SEISMIC BEHAVIOUR OF EXISTING MOMENT-RESISTING FRAMES WITH PLAIN ROUND REINFORCING BARS DESIGNED TO PRE-1970S CODES

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SUMMARY

Three as-built full-scale beam-column joints representing an existing reinforced concrete moment-resisting frame structure constructed in the 1950s in New Zealand were tested under simulated seismic loading. The three units were reinforced by plain round bars. One was an interior beam-column joint and the other two were exterior beam-column joints which were identical to each other but with alternative beam bar hook arrangements in the exterior columns. The beam-column joints when tested generally displayed low attainment of structural stiffness and strength. The low attainment of stiffness and strength was attributed to the slip of the plain round longitudinal bars through the joint core for the interior beam-column joint, and was attributed to the premature failure associated with the interaction of column bar buckling and the opening of the beam bar end hooks for the exterior beam-column joints, irrespective of the beam bar hook details. Compared to the deformed bars, the use of plain round longitudinal bars, although suppressing the joint and member shear failure, resulted in reduced structural stiffness and strength.

INTRODUCTION

There has been increasing emphasis in many countries on seismic assessment of existing reinforced concrete structures designed to the pre-1970s seismic codes in recent years [Aoyama, 1981; ATC, 1989; Hakuto, Park, and Tanaka, 1995]. This is a consequence of the significant advances in seismic design procedures since about the mid-1970s [Park and Paulay, 1975; Standards New Zealand, 1995] and of the observed severe damage to existing reinforced concrete structures, compared to those designed to current design codes [Park, et al, 1995].

A research program on Seismic Assessment and Retrofit of Existing Reinforced Concrete Structures has been under way at the University of Canterbury for many years, sponsored by Earthquake Commission of New Zealand. A number of other as-built reinforced concrete columns and beam-column joint assemblies [Hakuto, Park, and Tanaka, 1995; Rodriguez and Park, 1994; Wallace, 1996] have been tested under simulated seismic loading. Although the tests on the columns used plain round longitudinal bars [Rodriguez and Park, 1994], the previous tests on the beam-column joints used deformed bar reinforcement [Hakuto, Park, and Tanaka, 1995]. Plain round bar reinforcement was used in New Zealand until about the mid-1960s when deformed bar reinforcement became widely available. Compared to deformed bars, plain round bars have very low bond strength and the inadequate bond strength between the longitudinal reinforcement and the concrete may affect the probable seismic behaviour of reinforced concrete beam and column members and beam-column joints. Current seismic assessment procedures, which are based on the experimental information obtained from tests with deformed bar reinforcement, assume perfect bond between the longitudinal reinforcement and the concrete. The information on the effect of the use of plain round longitudinal bars on seismic behaviour of existing reinforced concrete structures is scarce and is urgently needed.

The aim of the research work reported here is to obtain the information on the possible seismic behaviour of existing reinforced concrete frame buildings reinforced by plain round bars, with the emphasis on investigating

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the effects of the utilisation of plain bar reinforcement on the attainment and maintenance of strength and stiffness.

This paper reports the results of the tests on three as-built beam-column joints reinforced by plain round bars. The effects of the plain round longitudinal bars used on the overall seismic behaviour of existing beam-column joint subassemblies and on the local behaviour of beam and column members and beam-column joints are outlined. The tested units were identical to Hakuto's test units [Hakuto, Park, and Tanaka, 1995] except that Hakuto's test units used deformed bar reinforcement.

DETAILS OF THE TEST UNITS

Three full-scale one-way beam-column joint units were constructed. One was an interior beam-column joint unit and the other two were identical exterior beam-column joint units but with alternative arrangements of the beam bar hooks in the exterior columns. Each unit was reinforced by plain round longitudinal and transverse reinforcement and the other reinforcement details were typical of 1950s construction in New Zealand. The interior beam-column joint unit is referred to as Unit 1. The exterior beam-column joint unit with the beam bar hooks bent away from the joint core in the exterior column is referred to as Unit EJ1. For this unit, the straight extension of the beam bars beyond the bends was four times the bar diameter, as was typical of pre-1970s construction in New Zealand. The exterior beam-column joint unit with the beam bar hooks bent into the joint core in the exterior column is referred to as Unit EJ2. For this unit, the straight extension of the beam bars beyond the bends was twelve times the bar diameter, as is current practice NZS3101:1995. The overall dimensions and reinforcing details of Units 1, EJ1 and EJ2 are shown in Figs.1, 2 and 3. These three units were respectively identical to the as-built units, O1, O7 and O6, tested by Hakuto [Hakuto, Park, and Tanaka, 1995] except that Hakuto used deformed longitudinal bars.

The concrete was normal weight. Each unit was cast in one stage in the horizontal plane. The concrete compressive cylinder strengths at the time of testing were 44 MPa, 34 MPa and 29 MPa for Units 1, EJ1 and EJ2 respectively. The properties of the reinforcement were the same for all three units. For the longitudinal bars, the bar diameter was 24 mm and the yield strength was 321 MPa. For the transverse reinforcement, the bar diameter was 6 mm and the yield strength was 318 MPa.

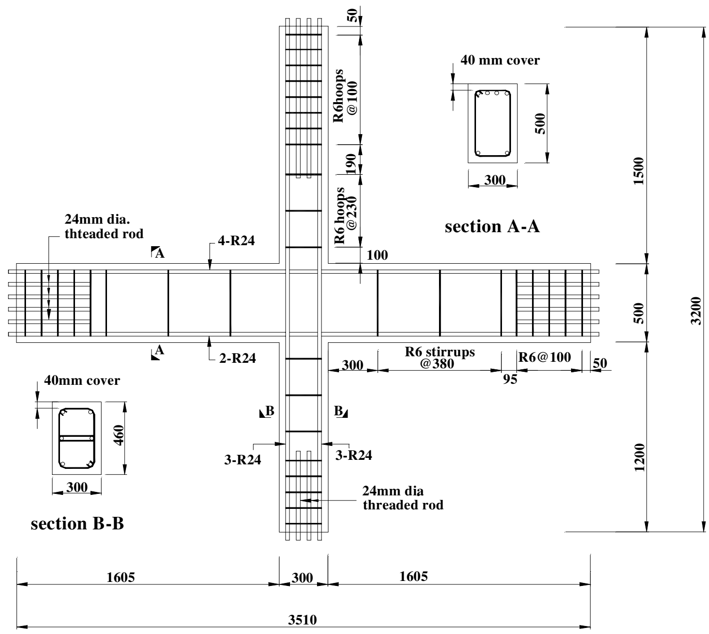
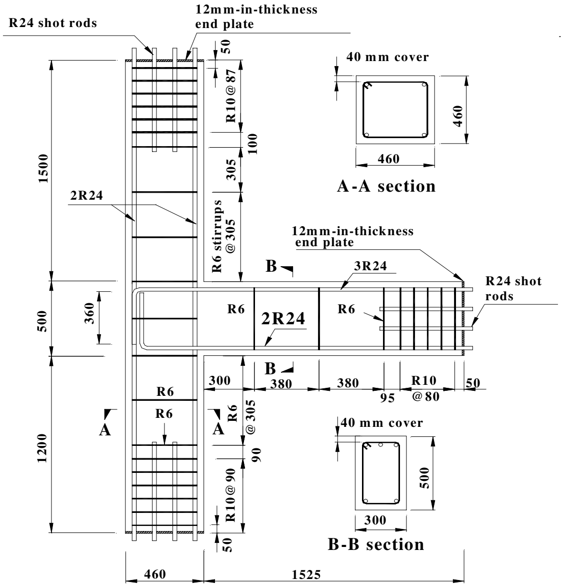


Fig. 1 Reinforcement Details of Unit 1



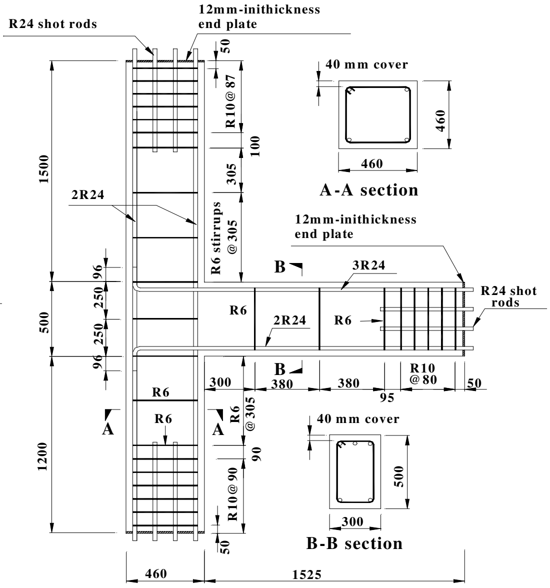


Fig.2 Reinforcement Details of Unit EJ1 **Fig.3 Reinforcement Details of Unit EJ2**

The member flexural strengths of the beams and columns of each unit were calculated assuming no bond degradation and using the measured material strengths, the code approach recommended by NZS3101: 1995 and a strength reduction factor ϕ of unity. The resulting theoretical storey horizontal load strength was 80 kN for Unit 1, which was governed by the column moment flexural strength, and it was about the same for Units EJ1 and EJ2, being about 67 kN when governed by the beam negative moment flexural strength, and 45 kN when governed by the beam positive moment flexural strength.

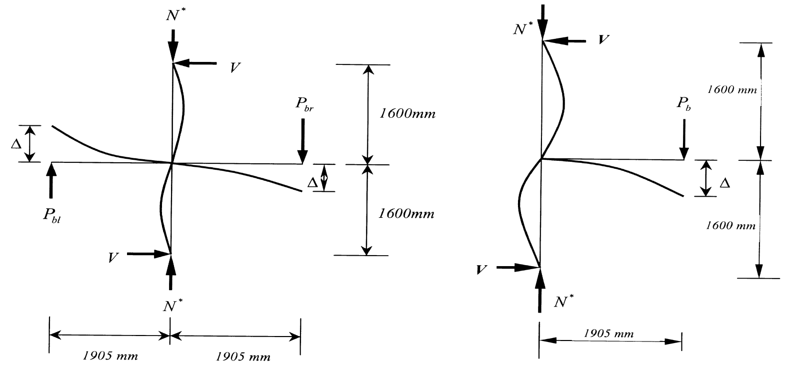
For each unit, the quantities of transverse reinforcement in the members were investigated using the method of NZS3101: 1995, and the quantities of transverse reinforcement in the joint were investigated using the method proposed by Park [Park 1997] because NZS3101: 1995 has not a method for calculating the shear strength of existing beam-column joints. For Unit 1, the beam shear strength was adequate, but the estimated shear strengths in the columns and the beam-column joint were respectively only 50% and 55% of the imposed shear forces on the columns and the joint when the theoretical storey shear strength of the unit was developed. For Units EJ1 and EJ2, the column shear capacities were adequate for both units, but the estimated shear strength in the beam was only about 20% of the imposed shear forces on the beam when the theoretical storey shear strength of the units was developed. The estimated shear strength in the beam-column joint of Unit EJ1 was only 38% of the imposed joint shear force, although the shear strength estimated for the beam-column joint of Unit EJ2 was adequate. For all three units, the spacing and diameter of the beam and column transverse reinforcement did not meet the requirements for the prevention of longitudinal bar buckling and the confinement of the compressed concrete, especially the requirement for the prevention of longitudinal bar buckling in the specific case of zero axial column load.

Furthermore, the development length of the longitudinal reinforcing bars within the interior beam-column joint of Unit 1 was very inadequate according to NZS3101: 1995, and this is aggravated due to the use of plain round longitudinal bars. The arrangement of the beam longitudinal bar hooks in exterior columns of Unit EJ1 did not meet the current code requirement.

TESTING OF THE UNITS

The units were tested under simulated seismic loading with zero axial column load. Seismic loading was simulated by vertically displacing the beam end(s) while the column ends were held against horizontal translation. The loading method is illustrated in Fig. 4. The first two loading cycles at the beam ends were load-controlled, involving one cycle to 50% of the theoretical flexural strength of the unit and one cycle to 75% of the theoretical flexural strength of the unit. These two cycles in the elastic range were followed by a series of deflection-controlled inelastic cycles comprising two full cycles at displacement ductility factors of 1, 2, 3 etc. The "first yield" displacement was found by extrapolating the measured stiffness at 75% of the theoretical flexural strength of the unit linearly up to the theoretical strength of the unit [Park, 1989].

The displacement components and member curvatures were monitored using linear potentiometers. For all tests, the beam fixed-end rotations (due to bond degradation in the joint core) were measured using pairs of linear potentiometers located next to the joint core. The joint shear distortion was measured by linear potentiometers, which were diagonally attached to the horizontal steel bars embedded in the concrete core. The member longitudinal and transverse reinforcement strains were also monitored by electrical resistance strain gauges.



(a) Interior Beam-Column Joint Test **(b) Exterior Beam-Column Joint Tests**

Fig. 4 Loading Method

TEST RESULTS OF THE AS-BUILT BEAM-COLUMN JOINTS

Cracking, Damage and Failure Mechanisms

Fig. 5 shows Unit 1, Unit EJ1 and Unit EJ2 at the end of testing. It is apparent that the actual performance of the beams and columns of all three units was dominated by flexure only and the joint cores of the three units were of good integrity until the completion of testing.

(a) Unit 1 (b) EJ1 (c) EJ2

Fig. 5 Final Appearance of Units 1, EJ1 and EJ2

For Unit 1, the damage in general was observed to concentrate in the columns in two ways. The major damage was caused by the wide horizontal flexural cracks in the columns above and below the joint panel as a result of the significant bond degradation and slip along the longitudinal column bars within and adjacent to the joint core. Damage was also caused by the vertical cracks running through the joint core along both layers of the column longitudinal bars, as a result of column bar buckling resulting from inadequate transverse restraint against bar buckling. In comparison, the observed damage to Hakuto's Unit O1, which was otherwise identical to Unit 1 but used deformed bars, tended to concentrate in the joint diagonal tension cracks due to lack of joint core shear reinforcement, and the final failure was initiated by the joint shear failure.

For Units EJ1 and EJ2, which were otherwise identical but had alternative arrangements of the beam bar hooks, the final failure of both units was controlled by premature failure associated with the concrete cracking along the beam bar hooks in tension, irrespective of the beam bar hook arrangements. Such concrete cracking occurred as a result of the interaction of the opening of the beam bar hooks in tension and column bar buckling adjacent to the joint cores. In comparison, the observed seismic behaviour of Hakuto's Units O7 and O6, which were otherwise identical to Units EJ1 and EJ2, respectively, but used deformed bars, was controlled by joint shear failure when the beam bar hooks were bent out of the joint core as for Unit O7 and by the shear failure in the beam and joint when the beam bar hooks were bent into the joint core as for Unit O6.

In summary, compared to the case with deformed longitudinal bars, the use of the plain round longitudinal bars, although suppressing the shear failure in the joint and beam, was found to enhance bond degradation and slip along the longitudinal bars, enhance the column bar buckling within and adjacent to the joint cores for both interior and exterior beam-column joint subassemblies, and facilitate the opening of the beam bar hooks in tension for exterior beam-column joints.

Overall Load versus Displacement and Drift Hysteresis Response

General

Figs.6 to 8 show the observed storey (horizontal) shear force versus storey (horizontal) displacement and drift hysteresis loops for Units 1, EJ1 and EJ2. Also shown in Figs. 6 to 8 are the theoretical storey horizontal load strengths of the units, Vi.

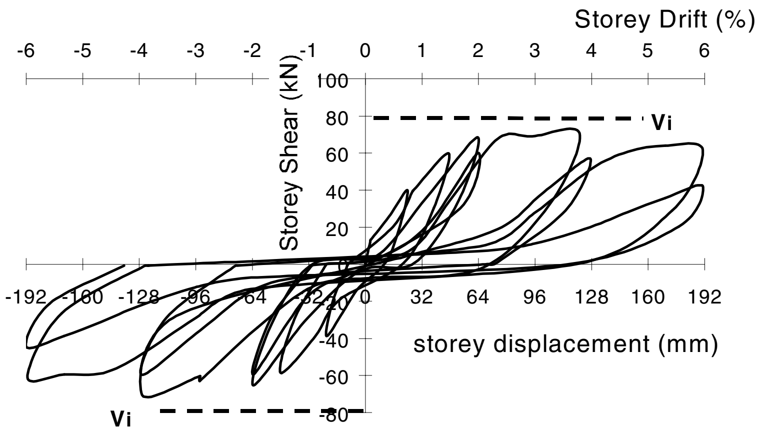


Fig. 6 Storey Shear versus Storey Displacement and Drift Hysteresis Loops of Unit 1

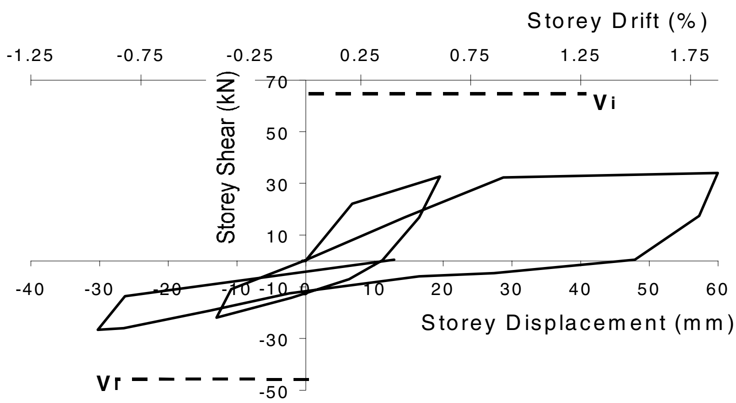


Fig. 7 Storey Shear versus Storey Displacement and Drift Hysteresis Loops of Unit EJ1

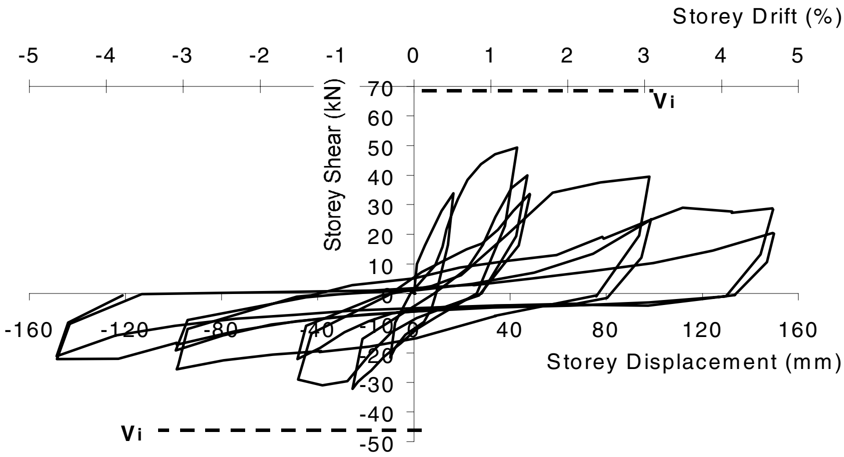


Fig. 8 Storey Shear versus Storey Displacement and Drift Hysteresis Loops of Unit EJ2

Test on As-Built Interior Beam-Column Joint Unit 1

Similar to Hakuto's test on Unit O1, significant pinching of the loops is evident for test of Unit 1. Whereas in the case of Hakuto's test on Unit O1 pinching was due to joint diagonal tension cracking and significant bond degradation along the longitudinal bars, the softness observed for Unit 1 at the beginning of each loading run was completely attributed to significant bond degradation and slip along the longitudinal bars within and adjacent to the joint core. Significant bond degradation and slip along the longitudinal reinforcement within and adjacent to the joint core of Unit 1 resulted in wide flexural cracks at the beam-column interfaces, leading to pronounced pinching of the loops before the commencement of the concrete contribution to the flexural compression. After the two faces of the major flexural cracks closed together, shear and compression could be transmitted along and across these cracks and the stiffness increased rapidly again.

The first yield displacement measured for the test of Unit 1 using the method described in section 3 was 57 mm. This is equivalent to a storey drift of 1.8%, and is nearly three times the first yield displacement of 20 mm predicted by the conventional theory. This theoretically predicted displacement at first yield did not include the contribution of the fixed-end rotations and it used an effective moment of inertia of 0.5Ig, where Ig is the gross sectional moment of inertia. In comparison, the first yield displacement observed for Hakuto's Unit O1 produced a storey drift of 1.2%, and this was about two times its theoretical value of 0.58%, which was also based on an effective moment of inertia of 50% of the gross value. Such a comparison suggests that the plain round bars used for Unit 1 resulted in the available stiffness at first yield to be only about two thirds of that with deformed bars. Apparently, the type of structure tested would be extremely flexible when plain round bars are used for longitudinal reinforcement. On this basis the displacement ductility factor calculated using the measured first yield displacement becomes meaningless. Storey drift becomes a much better index of the displacement of the subassemblies.

Unlike Hakuto's test on Unit O1 which attained the storey horizontal load strength equal to its theoretical storey horizontal load strength at a storey drift of 2%, the maximum storey horizontal load strength attained by Unit 1 occurred at a storey drift of about 4%, and was about 10% less than the theoretical storey horizontal load strength of the unit. The attained storey horizontal load strength at a storey drift of 2% by Unit 1 was about 15% less than the theoretical storey horizontal load strength of the unit. The lower strength attainment observed for Unit 1 was because severe bond degradation and slip along the longitudinal bars caused the plane section assumption to overestimate the member flexural strengths at the plastic hinges.

Tests on As-Built Exterior Beam-Column Joint Units EJ1 and EJ2

The first yield displacement could not be obtained for Unit EJ1 using the method described in section 3 due to the unattainment of the load of 0.75Vi. The first yield displacement measured for Unit EJ2 using the same method produced an equivalent storey drift of 1.5%, which was about four times the theoretical prediction which did not consider the effect of the fixed-end rotation and used an effective moment of inertia of 50% of the gross value. In contrast, the displacement at first yield observed for Hakuto's test on Unit O6 was equal to a storey drift of 0.42%, which was approximately two times the theoretical value predicted using the same method as for

Unit EJ2. This observation indicates that the available stiffness of the tested exterior beam-column joint assemblies with plain round bars was about 50% of that with deformed bars.

The maximum storey horizontal load strengths were attained at a storey drift of 1.5% by both Unit EJ1 and Unit EJ2, and they were 57% and 75% of their theoretical values respectively. Apparently, the arrangements of the beam bar hooks in exterior columns made a significant difference to the strength attainment when the units had zero axial column load. In comparison, the maximum storey horizontal load strengths attained by Hakuto's Units O7 and O6, which were respectively identical to Units EJ1 and EJ2 but used deformed bars, were respectively 75% and 100% of their theoretical values. This indicates that the use of plain round longitudinal reinforcement for the exterior beam-column joint units led to a decrease of 20% to 25% in the attained strengths.

Discussion of Performance

In summary, when compared to the deformed bars, the plain round bars used for the as-built beam-column joint units resulted in a significant reduction in the attained stiffness and strength. Typically, for the interior beam-column joint Unit 1, the observed stiffness at first yield was only two thirds of the value when deformed bars were used, and the attained storey horizontal load strength was only 85% of the value observed for the identical test but with deformed bars. For the exterior beam-column joint Units EJ1 and EJ2, the observed stiffness at first yield was only 50% of that when deformed bars were used, and the attained storey horizontal load strengths by Unit1 EJ1 and EJ2 reduced by 20 to 25% when compared to the identical cases with deformed bars.

Apparently, the adverse effect of plain round bars on the available initial stiffness and the attained storey horizontal load strength of exterior beam-column joint assemblies was much more severe, compared with that for interior beam-column joint assemblies. In addition, the adverse effect of the plain round bars used on the available initial stiffness of beam-column joint assemblies was more apparent than that on the attained strengths of the units, indicating that the stiffness performance of existing reinforced concrete frame structures with plain round bars could be more critical, compared to the strength performance.

Whereas in the case of Hakuto's tests on three as-built beam-column joints with deformed bars, the measured steel strains on longitudinal reinforcement agreed well with the theoretical predictions, the measured steel strains along the longitudinal reinforcement of the three beam-column joint units with plain round bars were generally larger than the theoretical values. This suggests that the actual flexural strengths of the member plastic hinges would be lower than their theoretical flexural strengths. In addition, the measured member deformation for all three units was mainly attributed to member flexure, and the member fixed-end rotation contributed up to 90% of the total member flexural deformation due to the severe bond degradation and slip along the longitudinal bars within and adjacent to the joint cores.

The observed diagonal tension cracking and the shear distortion of the joint cores of the three units were much less apparent, when compared to Hakuto's corresponding tests. Typically, the maximum joint shear distortions measured for Hakuto's Units O1, O7 and O6 were 0.77%, 3.5% and 1.5% respectively, whereas the measured maximum joint shear distortions were 0.37% for Unit 1, and 0.55% for Units EJ1 and EJ2. Evidently, the use of plain round bars resulted in a suppression of the joint shear failure, compared to the case with deformed bars.

CONCLUSIONS

Three full-scale beam-column joint units reinforced by plain round bars, including one as-built interior and two as-built exterior beam-column joint units, were tested under simulated seismic loading with zero axial column load. The units were respectively identical to the as-built beam-column joint units tested by Hakuto except that Hakuto used deformed bars. The tests showed that similar reinforced concrete frame structures with plain round longitudinal bars designed to outdated seismic codes would show low available stiffness and load strength, especially stiffness in a major earthquake. When compared to the case with deformed bars, the use of plain round bars, although leading to greatly improved joint and member shear performance, resulted in severe bond degradation and slip along the longitudinal reinforcement adjacent to and within the joint cores. As a result, member deformation can be approximately estimated by taking into account only the contribution of member fixed-end rotation. In this case, the plane section assumption made in conventional flexural theory was violated and the attained stiffness and strength reduced, especially the stiffness. The adverse effect of the use of plain reinforcing bars on the structural stiffness and strength was much more severe for the exterior beam-column joint units than that for the interior beam-column joint units.

For the interior beam-column joint Unit 1, the large fixed-end rotation due to severe bond degradation and slip along the longitudinal reinforcement was found to trigger the eventual failure of the unit. The attained storey horizontal load strength at a storey drift of 2% was 85% of the theoretical value estimated based on the flexural strength of the columns. The measured initial stiffness of the unit was only 30% of the theoretically predicted stiffness, which did not included the contributions of fixed-end rotation and joint shear deformation and assumed an effective moment of inertia of 50% of the gross value. Compared to the test on Unit O1 with deformed bars, the attained storey horizontal load strength and the available initial stiffness of Unit 1 decreased by 15% and 23% respectively due to the use of plain round bars.

For the exterior beam-column joint Units EJ1 and EJ2, which were identical but had alternative beam bar hook arrangements, the final failure was initiated by column bar buckling and opening of the beam bar hooks in tension, irrespective of the beam bar hook details. Different beam bar hook details in exterior column were found to have significant influences on the structural behaviour when the axial column load was low. The attained storey horizontal load strengths by Units EJ1 and EJ2 were 57% and 75%, respectively, of their theoretical values. Compared to Hakuto's Units O7 and O6, which were identical to Units EJ1 and EJ2 respectively but used deformed bars, the plain round longitudinal bars used for Units EJ1 and EJ2 led to a decrease up to 25% in the attained storey horizontal load strength of the units. The initial stiffness obtained for Unit EJ2 was only 24% of the theoretical value, and it was only 50% of the observed stiffness for Hakuto's O6. This observation demonstrates that the plain round longitudinal bars used for Unit EJ2 resulted in a decrease of 50% in the available initial stiffness.

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