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

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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

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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 ±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 (Max.Velocity)	Maximum Story Drift (%)	Ductility of Story
Service Limit State	Level 1 (25cm/s)	≦ 0.5 %	≦ 1.0
Design Limit State	Level 2 (50cm/s)	≦ 1.0 %	≦ 2.0
Ultimate State	Collapse or brittle failure of structural elements should be prevented.		

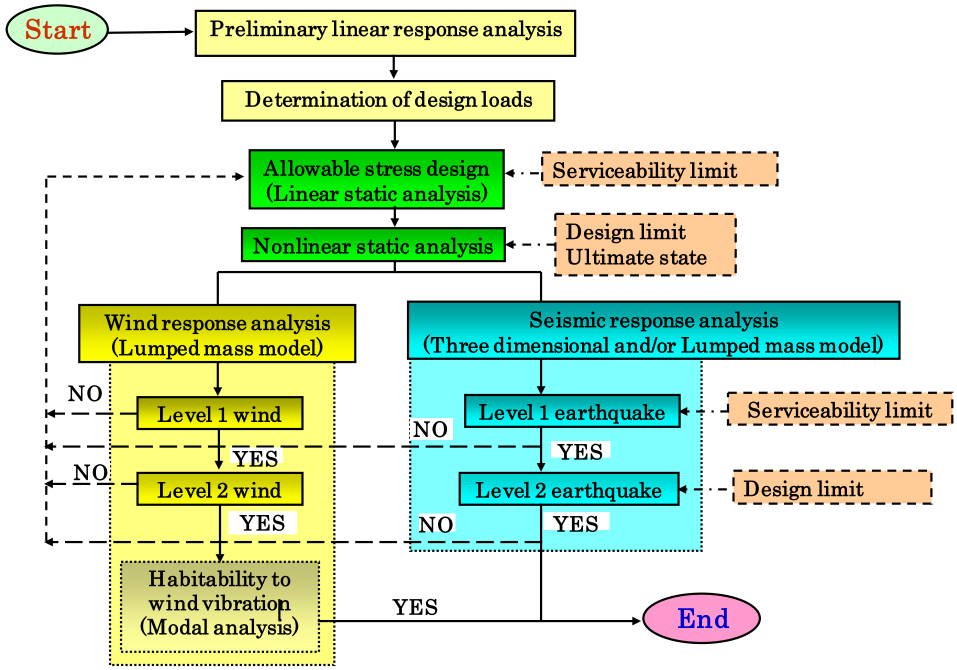

Figure1 –Typical Structural Design Procedure of High-rise RC Building

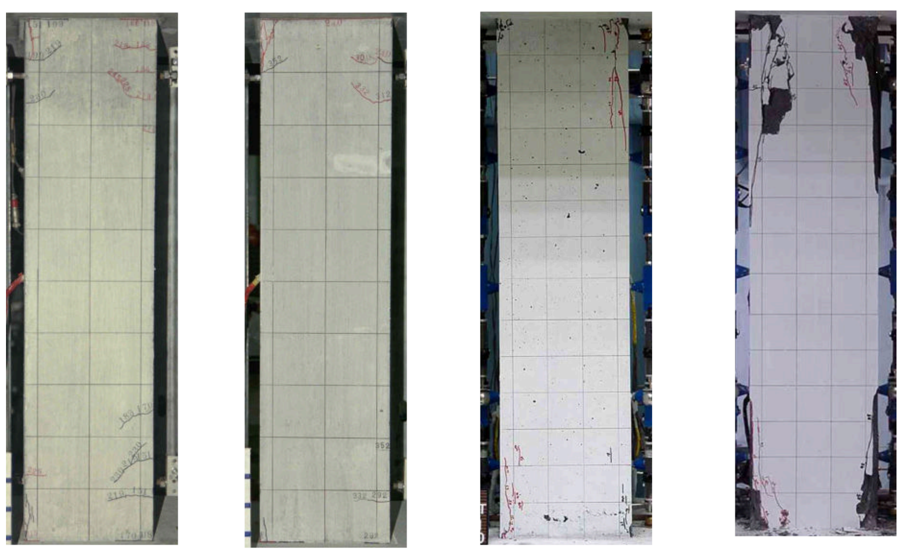

Figure2–Spalling of cover concrete for different concrete strengths at R=0.005 (axial load ratio =0.55~0.6)