0736

THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE WIDE BAND BEAM CONSTRUCTION

John S STEHLE[1], Kamiran ABDOUKA[2], Helen GOLDSWORTHY[3] And Priyan MENDIS[4]

SUMMARY

Reinforced concrete buildings designed with wide, shallow beams are an economically efficient form of insitu construction. In regions of high seismicity, however, designers are not permitted to use beams that are very wide compared to column width. This study investigates the performance of such a system in Australia, where this form of construction is allowed, and for which there are typically no detailing provisions for improved seismic performance. A sample 4 storey, 6 bay structure is analysed, and found to have satisfactory performance for a region of low seismicity. Also, a detailing strategy, which includes the debonding of reinforcing bars in the outer portions of the beam near the interior column joints, is recommended for regions of higher seismicity. The detailing strategy prevents torsional cracking from occurring. At the interior connection, it does, however lead to a different, more indirect load path being enforced, and hence checks for concrete crushing, serviceability deflection and P-Delta stability need to be made. Where P-Delta stability is a problem, additional stiffness may be provided by a structural wall or perimeter frame.

INTRODUCTION

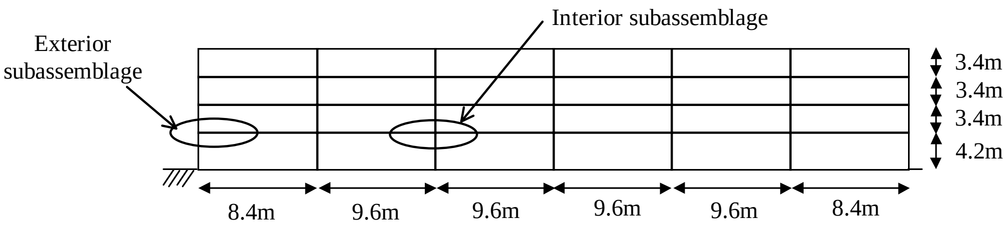
Beams that are very wide compared to the columns that they frame into are widely used in some regions of low seismicity. Australia is one such place, and these types of beams are often referred to as "band" beams. Band beams are popular for a number of reasons, the main ones being that inter-storey heights can be minimised and formwork simplified with no drop-panels required.

Band beams may not be used in regions of high seismicity due to restrictions on beam width specified in various codes [Abdouka and Goldsworthy 1997]. A number of researchers [Hatamoto et al. 1991; Gentry and Wight 1992; Popov et al. 1992; LaFave and Wight 1997; Quintero-Febres and Wight 1997] have investigated the performance of wide beam connections. Their research looked only at frames incorporating seismic detailing. This research differs in that there has been no special seismic detailing provided in the first series of connections tested, and for the second series of tests new detailing strategies are tested which improve the connections' seismic performance with a few unique changes to the detailing.

EXPERIMENTAL WORK

A four storey, six bay frame was designed according to current Australian design practice using a static earthquake force design method for a rock site with an effective peak ground acceleration of 0.1g, without the provision of extra detailing for seismic ductility. Dimensions are shown in Figure 1. Interior and exterior connection sub-assemblages of the frame at the first floor level were chosen for testing.

[1] *Dept. of Civil and Environmental Engineering, The University of Melbourne, email: stehle@civag.unimelb.edu.au*
[2] *Dept. of Civil and Environmental Engineering, The University of Melbourne, email: kamiran@civag.unimelb.edu.au*
[3] *Dept. of Civil and Environmental Engineering, The University of Melbourne, email: h.goldsworthy@civag.unimelb.edu.au*
[4] *Dept. of Civil and Environmental Engineering, The University of Melbourne, email: p.mendis@civag.unimelb.edu.au*



Bay width = 8.4 m in out of plane direction.
Beams: 2400mm wide, 400mm deep
Columns: 500mm square
Edge beams: 300mm wide, 600mm deep

Figure 1: Frame used in case study

Details of the first half scale interior test specimen are shown in Figure 3. Details for the first exterior specimen are similar (ref. XXXXX). A schematic diagram of the setup, showing how lateral and gravity loads are applied, is given in Figure 2.

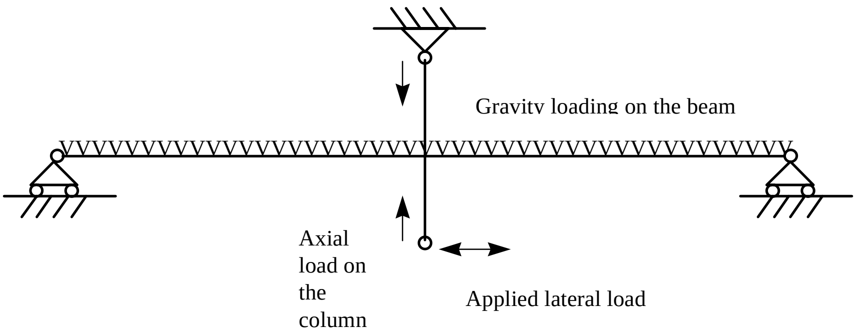


Figure 2: Schematic diagram of test setup

The first series of tests showed that torsional cracking was an important failure mode for both interior and exterior specimens. Failure is defined here as the development of crack widths greater than 1mm (for a half-scale specimen) in width at a drift ratio of 3% (note that 1% drift is equal to 19mm displacement of specimen). For such wide cracks, steel strains are very high and the possibility of steel fracture is of concern. The torsional cracks for the exterior connection were much worse than for the interior connection and were in the order of 5mm in width. The pullout of inadequately anchored bottom reinforcement also was a deficiency in the connections, with significant positive moment strength in the beams being unattainable. For the exterior connection, significant degradation of the subassemblage's overall strength occurred as a positive moment could not be sustained. The interior subassemblage's overall strength was not so drastically reduced as can be seen in Figure 5, in fact strength was maintained beyond the yield point due to the static redundancy of the connection.

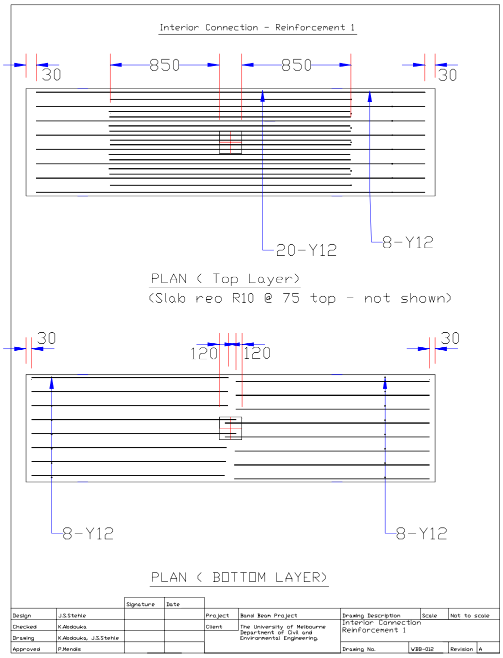


Figure 3: 1st interior specimen beam details

Figure 4: 1st interior specimen. Photo showing torsional cracking.

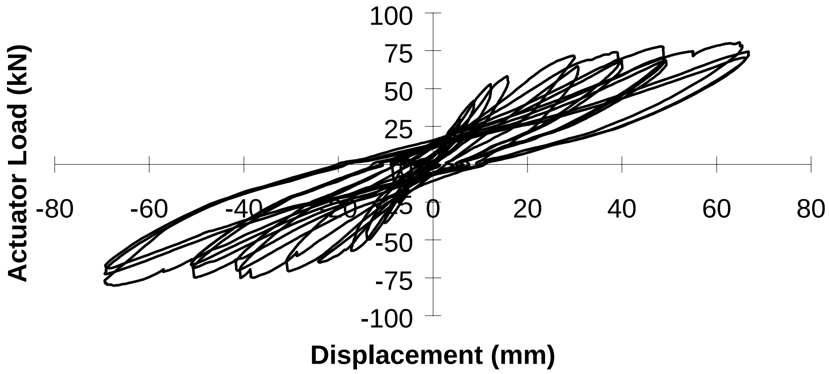


Figure 5: Hysteretic response of 1st interior specimen

The problems associated with the detailing adopted in the first series of tests were overcome with some very minor and cheap detailing modifications [Stehle et al. 1999]. In the exterior and interior connections, bottom bars were anchored to develop their full strength. For the exterior connection, beam reinforcement was concentrated closer to the column to reduce the torsional demand. The torsion on the side faces of the interior connection was avoided by debonding the beam bars in the outer width of the beam near the column joint region. This modification was the most radical as it is not a traditional method. Its impacts are discussed below. A third modification was made to the interior connection; ligatures were provided in the beam to provide extra shear strength for the reduced shear width of beam. Details of the second interior test specimen are shown in Figure 6. The performance of the interior specimen is summarised in Figure 7 and figure 8 where it can be seen that torsional cracking is completely avoided for the interior specimen. Torsional cracking in the exterior specimen was also controlled. The second exterior specimen also has improved strength for both directions of loading, since the beam now can develop significant positive moment capacity due to the bottom beam bars having better anchorage. Improved bottom beam bar anchorage in the interior specimen resulted in a higher specimen strength at high drift levels as shown in Figure 8.

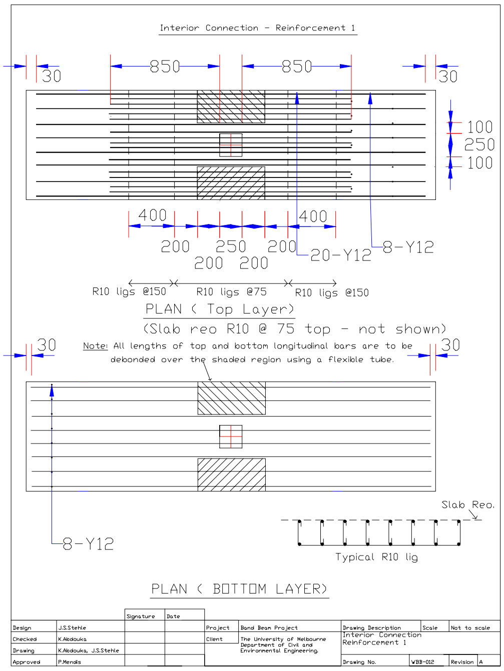


Figure 6: 2nd interior specimen beam detailing

Figure 7: 2nd interior specimen. Photo shows the absence of torsional cracking

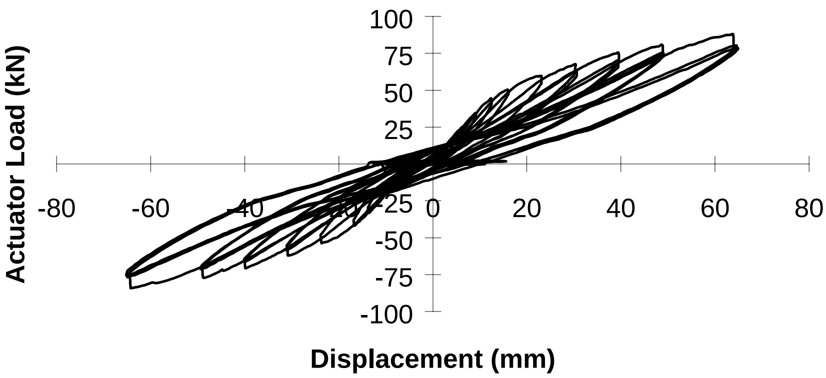


Figure 8: Hysteretic response of 2nd interior specimen

BAR DEBONDING

As the debonding of bars is a new approach, it requires special consideration. Normally in seismic design, engineers try to ensure that there is a strong bond between the concrete and the reinforcement. In this case, however, it is beneficial to have no bond between the bar and the concrete for a small region since, under these conditions, torsion cannot be developed on the joint's side face as can be seen in Figure 9. The out-of-balance moment over the debonded region must be zero since the bar force at each end of the debonding region is equal.

The bars are easily debonded by wrapping the bars with a flexible polyethylene tube. The tube does require some stiffness so that it will not buckle under the pressure of the wet concrete. It is desirable for the tube to be reasonably flexible however to prevent the tube from providing any significant structural strength. The ends of the tube require sealing with a suitable sealant so that wet concrete and water do not flow down the tube.

As a result of the outer bars being debonded, extra flexibility is introduced into the connection. The effect of this needs to be checked, especially with regard to deflections under serviceability gravity loading. A simple conservative calculation, using equation (1) shows that mid-span beam deflection will only increase by 3mm at most for the sample frame. A more rigorous analysis, taking into account a nonlinear moment-curvature relationship, shows that the deflection will increase by only approximately 2mm from an original deflection of 26mm (at full scale) with all bars bonded. With an allowable deflection of 36mm (L/250), such a small increase is not of concern.

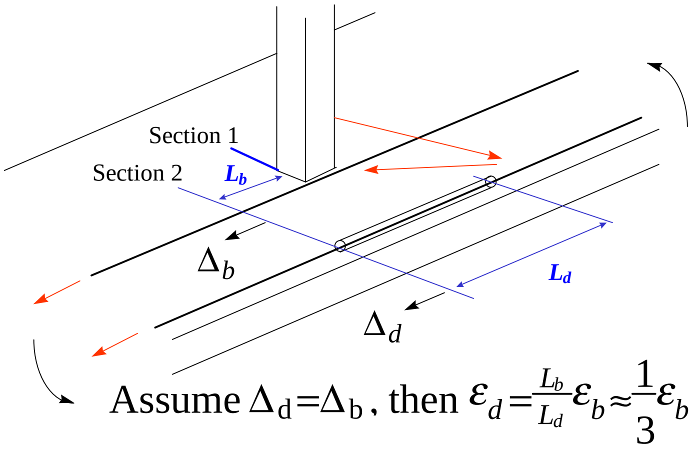


Figure 9: Diagram showing the new load path due to debonding

$$\Delta_{add} = \left(\varepsilon_{s,b1} - \frac{(\varepsilon_{s,b1} + \varepsilon_{s,b2})}{2} \right) \frac{L_b}{d} \frac{L}{2} \tag{1}$$

where,

Δ_{add} =additional deflection at midspan

$\varepsilon_{s,b1}, \varepsilon_{s,b2}$ =serviceability strain in beam bar passingthrough the column at sections 1 and 2 of beam when all bars are bonded (refer to figure 9)

L_b =length of bonded region from section 1 to section 2 in the beam (refer to figure 9)

L=clear beam span

d=depth to reinforcement

In terms of lateral loading, the serviceability deflection should also be checked. The lateral load may be due to wind or earthquake loading. A frame analysis may be adopted with a reduced beam stiffness for the length of the debonded zone. A value of stiffness can be calculated using equation (2). This expression is conservative for frame deflection if neither beam framing into the joint is subject to positive moment at the connection. Where one beam does have a positive moment, the equation appears to be a good approximation The expression is derived by assuming that the debonded reinforcement bar is being pulled from only one side of the joint, and that the section where the debonding ends has a uniform rotation for the full width of the beam (see Figure 9). Equation (2), is supported by observation of strain gauge readings on the beam reinforcement in the second test specimen. It was found that the strain in the debonded bars was approximately equal to one third of the strain in the bonded bars (see Figure 10) when there is a positive beam moment on one side of the joint, hence the debonded region has about one third of the stiffness of the bonded region. This one third ratio, is approximately equal to the ratio of the debonded length of bar to the bonded length of bar used in the test. For conservatism, cracked section stiffnesses should be used for all elements in the analysis, unless it can be otherwise justified.

For high drift levels, the debonded bars will eventually yield. Hence, to ensure that no column hinging occurs, the sum of the column moment capacities should be at least 1.4 times the sum of the moments of the beam moment capacities. Where it can be shown that the debonded beam bars will not yield until extremely high drift levels, then the full beam moment capacities may never be reached. In such cases, weaker columns may be used. For the second interior specimen this was the case. Column bar yielding did not occur until 3% drift which is not an acceptable drift level anyway so we can say that the column does not form hinges for a design earthquake. Accurately determining the drift at which column hinging occurs is a complex task, involving the consideration of nonlinear moment-curvature relationships, shear deformation and bar slip [Alsiwat and Saatcioglu 1992].

$$(EI)_{zone,eff,debonded} = (EI)_{zone,eff,bonded} \left(\frac{b_{bonded} + \frac{L_b}{L_d} b_{debonded}}{b} \right) \quad (2)$$

where,

$(EI)_{zone,eff,debonded}$ =Effective stiffness for debonded length of beam

$(EI)_{zone,eff,bonded}$ =Effective stiffness of beam when bonded

b=width of beam

b_{bonded} =width of beam over which bars are bonded

$b_{debonded}$ =width of beam over which bars are debonded

L_b =length of beam from column face to end of debonding zone

L_d =total length of debonding

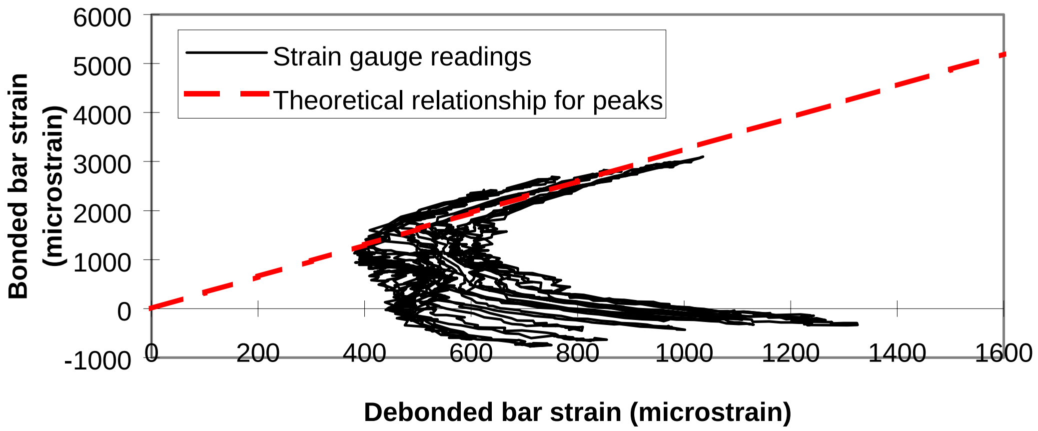


Figure 10: Comparison of bonded and debonded bar strains

Concrete crushing is another issue requiring consideration. Extra compression due to the debonded bars is applied to the compression regions in the beams either side of the joint since the tension in the bar is anchored beyond the joint, with the force being transferred to the joint region via a compressive strut (figure 9). There is only a limited concrete strut size possible in these regions to balance the tie forces. The dimensions of this region are given by the width of the bonded region by twice the depth of the reinforcement in compression. This is the maximum depth of the strut for which the debonded bars contribute zero moment to the far side of the joint. With a strut of these dimensions (450*50mm at half scale) and if all the beam bars yield then a compression stress of 44MPa is expected for the second interior specimen, taking into account the strength of the bars yielded in compression. This level of compression is above the measured compressive strength of the concrete which is 38MPa at 28 days. This is very high, since in strut and tie design, maximum compressive stresses of $0.36f'_c$ are usually allowed if no confinement steel is present. Since slab reinforcement and beam ligatures do provide some confinement, higher levels could be justified, up to $0.6f'_c$. During testing, it should be noted, that there was no evidence of concrete crushing. Also not all of the debonded bars yielded at the maximum 3.6% test drift. For design, a limit of $0.6f'_c$ seems prudent if beam ligatures and slab reinforcement bars are present.

P-Delta effects are very important in such flexible systems. Problems may be easily avoided by the use of a stiffer structural element such as a wall or perimeter frame. Nevertheless it is interesting to study the behaviour without such elements. Cho et al [Cho et al. 1998] provide a simple method of determining whether or not P-Delta effects are important. For the case study frame, it is found that P-Delta effects may be a problem, since P/V=70, and drift ratio=0.011 according to Australian design loads. The stiffnesses used for determining this drift, were the fully cracked stiffnesses of the members, and in the debonding region, the value given by equation (2), hence it is a conservative value of drift ratio.

ANALYTICAL WORK

A comprehensive series of time-history analyses were conducted on the sample 4 storey, 6 bay frame. The frame was modelled using "DRAIN2D", with beam elements being modelled by Takeda stiffness degrading elements, and column elements being modelled by beam-column elements which take into account the effect of axial load on moment strength, although this element has no stiffness degradation capability. The elements were calibrated to the experimental results, to ensure the overall hysteretic behaviour was well described. The structure's beams were subdivided into 6 elements, to account for variation in beam properties. Subdividing the beam allows hinges to occur within the beam span which is a possibility depending on the level of gravity load. Also a beam element, that bypassed the joint, was incorporated to model the portion of beam that was effective under gravity loads. In reality, the full width of the beam becomes effective under lateral loads also, however, this does not occur until very high drifts, so the strength increase was simply modelled as some extra strain-hardening in the plastic hinge. In general, element stiffnesses were chosen that were equal to or slightly greater than the fully cracked stiffness. For a design check of the structure, where nonlinear time-history analysis is employed, it would be conservative in terms of maximum drift and P-Delta effects to use the fully cracked section stiffness for all elements.

Over 10,000 earthquake records were examined from a database for suitability. Only corrected, free-field records, with simultaneous horizontal and vertical acceleration components were chosen with magnitudes and epicentral distances within the ranges shown in table 1. A set of records for rock sites and for soft soil sites was attained. An arbitrary level of seismic risk was allocated depending on the earthquake Richter magnitude, epicentral distance combination and the soil conditions. Level I was given for low seismicity, up to Level IV for very high seismicity (see Table 1). In total, 62 records were used from 31 recording stations during 24 events in 7 countries. The analysis for each of these records was performed for each horizontal component of the record separately when both components were available.

Table 1: Categorisation of earthquake records

	Rock (UBC Classification S1, S=1)				Soft Soil (UBC Classification S4, S=2)			
Richter	Epicentral distance (km)				Epicentral distance (km)			
Magnitude	10-30	30-70	70-120	120-500	10-30	30-70	70-120	120-500
4.5-5.5	**I**	-	-	-	**II**	-	-	-
5.5-6.5	**II**	**I**	-	-	**III**	**II**	-	-
6.5-7.5	**III**	**II**	**I**	-	**IV**	**III**	**II**	-
7.5-8.5	**IV**	**III**	**II**	**I**	-	**IV**	**III**	**II**

I Low seismicity **III** High seismicity
II Moderate seismicity **IV** Very high seismicity

The results of the analyses are summarised in Figure 11, according to the seismic risk. It is seen that the arbitrary classification of seismic risk gives a meaningful classification of results in terms of inter-storey drift. The low seismic risk category had less than 1% inter-storey drift. At this level of drift, no structural damage is expected according to the experimental results. Since no damage occurs for a low seismic event, it appears that even detailing, such as that adopted in the first series of tests is adequate. However, due to the possibility of a larger event occurring, it is recommended that minor detailing changes be made. These will involve little cost and will lead to a marked improvement in performance. They include the full anchorage of bottom reinforcement bars, as well as the provision of extra shear reinforcement in the columns, to ensure a catastrophic, brittle failure does not occur. The full detailing strategy that was adopted for the second series of tests is recommended only for higher seismic zones.

The other risk categories represent what may happen in events more extreme than provided by the code. It shows that the structure may still be adequate for a medium seismic event, however, for higher seismicity, there would be doubt for the structure's integrity. For regions with greater risk, the initial design should accommodate higher seismic forces, so improved performance could be expected for a high seismic event if designed correctly.

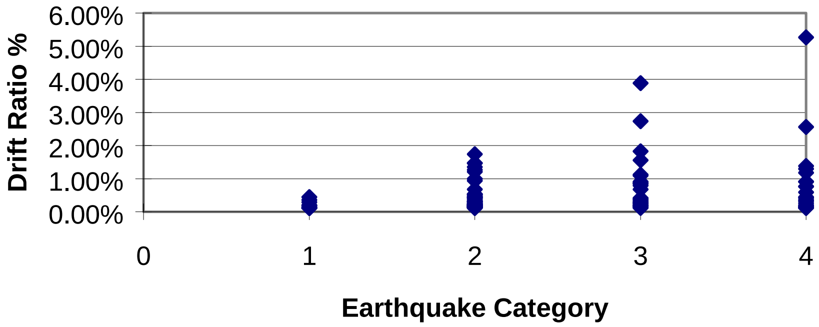


Figure 11: Peak drift ratio response for 4 storey, 6 bay frame vs. earthquake category

CONCLUSIONS

For regions of low seismicity such as Australia, very few detailing changes are required for wide beams to give satisfactory performance. All that is recommended is that all bottom beam bars are anchored to develop their full yield capacity at the beam supports, and for column shear reinforcement to be provided to prevent shear failure in the case of a soft-storey mechanism forming.

For regions of higher seismicity, the use of wide beams is advocated if the relevant design checks have been made relating to torsional cracking at the exterior connection, and for interior connections where bar debonding is employed: P-Delta stability, concrete crushing, and serviceability deflections. For most cases, it would be preferable to use such a flexible frame as a secondary seismic restraint.

REFERENCES

1. Abdouka, K. and Goldsworthy, H. (1997). Seismic Effects on Wide Band Beam Construction in Australia: Review. *15th Australasian Conference on the Mechanics of Structures and Materials*, Melbourne, Australia, Balkema,: 311-316.

2. Alsiwat, J. M. and Saatcioglu, M. (1992). “Reinforcement anchorage slip under monotonic loading.” *ASCE, Journal of Structural Engineering* 118(9): 2421-2438.

3. Cho, J. H., Moss, P. J. and Carr, A. J. (1998). Assessment of P-Delta effects in framed structures subjected to earthquake motions. *Australasian Structural Engineering Conference*, Auckland, New Zealand,: 705-712.

4. Gentry, T. R. and Wight, J. K. (1992). *Reinforced Concrete Wide Beam-Column Connections Under Earthquake-Type Loading* The University of Michigan USA.UMCEE 92-12

5. Hatamoto, H., Bessho, S. and Matsuzaki, Y. (1991). *Reinforced Concrete Wide-Beam-to-Column Subassemblages Subjected to Lateral Load*. Design of Beam-Column Joints for Seismic Resistance. Detroit Michigan USA, ACI Publications SP-123**:** 291-316.

6. LaFave, J. M. and Wight, J. K. (1997). *Behaviour of Reinforced Concrete Wide Beam-Column-Slab Connections Subjected to Lateral Earthquake Loading* The University of Michigan USA.UMCEE 97-01

7. Popov, E., Cohen, E., Koso-Thomas, K. and Kasai, K. (1992). “Behaviour of Interior Narrow and Wide Beams.” *ACI Structural Journal* 89(6): 607-616.

8. Quintero-Febres, C. G. and Wight, J. K. (1997). *Investigation on The Seismic Behaviour of RC Interior Wide Beam-Column Connections* The University of Michigan USA.UMCEE 97-15

Stehle, J. S., Abdouka, K., Goldsworthy, H. and Mendis, P. (1999). The Seismic Performance of Reinforced Concrete Frames with Wide Band Beams. *Second International Symposium on Earthquake Resistant Engineering Structures*, Catania, Italy, WIT Press,: 113-122.