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Steel bridge columns with pre-selected plastic zone for seismic resistance

Sheng-Jin Chen *, Jie Chen

Department of Construction Engineering, National Taiwan University of Science and Technology, P.O. Box 90-130, Taipei, Taiwan

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ABSTRACT

In the current design practices, the inelastic deformation of steel bridges under seismic load usually concentrates toward the bottom of the column, which is typically under the ground level. However, it is difficult to inspect or repair if damages occur in this portion. This study examined the behavior of bridge columns with a pre-selected plastic zone away from the bottom of bridge columns. The pre-selected segment is designed following the strength requirement of the bridge column, which is subjected to seismic force. Under strong ground excitation, the inelastic deformation of the bridge structure is confined in the pre-selected segment. Special detailing is adopted to enhance the ductility and energy dissipation capacity of the bridge column. After earthquakes, it is easier to perform inspection, to retrofit, or to replace the damaged segment, if necessary. A series of experimental studies were performed to examine the behavior of the proposed steel bridge columns with pre-selected energy dissipation segments. From these studies, it is shown that the seismic resistance of a bridge column can be enhanced effectively by adopting this design method.

1. Introduction

Bridges are important elements of the transportation system. However, bridges are also vulnerable to the earthquake excitation. If bridges were damaged during the earthquake, this may obstruct the operation of the rescue works or even the reconstruction after the disaster. Since bridges are one of the major parts of the transportation systems, reliable bridge structures are essential to keep the function of the transportation system in the event of earthquake. In the seismic active area, the seismic resistance capability is the major concern in the design and construction of bridge structures. Due to the requirements of stiffness and strength, bridge girders are usually stronger than bridge columns. During earthquake excitation, bridge columns are usually subjected to higher stresses and inelastic deformation may occur at the bridge columns. The performance of a bridge column under seismic load may govern the function of the bridge or even the transportation system. A steel bridge column is usually anchored to the reinforced concrete foundation or the cap of the pile foundation. During the earthquake excitation, maximum moment is induced at the bottom of the bridge column and this may lead to the failure of the column. Although steel structures are usually considered as possessing ductile behavior, steel bridges also suffered from damages during past earthquakes. For example, during the Kobe earthquake that occurred at Kobe, Japan, 1995, many bridge columns suffered from different degrees of damages [1]. These included local buckling of the bridge column, fracture at weld or steel plate, failure of anchor bolts, and even the collapse of the bridge pier. These failures indicate that ductile material does not ensure ductile performance of steel bridges. Proper detailing is necessary to ensure ductile performance of the steel bridges. Research works have been carried out to examine the effect of the width-to-thickness ratio of component plates of bridge columns, slenderness of columns, effect of the filled-in concrete, and even the welding detailing of the component plates of steel bridge columns [2–7]. These studies provide methods to enhance the seismic behavior of the bridge columns by improving the sectional properties of the column to avoid pre-mature buckling or fracture. Application of newly developed low-yield-point steel on the steel bridge column was also examined [8]. A new type steel plate, the longitudinally profiled steel plate (tapered plate), has been suggested in order to provide larger yielding zone on the bridge pier [9]. In the retrofit of the bridge columns, a small segment that is weaker than the remaining part of the pier is suggested so that yielding will occur in this segment [10].

Due to the nature of seismic moment along the height of the column, the maximum moment usually occurs at the bottom of the column and governs the design, as shown in Fig. 1. The inelastic deformations due to seismic moment are concentrated toward the bottom of the column, especially around the area of the joint between the column and the foundation. The joint between the bridge column and the foundation is usually under the ground level. After the earthquake, it is difficult to inspect the soundness or to retrofit the column at this portion. Besides, even

* Corresponding author. Tel.: +886 2 2737 6563; fax: +886 2 2737 6606.
E-mail address: sjchen@mail.ntust.edu.tw (S.-J. Chen).

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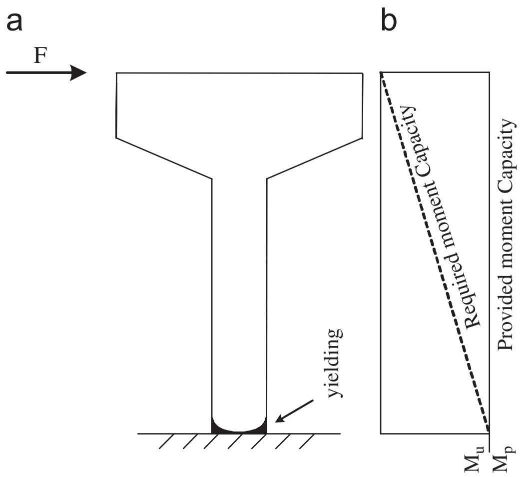

Fig. 1. Moment profile of bridge column under earthquake loading.

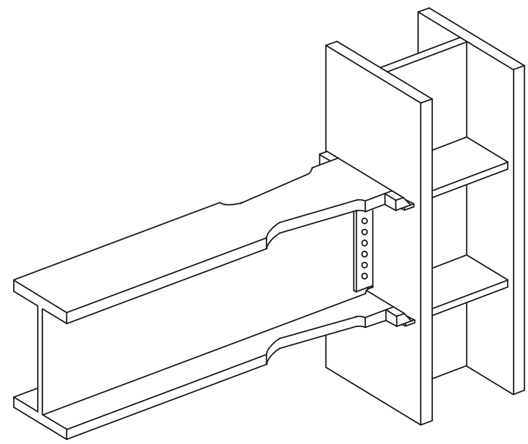

Fig. 2. Sketches of ductile beam-to-column connection.

in the periodical inspection and maintenance, it is difficult to detect a defect or crack at the bottom of the column. In recent design practice, in addition to the requirements of structural strength, constructability and inspectability need to be considered in the design of a bridge. It is also suggested that the inelastic response should occur at locations that are designed to provide ductile energy absorbing response [11]. However, there is no explicit design regulation or guideline for the designer to follow. This reported research is aimed at developing a simple method for the design of a steel bridge column with pre-selected plastic segment so that the inelastic deformation is confined to this portion. The pre-selected plastic segment is designed following the seismic moment requirement so that an enlarged plastic zone can be obtained and the seismic resistance capacity of the bridge column can be enhanced. Besides, the pre-selected segment is similar to a "fuse" so that it is easier to inspect or repair, if necessary, after the earthquake.

2. Steel bridge columns with pre-selected plastic zone

Due to its high strength and superior ductility, steel has been widely used in the construction of buildings and bridges in the seismic active area. However, in the Northridge earthquake and Kobe earthquakes, brittle performance of steel buildings and bridges was observed [1,12]. Some of the structures experienced cracks on the weld around the area of maximum stress. Many steel buildings were found to have brittle fracture at beam-to-column connections [12]. After the Northridge earthquake, the design of beam-to-column connection has been emphasized on moving the plastic hinge away from the joint between beam and column where stress is the highest. Recently, the reduced beam section method in which part of the beam flanges are trimmed at the pre-selected position has become very popular in the design of seismic moment connections [13–15].

Many different types of reduced beam section method have been suggested for beam-to-column connection. Fig. 2 shows the connection method suggested by the first author [14,15]. This connection method can be explained by examining the moment distribution of a typical frame as shown in Fig. 3. Fig. 3a shows the seismic moment distribution along the span length of the beam. It is shown that the maximum seismic moment occurred at the beam-to-column joints. Fig. 3b shows the inelastic zone of a traditional plastic hinge formed at the joint. Assuming an elastic-perfect-plastic steel material, the amount of the plastic zone formed at the beam-to-column joint depends on the shape factor of the beam section (the ratio of plastic modulus to the section modulus) and the moment gradient along the beam. Besides, the plastic hinge is formed at the junction between column and beam where lots of stress raisers, such as geometry change, welding access holes, bolt holes, and even weld defects, exist. These stress raisers usually deteriorate the capability of the plastic deformation of the connection. Fig. 3c shows the ductile beam-to-column connection method, in which that the beam flanges are trimmed, so that the provided moment capacity at the pre-selected area is equal to the seismic moment demand. By this arrangement, uniform yielding at the pre-selected area can be obtained. From the plastic zone shown in Fig. 3c, one can find that the reduced beam section method can enhance the capacity of plastic energy dissipation at the pre-selected location. Furthermore, even with the reduced beam sections, its strength can be the same as that of the original prismatic beam, since the beam is designed to let the provided strength equal the required moment strength. This connection method has been widely used in the construction of modern high-rise buildings. The 101-stories building, Taipei Financial Building, is one of the buildings adopting this connection method [16].

The reduced beam section method not only enhances the energy dissipation capacity of beam-to-column connections, but also simplifies the inspection and retrofit work after the earthquake. The same concept can be adopted for the seismic design of a bridge column. Fig. 1 shows a typical bridge column under seismic load. Fig. 1b shows the moment distribution of the bridge pier under seismic excitation. One can easily find that the seismic moment profile of Fig. 1b is similar to that of Fig. 3b. The maximum stress tends to concentrate toward the bottom of the column and only limited plastic energy dissipation capacity is obtained. Moreover, since the joint between the bridge column and the foundation is usually under the ground, it is difficult to perform the inspection at this portion. Fig. 4 shows the proposed design method, in which the plastic hinge is moved away from the bottom of the column. The strength of the pre-selected plastic zone is designed to let the provided moment strength equal the seismic moment demand. By this method, the pre-selected zone can be plasticized uniformly without strength reduction, as shown in Fig. 4b. This ductile segment is plasticized prior to the occurrence of any plastic deformation in the other part of the column and greatly simplifies the inspection and repair work after the earthquake. Further more, the length of the pre-selected plastic zone along the column height can be designed according to the demand of energy dissipation. By adopting this new design method, the inelastic zone can be moved away from the foundation and the ductility of the column can be enhanced by

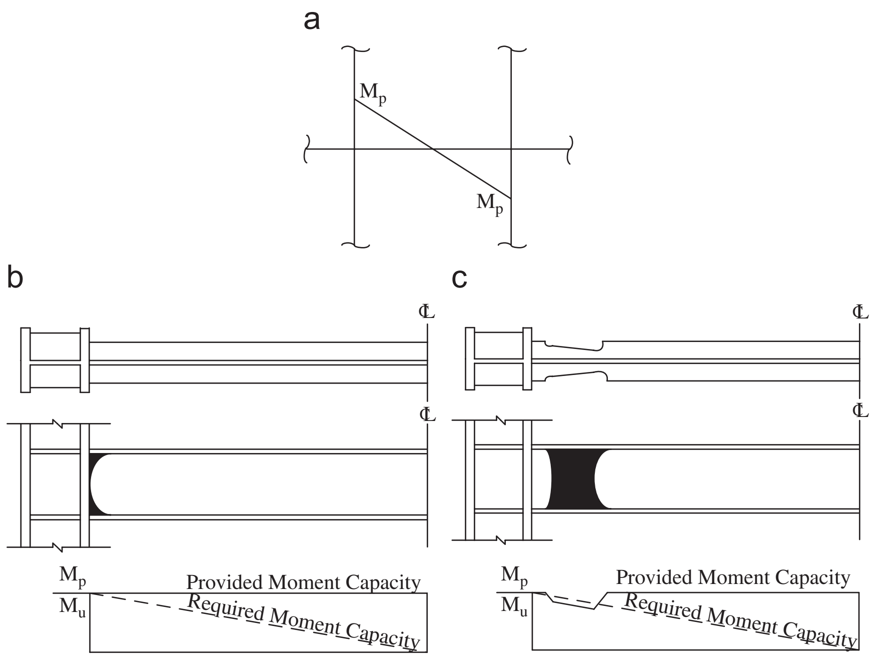

Fig. 3. Plastic behavior of beam-to-column connection (black zone representing plastic zone).

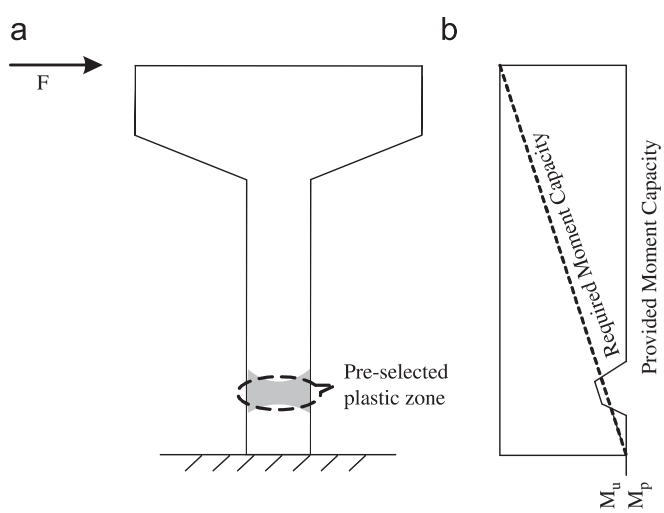

Fig. 4. Bridge column with pre-selected plastic zone.

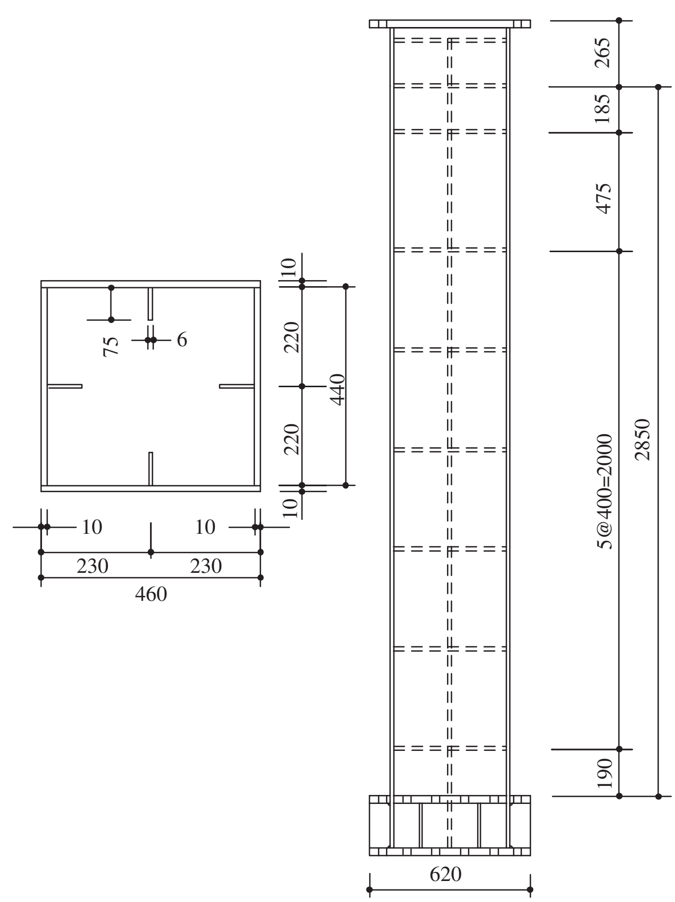

Fig. 5. Steel bridge column specimen.

designing a segment of uniform yielding. Experimental studies were performed to examine the seismic performance of this new type of bridge columns.

3. Experimental studies of bridge columns under seismic load

A series of experimental studies was carried out to examine the behavior of the proposed steel bridge column with pre-selected plastic segment (Figs. 5–7). Specimen No. 1 adopts the conventional design. Specimen No. 2 is designed with a pre-selected plastic segment 590 mm away from the junction of the column and foundation (Fig. 6). The length of the ductile segment is 133 mm. The reduced moment capacity is achieved by trimming the stiffeners in the pre-selected zone, as shown in Fig. 6. The required seismic moment and the provided seismic moment capacity are similar to those shown in Fig. 4b. It is noted that the stiffener of Specimen No. 2 is trimmed un-symmetrically. This is due to the nature of seismic moment demand, which is gradually decreasing away from the foundation. The strength of the non-uniform segment is proportional to that of the moment gradient, resulting in a more uniform stress distribution on the pre-selected

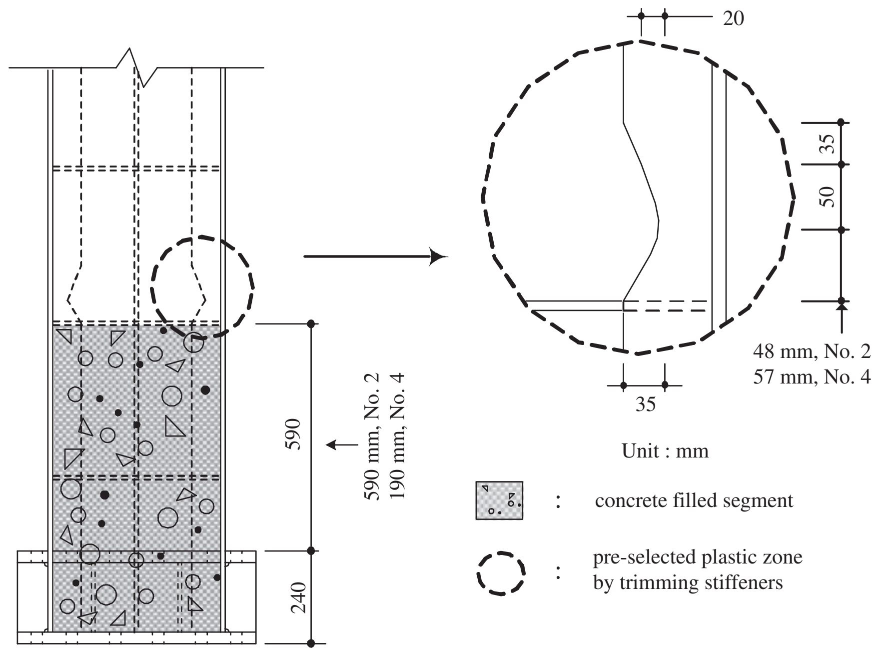

Fig. 6. Details of Specimen No. 2 and No. 4.

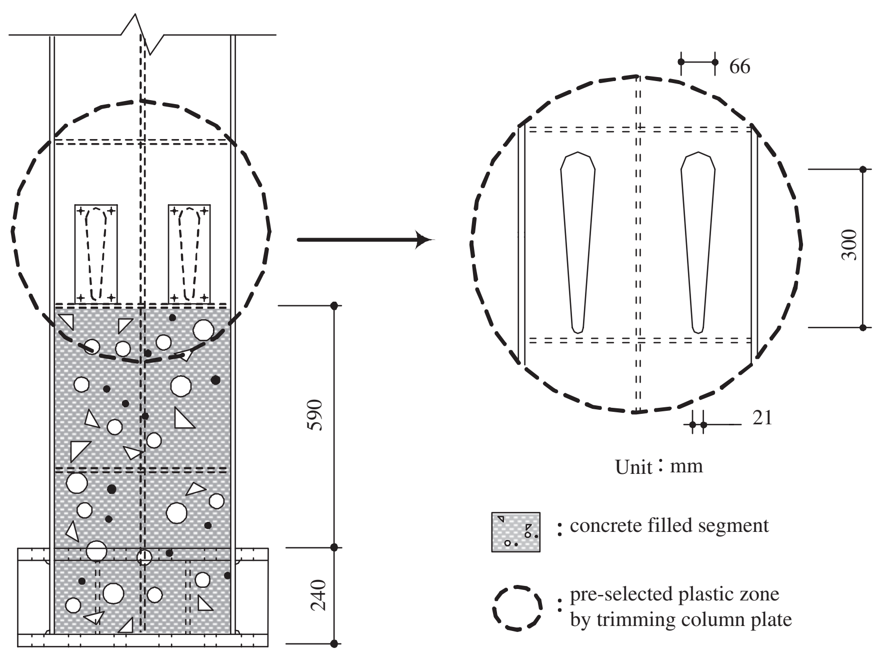

Fig. 7. Details of Specimen No. 3.

segment. The energy dissipation of the bridge column is greatly improved due to uniform stress distribution at the pre-selected plastic zone.

Fig. 7 shows the design of Specimen No. 3. Different from that of Specimen No. 2, Specimen No. 3 is designed to have strength reduction through the cut-off of the column plates. This method is more effective in reducing the seismic flexural strength of the column at the pre-selected zone. This is due to the fact that the column plates are at the extreme fibers from the neutral axis of the section and its thickness is usually larger than that of the stiffeners. However, unlike trimming the stiffener, cutting the column plate may decrease the axial strength of the column. It is necessary for the bridge column to provide enough strength for the gravity load, that is, the dead load and the traffic load. It is assumed that the strength ratio of the column (required strength to provide strength) under gravity load is less than 0.85 for bridges in seismic active area. In the design of Specimen No. 3, the axial strength is kept larger than 0.85 of the original strength of the bridge column. Besides, the holes on the column plate may lead to the moisture or even debris get into the steel column. It is suggested to provide cover plates as usually used for manholes (Fig. 7).

In this study, all the bridge columns are filled with concrete in the bottom of the columns. These concretes are usually used to increase the stiffness of the column in order to avoid damages when hit by vehicles or by floating debris if the bridge is in the river. The filled-in concrete will increase the strength of the column. Specimen No. 4 is selected to examine the effect of

the height of the filled-in concrete. The height of the filled-in concrete of Specimen No. 4 is 190 mm, which is much less than that of Specimen No. 1–No. 3, 590 mm. By this arrangement, the pre-selected zone is closer to the foundation and is much prone to the stress concentration at the bottom of the column. Besides the filled-in concrete, the basic design of Specimen No. 4 is similar to that of Specimen No. 2, as shown in Fig. 6. Design parameters of all specimens studied are listed in Table 1. The test set-ups are shown in Fig. 8.

Table 1
Summary of design of specimens

Specimen	Material property (MPa)		Filled-in concrete height (m)	
	Steel	Concrete		
No. 1	$\sigma_y = 360$ $\sigma_u = 500$	$fc' = 37.0$	0.59	Traditional steel bridge
No. 2				Tapered longitudinal stiffener
No. 3				Cut-off column plate
No. 4		$fc' = 40.7$	0.19	Tapered longitudinal stiffener

Note: Section size (mm): 460 × 460 × 10 (depth × width × thickness); longitudinal stiffener (mm): 75 × 6 (depth × thickness); diaphragm (mm): 6 (thickness); vertical load (kN): 1020.

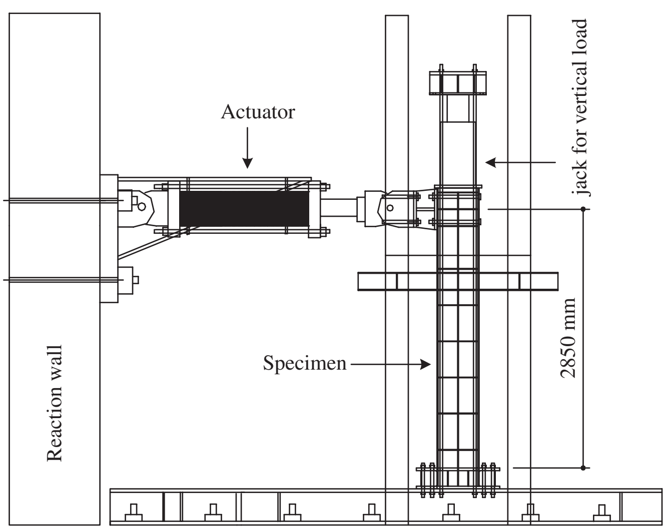

Fig. 8. Experimental set-ups.

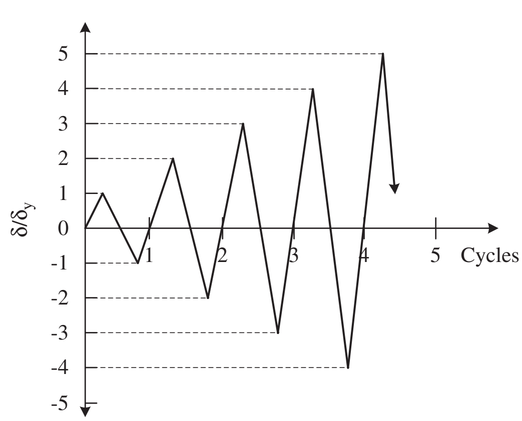

Fig. 9. Loading protocol.

All the specimens are subjected to cyclic load with increasing load/displacement amplitudes that simulated the recursions of earthquake load. The loading protocol is shown in Fig. 9. The loading tests are terminated when significant strength decay or fracture occurred. Fig. 10 shows the hysteresis behavior of all specimens studied. Table 2 shows the summary of the experimental results.

4. Discussion of experimental results

The experimental results are examined based on the stiffness, strength, stress distribution, effects of filled-in concrete, and energy dissipations. The following is the discussion of these parameters.

4.1. Stiffness

All the specimens studied are having the same material and the same dimensions. The major difference is the adoption of the pre-selected plastic zone for Specimen No. 2–No. 3. These specimens are having either the trimmed stiffener or the cut-off of a portion of the column plate at the pre-selected location. The reduction of the section will reduce the stiffness of the bridge column. The lateral stiffness from both experimental results and finite element analysis using ANSYS [17] program is shown in Fig. 11. Specimen No. 4 is not included in Fig. 11, since its height of filled-in concrete is different from the rest of specimens. The stiffness shown in Fig. 11 is normalized with the stiffness of Specimen No. 1, which is a conventional bridge column without pre-selected plastic zone. It is found that the reductions on the lateral stiffness due to the reduced section at the pre-selected segment are only 1.4–8.4%, which can be ignored in engineering practice.

4.2. Strain distribution on the bridge pier

Fig. 12 shows the typical strain distribution of Specimen No. 1 and No. 3 along, the length of the bridge column. Specimen No. 1 is a conventional bridge column, while Specimen No. 3 adopted a pre-selected plastic segment. It is found that larger and more uniform strain is obtained in the pre-selected area of Specimen No. 3 as compared with that of Specimen No. 1. It is seen that the proposed method with a pre-selected plastic zone is able to reach inelastic behavior prior to the other portion of the pier, and it is easier for inspection and repair, if necessary.

4.3. Strength

In order to provide a pre-selected plastic zone at a location away from the foundation, it is necessary to reduce the strength of the column at that location. The reduction on the section size does not reduce the seismic resistance capacity since the demand strength is also reduced. This is shown in Fig. 4b as the provided strength at the pre-selected area is equivalent to that of the required strength. Table 2 shows the experimental results of all the specimens tested. It is found that the strengths of the proposed connection method are about 1.23 of the nominal design strength or 1.17 of the calculated design strength based on the true material properties. The experimental results also show that the strengths of all the specimens studied are very consistent.

4.4. Energy dissipation and failure mode

The hysteresis behaviors of all the bridge columns studied are shown in Fig. 10. It is found that the proposed connection

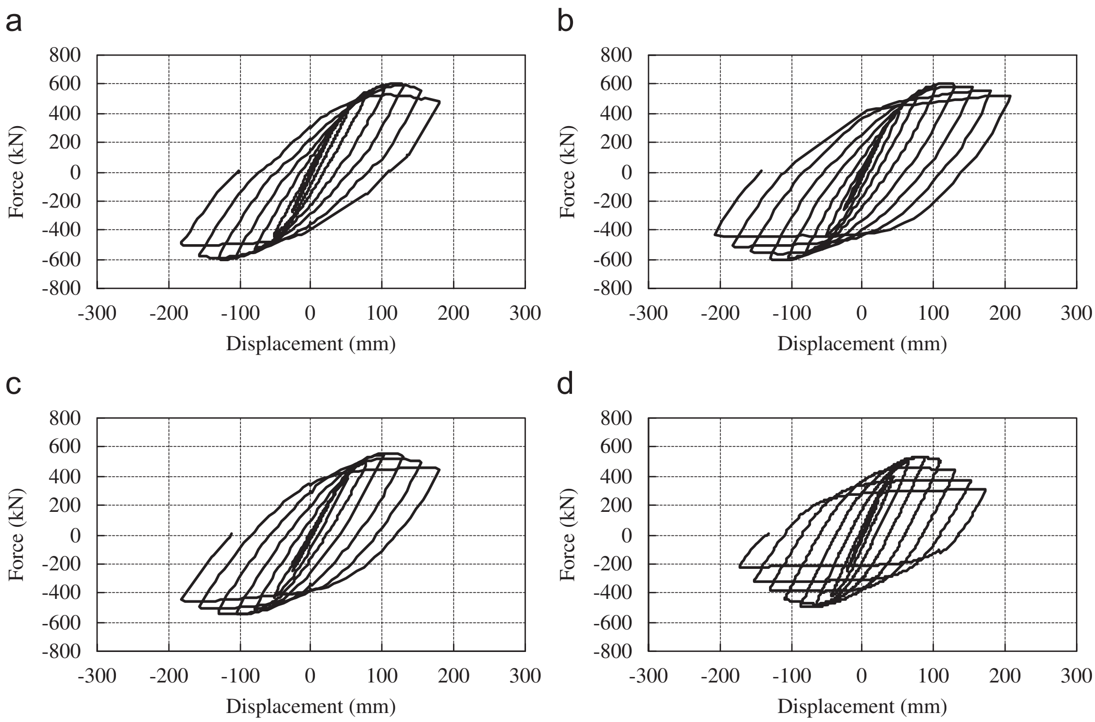

Fig. 10. Hysteresis behavior of specimens.

Table 2
Summary of experimental studies

Specimen	P_n (kN)	P_m (kN)	P_{exp} (kN)	P_{exp}/P_n	P_{exp}/P_m	Ductility (δ_u/δ_y)	Energy (kN m)	Failure mode
No. 1	486.5	510.1	603.0	1.24	1.18	7.0	493.4	Fracture at column base
No. 2	486.5	510.1	603.7	1.24	1.18	8.0	763.9	Plate buckling
No. 3	466.3	489.0	553.4	1.19	1.13	7.0	521.4	Plate buckling
No. 4	413.3	433.4	515.1	1.25	1.19	8.0	570.3	Plate buckling

Note: P_n: nominal strength; P_m: strength based on true material strength; P_{exp}: experimental strength. Average $P_{exp}/P_n = 1.23$; Average $P_{exp}/P_m = 1.17$, δ_u: maximum displacement where significant strength decay or fracture occurred. δ_y: yielding displacement.

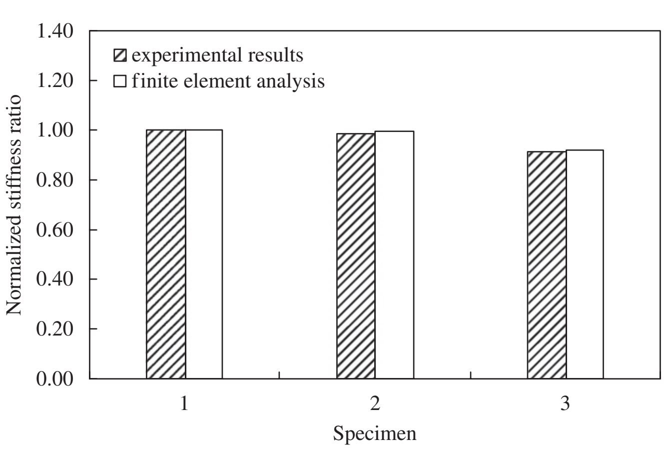

Fig. 11. Stiffness Specimen No. 1–No. 3.

method is able to provide a stable energy dissipation under cyclic load. The comparison of the energy dissipation is shown in Fig. 13. From Fig. 13, it seems that the proposed method has better energy dissipation capacity than that of the traditional design. Besides, Specimen No. 1 failed due to the fracture at the junction of the column and the anchor frame where the maximum seismic moment occurred. As explained previously, this location is usually under the ground level and is difficult to inspect and repair. Specimen No. 2–No. 4 failed due to the excessive plastic deformation at the pre-selected area, which is easier to inspect or repair if necessary. Fig. 14 shows the inelastic deformation around the pre-selected segment. The average energy dissipation capacity of Specimen No. 2–No. 4 is about 54.8%, 5.7%, 15.6% more than that of Specimen No. 1. It is noted that, although inelastic deformation of Specimen No. 3 is confined to the pre-selected segment, its energy dissipation capacity is only 5.7% larger than Specimen No. 1. This is because the reduced section strength of Specimen No. 3 was done by the cut-off part of the flange plate and leads to local buckling of the flange. The local buckling of the flange plate interferes with its energy dissipation capacity. It is suggested to reduce the column strength by trimming the stiffener as in Specimen No. 2 and No. 4. By this arrangement, the energy dissipation capacity of the bridge column can be greatly increased.

4.5. *Effect of filled-in concrete*

In this study, all the bridge columns are filled with concrete at the bottom of the column. These concretes certainly increase the

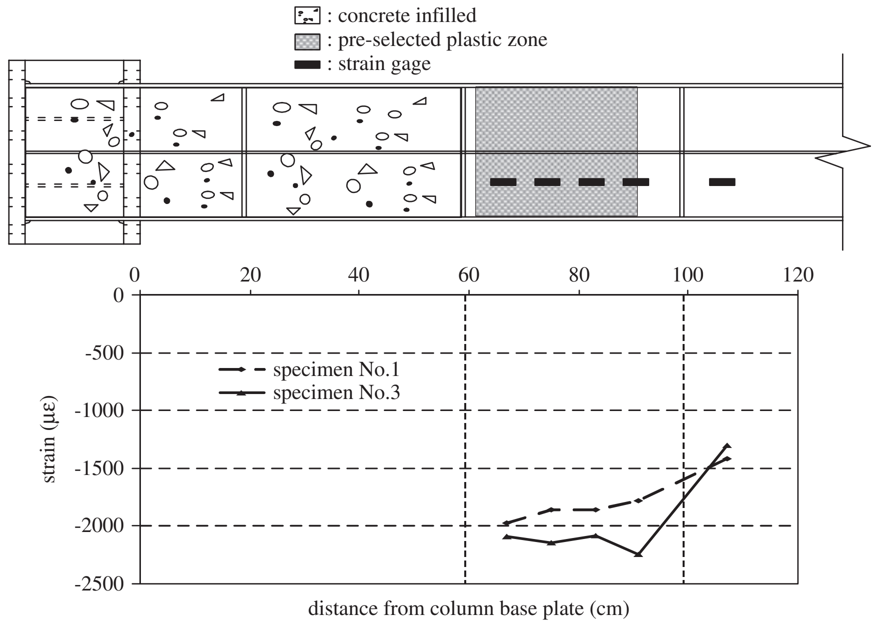

Fig. 12. Strain of Specimen No. 1 and No. 3.

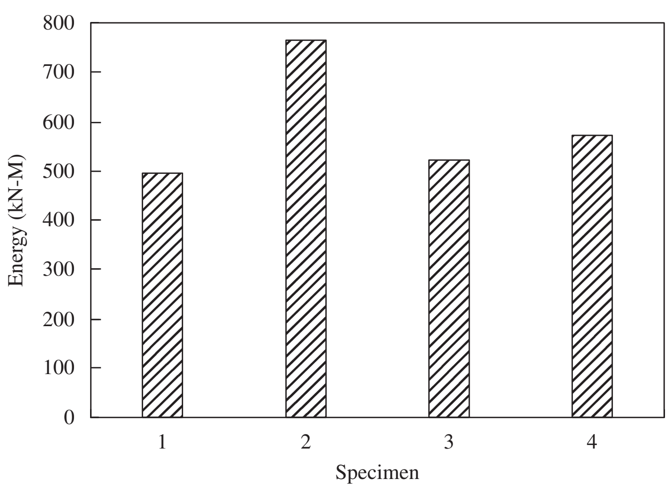

Fig. 13. Energy dissipation capacity.

Fig. 14. Local buckling and plasticization at the pre-selected segment.

strength of the column. Specimen No. 4 is selected to examine the effect of the height of the filled-in concrete. The height of filled-in concrete of Specimen No. 4 is 190 mm, which is much less than that of Specimen No. 1–No. 3, 590 mm. The pre-selected zone of Specimen No. 4 is closer to the foundation, which is much more prone to the stress concentration at the bottom. From the hysteresis behavior shown in Fig. 10d, the strength comparison, and the energy dissipation capacity shown in Table 2 and Fig. 13, it is seen that the height of filled-in concrete has only little effect on the seismic performance of the proposed connection methods.

5. Summary and conclusions

Due to the characteristics of seismic force on the bridge column, inelastic deformation is concentrated toward the bottom of the column, which is usually under the ground level. After the earthquake, it is difficult to inspect the damages at the bottom of the column. Providing a pre-selected ductile segment away from the bottom of the column can simplify the inspection or repair work after the earthquake. This ductile segment is designed following the seismic moment gradient, and uniform yielding can be obtained at the segment. From the experimental studies, it is found that reduced column section by trimming the stiffener or the column plate is able to ensure that the inelastic deformation occurred at the pre-selected region prior to any inelastic deformation occurrind at other parts of the column. It is also found that trimming the stiffener results in better energy dissipation capacity as compared to that of cutting the column plates. The experimental studies also demonstrated that by trimming the stiffener at the pre-selected segment it is able to provide a better energy dissipation capacity up to 54.8% larger than that of the traditional design. Although the column section is reduced at the pre-selected area, the stiffness

and strength are about the same as those of the original bridge pier.

Acknowledgments

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