Earthquake response of building parts

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R.H. Shelton, S.G. Park & A.B. King

Building Research Association of New Zealand

ABSTRACT: A suite of buildings was designed in accordance with the earthquake provisions of the draft joint NZ Australia Loadings Standard. Inelastic time history analyses were carried out using the Ruaumoko 3D program, and selected earthquake records. From the output of these analyses, interstorey drifts, floor accelerations and response spectra were obtained, and a force based equation developed. This equation forms the basis of the provisions of section 9 of Part 4 of the Standard .

1 BACKGROUND

The adaptation of the earthquake provisions from the current version of NZS 4203 (SNZ 1992) into Part 4 of the joint Australian/New Zealand loading standard (SA/SNZ 2000) provided the impetus for a fresh look at the requirements for the design of parts.

In common with other international standards (NEHRP1997), (NBC 1995) (ASCE 1998), (ICC 2000), NZS 4203 adopts the conventional force-based procedure to determine earthquake design actions on parts. There is some concern among researchers that such an approach may not be a good predictor of damage to building parts. The anomaly may be due to high floor acceleration pulses of very short duration and with very small displacements, often caused by building response in the higher modes. Such phenomena have been observed in real floor response records (eg Naeim 1996), and are also seen in both elastic or inelastic analyses, but do not necessarily result in actual damage. The project did not directly address this issue, which is clearly in need of detailed investigation.

There is a perception that the parts provisions of NZS 4203 are difficult to apply particularly since they require detailed information from the seismic design of each specific building. This is a major impediment to the designer of "off-the-shelf" items that account for a significant portion of parts that are installed in new buildings.

Also, the treatment of floor accelerations where the building has been designed for overstrength is not clear. The default value of $\mu = 1.0$ used in the equation of floor acceleration to account for overstrength, which is almost universally used by designers, can be shown to result in an overestimation of floor accelerations by a factor of up to 3.

In contrast to NZS 4203, the other standards referenced above calculate a force coefficient for the part by means of a multifactor equation. Generally such equations contain terms quantifying the maximum ground acceleration for the site, the response of the building (depending on period), the response of the part (depending on flexibility or ductility), and a risk factor for the part. To complete the Newtonian analogy, the coefficient (which effectively is acceleration) is then multiplied by the operating weight of the part.

2 PROJECT OVERVIEW

For completeness the overall project may be summarized below, with a more detailed account given by King et al (2002):

A suite of buildings was designed in accordance with the earthquake provisions of the draft joint Loading Standard (SA/SNZ 2000). The buildings consisted of 3, 10 and 20 storey buildings in reinforced concrete and steel, and their design and issues associated with them are described by Bell et al (2002). The concrete buildings had a moment resisting frame in one direction and shearwalls in the other, while the steel buildings also had moment resisting frames, with eccentrically braced frames in the other direction. Both the shearwalls and braced frames were offset from a symmetrical position to introduce eccentricity into the building structure. The buildings were designed for shallow and deep soil sites in Wellington and shallow soil sites in Auckland, at the limited ductile and fully ductile response levels (where possible) within the specified minimum strength provisions of the draft standard (SA/SNZ 2000).

Three ground-motion records were selected for each site at each of the ultimate, half ultimate, and serviceability limit states under contract by Geological and Nuclear Sciences. The process of selection and scaling to match the hazard spectra specified by the draft standard (SA/SNZ 2000) are described by King et al (2002).

Each building was then subjected to a time history analysis, using as input, the chosen ground motions, scaled as described above. Analysis was undertaken using Ruaumoko 3D programme (Carr 2001), and incorporated actual material properties rather than ideal, Pdelta effects, and masses offset 0.1 times the building width in the appropriate direction.

Purpose written extraction software was then used to generate building deflection profiles and interstorey drifts, floor accelerations and response spectra for selected floor levels, as described later in the paper.

The following paragraphs discuss each of the critical issues considered in processing the results of this project into clauses suitable for inclusion in the Parts section of the draft Standard.

3 RISK FACTOR/CONSEQUENCES OF FAILURE

The failure of a non-structural building part under earthquake actions may have varying consequences. For example, the collapse of the library shelving of Figure 1 is likely to be a hazard to fewer people than will the falling of the cladding panels onto the footpath as shown in Figure 2.

The criteria used for the classification of parts, and assigning the risk factor, R_p, were :

- Parts representing a hazard to crowds, or parts able to fall more than 3 metres onto an accessible area. (For example, an auditorium ceiling or cladding panels over a footpath).
- Parts representing a hazard to individuals within the building, or those necessary for the continuing function of life safety systems. (Library shelving, or medical gas lines)
- Parts required for operational continuity, or whose failure would have disproportionate consequences. (For example, a cool store chiller, or a leaking water pipe) In these cases the risk factor may be a commercial decision requiring input by the owner.
- Other parts.

Figure 1. Nisqually earthquake.

Figure 2. Kobe earthquake.

In contrast, the consequences of failure to the community is a function of the building itself, and is covered separately by the building return period factor, R; thus the effect on the design of the part is cumulative.

4 ACTIONS ON PARTS

Under earthquake induced action, a part may be required to resist either a horizontal or vertical inertial force, or a force induced by the deflections of the primary structure if the part is supported at different levels, for example, a riser pipe, or cladding panel.

Of most relevance to this paper, is the equation proposed to determine the horizontal force. It may take the form:

$$F_{ph} = C(0) C_f C_p R_p W_p \,, \tag{1}$$

where:

- $C(0)$ is the site hazard coefficient, with period T = 0,
- C_f is the floor acceleration coefficient,
- C_p is the part response coefficient,
- R_p is the part risk factor,
- W_p is the weight of the part.

The factors making up the equation are described below.

5 GROUND MOTION

It is necessary to have a suitable reference point for the calculation of the forces on a part. The maximum value of the input ground motion record (effectively the peak ground acceleration) is one possible parameter, and was used by Rodriguez et al (2000) in a study of floor accelerations. However this is a variable quantity depending on the record chosen as input to a time history analysis. A well defined value, readily available to the designer, is the level of earthquake hazard at the site, defined by the loading standard as the elastic site hazard coefficient at zero period, C(0) for the appropriate return period.   This is determined from

Section 3 of the draft standard as the product of the spectral shape factor, $C_h(0)$, the seismic zone factor Z, and the return period factor R for the given importance category of the building. This is as defined in equation 3.1 in the draft standard, with the near fault factor N(T,D), equal to 1.0 for zero period.

6 BUILDING RESPONSE

One of the main objectives of the project was to determine floor accelerations and interstorey drifts of a range of representative buildings. The analysis procedure has been briefly described above, and the full list of buildings with the pertinent features of each is detailed in King et al (2002).

6.1 *Data extraction*

Processing the outputs of the numerous time history analyses proved quite a challenge because of the very large amount of data involved (nearly 900 MB of data file per analysis run).

To automate the task as far as possible, purpose written software was used to interrogate each output file and extract the relevant data.

6.1.1 *Displacements*

Displacements for each corner node, at each time step were extracted from the analysis output file, and the maximum interstorey drifts in the two orthogonal directions, plus the vectorial resultant, were computed and stored in a text file. This information will be available to verify the procedures proposed for computing the building's interstorey drifts based on the output of an equivalent static, or modal response spectrum analysis.

Deflection profiles at various times were also obtained at the same locations, although need not be discussed in the context of building parts.

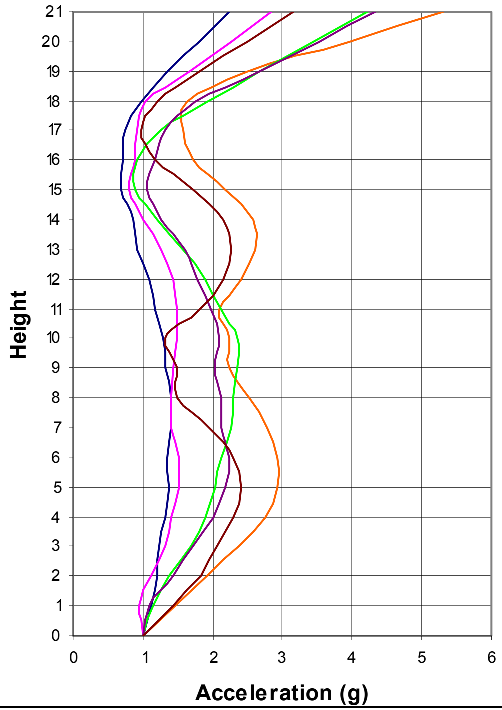
6.1.2 *Accelerations*

Relative accelerations (x and y and z rotation) at each analysis time step were extracted from the analysis outputs for a central node at each floor level. Ground accelerations were then added to obtain total accelerations, and the accelerations at the corners then calculated using the expression:

$$x_{t,corner} = (x_g + x_r)_{center} \pm z_{g,center} * r \qquad (2)$$

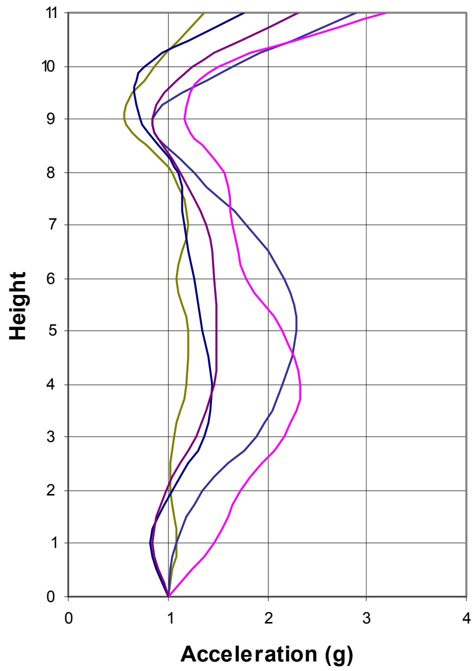
where x = acceleration in x direction; z = rotational acceleration; r = distance to corner; and subscripts equate to the t = total, g = ground, and r = relative accelerations.

The maximum floor accelerations so calculated were plotted against the height of the building to give a visual representation of the amplification of the ground motion by the building. Figure 3 gives a typical example for each height of building studied in the project. The six lines on each figure represent three earthquake records, each with its principal component applied in the two orthogonal directions. To obtain the building amplification independently of the input motion, the acceleration values were normalized against the elastic site hazard coefficient as discussed above.



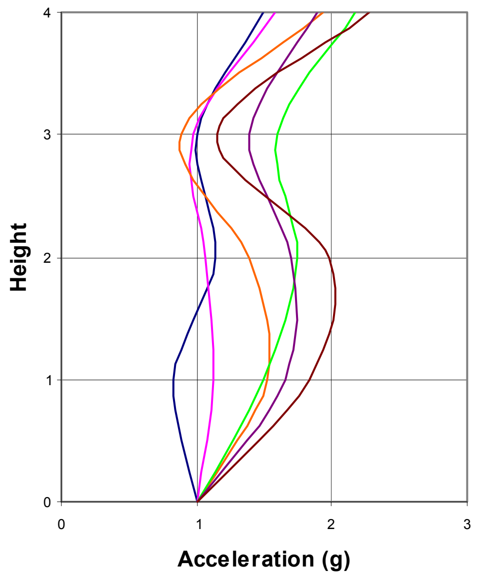
20 storey reinforced concrete building,

μ = 3, Wellington, Class C soil.



10 storey reinforced concrete building

μ = 6, Wellington, Class C soil.



3 storey reinforced concrete building

μ = 3, Wellington, Class C soil.

Figure 3. Floor accelerations for example buildings.

The plots of Figure 3 may be compared with the data shown on Figure 4 reproduced from Drake and Bachman (1995). Each dot represents the peak acceleration recorded in one of 150 Californian buildings subjected to peck ground accelerations greater than 0.25g in one of 16 earthquake events between the 1971 San Fernando Earthquake, and the 1994 Northridge Earthquake. The accelerations are normalized to the peak ground acceleration recorded at the

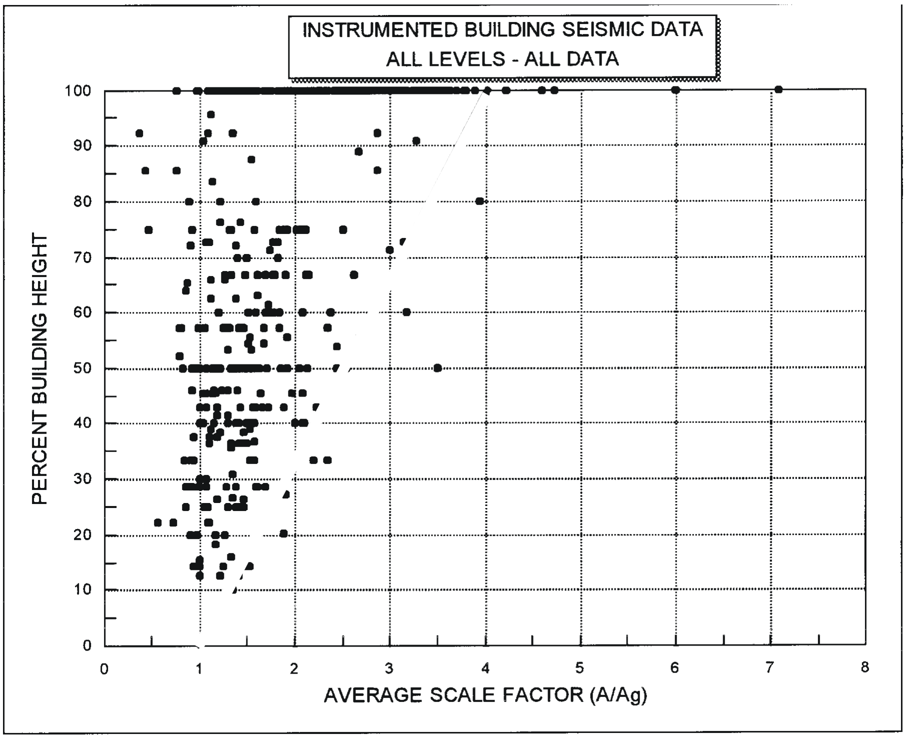


Figure 4. Recorded floor accelerations. From Drake and Bachman (1995)

site during the same event. The similarity with the calculated data on Figure 3 is apparent, although direct comparisons are not possible because nature of each building is unknown and the normalizing method is slightly different.

The equations proposed for floor acceleration coefficient in the draft standard approximately envelope the data from Figure 3 and similar plots, although this work is not yet completed.

7 PART RESPONSE

Using a force based approach, the response of non-structural parts to the building floor motions is most easily characterized by acceleration response spectra.

Purpose written software was used to calculate elastic response spectra at selected floor levels, and selected levels of damping. The software used the nodal accelerations extracted from the analyses outputs as the input to the equations of motion. Figure 5 shows the resulting elastic response spectra, calculated with 5% damping, for a 10 storey reinforced concrete building designed for a Wellington intermediate soil site (Class C). The plots on the left show action in a direction parallel to the shear walls, and those on the right parallel to the moment resisting frames. The six lines on each plot represent the three earthquake input records, with the principal component applied in the two orthogonal building directions. The vertical lines are the building periods from the first to the 4th mode.

This result may be compared with the spectra of Figure 6, reproduced from Naeim (1996). These spectra were computed (at 5% damping) from floor accelerations measured at different levels in a range of instrumented buildings during the 1994 Northridge earthquake. The report was obtained from the Strong Motion Instrumentation Program (SMIP) run by the State of California, Department of Conservation, Division of Mines and Geology.

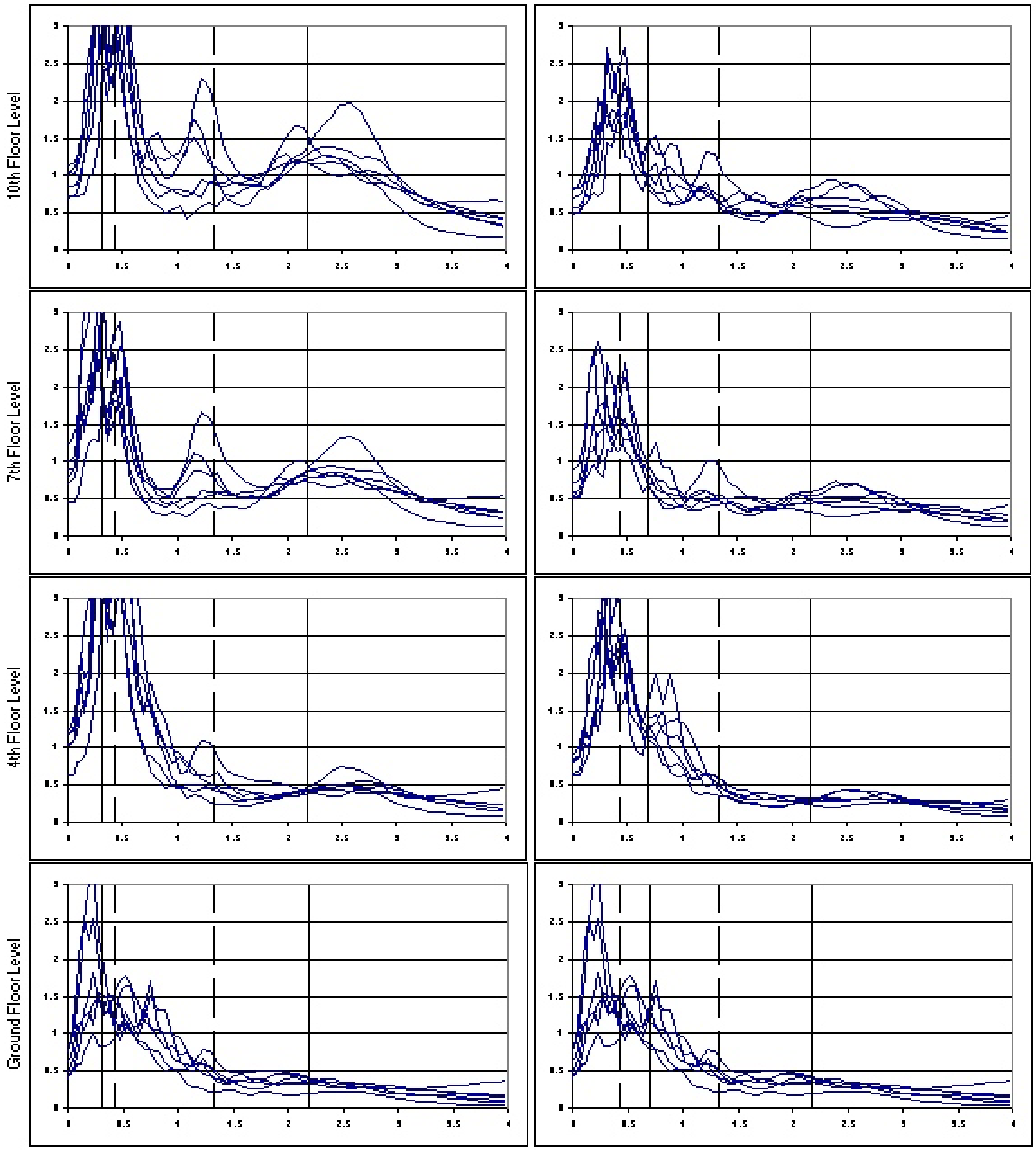
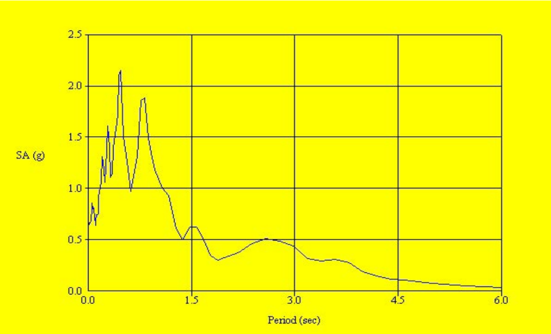
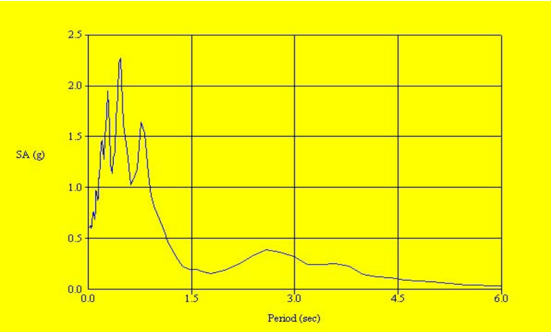


Figure 5. Floor response spectra.

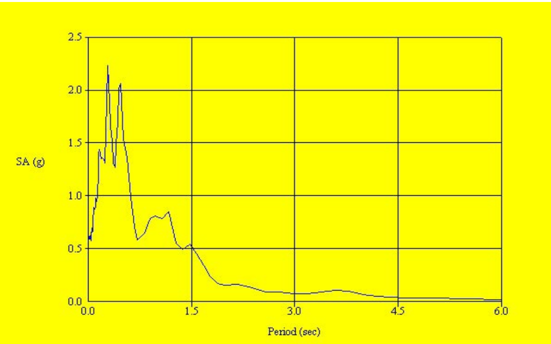
The same trend is evident in the two figures, although the Northridge examples have lower accelerations because the ground motions were less intense than the hazard level used for the current study. In particular, the lack of "resonance" at the building first mode period is absent from both sets of spectra. This is an example of how the yielding of the building structure, particularly in the frame direction, has damped out much of the response at the first mode period, thus accentuating the response at higher modes and a proliferation of short period acceleration spikes that appear typical There is some evidence that the correlation between accelerations determined from elastic floor response spectra and actual damage to building parts is very tenuous, although comparative studies are few. Such lack of correlation is shown in Figure 6, where accelerations in the order of 2g did not produce any significant damage to non-structural elements. The explanation may lie in the very short duration of the motions producing these peaks, and the displacements associated with them are also very small. Considerable work on this aspect of the project remains to be done, but until this is more advanced, the proposals put forward to the draft standard incorporate an equivalent of the "S_p" factor to account for the discrepancy.



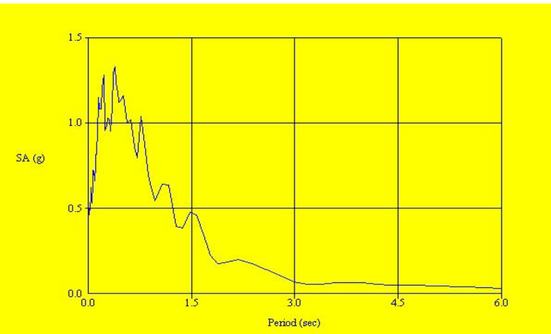
Roof (14th Floor)



8th Floor



2nd Floor



Basement Level

Acceleration response spectra

- Built in 1965 with the first floor spandrel girders modified by post-tensioning after the 1971 San Fernando earthquake.
- Moment resisting concrete frames in both directions.
- The geology of the site is alluvium.
- The building is situated 9 km from the epicenter.
- Plan dimensions of 209 ft x 125 ft.
- Structural members (beams, columns, walls and floor diaphragms) experienced moderate damage in the form of cracking.
- Damage was repaired by epoxy grouting.
- No damage to non-structural elements such as pipes, HVAC, air conditioning.
- The natural period of the structure in the direction described by the graphs opposite is about 2.6 seconds.

Figure 6 Performance in Northridge Earthquake.

8 CONCLUSIONS

A high level of understanding of inelastic behaviour of buildings designed to the draft standard has been achieved, and techniques developed from progress so far on this project, and some recommendations have been submitted for Part 4 of the draft joint loading standard. However much work remains to complete the project, to achieve the level of robustness required.

9 ACKNOWLEDGEMENTS

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