ENHANCEMENT OF FLEXURAL DUCTILITY OF REINFORCED CONCRETE BRIDGE COLUMNS

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Abstract: This paper introduces some new technologies aiming of enhancing the seismic performance of reinforced concrete bridge columns. Introduced are the interlocking spiral columns with a large cross section, effect of unbonding of longitudinal bars at the plastic hinge, prestressed concrete columns, and an isolator built-in column. Although they are still in the research and preliminary implementation stage, it is expected to extend the new technology into practice.

1. INTRODUCTION

The extensive damage to bridges during the recent earthquakes in Northridge, USA (1994), Kobe, Japan (1995), Chi-Chi, Taiwan (1999), and Kocaeli and Duzce, Turkey (1999), revealed the vulnerability of bridges under extreme ground motions. A number of bridges suffered extensive damage as a result of the insufficient shear strength and ductility capacity of reinforced concrete columns. Based on the lessons from the recent earthquakes, the seismic design methodology was extensively revised worldwide.

Since the enhancement of reinforced concrete piers/columns was one of the most important aspects in the revision of the seismic design codes, various unique technical developments for reinforced concrete columns with verification through loading tests have been conducted. Several new technologies are presented in this paper. Although most of them are still in the research stage, it is important to extend the current technology.

2. INTERLOCKING SPIRAL COLUMNS WITH LARGE CROSS SECTIONS

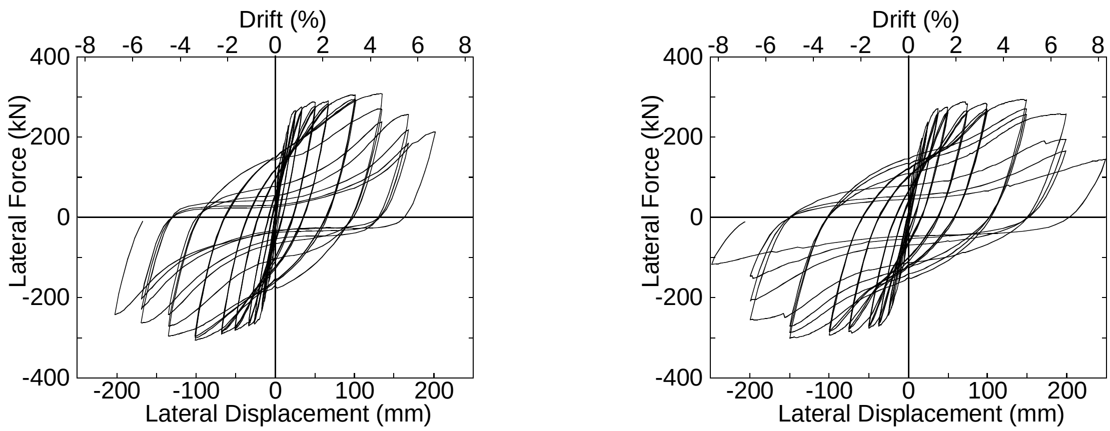
Interlocking columns have been extensively implemented in New Zealand, USA and other countries (Tanaka and Park 1993, Priestley et al 1996). The interlocking spirals confine the core concrete to enhance the ductility of reinforced concrete columns. Prior to the 1995 Kobe earthquake, spirals were not used in Japan because rectangular columns were generally preferred and because column diameters are generally larger. Since the 1995 Kobe earthquake, the interlocking spiral columns have been recommended in the design codes (Japan Road Association 1996 and 2002), and various studies have been conducted (Yagishita et al 1997, Fujikura et al 2000).

Interlocking spiral columns with large sections were constructed at Kamanashi bridge. Each column consists of 2 spirals with a diameter of 6 m and is 8.5 m wide and 6 m long in the transverse and the longitudinal directions, respectively. Since these columns were much larger in size than the interlocking columns which had ever been constructed elsewhere, a unique experimental test was conducted by the Japan Highway Public Corporation (JH) in conjunction with the construction of the bridge. Since assemblage of the interlocking spirals requires a special skill, an onsite assemblage test of large diameter interlocking spirals was conducted (Shito et al 2002).

In the cyclic loading test, several model columns with interlocking spirals were loaded independently in the transverse and the longitudinal directions. The model columns were 2.7 m tall (effective column height) and 900 mm wide and 600 mm long in the transverse and longitudinal directions, respectively. They consisted of two spirals with a diameter of 600 mm. They were about 1/10 geometrically scaled models. The concrete strength was 28.1-39.7 MPa. The volumetric tie reinforcement ratio was varied 0.19%, 0.29% and 0.52% with the longitudinal reinforcement ratio being 1.63%. A 900 mm wide and 600 mm long standard rectangular column was also constructed for comparison. In addition to ties, cross ties were laterally spaced at every 158-196 mm interval in the standard rectangular column. The concrete strength of the rectangular column was 39.8 MPa. The longitudinal reinforcement ratio was 1.18% and the volumetric tie reinforcement ratio including the cross ties was 0.88%.

Fig. 1 compares the lateral force vs. lateral displacement hystereses of the interlocking spiral column (volumetric tie reinforcement ratio is equal to 0.29%) and the standard rectangular column under a cyclic loading in the longitudinal direction. The lateral restoring force is stable until 4.5% drift in the rectangular column, while it is stable until 5% drift in the interlocking spiral column. A similar test was conducted to verify that the interlocking columns exhibit stable hysteresis under a cyclic loading in the transverse direction.

Since the diameter of the interlocking spirals is large at Kamanashi bridge, an onsite assemblage test of the interlocking spirals was conducted, using a 4.5 m wide and 3 m long column consisting of two interlocking spirals with a diameter of 3 m. Two spirals were interlocked after being hung separately using a balanced lever, and they were set in position from the top of longitudinal bars. The spirals were temporally fixed to hanging cables so that they were set with an expected vertical interval. This construction procedure was successfully implemented on the interlocking column.



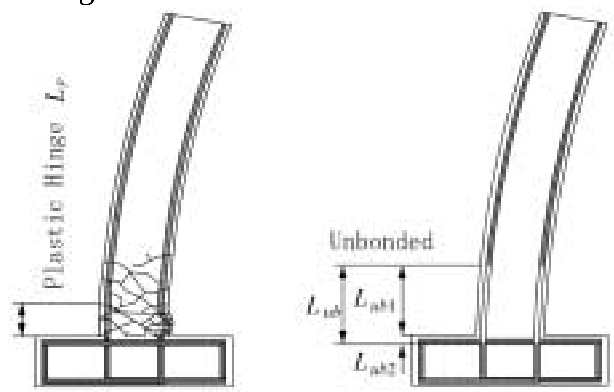
(a) Standard Columns (b) Interlocking Spiral Columns with Volumetric Ratio of 0.29%

Fig. 1 Effect of Interlocking Spirals

3. UNBONDING OF LONGITUDINAL BARS AT THE PLASTIC HINGE

In a reinforced concrete column, damage of the longitudinal bars progress from local bucklings to rupture in the plastic hinge under an extreme earthquake excitation. The bond between the longitudinal bars and the concrete results in the concentration of damage to the longitudinal bars at a specific localized interval where the local buckling occurrs.

One of the measures of mitigating such a concentration of damage to the longitudinal bars is to unbond the longitudinal bars from the concrete at the plastic hinge (Takiguchi et al 1976). By appropriately unbonding the longitudinal bars at the plastic hinge region with a length L_{ub} as shown in Fig. 2, the deformation of the longitudinal bars may be reduced by avoiding the concentration of strain as a result of averaging the strain in the interval L_{ub}. The unbonding may be achieved by wrapping the longitudinal bars with plastic materials. Protection is required for corrosion of the unbonded longitudinal bars.



(a) Standard Column (b) Unbonded Column

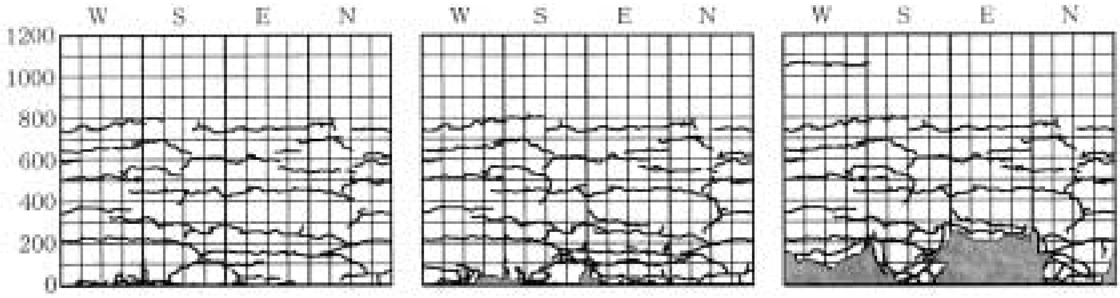
Fig. 2 Unbonding of Longitudinal Bars

Fig. 3 shows the effect of unbonding the longitudinal bars in a 1.45 m tall square column with a width D equal to 400 mm (Kawashima et al 2001). Although several tests were conducted, only two cases are presented here. The concrete strength was 24 MPa. The longitudinal reinforcement ratio was 0.95%, and the volumetric tie reinforcement ratio was 0.77%. The longitudinal bars were unbonded within a distance equivalent to the column width D. In the standard column, the covering concrete started to significantly spall off at $8\delta_y$, in which δ_y is the yield displacement of the standard column. Since δ_y is equal to 6 mm, 1% drift corresponds to $2.3\delta_y$. The column was cyclically loaded 3 times at each loading displacement δ_y, $2\delta_y$, $3\delta_y$, …., until failure. The same loading hysteresis was used for both the standard and the unbonded columns.

The concrete failed within about 200 mm from the bottom after $11\delta_y$ (=4.8% drift) in the standard column. On the other hand, the failure of concrete was much less in the unbonded column than the standard column. The covering concrete failed no higher than120 mm from the bottom even after $13\delta_y$ (=5.7% drift). The strain measured at 25 mm from the bottom built up over the yield strain at the first load excursion of $2\delta_y$. On the other hand, the strain on a longitudinal bar that was unbonded within a length of D was much smaller than the strain on a longitudinal bar in the standard column. The strains were similar, although not the same, at 25 mm and 175 mm from the bottom in the longitudinal bar in the unbonded column. The strains on the longitudinal bars became larger than 6,000μ at 25 mm and 175 mm from the bottom at the first excursion of $2\delta_y$ and $3\delta_y$ loadings, respectively.

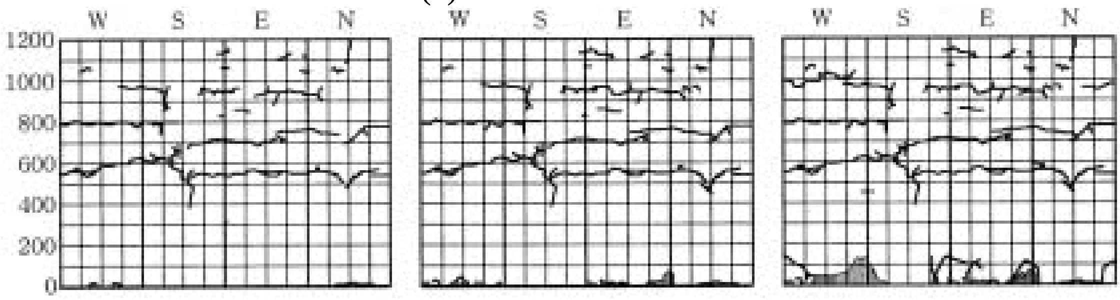
An important feature of the unbonded column is a rocking response of the column relative to the footing. Since the longitudinal bars were unbonded within a length of L_{ub}, the longitudinal bars in tension pulled out from the column, which resulted in a dominant rocking response of the column. As a result of the small flexural deformation, the flexural failure of the column was limited.

Fig. 4 compares the lateral force vs. lateral displacement hystereses. The restoring force of the standard column started to deteriorate at $9\delta_y$ (=3.9% drift), while the restoring force was stable until $11\delta_y$ (=4.8% drift) in the unbonded column.



(a) 5 δ_y (=2.2% drift) (b) 8 δ_y (=3.5% drift) (c) 11 δ_y (=4.8% drift)

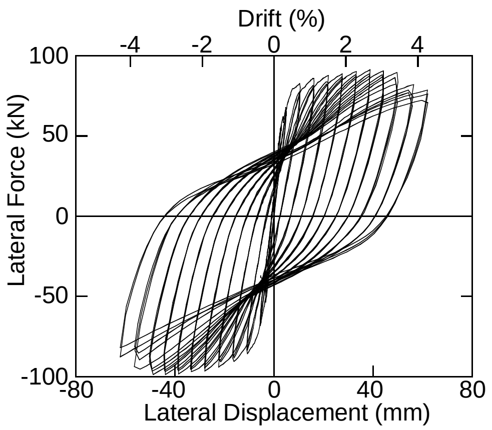
(1) Standard Column



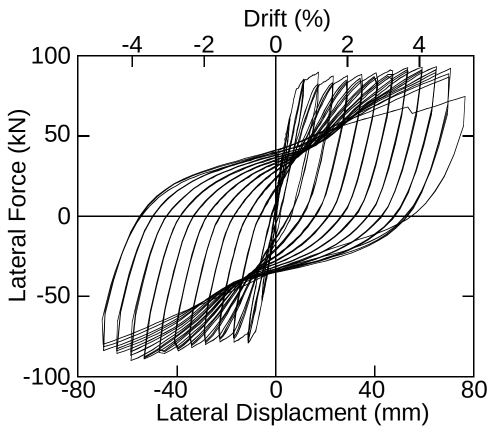
(a) 5 δ_y (=2.2% drift) (b) 8 δ_y (=3.5% drift) (c) 11 δ_y (=4.8% drift)

(2) Unbonded Column (Loaded in EW Direction)

Fig. 3 Progress of Damage



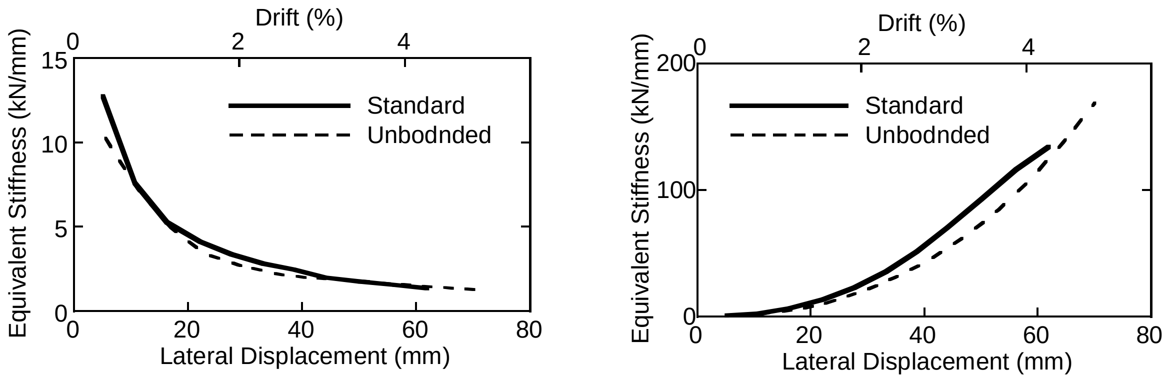
(1) Standard Column



(2) Unbonded Column

Fig. 4 Lateral Force vs. Lateral Displacement Hystereses

As a result of the deformation of the unbonded longitudinal bars in the plastic hinge, the initial lateral stiffness is slightly smaller in the unbonded column than the standard column. Fig. 5 compares the equivalent lateral stiffness and the accumulated energy dissipation between the unbonded and standard columns. The equivalent lateral stiffness is defined here as the secant stiffness between the maximum and minimum displacements in a hysteresis loop at each loading displacement. Although the equivalent lateral stiffness is slightly smaller in the unbonded column than the standard column when the lateral displacement is smaller than 1% drift, the difference between the two columns becomes small as the lateral displacement increases. This is due to the larger deterioration of the standard column. The difference of the accumulated energy dissipation between the two columns is negligible.



(1) Equivalent Lateral Stiffness (b) Accumulated Energy Dissipation

Fig. 5 Equivalent Lateral Stiffness and Energy Dissipation

Although the longitudinal bars were unbonded in the plastic hinge of the column in the above example, it is feasible to unbond the longitudinal bars in the footing or partly above and below the footing. Similar results were obtained by unbonding the longitudinal bars inside a footing (Hoshikuma et al 2000).

Based on the studies, it is considered that the unbonding is an effective means to increase the ductility capacity of columns by properly choosing the unbond length L_{ub}.

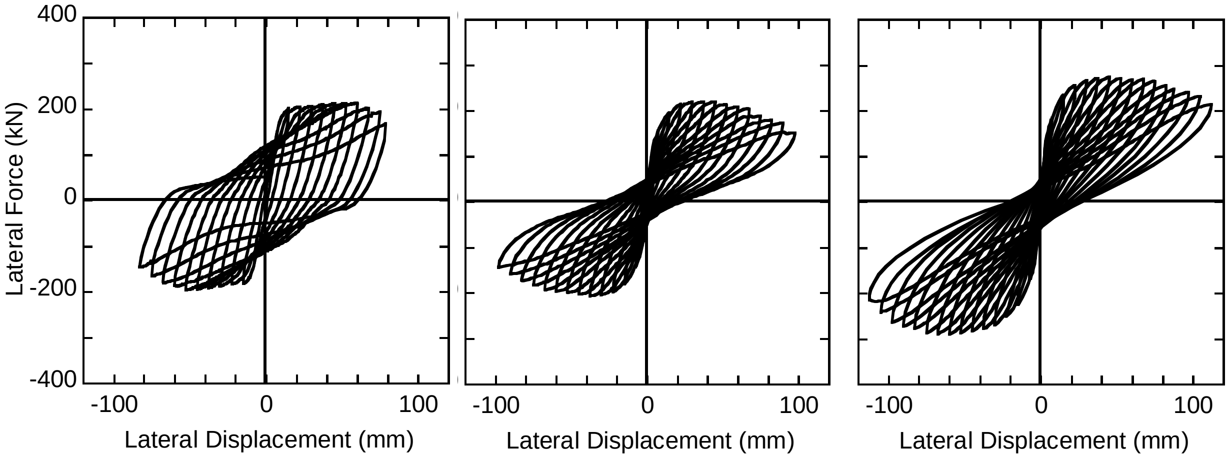
4. PRESTRESSED CONCRETE COLUMNS

It is well known that prestressed concrete members exhibit stable seismic performance under a combined action of shear and flexure. Consequently, it is anticipated that the flexure and the shear capacities can be enhanced in the prestressed concrete columns in comparison to the standard reinforced concrete columns. It is also anticipated that residual displacements after an extreme earthquake may be smaller in prestressed concrete columns than reinforced concrete columns. It is expected to reduce construction periods by using precast concrete segments.

However, the prestressed concrete columns have been seldom constructed throughout the world in spite of their merits. Lack of practice and possible cost increase may be the reasons for limiting the implementation of prestressed concrete columns. It is also sometimes pointed out that the energy dissipation is less in prestressed concrete columns than reinforced concrete columns because fewer concrete cracks dissipate less energy.

To verify the seismic performance of prestressed concrete columns, an extensive experimental and analytical study was conducted (Ikeda 1998, Ikeda et al 1998, Mutsuyoshi et al 2001). In the loading test, rectangular prestressed concrete columns with an effective height of 1.5 m and a section of 400 mm by 400 mm were constructed. The concrete strength, the prestress and bond/unbond of the PC cables were studied as parameters.

Fig. 6 shows the effectiveness of the prestressed concrete columns in terms of the lateral force vs. lateral displacement hysteresis. The columns were subjected to an axial load (dead load of the superstructure) equivalent to 1MPa, and the prestress was either 4 or 8 MPa. They failed in flexure. The hysteresis of a standard reinforced concrete column is also presented here for comparison. A remarkable feature of the prestressed concrete columns is the rest-position oriented unloading hystereses. If one defines the unloaded residual displacement as a residual lateral displacement of a column when the lateral force is equal to zero after unloaded from a maximum lateral displacement, then the unloaded residual displacement is significantly smaller in the prestressed concrete columns than the standard reinforced concrete column. Fig. 7 shows how the unloaded residual displacement decreases as the prestress increases in the prestressed concrete columns. It is obvious from a nonlinear dynamic response analysis that the limited unloaded residual displacement contributes to reduce the residual displacement of a bridge after an extreme earthquake. This contributes to satisfy the requirement of residual drift after an earthquake (Kawashima et al 1998, Japan Road Association 1996 and 2002).



(a) Standard Column (b) Prestress=4Mpa (c) Prestress=8Mpa

Fig. 6 Effect of Prestressing on the Hysteretic Behavior [Ikeda 1998, Ikeda et al 1998]

Number and size of concrete cracks were smaller in the prestressed columns than the standard reinforced concrete column during the loading and unloading reversals. The restoring force remarkably decreases when longitudinal bars locally buckle in the standard reinforced concrete, while such a remarkable deterioration of restoring force does not occur in the prestressed columns. Fig. 8 shows that the accumulated energy dissipations normalized by the peak restoring forces is smaller in the prestressed columns than the standard reinforced concrete columns as anticipated inherent to the rest-position oriented hysteretic behavior. This effect has to be considered in design based on the total response of a bridge system.

From the study, various merits of certain prestressed concrete columns were found. Those merits support the implementation of prestressed concrete columns.

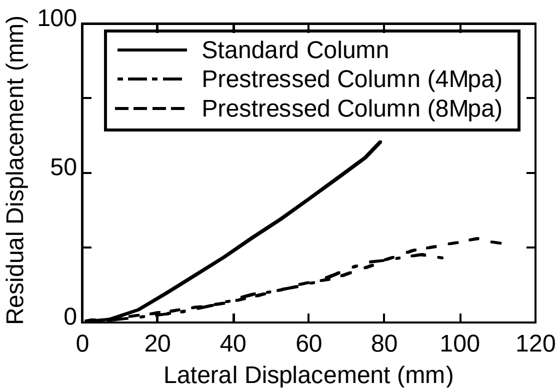


Fig. 7 Effect of Prestressing on the Residual Displacement of Unloaded Columns [Ikeda 1998, Ikeda et al 1998]

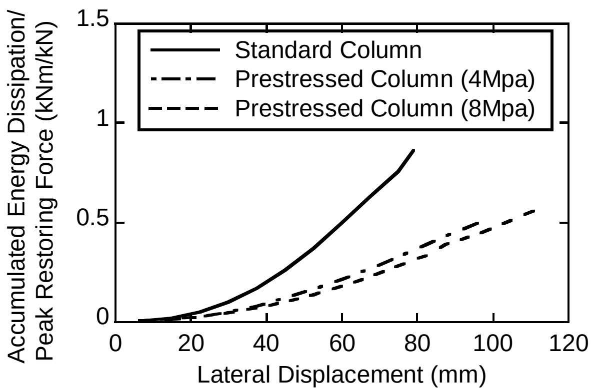


Fig. 8 Effect of Prestressing on the Accumulated Energy Dissipation/Peak Restoring Force [Ikeda 1998, Ikeda et al 1998]

5. ISOLATOR BUILT-IN COLUMNS

Since the hysteretic behavior of a reinforced concrete column occurs only at the plastic hinge, it is interesting to replace the concrete in the plastic hinge by an appropriate material that provides enough deformation and energy dissipation so that the flexural deformation in the rest of a column is limited. The material has to be sufficiently softer than the reinforced concrete column in order to reduce the flexural deformation of the column. By appropriately choosing the stiffness and strength of the material, it is expected that the reinforced concrete column at the plastic hinge becomes free from damage under an extreme earthquake excitation. Several efforts have been already initiated for such a purpose. The major technical importance is what material should be used for the replacement of reinforced concrete at the plastic hinge. It must be sufficiently stable under repeated seismic loading with large strains, and durable for long term use. It is preferable if energy dissipation is available associated with the deformation of the material.

A material studied is a high damping rubber that is used for standard high damping rubber bearings for seismic isolation. The high damping rubber meets several requirements described above. The high damping rubber may be provided in the form of a rubber block or a laminated rubber. If one sets a high damping rubber unit at the bottom of a cantilever column, the column deforms as shown in Fig. 9 under a lateral seismic force. The longitudinal bars are continuous through the rubber unit. Prestressed tendons may be effective to prevent a sudden deterioration of the restoring force and decrease the residual displacement.

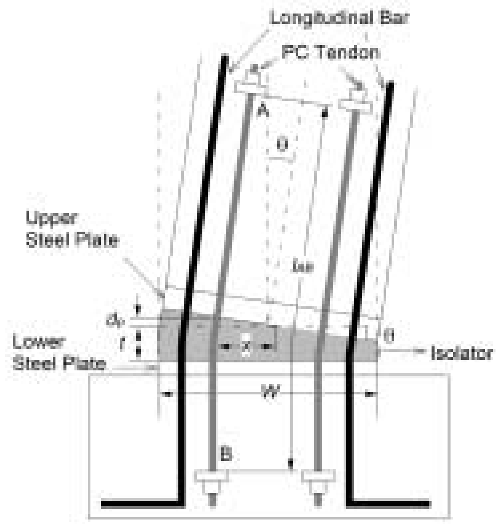
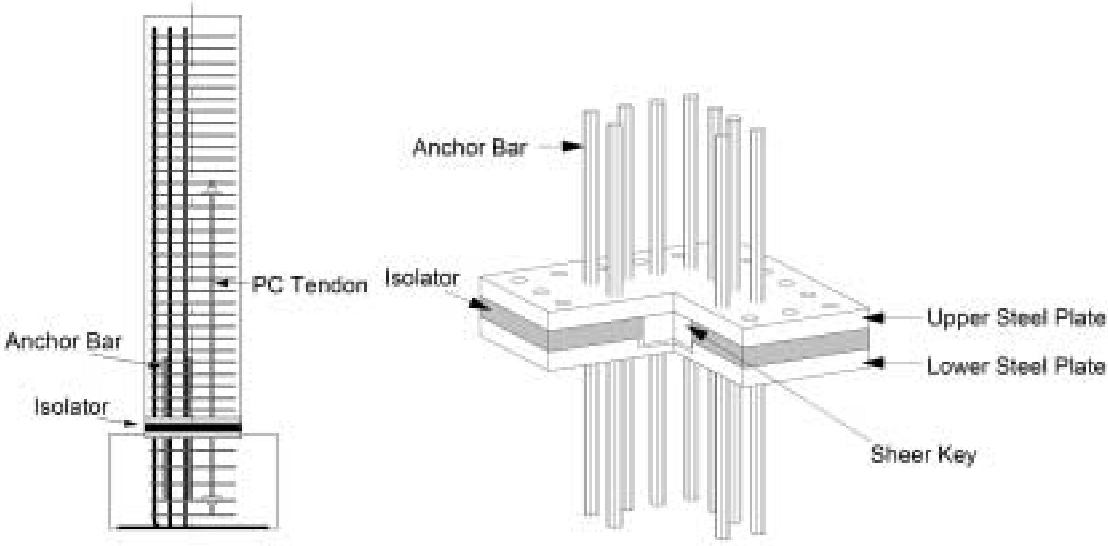


Fig. 9 Isolator Built-in Column

The rubber unit does not resist tension if it is not anchored to the column and the footing. Since contact of the rubber unit with the column and the footing is limited if the rubber unit is not anchored to the column and the footing, slippage and rotation of the column relative to the footing occurs once the longitudinal bars yield under a cyclic lateral loading. Hence, the upper and lower steel plates which are galvanized to the rubber unit are anchored to the column and the footing by anchor bolts. The longitudinal bars need to be continuous through holes in the steel plates and the rubber unit.

Laminated rubber units may be used if the rubber unit is thick. The steel plates in the laminated rubber unit may prevent the local buckling of the longitudinal bars when they are subjected to alternative tension and compression. Shear-keys may be required to prevent an excessive lateral displacement of the column relative to the footing when the rubber unit is thick. Since such a column is nearly equivalent to a built-in high damping rubber isolator, it is called here an *isolator built-in column* (Kawashima and Nagai 2002, Yamagishi and Kawashima 2004).



(a) Reinforcements (b) Rubber Unit with Anchor Bars
Fig. 10 Model Columns

A difficult barrier of the isolator built-in column is the deformation of the longitudinal bars. As a consequence that the column is supported by a flexible rubber unit, the longitudinal bars in the rubber unit are subjected to compression due to the self-weight of the structure. The longitudinal bars in the rubber unit are also subjected to repeated tension and compression with larger strain amplitude than a standard reinforced concrete column under an extreme earthquake excitation. Hence, it is likely that the longitudinal bars locally buckle and rupture in the rubber unit. Consequently a special attention has to be paid to prevent the premature failure of the longitudinal bars in the rubber unit. Use of special steels with the enhanced ductility may be effective.

If the stiffness of the rubber unit is sufficiently smaller than the stiffness of the column, major deformation under a lateral seismic force occurs in the rubber unit with the deformation of the column being limited. This results in the rocking response of the column similar to the unbonded column. Representing a rotation of the column as θ, the lateral displacement of the column at the top is $H \cdot \theta$ under a lateral force, in which H represents the column height. Since the drift $d_r \approx H \cdot \theta / H = \theta$, if one expects to have stable response of the column until a drift of d_r, the strain at the compression fiber of the rubber unit ε_r is

$$\varepsilon_r = \frac{\alpha W}{t}\theta \tag{1}$$

where W is the column width, t is the thickness of the rubber unit, and α is defined as $\alpha = x / W$ in which x is the distance from the neutral axis to the compression fiber. Since the rubber unit shows the extensive strain hardening under high compression, its effect has to be included in the evaluation of stress $\sigma_r = f(\varepsilon_r)$ corresponding to the strain by Eq. (1). Deformation characteristics of rubber units under high compression as high as 120 MPa was studied to determine $f(\varepsilon_r)$.

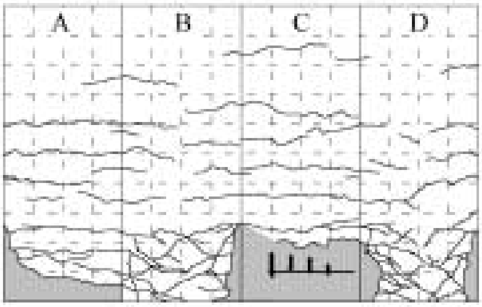
Consequently, the following relation has to be satisfied to avoid failure of concrete of the column

$$\sigma_r < \sigma_{cc} \tag{2}$$

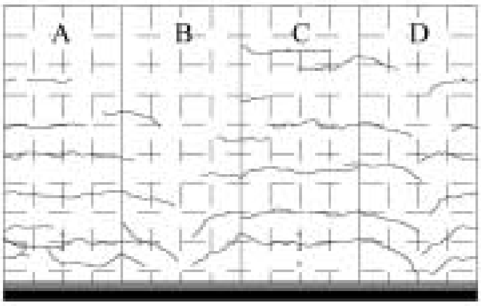
where σ_{cc} represents the concrete strength.

On the other hand, from Eq. (1), the rubber unit must be thicker than the following value so that it is stable under the repeated compression corresponding to the lateral drift d_r.

$$t_{\min} > \frac{\alpha W}{\varepsilon_r} \cdot d_r \tag{3}$$

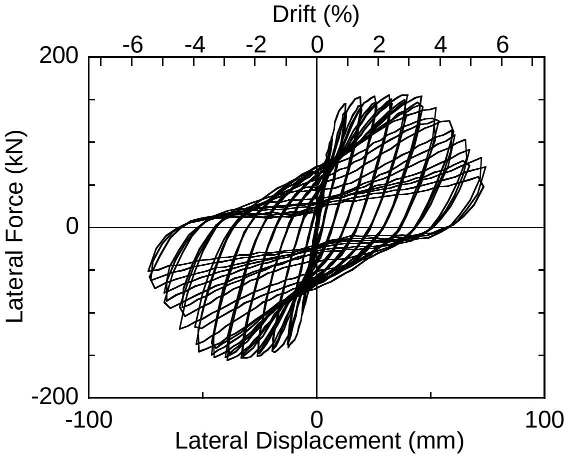


(a) Standard Column

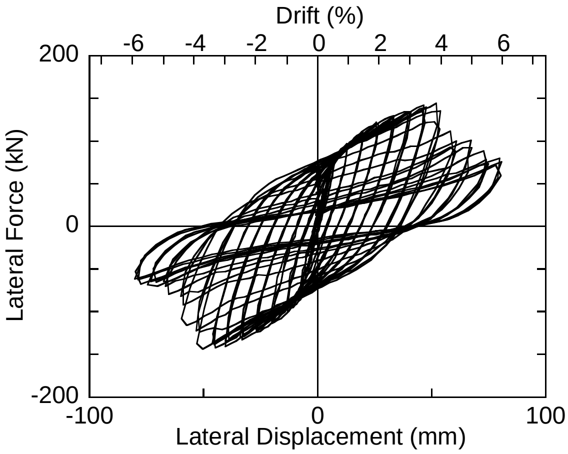


(b) Isolator Built-in Column

Fig. 11 Failure of Columns after 4% Drift (Loaded in AC Direction)



(a) Standard Column



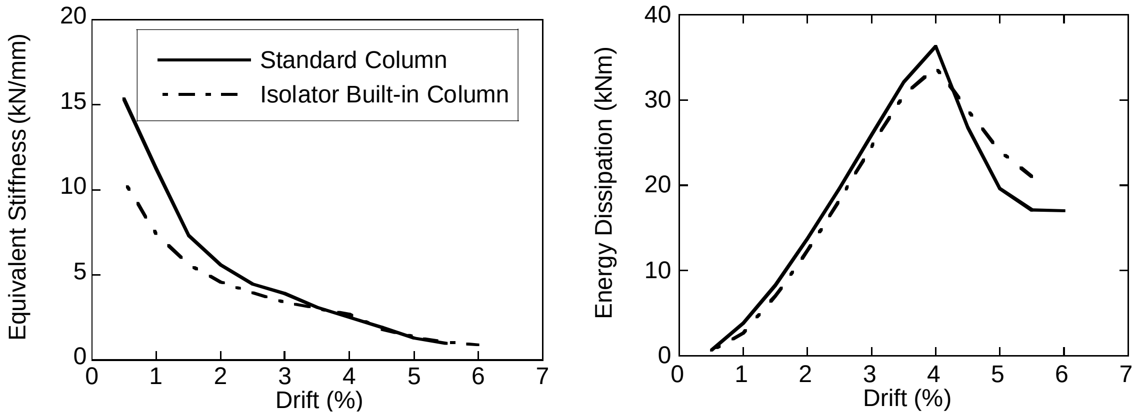
(b) Isolator Built-in Column

Fig. 12 Lateral Force vs. Lateral Displacement Hystereses

A series of seismic loading tests was conducted to verify the performance of the isolator built-in columns. Constructed were 1350mm tall (effective height) model columns with a 400mm by 400mm rectangular section as shown in Fig. 10. They were designed so that the hystereses are stable until 4% drift. As a consequence, 30 mm and 60 mm thick damping rubber units were used with an initial shear modulus of 1.2 MPa. Those rubber units are often used for seismic isolators in bridges. The longitudinal reinforcement ratio was 1.58%, and the volumetric tie reinforcement ratio was 0.79%. A shear-key was provided at the center, and four prestressed tendons were provided at the four corners.

Fig. 11 compares the failure of the isolator built-in column and the standard column after 4% drift loadings. Extensive failure of the concrete occurs until 4% drift at the compression fiber in the standard column. The longitudinal bars start to rupture at 5.5% drift, which results in the significant deterioration of restoring force. On the other hand, the failure of concrete is much limited in the isolator built-in column until 4% drift. However the longitudinal bars start to rupture in the rubber unit at 4.5% drift. The use of ductile steel is required to mitigate the rupture of the longitudinal bars as a result of concentration of strain at the bars in the rubber unit.

Fig. 12 compares the lateral force vs. lateral displacement relations of the two columns. A remarkable change of the shape of the hysteresis loops is seen. The lateral force is almost the constant in the post-yield zone in the standard column, while it increases as the lateral displacement increases in the isolator built-in column. The extensive deterioration of the restoring force at 4.5% drift results from the rupture of longitudinal bars in the isolator built-in column. An important difference of the isolator built-in column is the smaller initial stiffness, as shown in Fig. 13 (a), due to the soft deformation of the rubber unit. However, since the stiffness of the standard column deteriorates due to failure of the concrete, the lateral stiffness of the standard column becomes close to that of the isolator built-in column over 2.5% drift. The energy dissipation per load reversal is nearly the same between the isolator built-in column and the standard column as shown in Fig. 13 (b).



(a) Equivalent Stiffness (b) Energy Dissipation

Fig. 13 Effect of Isolator on the Equivalent Stiffness and Energy Dissipation

6. CONCLUDING REMARKS

The preceding pages introduced some new attempts to enhance the ductility capacity of reinforced concrete columns. It is noted that they are only a part of the efforts, and that many other practical and effective measures are being developed. Since 1995, the seismic design practice has extensively changed based on the technical development conducted in the last two devades. It was the major lesson of the recent earthquakes that it is important to have an insight to hold account in great un-experienced damage of bridges in the past.

Acknowledgements:

The author expresses his sincere appreciation to Professor Katsuki Takiguchi for introducing the concept of unbonding longitudinal bars. The author also expresses his sincere appreciation to Dr. Shoji Ikeda, Professor Emeritus, Yokohama National University, Mr. Yoshinori Igase, Japan Highway Public Corporation, and Mr. Takuya Mori, PS Concrete for providing him with the variable information and materials.

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