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

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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 undamaged 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 (2). 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

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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 localized 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 lower levels, so that the natural periods 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 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 fundamental period 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 defined. He also 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 due to 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 own foundation. 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 own foundation. In the appended discussion on the paper entitled 'The Effect of Earthquakes on Buildings with a Flexible First Storey', Martel considered the response of an undamped oscillator, by means of direct integration of the equations of motion over each time step and including the stiffness of the remaining columns of the structure. He also 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 due to 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.

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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 the
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 earth­
quake, therefore, the system would hopefully
behave as a rigid body on a bilinear
hysteretic foundation, and have all damage
confined to these specially designed founda­
tions. 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
storey, 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 on its
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 hysteretic support, 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, an increase in the yield force
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 the isolation system (given 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 the 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
was the reflection effect, which resulted from
elastic vibrations 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
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 primarily
on the elastic component of the base shear
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 just above the first storey.
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 earth­
quake attack. Five different systems were
examined for reducing the vibrations trans­
mited into a structure from the ground. These
were:
was absolutely controlled. He also suggested were, in order of decreasing effectiveness, hysteretic support mechanism in reducing the mental effect because of the system having a set spectral density characteristics. A linear log-log relation­ship was found between $y$, the coefficient of friction of the mass on the foundation, and the R.M.S. offset. The authors pointed out that the force transmitted to the structure 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 upon 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 the El Centro (N.S. 1940) earthquake. 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 relationship was found between $\nu$, the coefficient of friction of the mass on the foundation, and the R.M.S. offset. The authors pointed out 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 $ug$, where $g$ is 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' type. Natural rubber bearing springs gave the 'best' overall performance when compared with steel, polychloroprene and butyl rubber springs in tests on their transmissibility, internal damping capacity, creep resistance, 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 to 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°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 springs 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 limit the slip 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 on an earthquake. The form of these stabilisers was not revealed nor was it stated how they were 'automatically disconnected' or for that matter, reconnected.

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. (10) 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 Wirshing 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 to be 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 mechanism, unlike those derived 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 on a 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 ductility demands were very much less than those in a resonant appendage 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 unisolated were 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 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. 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 Holland 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.) 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. 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, M0, is of no significance as it is assumed to be rigidly attached, and move with the ground. However, for an isolated structure M0 is attached to the surface of the isolation system and does take part in the dynamic response of the structure. It has been shown that reductions in the interstorey stiffness of the upper floors had little effect on the responses of isolated structures. In this work γ is taken as 1/6.

The Base Isolation System illustrated in Figure 1 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, k1 = the elastic stiffness of the isolation system before yield, k2 = 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 k1 = 5.0 W m−1, k2 = 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, T0(UI), and the number of storeys in the structure is set by the relation N = 10 x T0(UI).

When such a structure is isolated but the isolation system is unyielded, (IUV), the natural period of mode i becomes T0(IUV). However once the yield point of 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 yield, the pseudo period of 'mode' i of an isolated-yielded structure T0(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 k2.

Viscous damping is applied to the structure by a direct method of Wilson and Penzien 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 isolation system. The 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', $B_p$, 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 $B_p = 0.0$). The dynamic solution routine determines the actual (normalised) shear distributions for the $N$-storey wide structure and from these the fundamental period of the structure, relative to that at the base, can be quantified. 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 $B_p = 0.3$.

### 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 for low $T_1(UI)$ 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, relative to that at the base, it is hoped to avoid the large seismic generated forces which would otherwise arise. The locations 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 the bulge based approaches, arithmetic and absolute value addition, were used and the results summarised in Figure 4 where the quantity $R_j$ 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 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 distribution (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 $B_p$ varies when increasing fractions, $R_2$, of the second shear mode shapes (without regard to sign) are added to the first shear mode shape for both unisolated and isolated structures. For isolated structures the isolated-yielded shear mode shapes were used. Now from Figure 5 it can be seen that as $T_1(UI)$ decreases or $R_2$ increases, the variation in $B_p$ with $T_1(UI)$ decreases although $B_p$ still increases with $R_2$ for each $T_1(UI)$. For unisolated structures the value of $B_p$ 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. Within isolated structures, the contribution of the second mode to 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, $B_2$ 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 mid and upper levels of the structure. 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 $B_2$ 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 acceleration, 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 not 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 $T_1(UY)$ will have little effect on the magnitude of the first mode response. The second mode responses, as shown in Figure 4, change markedly with $T_1(UY)$ 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 $T_1(UY)$ as can be seen in Figure 8. As $T_1(UY)$ 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 distribution of unisolated structures however increase only slowly with $T_1(UY)$ because, as noted earlier, second mode effects are significant at the structure base and to prevent the resultant shear distributions from changing greatly from the basic first mode shape which has a value of $B_2$ of the order of 0.3. The inset of Figure 8 depicts the different styles of the variation of $B_2$ with $T_1(UY)$ for isolated and unisolated structures.

It can be seen from Figure 8 that while the inverted triangle of loads technique of distributing structure shear (having a $B_2$ of 0.3) gives approximately correct shear distribution for isolated structures of $T_1(UY) < 1.2$ seconds, it is non-conservative for isolated structures of $T_1(UY) > 0.4$ 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 ($T_1(UY) < 0.4$ seconds) as not only is the maximum response reduced from a very high value but also the distribution of this lowered shear is made more triangular (i.e. $B_2$ is decreased).

2.3 The Response of Appendages

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 (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 at a viscous damped appendages of periods ranging from 0.05 to 4.0 seconds mounted on the top floor of isolated and unisolated structures of $T_1(UY) = 0.6$ 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 $T_1(UY) > 0.2$ 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 $T_1(UY) = 0.6$ 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 significant if 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 $T_1(\text{UI}) \geq 1.2$ 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' are the maximum values of curves (isolated and unisolated) like those of Figure 9 but for structures of different $T_1(\text{UI})$ and for the ground and middle, 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 floors 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 $T_1(\text{UI})$ on the same isolated structure (i.e. an appendage which was first mode resonant for the same period when unisolated). It can be seen, particularly for the lower levels of a structure, that the responses of an appendage of period equal to $T_1(\text{UI})$ 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 $T_1(\text{UI}) > 0.2$ 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 $T_1(\text{UI})$ 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 $B_0$ one first takes

$$\theta = \cos^{-1}(1 - B_0),$$

where the angle $\theta$ 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,

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

where $x_{\text{crit}} = 1 - \cot \theta$.

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

$$s_{e_i} = \begin{cases} \sum_{j=1}^{N} F_j; & i > k \\ \frac{\sum_{j=k+1}^{N} F_j}{1.0}; & i \leq k \end{cases}$$

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

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


Figure 2 illustrates standard shear distributions for a six storey structure together with the bulge defining parameters, $B_\text{D}$. 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 $0=45$-triangle of loads technique for distributing structure shears and has a Bulge Defining Parameter of $B_\text{D} = 0.3$.

**REFERENCES**


Paper received 29 September, 1978.
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.
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 $T_1 (UI) = 0.1 \times N$ SEC.

FIGURE 7: MAXIMUM NON-DIMENSIONALIZED SHEAR, $S$, IN AN NSTOREY, 5% VISCOUS DAMPED, SHEAR STRUCTURE OF FUNDAMENTAL PERIOD WHEN UNISOLATED OF $T_1 (UI) = 0.1 \times N$ SEC.

FIGURE 8: SHEAR DISTRIBUTION BULGE DEFINING PARAMETER, $B_D$, VERSUS THE FUNDAMENTAL PERIOD 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

FIGURE 11: 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