

BASE ISOLATION OF AN EXISTING 10-STOREY BUILDING TO ENHANCE EARTHQUAKE RESISTANCE

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ABSTRACT

The Rankine Brown Building at Victoria University of Wellington, New Zealand houses the University Library and is pivotal to the operation of the University. Built in the early 1960s, the building was of innovative form and construction for its time, with long span precast concrete waffle slabs over 10 floors supported on 16 main columns. Three years ago a review of the building structure suggested that increased protection of the building would be required to meet the University's expectation for operational continuity after a moderate or major earthquake event in the Wellington area. This paper outlines:

Probable areas of structural yielding in a major earthquake event.

Structural analysis for base isolation using earthquake time history records.

Details of base isolation bearings.

Installation of the base isolation bearings.

Other structural details that allow movement of the superstructure to take place due to base isolation.

1 INTRODUCTION

The Rankine Brown building at Victoria University of Wellington, New Zealand houses the main university library including book storage and study facilities. This building and its functions are pivotal to the operation of the university.

The building was constructed in the early 1960s of reinforced concrete with an innovative form for its time. Long span waffle slabs over 10 floors supported on 16 main columns provide open plan spaces with live load capacity in the order of 5 kPa.

In 2000 a review of the building structure identified the need for increased protection of the building to meet the university's expectation for operational continuity after a moderate or major earthquake event in the Wellington area. The library is critical to the University's ongoing operations and any long term disruption to that facility would severely impact upon the University's function. The need to reduce sharply increasing earthquake insurance premiums is another driver for the university to provide enhanced earthquake resistance for the building.

The effect that a major earthquake, or for that matter any

disruptive disaster, can have on a university library is described by M.M. Finley in a paper presented to the 8th Annual Federal Depository Library Conference, April 1999, titled "Disaster Planning for Libraries: Lessons from California State University, Northridge". For various reasons the library took 8 years to fully recover from the 1994 Northridge earthquake.

From this perspective, Victoria University initiated a study to identify options for the toughening of the Rankine Brown building structure against earthquake effects and reviewed the cost/benefit of any such work. Out of this study came the decision to base isolate the building. This paper outlines aspect of the base isolation design and installation for the Structural Upgrade project.

2 BUILDING DESCRIPTION

The form of the building structure is set out in Figures 1 and 2. The Rankine Brown building was originally constructed with 10 floor levels made up of:

- 2 basement floors (Levels 0 & 1) of 46.16 x 23.98

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metres and 50.06 x 30.43 metres respectively, built into the sloping site.

- Ground floor (Level 2) of 69.54 x 30.59 metres with level access from the west side only.
- First tower floor and podium roof (Level 3) of 70.38 x 31.42 metres.
- 6 tower floors (Levels 4 to 9) of 63.13 x 19.95 metres each.

A roof frame of structural steel covers the tower area. Subsequent to the original construction, the first tower floor (Level 3) was extended over the roof of the ground floor podium in lightweight framing.

The nine suspended levels of the Rankine Brown building were formed principally from reinforced, precast concrete inverted "U" shaped units 890 mm wide centre to centre, and 584 mm high with 38 mm thick floor topping. Each precast unit rib was pre-stressed longitudinally. Groups of units were post-stressed together transversely when in place on the building. Vertical support was provided by 16 main columns, 914 mm square, set out in 2 rows of 8 columns with longitudinal spacing of 8.001 metres and transversely at 12.624 metres centres. The in situ concrete of each column was placed from mid-height of one floor to mid-height of the next within precast concrete shell elements. Conventional bar reinforcing was fitted down through holes in the column-head section of the precast waffle units and lapped over the column mid-height section. Refer to Figure 3. A full description of the original building design and details of construction are provided by Allardice (1966).

As part of the Structural Upgrade project, the two basement levels were excavated (refer Figure 4) and extended to the southern perimeter of the superstructure. This allowed access to the base of the columns for the installation of the base isolation bearings. The extension of the suspended floors was constructed with the Stahlton Flooring system supported on the original primary beams or new secondary reinforced concrete beams that were fixed into the main columns or new perimeter columns.

3 HORIZONTAL RESISTANCE FOR UNMODIFIED BUILDING

The building's horizontal load resistance is provided by frame action of each column with the three pre-stressed waffle ribs at each floor in the transverse direction of the building, and the three post-stressed waffle ribs at each floor in the longitudinal direction. Refer to Figure 3. Analysis showed that under sway action the ribs further from the column did not provide significant resistance due to the lack of torsional stiffness in the transverse ribs back to the column.

In the transverse direction (parallel to the main axis of the precast floor units), the negative flexural restraint at the face of the column head is provided by "harped" stressing tendons

draped down away from the column head. These tendons are cast into the side face of the floor unit webs and are not restrained by vertical stirrups. For positive moment, restraint is provided by Tentor bars that are cast into the ribs. Fifty percent of these bars are cut off in the order of two rib depths from the face of the column head.

In the longitudinal direction, both the negative and positive flexural restraint is provided by post-stressing tendons set in the horizontal plane through the column head. The tendons are continuous through the length of adjacent ribs for one full bay between pairs of columns and one half bay on each side of the columns before being anchored off at mid span. The tendons were then fully grouted.

In both the transverse and longitudinal direction, the column flexural strength is significantly greater than the flexural capacity of the waffle ribs framing into the column head.

Material strengths noted for design were:

- Column *in-situ* Concrete Strength: 39 MPa
- Pre-cast Waffle Floor Unit Concrete Strength: 39 MPa
- Mild Steel Main Reinforcing Bars (ultimate): 253 MPa
- Tentor Deformed Reinforcing Bars (ultimate): 420 MPa
- 5 mm diam. Pre-Stressing Wire (ultimate): 1400 MPa
- 7 mm diam. Post-Stressing Wire (ultimate): 1400 MPa

4 PROBABLE PERFORMANCE IN A MAJOR EARTHQUAKE EVENT

Under Code Loading (Standards New Zealand 1992) requirements for a "time history" analysis, an overall displacement ductility of 3 or greater is required for the moment demand to stay within the ultimate moment capacity of waffle ribs plus floor topping. This places a curvature ductility demand of 10.2 on the transverse rib at the column-head hinge zone. This is sustainable for both positive and negative bending provided the number of cycles of extreme loading does not open up a positive moment crack against the column head that allows vertical shear failure to occur. This sliding type failure is inhibited by the stressing-induced compression across the crack face. The curvature ductility demand on the longitudinal rib at the column head hinge zone is 6.4. This demand cannot be sustained by the rib section due to failure of the stressing strands under both positive and negative moment. This ductile demand in a significant number of waffle rib hinge zones is such that high levels of damage are expected, but probably not collapse of any floor slab.

Bond length of the fully bonded stressing tendons has an effect on the ductile capacity of the cracked waffle rib section. It was found during construction for the Structural Upgrade where a number of tendons were de-stressed, that bond lengths were relatively short - thereby giving reduced ductile capacity.

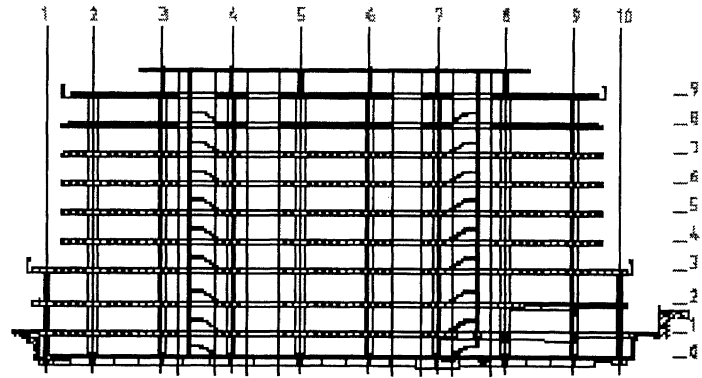
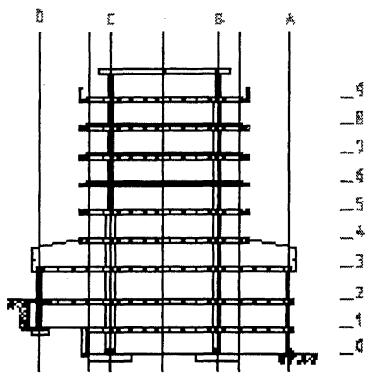


Figure 1: Building cross section.

Figure 2: Building long section.

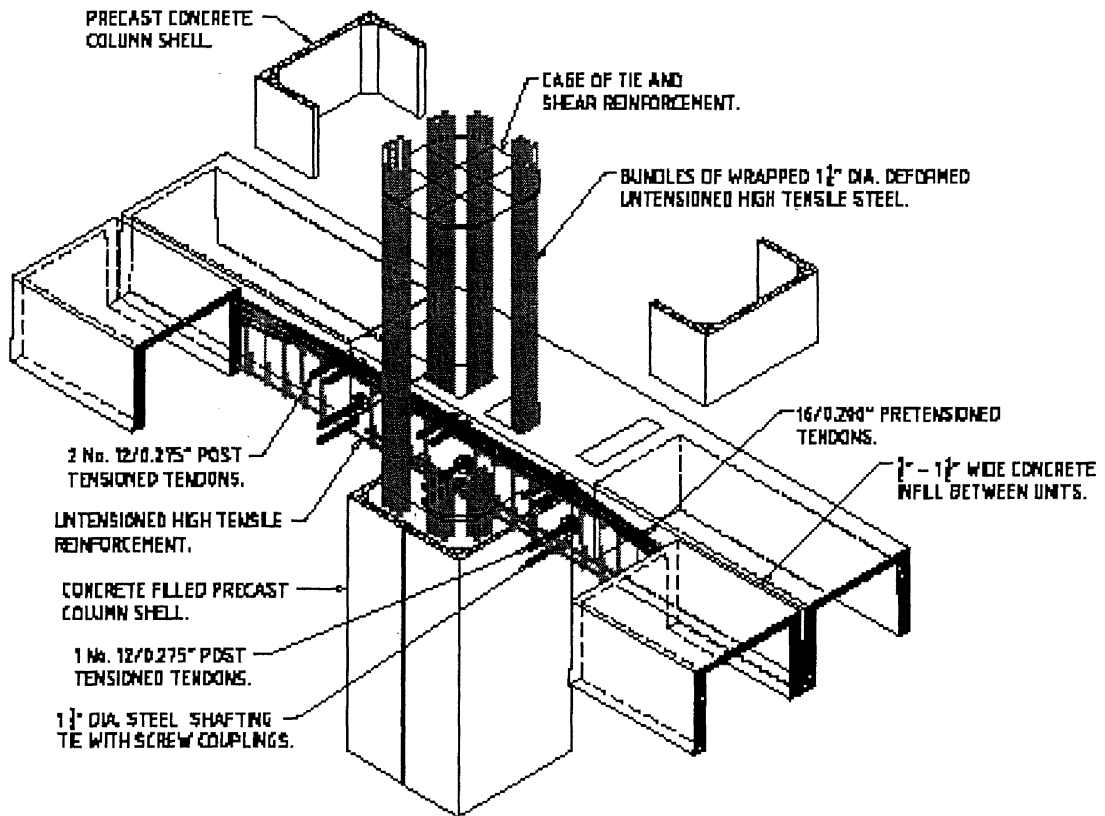


Figure 3: Column head detail.

In a Code-level earthquake, the extent of damage would be such that the building would most probably be more economic to demolish and replace than to repair.

5 STRUCTURAL ANALYSIS FOR BASE ISOLATION: TIME HISTORY RECORDS

The 1st mode dynamic period of the Rankine Brown building is 1.4 seconds in the longitudinal direction and 1.8 seconds in the transverse direction. To reduce the base shear by 50 % of current Code provisions required a period shift to approximately 3 seconds with the base isolation bearings. Even with this period shift there was a concern that in a major earthquake event the superstructure response could be such as to induce significant ductile demand on the waffle ribs framing into the column heads. To gain a better understanding of the level of earthquake protection that was most probably going to be provided by the base isolation bearings, it was imperative that a computer based "time history" analysis be carried out.

Shortly after the building was completed, small amplitude vibrators were used to check the dynamic characteristics of the building. This testing showed (Allardice 1966) that the 1st mode period in the longitudinal direction was 0.60 seconds, where as transversely it was 0.7 seconds with damping at 2 % critical. It was assumed that a greater percentage of the un-cracked floor system contributed to the frame stiffness, and that cladding, partitions, lift shafts and stairs contributed to stiffening the building at low amplitude displacements. This initially high stiffness in the first phase of a major earthquake event was considered at the preliminary design stage for the base isolation system. The assessment was that the greater relative shift in dynamic period between the superstructure and the base isolation plus superstructure was a good thing, and after a small number of excursions into the inelastic range, the superstructure would soften sufficiently to increase the natural dynamic period.

For the "time history" analysis, The Institute of Geological and Nuclear Sciences were engaged to provide a suite of accelerograms that represented a range of strong motion earthquake events likely to be experienced by the Rankine Brown building site. These records took into account appropriate representation of long-period components of the motion that would be important for the base isolation performance. The accelerograms provided (McVerry 2002) were based on:

- The Izmit record from the magnitude 7.4 Kocaeli, Turkey earthquake of 17 August 1999.
- The Lucerne record from the magnitude 7.3 Landers, California earthquake of June 1992.
- La Union record 16 km from the rupture plane of the 8.1 Michoacan, Mexico earthquake of September 1985.
- El Centro 1940 record.

The first two records are representative of the motions that

may be expected from rupture of the Wellington Fault that is only 900 metres to the northwest of the building site, or the Ohariu Fault further to the west. The Izmit record was from a site close to the epicentre with most of the rupture propagation away from the recording station. The Lucerne record contains a strong long-period forward-directivity pulse resulting from propagation towards the recording station. A similar pulse is expected at the Rankine Brown building for a rupture on the Wellington Fault propagating southwards from an epicentre near Kaitoke and terminating in Cook Strait.

The La Union record represents a greater magnitude earthquake of around magnitude 8 on the subduction interface beneath Wellington. It also represents a greater earthquake on the Wairarapa Fault 24 km to the southeast.

These sources of earthquake from the Wellington, Ohariu and Wairarapa Faults and the subduction interface contribute the majority of earthquake hazard to the Rankine Brown building with return periods of about 500 years or longer. They provide long-duration earthquake motions that are strong in long-period content.

The El Centro record has been long used as a de facto standard strong-motion accelerogram. It was included mainly for the purposes of comparison.

The scale factors for the accelerograms were determined to provide the best fits of their spectra to target spectra based on a seismic hazard analysis for the site. The target spectra correspond to the spectra required for various return periods for rock sites in central Wellington in the draft Australia-New Zealand Loadings Standard (Standards Australia/Standards New Zealand 2002) with two alternative sets of modifications to account for near-fault effects.

6 RESPONSE - TIME HISTORY ANALYSIS

A three-dimensional model was developed for the whole building to carry out "time history" dynamic analysis in ETABS v8.01. This allowed an assessment of the dynamic performance of the building when subjected to the suite of accelerogram records and supported by base-isolation bearings of varying characteristics located in the base of each of the 16 main columns.

One of the objectives of the analysis was to gain an understanding of the effect of the base isolation on reducing the moment demand on the waffle ribs where they frame into the column head. In this area the moment-curvature relationship of the ribs is significantly different for positive (as against negative) rotation. The ETABS program cannot model non-linear behaviour in beams with this difference. Therefore, a model was set up in STRAND 7 where the moment-curvature relationship could be more accurately modelled.

The analysis and design was an iterative process to gain an optimum compromise of initial and ultimate stiffness of the base isolation bearings to minimise the response of the building, and hence to gain an acceptable level of protection for the waffle rib hinge by reducing the ductile demand. A

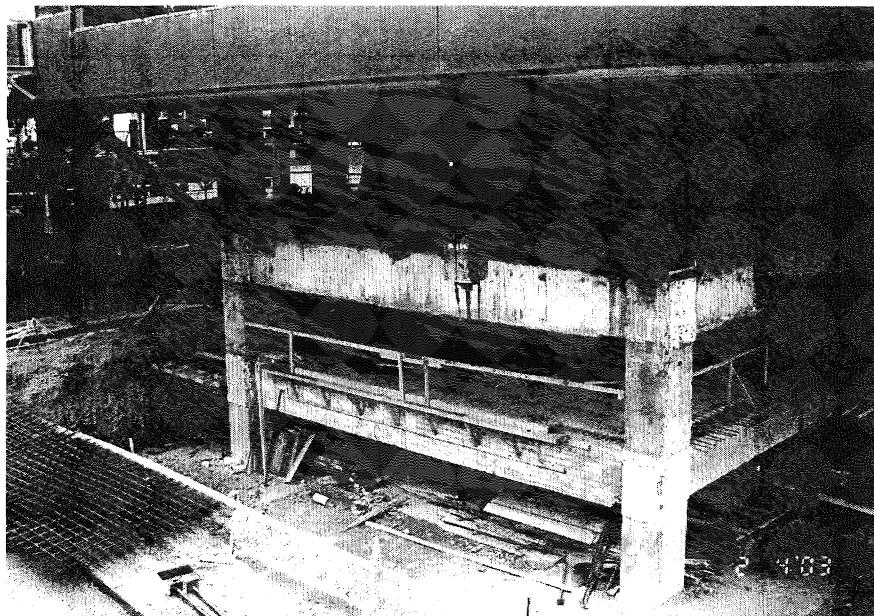


Figure 4: Southern extension of two basement levels. Excavation exposed original column and beams.

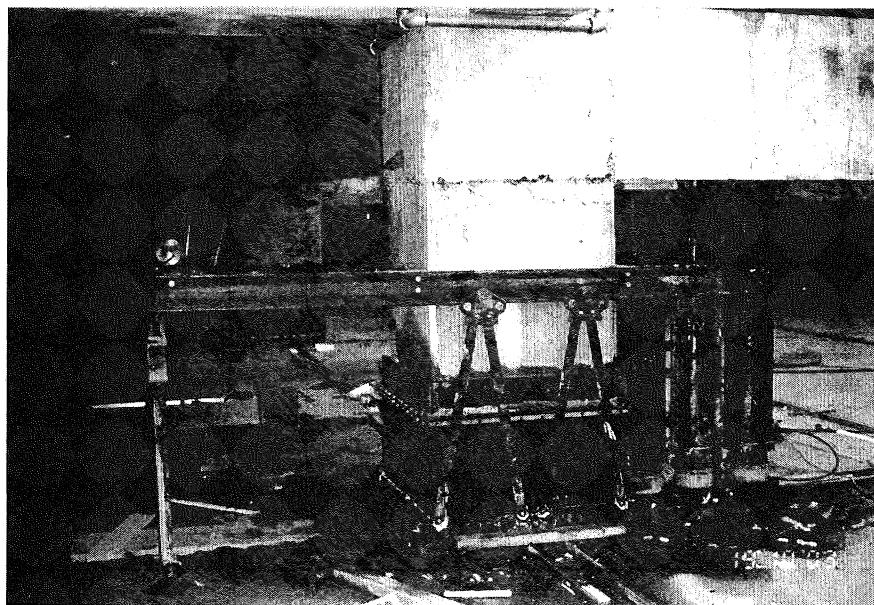


Figure 5: Adjustable trolley for removing cut column section and introducing base isolation bearing.

reduction of the order of 30 % in peak moment demand was achieved with a bearing that had an:

- initial stiffness of 12.4 MN/m up to 500 kN shear load,
- ultimate stiffness of 1.80 MN/m up to 500 mm of total shear displacement.
- axial stiffness of 2,600 MN/m.

7 BASE ISOLATION BEARING

The required design characteristics were achieved with a rubber/steel plate laminate bearing:

- 970 mm square in plan.
- 478 mm high, made up of:
 - top steel plate 25 mm thick.
 - 24 rubber laminates 14 mm thick.
 - 23 steel laminates 4 mm thick.
 - bottom steel plate 25 mm thick.
- 4 – 133 mm diameter lead plugs inserted for the full depth of the bearing.
- weighing 13 kN.

The bearings were manufactured by MIN Industries Sdn. Bhd., Malaysia to the requirements of Robinson Seismic Ltd., Lower Hutt. 18 bearings were produced, with the first two undergoing prototype testing to full load and displacement (11,850 kN axial load, 1,690 kN shear load, 587 mm shear displacement). From this testing a minor adjustment was made to the rubber formulation to increase the stiffness of the bearings. The first two production bearings were tested to 220 mm and 440 mm displacements so that a comparison could be made with the results of the prototype testing. The remaining 14 production bearings were tested to 220 mm displacements. All testing was carried out by Robinson Engineering on their test rig at the MIN Industries Sdn. Bhd. Laboratory, and were independently witnessed by Dunning Thornton, Malaysia Sdn. Bhd.

8 INSTALATION OF BEARINGS

McKee Fehl Constructors won the contract to carry out the Structural Upgrade of the Rankine Brown building in August 2002. The 14-month contract included the supply and installation of the base isolation bearings into the base of the 16 main columns. Refer to Figures 5, 6 and 7.

The tender documents outlined a methodology for the support of the superstructure, cutting of the column to provide a slot for the bearing, and fixing of the bearing element to the remaining sections of column. The tenderers were requested to submit alternative methods if they considered that they had a more practical and cost-effective

approach. The details of whatever method was to be used were “design and build”. The draft tender programme allowed for two bearings to be installed in a 4-week period. With innovative techniques and equipment, the contractor was able to prop and cut the column and install a bearing in 4 ½ days. This work took place while the building was occupied and fully operational. Therefore, security, noise, dust and water control were issues that were managed by good co-operation between the contractor and the University.

The sequence of operations for the installation of each bearing was:

1. The location of temporary propping to take the full axial load of the column (refer to Figure 6). 3 props and associated hydraulic jacks were fitted on the north and south sides of the Level 0 column.

2 props and associated hydraulic jacks were fitted on the north and south sides of the Level 1 column.

1 prop and associated hydraulic jacks were fitted on the north and south sides of the Level 2 column.

This allowed in the order of one third of the axial load to be picked up from the column into the props through the existing column heads at Levels 1, 2 and 3. The post-stressing of the waffle slab through the column head allowed this level of load transfer to take place between column and offset props. The props were 200 mm square hollow steel sections with a 9 mm wall thickness. The individual jacks and props could be manhandled into place and shimmed with no interference to the library operation. The props were non-intrusive and only required minor shifting of furniture.

2. The jacking of props to pick up the column load. Each set of floor jacks was connected to the one manifold to ensure the same load was in each prop at any given time. Once the required load was applied, a locking ring on the ram was screwed down so that the load could be taken safely through a mechanical system, and the hydraulic pressure released. The jacking sequence allowed the load to be picked up progressively, without overloading any one column head: 1/3 of load at Level 0, 1/3 of load at Level 1, 1/3 of load at Level 2, 2/3 of load at Level 1, full load at Level 0 to produce tension in the column at Level 0. Tension in the column was identified with a crack opening up at the mid-height of the column on the plane of the original construction joint. Tension occurred when in the order of 8,000 kN was applied through the Level 0 props. Good agreement was obtained between elastic rebound of the column lengthening and load pick-up on the props within the storey height.
3. The provision of protection barriers and environmental controls (air extract) around each column, particularly in library storage areas.
4. The use of a wire-saw to provide two cuts (lower first,

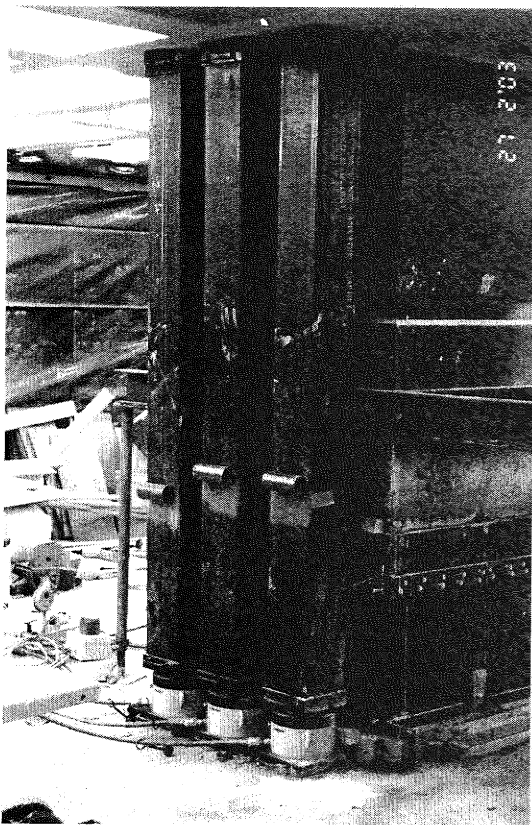


Figure 6: Props and jacks at Level 0. Three props each side of column.

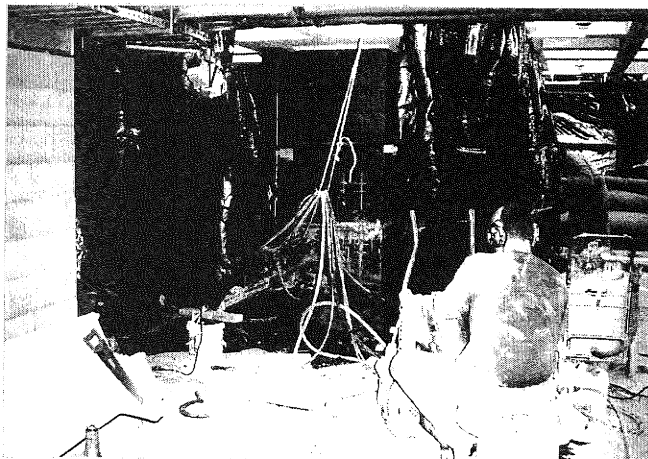


Figure 7: Set up of wire-saw to provide horizontal cut through main column.

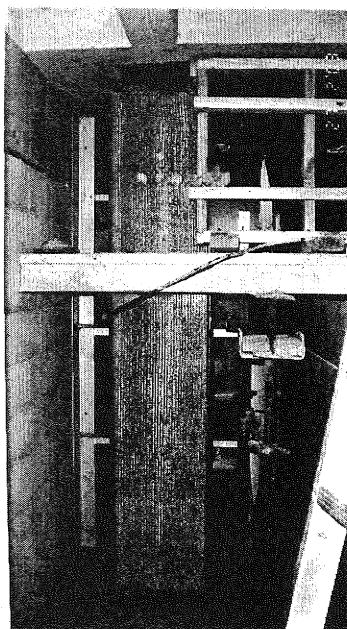


Figure 8: Podium perimeter column (original) with composite fabric wrap and hinge top and bottom.

then upper) through the column to isolate a section of the column that allowed space for the base isolation bearing. The continuous wire of the saw had diamonds embedded in nodules along the wire and was driven by a hydraulic motor to pull the wire around the column at a constant speed; approximately 5 m/s. Tension was continuously adjusted to apply a constant pressure to the cutting surface. Refer to Figure 7. The wire cuts through the concrete and steel reinforcing of the column in less than four hours. The resultant cut was generally planar - except where the wire rode up on an original stirrup that had not been placed in a true horizontal plane.

5. The lifting out of the section of column by fixing to a trolley as shown in Figure 5. The rails on which the trolley was supported were adjusted for height so that the section of concrete could be rolled out onto a flatbed trolley for removal from the building.
6. The fitting of a steel "shoe" to the upper section of column and temporarily holding it in place. This shoe provided the shear connection between the bearing and the existing column.
7. The introduction of the base isolation bearing into the gap with the use of the trolleys that had removed the concrete block. The shoe was lowered on to the top surface of the bearing and fixed together with bolting plates. The bearing height and level was adjusted with screw jacks supporting the ends of the rails that supported the trolley.
8. The grouting solid of the space between the inside of the shoe and the upper column section with cementitious grout, and similarly the space between the bottom plate of the bearing and the lower section of the column.
9. The removal of the trolley once the grout had gained the required strength. The props were then de-stressed in the reverse sequence of loading to allow the column load to be applied to the bearing. The compression in the bearing (less than 4 mm) was taken up by progressive settlement of the whole building by that amount. Bearing installation was one column at a time, working from the centre of the building towards each end, and not allowing one end to progress more than two columns ahead of the other. An exception was made for the southern end where the last four bearings were installed into the columns that had originally been back-filled and subsequently excavated as part of this Structural Upgrade to provide additional floor area.
10. The fabrication and installation of heavy steel angles to fix (by bolting) the base plate of the bearing into the foundation pad of the column.

9 PERIMETER COLUMNS SUPPORTING PODIUM

Supporting the perimeter of the three-storey podium are a

series of 457 mm square reinforced concrete columns. The column width was considered too small to fit a sliding bearing that could cater for 600 mm of relative horizontal movement and an ultimate axial load in the order of 2,260 kN. Therefore, the columns were modified to allow the column to sway. This was assisted by the original reinforcing at the top and bottom of each column being detailed as a hinge. In the lowest level of relative movement between ground and the superstructure, the concrete around the hinge reinforcing was cut out and the bundle of eight bars wrapped with carbon fabric saturated with epoxy resin to inhibit buckling of the bars. Refer to Figure 8. The column faces between the hinge points were also wrapped with glass fabric saturated with epoxy resin to enhance the shear capacity of the column to ensure that yielding of the hinge bars occurred under sway action without column failure.

10 SUPERSTRUCTURE SEPARATION

To allow the superstructure to sway freely plus or minus 600 mm on the base isolation bearings, the building was separated from the ground around the northern and southern perimeter at Level 1, and the western perimeter at Levels 1 and 2. The eastern perimeter was open down to Level 0. The separation was generally provided by cutting out the capping slab over the services space between the retaining wall and the inner concrete block masonry "dry wall" to the occupied spaces in the basement area. The capping slab was replaced with sliding steel cover plates fixed to the superstructure, but free to slide over the top of the retaining wall below. During an earthquake that causes significant movement of the base isolation bearings, some damage will occur along the western perimeter to the concrete block paving at Level 2 or the internal finishes at Level 1 that cover the steel plants. Damage will also occur to the frangible walls that cross the seismic movement gap.

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