PRELIMINARY EXPERIMENTAL AND NUMERICAL INVESTIGATION ON THE SEISMIC RESIDUAL CAPACITY OF REINFORCED CONCRETE BEAM-COLUMN JOINTS

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SUMMARY

This paper presents preliminary results of an experimental campaign as part of an ongoing research project aiming at investigating the seismic residual capacity of reinforced concrete frames, in terms of number and intensity of aftershocks the structure could withstand following a major earthquake. Full-scale (capacity designed) beam-column joints extracted from a 22-storey reinforced concrete frame building, constructed in late 1980s at the Christchurch’s CBD were tested under quasi-static displacement controlled lateral loading. Subsequently, numerical and nonlinear finite element (FE) analyses were performed and calibrated based on the experimental results.

INTRODUCTION

According to capacity design principles, structures are designed to withstand major earthquakes by developing inelastic action and energy dissipation in concentrated regions referred to as plastic hinges. Therefore, in agreement with current seismic performance-based design guidelines [1], structural damage is expected to occur. In principle, modern structures should be capable of remaining fully operational (i.e. with negligible structural and non-structural damage) after frequent earthquakes, operational (i.e. with some non-structural damage without significant structural damage) after occasional ones, and allow for life safety (i.e. without collapsing) during a rare or design level earthquake.

The above philosophy implicitly means that modern structures should also be able to withstand several frequent and/or occasional earthquakes over their life span, and that they might suffer some level (minor-to-moderate) of damage and require some post-earthquake structural and non-structural repairs. Nevertheless, despite the availability and recent development of seismic assessment and rehabilitation guidelines, they are mainly focused in mitigating the seismic risk of existing buildings designed prior to capacity design principles. Very little information and assistance is provided for a) assessing the residual capacity of damaged modern buildings, and for b) selection and implementation of a reliable repairing technique capable of bringing (either totally or partially) the structure back to its pre-earthquake condition.

As part of an ongoing research project aiming at investigating the seismic residual capacity of reinforced concrete structures, this paper presents preliminary results of an experimental and numerical campaign on “modern designed” beam-column joints extracted from a 1980s multi-storey reinforced concrete frame building. The main objectives of the overall project are: a) the evaluation of the residual capacity of existing reinforced concrete buildings to sustain subsequent aftershocks and/or other design level earthquake during the remaining life of the building, and b) the identification and better understanding of the effectiveness of epoxy injection techniques, widely proposed and adopted in practice, for partly or fully restoring the
seismic capacity of moderately damaged reinforced concrete members. This paper presents preliminary results on the first objective.

BUILDING DESCRIPTION

The PWC building (see Figure 1) was a 22-storey structure built in the 1980s and located on Armagh Street in the Christchurch’s Central Business (CBD) area. The lateral system comprised precast perimeter reinforced concrete (RC) frames with wet joints (typical of emulation of cast-in-place approach) in the beams at mid-span. The gravity system comprised precast double-tees with a reinforced concrete topping, supported on steel beams and concrete columns. It was designed following capacity design principles. The perimeter frames had a hoop detail in the beam-ends intended to relocate the plastic hinge 500mm away from the column face, so to avoid excessive demand and damage at the beam-column joint. The foundation system consisted of raft foundations.

During the CES, the building appeared to behave as expected, with the beams developing plastic hinges at both ends over the full height of the structure, with a general trend of diminishing level of damage along the elevation. The columns or joints did not show any signs of damage. The building experienced more damaged in the EW direction, consistent with the direction of the strongest components recorded in the surrounding area. Maximum observed residual cracks varied between 0.8mm and 20mm wide in the EW direction, and between 0.4mm and 8mm in the NS direction. Interestingly enough, most of the observed cracks were
within, instead of outside, the plastic hinge relocation detail (see Figure 2) which thus apparently did not work as intended per the original design.

Residual drifts and tilting (due to liquefaction and lateral spreading) were also observed. More detailed information on the observed damage can be obtained in [2] [3]. The building was considered uneconomical to be repaired and consequently demolished in 2012. Four “H frames” were extracted from the 16th floor level during the demolition process for experimental purposes (see Figure 1).

Figure 2. Typical damage observed in the superstructure of the PWC building, level 9 interior unit H4 (left) and level 4 corner unit H2 [3].

EXPERIMENTAL PROGRAM AND TESTING PROCEDURE

Test Specimen

The “H frames” extracted during the deconstruction process were later cut in two “T-shape” specimens due to laboratory crane capacity limitations. Each of the “T-shape” specimens tested weighs between 10.5ton and 13ton. The beams are 2550mm long (measured from the column face to the point of load application), 575mm wide by 1100mm high. The (main) longitudinal reinforcement consists of top and bottom 4 D-28 straight bars and 2 additional D28 hooked bars (within the plastic hinge relocation detail); the transverse reinforcement consists of 2 R-12 stirrups (one interior, one exterior) spaced at 150mm crs. There is also secondary reinforcement detailed such that it provides vertical support to the flooring system (see Figure 3). The columns are 2700mm long, either 1100mm square (at the building’s corners) or 1100mm by 800mm (at the building’s interior columns). The nominal steel yield strength and concrete compressive strengths, as specified in the drawings, are 300MPa and 30MPa, respectively.

Figure 3. Typical section and elevation view of beams tested [4].
Three beam column joints out of the eight extracted (i.e. four “H” frames) were initially tested at this stage. The first specimen showing no visible residual cracks, was cyclically tested in its as-is condition. The other two specimens which showed residual cracks varying between hairline and 1.0mm in width, were subjected to cyclic loading to simulate cracking patterns consistent with what can be considered moderate damage. The cracked specimens were then repaired with an epoxy injection technique and subsequently retested until reaching failure. This paper presents preliminary results on the first specimen and on the nonlinear finite element (FE) modelling of a subassembly with the same characteristics.

Experimental program

The first specimen (Test 1) corresponds to one of the frames oriented in the N-S direction, with no visible residual cracks and consequently considered as slightly damaged. A quasi-static displacement-controlled cycling loading protocol was applied at the beam-end (increasing “total” beam rotations of ±0.1%, ±0.2%, ±0.5%, ±0.75%, ±1.0%, ±1.5%, ±2.0%, and ±2.5%, see Figure 8) as per the acceptance criteria of the American Concrete Institute (ACI) [5]. The actuator was located approximately at the theoretical inflection point at mid-span of the beams oriented in the NS direction of the building.

As shown in Figure 4, the reaction frame used for the tests consisted of steel braced frames anchored to the RC strong floor. The beam-column joint was placed horizontally on top of steel beams and clamped to the braced frames with steel channels and post-tensioned Macalloy bars. The beam-end was vertically supported on Teflon pads. A steel frame with a roller prevented the beam-end to uplift due to any accidental eccentricity that might occur at the actuator-to-beam connection. No axial and gravity loads were applied at the column (in addition to the resultant force from the post-tensioned bars) and beam, respectively.

Instrumentation

The instrumentation consisted of 33 linear potentiometers for measuring displacements at different points along the beam and beam-column joint (required for further estimation of rotations and shear deformations), 1 load cell for measuring the applied load in the actuator, and 2 rotational potentiometers at the beam-end for measuring beam elongation and applied displacements. During the deconstruction process, the beams adjacent to the extracted “H frames” were cut-off approximately at the column face. Therefore, in some of the specimens the flexural beam capacity relies upon straight D28 bars developed over a length of 1100mm (i.e. without a standard hook), just above the minimum required. Two spring potentiometers were located at the bars at the cut-off section for measuring bar slips during the test, if any.

Three additional rotary potentiometers were strategically installed to capture rigid body translations and rotations of the specimen (due to axial elongation of the post-tensioned bars, slip at the reaction frame-to-strong floor connections, the specimen setup, and any other
deformation in the reaction frame that might occur during the test). This translation and rotation is further translated into an equivalent lateral displacement at the beam-end and extracted from the “total” applied displacement.

**NUMERICAL INVESTIGATION**

The nonlinear FE code MASA developed at the Institute for Construction Materials (IWB) of the University of Stuttgart [6], was used in this study. The concrete is modelled according to a microplane model, a three-dimensional (3D) microscopic model in which the material is characterized by uniaxial relations between the stress and strain components on planes of various orientations called “microplanes”. The smeared-crack concept was used for the modelling of the cracking of the concrete, and the reinforcing bars were represented with one-dimensional (1D) truss elements with a three-linear constitutive law.

The bond between the longitudinal reinforcement and concrete was modelled using discrete bond elements consisting of 1D nonlinear springs (see Figure 5). For transverse reinforcement a rigid connection between steel and concrete was assumed, neglecting the influence of the relative displacement between the stirrups and the concrete [7]. This discrete bond model is able to predict the bond behaviour of deformed bars under monotonic and cyclic loading; the bond deterioration is assumed to occur after some slip due to mechanical damage in the concrete-to-steel interface surrounding the ribs [8].

Hexahedral elements with side lengths of approximately 25mm in the joint area and the plastic hinge regions, and 75mm elsewhere were used to create the mesh of the elements. Linear elastic elements were used at the vicinities of the supports and the point of load application so that local failure of concrete elements due to excessive stresses is avoided. Mirror symmetry (i.e., symmetry about a vertical plane across a mid-section in the beam-column joint) was used to reduce the total number of nodes and elements and thus the required computational time.

![Figure 5. Microplane model: a) load transfer over a number of idealized contact planes; b) spatial discretization of unit-volume sphere by 21 microplanes [6]; c) Discrete bond model in MASA [8].](image)

**NUMERICAL – EXPERIMENTAL COMPARISON**

**Cracking pattern**

Cracks of 0.1mm in width opened up at 0.2% “total” beam rotation. These cracks might be pre-existing ones (earthquake induced) that closed due to the low level of inelastic action. At 1.5% total rotation most of the deformation was concentrated at a single diagonal crack 4-12mm wide (see Figure 6), and shear distortions became more evident. The reason of these diagonal cracks could be stress concentration due to the existence of a socket intended to provide support to non-structural components, as well as excessive principal tensile stresses as a result of the diagonal compression strut induced by the hooked bars within the plastic hinge relocation detail. It was not possible to test the specimen up to failure due to the excessive and
unexpected shear deformation (and sliding shear mechanism) of the specimen. The reaction frame was later modified (for Tests 2 and 3) in order to accommodate such displacement. Worth noting that some of the specimens were also part of the structure’s gravity system, and the inclusion of the gravity load effect during the test would have triggered the specimen’s shear failure at an earlier stage.

1.5% “total” beam rotation 2.5% “total” beam rotation

Figure 6. Residual racking pattern observed during Test 1, at 1.5% and 2.5% “total” beam rotation.

Figure 7 shows snapshots of the cracking patterns in the specimen during the experimental campaign at beam rotations (in “total” displacement units) of 0.5%, 1.0%, 1.5%, and 2.5%. It is evident how the diagonal crack is almost imperceptible at about 1.0% beam rotation, and dramatically increases in size at 1.5% and subsequent rotation level. Figure 7 also shows the expected cracking pattern obtained with the FE model. As seen in this figure, the simulation agrees reasonably well with the observations. Shear distortions became more critical after 1.5% beam rotations. Most of the damage occurs within the plastic hinge relocation detail, with some cracking developing outside this region. The main difference between the observations and simulation is that MASA is not able to capture the appearance of the diagonal crack propagating from the socket.

Hysteretic behaviour

Figure 8 shows the cyclic lateral force-displacement response from the quasi-static test (light-grey curves) measured in “effective” displacement units (i.e., the “total” applied displacement minus the equivalent lateral displacement at the beam end due to rigid body translation and rotation). The onset of nonlinearity occurs at about 0.5% “effective” beam rotation. The hysteresis loops show a fat and fairly stable shape with some pinching at higher rotation levels. This pinching might be attributed mainly to the opening and closing of the diagonal cracks, in combination with bond-slip degradation. Figure 8 also shows the cyclic curve obtained from numerical analysis. Although the model is not able to capture the pinching at higher rotation levels (due to the formation of diagonal cracks) and there is some strength degradation at the last cycle (not observed during the test), the model is able to capture the hysteretic behaviour at lower rotation levels as well as some features at higher rotation levels such as the loading and unloading stiffnesses.

Figure 8 also shows the axial beam elongation measured during the test. The maximum elongation 1 was 21mm, equivalent to 0.85% increase in beam length, respectively. It is worth noting that the testing apparatus allowed for free beam elongation without any restraint action from the floor diaphragm as in fact would occur in the real building. Therefore, while the results are important to develop a better understanding of the behaviour of a “free” subassemblies, the beam elongation results are not fully representative of what we would have observed following the earthquakes.
Figure 7. Observed damaged at different beam rotations in "total" displacement units (left); deformed shape (scaled-up) and cracking pattern obtained with the finite element simulation (right).
CONCLUDING REMARKS

This paper presents preliminary results of an experimental and numerical investigation on modern designed beam-column joints extracted from a 1980s 22-storey reinforced concrete frame building in the Christchurch’s Central Business District (CBD), damaged after the 2010-2011 Christchurch earthquakes sequence (CES).

The specimen tested failed in a flexure-shear mechanism. Severe diagonal cracking was developed within the plastic hinge relocation detail due to the existence of a socket intended to provide support to non-structural components, and to excessive principal tensile stresses as a result of the diagonal compression strut induced by the hooked bars details.

The results obtained with nonlinear finite element (FE) simulation agreed reasonably well with the experimental observations. Parametric analyses are under development to investigate how the beam length-to-depth aspect ratio, transverse reinforcement and stirrups spacing within the plastic hinge relocation detail influence the nonlinear behaviour of the specimen.
REFERENCES


