THE DESIGN AND CONSTRUCTION OF A BASE-ISOLATED CONCRETE FRAME BUILDING IN WELLINGTON, NEW ZEALAND.

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SUMMARY

The paper describes the design and construction of the William Clayton Building, a four storey ductile reinforced concrete frame structure isolated from its foundations by 80 elastomeric bearings incorporating lead energy dampers. Results from inelastic, time-history analyses for models of the isolated and non-isolated structures are compared for several input earthquake motions. The benefits of bearings and dampers in reducing the isolated structure's response, as well as constructional aspects are described.

INTRODUCTION

The building is a 3 storey and basement reinforced concrete frame structure of 97 by 40m plan dimensions, with column grid lines at 7.2m centres. The completed building, named the William Clayton Building after New Zealand's first Government Architect, is shown in Figure 1. The structure is the first building known to be designed with base-isolation incorporating elastomeric rubber bearings and lead energy dampers to reduce the structure's seismic response. The 80 bearings and their included dampers are positioned, one under each column, in a crawl space below the basement beams and sit on foundation pads which are on the 7.2m grid and are joined by a grid of ground beams. The building contains mainly large open plan offices and the interstorey heights are 5m from basement to ground floor and are all 4m above ground floor. Many internal beams of the first, second and roof levels are haunched to allow extra space for the mechanical services.

Deep precast concrete cladding panels are separated from the beams and slabs to allow the structure to act as a beam-hinging ductile frame. Steel portal framed plant rooms are situated on the second and roof levels. Figure 2 shows a transverse section through the building.

BUILDING ISOLATION

The theoretical and experimental research on the utilisation of natural rubber bearings as isolators and the development of lead and steel energy absorbers completed by the N.Z. Department of Scientific and Industrial Research at the Physics and Engineering Laboratory has been published (1) to (3). During the William Clayton Building design the DSIR developed and tested an elastomeric bearing including a lead plug as the energy absorber (4) (5). The cylindrical lead plug deforms in shear between the internal steel plates within the rubber bearing, while the bearing which is very stiff vertically carries the column axial load.

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The analysis value of the bearing's shear stiffness (6) was taken as approximately equal to the average axial dead plus seismic live load per metre of shear deformation (1525 kN/m). The recommended force at zero displacement of the lead damper is about 5% of the average axial load per bearing (76 kN) and is shown as F(a) on the idealised bearing and damper hysteresis curve in Figure 3. Note that the plastic stiffness of rubber and lead is greater than the rubber alone due to the lead not being elastic-perfectly plastic. The maximum design wind speed would cause a base shear of about 0.03g thus the lead plugs should not yield during severe wind storms.

The principal properties of the rubber/bearing lead damper (see Figure 4 for sectional details) used in the analyses were a stiffness of 10000 kN/m up to yield and a plastic stiffness of 2500 kN/m while the yield force (F_y) was 101 kN. Two bearings and dampers were selected at random from the 82 manufactured in Christchurch, N.Z. and their average principal properties found from cyclic testing (Robinson & Tucker, Ref.5) were 119 kN and 1380 kN/m for F(a) and plastic stiffness respectively.

However, the value of F(a) is partially dependent on the vertical load and shear displacement in the first complete cycle and for a shear strain of 50% (shear displacement to bearing height = 0.50) and a compressive stress of 3 MPa on the total bearing area, a better approximation of the bearing shear force at zero displacement, F(a) = 87 $\pm$ 9 kN was found (5). This implies a first yield force of 101 $\pm$ 9 kN in the actual bearings. Thus the lead dampers will first yield at a base shear level of approximately 8% of gravity (identical to the analysis values) while the post-yield stiffness will be about 55% of the assumed design plastic stiffness. This means the building will reach the limits of its isolation from the basement walls (150mm around the building periphery) at a lower base shear level than anticipated. From the bearing tests (5), the vertical compressive stiffness of combined bearing and damper was $6 \times 10^5 \pm 5 \times 10^4$ kN/m.

DESIGN AND MAXIMUM CREDIBLE EARTHQUAKES

The 1940 El Centro earthquake (N-S component) has been used as the basic earthquake for design codes and dynamic analyses for many years and indeed its response spectra forms the basis of the New Zealand seismic code coefficient (7). The building is situated very near a Class I fault on alluvial soils which extend to a depth of at least 100m. After due consideration of these site conditions and the somewhat experimental nature of the building 1.5 El-Centro of 10 seconds duration was used as the design earthquake with a peak acceleration of 0.523g. The maximum credible earthquake used was the artificial A1 motion (8) (40 seconds duration) which is equivalent to a magnitude 8 or greater earthquake in the vicinity of the fault.

STRUCTURAL ANALYSES

Typical 2-dimensional transverse frames were modelled and inelastic, bi-linear, dynamic time-history analyses were performed using the University of California, Berkeley's computer program DRAIN-2D (9) on the Ministry of Works and Development's IBM 270/168 computer. The bearing/dampers were modelled as horizontal truss members with the bi-linear hysteresis curve shown in Figure 3.

Full dynamic analyses assumptions and results are given in a previous paper by the author (10).

The principal modes of the structure on its bearings are the structure's period of vibration vibrating as a rigid mass on its bearing and dampers (T = 2.0 seconds) and the natural period of the frame structure alone (T = 0.3s). The actual fundamental period of the isolated structure varies between 0.8 and 2.0s, depending on the amplitude of the lateral vibration. 3% of critical damping was used for the isolated structure. The comparitive non-isolated structural analysis used 3 and 5% of critical damping corresponding to the structure's first and second mode periods (T_1 = 0.3s and T_2 = 0.08s).

The maximum base shear of the isolated building under 1.5 El-Centro and Al earthquake were 19.7% and 26% of the model building frame's dead plus seismic weight respectively. The 1.5 El Centro base shear is nearly identical to Skinner's (6) single mass linear resonator with base isolation under the same earthquake input and with the same 3% critical damping.

Figure 5 shows the sum of the column shear forces up the building for the isolated and non-isolated structures under the aforementioned design and credible earthquake input motions. Also plotted is the NZ Loading Code (7) static shear distribution for a base shear coefficient of 0.19, the base shear level for which a non-isolated building would have been designed. Note that the maximum base shear for the non-isolated structure was twice the maximum base shear for the isolated structure and the shear envelopes for the isolated building suggest a near uniform distribution of lateral force up the structure rather than the usual static inverted triangular distribution for non-isolated buildings.

The maximum bearing/damper shear deformation was 105mm and 151mm for 1.5 El Centro and Al earthquake respectively while during the 40s Al response there were 13 peaks at shear deformations of $\pm$ 100mm or greater.

Under 1.5 El Centro only the roof beam yielded and there was no hinge reversal. The applied vertical loads at roof level are high due to the weight of the plant rooms while there is very little dead and live loads on the lower floor beams due to the 1-way precast slab spanning in the transverse direction. The comparitive non-isolated building's response showed reversed hinging in all beams at ground level and above and some column hinges also formed.

The maximum interstorey drifts under both input earthquakes were about 10mm ($\Delta/h \simeq 0.002$) and were uniform over the isolated structure's height. By comparison the maximum interstorey drift recorded for the non-isolated model was 52mm between second floor and roof level. An overall structural deflection ductility demand of 1.6 was required for the isolated building and 7.6 for the non-isolated case.

BUILDING DESIGN

The design earthquake, 1.5 El Centro, produced a maximum dynamic base shear equivalent to nearly 0.20 W_t (W_t is total dead plus seismic live load) and this was used as the design static shear force which was equal to the non-isolated NZ Code base shear coefficient with an additional 20% allowance for torsion. The exterior transverse frames of a non-isolated structure would have required a 54% increase in base shear due to torsional aspects due to the structure's elongated plan. The design actions were found from elastic analyses on typical transverse and longitudinal frames under the approximate triangular distribution of lateral seismic force up the building, zero level being taken at the bottom of the bearings. The design engineer felt this conservative approach was necessary because of the building being a base isolated prototype. The capacity design approach was used in the column flexure and beam and column shear design but only uniaxial beam yielding was assumed. Some of the then current regulations regarding beam-column joint design were relaxed in the final design due to the lack of beam hinging evident in the dynamic analyses. If the full joint shear requirements had been used all beams would have been at least 200mm deeper so that the necessary joint ties could all be fitted between the top and bottom beam bars.

After completion of the design an extra analysis was run with the structural model under the March 4th 1977 Romanian Earthquake (Bucharest) N-S record (M7.2). This record was obtained from a seism ograph situated in a basement founded on a 12m strata of clay over deep loess and alluvium and recorded a peak acceleration of only 0.20g. However, the response spectra show peak responses at an undamped natural period of approximately 2sec; that is the approximate period of the isolated building on its energy dampers. The analysis (using 10s of record) produced a maximum shear deformation of 262mm in the bearings and two reversed cycles of shear displacements greater than 150mm. Thus if such an earthquake was experienced in Wellington, the isolated building would be expected to run into the basement retaining walls several times; the maximum velocity at impact being about 0.30m/s from the analysis. Some damage to the retaining walls would result but would not lead to a major failure of the building falling off its bearings. For the short time the building is in contact with the surrounding walls the building will respond as a normal ductile frame. It should be noted that an earthquake with a similar response spectra to that of the 1977 Romanian Earthquake is extremely unlikely in the Wellington district.

BEARING AND GENERAL CONSTRUCTION DETAILS

Although no column tensile forces were evident in the computer time-history analyses (including the vertical acceleration components), 40mm diameter steel dowels were welded to the bearing's top and bottom fixing plates. These dowels (4/plate) protruded about 18mm into the bearing in an effort to eliminate any chance of the building jumping off its bearings during vertical accelerations. The foundation - basement beams were designed so that large flat jacks could be positioned between them, thus allowing any bearing to be removed during the building's life. The contractors found that the volume of the lead plug needed to be about 1% greater than the hole in the bearing to allow a tight fit; the lead plugs were inserted using a press.

The 150mm isolation gap between the building and surrounding entrance ways were usually detailed with a sliding grill over the gap which also covered peripherial drains. Special jointing systems for the ret iculation services had to be devised to sustain movements of ± 150mm horizontally without failure. The lift pit over run was constructed as an open-topped box suspended from the basement slab.

Figure 6 shows the bearing and lead plug in position with the top fixing plate with dowels projecting about to be positioned on the bearing. No major hold ups due to the bearings or their positioning were experienced during construction and the building was completed ahead of schedule.

CONCLUSIONS

The construction of the William Clayton Building has shown that the use of rubber bearings and lead plug dampers to base-isolate a structure is a viable means of reducing the seismic response of low-rise structures. Not only can smaller frame members be used with less complex transverse reinforcement but the need for separation of non-structural components is greatly reduced. The approximate cost of the bearings/dampers, their fixings, installation and the extra foundation beams and excavation was estimated at 4% of the total building cost. This extra cost is easily equated by the savings from smaller structural members, less reinforcing and the reduction in non-structural and architectural detailing.

In future base-isolated buildings it may be possible to position the bearings and dampers at the top of the basement columns for example, thus eliminating the extra sub-basement beams and foundation pads. Recommendations for the design and construction of base-isolated structures can be found in reference (11).

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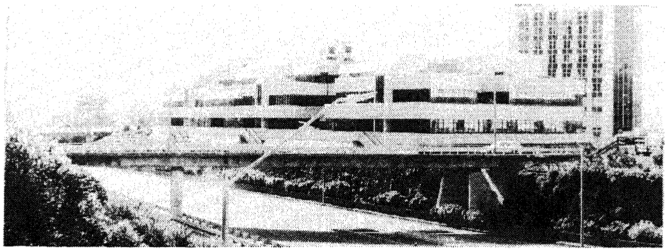

Figure 1: William Clayton Building as at July 1983

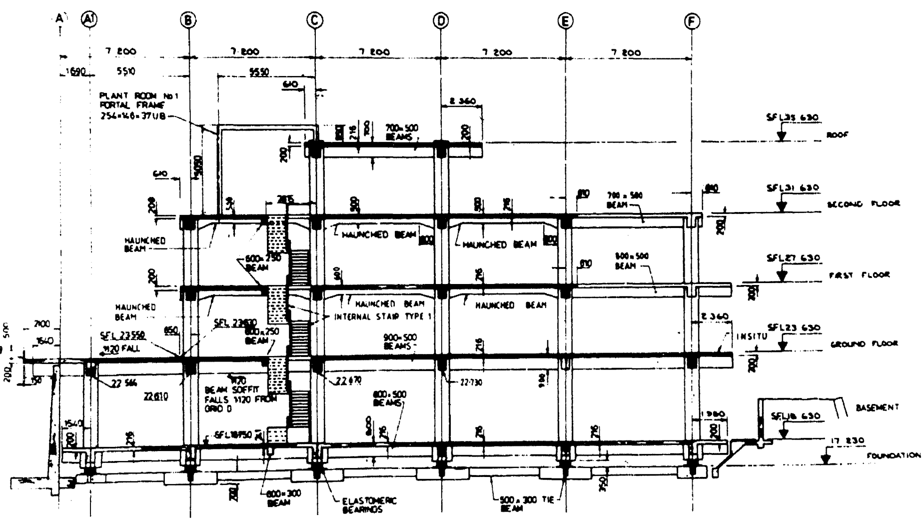

Figure 2: Transverse Section Through the Building

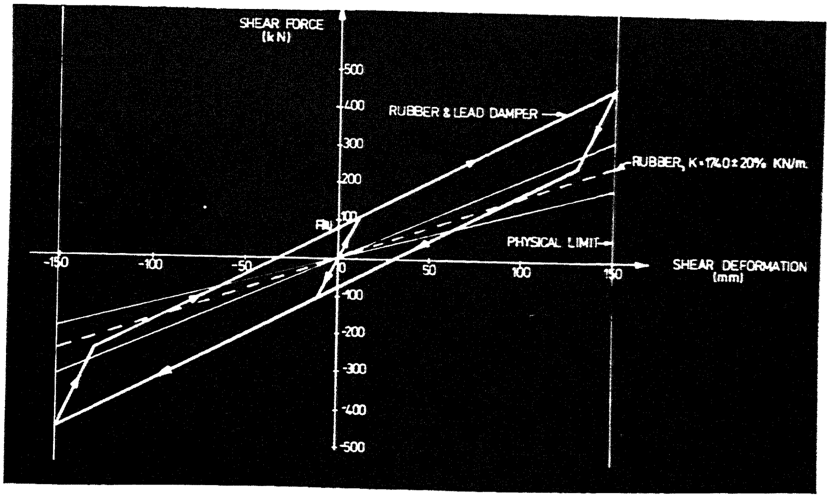

Figure 3: Idealised Bearing/Damper Hysteresis Curve Used in Dynamic Analyses

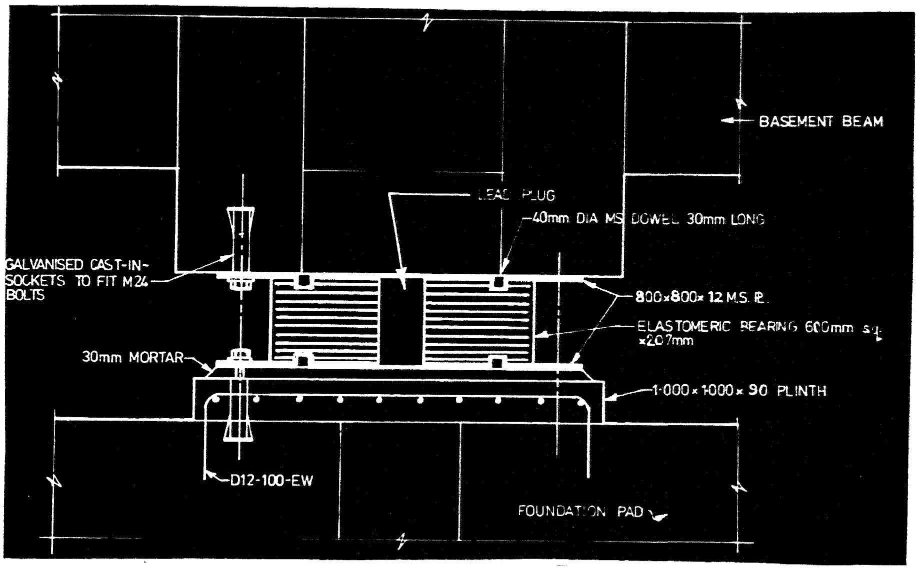

Figure 4: Section Through Bearing, Damper and Fixing Plates

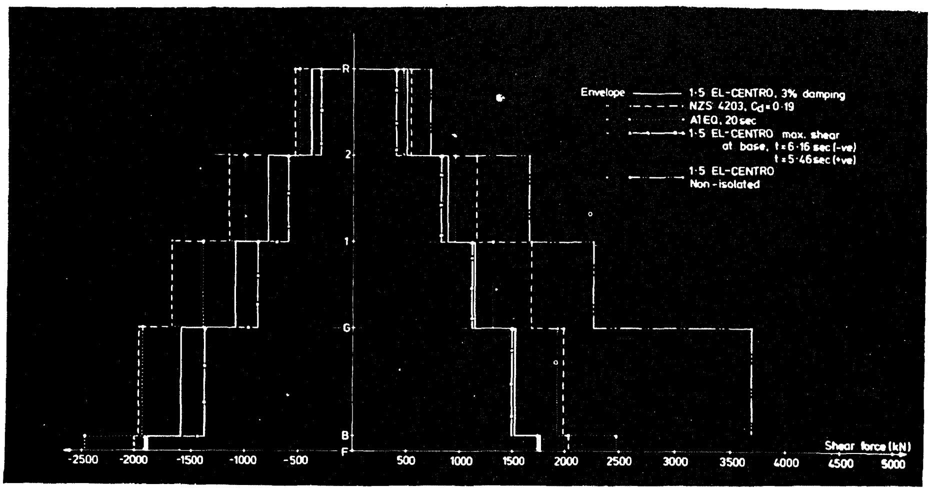

Figure 5: Maximum Shear Envelopes up the Building Under Various Earthquakes

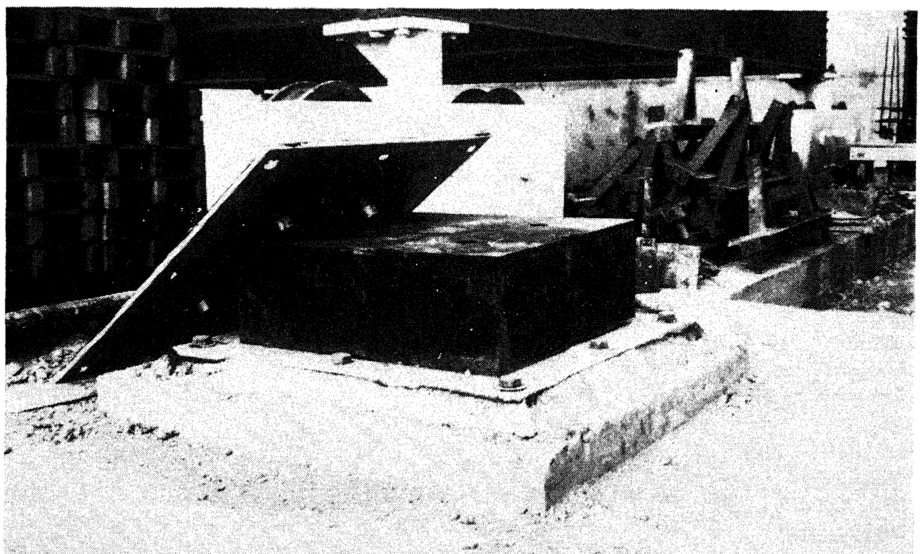

Figure 6: Bearing and Lead Plug in Position with Top Fixing Plate Awaiting Positioning