APPLICATIONS OF HIGH STRENGTH CONCRETE IN SEISMIC REGIONS

P A MENDIS[1] And C PANAGOPOULOS[2]

SUMMARY

The use of high strength concrete (with f'_c>50MPa) is very common in buildings and other structures designed recently. Economy, superior strength, stiffness and durability are the major reasons for its popularity. Structural Engineers are presently exploring the benefits of using this efficient material in various applications. The main aims of this paper are to highlight the applications of high strength concrete in earthquake resistant frame structures and study the behaviour of important high strength concrete elements in those structures. A case study is presented to show the comparative difference in seismic performance when high strength concrete is used as a replacement for normal strength concrete in a typical frame structure.

INTRODUCTION

The advancement of material technology and production has led to higher grades of concrete strengths. The use of High strength Concrete (HSC) elements (f'_c>50MPa) for concrete structures has proven very popular, with strengths of concrete up to about 130MPa used around the world. These concretes can be produced using conventional production procedures. The main advantages of HSC include higher strength and higher stiffness, improved durability, cost efficiency, reduced creep and drying shrinkage, better impact resistance and better resistance to abrasion. However, as shown in Figure 1, HSC is less ductile compared to normal strength concrete.

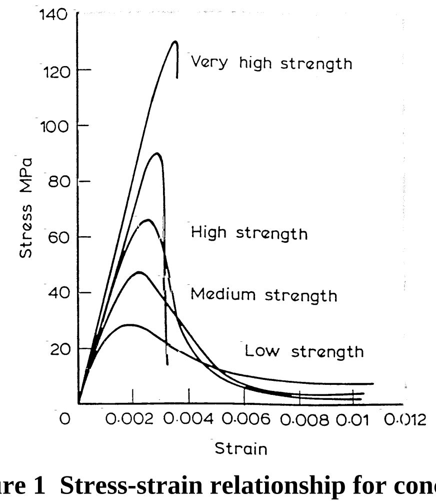


Figure 1 Stress-strain relationship for concrete

High strength concrete is structurally a different material and rules applicable to normal strength concrete are not always conservative when applied to high strength concrete. Due to the variations in fracture modes, microstructure and the differences brought about by various additives like silica fume, fly ash, superplasticizers, etc, the empirical design rules originally intended for concrete strengths f'_c<50MPa, needs to be re-evaluated. One of the reasons why some structural engineers are reluctant to use high strength concrete is due to the lack of provisions in Concrete Standards to cover high strength concrete.

There are a number of advantages of using HSC in earthquake resistant structures. Concrete structures are inherently heavy and hence have the potential to induce substantial inertial forces. High strength concrete

[1] *Dept of Civil & Environmental Engineering, The University of Melbourne, Australia Email: P.Mendis@civag.unimelb.edu.au*
[2] *Dept of Civil & Environmental Engineering, The University of Melbourne, Australia*

members will have the distinct advantage of reducing these inertialloads due to the reduction in member sizes. Higher elastic modulus will reduce the drift due to lateral load. Other aspects relevant to high strength concrete members are discussed in the next section. This discussion is limited to members in reinforced concrete frames. However, generic conclusions on the behaviour of high strength concrete members are applicable for other types of structures.

BEHAVIOUR OF HIGH STRENGTH CONCRETE MEMBERS

Flexural Members

Although the moment capacity of a flexural member is not significantly increased by using higher strength concrete, the deflections are reduced thus allowing larger spans. Despite high strength concrete being a more brittle material, flexural members cast with high strength concrete exhibit greater rotational ductility due to lower neutral axis depths [Pendyala et al., 1996]. In seismic design, a beam mechanism is preferred to a column sidesway mechanism. The formation of beam hinges is assured by having a strong column-weak beam system. Therefore, higher rotational ductilities of HSC can be an advantage in primary frames.

Columns

Ductility

Reinforced columns which carry high axial loads can have a marked reduction in cross section size and the amount of longitudinal steel reinforcement can be substantially reduced when high strength concrete is used [Park, 1998]. As mentioned earlier, the main concern regarding the use of high strength concrete is the reduction in ductility with the increase in compressive strength observed under uniaxial compression. However, it can be shown, experimentally and theoretically that ductility demands of heavily loaded high strength concrete compression members can be satisfied by providing additional ties [Kovacic, 1995].

Moment-curvature analysis can be used to find the curvature ductility of the critical regions of concrete members. Using the reliable strength in the post ultimate region, the curvature ductility can be defined as the ratio of curvature at 80% of the ultimate moment in the post peak region to the yield curvature. By using good detailing, the post ultimate behaviour of the section can be made quite favourable and thus increasing the reliable curvature ductility.

$$\text{Curvature Ductility} = \frac{\phi_{0.8M_u}}{\phi_y} \tag{1}$$

A typical internal column of an OMRF frame was studied. The column dimensions were 500x500mm and the reinforcement consisted of 12Y16 longitudinal bars and Y12 ligatures at 240mm spacing. Figures 2 and 3 are typical moment-curvature graphs for 50 and 100MPa columns with variable axial loads. Common to both figures is the evidence that as the axial load level on a reinforced concrete column increases, a smaller curvature is achieved and as a result, the ductility capacity of the column progressively reduces. However, the distinguishing feature of the moment-curvature behaviour is that the 50MPa columns can be seen to act in a relatively more ductile fashion as compared to the 100MPa columns with the same axial load level, consistent with the fact that high strength concrete is an inherently less ductile material. Table 1 shows the curvature ductilities calculated using Equation (1).

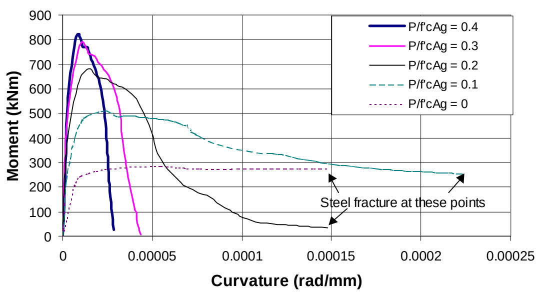


Figure 2 Moment-curvature for 50MPa columns

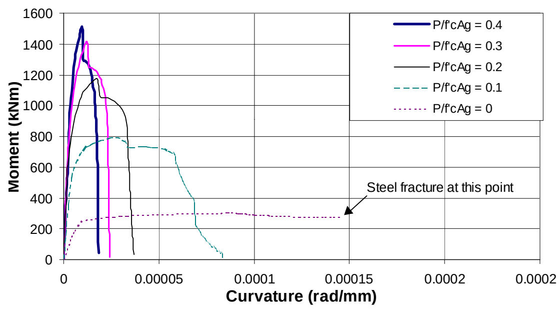


Figure 3 Moment-curvature for 100MPa columns

Table 1 Curvature Ductility Capacities for a Typical Column

P/f'_cA_g	50MPa Concrete Columns	100MPa Concrete Columns
0	27.4*	28*
0.1	8.9	6.8
0.2	4.8	3.4
0.3	2.9	2.4
0.4	2.7	2.1

* Steel fracture occurs prior to 20% reduction in moment capacity

Defining the appropriate level of ductility in a concrete column may depend on the importance of the structure or the level of earthquake risk in a particular area. The design of lateral reinforcement in a column has to be based on three criteria:

- To prevent longitudinal steel from buckling: The strain at which buckling of longitudinal steel occurs is independent of the compressive strength of concrete. Therefore, the spacing requirements suggested for NSC are applicable to HSC columns.
- To prevent fracture of lateral steel: Using an energy balance method it was shown that increasing compressive strengths from 50 to 80MPa causes only a small reduction in the hoop fracture strain [Kovacic, 1995].
- To provide adequate confinement for ductility: A formula was proposed by Mendis & Kovacic [1999] to calculate the spacing of lateral reinforcement by modifying the present requirements in AS3600 [1994]. In the method suggested in the paper, adequate ductility is provided for a high strength concrete column by comparing its moment-curvature characteristics with that of a normal strength concrete column. The level of ductility obtained from a 50MPa concrete column is therefore also ensured for the same column made with high strength concrete. Limits are also determined for the longitudinal steel buckling and lateral steel fracture.

Shear

There is the expectation that the curvature ductility demand would be met if there were adequate capacity. A brittle shear failure may ensue, even if there has been some ductile response, if there is inadequate lateral reinforcement at the critical section. The role of axial load can be significant. Under cyclic loading, the axial compression contributes to the closing of both flexural and shear cracks, thus mitigating premature failures due to sliding shear. High axial loads limit the sections post ultimate response thus reducing the available ductility.

Models to predict shear strength in columns that include axial load and the hinge rotation/curvature have been used in the Japanese design code [1992] and a model proposed by Priestley [1995]. Both models try to account for the degradation in the concrete's shear strength contribution as the rotational/curvature ductility of the section increases.

Priestley's model for shear strength uses the contribution of the concrete, lateral reinforcement and axial load.

$$V_n = V_c + V_s + V_p \qquad (2)$$

$$V_c = 0.8 A_g k \sqrt{f'_{ca}} \qquad V_s = \frac{A_v f_{yh} k D'}{s} \cot 30^o \qquad V_p = P \tan\alpha \qquad (2a)$$

As seen HSC will enhance the concrete contribution thus improving the shear behaviour. A comparison of the shear force corresponding to the column flexural strength ($V_f=2M_y/L$) to the calculated shear capacity can be made. The use of higher strength concrete leads to the use of smaller cross-sections and consequently lower yield values. This reduces the column flexural shear strength, and reduces the possibility of shear failure.

Beam-Column Joints

Stresses

Reinforced concrete beam-column joints cannot be considered in isolation, but must be considered as an integral part of the frame. It is preferable that these regions remain strong so that energy will be dissipated in the adjacent members rather than in the joint. Despite this preference for the joint to remain strong, it is inevitable that there will be some participation of inelastic shear and bond mechanisms in the hysteretic behaviour of a moment resisting frame. Shear forces in joint cores are much greater than in the adjacent columns and the moment gradient in the joint core is much larger than in the adjacent column or beam [Cheung et al, 1993].

Priestley [1995] has reviewed the test data for interior joints and concluded that with lightly reinforced beams, or with columns with high axial force levels, joint cracking may not develop if the principal tension stress in the joint is less than $0.29\sqrt{f_c^{'}}$. Beam-column joints with high shear stress levels tend to fail in shear regardless of the amount of transverse reinforcement. The reason for failure is the principal compression stress, and it is thus more logical to limit this directly, rather than through the shear stress, which does not recognise the influence of axial compression. Priestley [1995] suggested a limit of $0.5f^{'}_c$ for the principal compression stress. For beam-column joints with principal tension stress greater than $0.29\sqrt{f_c^{'}}$ and principal compression stress less than $0.5f^{'}_c$ failure may be due to joint shear, bond slip of rebar through the joint or beam flexural ductility. However, high strength concrete will significantly reduce the chance of joint cracking (Figure 4(a)).

According to the New Zealand Code [1995] two different mechanisms, the strut and the truss mechanisms, combine to resist the applied shear forces. These mechanisms are described in detail in Cheung et al. [1993]. The strut mechanism relies on a diagonal concrete strut to transfer shear forces across the joint at an angle close to that of the potential corner-to-corner failure plane. The compressive force is generated partly by bond forces from the beam and column longitudinal bars but primarily by concrete compressive forces from the flexural members at the two corners as shown in Figure 4(b). The remaining bond forces from the beam and column bars are transferred to the core concrete mainly outside the shaded area of Figure 4(c). The diagonal compressive field which sustains these bond forces can be generated by a truss mechanism which involves the participation of horizontal reinforcement (normally joint ties), vertical reinforcement (column intermediate bars) and numerous diagonal concrete struts. The compressive strength of the diagonal concrete struts will be considerably less than $f^{'}_c$ due to the presence of tensile strains in both the horizontal and vertical directions. High strength concrete will improve the contribution of the strut mechanism thus reducing the requirement of a large number of ties and column intermediate bars.

The positive effect of high strength concrete is included in the NZ recommendations, which are applicable up to 70MPa concrete. According to the NZ code, the amount of required shear reinforcement is given by

$$A_{jh} = \frac{6v_{jh}}{f_c^{'}}\alpha_i \frac{f_y}{f_{yh}} A_s^* \tag{3}$$

where v_{jh} = nominal horizontal shear stress in joint core and A^*_s = greater of area of top or bottom beam reinforcement passing through the joint, excluding the area of bars in effective tension flanges. And where

$$\alpha_i = 1.4 - 1.6\frac{C_j N^*}{f_c^{'} A_g} \tag{4}$$

whereby the beneficial effects of the axial compression load acting on the column above the joint may be included, $C_j = V_{jh}/(V_{jx} + V_{jz})$, V_{jx} = total horizontal joint shear in x direction, V_{jz} = total horizontal joint shear in z direction.

U.S. practice differs, whereby it is assumed that the joint shear is carried primarily by a diagonal compression strut that, if confined by transverse reinforcement, will be capable of carrying a certain amount of joint shear force. According to ACI-ASCE 352 recommendations [1996], for a joint confined on all four faces, the total shear stress is limited to $1.66\sqrt{f_c^{'}}$ MPa. In the ACI recommendations, the lateral ties required for confinement is given by the following formula.

$$A_{sh} = 0.3\frac{s_h h^{''} f_c^{'}}{f_{yh}}\left(\frac{A_g}{A_c} - 1\right) \qquad \text{and} \qquad A_{sh} \geq 0.09\frac{s_h h^{''} f_c^{'}}{f_{yh}} \tag{5}$$

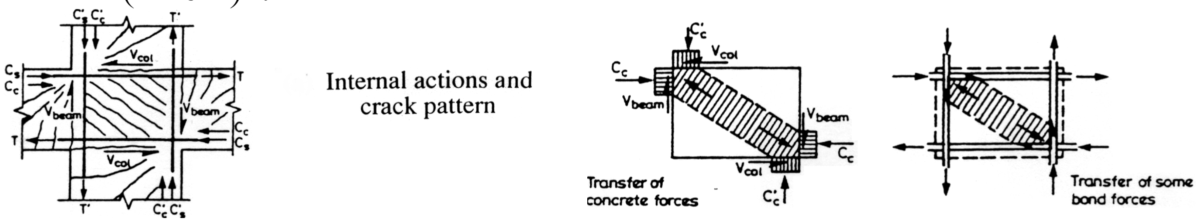
As seen the amount of transverse reinforcement required is inversely proportional to the concrete strength. This formula is based on research conducted in the 1930s. The formula has to be revised so that it is applicable to HSC.

Anchorage of beam and column bars within joint cores

With cyclic loading in regions of high seismicity, large bond forces may develop in the joint core at an interior joint leading to bond degradation and excessive slip of the bars, resulting in yield penetration into the joint region. This is because in frames designed to dissipate energy via a beam sidesway mechanism, large steel stresses need to be transferred by bond to interior joint cores over relatively short lengths of beam longitudinal bars. In the limit, the bar may be stressed to yield in tension on one side of the joint core, and to yield in compression on the other side. In order to reduce the amount of slippage, strict rules can be used to limit the ratio of the bar diameter to the column depth, with some relaxation of this if a large compressive load acts on the column [Park, 1992].

Fujii et al. [1998] recommended the following formula for bond requirements in HSC beam-column joints.

$$\frac{d_b}{D} \leq 1.34\left(1+\frac{N}{A_g f_c}\right)\frac{f_c^{2/3}}{f_y} \tag{6}$$



(a) (b) (c)

Figure 4 (a) Joint cracking, (b) Strut mechanism, (c) Truss mechanism

NZ code [1995] also specifies some requirements to limit the diameter of the bars passing through the joint.

Secondary Frames

According to ACI 318 [1995], frame members assumed not to contribute to lateral resistance shall be detailed to resist moments induced in those members when subjected to twice the lateral displacement under the factored lateral forces. When high strength concrete is used, the member sizes can be curtailed to yield a comparatively flexible frame. This frame in turn will attract significantly reduced forces.

CASE STUDY

A typical 6 storey building shown in Figure 5 was chosen for the case study and designed for two different ductility levels in accordance with the recommendations of the Australian Concrete Code, AS 3600 and Earthquake Code, AS 1170.4. The response modification factors are 4 and 6 for the Ordinary Moment Resisting Frames (OMRF) and Intermediate Moment Resisting Frames (IMRF) respectively. Frame types and section sizes are given in Table 2. For a direct comparison in the response of the structure the columns were designed with 50MPa and 100MPa concrete and detailed accordingly. Reinforcement details are given in Table 3. As the reinforcement yield strength is the nominal value used for design purposes, the analyses were also carried out with the same section detailing, but assuming a yield value for the reinforcement at the upper limit of 650MPa.

In order to assess the frames realistically, the member stiffness values should be close to that at yield to allow the formation of hinges to conform to assumed distributions. Effective second moment of areas of I_{eff}=0.4I_g for beams and I_{eff}=0.6I_g for columns [Paulay and Priestley, 1992] were used in the analyses. In addition, to be consistent with the present practice, frames were analysed with gross-section properties.

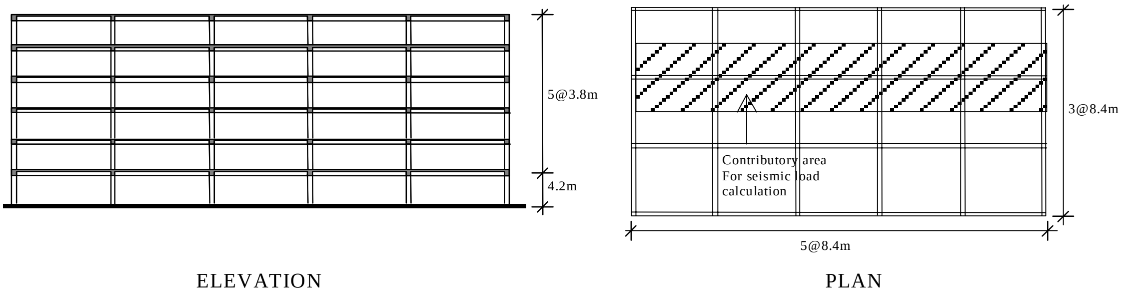


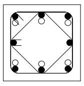
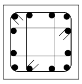
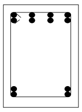
Figure 5. Structure configuration

The seismicity is taken typical of that for a low to moderately active seismic region, with a design peak ground acceleration (500 year return period) of a = 0.11g. The lateral seismic load at each storey level was calculated using the inverse triangular distribution based on the equivalent static method of AS 1170.4.

Table 2 Frame types and section sizes

Frame	Structural Type	Beam size	Beam f'_c (MPa)	Column size	Column f'_c (MPa)	f_{sy} design (MPa)	f_{sy} analysed (MPa)
1	OMRF	400x700	32	500x500	50	500	500 & 650
2	OMRF	400x700	32	400x400	100	500	500 & 650
3	IMRF	400x700	32	450x450	50	500	500 & 650
4	IMRF	400x700	32	400x400	100	500	500 & 650

Table 3 Detailing of Ground Floor Columns and First Storey Beams

		Frame			
	Detailing	1	2	3	4
External Column		8Y28 R10@220 or Y12@420	8Y28 Y12@400	8Y28 R10@100 or Y12@100	8Y24 Y12@80
Internal Column		12Y28 R10@220 or Y12@420	12Y20 Y12@300	12Y32 R10@100 or Y12@100	12Y16 Y12@80
Beam End		Top: 4Y28 + 4Y24 Bot: 2Y28 R10@100 or Y12@150	Top: 4Y28 + 4Y24 Bot: 2Y28 R10@100 or Y12@150	Top: 4Y24 +4Y24 Bot: 2Y20 + 2Y20 R10@150 or Y12@150	Top: 4Y24 + 4Y24 Bot: 2Y20 + 2Y20 R10@150 or Y12@150

The distribution of lateral forces is proportional to the gravitational load of the structure (Dead Load + 40% Live Load). Gravity loads dominate the structure's design, and as such, the beam sizes did not alter. Any dimensional reductions of columns gained through the use of high strength concrete did not translate into a major reduction in earthquake loading (reductions of 1.8% and 0.8% in gravity loads for OMRF and IMRF respectively).

Dynamic Analysis Results

The El Centro earthquake (PGA=0.33g) was selected to represent a major earthquake. The time history analyses were performed using the non-linear dynamic structural analysis program RUAUMOKO [Carr, 1999]. The members were modeled using the standard beam and column elements provided in the program.

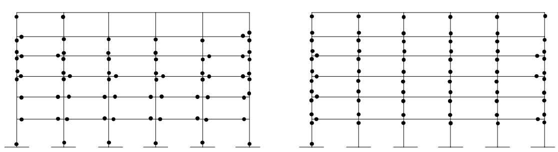
The following response parameters are presented to illustrate the behavioural nature of the structures designed for different ductility levels. The roof displacements of the gross section stiffness frames do not differ substantially and are in all four cases less than 0.85% of the building height (0.0085h). It has been suggested by De Stefano et al. [1995] that the onset of severe structural damage occurs approximately at an overall (roof) displacement of 0.01h, hence the damage levels in the frames considered here are expected to be moderate. The frames analysed with the effective stiffness showed much greater variability in displacements, being as high as 1.38%, thus indicating severe damage levels.

In both the OMRF and IMRF, the response with the higher steel reinforcement yield value showed an increase in column hinging. The relative increase of moment capacities in the sections is not uniform. The beams exhibit a substantial increase in capacity allowing the columns to yield first. Figures 6 and 7 show the locations of hinges that have occurred through the earthquake input. The utilisation of HSC lead to a smaller section and a reduction in longitudinal reinforcement, which has lead to a lower yield moment. It can be clearly seen that there is a shift to column hinging with higher strength concrete columns. This is not the favoured seismic response, although, the curvature ductility capacities of all sections were not exceeded. The only exception being frame 3 with EI_{eff}, where R10 lateral reinforcement would not provide sufficient capacity, although Y12 lateral reinforcement at the same spacing is sufficient to prevent flexural failure due to the increase in confinement pressure of the core.
Generally, the response and extent of hinging of frames using the effective stiffness was very similar to those with gross stiffness. The trend indicates that due to reduced stiffness, smaller forces are attracted by the sections and consequently have reduced curvature ductility demands. The exception is frame 3 where there is a shift from high beam demands to high column demands. The response with f_{sy}=500MPa was extensive external column hinging (GL-L5), internal column hinging (L4-L5) and beam hinging (L1-L3). With f_{sy}=650MPa, there was additional internal column hinging all the way up the structure with reduced beam hinging. The moment

Table 4 Time History Analysis Results Using El Centro (PGA=0.33g)

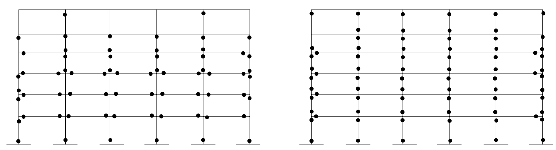
			Gross Stiffness (EI_g)				Effective Stiffness (EI_{eff})			
Frame	Structure Type	Steel Yield	Max. Displacement & Ductility Demand		Max. Curvature Ductility Demand		Max. Displacement & Ductility Demand		Max. Curvature Ductility Demand	
			Δ_{max} (mm)	Δ_μ	$\phi_{\mu\ beam}$	$\phi_{\mu\ column}$	Δ_{max} (mm)	Δ_μ	$\phi_{\mu\ beam}$	$\phi_{\mu\ column}$
1	OMRF	500	196	4.1	6.2	4.8	280	3.6	6.1	3.2
		650	180	3.7	8.9	4.1	320	4.1	4.7	2.7
2	OMRF	500	177	2.9	9.0	5.1	201	2.7	3.9	3.2
		650	168	2.7	6.2	5.4	206	2.8	2.3	3.1
3	IMRF	500	180	3.0	9.0	3.2	198	5.0	2.4	12*
		650	186	3.1	2.8	10.8	186	3.5	2.8	10.8*
4	IMRF	500	161	3.5	7.7	5.4	181	2.4	3.2	4.8
		650	162	4.0	4.3	5.3	206	2.8	2.3	2.7

capacities of beams and columns with low axial loads changed significantly with increasing reinforcement. However, lower storey columns with high axial loads remain almost constant as concrete failure governs for high axial loads. Therefore, there is a shift towards column hinging, especially at lower levels. More details are presented elsewhere [Panagopoulos and Mendis, 1999].



50MPa Column 100MPa Columns

Figure 6. OMRF (*EI_g*) Dynamic hinge formation



50MPa Columns 100MPa Columns

Figure 7. IMRF (*EI_g*) Dynamic hinge formation

Shear capacities of the columns were assessed using Priestley's model [1995]. (Eqn. 2). The design of the OMRF using higher strength concrete allowed the use of smaller column utilising the concrete's favourable load carrying capacity. Although the reduction in size reduced the shear strength of the HSC columns, a higher concrete strength compensated for that reduction. The sections that needed attention were on the lower levels which contained low yield lateral reinforcement. These sections displayed a low curvature ductility capacity, and the values of shear strength and flexural shear strength were close. According to Priestley's model, no shear failure would be expected, although the calculations are based on nominal values.

Eqns. 3, 4 and 5 were used to assess the Beam-column joints in the frame assuming the concrete strength within the joint is equal to the column strength. For frame No.1 (concrete strength of 50MPa in the joint), the required amount of horizontal shear reinforcement within an interior beam-column joint to resist the maximum shear force is equal to 4272 mm^2. According to the ACI318, the total horizontal reinforcement required is equal to 1831 mm^2. For Frame 2 (Concrete strength of 100 MPa) these values are 3891 and 2870 respectively. As mentioned earlier ACI formula gives incorrect results for high strength concrete joints.

Bond transfer within the beam-column joints were assessed using Eqn. 6 suggested by Fujii et al. [1995] and NZ code [1995]. For OMRF with 50MPa joints the maximum bar sizes are 46mm according to Fujii et al. formula and 21mm with NZ formula. For OMRF with 100MPa joints these diameters are 56 and 23mm respectively. As seen the two methods give significantly different allowable bar diameters. However, larger bar diameters are allowed in HSC joints.

CONCLUDING REMARKS

With increasing concrete compressive strength, the section sizes and reinforcement ratios in columns can be effectively reduced. This changes the balance of beam and column flexural capacities, which in turn may change the response from extensive beam hinging to extensive column hinging. Column hinging is prevalent and greater detailing may be needed in such potential plastic hinge regions. However, frames with HSC columns, in this case study, performed well in satisfying ductility demands. Although the column sizes were reduced, the maximum displacements were slightly reduced due to higher elastic modulus in HSC and slightly lower earthquake loads. There are other benefits in using high strength concrete such as improved shear capacity of columns, stronger beam-column joints, larger bar diameters allowed within a joint to transfer bond and higher rotational ductilities in flexural members. High strength concrete can be an attractive option to reduce the member sizes in secondary frames not resisting earthquake loads. These frames will attract significantly reduced forces due to the increase in flexibility. However as high strength concrete is not adequately covered in most concrete design standards, accurate guidelines are required, which will enable designers to make full use of this efficient material.

REFERENCES

ACI 318M (1995): Building Code Requirements for Reinforced Concrete, American Concrete Institute, USA.

ACI-ASCE Committee 352 (1996), *Draft, Recommendations for design of beam-column-slab connections in monolithic reinforced concrete structures*, Detroit, USA

Architectural Institute of Japan (1992), *AIJ Structural Design Guidelines for Reinforced Concrete Buildings.*

AS1170.4 (1993), *Minimum design loads on structures- Earthquake Loads*, Standards Association of Australia, Sydney, Australia.

AS3600 (1994), *Concrete Structures*, Standards Association of Australia, Sydney, Australia.

Carr, A.J. (1999), “RUAUMOKO”, Computer Program Library, University of Canterbury.

Cheung, P.C., Paulay, T. and Park, R. (1993), “Behaviour of beam-column joints in seismically-loaded RC frames”, *The Structural Engineer*, 71, 8, pp 129-138.

De Stefano, M., Faella, G. and Realfonzo, R. (1995), “Seismic response od 3D RC frames: Effect on plan irregularity”, *European Seismic Design Practice*, ed. Elnashai, A., pp 219-226.

Fujii, H., Noguchi, H. and Morita, S. (1995), “Bond and anchorage of reinforcement in high-strength concrete”, *SP-176: High-Strength Concrete in Seismic Regions*, USA, pp 23-43.

Kovacic, D. (1995), *Flexural ductility of high strength concrete columns*, MEngSc Thesis, University of Melbourne, Australia.

Mendis, P. and Kovacic, D.A. (1999), “Lateral reinforcement spacing for high-strength concrete columns in ordinary moment resisting frames”, *Accepted for publication in Journal of Structural Engineering*, Institution of Engineers, Australia.

NZ3101 (1995): *Design of concrete structures*, *Vol 1 & 2*, Standards Association of New Zealand, Wellington.

Panagopoulos, C. and Mendis, P. (1999), *Applications of high strength concrete in earthquake resistant structures – A case study*, Research Report, Dept. of Civil and Environmental Engineering, University of Melbourne, Australia.

Park, R. (1992), "Capacity Design of Ductile RC Building Structures for Earthquake Resistance", *The Structural Engineer*, 70, 16, pp 279-285.

Park. R. (1998), “Some current and future aspects of design and construction of structural concrete for earthquake resistance”, *Australasian Structural Engineering Conference*, Auckland, pp 1-16.

Paulay, T. and Priestley, M. (1992), *Seismic design of reinforced concrete and masonry buildings*, John Wiley & Sons, New York.

Pendyala, R., Mendis, P.A. and Patnaikuni, I.(1996), “Full range behaviour of high-strength concrete flexural members, part 1”, *ACI Structural Journal*, 93, 1, pp 30-35.

Priestley, M.J.N. (1995), “Displacement-Based seismic assessment of existing reinforced concrete buildings”, *Proceedings of Pacific Conference on Earthquake Engineering*, Australia, pp 225-244.

NOTATIONS

k	factor accounting for the degradation with increasing ductility
α	is the inclination of the line joining centres of compression of top and bottom faces of the column
A_g	gross area of section
A_c	area of core measured from the outside edge to outside edge of lateral reinforcement
A_v	area of lateral reinforcement
f_{yh}	specified yield stress of lateral reinforcement
f_y	specified yield stress of reinforcement
s, s_h	centre to centre spacing of lateral reinforcement
P, N, N^*	axial load
D', h''	core dimension of tied column, outside to outside edge of bar, perpendicular to the lateral reinforcement
V_f	shear force due to flexure with yielding at both ends of a section
f'_c, f'_{ca}	compressive cylinder strength of concrete
M_y	yield moment
$\phi_{0.8Mu}$	curvature at a post peak moment at 80% of the maximum moment capacity
ϕ_y	curvature at first yield
v_{jh}	nominal horizontal shear stress in joint core
f_y	yield strength of longitudinal reinforcement
f_{yh}	yield strength of horizontal joint reinforcement
D	section depth
d_b	nominal diameter of bar