HIGHLIGHTING THE NEED FOR MULTIPLE LOADING PROTOCOLS IN BI-DIRECTIONAL TESTING

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ABSTRACT

Major earthquakes, such as the Canterbury and Kaikoura events recorded in New Zealand in 2010-2011 and 2016 respectively, highlighted that floor systems can be heavily damaged. Quasi-static cyclic experimental tests of structural sub-assemblies can help to establish the seismic performance of structural systems. However, the experimental performance obtained with such tests is likely to be dependent on the loading protocol adopted. This paper provides an overview of the loading protocols which have been assumed in previous experimental activities, with emphasis on those adopted for testing floor systems. The paper also describes the procedure used to define the loading protocol applied in the testing of a large precast concrete floor diaphragm as part of the ReCast floor project jointly conducted by the University of Canterbury, the University of Auckland and BRANZ. Subsequently the limitations of current loading protocols for bi-directional testing are discussed. The relevance of local seismicity on bidirectional demand is demonstrated by examining a large dataset of records from the RESORCE database. It is concluded that bi-directional experimental testing be undertaken using at least two loading protocols that impose different ratios of demand in orthogonal directions.

INTRODUCTION

As highlighted by the 2010-2011 Canterbury and 2016 Kaikoura earthquake events in New Zealand, traditional floor systems can experience extensive damage and pose a significant life-safety risk, [1–5]. Kam et al. illustrated several examples of extensive floor diaphragm damages, typically induced by beam-elongation effects associated with ductile beam hinging mechanisms, [2]. Corney et al. and Fleischmann et al. also conducted a damage evaluation of the 2010–2011 Canterbury earthquake sequence, with special attention to the damages to precast concrete structures, [3,6]. Floor systems with up to 20mm wide longitudinal cracks in the topping slab have been reported, which resulted in the fracture of the topping slab mesh reinforcements. Damage to concrete buildings with precast floors during the 2016 Kaikoura earthquake have been analysed and reported by Henry et al. in [4] who observed several cases of significant beam elongation and associated support beam rotation which induced damages to precast floor unit supports. In one of the cases, this caused the loss of support for double-tee units.

As a result of the extensive damages observed, the research community has been making efforts to better identify the expected seismic performance of floor diaphragms and the efficacy of adopted and alternative retrofit solutions. Previous relevant research that involved large experimental tests was conducted at the University of Canterbury by Bull et al., [7–9]. In [10], the influence of beam elongation on seat width requirements for precast floor slabs was the principal issue of concern. In [11], the typical detailing used to attach the precast hollow-core units to the perimeter frames has been investigated and the test results have shown that collapse of precast concrete floors may be expected. In [7] the results of a large experimental test on a full-scale super-assemblage have been presented and de-

tails have been proposed for attaching the hollow-core units to the supporting beams to try to improve the performance of the hollow-core units. In [8], new end beams with larger (75mm) seating depths have been considered and examined together with a simple (pinned) connection between the hollow-core units and the beam adopted. The efficacy of the presented detailing improvements was also pointed out. Finally, in [9] it is highlighted that with simple detailing enhancements, significant improvement in the seismic performance of hollow-core floor systems can be expected. A summary of the research programs and key design and assessment provisions for hollow-core floors in New Zealand was recently presented in [12,13].

As also highlighted by some of the work already mentioned, at a reduced or full scale, quasi-static cyclic experimental tests on structural sub-assemblies have been conducted by a number of researchers to gain insights into the seismic performance of structural systems. However, a critical aspect of quasi-static cyclic testing procedures is associated with the dependency of the performance on the characteristics of the loading protocol assumed. Ingham et al. [14] investigated the influence of loading histories on the performance of plastic hinges in a reinforced concrete beam, considering twelve loading histories employed in the United States, Japan and New Zealand and artificially generated histories derived from recorded earthquake ground motions. The study demonstrated that the ultimate displacement of a test sub-assemblage was dependent on the applied loading procedure. More recently, Pujol et al. [15] tested 16 reinforced concrete columns with different displacement histories and also showed that the latter had a strong influence on the resulting drift capacity of the columns.

The identification of appropriate loading protocols for testing floor diaphragms with quasi-static procedures is therefore of high

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relevance, and an improved characterization of bi-directional loading protocols would be of value for researchers to obtain reliable seismic performance predictions.

This paper presents an overview of the most common loading protocols that have been proposed in literature and implemented in quasi-static testing procedures. Unidirectional and bidirectional loading protocols intended to replicate both the demand imposed by ordinary and near-fault ground motions are reviewed. The procedure adopted to determine the demand imposed by the Kaikoura earthquake on a typical building structure of the Wellington area, New Zealand, and implemented during the experimental test of the ReCast project, is described. The main parameters required to be identified for suitable bidirectional loading protocols are pointed out and one of the major aspects, the ratio between the peak displacement demands in the two main directions of testing, is analysed.

REVIEW OF UNIDIRECTIONAL AND BIDIRECTIONAL LOADING PROTOCOLS

A large variety of unidirectional and bidirectional loading protocols have been proposed in the literature to replicate the effects of both ordinary (far-field) and near-fault ground motions. The next section provides an overview of some of the most commonly implemented unidirectional loading protocols, such as the the ACI 374.2R-13, the CUREE-Caltech protocol[16], the modified CUREE-Caltech protocol [17] and the FEMA 461 Interim Testing Protocol [18]. The subsequent section presents, in turn, a review of the bidirectional loading protocols which have been proposed and applied in the literature in previous experimental activities.
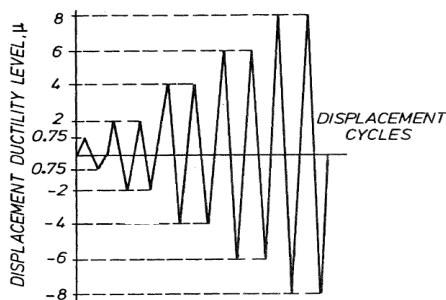

Figure 1: Displacement history used for quasi-static cyclic loading tests of columns and structural assemblages at the University of Canterbury, as reported by Park [19].

Unidirectional Loading Protocols

A quasi-static loading pattern for testing concrete elements and structural sub-assemblies that has been used for many years in New Zealand and also overseas, is depicted in Figure 1, as reported by Park in [19]. This loading protocol was specified in terms of ductility level, and as such it required the yield displacement to be identified. Park suggested to determine the yield displacement using the mean measured secant stiffness at 75% of the theoretical ultimate load or at first yield of the steel, whichever is least. The ductility level was increased step-wise and two cycles for each ductility level were applied. The strength requirement of the test was that the reduction of the strength not exceed 20%.

A deformation history for tests under unidirectional load reversal, similar to the one described by Park in [19], is proposed in the Guide for Testing Reinforced Concrete Structural Elements under Slowly Applied Simulated Seismic Loads, ACI 374.2R-13,
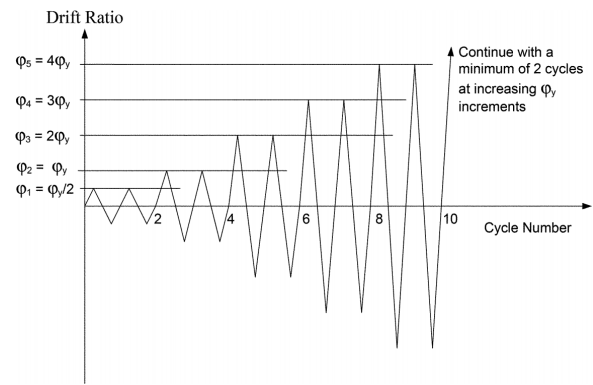

Figure 2: ACI374.2R-13 deformation history for tests under unidirectional load reversal.

[20]. The drift ratio is recommended to be specified considering the particular failure mechanism or performance objective investigated so that significant changes in specimen behaviour can be captured . Two or three cycles at each deformation level are typically appropriate. To capture performance within the elastic range of deformations an initial cycle approximately at one-half of yielding displacement or strength is suggested. The increase in subsequent drift levels is recommended to be in increments of ductility, as in [19].

Krawinkler et al. also argued that the choice of a testing program and associated loading history depends on the purpose of the experiment, type of test specimen, and type of anticipated failure mode (e.g., rapid strength deterioration such as caused by fracture, or slow strength deterioration such as caused by local buckling). For testing of steel beam-to-column subassemblies, in [17], Krawinkler et al. developed a multiple-step test, named the CUREE loading protocols, in which the deformation history consists of stepwise increasing deformation cycles as shown on the left side of Figure 3. The deformation parameter to be used to control the loading history is the inter-story drift angle. The protocol involves six cycles at a drift level equal to 0.00375, six cycles at a drift of 0.005, six cycles at a drift of 0.0075, four cycles at a drift of 0.01, two cycles at a drift of 0.015, two cycles at a drift of 0.02 and two cycles at a drift of 0.03. The test continues with increments of drift equal to 0.01, and with two cycles at each step, up to the maximum displacement of interest. A different loading protocol was also proposed to simulate the effects of near-fault events, as shown on the right side of Figure 3. The basic loading protocol was constructed assuming the SAC model buildings (3, 9, and 20 stories) designed for Los Angeles and Seattle, and the SAC ground motions records (10/50 and 20/50), [16]. The near-fault loading history was constructed based on the response of the SAC model buildings to the SAC near-fault ground motions for the Los Angeles location, [16].

Richards and Uang [21] conducted an analytical study to develop a loading protocol to be used for experimental testing of short links in eccentrically braced steel frames. Three eccentrically braced frames with short shear links were designed and modelled. Nonlinear time-history analyses were performed under twenty LMSR (large magnitude, M, small distance, R) Los Angeles records that have been used in the PEER projects, [22]. A new loading protocol was developed with a larger number of cycles, fewer large inelastic cycles and reduced cumulative rotation demand with respect to the AISC 2002 loading protocols.

For wood-frame structures, Krawinkler et al. [17] proposed a modified version of the previous CUREE loading protocols, as shown on the left and right sides of Figure 4, respectively. The
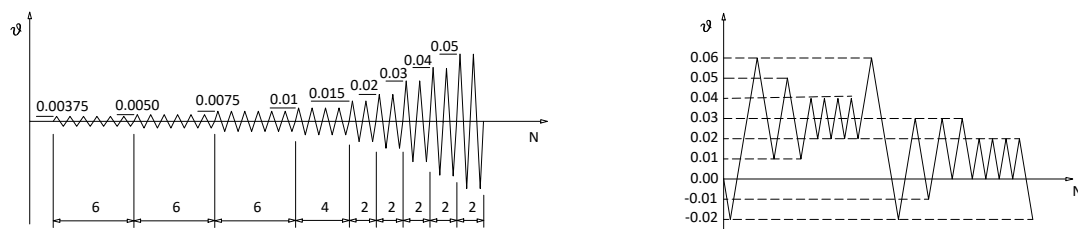
Figure 3: CUREE loading history for multiple-step test representing the effects of ordinary (on the left) and near-fault (on the right) ground motions.
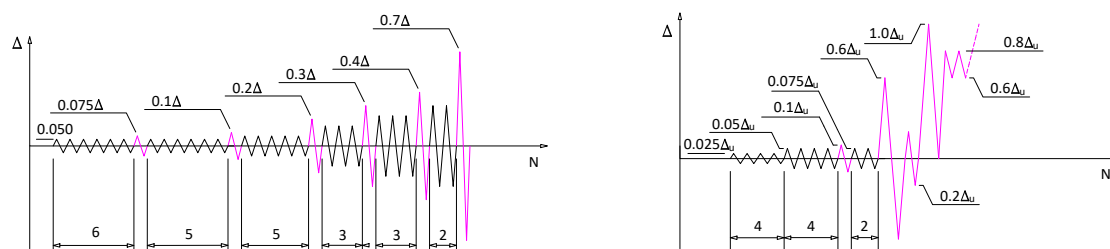

Figure 4: Modified CUREE loading history (Krawinkler 2001) for multiple-step test representing the effects of ordinary (on the left) and near-fault (on the right) ground motions.

modified loading histories are defined in terms of a reference deformation, Δ, which is the maximum deformation capacity the specimen is expected to sustain. The loading history for ordinary ground motions includes initial cycles, primary cycles, and trailing cycles. Initial cycles are executed at the beginning of the loading history to check the instrumentation (loading equipment, displacement, and force transducers) is working properly. A primary cycle is a cycle that is larger than all of the preceding cycles and is followed by smaller cycles, which are trailing cycles. All trailing cycles have an amplitude equal to 75% of the amplitude of the preceding primary cycle. All cycles are symmetric, with identical positive and negative amplitudes, and the test should be conducted with deformation control. The loading protocol in Figure 4, representing the effects of ordinary ground motions, includes four initial cycles at 0.05Δ, seven primary cycles at 0.075Δ, 0.01Δ, 0.02Δ, 0.03Δ, 0.04Δ, 0.06Δ and 1.0Δ. The first two primary cycles are followed by three cycles while the remaining five primary cycles by two cycles, all of them at 75% of the amplitude of the primary cycle. The loading protocol described, and shown on the left side of Figure 4, is an abbreviated version of the complete basic loading protocol proposed, [17], which in terms of the number of cycles and their amplitudes, is compatible with the original CUREE loading protocol introduced and shown on the right side of Figure 3.

In an effort to improve the state of the art in wood frame construction, Gatto and Uang [23] investigated the capacity of wood-frame structures with lateral resisting system composed of shear wall system. Quasi-static experimental tests with different loading protocols have been conducted and it was found that the performance of wood-frame shear walls is highly dependent on the loading sequence, and in particular protocols with a large number of equal-size cycles were shown to be the most demanding.

More recently, Krawinkler [24] highlighted that although several loading protocols have been developed and presented in the literature there is still a need to revisit them. The majority of existing cyclic loading protocols are appropriate for design level response and, as such, they are not suitable to simulate responses

close to collapse. If near-fault compatible loading protocols are excluded, loading histories are typically symmetric while the response of structures close to collapse is not at all symmetric. If component behaviour close to collapse is of primary interest, then in many (but not all) cases a typical component response history will be very different from the one a component experiences in a design level earthquake [25]. Consequently, it was recommended that ad-hoc loading protocols should be developed or, if collapse modelling is a primary objective, tests with existing cyclic loading protocols should be supplemented with monotonic tests. Suzuki and and Lignos [26] developed collapse-consistent protocols for experimental testing of steel columns. In this study, the loading protocol was developed assuming as critical failure mechanism the inelastic local buckling of the steel column. The developed loading protocol takes into account the effects of constant and variable axial load coupled with lateral drift demand.

The amplitudes and number of cycles of a loading protocol are not the only issue of relevance to obtain reliable evaluations of the seismic performance. Specified the number and amplitudes of the cycles, for example by statistical analysis, the performance obtained can be dependent by the specific sequence adopted to apply those cycles. Excursions with the same amplitude occurring at different times will contribute differently to structural damage. To take into account this issue Hutchinson et al. [27] proposed a damage index concept. A procedure to determine loading protocol was proposed that consists of (i) ground motion selection and scaling, (ii) representative building selection and modeling, (iii) nonlinear structural dynamic response calculations, and (iv) modified simple range counting to derive amplitude count information. The procedure was applied to determine a loading protocol for windows systems alternative to those proposed in FEMA 461.

Bidirectional Loading Protocols

As it was the case for unidirectional loading protocols, several bidirectional loading protocols have been proposed in the literature. Rodrigues et al. [28] observed seven commonly used
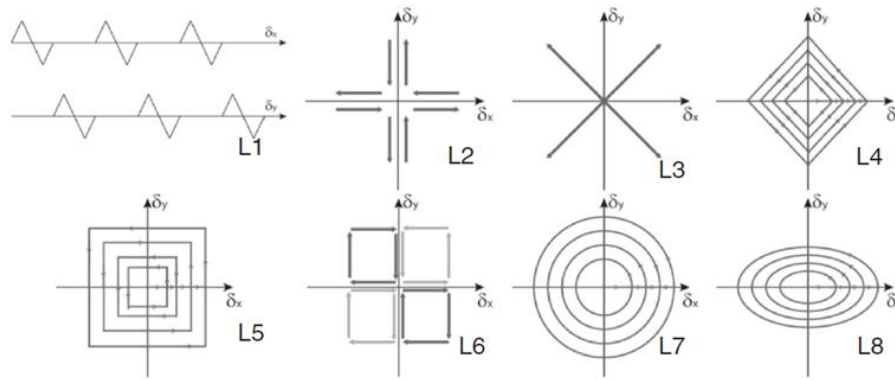
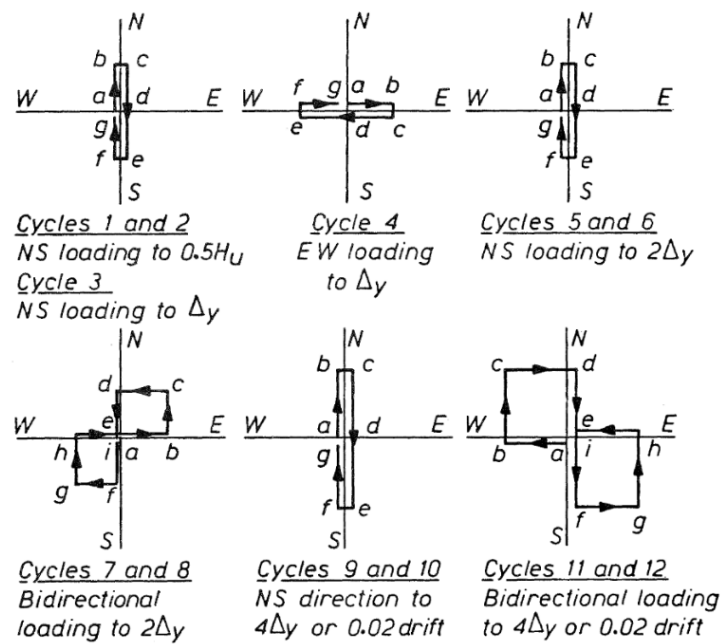*Figure 5: Bidirectional orbital patterns identified by Rodrigues et al. [28].*

Figure 6: Bidirectional displacment history used for quasi-static cyclic loading tests of the US-New Zealand-Japna-China collaborative research project on reinforced concrete beam-column joints, as reported by Park [19].

orbital patterns namely, cruciform, diagonal cruciform, Rhombus, expanding square, square in each quadrant, circular and elliptical paths, Figure 5.

A bidirectional displacement history used for quasi-static cyclic loading tests, agreed by the US-New Zealand-Japan-China collaborative project on reinforced concrete beam-column joints [29], was reported by Park [19]. As shown in Figure 6, this loading protocol consists of 12 cycles which are specified in terms of ductility level: the first six involve unidirectional loadings in either the north-south or east-west direction while the remain six (mainly) bidirectional loadings.

Clover leaf bidirectional loading protocols have been largely adopted in the past. Akguzel and Pampanin [30] conducted a series of quasi-static cyclic experimental tests with both unidirectional and bidirectional loading protocols to evaluate the effects of the variation of axial load and bidirectional loading on the seismic performance of reinforced concrete exterior beam-column joints retrofitted with fiber-reinforced polymer sheets. The unidirectional lateral displacement history consisted of two cycles at

increasing drift levels (0.1, 0.2, 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, and 4.0%) and a clover leaf orbital pattern, as shown in Figure 7, was adopted. One complete cycle of the clover shape was circumscribed at each specified drift level. During the bidirectional tests, specimens were subjected to a total of two excursions into the positive and negative direction in the x-axis and y-axis during each complete clover-shape cycle. The clover leaf load protocol was expressed in polar coordinates employing a sinusoidal curve expressed by $r(\theta) = Rsin(2\theta)$ where R represented the target radial displacement -equal to the magnitude of the maximum displacement vector at an angle of 45 degrees to the principal axis of the column at the particular drift level- and θ the angle of any point to the principal axes along the loading path. Bi-directional clover leaf loading protocols have also been used by Solberg et al. [31] to evaluate the seismic performance of a damage-protected highway bridge pier in the presence of bi-directional earthquake excitations.

Raza et al. [32–34] tested a building column under constant axial load and several bidirectional lateral actions. Three different bidirectional loading protocols, namely the linearized circular
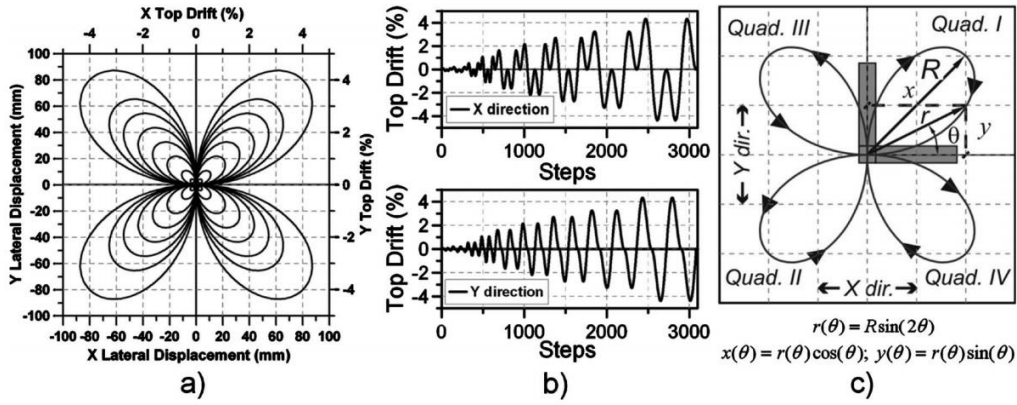
Figure 7: Loading protocols reported in Akguzel and Pampanin [30] for bi-directional quasi-static tests: (a) loading pattern; (b) displacement components; and (c) schematic representation of the cloverleaf orbital pattern.
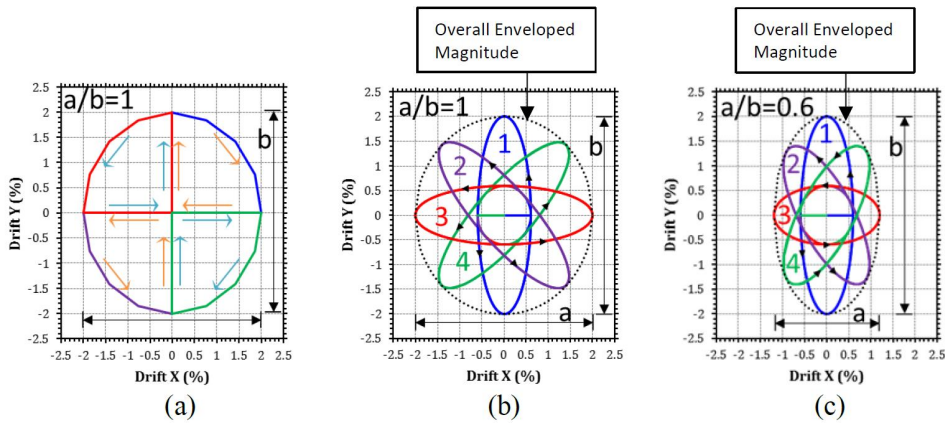

Figure 8: Loading protocols reported in Raza et al. [32] for bi-directional quasi-static tests.
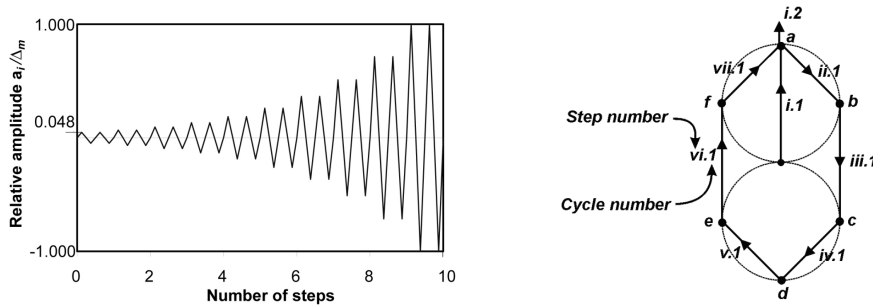

Figure 9: FEMA 461 loading protocol and orbital path.

path, Figure 8 on the left, the octo-elliptical path with equal displacement demand in the two orthogonal directions(a=b), Figure 8 central, and the octo-elliptical path with displacement demand in the main directions in the ratio a/b=0.6, Figure 8 on the right. In this paper, it is argued that the appropriate value of the ratio a/b should actually consider the seismicity of the region. Readers are referred to Raza et al. [32] for additional details into these bidirectional loading protocols.

Loading protocols for unidirectional and bidirectional quasi-

static experimental tests have been also included in the FEMA 461 Interim Testing Protocols for Determining the Seismic Performance Characteristics of Structural and Nonstructural Components document [18]. The quasi-static cyclic loading protocol presented in this document is intended to determine the performance characteristics of components, the behaviour of which is primarily controlled by the application of seismic forces or earthquake-induced displacements. The quasi-static cyclic procedure presented includes both a unidirectional loading protocol,
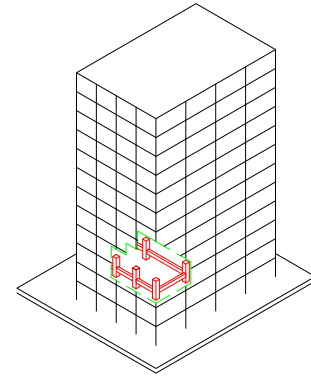
Figure 10: Global view of the building structure sub-assembly tested and building model geometry.

Figure 9 on the left, and instructions on the orbital pattern to be followed when bidirectional tests are conducted, Figure 9 on the right. The unidirectional loading history consists of repeated cycles of step-wise increasing deformation amplitudes. Two cycles at each amplitude shall be completed. The targeted smallest deformation amplitude, Δ_0, of the loading history must be safely smaller than the amplitude at which the lowest damage state is first observed. At the lowest damage state (or δ/h=0.0015 if no data exists) at least six cycles must have been executed. The targeted maximum deformation amplitude, Δ_m, of the loading history is an estimated value of the imposed deformation at which the most severe damage level is expected to initiate. This value must be estimated prior to the test. The number of steps in the loading history is usually equal to ten or larger. The number of cycles is therefore twenty or larger. The amplitude of the cycle a_{i+1} of the step i+1 is given by the following equation:

$$a_{i+1} = 1.4 \cdot a_i \tag{1}$$

where a_i is the amplitude of the preceding step, and a_{i+1} is the amplitude of the step closer to the target displacement, Δ_m.

Finally, ACI 374.2R-13, [20], recommend the same hexagonal orbital pattern presented by FEMA 461 in [18] and reported on the right side of Figure 9. This orbital pattern is meant to be associated with the deformation or ductility levels specified in the unidirectional loading protocol presented in Figure 2.

THE RECAST PROJECT LOADING PROTOCOLS

This section presents a procedure for developing drift protocols to evaluate the performance of floor systems under a specific earthquake excitation. The proposed procedure starts with the identification of the seismic hazard and a representative structure, and involves the evaluation of the seismic demand imposed on the building structure sub-assembly being tested with a quasi-static test. The demand obtained is post-processed and simplified to obtain the final drift protocol. The procedure is described assuming a typical New Zealand mid-rise building in Wellington. The results obtained have been applied to determine the loading protocol assumed for the large-scale experimental test conducted as part of the ReCast project, [35–38], as summarized in [39].

The ReCast project is a joint project between the University of Canterbury and the University of Auckland with support from BRANZ, that aims to (i) deepen the understanding of the seismic behaviour of precast concrete hollow-core floor diaphragms and (ii) identify the performances of common retrofit solutions adopted for precast concrete hollow-core floors in New Zealand. Additional details of the project can be found in [37–39]. The research project involves a series of experimental activities on

a large-scale building sub-assembly, Figure 10, conducted at the structural engineering laboratory (SEL) of the University of Canterbury in Christchurch. This section presents the procedure adopted to determine the first of the loading protocols assumed, with the aim of investigating the effects of the Kaikoura earthquake on precast concrete floor diaphragms of typical medium-rise buildings located in the Wellington area.

As previously noted, the 2016 Kaikoura earthquake caused significant damage to precast concrete floor diaphragms in the Wellington area. In view of this, for the ReCast project it was decided to consider a loading protocol that would impose displacement demands compatible with the demand imposed by the Kaikoura earthquake on typical medium-rise building structures located in the Wellington area. This required (i) the design of a typical building structure, for seismic demands and design criteria specified in the previous New Zealand standard NZS 4203:1984, (ii) the development of a complete three dimensional FEM model of the building structure, and (iii) evaluation of the displacement demand imposed on this representative building by the Kaikoura earthquake by conducting nonlinear dynamic analysis. The displacement demand obtained, at the sub-assembly level, was later simplified and regularized to identify the sequence of cycles, and their amplitude, required to simulate the demand imposed.
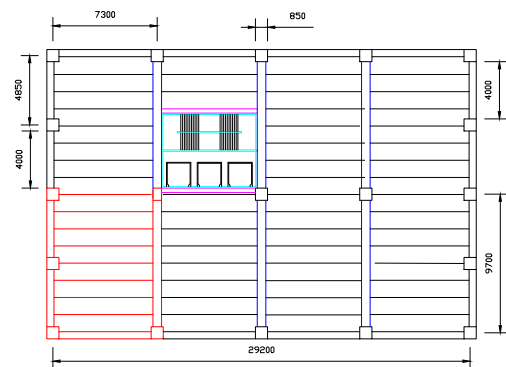

Figure 11: Global view of the building sub-assembly tested during the ReCast floors project.

Seismic Design of a Typical Structural System for Medium-rise Buildings in Wellington

To identify a typical existing mid-rise building structure, a simulated seismic design was conducted according to the New Zealand Standard NZS4203:1984. This required a series of as-

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sumptions that have been made assuming a hypothetical building located in Wellington.

Considering the dynamic characteristics of the Kaikoura earthquake, which was particularly demanding for medium-rise buildings, it was decided to consider a thirteen storey building. The lateral resisting system, both in the transverse and longitudinal directions, was taken to be composed of concrete moment resisting frames. The lateral load resisting system is provided around the building perimeter, and the internal columns are assumed to carry gravity loads only, i.e. gravity columns. As shown in the generic floor plan layout reported in Figure 11, internal beams have been assumed only in the transverse direction. The floor system consists of precast concrete hollow-core floor units spanning perpendicularly to the internal beams. An allowance for stairs and elevators was also considered, as reported in Figure 11.
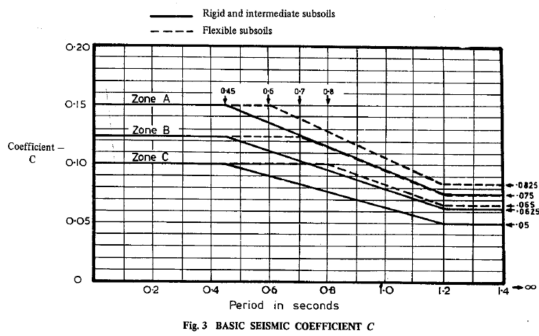

Figure 12: Basic Seismic Coefficient C, NZS 4203:1984.

In line with design provisions of the New Zealand Standard NZS 4203:1984, the seismic (base shear) demand, V_{base}, was obtained from the building seismic weight, $W_{Seismic}$, and the seismic coefficient, C_d, via Equation 2.

$$V_{base} = C_d \cdot W_{Seismic} \qquad (2)$$

The seismic coefficient, C_d, in turn, is specified in terms of four factors, as shown in Equation 3,

$$C_d = C \cdot R \cdot S \cdot M \qquad (3)$$

The *basic seismic coefficient C* is related to seismic zone, subsoil flexibility, and building fundamental period T. A value of C=0.0825 was obtained assuming the building is located in a high seismic risk zone (A) and has a fundamental period, T, longer than 1.2s, Figure 12. The *risk factor R* was assumed equal to 1 (buildings with normal occupancy), the *structural type factor S* equal to 0.8 (ductile frames), and the *structural material factor* equal to 0.8 (reinforced non-prestressed concrete). A base shear coefficient $C_d = 0.0528$ was obtained as summarized in Equation 4.

$$C_d = 0.0825 * 1 * 0.8 * 0.8 = 0.0528 \qquad (4)$$

The load analysis in the seismic combination resulted in a total building seismic weight of 62859kN. The weight of the generic floor was obtained summing the weight of the structural elements, beams, and columns, to the weight of the slab. The weight of the structural elements was obtained considering their geometry and a unitary concrete weight of 24kN/m3. The unitary floor weight, which includes the weight of 200 mm hollow core precast elements and an additional 1.95kPa of superimposed load, was 4.65kPa. As a result, the unitary floor weight (including columns
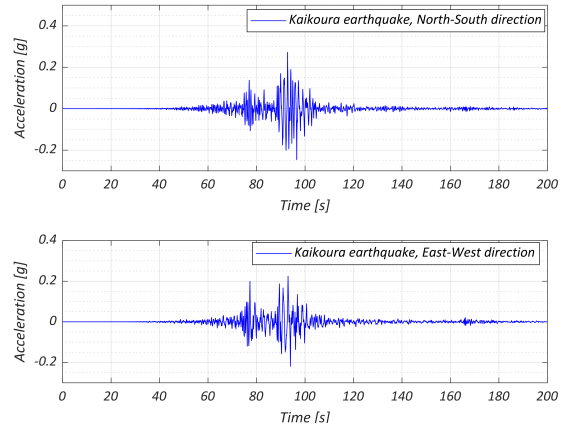

Figure 13: Kaikoura earthquake records recorded at the BNZ building in Wellington; N-S component (top) and E-W component (bottom).

and beams) was 9.2kN/m2, the total weight of the generic floor was 5238kN, and the slab weight was 2635kN.

The structural layout for the building consists of a symmetrical rectangular plan with main dimensions 29.2 m by 19.4 m, as shown in Figure 11. Constant bays lengths of 4.85 m and 7.3 m have been assumed respectively for the transverse and longitudinal directions. The inter-story height was assumed constant and equal to 4 m, as shown in Figure 14.

The gross section dimensions of beams and columns at the different levels of the building were assumed equal, for ease of construction and architectural aspects, but the longitudinal reinforcement ratios change over the building height. For simplicity, reinforcement ratios of beams and columns have been changed only at half of the building height, and from level 7 to 13 the longitudinal reinforcement ratios of beams and columns are reduced to 2/3rds of the values assumed at the lower floors. The beams' longitudinal reinforcement ratio is 2 times the minimum longitudinal steel content prescribed by the New Zealand standard. The primary beams of the building perimeter have been assumed 500 mm wide and 750 mm deep. The secondary internal beams are 550 mm deep and 600 mm wide. The external columns have a square section of 850mm each side while internal gravity columns have a 600mm square section.
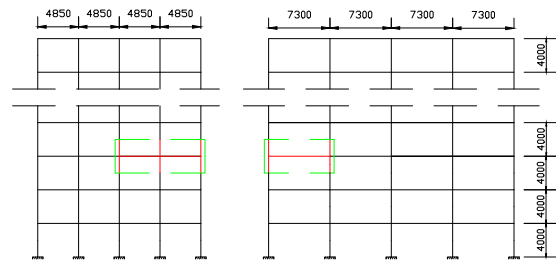

Figure 14: Building model transverse and longitudinal sections.

Assessing the Likely Response to Kaikoura Earthquake Ground Motion

Modelling and Analysis Assumptions

With the design finalized, the resulting building structure was modelled in OpenSees, [40,41], to determine the seismic performance under the Kaikoura earthquake record. The geometry of the OpenSees building model assumed is summarized in Figure 14, for both transverse and longitudinal elevations, and in Figure 11, for the generic plan. Beams and columns have both been modelled in Opensees using force-based fibre elements (nonlinearbeamcolumn elements in Opensees), specifying five integration points along each element. For modelling the section properties of the concrete core, the *Concrete02* concrete material model in OpenSees, [40,41], with concrete compressive strength equal to $f'_c = 25$Mpa, and ratio of the residual to ultimate maximum stresses equal to 0.2, was assumed. For modelling the (reduced) contribution to the section capacity of the concrete cover, the ratio between confined and unconfined concrete strength was set equal to 1.3. The *steel02* steel material model in OpenSees, [40,41], was assumed to model the longitudinal reinforcement and the yield strength was set equal to $f_y = 300$Mpa. The number of fibers was assumed equal to 12 for the core and to 2 for the cover, for both in-plane directions of the sections. Concentrated longitudinal and transverse mass properties have been assigned to the different nodes of the floor considering the plan layout shown in Figure 11. The damping model assumed was tangent-stiffness-proportional, [42], and the viscous damping ratio was set equal to 0.05. The Newmark time-stepping method (with γ=0.5 and β=0.25) was adopted with an integration time-step of 0.005s. Small displacement analyses were conducted. The building was found to have fundamental periods of 2.0s and 1.7s in the longitudinal (long bays) and transversal (short bays) directions, respectively.
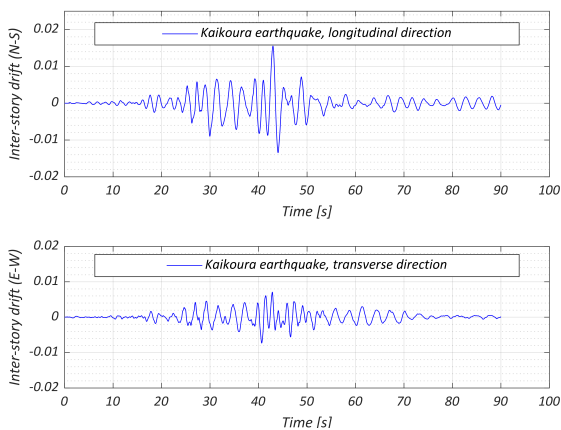

Figure 15: Inter-story drift time history at level 3, North-South (top) and South-East (bottom) directions for the Kaikoura earthquake.

Ground Motion for Development of Phase 1 Loading Protocol of the ReCast Project

The 2016 Kaikoura earthquake event caused significant damage to floor systems of buildings located in Wellington. To identify the demand imposed by this earthquake event and to replicate it experimentally, the typical medium-rise building structure designed in the previous section was subjected to both horizontal components of the Kaikoura earthquake recorded at the BNZ building location in Wellington. Both the North-South and East-West earthquake record components are shown in Figure 13, at
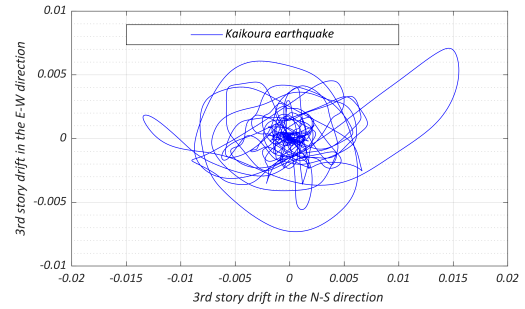

Figure 16: Bi-directional inter-story drift time history at the 3rd story for the Kaikoura earthquake.

the top and bottom respectively.

Analysis Results for Kaikoura Earthquake Motion

The main results of interest from non-linear time-history (NLTH) analyses are the interstory-drift at the level of the building where the test sub-assembly was supposedly located. Figure 15 presents the storey drift at the 3rd storey of the building for the two orthogonal directions as a function of time. Observe that for the longitudinal response direction the peak storey drift was estimated to be just over 1.5%. This would appear to align broadly with the peak response of other buildings subjected to the Kaikoura earthquake in Wellington. Figure 16 instead presents the combined N-S and E-W drift response at the third storey.

Interpreted Loading Protocol for Phase I of the ReCast Floors Experimental Testing

The loading protocol adopted to replicate the effects of the Kaikoura earthquake record was essentially based on the unidirectional drift response of the analysed building registered in the longitudinal direction, reported at the top of Figure 15. The inter-story drift time-history registered in the longitudinal direction between 42 and 45 seconds presents a large cycle with maximum drift approximately equal to 1.5%. The same plot, between 25 and 42 seconds, shows a series of cycles, approximately 8, with drift amplitudes between 0.45% and 0.8%. The first two/three of them have slightly lower amplitudes and the remaining five larger. These cycles have been approximated with three cycles at 0.5% and 5 cycles at 0.75%. Between 16 and 25 seconds, the inter-story drift time-history presents a few cycles with maximum amplitudes slightly larger than 0.2%. These cycles have been approximated by two cycles with amplitude 0.25%. Finally, after the main cycles, between 45 and 50 seconds, it can be seen that two relatively large cycles occur, with amplitude between 0.6% and 0.75%, and these have been approximated by two cycles with a drift level equal to 0.75%. In summary, as shown in the plot at the left side of Figure 17, the simplified unidirectional loading protocol consisted of 2 cycles at a drift level equal to 0.25%, 3 cycles at 0.50%, 5 cycles at 0.75%, 1 cycle at 1.50%, and 2 cycles at 0.75%. The applied loading protocol also included 2 preliminary cycles at 0.1% to check all the instrumentation was working properly.

The cycles presented on the left side of Figure 17 have been applied in both the horizontal directions according to the orbital pattern presented on the right side of Figure 17, [33]. This orbital pattern has equal maximum amplitudes in both directions of testing and, as such, is likely to be more representative of the
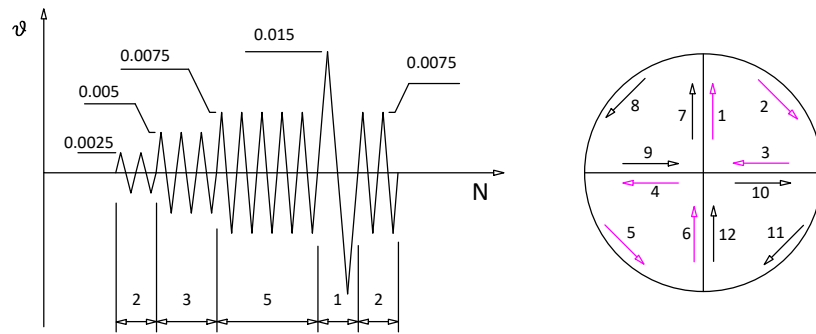
Figure 17: Interpreted Kaikoura unidirectional loading protocol and orbital pattern assumed.

effects of ordinary far-field ground motions (like the Kaikoura earthquake for a building located in Wellington) than near-field ones.

The final drift levels were specified to also be consistent with more standard loading protocols proposed in the literature. This consistency of drift levels made the results of the experimental test useful not only to replicate the effects of the Kaikoura earthquake but also to permit the comparison with experimental results obtained with more standard loading protocols.

Because the objective of the first loading protocol was to replicate the effects of the Kaikoura earthquake on a typical building located in Wellington, where several types of damage to precast concrete floors with hollow-core elements have been observed, this loading protocol was terminated with the last two important cycles (with amplitude assumed equal to 0.75%), according to the drift response presented in the plot at the top of Figure 15.

The Kaikoura loading protocol was followed by a standard progressive loading protocol that consisted of 2 cycles at 1.5%, two cycles at 2.0%, two cycles at 2.5%, two cycles at 3.0%. At the end of the Kaikoura loading protocol, in between the progressive loading protocol, the floor system was also subjected to a shear type of deformation pattern, termed a rhomboid loading protocol.
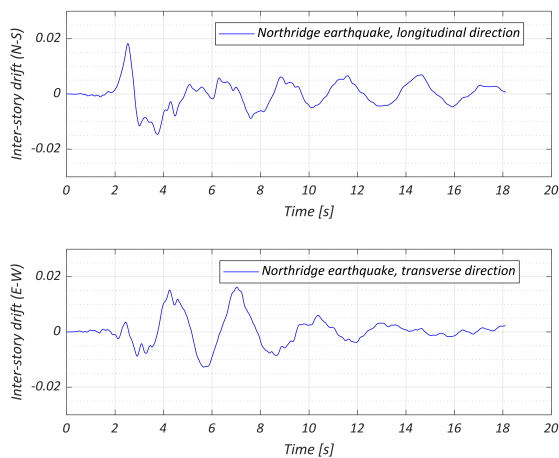

Figure 18: Inter-story drift time history at level 3, North-South (top) and South-East (bottom) directions for the Northridge ground motion.

Assessing the Likely Response to Northridge Near-fault Ground Motion

Ground Motion for Development of Phase II Loading Protocol of the ReCast Project

After completing phase I testing, it was recognised that the phase I loading protocol may have been favouring the resistance of the frame against negative moment failure. In particular, the ratio of the peak displacement demand demands in the two main framing directions for the testing phase I was equal to $a/b = 1$, and this had important consequences on the specific failure mechanism of the floor system activated, as will be explained later.

In phase II, it was decided to consider a loading protocol compatible with a ground motion with a near-field velocity pulse, that pushes the frame more in one main direction than the other. This type of ground motion is quite feasible for Wellington considering the local seismicity (faulting). The 6.7 magnitude 1994 Northridge earthquake accelerograms recorded at the Rinaldi receiving station are an example of a near-fault ground motion recording and hence, were used to to develop a loading protocol for phase II testing. This was done by re-running time history analyses of the model described previously with the as-recorded N-S component of ground motion applied in the longitudinal frame direction (see Figure 19).

Analysis Results for the Northridge Earthquake Motion

The variation in story drift demands at the 3rd storey of the building predicted from the NLTH analyses are shown in Figure 18. Note that there is a significant drift demand in the N-S direction before the drift demand in the E-W direction has developed significantly.

Interpreted Loading Protocol for Phase II of the ReCast Floors Experimental Testing

The loading protocol adopted to replicate the effects of near-fault earthquakes was based on both the unidirectional responses to the Northridge earthquake record registered in the longitudinal and transverse direction, reported in Figure 18. The drift history in the longitudinal direction, reported in the plot at the top of Figure 18, presents a peak in the drift response approximately equal to 2% between 2 and 3 seconds, in the positive direction, and a peak drift in the negative direction, approximately equal to 1%. Between two and three seconds, the same response in the transverse direction, plot at the bottom of Figure 18, was assumed negligible. A single unidirectional pulse with drift levels equal to 2% in the positive direction and 1% in the negative direction was assumed to represent the response during the first
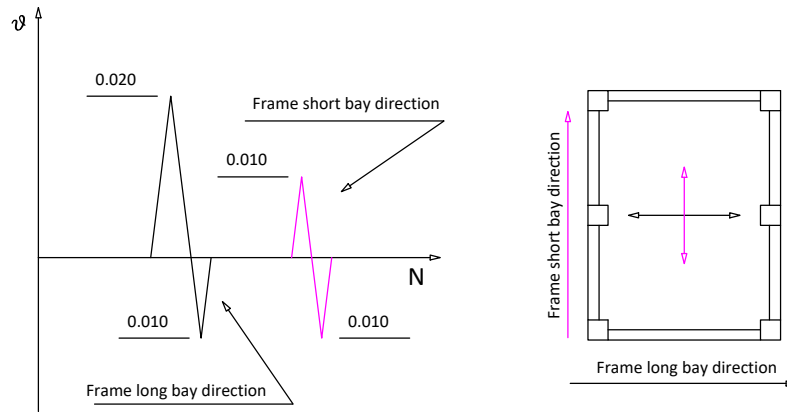
Figure 19: Interpreted Northridge unidirectional loading protocol and orbital pattern assumed.

loading phase of the near-filed compatible loading protocol.

As shown in the plot at the bottom of Figure 18, the same drift history in the transverse direction, for the same earthquake record, presents two peaks in the positive direction and one in the negative direction at a drift level approximately equal to 1.5% between 4 and 6 seconds. Between four and six seconds, the response in the longitudinal direction, plotted at the top of Figure 18, can be assumed negligible. This phase of the response was approximated and simplified in the second testing phase characterized by a complete unidirectional cycle with drift levels equal to 1%, in both the positive and negative directions.

In summary, the loading protocol adopted to replicate the typical effects of near fault earthquake records, as shown in Figure 19, consisted of two subsequent and independent unidirectional pulses. The first, unsymmetrical, with drifts equal to 2% in the positive direction and -1% in the opposite direction was applied in the frame longitudinal direction only.The second, symmetrical, with maximum drift level equal to $\pm 1\%$ in the transverse direction only.

Figure 20: Negative moment failure mechanism of hollow-core precast concrete units, from [43].

Failure Modes of Precast Concrete Floor Systems and Dependency of the Activated Failure Mechanism on the Loading Protocol Assumed.

Three are the major failure modes of precast concrete hollow core floors induced by seismic excitation highlighted in [13]: Loss of seating (LOS), [44], Positive Moment Failure (PMF), [7], and Negative Moment Failure (NMF), [43,45]. In the ReCast project, particular attention was paid to the negative moment failure at the connection between hollow-core elements and supporting beam (along the main direction of the hollow-core unit), Figure

20. This major failure mechanism was not observed during the first testing phase, which simulated the effects of the Kaikoura earthquake. An analysis of the registered experimental response has led to the conclusion that this was due to the softening of the torsional stiffness, and induced rotation of the supporting beam caused by the circular orbital pattern assumed. The beam rotation drastically reduced the negative moment demand on the hollow-core beam connection and the associated failure mechanism (the negative moment failure mechanism) was not activated. In the second test, conducted to replicate the effects of near-fault ground motions -that started with a big unidirectional pulse in the longitudinal directional without any preliminary crescendo drifts in any of the other testing directions- the negative-moment failure mechanism was observed as expected. This is a clear example of the dependency of the seismic performance obtained by experimental tests on the specific loading protocol assumed.

LOADING PROTOCOLS TO HELP INFORM RISK ASSESSMENT

The results of experimental testing will typically be used to inform engineers of the capacity of a structure which can in turn be used as part of seismic risk assessments. The previous sections have highlighted the fact that the capacity, and therefore seismic performance, of a structure can be affected by the loading protocol adopted. In the ReCast project, for example, it was shown that different failure mechanisms are activated in floors using different loading protocols and in previous studies [15] it has also been shown that not only the demand but also the capacity can depend on the displacement history. As such, it is apparent that an ideal loading protocol should reflect the characteristics of the local seismic hazard and the structure tested should be representative of a large ensemble of possible geometric and mechanical characteristics.

One risk-oriented approach could be to define the loading protocol to reflect the characteristics of the seismic hazard at a specific location. For example, the number and relative amplitude of, say, drift cycles to be imposed could be evaluated via statistical analysis of analytical results obtained from an ensemble of earthquake ground motions representative of the expected earthquake hazard and considering representative structural systems. The multidirectional conditional spectrum [49], which defines the demands at different orientations with respect to that of the maximum response, could be of use for such an endeavor. Krawinkler [24] points out, however, that there is no unique and "best" loading history, because no two earthquakes are alike and because the specimen may be part of many different structural configurations.
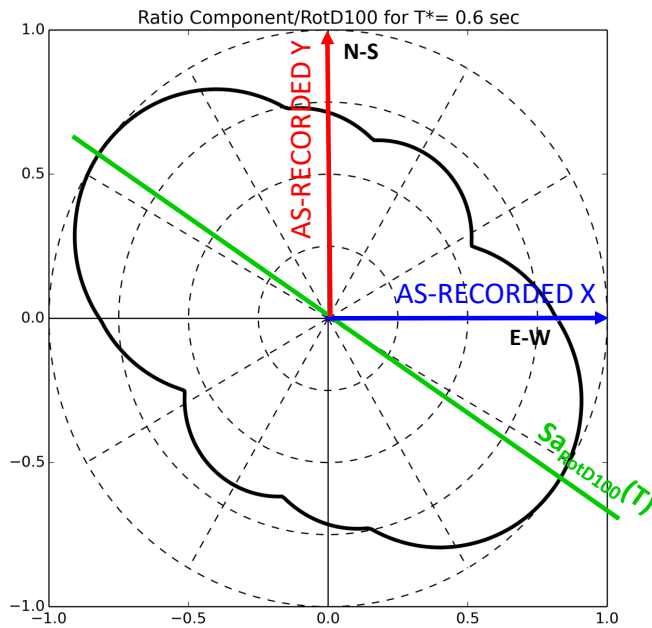
Figure 21: Ratio of spectral acceleration demands at different orientations with respect to the demand at the direction of maximum response, $Sa_{RotD100}$, for an oscillator period of 0.6 seconds subject to an aftershock of the 1995 Kozani earthquake (Greece) (record from the RESORCE database).
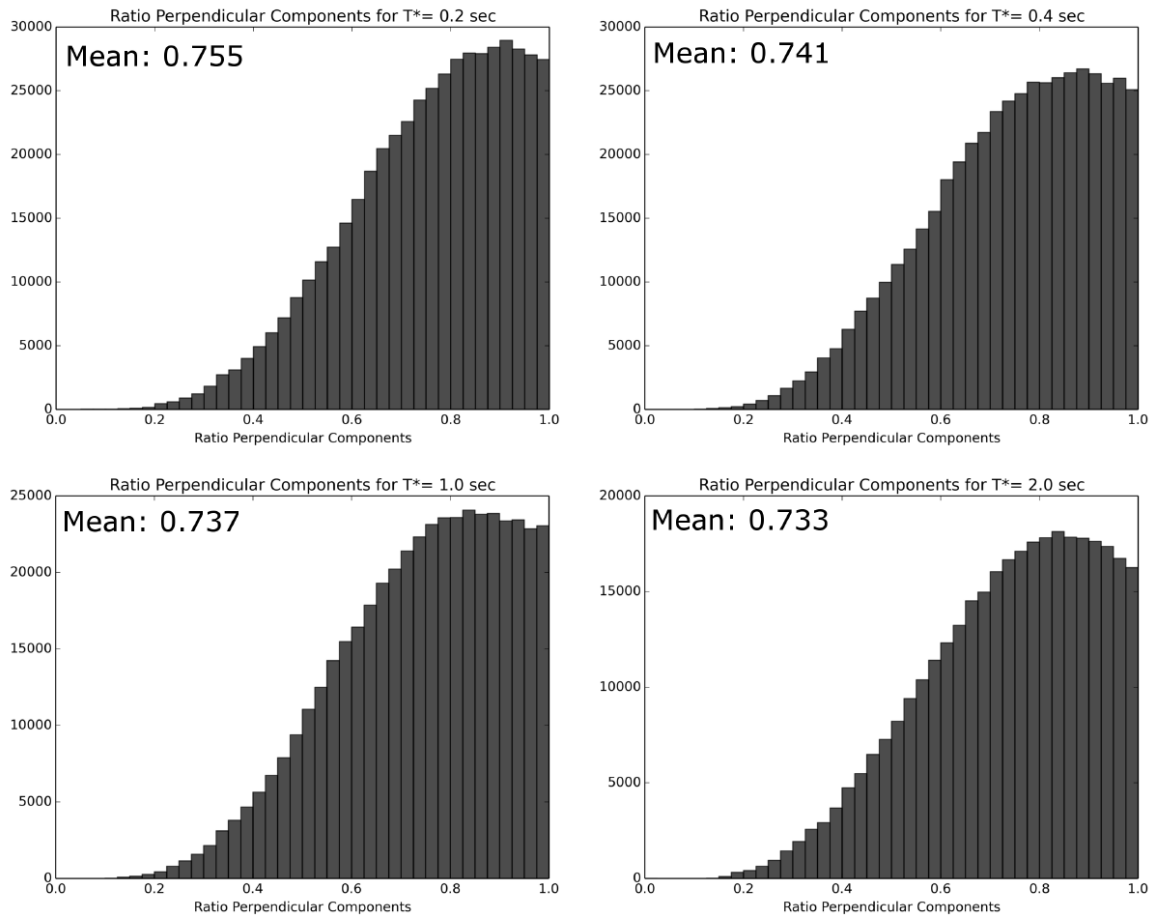

Figure 22: Ratio of smallest to largest spectral demands of perpendicular horizontal components of all records of the RESORCE database [46] classified as non-pulse-like rotated around all non-redundant orientations. From [47].
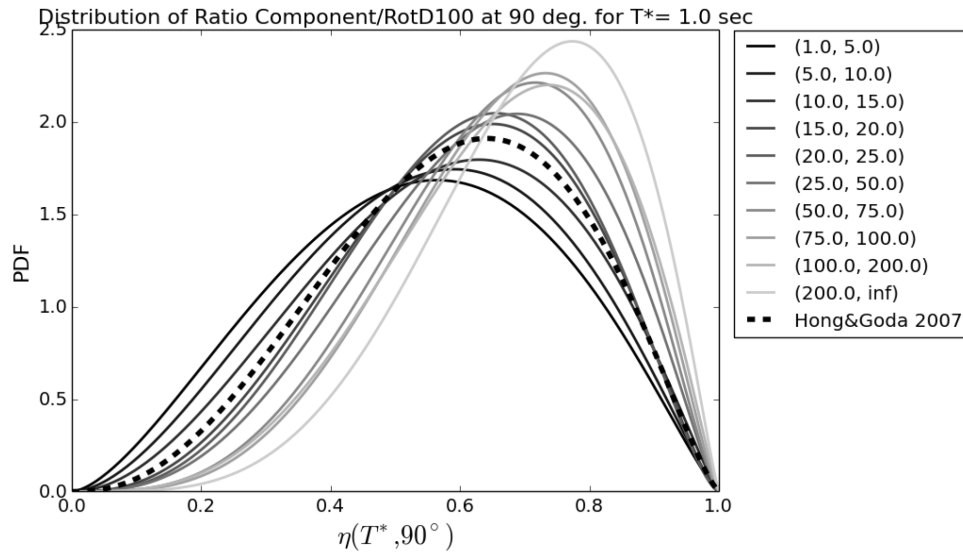
Figure 23: Comparison between beta distributions of the a/b ratio of spectral demands at T=1.0 s at the direction of maximum response and its perpendicular fitted with records from RESORCE [46] binned according to their epicentral distances (km) and that of Hong & Goda [48]. From [47].

Furthermore, a fully risk-oriented approach would create significant complexity in practice where experimental testing usually seeks to quantify the capacity of structural specimens for use at a range of locations, in buildings with a range of periods and site-soil conditions.

A more pragmatic risk-oriented approach to the definition of bi-directional loading protocols that is proposed herein (and that has, to a certain extent, already been adopted in the ReCast project) is to undertake testing considering at least two loading protocols that could be representative of the range of earthquake demands at a site. For example, one might expect that sites could be affected by near-fault earthquakes of moderate magnitude as well as long-distant large magnitude earthquakes with large shaking duration. If loading protocols are therefore specified to be representative of these two scenarios, two different mechanisms and capacity values may be obtained and subsequently, when risk-assessment is undertaken, these capacity values could be considered against the likelihood of different contributors to the hazard (i.e. considering the likelihood of near-field ground motions relative to the likelihood of large magnitude far-field motions). If additional experimental testing can be afforded using a protocol that is intermediate to the two extreme protocols then this will provide a third hazard-dependent capacity estimate and the uncertainty in the risk assessment could be reduced accordingly. To highlight the importance of the seismic hazard on the protocol definition, the next section examines the ratio of peak displacement demands in orthogonal directions as obtained from analysis using a large database of ground motions.

Ratio of Peak Displacement Demands for Bidirectional Loading Protocols

One of the main issues in the definition of suitable bidirectional loading protocols for bi-directional testing is the appropriate quantification and selection of the ratio a/b, shown in Figure 8, between the peak displacement demands in the two orthogonal testing directions, as highlighted by the experimental responses registered during the ReCast project. A bidirectional loading

protocol characterized by equal displacement demands in both the directions of testing (a/b=1) could be non-conservative for a specific failure mechanism with respect to a loading protocol with preponderant displacement demand in one direction. The question of the appropriate a/b ratio to adopt for testing therefore depends on the variability of ground motion demands at different orientations and how this variability may relate to hazard parameters that can be evaluated and quantified.

A first matter that needs to be agreed upon is the definition of the peak demand, b, to be used in the calculation of the ratio a/b. Works into directionality by [48] and [50] have shown the convenience of calculating the ratio of demands at different orientations in terms of ratios with respect to the peak spectral acceleration demand considering all possible angles of incidence, $S_{aRotD100}$. The peak demand at other angles can then be plotted as a ratio of $S_{aRotD100}$ and the values are all contained within a circle of unitary radius as shown in Figure 21 for an aftershock of the 1995 Kozani earthquake in Greece. In this example, the a/b value in the directions of maximum response and its perpendicular is approximately 0.7.

In order to highlight the dependency of the ratio a/b on a series of relevant seismological parameters, the 5532 records of the RESORCE database [46] available in 2015 (at the time this work initiated) were classified into groups according to their moment magnitude, epicentral distance, rupture mechanism, soil class, and pulse-like behaviour, as described in [47]. Figure 23 reports the a/b values obtained for these records at the direction of maximum response and its perpendicular as a function of the epicentral distance. The results indicate that smaller ratios of a/b tend to occur at closer epicentral distances. This suggests the existence of higher degrees of polarisation in the near field than in the far field, an observation that is consistent not only with the physical properties of wave propagation but also previous studies [51]. Similarly, Nievas [47] also observed stronger polarisation (smaller a/b at the direction of maximum response and its perpendicular) for smaller magnitudes (at short periods) and stronger spectral demands. However, this observation may be an

artefact of the correlation between shorter epicentral distances and magnitude/demand in the records used and requires further investigation.

Adopting a/b ratios for testing based solely on statistics obtained for the direction of maximum response and its perpendicular would imply that the principal directions of the analysed building or component are aligned with these two directions of ground motion, which cannot usually be known a-priori (apart from trends observed in the near-field, e.g. [51], if the location and orientation of the building are defined). Calculating a/b ratios at all non-redundant orientations leads to plots like those shown in Figure 22 for a range of SDOF oscillator periods. Note that there is a significant distribution in the a/b values even though mean values appear to be around 0.75. Both Figures 22 and 23 show that identifying a single value of a/b that is representative of the hazard would not capture the effects of uncertainty in demands on the capacity, as very large ranges are all possible. The choice of the a/b ratio for bidirectional testing should therefore consider means of characterising a range of demands that can be used to reflect the regional seismicity (and in particular whether deaggregation of the hazard suggests that both near field and far field motions are dominant) and uncertainty in the orientation of buildings relative to the peak shaking demand directions.

The results of this section support the earlier recommendation that bi-directional experimental testing be undertaken using at least two loading protocols that could be representative of the range of earthquake demands at a site. The results shown in Figures 22 and 23 suggest that suitable lower- and upper-bound values to a/b could be 0.30 and 0.90 respectively and yet settling on precise values is difficult and somewhat arbitrary. Sound judgement is fundamental as well to anticipate the possible effects that changes in the loading protocol could induce on the specific failure mechanisms investigated, so as to select ratios that are relevant to the context. The key finding is, nevertheless, that at least two widely-spaced values of a/b be adopted in testing such that the resulting mechanisms and associated capacity values can be considered against the likely hazard at a site, thereby reducing the uncertainty in subsequent risk-assessments.

SUMMARY AND CONCLUSIONS

This paper presented an overview of some of the principal loading protocols that have been implemented in quasi-static experimental tests to evaluate the seismic performance of structural system via quasi-static tests, for both unidirectional and bidirectional loading protocols.

The work has been undertaken as part of efforts to better understand the seismic performance of retrofitted precast concrete floor systems, as pat of the ReCast project. To establish a loading protocol for testing of a large precast concrete floor diaphragm and frame system at the University of Canterbury a hypothetical 13 story RC frame building was first designed according to NZ standards in use in the late 1980's. Subsequently, a 3D numerical model of the building was developed and subjected to non-linear time history analyses using ground motions recorded in Wellington during the 2016 Kaikoura earthquake. By examining the drift response at the 3rd story of the building in the North-South and East-West directions, the number and magnitude of cycles to be used in the ReCast testing was set. Subsequent loading histories applied to the frame are also described.

The last part of the paper identifies a number of limitations with current loading protocols for bidirectional testing. The results obtained during the two phases of the ReCast project described in this paper, highlighted the importance of considering an appropriate ratio between the peak displacements in the main directions of testing, as this can induce different failure mechanisms and seismic performances. Furthermore, by examining the bi-directional demands imposed by 5532 earthquake records, it is shown that the ratio (a/b) of peak displacement demands in perpendicular directions can vary greatly (with a mean value of a/b equal to 0.75). Furthermore, the results showed that smaller a/b values are expected at closer epicentral distances. In light of the results obtained, a risk-oriented approach to the definition of bi-directional loading protocols was proposed that consists of testing with at least two loading protocols representative of the range of expected earthquake demands at a site, such as ordinary far-field and near-fault ground motions. By testing with at least two widely-spaced values of a/b the resulting mechanisms and associated capacity values can be considered against the likely hazard at a site, in subsequent seismic assessment calculations, thereby improving the accuracy of the risk-assessment process.

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