

# EFFECTIVENESS OF VIBRATION CONTROL BRIDGES IN HIGH-RISE BUILDINGS

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## **ABSTRACT:**

The objective of this research is to investigate the effectiveness of the control bridges in high-rise buildings under seismic loading and, more specifically, how their effectiveness changes with varying critical parameters. Achieving this will involve a numerical analysis of structural dynamics, through the finite element analysis software Strand7. This is achieved by a comparative study, modelling an experiment that has been carried out on the control bridge method in Strand7. Results will be compared and analysed to explain any discrepancy seen, which is to be expected as any numerical model has some variation from reality. Once validated, the Strand7 program is then used to carry out a parametric study. The effect of varying the proportional height of the two buildings coupled on the effectiveness of the method will be investigated.

## **1 INTRODUCTION**

In recent years, attention has been made to reduce the vibration of various flexible structures such as vibratory platforms, tall buildings, bridges and tower structures. The inherent damping of these structures is low and unable to withstand the vibration caused by disturbances like earthquake motion or strong winds. This necessitates a suitable protective scheme to produce sufficient control force to suppress not only the first mode but also higher modes.

One of the more recent damping types is the control bridge, or coupling concept. This involves connecting buildings of differing dynamic response characteristics together in order to reduce their lateral vibrations due to wind or earthquake loading. It has many attractive properties such as requiring little or no floor space within either building or providing better access between higher storeys of buildings. However, its use is limited to fairly specific situations and as such has not yet been widely implemented.

Hence, this paper aims to investigate the effectiveness of the control bridges in high-rise buildings under seismic loading and, more specifically, how their effectiveness changes with varying critical parameters. Achieving this will involve a numerical analysis of structural dynamics, through the finite element analysis software Strand7. Comparison test was carried out for the performance of numerical modelling. Once validated, the Strand7 program is then used to carry out a parametric study. The effect of varying the proportional height of the two buildings coupled on the effectiveness of the method will be investigated.

## 2 CONTROL BRIDGE DAMPING

The concept of connecting adjacent buildings with a damper in order to reduce the vibrations in each of them is a relatively recent one and its actual application is even more so. It was first proposed independently by two researchers, Kunieda (1976) and slightly earlier by Klein et al. (1972). The first application was not until 1989, with the Kajima Intelligent Building complex in Tokyo, Japan. Subsequently, other low-rise applications such as a four building coupling application at Konoike Headquarters in Osaka and the more recent, completion of Harumi Triton Square buildings in Japan.

The control bridges function based on the different responses of the differing structures' differing dynamic responses to the same excitation. That is, with different motion of two adjacent structures connected by a damper, a velocity is developed across this damper, results in a damping force that dissipates the energy of the motion.

This approach to damping can achieve sufficient control under the relative low-frequency of earthquake loading (Seto and Watanabe, 2000), whilst causing little reduction in usable floor space and allowing traffic between structures at high levels. The allowance for traffic can cause not only an increase in safety through alternative evacuation, but also efficiency of movement.

Luco and De Barros (1998) proposed the linking of two buildings with passive dampers distributed up the height of the shorter structure and found the optimum damping values. Seto and Watanabe (2000) outlined the problems with spill over of response into higher modes that were known to occur with active damping of tall buildings.

Finally in Christenson et al. (2006), the control force applied for both active and passive control systems were compared, for the varying building and connector configuration. Control force is significant in this type of damper, as it is applied as a horizontal point load to the structure and as such the elements to which the coupling is connected must be able to withstand this load. Whilst with fully active control as seen here, the large extra reduction in vibration was accompanied by an increase in control force, promising indicators for semi-active dampers were seen. These results were that with active control only providing equivalent damping as the passive system, as little as half the passive forces were seen for the limited active damper.

## 3 COMPARATIVE STUDY - VERIFICATION OF NUMERICAL MODELLING

The finite element modelling technique is required to test the control bridge method. This involves dividing structures into a number of finite elements, with points of interest defined by nodes. For this paper, the front-end software program Strand7 is used. To achieve this, an existing experiment is required and the experiment chosen is a small-scale shake table test, found in Christenson (2001).

### 3.1 Experimental Set-up

Figure 1 shows the set-up used in the experiment to be modelled for comparison and validation purposes. Although a small, two-storey representative frame, it has some characteristics that are indicative of a much taller structure. It's basic structure is very flexible, with thin aluminium columns connected to much stiffer plexiglass. This is intended to represent a real, tall structure, where the lateral stiffness is provided in the slabs or beams, with vertical support elements being somewhat slender and flexible. The extra mass that is added to building one is for the purpose of making the structures differ, dynamically.

This apparatus was tested with a scaled variation of the North-South recorded motion of the El-Centro earthquake, as well as three others. Only the El-Centro data is used for comparison herein. These are all to be used in a comparison with the finite element model, although active control is expected to be much more effective than the passive damping used in the Strand7 solutions.

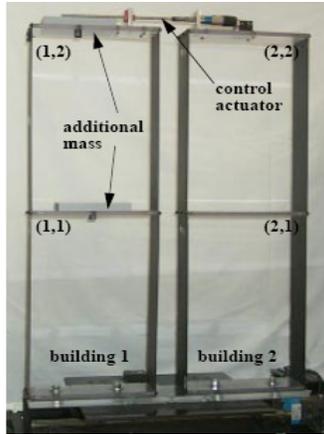


Figure 1 Experimental Set-Up (Christenson, 2001)

Plan size: 305 × 108mm,  
 Height: 980mm,  
 Inter-story height : 490mm,  
 Plexiglass thickness: 12.7mm  
 Aluminium thickness: 1.59mm  
 Separation distance: 75mm

Storey Masses

- $m_{11}$  3.22kg
- $m_{12}$  3.45kg
- $m_{21}$  0.47kg
- $m_{22}$  0.83kg

where  $m_{ij}$  indicates the mass on storey  $j$  of building  $i$

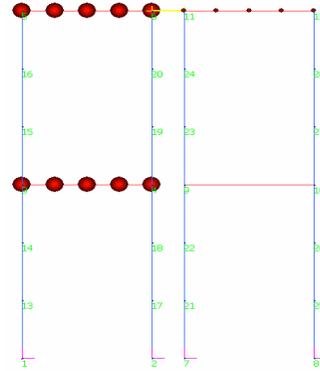


Figure 2 Numerical model using Strand 7

### 3.2 Numerical Modelling

Nodes and beam elements are created in modelling the experiment. Vertical and horizontal elements are made to link these nodes, with the horizontal elements different properties to the vertical ones. On consideration of this, as the ‘columns’ are so flexible and will be excited by fairly rapid frequencies, a single beam element was considered insignificant to properly model their behaviour. This results in a model of 29 nodes, with 28 beam elements in the case of no control bridge, and 29 elements when the control bridge is included, the case shown in Figure 2.

A summary of the material properties for the two types of building is summarized in Table 1.

**Table 1 Model Properties for Comparative Study**

Material	Modulus (Mpa)	Poisson's Ratio	Density (kg/m <sup>3</sup> )	Viscous Damping (Ns/m <sup>3</sup> )
Aluminium	6.9x10 <sup>4</sup>	0.334	2700.0	10
Plexiglass	2.96x10 <sup>3</sup>	0.333	1123.5	10

### 3.3 Comparative Results

#### 3.3.1 Natural Frequencies

After modelling the structure and carrying out the linear transient dynamic solver, results are produced so as to most easily compare with those given for the experiment. First to be found, to give an initial idea of whether the models are reasonable representations of reality, are the natural frequencies. These are shown in Table 3 and are quite self-explanatory. Only the first two modes are presented in Christenson (2001), as these are the modes likely to be excited by the accelerations applied. A good correlation can be seen in the 2<sup>nd</sup> modes of both buildings, with some variation from the first mode. The model does significantly underestimate the first mode of each building (by about 30%), this is most likely due to some difference between the restraint conditions or the properties entered and those for the materials used in experiment. Further results on the response to excitation will indicate whether the model is actually a reasonable representation of the experiment.

**Table 2 Natural Frequencies (Hz) of Actual Structures and from Strand7 Model**

Mode	Building 1		Building 2	
	Actual	Model	Actual	Model
1 <sup>st</sup>	0.90	0.63	1.85	1.22
2 <sup>nd</sup>	2.70	2.75	5.73	5.58

### 3.3.2 RMS Accelerations

A broad overall measure of the total acceleration response, in terms of root mean square of acceleration is compared. Table 3 shows this comparison most clearly. The RMS value over the entire 40 second recording period is used for the model calculations, as no designation otherwise is given in Christenson (2001).

**Table 3 Experimental RMS and Finite Element Model RMS Accelerations**

	Un-Coupled		Rigid Connection		Passive Damper	
Building 1	Exp	Comp	Exp	Comp	Exp	Comp
Level 1	1.42	1.23	0.76	1.64	0.52	0.70
Level 2	1.49	0.94	1.00	1.76	0.51	0.68
Building 2	Un-Coupled		Rigid Connection		Passive Damper	
Level 1	2.29	2.08	2.48	1.43	1.12	0.63
Level 2	2.09	2.35	1.03	1.76	0.87	0.77

A reasonable correlation can be seen between the numerical analysis and test result, although not incredibly close. With the rigid and damped values being somewhat different in the numerical situation to the actual one, as outlined, their discrepancies are not unexpected. The un-coupled results are fairly close with the differences within acceptable limits, particularly considering the difficulties outlined in the modelling process. Yet further information on the response, in the form of time-histories of the responses, will give greater idea of the accuracy of the finite element analysis and Strand7 software package. One general concept that can be seen to correlate well is the effectiveness of this type of damper in an agreeable situation. Clearly, rigid connections are a trade-off with one building gaining the benefit of the other that is not excited as close to its natural frequency, but increasing its own vibration. When the connection is damped, the trade-off no longer occurs, but a greater decrease in RMS accelerations is seen in both.

## 4 PARAMETRIC STUDY

### 4.1 Varying proportional height

The main outcome of this study, which the previous research and numerical comparative study builds towards, is an assessment of the impact of varying parameters on control bridges' effectiveness as a structural control mechanism. The parameters to be varied are schematically shown in Figure 3. They are proportional height ( $h_2/h_1$ ) and spacing between the two buildings. As shown in the figure, the height  $h_1$  will remain constant, with  $h_2$  varying as well as the distance between the buildings. Also remaining constant is  $h_c$ , the coupling height, at the top of the shorter building as this has been shown by previous research to be the most effective position. When the spacing is varied, it will be with both  $h_1$  and  $h_2$  constant, at  $h_2 = 0.5h_1$ , at which past research suggests the damping will be at or near optimum effectiveness.

Base acceleration used for the study is the included table in Strand7, representing the El Centro earthquake motion. It is an acceleration that represents real seismic excitation in scale and its broad range of frequency so will give an indication of structural performance under actual earthquake conditions. This acceleration is the same acceleration used in all cases in the following studies.

### 4.2 High-Rise Base Model

In order to have a degree of reflection of a real life application in the modelling of the high-rise structures involved in the parametric study, they are based on a suggested section of a moment resisting frame (MRF). Spencer et al. (1999) stated, in an article outlining a benchmark study for assessing structural dampers, that an in-plane finite element model of a building's moment resisting frame considering linear response is said to be a good approximation for actual response. Also in this article, a specific structure is outlined to be used for this benchmark study. It is a 20 storey building

designed, although not constructed, for the SAC Phase II Steel Project in Los Angeles. It meets all code including seismic code for the relatively seismically active region. Its presentation as in the article is given in Figure 4, which is used as a basis for the finite element modelling

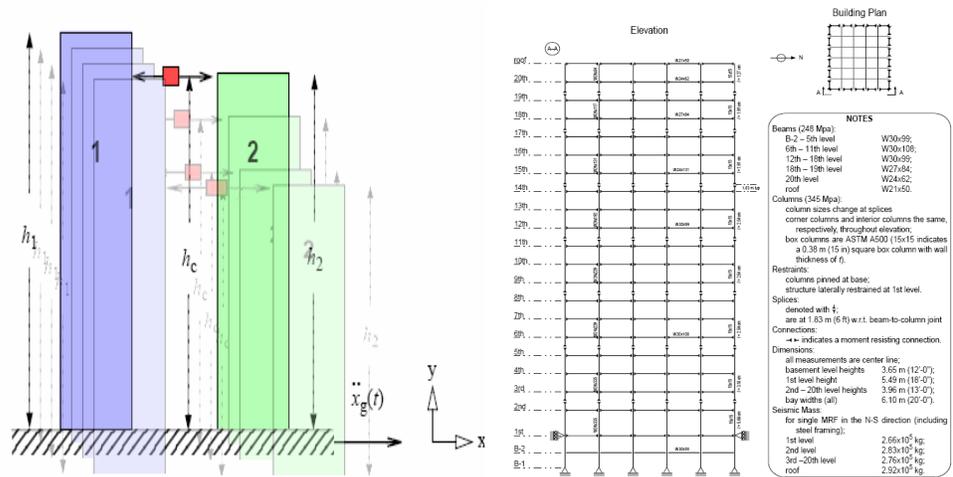


Figure 3 Schematic of Parametric Study      Figure 4 Los Angeles 20 Storey Building MRF (Spencer et al., 1999)

For this research, the height of building one is chosen as fifty stories, so the structure is extended to this height. Also, to somewhat offset this and simplify the model, one of the largest element sizes for each type (edge column, interior column and beam) and the largest seismic mass, are taken up throughout the model without any tapering of their size and strength. This is seen to stiffen the structure in comparison to the scaling of the twenty storey element sizes, which would be insufficient for the increased loads. As a result, nodes at the splices are deemed unnecessary as there is no change in element size or type. Obviously, this is not a very exact measure to represent the heightening of the building and is likely to be overly flexible, but with the constant variation in height required in the parametric model consistency is required to prevent constant re-modelling and increase efficiency. As it is for a parametric model, the fact that it gives some realistic representation is good, whilst exact re-creation is unnecessary, as long as consistency is maintained so accurate comparisons can be made.

The model is given full fixity at its base, with the removal of lateral restraint at the ground level of the designed structure, two storeys above its base. Basement levels are removed to simplify the structure in general and also because the prescribed system has some ambiguity over where exactly seismic loads will be applied in reality. Both the bottom of the basement as well as the lateral restraint points would receive seismic acceleration, a fact which would hinder confident understanding of the behaviour, with experience up to this point in the simple base-acceleration case. Finally, the dimensions of the structure are also rounded to whole numbers, as this assists modelling and this is not a specific analysis of this building or a design procedure, so exact measurements are unnecessary.

Measurements, both floor-to-floor heights and bay widths, have been rounded and standardised, resulting in five bays, each of six metres width and floor-to-floor heights of four metres throughout the model. All 551 elements are beam elements, as it is a two-dimensional frame to be analysed. It has three different properties, being edge columns, interior columns and beams. These are taken as the largest size given in the previous outline of the Los Angeles steel building for each type. Nodes are placed at all intersections of beams. When shorter buildings are used in the parametric study, the same length dimensions and beam properties are used. Although this does not represent real-life, it gives consistency to make some conclusions on the variation of the property intended to vary only, without any other effects impacting on the results. Standard spacing between buildings is 20m.

After creating this model, the non-structural mass must be added. This again is standardised, with the total seismic mass per floor taken as the maximum, of  $2.76 \times 10^5$  kg. That stated seismic mass includes both structural and non-structural, but to apply it to the model, the non-structural mass is calculated

and applied in order to assign the structural elements their mass separately, via their density.

To do this, the structural mass is found then subtracted from the total mass. Structural mass per floor is found as the product of each member's area, length and density, summed for each relevant member on a floor. This is found as 12 508 kg. When subtracted from the total, the value for non-structural mass per floor is found as  $2.63 \times 10^5$  kg. Finally, this is applied evenly across the beams as non-structural mass in Strand7, with a value of 8 767 kg/m.

#### 4.3 Analysing Response Acceleration Seeking Position of Maximum

In order to carry out the parametric study, some indication of trends of acceleration over the height of the buildings is needed, as the maximum acceleration in either building is the response sought. To do this, the case of  $h_2/h_1 = 0.5$  is taken and accelerations at  $1/4h$ ,  $1/2h$ ,  $3/4h$  and  $h$  of each building are recorded and compared to find any trend indicating where the maximum acceleration is likely in all cases.

Both RMS accelerations and total acceleration response graphs are compared for assessment, for 60 seconds of analysis, where the base excitation occurs over approximately the first 30. Both the connected and free (not connected) cases are tested, to observe whether this effects the position of the maximum acceleration. This is not a fool-proof method of determining where maximum response occurs in all models but with clear-cut results, as it is a middling example of the models, it gives a good indicator of the approximate position. Also, basic knowledge and research would suggest that the top of the tallest building is the critical position, so if the test agrees with this, it gives further conviction to the conclusions.

A summary of the results is shown in Table 4, clearly showing that, considering RMS accelerations, the top of the taller building displays the greatest accelerations in both the free and connected situations. This strongly suggests that this is the position where the greatest acceleration occurs throughout the measured period. It is reinforced by observing the acceleration time histories for each case also, with the acceleration of building 1 at height  $h$  being the greatest at almost all times throughout the sixty second period.

**Table 4 RMS Accelerations at Varying Heights over Both Buildings**

	Building No. and Height of Measured Acceleration							
	Building 1				Building 2			
	$0.25h$	$0.5h$	$0.75h$	$h$	$0.25h$	$0.5h$	$0.75h$	$h$
<b>Free</b>	1.82	1.30	0.97	2.04	0.54	0.60	0.69	0.89
<b>Connected</b>	1.20	0.82	0.74	1.37	0.60	0.66	0.67	0.84

## 5 VARIATION OF PROPORTIONAL HEIGHT

### 5.1 Models Used

The major study carried out in this parametric study involves the variation of proportional building heights. It involves maintaining building one at a constant height of fifty storeys and 200m, whilst varying building two's height. The height of building two is varied from 40m (10 storeys) up to 200m in increments of 20m (5 storeys). Models are created for each of these scenarios, both coupled and uncoupled. Table 5 outlines the names used for these models in this paper, with Figure 5 following showing visually the un-coupled models. It is also important to note that all damped models have the coupling at the top of the shorter building.

**Table 5 Model Names**

Building		Coupling Situation	
1 Height	Building 2 Height	Un-Coupled	Damped
50 storeys	10 storeys (0.2h <sub>1</sub> )	U02	D02
50 storeys	15 storeys (0.3h <sub>1</sub> )	U03	D03
50 storeys	20 storeys (0.4h <sub>1</sub> )	U04	D04
50 storeys	25 storeys (0.5h <sub>1</sub> )	U05	D05
50 storeys	30 storeys (0.6h <sub>1</sub> )	U06	D06
50 storeys	35 storeys (0.7h <sub>1</sub> )	U07	D07
50 storeys	40 storeys (0.8h <sub>1</sub> )	U08	D08
50 storeys	45 storeys (0.9h <sub>1</sub> )	U09	D09
50 storeys	50 storeys (1.0h <sub>1</sub> )	U10	D10

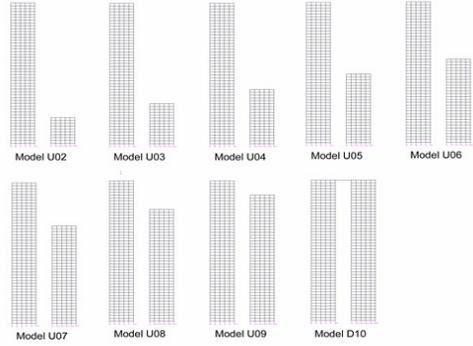


Figure 5 Some Models used for Varying Height Parametric Study

### 5.2 RMS Acceleration Results

In order to quantify the effects of the control bridge effectively, the RMS Accelerations at the top of each building (which is the location of the worst case acceleration in most situations) for each case are compared. This enables the comparison of a single value, which accurately represents the effect the damping system has. The difference in RMS accelerations from free to damped is the value used here to identify the effectiveness, in terms of proportional RMS reduction of the free response as a percentage. A summary of the results is shown in Table 6.

**Table 6 Changes in RMS Acceleration for Different Proportional Heights**

$h_2/h_1$	$\Delta$ RMS Acceleration, Building 1 (% of Free RMS Acceleration)	$\Delta$ RMS Acceleration, Building 2 (% of Free RMS Acceleration)
0.2	-24.84%	66.99%
0.3	42.96%	41.91%
0.4	-6.29%	39.40%
0.5	32.69%	5.92%
0.6	20.93%	-11.67%
0.7	24.21%	34.73%
0.8	16.88%	35.90%
0.9	29.77%	36.91%
1.0	0.01%	-0.01%

In summary, there are some simple inferences that can be made about the height effect on this control strategy. Noticeably, when the ratio is 0.2, the damping effect on the taller building is negative, meaning it actually increases the accelerations. For this setup (models U02 and D02), the shorter building does receive a significant amount of damping though. This can be understood intuitively, as the large, massive fifty storey structure can easily have a great effect on the smaller and much lighter ten storey one. Also, as the structures become too similar in height, when  $h_2/h_1$  approaches one, there is a large decrease in the damping that occurs.

The only other implication that can be seen at this point is for the majority of proportional heights between these, it is a quite efficient method, achieving up to 40% reduction in RMS acceleration for each building. It seems clear that the efficiency is fairly independent of proportional height apart from these general inferences and a greater understanding is achieved when the building's natural frequencies, shown in Table 7, are included in the investigation. Also shown is the comparison of natural frequencies (considering only the pertinent comparisons of the first three modes) between the two buildings, in Table 8, which is even more informative. In table 8,  $f_{nij}$  defines the  $j^{th}$  natural frequency of building  $i$ .

**Table 7 Natural Frequencies (Hz) of Un-Connected Models**

	Building 1	Building 2								
		$h_2/h_1$								
		0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
Natural Frequency 1	0.079	0.536	0.349	0.254	0.196	0.158	0.130	0.108	0.092	0.079
Natural Frequency 2	0.266	1.643	1.064	0.777	0.605	0.491	0.410	0.349	0.303	0.266
Natural Frequency 3	0.513	2.866	1.856	1.369	1.082	0.891	0.756	0.654	0.576	0.513
Natural Frequency 4	0.739	4.175	2.662	1.952	1.539	1.269	1.079	0.937	0.827	0.739

**Table 8 Important Natural Frequency Ratios**

$h_2/h_1$	$fn_{21}/fn_{11}$	$fn_{21}/fn_{12}$	$fn_{21}/fn_{13}$	$fn_{22}/fn_{12}$	$fn_{22}/fn_{13}$	$fn_{23}/fn_{13}$
0.2	6.7720162	2.0142181	<b>1.045656</b>	6.172466	3.204358	5.590647
0.3	4.4092184	1.3114451	0.68082	3.998873	2.075965	3.620113
0.4	3.211359	<b>0.9551627</b>	0.495861	2.919978	1.51587	2.670435
0.5	2.4823424	0.7383295	0.383295	2.273769	<b>1.180399</b>	2.109711
0.6	1.9904698	0.5920305	0.307345	1.844168	<b>0.957377</b>	1.738617
0.7	1.6363963	0.4867175	0.252673	1.539047	0.798977	1.474429
0.8	1.3701337	0.4075224	0.21156	1.312125	0.681173	1.276524
0.9	<b>1.1636257</b>	0.3461002	0.179674	<b>1.137454</b>	0.590495	<b>1.122594</b>
1	1	1	1	1	1	1

Those values in bold identify ratios that are close to one, which are seen as possibilities to adversely affect the damping of the model. The more critical values are those shaded, in which cases major problems with damping are expected. These shaded elements of the table can be immediately correlated with the worst points, all three of them being points where negative damping is occurring. Also, at  $h_2/h_1 = 0.5$ , where a possibility of poor damping performance is expected, however, for the case of  $h_2/h_1 = 0.9$ , none can be seen in the data shown, with the damping performing as well or better than for the other values which have no natural frequencies coinciding.

At the points where these drastic reductions in damping ability occur, there is some variance in the specifics of the effect the coinciding natural frequencies have. These include which building receives the decrease in damping, or even increase in RMS acceleration and the degree to which the decrease occurs. Firstly, the mostly unexpected increase of responses that occurs in the results will be addressed.

Also to be noted in the presentation of RMS acceleration results is the maximum efficiency occurring for both buildings simultaneously at  $h_2/h_1 = 0.3$ . At this point both achieve greater than 40% reduction in RMS accelerations. The first, possibly main reason for this is that none of the first four natural frequencies of the two buildings are close to coinciding with one another. This almost occurs when  $h_2/h_1 = 0.7$  also, however the fourth frequency of building one is in fact very close to the third of building two in this case.

The models U05 and D05 do not see the greatest reduction is that there is a close ratio of frequencies, with the 2<sup>nd</sup> natural frequency of building two being close to the 3<sup>rd</sup> of building one. Whilst the 3<sup>rd</sup> mode vibration of building one may not be a critical vibratory mode for the first building, the second building's second mode may cause a great amount of vibration in it. This conclusion is reinforced as the next case,  $h_2/h_1 = 0.6$ , has a closer correspondence between these frequencies and a much more drastic loss of damping is seen for building two. Indeed, a similar situation to that described previously for building one when  $h_2/h_1 = 0.4$  occurs. Also, the decrease in building one's damping that occurs suggests that this matching up of frequencies is also having a small effect on it, suggesting that although not the principal vibratory mode, loss of damping of its third mode still has an adverse effect on building one's damping. This is seen at  $h_2/h_1 = 0.4$ , where the first frequency of building two is coinciding with the second natural frequency of building one and thus little damping can be seen by it on the main contributor of building one's vibration. This frequency is approximately 0.25Hz and

probably is one of the major frequencies of the applied excitation. Also, the first mode of vibration will be excited to a greater magnitude, generally and so the second building's first frequency is important to the damping of the buildings in all cases. With the greatest excitation comes the greatest ability to provide damping force, with this type of damping system.

Contrarily, the first frequency of building one is of negligible import. This is simply so low that it is very unlikely to be excited at all, so has been seen to have little impact on the vibration or vibration control at all. In the case of the first building, the lowest excited mode is the second and it therefore has the greatest impact in general. In terms of RMS accelerations, this system is effective, where natural frequencies do not coincide and the buildings are dynamically dissimilar. Even around points where this does not occur, it is very effective, for example when the buildings are close to the same height, at  $h_2/h_1 = 0.9$ , the second greatest damping of both buildings is seen. With more models at smaller intervals, the points of great reduction in damping are likely to be more clearly shown as exceptions to the general reasonably constant trend of damping, not to show that these dips in damping are occurring over large variation in proportional height.

## 6 CONCLUSIONS

This paper focused on the effectiveness of vibration control Bridge using numerical analysis. The comparative results indicate that Strand 7 program can be used to predict the behaviour of the control bridge and represent the real physical performance of structures with control bridge damping.

It is observed from parametric study that natural frequencies of the two buildings are critical. Where the two buildings do not have coinciding natural frequencies that are critical under the excitation applied, quite effective damping of both RMS and peak accelerations are clearly seen as long as the shorter building is at least 30% the height of the taller one. Up to 40% reduction of the un-damped peak and RMS accelerations are witnessed in the taller of the buildings, which is most likely to be the critical one in application of this technique.

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