Seismic behaviour of a 22 storey building during the Canterbury earthquakes

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ABSTRACT: The seismic behaviour of a 22 storey building during the 2010-11 Canterbury earthquake sequence is investigated. The building, which consists of perimeter seismic resistant concrete frames and interior concrete column and steel beams gravity frames, was constructed in the late 80s and was substantially damaged during the 22nd February 2011 event. The observed damage included plastic hinges at the beam-column joints of most of the floors and a residual tilt due to permanent displacements in the foundation soils as well as residual interstorey drift. Following the conventional design practice, this paper focuses on the superstructure performance only, ignoring in the initial set of analyses, the effects of soil-foundation interface response which are being investigated as part of a wider research investigation. Non-linear time-history analyses using bi-directional input motion were carried out using a fixed-base three-dimensional lumped plasticity structural model. Different modelling assumptions on the beam-column joint configurations were considered to investigate their effect on the seismic response of the building. The numerical findings are compared with the damage observed during a detailed inspection of the exposed structural members prior to full deconstruction of the building.

1 INTRODUCTION

The Christchurch February 2011 earthquake had a drastic impact on the life of many people, on the built environment and on the national and international engineering community. Several reinforced concrete (RC) buildings were severely damaged (Kam, et al. 2011) and two of them collapsed (Beca 2011, Hyland and Smith 2012) causing 133 out of the 185 fatalities. After two years from the main event most of the city high-rise buildings have been demolished and parts of the central business district (CBD) are still closed to public. Yet, during the last two years the engineering and scientific community has been active. Since the main earthquake struck, a number of reconnaissance works have been published to disseminate preliminary observations (e.g. Special Issue of the NZSEE Bulletin 2011) and technical recommendations on different specific aspects (Canterbury Earthquakes Royal Commission 2012), in the attempt of learning from the past events, understanding the limitations of the current practice and trying to improve the seismic design process.

As part of the above described background, this paper aims to investigate the seismic performance of a multi-storey RC building during the Christchurch earthquake. Firstly, the building is described and the observed damage is summarized. Then, after a description of the adopted numerical model, the main results are outlined and discussed.

2 THE BUILDING

The investigated structure is a 22 storey building (including the basement) constructed in mid-late 1980s (construction completed in 1989). The plan layout is rectangular and mostly repeated along the building elevation, with the exception of an enlargement from the basement to the 3rd storey. Figure 1a shows the typical storey plan between storeys 4 and 21; the building cross sections are shown in

Figure 2.

The seismic resistant structural system is composed of perimeter reinforced concrete frames in both the east-west (grids H and B) and north-south (grids 2 and 7) directions. The frames are composed of precast beams (typically 1100mm deep, with a span length ranging from 6 to 7m) supported by in cast-in-situ columns. The corner columns are square (1100x1100mm), while the perimeter columns are rectangular, sized 1100x800mm (Fig. 1b). Figure 1c shows the typical precast beam (575x1100mm). At all the floors, the beam reinforcement at each beam-column (BC) joint interface is detailed for a plastic hinge relocation at 500mm from the column interface (Fig. 1d). The beams are connected at mid-span through emulative cast-in-situ joints.

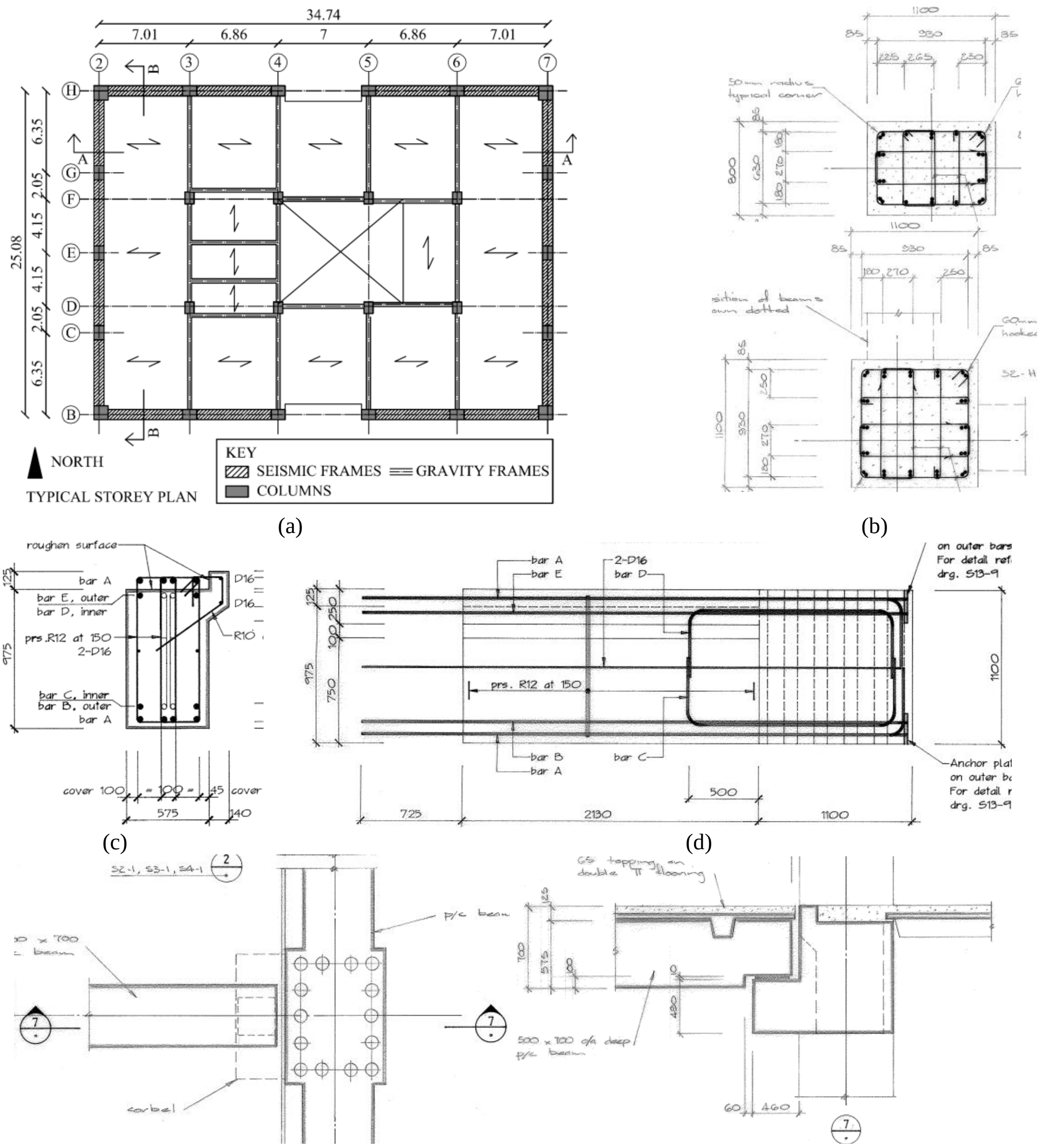


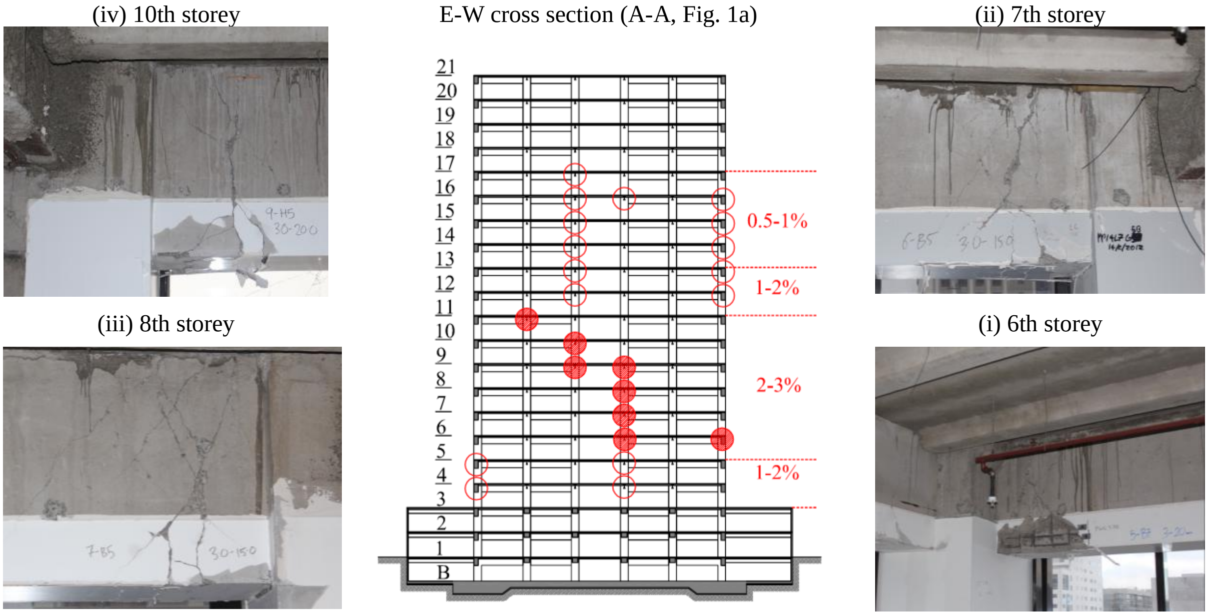
Figure 1 Structural layout and details of the case-study 22-storey RC building : (a) typical storey plan; (b) typical corner and internal columns; (c) typical beam; (d) plastic hinge relocation detail; (e) decoupling detail between the tower and the 3 storeys building enlargement at the bottom.

The suspended floors consist of precast double tees, 200mm deep with a 65mm thick cast-in-situ reinforced topping, spanning about 7m along the east-west direction. The floors are supported by the north-south frames (including the perimeter seismic frames). The gravity frames are mostly composed of steel UB elements (grids 3, 4, 5, 6, F and D, Fig. 1a) supported by cast-in-situ RC columns. The connection between the tower and the building podium (Fig. 2) is detailed as shown in Figure 1e; a seismic gap was designed to prevent the two building interacting each other.

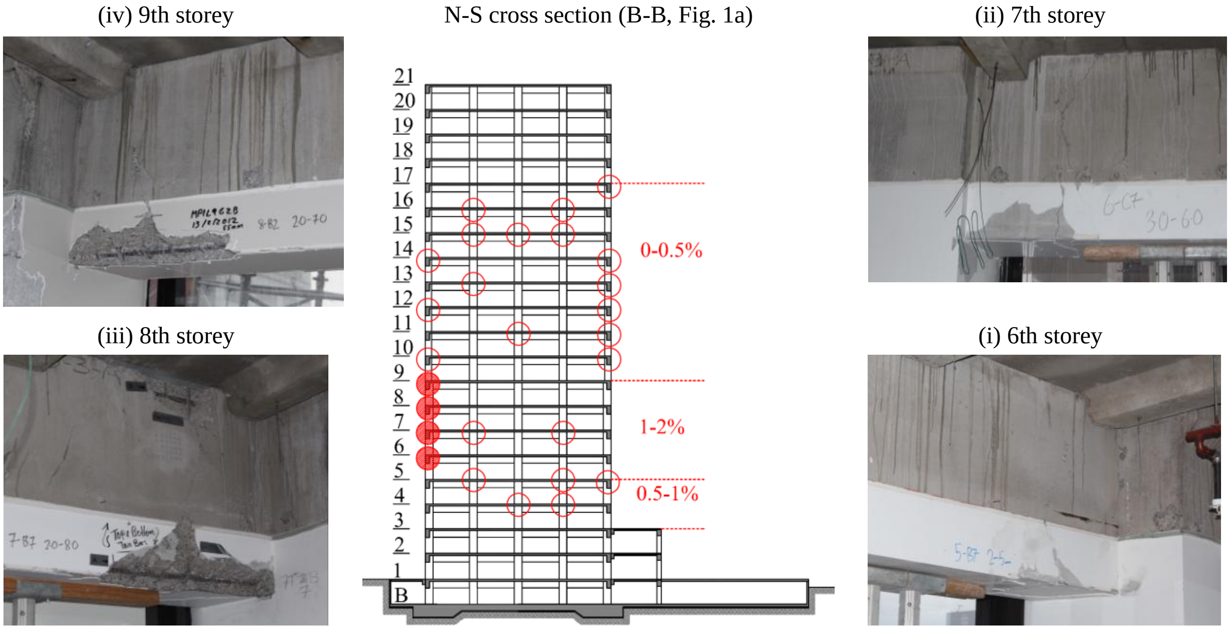
The building sits on raft foundations having variable thickness (1800, 900 and 400mm beneath the seismic frames, the central core and the basement enlargement respectively).

3 OBSERVED DAMAGE

During the February 2011 Christchurch earthquake, the building responded consistently with a "text-book" type weak-beam strong-column mechanism, developing plastic hinges at the beam-column joints of most of the storeys (e.g. Fig. 2).



(a)



(b)

Figure 2 Damage assessment of the building: east-west (a) and north-south (b) cross sections with the indication of the most damaged BC joint locations at each storey (red circles), few selected photographs and judgement estimation of the interstorey drift. The dashed circles show where the maximum crack widths were observed.

A detailed damage inspection of the exposed structural skeleton and individual members (e.g. after all linings, contents, non-structural elements, partitions, carpets etc. where completely removed for a clean disposal of the debris) prior to full deconstruction, was carried out in July 2012. Because of safety issues, and the fact that controlled demolition had been started at the higher levels, the inspection was limited to the accessible BC joints (most of them) located between the 4th and the 17th storeys. The crack width range (referred to as "minimum" and "maximum" values), was recorded at each BC joint. Table 1 reports the maximum observed values at each storey and average of the maximum values observed in the BC joints of the same storey (providing indication of a storey trend). Although the damage was widespread throughout the building, the maximum inelastic demand seems to have concentrated between the 6th and the 11th storeys (Table 1, grey highlighting and Fig. 2, dashed circles). However, not necessarily the crack width is an indication of larger drift demand; for equal or higher interstorey displacement demand, if the plastic hinge length grows, the crack width become smaller. The building shows more damage in the east-west direction (grids B and H), consistently with the principal directivity of the ground motion recorded at the four CBD stations at close distance to the building (Kam, et al. 2011).

Figure 2 illustrates the location of the most damaged BC joints at each storey (red circles) and some of the correspondent photographs. Both single crack and spread hinges can be noticed. Consistently with the hinge relocation reinforcement detail (Fig. 1d), the cracks occurred at distance from the column interface (about half of the beam depth). The reported drift demand is at this stage a preliminary estimation based on qualitative comparison with laboratory test experience on RC BC joints.

Table. 1 Observed crack widths (mm)

Level	**4**	**5**	**6**	**7**	**8**	**9**	**10**	**11**	**12**	**13**	**14**	**15**	**16**	**17**
Maximum values														
Grid B	4	7	20	15	15	15	10	5	7	10	2.5	2.2	2.2	1.4
Grid H	5	5	15	9	15	20	20	15	10	2.5	3	2.5	0.8	1.2
Grid 2	3	2	3	4	7	7	0.7	0.4	0.6	0.5	1.1	0.8	0.5	0.8
Grid 7	1.6	2	5	6	8	7	3	0.6	1	0.6	0.7	0.5	0.4	/
Averages of Maximum values														
Grid B	2.4	2.7	7.9	8.0	7.9	8.5	6.3	2.9	2.6	2.6	1.5	1.0	1.4	0.6
Grid H	2.4	3.3	5.3	7.0	9.8	10.9	8.8	6.5	5.1	1.7	1.6	1.0	0.6	1.2
Grid 2	1.8	1.6	1.8	3.5	3.8	3.6	0.5	0.4	0.5	0.5	0.6	0.4	0.4	0.4
Grid 7	1.5	1.9	2.5	3.5	3.4	2.4	0.8	0.4	0.5	0.5	0.5	0.4	0.3	/

4 NUMERICAL MODEL

The building seismic response has been investigated through nonlinear time-history analyses of the three-dimensional numerical model (Fig. 3). The numerical simulations have been carried out using Ruaumoko 3D (Carr 2010).

Each structural member is represented about its principal inertia by lumped plasticity Giberson elements behaving according to the modified Takeda hysteresis rule (Fig. 3). The moment-curvature of each element has been calculated and bi-linearized using the software Cumbia (Montejo and Kowalsky 2007). The adopted constitutive models take into account both the concrete confinement (Mander, et al. 1988) and strain hardening (King 1986). In the adopted bi-linearization the elastic stiffness is secant to the first yielding and the yielding happens when either the strain in the steel is $\varepsilon_s=0.015$ or the strain in the concrete in compression reaches $\varepsilon_c=0.004$.

The structural members are elastic (with secant stiffness to yielding) about their secondary inertia, with exception of the base columns which are allowed to yield, consistently with a frame type seismic behaviour.

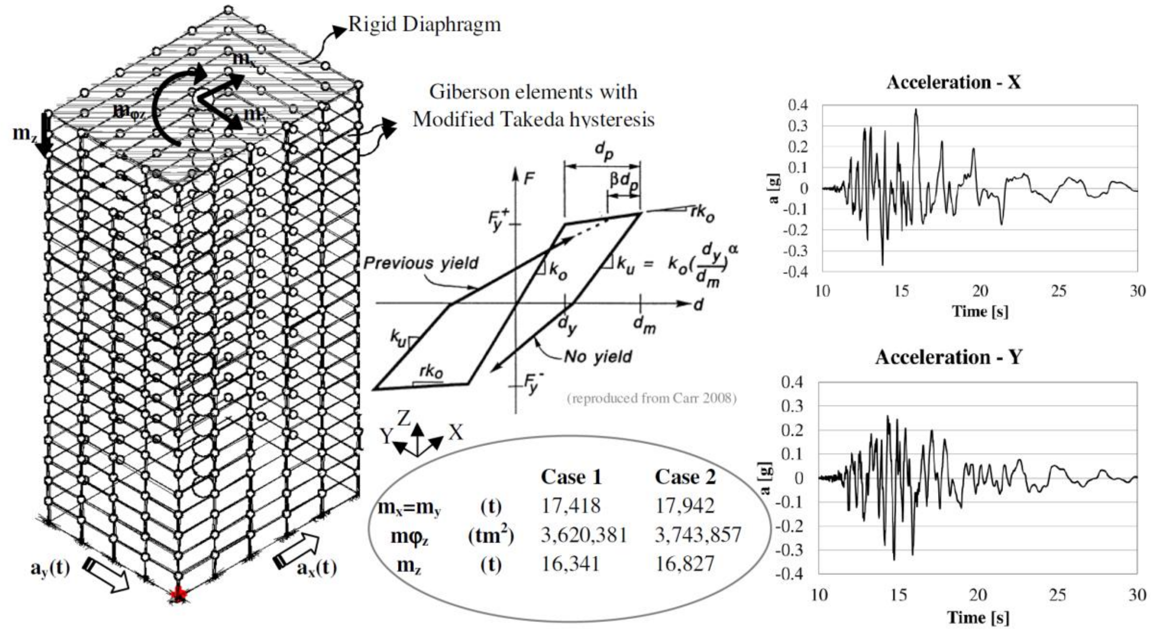
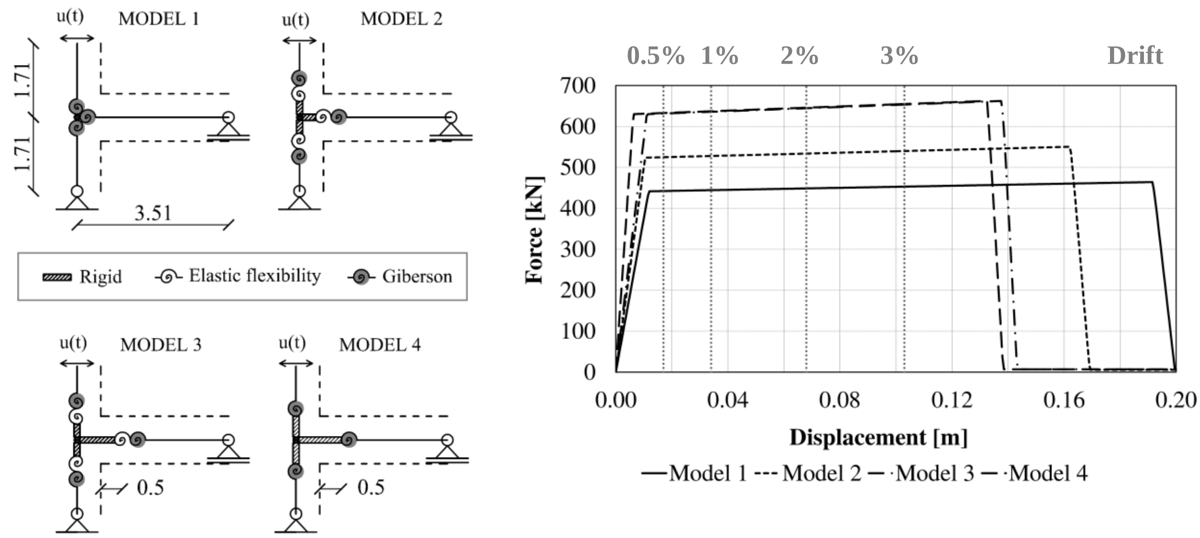


Figure 3 Sketch of the adopted numerical model including the assigned time-histories.

The floor diaphragms are modelled as rigid in plane (Fig. 3). The horizontal and rotational masses (respectively m_x, m_y and $m_{\varphi z}$) are storey by storey assigned to dummy nodes in the floor centres; the horizontal and torsional degrees-of-freedom of the seismic frames are then slaved to these dummy nodes. The vertical mass (m_z) is applied joint by joint to take into account the vertical inertia being activated by the swaying of such a tall building. The seismic weights are calculated according to the current New Zealand Standards (NZS1170.5:2004). Figure 3 summarizes the calculated masses. Case 1 considers only the tower in the calculations, while Case 2 considers some additional mass at the 2nd and 3rd storeys coming from the building podium floors (assumed to be 70% of the floor podium mass).

The elastic damping has been fixed at 5% at each main vibration mode (Wilson and Penzien 1972).



(a) (b)

Figure 4 (a) Investigated BC joint model alternatives: Model 1, Giberson from centre to centre; Model 2, rigid link over half of the column depth, elastic rotational flexibility representing ¼ of the column depth and Giberson; Model 3, rigid link up to the plastic hinge section, elastic rotational flexibility of ¼ of the column depth + the hinge relocation length and Giberson; model 4, rigid link up to the hinge relocation section and then Giberson. (b) Force-displacement diagrams of the investigated BC joints models.

With regard to the already mentioned plastic hinge relocation detail (Fig. 1d), quasi-static analyses have been performed on four BC joint configurations (Fig. 4a) to investigate the effect of such a detailing on the subassembly seismic response. The four models have increasing rigid link length to investigate the panel zone stiffness effect (model 1 to 4) and include rotational elastic flexibilities to consider h/4 of the column depth as flexible (model 2) and the flexibility of the relocation hinge length (model 3). Figure 4b shows the quasi-static analysis results: (i) the elastic stiffness of models 2 and 3 is in between the two extreme cases (model 1 and model 4); (ii) the shear transferred by the system increases proportionally to the relocation length (due to the moment given by the Giberson element shear).

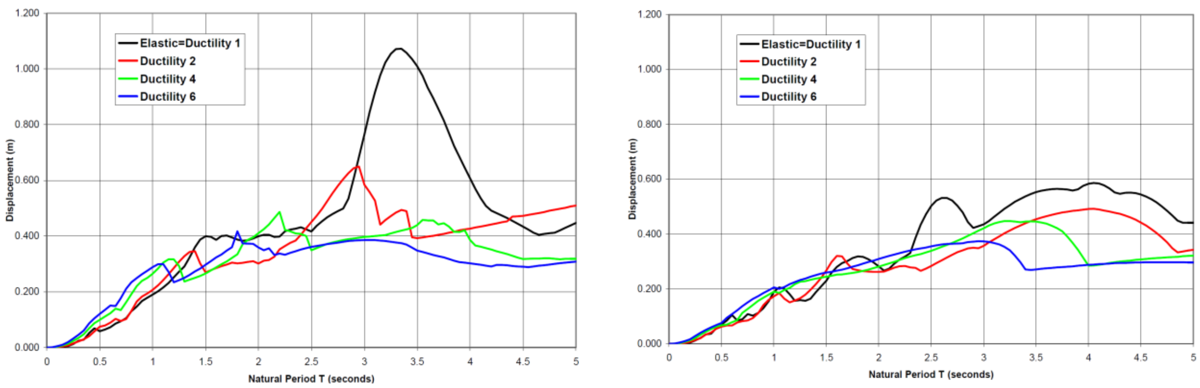
Table 2 summarizes the mechanical parameters in the investigated subassemblies. Notably, when the hinge relocation is accounted for properly (model 3), both the initial stiffness and the force yielding increase of about 50% with respect to the most flexible system (model 1); the yielding displacement tends instead to be slightly reduced. This effect cannot be neglected and model 3 has thus been incorporated in all the 3D models.

Table. 2 Summary of the beam-column joint properties.

	k_0 (kN/m)	F_y (kN)	Δ_y (m)	k_0 (%)	F_y (%)	Δ_y (m)
Model 1	3.68E+04	4.42E+02	1.20E-02	100%	100%	100%
Model 2	4.91E+04	5.24E+02	1.05E-02	133%	119%	88%
Model 3	5.52E+04	6.30E+02	1.15E-02	150%	143%	96%
Model 4	9.81E+04	6.30E+02	6.50E-03	267%	143%	54%

The numerical model has the main free vibration periods of 2.8, 2.6 and 1.8 seconds along the X, Y and about Z directions, respectively. The 90% of participating mass is reached at the 10^{th} vibration mode. The introduction of the additional mass at the 2^{nd} and 3^{rd} storeys (Case 2, Fig. 3) almost has no effect on the vibration periods. The consideration of large displacement effects slightly decreases the free vibration frequencies.

The Christchurch Hospital record (CHHC) has been used for the time-history analyses (Fig. 3). It has been selected because of soil property similarity (shallow gravel). The correspondent inelastic response spectra generated for the Modified Takeda hysteresis (Carr 2011) provide an indication of the displacement to be expected from the numerical analyses (Fig. 5a, b). The displacement varies between 400 and 600mm and from 300 to 350mm along the east-west and north-south directions respectively.



(a) (b)

Figure 5 Inelastic displacement response spectra (CHHC) generated for the Modified Takeda hysteresis: (a) East-west and (b) North-south components (Carr 2011).

5 RESULTS OF THE NUMERICAL ANALYSES

Figure 6 shows the analysis results including the deflected shape at maximum displacement, displacement and drift envelopes and the moment-curvature relationship of two selected beams. The two primary models (Case 1 and 2) have been both investigated with and without taking large displacements effects into consideration. The results of the four modelling approaches do not vary significantly as expected from the negligible initial period shift.

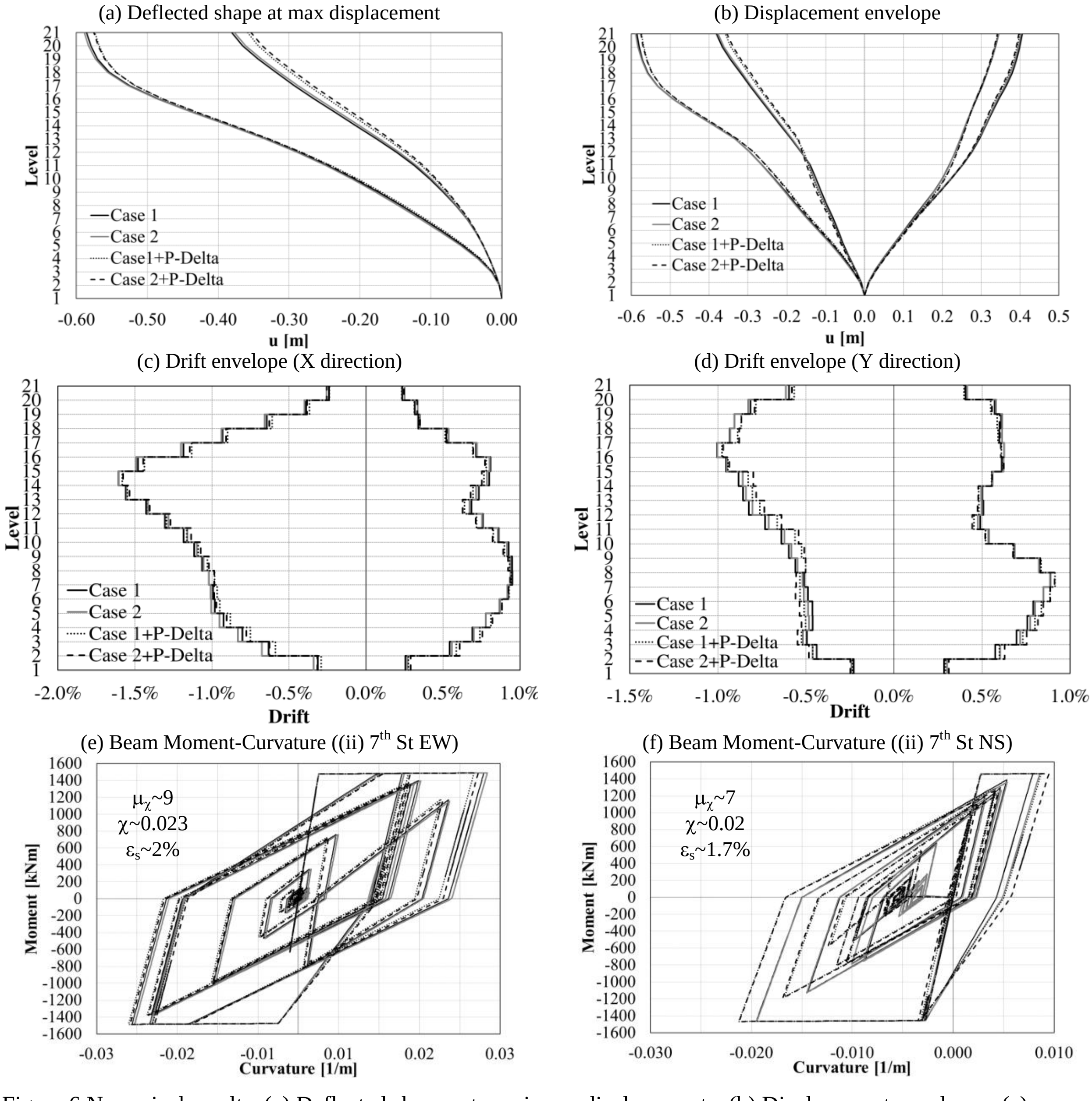


Figure 6 Numerical results: (a) Deflected shapes at maximum displacements; (b) Displacement envelopes; (c) Drift envelope along X direction; (d) Drift envelope along Y direction; (e) (f) Moment-curvature of the beams (ii) shown in Figure 2.

Figure 6a shows the deflected shape at maximum displacement. The displaced configuration resembles a wall type response; this is consistent with the normal elastic modes of the frame and suggests overstrength and larger stiffness at the lower storeys with no development of plastic hinges at the column bases during the shaking (as it would be instead expected from a typical beam-sway mechanism of a frame system).

The columns have dimensionless axial load coefficients ν ($\nu = N/(f'_c A_c)$) ranging from 0.17-0.28 at the base to 0.01-0.06 at the roof; their strength is thus affected by the axial load which contributes to create overstrength at the lower storeys. Consistently, the secant to yielding stiffness decreases from 0.5 to 0.2 of the gross inertia from the 1st column to the top one.

The amount of maximum displacement is in reasonable agreement with the expected one from the inelastic displacement spectra in both the directions (Fig. 5 a, b). However they appear to be on the lower side when compared with the actual level of damage observed in the plastic hinges. The displacements correspondent to the estimated average drifts (1.5% EW and 0.6% NS) are 900 and 400mm in the east-west and north-south directions respectively. Looking at Figure 5, while the latter could be achieved shifting the 2nd mode to 2.9s, there is not any inelastic spectrum suggesting 900mm displacement. However, the inelastic spectra shown in Figure 5 are generated for fixed-base single-degree-of-freedom (SDOF) systems.

The displacement envelope shown in Figure 6b reveals that the maximum positive and negative displacements are similar along the north-south direction but different in the principal direction.

Figures 6c, d show the drift envelopes (as obtained by this fixed-base numerical model) over the building elevation. In the east-west direction, the drift goes beyond 1% between the 5th and the 17th storeys, attaining the peak of 1.6% at the 15th storey. This damage distribution does not entirely agree with the observed damage. Although it is true that the most damaged beams were found between the 6th and the 11th storeys, the numerical model seems to underestimate the speculated maximum drift. In addition, no damage peak was found around the 15th storey.

Similar observations can be made for the north-south direction, where 1% drift is only reached at the 17th storey.

Interestingly, while the drift distribution in both the positive envelopes does qualitatively agree with the expectations, the negative envelope profiles do not, presenting opposite trends.

The moment-curvature diagram of two selected beams (i.e. (ii), Figs 2a, b) has been plotted (Figs 6e, f); during the field survey, these beams were observed to be among the most damaged. The curvature level they reached during the numerical analyses suggests that the bars are above yielding but not close to the ultimate limit state as previously speculated.

The in-site observed damage pattern is consistent with a frame type response, where the maximum drift occurs at the lower storeys and the deflected shape presents hinging at the base. However, the numerical results do not confirm this pattern. Therefore, the numerical investigation needs to be further extended to cover all the possible uncertainties which could affect the seismic response. A nonlinear flexible foundation should be added beneath the building; this will allow the consideration of the foundation-structure system period shift and the possible change in the response mechanism due to permanent deformations at the foundation level during the ground motion. The effect of axial load variation during the dynamic analysis should be taken into account; in fact, if the columns are in tension, their yielding moment would reduce drastically and trigger perhaps plastic hinges at the column bases. Consistently, the vertical acceleration component should also be added. Finally, the observed damage is the result of a sequence of events, which started in September 2010 and ended in December 2011. The consideration of the complete event series would consider the effects of structural stiffness degradation.

6 CONCLUSIONS

A 22 storey building which was significantly damaged during the February earthquake has been inspected prior demolition and subsequently numerically investigated. The damage assessment survey led to an estimation of the maximum drift demand based on the observed crack widths.

Although the maximum numerical displacement is in agreement with the inelastic design spectra for fixed-base SDOF systems, the extent of maximum drift is underestimated and the qualitative drift distribution does not entirely reflect the observed damage pattern. Where most of the damage has been observed (between the 6th and 11th storeys) a drift demand ranging from 2 to 3% has been estimated; at those storeys, the numerical drift ranges from 1% to 1.3%, yet causing, in two selected beams at the 7th storey, a strain in the steel to be above yielding (about 2%). While the numerical peak inelastic demand occurs in the top part of the building (15th storey), the observed maximum crack widths suggest a frame-type damage pattern.

Given the above mentioned results, further numerical investigations are needed to cover the limitations of the described numerical simulations. In fact, certain aspects such as the nonlinear soil-foundation interface behaviour, axial force-moment interaction, vertical acceleration and strength degradation could have affected the building seismic response to give the observed damage pattern.

7 ACKNOWLODGEMENTS

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