SEISMIC RESPONSE OF BNZ BUILDING IN WELLINGTON FOLLOWING THE 2016 KAIKOURA EARTHQUAKE

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ABSTRACT

This paper presents an engineering account of the seismic response of the BNZ building during the 2016 Kaikōura Earthquake. The building is part of the Centreport complex located on Waterloo Quay in Wellington. The building, constructed c2006, is a six storey ductile concrete moment frame structure supported on pile foundations. Within the superstructure footprint there are three independent structures connected together by steel link bridges.
Unlike many other buildings in Wellington region that suffered damage, the BNZ building was well equipped with resources that enabled live tracking of its response using accelerometer instruments and CCTV cameras. Additionally, information collected after the event through extensive damage mapping and surveys, material testing and analytical investigation provided useful insight into the building behaviour and subsequent damage.
Some of the main observations include; ratcheting behaviour in the plastic hinge zones resulting from unequal cyclic demands on the beams loaded with large gravity loads, sagging of floor slabs near column lines, global torsional response at the level of foundation due to interaction between unimproved and improved sub-soil regions and interaction of the three independent structures during cyclic sway.
This paper also present the results of analytical studies that were undertaken to provide further insight into the damage that occurred in the building and highlight the causes.

THE BNZ BUILDING AT HARBOUR QUAYS

The BNZ Harbour Quays building is a modern six-storey commercial office building constructed c2006 and located at 60 Waterloo Quay on the Wellington waterfront. The structure comprises three reinforced concrete framed piers separated by two full-height atria spaces. Piers 1 and 2, and piers 2 and 3 are interconnected via steel 'link bridges' at both ends of each level. The link bridges are steel beams with an in-situ topping slab and are pinned at each end to allow rotation in a horizontal plane at the supports at each end of the span. The link bridges were originally designed to tie the three piers during longitudinal (North-South) seismic drift, however, in order to maintain the geometry, the three piers also displace in-phase in the transverse direction albeit with some time lag.

Figure 1 shows the west elevation of the building as visible from the Waterloo Quay frontage.

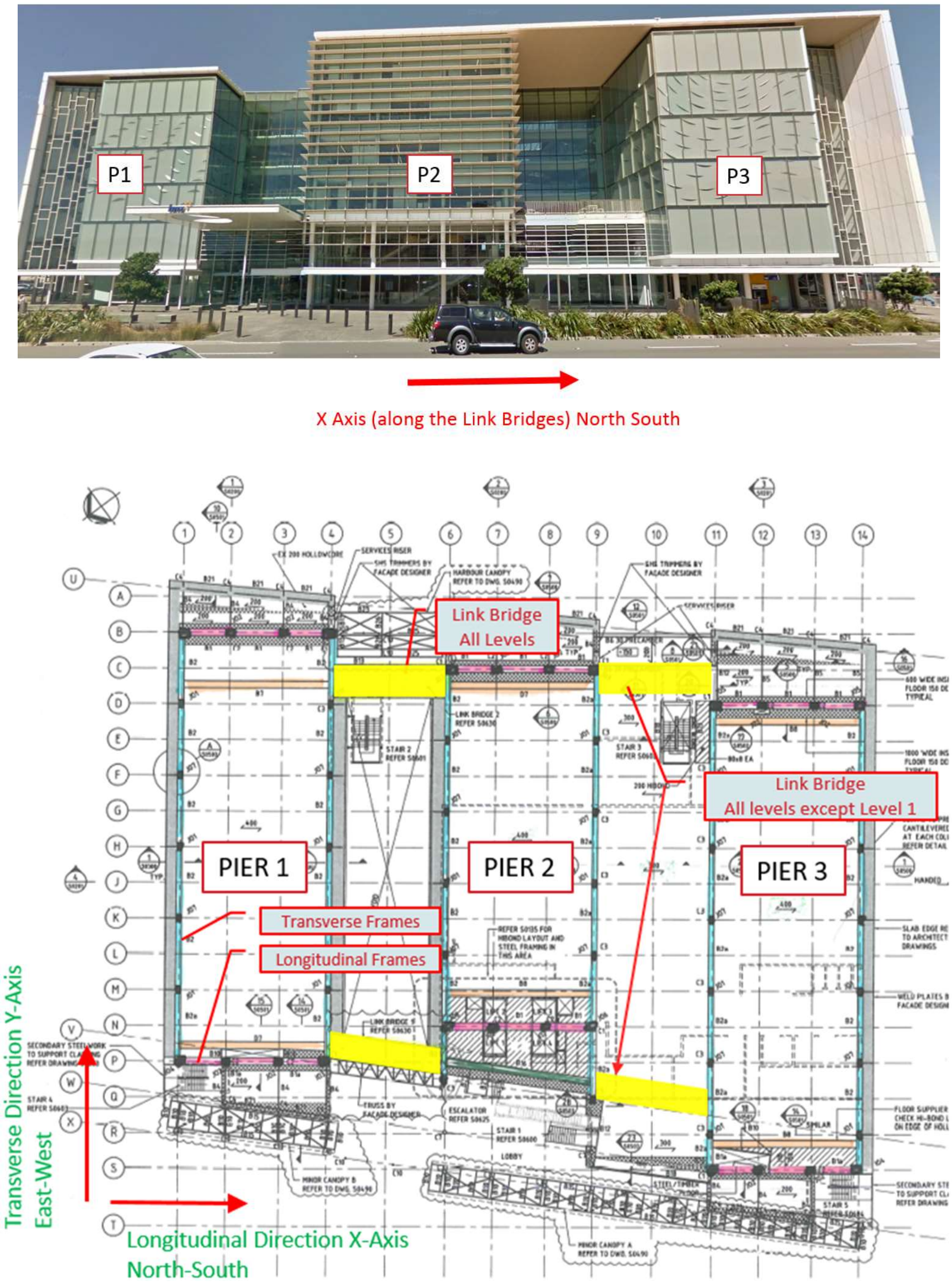


Figure 1. West Elevation (Waterloo Quay Side) and Plan at Level 2

Reinforced concrete moment resisting frames (MRFs) provide the main lateral resisting system with a pair of perimeter MRF in the longitudinal and transverse directions of each of the three piers. The frames consist of precast beams and in-situ column and joint elements designed and detailed for high ductility with μ=6.0 in both directions. Figure 2 shows the typical beam reinforcement details for the longitudinal moment resisting frames. The spacing between longitudinal frame columns varies between 5.4 to 6.0 meters whereas in the transverse frames the columns are 8.1 meters apart. The typical floor system is 400 mm thick hollowcore slab spanning 16.8 meters between the transverse frames with a topping slab thickness of 70mm.

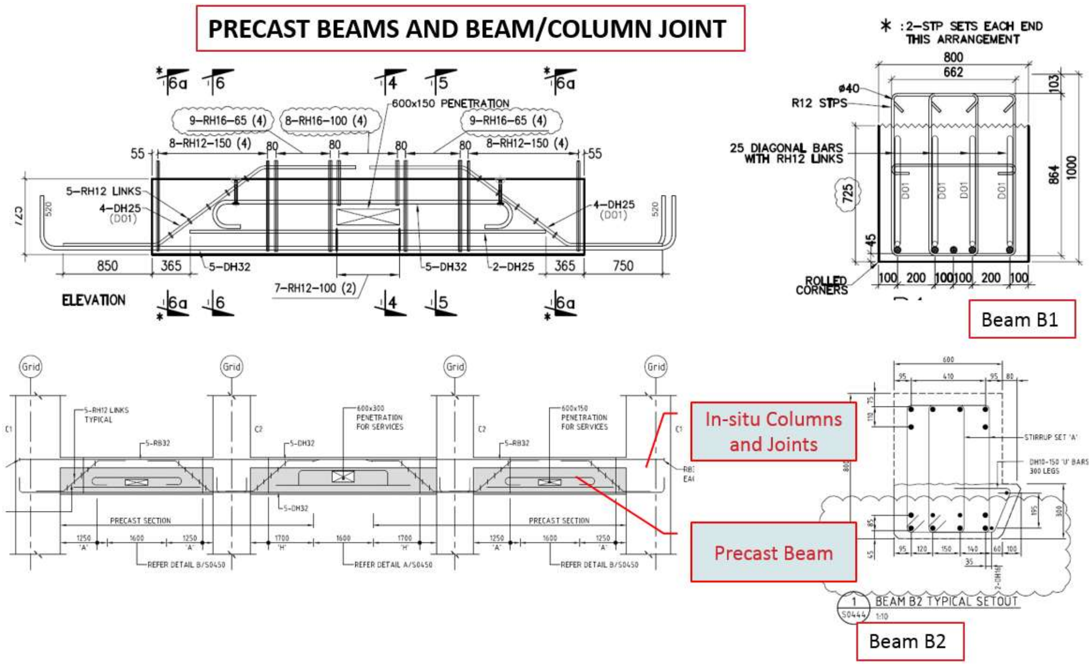


Figure 2. Typical Precast Beams and emulated beam-column joint details

GROUND CONDITIONS AND FOUNDATIONS

The BNZ building is founded on reclaimed land which is approximately flat with ground surface level approximately 2 m above mean sea level. The depth of the reclamation fill varies from 2 m to 5 - 8 m and comprises 1 - 2 m thickness of gravel sand with silt and 4 - 6 m thickness of interbedded sand and silt. The site subsoil classification in terms of NZS 1170.5 is Class D. Towards the harbour frontage ground improvements were carried out comprising 1000 mm diameter stone columns arranged in a 2.25 m triangular grid on plan over an 18m wide strip behind the original 1908 seawall. The overall foundation and ground improvement arrangement is shown in Figure 3.

The building foundations comprise 12 - 15 m deep bored concrete piles with varying diameter between 800 mm to 1500 mm, together with some concrete-filled steel tube driven piles. The piles are tied together at ground level by reinforced concrete ground beams. The ground floor slab is typically supported by the ground beams. Figure 4 shows the pile layout plan.

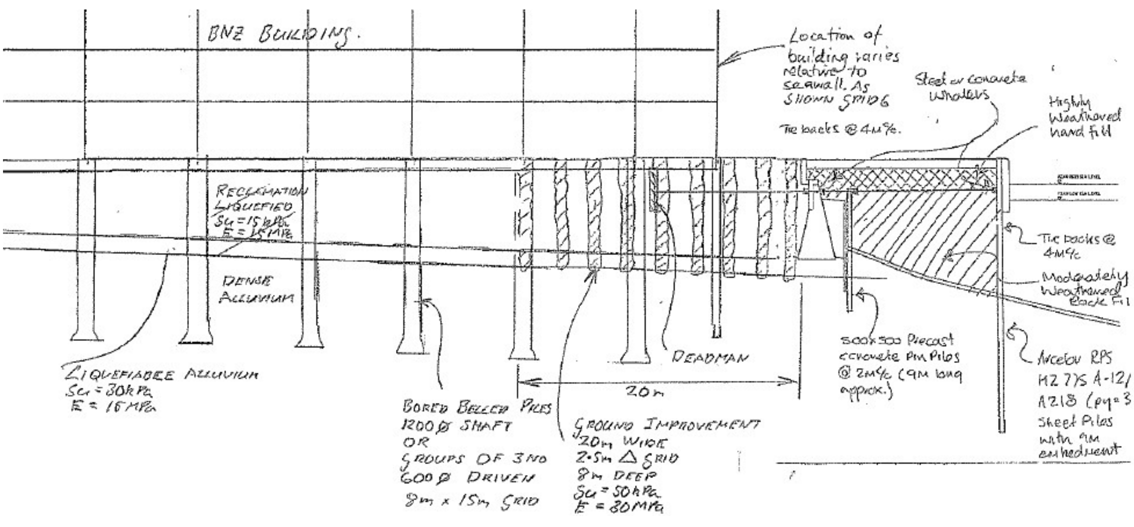


Figure 3. Typical cross section through the site (extract from 2006 T&T report[5])

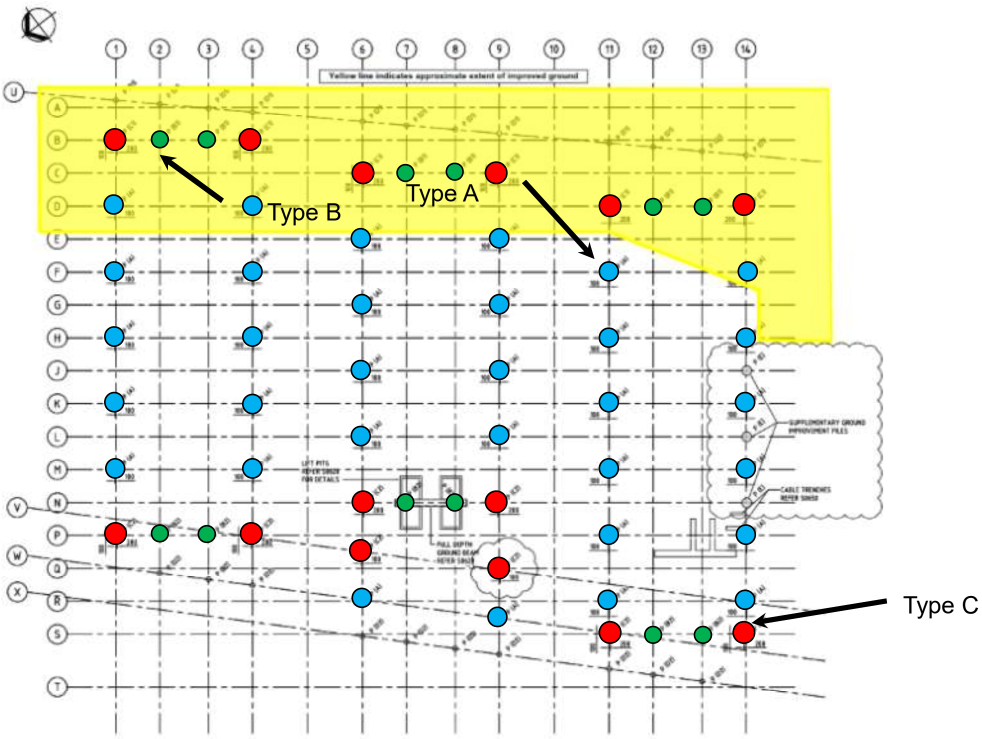
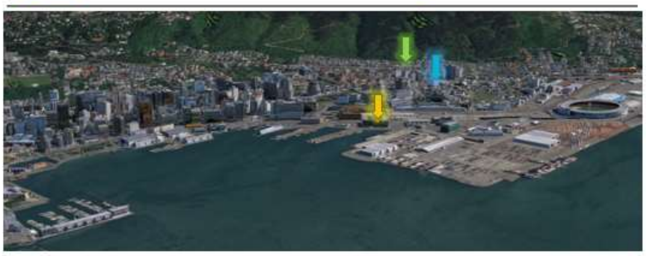


Figure 4: Pile layout: Blue = Pile Type A (1.2 m dia); Green = Pile Type B (0.8 m dia); Red = Pile Type C (1.5 m dia); Yellow = Ground Improvement Zone (Structural Drawings)

EFFECTS OF KAIKOURA EARTHQUAKE ON WELLINGTON WATERFRONT STRUCTURES

The BNZ Building suffered structural damage during the November 2016 Kaikoura earthquake. The ground motion intensities experienced by the structures founded on the reclamation fill near the waterfront region were approximately equivalent to or larger than the ultimate design demands corresponding to a 1-in-500-year event (ULS Elastic Spectra) in the period range between 1 to 2 seconds [6]. Figure 5 presents the comparison of spectral ordinates from elastic spectra from the New Zealand standards for Wellington CBD [4] to the ground motion records from Kaikōura earthquakes on the representative soils classes of A/B, C and D. The comparison of spectral ordinates show clear trend of ground motion amplification near the waterfront land classified as soil class ‘D’ in contrast to soil types ‘A & B’ and ‘C’ when compared with the spectra from Kaikōura earthquake.

Station	Surface Geology	V_{s_30} [ms^{-1}]	Soil Classification	Duration [D_{595}]
POTS	Rock Site	400-2000	A & B	
WEMS	Shallow Deposit (~50m depth)	250-750	C	Kaikoura ~ 22 sec
CPLB	Reclamation Fill	125-300	D	



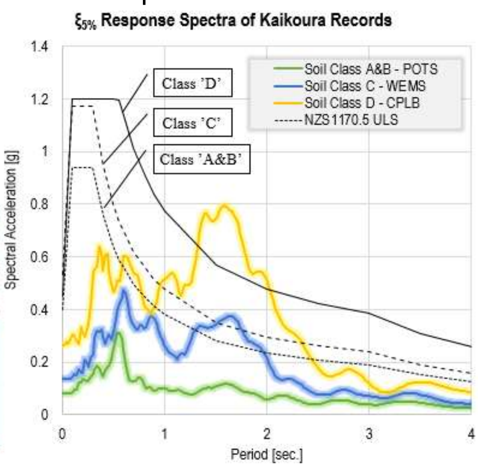


Figure 5. Comparison of spectra at different soil classes between Kaikōura and NZS 1170.5

The effects of the Kaikoura earthquake have been investigated in detail and there are some excellent publications on the topic [7] [8] [9]. The following is an extract from Bradley et al 2017[7].

"Despite being approximately 60km from the northern extent of the causative earthquake rupture, the ground motions in Wellington exhibited long period (specifically T = 1−3s) ground motion amplitudes that were similar to, and in some locations exceeded, the current 500 year return period design ground motion levels. Several ground motion observations on rock provide significant constraint to understand the role of surficial site effects in the recorded ground motions. The largest long period ground motions were observed in the Thorndon and Te Aro basins in Wellington City, inferred as a result of 1D impedance contrasts and also basin-edge-generated waves. Observed site amplifications, based on response spectral ratios with reference rock sites, are seen to significantly exceed the site class factors in NZS1170.5:2004 for site class C, D, and E sites at approximately T=0.3-3.0s. The 5-95% Significant Duration, D_{s595}, of ground motions was on the order of 30 seconds, consistent with empirical models for this earthquake magnitude and source-to-site distance. Such durations are slightly longer than the corresponding D_{s595} = 10s and 25s in central Christchurch during the 22 February 2011 Mw6.2 and 4 September 2010 Mw7.1 earthquakes, but significantly shorter than what might be expected for large subduction zone earthquakes that pose a hazard to the region".

OBSERVED & RECORDED RESPONSE OF BNZ BUILDING DURING KAIKOURA EARTHQUAKE

Following the November 2016 Kaikōura Earthquake, the BNZ building was subject to extensive investigations by several parties to determine the extent of damage. The information collected and presented herein is through the following sources.

Table 1: List of investigations and source of information

Investigation	Collected Data
Non-intrusive visual inspection	Crack mapping to beams, columns and floor units. [conducted by Beca and independently verified by WSP Opus on a sample of data]
Accelerometer Instrumentation Records	Recorded acceleration time histories at the base and floor levels. [Data is courtesy of GNS Science and University of Canterbury]
Material Testing	Tensile tests, Leeb and Vickers Hardness testing on reinforcement of the topping slab and the bottom reinforcement of beams to determine loss of strain capacity. [Conducted by Holmes Group and University of Canterbury]
Verticality Survey and Laser Point Cloud Survey	Column verticality, global settlement, floor level sagging, point cloud survey. [Conducted by Beca and WSP Opus]

Accelerometer Instrumentation Records

The BNZ building had an array of accelerometers at different locations as part of the GeoNet Building Instrumentation Program [10]. These were to monitor and infer local and global response parameters including floor and base accelerations, story displacements and modal properties through acceleration time histories recorded during earthquakes. The locations of these instruments are shown on a schematic plan and elevation in Figure 6 below (extracted from reference [10]). Note that the piers are noted as bays in [10] however they have the same numbering as in this publication. The most extensive array of instruments is located in the least damaged pier, Pier 3.

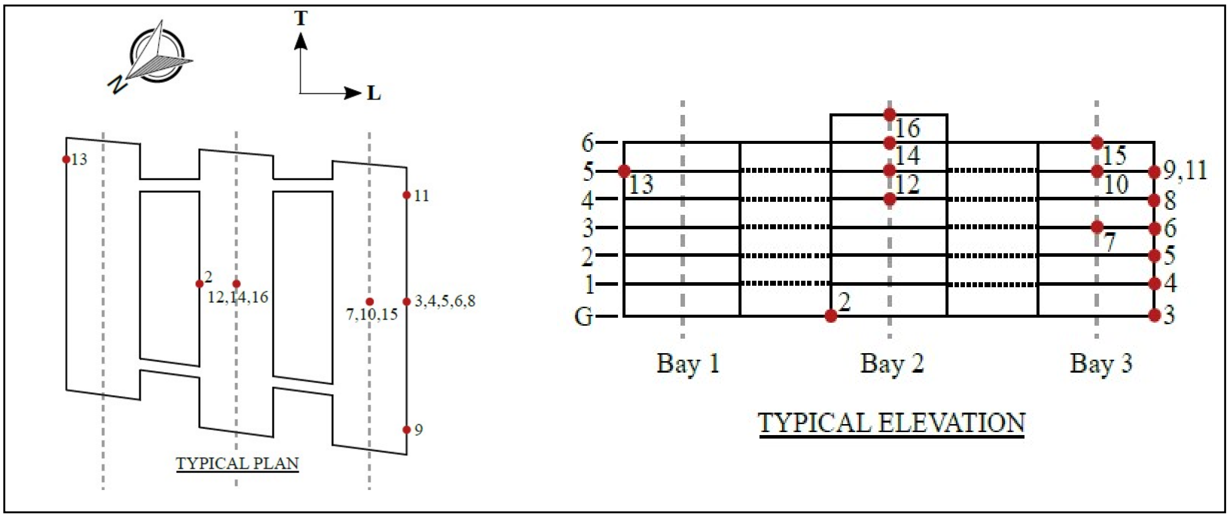


Figure 6. Accelerometer locations (from [10])

On Pier 3 the peak floor acceleration was recorded as 1.02g in the east-west direction and peak building displacement of 227 mm longitudinally and 243mm transversely were recorded at the top (level 5) [10]. Maximum peak interstorey drifts of 1.73% longitudinally and 1.71% transversely were also recorded and reported for Pier 3 [10].

In addition to the recorded instruments, the security cameras placed around the port also recorded CCTV footage of the Centreport buildings undergoing deformations during the seismic event.

Crack Mapping

Crack Mapping of Beams

Detailed damage observations, mainly the beam crack mapping have been recorded by Beca. For the primary frames, the lower portion of the plastic hinge regions of most of the beams have been inspected and photographed which depicts a range of observations, including the absence or presence of spalling and visible reinforcing and the maximum crack width at each beam end. Figure 7 shows the example of cracking in beams and columns photographed after the Kaikoura earthquake. Figure 8 presents colour-graded damage elevations of the primary frames. The colour grading is based on the residual crack mapping.

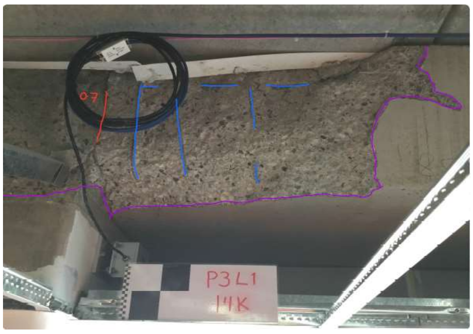
Crack Mapping of Hollowcore floors

A significant proportion of the precast hollowcore floor units have sustained damage due to deflection incompatibility between the units and the frames due to number of mechanisms including building drift, frame elongation, beam rotation, cracking of topping and straining of diaphragm steel around the perimeter. The key conclusions drawn from the observations were;

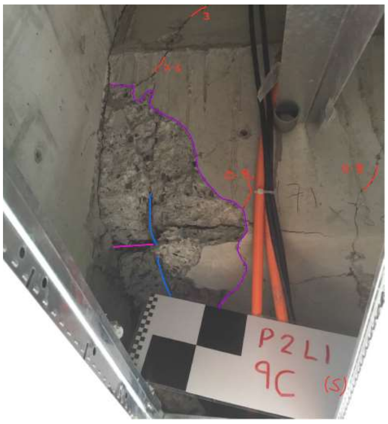
- Reduction in strain capacity of reinforcement in the floors; high mesh wire strains, longitudinal and splitting cracks in the units.
- High strains in the column tie bars

- the integrity of the gravity load system over a significant proportion of the building is compromised due to transverse cracking in the hollowcore units
- Portions of the topping slabs may have delaminated from the precast units beneath.


Pier 2 L1 Grids 6 and V beam



Pier 3, Grid 14/K Level 2 beam



Pier 2, Grid C/9 Level 2 beam

Figure 7. Cracking and spalling in the concrete beams

Post Seismic Building Survey

The surveys included an external feature level survey, internal horizontal and vertical assessment, column base level comparison, slab level surveying, construction set out benchmark comparison, gridline spacing checks and atrium column verticality survey.

- Global settlement range of 51mm to 66mm was evaluated indicating a differential settlement range of 15mm.
- The total level change to column bases between the time of construction and following the Kaikoura earthquake were 43mm for Pier 1, 49mm for Pier 2 and 45mm for Pier 3.
- Beams were surveyed at the column supports and at midspan along grids 4, 6, 9 and 11 at Level 2. Relative to the average level at the supports for each beam, the midspan sag measurements range from 8mm to 29.5mm.
- Significant sagging was also measured over the floor slabs. In particular, a noticeable slope between grids C and D in Piers 1 and 2, where floor level differences of between 40 - 70mm occurred over approximately 4m with about 1.0 - 1.75% crossfall.
- The average frame elongations per bay in transverse frames were 6.0mm, 10.8mm and 2.8mm in piers P1, P2 and P3 respectively. The transverse frames in pier P2 have undergone up to 100mm plan extension.

- Atrium Column verticality was surveyed and average residual drift calculated between the column bases and top were 0.16%, 0.21% and 0.08% in piers P1, P2 and P3 respectively.

Beam maximum crack widths

- 0.1mm ≤ largest crack < 0.5mm
- 0.5mm ≤ largest crack < 1.0mm
- 1.0mm ≤ largest crack < 2.0mm
- largest crack ≥ 2.0mm
- No information available

Steel Testing Summary:

Leeb: In-situ testing of beam bottom bars by Holmes Solutions.

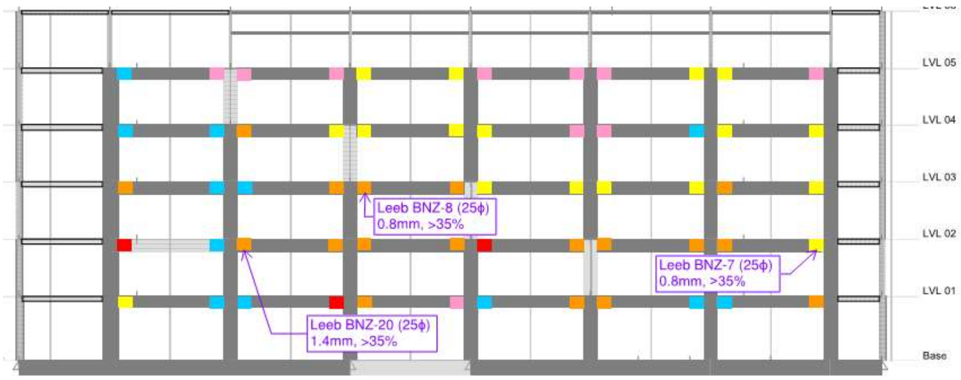
UTM: Laboratory testing of reinforcing using a Universal Test Machine and video extensometer.

Vickers & Tensile: Laboratory testing of mesh and saddle bars in floor diaphragm by the University of Canterbury.

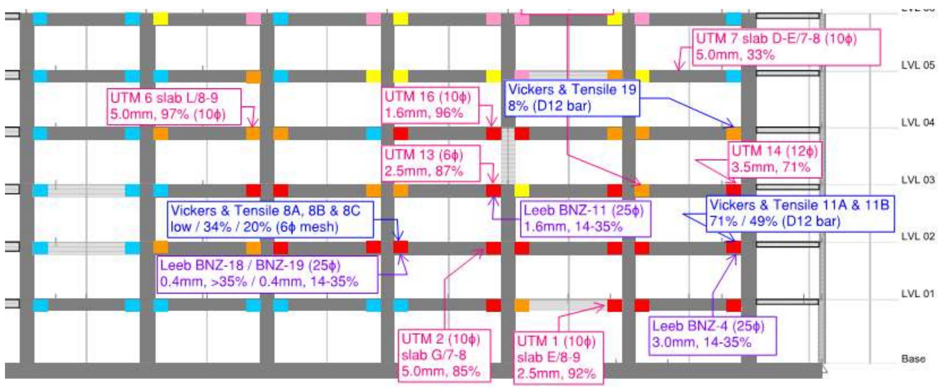
Holmes Solutions Leeb Test No. (bar dia.)
max crack width, predicted lost strain capacity

Holmes Solutions Tensile Test No. (bar dia.)
max crack width, Loss of strain capacity

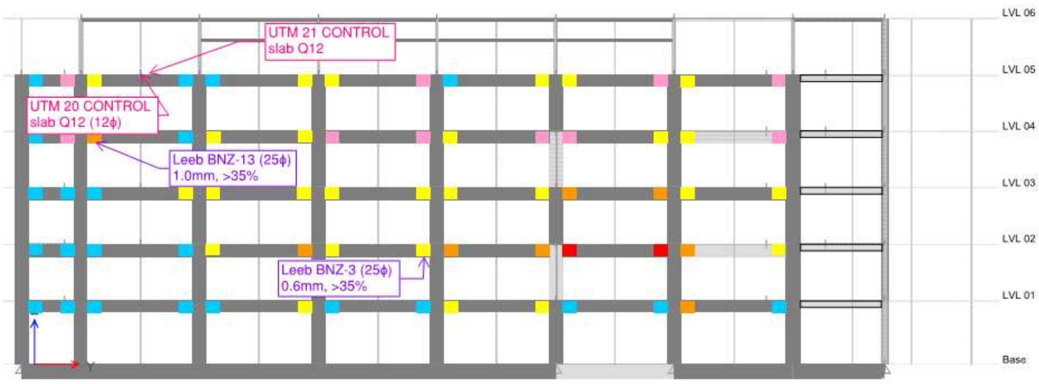
UC Vickers & Tensile Test No.
Loss of plastic strain capacity (bar dia.)



Pier 1 – Grid 4



Pier 2 – Grid 9



Pier 3 – Grid 14

Figure 8. Transverse frame beams (worst damaged) maximum residual crackwidth and reinforcement testing results

Façade Damage
A limited visual survey of the building façade systems found misalignment and gasket displacement has occurred to the storefront and stair tower glazing. Transitions and joints between the different enclosure systems have been damaged, including soffits and seismic expansion joints. The atrium glazing was out of alignment and it is likely to require removal in order to be repaired. Severe damage has occurred to the unitized curtain wall system in various locations around the façade, with misalignment of mullions and panels, damage to joinery and unit splice sleeves, cracking of metal rainscreen panels and reductions in movement tolerances due to element residual displacements.

Pile and Foundation Damage
Site survey and observations since the 2016 Kaikōura earthquake have concluded:

- Ground surface surveying indicated maximum vertical settlement of 30 mm and lateral movement of 20 mm at the BNZ site (Cardno, 2017). There were no observations of severe lateral spreading or liquefaction.
- A post-earthquake geotechnical assessment by Tonkin & Taylor (letter dated 13 June 2017), which identified residual transient lateral deformations of the pile heads relative to the ground surface of 0 – 35 mm.
- Excavation of the top 1 meter of a central pile (Pier 2, grid J9) identified no cracking of the pile, however some cracks in the ground beam were observed. It should be noted that Pile Type A was pinned in the longitudinal direction and hence was subject to smaller bending moments and may not be appropriate to be used as an indicator of damage to all piles.

Despite the lack of observable damage, the BNZ building was subject to ground accelerations of up to 0.24 g (Cubrinovski et al., 2017) and cyclic ground displacements of up to 23 cm (GeoNet strong motion network, determined from double integration of the acceleration time series conducted by the University of Canterbury). The combined kinematic and inertial forces on the foundations from these loads may have been sufficient to cause damage to the piles.

Reinforcing Steel Tests
Three sets of reinforcing steel testing have been performed on floor slab and beam bars around the building. The University of Canterbury carried out Vickers hardness tests and tensile tests in their laboratory on sampled lengths of mesh reinforcement and saddle bars taken from floor slab locations adjacent to the transverse frames. Holmes Solutions carried out in-situ Leeb hardness tests on longitudinal bottom bars in the MRF beams, as well as laboratory tensile testing of slab bars and mesh using a universal test machine and video extensometer measurement. The test locations on elevation are shown in Figure 5.

<u>University of Canterbury Vickers Hardness and Tensile Testing</u>
The University of Canterbury report *Assessment of Damage to Rebar in the BNZ Harbour Quays Building* Vickers hardness testing was carried out on floor slab bar and mesh samples on levels 2, 3 and 4 of pier 2 and it concluded that there is evidence of an increase in hardness for each sample tested. The majority of the tests showed a loss of strain capacity between 20% and 73%.

<u>Holmes Solutions In-Situ Leeb Hardness Testing</u>
The Holmes Solutions Leeb hardness testing were conducted on in-situ samples of the beam bottom longitudinal reinforcement distributed around the building. Out of sixteen, eleven tests

(69%) returned a high damage result indicating strain capacity losses of 35% or greater. A further four tests (25%) returned medium damage results, indicating strain capacity losses between 14% and 35%, while one test indicated low damage with strain capacity loss less than or equal to 13%.

<u>Holmes Solutions Destructive Tensile Testing</u>

The Holmes Solutions Destructive Tensile testing is similar to the University of Canterbury testing, however the methodology and reporting of results differs. The Holmes Solutions method outputs stress-strain responses incrementally over a series of regions along the bar samples. From these outputs, the peak strain in the worst region of the bar can be compared to the peak strain in the ‘parent’ steel, which the methodology estimates to be sufficiently remote from the damaged regions to be considered undamaged. The difference between these peak strain results provides the likely maximum loss of strain capacity in the sample.

The tensile testing was carried out on reinforcing bar and mesh samples. Across the test results, the minimum loss of strain capacity, calculated as the percentage difference between the most damaged portion strain capacity and the undamaged strain capacity, was 33%. Three-quarters of the test results showed a loss of strain capacity of over 70%.

ANALYTICAL STUDIES

Purpose of Analytical Studies

The main objective of understanding the seismic response of BNZ building was to assess the damage to the building incurred during the earthquake. It was a unique project requiring answer to the question of ‘What was lost in the building’s structure as a result of the earthquake’. This required determining the extent, severity and type of each damaged in all elements in physical terms to examine the extent of reinstatement required. The accelerometer recording were mostly concentrated within Pier 3 and therefore complete interstory displacement response was only available for Pier 3. This pier also happened to suffer the least damage. The detailed testing information was only available for a small sample of the building due to practicality reasons.

Therefore analytical studies were conducted to understand the response of the building in relation to the observed damage and fill the information gaps where response or damage were unknown. The analytical studies were required to reduce the inconsistency between observable residual crack widths, and actual damage during the event which can be associated with multiple criteria including: low cycle fatigue, energy dissipation capacity, peak crack widths, extent of plastic deformation in the plastic hinge regions, and the reduction in stiffness. These criteria have been combined to provide an improved understanding of the full extent of permanent structural damage to the building.

Numerical Model

A 3D numerical model of the structure has been developed using ETABS 2016 v 16.20 to complete to analyse the structure using linear and nonlinear analysis procedures. The rotational stiffness of foundation at the base of columns is incorporated by modelling the ground beams at the base of the columns whereas foundation flexibility was incorporated by means of ‘compression-only’ Winkler-Springs. The probable material properties for concrete were taken from the NZSEE Guidelines (July 2017) [1] whereas reinforcement strengths were based on the available laboratory tests results.

Cracked section properties were taken in accordance with NZS 3101:2006 [3] and the rigid joint offsets were provided to the beams-column joints in accordance with ASCE 41-13 Cl 10.4.2.1 [2].
A target equivalent viscous damping ratio of 3.5% was used for the nonlinear direct integration time history analysis in accordance with the procedures outlined in ASCE 41-13 chapter 7[2]. Rayleigh damping was adopted consisting of mass and stiffness proportional damping with 80% of the mass participation within the target damping ratio.
Plastic Hinge lengths of beams and columns, in order to calculate reinforcement strains, were determined using empirical equations from Priestley et al. (2007)[11] which are outlined in NZSEE Guidelines[1]. Lumped plasticity hinges are provided, one at each end of the beams and at the base of columns [12]. Additionally, a sensitivity study is carried out on single bay frames using fibre hinges to assess the extent of plasticity compared with that of observed damage. The results shows concentration of plasticity within a short region (approximately 200-250mm) at the top end of beam due to large gravity load demands and ratcheting effects discussed further. The hysteresis response of the plastic hinges during the nonlinear time history analysis is captured using 'Pivot' hysteresis model [13][14].
The seismic demands used in various analysis methods are based on the ground acceleration time history of the Kaikoura Earthquake from the recording station located adjacent to the BNZ building. This is identified as Channel 01 and is considered to be an appropriate free field acceleration ground motion due to its proximity to the building.

Analysis Methodology

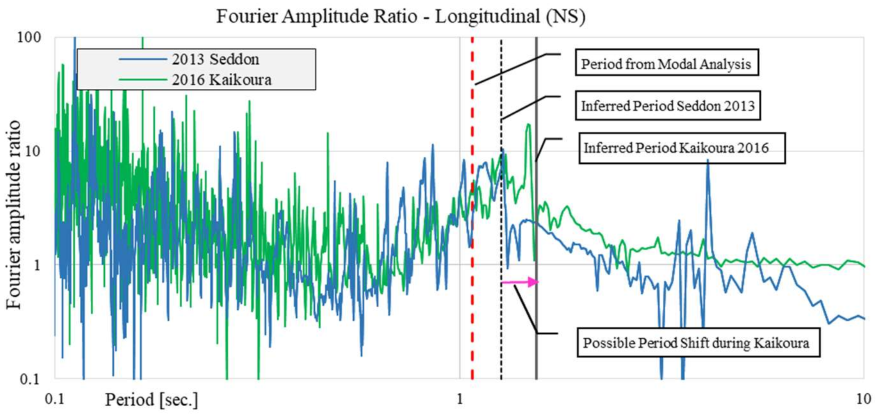
In order to understand the response of BNZ building the overall analysis methodology adopted was divided in to several stages each with its own specific objective. Initially the post-processed instrumentation records obtained from [10] were assessed in order to understand the key response parameters (including period of vibration, peak displacements and base shear demands, ductility demands etc.). A linear response spectrum analysis was performed using the elastic spectrum from the Kaikoura records to capture the global response parameters. As the damage observations indicated that the building has suffered damage beyond its yield capacity, a nonlinear pushover analysis was carried out to calculate the yield and post yield response of the building and to calibrate the model with recorded response. Following the initial analyses, a nonlinear time history analysis on the full 3D structure model was conducted to evaluate the post-elastic behaviour of the building under the Kaikoura Earthquake seismic ground motion and calibrated to the recorded response. The results were then used to determine the extent of plasticity and subsequent damage in the yielding members of the concrete MRFs utilizing several approaches available in the literature and guidelines.

GLOBAL RESPONSE AND MECHANISM

Fundamental Period of Vibration

The vibration period for piers 2 and 3 are inferred from the processed acceleration time history records using the ratio of their Fourier transformation between the base and at the top storey (Level 5) and the results are presented in Figure 9. Level 5 (19.75m above ground level) has been chosen because the recorded data is available for both piers at top store (Level 5) and at the base to enable the comparison and because the roof displacements gives a better approximation of the overall mode of vibration for a first mode dominant structure. For the purposes of identifying the modes, the records from the 2013 Seddon earthquake have been used to eliminate the possible effects of period lengthening due to progression of damage in several cycles during the November 2016 Kaikoura earthquake. The period of vibration is inferred from the common peak in these plots which occurs in the Fourier transformation ratio of both piers at the same time to be able to avoid the influence of local peaks and spikes. The unsmoothed Fourier spectra have been used for this analysis. The limitations in using such techniques for period inference are discussed in [10].

Using the Fourier transformation, the fundamental period of vibration for the longitudinal and transverse directions has been estimated to be approximately 1.25 seconds and 1.4 seconds respectively. These values generally compare well with the 3D modal analysis results indicating that the response is slightly stiffer in the longitudinal direction compared to the transverse direction, which also aligns with the original design and the displacement records.



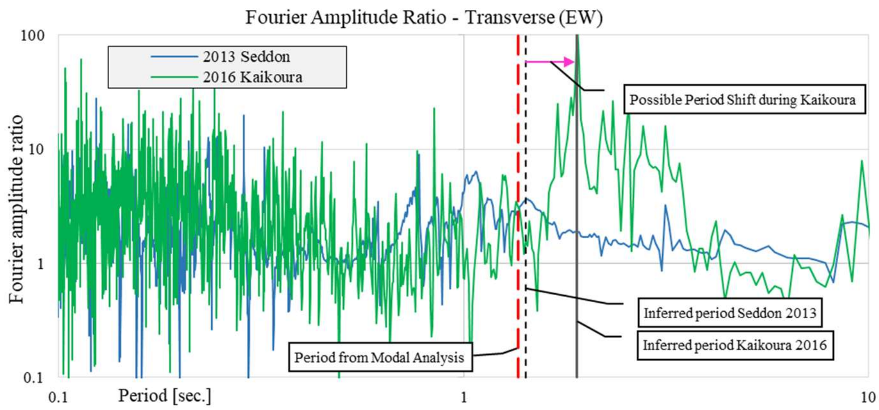


Figure 9. Fourier Transformation Ratio of Recorded Accelerometer Data

Global Seismic Demand on the Structure

The ground motions under the November 2016 Kaikoura Earthquake recorded near the building site had a peak ground acceleration of about 0.24g. The response spectral ordinates (ξ=5%) of the free-field acceleration time history records and the records at the base of Piers 2 and 3 are plotted and compared with the NZS 1170.5 ULS elastic and design spectra in Figure 10. The solid red and black lines represent the transverse (E-W) direction free field and base demand respectively, and the dotted lines represent the corresponding demand in the longitudinal (N-S) direction. The green vertical lines indicate the fundamental period of the building in the two direction as determined from the modal analysis. The blue solid and dashed lines represent the ULS elastic and design spectral ordinates as per NZS 1170.5 respectively.

Comparing the plots for both piers and for the base and free field spectra, it can be observed that the intensity of ground shaking has exceeded the ULS demand in the transverse direction for periods between 1.0 to 1.5 seconds. The elastic base shear demands were nearly eight times higher than the designed base shear (as per [4] by taking k_{μ} =6.0 and S_p = 0.7). This is due to amplification of the ground motion in the low frequency range associated with the deep soft-soil deposits on which the building is founded. The building-base spectral ordinates are amplified as compared to the free-field spectral ordinate primarily in the transverse direction for period range greater than 1.0 seconds. Refer to [10] for calculations of spectra and the discussions around possible reasons including soil-structure interaction resulting in differences between the free-field and building-base spectral ordinates.

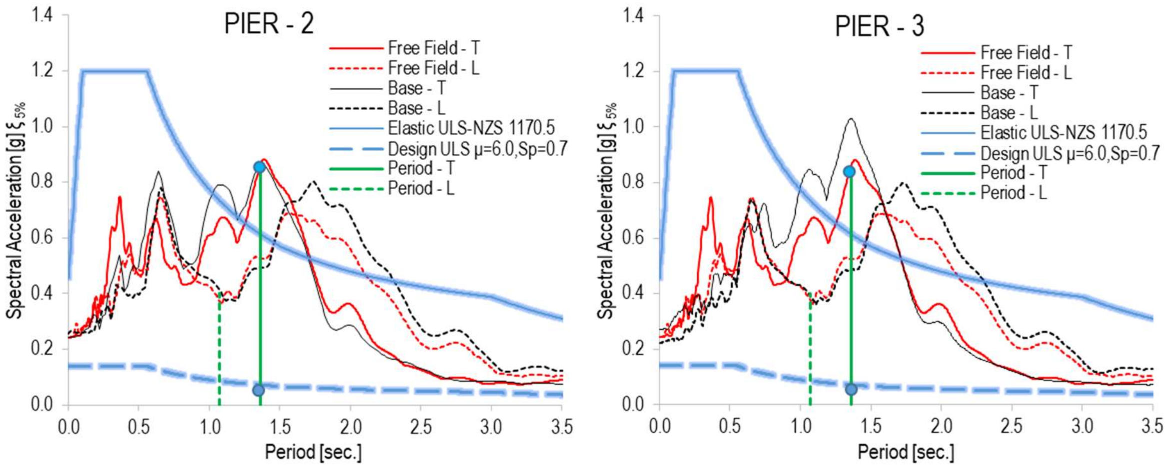


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Figure 10. Elastic Acceleration Response Spectra Comparison for Piers 2 & 3 (from [6])

Displacement Response of the Building

The storey displacement time histories for top level (level 5) relative to the base are plotted in Figure 11 for all three piers in both principal directions. Note that this is only presenting the peak ground motion duration.

The comparison between the two principal directions indicates that in the longitudinal direction, the direction of motion of the three piers is *in-phase* with each other in all cycles. In the transverse direction the three piers have a small *time-lag* (with Pier 2 lagging behind piers 1 and 3) between the peaks despite displacing in the same direction as well as reaching the peak displacement in the same cycle. This is consistent with the expected response of the building based on the modal analysis results, which indicate up to 80% of the mass participates in the first mode in the longitudinal direction where the building piers are designed to displace together by virtue of the link bridge.

Larger displacements are recorded in the transverse direction which is consistent with the greater intensity of shaking in the transverse direction and also correlates well with the observations. Of the three piers, the peak displacement of Pier 2 peak displacements is the highest, which is consistent with the larger seismic mass and lesser stiffness of lateral load resisting system by having one frame bay less than piers 1 and 3. Pier 2 also have longer period of vibration compared to Piers 1 and 3. The next highest peak floor displacement at Level 5 occurs in Pier 1, followed by Pier 3. In transverse direction the first mode mass participation of up to 73% is split across two translational modes due to difference in fundamental periods between Pier 2 versus piers 1 and 3.

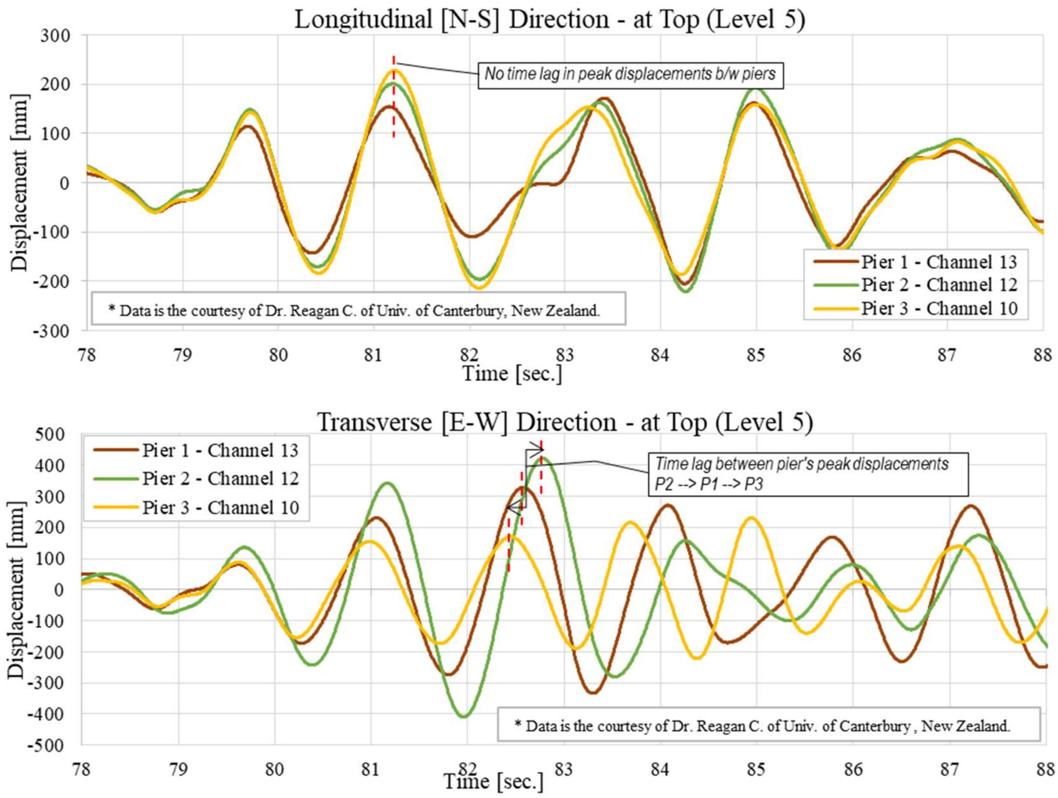

Figure 11. Recorded displacement time-histories at the top (Level 5) of all three piers

Figure 12 plots the peak recorded displacements for the three piers from accelerometer records on the building. Also shown are the estimated yield displacement profile from a pushover analysis, and the peak displacements from the ETABS non-linear 3D time history analysis using recorded Kaikōura accelerometer records. The peak roof displacement ductility demands during Kaikōura earthquake ranged between 3.0 and 4.2 for the three piers in the transverse direction frames where most damage was observed. The ductility demands for lower storeys is expected to be higher where most of the plastic deformations took place.

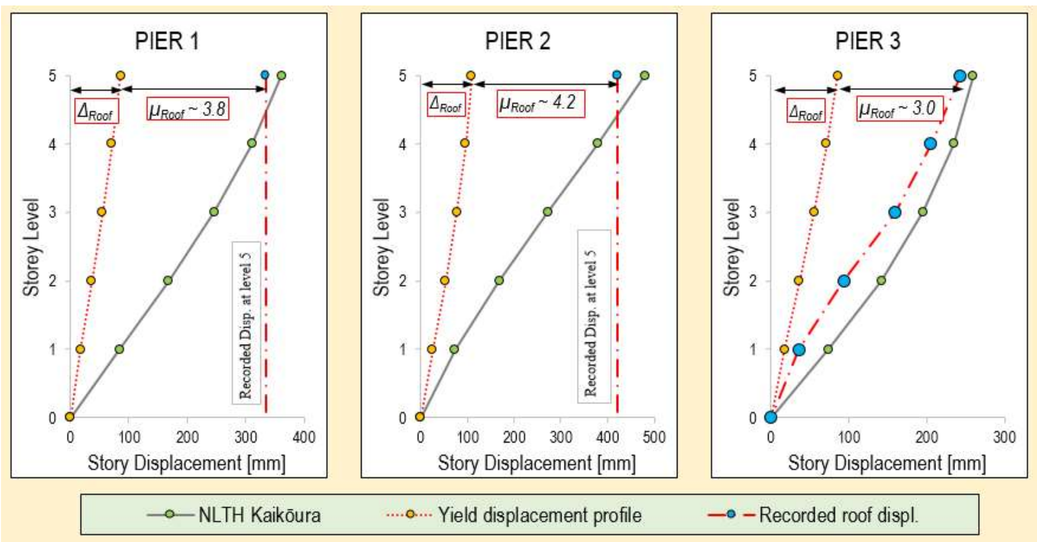

Figure 12. Ductility demands at roof level in each pier in transverse (EW) direction ([6])

Comparison of Records and Analytical Model

The comparison of story displacement time histories (peak duration only) at the Level 5 diaphragm centre of mass (CoM) between the records and the nonlinear direct integration time history analysis (NLTH) under the November 2016 Kaikoura earthquake free-field ground motion are presented in Figure 13. The displacement time histories from the model appears to have reasonably good correlation with that of the recorded response. The trend of the overall building displacement among the piers is also similar to the records.

Figure 13. Displacement Time Histories from Records and NLTH Analysis for the 3 piers

Effect of Foundation Flexibility on the Structure Response

The building response observed from the instrumental records at Level-5 in Pier 3 showed larger displacements on the Waterloo Quay side compared to Harbour-side frames in the longitudinal direction. Figure14 presents this torsional response at foundation level through pile head displacements variability in longitudinal direction across the entire building footprint. It shows the relationship between the foundation response of each pier and the superstructure peak displacements. The time at which the displacements have been taken from the numerical model is based on the instance of peak displacement in each direction from the records as well as from the analytical model in Pier 2 (being the governing pier in terms of total sway).

The foundation displacements (UX) shows a slight trend of differential lateral displacement among harbour and waterloo quay frames. As expected, the piles grounded in the softer (non-remediation) zone of soil, all waterloo quay side frames, undergoes more lateral displacement compared with ones within the remedial zone i.e. harbour side frames. The difference is about 15-20 mm given the nominal values of lateral stiffness of piles, which is expected to vary depending on the sensitivity of the soil parameters.

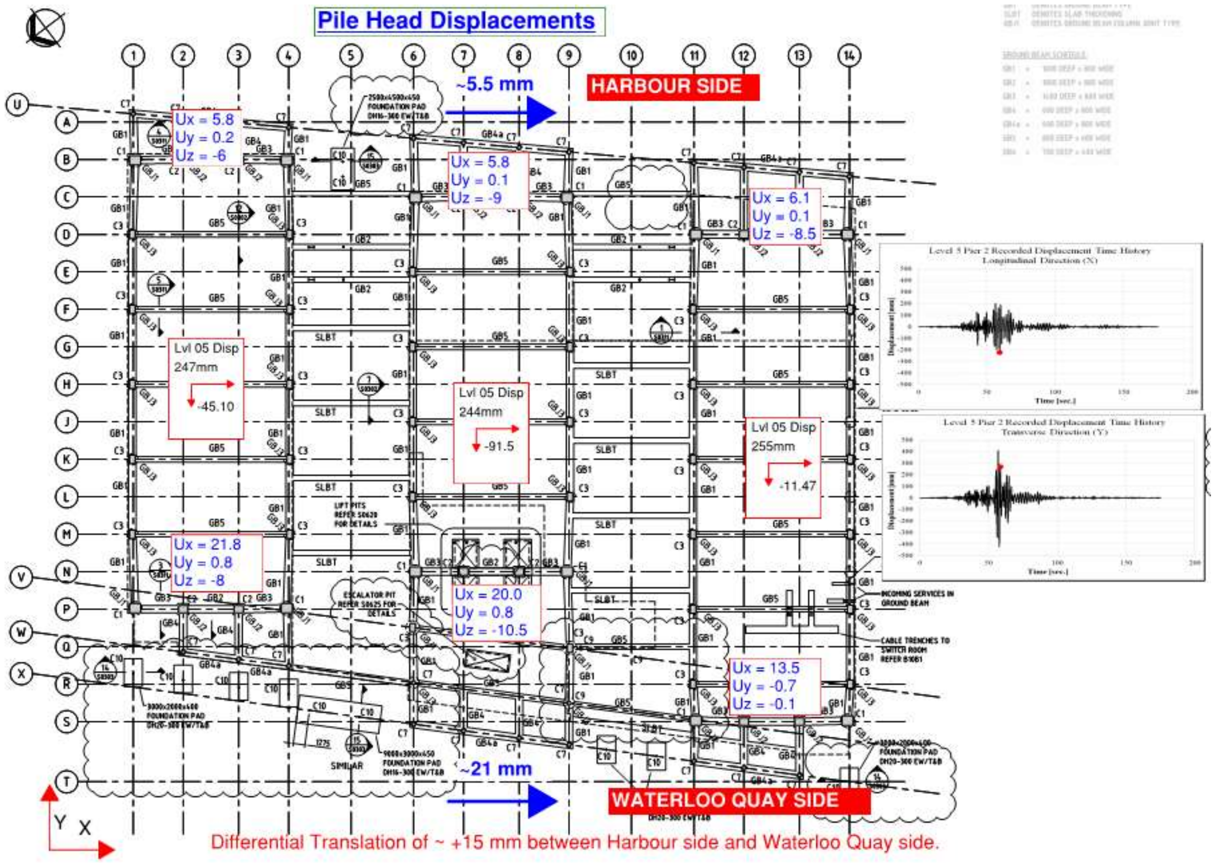

Figure 14. Pile head displacements at the instance of peak recorded roof (level 5) displacements in longitudinal direction

The evidence of foundation torsional response are also supported by the accelerometer records. Figure 15 presents the recorded displacements (w.r.t. ground) on the top storey slab (Level 5) in Pier 3 in the longitudinal (N-S) direction. The displacement time-histories indicate a clear trend of larger floor slab displacement near the Waterloo Quay end as compared to the Harbour side end with differential displacements of about 100 mm at some peak instances between the two ends of floor slab. It is interesting to note the accumulation of differential displacement as we move to the upper levels, with the differential displacements on floor diaphragm amplified from 15mm at the base to 100mm at the top floor level.

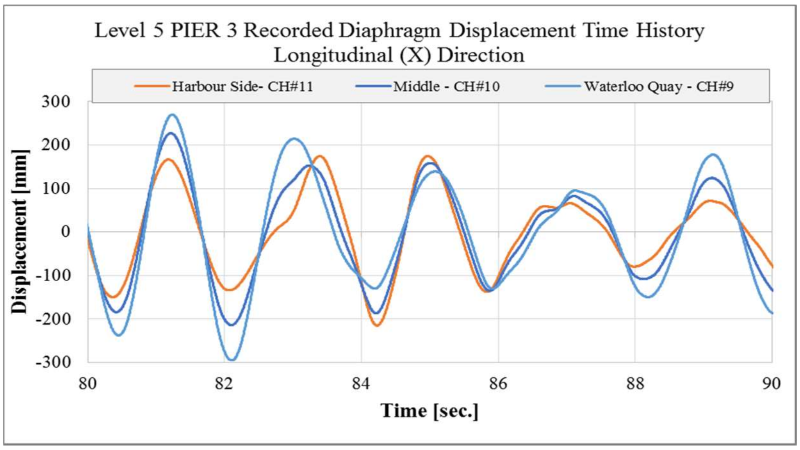

Figure 15. Recorded diaphragm displacements on Pier 3 at roof level in longitudinal direction

The analysis also indicates that the soil surrounding the pile in the non-remediation zone may have undergone deformations beyond the expected soil yield strains.
Contrary to longitudinal direction, in the transverse direction there is no significant torsional behaviour observed at the foundation level.

Response of Ductile Reinforced Concrete Moment Resisting Frames

Behaviour of Plastic Hinges at Beam Ends
In BNZ Building, the transverse frames suffered greater damages as compared to the longitudinal frames consistent with the imposed demands during Kaikōura earthquake as discussed above. The transverse frame beams were also loaded with large gravity loads as they supported the 400mm deep hollowcore floor units spanning 16.8 meters.

The damage observation on the plastic hinging zone indicated that the beams have suffered large cracks (0.5-3mm residual crackwidth) on both bottom and top of beams. At the top of the beams, cracking is sparsely distributed over a shorter distance (~200-250mm) with fewer cracks. At the bottom of the beam, the cracking pattern is usually well distributed within ~500mm plastic hinging zone. Moreover, the beams were surveyed at the column supports and at midspan along grids 4, 6, 9 and 11 at Level 2 and were found that relative to the average level at the supports for each beam, the midspan sag measurements range from 8mm to 29.5mm. The sagging measured is consistent with the behaviour expected for these gravity load-dominated beams. This is consistent with the behaviour of uni-directional plastic hinge region under in-cycle deformations or the so-called *'Ratcheting'* of the member have occurred in the plastic hinge zones of the beam.
A description of the failure mechanism associated with a moment resisting frame with large gravity loads is provided below along with an illustrative sketch in Figure 16.

Ratchetting due to In-cycle displacement demands
First seismic loading cycle (for transverse frames in all piers):

- During seismic loading, the negative moment (top) demand at one end of the beam increases (LHS in Figure 16). Under sufficiently large shaking, the top reinforcing bars of the beam yield first and any further increase in demand results in nonlinear deformation. This can be observed as rotation of the section which leads to cracks opening on the top face of the beam and possible compression zone spalling of the cover concrete on the bottom face.
- At the opposite end of the beam (RHS), the positive moment (bottom) demand at the column face increases after overcoming the negative moments from gravity loads, and therefore this end will not reach yield until after the LHS has yielded and has undergone nonlinear rotation. As the seismic demand increases in the same direction yielding of the bottom bars of the beam will occur over a larger area offset from the face of the column. The yielding region on the RHS will shift closer to the column face as the LHS continues to rotate leading to a concentration of yielding near the column face as the demand approaches the probable maximum capacity of the section.

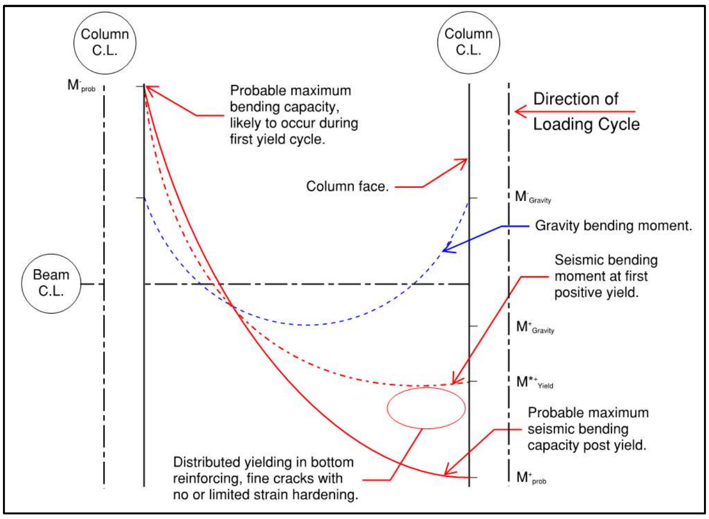

Figure 16. Illustrative bending moment demands on heavily loaded beam in seismic frame

Repetitive reversal of cycles:

- When the seismic load reverses the red lines in Figure 16 also reverse to match and this leads to the same damage repeating on opposite faces at the top of the beam.
- With each inelastic load cycle the process repeats, and the top bars continue to have relatively larger residual plastic deformation than the bottom bars and eventually concrete crushing on the bottom face leading to increasing downwards rotation of the beam at the column face leads to the so called member level 'ratcheting' with unequal plastic deformation for the top and bottom of the beams.

An illustrative example of the expected damage is shown in Figure 17 below.

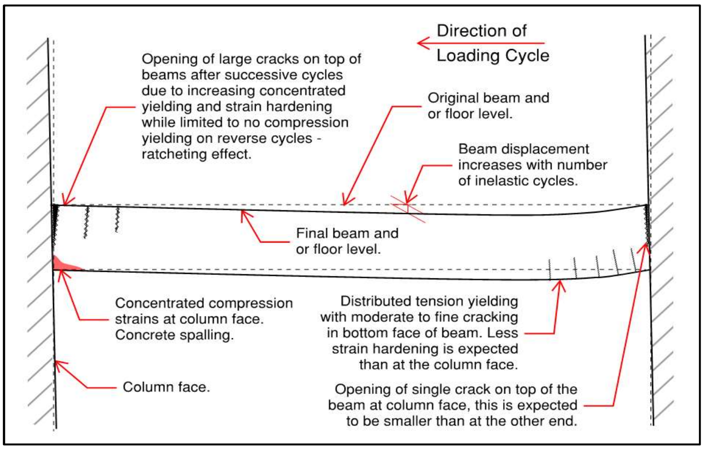

Figure 17. Idealised Beam Hinge Damage after First Major Yield Cycle

To support the hypothesis and to understand the sensitivity of the plastic hinge zone on the damage extrapolation using numerical analysis, a Joint sensitivity check was conducted using SAP 2000. The beam was discretised into a number of 100mm long finite elements within the plastic hinge region as defined in Priestley et al. (2007)[11]. Fibre hinges were assigned to each member based on effective material properties.

In order to capture the beam-column joint behaviour, zero-tension concrete was assigned to the beam end sections at the bottom to represent the cold joint between the precast portion of the beam and the column (ref. Figure 18). The beam section was modelled to include the effective flange width which was calculated in accordance with [3].

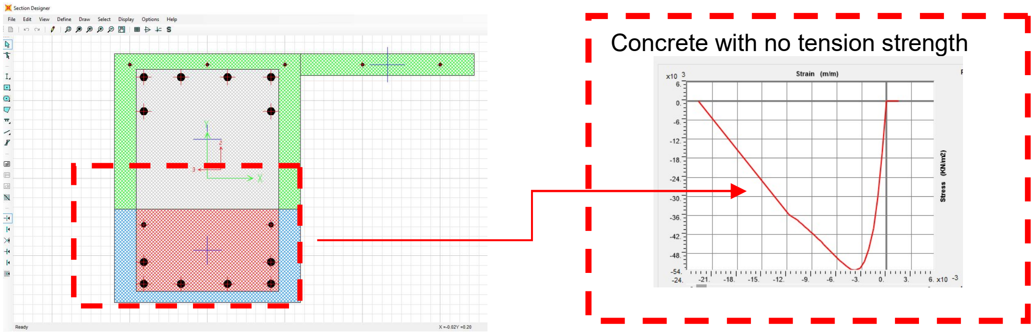

Figure 18. End beam section as modelled in SAP2000.

A full bay was modelled to account for the contribution of gravity loads. A displacement corresponding to 2.0% story drift was applied to the top of the columns.

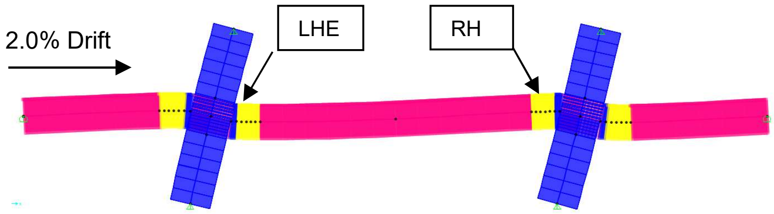

Figure 19. 2D view of the Beam-Column Joint Model as Displaced

The results presented in Figure 20 shows that in the transverse direction the plastic demands on the top of beams (RHE) are higher compared to the bottom of the beams (LHE). The yielding region was found to extend up to 500 mm in length with all hinges yielding. However, the total plastic rotation demand is well distributed in the bottom of the hinge regions. In contrast, at the top of the beam hinge regions, plasticity is concentrated within a plastic hinge length of 100mm to 200mm. These results are in line with the observed damage.

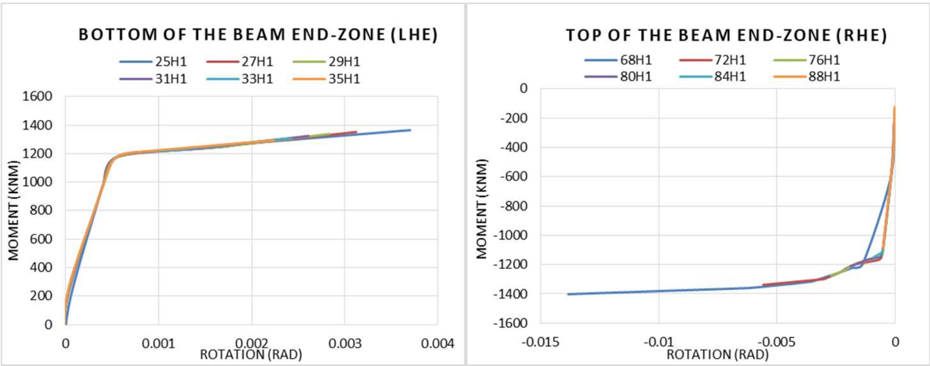

Figure 20. Total rotation demands in the Plastic hinges zones at the top and bottom of beam

Frame Elongation due to cyclic displacement demands

Comparative measurements were made between the construction set out markers and the design dimensions. Analysis of the survey measurements along transverse gridlines shows that the frames in Pier 1 elongated an average of 2.8mm per bay along Grid 1 and 6.0mm along Grid 4. Elongation in Pier 3 was lower with an average of 2.8mm per bay along Grid 11 and 0.5mm per bay along Grid 14. Pier 2 showed the largest elongation with an average of 10.8mm per bay along Grid 6 and 8.7mm per bay along Grid 9. The transverse frames have elongated up to 100mm in Pier 2, with lesser amounts in Piers 1 and 3. These average elongations are consistent with the observed frame plastic hinge cracking damage and the distribution of differing hinge crack width severity around the building.

Diaphragm Behaviour

The main cause of damage to the floor units and associated cast-in concrete toppings is interaction with the supporting frames. As the frames displace in both directions during earthquake shaking, the bemas undergo rotations, plastic hinging and elongation and as a result impose stresses to the unit through rotating it and reducing seating length at the support ledge. The effects of the beams elongating are schematically shown in Figure 21.

<u>Beams transverse to floor units, supporting the ends of the units.</u>

These supporting beams will rotate about their longitudinal axis (following the sway of the columns these beams are attached to). This rotation impose further deformations and forces on the units, the topping, and the reinforcing bars within and passing from the beams in to the floor structure. The imposed deformations at the ends of the units can cause longitudinal splitting in the tops and bottoms of the unit (the “flanges”), and in the vertical elements of the units (the “webs”). These longitudinal cracks can run the full length of the units. Longitudinal cracks in the webs can result in parts or all of the bottom of a unit to fall out of the floor.

<u>Units supported on or next to plastic hinge zones of RC beams</u>

Zones of permanent elongation are caused by the longitudinal bars of the beam undergoing plastic deformation beyond yield. These zones are called “plastic hinge zones”.

Elongation of plastic hinge zones can push beams supporting precast floor units apart. This reduces the seating length between the precast units and support ledge. The rotation of the supporting beam further stresses the unit and the supporting ledge. See Figure 21.

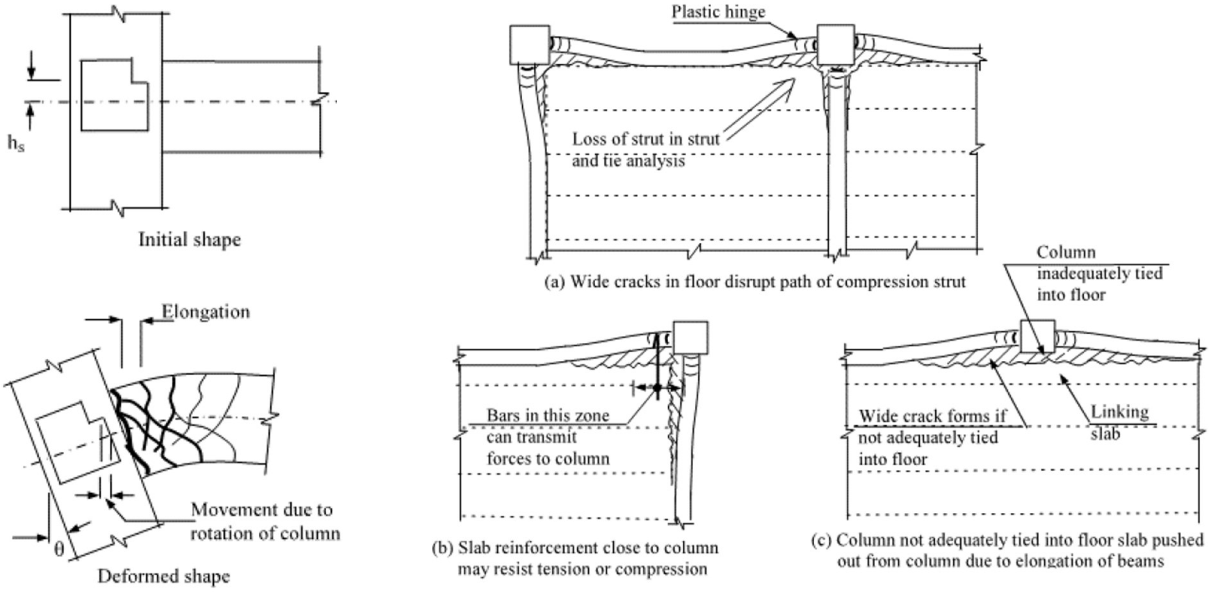

Figure 21. Beam elongation forming large cracks and effect of frame elongations

Figure 22 (part of the crack mapping undertaken by Beca) illustrates the effects of frame elongation/deformation. As the beams elongate next to the columns, the floor gets significantly stretched next to the column, forming many cracks (labelled A in the figure). This elongation also stretches the whole floor resulting in cracks that run the full length of the floor unit. This is exacerbated near the column due to the high rotations in the plastic hinge zone of the supporting beam. The floor units are deep and therefore stiff but brittle – there is no reinforcement across the unit. This longitudinal crack is observed in both the cast-in-place topping and top and bottom flanges of the unit (labelled B).

Other cracks form in the topping immediately above where units meet side-to-side (labelled C). It was noted that these cracks are originally formed by shrinkage of the cast-in-place concrete, but the crack had widened during the earthquake. Similarly, new cracks will form at these locations because of future earthquakes. When a building is being displaced by a major earthquake, the floor plates are stretched “dilates”, compressed and racked (going rhomboidal in shape). The concrete of the topping is the weakest at the line where the units meet, hence it is common to have many full length cracks forming at these locations all over the floor plate. This was widely observed in the BNZ building.

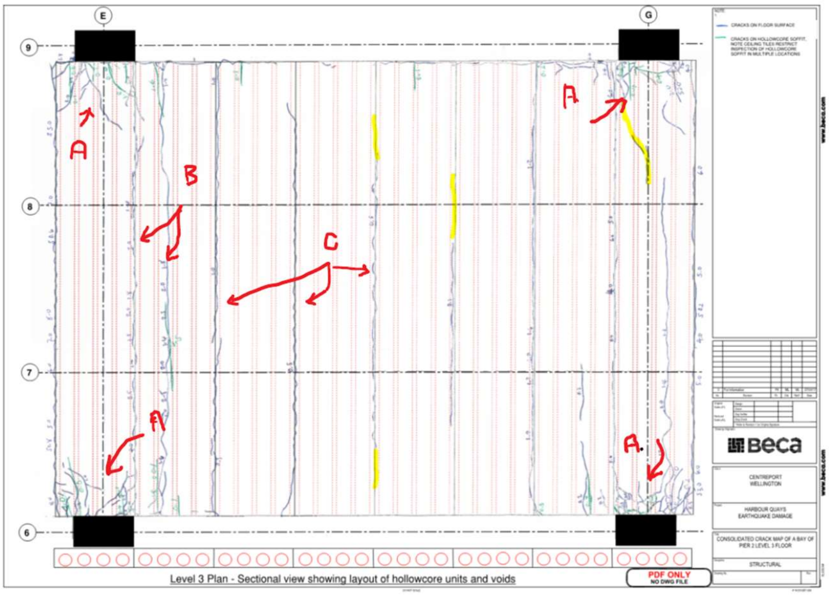

Figure 3: Beam elongation forming a large number of cracks at the beam to column junctions (A), longitudinal cracks in the topping and units below (B) and cracks in the topping where the sides of the units meet (C). [3]

Figure 22. Pier 2 Level 3 floor slab (part) crack maps

A lesser cause of damage is due to the diaphragm action itself. The diaphragm has quite a high aspect ratio (17m x 50m approximately) and it was expected to see some local damage due to strut and tie demands. However, the diaphragm has been designed and detailed for the high demands expected and no particular crack pattern could be observed. We think that the floor diaphragm also acted as a sort of Vierendeel Truss, with moment capacity being provided by the transverse beams and the reinforcement connecting the units and topping to the transverse beams.

<u>Loss of end restraint of hollow core at beam supports</u>

Significant sagging was also measured over the floor slabs. In particular, a noticeable slope is evident in the floor between grids C and D in Piers 1 and 2, where floor level differences of between 40 - 70mm occur over approximately 4m resulting in an approximate 1.0 - 1.75% crossfall. Between the floor perimeter areas and the centre of floor plates the floors generally fall from column locations towards the floor at midspan of floor units supported from the midspan of primary frame beams.

CONCLUSIONS

This building is one of only a few instrumented buildings in New Zealand which have been subjected to large earthquakes. The information available has helped inform our understanding of the building's response, including some of the mechanisms, and the resulting damage to the building. Further analysis to help understand the response across all floor levels of the three piers has shown generally good consistency with the instrumented records.

The damage to the reinforced concrete frame is largely as expected for a flexible structure designed and detailed for a high level of ductility subjected to earthquake demands in excess of the buildings ultimate limit state capacity.

Due to the building being essentially 3 buildings tied by links in one direction, there is consistent drift and damage in the longitudinal direction and in the transverse direction the central pier is most damaged due to its higher mass and lower stiffness.

The instrumentation, analysis and observations has provided an insight into other known effects including:

- In-cycle degradation of capacity due to 'ratchetting' in the beams under high gravity moments at plastic hinge regions
- Torsional response at foundation level due to a differential in soil stiffness between an area of improved and unimproved ground and its subsequent effects on the super structure displacements
- Damage to floor diaphragms and precast hollow core floor units, including permanent vertical deformation

ACKNOWLEDGEMENT

The authors acknowledge the contribution of Harbour Quays F1 F2 ltd., Beca Ltd. [15], Holmes Consulting Group [16], Tonkin & Taylor [5], Fletcher Construction and Dr. Reagan Chandramohan from University of Canterbury [10] for the provision of factual investigation and instrument post-processing data on the building.

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