]). The analysis accounts for the residual stresses due to welding (Mamaghani [5]). The details of the numerical analysis and comparison between test results and analytical results have been reported in the works by Mamaghani [5].

Numerical Results

The three retrofitting techniques are examined using numerical results obtained by employing the strength and ductility evaluation procedures and numerical method discussed in this paper as well as the test results available in the literature. Figures 7a-c shows the results for the cantilever column of $R_f = 0.750$, $\bar{\lambda} = 0.276$, and $h_c / h = 0.3$, with longitudinal stiffeners obtained from test and analysis.

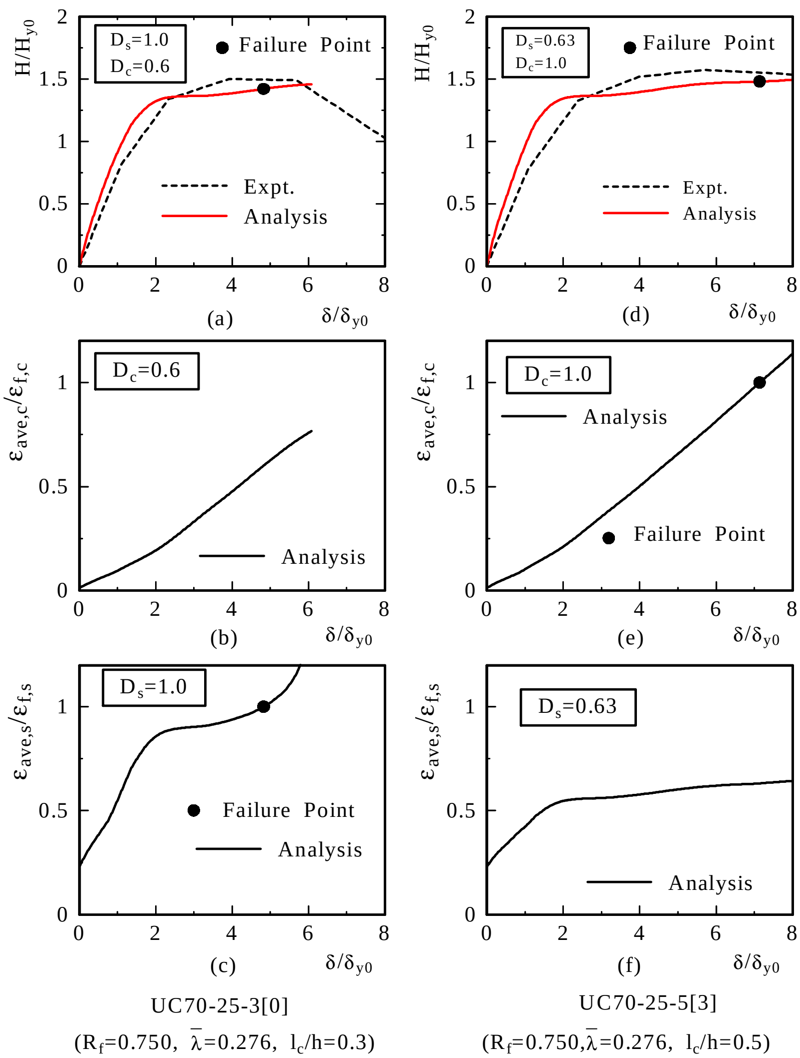


Figure 7. Comparison of tests and analyses: Improvement of ductility by partially concrete-filled retrofitting technique.

Figure 7a compares normalized lateral load-lateral displacement (H - δ) curves for the experiment and analysis. In this figure the envelope curve (dashed line) of the hysteretic behavior of the cyclic experiments (Usami [1]) is used for comparison. Figure 7b shows the concrete damage index $D_c = \varepsilon_{ave,c} / \varepsilon_{f,c}$ versus normalized lateral displacement. Figure 7c shows the steel damage index $D_s = \varepsilon_{ave,s} / \varepsilon_{f,s}$ versus normalized lateral displacement. As described in the previous section, the failure of the column is attained when any one of these indices first reaches unity. The failure points marked in Figures 7a and 7c correspond with the ultimate displacement of the columns at failure, taking place in the hollow section just above the concrete fill level with $D_s = 1.0$ (Figure 7c), while $D_c = 0.60$ (Figure 7b). Figures 7d-f compares the results for the same column with the fill concrete height of $h_c = 0.5h$. In this case the failure of the column is attained in the concrete-filled section with $D_c = 1.0$, (Figure 7e), while $D_s = 0.65$ (Figure 7f). As shown in Figure 7a and 7d, the analytical model closely simulates the test results. The failure modes are the same in both the experiments and analyses indicating the accuracy of the analytical model employed.

Table 1 Effects of residual stress, R_f, $\bar{\lambda}$, h_c/h on strength, ductility capacity and failure mode.

Specimen				Without R.S.		With R.S.		Without R.S.		With R.S.		Failure mode
	h_c/h	R_f	$\bar{\lambda}$	H_u/H_{y0}	δ_u/δ_{y0}	H_u/H_{y0}	δ_u/δ_{y0}	D_s	D_c	D_s	D_c	
UU2	0.3	0.664	0.362	1.36	4.43	1.36	4.08	1.0	0.57	1.0	0.49	Steel
UU3	0.5	0.664	0.362	1.41	6.36	1.41	6.79	0.35	1.0	0.64	1.0	Concrete
UU5	0.3	0.664	0.577	1.24	3.53	1.22	3.14	1.0	0.49	1.0	0.45	Steel
UU7	0.3	0.854	0.381	1.40	2.96	1.34	2.02	1.0	0.33	1.0	0.17	Steel

As shown in Figure 7a and 7d, the ductility of the column δ_u/δ_{y0} is improved by 48 percent (from 4.83 to 7.14) when the height of concrete fill is increased from 30 to 50 percent of the column height ($h_c = 0.3h$ to $h_c = 0.5h$) while the change in column ultimate strength is 4.2 percent, which is negligibly small (H_u/H_{yo} changed from 1.42 to 1.48). This is attributed to the fact that failure occurs in hollow steel box sections for lower h_c/h ratios ($h_c/h = 0.3$) just above concrete section while for higher h_c/h ratios ($h_c/h = 0.5$) failure occurs at concrete-filled sections near the base of the column. This satisfies the main idea underlying partially concrete-filled retrofitting technique, namely to increase the ductility for resisting strong earthquakes without substantially increasing the strength.

Examples of the numerical results obtained for other experiments are summarized in Tables 1. With reference to Table 1 and based on the numerical analysis and test results the effects of some important parameters on the strength and ductility capacity of the retrofitted columns are summarized as follows:

1. Failure modes are the same in both the experiments and analyses indicating the accuracy of the analytical model employed; Figure 7 and Table 1.
2. Failure occurs in hollow steel box sections for lower h_c/h ratios ($h_c/h = 0$ □ 0.3), Figure 7 and Table 1.
3. Failure occurs in concrete-filled sections for higher h_c/h ratios ($h_c/h = 0.5$ and above), Figure 7 and Table 1.
4. Residual stress has the effect of reducing the ductility of columns in which failure occurs in hollow steel box sections ($h_c/h = 0.3$). However, the residual stress causes an increase in ductility when the governing failure mode is concrete failure ($h_c/h = 0.5$). The effect of residual stress becomes more apparent with the increase in width-to-thickness ratio R_f.
5. Ductility of the column is improved when the height of concrete fill is increased from 30 to 50 percent of the column height while the change in column ultimate strength is negligible; Figure 7 and Table 1. This satisfies one of the main ideas underlying partially concrete-filled retrofitting technique: namely to increase the ductility for resisting strong earthquakes without substantially increasing the strength.
6. The ductility capacity substantially decreases with an increase in column slenderness ratio $\bar{\lambda}$ and width-to-thickness ratio R_f, where steel failure is the governing failure mode.
7. The results from experiments and analysis suggest that the optimum ductility of partially concrete-filled, thin-walled, steel bridge piers can be attained when concrete failure is the governing failure mode with $D_c = 1.0$ and the damage sustained by steel is very close to unity. That is, using the discussed retrofitting techniques and ductility evaluation program in this paper, the optimum seismic design and ductility capacity of such bridge piers can be achieved by arranging the bridge pier parameters such as R_f, $\bar{\lambda}$ and h_c/h to satisfy this optimum failure mode that provides enhanced ductility capacity.

SEISMIC VERIFICATION METHOD

To verify seismic performance of a bridge pier, the following condition for supply (earthquake-resistance) capacity S, provided by the structure and demand capacity D, required by an earthquake should be satisfied:

$$S \geq D \tag{15}$$

The demand capacity D, for a specific earthquake motion, can be obtained from the discussed pushover analysis by determining the H - δ curve, in which the ultimate displacement of the bridge pier δ_u corresponds to the ultimate strength H_u at failure, Figure 8. The factored lateral displacement δ_f, can be obtained by applying a safety factor against the ultimate displacement δ_u, and its corresponding factored lateral strength H_f can be obtained from the H - δ curve. Based on the equivalence in energy absorption capacity criterion, the supply and demand capacities are given by:

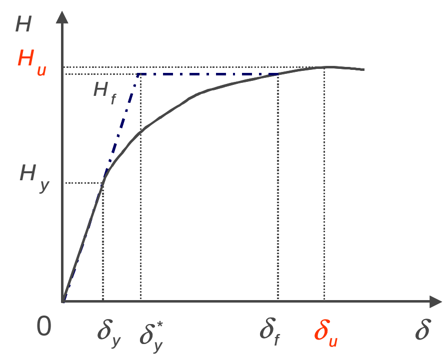


Figure 8. H - δ curve

$$D = k_{hc} W \tag{16}$$

$$S = \sqrt{2\mu_f - 1}\, H_f \tag{17}$$

$$\mu_f = \frac{H_y}{H_f} \frac{\delta_f}{\delta_y} \tag{18}$$

where k_{hc} = seismic coefficient; W = equivalent weight of both the superstructure and pier; μ_f = factored global ductility. This condition should be met for the design of both new and retrofitted structures.

CONCLUSIONS

The paper deals with seismic design and retrofit of thin-walled steel tubular columns supporting superstructures in highway bridges. The important characteristics of the thin-walled steel tubular columns are noted and the basic seismic design and retrofitting concepts of such structures are presented. Three seismic retrofitting techniques of the thin-walled steel tubular columns, namely, stiffeners strengthening technique, partially concrete-filled columns technique, and energy absorption segment technique are outlined. Adoption of each technique is examined and discussed. A seismic design method for ultimate strength and ductility evaluation of retrofitted thin-walled steel tubular columns is presented. The method involves an elastoplastic pushover analysis and definition of failure criterion taking into account local buckling and residual stresses due to welding. The application of the method is demonstrated by comparing the computed strength and ductility of some cantilever columns with test results. The method is applicable for both the design of new and retrofitting of existing thin-walled steel tubular columns. The effects of some important parameters, such as width-to-thickness ratio, column slenderness ratio, height of infill concrete, residual stress, arrangement of additional longitudinal stiffeners and energy absorption segment on the ultimate strength and ductility of thin-walled steel tubular columns are presented and discussed. It is concluded that the presented retrofitting techniques and seismic design evalaution method can be practically implemented for both the design of new steel bridge piers and the retrofit of existing bridge piers composed of thin-walled steel tubular columns.

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