

**DESIGN ISSUES AND APPLICATION OF HIGH STRENGTH CONCRETE FOR HIGH RISE BUILDINGS**

Hideki Kimura, Yuji Ishikawa, Hiroto Takatsu and Hassane Ousalem

**ABSTRACT :**

More than 500 high-rise RC buildings with height exceeding 60m (197feet) have been built since early 70's in Japan. The number of stories sometimes exceeds 50. Use of base isolation systems or vibration control devices in high-rise RC buildings has significantly increased since 1995 Kobe Earthquake. Ultra-high-strength materials have also been used in such buildings. The specified concrete strength of 150 MPa (21800psi) or higher is currently practiced and SD685 deformed bars of 685 MPa nominal yield strength are used as the main reinforcing bars.

Such buildings are subjected to intensive large axial and lateral loads in case of sever earthquakes and strong winds, particularly at their lower stories where exterior columns experience varying high-axial loads shifting from compression to tension. Furthermore, as concrete strength increases, fire resistance decreases and cracking behavior of RC members changes which affects the structural performance. A lot of experimental studies with regards to such columns and subassemblies have been carried out to investigate their structural performance and to establish appropriate design methods.

This paper presents some design issues related to the application of high-strength materials to RC columns or subassemblies. It also emphasizes recent research works and design methods for the application of ultra-high-strength concrete for high rise RC buildings.

**Keywords:** high-rise building; high-strength concrete; high-strength steel bar; precast concrete ; reinforced concrete column; beam-column subassembly; seismic design

## Biography:

**Hideki Kimura** is a General Manager at Research and development Institute, Takenaka Corporation. He received his PhD from Tohoku University. He is a member of winner building team for the 2010 fib awards for outstanding concrete structures. His research interests include seismic behavior and earthquake-resistant design of high-strength reinforced concrete structure . He is also interested in the application of fiber reinforced concrete to RC structures.

**Yuji Ishikawa** is a Chief Researcher at the Department of Construction Technology, Research and development Institute, Takenaka Corporation. He received his PhD from The University of Tokyo. His research interests include seismic behavior of high-strength reinforced concrete structures and concrete filled steel tube structures using high-strength concrete.

**Hiroto Takatsu** is a Chief Researcher at the Department of Construction Technology, Research and development Institute, Takenaka Corporation. He received his ME from Kyoto University. His research interests include seismic behavior of high-strength reinforced concrete structures and prestressed concrete systems using high strength concrete.

**Hassane Ousalem** is an Assistant Chief Researcher at the Department of Construction Technology, Research and development Institute, Takenaka Corporation. He received his PhD from The University of Tokyo. His research interests include seismic behavior of structures, high-strength reinforced concrete, evaluation and retrofit of existing RC structures and masonry buildings.

## INTRODUCTION

Although Japan is located in a high density seismic area, more than 500 high-rise RC buildings with height exceeding 60m (197feet) have been designed and constructed since early 70's. The number of stories sometimes exceeds 50, making the best of technology developments. Base isolation and/or vibration control devices are also adopted. Their use in high-rise RC buildings has significantly increased since the 1995 Kobe Earthquake. High-strength or ultra high-strength materials have been used. The specified concrete strength of 150 MPa (21800psi) or higher is currently practiced and SD685 deformed bars of 685 MPa (99400psi) nominal yield strength are used as the main reinforcing bars (Kimura et al. 2010).

Using such high-strength materials in columns of high-rise buildings to sustain the axial load is effective and makes possible designing them with a relatively small cross section even for 100-story-class buildings. Therefore, to optimally combine high-strength reinforcement with high-strength concrete in columns, beams and other subassemblies of actual buildings, and insure for these structural elements high performances in terms of strength and ductility, adequate design methods should be established. Various tests and analyses have been carried out in Japan on different structural elements and configurations and helped to validate the seismic safety of the established design methods. This paper addresses some aspects related to the implementation of structural elements of high-strength materials in actual structures, as well as some issues that should be focused on in the future.

## STRUCTURAL DESIGN OF HIGH-RISE RC BUILDINGS IN JAPAN

In Japan, generally, buildings exceeding a height of 60m (197feet) are considered as high-rise buildings. To design properly such buildings and ensure a sufficient safety, the government established a committee named “tall buildings evaluation committee” that is composed of academics and professionals. This committee is in charge of examining and approving the design of such important buildings. The flowchart in Figure 1 shows the typical procedure followed in Japan to design tall buildings. First, design loads are determined and allowable stress design is carried out to determine the size of structural elements and the necessary reinforcement. Next, 2D and 3D nonlinear static and pushover analyses are carried out where the relationship between the story shear force and story drift is determined for all stories. Then, the ultimate stage of the building is identified. Wind and seismic response analyses, considering a multi-degree-of-freedom lumped-mass shear-type model, are then performed to confirm the appropriateness of the building response regarding the serviceability and design limit conditions. The procedure is concluded when all limit conditions are fulfilled.

When the performance of a high-rise building is investigated for seismic loads, three limit conditions are set up, namely:

1. The building may experience, at least once during its service period, a maximum probable minor earthquake of level 1 (maximum velocity equivalent to 25cm/s [9.75in/s]). Yield hinges should not develop within the main structural elements, while damage, cracks and deformations should be kept below a certain level to satisfy service limits.
2. For a maximum credible earthquake of level 2 (maximum velocity equivalent to 50cm/s [19.7in/s]), yield hinges should not develop within the main structural elements, while damage, cracks and deformations should be kept below a certain level to satisfy design limits.
3. For an ultimate earthquake exceeding level 2, collapse or brittle failure of structural elements should be prevented to satisfy ultimate limits. The axial compression and tension stresses in columns are to be less than 0.65 of concrete strength and 0.7-0.9 of main bar's yield strength, respectively.

Therefore, static and dynamic designs are achieved to meet each limit condition. For instance, the dynamic design criteria relative to seismic and wind loads in terms of maximum story drift angle should be less than 1/200 and 1/100, respectively, for level 1 and level 2 load types.

The aimed seismic performance, in terms of story drift and ductility, at different limit states that correspond to different velocity input levels is listed in Table 1.

## USE OF HIGH STRENGTH MATERIALS FOR COLUMNS

Structural seismic tests of columns in Japan are commonly performed using pantograph loading systems that avoid rotation movements of columns' ends when combining horizontal loadings to axial ones. The early stage when the development research program of high-rise RC buildings was launched, lateral loading tests on columns were generally carried out with constant axial loads. Then later, many of such tests were carried out with varying axial loads to simulate the actual axial loading of peripheral columns. Furthermore, because the adoption of high-strength materials shifted the weak part within the structural construction to the beam-column joints, tests of structural elements including subassemblies were carried out under varying axial loadings with tension forces. The coming section illustrates the practical use of columns with high-strength materials through an experimental test.

### **Flexural strength**

As to the flexural strength of high-strength concrete columns subjected to high-axial loads, Kimura et al. (1996) reported that for concrete strengths between 40 (5800) and 110MPa (16000psi) the evaluated flexural strengths of columns subjected to an axial load ratio of about 0.3 using AIJ equation (AIJ 2009) and ACI code (ACI 318-11) coincide with measured values, and when the axial load ratio is about 0.6 the evaluated strengths are underestimated by both codes, where the ratios of the test to the calculated values are in the range 1.31~1.74 and 1.08~1.55 for AIJ and ACI, respectively. In the contrary, for concrete strengths between 150 (21800) and 200MPa (29000psi), both evaluated strengths are overestimated by both codes, as reported by Kimura et al. (2007) and Komuro et al. (2004).

Figure 2 illustrates the crack conditions at a lateral drift angle of 1/200 of columns made of concrete strengths of 40 (6000), 60 (8700), 100 (14600) and 150MPa (21800psi) and subjected to a constant axial load of a ratio of about 0.6 and lateral loading. When concrete strength exceeds 100MPa (14500psi) remarkable spalling of concrete cover is observed during loading.

Figure 3 shows bending-shear test results of an RC column made with 150MPa (21800psi) class concrete carried out by Komuro et al. (2004). The column's response to a lateral reversed cyclic loading simulating earthquake loads is presented in terms of lateral force and lateral drift angle where a common property of such columns is shown. When cover concrete crushing, appearing as a first peak in the figure, occurs before the lateral drift angle 1/100, it is accompanied by a large drop in shear force, then followed by a second peak. Furthermore, they reported that the equations suggested in ACI code (ACI 318-11) based on the stress block of the whole cross section, do not evaluate appropriately the ultimate flexure strength obtained by tests, in the contrary the strain compatibility analysis of a layered column cross section evaluates appropriately the two previously mentioned peaks.

To prevent the phenomena of early spalling of cover concrete and drop in strength some solutions are suggested like, concrete containing steel fibers is used for elements (Kimura et al. 2007), or elements are covered by steel tubes (Yamauchi et al. 2000). Furthermore, relatively to the core concrete of precast elements, lower strength concrete is used for external shells.

Figure 4 shows the test results of two columns made of 200MPa (29000psi) class concrete in terms of lateral force and lateral drift angle (Kimura et al. 2007). The concrete of one column is without steel fibers while the concrete of the other one contains 1% volume of steel fibers. The lateral loading test is carried out under a varying axial load. The maximum axial load on the compression side has a ratio of 0.6, while on the tension side it reaches 80% of the yield strength level of main reinforcement. An increase of 50% in the flexural strength of the column is obtained due to steel fibers that maintain the concrete compression level associated with the effective depth of reinforcement because spalling of cover concrete is difficult to occur. By considering the cover concrete, it is expected to evaluate appropriately the displayed flexural strength. Furthermore, Kimura et al. (2007) reported that, by using steel fibers, the observed damage at the bottom of columns is reduced, as shown in Figure 4 and the width of flexural cracks becomes small due to their spreading.

### **Deformation capacity**

To ensure a good ductility performance of RC elements, energy dissipation is aimed through plasticization of reinforcement without brittle failure of concrete. While it is easy to decide the flexural yielding of beam elements by limiting the tensile reinforcement ratio to the balanced value, the flexural yielding of column elements varies with the axial load ratio, their energy dissipation capacity is not sufficiently proven to be always related to the plasticization of tensile reinforcement and, as another shortcoming, their deformation capacity reduces with increasing axial load ratio. From this point of view, it cannot be said that the evaluation method of the deformation capacity is well established for RC columns and beams made of normal strength concrete, although a lot of research has been carried out.

Similarly, to ensure a good ductility to column elements made of high-strength concrete for the development of high-rise RC buildings, various research works have been conducted in Japan. For columns intended to experience a prior flexural yielding, a research program was carried out to investigate various aspects, among them: 1) ensure toughness and compression bending strength, 2) prevent shear failure beyond flexural yielding, 3) prevent bond failure beyond flexural yielding, and 4) prevent early buckling of compressive reinforcing bars. Among the factors that were nominated influencing the ductility of RC column elements beyond flexural yielding are the transverse reinforcement ratio, transverse reinforcement strength, axial load ratio, concrete strength, main reinforcement strength and loading history (constant or varying axial load).

Ishikawa et al. (2008) investigated the ultimate deformation capacity of reinforced concrete columns made of high strength materials based on 115 test data with maxima of concrete strength and main bars yield strength of 160MPa (23200psi) and 980MPa (142100psi), respectively. The ultimate deformations of the database were approximately related to five factors, namely, the 1) axial load ratio, 2) amount of transverse reinforcement, 3) main bars' strength, 4) axial loading type (constant or varying) and 5) concrete strength that lead to an estimated equation. It is worth to mention here that the ultimate drift angle was obtained from the shear force-lateral drift angle envelope curve beyond the maximum strength when the shear force became 80% of the maximum.

Figure 5 shows a comparison of envelope curves of three columns made of 120MPa (17400psi) concrete strength, similar reinforcement and reinforcement strength, but with different transverse reinforcement ratios: 0.7, 0.9 and 1.2%. The columns were subjected to a constant axial load ratio of 0.6. The ultimate drift angle increases with increasing transverse reinforcement ratio. Figure 6 illustrates such relationship based on results of 6 tested columns.

Figure 7 shows a comparison of envelope curves of two columns made of 115MPa (16700psi) concrete strength, similar reinforcement and reinforcement strength, but subjected to different axial load ratios with constant values (0.4 and 0.6). The ultimate drift angle decreases with increasing axial load ratio. Figure 8 illustrates such relationship based on results of 4 tested columns.

Figure 9 shows a comparison of envelope curves of three columns made of similar reinforcement, reinforcement strength and reinforcement amounts (transverse reinforcement ratio of 0.9%), and subjected to the same constant axial load ratio of 0.6, but made of different concrete strengths (70 [10000], 120 [17400] and 150MPa [21800psi]). The ultimate drift angle decreases with increasing concrete strength. Figure 10 illustrates such relationship based on results of the 3 tested columns.

Figure 11 shows a comparison of envelope curves of two similar columns but with different strengths of main reinforcement. The columns were subjected to a constant axial load ratio of 0.6. The ultimate drift angle increases with increasing main reinforcement strength. Furthermore, use of higher strength of main reinforcement results in higher ductility and delays early buckling of compressive reinforcing bars, as shown by SD390 and SD685 bars (nominal yield strength of 390 and 980 MPa [56600 and 99300 psi] respectively).

As to the effect of the loading type, it is pointed out those columns with varying axial load from compression to tension show higher ductility than columns with constant axial loads, for the same maximum compression axial load, as illustrated in Figure 12.

Figure 13 compares test results with calculated values based on the suggested equation. A precision of 93% for a  $\pm 30\%$  dispersion is obtained where the average and COV values of the equation are 1.01 and 16.6%, respectively.

### **Effect of varying axial load**

During earthquakes, peripheral columns in high-rise buildings are subjected to a state of combined overturning moments and varying axial loads in which compression alternates with tension. For the ultimate state of buildings in Japan, the design considers the axial compression stress level in columns should be below 65% of the strength of

concrete, while the axial tension stresses developing in the main bars should be between 70 and 90% of their yield strength. In the case of expecting large tension stress, longitudinal reinforcement to resist tension must be arranged at the center of the column cross section.

As presented previously, for the same maximum axial load ratio on the compression side, columns show larger deformation capacity when subjected to a varying axial load from compression to tension than when subjected to a constant one. However, main reinforcement should be kept below yielding when axial load turn into tension side.

High-strength concrete makes cross sections of structural elements relatively small and high-strength steel makes stress level in the elements higher.

As a result, the beam-column joints will become weak parts in the structural system. Research studies related to the properties of such structural subassemblies under varying axial loading are limited, especially under axial tension loading. The work of Ousalem et al. (2009) showed that under axial tension load the shear strength of these subassemblies is lower than the one under compression load. Two half scale exterior beam column joints of high-rise buildings (Figure 14) were tested under lateral loading and extreme varying axial load conditions (Figure 15) alternating between high compression (compression stress reached 65% of concrete strength) and high tension (tensile stress in main bars reached 90% of yield stress), as shown in Figure 16. Both subassemblies were made of 70MPa (10200psi) concrete strength and the main reinforcing bars of beams were provided with mechanical anchors. Because the test parameter was the level of applied axial tension load (tensile stress in main bars: 90% of yield stress), the column's main reinforcing bars of one subassembly were of SD490 type (nominal yield strength of 490MPa [71100 psi]) while the ones of the other subassembly were of SD685 type (nominal yield strength of 685MPa [99300 psi]). The subassemblies were designed to yield their full flexural capacity and present a bending yield mode of beams when the axial load is on the compression side, while the shear strengths of beam column joints were designed to be 75% higher than the shear force level corresponding to beam yielding.

Figure 17 illustrates the shear force-lateral drift angle relationship of the subassembly with SD490 type (nominal yield strength of 490MPa [71000 psi]) bars in columns. Even under extreme varying axial loading, the subassembly showed a good hysteresis properties within the story drift angle of 30/1000 rad. . However, when compared to subassemblies under only axial compression load, concrete degradation is considerable for those under varying axial load without tension side and the possibility to experience shear failure and loss of axial bearing capacity for either large lateral deformation of  $R=30/1000$  or excessive lateral reversed cycles is pointed out.

### **Use of Steel fiber concrete**

For concrete strengths higher than 100MPa (14500psi), the authors recommend the use of steel fibers to prevent early spalling of cover concrete, control the degradation of flexural strength and control damage by smearing developed cracks, besides its contribution to the increase of shear strength. Adding steel fibers is also useful when reduction of reinforcement amount and prevention of steel bars' congestion is aimed. Furthermore, high fire resistance performance for 150-200MPa (21800-29000psi) strength class concrete is effectively ensured when organic fibers are combined with steel fibers. The findings of Takatsu et al. (2006) relative to the structural performance of columns made of 150MPa (21800psi) class steel fiber concrete are summarized below.

1. Introduction of steel fibers into concrete mix increased the lateral load carrying capacity. , The maximum strength of the columns increased proportionally to the volumetric ratio of steel fibers.
2. Adding steel fibers into concrete mix delayed early spalling of cover concrete. Moreover, surface crack width was reduced in the specimens with SF comparing to those without SF.
3. The strains of transverse reinforcement tend to decrease as the volumetric ratio of SF increase. It is possible that steel fibers contribute to the confinement of concrete in the column.

4. For the constant axial loading cases, adding of steel fibers improved the column ductility, but for varying axial loading cases, the ductility was not influenced by steel fibers.

As to beam-column joints, adding steel fibers affected considerably their performance.

Figure 18 shows a comparison of interior beam-column joints using 150 MPa (21800psi) concrete with and without steel fibers (Takatsu et al. 2009). The joint panels do not contain any hoops. The figure and photos at the top present, respectively, the response and the damage of a beam-column joint without steel fibers. The figure and photos at the bottom show, respectively, the response and the damage of a beam-column joint containing 1% volume of steel fibers. The cracks were traced only on the right side of the specimens during test to show the actual crack conditions without any effect of marking. Both specimens were designed to fail in shear at the beam column joint. The specimen with steel fibers had larger shear strength and less damage than the specimen without steel fibers.

### **Fire resistance**

As it is well known, high strength concrete experiences explosions during fire due to the enclosed amount of water. When the planned concrete strength is higher than 80 MPa (11600psi), organic fibers, like Poly-propylene fibers, are added to the concrete. During fire, the fire-resisting performance is significantly improved. The mechanism is based on the creation of voids due to melting of the organic fibers during fire, allowing the encased water to stream out without inducing any inside pressure as shown in Figure 19. The concrete of 150MPa (21800psi) strength or higher should be provided with organic fibers as well as steel fibers to be able to resist fire and improve the fundamental structural performance. Steel fibers prevent the splitting of concrete when subjected to high stress conditions and enhance the properties of the total concrete surface and consequently the effective concrete section as shown in Figure 20.

## **CONCLUDING REMARKS**

A lot of expertise for RC high-rise buildings using high strength materials has been accumulated through many laboratory tests and practical works in Japan. The developed research and planning regarding the seismic hazard, the evolution of structural materials, computation methods and tools have provided a confident construction system that is facing the increasing demand for tall buildings.

However, when adopting high-strength concrete in the construction of high-rise buildings, it is to be noticed that some issues still should be investigated further. These issues as well as the existing practical knowledge are pointed out in this report and summarized below.

- 1) Attention should be paid to columns made of high strength concrete exceeding 100MPa(14600psi). Because of early spalling of cover concrete, the flexural strength of such columns can be smaller than the evaluated strength based on the existing equations.
- 2) While no common equation for the evaluation of deformability of columns made of high-strength concrete has been established, an appropriate formulation based on a large number of test data that considers the effects of various factors, namely, the transverse reinforcement ratio, transverse reinforcement strength, axial load ratio, concrete strength, main bars' strength and axial loading type (constant or varying), is suggested.
- 3) Deformability of columns subjected to varying axial loads, which alternate between high-compression and high-tension, is larger than that of columns subjected to constant axial loading with similar compression axial load ratio. However, when beam-column joints subjected to varying axial load with tension side, concrete degradation in joint is considerable and shear failure may occur at large lateral drift angle or under large number of lateral loading cycles.

4) For columns, adding steel fibers to high-strength concrete exceeding the strength of 100MPa (14600psi) is very effective in terms of structural performance, damage control and fire resistance. It prevents early spalling of cover concrete and controls the degradation of flexural strength as well as damage by smearing developed cracks. It contributes to the increase of shear strength, reduces the amount of transverse reinforcement and is useful when congestion of reinforcement bars is to be prevented. Furthermore, high fire resistance performance for 150-200MPa (21800-29000psi) strength class concrete is effectively ensured when organic fibers are combined with steel fibers.

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**Table 1 – Seismic Criteria for Different Limit States**

|                           | Input Motion<br>(Max.Velocity)                                             | Maximum<br>Story Drift<br>(%) | Ductility<br>of Story |
|---------------------------|----------------------------------------------------------------------------|-------------------------------|-----------------------|
| Service<br>Limit<br>State | Level 1<br>( 25cm/s )                                                      | $\leq 0.5 \%$                 | $\leq 1.0$            |
| Design<br>Limit<br>State  | Level 2<br>( 50cm/s )                                                      | $\leq 1.0 \%$                 | $\leq 2.0$            |
| Ultimate<br>State         | Collapse or brittle failure of structural<br>elements should be prevented. |                               |                       |

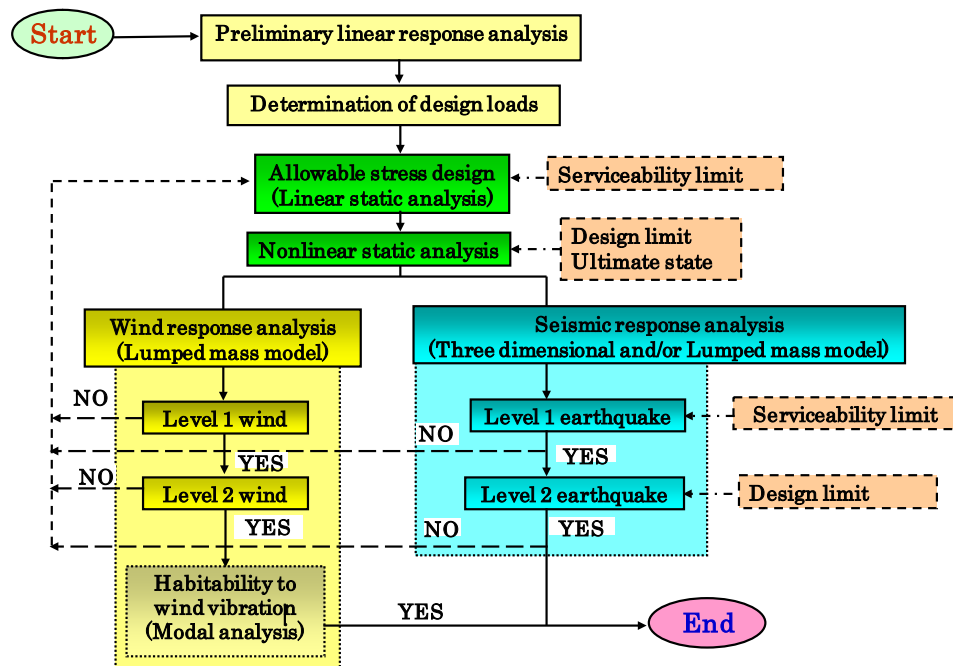


Figure1 –Typical Structural Design Procedure of High-rise RC Building

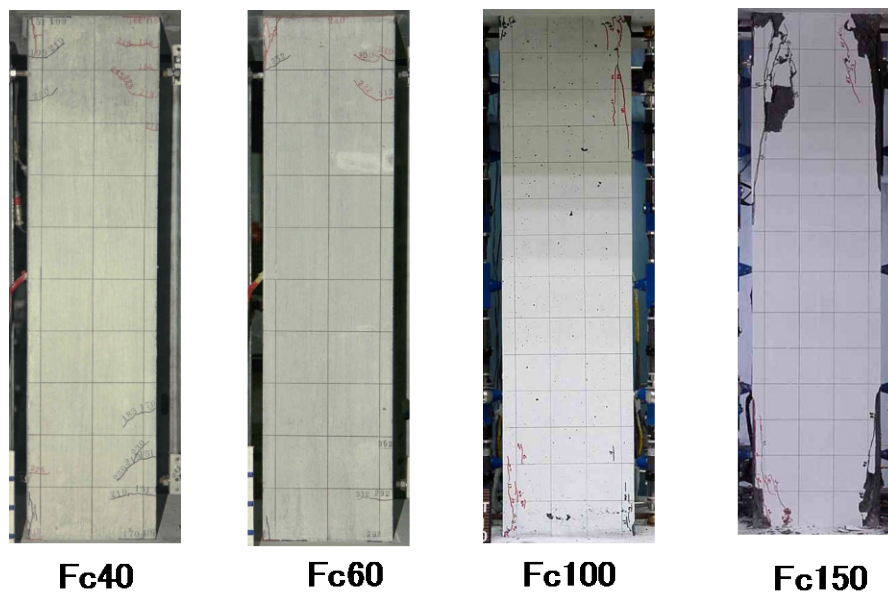


Figure2–Spalling of cover concrete for different concrete strengths at R=0.005 (axial load ratio =0.55~0.6)

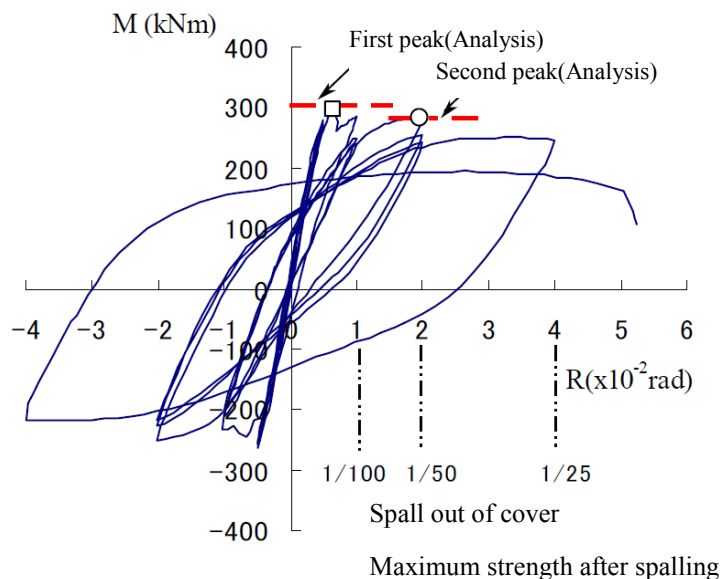


Figure3–Response of 150MPa concrete class column to lateral loading test (axial load ratio=0.33)  
(Note: 1 kNm=0.738 lbft)

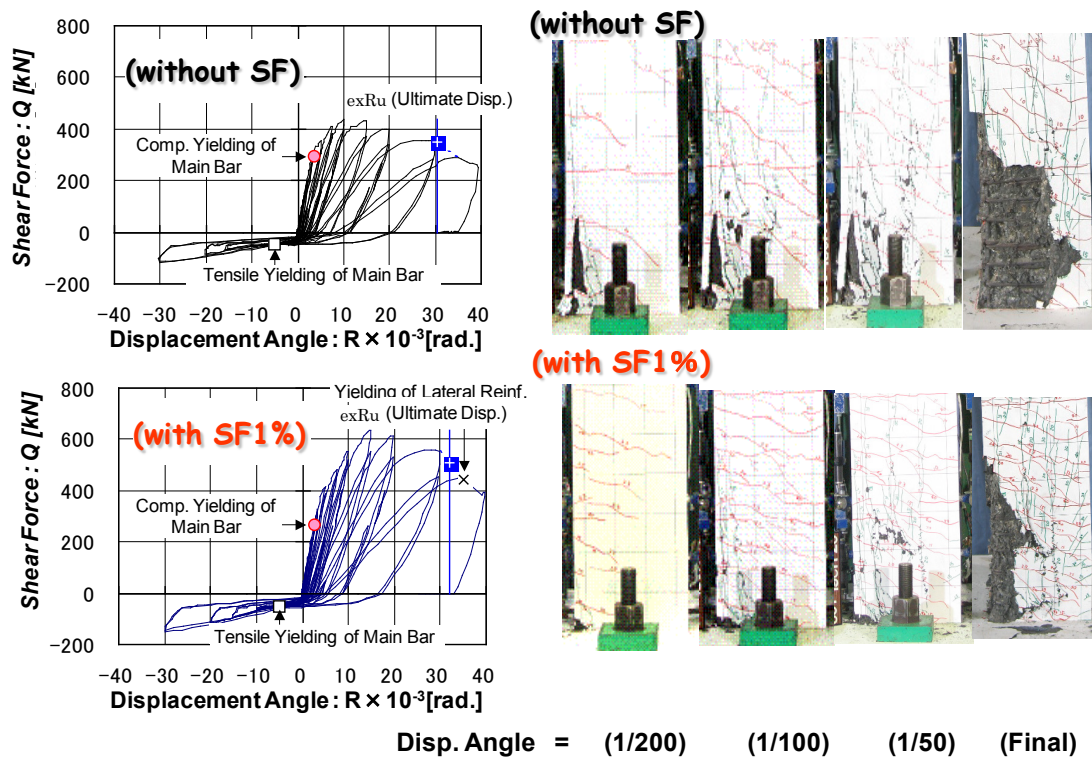


Figure4–Effect of Adding Steel Fibers on RC Column with 200 MPa Concrete. (Note: 1 kN=0.225 lb)

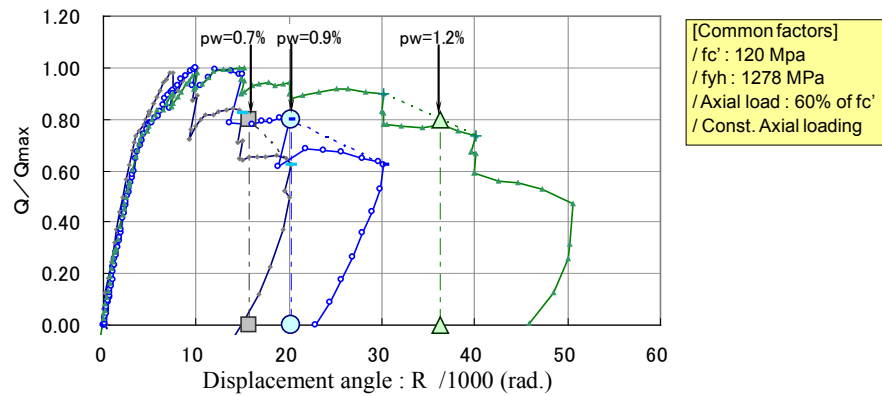


Figure5–Comparison of envelope curves of columns with different transverse reinforcement ratios (axial load ratio  $\eta = 0.6$ )

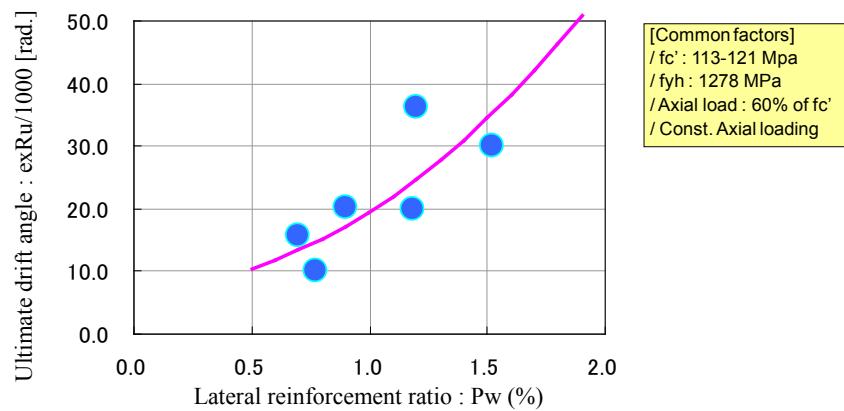


Figure6–Relationship of ultimate drift angle and transverse reinforcement ratio (axial load ratio  $\eta = 0.6$ )

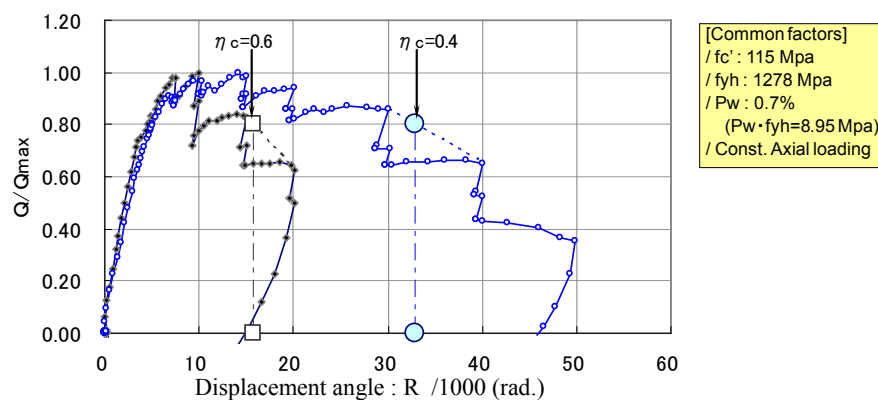


Figure7–Comparison of envelope curves of columns with different axial load ratios

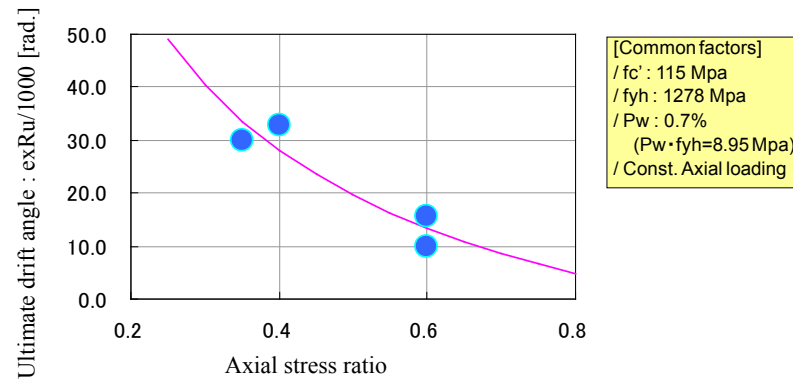


Figure8–Relationship of ultimate drift angle and axial load ratio

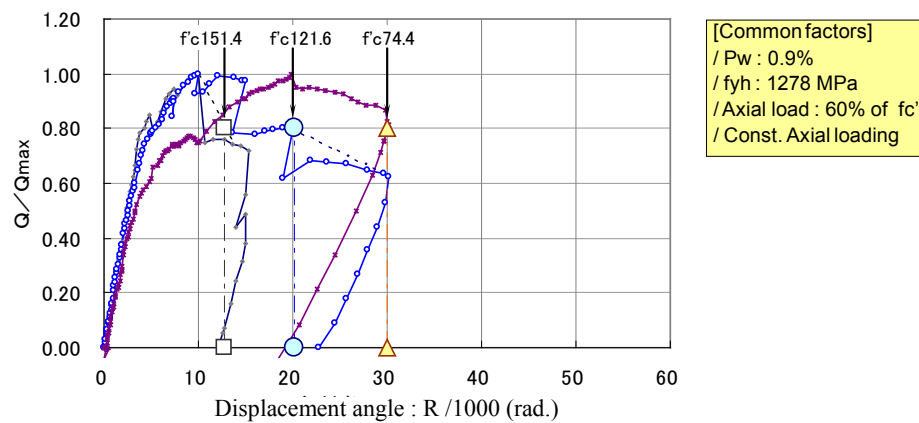


Figure9–Comparison of envelope curves of columns with different concrete strengths (axial load ratio  $\eta = 0.6$ )

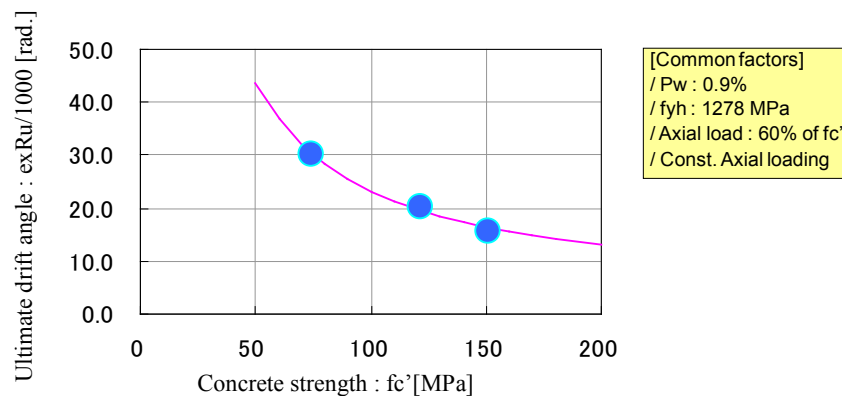


Figure10–Relationship of ultimate drift angle and concrete strength (axial load ratio  $\eta = 0.6$ )

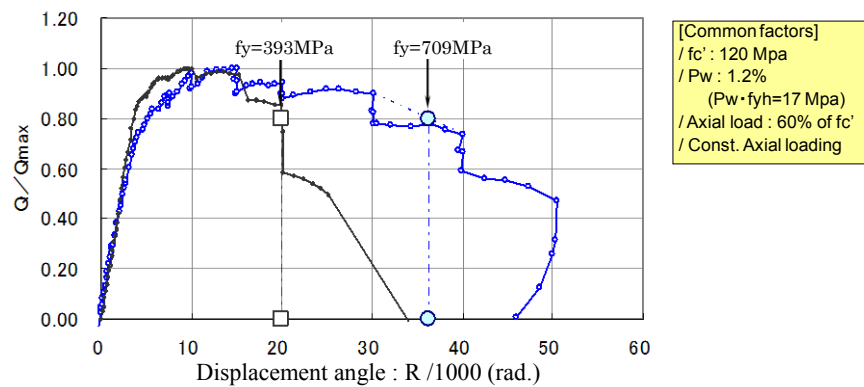


Figure11–Comparison of envelope curves of columns with different main bars' strengths (axial load ratio  $\eta = 0.6$ )

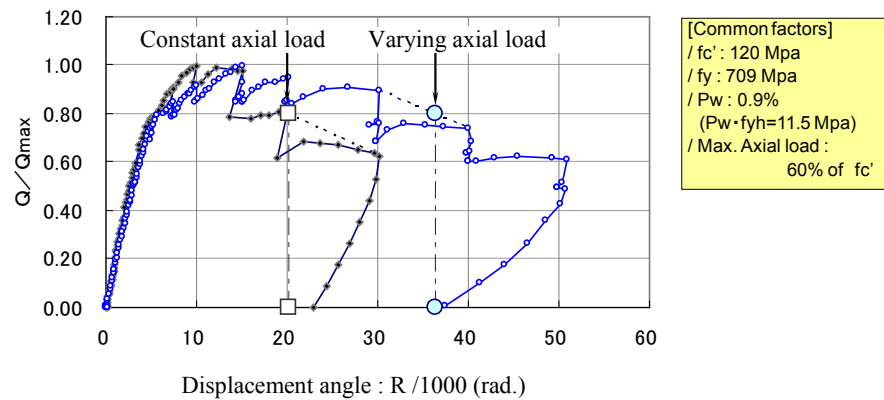
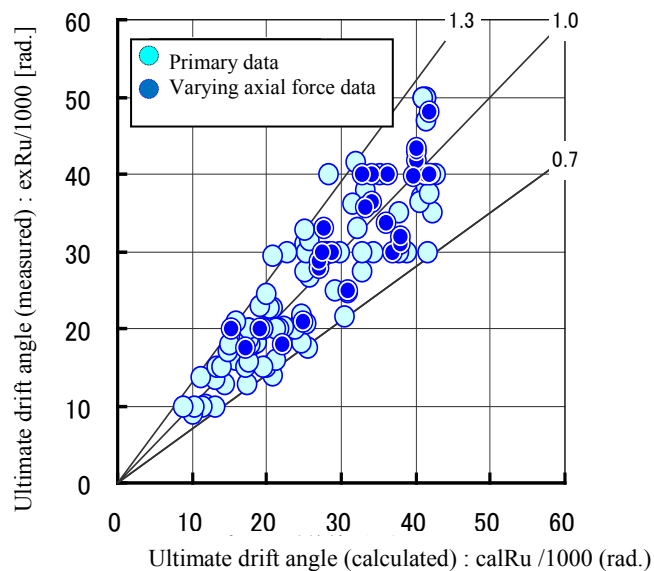


Figure12–Comparison of envelope curves of columns with different axial load histories (constant and varying axial loads)



$$R_u = \phi_b \cdot \phi_s \left( 22.5 \left( \frac{p_w \cdot f_{yh}}{f'_c} \right) \frac{(f_y)^{0.25}}{\eta_c} \lambda_{tu} + 2.5 \right) \leq 44$$

[ $\times 10^{-3} \text{ rad}$ ]

$$\lambda_{tu} = 10 \sqrt{\varepsilon_y \cdot \eta_t} + 1.1$$

$$\phi_b = 2.0 \left( 0.2 - \left( \frac{\eta_c \cdot f'_c}{n \cdot p_g \cdot f_y} \right) \left( \frac{S}{d_b} \right)^2 \right) + 1.0 \leq 1.0$$

$$\phi_s = 30 p_w \left( \frac{n_h - 2}{n_h} \right) \left( \frac{n_s - 2}{n_s} \right) + 0.95 \leq 1.1$$

$p_w$  : Transverse reinforcement ratio       $f_y$  : Main bars' strength [MPa]

$f_{yh}$  : Effective strength of transverse reinforcement [MPa]  $\leq 125 \sqrt{f'_c}$

$\eta_c$  : Compression axial load ratio       $f'_c$  : Concrete strength [MPa]

$p_g$  : Main bars' ratio [%]       $f_y$  : Main bars' yield strength [MPa]

$S$  : Transverse reinforcement spacing [mm]       $d_b$  : Main bars' diameter [mm]

$n$  : Modular ratio       $\lambda_{tu}$  : Amplification coefficient for varying axial load

$\varepsilon_y$  : Main bars' yield strain       $\eta_t$  : Tension axial load ratio

( $= N_t / N_{ty}$ ,  $N_t$  : Tension axial load,  $N_{ty}$  : Tension axial strength)

Figure13– Evaluation method of ultimate drift angle based on Ishikawa et al. (2008).

(Note: 1 mm=0.03949in. ; 1 MPa= 145 psi)

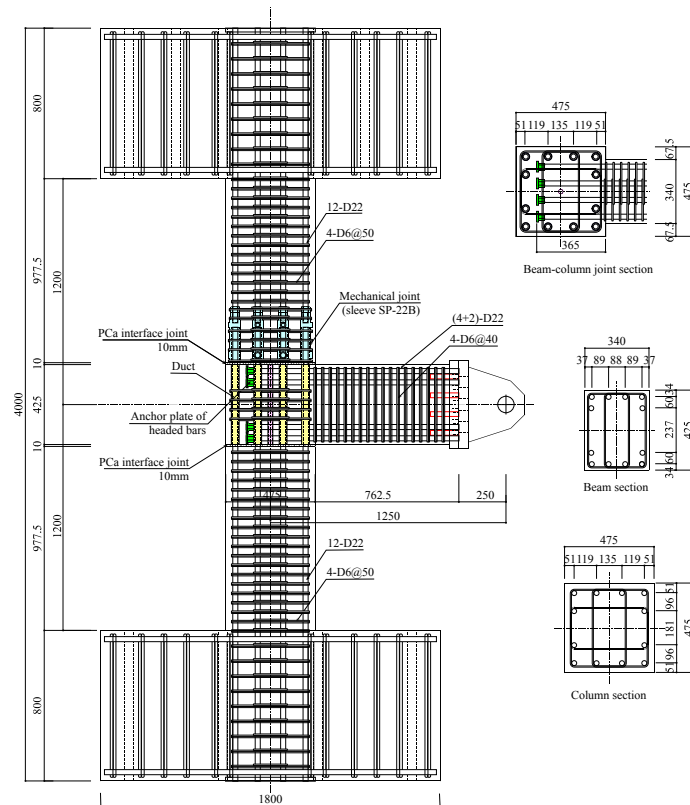


Figure14–Geometry and detailing of specimens. (Note: 1 mm=0.03949 in.)

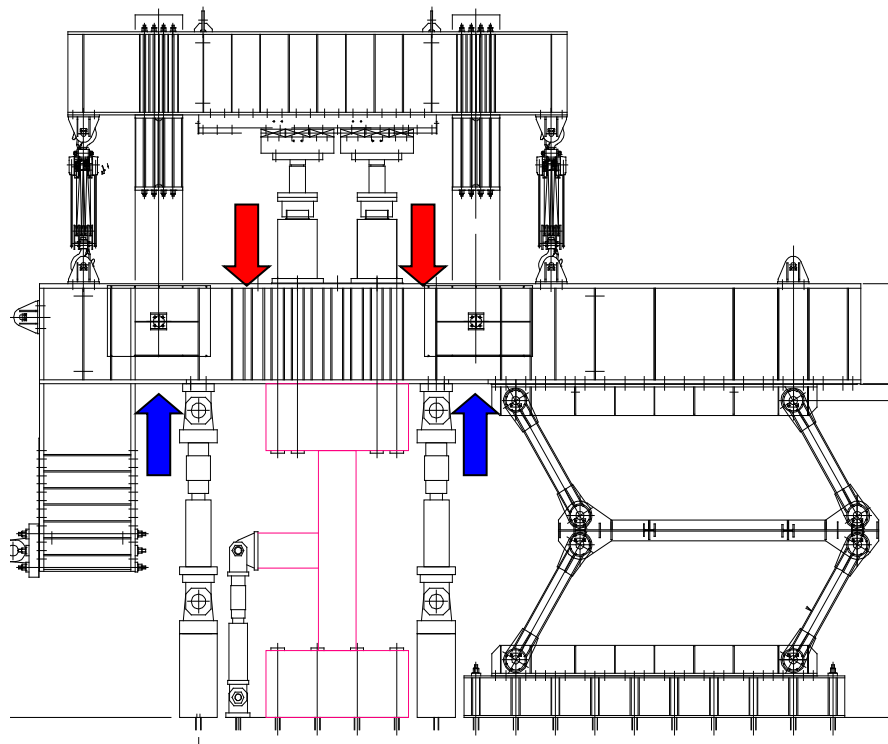


Figure15–Loading setup

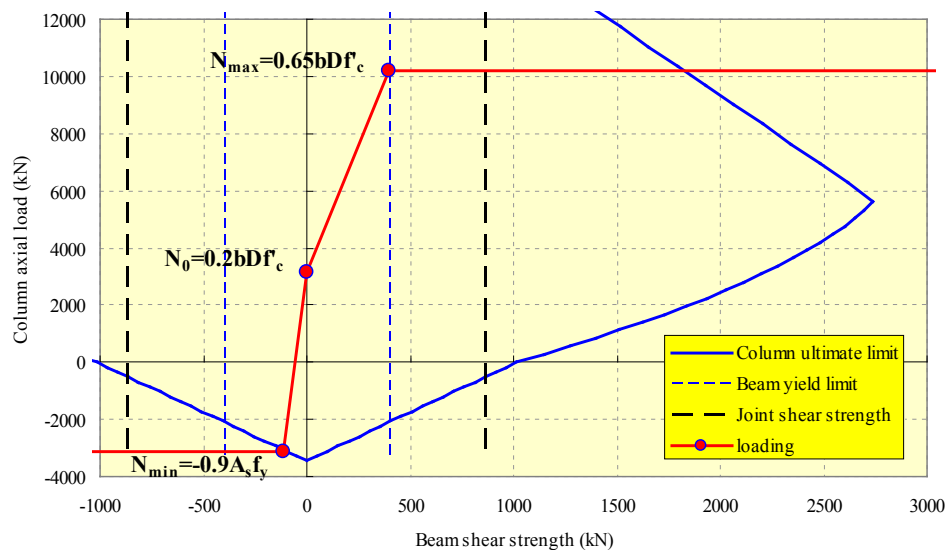


Figure16–Axial load course and interaction diagram. (Note: 1 kN=0.225lb)

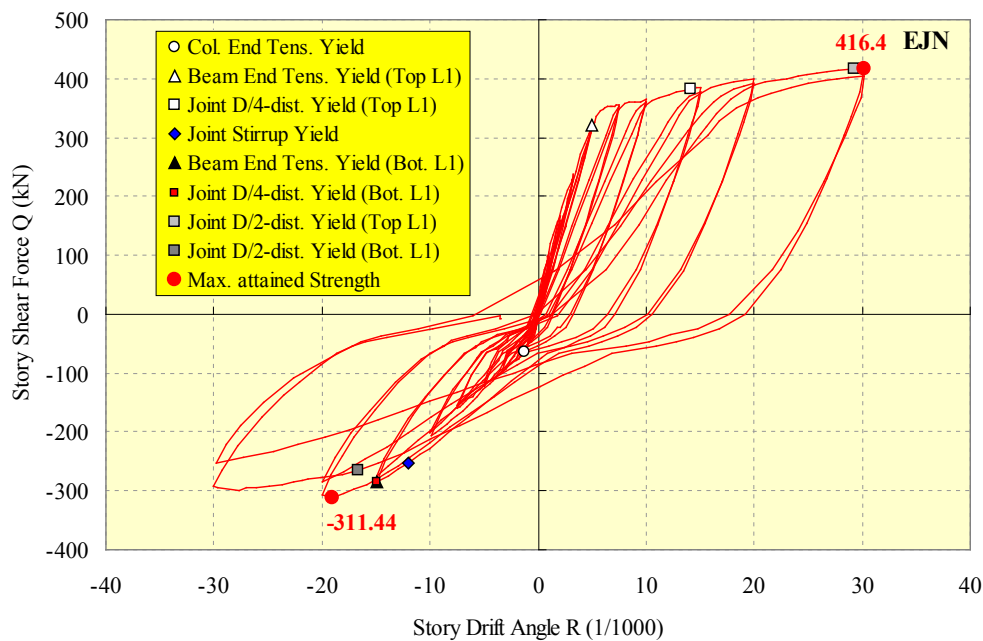


Figure17–Story shear force response. (Note: 1 kN=0.225 lb)

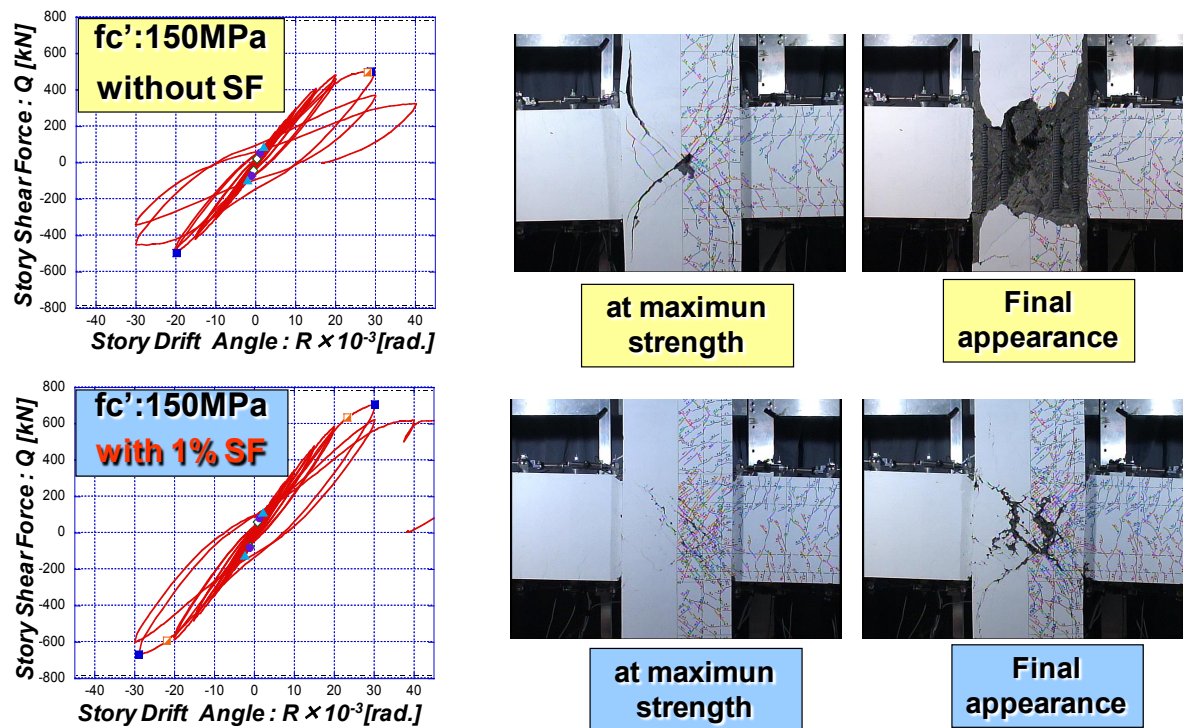


Figure18–Comparison of Beam-column Joints Using 150 MPa Concrete with and without Steel Fibers. (Note: 1 kN=0.225 lb)

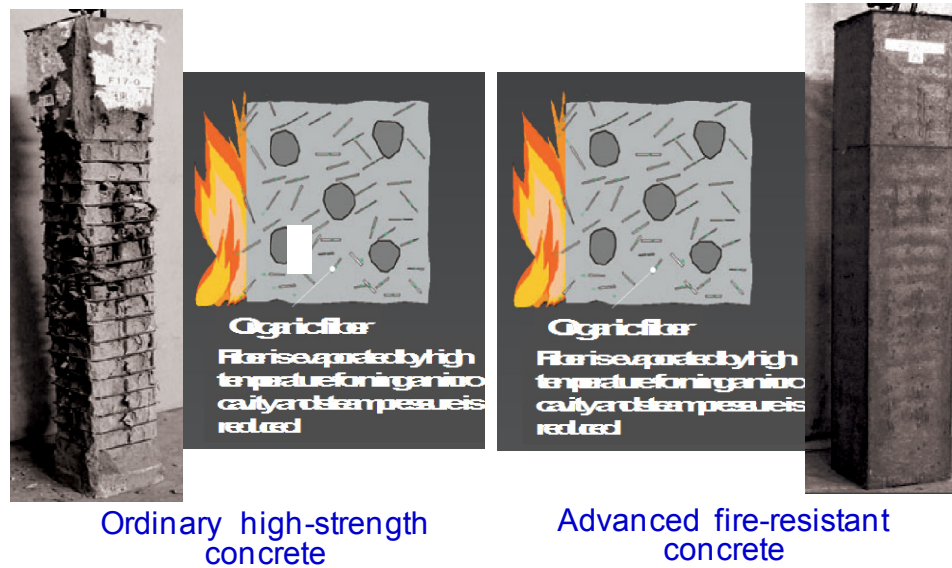


Figure19–Comparison of Fire Resistant Concrete Column and Ordinary Concrete Column

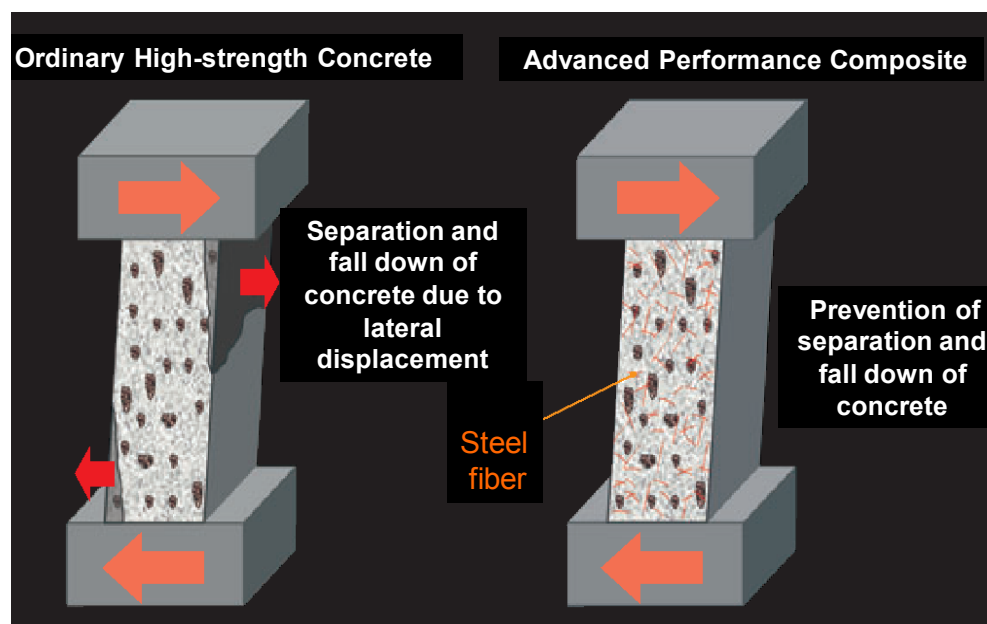


Figure20–Comparison of Damage to Columns with and without Steel Fibers



## Design of Seismic Confinement of Reinforced Concrete Columns Using High Strength Materials

Shyh-Jiann Hwang<sup>1,2</sup>, Guann-Jye Hwang<sup>3</sup>, Feng-Chan Chang<sup>4</sup>, Ying-Chang Chen<sup>4</sup>,  
Ker-Chun Lin<sup>5</sup>

<sup>1</sup> Professor, National Taiwan University, Taiwan

<sup>2</sup> Deputy Director, National Center for Research on Earthquake Engineering, Taiwan

<sup>3</sup> Master Student, National Taiwan University, Taiwan

<sup>4</sup> Master, National Taiwan University, Taiwan

<sup>5</sup> Researcher, National Center for Research on Earthquake Engineering, Taiwan

### Synopsis:

Column plays an important role as a vertical and lateral load carrying member in building structures. In order to reduce the dimension of the lower-story columns of high rise buildings, the use of high strength materials has become an effective measure. However, most of the available experimental data on the high strength columns came from the testing of reduced-scale specimens due to the limited testing capacity in laboratory. This research reports an experimental study of 5 full-scale specimens (600×600 mm [23.6×23.6 in.]) of columns subjected to quasi-static cyclic loading under high axial load up to  $0.67A_g f'_c$ . The main purpose of this research is to study the confinement effect and lateral drift capacity of reinforced concrete columns made of concrete with  $f'_c > 80$  MPa [12 ksi], longitudinal steel with  $f_y = 685$  MPa [100 ksi] and transverse reinforcement with  $f_{yt} = 785$  MPa [114 ksi]. Finally, based on test observations, a design equation on the seismic confinement of column is recommended.

**Keywords:** Column confinement, Deformation capacity, High axial load, High rise building, High strength material.

### Biography:

**Shyh-Jiann Hwang**, F.ACI, is a professor of the Department of Civil Engineering, National Taiwan University, Taipei, Taiwan. He also serves as deputy director at National Center for Research on Earthquake Engineering, Taipei, Taiwan. He received his PhD from the University of California at Berkeley, California. His research interests include seismic behavior of beam-column joints, shear strengths of reinforced concrete members, and seismic design and retrofitting of concrete structures.

**Guann-Jye Hwang** is a master student of the Department of Civil Engineering, National Taiwan University, Taipei, Taiwan. His research is mainly on the confinement effect of reinforced concrete member.

**Feng-Chan Chang** received his MS from the Department of Civil Engineering, National Taiwan University, Taipei, Taiwan. His research was on the confinement of RC columns using high strength materials.

**Ying-Chang Chen** is a structural engineer of the Sinotech Engineering Consultant, Ltd., Taipei, Taiwan. He received his MS from the National Taiwan University, Taipei, Taiwan. His research was on the confinement of RC columns using high strength materials.

**Ker-Chun Lin** is a researcher of Building Structures Division, National Center for Research on Earthquake Engineering, Taipei, Taiwan. He received his PhD from the National Taiwan University. His research interests include seismic behavior of beam-column joints for RC and steel structures, mechanical anchorage of headed bars on beam-column joints subjected to seismic loading, bond characteristics of high strength reinforcement and concrete, and large-scale structural experiments.

## INTRODUCTION

In a metropolitan area with a high population density, construction of high-rise buildings for residential purpose is a necessity especially for urban renewal. Reinforced concrete structure is preferred for a residential building due to its versatility to accommodate architectural elements. For columns at the lower part of high-rise buildings, the use of high-strength concrete (HSC) is required due to high axial loads. However, HSC fails in brittle manner and results in small deformation capacity of columns with potential plastic hinges. Therefore, how to improve the deformation capacity of HSC columns subjected to high axial loads becomes an important issue to be solved.

Traditionally, in concrete columns of special moment frames, transverse reinforcement is required to enable the column to sustain large deformation demands without degradation in lateral load resistance. It is questionable whether the successful experience in normal-strength concrete column can also apply to the HSC columns with high axial loads. In the 1980s, New RC Project<sup>1</sup> was carried out as a Japanese National Project to establish the design and construction guidelines for high-rise buildings up to 200 meters high. Several research programs to improve the deformation capacity of HSC columns by lateral confinement were carried out in Japan<sup>2</sup>. However, most of the available experimental data on the high strength columns came from the testing of reduced-scale specimens due to the limited testing capacity in laboratory.

This paper reports an experimental study of 5 full-scale specimens (600×600 mm [23.6×23.6 in.]) of columns subjected to quasi-static cyclic loading under high axial load up to  $0.67A_g f'_c$ . Objective of this research is to study the confinement effect and lateral drift capacity of reinforced concrete columns made of concrete with  $f'_c > 80$  MPa [12 ksi], longitudinal steel with  $f_y = 685$  MPa [100 ksi] and transverse reinforcement with  $f_{yt} = 785$  MPa [114 ksi]. Finally, based on test observations, a design equations on the seismic confinement of column is recommended.

## TEST PROGRAM

National Center for Research on Earthquake Engineering (NCREE) has launched the Taiwan New RC project since 2007 to promote the usage of high strength materials for RC high rise buildings. The improvement of deformation capacity of HSC columns by lateral confinement was taken up as one of the research programs in the project. Chang<sup>3</sup> and Chen<sup>4</sup> conducted the reversed cyclic loading tests on 13 HSC columns subjected to high axial loads in NCREE. Five columns in three pairs, selected from above tests<sup>3,4</sup>, were reported to compare the effects of confinement amount, confinement effectiveness and concrete strength.

For this selected group, column axial loads ranged from 0.42 to 0.67  $A_g f'_c$  and concrete strength ranged from 83 MPa [12 ksi] to 112 MPa [16 ksi]. The yield strengths of longitudinal and transverse reinforcement were 685 MPa [100ksi] and 785 MPa [114 ksi], respectively. These high strength reinforcement are complied with the Japanese standards<sup>5</sup>, and the typical stress-strain curves are presented in **Fig. 1**. Properties of test specimens were summarized in **Table 1**. Column dimension was 600 by 600 mm [23.6×23.6 in.]. Two types of columns were used, one was tied column and the other was spirally reinforced column, as shown in **Fig. 2**. The height-to-depth ratio of column was 3 and the ratio of longitudinal reinforcement was 2.25%. Transverse reinforcement was uniformly distributed along height, as shown in **Fig. 3**.

Specimen B5 was detailed with the column hoops (**Fig. 2a**) as per the ACI 318 requirement<sup>6</sup>, but not conforming to the provisions on strength limitation. The ratio of transverse reinforcement  $\rho_h$  for Specimen B5 was 0.87% and its spacing was 140 mm [5.5 in.]. It was assumed that the larger amount of confinement can also improve the deformation capacity of HSC column. Specimen T70-N46-D3 used the same cross section as that of Specimen B5, but the ratio of transverse reinforcement  $\rho_h$  was increased to 1.91% with a spacing of 100 mm [3.9 in.].

The ability of confining steel to maintain core concrete integrity and to provide lateral support of longitudinal reinforcement is related to the end anchorage of confining steel. A column crosstie with 90° hook will lose its confining ability after spalling of cover. ACI 318 Code<sup>6</sup> does not account for this ineffectiveness of confinement in determining the required amount of confinement. Specimen T70-N42-D4 was intended to test the effectiveness of the hoop detailing on end anchorage. The confining steel of Specimen T70-N42-D4 was similar to that of Specimen T70-N46-D3, but the end anchorage of crossties was changed to 180° hook.

A new type of spirally reinforced column, termed as multiple-spiral column<sup>7</sup>, was used in Taiwan precast industry. This interlocking spiral confinement for rectangular columns (**Fig. 2b**) was found to be very effective in providing confinement for columns subjected to axial load and bending. Specimens S70-N67-D2 and S100-N50-D2 employed the same amount and detailing of confining steel (**Fig. 2b**) but cast in concrete with different compressive strength. The testing purpose of these two specimens was to compare the effect of concrete strength on the deformation capacity.

All the specimens were tested in NCREC using the Multi-Axial Testing System (MATS), as depicted in **Fig. 4a**. For column tests, MATS can provide a vertical load up to 40,000 kN [9,036 kip] and a reversed lateral load of 4,000 kN [903 kip]. Columns were tested (**Fig. 4b**) using double curvature bending under a constant axial load. Lateral drifts were imposed on specimens by displacement-controlled actuators with a loading history (**Fig. 5**) suggested by ACI 374 Committee<sup>8</sup>.

In concrete columns of special moment frames, transverse reinforcement not only provides confinement effect but also resists shear forces. The current ACI 318 Code<sup>6</sup> treats these two requirements independently and simply chooses the larger amount to detail the transverse reinforcement of columns. However the confinement requirement and the shear requirement on transverse reinforcement might influence each other. In order to include the shear effect on confinement requirement empirically, this test program intended to enhance the acting shear for test specimens to a certain extent. Using the column height-to-depth ratio of 3 and imposing double curvature bending can enlarge the acting shear effectively. **Table 2** illustrates the flexural-to-shear strength ratios of test specimens. Shear strength was calculated according to ACI 318 Code<sup>6</sup>, whereas flexural strength was computed as per ITG-4.3R-07<sup>9</sup>. As indicated by **Table 2**, the flexural-to-shear strength ratio ranged from 0.40 to 0.67 for this test program.

## TEST RESULTS

**Figure 6** presents the crack pattern and the photo at the conclusion of test for Specimen T70-N42-D4 which is typical for all the tied columns in this test program. Vertical cracks along column axis indicates the strong influence of high axial loads (**Fig. 6a**). Buckling of column longitudinal bars and fracture of transverse reinforcement (**Fig. 6b**) are the evidences of flexural compression failure.

The relationships of the lateral load  $V$  versus the drift ratio  $D$  for all the tied columns are illustrated in **Fig. 7**, where the lateral load  $V$  is the acting shear on the column and the drift ratio  $D$  is defined as the relative deflection between two end points of the column divided by the clear height of the column. The ultimate drift, labeled in **Fig. 7**, is defined as the drift level that the lateral load resistance of column should not degrade by more than 20 % from the peak capacity. The related values of the peak loads and the ultimate drifts measured during tests are presented in **Table 3**.

The detailing of the column hoops for Specimen B5 followed the requirements of ACI 318 provisions<sup>6</sup>, but **Fig. 7a** shows an insufficient deformation capacity of Specimen B5. Increased amount of confinement of Specimen

T70-N46-D3 can improve the deformation capacity of column, as shown in **Fig. 7b**. Deformation capacity of column can be further enhanced if the crossties of column are equipped with seismic hooks at both ends, as for Specimen T70-N42-D4 (**Fig. 7c**).

**Figure 8** presents the crack pattern and the photo at the conclusion of test for Specimen S70-N67-D2, and **Fig. 9** shows the relationships of the lateral load versus the drift ratio for all the multiple-spiral columns. Dominant vertical cracks, rebar buckling and fracture of transverse reinforcement were observed (**Fig. 8**) for all spiral columns. Therefore, the failure mode of the multiple-spiral columns is also defined as flexural compression failure. **Figure 9a** shows a satisfactory deformation performance of Specimen S70-N67-D2, even with a high axial load up to 67% of  $A_g f'_c$ . Increased concrete strength of Specimen S100-N50-D2 decreases the deformation capacity of column, as shown in **Fig. 9b**.

## DISCUSSION

A performance target on lateral drift is needed in evaluating the adequacy of confinement provisions. ACI 318 Code<sup>6</sup> requires a concrete building to achieve a collapse prevention performance level when it is subjected to the Maximum Considered Earthquake. For the performance level of collapse prevention, it is acceptable that lateral load resistance of columns forming part of a special moment frame should not degrade by more than 20% from the peak capacity<sup>10</sup>. Current codes usually specify a 2% drift limit for most building types. Since Maximum Considered demands are 1.5 times Design Basis demands, a 3% drift limit is adequate for the demand of Maximum Considered Earthquake. Therefore, 3% column drift at 20% degradation in lateral resistance can be used as a performance target when evaluating the adequacy of confinement provisions<sup>11</sup>.

### Effect of confinement amount

Using a database of 145 rectangular column specimens subjected to reversed-cyclic loading<sup>12</sup>, Elwood et al.<sup>11</sup> already indicated that the confinement provisions in ACI 318 Code<sup>6</sup> do not necessarily ensure adequate deformation capacity, particularly for columns with high axial load. Performance of Specimen B5 shows a repeated observation for column using high-strength materials with high axial load. Specimen B5 was designed according to ACI 318 Code<sup>6</sup>, but its ultimate drift only reached 1.25% (**Table 3**). On the other hand, Specimen T70-N46-D3 with larger amount of confinement can achieve an ultimate drift level of 2.78% (**Table 3**). It had been reported that the ability of the HSC column core to sustain compressive strains can be increased with increasing confinement pressure<sup>2</sup>. As a consequence, larger amount of confinement can yield better deformation capacity of HSC columns. **Figure 10a** shows the comparison of envelopes of hysteresis curves for Specimens B5 and T70-N46-D3.

It should be noted that Specimen B5 was equipped with a transverse reinforcement  $\rho_h$  of 0.87% and subjected to an axial load of 17,119 kN [3,867 kip], whereas Specimen T70-N46-D3 had a  $\rho_h$  of 1.91% and an axial load of 14,515 kN [3,279 kip]. The better deformation capacity of Specimen T70-N46-D3 might be attributable to both the larger amount of confinement and the lower axial load level. Strains associated with axial load are additive to the compressive strains associated with lateral deformation. High axial load can easily consume the deformation capacity provided by confinement. The dependence of the axial load demand on amount of confinement should be fully recognized in evaluating the adequacy of design provisions for confinement<sup>11</sup>.

### Effect of confinement effectiveness

The ability of confining steel to maintain core concrete integrity and to increase deformation capacity is also related to the confinement effectiveness. Column longitudinal bars anchored by hooks of crossties that are greater than 135° can result in better confinement for the core concrete compared with longitudinal bars supported by 90° hooks. This is particularly important for columns with axial loads above the balance point and controlled by compression failure of the concrete.

The design of transverse reinforcement was the same for both Specimen T70-N46-D3 and Specimen T70-N42-D4, and the only difference was the detailing of crossties. Specimen T70-N46-D3 employed the crosstie having a 135° hook at one end and a 90° hook at the other end, but Specimen T70-N42-D4 used the crosstie having a 180° hook at both ends. These two specimens were subjected to the same axial load during tests (**Table 1**). Specimen T70-N46-D3 attained a lateral drift of 2.78%, which is disqualified as judged by the performance target of 3%. On the other hand, Specimen T70-N42-D4 achieved a satisfactory drift level of 3.77% (**Table 3**). **Figure 10b** shows the comparison of envelopes of hysteresis curves for Specimens T70-N46-D3 and T70-N42-D4. This comparison is an evidence that confinement effectiveness is a key parameter to determine the behavior of confined concrete. Therefore, confinement effectiveness should be considered in determining the required amount of confinement.

Effect of concrete strength

HSC generally shows brittle failure in compression, that is, the higher the concrete compressive strength results in the lower usable compressive strain. If two identical columns except for concrete strength are tested using the same loading, it is suspected that the column with higher concrete strength will result in smaller deformation capacity. The testing pair of Specimens S70-N67-D2 and S100-N50-D2 was intended for this purpose.

Specimens S70-N67-D2 and S100-N50-D2 were constructed identically but cast with different concrete strength (**Table 1**). The former used  $f'_c = 83$  MPa [12 ksi] and the latter used  $f'_c = 112$  MPa [16 ksi]. Both specimens went through the same axial load of 20,117 kN [4,544 kip] during reversed cyclic tests. Specimen S70-N67-D2 with lower concrete strength attained a satisfactory drift level of 3.86%, even with a high axial load of 67%  $A_g f'_c$  (**Fig. 9**). However, Specimen S100-N50-D2 with higher concrete strength reached only a drift level of 1.58% (**Table 3**). **Fig. 10c** shows the comparison of envelopes of hysteresis curves for Specimens S70-N67-D2 and S100-N50-D2. This comparison clearly shows that the brittleness nature of HSC should be considered in the design provisions of column confinement.

## DESIGN IMPLEMENTATION

Based on the observations in this test program, It is recognized that level of axial load, confinement effectiveness and concrete strength should be considered in determining the required amount of confinement of HSC columns. However, ACI 318 Code<sup>6</sup> does not account for axial load demand and confinement effectiveness. To address the above deficiencies in the confinement provisions of ACI 318 Code<sup>6</sup>, the CSA A23.3-04 equation for column confinement<sup>13,14</sup> seems to be a good alternative as follow.

$$\frac{A_{sh}}{sb_c} = 0.2k_p k_n \frac{f'_c}{f_{yt}} \frac{A_g}{A_{ch}} \geq 0.09 \frac{f'_c}{f_{yt}} \quad (1)$$

where:  $k_p = P / P_o$   
 $P_o = \alpha_1 f'_c (A_g - A_{st}) + A_{st} f_y$   
 $\alpha_1 = 0.85 - 0.0015 f'_c \geq 0.67$   
 $k_n = \frac{n_\ell}{n_\ell - 2}$

$A_{ch}$  = cross-sectional area of a structural member measured to the outside edges of transverse reinforcement;  $A_g$  = gross area of column;  $A_{sh}$  = total cross-sectional area of transverse reinforcement (including crossties) within spacing  $s$  and perpendicular to dimension  $b_c$ ;  $A_{st}$  = total area of nonprestressed longitudinal reinforcement;  $b_c$  = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area  $A_{sh}$ ;  $f'_c$  = specified compressive strength of concrete;  $f_y$  = specified yield strength of longitudinal reinforcement ( $f_y \leq 550$  MPa);  $f_{yt}$  = specified yield strength of transverse reinforcement ( $f_{yt} \leq 500$  MPa);  $n_\ell$  = number of longitudinal bars laterally supported by corner of hoop or hook of crosstie;  $P$  = axial compressive force on column;  $P_o$  = nominal axial strength at zero eccentricity; and  $s$  = center-to-center spacing of transverse reinforcement.

Equation (1) does account for the previous concerns, such as level of axial load, confinement effectiveness and concrete strength. For higher axial load  $P$ , Eq. (1) requires larger amount of  $A_{sh} / (sb_c)$ . For HSC column, the value of  $P_o$  calculated by Eq. (1) is relatively smaller. Thus amount of confinement is relatively larger for HSC column by Eq. (1). For column with a smaller number of laterally supported longitudinal reinforcement around the perimeter of the column core, the value of  $n_\ell$  is smaller and the amount of confinement is larger per Eq. (1).

Based on the observation in this test program and the customary practice in Taiwan, Eq. (1) can be further modified. Firstly, this test program indicates that, for columns failed in flexural compression, longitudinal bars supported by 90° hooks are inferior to those anchored by seismic hooks. The definition of  $n_\ell$  of Eq. (1) doesn't distinguish 90° hook from seismic hook. This deficiency can be addressed by excluding 90° hook from  $n_\ell$ . Secondly, Taiwan precast industry tends to use larger diameter rebars concentrated on the corners of column, which can facilitate fabrication. These widely-spaced longitudinal bars decrease the effectiveness of confinement and the associated compensation is needed for this shortcoming. Thirdly, the limit of 500 MPa for  $f_{yt}$  of Eq. (1) is too conservative and is a barrier for the promotion of high strength steel. Lastly, in Taiwan, the  $P_o$  of normal-strength

concrete column is estimated per ACI 318 Method<sup>6</sup>. It seems unnecessary to modify the calculation of  $P_o$  for the column using normal-strength concrete as Eq. (1) does.

To address the above issues, Eq. (2) is proposed to design the amount of confinement.

$$\frac{A_{sh}}{s b_c} = 0.2 k_p k_n \frac{f'_c}{f_{yt}} \frac{A_g}{A_{ch}} \geq 0.09 \frac{f'_c}{f_{yt}} \quad (2)$$

where:  $k_p = P / P_o$   
 $P_o = \alpha_1 f'_c (A_g - A_{st}) + A_{st} f_y$   
 $0.7 \leq \alpha_1 = 0.85 - 0.0022(f'_c - 55) \leq 0.85$

$$k_n = \frac{n_{ls}}{n_{ls} - 2} \frac{300 + h_x}{500}$$

$f_{yt}$  = specified yield strength of transverse reinforcement ( $f_{yt} \leq 700$  MPa);  $h_x$  = maximum center-to-center horizontal spacing of crosstie or hoop legs on all faces of the column  $[(300 + h_x)/500 \geq 1; h_x: \text{mm}]$ ;  $n_{ls}$  = number of longitudinal bars around the perimeter of column core which are laterally supported by the corner of hoops or by hooks anchored in the core. Calculation of  $P_o$  of Eq. (2) was suggested by ITG-4.3R-07<sup>9</sup> and provision on  $h_x$  was recommended by Elwood et al.<sup>15</sup>.

**Figures 11a** and **11c** present the evaluations of Eqs. (1) and (2), respectively, for tied columns using the PEER Structural Performance database<sup>12</sup>. **Figures 11b** and **11d** show the comparisons of Eqs. (1) and (2), respectively, with the present tests. The performance target is shown as a horizontal line at a 3% drift ratio. Data in the bottom-right quadrant represent columns with more confinement reinforcing than required by the provision but exhibiting drift capacity below the target, thus indicating that the provisions may be considered unconservative for these cases. **Figures 11a** and **11c** indicate that confinement in amounts based on Eqs. (1) and (2) generally provides deformation capacities in excess of the target drift ratio of 3%. **Figure 11b** indicates that Eq. (1) is conservative as compared with the present tests, but **Figure 11d** shows that Eq. (2) is justified by the present tests.

## CONCLUSIONS

A testing program consisted of 5 full-scale specimens (600×600 mm) of HSC columns subjected to quasi-static cyclic loading and high axial load was carried out to study the confinement effect and lateral drift capacity. Test specimens were made of concrete with  $f'_c > 80$  MPa [12 ksi], longitudinal steel with  $f_y = 685$  MPa [100 ksi] and transverse reinforcement with  $f_{yt} = 785$  MPa [114 ksi]. The following conclusion regarding the confinement of HSC columns are drawn from this study:

1. Although HSC fails in brittle manner in compression, HSC columns under high axial loads can still attain a satisfactory drift level of 3% with adequate confinement.
2. Lateral deformation capacity of HSC columns is increased with increasing amount of confinement. Strains associated with axial load are additive to the compressive strains associated with lateral deformation. The dependence of the axial load demand on amount of confinement should be fully recognized.
3. For the concern of deformation capacity of the columns failed in flexural compression, longitudinal bars supported by 90° hooks are inferior to those anchored by seismic hooks.
4. Lateral deformation capacity of HSC columns is decreased with increasing concrete strength due to the brittleness nature of HSC.
5. A design equation on seismic confinement of column, which can generally provides deformation capacities in excess of the target drift ratio of 3%, is proposed as Eq. (2).

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# NOTATION

|              |                                                                                                                                                         |
|--------------|---------------------------------------------------------------------------------------------------------------------------------------------------------|
| $A_{ch}$     | = cross-sectional area of a structural member measured to the outside edges of transverse reinforcement                                                 |
| $A_g$        | = gross area of column                                                                                                                                  |
| $A_{sh}$     | = total cross-sectional area of transverse reinforcement (including crossties) within spacing $s$ and perpendicular to dimension $b_c$                  |
| $A_{st}$     | = total area of nonprestressed longitudinal reinforcement                                                                                               |
| $b_c$        | = cross-sectional dimension of member core measured to the outside edges of the transverse reinforcement composing area $A_{sh}$                        |
| $f'_c$       | = specified compressive strength of concrete                                                                                                            |
| $f_y$        | = specified yield strength of longitudinal reinforcement                                                                                                |
| $f_{yt}$     | = specified yield strength of transverse reinforcement                                                                                                  |
| $h_x$        | = maximum center-to-center horizontal spacing of crosstie or hoop legs on all faces of the column                                                       |
| $n_\ell$     | = number of longitudinal bars laterally supported by corner of hoop or hook of crosstie                                                                 |
| $n_{\ell s}$ | = number of longitudinal bars around the perimeter of column core which are laterally supported by the corner of hoops or by hooks anchored in the core |
| $P$          | = axial compressive force on column                                                                                                                     |
| $P_o$        | = nominal axial strength at zero eccentricity                                                                                                           |
| $s$          | = center-to-center spacing of transverse reinforcement                                                                                                  |
| $\rho_h$     | = area ratio of transverse reinforcement of column                                                                                                      |

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Table 1–Properties of test specimens

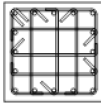
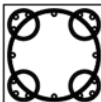
| Specimen Id. | Section Type                                                                      | $P$<br>kN<br>(kip) | $f'_c$<br>MPa<br>(ksi) | $\frac{P}{A_g f'_c}$ | $f_y$<br>SD685, MPa<br>(ksi) | $f_{yt}$<br>SD785, MPa<br>(ksi) |                            | Hoop<br>spacing, mm (in.) |                     |
|--------------|-----------------------------------------------------------------------------------|--------------------|------------------------|----------------------|------------------------------|---------------------------------|----------------------------|---------------------------|---------------------|
| B5           |  | 17,119<br>(3,867)  | 83.4<br>(12.1)         | 0.57                 | 744<br>(108.2)               | 817.3<br>(118.8)                |                            | D13@140 (5.5)             |                     |
| T70-N46-D3   |                                                                                   | 14,515<br>(3,279)  | 88.5<br>(12.9)         | 0.46                 | 735<br>(106.9)               | 820<br>(119.2)                  |                            | D16@100 (3.9)             |                     |
| T70-N42-D4   |  | 14,515<br>(3,279)  | 95.1<br>(13.8)         | 0.42                 | 735<br>(106.9)               | 820<br>(119.2)                  |                            | D16@100 (3.9)             |                     |
| S70-N67-D2   |  | 20,117<br>(4,544)  | 83.1<br>(12.1)         | 0.67                 | 739<br>(107.4)               | Large<br>Spiral<br>(SD785)      | Small<br>Spiral<br>(SD490) | Large<br>Spiral           | Small<br>Spiral     |
| S100-N50-D2  |                                                                                   | 20,117<br>(4,544)  | 112.1<br>(16.3)        | 0.50                 | 739<br>(107.4)               | 820<br>(119.2)                  | 625<br>(90.9)              | D16<br>@45<br>(1.8)       | D13<br>@45<br>(1.8) |

Table 2–Calculated flexural-to-shear strength ratio of specimens

| Specimen Id. | Shear Strength   |                  |                  | Flexural Strength          |                     | $\frac{V_{mn}}{V_n}$ |
|--------------|------------------|------------------|------------------|----------------------------|---------------------|----------------------|
|              | $V_c$<br>kN(kip) | $V_s$<br>kN(kip) | $V_n$<br>kN(kip) | $M_{nITG}$<br>kN-m(kip-in) | $V_{mn}$<br>kN(kip) |                      |
| B5           | 2151 (486)       | 1016 (230)       | 3167 (715)       | 1907 (16960)               | 2119 (479)          | 0.67                 |
| T70-N46-D3   | 1944 (439)       | 2010 (454)       | 3954 (893)       | 2178 (19370)               | 2420 (547)          | 0.61                 |
| T70-N42-D4   | 2015 (455)       | 2083 (471)       | 4098 (926)       | 2296 (20420)               | 2551 (576)          | 0.62                 |
| S70-N67-D2   | 2398 (541)       | 1928 (436)       | 4326 (977)       | 1545 (13740)               | 1717 (388)          | 0.40                 |
| S100-N50-D2  | 2785 (629)       | 1963 (443)       | 4748 (1072)      | 2317 (20606)               | 2574 (581)          | 0.54                 |

Table 3–Test results

| Specimen Id. | Peak Load          |                | Ultimate Drift    |               |
|--------------|--------------------|----------------|-------------------|---------------|
|              | $V_{peak}$ kN(kip) | $D_{peak}$ (%) | $V_{ult}$ kN(kip) | $D_{ult}$ (%) |
| B5           | 2835 (640)         | 0.72           | 2649 (598)        | 1.25          |
| T70-N46-D3   | 3151 (712)         | 2.46           | 2698 (609)        | 2.78          |
| T70-N42-D4   | 3312 (748)         | 1.63           | 2650 (599)        | 3.77          |
| S70-N67-D2   | 3302 (746)         | 1.66           | 2642 (597)        | 3.86          |
| S100-N50-D2  | 3064 (692)         | 0.68           | 2451 (554)        | 1.58          |

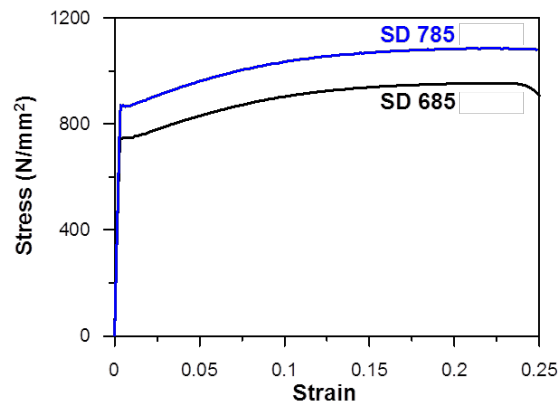


Fig. 1 –Typical stress-strain curves of high strength reinforcement

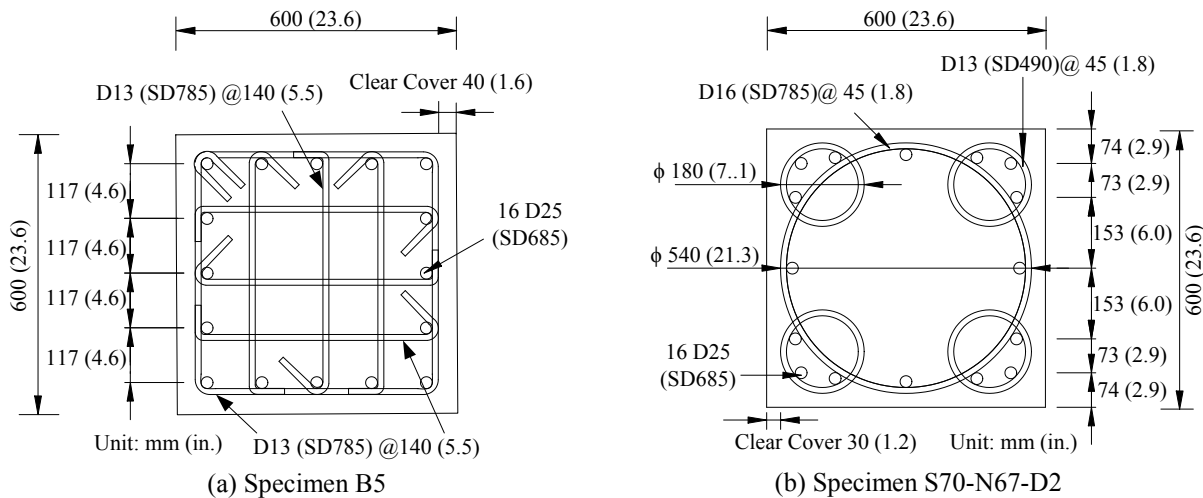


Fig. 2–Column cross section

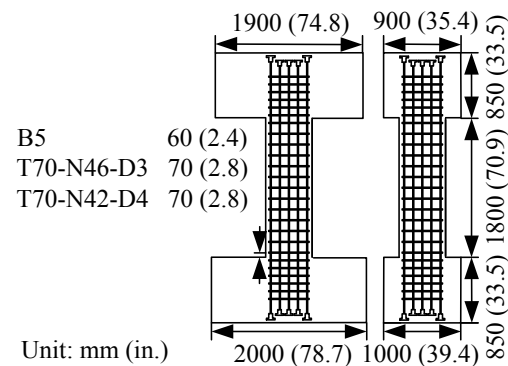
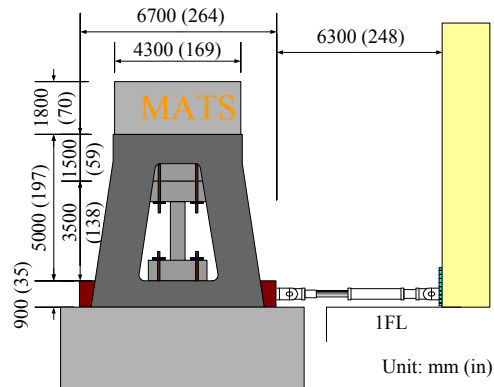


Fig. 3–Elevation of tied columns



(a) MATS testing facility



(b) Photo of test setup

**Fig. 4–Test setup**

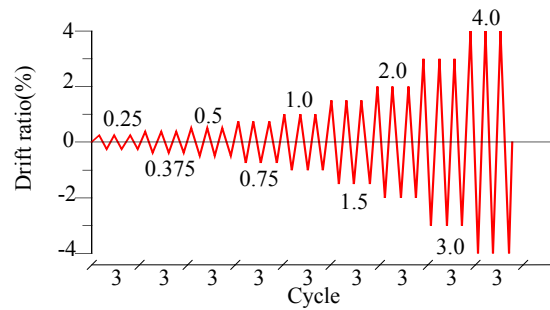
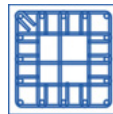


**Fig. 5–Loading history**



(a) Crack pattern



(b) Photo at conclusion

**Fig. 6–Specimen T70-N42-D4**

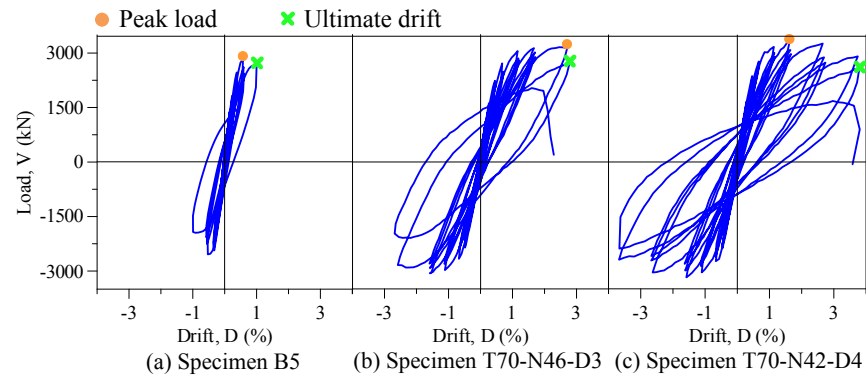


**Fig. 7–Load-versus-deflection response of tied columns**

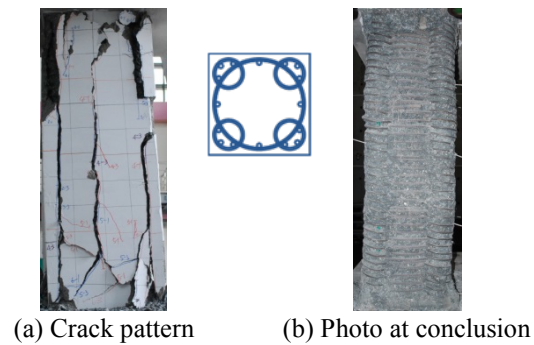


**Fig. 8–Specimen-S70-N67-D2**

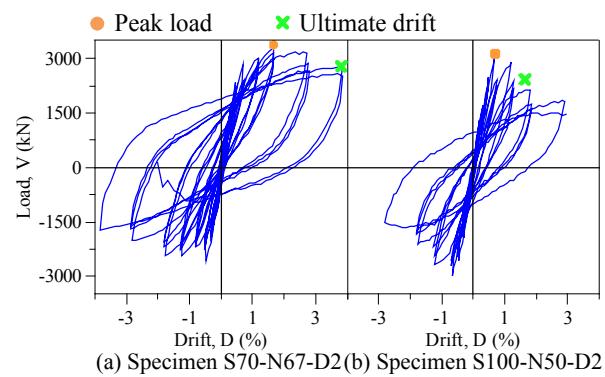


**Fig. 9–Load-versus-deflection response of multiple-spiral columns**

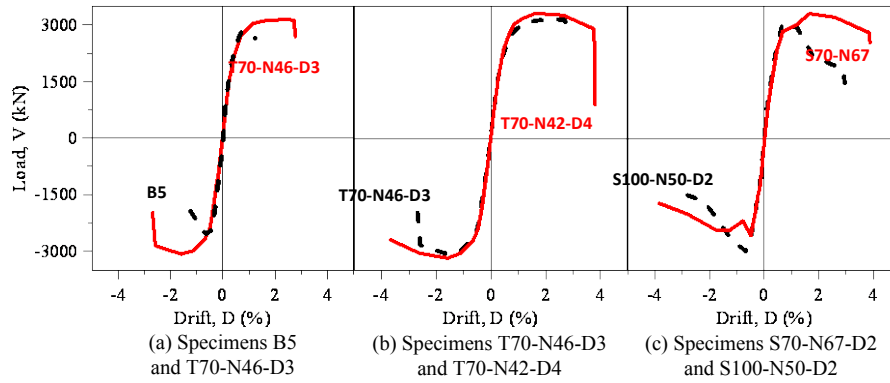
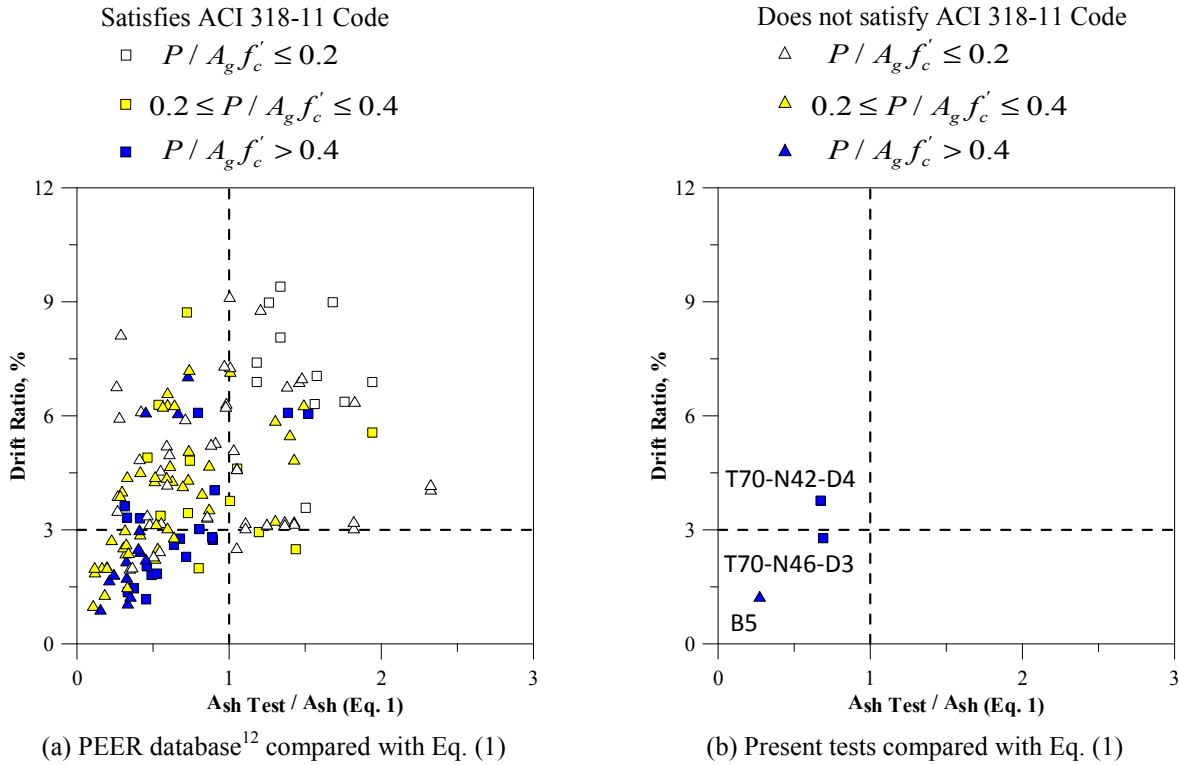
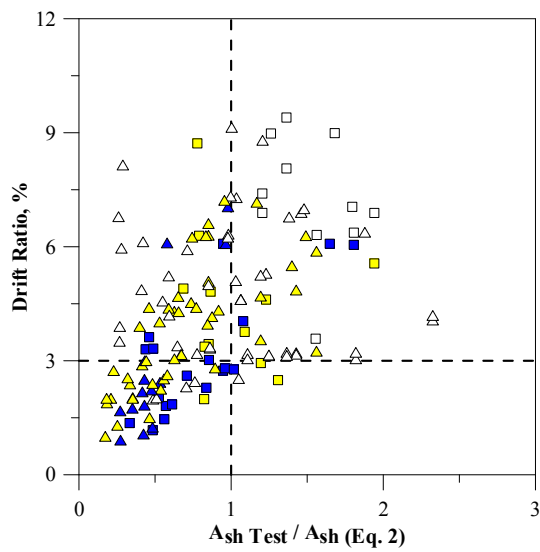
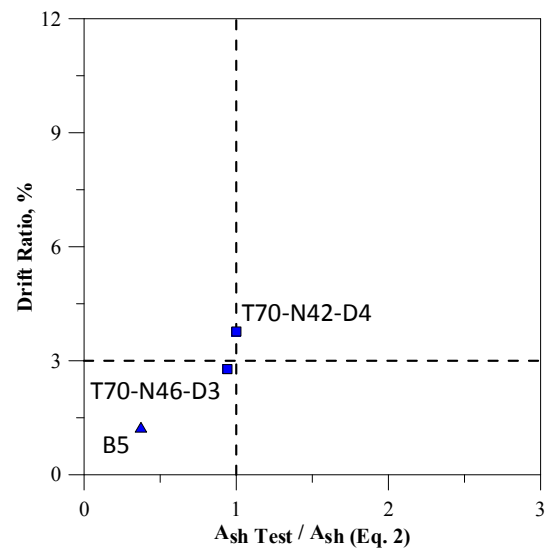


Fig. 10—Comparison of envelopes of hysteresis curves





(c) PEER database<sup>12</sup> compared with Eq. (2)



(d) Present tests compared with Eq. (2)

**Fig. 11—Drift ratio capacity versus confinement provision for tied columns**

**Numerical estimates of the seismic response of building structures  
reinforced with high-strength steel**

*Jeffrey M. Rutenberg and Santiago Pujol*

**SYNOPSIS:** The use of high-strength longitudinal reinforcement—having a specified yield stress between 80 and 120 ksi—in concrete elements has been shown to allow for the use of lower reinforcement ratios leading to reductions in fabrication costs and congestion. This is especially relevant to structures built in seismically active regions in which reinforcement ratios are typically higher than in structures in regions with a lower seismic risk. Recent research initiatives related to the use of high-strength reinforcement have largely been focused on the response of isolated elements instead of the response of building frames.

This paper presents results from a suite of numerical analyses designed to investigate the effects of high-strength longitudinal reinforcement on overall building frame response. Using steel with a higher yield stress allows for reductions in reinforcement ratio. Those reductions, in turn, cause a decrease in post-cracking stiffness. To investigate the effects of this relative softening, a series of multiple-degree-of-freedom models were proportioned to represent idealized frames reinforced with high-strength steel. Nonlinear dynamic analyses were conducted to estimate their response to a set of seven strong-motion accelerograms. It is shown that increases in drift demands related to the use of high-strength steel range from negligible to approximately 20 percent, depending on a number of factors including base shear strength, ground motion intensity, and extent of high-strength steel use. This increase in drift demand 1) is modest compared to the uncertainties associated with predicting ground motion intensities and 2) needs to be confirmed through experiments.

**KEYWORDS:** Nonlinear dynamic analysis; ASTM A1035; steel congestion; seismic response

## BIOGRAPHY

ACI Member **Jeffrey M. Rautenberg** is a project engineer at Wiss, Janney, Elstner Associates in Emeryville, California and is a member of ACI Committee 374. He studied at Purdue University where he received his BSCE, MSCE, and PhD between 2004 and 2011.

ACI Member **Santiago Pujol** is an Associate Professor in Civil Engineering at Purdue University. He is a member of ACI committees 314, 441, and 445.

## INTRODUCTION

The use of reinforcing steel bars with a nominal yield stress of 60 ksi [415 MPa] in concrete structures has been dominant in the United States for a number of decades. With the recent introduction of new standards, specifically ASTM A1035<sup>1</sup> Grades 100 and 120 ksi [690 and 830 MPa], interest in steels with a higher nominal yield stress has increased. Notable advantages of using high-strength steel as reinforcement for concrete members instead of conventional-strength steel are 1) reduced congestion of reinforcing steel, 2) reduced shipping costs, 3) reduced installation costs, and 4) reduced environmental impact.

The amount of research conducted on high-strength steel reinforcement is modest, especially for use in members used to resist earthquake-induced loads. In this paper, we present the results of a numerical study of the effects of using Grade 120 ksi [830 MPa] steel as reinforcement in concrete beams and columns of building frame structures. For the purposes of the analyses presented, the stress-strain relationships for reinforcing steel have been idealized as

$$|f_s| = E_s |\varepsilon_s| \leq f_y$$

for conventional-strength steel, and as

$$|f_s| = E_s |\varepsilon_s| \left[ A + \frac{1 - A}{[1 + (B \cdot |\varepsilon_s|)^C]^{1/C}} \right] \leq f_u$$

for high-strength steel<sup>2</sup> (Figure 1), where:

|                 |   |                                                          |
|-----------------|---|----------------------------------------------------------|
| $f_s$           | : | stress in the steel,                                     |
| $E_s$           | : | steel elastic modulus,                                   |
| $\varepsilon_s$ | : | strain in steel                                          |
| $f_y$           | : | steel yield stress defined using the 0.2%-offset method, |
| $f_u$           | : | steel tensile strength,                                  |
| $A$             | : | 0.0015,                                                  |
| $B$             | : | $\frac{E_s}{1.2 f_y}$ , and                              |
| $C$             | : | 2.6                                                      |

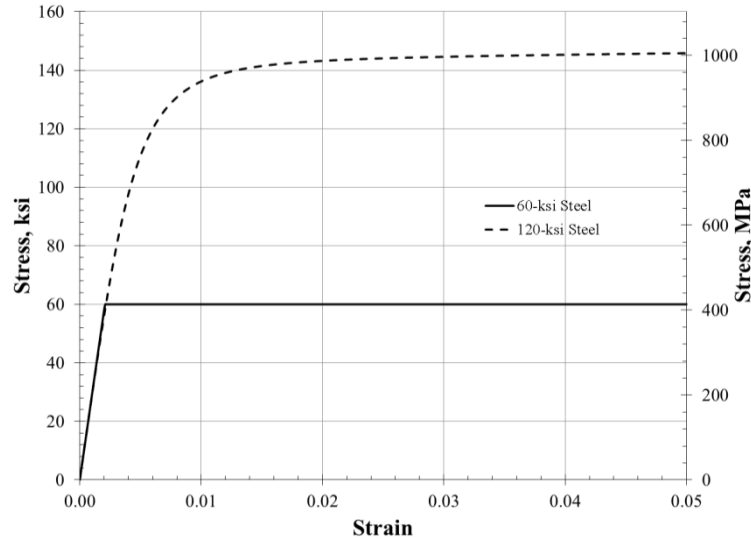


Figure 1—Idealized stress-strain response of ASTM A706<sup>3</sup> Grade 60 and ASTM A1035<sup>1</sup> Grade 120 steels

### EFFECTS OF HIGH-STRENGTH STEEL REINFORCEMENT ON COMPUTED RESPONSE

The use of high-strength steel in concrete elements allows for benefits related to a reduced reinforcement ratio for a given set of demands; however, it begs consideration of the potential consequences, including a possible increased likelihood of bar buckling, a decreased fracture strain of the steel, and—the primary focus of this paper—a reduction in post-cracking flexural stiffness associated with reducing the cross-sectional area of steel in a concrete member.

When subjected to a bending moment less than that required to cause cracking, a concrete element is typically idealized as having a homogenous cross section with linearly varying stress and strain distributions. This is depicted at the top of Figure 2 for a hypothetical beam. The bending stiffness is commonly assumed to be proportional to the moment of inertia of the gross cross section. After the cracking moment of the section is exceeded, the concrete below the neutral axis is ignored, and the resulting stiffness is then thought of as being proportional to the transformed moment of inertia of the cross section, taking into account the distribution and stiffness of the reinforcing steel (bottom of Figure 2). The use of high-strength steel allows for a reduction in the amount of steel required to resist a given moment demand. This reduction in steel leads to a lower post-cracking bending stiffness.

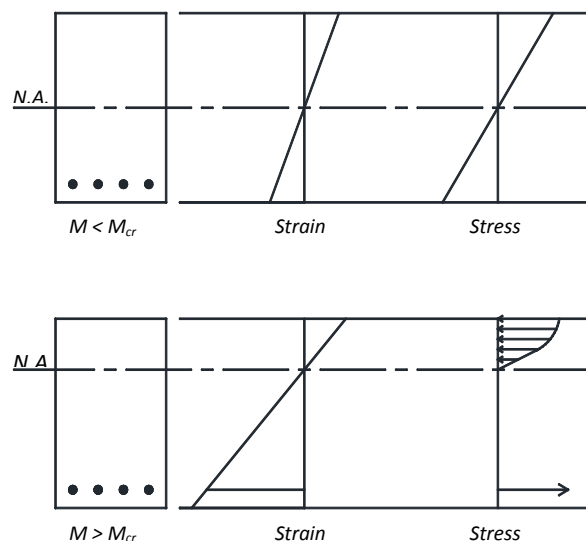


Figure 2—Representation of cross-sectional response to an applied bending moment; applied bending moment less than (top) and greater than (bottom) that required to cause cracking in the extreme tension fiber of the concrete

The reduction in post-cracking stiffness is addressed below from three different vantage points—cross-sectional response, member response, and overall frame response.

### Cross-sectional response

Determining the effects using high-strength steel on the response of a particular cross section is straightforward but is best illustrated through an example. Figure 3 shows two 24-inch- [610-mm-] square cross sections of hypothetical columns—Section A is reinforced with ASTM A706 Grade-60 steel while Section B is reinforced with high-strength ASTM A1035 Grade-120 steel but has half the reinforcement ratio. All assumptions made for analyzing Section A were also made to analyze Section B, with the exception of the stress-strain behavior of the steel reinforcement. The steel in Section A was assumed to follow an elasto-plastic relationship with a yield stress of 60 ksi [415 MPa] (solid line in Figure 1), and the steel in Section B was assumed to follow the dashed curve shown in Figure 1.

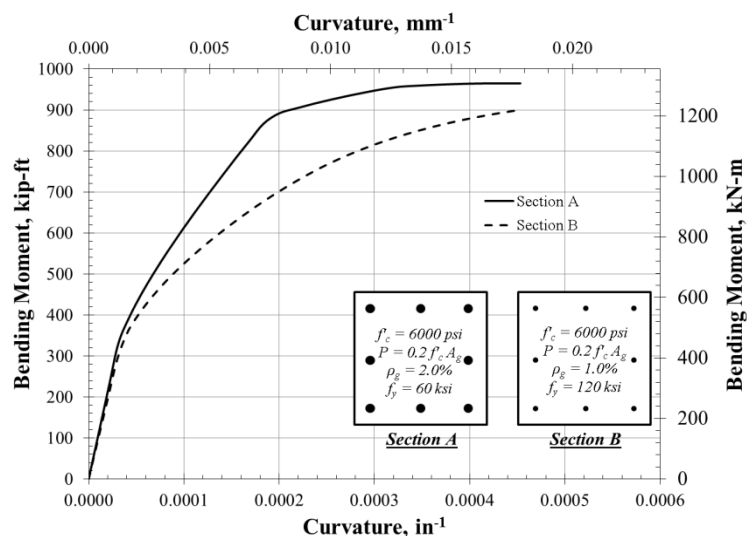


Figure 3—Computed moment-curvature relationships for column sections reinforced with Grade-60 (Section A) and Grade-120 (Section B) longitudinal reinforcement; limiting concrete compressive strain equal to 0.004 for both curves

The moment-curvature response of the two sections, shown in Figure 3, shows what was described previously—after cracking, the section reinforced with conventional-strength steel is stiffer than the section reinforced with (less) high-strength steel. It can also be observed that the ultimate curvature and peak moment capacity of the two sections are nearly identical. Thus, in terms of sectional response, the primary effect of high-strength steel on performance—if reinforcement ratio is reduced proportionally to yield stress—is the reduction in post-cracking stiffness.

### Member response

In the elastic range of response, estimating deformation of a beam or column subjected to a bending moment gradient is typically performed using the gross and/or cracked moments of inertia of the section, depending on the applied bending moment. It can thus be inferred that changing the amount of steel reinforcement and, in turn, changing the cracked moment of inertia of the section, will change the overall member response.

After yielding, estimating the response of the member becomes more difficult. It is often assumed that a so-called plastic hinge forms in the region of highest moment demand. Plastic rotations occur, which lead to additional deflection. Properties ascribed to the plastic hinge have been proposed by many researchers, but they have invariably been tied to experimental data. This presents a problem when trying to estimate post-elastic deflections in a member reinforced with high-strength steel because few research initiatives have been conducted on the subject.

Tavallali<sup>4</sup> and Rautenberg<sup>5</sup> studied the linear and post-linear behavior of beams and columns reinforced with high-strength steel and subjected to cyclic loads. Their observations confirmed that both bending moment capacity and deformation capacity are similar for members reinforced with conventional Grade-60 steel and members reinforced with (less) higher strength reinforcement. The primary difference observed was related to post-cracking stiffness.

For beams, the secant stiffness<sup>1</sup> of a member reinforced with high-strength Grade-97 steel was on average 70 percent of the secant stiffness of a member reinforced with conventional-strength steel. In columns reinforced with Grade-120 steel, that ratio was 50 percent. These observations support the results from the moment-curvature analyses discussed above.

#### Overall frame response

To project the results of moment-curvature analyses and experiments of isolated elements to the case of a complete frame requires further assumptions and analyses. Lepage, et al.<sup>6</sup> conducted nonlinear analyses of single-degree-of-freedom (SDOF) systems to estimate the effects of variations in initial stiffness on drift demand. The results from this investigation suggested that the nonlinear drift demand of SDOF systems with smaller initial stiffness is greater than the nonlinear drift demand of similar SDOF systems with larger initial stiffness. The drift increase is a function of a number of factors, including the ratio of initial stiffnesses, intensity of input motion, input motion frequency content, base shear coefficients, and the ratios of pre-to-post-yield stiffness.

Rautenberg<sup>5</sup> conducted a numerical investigation of multi-story frame buildings in which the columns were modeled as being reinforced with high-strength steel while assuming the beams were reinforced with conventional Grade-60 steel. He found that the resulting increase in drift demand was negligible. The primary reason for this was that the overall stiffness of a frame-type structure is typically governed by the stiffness of the beams more than by the stiffness of the columns. The use of high-strength steel in the columns allowed for a reduced reinforcement ratio, and the use of Grade-60 steel in the beams prevented a critical decrease in global building stiffness.

The effects of using high-strength steel in all members (beams and columns) of a moment-resisting frame are investigated in the following section.

## NUMERICAL SIMULATIONS

#### Model descriptions

Two sets of four idealized building frames were modeled using LARZ<sup>8,9</sup>, a software package for nonlinear dynamic analysis of two-dimensional structural models. One set of models, designated with the letter “C”, represented structures having columns and beams reinforced with conventional-strength (Grade-60) reinforcing steel. The other set of models, designated with the letter “H”, represented structures having columns and beams reinforced with high-strength (Grade-120) reinforcing steel but with approximately half the steel assigned to C models. Each of the two sets of models comprised four individual models—one having three stories, one having six stories, one having twelve stories, and one having twenty stories—for a total of eight analysis models. The models were given names reflecting both the steel grade assumed and the number of stories. For example, model H12 is a model of an idealized twelve-story building with high-strength steel in beams and columns.

Assumptions and analysis parameters used in the models included:

- Columns were square in cross section and had dimensions such that the gravity-induced axial loads were approximately  $0.10f'_cA_g$ , where  $f'_c$  is the nominal concrete compressive strength (5000 psi [34 MPa]) and  $A_g$  is the gross cross-sectional area of the column;
- All columns had cross sections at least 18 inches [460 mm] square;
- Column sizes varied from floor to floor such that the axial load ratio on interior columns was nearly constant, but all columns on a given floor had identical cross-sectional dimensions;
- Columns had overall reinforcement ratios equal to 2.0% for conventional-strength steel and 1.0% for high-strength steel;
- Story height (from top of slab to top of slab) was 12 feet [3.7 m];
- Each frame had four 24-foot [7.3-m] bays; the tributary width to each frame was also 24 feet [7.3 m];
- Beams were modeled with rectangular cross sections 18-inches [460-mm] wide and 24-inches [610-mm] deep;
- Beams had top and bottom reinforcement ratios equal to 1.0% for conventional-strength steel and 0.5% for high-strength steel;
- All buildings weighed 125 pounds per square foot [6.0 kPa];
- An equivalent viscous damping ratio of 5% of critical was assumed;

<sup>1</sup> The slope of a straight line joining the origin and the yield point in the load-deflection curve

- Geometric nonlinearities related to the so-called P- $\Delta$  effect were included in the analyses; and
- Strong-column-weak-beam requirements defined in current building codes were not enforced at all beam-column joints.

A moment-curvature relationship was constructed for each of the beams and columns in the models, and a tri-linear approximation was used as input into LARZ. The three segments of the tri-linear approximation consisted of a pre-cracked portion, in which the gross cross-sectional properties of the members controlled the behavior; a cracked portion, in which the stress in the steel is less than that required to cause yield, but the tensile stress in the concrete exceeds its tensile capacity; and a post-yield portion, in which the stresses in the steel exceed the yield stress. The curvature at yield of structural members assumed to be reinforced with high-strength steel was set to twice the curvature at yield for comparable members reinforced with (twice as much) conventional-strength steel—i.e., the secant stiffness (defined above) was reduced by half in elements reinforced with high-strength steel. The bending moment capacity at ultimate was assumed to be 1.2 times the bending moment associated with yield for all members. The tri-linear relationships input into LARZ were used with the concrete model proposed by Takeda et al.<sup>10</sup> (Figure 4) to represent hysteretic response, in which, the value  $\alpha$  was set equal to 0.4, as proposed by the Takeda et al., for all members.

In addition to member displacements and rotations related to the moment-curvature distribution over a given member's length, additional rotation was allowed at each joint to account for slip between concrete and steel in joints. The available test data<sup>4,5</sup> were collected from specimens in which the configuration of the test setup resulted in slip of the reinforcement that does not necessarily correlate well with the slip likely to occur in a moment-resisting frame. So in order to simplify the analyses and reduce the number of variables, the amount of additional rotation related to slip was set to 0.01 at ultimate for all members, regardless of the yield stress assumed for the reinforcing steel. The additional flexibility of members related to the reduction in reinforcement ratio was captured in the models with the different stiffnesses in the moment-curvature diagrams.

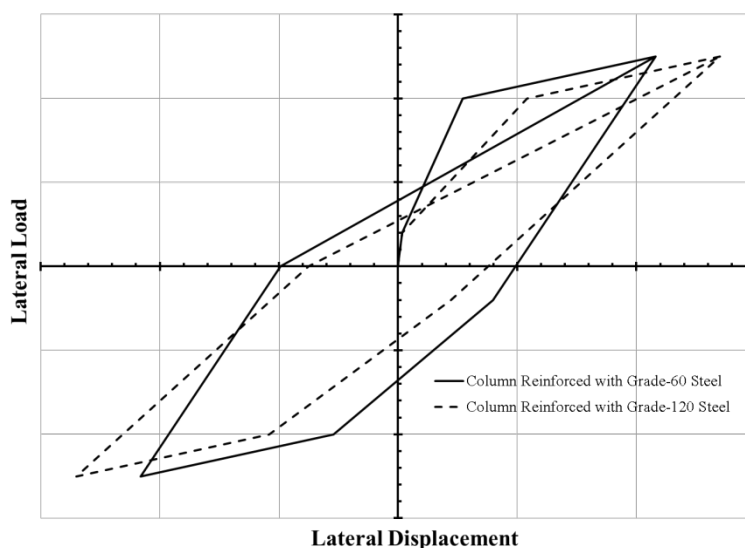


Figure 4—Hypothetical response of columns with different post-cracking stiffnesses assuming hysteretic behavior proposed by Takeda et al.<sup>10</sup>

Table 1 shows the periods of each structure based on gross cross-sectional properties and the computed base shear strength coefficients, assuming an applied lateral load increasing linearly from the base.

Table 1—Computed model periods and base shear strength coefficients

| Model | Fundamental Period<br>s | Base Shear Strength Coefficient |
|-------|-------------------------|---------------------------------|
| C03   | 0.30                    | 0.33                            |
| H03   |                         | 0.32                            |
| C06   | 0.48                    | 0.23                            |
| H06   |                         | 0.22                            |
| C12   | 0.87                    | 0.14                            |
| H12   |                         | 0.13                            |
| C20   | 1.29                    | 0.082                           |
| H20   |                         | 0.077                           |

#### Input ground motions

The response of each model was estimated for the series of seven un-scaled ground motions shown in Table 2<sup>7</sup> for a total of fifty-six analyses. Figure 5 shows the computed spectral acceleration versus spectral displacement for each motion. These motions were chosen to encompass a wide range of intensities, frequency contents, epicentral distances, and soil conditions.

Table 2—Summary of input motions to numerical simulations

| Earthquake              | Date           | Reported Magnitude | Epicentral Distance<br><br>mi<br>[km] | Direction | Reported PGA<br><br>g | Reported PGV<br><br>in/s<br>[cm/s] | Reported PGD<br><br>in<br>[cm] |
|-------------------------|----------------|--------------------|---------------------------------------|-----------|-----------------------|------------------------------------|--------------------------------|
| El Centro California    | May 18, 1940   | 6.9                | 7.6<br>[12.2]                         | NS        | 0.35                  | 13.2<br>[33.5]                     | 4.3<br>[10.9]                  |
| San Fernando California | Feb. 9, 1971   | 6.6                | 2.2<br>[3.5]                          | S74W      | 1.08                  | 22.7<br>[47.7]                     | 4.3<br>[10.9]                  |
| Mexico City Mexico      | Sept. 18, 1985 | 8.1                | 23.8<br>[38.3]                        | NS        | 0.14                  | -- <sup>†</sup>                    | -- <sup>†</sup>                |
| Loma Prieta California  | Oct. 17, 1989  | 7.0                | 1.7<br>[2.7]                          | EW        | 0.48                  | 18.7<br>[47.5]                     | 4.5<br>[11.4]                  |
| Northridge California   | Jan. 17, 1994  | 6.7                | 8.0<br>[12.9]                         | NS        | 0.45                  | 24.1<br>[61.2]                     | 7.0<br>[17.8]                  |
| Kobe Japan              | Jan. 16, 1995  | 6.9                | 0.6<br>[1.0]                          | EW        | 0.60                  | -- <sup>†</sup>                    | -- <sup>†</sup>                |
| Duzce Turkey            | Nov. 12, 1999  | 7.1                | 10.9<br>[17.5]                        | NS        | 0.73                  | -- <sup>†</sup>                    | -- <sup>†</sup>                |

<sup>†</sup>No value reported

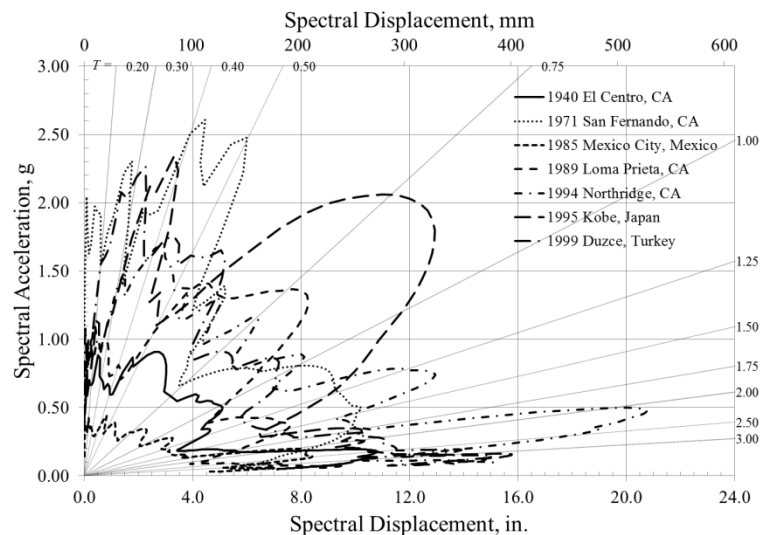


Figure 5—Linear spectral displacement vs. spectral acceleration plots for all input ground motions assuming a 5% equivalent viscous damping ratio

## RESULTS AND DISCUSSION

In each of the fifty-six analysis runs, the peak computed roof displacement was normalized by the assumed height of the structure to get a peak mean drift ratio. The effect of using high-strength steel was then evaluated by comparing peak mean drift ratios from *C* models to peak mean roof drift ratios from *H* models (Figure 6).

Points lying on the solid line in Figure 6 represent cases in which the peak mean drift ratio in a *C* model was the same as that in the comparable *H* model. Points above that line represent cases in which the peak drift ratio in an *H* model was larger than in the comparable *C* model. Points below the line represent the opposite case. The slope of the dashed line—1.15—corresponds to the average ratio of peak drifts for *H* models to *C* models for all analysis runs.

We refer to the ratio of peak drift in an *H* model to peak drift in a *C* model as the “high-strength steel penalty.” For the nonlinear multiple-degree-of-freedom systems cases investigated here, that penalty amounted to 15 percent (on average). To put this figure in context, consider that the maximum difference in drift between two linear oscillators with short periods (i.e. responding in the constant-acceleration region of a typical idealized response spectrum), one of which is half as stiff as the other, is likely to be  $(\sqrt{2} - 1) \cong 40\%$ .

The results show that as the number of stories in the model is increased, the “high-strength steel penalty” decreases. Specifically, the penalty for 3-story building models was 1.3 but was 1.0 for 20-story building models. The primary reason for this is that as the number of stories increases, so does the number of structural elements participating in the lateral load resistance. When more structural elements are participating, a relatively lower percent of those members experience cracking and yielding than in structures with fewer members. This allows the structure to respond at an effective period that is shorter than that corresponding to the structure with all members cracked and/or yielding.

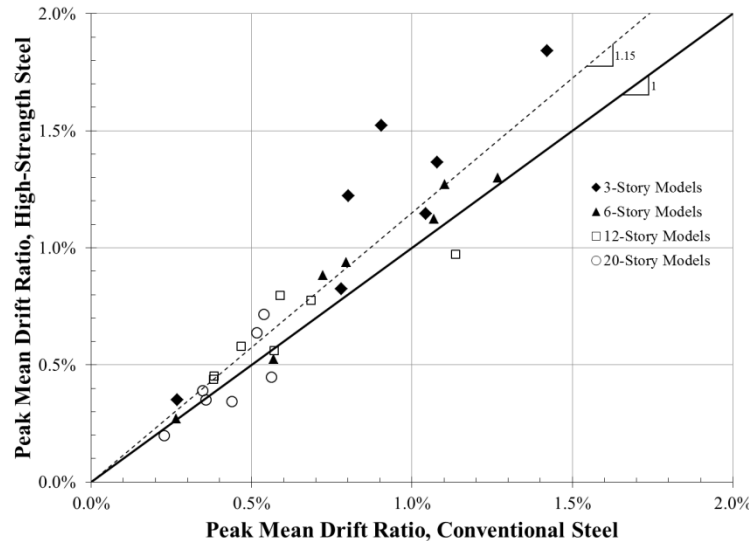


Figure 6—Comparison of peak computed roof drift ratios for C and H models

The results obtained in this study (Figure 6) lie between results from similar studies done by Lepage et al.<sup>6</sup> for SDOF systems and by Rutenberg<sup>5</sup> for models of idealized multiple-degree-of-freedom systems (representing frames with columns reinforced with high-strength steel and beams reinforced with conventional Grade-60 steel). In the former, Lepage estimated the high-strength steel penalty to be approximately 28 percent (on average), and in the latter, Rutenberg estimated the high-strength steel penalty to be approximately 3 percent (on average).

### CONCLUSIONS

The results from a series of fifty-six nonlinear time history analyses were used to evaluate the effect of using high-strength steel in multi-story frame structures on computed peak roof displacements. Within the scope of the variables tested, the following conclusions are made:

- 1) The mean peak roof displacements computed for models assumed to have members reinforced with high-strength steel was on average 1.15 times the mean peak roof displacements computed for comparable models assumed to have members reinforced with (twice as much) conventional-strength steel.
- 2) The computed effect of using high-strength steel on the peak roof displacement demands decreased as the number of stories—and number of structural members—increased.
- 3) The results obtained need to be tested against tangible physical evidence.

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## High-Strength Concrete Columns Confined with Spirals

Lonnie A. Marvel and Riyadh A. Hindi

**Abstract:** This paper experimentally investigates the behavior of high-strength reinforced concrete columns confined using a new cross spiral confinement technique. The new cross spiral confinement technique uses two opposing spirals to confine circular concrete columns enhancing their strength and ductility, and increasing spiral spacing to facilitate the flow of fresh concrete. The new confinement arrangement is experimentally evaluated and compared to the conventional single spiral confinement arrangement. Twenty-one circular high-strength reinforced concrete columns with four different spiral spacings and longitudinal reinforcement ratios were tested under monotonic axial loading. Seven specimens utilized the conventional single spiral confinement, used as control specimens, while the remaining specimens utilized the new cross spiral arrangement. The new arrangement enables an increase in spiral spacing while maintaining the same volumetric confinement ratio as the conventional. Alternatively, doubling the volumetric confinement ratio without violating ACI 318-08<sup>1</sup> requirement for minimum spiral spacing. The study showed that the new cross spiral arrangement with the same volumetric confinement ratio as the conventional spiral obtained similar ultimate stress values while it attained about a twenty percent increase in ultimate displacement. The cross spiral confinement using twice the volumetric confinement ratio greatly outperformed the conventional spiral in all aspects.

**Keywords:** axial loading; cross spirals; ductility; concrete, high-strength; spiral reinforcement

**Biography:** **Lonnie A. Marvel** earned a BSCE degree in 2006 and a MSCE in 2008 from Bradley University, Peoria, IL. He is a structural engineer in Peoria, IL.

**Riyadh A. Hindi** is a Professor of Civil Engineering at Saint Louis University in Saint Louis, MO. He is a member of ACI committees 341, Earthquake Resistant Concrete Bridges; 342 (secretary), Evaluation of Concrete Bridges and Bridge Elements; 441, Concrete Columns; and Joint ACI-ASCE Committee 343, Concrete Design. His research interests include behavior of reinforced concrete, seismic behavior of reinforced concrete bridges, damage analysis and experimental testing of reinforced concrete elements.

### INTRODUCTION

The lateral expansion of an axially loaded concrete column is resisted by lateral confinement, imposing confining stresses on the concrete. The state of stress created by lateral confinement delays the crushing of concrete thus enhancing the strength and ductility of the column. The effectiveness of the lateral resistance is determined based on the method of confinement used. Spirals have proven to be the most effective means of confinement.

Spiral columns, constructed according to the ACI 318-08 concrete building code<sup>1</sup>, have sufficiently close spirals to place the concrete in a tri-axial state of stress. Concrete under tri-axial compression becomes stronger and has higher ductility than concrete under uni-axial compression. The ACI 318 building code<sup>1</sup> specifies minimum spiral spacing of 25 mm (1 in.) for all reinforced concrete columns. This minimum limit is to ensure constructability; concrete will flow freely and bond to all the reinforcing steel. This minimum spacing for spirals may limit the ductility in conventionally confined columns. This is relevant for structures located in high seismic regions where ductility is essential. The ACI 318<sup>1</sup> standards describe a column that may not have enough confining reinforcement.

The use of high-strength concrete has been hindered by the space limitation of ACI 318<sup>1</sup>. The strength gain of high-strength concrete comes at a cost in a loss of ductility. High-strength concrete is more brittle compared to normal strength concrete. This brittleness requires more lateral confinement in order for the column to develop adequate ductility. This has greatly limited the use of high strength concrete in areas where ductility is needed.

Seismic design relies heavily on ductility. Limitations imposed on the spiral spacing have their effects on the design of structures in seismic regions. The stringent guidelines placed on the spacing of spirals and the leveling of the seismic design codes across all regions have increased the need for highly ductile concrete, especially high-strength concrete, without compromising constructability. These limitations create a role for a more efficient means of confinement, such as that proposed, to meet the demands of the column without creating unnecessary congestion.

Hindi<sup>2</sup> proposed the use of two opposing cross spirals in lieu of the conventional single spiral. The spacing of the cross spiral could be manipulated to either increase the behavioral characteristics of the column or

constructability. Where high ductility is needed, the cross spirals could be used each with a spacing of  $S$ , thus effectively doubling the volumetric lateral confinement ratio without violating the minimum spacing limitation. In areas where constructability is an issue, using a cross spiral each with a spiral spacing of  $2S$  could be used. The net effect would maintain the conventional single spiral volumetric confinement ratio. This would enhance the constructability without altering the load carrying characteristics of the column. Piles would also benefit from an increased spacing which would allow more space for spiral slippage and movement without affecting the confining capability of the spiral. In addition, heavily reinforced beam-column connections would become less congested, thus, more easily constructed if the spiral spacing could be increased without affecting the load carrying capacity of the beam-column.

Various methods of lateral confinement have been tested: hoops, welded wire mesh, high-strength strands, and spirals<sup>3-6</sup>. These tests were conducted for various reasons: to verify code adequacy, to develop new methods and ultimately, to increase the strength and ductility of normal and high-strength concrete columns.

Hindi et al.<sup>7</sup> tested the new cross spiral confinement. Small scale reinforced normal weight concrete specimens were tested under pure axial compression. The experimental results revealed, when comparing the conventional single spiral confined columns to the cross spiral columns with the same amount of volumetric confinement ratios, similar behavior in strength and ductility between the specimens. In addition, the cross spiral confined specimens with the same spiral spacing as the conventional single spiral specimen, twice the volumetric confinement ratio, displayed a slight increase in strength and significantly higher ductility.

The objective of this paper is to experimentally investigate the axial behavior of high-strength concrete bridge columns using the cross spirals confinement and to compare the results to the conventional single spiral confinement in accordance with the ACI 318-08 building code<sup>1</sup>. In addition, the experimental results were compared to three different confinement models: Mander et al.<sup>8</sup>, Bing et al.<sup>9</sup> and Razvi et al.<sup>10</sup>, to determine if a new confinement model is warranted for the new cross spiral confinement.

### RESEARCH SIGNIFICANCE

This paper experimentally investigates the axial behavior of high-strength concrete confined with a new cross spiral arrangement. It's demonstrated that the proposed new confinement arrangement offers the possibility to confine high-strength concrete columns to achieve greater strength and strain capacity compared to conventional single spiral confinement. In addition, the cross spiral confinement may be used to allow for greater spiral spacing without compromising strength and strain capacity.

### EXPERIMENTAL PROGRAM

Twenty-one reduced-scale reinforced high-strength concrete circular columns with four different longitudinal reinforcement ratios and four different confinement reinforcement ratios were built and tested. The columns were subjected to monotonic axial loading to study the influence of the new confinement on both the axial strength and vertical displacement. This was to support a hypothesis regarding use of reinforced high-strength concrete bridge columns confined with the new cross spiral confinement as compared to the conventional single spiral confinement. The hypothesis was that the new cross spiral confinement technique with a spacing of  $2S$  would perform similar to the conventional single spiral technique. In addition, cross spirals with a spacing of  $S$  would significantly enhance the strength and deformability of reinforced concrete columns as compared to the conventional single spiral confinement.

#### Specimen details

The columns specimens considered in this research were designed and built based on ACI 318<sup>1</sup> specifications. To test the proposed confinement configurations, seven of the twenty-one specimens were constructed using the conventional single spiral and were used as control specimens. Figure 1 shows a comparison between the conventional single spiral configuration and the cross spiral configuration. The remaining fourteen columns were constructed using the cross spiral confinement. All specimens had the same overall dimensions with the only variations being the longitudinal reinforcement ratios and spiral spacings. All specimens had diameters of 350 mm (13.78 in.) and lengths of 1000 mm (40in.). Either 8, 10, 12, or 14 -  $\Phi 15.9$  mm, No. 16 ( $\Phi 0.625$  in, No. 5) longitudinal reinforcing bars were used and evenly spaced over a diameter of 250 mm (9.84 in.), giving longitudinal steel ratios of 0.016, 0.020, 0.024 and 0.028. A typical concrete bridge column is practically reinforced with a longitudinal steel ratio of 0.02. Therefore, the specimens tested cover the reinforcement range used in practical design. The confining spiral used was  $\Phi 9.5$ mm, No. 10 ( $\Phi 0.375$  in, No. 3) with four base spiral spacings 40, 50, 55, or 60 mm (1.58, 2.0, 2.17 or 2.36 in.), covering the allowable spiral spacing set forth in ACI-318<sup>1</sup>. Table 1 summarizes the specimens' details.

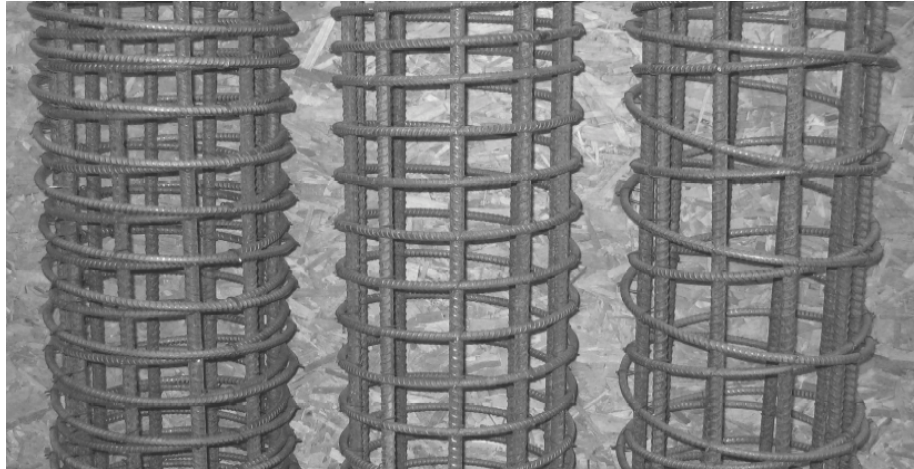


Fig. 1—Specimen Comparison, Cross Spiral Spacing of S, Single Spiral Spacing of S, Cross Spiral Spacing of 2S

Table 1—Specimen Details

| Specimen                                                                       | Spiral Type | Effective Length, mm (in.) | Spiral Spacing, mm (in.) | $\rho_l$ | $\rho_s$ | Avg. 28 Day $f'_c$ , MPa (ksi) |
|--------------------------------------------------------------------------------|-------------|----------------------------|--------------------------|----------|----------|--------------------------------|
| 50S-8                                                                          | Regular     | 1000 (40)                  | 50 (2)                   | 0.016    | 0.021    | 67.3 (9.76)                    |
| 100X-8                                                                         | Cross       | 1000 (40)                  | 100 (4)                  | 0.016    | 0.020    | 67.3 (9.76)                    |
| 50X-8                                                                          | Cross       | 1000 (40)                  | 50 (2)                   | 0.016    | 0.040    | 67.3 (9.76)                    |
| 40S-10                                                                         | Regular     | 1000 (40)                  | 40 (1.6)                 | 0.020    | 0.026    | 67.3 (9.76)                    |
| 80X-10                                                                         | Cross       | 1000 (40)                  | 80 (3.2)                 | 0.020    | 0.026    | 67.3 (9.76)                    |
| 40X-10                                                                         | Cross       | 1000 (40)                  | 40 (1.6)                 | 0.020    | 0.052    | 67.3 (9.76)                    |
| 50S-10                                                                         | Regular     | 1000 (40)                  | 50 (2)                   | 0.020    | 0.021    | 67.3 (9.76)                    |
| 100X-10                                                                        | Cross       | 1000 (40)                  | 100 (4)                  | 0.020    | 0.021    | 67.3 (9.76)                    |
| *50X-10                                                                        | Cross       | 1000 (40)                  | 50 (2)                   | 0.020    | 0.041    | 63.2 (9.17)                    |
| 55S-10                                                                         | Regular     | 1000 (40)                  | 55 (2.2)                 | 0.020    | 0.019    | 67.3 (9.76)                    |
| 110X-10                                                                        | Cross       | 1000 (40)                  | 110 (4.4)                | 0.020    | 0.019    | 67.3 (9.76)                    |
| 55X-10                                                                         | Cross       | 1000 (40)                  | 55 (2.2)                 | 0.020    | 0.038    | 67.3 (9.76)                    |
| 60S-10                                                                         | Regular     | 1000 (40)                  | 60 (2.4)                 | 0.020    | 0.017    | 67.3 (9.76)                    |
| 120X-10                                                                        | Cross       | 1000 (40)                  | 120 (4.8)                | 0.020    | 0.017    | 67.3 (9.76)                    |
| 60X-10                                                                         | Cross       | 1000 (40)                  | 60 (2.4)                 | 0.020    | 0.034    | 67.3 (9.76)                    |
| 50S-12                                                                         | Regular     | 1000 (40)                  | 50 (2)                   | 0.024    | 0.021    | 67.3 (9.76)                    |
| 100X-12                                                                        | Cross       | 1000 (40)                  | 100 (4)                  | 0.024    | 0.020    | 67.3 (9.76)                    |
| *50X-12                                                                        | Cross       | 1000 (40)                  | 50 (2)                   | 0.024    | 0.040    | 63.2 (9.17)                    |
| 50S-14                                                                         | Regular     | 1000 (40)                  | 50 (2)                   | 0.028    | 0.021    | 67.3 (9.76)                    |
| 100X-14                                                                        | Cross       | 1000 (40)                  | 100 (4)                  | 0.028    | 0.020    | 67.3 (9.76)                    |
| 50X-14                                                                         | Cross       | 1000 (40)                  | 50 (2)                   | 0.028    | 0.040    | 67.3 (9.76)                    |
| *indicates specimens with a 28 day compressive strength of 63.2 MPa (9.76 ksi) |             |                            |                          |          |          |                                |

The columns were grouped based on the spacing of the conventional spiral spacing. For each comparison group a cross spiral with the same spiral spacing as the conventional column and a cross spiral with twice the spiral spacing was built and tested. This comparison group was created to examine the axial effects of the cross spiral configurations as compared to conventional single spiral construction.

To force failure to occur in the middle portion of the specimens, the spiral spacing at the ends of the column (0.25L) were spaced at 0.7 times the spiral spacing stated within the middle portion of the column (0.5L) as shown in Figure 2. With the only variation between the columns being the spiral configuration and the spacing, the cross spirals columns had less concrete cover compared to the single spiral columns, maintaining a constant diameter of the longitudinal reinforcement for all columns. Since the core area is calculated based on the diameter of the centroid of the confining spiral bars, the core area also varied between cross spiral columns as compared to single spiral columns. These variations were considered in the results of the experiment. The columns had an aspect ratio ( $d/L$ ) of 0.35.

The longitudinal reinforcing bars used in this study were standard  $\Phi 15.9$  mm, No. 16 ( $\Phi 0.625$  in, No. 5) Grade 60 deformed bars. The spiral reinforcing bars used were standard  $\Phi 9.5$  mm, No. 10 ( $\Phi 0.325$  in, No. 3) Grade 60 deformed bars. Both rebar sizes were tested under tension to obtain the stress-strain relationships. The longitudinal reinforcing bars had actual yield strength of 465 MPa (67.4 ksi) and the spiral reinforcing bars had actual yield strength of 467 MPa (67.7 ksi). Figure 2 is a schematic of the test specimens.

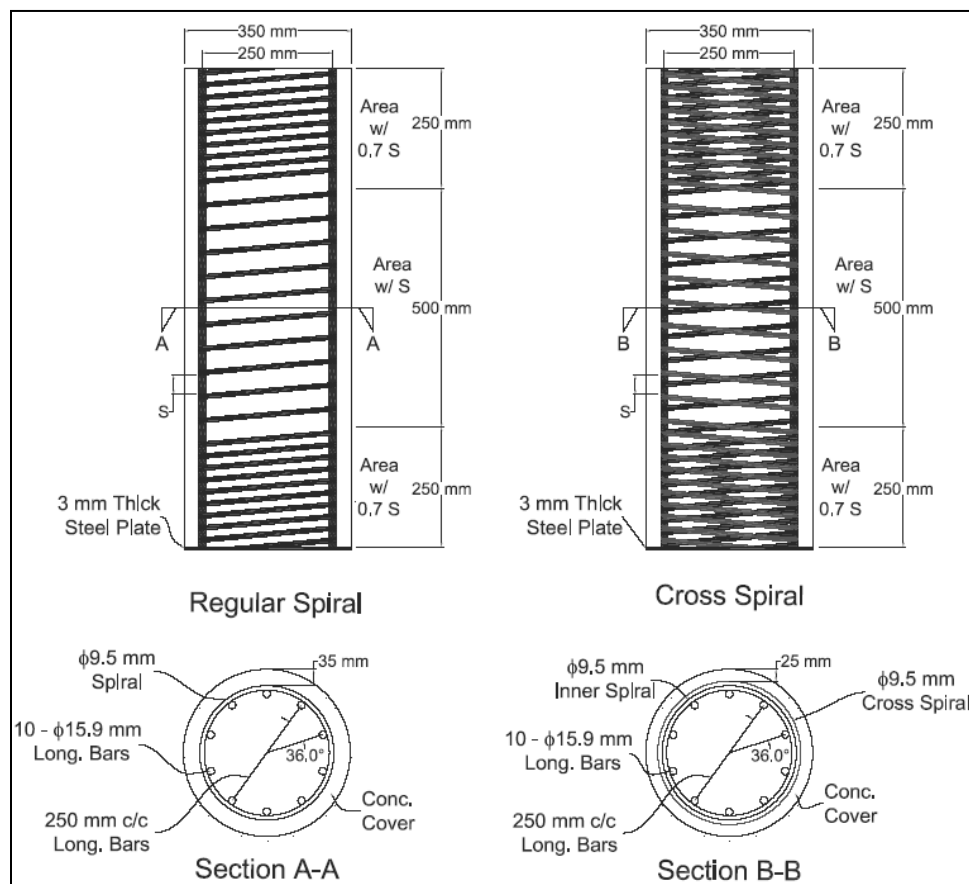


Fig. 2—Schematic of Test Specimen

The bridge columns were cast in two separate pours. A ready-mix concrete with maximum aggregate size of 19 mm (3/4 in.) was used. The specified concrete strength,  $f'_c$  was 70 MPa (10 ksi). The actual 28-day compressive strength for the first pour was measured at 67.3 MPa (9.76 ksi) and 63.2 MPa (9.12 ksi) for the second pour. Six standard cylinders were cast with the first pour and 4 cylinders were cast with the second pour. In addition, nineteen of the columns were cast with the first pour and two were cast with the second pour. The stress strain relation of the concrete was determined from Chang and Mander<sup>11</sup>. To distinguish between the 21 specimens a labeling scheme was created where the number at the beginning of the specimen title designates the spiral spacing,

the letter following represents the spiral configuration, S for a single spiral and X for a cross spiral, the final number at the end designates the number of longitudinal bars present within the specimen. Table 1 displays the properties of each individual column.

### Test setup and loading

Each column was instrumented with three strain gauges. One gauge was placed on the longitudinal bars in the direction of loading. The gauges were placed in the vertical center of the column 500 mm (19.7 in.) from either end. These gauges were placed to read the strain of the longitudinal rebar in the plastic hinge region. The other two gauges were placed on the spiral in the middle of the column, 500 mm (19.7 in.) from either end. In the case of the single spiral bridge columns the two strain gauges were placed 180 degrees from each other. For the cross spirals one strain gauge was placed on the inner spiral and the other on the cross spiral 180 degrees away.

Two linear potentiometers were fitted to the column, one to measure the vertical displacement of the inner quarter of the column, and the other to measure the vertical displacement of the inner half of the column as shown in Figure 3. The potentiometers were placed 180 degrees away from each other. Both potentiometers were used to measure vertical deformation in the column and to ensure bending moment was not induced during loading. In addition, vertical deformation of the testing machine ram head was recorded to obtain overall displacement of the columns. Figure 3 is an illustration of the test setup.

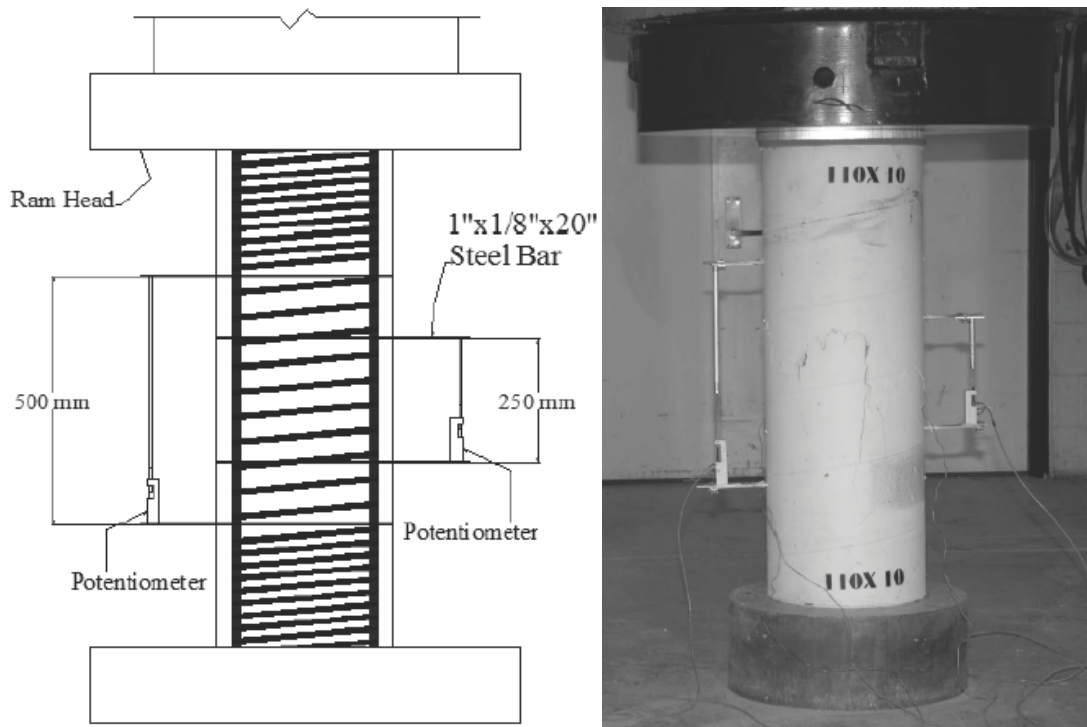


Fig. 3—Test Setup

The loading of the columns was monotonic axial loading. A constant loading rate was maintained until complete failure of each column was reached. The base of the column was centered under the loading ram. Then the two potentiometers and the three strain gauges were electronically attached to the data acquisition center. Readings of the instruments were recorded every one-second during the testing.

### RESULTS AND COMPARISONS

The experiment was designed such that various comparisons could be made between the characteristics of traditional versus cross spiral confinement arrangements. The specimens were grouped by various means. The base comparison group consisted of one traditional single spiral column with a spiral spacing of S used as a control specimen, a cross spiral with a spacing of 2S and a cross spiral with a spacing of S. This base comparison with the control specimen was made for all the specimens. The base comparison group enables a comparison of the new cross spiral configuration as compared to the conventional single spiral configuration. In this comparison group a

comparison is made between the conventional single spiral column and a cross spiral column with the same amount of volumetric confinement ratio while increasing the spiral spacing; in addition to making a comparison between the conventional spiral configuration to a cross spiral with twice the confining volumetric reinforcement ratio while maintaining the same spiral spacing. In order to study the effect of various spiral spacings a grouping was created based on the traditional spiral spacing,  $S$ , with various spiral spacings, while maintaining the longitudinal reinforcement ratio. The spacing for this group was 40, 50, 55, and 60 mm (1.58, 2.0, 2.17 and 2.36 in.) with a longitudinal reinforcement ratio of 0.02. In order to study the effect that the longitudinal reinforcement ratio had on the confined behavior a grouping was created with various longitudinal reinforcement ratios while maintaining the transverse reinforcement ratio. The longitudinal reinforcement ratios in this grouping were 0.016, 0.02, 0.022 and 0.024 with a constant spiral spacing of 50 mm (2 in.).

The stress in the core had to be calculated by assigning a portion of the total force to the core only. The total axial force carried by the column consists of various components; concrete cover, longitudinal reinforcement, and the concrete core. In the analysis it was assumed that all components experience the same axial strain. From this assumption, the axial force in the core was calculated by subtracting the axial force of the longitudinal reinforcement and the concrete cover from the total axial force at every strain value. The core axial force was then divided by the core area to calculate the core axial stress. These calculations were made using the following equations:

$$P_{\text{core}} = P - P_{\text{steel}} - P_{\text{cover}} \quad (1.)$$

$$\sigma_{\text{core}} = \frac{P_{\text{core}}}{A_{\text{core}}} \quad (2.)$$

Where  $P$  = total axial load,  $P_{\text{cover}}$  = axial load carried by concrete cover,  $P_{\text{core}}$  = axial load carried by concrete core,  $P_{\text{steel}}$  = axial load carried by longitudinal reinforcing bars,  $\sigma_{\text{core}}$  = core axial, and  $A_{\text{core}}$  = Area of concrete core enclosed by the centerline of the spiral(s).

### Damage progression

The specimens displayed very similar damage progression and failure modes. Strain was measured based on axial displacement of the linear potentiometers and the overall displacement of the ram head. All of the bridge columns displayed visible signs of spalling at an axial strain of about 0.002. When the axial strain reached about 0.003 the concrete cover displayed severe spalling. All of the bridge columns failed due to rupture of the spiral(s) which led to buckling of longitudinal reinforcement and ultimately failure of the concrete core. The specimens displayed the common shear fracture of columns under axial compression. Figure 4 is a typical specimen at failure.



Fig. 4—Typical Specimen at Failure