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New Zealand Tests on Full-Scale Reinforced Concrete Beam-Column-Slab Subassemblages Designed for Earthquake Resistance

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Synopsis: As part of a United States/New Zealand/Japan/China collaborative research project, interior and exterior beam-column joint subassemblages with floor slabs of prototype two-way and one-way reinforced concrete building frames were designed for earthquake resistance using the current New Zealand concrete design code, NZS 3101:1982. Three full-scale subassemblages as designed were constructed and tested under quasi-static cyclic loading which simulated severe earthquake actions. The overall performance of each subassemblage during the tests was satisfactory in terms of strength and ductility. The joint core and column remained essentially undamaged while plastic hinges formed in the beams. The strong column-weak beam behaviour sought in the design, desirable in tall ductile frames designed for earthquake resistance, was therefore achieved. Although the joint cores of the subassemblages remained in the elastic range, joint core shear deformations contributed significantly to the interstorey drifts. Also, a significant proportion of the slab bars in tension contributed to the negative moment flexural strength of the beams. The performance of the one-way joint was superior to the performance of the two way joints.

Keywords: Beams (supports); capacity, columns (supports); cyclic loads; ductility; earthquake-resistant structures; frames; hinges (structural); joints (junctions); reinforced concrete; slabs; tests

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INTRODUCTION

Three full-scale reinforced concrete beam-column joint subassemblages with floor slabs were designed in compliance with the current New Zealand concrete design code for ductile moment resisting frames and tested under simulated earthquake loading. The structural dimensions and the quasi-static cyclic loading history followed the guidelines of the United States/New Zealand/Japan/China collaborative research project on the seismic design of reinforced concrete beam-column joints, as agreed at meetings of the principal investigators of the project held in the United States in 1984, Japan in 1985, New Zealand in 1987 and the United States in 1989.

In New Zealand a strong column-weak beam concept is used in the design of tall ductile moment resisting frames. The required flexural strength of the plastic hinge regions in the beams and column bases is first determined using the combinations of design seismic and gravity loadings. A capacity design procedure is then used to calculate the required shear strengths of beams, columns and joints, and the required flexural strengths of columns, with the aim of ensuring that inelastic deformations occur only at the chosen plastic hinge regions (1,2,3). The design forces for shear and column flexure are derived considering the possible flexural overstrengths of the plastic hinge regions of the beams, based on the as detailed reinforcing layout and the possible strengths of the longitudinal reinforcing steel taking into account yield strengths greater than specified and strain hardening. In addition, the effects of higher modes of vibration and of seismic loading acting concurrently along the two axes of the building are taken into account when determining the column actions. In the three subassemblages tested, plastic hinges were designed to form in the beams at the column faces and the beams, columns and joints were reinforced accordingly.

In the joint core of each of the three subassemblages, reinforcement was placed to resist the total design horizontal shear forces acting across the joint core, as is required by the New Zealand concrete design code (1) when the axial load level on the columns is less than $0.1f'_cA_g$. Thus a considerable quantity of horizontal hoops was necessary in each joint core. A smaller amount of transverse reinforcement would be required in the joint core if the ACI building code (4) was followed. The New Zealand code (1) requirement is based on the observation that, when the axial load on the column is low, at large inelastic cyclic displacements the shear resistance provided by the diagonal concrete compression strut across the joint core diminishes, while that by a truss mechanism formed by the joint core reinforcement becomes dominant (2,3,5,6,7). When the axial column load is large this degradation of the diagonal compression strut mechanism is not so marked. To examine the worst case of joint core behaviour, no axial compression was applied to the upper columns of the subassemblages tested.

An important feature in the New Zealand code (1,2) is that the longitudinal slab bars within a prescribed width of slab on each side of the column can be counted on to act as part of the negative moment tension reinforcement of the beam. To study this slab effect, the layout of the slab reinforcement in each of the subassemblages followed closely that of prototype construction.

The aim of the tests was to examine the behaviour during major earthquake loading of subassemblages designed according to the New Zealand concrete design code (1), for comparison with the results obtained from all four countries of the collaborative research project. A particular aspect of interest was the effects of the presence of transverse beams and floor slabs on the behaviour of the subassemblages. Previous tests in New Zealand (5,6,7) have involved beam-column joint subassemblages without floor slabs and primarily of plane (one-way) frames.

TEST PROGRAMME

Description of Test Units

Each of the three subassemblages tested represented types of beam-column joints in three-dimensional moment resisting frames of buildings. The three subassemblages were as follows:

Unit 1D-I : Interior beam-column-slab joint of a one-way frame.
Unit 2D-I : Interior beam-column-slab joint of a two-way frame.
Unit 2D-E : Exterior beam-column-slab joint of a two-way frame.

Details of the three Units, as designed, are shown in Figs. 1 to 3. The Units simulated full-scale subassemblages of frames with 3.5 m (11.5 ft) interstorey height. The beam shear span provided in the Units was only about two-thirds of the distance to the midspan of an assumed prototype which had a beam span of 6 m (19.7 ft). Also, the cantilever span of the one-way slab of Unit 1D-I, which modelled the topping slab of a ribbed floor system, was slightly less than one-third of an assumed prototype which had a 6 m (19.7 ft) span between two frames.

The reinforcing details are shown in Figs. 1, 2 and 3. The reinforcement ratios are listed in Table 1 and the measured properties of the reinforcing steel and concrete are shown in Table 2 and 3.

Design Features of Test Units

The main features of the design of the Units are summarized below.

In all Units the longitudinal reinforcement in the webs of all east-west beams (east beam only in Unit 2D-E) were kept identical. This enabled a direct comparison of the behaviour of exterior (Unit 2D-E) and interior (Units 1D-I and 2D-I) joints.

The ratios of the diameter of the longitudinal beam bars to the column depth was d_b/h_c = 1/25 for all Units. The ratio of the diameter of longitudinal column bars to the beam depth was d_b/h_b = 1/22.9, 1/19.6 and 1/19.6 for Units 1D-I, 2D-I and 2D-E, respectively. The New Zealand code (1) requirement for these ratios is $d_b/h_c \leq 1/25$ and $d_b/h_b \leq 1/20$.

The design shear forces for the beams and columns of the Units were calculated assuming that all the longitudinal beam and slab bars in tension were stressed to 1.25 times the specified yield strength of the steel. That is, to 1.25 x 275 MPa (1.25 x 40 ksi) in these Units. Generally, the quantity of transverse reinforcement in the beams and columns was governed by the limitations on the maximum spacing of transverse reinforcement for concrete confinement and for lateral restraint of compression reinforcement.

The design shear forces for the beam-column joint cores were also calculated assuming that the effective longitudinal reinforcement in the beams was stressed to 1.25 times the specified yield strength of the steel. The design horizontal and vertical shear forces so calculated are listed as V_{jh} and V_{jv} in Table 4 and are compared with the shear resistance provided by the horizontal and vertical joint core reinforcement, V_{sh} and V_{sv}. It should be noted that the intermediate longitudinal bars of the column were considered to be effective in resisting vertical shear in the joint cores. For horizontal shear, the joint hoops present in each Unit provided resistance almost identical to the design shear force.

To predict the theoretical nominal (ideal) strengths of the Units, the nominal (ideal) flexural strengths of the beam sections were calculated using the measured material properties of the concrete and steel. The New Zealand code (1) method was adopted which, like the ACI code (4), assumes an equivalent rectangular compressive stress block for concrete with a mean stress of $0.85f'_c$ and a maximum concrete compressive strain of 0.003. Since the floor slabs and beams were cast monolithically, they were expected to act integrally as T-beams. For negative bending moment producing tension in the top bars, the New Zealand code (1,2) considers that all longitudinal bars placed within a specified flange width act as tension reinforcement for the beam. The flange width is of varying magnitude depending on the structural configurations, and is illustrated in Fig. 4. These New Zealand code recommendations were followed in the theoretical calculations. In addition, for T-beams subjected to positive bending moment under seismic conditions, the effective width of the flange in compression was assumed in all cases to be twice the column width. Calculations showed that for positive bending moment, larger slab widths did not increase the moment capacity significantly. Flexural strength values estimated using other assumptions have been discussed in Refs. 8 and 9. The ratio of column flexural strengths to beam flexural strengths, obtained using the New Zealand code assumptions for effective widths of slabs, are listed in Table 5.

The nominal (ideal) strength of each Unit subjected to seismic loading is attained when a positive moment plastic hinge occurs in the beam at one face of the column and a negative moment plastic hinge occurs in the beam at the other face of the column, as in Fig. 5(b). In the figures showing the measured lateral load-lateral displacement hysteresis loops for the Units (Figs. 12, 13 and 14), this theoretical ideal strength is denoted as V_i when the New Zealand code recommendations are followed for the calculation of the width of flange in tension contributing to the negative moment strength of the beam. For comparison, those figures also show a theoretical ideal strength denoted as V_i^* which is calculated assuming that the longitudinal slab bars over the full width of the slab yield in tension and contribute to the negative moment flexural strength of the beam.

Loading Rig and Test Procedure

The Units were extensively instrumented in order to gather as much information as possible on the load-displacement responses, joint distortions, beam curvatures and strains in the reinforcing bars. Full details of the instrumentation can be seen in Refs. 8 and 9.

Figure 5(b) shows the manner of loading the Units by displacing the beam ends. Figure 6 shows a schematic configuration of the loading rig, which was designed so as to apply either unidirectional or bidirectional (i.e. orthogonal east-west and north-south) simulated seismic loading. Pins were provided in the two directions to enable the top and bottom ends of the columns of the Units to rotate in the loading directions. The beam ends, where the double-acting jacks imposed vertical loads, were also free to rotate and move laterally. Figure 7 shows a Unit during testing.

By considering the geometrical relationships shown in Fig. 5(b), the equivalent interstorey displacement Δ_c and interstorey shear V_c can be derived from the applied beam vertical loads, P_1 and P_2, and corresponding displacements, Δ_{B1} and Δ_{B2}, using the following expressions:

$$\Delta_c = \left(\frac{\Delta_{B1} + \Delta_{B2}}{\ell_1 + \ell_2} \right) \ell_c \qquad (1)$$

$$V_c = \left(\frac{P_1 \ell_1 + P_2 \ell_2}{\ell_c} \right) \qquad (2)$$

where the notation is as shown in Fig. 5(b).

During the tests, continuous corrections to the overall measured displacements were necessary to allow for the flexibility of the steel loading rig which caused some horizontal movement of the top of the Units.

Loading Sequence

The "first-yield displacement" $\Delta_{y,test}$ was determined by extrapolation from the measured displacement at 75% of the ideal strength V_i of the Unit, as shown in Fig. 8.

The New Zealand loadings code (10) permits a maximum interstorey drift (Δ_e/ℓ_c in Fig. 5(a)) of 0.32% computed for the frame assuming elastic behaviour under the code seismic loading. The experimentally measured interstorey drifts when the displacement ductility factor was μ = 1 (found from Δ_c/ℓ_c, where Δ_c is given by Eq. 1 with Δ_{B1} and Δ_{B2} given by the first yield displacement as defined in Fig. 8) was 0.45% for Unit 1D-I, 0.47% for Unit 2D-I and 0.35% for Unit 2D-E.

The quasi-static cyclic loading histories followed for the three Units are depicted in Figs. 9 to 11. In each test, the first two load cycles were to an imposed lateral load of about one-half of V_i. The first yield displacement $\Delta_{y,test}$, and the measured stiffness K_{test}, were determined in the first half of the third load cycle. Subsequent cycles were displacement controlled with increasing imposed displacement ductility factors $\mu = \Delta/\Delta_{y,test}$ and enabled observation of the performance of the Units at low as well as high ductilities. According to the New Zealand loadings code (10), a ductile structure should be able to undergo four cycles of loading to a displacement ductility factor of four in each direction, implying a cumulative displacement ductility factor demand of $\Sigma\mu = 32$. Furthermore, after those four cycles, the reduction in strength of each individual component should not exceed 30%, while that of the whole structure should not exceed 20%.

For Unit 2D-E (Fig. 11), loading was applied principally in the east-west direction (that is, perpendicular to the spandrel beam). The values for $\Delta_{y,test}$ and K_{test} were obtained for when the east beam was displaced downwards and were significantly different from those found for Unit 2D-I. In order to make a more meaningful comparison of the behaviour of Unit 2D-E with that of the other two Units, the subsequent imposed displacements for Unit 2D-E were to similar interstorey drift levels as for the other two Units.

TEST RESULTS

General Observations

The experimental (measured) hysteresis loops for column lateral load (storey shear) versus lateral displacement are shown in Figs. 12 to 14. For Units 2D-I and 2D-E both the north-south and the east-west load-displacement responses are given.

All three Units performed extremely well during the tests and easily satisfied the New Zealand loadings code (10) performance criterion for a ductile structure. Plastic hinges formed in the beams at the column faces when the Units were loaded into the inelastic range. Each column developed only fine cracks and remained in the elastic range. Figures 15 to 17 show overall views of the Units during testing when large ductilities had been imposed. Although in the joint core regions diagonal cracks and sometimes spalling of concrete surfaces were observed, the joint cores remained essentially intact.

Lateral Load-Displacement Response

For all Units, although there was a gradual degradation of stiffness during the testing, the shape of the hysteresis loops remained reasonably stable up to interstorey drifts of at least 3%. For Unit 2D-I the particularly flat and pinched curve for run 36 up to an interstorey drift of 4% (see Fig. 13(a)) was caused by some slippage of the east-west bottom beam bars. In other cases the pinching of loops which occurred was mainly associated with some buckling of compressed beam bars after cover concrete had spalled. Deterioration of the Units became significant only when the interstorey drift was well in excess of 3%.

The theoretical strengths V_i of the Units , which assumes an effective flange width in tension equal to that recommended in the New Zealand code (see Fig. 4), were consistently exceeded during the tests. The sources of strength enhancement were mainly from the strain hardening of the reinforcing bars in tension and from contributions of slab bars over a greater width than assumed in the New Zealand code. The theoretical maximum strength V_i^*, which assumes an effective flange width in tension equal to the entire width of slab, is also plotted in the figures. Evidently, since strain hardening of all the longitudinal reinforcement in the beams was inevitable, not all slab bars over the entire width of slab were contributing to the strength enhancement.

A feature of the hysteretic responses of the two-way Units 2D-I and 2D-E (see Figs. 13 and 14) was the reduction in peak load capacity by as much as 20% while the predetermined displacement ductility was being applied in the perpendicular loading direction. It is believed that apart from the effect of creep in the Units, this strength reduction was primarily caused by the changes in the contributions of the reinforcement in the slab to the flexural strengths of the beams while the loading was being applied in the perpendicular direction (11).

Comparison of the measured hysteretic responses of the three Units (Figs. 12 to 14) shows that the performance of the one-way interior (Unit 1D-I) was superior to that of the two-way exterior joint (Unit 2D-E), which in turn was superior to that of the two-way interior joint (Unit 2D-I). This observation is of interest since the joint regions of the three Units were similarly designed (Units 1D-I and 2D-I had identical transverse reinforcement). The effects of unidirectional and bidirectional loading, and of the presence of transverse beams and slab on the joints, are discussed in more detail elsewhere (11).

The contributions to the displacement of the Units were analysed as the sum of the deformations from the beams, columns and joint cores. In the tests, rotations in each beam over a length of 1.5 times the depth of beam from the column face were measured. This measurement included the plastic hinge rotations and the effect of beam bar deformations in and slip through the joint core. The elastic deformations over the remaining length of the beam and of the column were considered less significant and were estimated by calculations based on traditional elastic theory methods (5). For joint core shear distortions, experimental measurements were made only for Units 1D-I and 2D-E (north-south). Figure 18 shows the percentage contributions of these various deformation components for Unit 1D-I. Results for the other Units were similar. As expected, the major source of interstorey drifts were from beam plastic hinge deformations. Nevertheless, an important observation from Fig. 18 is that joint core shear deformations were significant. Joint core shear deformations accounted for about 26% of total interstorey drift at a displacement ductility factor $\mu = 2$. This proportion decreased to less than 20% at higher ductilities and became reasonably constant. Hence, although adequate transverse reinforcement was provided for the joint cores to remain mainly in the elastic range, it is evident that joints were far from rigid.

Strains in Reinforcing Bars

Typical strain variations in the horizontal joint hoops are illustrated in Figs. 19 and 20. The imposed ductilities are shown circled and a * on the ductility indicates bidirection loading. Figures 19 and 20 indicate a gradual and consistent increase in tensile strains as the level of imposed ductility increased. Hence the role of the hoops in resisting joint core shear forces through a truss mechanism, and in providing confinement of the joint core, became more significant as the ductility levels increased. In particular, type E legs (Fig. 19) and type B legs (Fig. 20) exhibited larger strains than the other types, evidently because they were subjected to greater bond forces from the beam bars. It is also noted that the hoop strains increased during imposed bidirectional loading (compare, for example, results for $\mu = \pm 3$ and $\pm 3*$ in Fig. 20). This would have been because, for example, the loading in the north-south direction would have caused dilation in the joint core with the formation of diagonal cracks. The east-west hoop legs were then required to provide the necessary containment, thus resulting in additional tensile strains. Hence the transverse beams did not provide significant confinement to the joint core. It is of interest that the joint core hoops generally did not reach yield. Yield was only reached at high displacement ductility factors in some hoops.

Strains were measured on the longitudinal beam and column bars in the joint cores. The measured strains indicated that those bars were adequately anchored. For the column bars away from the column corners of Unit 2D-I during unidirection loading, tensile strains less than the yield strain but of significant magnitudes were measured within the joint core. The participation of these bars in resisting vertical joint shear, analogous to the horizontal hoops, is therefore evident. Considerably higher strains generated in the column bars under bidirectional loading indicate that the column bars in the joint core were subjected to a more severe tensile stress state during bidirectional loading.

Strains measured on longitudinal beam bars and the slab bars during the tests were converted to stresses using a computer program (12) based on the Ramberg-Osgood stress-strain model. Figure 21 shows for example the stresses in some slab bars of Unit 2D-I. It is significant that a large proportion of those slab bars have tensile stresses, and the width of the slab with bars in tension exceeds that recommended by the New Zealand concrete design code (1,2), shown in Fig. 4. The mechanisms of slab contributions to the behaviour of beam-column assemblies is discussed in a separate paper (11).

CONCLUSIONS

1. The performance of the three beam-column-slab subassemblages, Units 1D-I, 2D-I and 2D-E, which were designed

according to the New Zealand concrete design code, provisions for ductile moment resisting frames, was very satisfactory during the cycles of quasi-static cyclic loading which simulated severe seismic loading. The performance of the one-way interior joint (Unit 1D-I) was superior to that of the two-way exterior joint (Unit 2D-E), which in turn was superior to that of the two-way interior joint (Unit 2D-I).

2. The very good performance of the beam-column-slab subassemblages was considered to have resulted from the relatively large quantity of joint core shear reinforcement provided, and the use of sufficiently small diameter longitudinal beam bars to avoid excessive slippage through the joint cores when plastic hinges developed in the beams at the column faces.

3. There was no evidence during the tests to indicate that the presence of floor slabs or beams in two directions provided significant confinement to the joint cores during bidirectional seismic loading.

4. Although the beam-column joint cores remained essentially in the elastic range during the tests, the contributions of joint core shear deformations to the interstorey drift of each Unit was significant and was as high as 26% of the total interstorey drift, reducing to about 20% of the total at large ductilities.

5. A significant proportion of slab bars in tension contributed to the negative moment flexural strength observed in the beams and hence to the enhancement of the flexural strength of each Unit. The lateral load strength of the Units was up to 39% higher than that calculated using the effective flange widths in tension assumed in the New Zealand concrete design code. In this calculation the measured material strengths and a strength reduction factor ϕ of unity were used. Some of this strength enhancement was due to strain hardening of the steel.

6. The strengths and stiffnesses of the Units reduced when bidirectional loading was applied, due to changes in the contributions of slab reinforcement.

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