13th World Conference on Earthquake Engineering
Vancouver, B.C., Canada
August 1-6, 2004
Paper No. 1920

SEISMIC CAPACITY OF BUILDINGS IN SINGAPORE DESIGNED PRIMARILY FOR GRAVITY LOADS

T. BALENDRA[1], M.J. PERRY[2], J.L. WILSON[3] and N.T.K. LAM[3]

SUMMARY

The recent Bengkulu earthquake (4 June 2000) was reportedly felt in many parts of Singapore. This earthquake, with a moment magnitude of 7.7 and centred 700km away, raised questions about what sort of threat a major Sumatran earthquake could pose to the buildings in Singapore.

In this study the seismic capacity of a 16-story frame-wall structure is investigated with the aim of determining the threat that a 'worst case' Sumatran earthquake could pose to buildings in Singapore. The structure has been designed for gravity loads only, with no provisions for seismic effects. The dynamic analysis software Ruaumoko is used to model the structure in three-dimensions. Conventional pushover and full dynamic time-history analyses are carried out to determine the ultimate base shear demand and failure mechanism of the structure. A comparison of the results obtained in the two analyses is also conducted to assess the suitability of using a pushover analysis to estimate the seismic capacity of such structures.

Results of the time-history analyses indicate that failure of the structure would occur due to shear failure in the wall members. The total base shear demand at failure of approximately 3% of the dead load of the structure corresponds to twice the demand due to the identified 'worst case' earthquakes. It is therefore found that the building possesses sufficient lateral strength to survive such an event.

It was found that due to a large twisting of the structure the inertial moments induced in the structure were substantial and had a large contribution towards the failure of the structure. The conventional pushover study, unable to represent these moments, therefore resulted in a gross overestimation of the structures capacity. A modified pushover analysis was proposed in which loading was applied as both a lateral load and a moment. In this way the effects of the significant twist component could be well approximated and a building capacity close to that obtained in the time-history analysis resulted.

[1] Department of Civil Engineering, National University of Singapore, Singapore 117576, cvebalen@nus.edu.sg, Tel: 65-68742158, Fax: 65-67791635
[2] Department of Civil Engineering, National University of Singapore, Singapore 117576
[3] Department of Civil and Environmental Engineering, University of Melbourne, Australia

INTRODUCTION

Singapore is located on a stable part of the Eurasian Plate, with the nearest fault in Sumatra about 400km away. It has been recognised however that buildings located far from earthquake sources may also be affected by earthquakes. The 1985 Earthquake that struck the coast of Mexico caused massive damage and loss of life in Mexico City, 350km from the epicentre. Singapore has never suffered any major damage in an earthquake, however it must be kept in mind that there have been no huge earthquakes in recent times, and the two greatest earthquakes in Sumatra both occurred over 100 years ago, a time when Singapore was without high-rise structures and reclaimed land.

Two possible sources of major earthquakes exist for Singapore. The first is the Sumatra fault that lies about 400km from Singapore at its closest point. This is a strike slip fault and it has been estimated that the maximum earthquake which would occur at this location would correspond to a $M_W = 7.8$ earthquake. The second is the subduction zone at the java trench approximately 600km from Singapore. This is where the Indian-Australian plate subducts under the Eurasian plate. The 1833 Sumatran subduction earthquake has been estimated at about $M_W = 8.9$ and this is used as an estimate of the largest likely event. In this study both these possible events have been considered.

There are several factors that could contribute to the potential damage to buildings in Singapore caused by these earthquakes. These factors are mostly related to the frequency content of the waves and the fundamental periods of the sites and buildings in Singapore. High frequency earthquake waves are damped out quickly as they move away from the source. Because of this any earthquake reaching Singapore is likely to have a very large proportion of low frequency waves. Thus the waves even with low peak ground acceleration could be quite destructive due to the large displacements. It has also been shown by Balendra et al [1] that Singapore soils have a large amplifying effect on these low frequency vibrations. Thus small vibrations at the bedrock may be amplified by about 10 times as they propagate to the surface. In addition the period of these sites, typically 0.7-1.6 seconds corresponds to the fundamental period of the buildings under study. This means severe vibration can occur due to multiple resonances developed in the building and soil.

Currently there is no requirement to consider seismic forces for design of buildings in Singapore. Wind loading corresponding to gusts of up to 32m/s or a notional load of 1.5% of the dead weight is commonly used to design for lateral stability. It is assumed that this design will adequately take care of any earthquake loads. For this reason a study of existing buildings must be carried out in order to determine the behaviour of such buildings if subject to earthquake motions. Although many studies have been carried out previously, very few consider full non-linear dynamic analysis of the structure and fewer still consider the building in three dimensions. This study is therefore important to determine how existing buildings in Singapore could be expected to perform if an earthquake were to occur.

The main objective of this study is to determine the risk that a large Sumatran earthquake could pose to the gravity load designed buildings in Singapore. This involved determining the seismic capacity of the structures compared to the demand imposed by the earthquakes identified. An asymmetrical 16-storey housing development board (HDB) building, designed and built in Singapore in the late 1960's has been considered. The building has been modelled in three dimensions and analysis carried out using the computer program Ruaumoko 3D [2]. It is also of interest to compare the results obtained from a three dimensional non-linear dynamic time-history analysis to those obtained from the more commonly used pushover type analysis, to determine if there exists any real benefit in performing full dynamic analysis when analysing these types of structures.

EARTHQUAKE MOTIONS

Bedrock motions that have been previously obtained through simulation [1] of the above mentioned 'worst case' earthquakes have been used. The two cases mentioned above were considered. These bedrock motions were converted to surface motions using the program SHAKE91 [3]. Local soil properties at Katong Park were chosen to correspond as closely as possible with the fundamental period of the building under test to allow for amplification of the critical frequency waves. The soil profile at the site comprises of more that 40m of soft clay overlain by a layer of loose sand. The relationships used for the soil shear modulus follows the work done by Hardin and Drnevich [4], and for damping follow Poulos [5]. Values suggested by Lam and Wilson [6] and Imai and Tonouchi [7] have been adopted for shear strain, damping and maximum shear modulus. The surface accelerograms and their respective spectral accelerations for the case of 5% damping are shown in fig 1. These accelerograms were then used as input to the time history analysis.

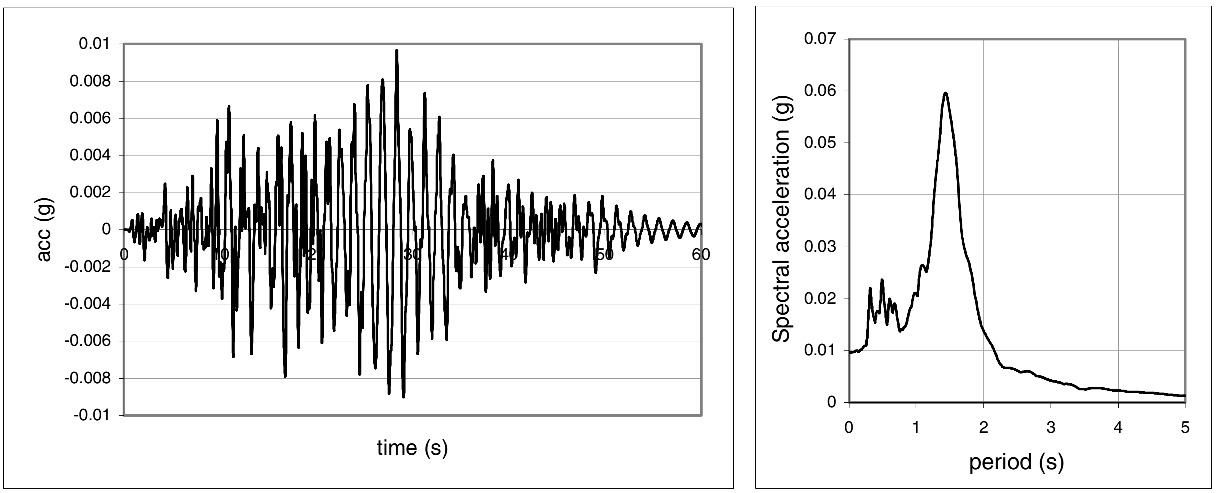

Fig 1a. Surface Accelerogram for M_W = 7.8 at 400km

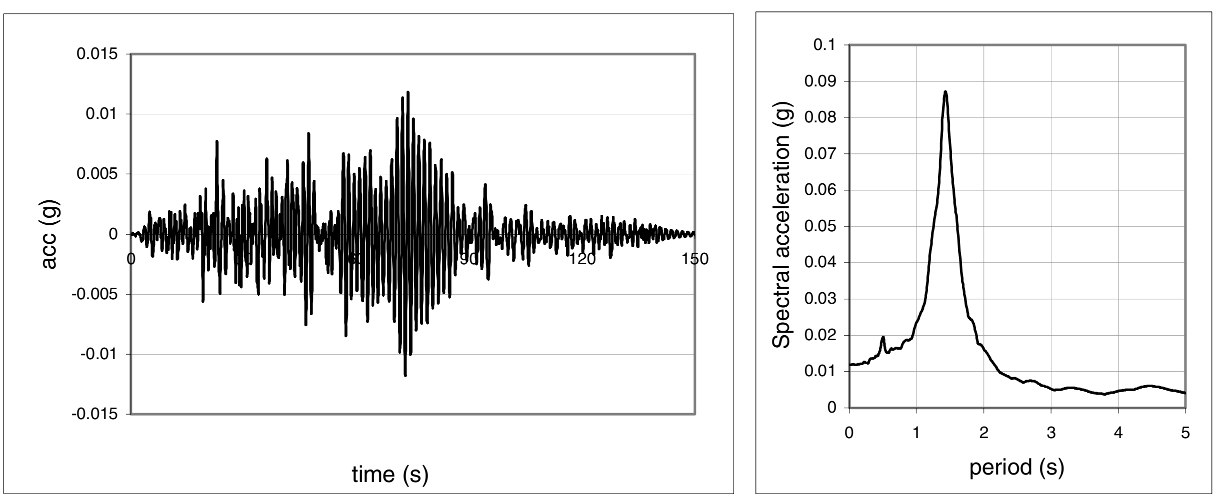

Fig 1b. Surface Accelerogram for M_W = 8.9 at 600km

MODELLING OF STRUCTURE

The building studied is a 16-storey asymmetrical concrete frame-wall structure. The layout of a typical floor of the building is shown in fig 2. It can be seen from figure 2 that the major source of lateral stiffness is likely to come from the walls at the right side of structure. It is also noted that the asymmetrical nature of the building may lead to a large twisting action of the frame under lateral load, an aspect of behaviour that would not be captured by a 2D analysis.

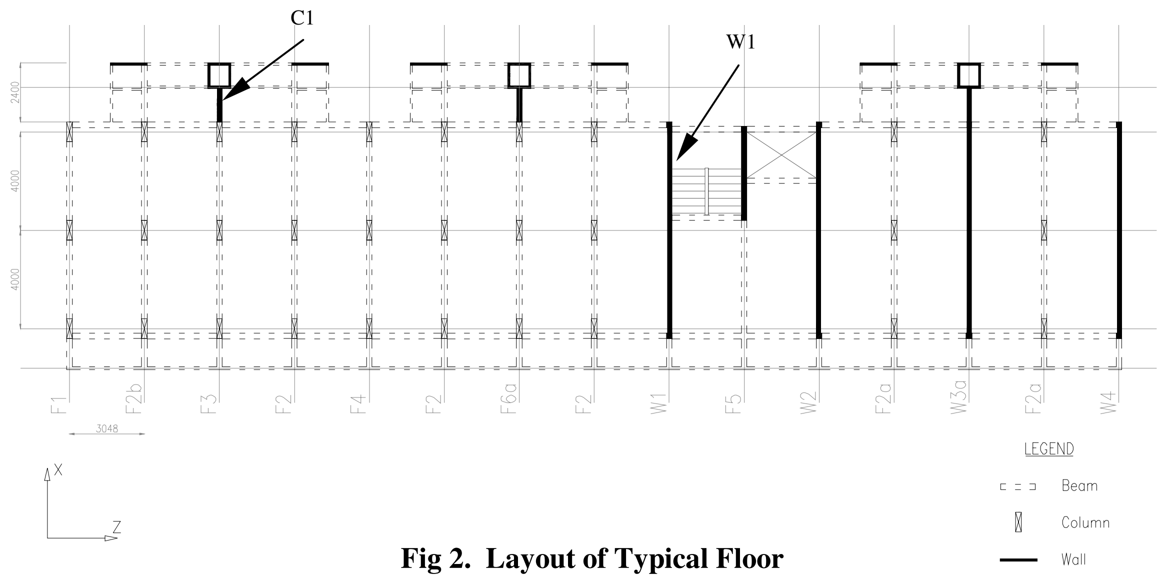

Fig 2. Layout of Typical Floor

The structure is modelled in three-dimensions using Ruaumoko and aims as far as possible to resemble the actual structure, while at the same time eliminating small details that will only add to computational time and effort. With this in mind the model has included all of the major beams, columns and walls in the structure but has eliminated or combined some of the smaller details, such as minor transverse beams which will have little effect on the response of the structure. Beams and columns in the structure have been represented by frame type, beam or beam-column members. Shear walls have been split into sections and modelled as vertical frame members connected horizontally by rigid links. This approximation of the walls allows for easier member connections while still preserving approximately the actual behaviour of the wall. The nodes on each floor have been slaved for x and z translation and y rotation. This is to simulate the effect of the floor slab at each level. In the pushover analysis the lateral load at each floor is applied at the master node at the approximate centre of mass of the floor.

In the model developed for the structure 113 different section properties were defined to represent each different cross section present in the structure. The definition of each cross section involved stiffness and strength parameters, basic member properties, hysteresis rules, yield surfaces, and strength degradation. The shear capacities of the members were not input into the program but were instead checked against the output values of shear demand in the members to determine if shear failure was taking place. Calculations of bending and shear strengths were based on BS8110 with the removal of partial safety factors. The stiffness and strength properties were based on a concrete compressive strength of 30MPa and reinforcement steel strength of 460Mpa for longitudinal steel and 250MPa for transverse steel. In the calculation of yield surface the steel strength is multiplied by a factor of 1.1. This is to account for the fact that steel strength is likely to be slightly higher than specified and also to account for strain hardening of the steel. The yield surface for beams is assumed independent of axial load whereas for columns the axial interaction is included by way of a quadratic beam-column yield surface. Degradation of the strength of the members is included in the analysis to reflect the damage that would occur in the members when the yield level of resistance is exceeded. The amount of reduction is based on the maximum ductility of the member. In this case, as the building under study is not designed for seismic loads, it is likely to have very large reduction in strength at low levels of ductility. Gupta et al [8] found that members with nominal ductility could experience a large loss in strength of 55-60% at reasonably low ductility levels. The variation of strength with ductility level adopted in this study is based on this finding and is shown in fig 3. Stiffness degradation is included using a modified Takeda hysteresis as shown in fig 4. Very little work has been done to identify appropriate hysteresis values for buildings of limited ductility. Values adopted here are therefore based loosely on the limited amount of other work that has been done in this area. It has been shown [9] however that for structures with large periods such as the one under study any error in estimation of these values are unlikely to have substantial effect on the structural response. The values adopted in table 1 are based on the works by Otani [9], Harries et al [10] and Filiatrault et al [11]. Constant damping of 1% of critical damping was also applied to the structure.

Table 1. Hysteresis parameters

	Beams	Walls/Columns
Unloading Stiffness, α	0.4	0.3
Reloading Stiffness, β	0.4	0.6
Bilinear factor, r	0.2	0.2

Mass and load are applied separately to the structure in Ruaumoko, the load being involved in creating stresses in the members and the mass contributing to inertia effects. Gravity loading has been taken as the common load case of the dead load and 40% of the imposed loads. These vertical loads are applied directly to the structure as uniformly distributed loads. Mass corresponding to the gravity load is also applied to the structure. Lateral loading is applied to the master node at each level for the case of the pushover analysis and as ground level accelerations for the dynamic time-history analysis.

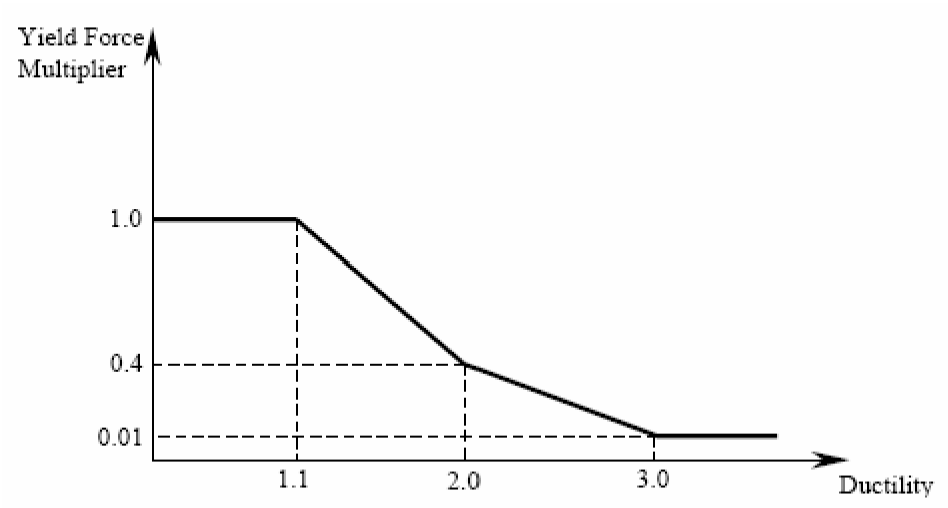

Fig 3. Strength Degradation

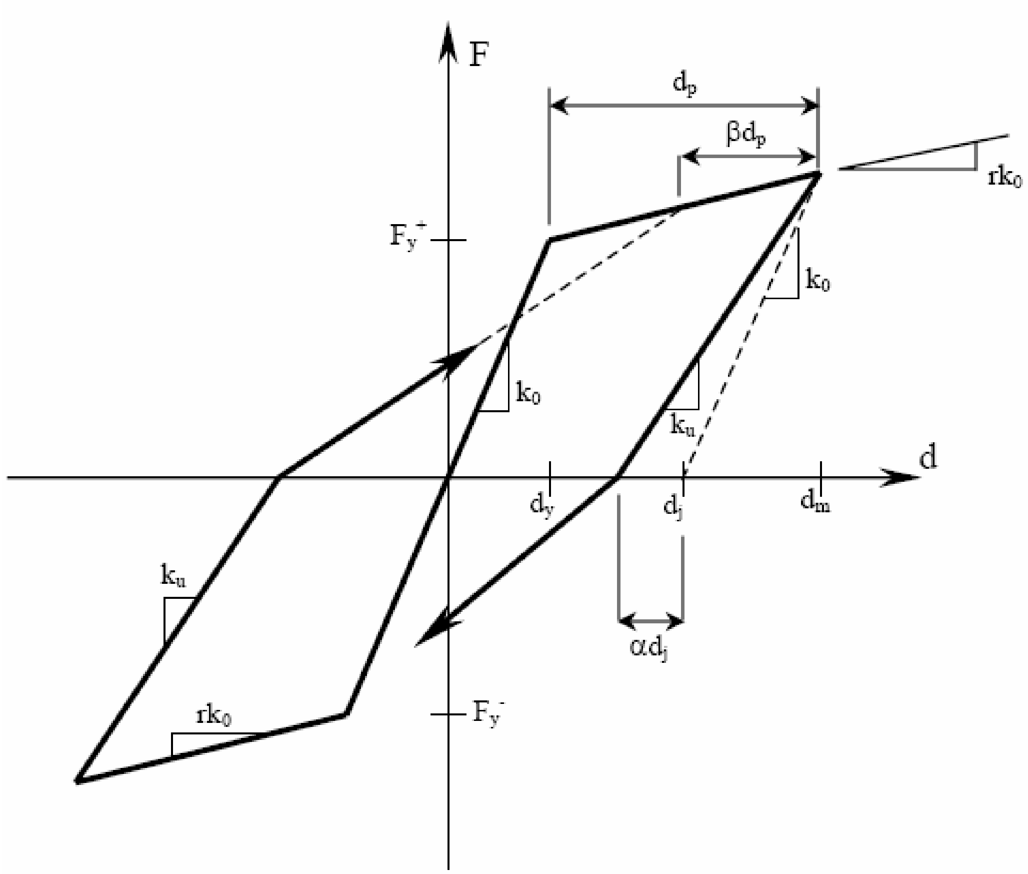

Fig 4. Modified Takeda Hysteresis

METHOD OF ANALYSIS

The analysis for this study is conducted using Ruaumoko in three main parts, namely a modal analysis, a pushover test and full dynamic time-history analysis. The results of the modal analysis showed the building to have a natural period of 1.77s in a coupled rotation and translation mode. This was due to the fact that the more flexible 'column end' of the structure moved more easily than the more rigid 'wall end' of the structure.

The program can carry out the time-history integration using either the Newmark constant average acceleration method or the explicit central difference method. In this study the Newmark constant average acceleration method has been chosen. This method has been shown to be unconditionally stable and has the advantage that not all degrees of freedom need to have an associated mass. The Newmark scheme used in the program has been modified from the original incremental equilibrium method to an equilibrium approach to ensure equilibrium is maintained at each time step.

The distribution of load in the pushover analysis is in accordance with the Australian standard loading distribution (AS1170.4) given as

$$F_x = \frac{G_{gx} h_x^{\ k}}{\sum G_{gi} h_i^{\ k}} V_b \qquad \begin{array}{ll} k = 1.0 & ; \quad T \le 0.5s \\ k = 1.0 + 0.5(T - 0.5) & ; \quad 0.5 < T < 2.5s \\ k = 2.0 & ; \quad T \ge 2.5s \end{array}$$

Where G_{gx} is the gravity load at storey x, h_x is the height of storey x above the base, and k is a parameter dependant on the fundamental period of the structure as given above. The loading distribution obtained for loading of 1% of total load is shown in fig 5. In the analysis this loading is scaled by applying a slow ramp loading function until failure occurs.

The lateral loading in the dynamic time history analysis was applied as input ground accelerations as described previously. In the analysis the acceleration ordinates were scaled and the analysis was rerun many times until failure occurred. In this way an idea of the seismic capacity with respect to the given input earthquake can be established. The integration time step of 0.01s used for the analysis was verified as adequate by convergence tests involving smaller time steps. The difference in the results obtained using smaller time step was insignificant.

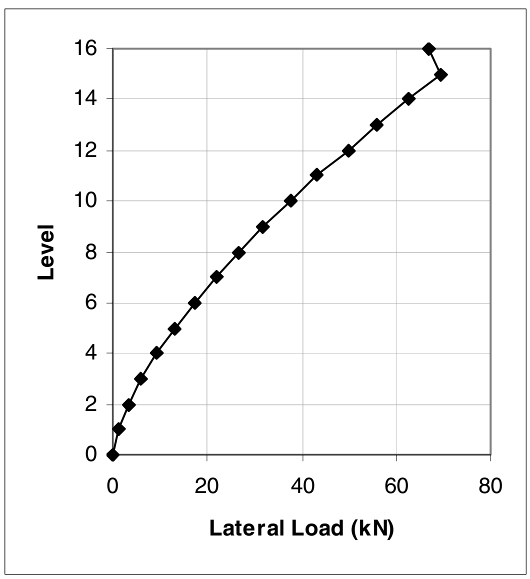

Fig 5. Loading shape for pushover analysis

In this study failure of the structure is identified when one of the following cases occurs

- A mechanism forms so that the structure becomes unstable (pushover analysis only)
- Local shear failure (based on BS8110) occurs in one of the wall or column members.
- Overall or inter-storey drift exceeds a limit of 2%.

Drift is limited to 2% to limit excessive P-Δ effects. In addition a large displacement analysis option was adopted in both the pushover and dynamic analyses. With the large displacement option the nodal coordinates are updated at every time step and the stiffness of the members are recalculated based on the changes in member forces and geometry.

RESULTS AND DISCUSSION

Pushover Analysis

The pushover analysis was performed for loading in both the positive and negative x directions. Failure was first identified as shear failure in the wall members C1 and W1 for loading in the positive and negative directions respectively. These members are indicated in fig 2. The loading at failure was around 6.9% of the dead load in both cases. The main feature noted during these tests was the large twist component that was observed due to the asymmetric nature of the structure. The ratio of twist (θb) and displacement (Δ) as shown in fig 6 was 1.03 in both loading cases at 3% dead load demand.

Dynamic Time-History Analysis

The base shear demand of the structure under the simulated earthquakes was found to be 1.67% and 1.38% of the dead load respectively. At this level of loading no failure of the structure occurred. The accelerations were then scaled to increase the demand and the analysis rerun. Failure was found to occur first in the same C1 and W1 members. For the 400km motion the failure occurred at a base shear demand of 3.30% corresponding to a scaling of 2.1 times the original motions. In the case of the 600 km earthquake the base shear demand at failure was 3.07% with corresponding scaling of 2.23 times. As in the case of the pushover analysis the twist component dominated the motion, with twist to displacement ratios of 1.60 and 1.61 for the earthquakes at 400km and 600km.

The load displacement behaviour can be viewed best graphically. Fig 7 shows the behaviour of the master node and a node located at the extreme left edge of the structure at the roof level. The points of first shear failure are shown in the figure as a diamond. It is clearly seen from the figure that although the pushover analysis seriously overestimated the structures capacity it does give a reasonable approximation of the displacement demand at failure. It is also noted that the twist component is much larger in the time-history analyses than in the pushover analysis.

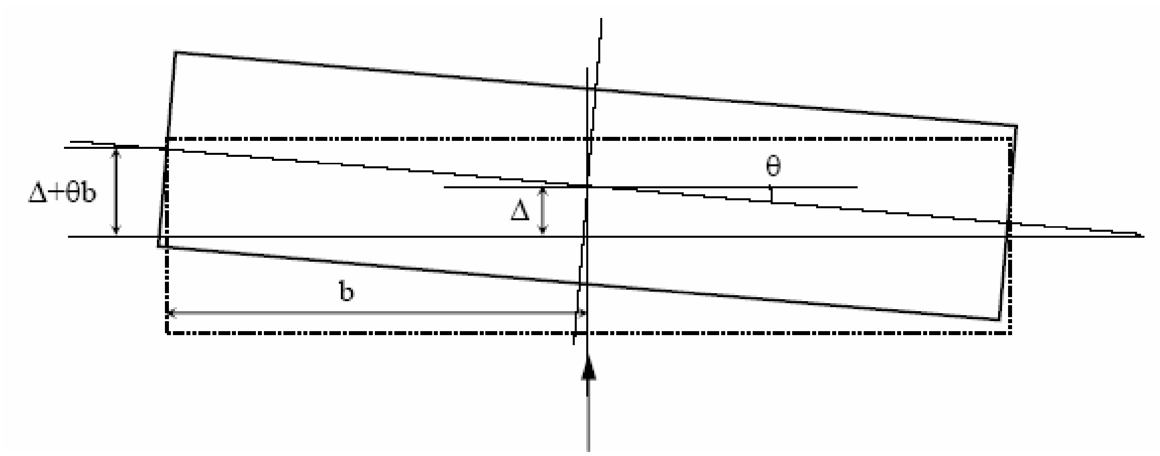

Fig 6. Twist-Displacement Relationship

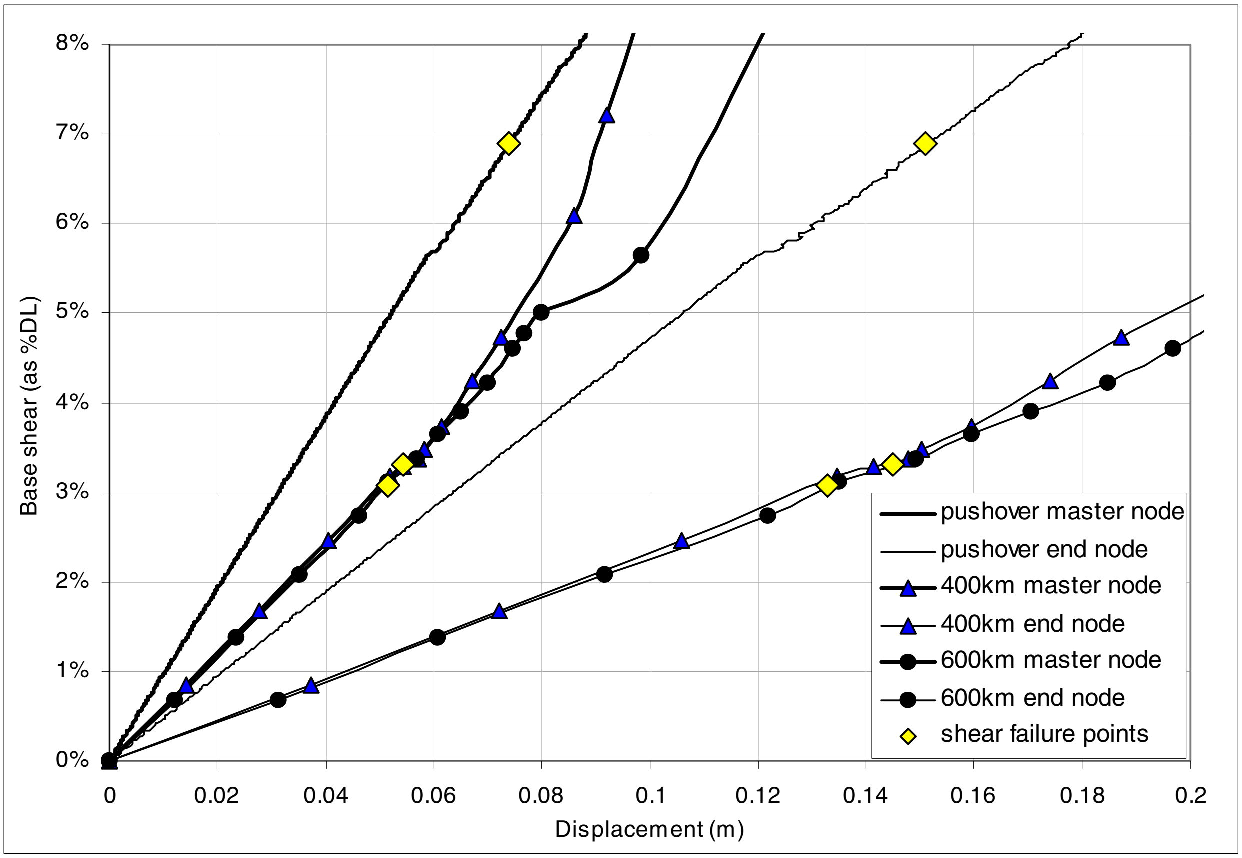

Fig 7. Load Displacement Behaviour

To investigate this increased twisting under dynamic loading the inertia forces and moments occurring in the structure were considered. In order to do this the linear and rotational accelerations at each floor level were multiplied by the floors mass and mass moment of inertia. The result for a snapshot taken for the case of the 600km earthquake scaled 2 times at maximum displacement is shown in fig 8. It is noted that the shape of the inertia forces closely matches the shape of the loading applied in the pushover analysis (dotted line), but in addition to this loading there is a large inertial moment. This moment can be thought of in terms of an eccentricity of load. If the moment at each floor is divided by the corresponding force an effective eccentricity of approximately -14.8m is obtained. This suggests that in theory if the structure is loaded 14.8 m towards the left of the master node the resulting displacements should be similar to those obtained using 3-D time history analysis.

Modified Pushover Analysis

In order to verify the above idea a modified pushover analysis was conducted. In this analysis a force and a moment were applied to the master node at each level. The force was, as before specified by the Australian code while the moment was simply taken as -14.8 times the applied load. Failure was identified in this case at 3.22%, very closely matching the result obtained in the time history analyses. The load displacement behaviour also closely resembled that of the time history analysis as is shown in fig 9. This result shows that if we can find a way of estimating the effective eccentricity we can then predict the failure load without the need for time consuming full dynamic analysis.

Failure Mode
In all of the tests conducted failure was initiated by inadequate shear capacity of the wall members. This was due to inadequate provision of shear links in the members primarily designed to carry axial compression. Further tests in fact showed that the capacity of the structure would be many times greater if shear failure could be avoided as a collapse mechanism would not form until 15-20% base shear demand.

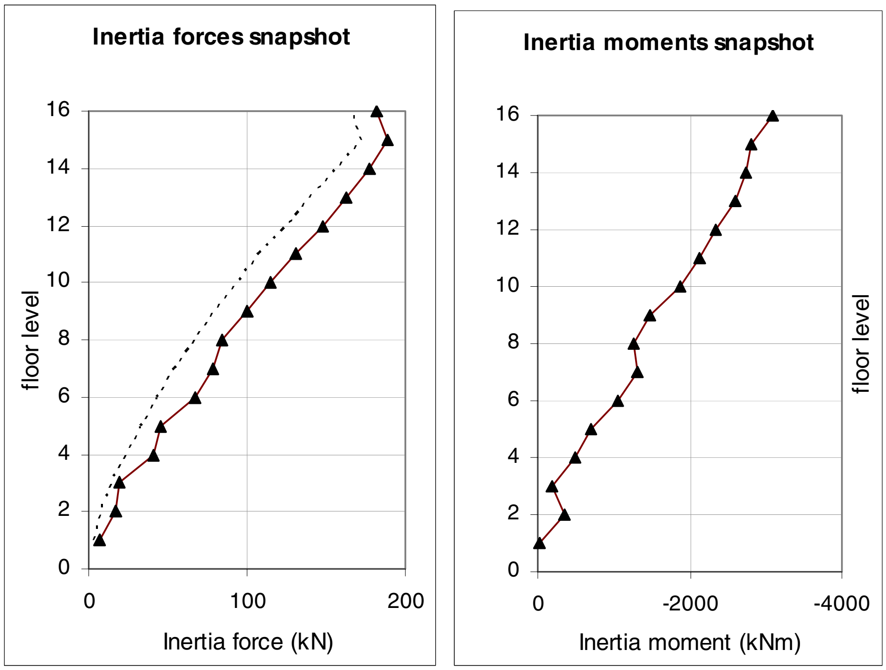

Fig 8. Snapshot of Inertia Forces and Moments

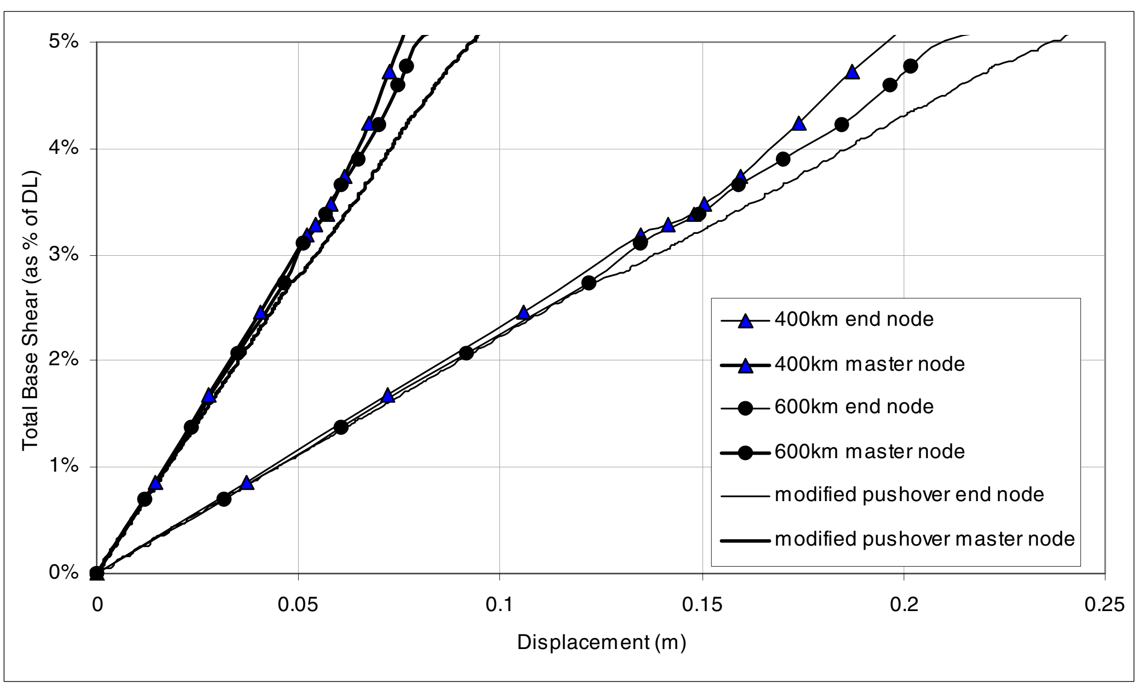

Fig 9. Modified Pushover Load-Displacement Behaviour

CONCLUSIONS

A 16-storey reinforced concrete frame wall structure has been analysed using both pushover and dynamic analyses. It was found that the building was inadequately designed for shear and hence failed due to local shear failure in the wall members. The base shear demand on the structure due to the identified worst case scenario earthquakes was found to be 1.37-1.68% of the structures dead load. The capacity of the structure was then found to exceed this by approximately a factor of two, showing the structure to have adequate capacity to survive the identified earthquakes.

Major differences were however found between the capacities of approximately 6.9% and 3.1% of structural dead load predicted using the pushover and dynamic time history analyses respectively. This difference was investigated and it was found that the asymmetric nature of the structure led to the generation of large inertial moments during the dynamic analysis. It was then shown that this could be accounted for in the pushover analysis by the inclusion of a proportional moment applied to the master node of the structure.

REFERENCES

1. Balendra, T., Lam, N. T. K., Wilson, J. L. and Kong, K. H. "Analysis of long distance earthquake tremors and base shear demand for buildings in Singapore", Engineering Structures 24, pp 99-108, 2002
2. Carr, A. J. "Ruaumoko, The Maori God of Volcanoes and Earthquakes, 3-Dimensional Version", Department of Civil Engineering, University of Canterbury, New Zealand, 6 May 2002.
3. Idriss, I. M. and Sun, J. I., "Users Manual for SHAKE91", Centre for Geotechnical Modelling, University of California, 1992
4. Hardin, B. O., and Drnevich, V. P. "Shear Modulus and Damping in Soils: Measurement and Parameter Effects", Journal of Soil Mechanics and Foundations, American Society of Civil Engineers, Vol.98, pp 603-624, 1972
5. Poulos, H. G. "Relationship between local soil conditions and structural damage in the 1989 Newcastle earthquake", Australian Civil Engineering Transactions, Institute of Engineers Australia, vol.CE33(3), pp 181-188, 1991
6. Lam, N. T. K. and Wilson, J. L. "Estimation of the site natural period from borehole records", Australian Journal of Structural Engineering, vol SE1(3), pp 179-199, 1999
7. Imai, T. and Tonouchi, K. "Correlations of N value with S-wave velocity and shear modulus", Proceedings of the 2nd European Symposium on Penetration testing, pp 24-27, 1982
8. Gupta, V.K., Nielsen, S. R. K. and Kirkegaard, P. H. "A preliminary prediction of damage-based degradation in RC structures", Earthquake Engineering and Structural Dynamics 30, pp 981-993, 2001
9. Otani, S. "Hysteresis Models of Reinforced Concrete for Earthquake Response Analysis", Journal of Faculty of Engineering, University of Tokyo, Vol. XXXVI, No. 2, pp 407-441, 1981
10. Harries, K. A., Mitchell, D., Redwood, R. G. and Cook, W. D. "Nonlinear seismic response predictions of walls coupled with steel and concrete beams", Canadian Journal of Civil Engineering 25, pp 803-818, 1998
11. Filiatrault, A., Lachapelle, E. and Lamontagne, P. "Seismic performance of ductile and nominally ductile reinforced concrete moment resisting frames. II. Analytical study", Canadian Journal of Civil Engineering 25, pp 342-352, 1998