RUBBER BEARINGS IN BASE-ISOLATED STRUCTURES - A SUMMARY PAPER

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SUMMARY

The paper discusses the use of rubber bearings in base isolated structures. In Japan alone there are likely to be 100 base-isolated buildings completed by the end of 1994 all of which will be on rubber bearings [Ref 1.2]. A distinction is drawn between the use of rubber bearings in bridges where they are subject to both the fatigue of traffic loading and diurnal temperature movements throughout their life, and their use in buildings where the loading is comparatively static until severe earthquake shaking occurs, which at a particular location is not likely to occur more than about once a century. Vertical load capacity and characteristics of vertical deflection in relation to rubber layer thickness and instability are discussed. Shear stiffness and deflection characteristics are outlined together with methods used for providing damping, including damping in lead-rubber bearings. The design of the high-stability aseismic bearing for loads of 1 tonne upwards is discussed in relation to its stability characteristics. It is suggested that tensile forces should be permitted in the short-term loading induced by earthquakes and this is confirmed by current Japanese practice in using fully bonded bearings. The development of self-damping rubber bearings is discussed.

1. INTRODUCTION

There has been a rapid expansion of the use of rubber bearings for the base isolation of structures against earthquake attack. The principal countries involved are Japan, U.S.A. and New Zealand. In Japan alone there are expected to be 100 buildings on seismic bearings by 1994 [2] while the U.S.A. has at least 6 bridges and 6 buildings isolated in the same way. In 1991 New Zealand has 40 bridges and 2 buildings on lead-rubber bearings.

Rubber bearings laminated with steel plates to control edge-bulging of the rubber have been used in bridges since the 1950's. These bearings are designed for diurnal and seasonal expansions and contractions of the bridge deck and standards introduced throughout the world have sought to allow for this particular design condition, a common requirement being that the lateral deflection of the deck should not exceed one half of the depth of the rubber, i.e. a shear strain in the rubber of 50% (See Fig.1). This requirement covers safety against tipping of the bearing, when the maximum deflection is maintained for a period of time, e.g. for a few hours in the afternoon of a hot day in summer, and also seeks to control the magnitude of tensile strains which will develop in the rubber at the bearing perimeter.

In earthquake prone areas the same bearings are used to isolate the bridge deck from the abutments essentially by promoting large horizontal deflections to give the deck a 'soft ride' during an earthquake. In New Zealand, the natural period of oscillation of the deck horizontally is designed to be in the range 1 to 2 seconds, i.e. a longer period than the predominant earthquake ground oscillation, which is most usually in the range 0.1 to 1 seconds (Fig.2). A shear strain of 100% is permitted in the rubber to cover the earthquake condition, which is double that required to cover day-to-day service conditions. The increase in allowable shear strain is justified in that the bearings are required to perform at some peak deflection only about once a century for a particular location, with other earthquakes generating smaller deflections at that particular site at intervals of perhaps 20 years. This is essentially an intermittent dynamic loading as opposed to the diurnal temperature loadings coupled with continuous traffic loadings for which codes of practice have been drawn up, and design requirements for base isolated structures need to be separately specified. Consideration should be given to allowing the bearings to carry vertical tensile forces by fully bonding the bearings to top and bottom fixing plates, a procedure which is now being done in current Japanese practice (Ref. 1 and 2 see para 6).

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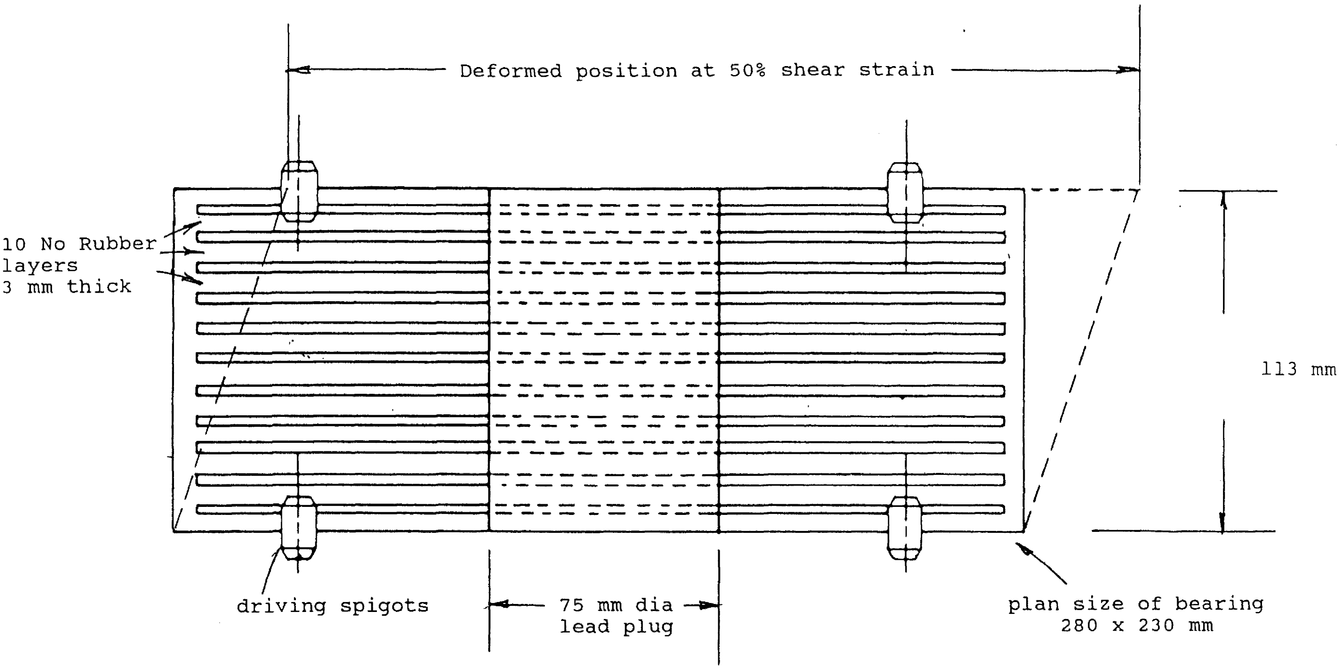


FIGURE 1 DEVIL'S CREEK BRIDGE BEARING, NEW ZEALAND. Design requirement for 50% shear strain in rubber.

(Design circa 1982, studs too far from centre) [8]

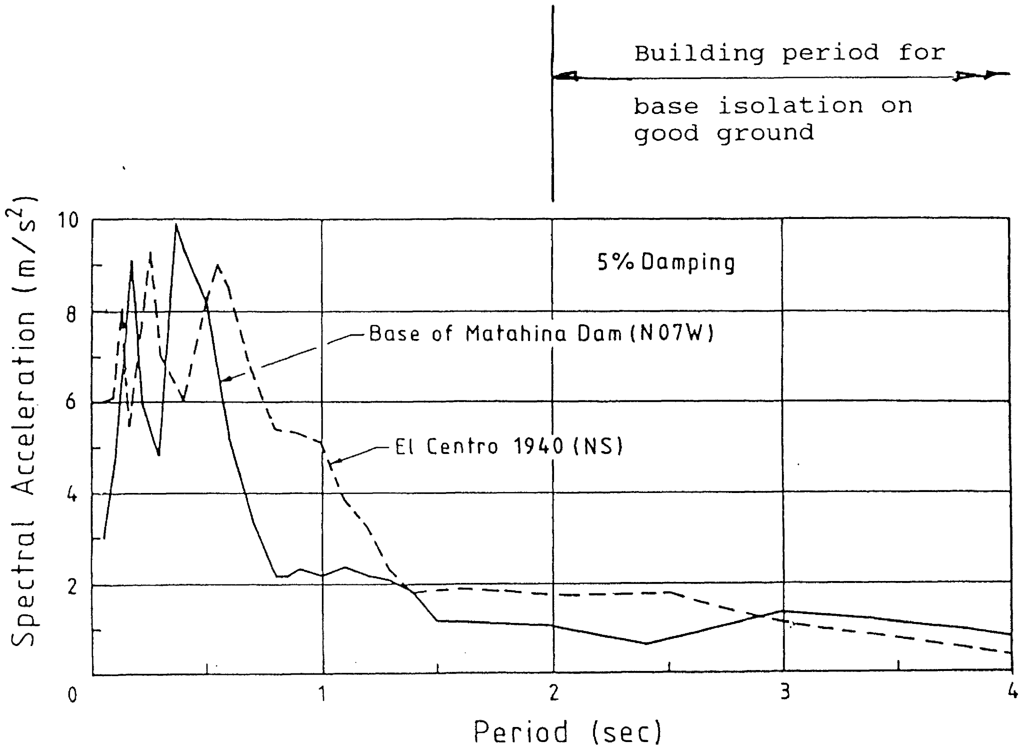


FIGURE 2 TYPICAL ACCELERATION RESPONSE SPECTRA FOR GOOD GROUND.

Matahina Dam Record for Edgecumbe earthquake of March 2, 1987, compared with that for El Centro 1940 (NS) [9]

In the case of a base-isolated building resting on rubber bearings, only very small temperature movements occur and wind effects would be expected to be less severe than the day-to-day live traffic loading effects for which specifications such as the U.K. BE1/76 [16] were drawn up. For instance, the bearings of the William Clayton Building in Wellington have not moved more than a few millimetres since their installation in 1981 and are in good condition, and have remained undeflected during the small earthquakes that have occurred since then. Thus for a base-isolated building a designer only has to allow for a brief period of large deflections which may occur only once during its lifetime and is arguable that the rated load for the bearings can be for a zero shear condition and, in consequence, of greater value.

2. GENERAL BASE ISOLATION DESIGN REQUIREMENTS

For structures resting on good ground, e.g. rock, weathered rock, firm hard gravelly clays in close proximity to a rock base, a desirable aim for earthquake isolation is to provide a natural period of oscillation in the structure laterally of at least 2 seconds by introducing flexibility at the base with appropriate damping to prevent the build-up of oscillations (Fig.2).

For softer ground the likely earthquake ground motion needs to be estimated either from readings taken from strong-motion earthquake recorders or from the depths of alluvium in relation to geological features and a decision made as to the practicability of base isolated methods. Where very deep soft alluvium is found eg. in parts of Mexico City, where the natural ground period of oscillation during earthquakes is in excess of 2 seconds, isolation methods may be impracticable. The favoured method of providing earthquake resistance in such circumstances is likely to be to build squat rigid buildings of a few storeys only, with plenty of shear walls, to provide a natural horizontal period in the building of the order of 0.2 to 0.4 seconds. The effectiveness of this method was shown up in the 1987 earthquake in Mexico City when all the old rigid masonry buildings survived the earthquake without damage whereas comparatively flexible buildings in the height range from 10 to 20 storeys suffered collapse or major damage.

The rigid building design for soft ground was referred to as the "pebble in the bowl of gelatin" technique by American and Japanese designers early in the century [Fig.3] Architect Frank Lloyd Wright effectively adopted this method for the Imperial Hotel in Tokyo in 1915 which survived the 1923 Tokyo earthquake but was subsequently demolished in 1928 as the piles were too short and it settled excessively in the soft alluvium.

Where the soft ground is shallow above rock, one method that has been employed in New Zealand is to use piles within hollow piles down to the rock base in order to achieve isolation. The inner piles carry the weight of the structure and are allowed to pivot about their bases to take up the clearance within the hollow piles. Damping of the oscillations is provided by steel or lead extrusion dampers at ground surface level, with a travel equal to the pile clearance. This method has been adopted for Union House in Auckland and the new Police Station in Wellington, which was completed in 1991 [4]. In Japan and the U.S.A. however, the application of base isolation techniques has been confined to the use of rubber bearings.

It is worth bearing in mind that maximum ground deformation is likely to occur in the alluvium around the perimeter of a valley, where flexible ground waves "break" against the rock in somewhat the same way as sea waves break on the beach, with an increase in both vertical and horizontal movement. In choosing a base-isolation system therefore, designers must make sure that it is not likely to be swamped by large vertical and horizontal deformations. Freeman, in 1932, [4] describing Californian earthquakes, noted "The localities most disturbed of all by an earthquake will be found in the narrow zone along the dividing lines between the hard ground and the soft ground where oscillations will rise highest against the resisting hard ground". Earlier, he had noted, following the Owen's Valley, California, earthquake of 1872, a "long ridge of earth 5 feet, more or less, in height, set up by the earthquake in localities near the zone of contact with the hard rock..." presumably referring to the scarp set up at the division between hard and soft ground.

In the "edge of valley" condition therefore, clearances for base isolated systems are less likely to be preserved, which leads to the conclusion that for the most severe earthquakes base isolation is reliably effective on good ground but that in other locations caution is required in applying the method, and would need to be backed up by a study of strong-motion ground records for the locality.

A Japanese record illustrating variation of earthquake responses on the surface across a valley is shown in Fig.5 [18]. It shows (a) a drop in frequency and a somewhat greater acceleration in the E.W. direction at sites 2 and 3 on peat near the centre of the valley as compared with adjacent gravel sites, with a maximum value of acceleration in the position adjacent to the rock at site 2. Similar records have been obtained at valley sites in New Zealand.

Rubber bearings are best employed for "good" ground situations as otherwise the flexibility required in the horizontal direction is so large that the building may tend to horizontal instability, which would be accentuated during wind loading. Isolation is not normally sought in the vertical direction, as structures can withstand a degree of vertical overload. In consequence bearings are usually designed to be stiff in that direction. (see para 3.3)

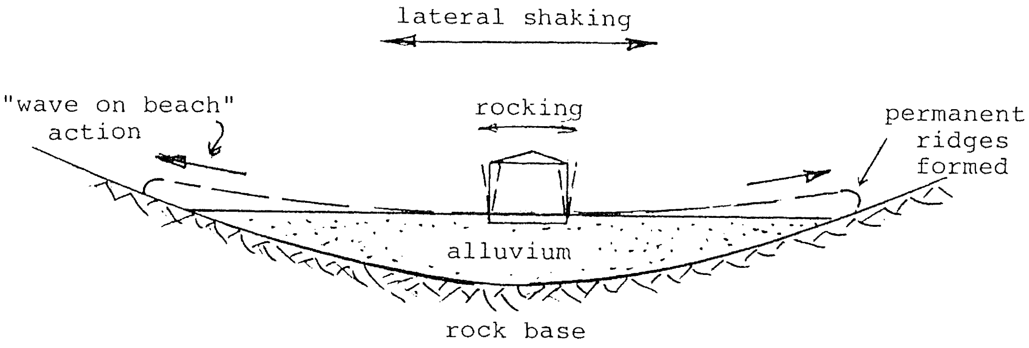


FIGURE 3 "THE PEBBLE ON THE BOWL OF GELATIN" CONCEPT FOR A VALLEY FLOOR

Lateral shaking causes a "sloshing" of the alluvium generating a long period oscillation. A rigid body founded on the alluvium will sway but not fail.

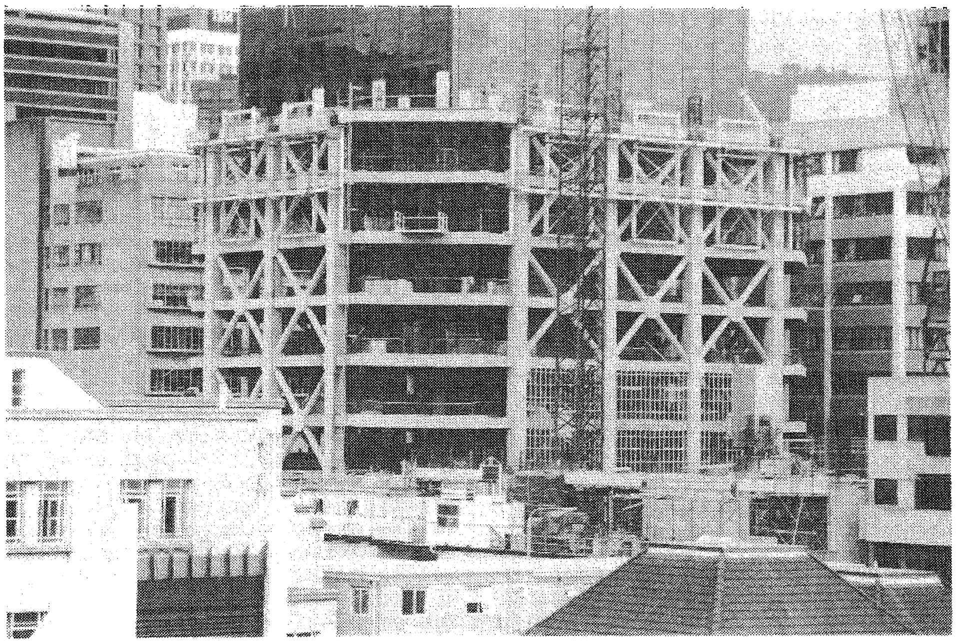
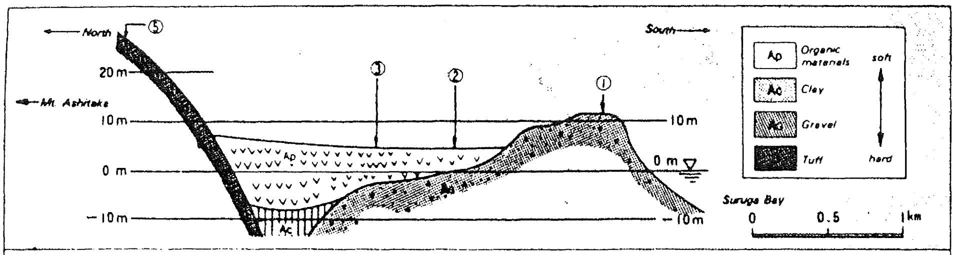
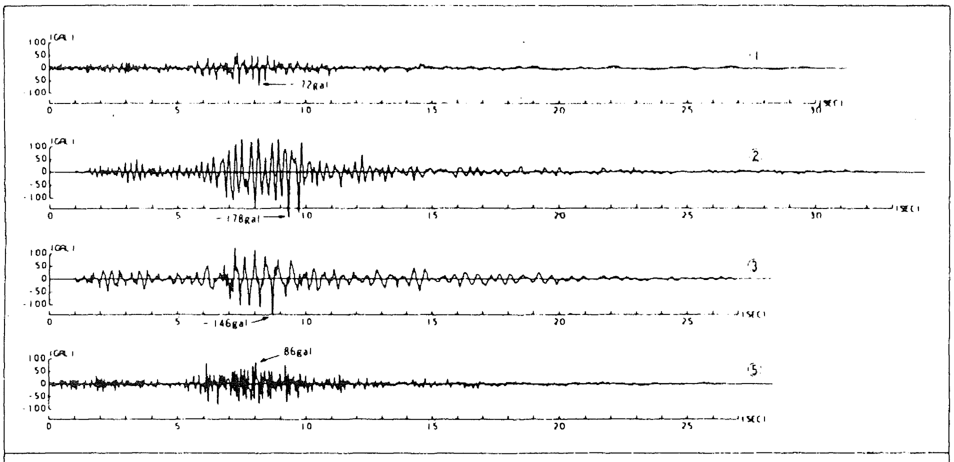


FIGURE 4 THE POLICE STATION, WELLINGTON, UNDER CONSTRUCTION, 1990.

The building is supported on pivoting piles with lead-extrusion dampers.



Geological condition of Numazu Site



Comparison of E-W components of records

FIGURE 5 RECORDS FOR A JAPANESE VALLEY.

The greatest recorded ground acceleration is seen to be near the edge of the alluvium, with the longest period at the centre. Maximum values of E.W. acceleration shown are: (1) hill 0.072g, (2) edge of valley 0.178g, (3) near centre of valley 0.146g, (5) hill 0.086g.
(Illustration courtesy Public Works Research Institute, Ministry of Construction, Japan [18]).

3. DESIGN CONDITIONS FOR RUBBER BEARINGS

3.1 Vertical load capacity

A rubber bearing employed in a base isolation system must carry that part of the dead weight loading which is projected down from the structure above the bearing, including the building contents, while at the same time performing considerable displacement in the horizontal direction.

For squat bearings, the vertical load capacity is controlled by allowable strains in a single layer of the elastomer, as the effect of increasing load is to cause more bulging at the exposed edges which eventually leads to splitting. One requirement of U.K. Bridge Specification BE1/76, which is a commonly used specification and the basis of other specifications, is that the maximum shear strain, e_t, due to total dead and live loading occurring at the edges of the bonded plates, shall not exceed ⅓ (for normal design loadings) to ½ (for abnormal loadings) of the elongation strain at break for the chosen rubber.

In view of recent favourable fatigue tests in shear in Japan, in which shear strains of up to 600% were applied (para 4.4), and testing in New Zealand at strains of up to 200% [8], it is possible that where appropriate research can be completed, these limits may be the subject of revision for base-isolated structures subjected to the rare earthquake loading.

In BE1/76, $e_t = e_q + e_b$

where e_q = the shear strain arising from the bulging effect arising from the effect of vertical load,

and e_b = the imposed shear strain arising from lateral deflection of the structure.

Also $e_q = 6Se_c$

where S = the shape factor for a component elastomer layer, defined as the ratio of the loaded area to the force-free (exposed) area at zero shear strain

and e_c = average compressive strain of the elastomer (in the vertical direction) between two adjacent reinforcing plates.

$$= \frac{\sigma}{E_c}$$

where σ = average compressive stress on the elastomer
= V/A, where V = applied vertical load and A is the cross sectioned area of the bearing,

and E_c = the effective modulus of elasticity of the elastomer (as defined below).

Thus in BE1/76, the vertical load capacity is limited by the allowable shear strain, which limitation is accentuated by the reduction in bearing area as the shear strain increases, given as

$$A\left(1 - \frac{\sigma L}{L} - \frac{\sigma B}{B}\right)$$

approximately, for a bearing of plan area A (= length L times breadth B) and lateral displacements σL and σB respectively, which increases the value of e_c and hence e_q.

A further requirement of BE1/76 is that the total compressive strain due to dead and live loading and bulk compression of the elastomer shall not exceed 0.1. This gives

$$e_c + \frac{\sigma}{E_\infty} < 0.1$$

where E_∞ is the bulk modulus of elasticity of the elastomer and e_c is the average compressive strain for the zero shear conditions. At a compressive strain of 0.1 a small departure from linearity is obtained on the stress/strain curves (Fig.6).

For taller bearings, where their height exceeds about one half of the least plan dimension, instability or overturning are more likely to be a criterion of failure, particularly with regard to the large horizontal displacements which may occur during an earthquake. In general, as the vertical load increases the shear capacity decreases and eventually buckling occurs. At the buckling load the capacity in shear is zero. A discussion of design considerations is given by Stanton and Roeder [5].

The confinement of the rubber between the bonded reinforcing plates dictates the mean vertical stress required to develop the allowable vertical strain of 0.1, as required by BE1/76; the thinner the rubber layers, ie. the closer the plates are together, the greater the stress required to develop the allowable strain [5], as is shown up in the calculation below.

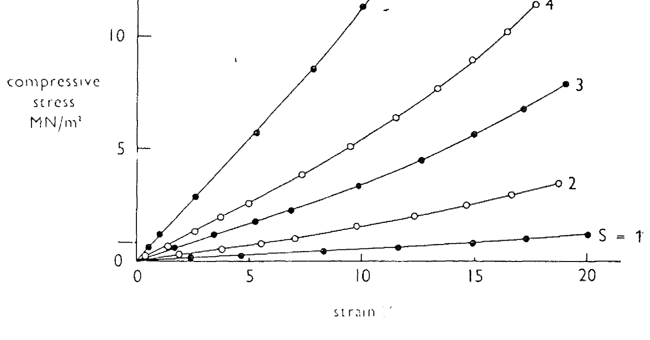


FIGURE 6 STRESS/STRAIN CURVES FOR RUBBER IN COMPRESSION RELATED TO SHAPE FACTOR S [6].

The rubber confinement determines the value of the effective compression modulus E_c of each rubber layer.

Lindley [6] gives:

$$E_c = E_0(1 + 2kS^2)$$

where E_0 = Young's modulus for unconfined rubber in a test machine for strains of a few per cent.
k = a numerical factor dependent on rubber hardness
S = Shape factor
= ratio of the loaded area to the force-free area for the layer

$$= \frac{LB}{2t(L+B)}$$

for a rectangular bearing of length L and breadth B and layer thickness t

$$= \frac{D}{4t}$$

for a circular bearing of diameter D

Values of E_0 and k are given in Table 1 below for a range of values of rubber hardness. With lower hardness values, smaller loads are required to reach a given strain, so varying the rubber hardness affords one way of assuring the same deflection in a number of bearings when column loads vary throughout a foundation, although measured stiffness of bearings as manufacturred often vary by ± 20% of those specified.

Lindley [6] further suggests that "the most accurate published value of the bulk modulus of gum natural rubber appears to be about 2.000 MPa" and this valuue is usually adopted for most codes of practice.

The effect of varying the thickness of the elastomer layers may be demonstrated by considering the stress σ required to develop the limiting compressive strain of 0.1 as required by BE 1/76. Allowing for the bulk modulus E_∞ gives:-

$$0.1 = \sigma\left(\frac{1}{E_c} + E_\infty\right)$$

$$\text{giving} \quad \sigma = 0.1\left(\frac{E_c E_\infty}{E_c + E_\infty}\right)$$

As an example, for the Devils Creek bearing illustrated in Figs 1 and 10, the details were:
Overall size: 280 x 230 x 113mm
Steel plate size: 266 x 216 x 3mm
Internal rubber layers 7mm thick

$$S = \frac{LB}{2t(LB)} = \frac{266 \times 216}{2 \times 7\ (266 + 216)} = 8.514$$

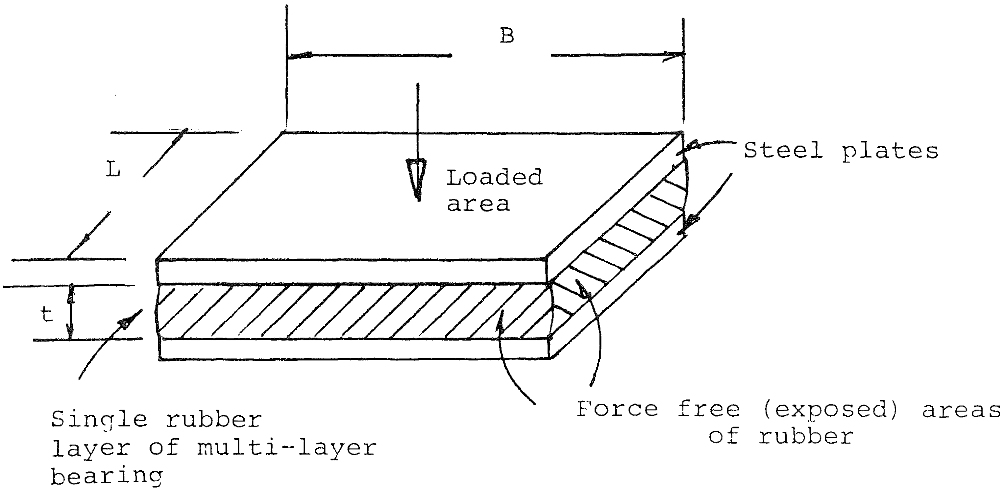


FIGURE 7 SHAPE FACTOR.

$$\textbf{SHAPE FACTOR} = S = \frac{\text{Loaded area}}{\text{force free area}} = \frac{LB}{2t\ (L + B)}$$

The load carrying capacity of a laminated rubber bearing is determined by the shape factor of a single layer for squat bearings. The shape factor controls the degree of bulging at the free surface.

TABLE 1 HARDNESS AND ELASTIC MODULI
[due to Lindley Ref.6]
Based on experiments for typical natural rubber spring vulcanizates.

Hardness IRHD ± 2	Young's modulus Eo MN/m2	Shear modulus G MN/m2	k numerical factor	Bulk modulus E_∞ MN/m2
30	0.92	0.30	0.93	1000
35	1.18	0.37	0.89	1000
40	1.50	0.45	0.85	1000
45	1.80	0.54	0.80	1000
50	2.20	0.64	0.73	1030
55	3.25	0.81	0.64	1090
60	4.45	1.06	0.57	1150
65	5.85	1.37	0.54	1210
70	7.35	1.73	0.53	1270
75	9.40	2.22	0.52	1330

k = 0.70 for an IRHD hardness of 53 for this particular bearing and
E_0 = 2.8 MPa approximately
and E_∞ = 2.000 MPa

Hence $E_c = 2.8\ (1 + 2 \times 0.70 \times 8.514^2)$
$= 287$ MPa

giving $\sigma = 0.1\ \frac{(287 \times 2000\ )}{(287 + 2.000)} = \underline{25.1\ \text{MPa}}$

If the size of the rubber layers is changed to 8mm then following the same calculation:-

$S = 7.45$
$E_c = 220$ MPa
and $\sigma = \underline{19.8\ \text{MPa}}$

Thus the limiting compressive stress is lowered from 25.1 MPa down to 19.8 MPa when the thickness of the rubber layers is increased from 7 to 8mm. Checks for the shearing condition also would need to be carried out.

This shows that for a given bearing plan size the allowable load can be increased by decreasing the thickness of the rubber layers, or, alternatively, for a given load, the plan size may be reduced by specifying thinner rubber layers, although stability under high shearing deformations then becomes an increasing problem (Figure 8b). Bearings in New Zealand have been manufactured with rubber bearing thicknesses in the range 7 to 16 mm while those manufactured in Japan and the U.S.A. have tended to have rubber layers about 7mm thick possibly in order to control resonance with

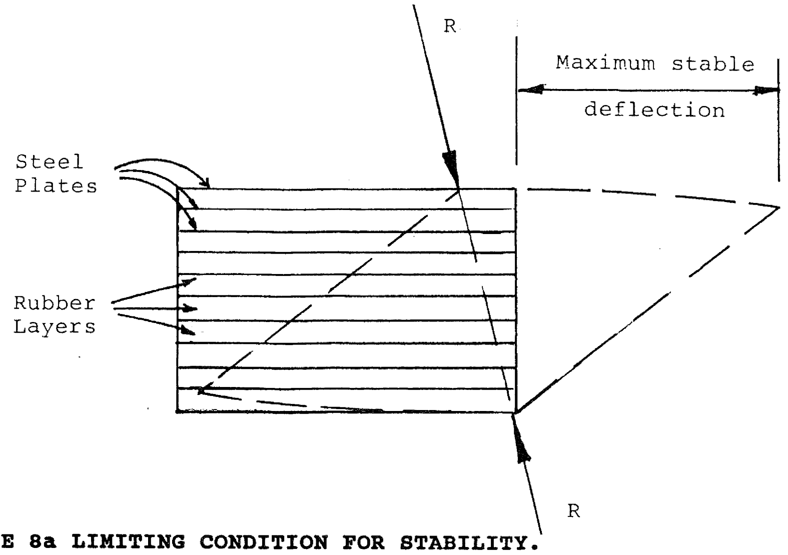


FIGURE 8a LIMITING CONDITION FOR STABILITY.

Overturning occurs when the net resultant thrust at peak deflection passes outside the opposite corners of the bearing.

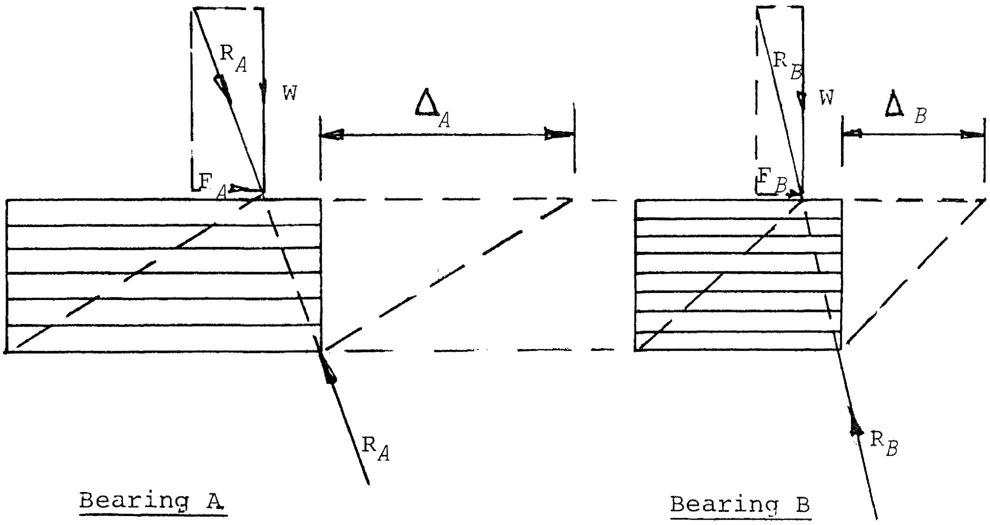


FIGURE 8b COMPARISON OF TWO BEARINGS.

Bearing B, having a smaller plan size, can be designed to have the same vertical load capacity as A by reducing the thickness of the rubber layers and steel plates to keep the same height of bearing and total thickness of rubber. But A will be more stable than B as Δ_A for instability exceeds Δ_B. There is some amelioration however as the reduced plan size of B makes for a lesser horizontal stiffness.

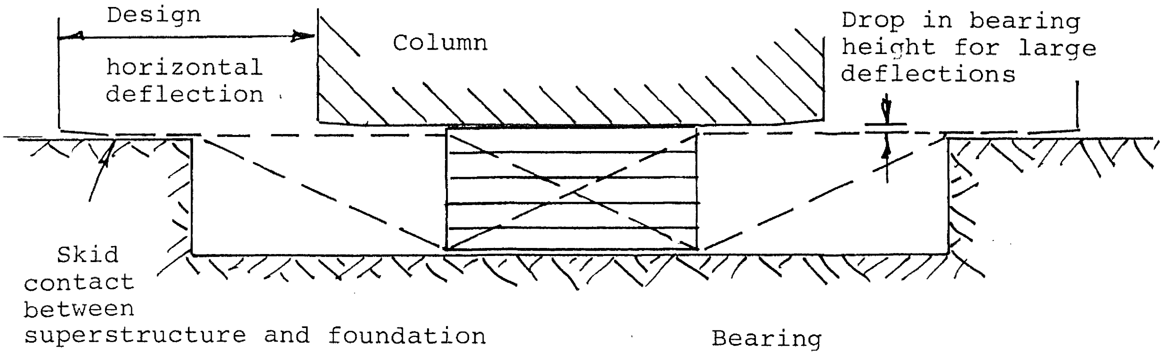


FIGURE 9 USE OF SKIDS.

The drop in bearing height allows the structure to slide on projecting skids for large vertical deflections.

vertical earthquake (See para 3.2), whereas in New Zealand bearings for the William Clayton building, completed in 1981, employed layers 15 mm thick. The provision of thin rubber layers makes for additional expense in bearing manufacture and designers and researchers should consider whether vertical resonance will be a problem when at the same time providing the required shear stiffness and preserving stability.

3.2 Instability under combined vertical and horizontal loading

While there are usually no instability problems for squat bearings with 10% limiting vertical strain, where the total height does not exceed about half the minimum plan dimension, it is apparent that, as the height increases, consideration of buckling arises (e.g. Ghent [7]). In addition, for large lateral strains associated with base isolation systems, the possibility of overturning arises. Essentially overturning will occur when the line of resultant thrust passes outside the limits of the base of the bearing (Fig 8)

Two methods are available to ensure stability by way of designing out the problem. Either (a) skids may be employed to transfer the vertical load to surrounding concrete upstands, taking advantage of the drop in bearing height for large deflections (see Fig.9 and para 3.3) or (b) the high-stability aseismic bearing (HSAB see para 5) may be used. In the latter instance the rubber is distributed to the extremities of the reinforcing plates in the form of circular discs to increase the plan size to any required extent within practical limits. For the two base isolated buildings constructed in New Zealand lateral deflection has been limited by collision with the perimeter retaining wall (see Figure 12c).

3.3 Vertical deflection

An endeavour is usually made to design isolation bearings as stiff as possible vertically in order to avoid the possibility of resonance with vertical earthquake oscillations. An alternative may be to allow greater deflection and a high degree of vertical damping to control lurching. The use of a shake table would establish a preference.

As an example of the calculation of the natural period for vertical motion, taking the tallest standard bridge bearing available in the plain size 609 x 609 mm, the relevant details taken from manufacturer's tables are: total height 391mm, internal rubber layer thickness 20mm, compression stiffness 194 kN/mm, total rated load at maximum shear (50% shear strain) of 1728 kN for UK HA bridge loading, which gives a static deflection of 8.9mm for this loading.

The natural period T vertically is then given by:

$$T = 2\pi\sqrt{\frac{0.0089}{g}} \quad \text{where } g = \text{acceleration due to gravity} = 9.8 \text{ m/s}$$

$$= 0.19 \text{ seconds}$$

If research shows that a much higher loading may be accepted for the comparatively static situation of a building, say the value of 4858 kN at zero shear appropriate to UK HB loading, quoted in the same manufacturer's tables, then, following the above calculation, the static deflection becomes 25.0mm and T becomes 0.325.

As the above values of period for typical design loadings are within the range 0.1 to 1.0 seconds for likely peak earthquake input (Fig.2), the use of thinner rubber layers, or a high degree of damping, is desirable, particularly where spigots are used to drive the bearing, as the spigots could leap out of their sockets. (Fig.10) This is an argument for fully-bonded bearings, (see para 6). To date lead-rubber bearings with internal rubber layers having thicknesses in the range of 7 to 16mm have been used in base isolated structures throughouut the world, with a tendency now to use the thinner layers. Bridges may employ thicker layers to accommodate bearing rotation.

An additional complication to the calculation of vertical deflection arises as full-scale shake-table tests at the University of California, Berkeley, have shown that as the shear deflection increases the vertical deflection increases, i.e. the structure dips down towards the extreme of the horizontal travel and raises up as it passes back through the neutral position, i.e. a bucking action occurs. (Fig.9) It is not necessarily a disadvantage however as hysteresis loops show that the action is associated with a loss of energy, which may compensate for any increase in vertical flexibility. The phenomenon depends on the temporary increase in compressive stress from corner to corner of the bearing in the line of the thrust, as demonstrated in Figs.10 and 11. Testing of full-sized bearings indicates that the rubber is capable of taking the transient overload. The clearance which must be introduced in the design of skids (see para 3.3) to relieve vertical load on the bearing must be the subject of tests for a particular design. Overlapping plates around the perimeter of the structure which bridge the clearance gaps need also be designed to allow for the drop in the structure for extreme horizontal deflection as the skidding effect will occur on all perimeter contact points.

When, in addition to earthquake protection, it is desired to either (a) limit the noise input into a building arising from underground trains or traffic transmitted by ground vibrations, or (b) limit the transmission of noise from an item of plant, e.g. a diesel generator, to surrounding buildings, these conditions require a vertical dead weight deflection, for zero shear, of at least 1cm in the bearings. The consequent softness may cause undesirable resonance with vertical earthquake oscillations, but, as very little amplitude of motion is associated with the noise isolation requirement, the bearings can be arranged to lock up vertically after a very small vertical motion in order to control the earthquake motion. This may be at the expense of a delicate instrument such as an

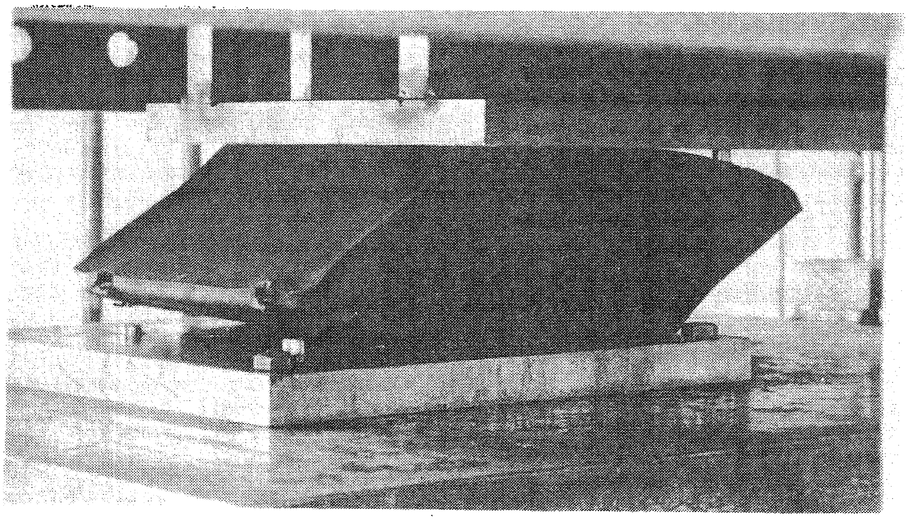


FIGURE 10 LEAD-RUBBER BEARING UNDER TEST TO 200% SHEAR STRAIN.
The driving studs in this early design are seen to be exposed by the deformation [8].

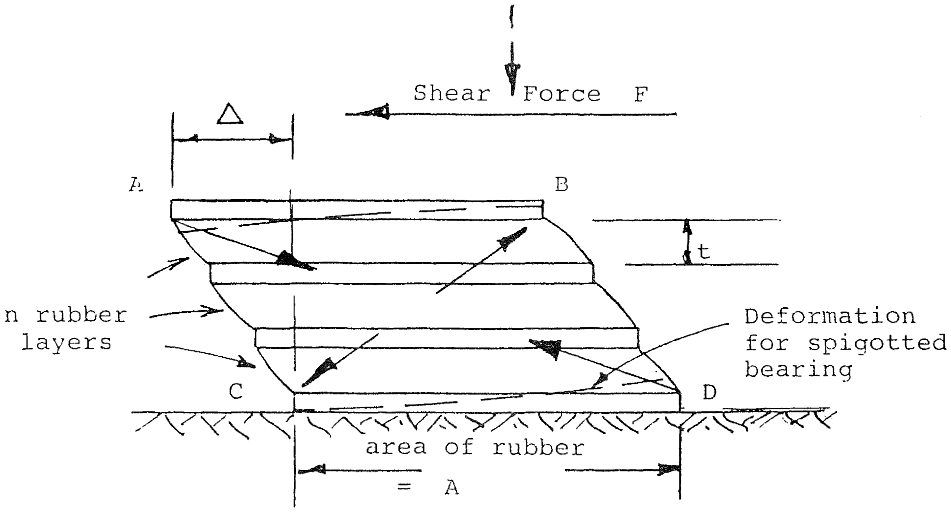


FIGURE 11a RUBBER BEARING IN SHEAR.

Shear stiffness $= k_s = \frac{F}{A} =$ GA/nt where G = shear modulus

The shear deformation causes diagonal dimension AD to extend and CB to compress. For spigotted bearings, the rubber resists this deformation and moves away from the structure at A and D, bending the steel plates. (See Figure 10).

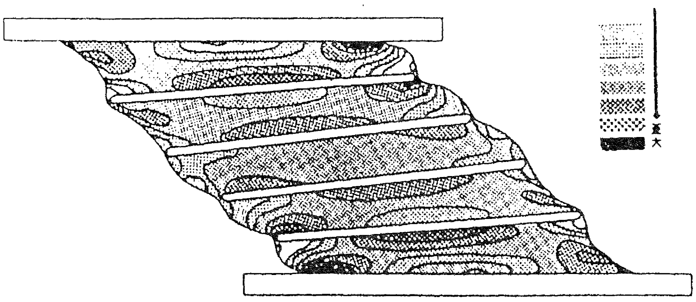


FIGURE 11b FINITE-ELEMENT STUDY OF STRESSES IN A BEARING UNDER LOAD

showing concentration of compressive and tensile stresses and tilt of reinforcing plates.
(Illustration courtesy Bridgestone Rubber Co., Japan)

electron microscope however, and sophisticated arrangements have been introduced by Japanese designers to provide for greater damping for vertical deflections.

3.4 Shear Stiffness

The shear stiffness of a rubber bearing used in a base isolated structure must provide an appropriate natural period of lateral oscillation for the structure, certainly in excess of 1 second and preferable in the order of 2 seconds, so that it does not resonate to those predominant frequencies which make up the earthquake oscillations.

The shear stiffness of a rubber laminate, K_s is directly proportional to its cross sectional area, A, and inversely proportional to its height, t (see fig 11, i.e.)

$K_s = GA/t$ where G is the shear modulus of elasticity [6]

For a number of layers, n, in a bonded stack in a bearing to give a total thickness T, i.e. $T = tn$ then $K_s = GA/tn$. The steel laminates in a sandwich do not influence the value of the shear stiffness.

Thus while a structure can be made very flexible laterally by increasing the height or decreasing the cross sectional area of a bearing, both actions would compromise stability (Fig.8) and a compromise is reached in design.

The periodic time T (seconds) for a supported mass M (tonnes) and a shear stiffness K_s, (kN/m) is given by

$$T = 2\pi\sqrt{\frac{M}{K_s}}$$

Evidently for a 1 tonne bearing (the lowest capacity at present available in a high stability seismic bearing, (see Para 5), the lateral stiffness required for a period of 2 seconds is 9.186 kN/m. This translates as a 0.1 kN (10 Kg weight) force to deflect the bearing laterally 1cm, which flexibility is easily achieved for a manufactured bearing by pushing with the foot on the top of the bearing, which is indicative that, where this type of bearing is used to isolate items of electrical or other plant in the open, then the effects of wind loading need to be checked, and adequate damping provided.

In practice the usual provision of yielding or frictional damping in the system reduces the effective periodic time to less than the value calculated by the above formula. It nevertheless serves initially as a means of selecting bearing stiffness, and periodic times calculated from it are quoted in manufacturer's tables.

3.5 Shear Deflection

As testing work at the Department of Scientific and Industrial Research, New Zealand, has indicated that laminated rubber bearings manufactured in New Zealand will not fail when cycled repetitively at 200% shear strain in the rubber,[8] it is suggested that design is based on a shear strain of 100% for a lateral deflection of ±150mm as a minimum, but that clearances around a structure should allow for at least ±300mm. If it is desired to design for a particular earthquake record, suggested as appropriate to a particular location, then the magnitude of the induced shear deflection can be calculated.

The allowance for shear deflection for the William Clayton Building in Wellington, completed in 1981 was 150mm prior to contact being made with the perimeter retaining wall. In hindsight the clearance could have been more than twice this with no instability in the bearings. Subsequently in 1985, following visits to New Zealand by designers, the San Bernardino Law & Justice Centre was constructed for movements in excess of 14in (350mm) depending on the location of the bearings, and the new Police Station in Wellington for 300mm using a pivoting pile system. In the most recent construction for the Wellington Newspapers Building in Petone, the clearance allowed around the building perimeter is 460mm (Figure 12c).

4. DAMPING METHODS

4.1 General

As a bearing laminated from natural rubber behaves as a pure spring with very little damping, the build-up of undesirable prolonged oscillation must be avoided by providing damping. Up to quite recently damping was normally provided external to the rubber, but natural rubbers with damping provided within the rubber itself were introduced in 1985 (see Para 5).

4.1.1 Lead-rubber bearing (LRB)

The most significant advance in damping came in the 1970s with the introduction of a lead plug down the centre of the bearing following research work at the DSIR in New Zealand (Figures 1 & 13) This method has been accepted worldwide and buildings and bridges have been constructed in New Zealand, California and Japan using the method, using bearings manufactured under licence in these two latter countries. It was first used in New Zealand in the William Clayton Building, completed in 1981. The method effectively superimposes a nearly rectangular hysteresis loop due to the lead on to the linear spring characteristic of the rubber (Figure 13), with consequent absorption of earthquake energy, and control of oscillations. The rounding of the resulting loop is dependent on "give" during strain transfer at the lead-rubber interface. A typical set of hysteresis loops for a 6 cycle earthquake imposed on a bridge bearing is shown in Figure 13b [8] giving values of damping up to 40% of critical. The transmission of deformation to the lead plug via the rubber and the steel plates dominates the loop shape for small strains, but at high strains the slope of the flat top is roughly that of the rubber spring alone.

The use of a lead plug locks up the bearing until the lead shears (at approximately 8

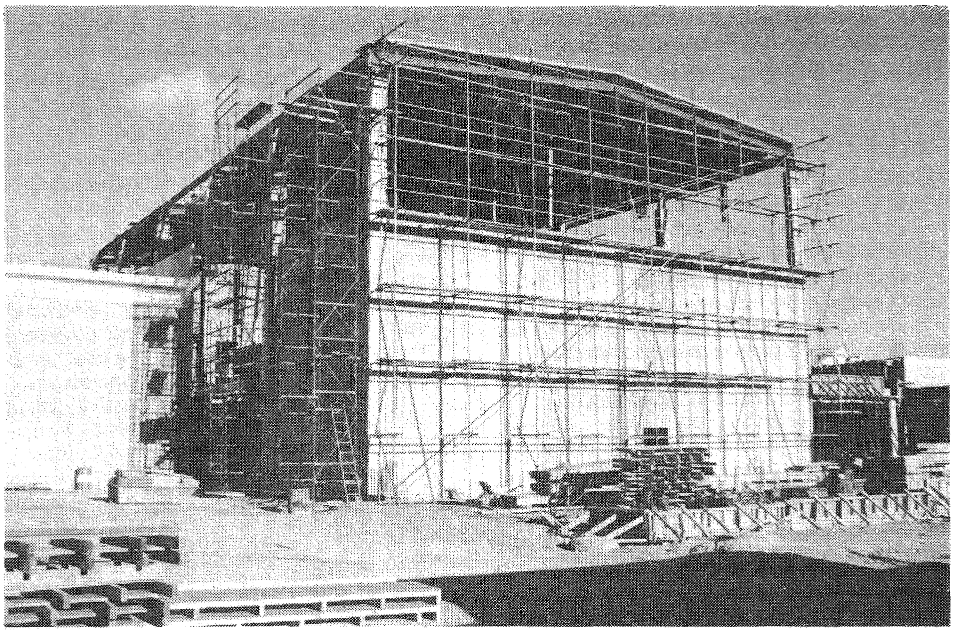


FIGURE 12a BUILDING FOR WELLINGTON NEWSPAPERS LTD UNDER CONSTRUCTION, 1990.

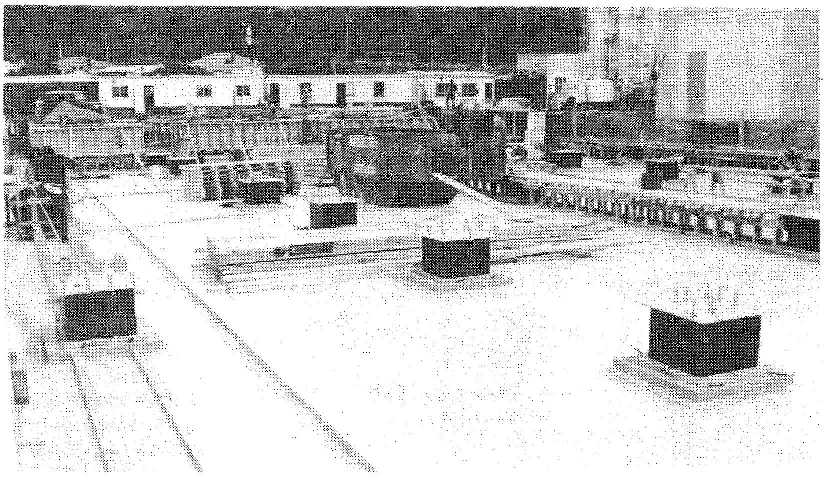


FIGURE 12b BEARINGS IN FOUNDATION OF THE WELLINGTON NEWSPAPERS LTD BUILDING, PETONE
Some have lead plugs, others not.
[Photos courtesy Skellerup Industrial Ltd.]

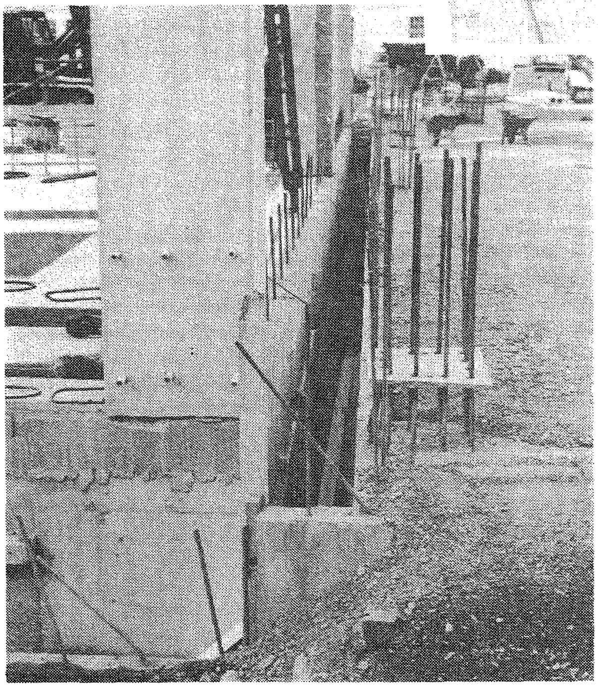


FIGURE 12c CLEARANCE AROUND BUILDING UNDER CONSTRUCTION FOR WELLINGTON NEWSPAPERS LTD.

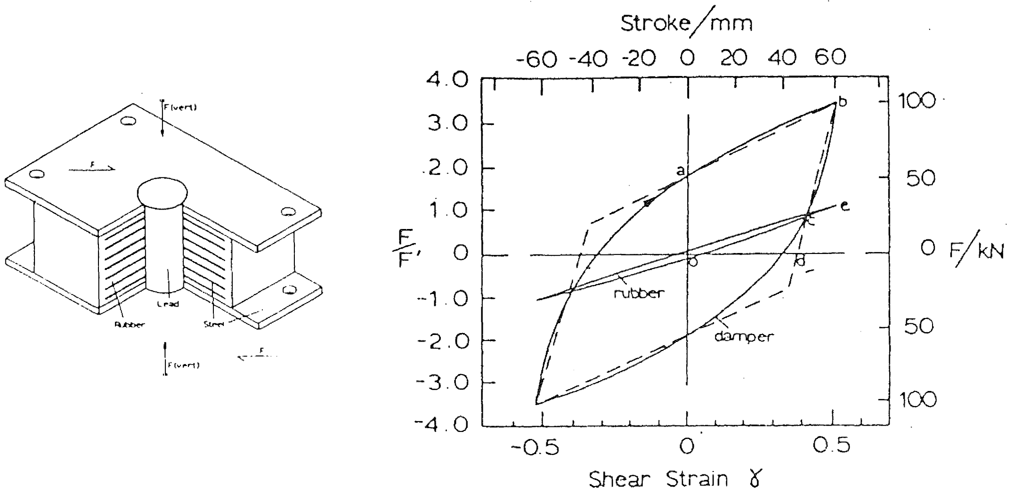


FIGURE 13a LEAD-RUBBER BEARING AND FORCE/DEFLECTION HYSTERESIS LOOP [17].

The outer loop represents the combined action of the lead and rubber.

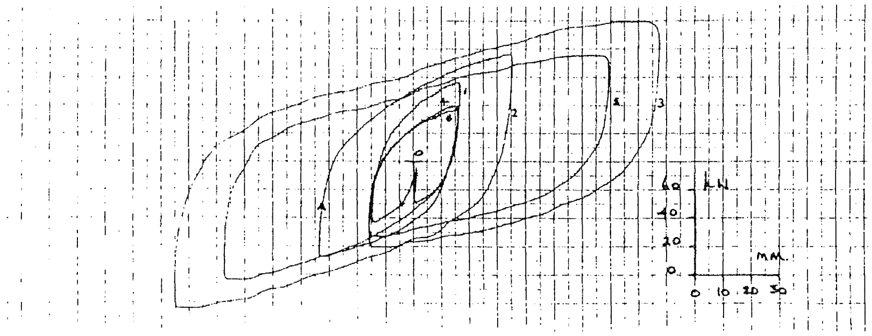


FIGURE 13b HYSTERESIS LOOPS FOR A SIX CYCLE EARTHQUAKE

at 1 Hz for a peak strain of ±125% in the rubber. Frequency 1 Hz, for the bearing shown in Fig.10 [8].

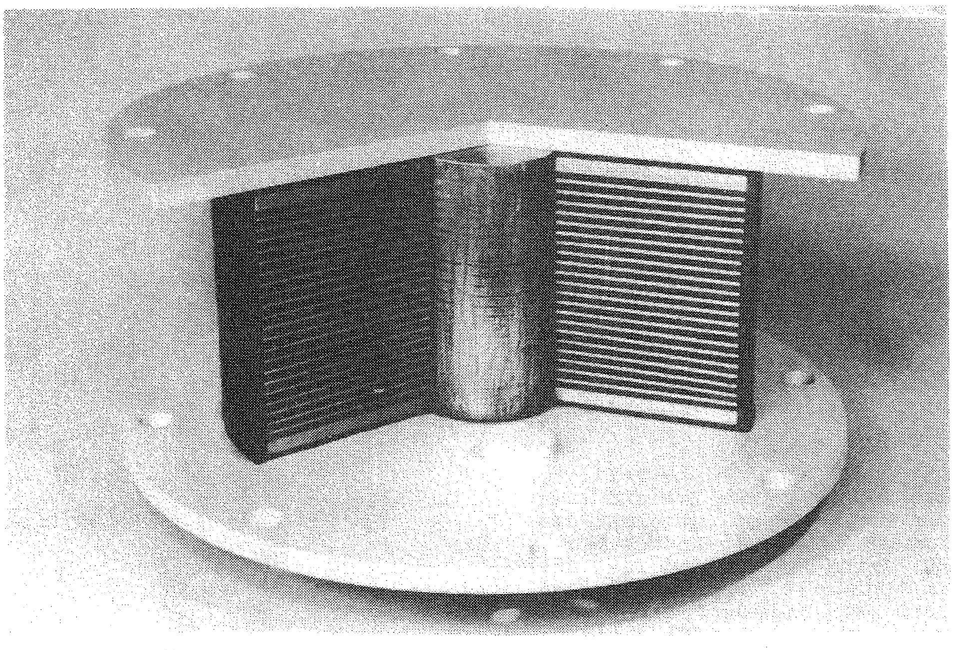


FIGURE 13c JAPANESE LEAD-RUBBER BEARING

Photo courtesy Oiles Industry Co. Ltd., Japan.

MPa in the lead). Thus a building on this type of bearing is protected for a disastrous earthquake and will not generally yield under the effects of wind, but responds in the same way as conventionally founded buildings for small earthquakes. Where industry requires amelioration of small earthquakes for the protection of industrial processes, Japanese engineers have introduced viscous damping in combination with steel dampers externally to the bearings. The viscous damping may be in the form of cylindrical (car-type) oil shock absorbers, or oil-bath types employing paddles, Fig.14. The onset of an earthquake initially affects the viscous damping elements, but after a chosen deflection is reached the steel dampers are introduced to control larger deflections. More recently the use of higher damping rubber has been introduced in Japan which is claimed to soften the effect of small earthquakes [Ref 1,2] (See para 4.4)

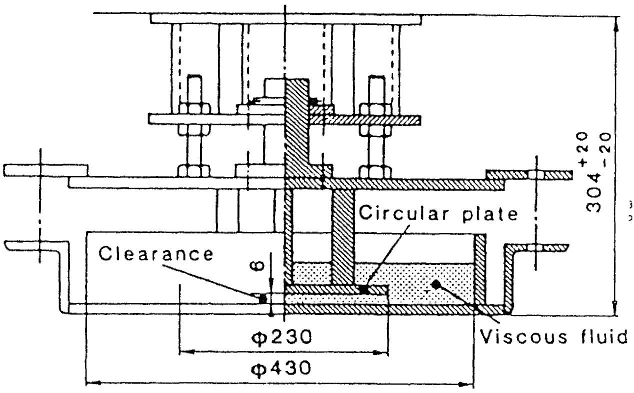


FIGURE 14 VISCOUS DAMPER [13].

4.1.2 Use of steel dampers

The use of steel dampers in Japan loaded into the plastic range in bending, has sparked off a continuation of testing work, which was initiated at the DSIR in New Zealand from 1970 with the main interest in Japan being the performance of round mild-steel bars, either manufactured as straights or bents, often fixed at one end and loaded as a cantilever at the other through a sliding ball joint which allows for the increase in length required as the structure deflects laterally.

The introduction of the high-stability aseismic bearing (HSAB) in New Zealand (see para 5) for the isolation of equipment has revived an interest in steel dampers here, as at present internal damping is not provided in the HSABs. For the isolation of electrical plant at Haywards Substation steel taper cantilevers [10] were designed by the DSIR with sliding ball joints allowing for the devices to be loaded in bending while undergoing lateral displacement. (See Fig.24)

Probably the most useful form of steel damper for use externally to rubber bearings, is the plain round bar fixed at both ends, for which design details are available [11] (Fig 15). For this type plastic hinges develop at the points of fixity as the steel progresses into yield. The required length extension during lateral displacement is obtained by providing a bend in the bar either as a half circle or in one application in Japan, [12] (Fig 16) as a complete circle. This causes the bar to progress into yield in tension as the deformation increases, which gives damping and restraint vertically as well as horizontally. A further hinge may then develop at the centre of the bend depending on its proportions. Parameters for the half-circle type were defined by early DSIR

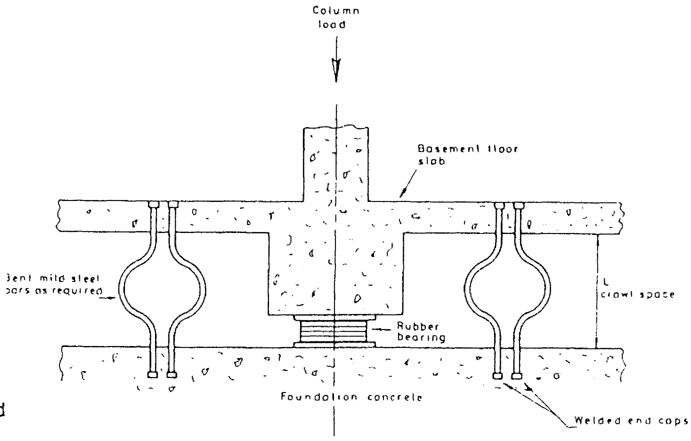


FIGURE 15a PROPOSED BASE ISOLATED METHOD USING BENT ROUND BARS [11].

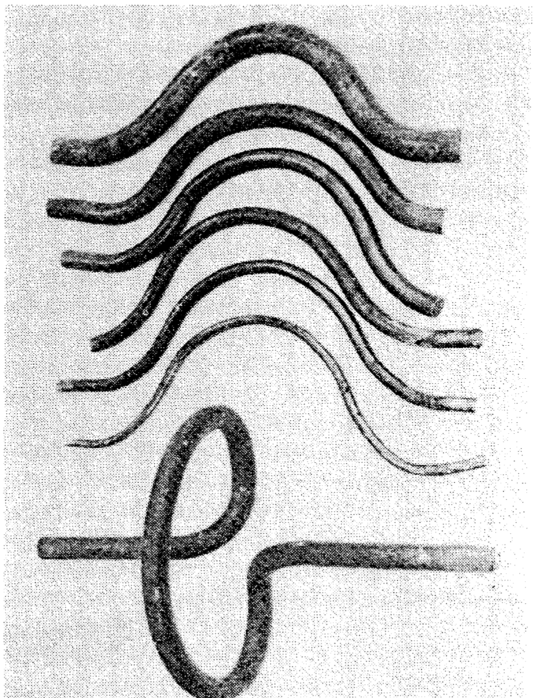
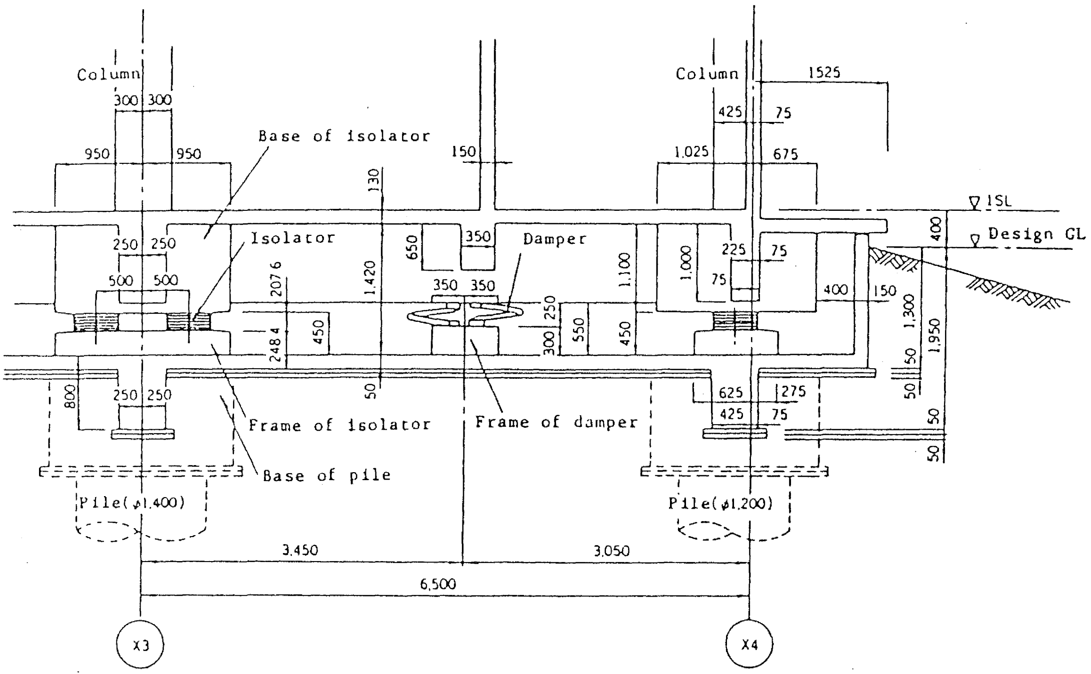


FIGURE 15b FAILED BENT-BAR DAMPERS FOLLOWING TESTING [11].



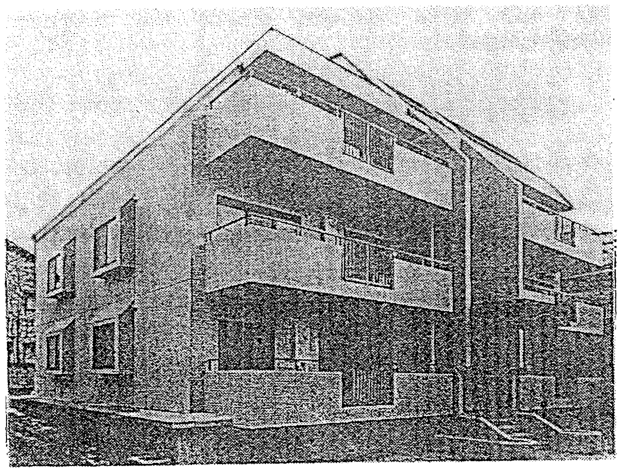


FIGURE 16 BENT BAR DAMPERS USED IN AN ISOLATION SYSTEM

for a four storey apartment block in Japan, which was subjected to an earthquake in 1987. The measured acceleration within the building was reduced "to one third or one fifth of that of an ordinary building". (Illustrations courtesy Okumura Corporation, Osaka, Japan, [12])

testing [11] but further work could usefully be carried out on the full-circle type used recently in Japan by incorporating this type of damper in large shake-table tests. The stiffness of the bent bars in tension could also contribute to the control of vertical resonance. (See para 3.2)

4.1.3 Comparison of damping methods

Various types of damper for industrial plants were compared in a paper by Japanese authors which was presented at the 9th World Conference on Earthquake Engineering in Tokyo in 1988 [12]. The types included a friction damper, a lead shear damper which was engineered to have a near-rectangular hysteresis loop, similar to the friction damper, a cylindrical oil damper and a viscous shear damper (Figure 14) all of which were tested using a model building on a shake table. It was concluded that "dampers having roundish shapes of hysteresis loops like the viscous shear damper are considered to have the most desirable properties for buildings of industrial facilities". For the viscous types:

(a) Attenuation of response accelerations of unsecured equipment within the building was more effective particularly for frequencies within the range 1 to 5 Hz.

(b) Viscous dampers respond with a greater force as the earthquake becomes greater, but at the same time are effective for even microvibrations.

Thus it was concluded that for semi-conductor manufacturing factories containing sensitive equipment, the viscous type would be most suitable. On the other hand for nuclear power plants the viscous shear damper was not recommended mainly because of the difficulty of manufacture in large sizes and because its damping force is dependent on the air temperature. For this application elasto-plastic steel dampers, lead-rubber bearings and high-damping rubber bearings were advocated.

In design the peak force provided by the damper is usually between 5% and 10% of the supported load.

4.2 Efficient damping in the LRB (lead-rubber bearing)

The use of thin rubber layers, having thicknesses of the order of 7mm, in lead-rubber bearings may have been influenced by early tests in New Zealand at University of Auckland, Central Laboratories, Ministry of Works and Development and DSIR, when thicknesses of rubber layers up to 16mm were used and showed a poor transmission of stress to the lead plugs and hysteresis loops of comparatively small area.

However the author collated the published results available and concluded that as the size of the bearings increased, so also could be the rubber layer thickness [14], at least from the point of view of efficient transmission of stress to the lead plug, aside from the need that thin layers may be required to ensure stiffness in the vertical direction. As an example a high value of shear stress was calculated for the lead plugs for the bearings in the William Clayton Building for which the layer thickness was 15mm.

Lindley [6] showed that the compressive stress in a rubber layer bonded between rigid plates rises from zero at the free edge of the rubber to a maximum value at the centre of the layer. It is this value of stress which confines the lead plug, thus illustrating the desirability of locating lead plugs at the centre of the bearing.

Further development of Lindley's formulae by the Author, [14], showed that the rubber stress at the centre of a bearing was approximately 3.0 V/A for circular bearings and 3.5 V/A for square or nearly square bearings, for rubber having the commonly used hardness of 53 IHRD and dead load V on bearing area A. This enabled rubber stresses to be calculated around the plugs for loads at and below the working loads in the bearings, which had been the subject of tests at DSIR and elsewhere. Results are illustrated in Fig 17, which shows the shear stress achieved in the lead (calculated from experimental hysteresis loops) plotted against the ratio pressure around plug : diameter of plug.

The graph shows that, provided this ratio exceeded 200 MN/m^3, the shear stress in the plug exceeded 11 MPa. At lighter loads below the working load in the bearing the stress transmission was less efficient, as hysteresis loops were relatively smaller in area, as illustrated by points on the left of the graph, when the confining effect of the rubber was less positive. A point of interest is that where the transmission is less efficient the hysteresis loops tend to be more rounded which would make for better attenuation of higher frequencies (see para 4.1) and this aspect could be further investigated.

4.3 High-damping rubber bearings for San Bernardino Building

High-damping rubber bearings to a design by the Malaysian Rubber Research and Development Board, were first used in the isolation system for the Law and Justice Centre, San Bernardino, California, by employing rubber designed to have a lower crosslink density and higher oil and carbon black content then with low-damping rubber.[15]

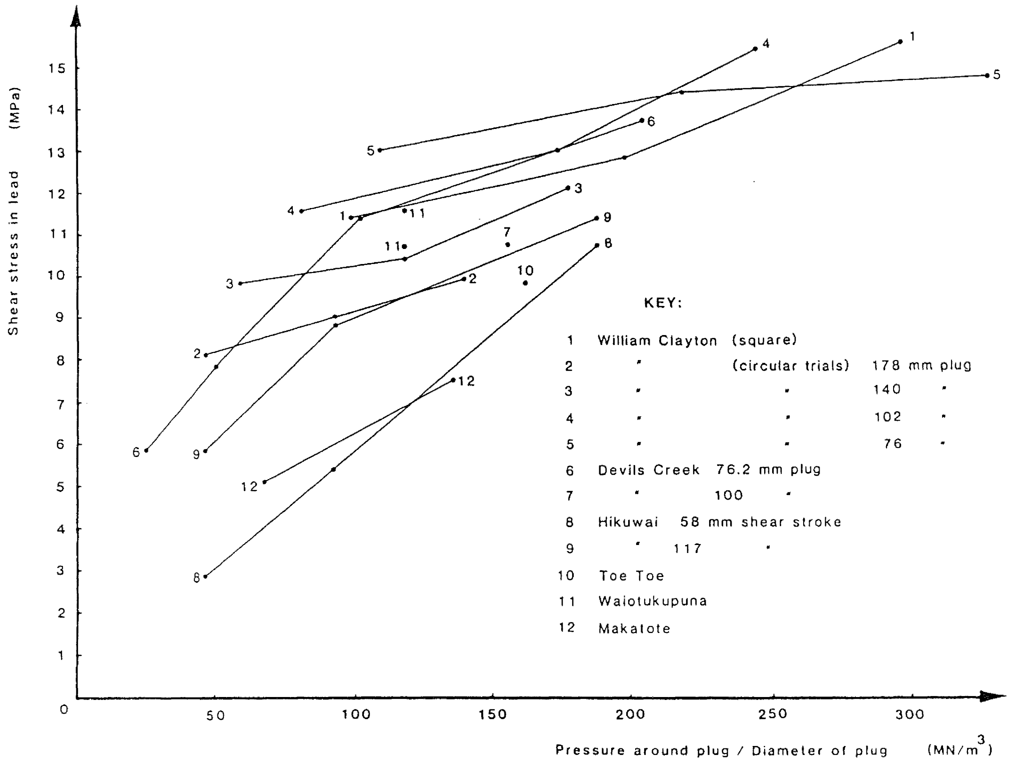


FIGURE 17 SHEAR STRESSES TRANSMITTED TO LEAD PLUG IN LEAD-RUBBER BEARING [14].

At small shear strains the bearings were stiffer and showed greater damping than at larger strains as shown in the following table, due to Derham [15] Damping in the range 16.7 to 9.2% of critical compares with approximately 2-3% for natural rubber bearings commonly used in bridges and structural applications.

TABLE 2 MODULUS AND DAMPING VALUES

Shear strain	Dynamic shear modulus MPa	% critical damping
0.02	5.2	16.7
0.05	3.1	18.3
0.10	2.2	16.6
0.20	1.6	14.1
0.30	1.4	12.0
0.50	1.2	10.8
0.70	1.0	10.2
1.00	0.9	9.7
1.25	0.9	9.2

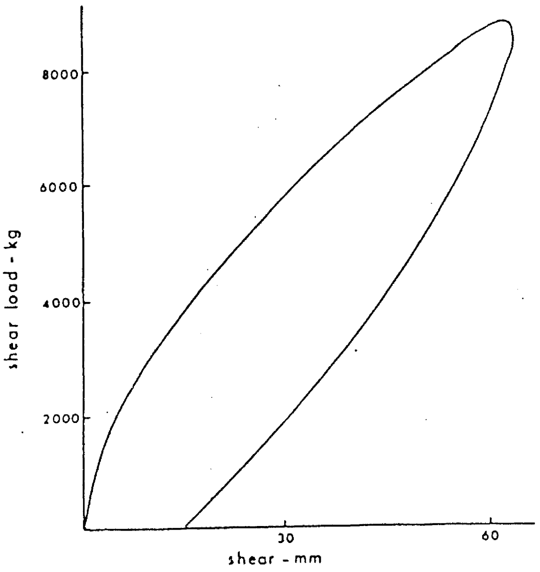


FIGURE 18 HALF-CYCLE LOOP FOR SELF-DAMPING NATURAL RUBBER BEARING

(Critical damping 11.2%, due to Derham [15]).

The increased stiffness and damping at low strains ensures some control of deflections induced by wind loading, estimated to have an amplitude of about 6mm, while at the same time offering damping to small earthquakes, as opposed to a partial lock-up with the LRB up to plastic yield of the lead. A hysteresis loop for a half cycle of shear deflection is shown in Fig.18 [16].

Bearings of this type have not been used in the U.S.A. since the construction of this building and LRBs have been preferred for other applications presumably on the basis of the greater damping which can be provided, which for one bridge in New Zealand was up to 40% of critical (derived from Fig.13).

4.4 <u>Self-damping rubber in Bridgestone HDMRB Bearings</u>

The need to protect nuclear power plants has spurred on the development of bearings in Japan and research into high-damping rubber compounds by Bridgestone has produced the HDMRB type of bearing using an unspecified formulation. Development proceeded by testing model bearings 132 mm diameter, 95.9 mm high, composed of 30 rubber layers 0.9 mm thick, ie. a manageable size for a large number of tests.[1]

Hysteresis loops reproduced in Fig.19 [1] for shear strains of up to 250%, gave values in excess of 15% of critical, ie. greater than the values for the San Bernardino bearings. As with these bearings, the HDMRB's are stiffer at low strains thus providing control of wind oscillations, while comparison of Fig.19(a) for the first cycles of strains and Fig.19(b) for second cycles shows that damping is less for second cycles.

Fig.19 also indicates the development of diagonal tensile strains in the bearings at high strains shown up by the turn-up at the peak of the strains, which is a desirable feature and often a characteristic of shock-absorbing systems. The bearings therefore have a lock-up characteristic, ie. they become stiffer with excessive deflections as a result of being fully bonded (Fig 20). A similar characteristic was obtained when dampers fabricated from bent steel bars were tested for large shear deflections at the DSIR in New Zealand in the 1970's.[11]

A log-plot of creep tests, based on about 1 year of results, also shows a similar characteristic to ordinary natural rubber (Fig.21), while temperature dependence is reported as favourable. Also while heat is generated within the bearings during deformation, it is suggested that this is not likely to be a problem for a few major cycles in an earthquake, a conclusion generally confirmed by earlier testing of damping systems at DSIR New Zealand in the 1970's when temperature rises were found to be quite small. The fatigue life of the rubber tested in shear for a single layer of rubber bonded between two metal plates is shown in Fig.21b. Even at 600% shear strain the life is shown to be of the order of 100 cycles, for which strain very large tensile forces and stress concentrations (Fig.11b) will exist across the rubber. No doubt normal natural rubbers tested in the same way would show a similar favourable performance.

Thus taking the results of one or two years of testing of HDMRB bearings, the prognosis for the HDMRB bearings appears to be good, but a few years of service with structures subjected to earthquakes are required to fully vindicate their performance. By comparison the lead-rubber bearings used in structures in New Zealand have been shown to be reliable over the years as natural rubber bearings of similar type (without the lead plug) have been in service in bridges in Europe since the 1950's.

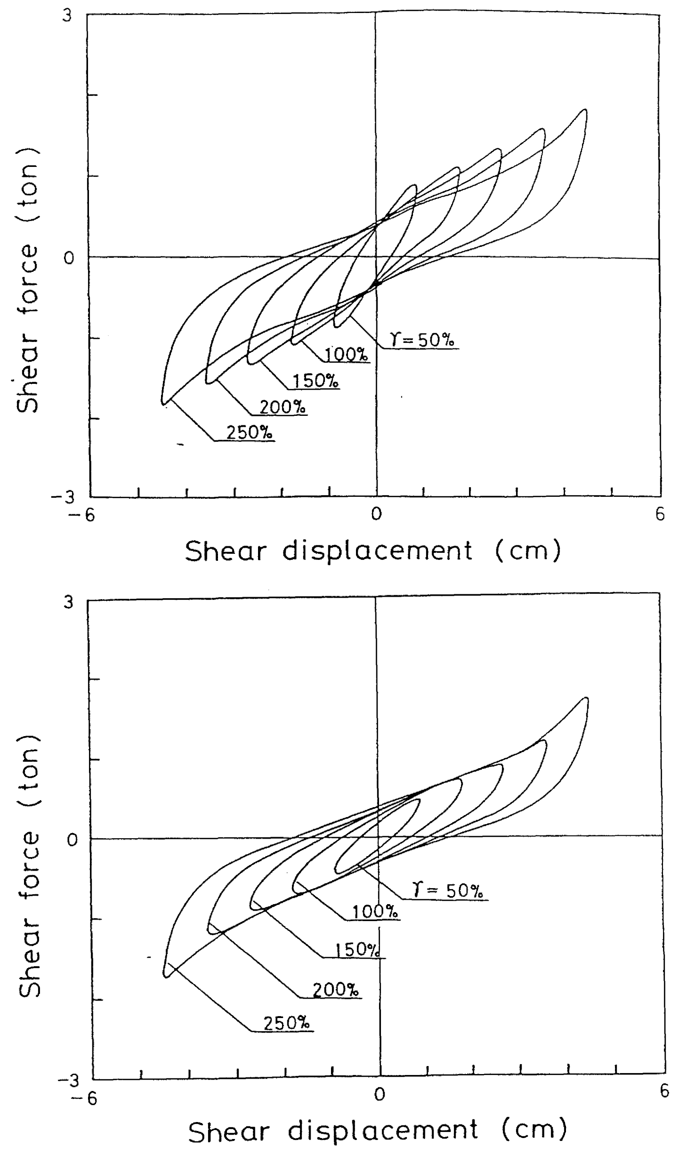


FIGURE 19 FORCE-DEFORMATION LOOPS FOR SELF-DAMPING HDMRB BEARINGS IN SHEAR FOR STRAINS UP TO 250%

(a) (*Above*) 1st cycles,
(b) (*below*) 2nd cycles [1]

So far HDMRB bearings have been installed in three buildings and two more were under construction in late 1990, and 5 to 6 more were planned for the following two years. One of these buildings will be a computer centre supported on 40 bearings, six stories high and weighing 20,000 tons, described as the heaviest base-isolated building in the world.[1]

One of the first buildings constructed was subjected to a small earthquake in 1987. Accelerations measured on the first floor of the building were about one quarter of those measured on the ground, which were up to 0.13g (Fig. 22). The bearings will therefore be useful in the Japanese computer industry in reducing the effects of small earthquakes, as some manufacturing processes in that industry are very sensitive to any vibration, and are costly to restart once interrupted. (See also para 4.1).

In early 1990 there were 20 existing base-isolated buildings in Japan with a further 25 planned or being constructed. Of the latter 25, 10 are planning to use Bridgestone HDMRB bearings, with the remainder being mainly on lead-rubber bearings.

5. HIGH STABILITY ASEISMIC BEARING (HSAB)

The high stability aseismic bearing (Fig.23a & b) was developed in New Zealand to maintain stability in bearings of comparatively small load capacity when subject to large horizontal displacements during an earthquake. The principle employed is to enlarge the plan size of the bearings by distributing the rubber in the form of circular discs around the perimeter of the steel laminates in order that the net thrust passes within the perimeter of opposing discs for significant horizontal displacements (cf. Fig 8). The production bearing has the appearance of a solid bearing (Fig.24) as a shroud of rubber is bonded over the outside, but is in fact hollow between the rubber laminates (Fig 23b). Prior to assembly the steel plates are fully vulcanised to avoid the possibility of corrosion in an unfavourable environment.

Damping is not at present provided within the bearing although experiments are being conducted with plastic materials contained within the bearing to provide the damping.

While this type of bearing was developed initially for a load capacity in the range 1 to 5 tonnes, the use of the principle in larger bearing sizes enables very large displacements to be accommodated in larger bearing sizes, the only limit on size being a manufacturing one. Presses are available in New Zealand to accommodate the manufacture of bearings up to 1m square, either for conventional laminated bearings or HSABs, which will enable stability to be assured up to approximately 0.5m of displacement in both types of bearing.

The first HSAB (Fig 23b) was designed for 1 tonne capacity and employed 4No/50mm diameter rubber discs 3mm thick in 19 layers, with 20 fully vulcanised steel separating plates each 3mm thick, which, allowing for a thickness of vulcanisation in excess of 1mm all around the plates, gave an overall depth of 193mm. This bearing was tested cyclically, using earthquake records, for a maximum displacement of ±15cm to a maximum shear strain in the rubber of 200% and many hundreds of cycles were completed without failure. Later four similar 5 tonne bearings designed for the isolation of filter banks at Haywards Substation were tested on a shake table at Works Consultancy Services Central Laboratories while supporting a 4 tonne concrete block, again without failure.

HSABs are manufactured with fully-bonded discs and mounted via vulcanised steel mounting plates top and bottom. It is likely therefore that, under extreme conditions the discs on a line across the longest diagonal will develop tensile forces. For short-term loading this is not

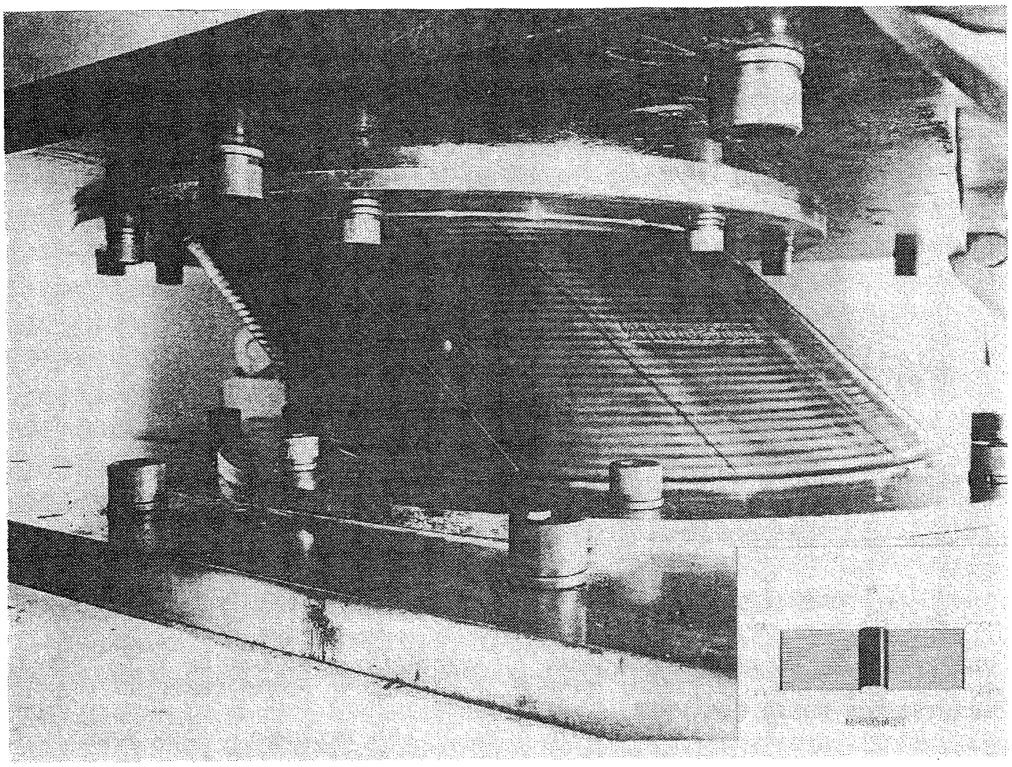


FIGURE 20 FULLY BONDED BRIDGESTONE BEARING UNDER TEST.

The extended diagonal length is clearly indicative of tension across the bearing.
(Courtesy Bridgestone Rubber Co., Japan)

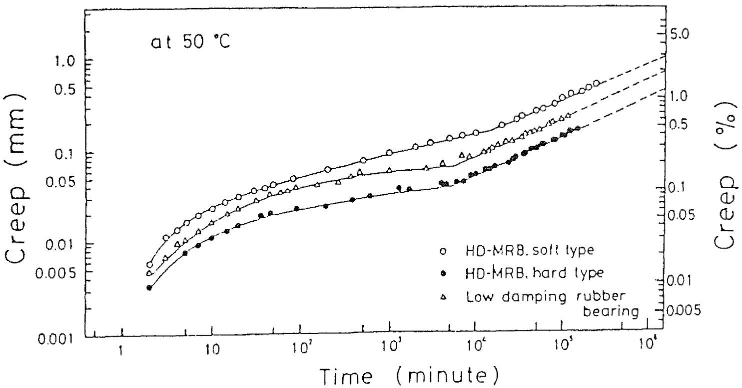


FIGURE 21a CREEP-TIME RESULTS FOR BRIDGESTONE HDMRB BEARING [1]

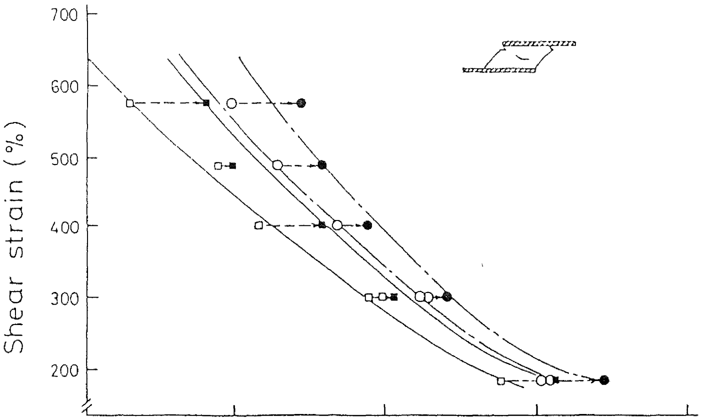


FIGURE 21b FATIGUE LIFE OF BRIDGESTONE HIGH DAMPING RUBBER IN SHEAR

Progressive failure (inset) is indicated from a crack at the corner to a split at the centre. Circles are for a cured condition, squares for a projected 90 years degradation [1].

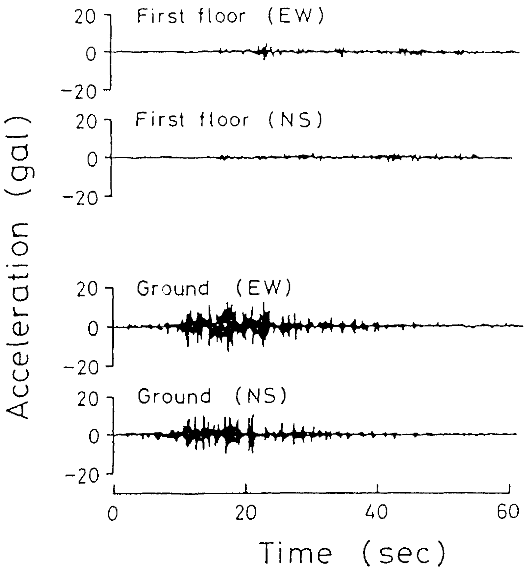


FIGURE 22 THE EFFECT OF A SMALL EARTHQUAKE ON A BUILDING ISOLATED WITH BRIDGESTONE HDMRB BEARINGS.

[due to 1]

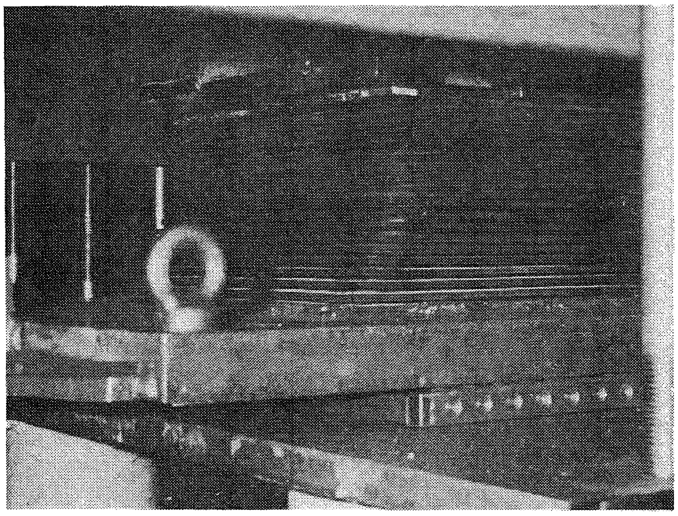


FIGURE 23a PROTOTYPE HSAB BEARING DURING SHEAR TESTING

(Photograph courtesy Skellerup Industrial Ltd.)

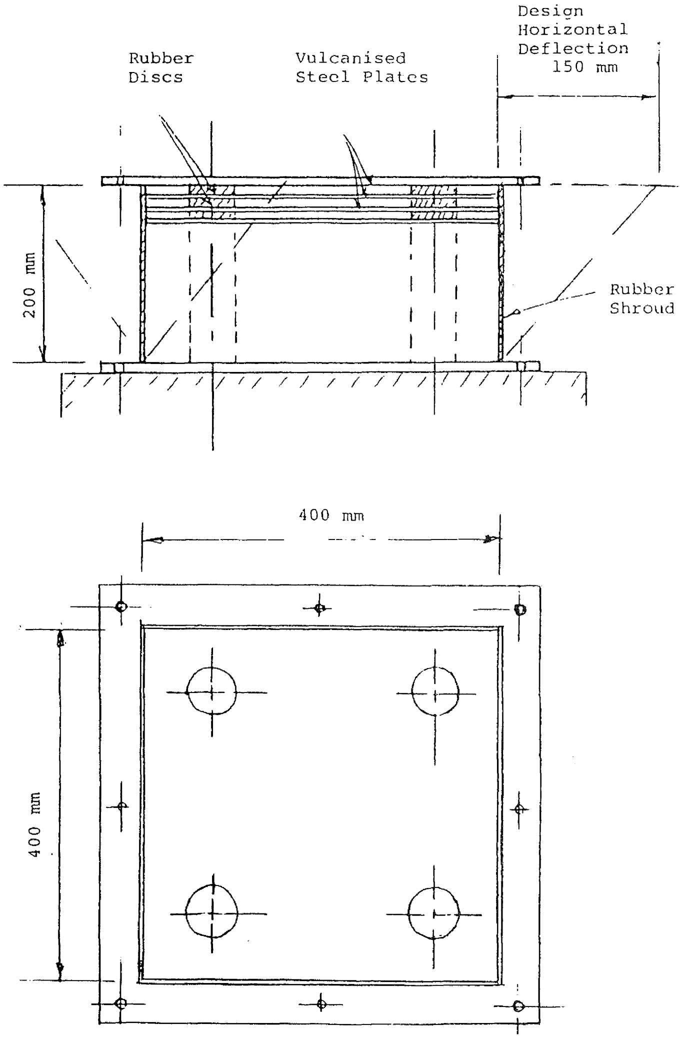


FIGURE 23b DETAILS OF PROTOTYPE HSAB BEARING

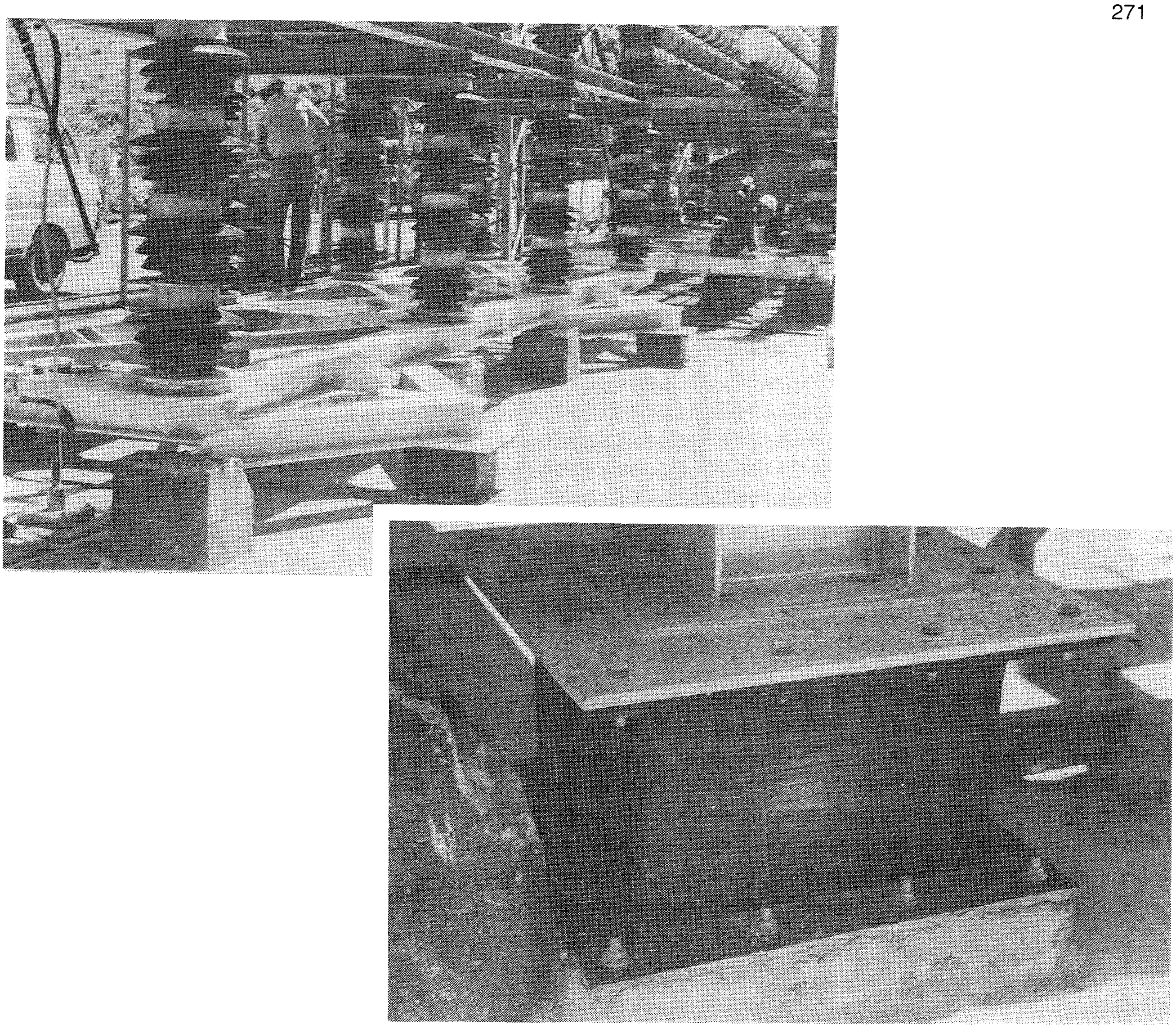


FIGURE 24 HSAB BEARINGS ISOLATING ELECTRICAL EQUIPMENT AT HAYWARDS SUB-STATION, NEAR WELLINGTON.

(a)(*top*) Rubber bearings in position, replacing the concrete mounting blocks which were later removed. A steel taper-cantilever damper, one of a pair for each filter bank, is shown at the bottom left, bolted to the floor. A sliding ball joint was used in the frame.
(b)(*bottom*) Rubber bearing.
(Photographs courtesy - Skellerup Industrial Ltd.)

considered to be detrimental, as noted below (Para 6) and the tests showed up no untoward effects.

The onset of shear deflections causes bending of the steel separating plates, which can be observed in cyclic shear testing, and for which an approximate calculation can be made. A research problem arises as to how thin the plates can be manufactured, since bending deflections are not necessarily detrimental provided stability is assured in the bearings. At present a minimum thickness of 3mm is used for the plates in New Zealand for manufacturing reasons. With thinner plates there is a reduction in weight and cost and a small improvement in stability (cf. Fig.8)

6. TENSILE FORCES IN RUBBER BEARINGS

Specifications such as the U.K. BE1/76 [16] recommend that rubber bearings are not subjected to tensile forces in the day-to-day temperature cyclic loading conditions prevailing on bridge decks. This gave rise to the use of dowels to transmit shearing forces to the bearings in order that the bearing can take up a deformed shape (Figs 10 & 11) under shear displacement. Thus the development of tensile forces at the

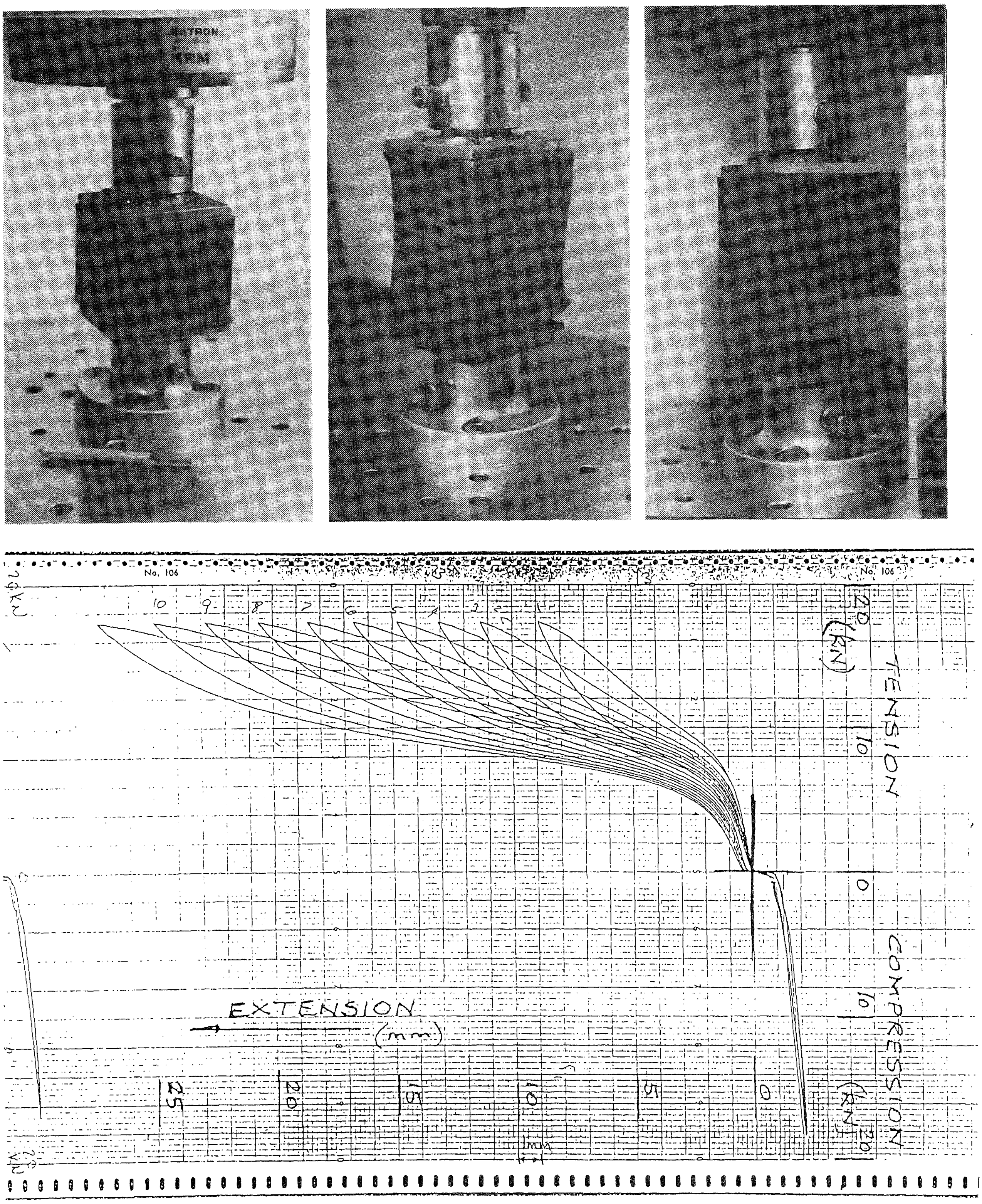


FIGURE 25 FAILURE OF A LAMINATED RUBBER BLOCK IN TENSION

(a) (*top*) Before, during and after failure
(b) (*bottom*) Record illustrating progressive tearing of rubber during cycling.
(Illustrations courtesy Skellerup Industrial Ltd.)

perimeter of the bearing is avoided as the rubber is allowed to move away from the mounting plate at its perimeter. Tests at DSIR in New Zealand in the early 1980s showed that dowel pins would pop out of their sockets if near the edges of the bearings (Fig.10 see also Para 3.2) so accordingly a recommendation was made that dowel pins should always be located within the middle third of the bearings in order to maintain the drive on the bearings.

On the other hand the limited research work which has been carried out so far on bearings in tension in New Zealand suggests that laminated rubber bearings will carry tensile forces, at least for transient loadings. This is therefore a worthwhile subject for research. As an example, the laminated rubber block shown in Fig 25(a), which was 100 x 100 x 90 mm in size, with 18 internal rubber layers, 96 mm square by 3mm thick, giving it a shape factors of 8 and a bearing capacity of 100 kN when calculated to the H_A loading requirement of BE1/76 [16] eventually failed in tension at a cyclic loading of ± 20 kN Fig 25(b). Ten cycles were applied at each of the stages ± 5, 10, 15 and 20 kN, each cycle taking about 1 minute. As the shape factor of 8 was identical to that of the full-scale Devil's Creek bridge bearing (Figs.1 and 10), which had 7 mm rubber layers, but a larger plan area, a similar performance would be expected on the full-scale. Again one of the 5 tonne bearings tested at the laboratories of the Works Consultancy Services Limited for the Haywards Substation project was "accidently" loaded in tension and failed when the tensile load was 4 tonne, by tearing of the rubber, i.e. it did not debond.

While conservative statements from the U.S.A. and the U.K. [Derham 15] suggest that tension should be avoided, they have not been based on published results of dynamic testing where only a few cycles are applied corresponding to the total number which may reasonably be expected during the life of a building. It is suggested that fully-bonded bearings are to be preferred for base-isolated buildings where there are no day-to-day temperature movements. Disaster conditions may cause some tearing of the rubber but shear forces will still be transmitted by portions of rubber remaining intact and there is no possibility of losing the drive by the disengagement of pins when foundations are deformed vertically by transient earthquake motions, or lurching of the structure occurs, or by local uplift of part of the structure.

This view has more recently been proferred by Japanese engineers [12] who have observed lurching in model tests, which is regarded as an unstable state, as the building may tilt a few degrees, with a tendency to lift off the edge bearings. In consequence most Japanese bearings are fully bonded to metal plates top and bottom, which are bolted in place. In addition the rubber is curved into the plates to avoid a stress concentration on the bonding (Fig 20).

7. OTHER USES OF RUBBER COMPONENTS IN MOUNTING SYSTEMS

7.1 Rubber Components as springs

Rubber components may also be used as springs in base-isolated designs for which application they are not loaded by the building weight. For these elements, the steel laminates would be omitted and solid blocks of rubber utilised, usually designed to shear in a horizontal direction. Spring elements may be used to ensure return to a central position for (a) in pivoting pile type of design of (b) when 'slippery' bearings employing polytetraflouroethylene, "Teflon", sliding elements, are utilised to support the building weight.

7.2 Rubber elements as buffers

Rubber elements may also be used as buffers to reduce the shock of two structural elements coming together. One application is the use of solid circular rings between bridge beams and an abutment. Severe distortion of the buffer provides increasing resistance as the two parts close together, while some damping is usually obtained by the use of particular cross sections suffering severe deformation.

Experience in the 1989 Loma Prieta earthquake in San Francisco indicated that items of plant utilising mounting sprigs and snubbers (small rubber buffers) were not wrenched from their mountings, whereas damage occurred in solidly mounted equipment. The accelerations experienced were not large, but there is a need to collate information on the effectiveness of this form of mounting in larger earthquakes.

8. CONCLUSIONS

While New Zealand led the world in the promulgation of base-isolation techniques in the 1970s and early 1980s this lead has now fallen away with greatly increased investigations in overseas countries, particularly Japan and the U.S.A. Reductions and constraints on fundamental research in DSIR and the Universities and the apparent inability of industry to fund research have further reduced the relative contribution of New Zealand. The following are notes on research projects which could usefully be carried out.

1. Research is required to determine the limiting parameters for allowable tension on the premise that fully-bonded, and bolted-in-place bearings will provide a tough resilient foundation. Tests so far completed in New Zealand suggest that rubber bearings will carry transient tensile stresses. Further testing should be carried out on fully-bonded bearings at high shear strains in order to establish limiting criteria. Japanese practice appears to confirm that fully-bonded bearings can be used.

2. The relationship between vertical bearing stiffness and lurching during horizontal motion should be investigated

and the effect of the introduction of damping in the vertical direction.

3. Thickness criteria for reinforcing plates should be considered. Primarily the plates carry tensile stresses which prevent the rubber from bulging at the edges, but they will also be subjected to bending the magnitude of which will depend on whether the bearings are spigotted or fully bonded. There is a special problem for high stability (HSAB) bearings, as the plates carry bending moments between the individual discs. A typical plate thickness is 3mm, and the process of bearing manufacture including bonding the rubber to the plates may dictate how much thinner the plates can be made.

4. Internal damping methods need perfecting for high stability (HSAB) bearings. A plastic infill between the discs has been suggested or small lead plugs down the centre line of the discs. Self-damping rubber could also be employed.

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