BASE ISOLATION - AN HISTORICAL DEVELOPMENT, AND THE INFLUENCE OF HIGHER MODE RESPONSES

D. M. Lee* and I. C. Medland**

ABSTRACT

In this paper the evolution of a technique for protecting a structure from earthquake attack is traced from its beginning through to its currently most effective form, and this form, the Base Isolation System, is compared to other currently available techniques.

The influence of higher mode effects in base isolated multi-storey structures is investigated and shown to be of considerable significance in determining the shear forces in the upper levels of a structure. Because of these higher mode effects the responses of appendages on isolated structures, while still being less than those for appendages on unisolated structures, can be significantly larger than previous 1-D analyses had suggested.

A standard set of distributions of inter-storey shear up a multi-storey structure is presented with each distribution being defined by a parameter which varies from zero to unity.

PART I HISTORICAL DEVELOPMENT AND REVIEW

1.1 Introduction

In his historical review paper Eiby[1] pointed out that prior to the 1920's when isolated papers by New Zealanders began to appear in the Bulletin of the Seismological Society of America, there was little New Zealand interest in the problems of seismic engineering. However the Murchison earthquake of June 16, 1929, which involved the loss of 17 lives, radically altered the then current view that earthquakes were of scientific interest only.

In the subsequent early attempts at earthquake resistant design, structures were designed for strength and rigidity with some success. Because such structures were also typically rather squat, they had very low periods of vibration and consequently were not severely excited by strong earthquake motions. Thus not only were the structural components of these structures left largely umdamaged after an earthquake, but because of the structure's stiffness there was also little internal movement and consequently little secondary damage.

In more recent times with the advent of taller, and for economic reasons, more flexible buildings, structural periods tended to lie in the region of dominant earthquake energy. This necessitated considering the structure as a dynamic system whose internal damping was of paramount importance in limiting its response to earthquake forcing. This internal damping is small for most structures[2], and for large earthquakes is insufficient to prevent accelerations from building up in the structure that stress it beyond its elastic limit. At present the earthquake design codes in use throughout the world[3,4] generally specify a minimum allowable elastic strength for a structure by means of static lateral loads that vary with the height or period of the structure. This elastic strength is typically much lower than the maximum force that would arise in a purely elastic structure during a moderate earthquake. When a structure so designed is attacked by an earthquake, it behaves elastically at first but because of its low inherent damping capacity the vibrations build up until inelastic (or plastic) deformation occurs. Current design practices allow the structure to develop plastic hinges at beam ends in order to give increased flexibility and energy absorbing capacity. However such inelastic deformation of the structure not only causes breakdown of the structural components but causes severe and expensive secondary damage as well. In many cases the force-limiting inelastic action of the structural components cannot occur until after great secondary damage (e.g. fracture of partitions, windows, facings, etc.) has occurred[5]. Moreover the ability of the structure to form the number and style of plastic hinges required is by no means guaranteed.

Thus, most current design rules depend on an earthquake resisting mechanism that is both destructive and of uncertain performance. While structures designed to these rules generally fulfil the designer's aim that there be 'no loss of life' during the earthquake, they achieve this by the deterioration of structural components.

Since flexibility and energy absorbing capacity are desirable attributes but the large deformations that these entail are not, it would seem logical to attempt to restrict

* Graduate Student, Department of Theoretical and Applied Mechanics, University of Auckland. (Now Post Doctoral Fellow in Earthquake Engineering, California Institute of Technology.)

** Senior Lecturer in Theoretical and Applied Mechanics, University of Auckland.

the flexibility and energy absorbing capacity of a structure to a single region and to allow the rest of the structure to maintain its integrity during an earthquake. Damage could then be localised and hopefully more easily repaired. The idea of designing a structure with a laterally flexible first storey, first proposed almost fifty years ago, is discussed more fully in Section 1.2.

1.2 The Soft First Storey Concept

The Soft First Storey Concept consists of designing a structure with laterally very flexible first storey columns while the stiffnesses of the remaining columns of the structure are kept at more usual design levels. When an earthquake attacks such a structure it is intended that severe deformations (elastic or inelastic) be restricted to the flexible first storey, leaving the remainder of the structure isolated from the high levels of lateral loading that would otherwise occur.

In 1929, the same year as the disastrous Murchison earthquake, Martel[6] published a paper entitled 'The Effect of Earthquakes on Buildings with a Flexible First Storey". Martel considered the response of an undamped single storey bent subjected to sinusoidal forcing. He found that with bents having natural periods of more than 2.3 times that of the impressed ground accelerations, the mass mounted on the bent underwent accelerations of amplitude less than 25% of that of the ground motions. However, when the period of the ground motion was greater than that of the bent, then as the bent flexibility increased the accelerations transmitted to its top increased. This was the first paper to deal directly with the idea that a structure be specially designed to resist earthquakes by shifting its fundamental period out of the region of dominant earthquake energy, so lowering the forces arising in the structure.

In 1935 Green[7] considered the effect of piecewise linear acceleration inputs on an undamped oscillator, by means of direct integration of the equations of motion over each time interval for which the ground acceleration was linearly defined. Green re-emphasised the importance of the need to separate the structural fundamental period from the dominant periods of the acceleration motions, if the accelerations arising in the structure were to be limited. The period of the acceleration input was shown to be as important a parameter as the acceleration amplitude itself. Furthermore, an irregular acceleration input gave greater accelerations and displacements in the structure than did a regular acceleration input. In the appended discussion on Green's paper, Johnson pointed out that the maximum deflection produced in a structure by a given ground motion would be relatively independent of the structure period only if the range of structure periods was sufficiently greater than the period of the ground motion, so that resonance effects were not significant. Other contributors to the same discussion (White, Kartzke and Smits) pointed out that large increases could occur in Green's calculated displacements when the structures considered were subjected to different types of acceleration forcing. Williams[7] in an experimental investigation of Green's work concluded that for the assumption of a rigid superstructure to hold true, the upper column stiffnesses had to be forty times that of the 'flexible first storey' and that the usefulness or not of the flexible first storey concept in practice would depend on the practical difficulties encountered.

And then, due mainly to the practical difficulties involved in implementing the soft first storey system, the idea was passed over for thirty years.

In 1965 at the 3rd World Conference in Earthquake Engineering held in Auckland and Wellington, Katsuta and Mashizu[8] proposed an earthquake isolation mechanism which consisted of mounting the structure on rollers and then anchoring it to its foundations with a set of high speed electro-hydraulic rams controlled by a seismograph. As an earthquake moved the structure foundation, the rams were to be activated to move the structure (relative to its foundation), so that its absolute movement was minimised. It was shown that neutral compensation for drift was required when the seismograph was mounted on the structure itself and that transmissabilities (ratio of the acceleration transmitted into the structure to the acceleration of the ground) of the order of 0.04 could be achieved for ground forcing periods of around one second; and of the order of 0.3 for ground forcing periods of around 0.1 seconds. The authors concluded that the system could be designed to be very effective if some knowledge of the frequency characteristics of likely attacking earthquakes was available. Although feasible technologically, the method was at that time considered impractical for economic reasons. At the same conference Matsushita and Izumi[9,10] expressed their concern at the sudden change in mass and rigidity distributions of tower structures where they join with their basements, which are often of large plan dimensions and fully or partially embedded in the ground. They showed that a lowered and more uniform distribution of interstorey shear could be obtained by utilising their 'Double Basement Method', whereby the tower part of the structure continued down through the basement but was structurally separate from it. This had the effect of adding two storeys or so of height to the tower, which both increased its fundamental period (and hence lowered the seismic forces applied to it, for most types of earthquake), and allowed a more uniform stiffness and mass distribution for the tower from foundation level (which also lowered the seismic shear response). When the stiffness of the newly created lower storeys of the tower was reduced to a value much less than that of the upper storeys of the tower (i.e. a flexible lower storey situation), the shears induced in the tower by seismic forces were reduced according to the degrees of flexibility in the lower storeys. To guard against possible instability the authors proposed filling the gap between the tower and basement with a soft flexible material so that under heavy earthquake loading the tower basement would be stabilised by the surrounding, stiff outer basement. The authors concluded their paper with the caution that the flexible lower level was not effective when long period components of the ground motion were predominant, and hence care had to be taken to try and determine in advance the type of ground motion likely to occur.

In 1969 Fintel and Khan[11] criticised the practice of designing structures to develop plastic hinges throughout their structural components as an earthquake energy absorbing mechanism. They proposed a soft storey system whereby the structure was supported at its base on flexible elastic pads, and then anchored to its foundation with columns specially designed to yield at their extremities during a moderate earthquake. In a moderate earthquake, therefore, the system would hopefully behave as a rigid body on a bilinear hysteretic foundation, and have all damage confined to these specially designed foundations. The authors examined a single mass mounted on a bilinear hysteretic support subjected to the El Centro (N.S. 1940) ground accelerations and found that when the period of the mass mounted on the unyielded system, T, was less than 0.8 seconds there was a rapid rise in acceleration response as T decreased and so the method discussed was applicable only to structures where the soft first storey was made sufficiently soft. For T = 0.8 seconds and with a mass having 10% of critical damping being mounted on a bilinear hysteretic support of yield level of 0.06 g. and having a ratio of stiffness in the yielded regime to stiffness in the unyielded regime (the bilinear slope ratio) of 0.05, the maximum accelerations of the mass were less than 30% of the input ground accelerations. The authors found that the single most important parameter of the soft storey system was the yield force of the bilinear supports, an increase in this quantity giving an almost linear increase in transmitted accelerations (for bilinear slope ratios < .1). It was also pointed out that for a perfectly elasto-plastic system, the maximum force that could be transmitted into the structure was the yield force of the support. (For the single mass model (i.e. rigid structure) considered, this does imply that the maximum force experienced by the mass is given by the yield force of the support, but if a flexible structure were to be mounted on the soft storey system then this would not necessarily be the case as internal resonant effects could occur.)

In 1970 Caspe[12] proposed an isolation system in which the structure was mounted on sets of roller bearing assemblies but fixed to its foundation by neoprene rubber pads and sets of horizontally mounted 'control rods' which prevented movement at the structure base during wind loading. These control rods, fabricated from mild steel, yield at the onset of strong earthquake motion, preventing the transmission of large destructive forces into the superstructure. A one degree of freedom (1 d.o.f.) oscillator mounted on this isolation system (giving an effective 2 d.o.f. system) was excited by the El Centro (N.S., 1940) accelerations and the influence of different isolation system parameters on the structure response investigated.

Chopra et.al.[13] considered the dynamic response of a series of yielding, eight storey, shear buildings subjected to simulated earthquake excitations. Their aim was to determine under what conditions a yielding first storey could adequately protect the upper storeys from significant yielding. Two classes of building were considered: stiff (0.5 seconds) and flexible (2.0 seconds), with the basic parameters considered in the yielding first storey being its yield force and bilinear stiffnesses. Their results indicated that a very low yield force level and an essentially perfectly plastic yielding mechanism were required in the first storey to prevent yielding in the superstructure. For the flexible structures, the displacements developed across the yielding first storey were very large. One factor noted by Chopra that had been overlooked by previous workers in the soft storey field, was that the shear wave transmitted into the superstructure by the ductile first storey (which may be limited by the yield force level during its initial propagation) would be reflected at the top of the building, and this reflection would tend to cause a doubling of the nett amplitude in a uniform building. Also noted by Chopra was the fact that in both the stiff and flexible buildings, the ductility factors (ratio of total displacement to the displacement at yield) in storeys immediately above the first were essentially independent of the yield level of the soft storey, if the bilinear slope ratio of the soft storey was 0.1. Thus the maximum shear force transmitted into the second storey did not decrease in spite of large reductions in the first storey yield strength. This implied that to ensure elastic behaviour in the second storey the forces transmitted from the foundation must be limited. It was evident that the large first storey drifts operating on the yielded stiffness of the soft storey still developed large base shears. To counteract this a bilinear slope ratio of 0.01 was used in the soft storey with the result that the ductility factors were reduced in all storeys but especially in those just above the first. The ductility factor for the second storey was then strongly affected by a decrease in yield strength. In conclusion the general observation was made that the ductility requirements in the lower storeys depended to a large extent on the total force (elastic and plastic) developed at the structure base, whereas the ductility requirements in the upper storeys (which result from whiplash or wave reflection effects) depended primarily on the elastic component of the base force. Thus the bilinear slope ratio was important in the lower storeys, while the yield force controlled the behaviour in the upper storeys. The artificially generated earthquakes used in the analysis all had the same spectral basis for generation and so no information on the effect of different categories (shock type, regular, periodic, etc.) of earthquakes on the system described could be obtained from the results. Also no direct comparison of this system could be made with those studied by the earlier researchers[6,7,9,10,11] because of the different types of soft storey systems investigated and the widely differing ranges of parameters examined.

Wirshing et.al.[14] attempted to illustrate the relative effectiveness of different techniques for shielding structures from earthquake attack. Five different systems were examined for reducing the vibrations transmitted into a structure from the ground. These were:

(i) A viscous damper between the ground and first floor.
(ii) A vibration absorber (an auxiliary oscillator designed to move out of phase with the structure and so limit its motion) on the roof of the building.
(iii) (i) and (ii) together.
(iv) An isolator system at the base whereby the structure is separated from its foundation by a horizontally acting spring support and viscous damper. The spring was elastic plastic with hardening.
(v) (ii) and (iv) together.

Five and ten storey structures of fundamental period 0.5 and 1.0 seconds respectively were excited by a range of artificial earthquakes with set spectral density characteristics and having the same average intensity as the El Centro (N.S. 1940) earthquake. The effectiveness of the five systems in reducing the vibrations transmitted into the structure were, in order of decreasing effectiveness, (iv), (v), (iii), (ii), (i). It was concluded that only the isolator system could give sufficient protection to prevent a structure from deforming into the plastic range during an El Centro type earthquake. Addition of the absorber to the roof with an isolator system already employed could have a detrimental effect because of the system having a pseudo period, when yielded, that was greater than that of the absorber itself. Only one set of mechanism parameters was used to investigate each of the protection mechanisms but it was considered that the order of preference would not be altered even with extensive parameter variation.

Williams[15] in a 'state of the art' paper summarised the problems posed by earthquakes and their effects on structures. He considered the isolation system proposed by Fintel and Khan[11] to be of uncertain stability under heavy earthquake loading and proposed instead an isolation method whereby the structure was mounted on slip pads so that the force transmitted to the structure was absolutely controlled. He also suggested the use of restoring springs to limit the slip of such a system. Husid and Sanchez[16] investigated the effect of viscous damping, an elastoplastic support mechanism and a bilinear hysteretic support mechanism in reducing inter-storey displacements. Their rather few results were similar to those obtained by Wirsching[14]. A rather bulky bilinear hysteretic damping mechanism was proposed. Crandall et.al.[17] considered the system as proposed by Williams[15]. The authors examined the Root Mean Square (R.M.S.) residual relative displacement after an earthquake, of a mass mounted on a randomly moving foundation, where the mass was linked to the foundation only by Coulomb friction. The R.M.S. residual relative displacement was predicted approximately both analytically and by computer simulation using an ensemble of artificial earthquakes of set spectral characteristics. A linear log-log relationship was found between μ, the coefficient of friction of the mass on the foundation, and the R.M.S. offset. The authors pointed out that if a structure's internal resonant frequencies were all above the cutoff frequency of the earthquake motion, then the horizontal acceleration of the structure would be limited to μg, where g = the acceleration due to gravity. This work forms a useful test case for the residual displacements likely to be encountered in an extreme form of bilinear hysteretic support.

Derham et.al.[18] examined four different materials with regard to their use as springs in a 'building mounted on springs'. Natural rubber bearing springs gave the 'best' overall performance when compared with steel, polychloroprene and butyl rubber springs in tests on their transmissability, internal damping capacity, creep resistance, resistance to ozone attack and constancy of modulus with temperature. A five storey test building mounted on laminated steel and rubber bearings (for greater vertical stiffness) and subjected to earthquake motions, had a maximum floor acceleration of 0.1 g. as compared to the value of 1.0 g. for the same building when unisolated. Also the isolated building did not tend to amplify accelerations in the upper storeys as did the unisolated building. A later paper by Waller[19] underlined the results of Derham's work and the desirability of shifting the natural period of a building above the region of dominant energy of most earthquakes. Delfosse[20] performed experimental tests and analyses on models of a twenty storey structure forced by the N21^{o}E component of the Taft, California earthquake of 1952, both with and without a so-called GAPEC SYSTEM (G.S.). This system, which consists of mounting the structure at base level on horizontally flexible, vertically rigid, laminated rubber pads having nonlinear but elastic stiffness characteristics, reduced the maximum shears and overturning moments in the structure by a factor of from five to eight. The natural periods of the two models considered were increased from 0.86 seconds and 1.15 seconds without the G.S. to 3.1 seconds and 5.0 seconds with the G.S. Delfosse proposed the use of wind stabilisers inserted at the same level as the isolator pads to fix the building against ordinary wind loads. These stabilisers were automatically disconnected from the structure when the base shear exceeds a maximum design load and are reconnected to the structure after an earthquake. The form of these 'stabilisers' was not revealed nor was it stated how they were 'automatically disconnected' or for that matter, reconnected.

Eidinger and Kelly[21] experimentally investigated the forced response of a scale model building mounted on laminated natural rubber bearings and excited by scaled El Centro (1940), Taft (1950), Parkfield (1966) and Pacioma Dam (1971) earthquake acceleration records. Rubber bearings of varying damping capacities were investigated and the influence of hydraulic dampers connected in series with these bearings also studied. It was found that the use of the rubber bearings significantly reduced the building's acceleration response in the horizontal direction while leaving its vertical response virtually unaltered.

1.3 An Appraisal of Alternative Protective Mechanisms

Most of the soft storey systems as discussed in Section 1.2 have had major practical drawbacks. The electrohydraulic servo-mechanism technique as described by Katsuta et.al.[8] was, by their own admission, prohibitively expensive. Also any slight failure or misadventure in the controlling seismographs could be disastrous. The shock absorbing soft storey concept of Fintel et.al.[11], while giving worthwhile reductions in earthquake generated forces, could suffer from stability problems particularly with regard to overturning under heavy excitation. Also the repair of the energy absorbing columns of such a system after an earthquake could prove rather expensive. This is also a drawback to the system proposed by Chopra et.al.[13]. The double basement method of Matsushita and Izumi[9,10], while giving some reduction in the seismic forces arising on a building, involves the use of special construction over two to three floors of the structure. The isolation system proposed by Caspe[12] requires regular maintenance of the ball bearing supports but these have the advantage that they could be very easily replaced by local jacking of the structure and removal of the exercise plate in the centre of each bearing assembly. The practical applications of Crandall's[17] slip pad system are likely to be somewhat limited due to the possibility of large displacements with earthquakes having a predominant drift. The frictional properties of these pads could also change with time and mechanical loading, adversely affecting the behaviour of the isolation system.

Now Wirsching and Yao[14] and Husid and Sanchez[16] showed that an isolation mechanism at the structure base gave the best protection from earthquake motions while Waller[19] demonstrated that laminated natural rubber mounts were the most suitable for use in supporting a structure for earthquake isolation. Accordingly in 1975, in an attempt to combine the best features of different earthquake protection mechanisms, Skinner et.al.[22] proposed a technique called 'Base Isolation' where the structure was supported completely on flexible laminated rubber pads at the foundation-structure base interface but anchored to its foundation by a new type of hysteretic damper[23,24] developed by the Physics and Engineering Laboratory of the D.S.I.R.

1.4 The Base Isolation System : State of the Art

The aim of Base Isolation is to provide a structure with an isolation system that restricts all plastic deformation to cheap, and especially, easily replaceable devices, allowing the structure proper to remain in the elastic region during even a strong earthquake. These devices have accurately defined stiffness and damping characteristics and give a highly reliable means of providing earthquake protection for a structure. The Base Isolation System has the advantage of not requiring any complex or delicate control mechanisms but works directly from the material properties of its components. Skinner et.al.[22] derived an approximate analytical technique for evaluating the shear response of a one-dimensional oscillator mounted on such a system and showed that exceedingly flat and very much reduced shear responses were obtained over a wide range of oscillator periods, for oscillators subjected to El Centro (N.S. May 1940) type accelerations. This independence of the response, on the period of the original structure (i.e. the period prior to Base Isolation), is one of the very desirable features of the system, as it considerably reduces the danger of a structure being excited by earthquake motions having dominant periods coincidental with that of the structure's fundamental period. With the Base Isolation (B.I.) system as studied by Skinner[22] the flexibility of the laminated rubber support pads was arranged so that the horizontal period of the oscillator mass mounted directly on the pads was 2.0 seconds (vertical period, 0.1 seconds), which is outside the region of dominant energy for most earthquakes. The hysteretic dampers prevent any pseudo-resonant effects from occurring and limit movement across the rubber pads to ± 9 cm. which is within the safe displacement range for the type of rubber pads proposed for use in B.I. systems.[25] It was not expected to be at all difficult to design plumbing, electrical and other service connections to accommodate such displacements. Using B.I. it was hoped that, because of the reduced demand for ductility in the structure, a wider choice of architectural forms and structural materials would be available to the designer, which would help to offset the cost of the B.I. system, estimated at 1% of the total structural cost for a typical ten storey structure.

In a later paper Skinner and McVerry[26] performed an actual dynamic analysis of a set of one storey, Base Isolated structures, verifying previous results,[22] as well as showing that if a Base Isolated Structure yielded during an earthquake, then the ductility demands were very much less than if the structure had been unisolated.

Nuclear power plants are a special class of structure where a very high resistance to earthquake attack is desired. Not only must the structure proper resist earthquakes but the individual components of the plant, many of which are inaccessible to standard damping methods (e.g. fuel rods, coolant pipes), must not suffer deterioration during an earthquake and prevent a safe shutdown. Skinner and McVerry[27] and Skinner, Tyler and Hodder[28] examined the effects of a B.I. system in reducing the seismic forces arising on a main mass (the nuclear plant) with an appendage (a fuel rod, pipe, etc.), subjected to scaled El Centro (N.Z. 1940) accelerations. With rubber pad flexibilities selected to give the main mass a period of 4.0 seconds when mounted directly on the pads, and a damper resistance of 15% of the weight of the main mass, it was found that the earthquake generated horizontal forces and deformations of the main mass could be reduced by a factor of about ten, and those in a resonant appendage by a factor of about forty. However, the forces and deformations arising in an appendage of period equal to that of the main mass when isolated, was not investigated. It is possible that such appendages might have their responses increased slightly with isolation although to a level greatly below that of a resonant appendage on a non-isolated main mass. The effect of variations in the yield level of the dampers was investigated for a range of El Centro amplitude and period scaling factors.

Priestley, Crosbie and Carr[29] performed dynamic analyses on four, eight and twelve storey cantilever shear structures supported on a B.I. system and found that the behaviour of such structures to the El Centro earthquake was more complex, due to the significance of higher mode effects, than had been previously believed on the basis of simple single degree of freedom models (22,26). The influence of some different types of B.I. systems, on the dynamic response to the El Centro motions, was also discussed and tentative design rules proposed for use with Base Isolated structures. Lee and Medland[30] considered the effect of different types of earthquakes on a range of Base Isolated multi-storey shear structures and proposed a set of design curves for estimating the maximum shear and the distribution of this maximum shear up the structure, when the structure was excited by an earthquake of known Housner Spectral Intensity (H.S.I.). The results showed that higher mode effects were in fact significant (as indicated by Priestley et al[29]) and that the H.S.I. gave a very good description of the ability of an earthquake to excite shears in a Base Isolated structure.

Kelly et.al.[31] performed an analytical and experimental investigation of a 20 ton, three storey, single-bay, moment-resistant, steel frame structure both with and without a B.I. system. For large earthquakes it was found that the structure's first mode period increased from 0.6 to 1.0 seconds and that the equivalent first mode damping was between 30% and 35% of critical. It was concluded from test results and from a consideration of the physical properties of the mild steel hysteretic dampers that a B.I. system having a post yield stiffness near to 5% of its elastic stiffness and a yield force of from 5% to 10% of the structure's weight should be optimal.

PART 2 ANALYSIS OF BASE ISOLATED MULTI-STOREY STRUCTURES

The responses of isolated and unisolated structures to the El Centro (N.S. May 1940) accelerations are evaluated and the relative dominance of higher mode effects in both cases compared. The responses of appendages on these isolated and unisolated structures are also evaluated and compared.

2.1 Structure Models and Parameters Used

The structure models considered in this work are N storey lumped parameter shear structures with rigid floor slabs of mass M mounted on vertically rigid columns of total interstorey stiffness K as shown in Figure 1. It should be noted that for an unisolated structure the mass concentrated at the ground floor, M_b, is of no significance as it is assumed to be rigidly attached to, and move with, the ground. However for a base isolated structure M_b is attached to the upper surface of the isolation system and does take part in the dynamic response of the structure. It has been shown[32] that γ, the ratio of M_b to the total structure mass above the isolators, N.M, generally had little effect on the responses of isolated structures. (It was further shown that reductions in the interstorey stiffness of the upper floors had little effect on the responses of isolated and unisolated structures.) In this work γ is taken as 1/6.

The Base Isolation System illustrated in Figure 1(a) can be mathematically defined as a single bilinear hysteretic support with the horizontal force displacement characteristics shown in Figure 1(b). The stiffness effects of the laminated rubber pads and hysteretic dampers are agglomerated into one unit such that,

k_1 = the elastic stiffness of the isolation system before yield,

k_2 = the elastic stiffness of the isolation system after yield,

Q = the static force required on the isolation system to cause initial yield,

and in this work the parameters used are $k_1 = 5.0\ W\ m^{-1}$, $k_2 = 1.0\ W\ m^{-1}$ and $Q = 0.05\ W\ m^{-1}$ where W is the total weight of the structure above the isolation system (i.e. N.Mg).

The dynamic properties of the structure proper (i.e. above the isolation system) are defined by reference to the fundamental period of the structure when unisolated, $T_{1(UI)}$, and the number of storeys in the structure is set by the relation

$$N = 10 \times T_{1(UI)}.$$

When such a structure is isolated but the isolation system is unyielded, (IUY), the natural period of mode i becomes $T_{i(IUY)}$. However once the isolation system yields one cannot strictly speak of 'natural periods' and 'modes of vibration' but in order to allow some understanding of the way in which the period characteristics of a structure change following yielding of the isolation system, the pseudo period of 'mode' i of an isolated-yielded structure $T_{i(IY)}$, is calculated using the structure proper mounted on only the elastic components of the yielded isolation system, i.e. the structure proper mounted on an elastic spring of stiffness k_2.

Viscous damping is applied to the structure by the direct method of Wilson and Penzien,[33] whereby the damping matrix is derived from the sum of a series of matrices, each of which is arranged to produce 5% of critical damping in each mode of vibration. For an unisolated structure the viscous damping matrix is simply defined using the mass and stiffness matrices of the structural system. However, for an isolated structure two viscous damping matrices are defined, one using the stiffness matrix with the B.I. system fully elastic, whilst the other uses the stiffness matrix containing only the elastic components of the yielded B.I. system. The dynamic solution routine alternates between these two damping matrices according to the state of the isolation system.

Shear forces in the structures are non-dimensionalised by the total weight of the structure above the base level (i.e. N.Mg. here) in order to allow the easy comparison of the responses of different structures. The largest such shear is denoted by S and usually, but not always, occurs at the structure base.

Now in order to be able to describe the effect of increasing higher mode dominance on the shear distributions of earthquake excited structures it is necessary to have a standard set of shear distributions, using which the sizes of shears in the upper storeys relative to that at the base can be quantified. Examples of such a set of standard shear distributions for a six storey structure are given in Figure 2. The 'Bulge Defining Parameter', B_D, used to define these shear distributions (as outlined in the Appendix) describes the 'bulge' of the resultant distributions away from the linear distribution of shear case (of $B_D = 0.0$). The dynamic solution routine determines the actual (normalised) shear distributions for the N-storey structure subjected to the given earthquake and then takes increasing values of B_D until the standard distribution so defined is just an overfit at all N storeys of the structure. This value of B_D for minimum overfit is taken as being representative of the 'bulge' of the actual shear distribution of the structure. It is worthy of note that the commonly used technique of defining the shear distribution of a structure by means of an inverted triangle of loads has an overfit standard shear distribution with a Bulge Defining Parameter of $B_D = 0.3$.

2.2 Higher Mode Effects

The B.I. system achieves its objective in two ways. It shifts the fundamental period of the structure out of the region of dominant earthquake energy and increases the structure's damping capacity. Figure 3 depicts how the first two natural periods of a base isolated structure (with the B.I. system both unyielded and yielded) change as $T_{1(UI)}$ changes. It can readily be seen that base isolation causes a wide separation of first and second mode periods, especially for the 'pseudo' periods of a structure on a yielded isolation system. This period separation is enhanced for structures of low $T_{1(UI)}$. For locations close to the centre of a shock, where seismic effects are likely to be largest, the dominant earthquake generated motions generally have periods of less than one second, and by increasing the fundamental period of the structure via base isolation to a value greater than 1.0 seconds, it is hoped to avoid the large seismic generated forces which would otherwide arise. The higher mode periods of a structure however are only slightly increased with base isolation, so moving them into the region of dominant earthquake energy, with the possibility of higher mode effects becoming more significant than in the corresponding unisolated structure.

In order to investigate the role of higher mode effects in isolated structures compared to unisolated structures, least squares fits were made to the actual shear distribution up the structure using its first three shear mode shapes. For isolated structures the isolated-unyielded shear mode shapes generally gave a better fit to the actual shear envelope than either the unisolated or the isolated-yielded pseudo shear mode shapes and hence were used to produce the plots for the isolated structures in Figure 3. Now because the shears comprising the actual shear envelope may not have all occurred at the same time (a problem noted by Clough[34]) one has the problem of deciding whether to arithmetically add the shear mode shapes (which assumes that the maximum shear forces occurred at the same time) or to add the absolute values of the shear mode shapes (which assumes that the maximum shears occur at different times) in performing the least squares fit. From the dynamic analyses it was noted that the maximum shear forces sometimes occurred almost simultaneously over the entire structure while at other times these maxima occurred at different times and were of different signs. In this investigation therefore both approaches, arithmetic and absolute value addition, were used and the results summarised in Figure 4 where the quantity R_i denotes the ratio of second (i=2) and third (i=3) shear mode amplitude to the fundamental shear mode amplitude for the best least squares fit of the first three modes to the actual shear envelopes. Parts (a) and (b) of both 1 and 2 in Figure 4 are sufficiently similar in character to allow general conclusions to be drawn regarding the relative importance of higher modes in isolated and unisolated structures.

A comparison of Figure 4 (a-1) and (b-1) to (b-2) shows that higher mode contributions are indeed generally greater for isolated structures than for unisolated structures and that the peak values of second and third mode contributions occur at lower values of $T_{1(UI)}$ for isolated structures than is the case for unisolated structures. Using the standard shear distributions (as in Figure 2), the effect of this increasing higher mode dominance on the shears in the upper floors of the structure, relative to that at the base, can be quantified.

Figure 5 shows how B_D varies when increasing fractions, R_2, of the second shear mode shapes (without regard to sign) are added to the first shear mode shapes for both isolated and unisolated structures at $T_{1(UI)}$ equal to 0.3, 0.6 and 0.9 seconds. For the isolated structures the isolated-unyielded shear mode shapes were used. Now from Figure 5 it can be seen that as $T_{1(UI)}$ increases, the bulge (and hence B_D) of the first mode shear response shape (i.e. B_D at $R_2 = 0$) increases significantly for isolated but not for unisolated structures. Also with isolated structure, as higher mode contributions increase relative to the fundamental mode (i.e. R_2 increases), the variation in B_D with $T_{1(UI)}$ decreases although B_D still increases with R_2 for each $T_{1(UI)}$. For unisolated structures the value of B_D also increases with R_2 but less rapidly than for isolated structures.

Such a result can be understood by reference to Figure 6 which illustrates how the first two shear mode shapes change with both $T_{1(UI)}$ and base isolation. Comparison of the shear mode shapes in Figure 6(a), (b), and (c) reveals that isolated structures have lower bulge first mode responses than do unisolated structures, but, as with the unisolated structures, this bulge increases slightly with $T_{1(UI)}$. It is on the second shear mode shapes that base isolation has the greatest effect however. With unisolated structures, the contribution of the second mode at the structure base is almost the maximum value for that mode and, moreover, increases with $T_{1(UI)}$ whereas with isolated structures the contribution of the second mode at the base is very much less than the maximum value for that mode and decreases as $T_{1(UI)}$ increases. (The third shear mode shapes for both isolated and unisolated structures behave in a similar fashion to their respective second shear mode shapes.) Thus as higher mode effects start to become significant in isolated structures, B_D for the resulting structure shear distribution should increase rapidly from near zero (for the pure first mode response) because significant shear contributions from the higher mode responses occur mainly in the middle and upper storeys. In unisolated structures, however, second mode effects are significant at the structure base and so tend to prevent resultant shear envelopes from changing greatly from the first mode shear shapes which have B_D of the order of 0.3.

The dramatic effect of B.I. in reducing the transmission of earthquake motions into a structure can be seen in Figure 7 where the maximum non-dimensionalised load, S (= Maximum shear in the structure/ Weight of structure), that arises in an elastic structure subjected to the El Centro accelerations, is reduced by a factor of up to 7.8.

Equally as dramatic as the reduction in S is the constancy of S with $T_{1(UI)}$ for isolated structures, a result that is by no means true for unisolated structures. This is due to the fundamental 'mode' period of the isolated-yielded structure being increased to approximately 2 seconds regardless of $T_{1(UI)}$. The El Centro acceleration response spectrum is rather flat in this period region and so changes in $T_{1(IY)}$ will have little effect on the magnitude of the first mode response. The second mode responses, as shown in Figure 4, change markedly with $T_{1(UI)}$ but because the second shear mode shapes have little contribution at the structure base (where S normally occurs), S remains unaffected by these changes. The distribution of the shear forces up an isolated structure do however depend heavily on $T_{1(UI)}$ as can be seen in Figure 8. As $T_{1(UI)}$ increases the periods of the higher modes move into the region of dominant earthquake energy and their amplitudes become increasingly significant with respect to that of the fundamental mode (Figure 4). The 'bulge' of the resulting shear distribution therefore increases from near zero, for the pure fundamental mode response, to around 0.4 for these significant second mode contributions. The bulges of the shear distributions of unisolated structures however increase only slowly with $T_{1(UI)}$ because, as noted earlier, second mode effects are significant at the structure base and so tend to prevent the resultant shear distributions from changing greatly from the basic first mode shape which has a value of B_D of the order of 0.3. The inset of Figure 8 depicts the different styles of the variation of B_D with $T_{1(UI)}$ for isolated and unisolated structures.

It can be seen from Figure 8 that while the inverted triangle of loads technique of distributing structure shears (having a B_D of 0.3) gives approximately correct shear distributions for unisolated structures of $T_{1(UI)} \leqslant 1.2$ seconds, it is non-conservative for isolated structures of $T_{1(UI)} > 0.4$ seconds. Thus while Base Isolation dramatically reduces the maximum response in a structure it does not always reduce the shears in the upper levels of the structure by the same amount.

It can be seen from Figures 7 and 8 that Base Isolation is of most benefit to stiff structures ($T_{1(UI)} < 0.4$ seconds) as not only is the maximum shear, S, reduced from a very high value but also the distribution of this lowered shear is made more triangular (i.e. B_D is decreased).

2.3 <u>The Response of Appendages</u>

It was noted in Section 1.4 that previous workers, Skinner et.al.[27,28] had found that base isolation reduced the accelerations suffered by an appendage of period T mounted on a one degree of freedom (1 d.o.f.) oscillator, also of period T.

Here the effect of B.I. in reducing the accelerations of a wide range of different period appendages located in the ground, middle and top floors of a set of structures is investigated. Figure 9 depicts the maximum absolute accelerations of 3% viscous damped appendages of periods ranging from 0.05 to 4.0 seconds mounted on the top floor of isolated and unisolated structures of $T_{1(UI)} = 0.6$ seconds. The level of 3% of critical viscous damping was chosen as appendages are in general only lightly damped.

It can be seen in Figure 9 that Base Isolation greatly reduces the maximum accelerations of appendages located at the top of multi-storey structures. It was also found that for isolated structures of $T_{1(UI)} > 0.2$ seconds,the peak response of a set of appendages mounted on the top or ground floor always occurred for an appendage whose period of vibration coincided with the second 'mode' of vibration of the isolated structure (~ 0.3 seconds in Figure 9), i.e. a second mode resonant situation. This 'peak' is absent for appendages located on the middle floor of an isolated structure. An explanation of this behaviour can be gleaned from the displacement second mode shapes of unisolated and isolated-unyielded six storey structures of $T_{1(UI)} = 0.6$ seconds, shown in Figure 10.

For an unisolated structure the amplification of ground accelerations with respect to second mode effects will be negligible for appendages located at the ground level but significant, and similar in magnitude, for appendages located at the middle or top floors. For an isolated structure however the amplification of ground accelerations with respect to second mode effects will be negligible for appendages located at the middle floor but significant and similar in magnitude, for appendages located at the ground and top floors, because of the inherently different second mode shapes in Figure 10(b) from Figure 10(a).

Because of the effect of base isolation in shifting the fundamental period of a structure out of the region of dominant, earthquake energy, it is not unexpected that the acceleration response of appendages near the pseudo second mode period of an isolated structure should be greater than those for appendages of periods near the pseudo fundamental period. (A similar effect was noted for appendages on unisolated structures of $T_{1(UI)} \geq 1.2$ seconds).

Figure 11 depicts the 'peak' maximum absolute accelerations of the 3% damped appendages located on the ground, middle and top floors of isolated structures. These 'peak' accelerations are the maximum values of curves (isolated and unisolated) like those of Figure 9 but for structures of different $T_{1(UI)}$ and for the ground and middle, as well as the top floors. The tendency of appendages on isolated structures to have greater peak accelerations when located at the top or ground floors rather than on the middle floor is immediately apparent, as is the overall tendency for appendages on isolated structures to have responses of the order of those for similar oscillators located directly on the ground.

Figure 12 compares the peak maximum acceleration responses of appendages on an isolated structure, to the maximum acceleration response of an appendage of period $T_{1(UI)}$ on the same isolated structure (i.e. an appendage which was first mode resonant for the structure when unisolated). It can be seen, particularly for the lower levels of a structure, that the responses of an appendage of period equal to $T_{1(UI)}$ on an isolated structure can be very much an underestimate to the worst resonant situation, with responses of up to ten times this value occurring.

Hence in using 1 d.o.f. models, even if the appendage period was chosen to correspond to the pseudo fundamental period of the isolated structure, one would still not have the 'worst case' as for structures of $T_{1(UI)} > 0.2$ seconds, 'second mode resonant' appendages have the largest response.

Even with the effect of the higher modes dominating the accelerations of appendages on isolated structures, base isolation still has a very beneficial effect in lowering the level of appendage responses.

CONCLUSIONS

The Base Isolation System is a very practical, convenient, easily installed, maintenance free, highly reliable means of providing earthquake protection for a structure and has the advantage that it requires no complex or delicate control mechanisms but works directly from the material properties of its components.

Base Isolation reduces structure shear forces to 0.1 W by suppressing the fundamental mode response of the structure but this can increase the influence of the second mode on the overall structure response. The degree to which second mode effects become dominant depends on the correspondence or otherwise of the second mode period of the isolated structure with a high energy period of vibration of the forcing earthquake. An increase in the fundamental period of the structure proper, or a decrease in the horizontal stiffness of the isolation system, or both, serve to increase the second mode period of an isolated structure which generally increases the changes of having larger amplitude second mode vibrations and hence the chance of second mode dominated structure responses. This increasing significance of second mode effects means that not only can the 'bulges' of shear distributions be increased following base isolation but also that the peak responses of appendages on isolated structures will typically occur for 'second mode resonant' appendages.

It can therefore be seen that Base Isolation is of most benefit to structures of low $T_{1(UI)}$ where second mode periods are sufficiently low so as to not cause an increase in second mode significance following isolation.

APPENDIX

To evaluate a standard shear distribution having a given bulge defining parameter B_D one first takes

$$\theta = \cos^{-1}(1 - B_D) ,$$

where the angle θ defines the position of the swinging arm OC in Figure A-1. The static horizontal load, F_i, to be applied above column i is the distance, at level i, from the ordinate AC to the swinging arm OC. Only contributions above the higher of points C or D are considered.

Thus from Figure A-1 one obtains,

$$F_i = \begin{cases} (i/N - x_{crit}) \tan\theta \; ; & i/N > \max(x_{crit}, 0.0) \\ 0.0 \; ; & i/N \leq \max(x_{crit}, 0.0) \end{cases}$$

$$\text{where } x_{crit} = 1 - \cot\theta \qquad \text{(A-1)}$$

and the normalised shear distribution which results from this force distribution is,

$$se_i = \begin{cases} \left(\sum_{j=1}^{N} F_j\right) \Big/ \left(\sum_{j=k+1}^{N} F_j\right) \; ; & i > k \\ 1.0 \; ; & i \leq k \end{cases}$$

$$\text{where } k = \max \begin{cases} N(1 - \cot\theta), \text{ rounded down to the next integer.} \\ 0.0 \end{cases}$$

$$\text{(A-2)}$$

On evaluation of the summation terms, Equation A-2 becomes,

$$se_i = \begin{cases} \dfrac{\tan\theta\left[-N(N+1) - i(i-1) + 2Ni\right] + 2N(N-i+1)}{\tan\theta\left[-N(N+1) - k(k+1) + 2N(k+1)\right] + 2N(N-k)} & ; \ i > k \\ 1.0 & i \leq k \end{cases}$$

$$\text{where } k = \max \begin{cases} N(1 - \cot\theta), \text{ rounded down to the next integer.} \\ 0.0 \end{cases} \qquad \text{(A-3)}$$

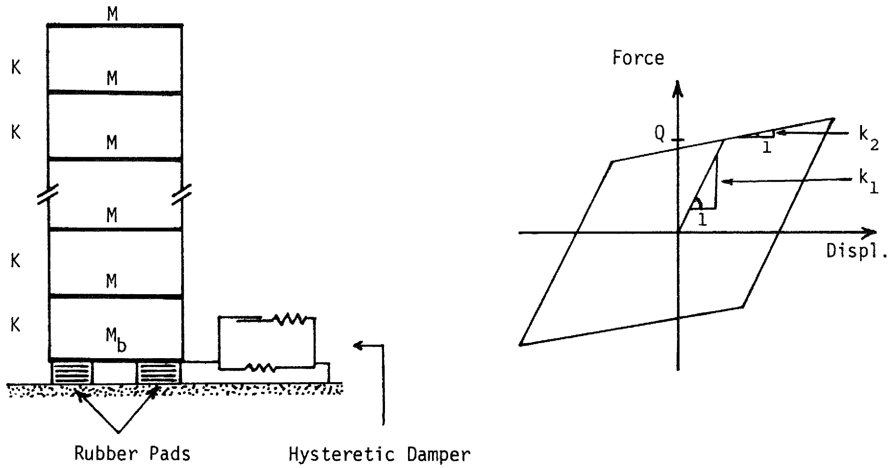
Figure 2 illustrates standard shear distributions for a six storey structure together with their bulge defining parameters, B_D. It should be noted that the method of defining these standard shear distributions ensures that the shear at any level of the structure never exceeds the base shear. The $\theta = 45^o$ situation is the commonly used inverted triangle of loads technique for distributing structure shears and has a Bulge Defining Parameter of $B_D \simeq 0.3$.

REFERENCES

1. G. A. Eiby, "A History of Anti-Seismic Measures in New Zealand", Bull. N.Z. Nat. Soc. Earthq. Engng, Vol. 8, No. 4, pp. 255-259 (December 1975).
2. G. W. Housner, "Analysis of Structural Response and Instruments", Summary Report, Proc. 3rd Wld. Confr. Earthq. Engng, Vol. II, (1965).
3. "Earthquake Resistant Regulations of the World", Compiled by the Organising Committee of the 2nd Wld Confr. on Earthq. Engng, (1960).
4. "New Zealand Standard 4203 (1976), Part 3: Earthquake Provisions, 3.4.6, Distribution of Horizontal Seismic Forces.
5. J. A. Blume, "Structural Dynamics in Earthquake Resistant Design", Trans. A.S.C.E., Vol. 125, p. 3054, pp. 1088-1139, (1960).
6. R. R. Martel, "The Effects of Earthquakes on Buildings with a Flexible First Storey", Bull. Seism. Soc. Am., Vol. 19, pp. 167-178 (1929).
7. N. B. Green, "Flexible First Storey Construction for Earthquake Resistance", (with Discussion by Johnson, Bednarski, White, Kartzke, Smits, Williams), Trans Am. Soc. Civ. Engrs, Paper No. 1906, pp. 654-674, (1935).
8. C. Katsuta, N. Mashizu, "Earthquake Isolation Method of Structure by a High Speed Electrohydraulic Servomechanism", Proc. 3rd Wld. Confr. on Earthq. Engng, Vol. II, pp. 276-297, (1965).
9. K. Matsushita, M. Izumi, "Some Analyses on Mechanisms to Decrease Seismic Force Applied to Buildings", Proc. 3rd Wld Confr. on Earthq. Engng, Vol. II, pp. 342-359, (1965).
10. K. Matsushita, M. Izumi, "Deflection Controlled Elastic Response of Buildings and Methods to Decrease the Effect of Earthquake Forces Applied to Buildings", Proc. 3rd Wld. Confr. on Earthq. Engng, Vol. II, pp. 360-372, (1965).
11. M. Fintel, F. R. Khan, "Shock Absorbing Soft Storey Concept for Multi-Storey Earthquake Structures", Proc. Am. Concr. Inst., Vol. 66, Title No. 66-29, pp. 381-390, (1969).
12. M. S. Caspe, "Earthquake Isolation of Multistorey Concrete Structures", Proc. Am. Concr. Inst., Vol. 67, Title No. 67-61, pp. 923-933, (Nov., 1970).
13. A. K. Chopra, D. P. Clough, R. W. Clough, "Earthquake Resistance of Buildings with a Soft First Storey", Int. J. Earthq. Engng Struct. Dyn., Vol. 1, pp. 347-355, (1973).
14. P. H. Wirsching, J. T. P. Yao, "Safety Design Concepts for Seismic Structures", Comput. Struct., Vol. 3, pp. 809-826, (1973).
15. J. H. Williams (Jnr), "Designing Earthquake Resistant Structures", Tech. Rev., Vol. 76, No. 1, pp. 37-43, (1973).
16. R. Husid, J. Sanchez, "Earthquake Response Reduction in Buildings with an Elastoplastic Storey", Reg. Confr. Tall Bldgs, Bangkok, pp. 671-682, (January 1974).
17. S. H. Crandall, S. S. Lee, J. H. Williams (Jnr), "Accumulated Slip of a Friction Controlled Mass Excited by Earthquake Motions", Trans. A.S.M.E., Jour. Appl. Mech., Vol. Al, Series E, No. 4, pp. 1094-1098, (1974).
18. C. J. Derham, L. R. Wootton, S. B. B. Learoyd, "Vibration Isolation and Earthquake Protection of Buildings by Natural Rubber Mountings", N.R. Tech. Vol. 6, No. 2, pp. 21-33, (1975).
19. R. A. Waller, "Buildings on Springs", Sound Vibrn., Vol. 9, No. 3, pp. 22-25, (1975).
20. G. C. Delfosse, "The Gapec System : A New Highly Effective Aseismic System", Proc. 6th Wld. Confr. on Earthq. Engng, New Delhi, India, Vol. 3, p. 163, (1977).
21. J. M. Eidinger, J. M. Kelly, "Experimental Results of an Earthquake Isolation System Using Natural Rubber Bearings", Report No. UCB/EERC-78/03, Univ. of Calif., Berkeley. (Feb. 1978).
22. R. I. Skinner, J. L. Beck, G. N. Bycroft, "A Practical System for Isolating Structures from Earthquake Attack", Int. J. Earthq. Engng Struct. Dyn., Vol. 3, pp. 297-309, (1975).
23. R. I. Skinner, J. M. Kelly, A. J. Heine, "Energy Absorption Devices for Earthquake Resistant Structures", Proc. 5th Wld. Confr. Earthq. Engng, Rome, Vol. 2, pp. 2924-2933, (1973).
24. R. I. Skinner, J. M. Kelly, A. J. Heine, "Hysteretic Dampers for Earthquake Resistant Structures", Int. J. Earthq. Struct. Dyn., Vol. 3, pp. 287-296, (1975).
25. R. G. Tyler, "Dynamic Tests on Laminated Rubber Bearings", Bull. N.Z. Nat. Soc. Earthq. Engng, Vol. 10, No. 3, pp. 143-150, (September 1977).
26. R. I. Skinner, G. H. McVerry, "Base Isolation for Increased Earthquake Resistance of Buildings", Bull. N.Z.

Nat. Soc. Earthq. Engng, Vol. 8, No. 2, pp. 93-101, (1975).
27. R. I. Skinner, G. H. McVerry, "A Practical System for Isolating Nuclear Power Plants from Earthquake Attack", Nucl. Engng and Design, Vol. 36, No. 2, pp. 287-297, (1976).
28. R. I. Skinner, R. G. Tyler, S. B. Hodder, "Isolation of Nuclear Power Plants from Earthquake Attack", Bull. N.Z. Nat. Soc. Earthq. Engng, Vol. 9, No. 4, pp. 199-204, (1976).
29. M. J. N. Priestley, R. L. Crosbie, A. J. Carr, "Seismic Forces in Base Isolated Masonry Structures", Bull. N.Z. Nat. Soc. Earthq. Engng, Vol. 10, No. 2, pp. 55-68, (1977).
30. D. M. Lee, I. C. Medland, "Base Isolation for Earthquake Protection of Multi-Storey Shear Structures", Proc. 6th Aust. Confr. on Mech. & Structs of Mats, Vol. 1, pp. 181-188, (1977).
31. J. M. Kelly, J. M. Eidinger, C. J. Durham, "A Practical Soft Storey Isolation System", Report No. UCB/EERC-77/27, Univ. of Calif., Berkeley. (Nov. 1977).
32. D. M. Lee, "The Effect of Base Isolation on Multi-Storey Shear Structures", Ph.D Thesis, University of Auckland, School of Engineering, (July, 1978).
33. E. L. Wilson, J. Penzien, "Evaluation of Orthogonal Damping Matrices", Int. J. Num. Meth. Engng, Vol. 4, pp. 5-10 (1974).
34. R. W. Clough, "On the Importance of Higher Modes of Vibration in the Earthquake Response of Tall Buildings", Bull. Seism. Soc. Am., Vol. 45, No. 4, (October 1955).

Paper received 29 September, 1978.



(a) (b)

FIGURE 1(a) AN N STOREY, LUMPED PARAMETER, BASE ISOLATED, SHEAR STRUCTURE WITH ISOLATION SYSTEM CHARACTERISTICS AS IN (b)

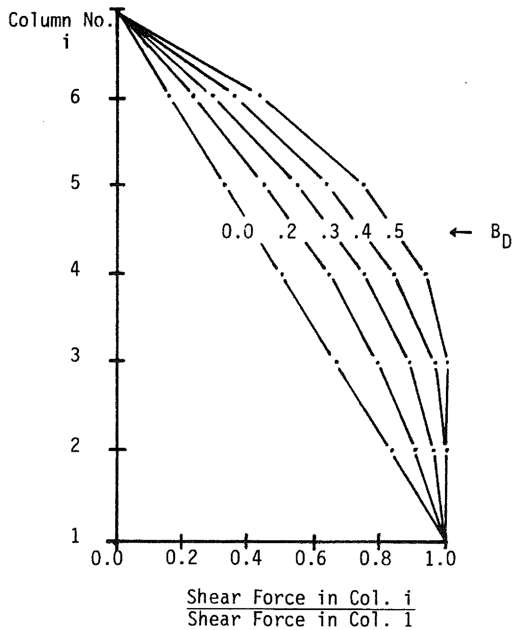


FIGURE 2: VARIOUS STANDARD SHEAR DISTRIBUTIONS FOR A SIX-STOREY STRUCTURE

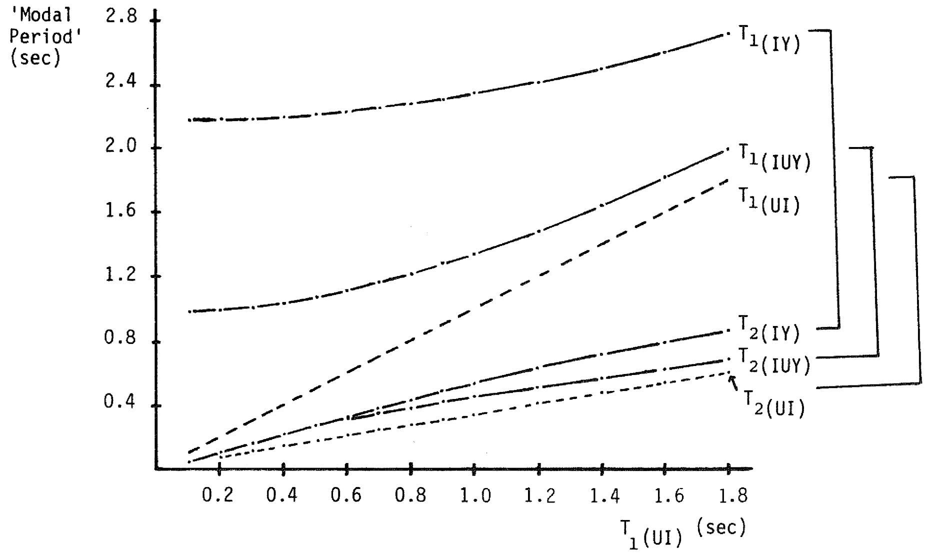
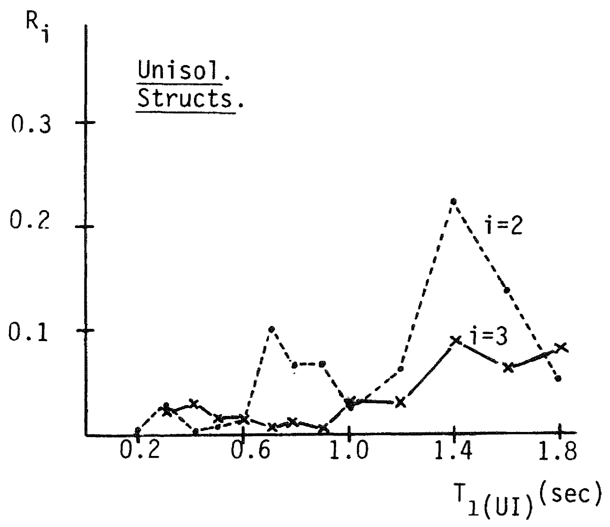
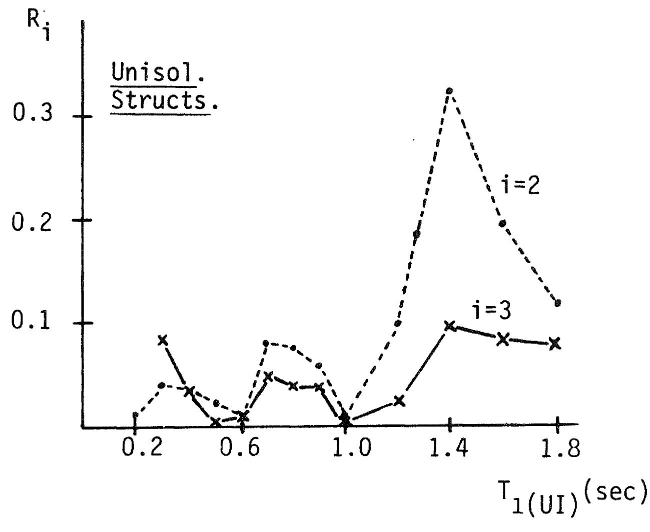


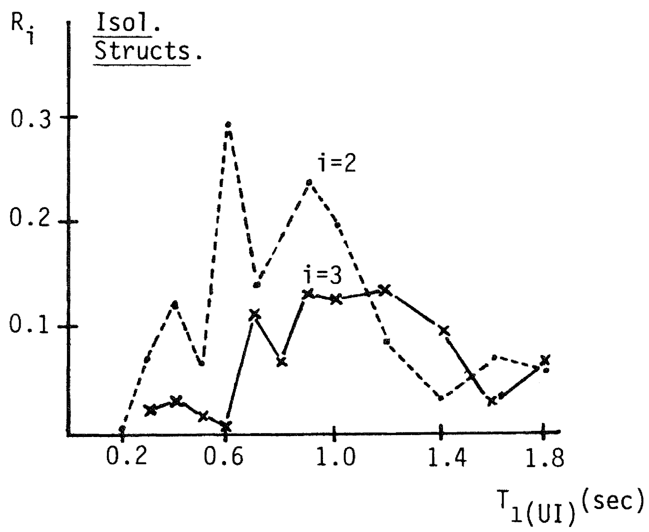
FIGURE 3: ISOLATED AND UNISOLATED FIRST AND SECOND MODE STRUCTURE 'PERIODS' VERSUS THE FIRST MODE UNISOLATED PERIOD



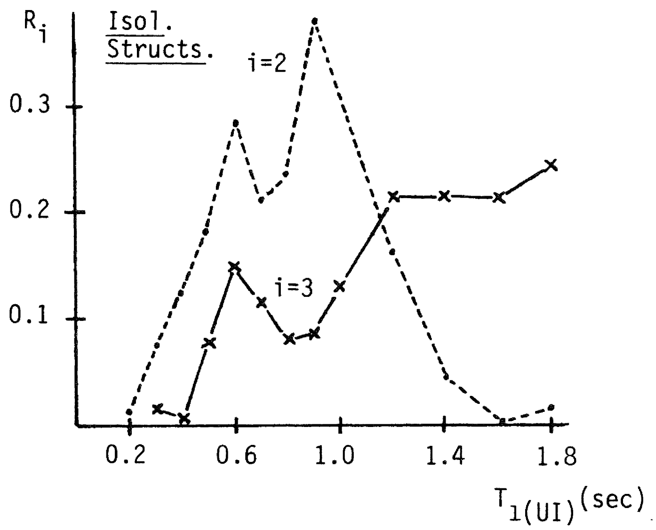
(a-1) Arithmetical addition of mode shapes.



(b-1) Absolute value addition of mode shapes.



(a-2) Arithmetical addition of mode shapes



(b-2) Absolute value addition of mode shapes.

FIGURE 4: RELATIVE AMPLITUDE, R_i, OF SHEAR MODE i TO THE FUNDAMENTAL SHEAR MODE FOR THE LEAST SQUARES FIT OF THE FIRST THREE SHEAR MODES OF VIBRATION TO THE ACTUAL SHEAR DISTRIBUTION UP THE STRUCTURE

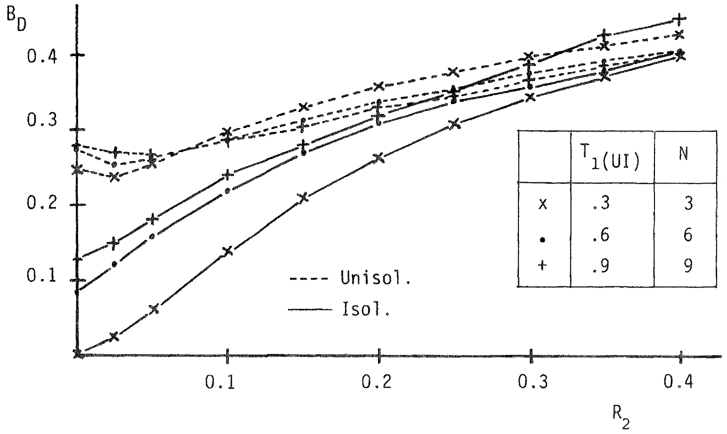


FIGURE 5: BULGE DEFINING PARAMETER, B_D FOR SHEAR DISTRIBUTIONS FORMED BY ADDING A FRACTION, R_2 OF THE MAGNITUDE OF THE SECOND SHEAR MODE SHAPE, TO THAT OF THE FIRST

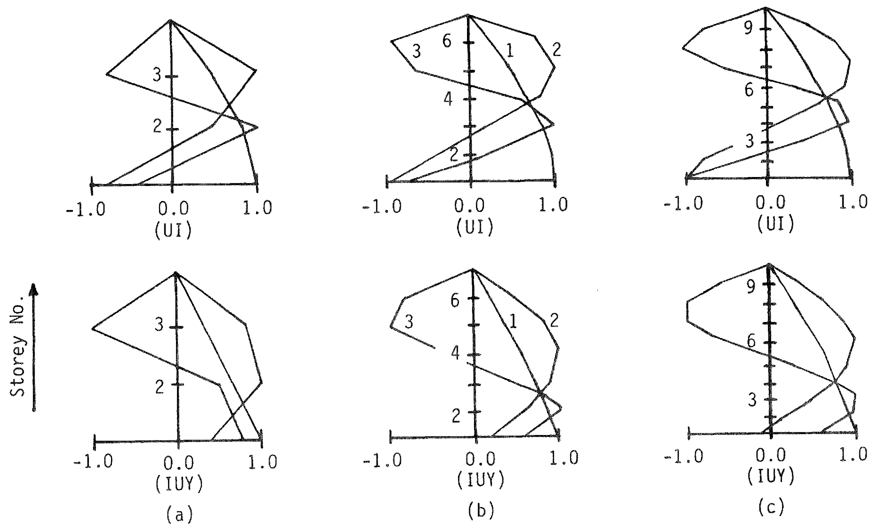


FIGURE 6: THE FIRST THREE SHEAR MODE SHAPES FOR UNIFORM, UNISOLATED (UI) AND ISOLATED UNYIELDED (IUY) (a) N=3, (b) N=6, (c) N=9 STOREY STRUCTURE OF $T_{1\ (UI)}$ = 0.1 x N SEC.

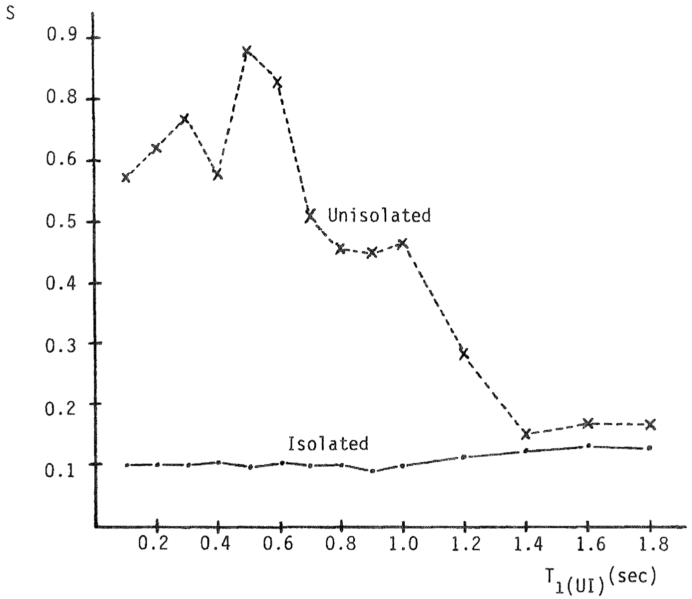


FIGURE 7: MAXIMUM NON-DIMENSIONALIZED SHEAR, S, IN AN NSTOREY, 5% VISCOUS DAMPED, SHEAR STRUCTURE OF FUNDAMENTAL PERIOD WHEN UNISOLATED OF $T_{1\ (UI)}$ = 0.1 x N SEC.

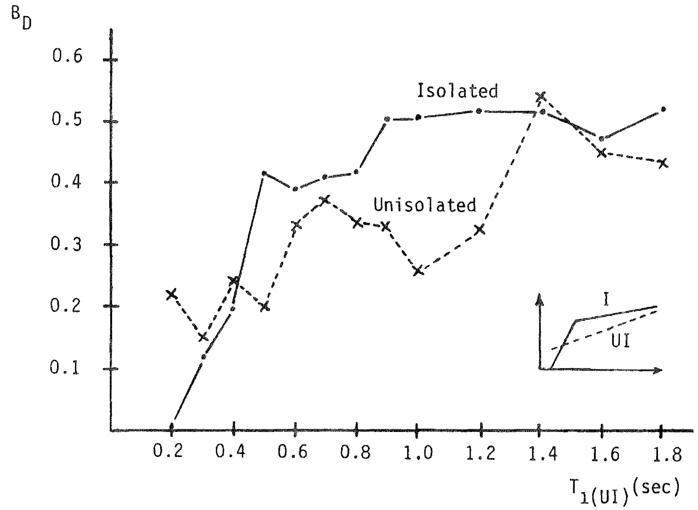


FIGURE 8: SHEAR DISTRIBUTION BULGE DEFINING PARAMETER, B_D, VERSUS THE FUNDAMENTALPERIOD OF THE UNISOLATED STRUCTURE (FOR ISOLATED AND UNISOLATED STRUCTURES)

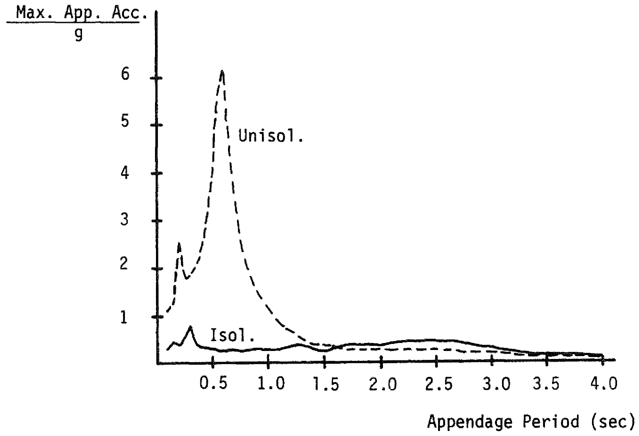


FIGURE 9: MAXIMUM ABSOLUTE ACCELERATION OF 3% VISCOUS DAMPED APPENDAGES LOCATED ON THE STOP FLOORS OF ISOLATED AND UNISOLATED SIX STOREY STRUCTURES OF $T_{1\,(UI)}$ = 0.6 SEC.

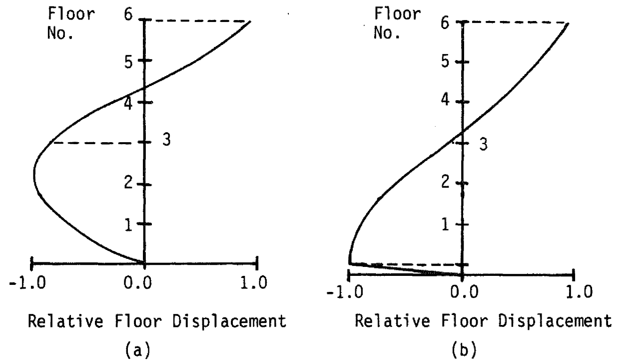


FIGURE 10: DISPLACEMENT SECOND MODE SHAPES FOR A SIX STOREY (a) UNISOLATED AND (b) ISOLATED-UNYIELDED STRUCTURE

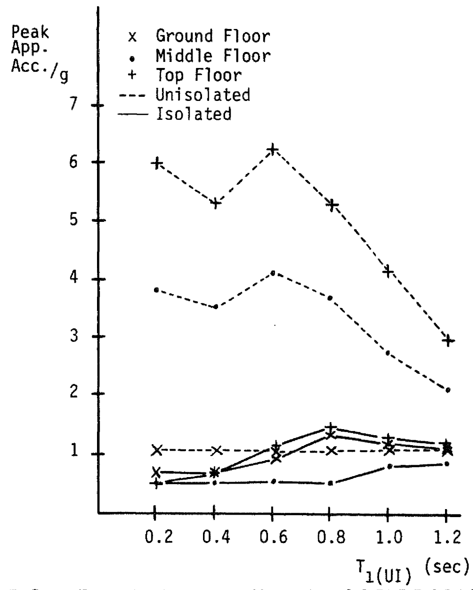


FIGURE11: PEAK MAXIMUM ACCELERATIONS OF 3% DAMPED APPENDAGES LOCATED ON VARIOUS FLOORS OF ISOLATED AND UNISOLATED STRUCTURES

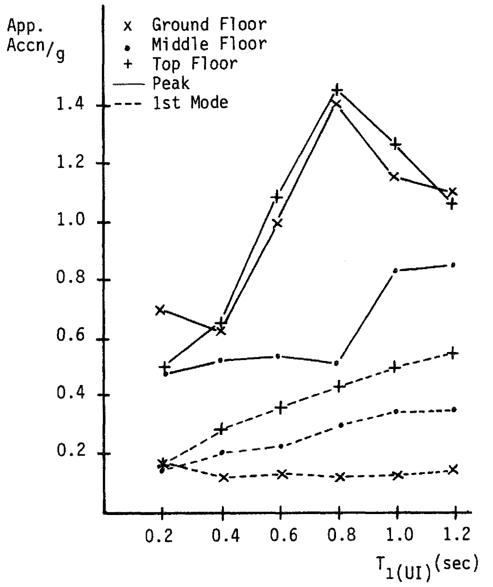


FIGURE 12: PEAK AND 1ST MODE RESONANT MAXIMUM ACCELERATIONS OF 3% DAMPED APPENDAGES LOCATED ON ISOLATED STRUCTURES

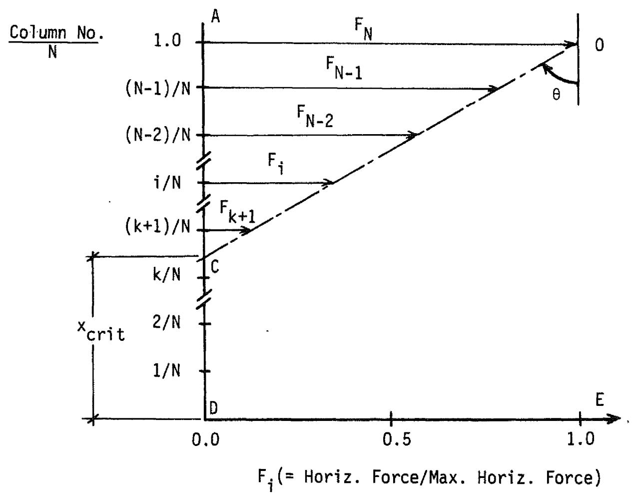


FIGURE A-1: DISTRIBUTION OF HORIZONTAL LOADING FOR THE STANDARD SHEAR DISTRIBUTIONS