Technical report

Use of High-Strength Bars for the Seismic Performance of High-Strength Concrete Columns

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Abstract

To accommodate the ever increasing height of high-rise buildings and required large column-spacing, and assure a high structural performance to supporting elements, use of high-strength materials is sought as a solution. Accordingly, an experimental investigation of the effectiveness of very-high-strength steel bars in improving the performance of ultra-high-strength concrete columns is described. The concrete of 171 MPa strength contained steel fibers. Two grades of high-strength steel bars were used for longitudinal bars in columns. While the SD685-grade is already used in practice, the SD980-grade is still under development. The 1/4 scaled columns were subjected to high levels of compression and tension, and to cyclic lateral loads with an anti-symmetric double curvature bending. The tested columns proved to be ductile and showed good performances. The maximum recorded lateral strength values were at least 30% and 10% greater than those obtained by using ACI and AIJ equations, respectively. The advantage of using steel fibers was apparent by the limited and narrow cracks even at large lateral drifts. It is also suspected to have an impact on preventing buckling of the longitudinal reinforcement. The SD980-grade bars were found very effective in terms of tension axial strength, delay of crack evolution and shear strength degradation, and to be slightly less effective than the SD685 grade bars in terms of shear strength and energy dissipation.

1. Introduction

The crucial need of high-strength materials (HSM) to achieve high structural performance has been the consequence of the ever-increasing need for high-rise buildings in order to accommodate the vertical expansion of metropolitan cities due to over-inflated prices of land, while technically overcoming the anticipated level of induced loads due to diverse causes. For example, the varying axial load that may be experienced by external columns during a severe earthquake could reach, besides a high axial compression level, a very high axial tension level. Such high level, in case of reinforced concrete (RC) buildings can only be resisted safely by appropriate high-strength reinforcement.

The historical progress and technical characteristics of HSM, particularly those used in reinforced concrete were well illustrated in different previous works (Okamura *et al.* 2005; Schmidt and Fehling 2005; Mertol *et al.* 2005). The recorded limited use of HSM in RC structures has been mainly due to the fact that they are not covered by current regulations, which is the result of

the lack of research data rather than the inability of the materials to perform their intended function. The social demand for the use of HSM in earthquake prone areas has become significant, following recent catastrophic events, in order to reduce the life cycle cost of high-rise buildings. As a result, the use of HSM is increasing in RC structures leading to the need for extension of relevant design regulations to include the use of these materials in seismic regions.

To reach that purpose, analytical and experimental works investigating the behavior of high strength concrete (HSC) columns have been carried out. Konstantinidis (2005) made an analytical study on the seismic response of twenty-two 15-story buildings assuming different element/material properties, where HSS/HYS reinforcements were used in all elements except in beams (normal yield). It was concluded that the benefit of utilizing high strength/yield steel (HSS/HYS) as transverse reinforcement was not fully exploited for the beams due to the code requirements for the maximum spacing inside the critical regions, while utilizing HYS as longitudinal reinforcement in the columns kept the response essentially elastic. Sun and Fukuhara (2005) carried out tests on four RC frames simulating the behavior of HSC elements at the lower story of high-rise buildings. Their work investigated steel tube confinement as an alternative to conventional confinement in order to overcome steel congestion problems and concrete splitting. It was concluded that the confinement of HSC columns by steel tube and use of high-strength steel as longitudinal bars assured ductile seismic performance to the frames under high-axial compression and reduced the residual story deformation, without

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providing any cost information. To achieve similar objectives, Ishikawa *et al.* (2000) and Takatsu *et al.* (2006) investigated the influence of steel fiber (SF) volumetric ratio on columns using HSS (SD685 for main bars and SPBD1275/1420 for stirrups) and with concrete strength ranging from 110 MPa to 160 MPa. It was concluded that, as the SF volumetric ratio increased, column ductility increased, early spalling of concrete cover was delayed, surface crack widths were reduced and the lateral capacity increased.

2. Experimental program

2.1 Specimens

Three 1/4-scaled column specimens, 280 mm x 280 mm (b x D) in cross section and 1120 mm (L) in height corresponding to a shear span ratio of 2, served as test specimens. They were built in the horizontal position. All units were designed according to the Japanese codes (AIJ 1990; AIJ 1999) and considered representative of those most commonly used in the first story of tall buildings. Therefore, to prevent buckling the spacing of transverse reinforcement was decided by reference to the results of previous experimental works (Ishikawa *et al.* 2000 and Takatsu *et al.* 2006). **Figure 1** depicts a typical column detailing of the tested specimens. The difference between specimens, as shown in **Table 1** was the grade of main longitudinal bars and the type of axial loading. All columns had the same transverse and longitudinal reinforcement ratios i.e. $\rho_w = 1.22\%$ (consisted of a 7.1 mm-diameter peripheral tie and two crossing legs at every 35 mm) and $\rho_g = 2.92\%$ (consisted of 8 D19 deformed bars of 19 mm nominal diameter). All specimens were made of an ultra HSC aiming a strength of 200 MPa. After casting the concrete into the metallic forms, the specimens were subjected to a thermal curing using a steam boiler and reproducing the actual practice in the field where the only difference was in the size of the elements. Therefore, because the size of actual elements (relatively massive elements) was four times the tested ones, the thick placement would generate more quickly heat in a way the heat generation rate would be higher than the heat loss rate and consequently the curing temperature level in the field would be different from the temperature of the tested elements. Furthermore, as long as the strength of elements was the concern where the durability was not aimed and for test cost effectiveness, the thermal curing time was limited to a minimum possible. Such curing was applied, herein, primarily to prevent the rapid lost of moisture from the concrete, which contained high-cementitious materials, during the period in which it would gain strength and provided an environment that balanced the quick heat loss from the concrete for relatively not thick placement, preventing large temperature difference that may cause or contribute to cracking. Considering the previously mentioned points including the amount of the various material components and admixtures, a 72-hour thermal

curing under constant temperature plateau of 90°C was applied. Four hours (almost 16°C/hour) were necessary to raise the temperature from the ambient level to the designated level, while 7 hours (10°C/hour) were consumed in the reverse phase. The temperature at the beginning of the plateau could not be stabilized easily and the temperature went up to an average of 94°C for almost 10 hours. The maximum temperature as well as the upward and downward slopes were decided previously at the laboratory according to the proportions of the materials contents to insure a complete hydration reaction, particularly as the water content was low.

2.2 Materials

The concrete mix consisted of a maximum crushed stone coarse aggregate size of 13mm, Silica fume cement, with 13% of water/cement ratio and 1% volumetric ratio of steel fibers, resulted in the average density of 2.51 t/m³. A 3.5% of water reducing agent was added to the mixture. The average concrete compressive strength f_c and Young's modulus E_c of the standard concrete cylinders a few days before column loading were 171.3 MPa and 4.52×10^4 MPa, respectively. The filaments of wire mixed with the concrete consisted of high tensile

Fig. 1 Geometry and detail of test specimens [mm].

Table 1 Steel grade and axial loading type of specimens.

Specimens	Main bar grade	Axial loading
Unit 700	SD685	Varying
Unit 701	SD980	Varying
Unit 702	SD980	Constant

steel fibers called Super performance Dramix® RC-80/30-BP. It is a high carbon wire fiber, of 30 mm long and 0.38 mm diameter with hooked ends and glued in bundles, recommended for applications with concrete strength higher than 60 MPa. The minimum tensile strength (information of material producer) on the wire was 2300 MPa.

As for the reinforcing steel, it consisted of two types of deformed bars for longitudinal reinforcement, i.e. SD685 and SD980 and one type of deformed bars for transverse reinforcement (SPBD1275/1420). **Table 2** and **Fig. 2** show the main properties of the tested steel bars.

2.3 Test setup

Figure 3 shows the general view of the test setup as it was conducted. The upper and lower column stubs were, respectively, tied up and down to a rigid steel loading girder and to a strong floor by 16 and by 8 high-strength steel bars post-tensioned to a total force of 4000 kN. The axial compression load on each specimen was applied by means of two laterally sliding 5000 kN hydraulic jacks bearing against the rigid steel girder at the top of the upper stub and a load transfer girder sitting on top of the jacks. The axial tension load on specimens subjected to varying axial load was applied by means of two 2000/1000 kN jacks (2000kN pushing and 1000kN pulling capacities) bearing against the strong floor and the rigid steel girder at the top of the upper stub. A 1000

Fig. 2 Characteristics of reinforcements.

Fig. 3 An overview of loading system.

kN hydraulic actuator was employed to supply the cyclic reversed lateral force, which was applied at the mid-height of the column. The loading system was provided longitudinally with a pantograph to prevent any in-plane rotation of the loading rigid girder, while laterally a system also was provided to prevent any out-of-plane movement of the loading rigid girder during testing.

2.4 Instrumentation

The tested specimens were instrumented externally and internally. Linear variable displacement transducers (LVDTs) were installed to measure the relative displacements between different selected locations of the column along its height, axially and laterally. Electrical resistance strain gauges were placed at several locations on the longitudinal reinforcement along the height of the column and in the lower stub while on the transverse reinforcement only three layers of ties and legs were instrumented at the column ends covering the hinge zone.

The vertical jacks for axial load and horizontal actuator for lateral load were fitted with load cells. Signals from the LVDTs, strain gauges and load cells were processed through a computerized data acquisition system.

2.5 Loading procedure

All the specimens were tested in a vertical position and loading limit was referred to their lateral capacities. The axial load N , when varied, was proportional to the lateral shear force Q , where the initial, tension and compression axial ratios, respectively, η_0 , η_t and η_c are illustrated in **Table 3** and **Fig. 4**. Although not proper at high-strain levels for high-strength materials, the curves of Q - N interaction diagram were obtained based on the plane section assumption and the stress-strain relations of concrete and reinforcement. The maximum compress-

Table 2 Properties of reinforcements.

Type	Elastic Modulus	Tensile strength	
	E_s 10 ³ MPa	σ_y MPa	σ_u MPa
D19 (SD685)	189	740	1000
D19 (SD980)	188	1201*	1287
φ7.1(SPBD1275/1420)	193	1361*	1430

Table 3 Applied axial load ratios.

Specimens	Initial load ratio	Minimum load ratio	Maximum load ratio
	η_0	η_t	η_c
Unit 700	0.2	-0.71	0.6
Unit 701	0.2	-0.71	0.6
Unit 702	0.3	-	0.3

$\eta_0 = N_0/(bDf_c)$, $\eta_c = N_c/(bDf_c)$, $\eta_t = N_t/(\rho_g bD\sigma_y)$ where N_0 , N_c and N_t are, respectively, the initial, compression and tension axial loads

Fig. 4 Q-N interaction diagram and course of axial load.

sion load was equal for Unit 700 and Unit 701 while the maximum tension force was higher in Unit 701. When the intended axial load was compression, only the 5000 kN jacks were activated, and then when the intended axial load became tension the 5000 kN jacks were disengaged letting only the 2000/1000 kN jacks into action and vice-versa. Laterally, the specimens were subjected to an anti-symmetric double curvature bending where the loading path was controlled by displacement. Reversed cycles with increasing lateral drift angle amplitude R from 0.1% till 5% were applied in one direction accounting for some cracking, pre-yielding and different ductility levels. Two cycles of loading were performed for each level until the ultimate capacity was reached. The failure state was defined as the level at which the capacity of the specimen during the loading cycle considered dropped by more than 20% of the maximum capacity of the specimen. The associated lateral drift angle is denoted by R_u , corresponding to a lateral displacement δ_u while the displacement ductility factor is expressed by

$$\mu_{\delta,u} = \frac{\delta_u}{\delta_y} = \frac{R_u}{R_y} \quad (1)$$

where R_y and δ_y denote the lateral drift angle and lateral displacement at yield, respectively.

3. Experimental results and discussion

3.1 General observations

The tested specimens experienced various stages throughout the loading procedure. The degradation in strength was due to concrete damage and possible buckling although no clear sign of main bars buckling was noticed in the damaged zones. The advantage of using SF is apparent by the limited and narrow cracks on the

whole surface till the lateral drift angle $R = -0.5\%$ and on the column's central face zone till the lateral drift angle $R = -1.5\%$. The type of SF used in these specimens, in comparison to another type of SF used in a previous experiment (Takatsu *et al.* 2006), might have an impact on preventing the buckling of main bars. The advantage of using HSS bars appeared through the delay of cracking. Although the advantage appeared slightly on the positive loading (column compression) side where the maximum shear strength increased by only 2%, the increase of strength on the negative loading (column tension) side reached 8%. As to the ultimate state, 5% increase in the ultimate lateral drift angle was noticed when HSS was used. Furthermore, it appeared that the contribution of concrete to dissipate energy was more significant than the longitudinal reinforcement. The applied maximum axial load levels brought the specimens relatively far beyond their balanced failure state and outside the region where more ductile behavior (conditions below the balanced failure state) and higher flexural strength would be suspected. Improving the performance of columns with HSS could be achieved if the increase in the strength of HSS were accompanied by a proportioned increase in the strength of concrete or a reduced axial load ratio. However, the bond degradation problem could limit the performance of SF RC columns with HSS, as long as the limit shear bond stress would be conditioned by the concrete strength.

3.2 Crack progression and damage

For the specimens under varying axial load, similar crack patterns were generally observed although there was some delay in crack propagation for the specimen with very HSS bars (Unit 701) beyond the lateral drift angle $R = -0.33\%$. This delay could be because of the large elasticity of HSS until a large deformation of the columns. Tension cracks developed mainly when load-

ing direction was on the tension side while crushing appeared when loading direction was on the compression side. On both specimens, as can be seen on the force response curves (**Fig. 6**), rupture of some main bars occurred probably inside the stub nearby the column-end sections, but were not visible when checked at the end of loading.

Unit 700 (specimen with SD685) experienced bending cracks, bending shear cracks, crushing of concrete and yielding of compression steel bars at column ends section at the lateral drift angles $R = -0.10\%$, -0.20% , 0.33% and 0.50% , consecutively. Yielding of the tension steel bars at the column-end sections occurred at the lateral drift angle $R = 2.60\%$. Yielding of steel bars extended from column-end sections till a distance $D/2$ for compression and tension bars corresponding to the lateral drift angles $R = 0.71\%$ and 3.71% , respectively, and till a distance D for only compression bars correspond-

ing to the lateral drift angle $R = 0.71\%$ during a higher cycle loading. Damage concentrated at the compression corners since the lateral drift angle $R = 0.75\%$ and discontinuous splitting cracks formed along the main-bars location after the lateral drift angle $R = 1.00\%$.

Unit 701 (specimen with SD980) experienced bending cracks, bending shear cracks and crushing of concrete, consecutively, at the lateral drift angles $R = -0.11\%$, -0.20% and 0.33% . Due to some probable deflections in strain gauges caused by thermal curing, yielding was recorded only on one central bar at $R = -3.00\%$ while all other main bars seemed to remain elastic. Damage concentrated at the compression corners since $R = 0.75\%$ and splitting cracks did not form along the main-bar locations.

For the specimen under constant axial load, Unit 702 (specimen with SD980), as compared to the specimens under varying axial load, the crack pattern was different

Fig. 5 Progression of cracks and damage.

due to on one hand the absence of the full tension force that induced uniformly distributed cracks along the column height of Unit 700 and unit 701, and on the other hand the relatively lower maximum compression level of the axial load that induced crushing at late stages. At first, small vertical cracks at the corners started at lateral drift angle $R = 0.20\%$, followed by bending cracks, bending-shear cracks, crushing of concrete and yielding

of compression steel bars at column ends section, consecutively, at the lateral drift angles $R = 0.33\%$, 0.60% , 0.75% and 2.00% . Damage concentrated mainly at the corners of the specimen-end regions and splitting cracks spread along the main-bar locations after the lateral drift angle $R = 1.00\%$. **Figure 5** shows the condition of the tested specimens at different levels of the lateral drift angle.

a) Unit 700

b) Unit 701

c) Unit 702

Fig. 6 Shear force-lateral drift response.

Table 4 Performance of specimens including P-Δ effect.

Specimens	$Q_{\max}$ kN	$R_{\max}$ %	R_u %	Q_{ACI} kN	Q_{AIJ} kN
Unit 700	995.9 (-241.2)	1.51 (-3.25)	3.53	725.8 (-113.5)	767.7 (-13.6)
Unit 701	1015.0 (-260.4)	1.50 (-3.05)	3.70	725.8 (-171.4)	915.3 (-21.0)
Unit 702	1045.9 (-1042.9)	3.02 (-3.06)	5.76 (-5.31)	787.6	1117.1

(x.x): value on negative loading side

Table 5 Performance of specimens excluding P-Δ effect.

Specimens	$Q_{\max}$ kN	$rQ_{\max}$	$R_{\max}$ %	$rR_{\max}$	R_u^* %	rR_u
Unit 700	1167.5 (-241.2)	1.17	2.66 (-3.25)	1.76	4.52	1.28
Unit 701	1195.9 (-260.4)	1.18	2.98 (-3.05)	1.99	4.97	1.34
Unit 702	1167.6 (-1166.4)	1.12 (1.12)	3.02 (-3.06)	1.00 (1.00)	8.31 (-6.63)	1.41 (1.25)

(x.x): value on negative loading side, *evaluated by extrapolation, $rX_{\max}$: ratio of X-value when P-Δ effect is not included to X-value when P-Δ effect is included

The crack-width could be recorded after the lateral drift angles $R = 0.2\%$ and 1.0% , respectively, for the specimens under varying axial load and the specimen under constant axial load. The maximum widths were recorded at the column-end regions, particularly those resulting from bending. However, crack width recording could not be kept continuous at different locations due to crushing, splitting and damage of concrete. In Unit 700, the cracks that were distant from crushed regions reached a maximum width of 0.1 mm while those inside the damaged region went beyond 1.5 mm before splitting of concrete. In Unit 701, the cracks that were distant from crushed regions had a limited width of 0.06 mm while those inside the damaged region reached a maximum width of 0.35 mm at the lateral drift angle $R = 2.0\%$. In Unit 702, crack-width was also limited and reached 0.03 mm for bending cracks and 0.45 mm for shear cracks in the column-end region for the lateral drift angle $R = -5.0\%$. Far from the column-end region, the vertical cracks reached a width of 0.25 mm at the mid-height of the column.

3.3 Hysteretic response of the specimens

The lateral load-lateral drift angle hysteretic responses of the specimens are shown in Fig. 6. Tables 4, 5 and 6 present the main results relative to each specimen. The global flexural behavior clearly indicates good performance characteristics where the specimens subjected to varying high intensity axial loads experienced stable hysteresis loops beyond the lateral drift angle $R = 3\%$, while the specimen subjected to lower constant axial load (half of the maximum varying load) went even

beyond the lateral drift angle $R = 5\%$. The test lateral strength values were between 30% to 40%, and 10% to 30% greater than those obtained by using the equation of ACI (2002) and the simplified equation of AIJ (1999), respectively. Both equations are based on the plane section assumption and the stress-strain relations of concrete and reinforcement, though such assumption is not appropriate at high-strain levels. As to the concrete block, a factor of 0.65 was considered for the height and a factor of 0.85 for the strength in the ACI formulation, on the other hand the simplified AIJ equation, similar to the ACI formulation, was used. Therefore, the evaluated shear strengths of the specimens, based on inelastic displacement concept, were not critical. The AIJ equation combines the strut-tie action to the arch action and considers an appropriate concrete softening factor for high-strength concrete.

When lateral loading direction was on the positive side (compression side for varying axial loading), Unit 702 showed higher lateral strength than Unit 700 and Unit 701 by 5% and 3%, respectively, while the strength

Table 6 Shear degradation on envelope curve.

Specimens	$pK_{Q, \text{degr.}}^*$ kN/mm	$K_{Q, \text{degr.}}^{**}$ kN/mm	r
Unit 700	20.2	13.1	0.65
Unit 701	18.8	10.9	0.58
Unit 702	8.0 (11.3)	4.6 (7.8)	0.58 (0.69)

(x.x): value on negative side, *P-Δ effect included, **P-Δ effect not included, $r = K_{Q, \text{degr.}} / pK_{Q, \text{degr.}}$

of Unit 701 was only 2% higher than the strength of Unit 700. As for the lateral deformability, Unit 702 showed the most ductile behavior. The evaluated displacement ductility factors, based on Park's work (Park 1989) were, respectively, 3.5, 3.7 and 5.2 for Unit 700, Unit 701 and unit 702. Unit 702 reached larger ultimate lateral drift angle than Unit 700 and Unit 701 by 63% and 56%, respectively, while the ultimate lateral drift angle of Unit 701 was only 5% greater than the ultimate lateral drift angle of Unit 700. At the end of the test, the specimens, after unloading from the maximum attained peak cycle, experienced residual lateral drifts. Unit 700 and Unit 701 returned to the lateral drift angles $R=2.50\%$ and 2.49% , respectively, after reaching the maximum lateral drift angle of 4.00% , while Unit 702 returned to the lateral drift angle $R=4.70\%$ after reaching the maximum lateral drift angle of $R=7.00\%$. As to the shear degradation in the columns beyond their maximum lateral strength (inclination of the envelope curve), Unit 702 experienced less degradation than the two other specimens due to the low intensity level of the applied constant axial load. The shear degradation in Unit 701 was 7% less than the degradation in Unit 700. This might be explained by the difference in the variation of the column section's compression area (related to the effective shear area) along the column height. The compressed area was approximately evaluated for both specimens using the recorded strains in the main bars. The results of this evaluation showed that after reaching the peak shear strength generally the compression area in Unit 701 was relatively wider than in Unit 700 and the strain levels at all sections along the height of the columns were lower in Unit 701 than in Unit 700. After the peak shear strength, such lower strain level (lower stress) on larger area would result in less damage to the cracked concrete and consequently less degradation of shear strength.

When the lateral loading direction was on the negative side (tension side for varying axial loading), Unit 701 showed slightly higher stiffness and reached higher strength at lower lateral drift but was less ductile than Unit 700 due to high applied tension load (30% higher in Unit 701 than in Unit 700). The test on Unit 701 was interrupted for safety reasons when the specimen showed a sudden decrease in strength on the tension side due to a rupture of the main bars that could neither be checked visually nor observed on the strain data. Due to the limited crack-width in Unit 701, compared to Unit 700, the specimen had slightly higher stiffness during the unloading and reloading stage that might be due to the slight variability in the actual strength of materials and the crack patterns.

Unit 702, under constant axial load, had a smooth and symmetric response on both loading sides. The axial loading level could be considered favorable and less damaging than the loading level applied in Unit 701. Higher constant axial load ratio on Unit 702 would probably induce a similar degradation response as in

Unit 701.

3.4 Strains in reinforcement

The recorded strains of reinforcement at different locations confirmed the lower values in HSS, as shown in Fig. 7 for the strains at the lower section of the columns. Generally, in the specimens subjected to varying axial load, recorded strains of the main bars were similar before the lateral drift angle $R=\pm 0.2\%$ (relative to the appearance of bending shear cracks). However, beyond that drift angle, strains were lower for extreme and center bars at column ends when SD980-HSS was used. This difference was explained by the large elasticity of HSS until a large lateral deformation of the columns. By reference to Fig. 2, SD980 bars would develop large forces (stresses) for small strains before reaching their own yield level and even after the yield level of SD685 bars, while SD685 bars would experience large strains due to the presence of the plateau after yielding. Actually, the early cumulated strains in Unit 700 (SD685) under the combined axial compression/tension load and

Fig. 7 Strain evolution in main bars.

lateral drifts, and the lower elasticity range of column's main bars resulted in larger strains than in Unit 701. For instance, at the column lower section, on the positive loading direction (axial compression $N=7500\text{kN}$) at the lateral drift angle $R=0.5\%$, the compressed bars of Unit 700 reached a strain of -4060μ (beyond yield level) while the opposite bars reached -800μ . When the loading was reversed (axial tension $N=-680\text{kN}$) at $R=-0.5\%$, the previously compressed bars turned to tension and reached a strain of 1480μ while the opposite bars reached 250μ . Comparatively, on the positive loading direction (axial compression $N=7480\text{kN}$) at the lateral drift angle $R=0.5\%$, the compressed bars of Unit 701 reached a strain of -990μ (far below yield level) while the opposite bars reached -430μ . When the loading was reversed (axial tension $N=-590\text{kN}$) at $R=-0.5\%$, the previously compressed bars turned to tension and reached a strain of 800μ while the opposite bars reached 300μ . By comparison of other recorded values at different lateral drift angles, it was found that while on the positive loading side, the compression axial load was similar for both specimens along the positive loading side, on the negative loading side the tension axial load was lower in Unit 701 than in Unit 700 due to the selected loading path (Fig. 4) below the lateral drift angle $R=-1.0\%$. A large difference in the tension axial load was noticed beyond the lateral drift angle $R=-1.3\%$. Therefore, the reason of the higher strains in Unit 700 than in Unit 701 were found to be due to the combination of the low elasticity level, the early cumulative strains during loading and the higher tension axial load at early loading stages in Unit 700 than in Unit 701.

Also, when lateral loading was on the positive side, compression or tension strains of HSS in the specimen under constant axial load (Unit 702 with lower axial load) were higher than those in the specimen under varying axial load (Unit 701 with higher axial load) due to the difference level in the applied axial force and the combined resulting moment. For convenience, Table 7 illustrates the maximum level recorded at different locations and heights.

As for the transverse reinforcement, the strains varied along the column height during loading and the SF proved its effectiveness by reducing the expansion of concrete and reducing the strains in the transverse bars. Figure 8 illustrates the strain evolution recorded in the stirrups at the column end section (Fig. 8-b) and at the mid-hinge zone (Fig. 8-a). The strains in the stirrups were higher than the strains in the individual legs. The strains in the steel bars parallel to the lateral loading direction were higher than the strains in the steel bars perpendicular to the loading direction. In specimens subjected to varying axial load, recorded strains were similar in both specimens before $R=\pm 1.0\%$ but lower for the specimen with SD980 bars beyond that lateral drift angle, particularly inside the hinge zone. The recorded maximum strain in both specimens was less than $0.5\epsilon_y$ (ϵ_y is the yield strain). In the specimen subjected

Table 7 Main bars maximum strain ratio ($\epsilon_{\max}/\epsilon_y$).

Specimens	Strain	Lower End*	D/2**	D**	Upper End*
Unit 700	C	2.50	1.30	2.00	2.50
	T	1.30	1.80	0.90	1.30
Unit 701	C	0.25	0.25	0.25	1.20
	T	0.25	0.25	0.25	0.50
Unit 702	C	1.50	0.75	0.25	0.55
	T	1.00	0.75	1.00	0.25

C: Compression; T: Tension; D: Column section height; *: Column end section; **: Distance from the column end section

Fig. 8 Strain evolution in transverse steel.

to constant axial load (Unit 702), when lateral loading was on the positive side, the strains were similar to those in Unit 701 particularly in the hinge zone and lower elsewhere. The recorded maximum strain was less than $0.33\epsilon_y$.

3.5 Deformability

The flexural and shear displacements of the specimens were measured using many LVDTs, which were arranged along the height of each specimen. Vertical and crossing LVDTs measured, respectively, the vertical deformation that allowed the evaluation of the curvature (flexure) and the lateral deformation (shear) that allowed the evaluation of the distortion angle. Other displacements of different origins were evaluated by subtracting the shear and flexural displacements from the measured total displacement.

The tested specimens reached large lateral deformability, allowing larger displacement ductility when low constant axial-compression load was applied (the case of Unit 702 with a ductility of 5.2). The deformability of the specimen under constant axial load was larger than the specimen under varying axial load due to the applied low level of the constant axial load, which was relatively close to the load corresponding to the balanced failure level, while the specimens under varying axial loads experienced maximum compression levels far higher than the balanced failure state, and were double of the level of the specimen under constant axial load (refer to Fig. 4).

Shear displacement decreased during loading due to dominant flexural performance and the flexural damage at the columns' ends. Average shear displacement at $R=1.0\%$ (-1.0%) accounted for almost 20% (30%) of the total displacement for Unit 700 and 20% (20%) for both Unit 701 and Unit 702.

Axially, the deformation was the highest (column shortening) at the neutral position ($R=0.0\%$) and the lowest at maximum drift angles when constant axial load was applied. For the specimens under varying axial load, the axial deformation was lower (shortening or elongation) in Unit 701 than in Unit 700 at the neutral position but similar for both specimens at the maximum drift angles, as shown in Fig. 9.

3.6 Energy dissipation

The ability of structures to withstand cyclic loading is usually measured in terms of the energy dissipation capacity, which is defined as the total energy dissipated during n loading-unloading cycles. The total dissipated energy is then the summation of the energy E_i dissipated within each cycle i . The total dissipated energy when expressed relatively to the maximum attained lateral load $Q_{\max}$ and the yield displacement δ_y is called the normalized energy dissipation capacity E_N , which is given by

$$E_N = \frac{\sum_{i=1}^n E_i}{Q_{\max} \delta_y} \quad (2)$$

Fig. 9 Evolution of axial deformation.

The yield displacement was determined based on the method described by Park (1989) according to the idealized bilinear backbone curve, as shown in Fig. 10. The intersection of the elastic line of the idealized curve and the test maximum strength line $Q_{\max}$ corresponds to the idealized yield point. The idealized lateral yield drift angle for Unit 700, Unit 701 and Unit 702 were found to be equal to 1%, 1% and 1.1%, consecutively. By considering these values, the cumulative normalized dissipated energies related to the number of loading cycles for the three specimens are shown in Fig. 11.

Besides the differences in steel bar characteristics and axial loading, the energy dissipation capacities of all specimens seem to be almost the same till loading cycle 20 that corresponds to the lateral drift $R=2.00\%$. Beyond that level, while no difference could be observed between Unit 700 and Unit 701, the energy dissipation of Unit 702 reached a higher level and doubled during the last two cycles. The high elasticity of HSS had reduced the dissipated energy until a large deformation of the columns and concrete dominated the dissipation, particularly when the axial compression load reached a high level.

3.7 Equivalent viscous damping

The ability of the specimens to dissipate energy and reduce the peak response amplitudes due to inelastic deformation caused by earthquake excitations can be reflected by the assessment of the equivalent viscous damping, which is assumed to represent the combined effect of all damping mechanisms present in the actual specimens during testing. The equivalent viscous damping ratio is given by

$$h_{eq} = \frac{E_i}{4\pi E_0} \quad (3)$$

where E_0 is the elastic strain energy stored in an equivalent linear elastic system when the maximum displacement is reached at cycle i .

Figure 12 shows the variation of the equivalent viscous damping ratio with the lateral drift angle for all the specimens. Although the slightly low residual drift recorded on Unit 701 compared to Unit 700, the specimens had similar damping ratios with slightly higher level in the specimen with constant axial load beyond $R=0.5\%$, as shown in the same figure.

4. Conclusions

The work presented herein aims to examine the behavior of HSC columns reinforced with HSS (SD980) longitudinal bars under high axial load and cyclic reversals. The experimental results from this study could form a valuable addition to the existing database, since to the best knowledge of the authors no publication has dealt with test data regarding the use of HSS bars in HSC columns and may contribute to the use of HSS in the design of tall buildings. Therefore, while such scaled

Fig. 10 Idealization of test envelope curve.

Fig. 11 Cumulative normalized dissipated energy.

Fig. 12 Evolution of equivalent damping.

specimens would represent appropriately the behavior of actual size of columns, the spacing of lateral confinement of the later should be considered appropriately in order to insure the buckling of main bars.

The use of SD980 steel bars in columns were found to be very effective in terms of tension axial strength, delay of crack evolution and shear strength degradation and to be slightly more effective than the SD685 grade bars in terms of shear strength and energy dissipation. Therefore, the column with SD980 was more ductile when subjected to low constant axial load. Such column could be more ductile and have larger load capacity if designed relatively far below the balanced failure point.

In addition, the following observations and conclusions can be drawn from the results of the experimental program.

- (1) The benefit of using SF is apparent by limited and narrow crack-widths observed before the lateral drift angle $R = -0.5\%$, particularly when SD980 steel is used. The type of SF used in the columns is suspected to have an important impact on preventing the buckling of longitudinal steel bars;
- (2) Using SD980 steel delayed the evolution of cracks outside hinge zone and beyond the lateral drift angle $R = -0.33\%$. Furthermore, damage concentration in columns was similar for both steel types till the lateral drift angle $R = 0.75\%$, beyond which it appeared more abrupt for columns with SD980 steel;
- (3) The axial loading type affected the crack development and damage concentration for columns of similar high strength steel type;
- (4) Using SD980 steel in columns induced lower column shear strength degradation due to the presence of wider column section's compression area (related to the effective shear area) along the column height and the development of lower strain level on column section;
- (5) Using SD980 steel in columns induced slightly higher ductility (larger ultimate drift angle R_u) and smaller residual deformation than when using SD685 steel;
- (6) Using SD980 steel in columns lowered column shear deformation in the lateral drift angle range $R = \pm 1\%$ more than when using SD685 steel, particularly on the tension lateral loading side;
- (7) Using SD980 steel in columns reduced strains in stirrups within the hinge region beyond the lateral drift angle $R = 1.0\%$, had little influence on energy absorption as compared to the column with SD685 steel, therefore, it lowered the axial deformation at the neutral position, but did not affect it at large drift angles; and
- (8) The test results of HSC columns with HSS reinforcement confirmed the appropriateness of the design equations in AIJ and ACI regulations. Test shear forces were 10% to 30% higher than the shear force evaluated using AIJ equation confirming the good precision of the equation. Test shear forces were 30% to 40% higher than the shear force evaluated using ACI equation showing the appropriateness safety margin of the equation.

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