Base isolation: the good, the bad and the ugly

Athol J. Carr

Professor (Emeritus), University of Canterbury, Christchurch.

Director, MC Consulting Engineers, Christchurch.

Director, Carr Research Ltd., Christchurch.

Arun M. Puthanpurayil

Beca Ltd., Wellington.

ABSTRACT

In the light of the recent Christchurch and Kaikoura earthquakes and the life safety and the economic uncertainties they pose, the need for low damage design has become more important. In theory a low damage design should provide life safety during the earthquake and a high seismic resilience and business continuity following the earthquake. Evidence from the Christchurch and Kaikoura earthquakes have shown that the conventional capacity design approach which satisfied the life safety requirements failed to meet the other requirements. Thus, there is an important need for innovative technologies to be applied to buildings to enhance their performance. One such technology is base isolation. Base isolation, as the name suggests, isolates the superstructure from the substructure or foundation by providing an energy absorbing flexible layer between them. The provision of the flexible layer results in increased period and additional energy dissipation which results in minimizing the seismic energy input into the system. At present, this technique has become relatively popular in structural design in parts of New Zealand. This paper presents the preliminary results of an ongoing study into the advantages and disadvantages of this technology with regards to its applications in various types of buildings founded in different soil types and locations. To this regard, case study results are presented for a 7-storey reinforced concrete framed structure founded on different soil types and in different locations. Special attention is given to the response of base isolation in soft soil and near fault conditions. It has been shown that on soft soils and sites with high near fault possibility this technology should be used with utmost care and might not give the same benefits as presently believed.

1 INTRODUCTION

In the light of the recent earthquakes like the Christchurch sequence of earthquakes 2010-2011, more than ever the need for seismic resilience has been highlighted. The loss of life, social and economic disruption caused by these earthquakes is unprecedented. To exemplify this statement, the deaths in the 2010 and 2011 Canterbury earthquakes were 185 as per record with 1000s injured and accompanied by damage and business disruption that amounted to be around $\geq NZ\$40$ billion which corresponds to approximately 20% GDP (Pampanin 2015). The recovery is still in progress in 2021, that is 10 years after the earthquake. The social disruption caused by the earthquakes is very significant.

Similarly, the $M_w = 7.8$, 14th November 2016 Kaikoura earthquake in New Zealand, although only resulting in two deaths, the earthquake-related damages to buildings and infrastructure were roughly estimated as

$\geq NZ\$15$ billion (MBIE document). For the sustenance of a modern society, these recurrence of disruptions and economical losses are unacceptable. Also, in the light of these recurring events, it has come to the forefront that as far as New Zealand is concerned, earthquakes are not rare events as treated by most parts of the world. So, the present reliance on capacity design philosophy which relies on "dissipation with degradation" is not an economically viable option.

Similar observations could be made from past events in other parts of the world; for example, the 1989 M_w 6.9 Loma Prieta earthquake caused 63 deaths and more than *US* $8 billion in direct damage (several buildings and bridges suffered total and partial collapse) (Wada et al., 2004). Similar observations were made for 1995 Kobe earthquake (*US* $102.5 billion in damage, 2.5% of Japan's GDP at the time) and the 1999 Chi-Chi earthquake which caused about *US* $10 billion worth of damage (Wada et al., 2004).

The huge economic losses caused by these earthquakes poses the engineering community with a big challenge which translates as, how do we make our infrastructure resilient to these unexpected disasters?

There is a general feeling amongst the public that the level of seismic damage incurred in these recent earthquakes (Christchurch sequence and Kaikoura earthquakes) are not acceptable and should be minimized. This expectation from society necessitates a complete paradigm shift from the dissipation by degradation philosophy of the present seismic design approach to dissipation without degradation to achieve both economic resilience and life safety. This shift of focus in the seismic design philosophy demanded by the public community may only be brought to fruition by adopting low damage technologies like base isolation, viscous dampers etc. But still the pressing question remains that which out of the available techniques of so called low damage options provides the highest resilience?

1.1 Motivation

Presently, base isolation is thought to be the "highly resilient" low damage solution. There is no doubt in the mind of the authors that, if base isolation is applied properly satisfying all the basic assumptions of the design process, then it will result in a structural system which is highly resilient. But if the assumptions under which base isolation works are violated, then it might not prove itself to be a highly resilient system. In this paper *a* very preliminary investigation into the resilience of base isolation is reported. It must be noted that the results reported in this study are part of an ongoing investigation into the efficacy and suitability of base isolation scheme for different type of structures. First the design philosophy and the benefits of base isolation is outlined. Then the issues arising from the violation of the assumptions of base isolation are described. This is followed by the results of a numerical study of a seven-story structure where both the beneficial and not so beneficial aspects of base isolation is highlighted.

2 INHERENT ASSUMPTIONS AND THE DESIGN PHILOSOPHY OF BASE ISOLATION

Base isolation is the technique of introducing flexible layers between the superstructure and the substructure which results in dynamically modifying the system response to a ground motion. The benefit of base isolation is mainly gained from the fact that the introduction of a nonlinear flexible layer between the superstructure and sub-structure results in period lengthening which means that the maximum participating mode (first mode) is moved to a region of lower seismic excitation. Figure 1.0 depicts this phenomenon.

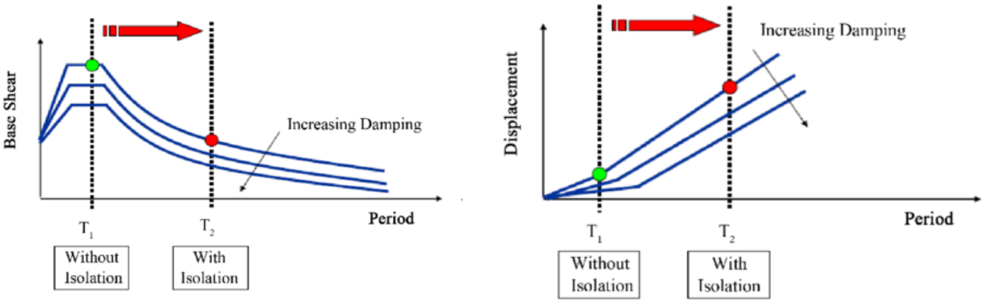


Figure 1: Conceptual illustration of the effect of base isolation layer on the pseudo-acceleration spectra and the displacement spectra. (adopted from Barmo et al. 2015)

A close observation of Figure 1 would immediately suggests that the major benefit is obtained from the period shift and the secondary benefit is obtained from the further available damping in the isolation layer. In other words, base isolation primarily relies on the period shift assuming the shown stylised spectral shape will be representative of the future earthquakes. So, in summary, base isolation relies on the following 3 assumptions:

1. There is no impact from foundation compliance or in other words the structure is founded on good ground.
2. Future earthquakes follow the standard stylised spectral shape with no anomalies.
3. There is clearance to accommodate the rigid body displacement of the system.

The third assumption is not really an assumption but a design outcome. When considering the resilience of the system, it is inherently assumed that there is enough clearance to accommodate the rigid body absolute displacement. Since high uncertainty is associated with this aspect, mainly due to the inability to correctly estimate the future earthquake induced ground displacement, here in this paper this is treated as an assumption. This paper presents preliminary results of an initial study on the second and third assumptions. As no explicit soil-structure interaction is considered, the first assumption is not investigated.

3 BENEFITS OF BASE ISOLATION

This section basically highlights the benefits of the base isolation in qualitative terms which will be further exemplified in the numerical section study.

3.1 The good

As most structures possesses maximum mass participation and hence the highest modal strain energy in the first mode, isolating this mode or shifting this mode from predominant energy components in the incoming ground motion to a lower energy zone aids in reduced migration of inertial forces to the superstructure. This approach as shown in the past earthquakes performs well when the earthquake follows the stylised spectrum based on which the design of the isolator is made. Of all the low damage passive techniques available currently, base isolation stands out mainly because rather than modifying the primary modal deformations (as is done by most of the other techniques), it basically modifies the system itself by introducing a boundary nonlinearity which actually converts the primary deformational mode of the superstructure into a rigid body mode. As the nonlinearity is introduced in the boundary, the effect of base isolation is very beneficial under the circumstances where the structure has a relatively short natural period of free-vibration and the earthquakes have an acceleration response spectra similar to that shown in Figure 1 and that the ensuing displacement spectra is not too different from that indicated in Figure 1.

4 ISSUES WITH BASE ISOLATION

Presently in New Zealand, more base isolated structures are being constructed. As mentioned in the previous section, although base isolation is beneficial, if the above stated assumptions are in danger of violation, there may be some negative effects. This section mainly outlines the issues with violation of the above stated assumptions and something the engineering community need to be aware of before deciding that base isolation as their solution.

4.1 The bad

In this section the first two assumptions are critically evaluated. The first assumption states that the structure is founded on good ground; in other words, there will not be any soil-foundation-isolator-structure interaction during the ground motion. Now what would happen if this assumption were not adhered to. The effect of this is illustrated qualitatively in Figure 2 and this effect was discussed by Filiatrault et al (1990) and Andriono and Carr (1991). In effect, the shift of the period phenomenon might result in amplification of the response mainly because of the change in spectral shape for the soft soil.

Now if we review so many of the structures that are currently being built in New Zealand, it may be seen that this assumption is not strictly adhered to. So many of the structures are founded on “C” soil or even softer ground. Though a “C” soil will not be classed as soft, reasonable soil compliance maybe expected which demands a soil compliance study. The justification is mainly done on the basis of using nonlinear time history analysis and evaluating the performance of the system with a soil model in the collapse limit state (1 in 2500 year return period) earthquake. The fundamental flaw in the present analyses as adopted by most of the practising engineers (communication with other engineers) is that they assume that the “C” soil will remain “C” itself in an Maximum Credible Earthquake (MCE) type event which is far from reality. This is absolutely not true as soil will have higher inelastic excursions and the same level of benefit will not be obtained by base isolating the system. Also capturing the effect of soil-foundation-isolator-structure interaction is a highly complex and uncertain problem which is still in research stage. Interested readers should refer to Andriono and Carr (1991), Luco J E (2014). No further study is presented in this regard in this paper as no SSI is considered in the numerical example.

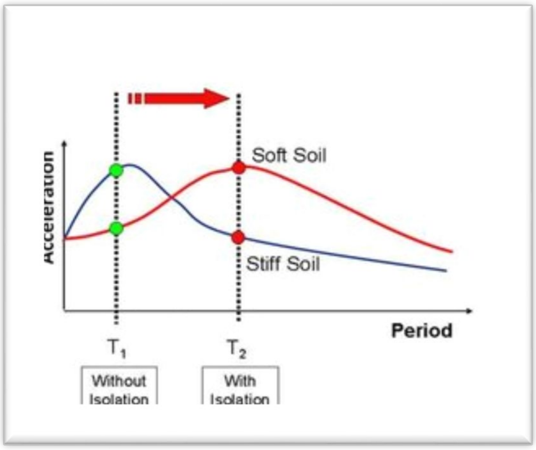
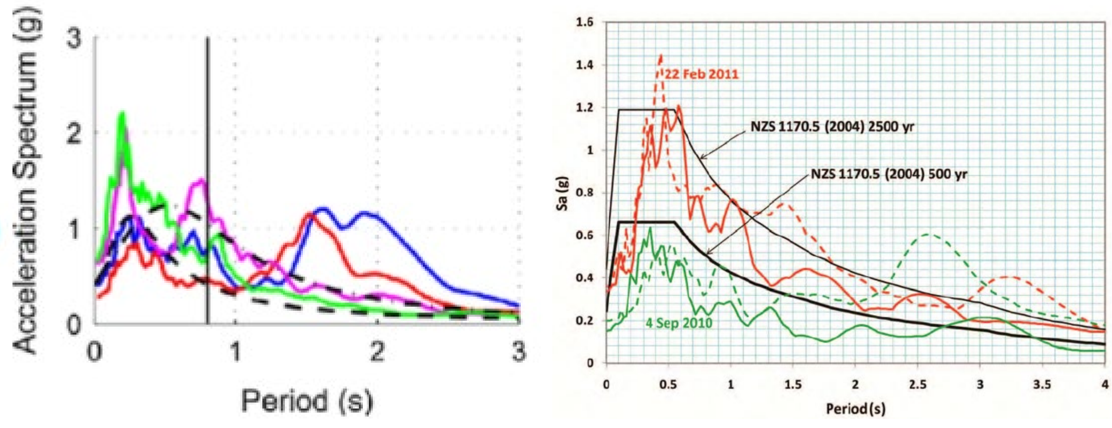


Figure 2: Effect of base isolation in soft soil (adopted from Barmo et al. 2015)

Second assumption relies on the future earthquakes have an accelerations spectra similar in shape to that shown in Figure 1.0 because the efficiency of period lengthening relies on the stylized spectral shape. Figure 3 depicts the response spectra for the Maule earthquake, 2010 in Chile and two of the main Canterbury earthquakes that occurred on 10th September 2010 and 22nd February, 2011. It can be clearly seen that these earthquakes do not follow that spectral shape. For e.g., the September 2010 earthquake had spectral components well above the 2500-year return period spectra around 2.5 to 3 seconds, which is the typical first mode period of a base isolated structure. So, there is a high probability that similar benefit would not be obtained for a base isolated system and in certain cases it may even be worse.



(a) (b)

Figure 3: Spectral plots for Maule 2010 earthquake (a) and 2010& 2011 Canterbury earthquakes (b).

4.2 The ugly

For the successful performance of base isolation, there should be enough clearance to accommodate the rigid body movement of the superstructure and that the available travel of the isolator will not be exceeded. There should also not be a risk of pounding between the structure and adjacent structures. In order to make a realistic estimate of the movement space required, we need to theoretically have a good estimate of the absolute displacement of the ground in the future earthquake. For a site which is predominantly subjected to far-field earthquakes which produces relatively small ground displacements (10-20 cms) this maybe more or less easy to establish; but for a site which is subjected to near-fault events which produces relatively large ground displacements (>1-3m), it is very difficult to arrive at a realistic movement space.

Hall (1995) studied the effects of base isolated structures on near-fault events and recorded the fact that base isolated structures experienced severe nonlinear behaviour and even there is potential for collapse. Through numerical study he showed that base-isolated structures exhibited drifts of the order of 5% when subjected to near-fault event like Lucerne 1993 (M_w=7.2) in which the ground displacement was of the order of 2.5m at 1.5 m/s (5 times the velocity of a normal earthquake). Similar results are obtained by the authors and presented in section 5.1.2.

In New Zealand, the Wellington CBD is a region where we can expect these levels of large ground displacements mainly due to the closeness of the CBD region to the Wellington fault. As outlined in NZS 1170.5 commentary, Wellington fault can generate velocities and ground displacements similar to or higher than that generated by Lucerne 1993. As the epistemic and aleatoric uncertainties of the ground displacement and velocity of a future earthquake emanating from the Wellington fault is extremely high, it would be prudent to provide larger movement space for buildings base isolated in Wellington CBD region accounting for these uncertainties.

5 NUMERICAL STUDY

A seven storey RC structure is used for the numerical study adopted from Dutta 2010. Figure 4.0a shows the geometric details of the structure and figure 4.0b shows the backbone of the isolator. This section mainly presents the results of the preliminary study which is completed on which further detailed investigation is progressing; nevertheless, some interesting observations are recorded so that the engineering community is aware about the *pros* and *cons* of adopting a base isolation scheme.

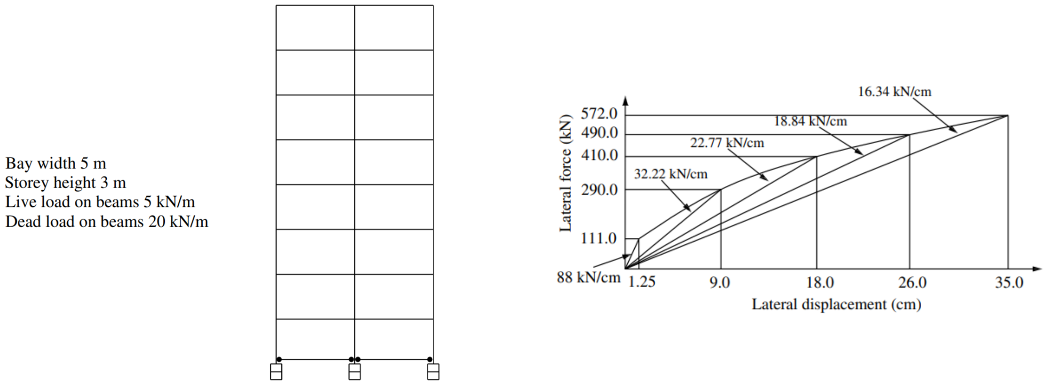


Figure 4: Geometric details of the structure (a); backbone of the isolator (b) (adopted from Dutta 2010).

Table 1.0 shows the details of the structure. Figure 4.0 also give the mass distribution adopted for the present study.

Table 1: cross section properties of the structure

Section	**Breadth x Depth(mm)**
Interior	300 x 500
Exterior	500 x 500

The fixed based elastic period of the structure is 0.9 secs, and the isolator is sized so that the inelastic secant period is around 2.6 seconds with the isolator displacement of 300mm. The elastic period of the structural system with the isolator incorporated is 1.0 second. The ground motions were selected from both shallow and deep soil suites specified for the Wellington region by Oyarzo et al. (2012) along with Mexican and Christchurch earthquakes added. The scaling is applied for a 500-year return period earthquake with a S_p=1.0.

5.1 Results and discussions

5.1.1 With no limit on the clearance for the isolator

This section consolidates the key results of the study with no limits on the movement space so that the isolated building will never impact on other structures and the base isolator travel is unlimited. It must be noted that this is only a theoretical case and in practice the available space for movement of the isolator would be limited. Section 5.1.2 presents the preliminary result of what happens if there is limit on the clearance. Only global responses in terms of roof displacements are plotted to get a qualitative understanding of the effect of base isolation in comparison to fixed base systems. Two cases are studied, one in which damping at the isolator level is 5% and the other one in which the damping at the isolator level is 15%. The 15% level damping is representative of adding additional damping in the isolator level whereas the 5% case represents the energy dissipation of the isolator in its elastic range.

Before delving into the results, some important observations on the chosen numerical example are recorded here:

- No soil structure interaction is considered in the present study.
- No vertical or rotational ground motions are considered in the present study. One of the features of commonly available isolation systems is that they can provide no isolation against vertical excitations, and for some, structural overturning effects may seriously impact on the behaviour of the devices.
- The results of the present preliminary study should only be considered as indicative, mainly because the chosen example is a simple 2D structure. In a 3D structure, more complex inelastic dynamic interactions will likely occur which are not covered in the present study and will be a function of the specific structure and location under consideration.
- Ground motions used for the present study are chosen based on different characteristics, like forward directivity, soft soil, unusual spectral shapes etc. The main purpose for this is to just show that engineers should be very careful when choosing base isolation as their default solution for low damage schemes, largely because unless these aspects are explicitly addressed, base-isolation may not provide a resilient solution.
- Due to space limitations, time history responses are only plotted for 15% damping (scenario with additional damping at isolator level), but key results of the worst response with the 5% are recorded in the text
- The results of a study with a movement stopper to represent the running out of the movement space for isolator are also presented.

5.1.1.1 The good

Figure 5a shows the roof displacement response of the 15% damped base isolated structure for the 1940 El Centro ground motion. Figure 5b shows the zoomed in plot. Both roof displacement and base displacement is less than the fixed base structure. Similar results are observed for the 5% damped structure as well. No beams or columns incur inelastic excursions limiting any damage in the 15% damping case and minor yielding is observed in the 5% damped case. El Centro 1940 is a very typical cyclic type of earthquake which has been used in classical literature and fits well within the stylised pattern of the classical acceleration spectrum. In a general sense, the probability of getting these type of ground motions which match the classical spectrum is high, except in some specific locations such as places like Wellington and similarly highly seismic locations around the world where the future ground motions have a likelihood of violating the globally adopted shape

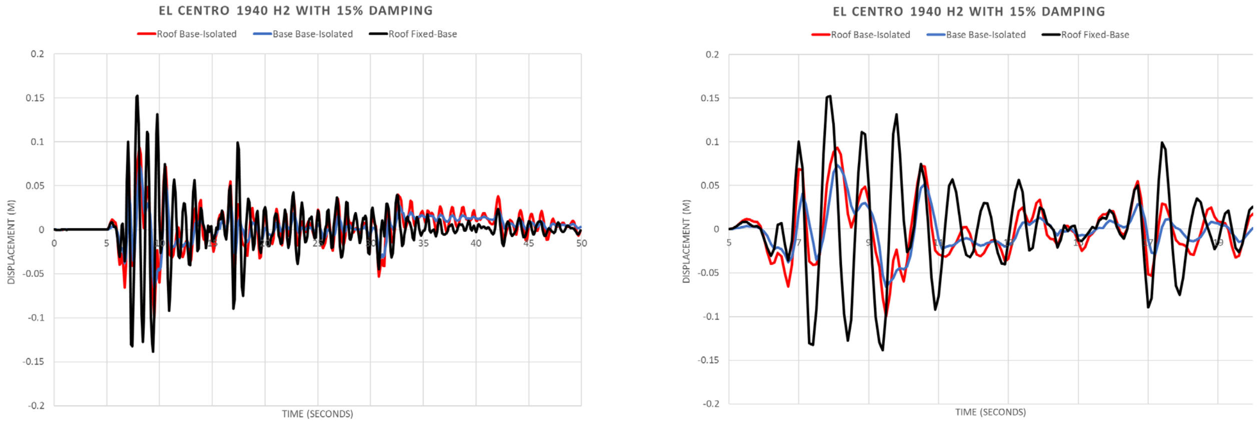


Figure 5: a) Roof displacement for El Centro 1940 with 15% base damping; b) Zoomed in plot

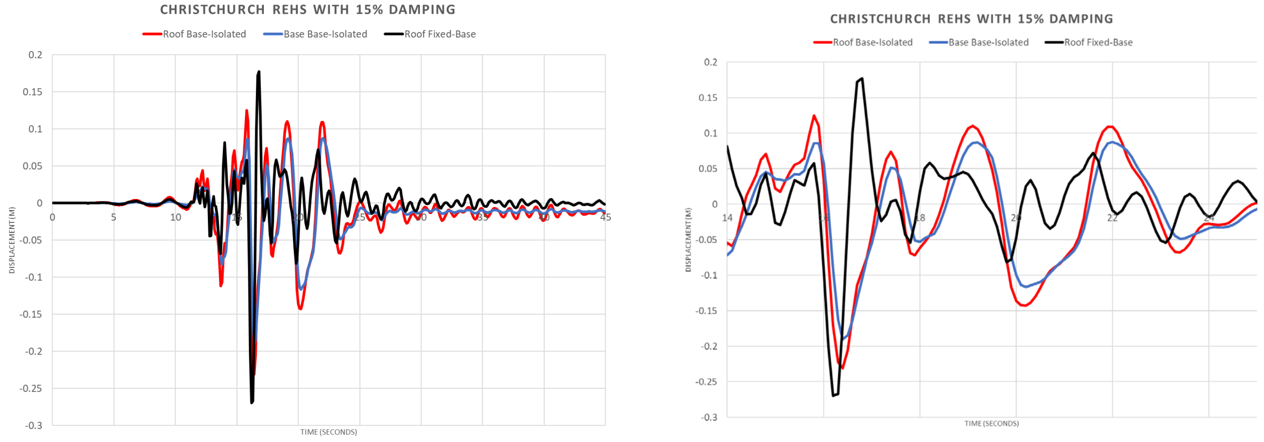
of the acceleration spectrum. Seismic engineering is still in its learning phase with regards to these types of ground motions.

5.1.1.2 The bad

In this section, first, the case with 15% damping is described followed by the system with 5% damping at the isolator level.

- *15% damped case*

In this section, the roof displacement response of 15% isolator damped system for the Christchurch earthquake with a bump in the acceleration spectrum is shown. In other words, this section mainly outlines the performance of the system in a ground motion with an unusual spectral shape. The expected level of response reduction is not obtained as shown in figure 6a. A zoomed in plot is shown in figure 6b and can be clearly seen that the response is not much reduced as expected from a base isolation scheme in comparison to Figure 5b. This clearly shows the excitation dependency of the base isolation scheme; in other words, how important it is for the future earthquake spectra to follow the shape of the design spectra.



(a) (b)

Figure 6: a) Roof displacement for Christchurch REHS recording with 15% base damping; b) zoomed in plot of the response.

Similar responses are shown by the 1985 Mexican city earthquake as well. Figures 7a and 7b shows the roof displacement response to the Mexican earthquake. It can be clearly seen that the full expected benefit is not obtained in the system by base isolating. As indicated by Andriono and Carr (1991) the consequences of this type of acceleration spectra reduces the benefit of base isolation but the energy dissipated by the isolator does limit the displacement and there is still some benefit from the isolation strategy. In this particular case the inter-storey drifts are still much less than those of the un-isolated structure.

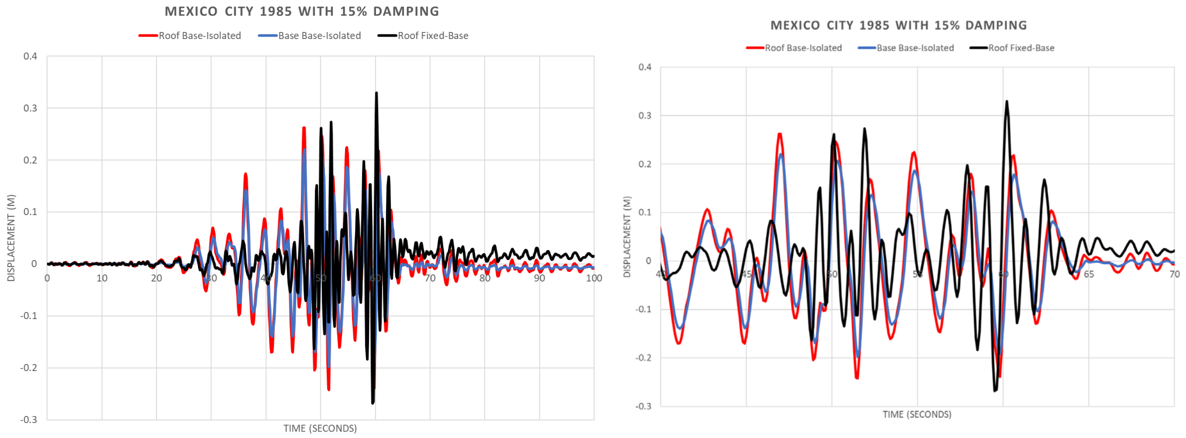


Figure 7: a) Roof displacement for Mexican earthquake with 15% base damping; b) zoomed in plot of the response.

As mentioned in the previous section, the probability of these type of unusual earthquakes is more likely in high risk seismic regions such as Wellington. Therefore, it is important that, if base-isolation is provided for such regions, sufficient additional damping needs to be provided. The damper force required for the present case study building is of similar magnitude to the isolator force. It must be noted, however, that the base isolation with sufficient base damping subjected to these earthquakes with unusual spectral shape does significantly reduce the level of inelasticity in the superstructure when compared with the fixed-base case.

One more interesting thing to note is that even the 15% damping at the base is not sufficient for reducing pounding issues as evidenced by figure 6a & 6b.

- *5% damped case*

The structure with 5% damping at the isolator level shows responses worse than the 15% damping attributed system Figures 6 and 7. This signifies the need to professionally design the required damping in the system to account for minimizing uncontrolled responses during a future event where the acceleration response spectra is different from the shape of the design spectra.

5.1.1.3 The ugly

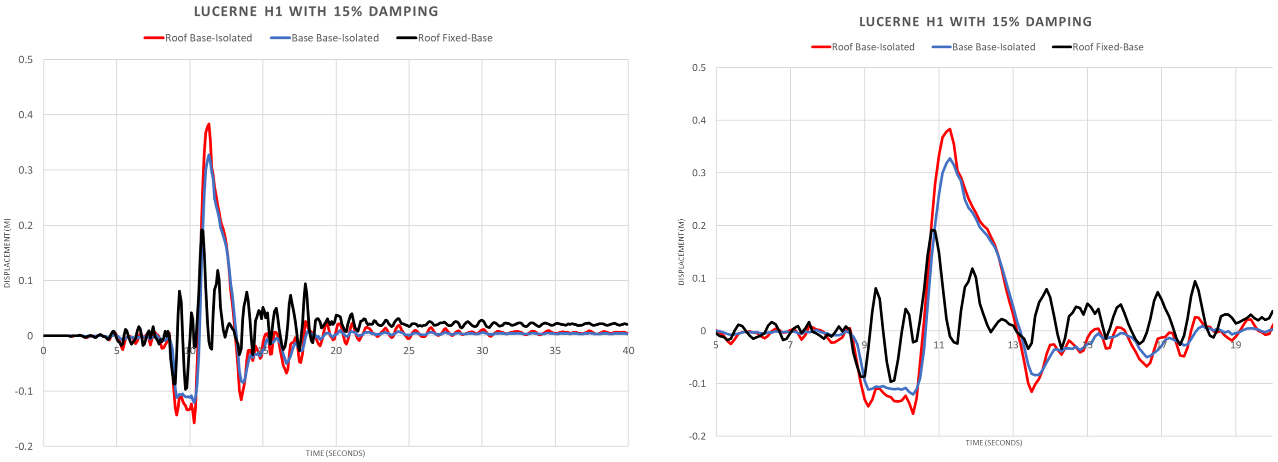


Figure 8:a) Roof displacement for Lucerne earthquake with 15% base damping; b) zoomed in plot of the response.

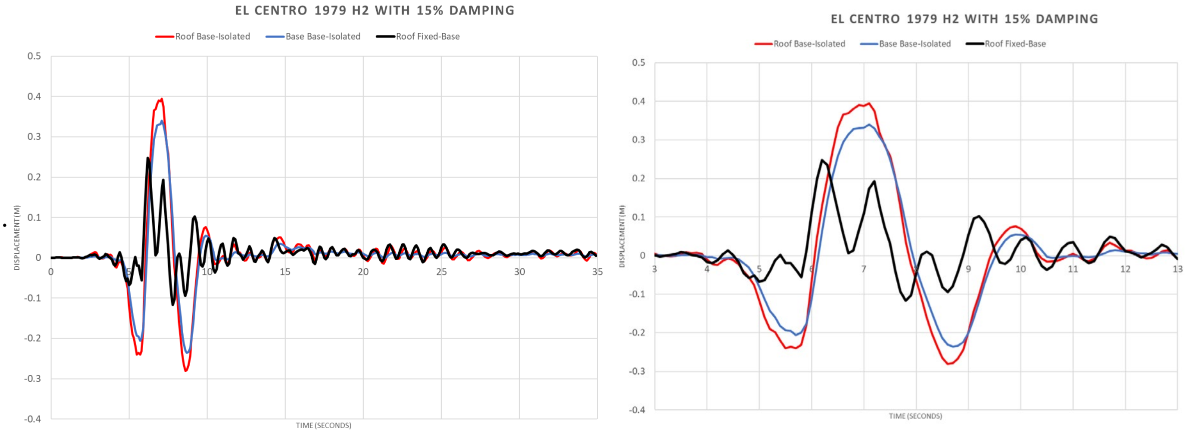


Figure 9:a) Roof displacement for El Centro 1979 earthquake with 15% base damping; b) zoomed in plot of the response.

- *15% damped case*

This section mainly refers to the response of structures with base-isolation system subjected to near-fault events with a forward directivity type pulse which is characteristic of a Wellington scenario.

Two earthquake records, Lucerne 1994 and El Centro 1979 are used for this study. Figures 8-9 represent the roof displacements and zoomed in plots of the roof displacement response. It can be clearly seen that these type of earthquakes causes large roof displacements for base-isolated structures and which are much larger than those corresponding to the fixed base structure. Around 30% of the structural elements appear to yield with minor ductility. The plots are for 15% damping.

- *5% damped case*

With the base isolator damping at 5% damping the roof displacements are of the order 650mm and the base displacement is of the order of 500 mm. More than 60% of the structural elements incur inelastic excursions and though not shown, much higher inter-storey drifts are observed.

5.1.2 With limits on the "movement space" for the isolator (practical scenario)

As shown in the previous section, the near fault earthquakes produce large displacements at the base. In a practical situation, the movement space available for the isolator is limited due to site constraints. So, to test the effect of running out of travel for the isolators and to study the effect of the isolator hitting the limit, a bi-linear gap element of three times the isolator stiffness was introduced. The gap was set at 300mm. It was seen that once the isolator hits this limit, shock waves travelled up the height of the building. All the elements incurred inelastic actions with high curvature ductility, of the order of greater than 20. The peak floor accelerations were of the order of >2.5g with peak inter-storey drifts >3%. It is interesting to note that similar results were obtained by Hall (1995).

5.2 Critical observations

This section consolidates some of the critical observations of the following study:

- It must be noted that base isolation is *a highly effective low damage solution* when the incoming ground motions follow the stylised spectra where response reduces with increasing system period.
- If the ground motion does not follow the stylised spectra or shows "anomalies" like a bump etc., it seems likely that base isolation may be less effective in damage control. Based on the present study, the system exhibits larger damage due to larger inelastic excursions and depending on the system the overall benefit might or might not seem to be very appealing when compared with a similar fixed-base system. Future more extensive studies presently progressing will confirm it in detail.
- If there is likelihood of a near-fault forward directivity type ground motion, the choice of base isolation should be made with utmost care as discussed in 5.1.1.3 and 5.1.2.
- In the present study, it is worth mentioning that the present ground motion scaling is only done for a 500-year return period earthquake. So, when there is likelihood of a near fault type motion, in order to reduce the uncertainty on the required movement space, it would be prudent to at least consider an earthquake with >2500-year return period. This requirement is exemplified in section 5.1.2. To simply get a qualitative outlook on this, for e.g., for the case study building, ignoring the soil-structure interaction, it might at least require a minimum of >1m movement space. If SSI is incorporated in the study, authors believe that a larger movement space (>>1 m) might be needed depending on the degree of soil compliance.
- When deciding the movement space for near-fault type events, it would be prudent to consider the soil-structure interaction phenomenon due to reasons highlighted in the previous point.
- It must be noted that when base-isolation is put on soft soil or soil with reasonable compliance, there is a chance of the combined soil-isolator-structure system reaching in resonance. In the present study the resonance effect of base-isolation due to soil structure interaction is not investigated. Future studies will address this aspect.
- The present study involves a simple 2D structure, but all structures are subjected to 3D bi-directional effects. So, when 3D design of a structure is done in a region where anomalous ground motions may happen or places subject to near-fault scenarios, the whole design should be done with utmost care and should be supplemented with a design de-sensitization process where all epistemic and aleatoric uncertainties are accounted for in the design. Utmost care will be required if a high degree of resilience is the objective.
- It must be remembered that earthquake engineering is still in a learning phase as far as anomalous ground motions or near-fault scenarios are considered. So, sufficient care should be taken in designing base isolation system which relies predominantly on the period shift characteristics to minimize the earthquake energy input into the system.

6 CONCLUSIONS

A preliminary investigation into the advantages and disadvantages of base-isolation technology with regards to its applications in various types of buildings founded in different soil types and locations are presented. It has been shown that on good soil and when the future earthquakes follow the design spectra shape, base isolation is a highly effective technology, but on soft soils and sites with a risk of near-fault effects this technology should be used with utmost care and might not give the same benefits as presently believed. For base-isolated structures located on sites subjected to near-fault scenarios it is considered a must that the designer arrives at the estimated movement space considering the undesirable effects of impact with surrounding structures, limits on isolator travel and while doing so considers appropriate soil definition in the design-analytical model. This is particularly important to ensure that the attributed resilience of this structural system is not overstated.

6.1 References

Andriono, T., Carr A.J. 1991. Reduction and distribution of lateral seismic inertia forces on base isolated multi-storey structures, Bulletin of NZ Society of Earthquake Engineering., 24,3, 225-237

Barmo A., Mualla, I.H., Hasan, H.T.,2015. The behavior of multi-story buildings seismically isolated system hybrid isolation(Friction, rubber and with the addition of rotational friction dampers)., Open journal of Earthquake Research,4,1-3

Filiatrault, A., Cherry S., and Byrne, P.M., 1990. "The Influence of Mexico City Soils on the Seismic Performance of Friction Damped and Base Isolated Structures", Earthquake Spectra, 6(2), 335-352

Hall, J.F. 1995. Near-Source ground motions and its effects on flexible buildings, Earthquake Spectra, Vol 11, No.4

Luco, J.E. 2014. Effects of soil-structure on seismic base isolation, Soil Dynamics and Earthquake Engineering,66:167-177

Oyarzo C., Mc Verry,G.,& Ingham J.2012. Seismic zonation and default suite of ground motion records for time-history analysis in the North Island of New Zealand. Earthquake Spectra, 28(2), 667–688.

Pampanin. S. 2015. Towards the "ultimate earthquake-proof" building: development of an integrated low-damage system: 321-358. http://dx.doi.org/10.1007/978-3-319-16964-4_13.

Wada, A., Huang,Y., & Bertero,V.V., 2004. Innovative Strategies in Earthquake Engineering. Bozorgnia, Y., Bertero, V.V.,(Ed), Earthquake Engineering, From Engineering Seismology to Performance-Based Engineering, (pp. 10-1-10-33). CRC Press, London.