Pacific Conference on Earthquake Engineering
New Zealand, 5 - 8 August 1987

WELLINGTON CENTRAL POLICE STATION
BASE ISOLATION OF AN ESSENTIAL FACILITY

A.W. CHARLESON,* P.D. WRIGHT,** R.I. SKINNER***
*School of Architecture, Victoria University (formerly M.W.D.)
**Ministry of Works and Development
***D.S.I.R.

SUMMARY

The decision to use base isolation and the subsequent analysis and design of the building are outlined. Seismic performance criteria and methodology to select design accelerograms for the inelastic time history analyses are discussed. Particular attention is paid to controlling the behaviour of the building in an extreme earthquake as part of ensuring the very high seismic performance required of an essential facility.

INTRODUCTION

The proposed Wellington Central Police Station consists of a ten storey tower block with a basement alongside a four storey car parking building. The two buildings are structurally independent, and are to be built in the Wellington inner city area. This paper only considers the design of the tower block.

The Client's brief emphasised the requirement for the building to continue to function in the event of a major disaster, such as an earthquake. The New Zealand loadings code [9] also requires a significantly enhanced seismic performance for essential facilities such as police stations "which are intended to remain functional in the Emergency Period for major earth-quakes". For this reason the Code requires the use of a Risk Factor R = 1.6 to increase the building strength, and thereby reduce the structural damage that would occur in such a major event. With the need for superior seismic performance thus emphasised the option using base-isolation was considered at an early stage in the development of the project.

DEVELOPMENT OF THE STRUCTURAL SYSTEM

Initially the number of structural options considered in depth was deliberately limited in order to meet the "fast track" programme for the project. The lateral load resisting structure is on the perimeter of the building to minimise possible conflicts with services requirements, to minimise the interstorey heights, and to give the client increased flexibility of useage. Further, the perimeter structure had to be heavily penetrated to provide natural light and views to all the individual offices. All the internal structure, beams and columns therefore carry gravity loads only and are sized accordingly. A typical floor plan is shown at Figure 1.

Three options considered for the structural system were a cross-braced frame, a moment resisting frame, and a base isolated cross-braced frame. The first option was discarded at an early stage because of the very large member sizes required and the difficulty of achieving a reliable post-elastic deformation mechanism. The perimeter moment resisting frame was

Volume 2, pp. 377 – 388.

considered in detail. Due to the 1.6 Risk Factor and there being only two or three frames in each direction, relatively large members were necessary to satisfy the Code strength and deflection requirements. Finally the base isolated cross-braced frame option was considered. As piling was already indicated by the foundation conditions, which consist of several metres of reclamation fill and marine deposits overlying weathered greywacke rock, it was decided to isolate the superstructure by placing it on long flexible piles and connecting it to the ground with horizontally functioning energy dissipating devices. This system is similar to that reported by Boardman et al [1]. The use of elastomeric bearings to base isolate the structure was considered inappropriate because of the high axial loads.

Initially for the preliminary design a simplified mathematical model of the structure was subjected to 2.0 times the El Centro 1940 N-S accelerogram using the inelastic time history computer program DRAIN 2D [4]. The factor of 2.0 was assumed to be a reasonable upper bound value until a more detailed evaluation of the site seismicity could be made.

The costs of the two options were compared on the basis of initial structural cost, although non-structural items such as the provision of seismic mullions for glazing and interior partition separation details were included in the cost of the non-isolated structure. It was found that there was a 10% saving in structural cost associated with the base isolated option.

Not only is the base isolated option cheaper, but it has a considerably enhanced earthquake resisting performance with the further benefit of reduced repair costs after a major event. No attempt was made to quantify this however.

DESCRIPTION OF SEISMIC STRUCTURAL SYSTEM

Lateral seismic inertial loads within the superstructure are resisted by reinforced concrete floor diaphragms and the perimeter cross-braced frames. The ground floor slab acts as a transfer diaphragm to introduce load to the piles which are connected to column downstands by steel pins. The piles, which are constructed in oversize diameter shafts, are founded in the weathered greywacke rock. Most piles are fixed against rotation at their bases in order to provide an elastic centring force. However the exterior piles to each perimeter frame are pinned at their bases so as not to form premature plastic hinges under the combination of flexure, caused by lateral displacement at the pile tops, and tensions generated by overturning moments.

Lead extrusion dampers are connected between the basement structure and the column downstands at the perimeter frames. Lateral loads from the columns are transferred through the dampers into the basement, and then via the non-isolated piles into the ground. The superstructure is therefore able to move freely in the horizontal plane except for the influence of the dampers, and cantilever piles which take a proportion of the lateral load directly to bedrock. Figures 2 and 3 show an elevation of the structural system and a plan of the basement structure respectively.

Lead Extrusion Dampers

The lead extrusion dampers are pin ended cylindrical telescoping units which function by forcing lead past a constriction within an outer sleeve when axially loaded. This type of damper was chosen because it offered a number of advantages over mild steel damping devices. First, there is no strain hardening which would increase yield strength and therefore the structural

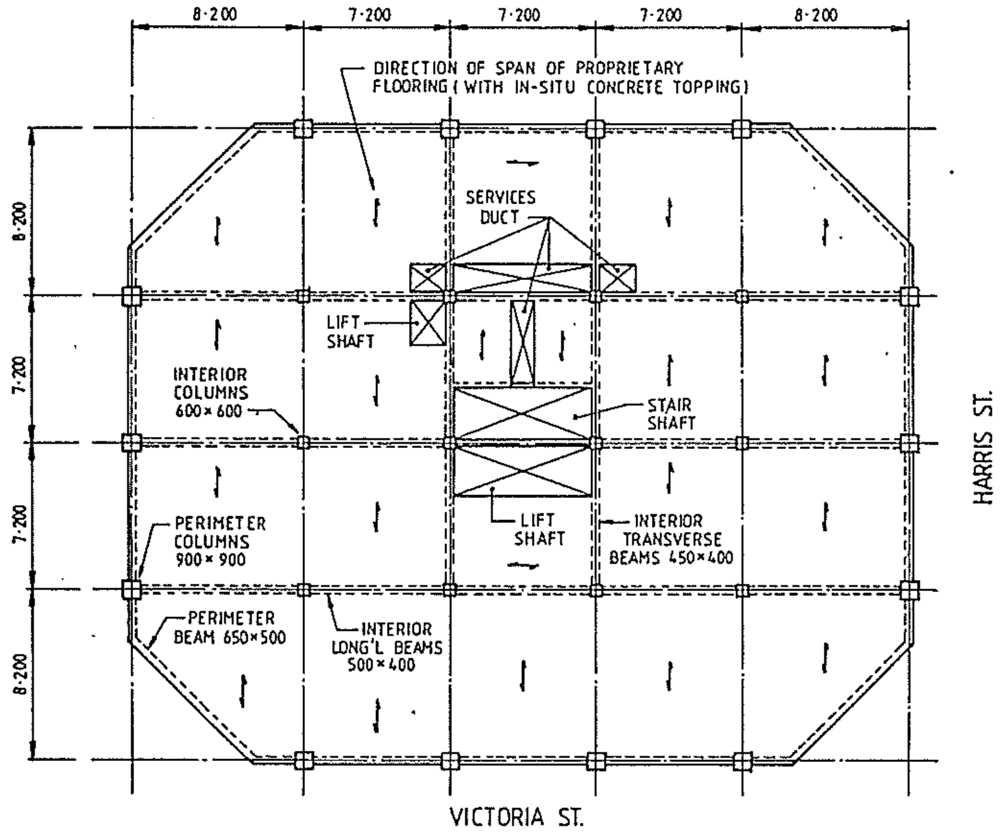

Figure 1 Typical floor plan

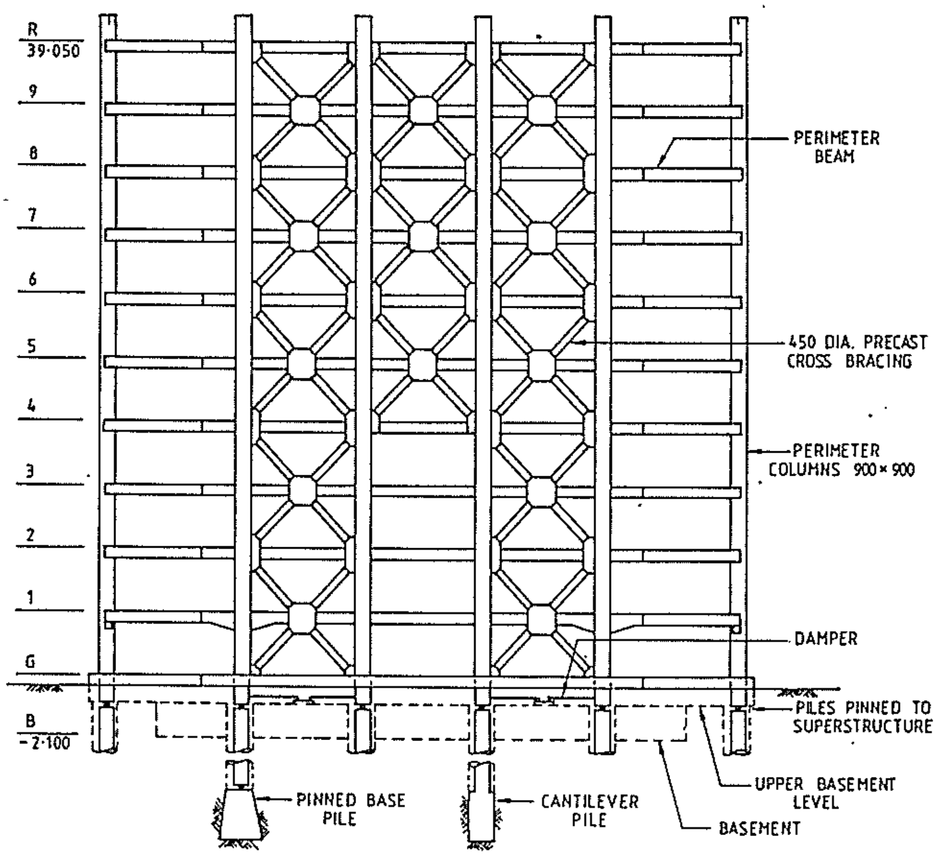

Figure 2 Elevation of long frame

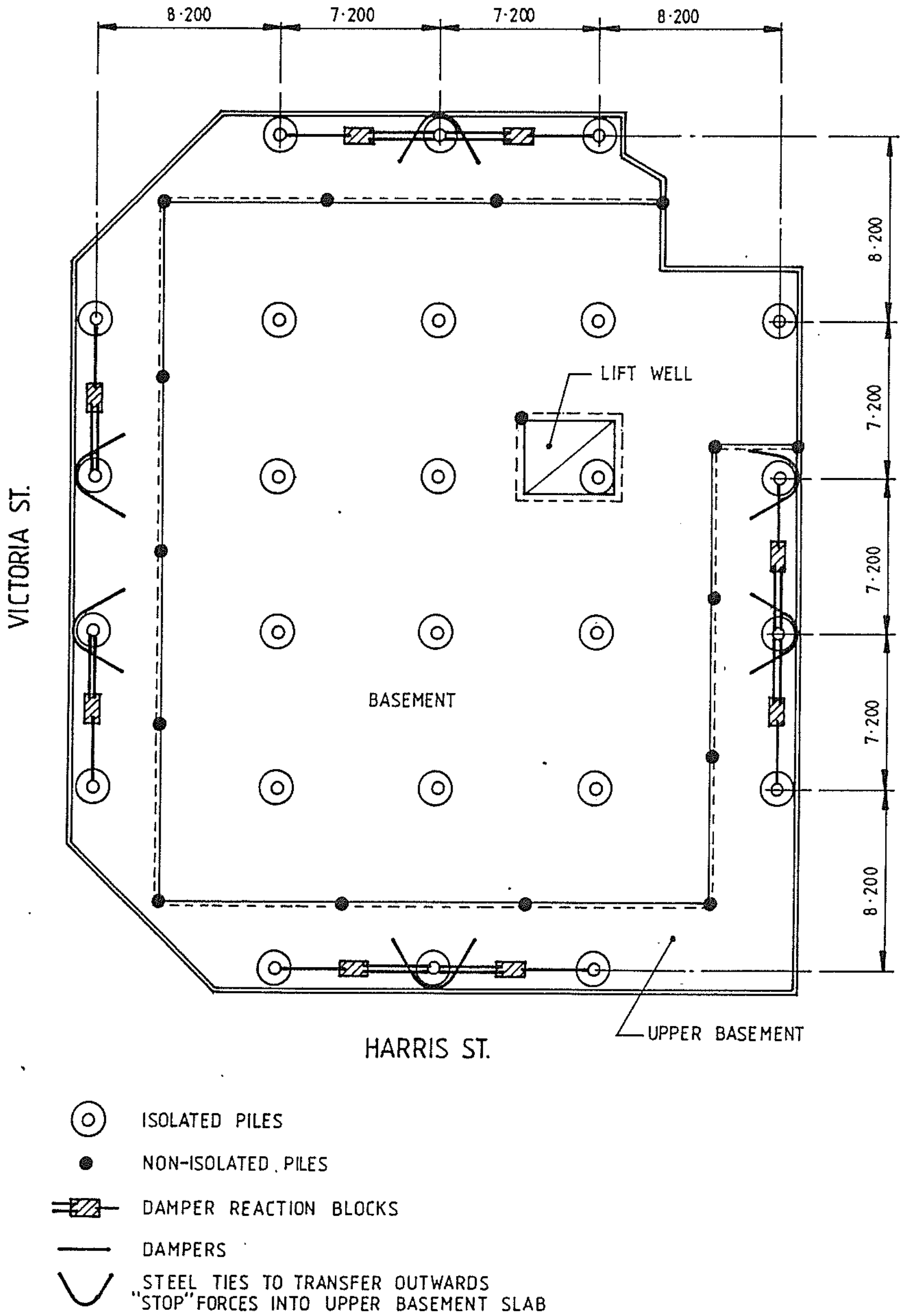

Figure 3 Basement plan

response, but rather, during intense earthquake shaking there will be some reduction of the yield strength due to heat build up in the device. Secondly, the lead will tend to creep under sustained load, allowing the building to move slowly back towards its original position if it is shaken off-centre. Another advantage of these dampers is in the transfer of load into the basement structure by direct tension and compression. These dampers have been used in several bridges, and are further described by Skinner et al [8].

The yield load of the dampers has been kept as low as possible while preventing yield under wind loading. A nominal yield load of 3000 kN (0.035 of the building seismic weight) has been provided in each orthogonal direction (six units of 250 kN yield force along each side of the building). The detailed design, construction, and testing of the 250 mm diameter dampers will be undertaken by the Department of Scientific and Industrial Research.

SEISMIC PERFORMANCE CRITERIA

The general seismic performance criteria were based on the requirements of the Client brief and the loadings code as previously outlined. It was necessary, however, to interpret these requirements in order to quantify the design ground motions. A two level approach was taken; the first to ensure the functionality of the building in the emergency period for major earthquakes, and the second to prevent collapse in an even larger event. The levels are defined as follows:

a) The building must be functional after an earthquake with a low probability of occurrance during the nominal building life of 50 years. A probability of 10% was considered appropriate.

b) The building must not collapse in a larger earthquake with an even lower probability of occurrance. A probability of 5% was adopted. This level will ensure the safety of occupants, and that the building will survive earthquakes larger than (a) above, but with increasing levels of damage.

The return periods for earthquakes corresponding to (a) and (b) are approximately 450 and 1000 years respectively.

The 450 year return period event will have an intensity greater than MM IX and is nominally equivalent to the return period implied by the SANZ Seismic Risk Committee's proposed revision to the spectrum for a non isolated building designed with a risk factor of 1.6 [11]. It is, however, less than would be implied by the current code, which is more conservative for long period structures. The 1000 year return period event has been recommended as appropriate for the design of certain essential facilities in the USA [3].

SELECTION OF DESIGN EARTHQUAKES

With the required seismic performance criteria defined it was necessary to choose a suite of accelerograms which, when appropriately scaled, would correspond to 450 year and 1000 year return period events. The methodology described in detail by McVerry and Skinner [6], is summarised below.

The baseline for the choice of accelerograms was the Standards Association of New Zealand Seismic Risk Committee's spectra for Wellington [5]. This Committee has produced 450 year and 1000 year 5% damped acceleration response uniform risk spectra modified with a constant psuedo-spectral velocity at periods beyond one second. Although the piles of the structure are founded on rock it was decided to use the more conservative Ground Class III spectra to allow for any response amplification which might occur in the four metre depth of reclamation fill and alluvium overlying the weathered rock. This amplification could increase the response of the basement structure.

The Caltech earthquake records [2] were searched for spectra with, where possible, similar shapes to the Risk Committee's spectra and with maximum strengths in order to reduce the influence of recorded noise. Five selected spectra were then scaled so that in the 2.0 second to 4.0 second period range the spectral acceleration approximated that of the baseline spectra. The 2.0 second to 4.0 second range was chosen as results from the dynamic analyses indicated that the isolated structure had an equivalent period of vibration of approximately 3 seconds. The scaling factors are therefore only suitable for such a long period structure as is being considered. The El Centro 1940 NS accelerogram (one of the five selected) was scaled by 1.4 and 1.7 to simulate earthquakes with 450 year and 1000 year return periods respectively.

As well as these accelerograms, account was taken of possible fault rupture in the vicinity of the site. This was done by combining the responses to design accelerograms with the response to accelerations simulating the fault displacement. The fault displacement was found to produce only a small increase in structural forces and displacements.

DESIGN OF STRUCTURE

Isolated Piles

Two types of pile have been used to support and isolate the superstructure. The 12 interior piles plus the intermediate piles at each exterior frame will cantilever from the rock and be "pinned" to the underside of the superstructure. The remaining eight piles at the ends of each perimeter frame are pinned at both ends. The pins proposed will consist of spherical steel seatings (ball and socket) with tensile loads being carried by prestressing cable.

Cantilever piles were chosen because it was considered desirable to have a centring force in excess of the P-delta induced horizontal forces resulting from the building sway. A net centring force of approximately 1.5% of the seismic weight is provided at a deflection of 300 mm (upper bound deflection for 450 year earthquake). Some yielding of the pile reinforcement will occur for the design 450 year earthquake, and hinging will occur at the 400 mm maximum displacement.

The piles are to be founded approximately 15 m below ground. This depth is well into the weathered greywacke rock, which begins approximately 5 m below ground level, but is necessary to achieve the required flexibility for the cantilever piles and to minimise the P-delta effect resulting from the horizontal component of the pile reactions at large horizontal displacements. The pile lateral stiffness was calculated using a program developed by Mander and Zahn [12] which included the effects of axial load, cracking and concrete tension stiffening.

To confirm that the assumption of base fixity of the piles could actually be achieved pressuremeter testing of the weathered greywacke using a fast loading rate was carried out during the site investigation. A finite element analysis of the pile base-rock area indicated that the base should be embedded 3 metres to ensure stress levels within the rock were acceptable and that the base rotation should not reduce the pile stiffness by more than 5% [7].

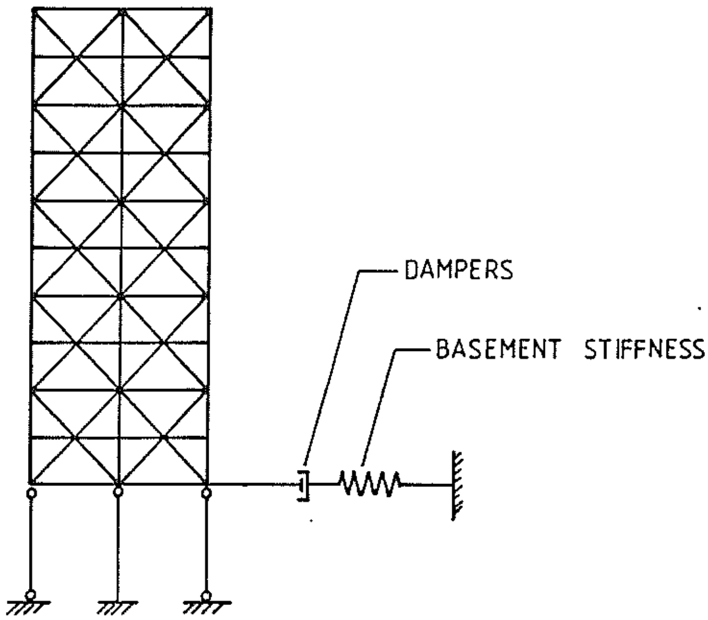

Figure 4 Schematic diagram of computer model-short frame

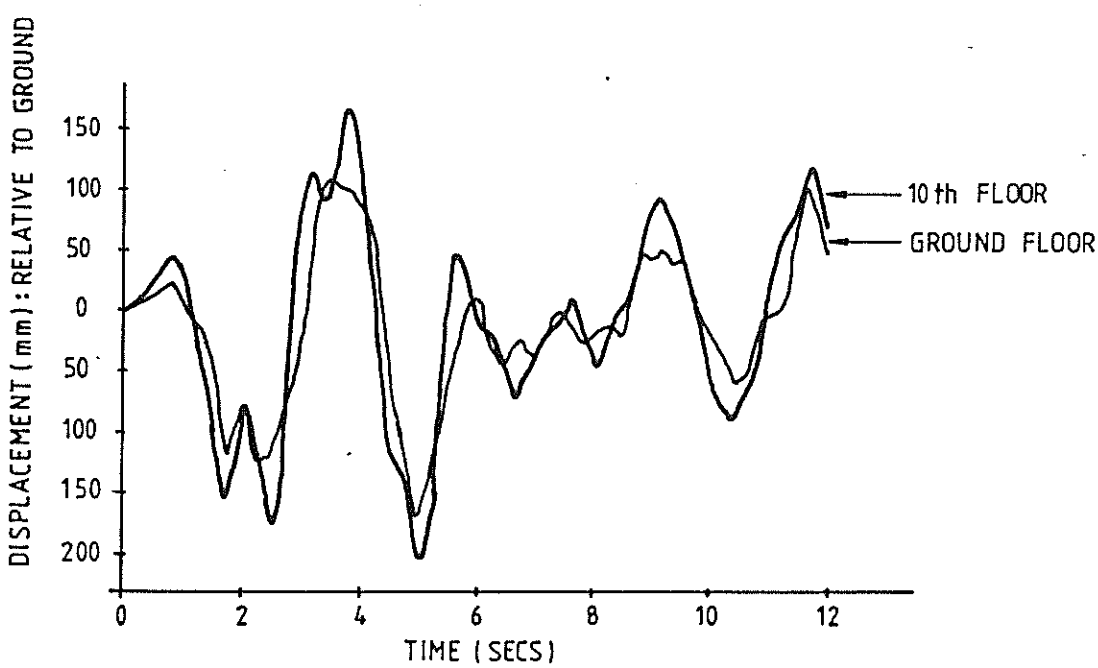

Figure 5 Displacement time history. El Centro 1940 N-S, scale factor 1·4 (450 year)

Basement

The basement is built at two levels. The lower level provides the area required to satisfy the Client's brief while the upper level provides a platform for mounting the dampers and reacting the superstructure stop forces. These forces will be transmitted from the column downstands to the upper basement slab via specially strengthened upper casings, and then throughout the basement by inplane shear and flexure of the upper basement slab.

Lateral load resistance is provided by friction and earth pressure against basement walls and the unisolated piles. The pile casings of the 12 interior isolated piles are separated from the basement slab to allow for horizontal movements while resisting vertical hydrostatic forces. The separation of these pile casings will reduce the potential overstrength lateral load resistance of the basement and encourage it to displace before serious damage can occur in the superstructure.

DISPLACEMENT LIMITING CONSIDERATIONS

The greatest design displacement, for a 1000 year return period earthquake, would almost cause the building downstands to reach the basement stops, (set at 375 mm to protect the 400 mm travel dampers, with 25 mm allowed for downstand deformations and construction tolerances); however the extent of the displacement demand remains the greatest uncertainty in the performance of the isolated building. As a safety factor against this uncertainty it is therefore appropriate to provide for whatever excess displacement, over a nominal 400 mm, can be accommodated at moderate cost.

The downstand and basement constraints have bilinear lateral load/deflection characteristics. These constraints were not included in the building model for time history analysis, however a simple dynamic analysis leads to an understanding of the mechanics associated with basement impact.

Basement Translation

Consider the basement translation X, due to an earthquake which would cause a superstructure displacement of 375 x f (mm) in the absence of the basement stop. (The factor f is roughly equal to a scale factor applied to an earthquake that will use all the 375 mm available clearance).

When the building kinetic energy is balanced against the energy required to displace the basement a distance X the following approximate results are obtained:

f	1.25	1.50	[1.75	[2.00
X,mm,	80	125	160]	200]

If the impulse generated by the basement mass is neglected, an extreme displacement factor f = 2.0 gives a building base shear of about 80% of the superstructure shear strength.

Impact Shear Pulse

When the mass of the basement is considered, it is found to send an attenuating pulse of horizontal shear up the building. The severity of the pulse depends on the displacement factor f and on the stiffness of the impacting downstands (and any associated buffer). With the total downstand stiffness at about 0.3 of the interstorey stiffness the impulse will be well below the building yield level and superstructure damage will be largely confined to the downstands. It would be necessary for the downstand stiffness to be greater than the interstorey stiffness to produce yielding of the superstructure; for f = 2.0 a ductility demand of about 1.5 at first storey level (due to impulse alone) would result.

The downstand operating conditions can be further improved by facing them with a lead sleeve about 25 mm thick (rubber appears unsuitable for this application), but at the time of writing it has not been decided whether this is warranted.

Basement Displacements

The most difficult problem with a large displacement (or earthquake) factor f is the substantial basement displacement which results. It is seen that for f = 1.5 (and 2.0), basement displacements of 125 mm (and 200 mm) are required. These displacements would be reduced by an increase in the basement's resistance to translation. The very low probability of some ductility demand on the building and the downstands would then be somewhat increased.

STRUCTURAL DETAILS

Cross-Braced Frames

All structural elements are of reinforced concrete construction. Columns and joints will be cast in-situ; cross-bracing will be precast, and the beams have been detailed to allow the use of either technique.

Precasting was chosen for the 450 mm diameter braces as it is the only viable means of achieving satisfactory concrete quality for the narrow inclined member. This in turn dictated the use of steel flats as reinforcement to allow bolting within the joint regions. A satisfactory solution was obtained by using either two or three flats which alternate at each bolted connection and are able to cross easily within the columns.

Isolated Piles

The piles will be constructed generally as conventional bored piles with permanent steel casings. The casings will be anchored into three metre deep bases cast against the rock and will provide a 400 mm clearance gap to the piles. The 800 mm diameter isolated piles will also be anchored into the bases, either fully fixed, or pinned, depending on their function. It is proposed that the isolated piles be cast in-situ using cardboard tube permanent forms.

The pin ended piles, designed to transmit large uplift forces will have their bases belled.

Pins

The isolated piles are to be pinned to the superstructure to minimise the length of pile necessary to give the required lateral flexibility, and reduce bending of the superstructure columns.

Two types of pin are being considered; a spherical seating and a hinging solid steel insert. At the time of writing, a final selection has not been made.

Non-Structural Elements

Low interstorey deflections (10 mm maximum), except between basement and ground floor, have generally obviated the need for seismic separation of non-structural elements. Precast stairs, however, have been detailed to allow one end to slide. Lift guides have been suspended from the superstructure, and are totally independent of the basement to allow up to 400 mm relative movement. Flexible flashings and services pipes are also required in this area, and between the tower block and adjacent car parking building.

Advantage has been taken of base isolation to reduce the fixing requirements for equipment at roof level. The design is based on peak accelerations taken from the time-history analyses rather than the Parts and Portions coefficients of the loadings code [10].

SEISMIC PERFORMANCE

As outlined in earlier sections a very high level of seismic performance can be expected from this base isolated structure.

The building has been designed to survive a major earthquake of approximately 450 year return period and remain fully functional. At this level of loading the superstructure responds elastically, and low interstorey deflections of approximately 10 mm will ensure only minor non-structural damage is sustained.

Some yielding of the superstructure will occur when the building is subjected to a 1000 year return period earthquake, but damage levels will remain low and consequently easily repairable. In an extreme, or unusual, earthquake event of very low probability resulting in the isolation clearance being exceeded, the superstructure will be arrested in a controlled manner without collapse. Significant damage can be expected to the column downstands, basement structure, piles and pile casings.

Large displacements between ground and superstructure are implied by base isolation, requiring large clearances at basement level. A maximum ground floor displacement of 300 mm was obtained from one of the 450 year design earthquakes.

Any earthquake that produces yielding of the damping devices is likely to leave the building off-centre. Creep of the lead in the devices under the sustained pile centring force will reduce this in time, but it may be necessary to return the building by jacking to achieve an acceptably small permanent offset.

ACKNOWLEDGEMENTS

The permission of the Commissioner of Works to report this work and the assistance of the Chief Structural Engineer and his staff at Head Office, Ministry of Works and Development is acknowledged.

REFERENCES

1. Boardman, P.R. et al. 1983. Union House - A cross-braced structure with energy dissipators. Bulletin of New Zealand National Society for Earthquake Engineering, Vol. 16, No. 2. Wellington.

2. California Institute of Technology, Earthquake Engineering Research Laboratory. Strong motion earthquake accelerograms digitised and plotted data. Pasadena, California.

3. Freeman et al. 1983. Seismic design of guidelines for essential buildings. Proceedings of the Eighth World Conference on Earthquake Engineering.

4. Kanaan, A. and Powell, G.H. 1973. General purpose computer program for an inelastic dynamic response of plane structures. EERC 73-6, University of California, Berkerley, California.

5. Matuschka, K.R. et al. 1985. New Zealand Seismic Hazard Analysis. Bulletin of the New Zealand National Society for Earthquake Engineering, Vol. 18, No. 4. Wellington.

6. McVerry, G.H. and Skinner, R.I. 1986. Design Accelerograms for Wellington Central Police Station. Physics and Engineering Laboratory Report No. 954. Wellington.

7. Ministry of Works and Development, Special Projects Office. 1986. Wellington Central Police Station foundation report. Wellington.

8. Skinner, R.I. et al. 1980. Hysteretic dampers for the protection of structures from earthquakes. Bulletin of the New Zealand National Society for Earthquake Engineering. Vol. 13, No. 1. Wellington.

9. Standards Association of New Zealand. 1984. Code of practice for general structural design and design loadings for buildings. NZS 4203. Wellington.

10. Standards Association of New Zealand. 1982. Code of practice for the design of concrete structures. NZS 3101. Wellington.

11. Standards Association of New Zealand. 13/5/86. Code of practice for general structural design and design loadings for buildings : draft for comment. DZ 4203. Wellington.

12. Zahn, F.A. et al. 1986. Design of reinforced concrete bridge columns for strength and ductility. Research report 86/7. Department of Civil Engineering, University of Canterbury, Christchurch.