SEISMIC ISOLATION IN NEW ZEALAND

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Bridges, buildings, and industrial equipment can be given increased protection from earthquake damage by limiting the earthquake attack through seismic isolation. A broad summary of the seismic responses of base-isolated structures is of considerable assistance for their preliminary design. Seismic isolation as already used in New Zealand consists of a flexible base or support combined with some form of energy-dissipating device, usually involving the hysteretic working of steel or lead. This paper presents examples of the New Zealand experience, where seismic isolation has been used for 42 bridges, 3 buildings, a tall chimney, and high-voltage capacitor banks.

Additional seismic response factors, which may be important for nuclear power plants, are also discussed briefly.

Introduction

The significant resonant periods of buildings, bridges, or industrial plants often lie within the period range of 0.1 to 1.0 s, which is also the dominant period range for the ground vibrations during many severe earthquakes. The damaging effect of such earthquakes on these structures can be markedly reduced by mounting the structures on isolating supports such as ball or roller bearings, or more practically on sliding bearings, flexible rubber bearing pads, or flexible piles. With the proper choice of the flexible support, the effective period of the structure can be increased considerably to a value of 1.5 s or usually greater, outside the period range of strong excitation for most earthquakes. This produces a marked decrease in the acceleration response of the structure, but isolation by increased support flexibility alone typically requires large displacements across the isolating components, sometimes up to half a meter or more. To achieve a practical isolation system with acceptable displacements, it is necessary to add mechanisms for providing increased stiffness under low loadings and energy dissipation (damping), and some centering force, during large-amplitude motions. The mechanism should also have sufficient stiffness under moderate loads to give acceptably small side-sway during wind gusts.

It must also be recognized that occasionally earthquakes give their strongest excitation for long periods. The likelihood of these types of motions occurring at a particular site can sometimes be foreseen, such as with deep deposits of soft soil which may amplify low-frequency earthquake motions, with the old lake-bed zone of Mexico City as the best-known example. With this type of motion, flexible mountings with moderate damping may increase rather than decrease the structural response. The provision of high energy dissipation as part of the isolation system gives an important defense against the unexpected occurrence of such motions.

Seismic Responses of Base-Isolated Structures

A broad overview of the consequences of base isolation for aseismic structures is of considerable assistance in the choice of a system appropriate to the design requirements and in the preliminary design of the base isolator and the structure.

Some dominant features of the seismic responses of base-isolated structures are illustrated in Fig. 1, and further features are covered in discussion. Figure 1 compares the seismic responses of an unisolated nonyielding structure with the corresponding responses when the structure has each of four different isolation systems; linear isolation with low or high damping, or bilinear isolation with low or high flexibility. Here, bilinear loops are used as an approximation to the load-displacement loops of isolators with yielding or sliding components.

A uniform shear structure is used for the response comparisons of F'g. 1, and the earthquake accelerations are from El Centro SO°E, May 18, 1940.

As given in Fig. 1, the first-mode period and damping of the unisolated structure are 0.6 s and 5% of critical, respectively. Assuming a rigid structure, the linear isolators give a period of 2.0 s and a damping of 5 or 20%. Again, with a rigid structure the first bilinear isolator stiffness K_{b1} gives a period of 0.3 s and the second bilinear stiffness K_{b2} gives a period of 1.5 or 3.0 s. The bilinear isolator has a yield-force/structural-weight ratio Q_y/W of 0.05.

The maximum seismic responses of the five structural systems are compared for displacements Y, for forces and shears divided by the structural weight, F/W and S/W, and for top-floor spectra, with 2% spectral damping.

While Fig. 1 shows that the isolators generally reduce the structural seismic responses by about five times, there are large differences in the character of the responses with different isolators. The bilinear isolators of Fig. 1 give large higher mode seismic loads, and large floor spectra in the important period range, 0.1 to 1.0 s. However the low levels of floor spectra shown in Fig. 1 for linear isolators may also be achieved with appropriate bilinear isolators. For example, the isolator elastic stiffness and yield force of case (v) may be reduced to give the parameters

$$T_{b1} = 0.9 \text{ s}, \; T_{b2} = 3.0 \text{ s}, \; Q_y/W = 2\% .$$

This isolator reduces the higher mode contributions to structural loads, and also the values of the floor spectra, to about 40% of the case (v) values, and hence to values comparable with those for the linear isolators of Fig. 1.

Nonlinear isolator damping should be used as far as possible since it is generally more effective in reducing isolator deformations, structural loads, and wind-sway than is linear damping. If higher-mode seismic responses or floor spectra are an important design factor, then the nonlinear damping should be restricted to a level which limits these responses to acceptable values. If higher levels of isolator damping are required these should be provided by supplementing the allowable nonlinear damping with linear damping.

For the cases with linear isolation, Fig. 1 shows that maximum level-r displacements and base-level shears, Y_r and S_b, are dominated by an approximately rectangular mode 1, and hence:

$$Y_r = D(T_1, \zeta_1) \tag{1}$$

$$S_b = MA(T_1, \zeta_1) , \tag{2}$$

where T_1, ζ_1 = period, damping, of isolated mode 1; D, A = displacement, acceleration, spectra of the earthquake; M = structural mass.

For the cases with bilinear isolation, the approximate maximum displacements and base-level shears are also controlled by an approximately rectangular mode 1. The base displacements Y_b may be approximated by using an effective mode-1 period and damping T_e and ζ_e, based on the diagonal slope and the area of the maximum bilinear loop, and then using a weighted displacement spectra. The base shear may then be obtained from the isolator parameters. This gives

$$Y_b \approx 0.85\ D(T_e,\ \zeta_e) \qquad (3)$$

$$S_b \approx Q_y + (Y_b - Y_y)\ K_{b2}\ , \qquad (4)$$

where

$$T_e = 2\pi\sqrt{[M/(S_b/Y_b)]} \qquad (5)$$

$$\zeta_e = 63.7\ (Q_y/S_b - Y_y/Y_b)\%\ , \qquad (6)$$

and

Q_y, Y_y = yield force, yield displacement, of bilinear isolator

K_{b2} = stiffness of yielded isolator.

The factor 0.85 in Eq. (3) has been chosen to give the best agreement with the isolator displacements for cases (iv) and (v) of Fig. 1. When the character of the isolator, the structure, or the earthquake differs greatly from those used for Fig. 1, then the factor 0.85 is likely to be substantially altered.

As one alternative, the base shear and displacement may be expressed simply in terms of bilinear earthquake spectra, as given for example by the family of response curves in a recent publication.[1]

Assuming a rectangular mode 1, the mode-1 shears and forces, S_{r1} and F_{r1}, may be derived simply from the base shear and the mass distribution. For approximately uniform shear-like structures, the ratio of overall/mode-1 shears and forces are indicated roughly by the corresponding ratios in cases (iv) and (v) of Fig. 1.

While the isolator parameters of Fig. 1 are typical of those that may be used for a wide range of aseismic structures, a much wider range of isolation systems may be appropriate for a range of special aseismic design problems. Additional structural problems may include inherently low strength or ductility, or a low tolerance for distortion. The seismic attack may be increased in certain parts of the structure by setbacks, structural taper, or torsional unbalance. Again, the site may give unfavorable earthquake vibrations. Such effects may call for a change in the character or the degree of the isolation provided.

A much broader overview of the effects of base isolation is included in a forthcoming publication by the authors, Skinner, Robinson and McVerry.[2]

Structures Isolated in New Zealand

In New Zealand, isolation has been achieved by a variety of means: transverse rocking action with controlled base uplift; horizontally flexible elastomeric bearings; and flexible sleeved-pile foundations. Damping has been provided through hysteretic energy dissipation arising from the plastic deformation of steel or lead in a variety of devices such as steel bending-beam and torsional-beam dampers, lead plugs in laminated steel and rubber bearings and lead-extrusion dampers.[3,4] Seismic isolation has often been considered as a technique for "problem" structures or industrial facilities requiring a special seismic design approach. The design difficulties may relate to the structure's function (e.g., sensitive or high-risk industrial or commercial facilities such as computer systems, semiconductor manufacturing plants, biotechnology facilities, and nuclear power plants), its special importance after an earthquake (e.g., hospitals, disaster-control centers such as police stations, bridges providing vital communication lines), or other special problems (e.g., seismic strengthening of existing structures). Seismic isolation may indeed have particular advantages over other approaches in these circumstances, sometimes being able to provide much better protection under extreme earthquake motions than other techniques. However, its economic use is by no means limited to such structures.

In New Zealand, by far the most common use of isolation has been in ordinary two-lane road bridges of only moderate span, justifiable purely on the economics of construction with cost savings of typically 5-10% over conventional designs. In many cases, isolation provides superior seismic performance at reduced cost, because the reduction of forces by isolation and energy dissipation means that the structure need be designed only for elastic or limited-ductility seismic response and there is no need to accommodate large structural deformations.

Several examples of the application of seismic isolation in New Zealand are discussed in the following sections.

Road Bridges

Forty-one road bridges and one rail bridge in New Zealand have been seismically isolated. A number of examples of seismic upgrading by retrofitting isolation systems are included.

By far the most common form of isolation system for bridges uses lead-rubber bearings,[5,6] usually installed between the bridge superstructure and the supporting piers. The bearings (Fig. 2) consist of a many-layered sandwich of steel plates and rubber sheets vulcanized together, with a lead plug inserted in the center. The lead plug is subjected to a shear deformation under horizontal loading, providing considerable energy dissipation when it yields under severe earthquake loading. The lead-rubber bearing combines the functions of isolation and energy dissipation in a single compact unit, while also supporting the weight of the superstructure and providing an elastic restoring force. Many unisolated New Zealand bridges use elastomeric bearings to accommodate thermal movements, so little modification to standard structural forms has been necessary to incorporate seismic isolation bearings apart from making provisions for the increased superstructure displacements required under seismic loading. As well as providing energy dissipation in large movements, the lead plug also stiffens the bearing under low lateral forces up to its yield point, reducing the displacements under wind and traffic loading. One concern that has been reported with lead-rubber bearings is the need for sufficient vertical load to provide a confining action on the lead core. With a well-designed bearing with rubber layers of the order of 10-mm thickness, the effect of vertical load on the hysteresis loops is small. However, with rubber layers of around 20 mm or greater thickness, the hysteretic energy dissipation capacity is reduced markedly when there is insufficient compressive confining load.

Further information on the seismic isolation of road bridges in New Zealand, including case studies and design procedures, is given by Blakeley[7] and Billings and Kirkcaldie.[8]

South Rangitikei Rail Bridge

The South Rangitikei Rail Bridge, which was opened in 1981, is an example of isolation through controlled base-uplift in a transverse rocking action. The bridge is 70-m tall, with six spans of prestressed concrete hollow-box girder, and an overall length of 315 m. The stresses, which can be transmitted into the slender reinforced concrete H-shaped piers under earthquake loading, are limited by allowing them to rock sideways, with uplift at the base alternating between the two legs of each pier. The extent of stepping and the associated lateral movement of the bridge deck is limited by energy dissipation provided by the hysteretic working of torsionally yielding steel-beam devices connected between the bottom of the stepping pier legs and the caps of the supporting piles, as shown in Fig. 3.

The stepping action reduces the maximum tension calculated in the tallest piers, for the 1940 El Centro NS record, to about one-quarter that experienced when the legs are fixed at the base, and unlike the fixed-base case there is little increase in base-level loads for stronger seismic excitations. The dampers reduce the displacements to about one-half of the undamped case, and the number of large displacements to less than one-quarter. The maximum displacement at the deck level for the damped stepping bridge is about 50% greater than for the fixed-leg bridge.[9]

The 24 energy dissipators operate at a nominal force of 450 kN with a design stroke of 80 mm. The maximum uplift of the legs is limited to 125 mm by stops. The weight of the bridge at rest is not carried by the dampers but is transmitted to the foundations through low-thickness laminated-rubber bearings, whose primary functions are to allow rotation of each unlifted pier foot, and to distribute loads at the pier/pile-cap interfaces.

The stepping action was very effective in reducing seismic loads on this bridge because its high center of gravity resulted in a nonisolated design which was strongly dominated by overturning moments at the pier feet. The hysteretic damping during stepping was quite effective because the estimated self-damping of the stepping mechanism was quite low.

William Clayton Building

The William Clayton building in Wellington, completed in 1981, was the first building to be base isolated on lead-rubber bearings. The bearings are located under each of the 79 columns of the four-story reinforced concrete frame building which is 13 bays long by 5 bays wide with plan dimensions of 97x40 m. Detailed descriptions of the building have been given by Megget[10] and Skinner.[11]

The pioneering nature of the building and its proximity to the active Wellington fault dictated that a conservative design approach be taken. The design earthquake was taken as 1.5 El Centro S0°E 1940, which was calculated as producing a maximum dynamic base shear of 0.20 times the total building weight W, which was selected as the design static base shear force. The artificial A1 record, which is intended to represent near-fault motion in a magnitude 8 earthquake, was considered as the "maximum credible" motion, producing a calculated maximum base shear of 0.26 W. Even though the calculated response of the base-isolated structure was essentially elastic for the design earthquake motions, a capacity design procedure as required for plastic design was used.

Horizontal clearances of 150 mm were provided before the base slab impacts on retaining walls. This corresponds to the maximum bearing displacement calculated for the A1 record, with 105 mm calculated for 1.5 El Centro. Water, gas and sewerage pipes, external stairways, and sliding gratings over the separation gap are detailed to accommodate the 150-mm isolator displacement.

The lead-rubber bearings lengthen the period of the structure from 0.3 s for the frame structure alone, to 0.8 s for the isolated structure with the lead plugs unyielded, and 2.0 s in the fully yielded state (i.e., calculated from the structural mass and post-yield stiffness of the bearings). The combined yield force of all the bearings and lead plugs was ~7% of the structure's dead plus seismic live load.

The maximum base shear for the isolated structure calculated for 1.5 El Centro of 0.20 W was half the value of 0.38 W for the unisolated structure. Only the roof beam yielded for the isolated structure with a rotational ductility of <2 and no hinge reversal. For both 1.5 El Centro and the A2 record, the maximum interstory drifts for the isolated structure were about 10 mm, about 0.002 times the story height, and were uniform over the structure's height. For the unisolated structure, the interstory drifts increased up the height of the building, reaching a maximum of 52 mm. The markedly reduced interstory drifts should minimize the secondary damage in the isolated structure and greatly simplified the detailing for partitions and glazing.

As a first attempt at base isolation of a building with lead-rubber bearings, the design of the William Clayton building was very much a learning experience. The design was conservative and, if repeated now, it is probable that more advantages would be taken of potential economies offered by the isolation approach to seismic design. Nevertheless, the design analysis demonstrated the improved seismic performance which can be achieved through isolation of appropriate structures. Moreover, the extreme-earthquake capacity could be extended substantially simply by increasing the base-slab clearance to 200 or 250 mm, in the light of subsequent tests on lead-rubber bearings.

Union House

The 12-story Union House,[12] completed in 1983, achieves isolator flexibility by using flexible piles within clearance sleeves. It is situated in Auckland alongside Waitemata Harbour. Poor near-surface soil conditions, consisting of natural marine silts and land reclaimed by pumping in hydraulic fill, led to the adoption of long end-bearing piles, sunk about 2.5 m into the underlying sandstone at a depth of about 10-13 m below street level, to carry the weight of the structure. Although Auckland is in a region of only moderate seismic activity, there is concern that it could be affected by large earthquakes up to magnitude 8.5 centered 200 km or more away in the Bay of Plenty and East Cape regions near the subduction zone boundary between the Pacific and Indo-Australian plates. Such earthquakes could cause strong shaking in the flexible soils at the site.

Isolation was achieved by making the piles laterally flexible with moment-resisting pins at each end. The piles were surrounded by hollow steel jackets allowing ±150 mm relative movement, thus separating the building from the potentially troublesome earthquake motions of the upper soil layers and making provision for the large base displacements necessary for isolation. An effective isolation system was completed by installing steel, tapered-cantilever dampers at the top of the piles at ground level to provide energy dissipation and deflection control. The structure was stiffened and strengthened using external steel cross-bracing. The increased stiffness improved the seismic responses, while the cross-bracing provided low-cost lateral strength since only limited ductility was required. The dampers are connected between the top of the piles supporting the superstructure and the otherwise structurally separated basement and ground-floor structure, which is supported directly by the upper soil layers.

As Auckland is a region where earthquakes of only moderate magnitude are expected nearby, the seismic design specifications for Union House are less severe than for many base-isolated structures. The maximum dissipator deflections in the "maximum credible" El Centro motion were 150 mm, with 60 mm in the design earthquake. The effective period of the isolated structure was about 2 s. Maximum interstory deflections were typically 10 mm for the maximum credible earthquake and 5 mm for the design earthquake.

Union House is an example of the economical use of base isolation in an area of moderate seismicity. An appropriate structural form was chosen to take advantage of the reductions of seismic force and structural deformations offered by the base-isolation option. The inherently stiff cross-braced frame is well-suited to the needs for a stiff superstructure in the base-isolated approach, which in turn makes the cross-bracing feasible because of the low ductility demands placed on the main structure through the use of isolation with energy dissipators. An important factor in the design of such isolation systems is an appropriate allowance for the displacement of the pile-sleeve tops with respect to the fixed ends of the piles. Other structural forms were investigated during the preliminary design stages, including two-way concrete frames, peripheral concrete frames, and a cantilever shear core. The cross-braced isolated structure allowed an open and light structural facade, and a maximum use of precast elements. The base-isolated option produced an estimated cost saving of nearly 7% in the total construction cost of NZ$6.6 million, including a construction time saving of three months.

Wellington Central Police Station

The new Wellington Central Police Station,[13] presently under construction, is similar in concept to Union House. The ten-story tower block is supported on long piles founded 15 m below ground in weathered graywacke. The near-surface soil layer consists of marine sediments and fill of dubious quality.

Again, the piles are enclosed in oversize casings allowing considerable displacements relative to the ground. Energy dissipation is provided by lead-extrusion dampers,[14] connected between the top of the piles and an embedded basement. A cross-braced reinforced concrete frame provides a stiff superstructure. The flexible piles and lead-extrusion dampers (Fig. 4) provide an almost elastoplastic force-displacement characteristic for the isolation system, which controls the forces imposed on the main structure.

The seismic design specifications for the Wellington Police Station are considerably more severe than for Union House in Auckland. The police station is an essential facility required to be in operation after a major earthquake. The New Zealand loadings code requires a risk factor R=1.6 for essential facilities. The site is a few hundred meters from the major active Wellington fault, and <20 km from several other major fault systems.

Functional requirements dictated that the lateral load-resisting structure should be on the perimeter of the building. Three structural options were considered, a cross-braced frame, a moment-resisting frame, and a base-isolated cross-braced frame. With the requirement for piling because of the foundation conditions, the base-isolation option looked attractive from the outset, although the perimeter moment-resisting frame was also considered at length.

The structure is required to respond elastically in a 450-yr return period motion, corresponding to a 1.4 times scaling of the 1940 El Centro accelerogram. The building must remain fully functional and suffer only minor nonstructural damage in this motion. This is assured by the low interstory deflections of ~10 mm. With an isolation system with a nearly elastoplastic force-deflection characteristic and a low yield level of 0.035 of the building seismic weight, it was found there was only a modest increase in maximum frame forces for the 1000-yr return period motions, corresponding to 1.7 El Centro NS 1940 or the 1971 Pacoima Dam record. The increase in force was almost accommodated by the increase from dependable to probable strengths appropriate to the design and ultimate load conditions. It is possible that some yielding will occur under the 1000-yr return period motions, but the ductility demand will be low, and specific ductile detailing was considered unnecessary. The Pacoima Dam record poses a severe test for a base-isolation system because it contains a strong long-period pulse, thought to be a "fault-fling" component, as well as high maximum accelerations. The Pacoima record imposes severe ductility demands on many conventional structures.

The degree of isolation required to obtain elastic structural response with these very severe earthquake motions requires provision for a large relative displacement between the top of the piles and the ground. A clearance of 375 mm was provided between the 800-mm-diam piles and their casings to give a reasonable margin above the maximum calculated displacements, 355 mm was calculated for one of the 450-yr return period accelerograms. Consideration was also given to even larger motions, when moderately deformable column steps will contact the basement structure, which has been designed to absorb excess seismic energy in a controlled manner in this situation.

The large displacement demands on the isolation system and the almost elastoplastic response required from the energy dissipators led to the choice of lead-extrusion dampers rather than steel devices, as used in Union House. In total, 24 lead-extrusion dampers each with a yield force of 250 kN and stroke of ±400 mm were required. This was a considerable scaling up of previous versions of this type of damper used in several New Zealand bridges. The bridge dampers had a yield level of 150 kN and a stroke of ±200 mm. The new model damper was tested extensively to ensure the required performance.

The base-isolated option was estimated to produce a saving of 10% in structural cost over the moment-resisting frame option. In addition, the base-isolated structure will have a considerably enhanced earthquake resistance. Moreover, the repair costs after a major earthquake should be low. Importantly, the base-isolated structure should be fully operational after a major earthquake.

Isolation of Other Structures

Approximately bilinear isolators, which provide most of the mode-1 damping, have been practical and convenient for current New Zealand applications. However, when an aseismic design is critically controlled by the responses of relatively light-weight substructures it is often appropriate to restrict the isolators to moderate or low levels of nonlinearity. For such isolators, it will sometimes be appropriate to provide a substantial part of the mode-1 damping by approximately linear velocity dampers. The restrictions would not preclude the use of moderate levels of bilinear damping. This may be provided by metal yielding or by low sliding-fraction forces. For example, the weight of an isolated structure might be carried on lubricated poly tetra fluoro ethlene (PTFE) bearings. However, to minimize resonant-appendage effects during relatively-frequent moderate earthquakes such PTFE bearings should be supported by flexible mounts, as in the laminated-rubber/lead-bronze bearings pioneered by French engineers. Further isolator components should include flexible elastic components to provide centering forces, and sometimes substantial velocity damping. But the latter components reduce the maximum extreme-earthquake base movements for which provision must be made.

Nuclear power plants contain critical light-weight substructures essential for their safe operation and shutdown, including control rods, fuel rods and essential piping. These can be given a high level of protection by appropriate base isolation systems, designed to give low levels of seismic response for higher vibrational modes of major parts of the power plants. Further, serious seismic problems arise with fast breeder reactors in which critical components are given low strength by measures designed to give high rates of heat transfer. For some breeder reactor designs, it may be desirable to attenuate vertical as well as horizontal seismic forces.

In this case, it may be practical to provide horizontal attenuation for the overall plant and vertical attenuation for the reaction vessel only. Since the dominant vertical earthquake accelerations have considerably shorter periods than the associated horizontal accelerations, displacements associated with vertical attenuation should be much smaller than those for horizontal attenuation.

Early papers on nuclear power plant isolation,[15,16] concentrated on the protection of the overall power plant structure but did not treat the problems with light-weight substructures, which arise from the seismic responses of higher modes of structural vibration. Structural protection may now be achieved with simpler alternative isolator components; for example, the use of lead-rubber bearings may remove the need for installing steel beam dampers.

Conclusions

The seismic responses which are most important for a wide range of structures have been discussed briefly. Examples of the application of seismic isolation in New Zealand were then discussed. This approach has now been used for 42 bridges, 3 buildings, a tall chimney, and electrical capacitor banks at a vital substation in the South to North Island high-voltage dc link. The New Zealand approach to isolation incorporates energy dissipation in the isolation system to reduce the displacements required across the isolating supports and to safeguard against unexpectedly strong low-frequency content in the earthquake motion. Combined yield levels of the hysteretic energy dissipators range from about 3 to 15% of the structure's weight, with a typical value of about 5%. Displacement demands across the isolators range from about 100 to 150 mm for El Centro-type motions, to about 400 mm for the Pacoima Dam record. Structural response can often be limited to the elastic range in the design level earthquake, with limited ductility requirements in extreme motions. Often, substantial cost savings of up to 10% of the structure's cost have been possible by adopting the isolation approach, as well as obtaining an expected improvement in seismic performance of the structure.

The seismic problems that arise with light-weight substructures have also been given considerable attention in New Zealand. In particular, the constraints on isolation systems required to give the substructures effective protection have been studied.

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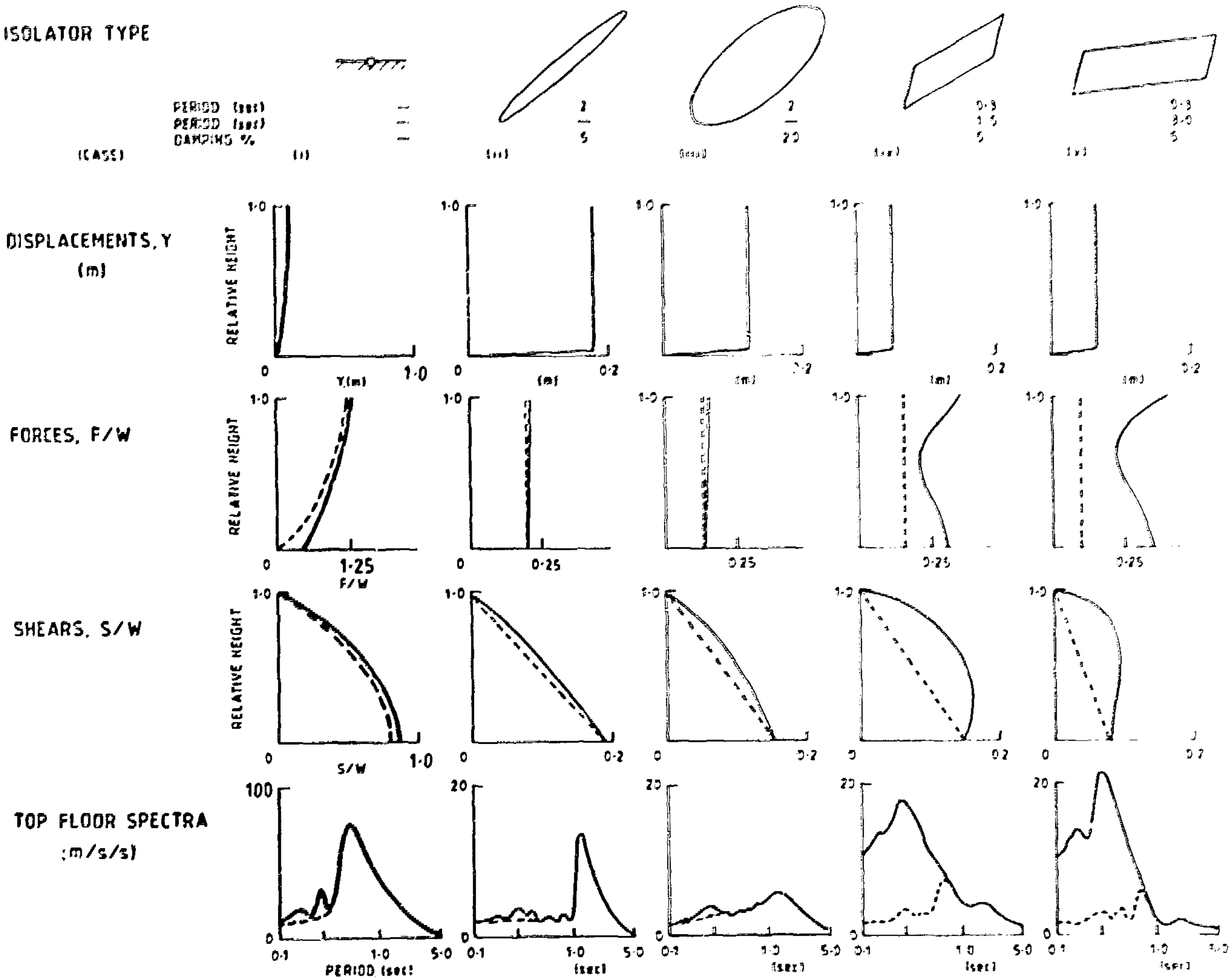

Fig. 1. Approximate Maximum Seismic Responses for El Centro S0°E May 18, 1940

Structure-uniform shear beam, of weight W; first period = 0.6 s, damping = 5% of critical.

Isolator-listed periods and dampings, when the structure is rigid; bilinear isolator has a yield-force/weight of 0.05.

Floor spectra are for the low damping of 2%.

Dotted curves are the seismic responses of mode 1 only.

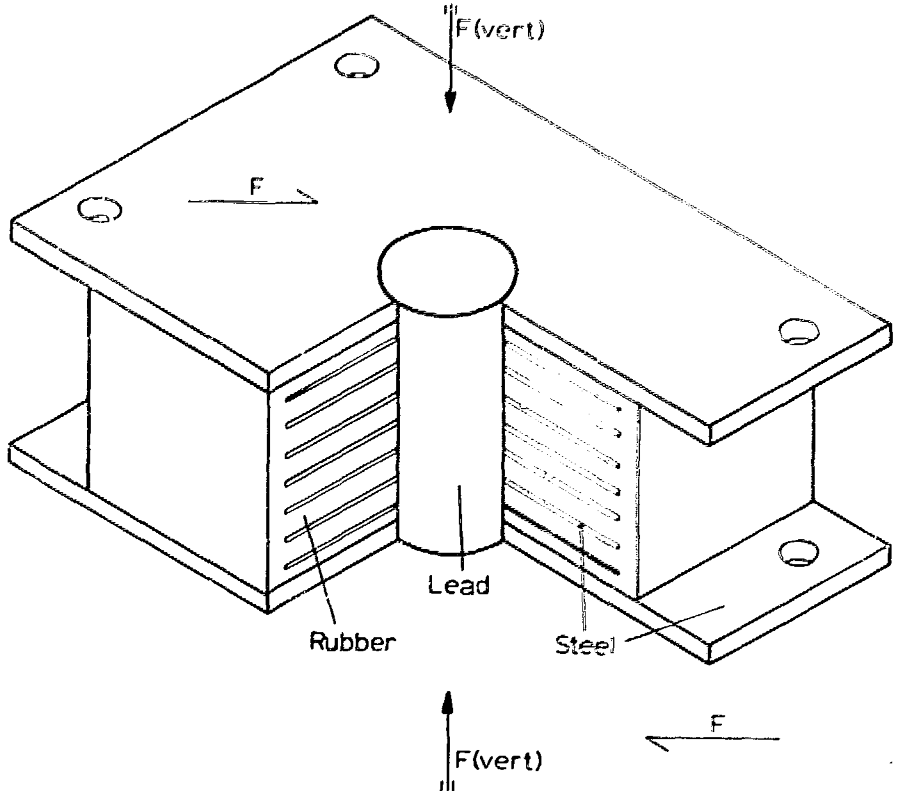

Fig. 2. Lead Rubber Bearing

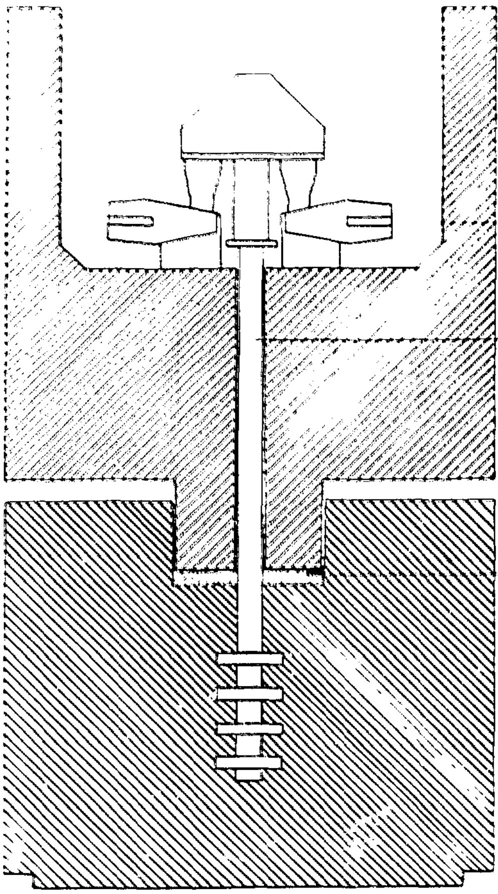

Fig. 3. Detail of a Foot of the Piers of the South Rangitikei Bridge

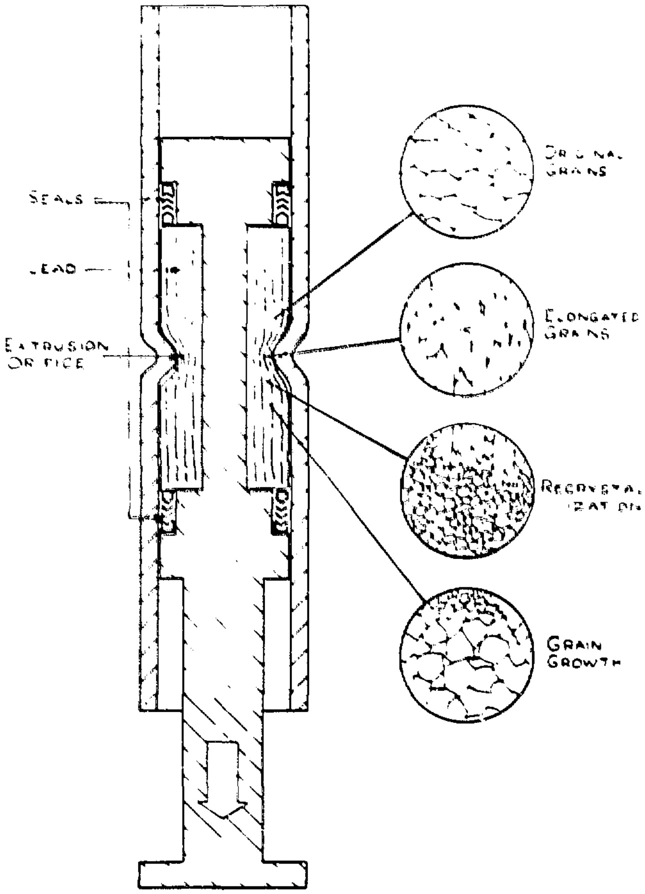

Fig. 4. Schematic Cross Section of a Lead-Extrusion Damper