

## UNION HOUSE — A CROSS BRACED STRUCTURE WITH ENERGY DISSIPATORS

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### ABSTRACT:

This paper describes the design and construction of a twelve storey office building which has base isolators (i.e. energy dissipators) and external cross bracing to resist lateral loads.

The computer analysis techniques used are described and some comparisons are made with non-base isolated structures.

The system shows a practical method of using base isolation and energy dissipation techniques in a multi-storey structure. The construction of the building has proved the system to be advantageous in terms of both cost and construction time.

### 1. INTRODUCTION:

Recent research has identified means of dissipating seismic energy in mechanical devices located within the structural system. The devices are intended to dissipate energy within themselves, leaving the structure as a whole undamaged. Thus the need for ductility in the structure is removed. Significant and detailed research, both of an analytical and experimental nature has been carried out and it is now feasible and economical to use energy dissipating devices as part of a commercial design.

Because little or no ductility demands are required of the super-structure, cross-bracing may be used. This both reduces lower mode effects and provides an economical structure. To allow the dissipators to function, the base of the structure must be free to move, and this is achieved by the use of long piles pinned at each end and separated from the ground.

The design described was developed by the office of Holmes Wood Poole & Johnstone, Christchurch. Final design was carried out in the latter half of 1981. A separate piling contract was completed between December 1981 and March 1982. The building itself started in March 1982, and in February 1983 the top floor (level 12), had been formed.

### 2. DESIGN OF BUILDING:

#### 2.1 Siting

The building is located in Quay Street, Auckland, and has a commanding view over the Waitemata Harbour and the North Shore. It was therefore architecturally important to keep the façade as open as possible.

The site is bounded on three sides by

streets and on the fourth side the building has been separated from its neighbour by some 12 metres, to satisfy Town Planning requirements. Such unusually large separations from boundaries have allowed modest fire rating requirements which do not include any solid or near solid walls. Complete freedom in choice of structure has thus been possible.

The site is located on the edge of an old valley formed in the underlying Waitemata Series sandstones. This valley has been filled with natural marine deposits and more lately has been reclaimed with pumped in hydraulic fill. The sandstone is about 10-13 metres below present street levels.

#### 2.2 Architecture

Refer to fig. 1. The building has 12 floors plus a ground floor/plaza, which is sited approximately 1.5 metres above footpath level. Beneath this there is a carparking basement. At the top of the building are two small additional floors for plant and lift machinery. It is approximately 25 metres square and has 3 bays in each direction of 8.4 metres. The columns are circular, and there are 4 internal columns supporting the floors. The services core, containing lifts, stairs and toilets, is located on the south side of the building, and is constructed of timber framing, fire rated with plaster-board.

The external window line is set back from the centreline of the exterior frames by 600 mm, to provide sunshading and access for window cleaning. The external frames and the bracing will be very clearly expressed and will be finished in a gloss white in the completed building.

#### 2.3 Structure-Gravity Loads

The typical floors consist of precast, pretensioned, hollow core planks with an insitu concrete topping, supported on precast beams at the edges of the building and insitu concrete beams running north-south only, internally. The beams are supported on circular columns, which were cast insitu, using concrete pipe sections as permanent formwork. The ground floor slab and basement slab are cast insitu, two way flat slabs, with no beams. Base-

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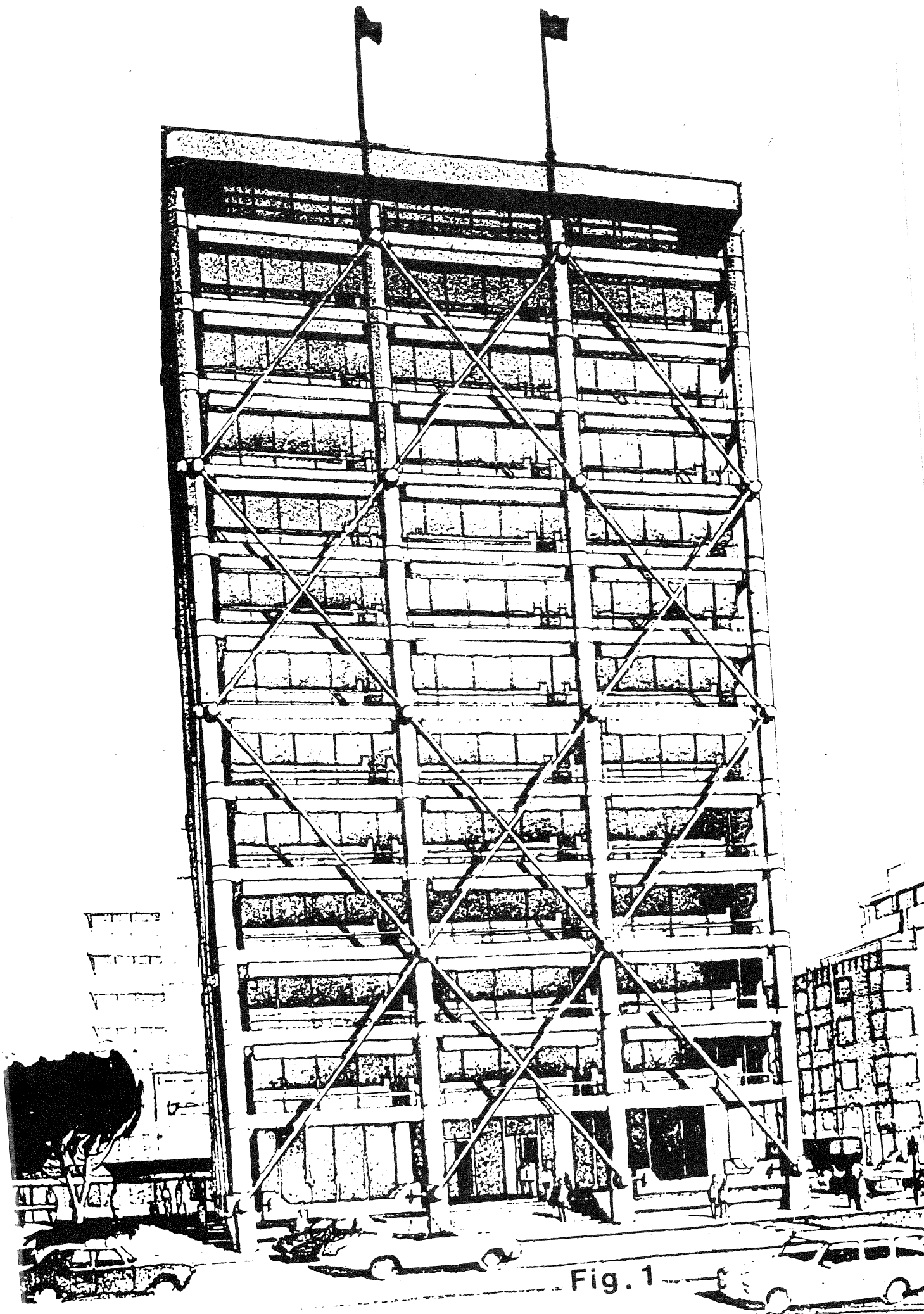


Fig. 1

ment walls were precast with insitu concrete jointing strips. Water-proofing has been successfully achieved with double PVC waterstops at every joint, and dense concrete cast in small sections.

The superstructure is supported by bored piles which are sunk about 2.5 metres into the sandstone. The basement ground floor structure floats on the overlying fill material.

#### 2.4 Structure - Lateral Loads

Refer to figs 2 and 3. Lateral loads in the superstructure are carried by a 4 bay high vertical truss on each of the four faces of the building. The truss is comprised of the columns which form the chords, every 3rd level of beams which form the verticals, and the diagonals themselves which are structural steel encased in concrete to form a circular section. Loads from floors between the node points at every third floor are carried by frame action to these nodes. At the base of the superstructure the bracing is connected directly to steel plate, cantilever type energy dissipators.

Each dissipator is anchored into a fabricated steel housing, which in turn is cast into a beam projecting above ground floor slab and firmly connected to it. The loads from the dissipators are carried from this beam into the ground floor slab, then to the basement walls and so into the surrounding ground. The basement/ground floor structure is not connected to the superstructure columns, except by the 16 dissipators. The columns pass outside the basement walls and internally columns pass through steel plate tubes which complete the waterproofing system. 150 mm separation has been allowed.

The columns are supported on piles, which allow movement of the superstructure relative to the ground. This is achieved by forming the load carrying pile inside a steel jacket which retains the ground away from the pile. Each pile has a hinge built in at each end to give adequate lateral flexibility. Various beams are provided at ground level to prevent column/pile buckling between 1st floor and bedrock. The corner piles are belled to resist uplift, and the rest are straight bored as no tensions are possible.

### 3. OTHER STRUCTURAL FORMS:

Other forms investigated during the preliminary design phase were two-way concrete frames, peripheral concrete frames, and a cantilevering shear core. The form finally chosen allowed an open and very light structural façade on all sides, and a maximum of precast elements. Cost and time comparisons showed also that a capital cost saving of \$300,000 and a construction time saving of 3 months representing \$150,000 approximately have also been achieved.

### 4. DESIGN BASIS:

#### 4.1 General

The overall approach is essentially

that advocated by the New Zealand National Society for Earthquake Engineering subcommittee on the design of base isolated structures(1).

#### 4.2 Ductility

The design of Union House avoids the need for providing ductility throughout the structure by ensuring that excess seismic energy can be absorbed at specific locations - the energy dissipators. The dissipators limit the amount of energy and thus force which can be transferred into the structure above, to a value which can be assessed. Yielding of the dissipator increases the period of the structure and seismically induced forces are decreased.

The building was analysed for two levels of seismic force. Firstly, a design earthquake was chosen, that represented the load level present in NZS 4203 for the Auckland area. This was called the design earthquake and the structure was designed to remain elastic during it, with the dissipator having minor excursions into its yield range. No damage is expected to the structure at this load level. Secondly, an earthquake was chosen that represented the maximum likely earthquake that may occur in the Auckland area. This is referred to as the "maximum credible" earthquake, and the building is designed considering this load as an ultimate load. Some limited yielding of members may occur and the dissipators are used to their full extent. Specific detailing for ductility in individual members is not done, as at worst they undergo strains only slightly past yield for 1 or 2 cycles.

Member actions from the maximum credible earthquake were always critical and controlled the design.

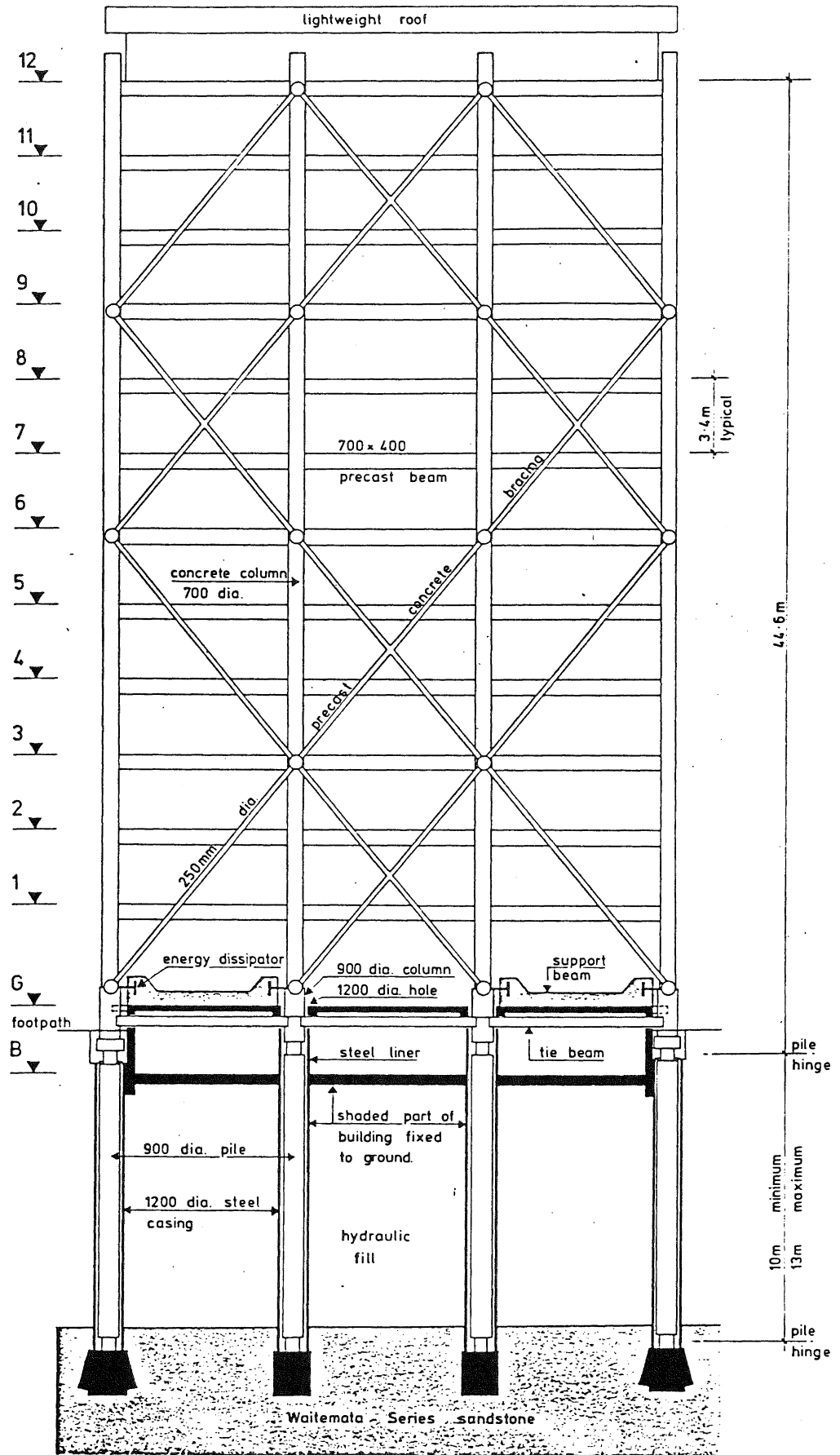
#### 4.3 Earthquake Choice

Dr J. Berrill of the University of Canterbury assisted by providing digitised earthquake records which he had assessed were appropriate to the site.

To achieve consistency of performance between this design and a building designed to NZS 4203, design earthquakes chosen to have similar spectra to that implied by NZS 4203 for Zone C were February 9, 1971 San Fernando Valley recorded at 646 South Olive Ave, and December 30, 1934 El Centro Imperial Valley. The earthquake chosen as the maximum credible earthquake was May 18, 1940 El Centro Valley, north south record, Scale Factor of 1.0. The earthquake chosen is a conservative estimate of the worst that could strike Auckland and its probability of occurrence is very small.

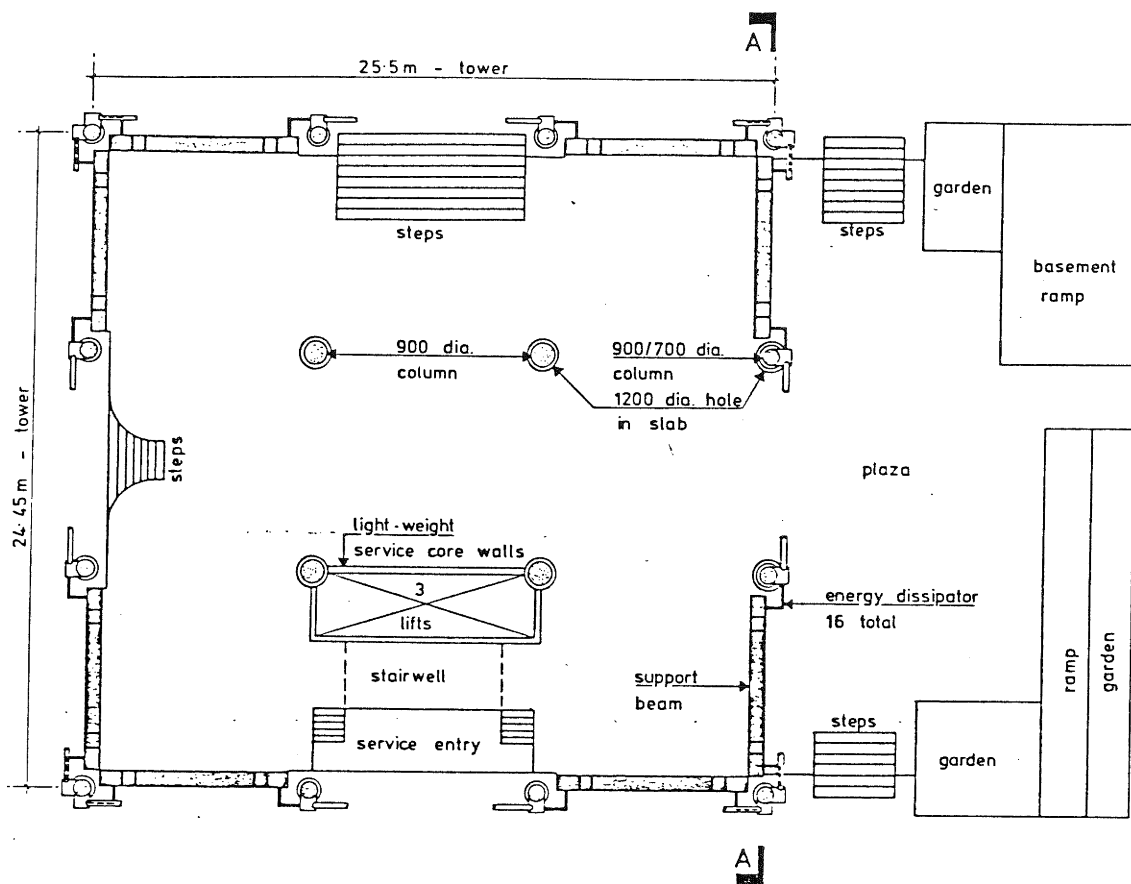
#### 4.4 Soil Parameters

As earthquake motions must be transmitted from the bedrock through the overlying fill to the structure the effect of that fill was considered in the analysis. The elasticity and damping values for the soil were not able to be given precise numerical values, so upper and lower bound values were assessed in the analysis.

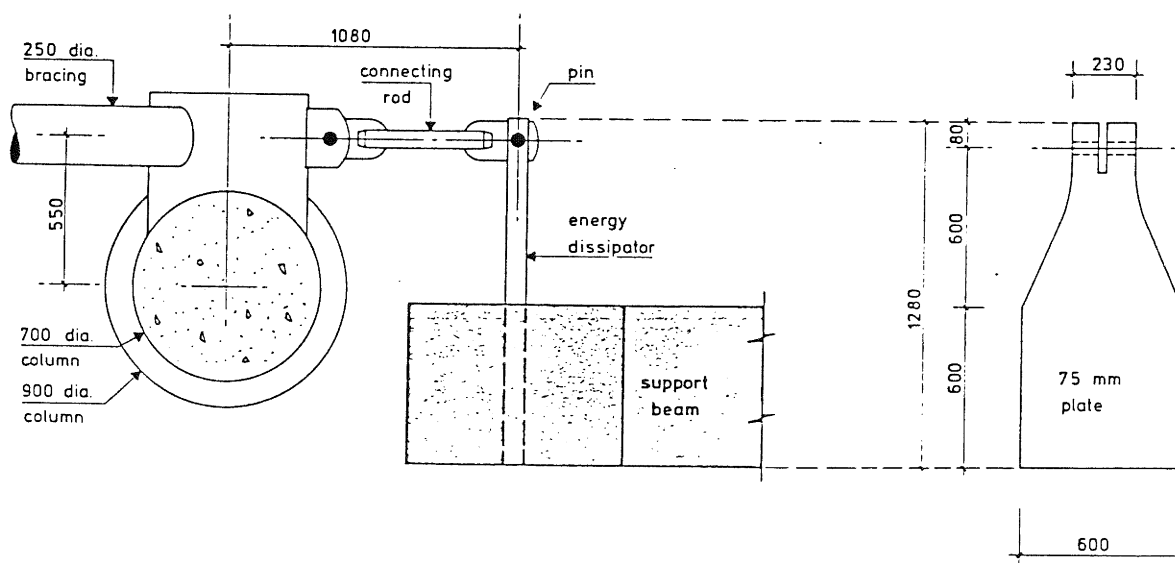


TYPICAL CROSS-SECTION A - A

Fig. 2



GROUND FLOOR PLAN



TYPICAL ENERGY DISSIPATOR ASSEMBLY PLAN

ENERGY DISSIPATOR

Fig. 3

Variations in soil stiffness were significant but variations in damping within the range were not.

## 5. ANALYSIS:

### 5.1 Preliminary Analysis

The preliminary analysis was carried out by hand without the aid of the computer. A plane frame was analysed for a triangularly distributed load of 0.06 g, and the dissipators were sized to resist 0.04 g. Wind loads amount to 0.025 g. The results enabled preliminary member sizes to be established and estimated costs to be found.

### 5.2 Static Analysis

When the full design was undertaken the first step was to check the static analysis noted in section 5.1 above using a plane frame computer programme. The results again checked member sizes, but more importantly provided a benchmark against which to assess results from later dynamic analyses. This was most useful for checking purposes. Since the frame was analysed with all beams, columns and braces modelled, a good indication of frame and bracing interaction was obtained. This will be discussed in more detail later.

### 5.3 Simplified Dynamic Analysis

A simple frame consisting of 1 bay in width, 4 stories high, with cross bracing in each bay, was modelled with the piles and their hinges, the dissipator, and the soil all included (fig. 4). The properties of the beams, columns and braces were assessed so that a reasonable approximation to the stiffness of the full frame was obtained.

The dissipator was modelled as a single bilinear elasto-plastic spring, with its properties being the summation of four real dissipators. The soil was modelled as an elastic spring, with a dashpot for damping, and mass lumped to it to simulate the soil between bedrock and basement, and the mass of the basement itself.

This frame was analysed for static loads initially, to check that it did have similar stiffness to the real frame, and then 6 further runs were made: 3 earthquakes each with two different soil springs. For each case deflections and forces in the members were compared, and the worst combinations identified. A back check with the full frame static analysis deflections indicated that preliminary member sizing would not need alteration.

### 5.4 Final Dynamic Analysis

The full frame as modelled in section 5.2 was then analysed under one design earthquake and the maximum credible earthquake with dissipator and soil spring in place, and with pile members modelled as bilinear elasto-plastic elements (fig. 5). At this stage the bracing could yield in tension only, and would carry no compressive force. The two critical combinations both had the more flexible soil

parameters and were San Fernando 1971 from 10 seconds, as the design earthquake; and El Centro 1940 from 0 - 12 seconds as the maximum credible earthquake. The full earthquake records were not required, as the simplified analysis revealed that nothing which might affect the design occurred outside of the times used.

Results from the full analysis were compared with the simplified structure and were found to be similar in both deflections and structure period, leading to the conclusion that the results were certainly accurate enough to identify the critical earthquake/soil combinations.

Subsequent to this final analysis the construction of the bracing was changed from steel tube to concrete encased structural steel. The bracing could now act in compression, so this section of the analysis for both earthquakes was repeated.

Finally a further programme was written to search for and identify the worst combination of forces in the braces at the joints, to allow their design. These combinations did not necessarily occur when brace forces were at their maximum.

## 6. COMPUTER ANALYSIS PROGRAM:

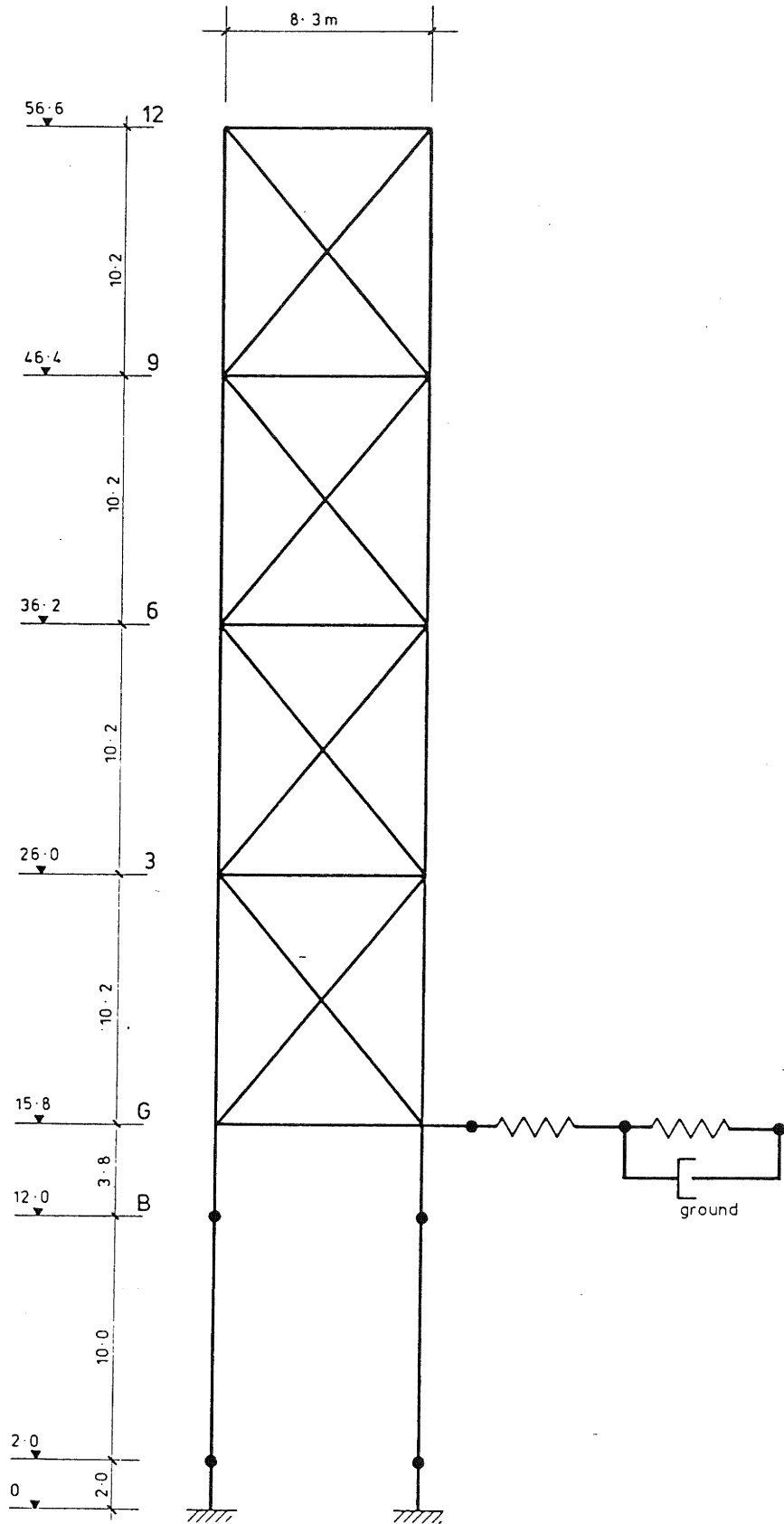
The inelastic seismic analyses reported in this paper were carried out using the non-linear dynamic analysis program 'RUAUMOKO'. This program for analysing an arbitrary shaped two-dimensional frame was initially written by Sharpe (4) but has been extensively modified during the past decade by Carr (3) to extend its capabilities into a very versatile analysis for use in both research and design.

The program is capable of considering both lumped and consistent mass(5) representations, has a wide range of damping models to represent the viscous damping characteristics of the structure (6) and has a wide variety of inelastic member models available to characterise the inelastic behaviour of the structural components of the frame.

Beam and beam-column members are usually considered as an elastic prismatic member with possible inelastic spring hinges at the ends. Yield of these springs is governed by yield moments at the ends of the beam members and in terms of an axial force-bending moment interaction diagram for beam columns. Bilinear, Ramberg-Osgood, Degrading Stiffness, Pinching and members developing slackness models are available for the hysteresis rules governing the post elastic behaviour of the hinges.

To allow for the representation of foundation behaviour or for the effects of localised viscous damping, a dashpot member may also be used. This has axial as well as end-rotation dashpot actions available. In these analyses such a member was used to model the dissipative characteristics of the soil between the bedrock below the structure and the basement.

For the initial analyses the bracing



COMPUTER MODEL : SIMPLIFIED FRAME

Fig. 4



members were modelled as 'no compression' members where tension yield would mean the development of slackness in the members. It was found, however, that tension yield was only just reached with the full El Centro excitation and thus when the design was modified to allow the braces to take both tension and compressions it was considered that the braces would remain elastic. This greatly simplified the analyses as the only members that needed to be assessed at each time stop to check for inelastic behaviour were the piles and the dissipator member. The braces were checked at the end of the analyses to verify that some of them had, in fact, reached yield.

In the analyses described a lump mass model was used for the mass representation. Proportional or Rayleigh damping was used, for its simplicity, in representing the viscous damping but with the parameters chosen to achieve realistic amounts of damping over the range of natural frequencies exhibited by the structure. With the inelastic behaviour limited to a few isolated locations in this structure care was required to ensure sub-critical damping in the higher modes of free vibration.

For all analyses a time step of 0.01 seconds was chosen for the time integration of the seismic response as experience has shown that such a value yields reliable results for structures of this type without exorbitant computational cost. The integration method used is Newmark's Constant Average Acceleration method which is unconditionally stable.

As the computer programme (3) is continually undergoing development it has been completely rewritten during the past year to enable these future enhancements to be achieved with ease but also all machine dependence has been removed enabling the programme to be run on any computer with a 16 bit (or greater) integer word length.

## 7. RESEARCH AND DESIGN:

The use of inelastic dynamic analyses for design is still a relatively rare procedure for several reasons.

Firstly, the cost of such analyses is considerable, particularly as many analyses are required to achieve a sufficient sample of the structures response to the different types and size of earthquake excitation. Secondly there is the problem of what earthquakes should be considered as being sufficiently testing for this structure. In most code spectra all the particular characteristics of appropriate earthquakes have been smoothed and averaged. This is not available to the user of a non-linear time-history response. Thirdly, the analyses cannot be carried out until the design is complete as all the yield levels as well as the post yield behaviour are required to be known before the analysis can be set up making the design machine difficult.

The designer of unusual structures is however left with little alternative as the assumptions of structural behaviour

implied in most codes may not be exhibited by his structure and thus the code design rules, as well as the design spectra may not be applicable. He must therefore convince himself that his structure exhibits no abnormal behaviour when subjected to a range of likely excitations and that the levels of protection provided to more conventional structures by the code loadings and design methods in accepted practice are available to his structure.

It is to be hoped that future research into structures of this type having base energy dissipation will lead to the formulation of design methods enabling designers to use the less expensive analyses available to the designers of more conventional structures.

## 8. ANALYSIS RESULTS:

### 8.1 Steel versus Concrete Bracing

Having to reanalyse the structure with braces which work in both tension and compression (concrete bracing), as compared to the original analysis with braces in tension only (steel bracing) gave some interesting comparisons. The concrete structure was stiffer as it effectively has twice as much bracing working. Interstorey deflections and total structure deflections were lower. The increased stiffness in the structure lead to slightly higher dissipator deflections (150 mm compared with 120 mm). The period of both structures was in the order of 2 seconds indicating that the dissipator dominates first mode effects.

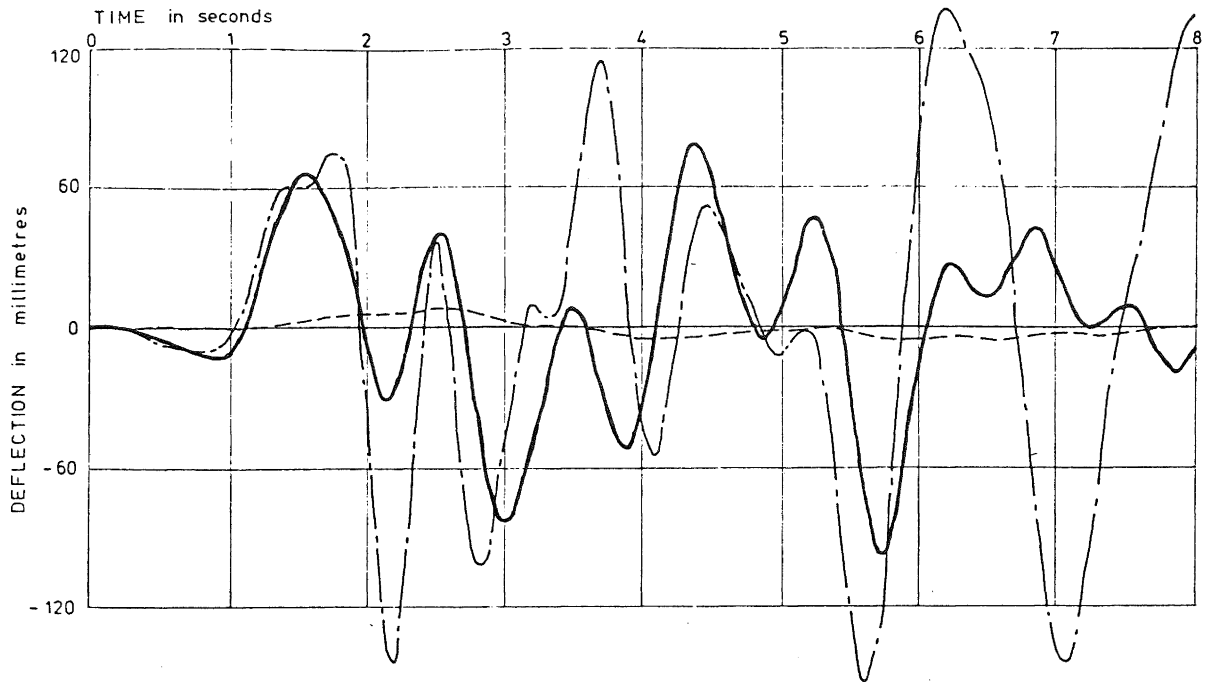
The graphs (fig. 6) relate to the concrete bracing case.

### 8.2 Structure with no Dissipator

A further analysis was carried out with the dissipator removed and the building restrained laterally only by the flexible piles. The results indicate that an almost perfectly isolated structure such as this experiences virtually no lateral load. Interstorey deflections (fig. 6) and hence bracing forces are very small. The structure is not unstable, and does not drift away in one direction only, although the deflection between ground and structure is large at 280 mm. Such a deflection would be difficult to accommodate. In any case some positive lateral load resisting mechanism must be provided to resist wind forces.

### 8.3 Rigid Dissipator

A computer run was done with a completely rigid dissipator i.e. the base of the structure is slaved to the motion of the basement. As expected the natural period decreased and bracing forces and interstorey deflections increased dramatically. Some smoothing of the earthquake input occurs between the bedrock and the basement, but there is insufficient flexibility in this mechanism to sufficiently damp out the forces which are imposed on the structure. At the end of the analysis (after 8 seconds of El Centro) the structure was still vibrating strongly whereas in

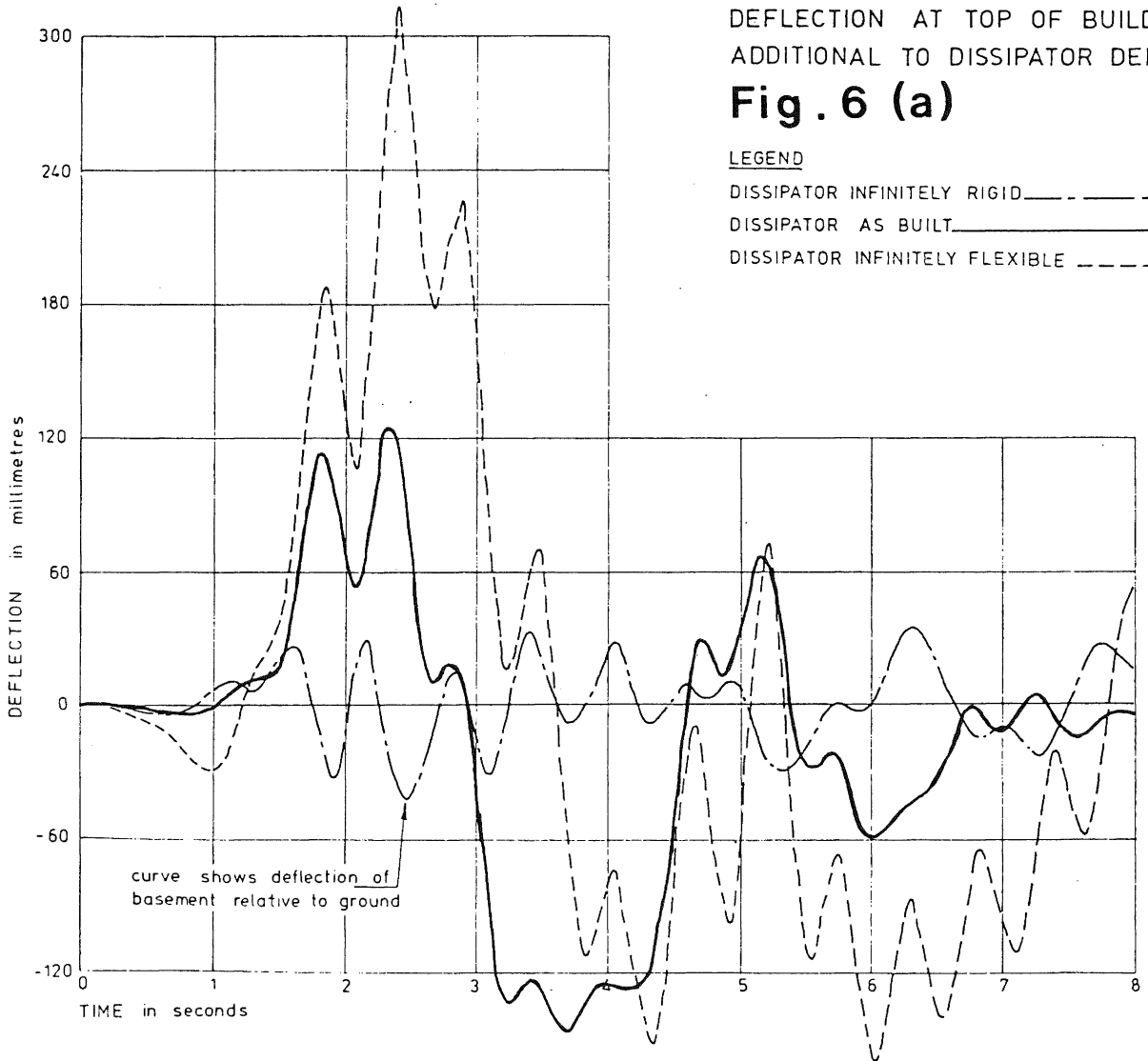


DEFLECTION AT TOP OF BUILDING  
ADDITIONAL TO DISSIPATOR DEFLECTION

**Fig. 6 (a)**

LEGEND

- DISSIPATOR INFINITELY RIGID -----
- DISSIPATOR AS BUILT \_\_\_\_\_
- DISSIPATOR INFINITELY FLEXIBLE - . - . - .



DEFLECTION AT DISSIPATOR

**Fig. 6 (b)**

all previous cases, the motion at this time was of no consequence.

This analysis shows that had dissipators not been used braces with more than twice the capacity they now have would have been required, or considerable ductility would have had to be built in.

#### 8.4 Comparisons

Refer to fig. 6; the choice of dissipator capacity is a good compromise between being too flexible (and allowing deflections relative to basement to be too large), and so stiff that forces in the structure are unnecessarily large. The rigid dissipator case emphasises the importance of keeping the dissipator strength as low as practically possible so that it will be fully effective in reducing structural frame actions. As well as indicating that a base isolation system without energy dissipation devices is not a practical possibility, the "no dissipator" case shows the importance of minimising the force which can be transmitted through the isolation system. Even these very flexible piles input some force into the structure.

### 9. DESIGN:

#### 9.1 Beams and Columns

The computer programme used (3) does a search throughout the entire time history analysis and prints at the end of each run the maximum and minimum shears, moments and axial loads occurring in every member, and the time during the earthquake record at which each occurs. The distribution of these forces up the building was plotted, for each bay or column line. To these were added forces from gravity loads.

For the beams the value of moment at the end of each beam was fairly constant except for some increase at each third (node) level. This is not surprising as these moments are induced by the sway of the frame which is nearly constant from floor to floor, with the bracing forcing the frame to deform in a shear mode predominantly. Accordingly it was decided to reinforce for a constant moment up the height of the building except for the top two floors. The value chosen was 540 kN.m, of which 140 kN.m is contributed by gravity loads. The actual moment diagram shows some overshoot at levels 3 and 9 above this value. This is not expected to be a problem as these higher moments occur only briefly (not longer than 5/100 of a second), once in the time history. Ductility will not be a problem as there are almost equal quantities of top and bottom reinforcing. Shears are low because of the long span of the beams, and nominal reinforcement is ample to carry all shear forces.

It is interesting to note that the seismic moments induced in the beams are significantly greater than the gravity load moments. This was unexpected, because the cross-bracing nodes are only 3 storeys apart. The rigorous analysis undertaken here has shown that such frames,

provided essentially for resisting gravity loads, will attract incidental forces in excess of those found by simple analysis, and will therefore be required to exhibit significant ductilities if adequate reinforcement is not provided.

A similar approach was taken for the columns. Bending moments were smoothed to a constant value. Axial loads showed sharp increases at each node point of the truss (every third level) so the reinforcing was designed accordingly. Biaxial effects on the corner column were considered. 3% reinforcing was placed in these columns, which gave them the capacity to resist the following load cases :

- (i) the most unfavourable moment and axial load separately in a uni-axial direction, and at 45° to the column axes (the same load, as the column is round). The latter results from 0.7 of the worst seismic force in both directions simultaneously. These actions are derived from the maximum credible earthquake.
- (ii) the most unfavourable moment and axial load resulting from the design earthquake in both directions simultaneously.
- (iii) the most unfavourable moment and axial load resulting from the maximum credible earthquake in one direction and one-third of this value in the other. Since this maximum value occurs uni-directionally once only and then for only 1/10 of a second, it was considered improbable that it could occur simultaneously in each direction.

The worst combination always involves tension in the corner column, and should occasional yielding occur this will not be significant.

Internal columns are not highly stressed and nominal reinforcing is sufficient. Similarly, as all shear forces are low nominal spiral reinforcing only is required.

There was no need to carry out capacity design procedures, and use actual beam yield moments and brace yield forces summed to give column actions, as the maximum likely forces had already been predicted in the analysis procedure. The energy dissipator/base isolation system effectively ensures that these forces will not be exceeded. Beam-column joints were not specially reinforced, as the majority will remain elastic. Should the occasional joint yield slightly in the worst of earthquakes, there is ample capacity for re-distribution of moments, and each beam can safely carry gravity loads in a simply supported condition.

When the bracing was changed to concrete from steel tube, the new analysis showed that interstorey deflections were slightly smaller and hence the beam seismic moments were also smaller. It was considered necessary to redesign the beam

only for a new total moment of 460 kN.m at the column face.

### 9.2 Braces

Maximum forces were given directly by the computer output and a search revealed that there was no advantage in reducing the size or strength of the brace up the building as except in the top level of bracing the maximum force varied only 10% between levels.

The original design was made in steel tube (200 mm O.D. API line pipe) which was to be site-welded between rolled steel plate joint units. Because of potential labour problems the braces were changed to concrete encased steel flats with bolted joints, which eliminated the site welding. The steel tube braces were intended to be filled with treated water to act as a heat-sink and provide the required fire resistance rating of 2 hours (7) (8). The whole bracing system was thus interconnected as a plumbing system to allow circulation of the water.

With the concrete encased braces, which are designed to act in both tension and compression, the braces were restrained back to the building to reduce the buckling length.

### 9.3 Bracing Joints

Refer to fig. 7. Moments generated by the eccentricity of the bracing beyond the column line are carried by the combination of a steel R.H.S. stub and reinforcing projecting into the joint region, and are taken out by columns and floor diaphragms. At the base of the building the energy dissipator is in the plane of the bracing, as because there is no floor-slab connected to the joint horizontal torsional moments must be minimised. The steel joints were fabricated in one piece and the braces connected via interlocking fingers of steel with friction grip bolts. Concrete encasement will be formed in place.

### 9.4 Energy Dissipators

This configuration of dissipator (fig. 3) was chosen because of its simplicity and the ease with which it could be replaced if required at any time in the future. The shape was selected to ensure that yielding will occur over the full length of taper, and that plastification moves in from the outside fibres of the steel as load increases, with non-yielding material obeying a linear stress strain law. In the analysis, step-by-step gradations of plastification are taken and the strain, curvature and moment in the section can be calculated. Hence the deflection at the end of the dissipator can be simply found. This information was plotted on graphs to give theoretical hysteresis loops. These were checked against those found experimentally and the size of the unit was checked and found to closely agree with that proposed in the charts prepared by Tyler (ref 2).

The unit is 75 mm thick, has a width and cantilever length of 600 mm and is made

from grade 43 C steel. It has a first yield force at the end of 231 kN. Full plastification corresponds to an equivalent static force of 0.04 x building mass for the 8 dissipators in each direction. At the design movement of + 150 mm the maximum strain in the steel is 3%.

Since previous tests had shown the welding usually initiated the failure of similar devices, the design completely avoids welding on the dissipator itself. Connection to the bracing is via a high tensile pin, close tolerance drilled through the dissipator plate. The fixed end is anchored into a sleeve fabricated from steel plate, which is built into the concrete beam. It has machine bearing plates welded into the two faces, through which the dissipator passes. The dissipator is removeable and could easily be changed if it remained in a bent condition after a major earthquake.

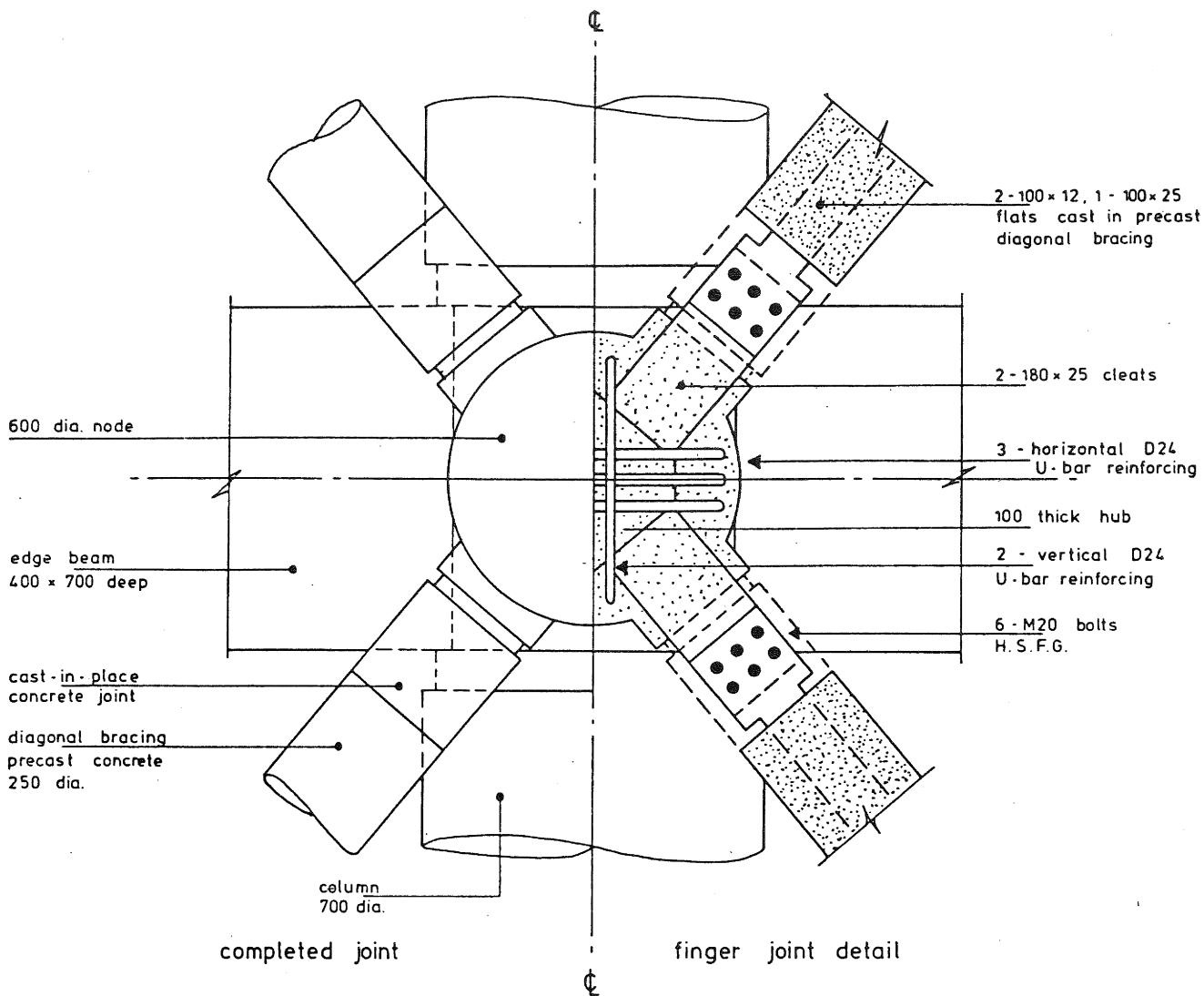
The dissipator was not tested as it was considered that sufficient testing of similar units had been carried out to give complete confidence in the behaviour of this particular design. The plate from which the units were manufactured was ultrasonically scanned to check for delamination or serious laminar inclusions (none were found) and bend, Charpy V-notch and tensile tests were performed to ensure it met specification requirements.

It is interesting to note that previous tests have shown similar dissipators able to endure several hundred cycles of loading. The analysis in this case shows that it need only withstand two or three cycles to full deflection and perhaps 10 more at limited deflection to ride through the full El Centro earthquake record.

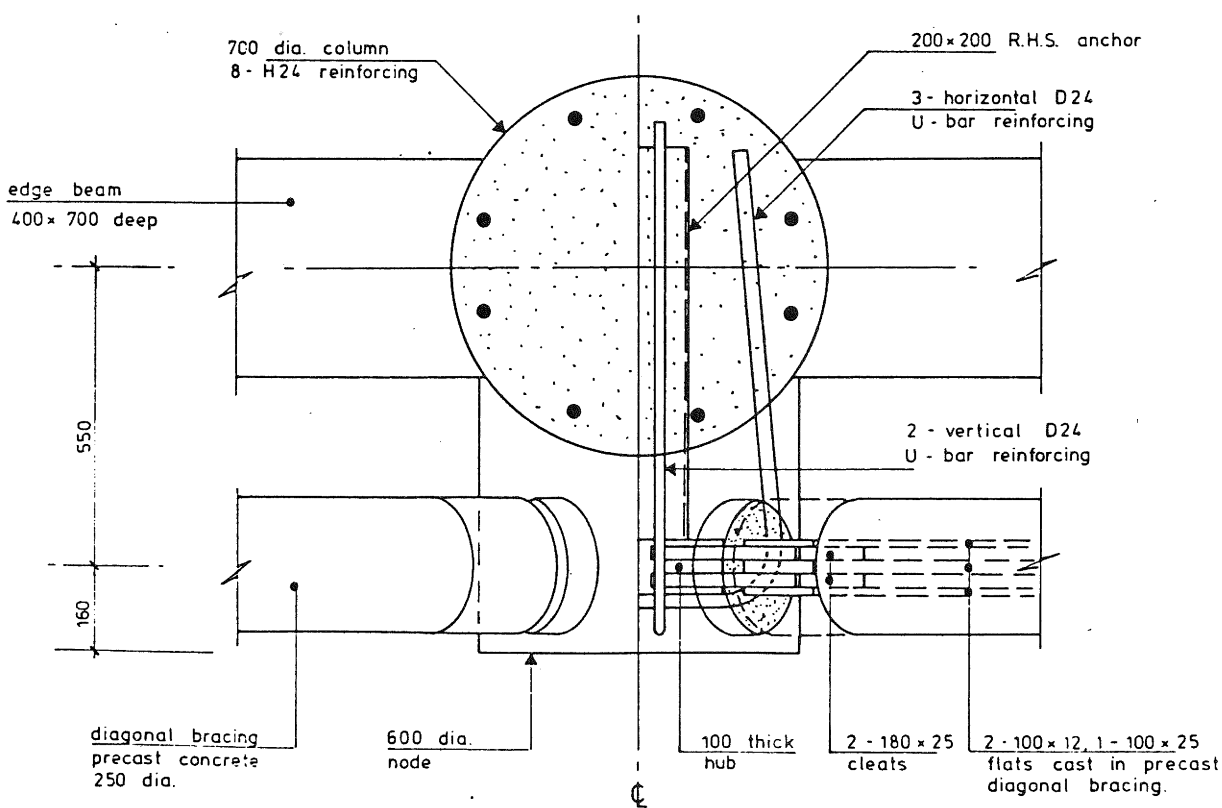
### 9.5 Piles

The piles are unique in that they both support direct vertical loads and also provide the base isolation system for the building. Because of their length only small curvature strains at each end are necessary to allow the required lateral movement. The piles were designed on a conventional basis but are enclosed in a tube to allow free lateral movement to take place. At each end a hinge is provided to allow lateral movement to take place whilst generating the minimum possible moment and hence shear in the pile. Such shear is undesirable because it inputs seismic load into the building which is not controlled by the dissipators.

On the exterior columns, the hinge is formed by necking the 900 mm diameter pile to 600 mm for a height of 600 mm. This area has the minimum possible vertical reinforcing, with much of it concentrated at the centre. It is well confined with a heavy closely spaced spiral. The reinforcing is all galvanised, as the hinge is below water level. This arrangement generates too much moment on the more heavily axially-loaded 4 interior columns, so here a solid steel pin (250 mm diameter), is carried through the hinge area and anchored into the pile and the concrete



BRACING JOINT ELEVATION



BRACING JOINT PLAN

Fig. 7

plug in the rock.

The combination of lack of lateral support and pins at each end suggested that pile buckling was a problem. Various beams and on one face, a horizontal concrete truss, were provided at the level of the top hinge to restrain it in position to reduce the buckling length to that of the pile itself - about 12 metres. Had this not been done the column/pile combination would be free to buckle as a 3 pin bifurcated strut between first floor and bedrock.

#### 10. ACCOMMODATING DEFLECTIONS:

The base isolation/energy dissipator combination allows movement of + 150 mm under the maximum credible earthquake, of the first floor relative to the ground floor and basement. In the design earthquake the movement is + 60 mm. Anything passing through this space must have separations of this order built in. Over the remainder of the building interstorey deflections are smaller (typically 10 mm for the maximum credible earthquake and 5 mm for the design earthquake) and no specific detailing is required.

Windows are hung from the first floor as they needed to penetrate the first floor ceiling. They are supported on fabricated steel Vierendeel girders which cantilever from first floor. A sacrificial flashing at their base keeps the weather out. Gibraltar board partitions are fixed to the ground floor and are stabilised by bracing in the plane of ceilings which surround them.

The lifts are carried on a steel space truss which cantilevers down from first floor into the lift pits. Guide rails are supported directly on these trusses and vertical loads from rails and buffers are carried on a sliding steel plate attached to the trusses. This is all necessary to ensure that the full length of the lift shaft always stays vertical to allow the lifts to function down to ground floor.

Services crossing from basement to first floor are confined to water supply and waste disposal. Relatively small pipes are needed and they will comfortably bend over the interstorey distance.

#### 11. CONSTRUCTION:

The job was tendered in two separate sections - a piling contract and a main building contract, so that final design and approvals could be completed during the piling. Piling was completed between December 1981 and March 1982, at a cost of \$455,000. The building contract started in March 1982 and is expected for finish in middle October 1983. The Contractor is Downer & Co. Ltd. The structure was complete by March 1983. The final cost is expected to be \$6.6 million. This cost is equivalent to approximately \$700/square metre.

##### 11.1 Piling

The piling was carried out by Gilberd

Hadfield Piling Co.

The site was covered with old driven timber piles which had to be removed before the new piles were bored. Their removal left vertical tunnels in the fill material which allowed the fill to run and build up uneven pressures around the permanent outer casing of the new piles. The casing was originally designed as stacked flush-jointed concrete pipe, intended to be sunk as a caisson as boring proceeded. It was necessary to change this to a continuous steel liner which was driven ahead of the boring and sealed into the bedrock.

A second problem emerged when the columns were to be poured. Because there is no pile cap the piles have to directly line with the column. Construction tolerance meant that this was not exactly achieved, and so the tops of the piles were moved sideways with light jacks to the correct position. The normal eccentricities allowed for in design were sufficient to cope with the realignment. It was interesting that the steel pin hinged piles were much easier to move than the concrete hinged ones, because of the strength of the uncracked concrete. Future designs would be better with all steel hinges.

##### 11.2 Beams and Columns

The beams were all precast and no problems were experienced. At the Contractor's request concrete pipe sections were used for permanent formwork to the columns. In all but the corners all of the spiral reinforcing required was incorporated into the pipes, making site placing of column reinforcing very easy.

Wherever possible precast concrete was used, and this resulted in significant time savings. At best a complete floor level was constructed in 6 working days.

##### 11.3 Bracing and Joints

The joints were very carefully placed and checked by survey before they were concreted in, and the later erection of the braces and bolting of joints was achieved smoothly. Concreting around the joint areas has yet to be completed and will be an interesting formwork exercise.

The Contractor was permitted to erect the beams and columns 3 storeys ahead of the bracing, which meant that the bracing could be installed when the crane was not required for other lifts.

#### 12. CONCLUSION:

This paper has described and discussed a successful application of base isolation and the attendant use of energy dissipators in a multi-storey building. Their use has enabled a cross braced lateral load resisting structure to be used, which needs only modestly sized principal members, and yet is by its nature very stiff. No specific ductility has been required in the structural frame - excess seismic energy is absorbed in the energy dissipation system.

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DISCUSSION:

J Hollings asked the following:

"In view of the ten metre saturated hydraulic fill shown one would expect there to be a real possibility of partial liquifaction or similar permanent ground movement in the "maximum credible earthquake", please comment on the consideration given to this.

What provision for limit stops for the energy dissipators and thus the structure as a whole was provided for such an event?"

Some limited consideration was given to the question of liquifaction of the ground. Tonkin and Taylor Ltd regarded the possibility as sufficiently remote that it was insignificant. The return period for the maximum credible earthquake was chosen to test the structure mathematically to ensure that adequate clearances

were provided at the energy dissipators and that no significant yielding would occur in the superstructure. It is a level that would not normally be considered for buildings in the Auckland area.

For the same reasons, there are no special limit stops for the energy dissipators. After 150 mm movement from the neutral position the piles will hit the liners in the ground and in the basement. The maximum deflection in the analysis was 130 mm for the maximum credible earthquake and 60 mm under the design earthquake representing a return period of 150 years. With adequate clearances, limit stops are felt to be unnecessary, and even undesirable.

B.J. Wood