Benefits of Vertically Distributed Isolation Devices for an 8-storey Structure with Complex Geometry

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SUMMARY:
The construction of structures with curved or irregular geometry is becoming increasingly common as modern architects strive to create iconic buildings. However, the realization of complex building forms creates significant challenges in regions of high seismicity, since this introduces significant uncertainties in the likely plastic mechanism, with questions on the likely performance of connections and doubts about the probable loadpaths. This paper examines the benefits of a novel seismic design strategy that uses isolation devices distributed vertically up a building between the exterior structure (of complex-geometry) and the heavy internal core. An innovative Direct DBD methodology is used to design an 8-storey case study building in which visco-elastic devices are used. Non-linear time-history (NLTH) analyses are then conducted on a 3-dimensional model of the building and the results indicate that the target displacement and drifts for both the internal and external structures are well controlled by the design solution.

Keywords: complex geometry, diagrid, displacement based design, visco-elastic damper.

1. INTRODUCTION

The realisation of structures with curved or irregular geometry is becoming increasingly common, with iconic buildings such as Guggenheim in Bilbao, the Seattle public library, the Birdsnest Stadium in Beijing, and the Gherkin in London, all distinguishing themselves by their unusual form and composition. This growing tendency to propose complex building forms creates significant challenges for structural engineers in regions of high seismicity, where it is highly recommended that structures are regular in form, with clearly established loadpaths and plastic mechanisms, in order to provide reliable seismic performance. If a structure possesses an unusual geometrical form, this introduces significant uncertainties in the likely plastic mechanism, with questions on the likely performance of connections and doubts about the probable loadpaths. Recently, Lago *et al*. (2010) undertook preliminary testing of a novel seismic design strategy for structures with complex geometry, that aims to achieve reliable seismic behaviour through the introduction of special seismic isolation devices vertically distributed up the height of the building. This paper extends on the work undertaken in Lago *et al*. (2010) by considering the 3D response of an 8-storey building with vertically distributed isolation devices. The structure is designed using a Direct displacement-based design (DBD) approach (Priestley et al. 2007) and non-linear time-history (NLTH) analyses are undertaken with bi-directional excitation in order to gauge the performance of the innovative structural scheme.

2. THE CONCEPT OF VERTICALLY DISTRIBUTED ISOLATION DEVICES

The seismic response of a building will be affected by its mass, stiffness, strength and deformation capacity. The proposed concept of vertically distributed isolation devices is to isolate a stiff, light exterior structure, which will have limited strength and inelastic deformation capacity, from the heavy mass at a building's core. One possible means of doing this is illustrated in Fig. 2.1, where a curved exterior structure is connected to a relatively regular, heavy interior structure through the use of special seismic isolation devices. The internal columns and beams would be detailed to resist gravity loads and undergo seismic movements without developing significant resistance, whereas the core walls and floors would be used as main lateral load resisting structural elements, together with the exterior structure. The isolation devices are sized to control the transfer of inertia forces from the core and limit the displacements imposed on the exterior structure. The use of the term "isolation" is not completely appropriate since both the exterior and the interior structures are directly connected to the ground and are therefore not base isolated. However, the term isolation is used to reflect the partial isolation of the complex exterior structure from the heavy internal portion of the building.

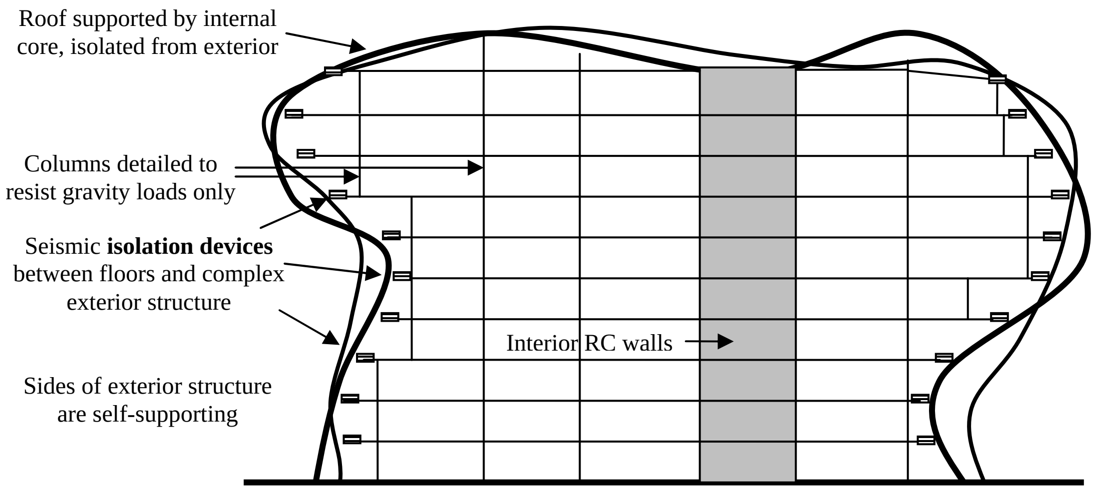


Figure 2.1. Vertically distributed isolation concept for a building with complex geometry

If very stiff and strong devices are used then the exterior structure will displace together with the interior structure, whereas if very flexible or low-strength devices are used then the exterior structure's displacements will be very small but the zone between the floor slabs and the exterior structure will have to sustain larger differential displacements. These concepts can be useful for optimising the properties of the seismic devices.

Lago *et al*. (2010) investigated the use of viscoelastic devices, which transmit both displacement- and velocity-dependent forces. Figure 2.2 depicts the relative displacements of the internal and external systems at two different instants of structural motion: (a) at peak core displacement and (b) peak core velocity (Fig. 2.2(a) and (b) respectively). By using viscoelastic devices, forces are transmitted to the external structure when the internal system velocity is a maximum (due to the velocity dependent (damping) characteristics of the devices) and when the internal system displacement is a maximum (due to the stiffness characteristics of the devices). This is beneficial since the relative displacements between the external structure can be limited but effective isolation can still be obtained. If, say, viscous damping devices were used instead of viscoelastic devices, the external system is not displaced when the core reaches displacement, implying that the connection between the exterior and the core will require detailing for much larger relative displacements.

Despite this apparent advantage of viscoelastic dampers compared to viscous dampers, there are some drawbacks related with these devices, since they have a temperature dependent behaviour and they are prone to creep when subjected to permanent loads (see Christopoulos and Filiatrault (2006) for further details). Temperature dependence studies are outside the scope of this work and it should be part of future research works. Instead, as discussed in Lago et al. (2010), in order to avoid creep problems for the type of structural system considered here, one could consider two different solutions: (i) a self-supporting exterior structure that does not impose gravity loads on the devices, or (ii) use of devices realised with multiple elements arranged to provide the equivalent of viscoelastic behaviour with, for example, the use of viscous dampers in parallel with flexible plates or beams.

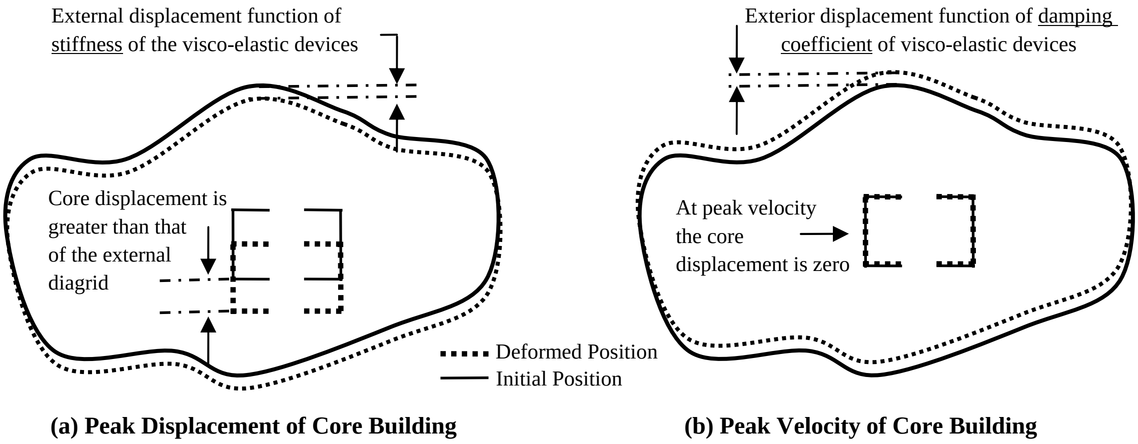


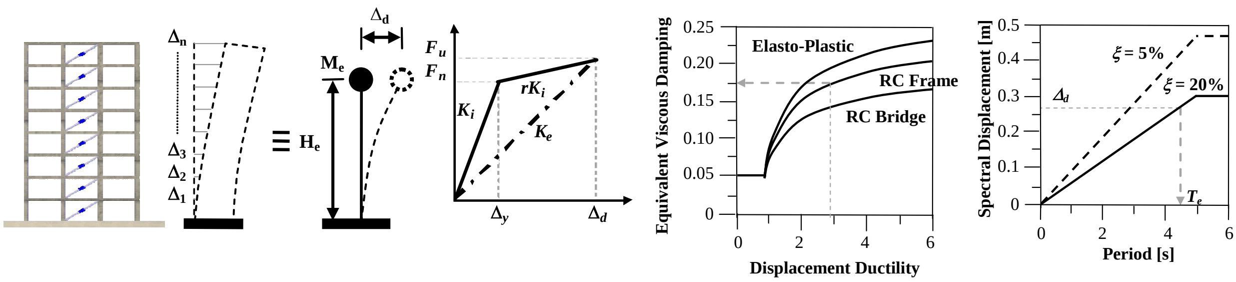
Figure 2.2. Plan view of a structural system at (a) peak displacement of core and (b) peak velocity of core when complex exterior structure is vertically isolated with visco-elastic devices (modified from Lago et al. 2010).

With the above in mind, it is considered that the system merits some development including the identification of a design methodology that can properly account for the non-linear response of the core together with the damping offered by the visco-elastic devices.

2.1. Design Procedure

One of the main difficulties faced in the design of structures with complex geometry is that every structure is likely to be different and as such, it is difficult (if not impossible) to assign rational behaviour factors or provide other general design guidelines for such systems. However, by adopting the vertical isolation strategy the level of complexity can be drastically reduced, since the non-linear behaviour can be designed to occur only in the ductile core structure. As such, what is required is a seismic design procedure that can properly account for the non-linear response of the core together with the damping and stiffness offered by the isolation devices and exterior structure. This section will briefly review a Direct DBD procedure (from Sullivan (2009) and Lago et al. 2010) that can account for these characteristics. In section 3 the method will be applied to a case study structure in order to verify its ability of controlling the structural response under bi-directional earthquake excitation.

The fundamentals of the Direct DBD from Priestley et al. (2007) are illustrated in Figure 2.3. The method utilises the substitute structure approach (see Shibata and Sozen, 1974) to characterize the non-linear response of a MDOF system with an equivalent SDOF system characterized by a secant stiffness K_e, and effective mass, m_e, together with a ductility dependent equivalent viscous damping value. These concepts are illustrated in Fig.2.3a to 2.3c.



(a) SDOF representation

(b) Effective stiffness, K_{eff}

(c) Equivalent Viscous Damping

(d) Displacement Spectrum

Figure 2.3. Fundamentals of DDBD (adapted from Priestley *et al.* [2007])

For the Direct DBD of complex geometry systems with vertically distributed isolation devices, one must first identify performance criteria which could include peak storey drift limits, strain limits of structural elements or residual deformation limits (see Sullivan et al. 2012). An expected displaced shape at peak response is then identified and the desired viscous damping level for the system is selected. The displaced shape of systems such as that shown in Fig. 2.1 can be set assuming that the walls will control the deformed shape (see Lago *et al*. (2010) for further discussion of this assumption). Consequently, for walls with aspect ratio (height divided by length) greater than 3.0, the design displacement profile is defined according to Eqn.2.1.

$$\Delta_i = \frac{\phi_{yW}}{2} h_i^2 \left(1 - \frac{h_i}{3H_n}\right) + \theta_p h_i \tag{2.1}$$

where h_i is the height of level i above the wall base, H_n is the total height, θ_p is the design plastic rotation of the wall base hinge and ϕ_{yW} is the wall yield curvature that can be obtained from sectional moment-curvature analysis or using approximate expressions provided in Priestley et al. (2007) and Sullivan et al. (2012). The design plastic rotation can be found from:

$$\theta_p = \theta_c - \frac{\phi_{yW} H_n}{2} \le \left(\phi_{ls} - \phi_{yW}\right) L_p \tag{2.2}$$

where θ_c is the code design storey drift (to limit damage to non-structural elements), ϕ_{ls} is the wall curvature limit for the design limit state and L_p is the length of the base plastic hinge in the walls (see Priestley et al. 2007 or Sullivan et al. 2012 for further details).

Knowing the displacement profile of the system the equivalent single degree of freedom properties of design displacement, Δ_d, effective height, H_e and effective mass, m_e, are computed as:

$$\Delta_d = \frac{\sum m_i \Delta_i^2}{\sum m_i \Delta_i} \tag{2.3}$$

$$H_e = \frac{\sum m_i \Delta_i h_i}{\sum m_i \Delta_i} \tag{2.4}$$

$$m_e = \frac{\sum m_i \Delta_i}{\Delta_d} \tag{2.5}$$

where Δ_i is the design displacement, m_i is the seismic mass, and h_i is the height, of level i.

To proceed with the design it is proposed that a desired level of system viscous damping, ξ_{sys} be selected based on engineering judgement (a value of between 20% and 30% can typically be effective). Having obtained the design displacement from Eqn.2.3 and having selected the design equivalent viscous damping, a highly damped design displacement spectrum should be obtained and an effective period identified, as illustrated in Figure 2.3d. There are many different rules in the literature to construct highly damped spectra and in this work the following expression is used in which the highly damped displacement spectrum, $S_{d,\xi}$, is obtained from the 5% elastic spectral displacement demands, $S_{d,5\%}$ as a function of the design value of the equivalent viscous damping ξ:

$$S_{d,\xi} = S_{d,5\%} \left(\frac{10}{5+\xi}\right)^{0.5} \tag{2.6}$$

With the required effective period known, it is possible to compute the necessary effective stiffness, K_e, and base shear, V_b, as follows:

$$K_e = 4\pi^2 \frac{m_e}{T_e^2} \rightarrow V_b = K_e \Delta_d \tag{2.7}$$

Note that, for simplicity, the above expression does not include an allowance for p-delta effects and interested readers should refer to Sullivan et al. (2012) for such detail.

With the design base shear established, the member strengths and the properties of isolation devices should be set in order to respect the initially assumed system damping value. To facilitate this, it is helpful to firstly choose the proportions of lateral load that will be resisted by the internal core and the exterior structure. As the exterior structure is of complex geometry and should remain in the elastic range, elastic analyses of the exterior system could be conducted to identify a safe design level of lateral force. Such elastic analyses should involve application of a set of equivalent lateral forces, F_i, distributed according to:

$$F_{e,i} = V_{Ext} \frac{m_i \Delta_i}{\sum_i m_i \Delta_i} \tag{2.8}$$

where m_i and Δ_i are respectively the mass and the displacement of floor *i*. A unit base shear can be used at first for V_{Ext} so that the ratio of the design force to the design resistance in all elements of the external frame can be computed. The maximum ratio will indicate the critical element and can also be divided into the unit base shear in order to estimate the maximum allowable transfer force, $V_{Ext,max}$. However, in practice one should aim to incorporate some margin of safety by adopting a lower transfer force than $V_{Ext,max}$, considering that higher mode effects, seismic excitation of the frame itself and capacity design considerations will tend to increase the demands on the exterior frames.

The safe transfer force could be considered a limit to both the velocity-dependent and stiffness-dependent forces in the isolation devices. To facilitate a direct DBD solution, the ratio of the stiffness-dependent transfer force to the total strain energy will be denoted using the symbol ψ as recommended by Lago et al. (2012). Assuming that the dampers and the structure have the same design displacement (which should be valid when dampers are provided at all floors) the proportion can be referred in terms of the total base shear instead of the total strain energy, such that:

$$V_{Tot} = V_{Stru} + V_{Ext} \rightarrow V_{Stru} = V_{Tot}(1-\psi) \tag{2.9}$$

where the total base shear at maximum displacement, V_{Tot}, is carried by the core structure, V_{Stru}, and by the elastic stiffness-dependent forces in the dampers, set equal to V_{Ext}.

At maximum velocity (zero displacement) it is useful to expression the proportion of total base shear carried by the viscous part of the dampers as β, such that:

$$V_{FD} = \beta V_{Tot} \tag{2.10}$$

The proportions indicated in Eqns. 2.9 and 2.10 are useful for the calculation of the system damping, as shown by the following equation (see Lago *et al*., 2012 for detailed derivation):

$$\xi_{sys} = \frac{\sum F_D \Delta_D}{2V_{Tot}\Delta_e} = (1-\psi)\xi_{Stru} + \psi\xi_{Ext} + \frac{\beta}{2} \tag{2.11}$$

where ξ_{Stru}, is the structural equivalent viscous damping constant of the system, given by the sum of the inherent damping and the inelastic structural damping (if the main structural system is entering its inelastic range); ξ_D, is the damper equivalent viscous damping value offered by the devices; ξ_{Ext}, is the external structure damping; and V_{Ext}, is the base shear carried by the external structural system (that is the same as the elastic portion carried by the viscoelastic dampers $V_{Ext} = V_{FD,el}$).

If the velocity-dependent shear obtained in this way is greater than the frame design limit (i.e. V_{Ext}), it is necessary to reduce the system damping value (to permit a reduction in the strain energy proportion β). Another solution instead can be to upgrade the design the external frame to carry the force imparted by the viscous part of the damper and repeat the design.

The velocity-dependent base shear obtained from Eqn. (2.2) is then used to find a set of floor damping forces, $F_{d,i}$, distributed using a modified form of Eqn. (2.3) in which the differential displacement profile between the core structure and the external frame, Δ_i-$\delta_{fr,i}$, replaces the design displacement of each level, Δ_i. With the required damping forces known, the relative damping constant, C_i, at each level can be determined with the following expression:

$$C_i = \frac{T_e F_{d,i}}{2\pi\left(\Delta_i - \delta_{fr,i}\right)} = \frac{T_e F_{d,i}}{2\pi\Delta_{damper,i}} \tag{2.3}$$

where $\delta_{fr,i}$ is the frame displacement at level i and $\Delta_{damper,i}$ is the damper displacement for level i. The damper displacement as utilized in Eqn. (2.3) assumes that between the external frame and the internal core the displacements are in phase. However, the response is out of phase and consequently it should be determined through the following expression from Lago *et al*., (2012):

$$\Delta_{damper,i} = \left| \Delta_i \sin\left[-\mathrm{atan}\left(\frac{\Delta_i}{\delta_{fr,i}}\right)\right] - \delta_{fr,i}\cos\left[-\mathrm{atan}\left(\frac{\Delta_i}{\delta_{fr,i}}\right)\right]\right| \tag{2.4}$$

Finally, the required damper elastic stiffness can be obtained using the elastic forces determined of Eqn. 2.8, as shown in Eqn. 2.5.

$$K_i = \frac{F_{e,i}}{\Delta_{damper,i}} \tag{2.5}$$

Having determined the required properties of the adding damping system the subsequent step is to design the core wall. The base shear carried by the core walls, V_{wall}, can be found directly from the system total base shear as: $V_{wall} = V_b - V_{Ext}$. Furthermore, the overturning moment carried by the walls, M_{wall}, can be calculated as the product of the respective base shears by the effective height, H_e.

At this point the DDBD procedure is complete, since the required strength for each structural system has been established. The next step would be to undertake capacity design of the elements not intended to yield and determine necessary reinforcement quantities. Furthermore, capacity design requirements are needed for the viscoelastic elements but this is outside the scope of this paper and interested readers should refer to Lago *et al*. [2012]. The above reviewed step-wise procedure for complex geometry structures with viscoelastic damping devices is summarized in Fig. 3.

3. APPLICATION TO AN 8-STOREY CASE-STUDY STRUCTURE

The seismic design procedure described in the previous section has been applied by Lago et al. (2010) to the 8-storey structure shown in Fig. 3.1 for 2D response. This paper extends on the work undertaken by Lago *et al*. (2010) to consider the 3D response of the building and a simplified modelling approach.

The structure consists of a curved exterior diagrid structure realised with steel elements, and two C-shape RC walls. A secondary framing system is also present in order to assist in resisting gravity loads. The diagrid designed to be self-supporting under gravity loads, and visco-elastic devices are distributed vertically up the height at the diagrid-floor interface. The seismic design of the building is carried out for a a peak ground acceleration of 0.4g using the soil type C EC8 type 1 spectrum (CEN, 2004) and a design storey drift limit of 2.0%. Spectra of a set of 7 pairs of spectra compatible accelerograms (from Lago et al. 2012) used for the NLTH analyses are shown in Fig. 3.2.

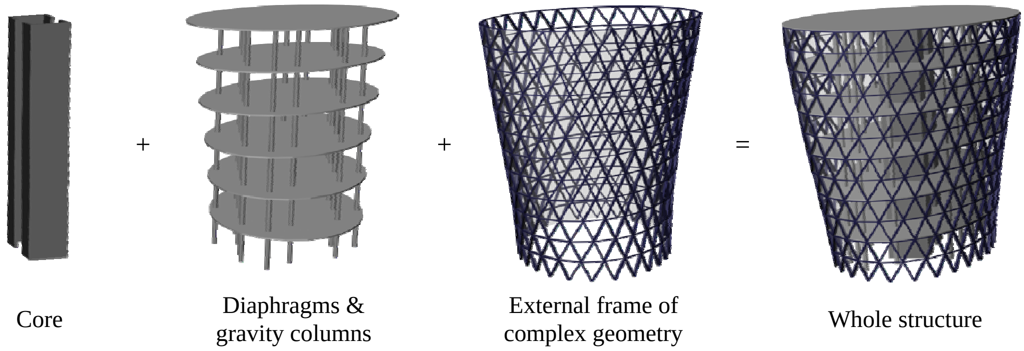


Figure 3.1. Case study structure with core and complex diagrid structural systems (after Lago *et al*., 2010)

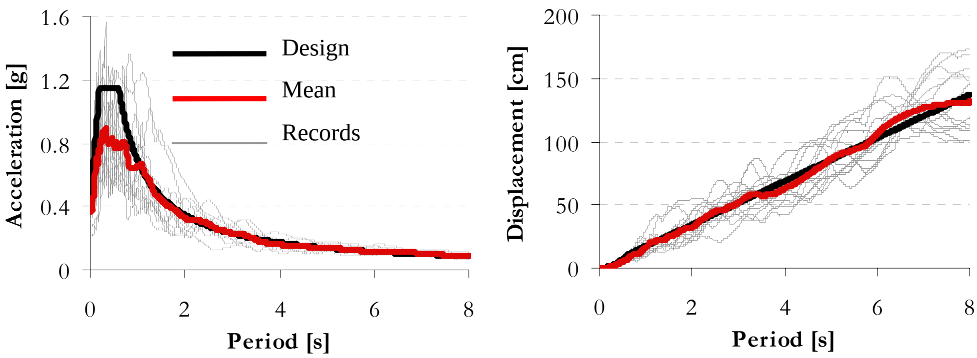


Figure 3.2. Ground motions pairs for NLTHAs: (a) acceleration and (b) displacement spectra (Lago *et al*., 2012)

To adjust the design for 3D effects, the design procedure of Section 3 was applied in the global X and Y directions separately, with torsion effects ignored owing to the relatively concentric mass, stiffness and strength distribution. The damper characteristics were set considering their contribution to the in-plane direction of the diagrid walls. In other words, the damper contribution associated with the out-of-plane response of the diagrid was ignored in the design. The bi-directional characteristics were, however, specified for the non-linear time-history analyses used to gauge the performance of the system, as explained in the next section. Following the above considerations the structure is equipped with four viscous dampers per floor (two per main principal direction as shown in Fig. 3.3).

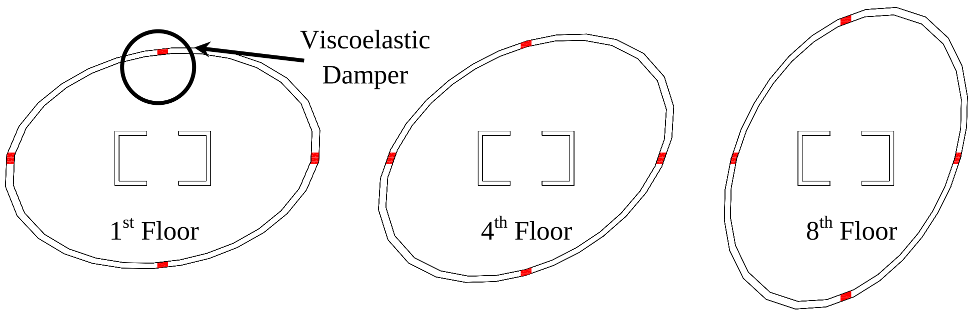


Figure 3.3. Plan views of the case study structure at three levels indicating damper locations (Lago *et al*., 2012)

4. SEISMIC PERFORMANCE OF THE NEW SCHEME

The design of the structural system in both principal directions has lead to define the properties of the viscoelastic devices in both directions. The relative design properties are shown in Table 4.1 and detailed calculations are provided in Lago *et al.* (2012).

Table 4.1. 8-storey design properties for both principal directions

Principal Direction	$M_{n,wall}$ (kNm)	I (m^4)	$C_{Damp.}$ (kNs/m)	K_{Damp} (kN/m)
X	192574	13.16	600	1913
Z	205203	11.94	343	698

The results for the major response parameters are shown for both directions in Fig. 4.1. The figures show that the average NLTHAs results are very close to the design values and the overall structural response is well predicted in terms of displacements and drifts. The situation is slightly different looking at the response in terms of wall shears and moments since higher modes effect amplify the demand. However, good capacity design methods should overcome this issue (see the capacity design requirements for structures with viscoelastic dampers proposed by Lago *et al*. 2012). Similar remarks can be made looking at the response for the isolation devices. The displacement demands are very well predicted but the velocity is underestimated due to differences between the pseudo-velocity and the real spectral velocity and higher modes effect (again, see Lago *et al*. 2012).

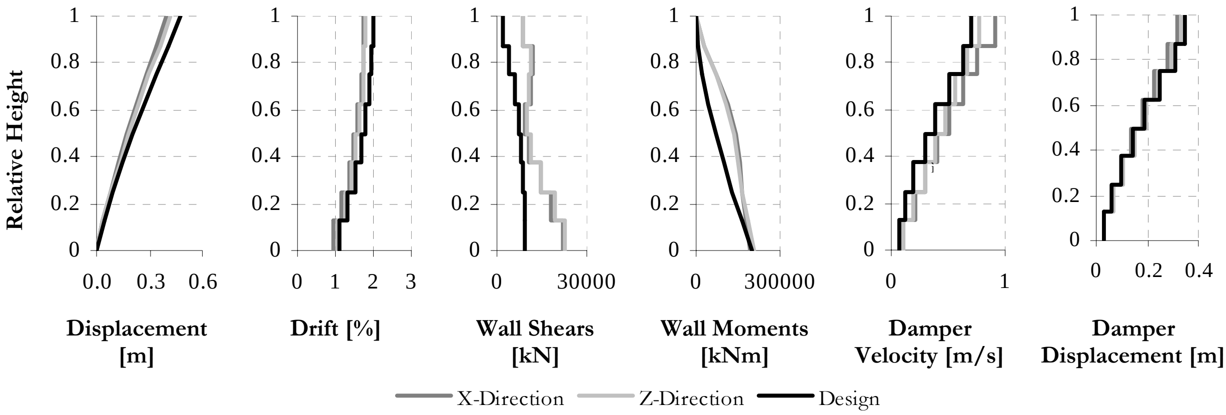


Figure 4.1. Case study structure NLTH results for both principal directions (Lago *et al*., 2012)

In light of these results, one notes that torsional effects were not predominant, illustrating a benefit of the vertical distributed isolation approach. Indeed, the situation is completely different in the case the internal and external system works together with the core system, as shown in Lago *et al*. (2010).

Having demonstrated the validity of the proposed solution for the structural case under consideration in the following section an alternative modelling approach is proposed that can be used as a more direct and easy way to deal with complex geometry structure proposed in this work.

4.1. Simplified Analysis Approach

Due to the modelling difficulties intrinsic with structures of complex geometry, a simplified model is proposed herein that can help for initial stages of design of these structural typologies in order to grasp the capacity of the isolation system to furnish a valuable scheme compared to other structural systems. The idea is to use simple MDOF models that have similar properties of the full complex geometry structure in such way the analyses can be run very quickly (compared with the long analyses of the full model). The proposed solution is shown in Fig. 4.2 where the complex geometry is simplified by a multi-stick model linked to the core by visco-elastic links.

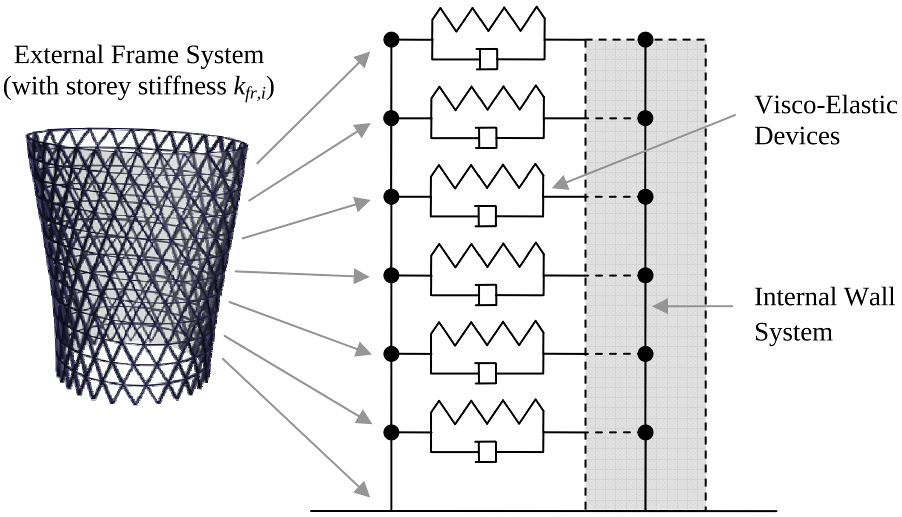


Figure 4.2. Complex geometry simplified model schematic view (Lago *et al*., 2012)

The properties of the external structure, as stick elements, are simply derived from the properties of the real structures (but in the case of preliminary design stages trial values of the stiffness can be defined). For the complex geometry structure considered in this work, the lateral stiffness of the elements is computed through the knowledge of the storey stiffness, $k_{fr,i}$, defined as the ratio of the external frame storey shear, $V_{fr,i}$, to the relative interstory displacement, $\delta_{fr,i}$, as follows:

$$k_{fr,i} = \frac{V_{fr,i}}{\delta_{fr,i}} \tag{4.1}$$

This simplified assumption is only an approximation to the storey stiffness but the NLTHA results presently shortly will illustrate that it is good enough for the sake of grasping the structural capacity of the system.

4.1.1 Analysis of the Simplified Scheme

As explained above, the exterior structure with complex can be simplified through a simply MDOF stick element system whose stiffness is defined by Eqn. (4.1) and for the 8-storey case study structure under analysis (Fig. 3) the properties obtained in this way are reported in Table 4.2.

Table 4.2. 8-storey simplified approach complex geometry equivalent stiffness

Level	EI (kNm^2) X-Direction	EI (kNm^2) Z-Direction
1	328565	20036
2	194605	46416
3	136639	41460
4	179956	51655
5	118735	102151
6	202362	139653
7	280073	201207
8	20398	217450

Knowing the properties of the external frame in the both principal directions and designing the internal core and the dampers with the same procedure described in Section 3, it is possible to gauge the performance of the simplified model for 3D response. The results of NLTH analyses are shown in Fig. 4.3 for the main structural response parameters in both principal directions. The figures show that the response is very close to that of the full geometric model and the same conclusions can be drawn about the design solution. Indeed, the displacements are very well predicted while the forces and damper velocity require the application of capacity design rules (see Lago *et al*. 2012).

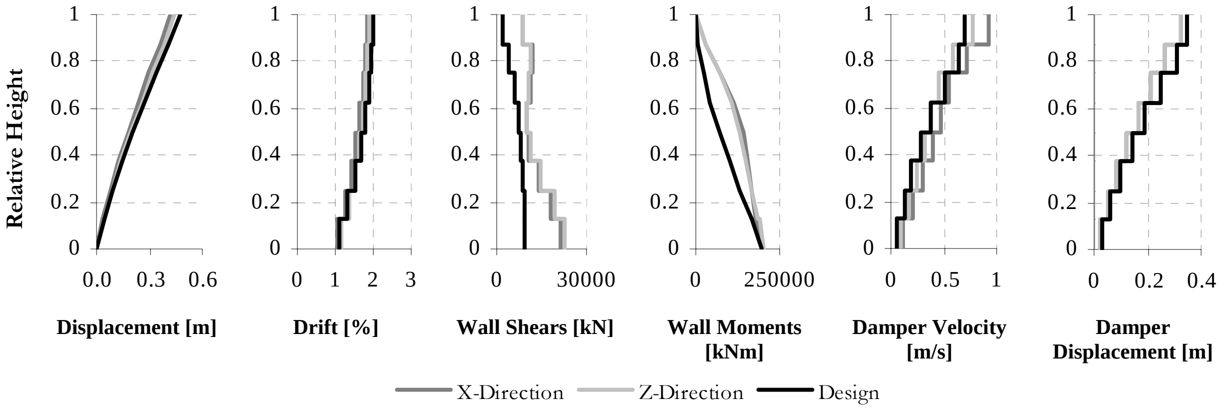


Figure 4.3. Simplified model NLTH results for both principal directions (Lago *et al*., 2012)

5. CONCLUSIONS

A novel seismic design solution for buildings with complex geometries has been reviewed. The innovative solution involves a vertical distribution of damping devices up the height of the complex structures to partially isolate the complex portions of the structure and effectively use the whole building system. A distinct advantage of this approach is that it would be applicable to most structures with complex geometry, provided that the exterior structure is self-supporting under gravity loads.

A new Direct DBD formulation for the systems with vertically distributed isolation devices has been developed, following on from the recommendations of Sullivan (2009) and Lago *et al*. (2010). In order to validate the method for 3D response, an 8-storey complex geometry case study structures has bee designed and subject to NLTHAs using a suite of spectrum-compatible accelerograms. The results confirm that the vertical isolation strategy can be an effective means of designing structures with complex geometry in seismic regions and that the proposed DDBD approach could provide effective control of the response. However, careful capacity design is required, as discussed in Lago *et al*. (2012). Moreover, the results show that the isolation method helps limit the torsional influence of the complex structure. Finally, an alternative simplified modelling approach has been introduced in order to avoid complex structural modelling at concept design stages. The model was subjected to NLTHAs and the results indicate that it could predict a response similar to that of a full geometric model.

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