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SEISMIC ASSESSMENT AND RETROFIT OF PRE-1970s CONCRETE FRAME BUILDINGS WITH PLAIN ROUND REINFORCING BARS

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Abstract

Technical Guidelines for Engineering Assessments "the Guidelines" are used for conducting seismic assessments of existing building structures in New Zealand (NZ) and the essence of the Guidelines is to determine the governing inelastic lateral resisting mechanisms of the structures. In assessing existing concrete frame buildings using the Guidelines, the information on probable strength and deformation capacities for individual members and subsystems was based on the tests on concrete members/assemblies reinforced with deformed bars. However, plain round bars had been used until early 1960 before deformed bars became available. When concrete frame members contain plain round bars, the failure modes and seismic resisting capacities at an element level or at a system level are significantly different from their counterparts containing deformed reinforcing bars. Assessing a concrete frame building containing plain round reinforcing bars, using the Guidelines, would likely lead to incorrect identification of structural weakness and subsequently mistargeted retrofit solutions.

This paper presents the findings about the effects of reinforcing bar profiles (plain round bars or deformed bars) on failure modes, strength and deformation capacities of concrete members and concrete systems. The study was conducted by comparing the test observations of as-built beam-column joint assemblies reinforced with plain round bars with those of otherwise identical beam-column joint assemblies but with deformed reinforcing bars. The study reveals that reinforcing bar profiles affect nearly every aspect of the seismic behaviour of reinforced concrete frame structures significantly. When deformed bars were used as longitudinal reinforcement, premature shear failures in the beam-column joints and/or in the flexural weaker members initiated the final failure of the concrete frame structures. However, when the longitudinal reinforcing bars are from plain round bars, degrading flexural behavior (strength and stiffness) of flexural weaker members and bar buckling, especially the column bar buckling adjacent to the joints, are likely to initiate the seismic behavior of the buildings. At the system level, as-built reinforced concrete frames reinforced with plain round bars could be expected to have completely different failure mechanisms, achieve significantly lower stiffness and lower strength, in comparison with those reinforced with deformed bars.

Keywords: Reinforced concrete frames, plain round bars, seismic assessment, retrofit

1. Introduction

Since 1990s, there have been significantly research efforts studying seismic behaviour and retrofit solutions of existing reinforced concrete (RC) buildings designed before introduction of the current generation of codes for the seismic design of building structures in 1970s [1, 2]. With regards to concrete buildings, RC frame structures are more susceptible to lose the structural instability in earthquakes because of the skeleton structural nature, in comparison with concrete wall structural systems. As such, greater efforts were made in studying the seismic performance of pre-1970s frame buildings than other structure types. In New Zealand (NZ), a number of simulated seismic loading tests on as-built and retrofitted reinforced concrete columns and beam-column joint assemblies had been conducted. This resulted in the production of the first guidance document on seismic assessment and strengthening of existing concrete buildings, "2006 NZSEE Guideline", in 2006 [3].

Since the introduction of "2006 NZSEE Guideline", a great deal of pre-1970s concrete buildings have been assessed nationwide in NZ. Concerns were raised about significant inconsistency of assessment results. Recent earthquakes in NZ, the Canterbury earthquakes in 2011 and the Kaikoura earthquake in 2016, provided new learning opportunities and renewed impetus to advance the assessment guidance "2006

NZSEE Guideline". Consequently, the assessment guidelines were updated and released in 2017 and became "The seismic assessment of existing reinforced concrete buildings" (The 2017 Guideline [4]), in order to provide a more reliable and consistent outcome of the seismic assessments. The 2017 guideline promotes the use of the Simple Lateral Mechanism Analysis (SLaMA) procedure, a displacement-based assessment procedure, to determine the global non-linear pushover capacity curve. At the elemental level, the SLaMA technique assesses the hierarchy of strengths related to different failure modes of individual members. At the system level, the SLaMA technique quantifies the seismic global behavior of a building by summation of simple representations of the capacities of the individual members/sub-systems. The assessment at the elemental level of the seismic behaviour of the individual members/sub-systems is the fundamental step for the entire assessment process.

Part C of the 2017 guideline sets out methodologies for engineers to conduct a detailed seismic assessment (DSA) where section C5 in part C provides guidance specific to reinforced concrete structures. The basic information used for undertaking the assessment at elemental level, as suggested in Section C5 of the 2017 Guideline, was mainly obtained, based on the experimental studies, which involved deformed reinforcing bars.

Plain round reinforcement was used in NZ until about the mid-1960s when deformed reinforcement became available. This means that a significant amount of pre-1976s RC frame buildings contain plain round reinforcing bars. Conventional theory used for evaluating reinforced concrete systems is established, based on the assumption of perfect bond between the longitudinal reinforcement and the surrounding concrete. Plain round bars have less bond strength than deformed bars. As such, the concrete frame structures reinforced by plain round longitudinal bars would be expected to have very different performance in earthquakes, in comparison with identical structures but reinforced by deformed longitudinal bars.

In this study, three pairs of simulated seismic loading tests of as-built full-scale beam-column joint assemblies were analysed. The three pairs of tests were selected from the past tests conducted within the programme "seismic performance of pre-1970s existing reinforced concrete frame structures" at the University of Canterbury [5, 6]. The two tests in each pair were identical except the surface profiles of the longitudinal reinforcement and the objective is to investigate the effects of the longitudinal reinforcing bars on the seismic behaviour of concrete frame members/systems.

This paper summaries the observed differences of simulated seismic tests on as-built beam-column joint subassemblies reinforced by plain round reinforcing bars, in comparison with the deformed reinforcing bars.

2. Details of the As-built Beam-Column Joint Assembly Units Aelected for This Study

2.1 General

Since early 1990s, a series of research projects in NZ had been designed to study seismic behaviour and retrofit methods of pre-1970s concrete frame structures in NZ and many simulated seismic loading tests on as-built and retrofitted reinforced concrete members and subassemblies had been undertaken to help inform the seismic assessment of existing concrete buildings and the seismic strengthening solutions [5, 6, 7, 8, 9].

Among many studies, two projects at the University of Canterbury each had conducted three simulated seismic loading tests of as-built beam-column joint assemblies. These two projects were respectively undertaken by Hakuto et al [5], and Liu et al [6]. The three tests by Liu were respectively identical to the three tests by Hakuto except that Hakuto used deformed longitudinal bars while Liu used plain round longitudinal bars. These three tests were selected to study the effects of the longitudinal reinforcement bar surface profile on the likely seismic performance of as-built reinforced concrete frame buildings.

2.2 Details of the three as-built beam-column joints assemblies selected in this study

The three as-built units selected for this study included one interior beam-column joint assembly and two exterior beam-column joint assemblies. These three as-built test units were all full-scale replicas of an example reinforced concrete frame building constructed in the 1950s in New Zealand and they represented sub-frame systems with a storey height of 3.2 meters and a bay width of 3.81 meters. The reinforcing details of these three as-built test units are shown in Figures 1, 2 and 3. Noted is that the reinforcing bars illustrated

in Figures 1 to 3 were plain round bars as used by Liu, and the three tests by Hakuto were identical except Hakuto used deformed longitudinal bars.

For the as-built interior beam-column joint assembly "Unit 1" shown in Figure 1, the beams were 500 mm in depth and 300 mm in width, and the columns were 300 mm in depth and 460 mm in width. The beams were unsymmetrically reinforced and had four and two 24 mm diameter 300 MPa plain round bars respectively in the top and the bottom. The beam transverse reinforcement was from 6 mm diameter 300 MPa plain round bars placed at 380 mm centres. The columns were symmetrically reinforced and had three 24 mm diameter 300 MPa plain round bars on each side. The column transverse reinforcement was from 6 mm diameter 300 MPa plain round bars placed at 230 mm centres. The joint core had no transverse reinforcement at all.

The two as-built exterior beam-column joint test units were Unit 2 and Unit 3, and they were illustrated in Figures 2 and 3. Unit 2 and Unit 3 were identical to each other except for the anchorage of the beam longitudinal bars in the exterior columns. In details, Unit 2 had the beam bar hooks bent away from the joint core in exterior columns while Unit 3 had the beam bar hooks bent into the joint core in exterior columns. The beam was 500 mm in depth and 300 mm in width, and the columns were 460 mm square. The beams were unsymmetrically reinforced and had three and two 24 mm diameter 300 MPa plain round bars respectively in the top and the bottom. The beam transverse reinforcement was from 6 mm diameter 300 MPa plain round bars placed at 380 mm centres, similar to the interior beam-column joint test Unit 1. The columns were symmetrically reinforced and had two 24 mm diameter 300 MPa plain round bars on each side. The column transverse reinforcement was from 6 mm diameter 300 MPa plain round bars placed at 305 mm centres. The joint core had one set of column transverse reinforcement at mid-height of the beam.

2.3 Test procedures

The test procedures used by Liu and Hakuto were fundamentally the same and both projects only simulated seismic loading with no axial load on columns. For the tests by Liu, seismic loadings were simulated by deflecting the beam end(s) vertically while the column ends were held against horizontal translation. For the tests by Hakuto, seismic loadings were simulated by deflecting the column ends horizontally while the beam end(s) were held against vertically deflection.

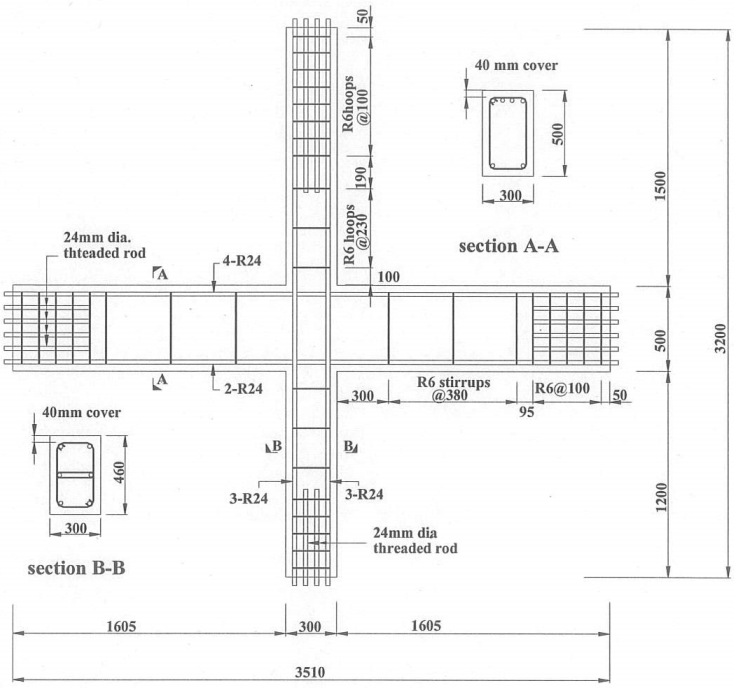


Figure 1 Reinforcing details of As-built test Unit 1 [6]

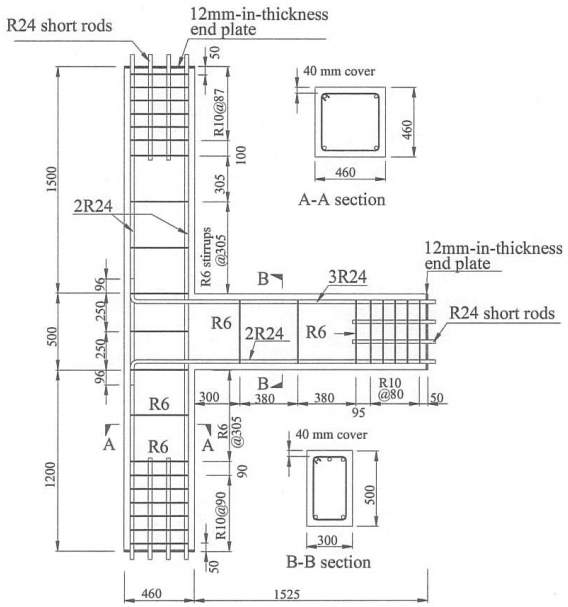


Figure 2 Reinforcing details of As-built Unit 2 [6]

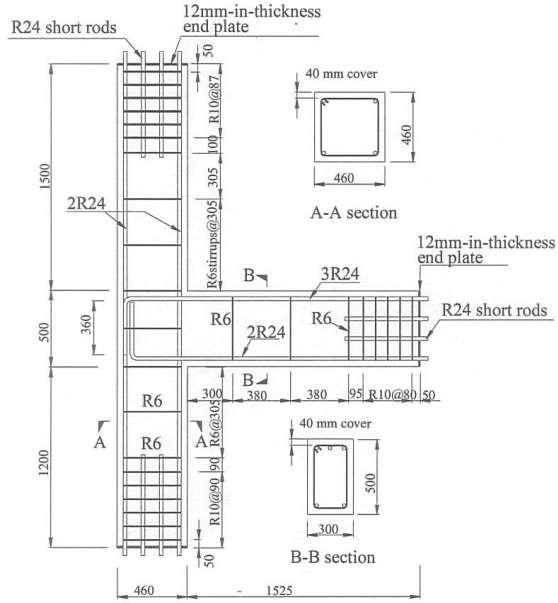


Figure 3 Reinforcing details of As-built Unit 3[6]

2.4 Theoretical considerations

2.4.1 Probable flexural strengths of beam and column members

The 2017 Guidelines suggest that the conventional flexural theory [10] and the probable material properties with the reduction factor of unity be used in determining the member flexural strengths, irrespective of the longitudinal reinforcing bars being plain bars or deformed bars. The determined flexural strengths of beams and columns of the test units are summarized in Table 1. Also listed in Table 1 are the flexural strength ratios at the joints, $\frac{\Sigma M_{column}}{\Sigma M_{beam}}$ in order to determine the post-elastic deformation locations. As evident in Table 1, that, during simulated seismic loading, hinges would be developed in the columns for Unit 1 but in beams for Unit 2 and Unit 3.

Table 1 lists the assessed flexural strengths of the beams and columns of the three test units

Test Units	Beam Positive moment (kNm)	Beam Negative moment (kNm)	Column Moment (kNm)	$\frac{\Sigma M_{column}}{\Sigma M_{beam}}$
Unit 1	129 (132)	250 (246)	108 (120)	0.6 (0.7)
Unit 2	129 (119)	190 (173)	120 (114)	2.0 (2.0) (+) 1.4 (1.4) (-)
Unit 3	128 (121)	189 (174)	119 (115)	2.0 (2.0) (+) 1.4(1.4) (-)

Note:
1. The values without brackets were the flexural strengths of the members for the tests with plain round longitudinal bars predicted by Liu while the values with the brackets were the flexural strengths of the members for the tests with deformed longitudinal bars predicted by Hakuto
2. Slight difference in the flexural strengths of beams and columns between Unit 2 and Unit 3 in both Hakuto's project and Liu's project was due to the difference in measured concrete strengths for the two tests.
3. + and – refer to beam positive bending direction and beam negative bending direction respectively.

The predicted storey shear strength for Unit 1 was 80 kN and 89 kN respectively for the test with plain round longitudinal bars and for the test with deformed longitudinal bars, this was governed by the flexural strength

of the columns. The predicted storey shear strengths for Units 2 and 3 when plain round longitudinal bars were used were about the same, being 45 kN for the positive cycles and 67 kN for the negative cycles. The predicted storey shear strengths for Units 2 and 3 with deformed longitudinal bars were about the same, being 42 kN for the positive cycles and 61 kN for the negative cycles.

2.4.2 Probable shear strengths of beam and column members

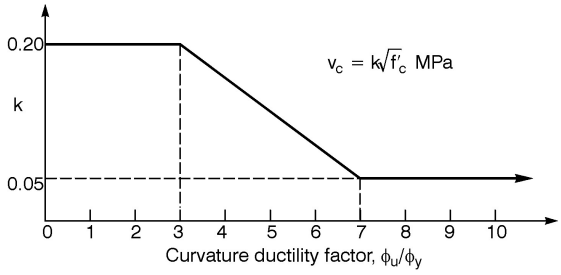
According to the 2017 Guidelines, the probable shear strengths of beams and columns are calculated using

$$V\mathrm{p} = 0.85\,(V_C + V_S + V_N) \qquad (1)$$

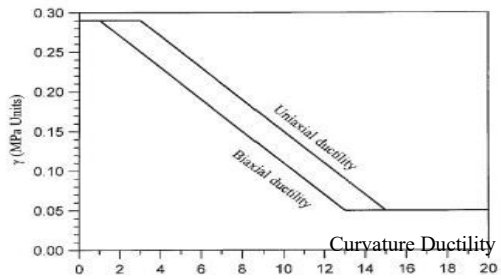
Where: Vc and Vs are the shear contributions provided by the concrete mechanism and steel shear reinforcement for both beams and columns; and V_N is the shear contributions provided by the axial compressive load N^* and it is only applicable to columns.

The shear contributions provided by the concrete mechanisms degrade with the increases of the imposed deformation demands. Shear strength degradation factors suggested in the 2017 Guidelines are shown in Figure 4(a) for beams and Figure 4(b) for columns. Figure 4 (a) was produced, based on test results from Hakuto et al. and Priestley et al. (1994), and Figure 4(b) was produced, based on the test results from Priestley et al. All the tests by Hakuto and Priestley used deformed longitudinal reinforcing bars.

The shear contribution from the steel shear reinforcement, Vs, is evaluated assuming that the critical diagonal tension crack is inclined at 45° to the longitudinal axis of the beams and 30° to the longitudinal axis of the columns. The shear resistance as a result of axial compressive load on the columns is taken as the horizontal component of the compression action in the compressive strut formed between the centroids of the concrete compressive forces of the column section at the top and bottom of the column.



(a) beams



(b) Columns

Figure 4 Shear strength degradation factor, $\boldsymbol{\gamma}$ (Reproduced from C5 of the 2017 Guidelines[4])

Section C5 of the 2017 Guidelines also has a method for assessing the probable shear strengths of joints. The probable horizontal joint shear strengths are dependent on the joint type (exterior or interior), the axial load on the columns and the imposed ductility demands on the members framed into the joints. The joint shear strength degradation factors recommended in C5 of the 2017 Guidelines is shown in Figure 5. Figure 5 was produced, based on experimental testing by Hakuto and Pampanin. Hakuto used deformed bars and Pampanin used plain round bars but different details from Hakuto.

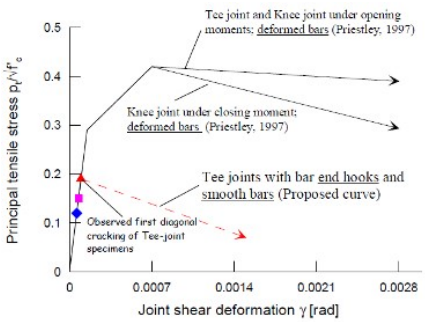


Figure 5 Shear strength degradation (Reproduced Figure C5.17 in the 2017 Guidelines [4])

For the three as-built beam-column joint assembly tests studied here, the predicted shear resistances of weaker flexural members and the joints, according to C5 of 2017 Guidelines, ranged from about 20% to 50% of the shear demands, indicating potential premature shear failures in the members and/or the joints. However, the three as-built beam-column joint assembly tests reinforced by plain round longitudinal bars had no reported premature shear failure either in the members or in the joints. As such, the detailed theoretical analyses of the shear performance of these three tests, although conducted in the original project by Liu, were not presented here in greater details.

2.4.3 Probable deformation capacities of beams and columns

The 2017 guidelines recommend that the deformation capacities of concrete members be calculated according to the methods suggested in NZS3101 [10]. According NZS3101, the deformation capacity of a concrete member at first yield is calculated based on the initial stiffness, the deformation capacity of a concrete member at ultimate state is calculated based on the deformation at first yield and post-elastic deformation calculated based on plastic hinge lengths.

NZS3101 allows for cracking effect in calculating initial stiffness and assumes effective sectional moment of inertia of concrete frame members to be a percentage of the gross sectional moment inertia. According to NZS3101, the effective moment of inertia, I_e, is $0.43I_g$ and $0.45I_g$ respectively for the beams and the columns of the three tests. Based on the effective sectional moment of inertia defined above and assuming that deformation contribution due to joint shear distortion was 20% of the total storey deflection, the initial stiffnesses of the three as-built tests were calculated. The obtained initial stiffness of test 1 (test on Unit 1), if expressed as the ratios of the storey shear strength versus storey displacement, was respectively 4.4 kN/mm and 4.8 kN/mm for the test with plain round longitudinal bars and the test with deformed longitudinal bars. The obtained initial stiffness of test 2 (test on Unit 2) was respectively 9.0 kN/mm and 8.5 kN/mm for the test with plain round longitudinal bars and the test with deformed longitudinal bars. The obtained initial stiffness of test 3 (test on Unit 3) was respectively 9.2 kN/mm and 9.0 kN/mm for the test with plain round longitudinal bars and the test with deformed longitudinal bars.

If the initial stiffness obtained as above is expressed as the yield deflection, which is the storey displacement when the theoretical strength is attained, the estimated yield deflection for test 1 is 19 mm, and the estimated average yield of positive cycles and negative cycles, is 6 mm for test 2 and 6 mm for test 3. Table 2 summarises the predicted initial stiffnesses and the first yield displacements.

Table 2 Predicted first yield displacements, Δ_y and predicted initial stiffnesses, K_p

Test Number	Unit Number	Δ_y (mm)	K_p (kN/mm)
Test 1	Unit 1	19	4.4 (4.8)
Test 2	Unit 2	6	9.0 (8.5)
Test 3	Unit 3	6	9.2 (9.0)

Note: The initial stiffness values, K_p, without brackets are the stiffnesses with plain round longitudinal bars while the initial stiffness values, K_p, with brackets are the stiffnesses with deformed bars.

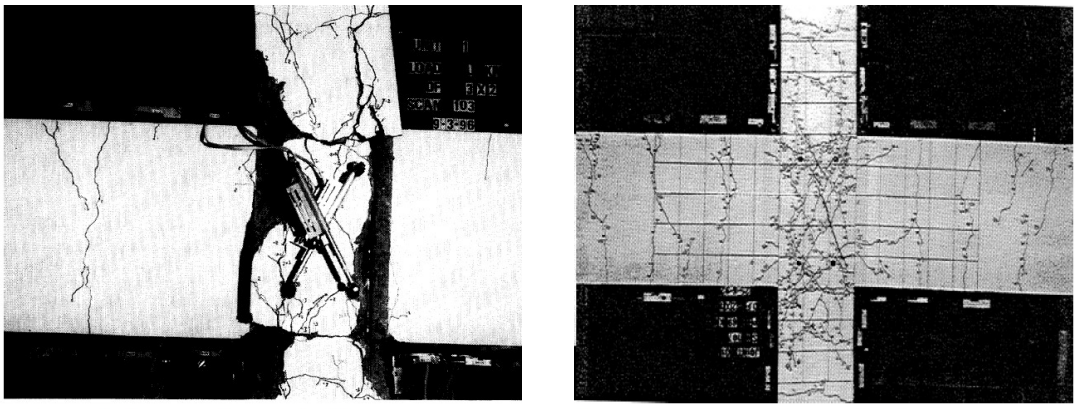
3. Observed Performance of Simulated Seismic Loading Tests on As-built Test Units

3.1 General

Figures 6 to 8 compares the final appearances of three pairs of as-built tests, at the completion of the simulated seismic testing where two tests in each pair were identical except the longitudinal bar profiles.

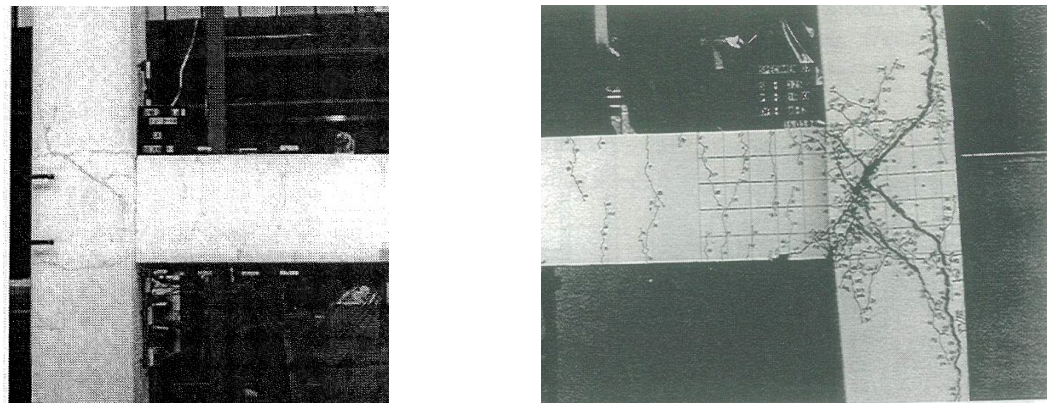
Clearly, the seismic performance of the as-built beam-column joint assemblies reinforced with plain round longitudinal bars was different from that with deformed longitudinal bars. When the as-built beam-column joint assemblies had deformed longitudinal reinforcing bars, premature shear failure in the joint zones of tests 1 to 3 and premature shear failure in the beam of test 3 occurred, and these resulted in the final failures of the as-built beam-column joint assemblies. In comparison, premature shear failure did not happen in either

members or in the joints throughout the entire testing for all three tests with plain round longitudinal bars, instead the degrading flexural behaviour of flexural weaker member(s) caused their final failures.



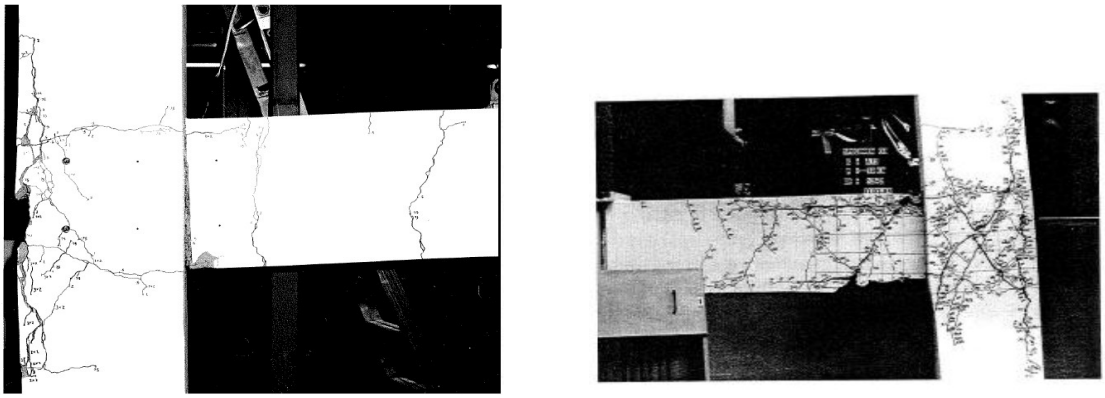
(a) Plain round longitudinal bars [6] (b) Deformed longitudinal bars [5]

Figure 6 Final appearances of test on Unit 1



(a) Plain round longitudinal bars [6] (b) Deformed longitudinal bars [5]

Figure 7 Final appearances of test on Unit 2 (beam bars bent away from the joint)



(a) Plain round longitudinal bars [6] (b) Deformed longitudinal bars[5]

Figure 8 Final appearances of test on Unit 3 (beam bars bent into the joint)

In comparison with the cases with deformed longitudinal reinforcement, the use of plain round longitudinal reinforcement caused the flexural deformation to concentrate on one single crack at the fixed-ends of members, rather than spreading over the commonly stated plastic hinge regions as desired. This occurred because inadequate bond strength between reinforcement and the surround concrete introduced much smaller actions to the concrete through bond than the cases with deformed bars, indicating the violation of perfect bond assumption thus the conventional flexural theory.

In summary, the seismic behaviour of as-built reinforced concrete frame buildings would be very different if the longitudinal bars have different surface profiles and properly validated methods should be developed in assessing the strength and deformation capacity of as-built members and systems with plain round longitudinal bars.

3.2 The storey shear versus storey deflection behaviour

Figure 9 shows the measured storey shear strength versus storey displacement and drift hysteresis loops for the three tests with plain round longitudinal bars where the theoretical storey shear strength of each test, Vi, calculated based on the flexural strengths of weaker members, is also shown.

As shown in Figure 9, the attained storey force strengths of the as-built beam-column subassemblies reinforced by plain round longitudinal bars were significantly lower than the theoretical predictions, being only about 85%, 70% and 75% of the theoretically predicted storey shear strengths respectively for the test 1, test 2 and test 3. When the obtained storey force strengths of the as-built beam-column subassemblies reinforced by plain round longitudinal bars were compared with those of the identical tests but with deformed longitudinal bars, it revealed that the use of plain round bars led to a storey shear strength reduction of 15% to 30%. This agrees well with the test evidence obtained from the simulated seismic loading tests on as-built columns reinforced by plain round longitudinal bars by Rodriguez [7]. The as-built columns tested by Rodriguez were also replicas of the 1950 building represented by tests 1 to 3 described above. Rodriguez concluded that the use of plain round longitudinal bars could cause the flexural strength attainment to reduce between 25% to 40%, in comparison with the similar systems reinforced by deformed reinforcement and detailed according to modern codes. This suggests that the conventional flexural theory as recommended in the 2017 Guidelines could significantly overestimate the flexural strength of concrete frame members. Table 3 summaries the attained storey shear strengths for the three as-built tests.

Table 3: Attainment of the storey shear strength of the beam-column joint assemblies

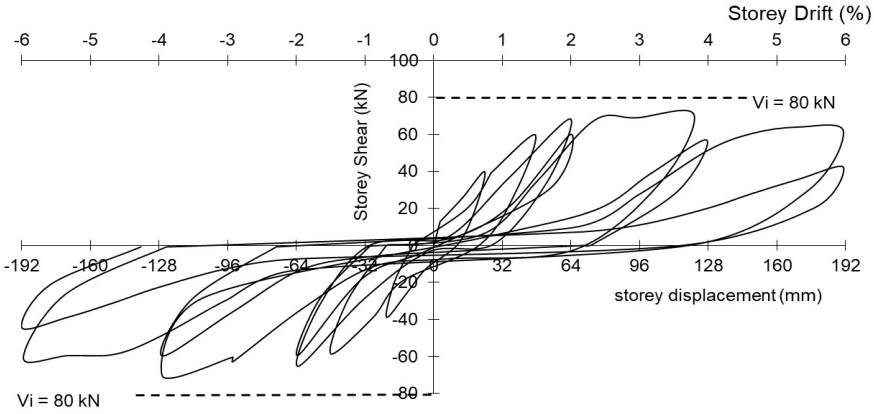
Test ID		V_p(kN)	V_m (kN)	R_V (%)
Test 1	Plain bars	80	68	85%
	Deformed bars	89	89	100%
Test 2	Plain bars	67(-) 45(+)	33(-) 22 (+)	50% 50%
	Deformed bars	62(-) 42(+)	46(-) 37(+)	75% 88%
Test 3	Plain bars	67(-) 45(+)	50 (-) 34(+)	75% 75%
	Deformed bars	62(-) 42(+)	64 (-) 47(+)	100% 110%

Notes: V_p is the predicted storey shear strength, V_m is the attained storey shear strength during the testing and R_V is the ratio of the attained strength versus the predicted strength.

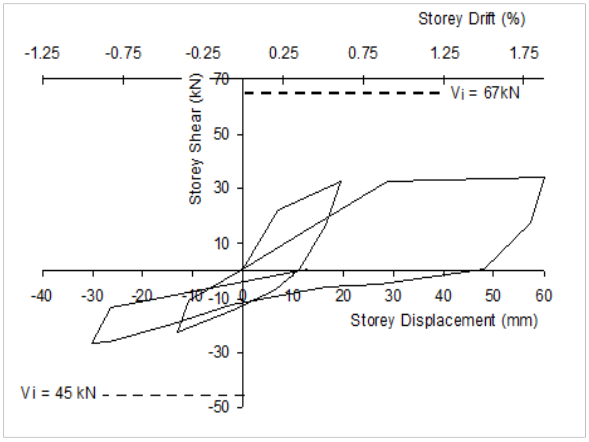
The deflection behaviour of the as-built test units was also examined in this study. The first yield displacements of the as-built tests with plain round longitudinal bars were obtained by extrapolating the measured stiffness at 75% theoretical flexural strength linearly up to the theoretical flexural strength as commonly used in NZ. The first yield displacements obtained as described above were 57 mm for the as-built interior beam-column joint assembly (test 1) and it was 48 mm for the as-built exterior beam-column joint assembly test with beam bars bent into the joint (test 3). The first yield displacement for the as-built exterior beam-column joint assembly test 2 could not be obtained as described above because abrupt strength

degradation occurred at the loading cycles of 50% of the theoretical strength and the test subsequently stopped. The first yield displacement for test 2 was then determined by extrapolating the measured stiffness at 50% theoretical flexural strength linearly up to the theoretical flexural strength and it was 40 mm. It has to be appreciated that the real yield displacement for test 2 could be a lot greater than 40 mm because of significant degradation after the attainment of 50% the theoretical storey shear strength.

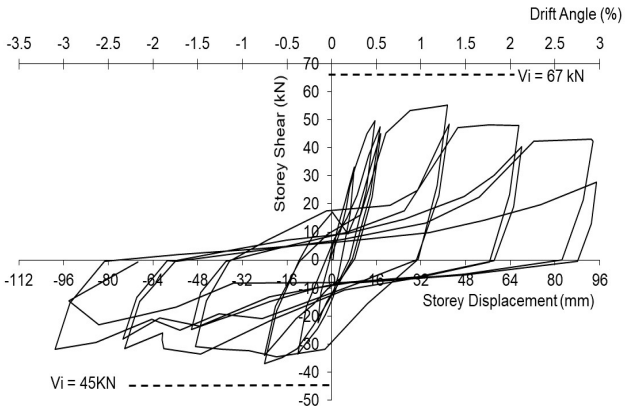
Based on the obtained yield displacements, the theoretically predicted first yield displacements and the attainments of the storey shear strengths, the initial stiffnesses were obtained for the as-built tests with plain round longitudinal bars. The obtained initial stiffnesses were expressed as the percentages of the theoretically predicted stiffnesses using NZS3101 method. The detailed comparison was listed in Table 4.



(a) Unit 1



(b). Unit 2



(c) Unit 3

Figure 9 Storey shear versus storey displacement and drift hysteresis loops of as-built units with plain round longitudinal bars [6]

The first yield displacements of the identical tests but with deformed reinforcing bars were also reported by Hakuto and they were 37 mm, 15.2 mm and 13.3 mm respectively for test 1, test 2 and test 3. Comparative study of Hakuto's tests and Liu's tests were conducted to study the effects of the reinforcing bar profiles on the initial stiffness. This was undertaken, based on the storey shear strength attainments (Table 3), the theoretically predicted first yield displacements (Table 2) and the obtained first yield displacements. The ratios of the obtained initial stiffnesses for the as-built tests with plain round bars versus those with deformed bars were also listed in Table 4.

Table 4 The first yield displacements and stiffness behaviour

Test ID	$\Delta_{y,p}$ (mm)	$\Delta_{y,m}$ (mm)	R_v %	K_P %	K_D %	K_{PR} (%)
Test 1	20	57	85%	30%	43%	55%
Test 2	6.3	40	50%	8%	38%	25%
Test 3	6.3	45	75%	11%	47%	22%

Notes: $\Delta_{y,p}$ is the predicted first yield displacement according to the NZS3101 method; $\Delta_{y,m}$ is the measured first yield displacement of the test with plain round longitudinal bars; R_V is the strength attainments of tests with plain round longitudinal bars, expressed as percentages of the theoretically predicted storey shear strength; K_P and K_D are respectively the stiffness attainments of tests with plain round longitudinal bars and the stiffness attainments of tests with deformed bars, expressed as the percentages of the theoretically predicted initial stiffness; and K_{PR} is the ratio of the obtained initial stiffness of as-built units with plain round longitudinal bars versus that with deformed longitudinal bars.

As shown in Table 4, the use of plain round longitudinal bars caused the initial stiffness to reduce by about 45% to 78%, in comparison with the use of deformed bars. The 2017 Guidelines suggest that, in assessing the existing concrete structures, the stiffness modeling of concrete frame members use the NZS3101 method. As revealed from this study, NZS3101 significantly overestimated the initial stiffness of as-built concrete frame systems and this is especially true when the frame members had plain round longitudinal bars.

3.3 Post-elastic behaviour

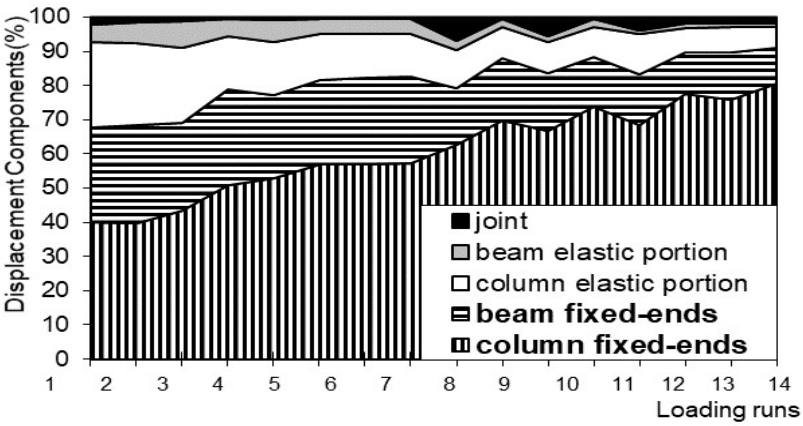
The detailed analysis of post-elastic behaviour revealed that the non-linear deformation progression matched with the observed damage progression as described in section 3.1.

For the as-built reinforced concrete members with plain round longitudinal bars, the post-elastic behaviour was mainly governed by the flexural behaviour at the fixed-ends (the beam-column interfaces) of the flexural weaker members, and the post-elastic deformation did not spread to a bigger region as conventionally called "plastic hinge region". The deformation components attributed to the fixed-end rotations of the flexural weaker member(s) grew significantly as the testing progressed and reached up to nearly 80% of the total deformation for both tests on Unit 1 and Unit 3. Noted is that test on Unit 2 was terminated, at the cycles to 50% of the theoretical storey shear strength, due to premature concrete tension failure above and below the column fixed end regions. As such, test on Unit 2 did not give a complete picture of the system performance. Figure 10 shows the measured contributions of different deformation sources at loading peaks, expressed as the percentages of the measured storey displacements, for the as-built tests with plain round longitudinal bars.

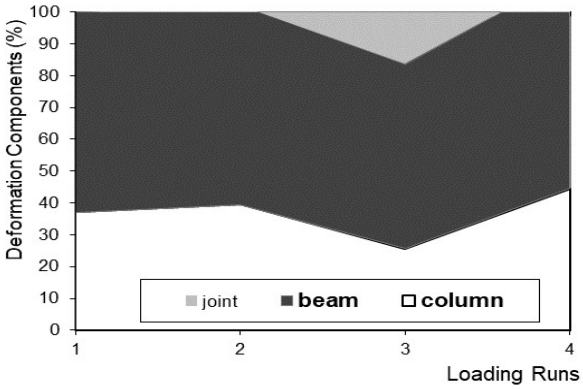
In comparison, the tests on three as-built units with deformed bars all reported steady increases in the measured contributions to the total storey deflections by shear deformations in the joints and/or in the members. This indicates that premature shear failure contributed to the final failure. For example, the test on Unit 1 with deformed bars revealed that the displacement component by the joint deformation kept increasing and reached up to 30% of the total deformation at the final stage of the testing. This was consistent with the observed evidence that the premature joint shear failure was partially responsible for the final failure as shown in Figure 6 (b). For the test on the as-built test Unit 2 with deformed beam bars bent away from the joint, the displacement component by the joint deformation reached up to 66% of the total deformation and this agrees well with the observed damage in Figure 7(b). In the case of the test on the as-built test Unit 3 with deformed bars, the obtained displacement components respectively by the joint shear deformation and the beam shear deformation kept increasing, and they reached respectively up to about 30% and 25% of the total deformation. This is again consistent with the observed damage progression in Figure 8(b).

Clearly, the non-linear performance of concrete frame buildings strongly depends on the surface profiles of the longitudinal reinforcing bars. As such, the seismic assessment of existing reinforced concrete frame buildings reinforced by plain round longitudinal bars should not completely rely on C5 of the 2017 Guidelines, which has a great focus on the iterative shear strength degradation check as the seismic actions progress. The assessment of the existing reinforced concrete buildings reinforced by plain round longitudinal bars should really address the low attainments of member flexural strengths and stiffness, as well as

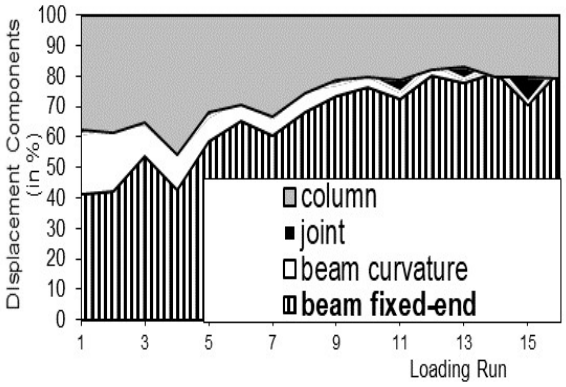
significant strength degradation associated with the potential column bar buckling and the bond degradation. The shear strength degradation as the deformation progresses, although essential if the buildings contain deformed longitudinal bars, is no longer an issue if the buildings contain plain round longitudinal bars.



(a) Test on Unit 1



(b) Test on Unit 2



(c) Test on Unit 3

Figure 10: Deformation components obtained when plain round longitudinal bars were used [6]

4. Discussions and Conclusions

The simulated seismic loading tests on the as-built reinforced concrete beam-column joint assemblies show that the post-elastic behaviour of existing reinforced concrete frame buildings could be strongly dependent on the surface profiles of the used longitudinal reinforcing bars. A great deal of the existing reinforced concrete building stocks in NZ contain plain round longitudinal bars because the deformed bars were not available before the 1960s. However, current seismic assessment procedure in NZ (C5 of 2017 Guidelines) was developed mainly based on the evidence associated with the deformed reinforcing bars. As a result, the assessment results could be misleading in predicting the likely seismic performance of the existing reinforced concrete structures reinforced with plain round longitudinal bars. There is a need for distinguishing the critical issues associated with the different reinforcing bar types.

When the existing reinforced concrete buildings contained deformed longitudinal bars, significant shear strength degradation was observed as the displacement progressed. As a result, the members which yielded in flexure first could end up with the premature shear failure, and the non-linear behaviour was significantly attributed to the shear failure in the members and the joint. This observation has been incorporated into the current seismic assessment procedure, C5 of the 2017 Guidelines.

When the existing reinforced concrete buildings contained plain round longitudinal bars, the premature shear failure in the members and joints was much less problematic and the seismic behavior of concrete frame systems was limited by the degrading flexural behaviour of flexural weaker members. The critical seismic issues in this case include low attainments of stiffness and flexural strengths of members, significant degradations of the stiffness and flexural strength, and severe bar buckling of the longitudinal reinforcing bars at the fixed-ends of the members, especially the columns.

Comparative study of simulated seismic tests on as-built full-scale beam-column joint assemblies revealed that the current methods suggested by the 2017 Guideline could significantly overestimate the initial stiffness of as-built concrete frame members, especially in the cases when the members contain plain round longitudinal bars. In details, the measured initial stiffness, when the deformed longitudinal bars were used, was about 38% to 47% of the theoretically predicted initial stiffness using current method recommended in the 2017 Guidelines. The measured initial stiffness, when plain round longitudinal bars were used, was only 11% to 30% of the theoretically predicted initial stiffness.

With regards to flexural strength attainment, the comparative study revealed that the current methods suggested by the 2017 Guideline could overestimate the flexural strengths of concrete frame members by 15% to 30%, when the members contain plain round longitudinal bars.

In comparison with concrete frame structures reinforced by deformed bars, low stiffness attainment and low strength attainment of concrete frame members with plain round bars could result in significant differences in expected seismic performance of the buildings. As such, more studies are needed to develop a proper seismic assessment procedure for existing reinforced concrete buildings with plain round longitudinal bars.

5. Acknowledgements

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