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PROGRESSIVE COLLAPSE ANALYSIS OF EXISTING BUILDINGS A PERFORMANCE BASED APPROACH

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Abstract

The paper describes the complexity of the seismic assessment and rehabilitation of three different existing buildings in New Zealand. The assessment was performed using Progressive Collapse Analysis. This method has been materialized into explicit requirements for redundancy in building codes. Conventionally, the engineering industry uses a simplistic procedure for most seismic assessments, which models only linear beam and column elements. This neglects the contribution of walls and slabs, leading to uneconomic solutions. Walls and slabs may be considered secondary members in other types of analysis but in progressive collapse analysis, walls and slabs often behave as primary members with slabs carrying load though membrane action and walls providing alternate load paths in case of loss or extensive damage of columns.

The buildings have been modelled using the "Applied Element Method" (AEM) [1, 2, 3]. This approach allows tracking of the structural collapse behavior passing through all stages of the application of loads including elastic stage, crack initiation and propagation in tension-weak materials, steel yielding, element separation and element collision. It has also the unique ability to accurately evaluate the dynamic Eigen modes accounting for the phenomenon of period elongation due to cracking of the structural elements during the ground excitation. Period elongation is a phenomenon that may alter significantly the response of the structures and the effects of the ground motions on the buildings. This is a significant breakthrough not only for the New Zealand industry but also for the international engineering community.

Extensive research was undertaken to overcome the modelling complexities to incorporate the specific building characteristics including riveted connections, slabs, infill panels, foundation and surrounding soil and to assess the performance of the structures using the state of the art methodology [4, 5, 6, 7]. A set of Numerical Integration Time History (NITH) analyses in compliance with AS/NZS 1170.5 [7] recommendations was completed for the Progressive Collapse methodology. Various geotechnical and material testing was undertaken to confirm the parameters used in the analysis. The ground motions were selected and scaled in accordance with Site Specific Seismic Hazard Assessments.

To validate the accuracy of the models, the results were checked against ASCE41-13 [8] acceptance criteria in conjunction with AS/NZS code requirements and limitations [7, 9]. The post-earthquake observation in one of the case studies were used to validate the results of our analysis. The results indicate the efficiency of the specific methodology to visualize the extent, magnitude and direction of any potential local or global collapse or crack occurrences within the structures and provide accurate insights on the performance of the buildings, leading to the most effective strengthening strategy.

This methodology also enables the engineers to safely design the egress routes away from falling debris, for the safe evacuation of the buildings during the earthquakes.

Keywords: Progressive Collapse Analysis; Applied Element Method; Performance Based Design; Period Elongation

1. Introduction

Harrison Grierson Consultants Limited (HG) was engaged by their clients in Wellington and Christchurch in New Zealand, to undertake Detailed Seismic Assessments of their buildings. Three different case studies are presented with this paper. The first two case studies, Hotel St. George and Munro Benge House, are located in Wellington and the third case study, The Park Towers, is located in Christchurch.

The main objectives of the assessments were to assess the seismic capacity of the existing buildings and provide the most effective strengthening scheme to upgrade the buildings to the New Building Standard

The assessments were based on extensive site investigations. Progressive Collapse Analysis with Numerical Integration Time-History (NITH) was used as the most suitable, sophisticated and state of the art analytical technique.

Our aim was to identify the critical structural weaknesses of the existing structures in regards to the acceptable performance based criteria, in order to develop a strengthening scheme that added strength and resilience whilst utilising the existing inherent capacity of the buildings.

2. Methodology

2.1 Progressive Collapse Analysis

Progressive collapse (aka: Disproportionate Collapse) occurs when a local failure spreads throughout a structure from element to element, eventually resulting in the collapse of either the entire structure or a disproportionately large part of it. Progressive collapse is caused by an abnormal or extreme loading event, typically due to earthquake, accidental impact, faulty construction, foundational failure, or violent changes in air pressure (i.e. Blast). Infamous examples of progressive collapse can be seen in such history events as the terrorist attacks on the A.P. Murrah Building (Fig. 1) and the Twin Towers of the World Trade Centre on September 11, 2001 or the Ronan Point building (1968).

Fig. 1 – AP Murrah Building Progressive Collapse

Utilizing the Extreme Loading® for Structures (ELS) proprietary software [10], we had the ability to accurately analyse the behaviour of the structures under extreme loads and to assess the risk of progressive collapse. ELS uses the Applied Element Method (AEM) [1, 2, 3] to track and analyse structural behaviour from its elastic stage through cracking, element separation and collision. This method was used for modelling the building.

Progressive collapse and its dynamic effects may be evaluated using time-history analysis. Therefore, Numerical Integration Time-History (NITH) in accordance with AS/NZS 1170.5:2004 [7] was used for the analysis.

2.2 Applied Element Method

In the AEM [1, 2, 3], the structure is modelled as an assembly of small elements made by dividing the structure virtually (Fig. 2 (a)). Adjacent elements are connected by a set of one normal and two shear springs located at shared surfaces between elements, representing the material behaviour. These nonlinear springs are representing stresses and deformations of a certain volume (Fig. 2(b)).

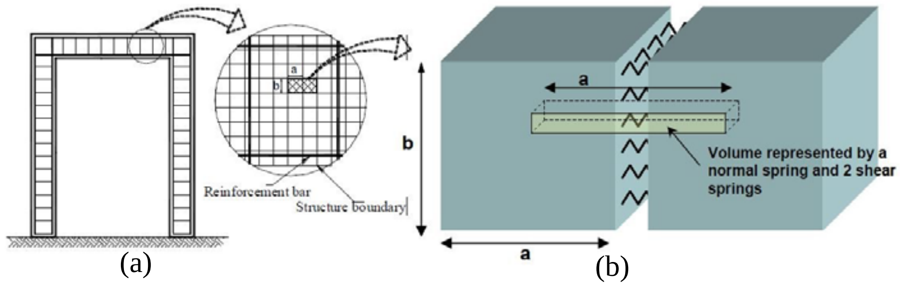


Fig. 2 – (a) Element Generation for AEM, (b) Spring Distribution and Area of Influence of springs

When the strain value at the connecting springs reaches the separation strain, springs are removed. When all springs connected to a specific surface are removed, the surface is considered a free surface and collision is allowed at this surface (Fig. 3).

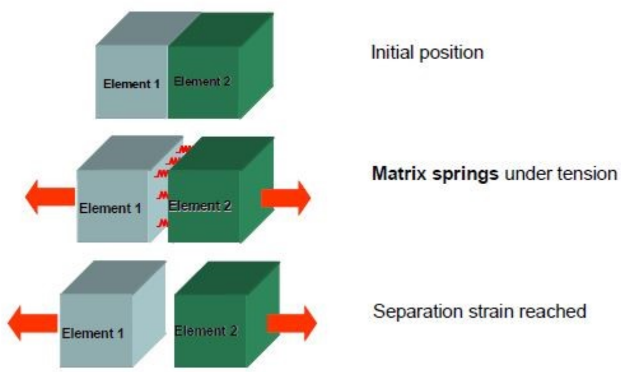


Fig. 3 – Element separation

The AEM is a stiffness-based method, in which an overall stiffness matrix is formulated and the equilibrium equations including each of stiffness, mass and damping matrices are incrementally solved in time domain for the structural deformations (displacements and rotations).

2.3 Extreme Loading for Structures (ELS) Software

ELS [10] is an analysis tool developed by Applied Science International, LLC, based on the AEM [1, 2, 3] method. It can perform nonlinear dynamic analyses for reinforced concrete, steel and composite structures. Modelling the building in ELS allowed us to monitor:

- ✓ Elastic to plastic deformation calculation
- ✓ Automated Plastic hinge formation
- ✓ Buckling and post-buckling under compressive loads
- ✓ P-Delta effect and large displacement consideration
- ✓ Crack propagation and separation of elements
- ✓ Collision and collapse of separated elements

2.4 Selection and Application of Ground Motion Records

Earthquake ground motion records used for time-history analysis were selected based on probabilistic site hazard assessments (PSHA) [12, 13] or the recommendations of AS/NZS 1170.5:2004 and the research carried out by collaboration of University of Auckland and GNS Science [11]. There are specific references in each case study.

The two horizontal components of the ground motions were appropriately scaled (factors k1, k2) in accordance with AS/NZS 1170.5 and used for the analysis. The vertical component of the ground motions were considered only at the third case study, due to the existence of a large cantilevered structure which was considered to be sensitive to these vertical ground motions.

3. Case study 1 – Hotel St George

3.1 Building Description

The Building at 124 Willis St, Wellington, is a 7-storey concrete encased steel frame structure constructed in 1929-1930 (Fig. 4) with an irregular L-shape configuration. It was designated for retail use on the ground floor and hotel accommodation on the upper storeys.

It is currently listed as a Class II heritage building by the New Zealand Historic Places Trust. Partial seismic strengthening of the building by addition of concrete shear walls in different locations of the building was undertaken in 2006.

The facade of the building consists of reinforced concrete infill panels with thickness of 230mm for the first 3 storeys and 150mm for the upper levels. It is penetrated by numerous window openings.

Fig. 4 – Aerial View of the building

The earlier strengthening scheme which involved adding 8 RC shear walls to the building was only partially completed. Walls 2 and 7 are the only walls extended full height while wall 3 terminates after level 1, and walls 1,4,5,6 and 8 are built up to level 4 (Fig. 5).

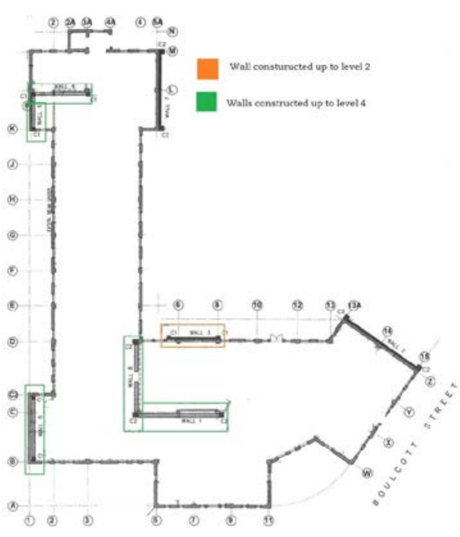


Fig. 5 – Typical Plan of Partial Seismic Strengthening

3.2 Selection and Application of Ground Motion Records

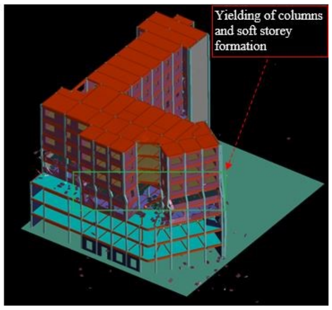
Earthquake ground motion records used for time-history analysis were selected based on the recommendations of AS/NZS 1170.5:2004 [7] and the research carried out by collaboration of University of Auckland and GNS Science [11]. The particular four ground motions used for the analysis are as follows (Table 1):

Table 1. Ground motions used for analyses

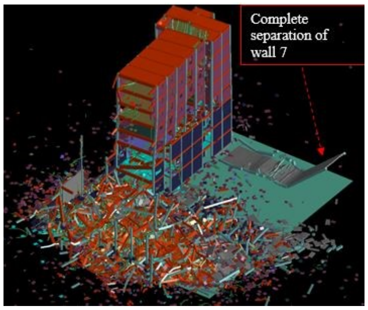
Zone	Record Name	Date	M (MW)	D (kM)	Fault Mechanism	Forward Directivity	Applicable Type of Soil
North NF	El Centro	19-May-40	7	6	Strike-Slip	No	C and D
North NF	Duzce	12-Nov-99	7.1	8	Oblique	No	C and D
North NF	Tabas	16-Sep-78	7.4	2	Reverse	Yes	C
North NF	La Union	19-Sep-85	8.1	16	Sub. interface	No	C

3.3 Analysis Results of Existing Building

Upon completion of the analysis of the existing building subjected to 100% of the selected ground motions it was found that the existence of critical structural weaknesses such as plan irregularity and vertical stiffness irregularity led to global or significant partial collapse of the building (Fig. 6 and 7).



(a)



(b)

Fig. 6 – (a) Formation of Soft Storey Mechanism (b) Global Collapse-100% Duzce Ground Motion

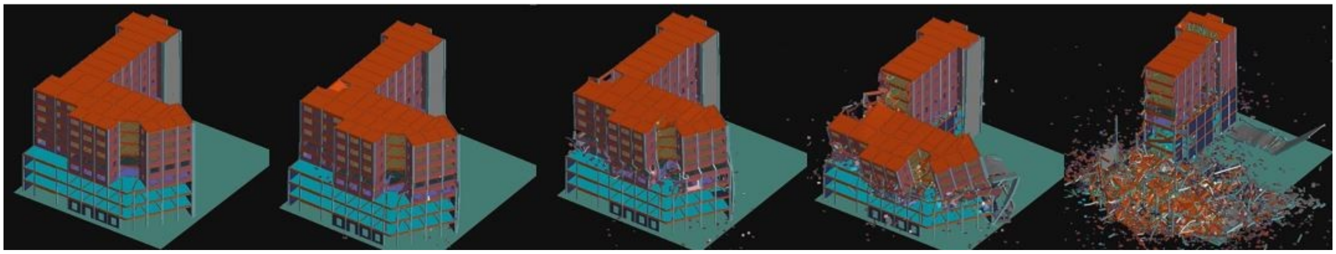


Fig. 7 – Sequence of the Existing Building Collapse Subjected to 100% Duzce Ground Motion

The next step of our investigation was to determine the building's rating with respect to the New Building Standard (NBS%). Therefore the building was subjected to a certain percentage of the scaled ground motions and the performance of the building reviewed. The procedure was completed in an iterative process to determine the performance level in which the building performed satisfactorily.

A Detailed Seismic Assessment of the building in accordance with NZSEE [9] and ASCE41-13 [8] performance criteria was then carried out to validate the results of the progressive collapse analysis. The comparison between the analyses demands and the flexural and shear capacities of beams, columns and shear walls, verified that the software predicted capacities up to member's collapse stage (member separation) with high levels of accuracy and are in compliance with the codes performance criteria.

The results indicated that the building can achieve a seismic rating of 45%NBS (New Building Standard or 45% of current code).

3.4 Strengthening Scheme

Following the analysis of the existing structure it was crucial to develop a strengthening scheme that would address the soft storey mechanism as well as the diaphragm deficiencies.

This required the consideration of the client's requirements (maximise leasable area whilst creating a strengthened structure) and the wider communities and local authorities interests in the heritage aspects of the area (minimise any changes to the exterior of the building fabric).

Exploring several different options for the seismic strengthening of the building resulted with the continuation of the existing RC shear walls up to the 8^{th} floor as the most effective solution. A comprehensive set of progressive collapse analyses using NITH method and the records mentioned in Table 1 were performed.

Interpretation of the attained results, showed that:

- Where the building was subjected to shallow crustal earthquakes without pronounced forward directivity features it experienced a higher level of damage.
- The extent of damage to the building subjected to 70% of the Duzce ground motion with the main component in the X direction is the most adverse.
- Extending the walls to the roof level, resulted in a better distribution of lateral loads among the resisting elements. Accordingly the main limiting factors of the building performance, the flexural failure of walls and the flexural and shear failure of the beams were resolved. Also, this measure mitigated the soft storey mechanism and the detachment of the existing shear walls (2006) from the building.

Despite the strengthening scheme mentioned above, we observed tension failure of the riveted connections in levels 7 and 8 between the beams and the column in the corner where the two wings of the building join, initiating damage. As a result of this loss of support the slab-diaphragm in this area fails and caused a partial local collapse (Fig. 8). Therefore, strengthening was required to the slab-diaphragm over the localised area where the two wings of the building meet.

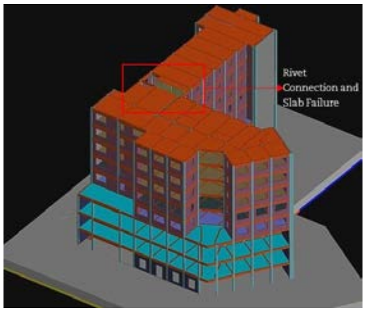


Fig. 8 – Partial Collapse after Continuing up the Shear Walls-70% Duzce Ground Motion

The final scheme includes the strengthening of the slab-diaphragm in this area to all levels above the ground level using 80mm shotcrete adequately connected to the existing structure.

4. Case study 2 – Munro Benge House

4.1 Building Description

According to the original structural drawings, the Munro Benge House is a 10-storey reinforced concrete structure built in 1962 with a basement plus plant and tank rooms located on the roof level of the building. The superstructure of the building is supported on shallow foundations including shallow pads and tie beams mostly founded on completely weathered greywacke rock. The basement, ground and first floor levels consist of in-situ concrete slabs reinforced with hot-rolled steel reinforcing bars. The levels above the first floor were constructed with precast floor elements with a 4'' (101.6mm) thick in-situ structural concrete screed layer cast on top and reinforced with 335 HRC fabric which consists of cold-drawn steel wires. Gravity and lateral loads are transferred mainly through a perimeter frame and an internal frame that runs from east to west near the lift shaft, stair well and toilet areas. Service core concrete walls are present from the basement to the roof level.

As part of our assessment, a detailed 3D model was created in the Extreme Loading for Structures (ELS) [10] proprietary software to represent the existing building and consists of all structural elements including the service core concrete walls from the basement to the roof level, precast concrete floor units and every reinforcing steel bar and HRC fabric layer present in all the concrete beams, columns, walls, floors and foundations. Fig. 9 shows the 3D model generated in the ELS proprietary software.

Fig. 9 – 3D ELS model of Munro Benge House

4.2 Selection and Application of Ground Motion Records

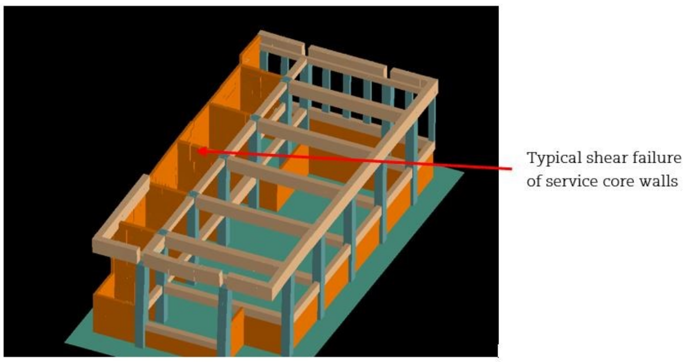
The ground motion records were selected by GNS Science [13] based on a site specific study and scaled to the requirements of NZS 1170.5 – the New Zealand seismic standard. After scaling with the record scale factor k1, the family scale factor k2 and a percentage reduction factor (%NBS), each ground motion accelerogram was used in two analyses; in the first analysis, the principal component (H1) and secondary component (H2) were applied in the X and Y directions (Fig. 9) respectively, and then in the second analysis, H1 and H2 were applied in the Y and X directions respectively (Table 2).

Table 2 – Selected ground motion accelerograms

Record Name	**Date**	**M(MW)**	**D(KM)**	**Fault Mechanism**	**Station Soil**	**Forward Directivity**
F02PS101 PS10 Denali (USA)	2002	7.9	2.7	Strike-Slip	Vs30 = 330m/s	Yes
F99604Z2 Duzce (Turkey)	1999	7.2	8	Strike-Slip	Soft soil	No
HHKD085 Hokkaido (Japan)	2003	8.3	46	Subduction Interface	Weak rock	No

4.3 Analysis Results of Existing Building

Initially, the structure was analysed at 100% of the Denali ground motion with H1 applied in the X direction and H2 applied in the Y direction. From this initial analysis, it was found that the service core concrete walls failed in shear and separated at the rear of the building due to shear failure of the reinforcing bars connecting these walls. As a result, it was determined that the seismic rating of the building would be below 100%NBS. The critical factors were determined to be the lateral shear capacity of the service core walls located on the southern side of the building and the shear capacity of the reinforcing bars connecting the walls. Fig. 10 shows the separation of the service core walls of the building.



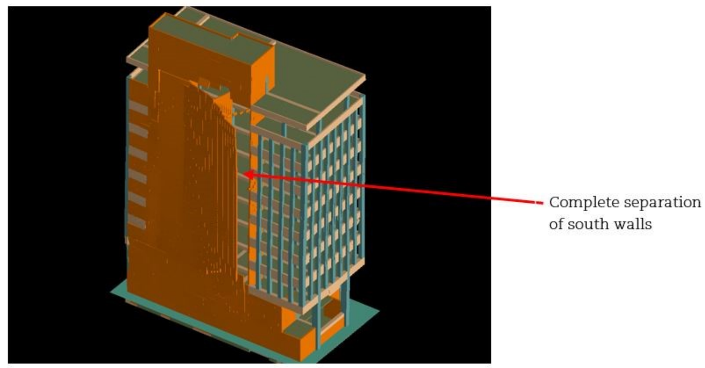


Fig. 10 – Shear wall failure (left) and complete wall separation (right)

Following the initial assessment at 100%NBS, another set of analyses was undertaken at 34% of the Denali, Duzce and Hokkaido earthquakes. All structural elements at every floor level were checked in the ELS proprietary software to identify plastic hinge zones that may have formed during the ground motion period. It was found that the structure performed mostly within the elastic range throughout the period. No failures were found in the post-analysis examination. On the basis of these findings, the Munro Benge House is not an 'earthquake prone' building as defined under the current New Zealand Building Act and related regulations as the seismic rating is calculated to be above 34%NBS (34% of current code requirements).

To obtain a more accurate rating, a final series of analyses was completed at 67% of the three selected ground motion accelerograms. Plastic rotations and inter-storey drifts were checked against the requirements of ASCE 41-13 [8] and AS/NZS 1170.5 [7]. Shear capacities of selected members and the bearing strength of the ground material were also checked against the demand actions. Based on the building meeting the stated criteria, it was concluded that the seismic rating of the building is 67%NBS. With 67%NBS, the Munro Benge House could continue to be occupied and is not required to be strengthened by law. As a result, this is a desirable outcome for the client.

When the client engaged Harrison Grierson, the Munro Benge House had already been assessed by another engineering consulting firm using the nonlinear pushover analysis procedure. The previous seismic rating was below 34%NBS defining the building was earthquake prone. This significant difference between the rating outcomes (34 and 67%NBS) was mainly due to the contribution of the reinforced concrete service core walls (shown in Fig. 10) located on the southern side of the building. These walls were completely neglected in the initial seismic assessment due to the uncertain behaviour of the existing HRC fabric reinforcement in the walls. In the subsequent ELS model, all of these walls were included.

In addition, it was found that the structure period was initially constant and then elongated due to concrete cracking and reinforcement yielding in all the analysis cases considered. Following each analysis, the period was obtained as a function of time. Fig. 11 and 12 show the structure period against time for the X and Y directions in the 34% and 67% cases respectively.

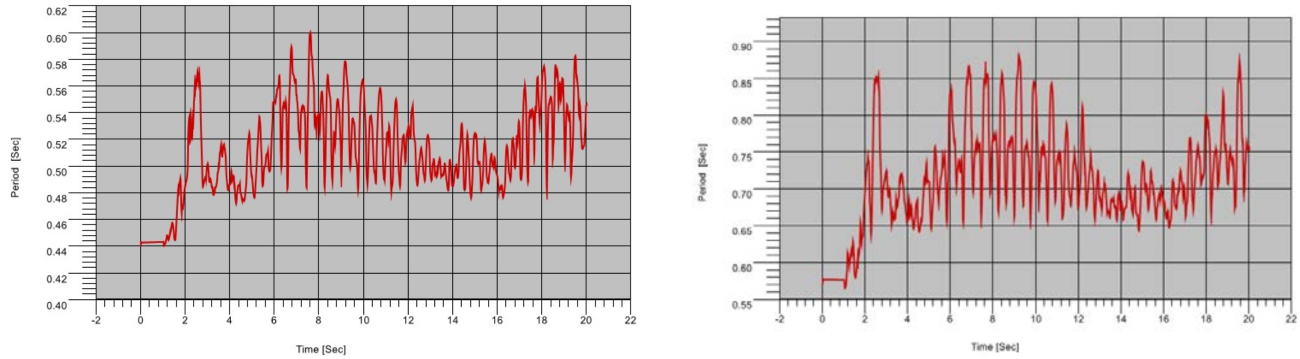


Fig. 11 – Structure period vs. time in the X (left) and Y (right) directions (34% cases)

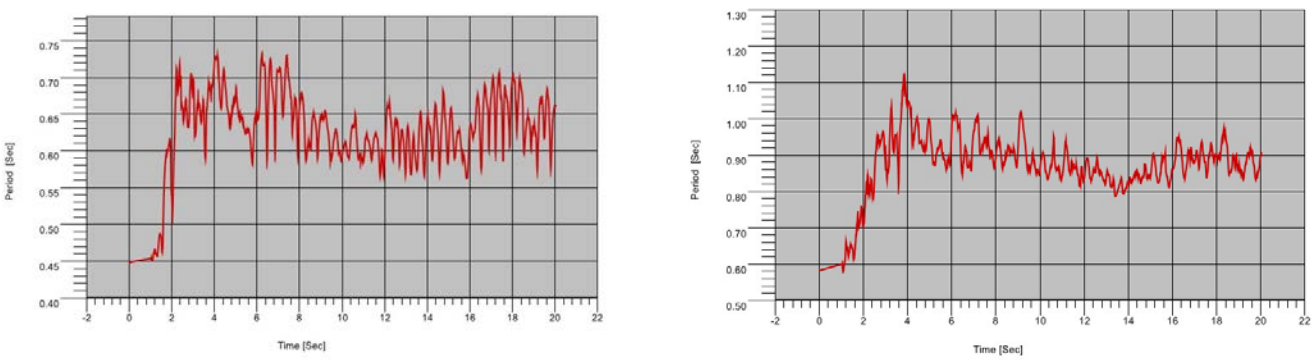


Fig. 12 – Structure period vs. time in the X (left) and Y (right) directions (67% cases)

According to GNS Science [13], the k2 scale factor was 1.25 for the building, and the k1 factor was determined based on the period of the structure. The periods used for the k1 factor calculation were 0.6s and 0.8s for the X and Y directions respectively for all the 34% analysis cases (Denali, Duzce and Hokkaido), and for the 67% cases, the selected periods were 0.7s (X) and 1.0s (Y). It was found that the k1 factor decreased with the period elongation and led to reduced ground accelerations, and therefore, a more desirable and accurate rating outcome was achieved for the building.

4.4 Strengthening Scheme

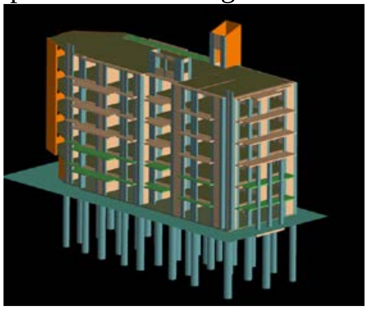
Even though strengthening was not required due to the performance meeting minimum requirements, a concept strengthening scheme was proposed for the building based on its seismic performance. As part of the scheme, the existing concrete service core walls would be thickened with additional reinforcement to increase lateral shear capacity, and new bolted steel bracket connections would be added between existing walls to prevent wall separation. Also, additional reinforced concrete shear walls would be constructed in the north-south direction to reduce plastic rotations of columns. The proposed scheme aimed to upgrade the seismic rating to 100%NBS (New Building Standard or 100% of current code requirements).

5. Case study 3 – Park Towers

5.1 Building Description

The Main Residential Tower (Fig. 13) is one of the buildings within the residential complex at 12 Latimer Square, Christchurch, also known as Park Towers. The Main Residential Tower is an eight storey residential block with precast concrete panels and composite floor slabs. All storeys (apart from light weight structure above level 7) are similar in plan with approximate dimensions of 27m by 10.7m. It was designed and constructed in c.1998.

During the 2011 Canterbury earthquakes the cantilevered part of the building was partially collapsed. Harrison Grierson were commissioned to check the residual capacity of the remaining building and re-instate the cantilevered part of the building to a new design.



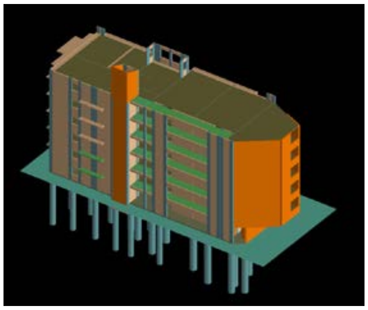


Fig. 13 – 3D ELS model of Park Towers

5.2 Selection and Application of Ground Motion Records

Earthquake ground motion records used for time-history analysis were selected based on the probabilistic site hazard assessment carried out by Golder & Associates [12]. Based on the recommendation from Golder & Associates, three ground motion records (Table 3) which have been appropriately scaled to New Zealand code, were taken into account for our analysis.

Table 3 – Selected ground motions

Record Name	**Date**	**M (mw)**	**D (km)**	**Velocity Pulse**
Parkfield, California (USA)	2004	6	4.23	Yes
Darfield, Canterbury	2010	7.1	6.0	No
Hector Mine, California	1999	7.13	104.95	No

5.3 Analysis results of existing building

Under all ground motions it was observed that the cantilevered part to the rear of the building collapsed. The collapse propagated from one of the precast walls due to the failure of the connections between the wall and the slab (see fig. 14). This was followed by the collapse of the cantilever slab at level 1 (see fig. 15) which propagated the total collapse of the cantilever part of the building (see fig. 16).

The damaged observed on site indicated a significant validation of our model under the real ground motion causing the collapse of the cantilever part of the building. The mode of collapse of our model was similar to the observed damage and crack patterns of the building due to the Canterbury Earthquake Sequence.

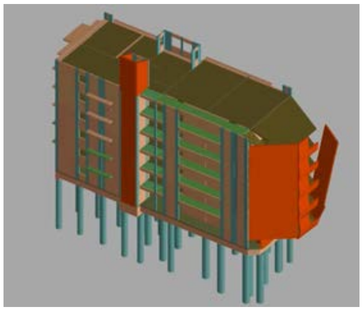


Fig. 14 – Collapse initiation

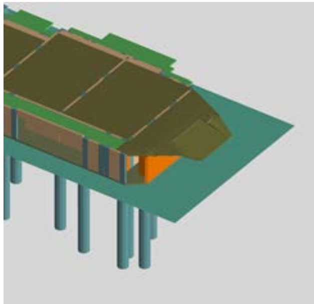


Fig. 15 – Level 1 slab collapse

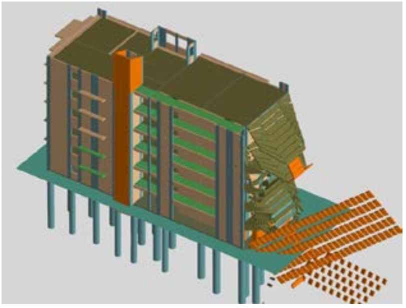


Fig. 16 – Full collapse of the cantiléver

Minor cracks to the precast walls, balconies and slabs were also observed.

Yielding of starter bars connecting the walls and the ground beams were also observed at some of the walls. The majority of the structural elements of the building (walls, slabs and beams) remained without visible damage.

5.4 Strengthening Scheme - Reconstruction of the Proposed Cantilever Part

Even though strengthening was not required for the remaining part of the building, we have been asked to redesign the cantilever part of the building. The remodelling of the building, including the new part indicated the successful choice of the new structural system in regards to the overall performance under the real ground motions.

6. Conclusion

Using the advanced capabilities of the Applied Element Method (AEM) [1, 2, 3] based Extreme Loading® for structures (ELS) [10] software we were able to create models fully utilising the concept of Performance Based Design. The AEM method is likely the future of sophisticated structural analyses available to structural engineers for damage assessment and progressive collapse analysis. It is one of the most advanced structural analysis tool capable of quantifying the level of damage sustained by a structure when it is subjected to a specific load case or ground motion. This allows the engineer to check the level of damage against the required performance objective for the particular load case. Ultimately this method gives significant time and cost savings over existing nonlinear time-history-based analysis.

The three case studies have demonstrated the ability of the methodology to reveal hidden modes of collapse within the structures, to visualise the extent and direction of damage and identify the significant influence of the period elongation of the structures during several ground motions. The comparison with the observed damage of our third case study, indicates a significant validation of our methodology against the damage occurred on the building during the 2011 Canterbury earthquakes.

This method of analysis makes evaluating multiple scenarios with multiple performance objectives (such as immediate occupancy, life safety, or collapse prevention) both practical and economical. The building owner can easily visualize the performance and the level of damage and determine whether it is acceptable for the specific hazard under consideration. This is an enhanced advantage unavailable using many other techniques, where the performance objectives are either qualitative or quantitative but difficult to visualize by the owners.

Conventionally, the engineering industry uses a simplistic procedure for seismic assessments, which models only linear beam and column elements. This neglects the contribution of walls and slabs, leading to uneconomic and/or inaccurate results. Walls and slabs may be considered secondary members in other types of analysis but in progressive collapse analysis, walls and slabs often behave as primary members with slabs carrying load though membrane action and walls providing alternate load paths in case of loss of columns. Our case studies demonstrated the significant influence of all these parameters on the seismic capacity of the existing buildings as well as on the cost for any seismic upgrades to the current building standard.

Considering the sophisticated modelling and analysis capabilities of AEM in ELS, it is particularly suited for historic structures where the strength of secondary elements is typically ignored in traditional modelling and analysis approaches.

In recent years progressive collapse analysis has developed into explicit requirements for redundancy in building codes all over the world.

In a comparison study, we have found that analysis using simplified finite element linear and nonlinear analysis suggested a significant increase in the strengthening scheme cost to satisfy current code requirements. Using more advanced analysis, like the Applied Element Method, shows significant reduction on the strengthening scheme cost and better ratings on the seismic capacity of the existing buildings while provides accurate insights on the performance of the buildings, enabling the engineers to safely design the egress routes.

7. Acknowledgements

We would like to express our appreciation to ASI [14] and in particular to Hatem Tagel-Din, Ph.D., President and Ahmed Amir Khalil, Ph.D., Senior Structural Consultant, for the peer review of our case studies and their useful comments.

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