SEISMIC ASSESSMENT OF A PRE-1970S REINFORCED CONCRETE FRAME BUILDING WITH PLAIN ROUND REINFORCING BARS

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ABSTRACT:

Seismic performance of an existing reinforced concrete frame building constructed in New Zealand in the late 1950s was assessed by conducting non-linear static and non-linear dynamic analysis. The non-linear behaviour of individual reinforced concrete members was based on the simulated seismic loading tests conducted on as-built reinforced concrete beam-column joint units, which represented the subject building and contained plain round reinforcing bars.

Both non-linear static push-over and non-linear dynamic analyses showed that the overall non-linear performance of the building was mainly dominated by the non-linear behaviour of the beams. The building would not develop a collapse mechanism prior to the attainment of the maximum local deformation capacity, and no ductility can be relied on. Comparison with the results for deformed longitudinal reinforcing case shows that the current seismic assessment procedure could significantly overestimate the building capacity if plain round longitudinal bars were actually present instead.

KEYWORDS: Seismic Assessment, Plain Round Bars, Existing Reinforced Concrete Structures

1. INTRODUCTION

There has been increased emphasis worldwide in recent years in the seismic assessment of existing reinforced concrete structures and strengthening where necessary to improve their seismic performance [A1, P2]. In the last two decades, considerable work has been done in developing the detailed seismic assessment procedures, resulting in the development of much advanced seismic assessment procedures. The advanced procedures are capable of considering the global structural behaviour in the post-elastic range, rather than the check-list type of screening which checks the individual members as for a working stress design concept.

In New Zealand (NZ), a multi-stage research programme on Seismic Assessment and Retrofit of Existing Reinforced Concrete Structures had been carried out at the University of Canterbury sponsored by the NZ Earthquake Commission for many years. A 1950s reinforced concrete frame building (referred to as the subject building) has been thoroughly investigated. A number of cyclic loading tests on as-built reinforced concrete columns and beam-column joint units with reinforcing details typical of this 1950s construction have been conducted [H1, L1], and the main bars used were deformed bars. The information obtained on the non-linear behaviour of the existing reinforced concrete members has been largely incorporated into the current seismic assessment procedure recommended by the New Zealand Society of Earthquake Engineering [N1]. However, the deformed bars were not commonly available before 1960s in NZ. The observed non-linear behaviour of existing members reinforced by plain round bars was very different from that with deformed bars. The current seismic assessment procedure in NZ does not have guidelines for determining the post-elastic behaviour of existing reinforced concrete members with plain round longitudinal bars. Therefore, a series of cyclic loading tests on as-built beam-column joint units with the details typical of the subject building were carried out using plain round longitudinal bars. The seismic assessment of the subject building was then conducted based on the observed behaviour, as a continuation of the research project [L1].

The paper presented here is the summary of the seismic assessment of the subject reinforced concrete building, in which the longitudinal reinforcing bars were plain round. Both non-linear static and non-linear dynamic analyses were carried out. The main objective of this assessment was to identify the effect of the use of plain round longitudinal reinforcing bars on the overall post-elastic performance of similar reinforced concrete buildings.

2. DESCRIPTION OF THE SUBJECT BUILDING

The subject building is a reinforced concrete frame building constructed in the late 1950s in Christchurch, New Zealand, and it has been thoroughly investigated in the past. Figure 1 shows the typical floor plan of this subject building. There are 5 spans in the X (longitudinal) direction, and each span is 4 m. There are 3 spans in the Y (transverse) direction and each span is 4.9 m. There are seven stories, and the storey height is 3.2 m, except that the first storey is 3.81 m high. The building plane configuration is reasonably symmetric in both the X and Y directions. The building was founded on large foundation beams and reinforced concrete piles. The design details show many deficiencies in terms of the current design standard [L1].

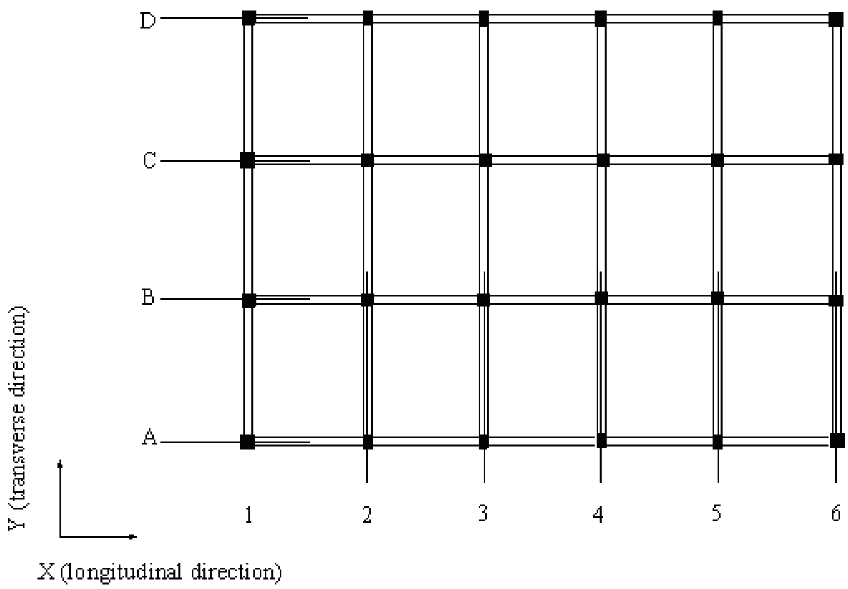
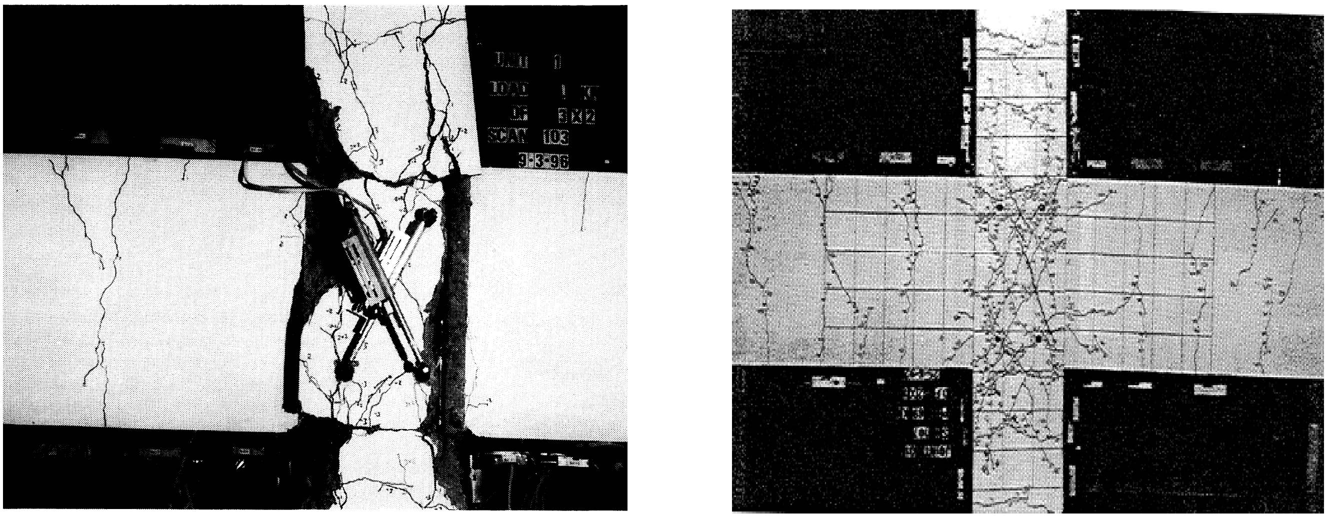


Figure 1 Typical Floor Plan

3. CHARACTERISTICS OF NON-LINEAR BEHAVIOUR OF AS-BUILT REINFORCED CONCRETE JOINT UNITS WITH PLAIN ROUND LONGITUDINAL BARS [L1]

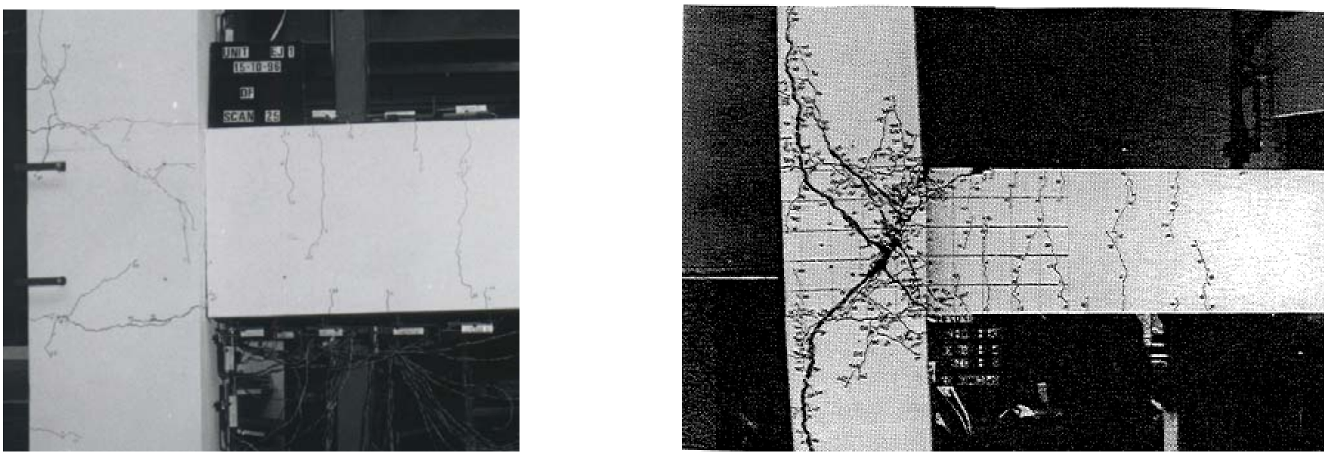
As part of this research programme, simulated seismic loading tests were conducted on six as-built full-scale reinforced concrete beam-column joint units, two interior and four exterior units. All the units had the reinforcing details typical of this subject building, and the longitudinal reinforcing bars were plain round bars in this test series, as was common in NZ before 1960.

The assessment of the test units based on both the current New Zealand code [N2] and the current seismic assessment procedure [N1] showed that the premature shear failure in the beams, columns and within the joint regions would take place well before the flexural failure in the beams and columns. Figures 2 and 3 respectively show the final appearance of one typical interior beam-column joint unit and one typical exterior beam-column joint unit. In Figures 2 and 3, the final appearance of the identical test units but reinforced by deformed bars are also shown for comparison [H1]. It is apparent that the predicted premature shear failure for the members and the joint regions, although occurred when the deformed longitudinal bars were used, did not occur when the plain round longitudinal bars were used. The observed post-elastic behaviour of the as-built reinforced concrete members with plain round longitudinal bars was mainly governed by the flexural behaviour at the fixed-ends (the beam-column interfaces), and the post-elastic deformation did not spread to a bigger region as conventionally called "plastic hinge region". The significant characteristics of the observed seismic performance of as-built units with plain round longitudinal bars was that column bar buckling and severe bond slip along the plain round longitudinal bars occurred adjacent to the joint region. Clearly, the current seismic assessment procedure in NZ [N1] would not give a good indication of the seismic performance of the existing reinforced concrete buildings reinforced by plain round longitudinal bars.



(a) with plain round longitudinal bars (b) with deformed longitudinal bars

Figure 2 Final Appearance of the As-Built Interior Beam-Column Joint Units



(a) with plain round longitudinal bars (b) with deformed longitudinal bar

Figure 3 Final Appearance of the As-Built Exterior Beam-Column Joint Units

Figure 4 shows the observed storey-shear versus storey-displacement hysteresis loops for an as-built interior beam-column joint unit. Again the observed behaviour of the otherwise identical unit but reinforced by deformed bars is shown for the purpose of comparison. It is seen that, due to the use of plain round longitudinal bars, the initial stiffness at first yield and the attained force strength were low. In comparison with the test on the otherwise identical unit but reinforced by deformed bars, the attained force strength was only 85% and the attained initial stiffness at first yield was about 65%. For the reinforced concrete members with plain round bars, the rotational ductility, rather than the curvature ductility, was found to be a much better deformation index, and the beams were found to have significant strength degradation when the rotational ductility reached about 5. The column bar buckling and severe bond slip along the plain round longitudinal bars had led to lower flexural strength and stiffness.

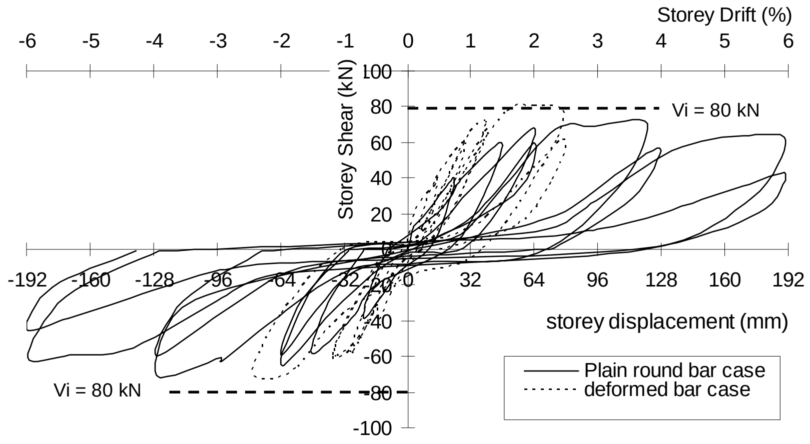


Figure 4 Observed Storey Shear versus Storey Displacement Curves

4. SEISMIC ASSESSMENT OF THE SUBJECT BUILDING

4.1 General

The assessment of the subject building was conducted by non-linear static push-over analysis and non-linear dynamic analysis, based on the observed non-linear behaviour of beam-column joint units. The non-linear structural analysis program RUAUMOKO was used [C2]. The non-linear static push-over analysis was to identify the post-elastic failure mechanism and determine the associated strength and deformation capacity when the longitudinal bars were from plain round bars. The lateral load pattern was approximated as a triangular distribution. The non-linear dynamic analysis was carried out for the subject building as well in order to provide a comparison with the static analytical results. The first ten second of the El-Centro 1940NS earthquake motion, which was scaled to the New Zealand seismic loading code NZS1170.5 for Soil Class C, was used.

4.2 Assumptions and Member Modelling

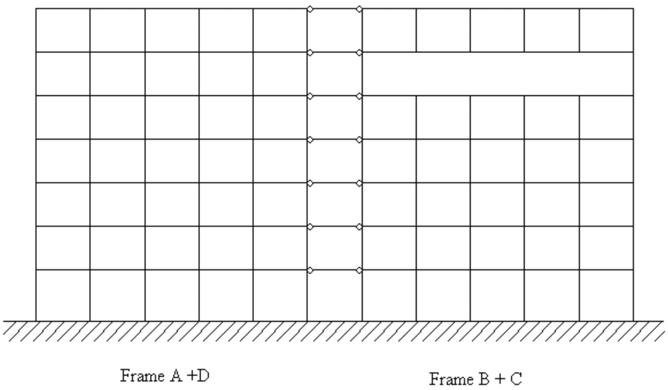


Figure 5 The 2D Structural Modelling

1. Two-dimensional analysis

The scope of this study was limited to the X direction, and a 2D analysis in the X direction was conducted. A 2D analysis was believed to be adequate because the building is relatively regular. Figure 5 shows the 2D structural modelling where frames A and D represents two exterior frames, and frames B and C represents two interior frames.

2. Rigid beam-column joints

Beam-column joints were modelled as infinitely rigid. This assumption was also believed adequate because simulated seismic loading tests on the laboratory test units showed good integrity of the joint panel when plain round longitudinal reinforcement was used.

3. No interaction between upper structure and foundation

4. All the masses were lumped at each floor level

5. Reinforced concrete member modelling

The beams and columns were modelled using Giberson's one-component model [G1, G2], which has non-linear rotational spring at each end. This was believed to be adequate because the observed test evidence was that the non-linear flexural deformation of the as-built reinforced concrete member was limited to the fixed-ends of the member, and no significant shear deformation was observed for the members and the joint panel. For the non-linear behaviour of the flexural springs of the one-component model, the flexural strength attainment at first yield was taken as 85% of the theoretical flexural strength. The determination of the member flexural initial stiffness was based on the test evidence [L1]. The beam stiffness at the first yield was taken as 50% of the theoretical prediction when the column axial load at the same joint is zero, and 75% of the theoretical prediction when the column compressive axial load is not less than 0.25 $A_g f_c'$, where A_g and f_c' are respectively the gross sectional area and the compressive concrete strength of the columns. For the cases between, the interpolation method is used to estimate the enhancement effect on the beam flexural stiffness at the first yield due to the compressive axial action on the transverse members. For the as-built columns, the flexural initial stiffness was taken as 75% of the theoretical initial stiffness, irrespective of the beam actions at the same joint. The hysteretic behaviour of the beam flexural non-linear spring was modelled using the Takeda slip model [T1], which has an unsymmetrical skeleton curve because the existing beams had unsymmetrical longitudinal reinforcement. The hysteretic behaviour of the column flexural non-linear spring was modelled using the Wayne Stewart model [S1], which does not vary the positive and negative flexural strength. Both hysteresis models can capture the bond-slip effect.

For the two hysteresis models, the parameters for defining the internal rules were calibrated against the test data using the program "Hysteresis" [C1] developed at the University of Canterbury.

4.3 Results of Non-linear Static Push – Over Assessment

Figure 6 shows the strength and deformation capacity curve for the subject building, in terms of the base shear and the drift at the roof level, obtained from the non-linear static push-over analysis as described previously.

The overall performance of the subject building was found to be limited by the beam local deformation capacity, rather than by the code-specified inter-storey drift limit or the formation of a collapse mechanism. At the roof drift equal to 0.9%, the maximum deformation demand on the beams was 5 in terms of the rotational ductility, which was the point when significant strength degradation was observed to start. At reaching the maximum rotation ductility of 5 in the beams, the hinging formation across the frame was mainly in the beams, and the collapse mechanism was not developed at this stage. At this stage, the maximum inter-storey drift was found to be 1.25%, which is less than the code specified inter-storey drift limit, 1.8%.

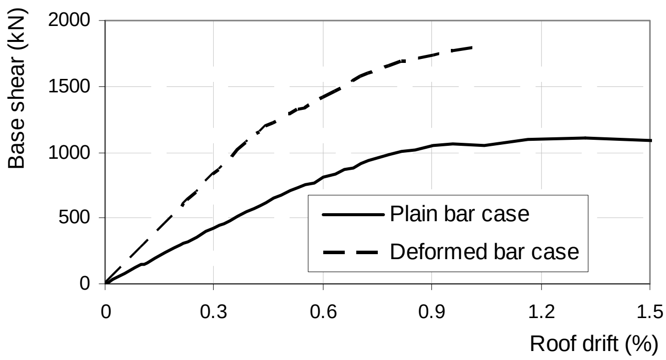


Figure 6 The strength and deformation capacity curve for the subject building

As seen in Figure 6, prior to reaching the roof drift of 0.9%, the force strength versus deflection curve is fairly linear so the roof drift of 0.9% could be assumed to be ductility of 1. At the development of the roof drift of 0.9%, the force strength in terms of the base shear capacity is 1050 kN, and this is equivalent to a seismic coefficient of 0.128. For this subject building, the fundamental period is T_1=1.86 s and the elastic design seismic coefficient to AS/NZS1170.5 [N2] is 0.16 assuming a soil class of C and a ductility of 1. Hence the subject building could meet about 80% of the current seismic standard.

Figure 6 also shows the strength and deformation capacity curve for the subject building, in terms of the base shear and the drift at the roof level, obtained by the non-linear static push-over analysis and assuming the use of deformed longitudinal bars [H1]. The probable strength of the building in terms of the base shear at developing the roof drift of 0.9% was 1800 kN, which is equivalent to a seismic coefficient of 0.22. At this stage, the maximum curvature ductility demand on the members was 10, and the observed test evidence showed that the existing members could achieve the required deformation capacity without any significant shear strength degradation. The fundamental period was T_1=1.32 s in this case and the elastic design seismic coefficient to AS/NZS1170.5 is 0.22. Therefore the subject building could meet the full current seismic standard, if the longitudinal bars were from deformed bars [H1].

It is clear from Figure 6 that the use of plain round bars led to a significant reduction in the force strength and the initial stiffness of the building, in comparison with the case using deformed longitudinal bars. The achieved force strength and the initial stiffness was about 60% of the values for the deformed bar case. The lower initial stiffness associated with the use of plain round longitudinal bars shifted the fundamental period from 1.32 s to 1.86 s, resulting in a reduced seismic demand (about 73%). However the use of the plain round longitudinal bars meant that the overall seismic performance of the building degraded to about 80% of that with deformed longitudinal bars. Therefore the seismic assessment of the existing reinforced concrete buildings using the current seismic assessment procedure would significantly overestimate the capacity of the building when the buildings actually contain plain round longitudinal bars.

4.4 Results of Non-linear Dynamic Assessment [L1]

The non-linear dynamic assessment under the first ten seconds of the El-Centro 1940NS earthquake motion[C2], which was scaled to the current New Zealand seismic loading code NZS1170.5 for Soil Class C, was conducted, and the P-Δ effect was investigated.

The analysis reveals that, in comparison with the case without including the P-Δ effect, the inclusion of the P-Δ effect has not made any noticeable changes in the overall seismic performance of the subject building. Namely the attained maximum base shear force strength, the maximum roof displacement demand and the maximum deformation demand on the members were basically the same, irrespective of the inclusion of P-Δ effect. This was mainly because the maximum inter-storey drift demands were lower than 1% except the bottom level. The

P-Δ effect would not be significant if the inter-storey drifts were small. The identified hinging formation is the preferred weak beam - strong column mechanism but not a collapse mechanism.

The attained maximum force strength in terms of base shear is 1280 kN and it equals to a seismic coefficient of 0.15. The maximum inter-storey drift demand was 1.35%, which occurred at the bottom level, and it is below the code specified inter-storey drift limit. The maximum deformation demand on the beams was 4.8 in terms of the rotational ductility and this is below the observed beam deformation capacity. Hence, the subject building would survive during a major earthquake of similar magnitude and similar vibration characteristics to the El Centro 1940NS earthquake motion.

Comparing the failure mechanisms reached by static and dynamic analyses shows that consideration of the effect of bond slip and therefore the strength degradation as for the dynamic analysis led to much more widely spread hinging across the building. The force strength in terms of base shear, obtained by dynamic analysis, was about 120% of that by static analysis. This reveals that the inclusion of the effect of bond degradation in the structural modelling led to more evenly spread hinging, therefore greater force strength because the analysis was terminated by the attainment of the maximum member local deformation capacity, in comparison with the non-linear static analysis.

5. CONCLUSIONS

The seismic performance of an existing reinforced concrete frame building constructed in Christchurch, New Zealand in the late 1950s was assessed by non-linear static push-over and non-linear dynamic analyses. The non-linear seismic behaviour of the existing members was modelled based on the observed test evidence. Seismic assessment of the existing reinforced concrete frame structure led to the following conclusions:

1. Unlike the case with deformed longitudinal bars, the non-linear seismic behaviour of the existing reinforced concrete buildings reinforced by plain round longitudinal bars was limited to the fixed-ends of the members, and premature shear failure would not occur in the members and joints. Hence the adequacy of the non-linear analysis is determined by the adequacy of the modelling of the flexural behaviour at the member fixed-ends. Therefore the current seismic assessment procedure in NZ, which has a great deal of assessing the possible premature shear failure, would not properly identify the failure mechanism of the reinforced concrete frame buildings with plain round longitudinal reinforcement.

2. Both non-linear dynamic and non-linear static analyses showed that the overall non-linear behaviour of the subject building was mainly dominated by the non-linear flexural behaviour of the beams.

3. The non-linear static (push-over) analysis showed that the structural post-yielding performance was governed by the member local deformation capacity, rather than by the code-specified deflection limit or the formation of the collapse mechanism. The seismic performance of the subject building only met about 80% of the current seismic standard. Of importance is that the seismic performance of the subject building with plain round longitudinal bars was about 80% of the similar buildings with deformed longitudinal bars. Therefore the current seismic assessment procedure could significantly overestimate the capacity of the existing reinforced concrete buildings with plain round longitudinal bars.

4. The non-linear dynamic analysis of the subject building showed that the building would survive during a major earthquake of similar magnitude and similar vibration characteristics to the El Centro 1940NS earthquake motion. Comparison of the analysis results of static and dynamic analysis shows that the inclusion of the effect of bond slip and strength degradation in the dynamic analysis resulted in more evenly spread of hinging and therefore slightly higher force strength.

5. The assessment of the subject building conducted here has not monitored the effect of the observed column bar buckling associated with the use of plain round longitudinal bars. It is recommended that

further research be carried out to develop a methodology of properly modelling the effect of column bar buckling on the overall non-linear behaviour of similar buildings.

ACKNOWLEDGMENTS

The research funding provided by the Earthquake Commission of New Zealand for this project and the Doctoral Scholarship provided by the University of Canterbury are gratefully acknowledged. Thanks are also due to Beca Carter Hollings & Ferner Ltd, Wellington, New Zealand, for the support. Special acknowledgement is made of the supervision and encouragement given by the late Prof. R. Park.

REFERENCES

[A1] ATC (1996). Seismic Evaluation and Retrofit of Concrete Buildings, Applied Technology Council, California, ATC-40

[C1] Carr, A. Hysteresis Program, the University of Canterbury, New Zealand

[C2] Carr, A. RUAUMOKO – Inelastic Dynamic Analysis, University of Canterbury, NZ

[G1] Giberson, M.F. (1969). Two Non-linear Beams with Definitions of Ductility, *Journal of the Structural Division*, *ASCE*, Vol. 95, No. ST2

[G2] Giberson, M.F. (1967). The response of Nonlinear Multi-storey Structures subjected to Earthquake Excitation, Earthquake Engineering Research Laboratory, California Institute of Technology Pasadena, California, *EERL Report*

[H1] Hakuto, S., Park, R. and Tanaka, H. (1995). Retrofitting of Reinforced Concrete Moment Resisting Frames, Research Report 95-4, Department of Civil Engineering, the University of Canterbury.

[L1] Liu, A. and Park, R. (2002). Seismic Assessment and retrofit of Pre-1970s Reinforced Concrete Frame Structures, Research Report 2002-1, Department of Civil Engineering, the University of Canterbury, New Zealand

[N1] NZSEE (2006). Assessment and Improvement of the Structural Performance of Buildings in Earthquakes, New Zealand Society of Earthquake Engineering

[N2] New Zealand Standard (2004). NZS1170.5 :2004, Structural Design Actions - Part 5: Earthquake actions – New Zealand

[P1] Park, R. (1981). Review of Code Developments in New Zealand, *Bulletin of New Zealand National Society for Earthquake Engineering*, Vol 104, No.3

[P2] Park, R. (1992). Seismic Assessment and Retrofit of Concrete Structures-United States and New Zealand Developments, *Proceedings of Technical Conference of New Zealand Concrete Society, Wairakei*, pp18-25

[S1] Stewart, W.G. (1987). The Seismic Design of Plywood Sheathed Shear Walls, Ph.D Thesis, Department of Civil Engineering, University of Canterbury, New Zealand

[T1] Takeda, T. Sozen, M.A. and Nielsen, N.N. (1970). Reinforced Concrete Response to Simulated Earthquakes, *J. Struct, Engrg. Div.*, *ASCE*, Vol 96, No 12