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Evaluation of the effect of smart façade systems in reducing dynamic response of structures subjected to seismic loads

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Abstract. To date the engineering community has seen facade systems as non-structural elements with high aesthetic value and a barrier between the outdoor and indoor environments. The role of facades in energy use in a building has also been recognized and the industry is also witnessing the emergence of many energy efficient facade systems. This paper will focus on using exterior skin of the double skin façade system as a dissipative movable element during earthquake excitation. The main aim of this study is to investigate the potential of the façade system to act as a damper system to reduce earthquake-induced vibration of the primary structure. Unlike traditional mass dampers, which are usually placed at the top level of structures, the movable/smart double skin façade systems are distributed throughout the entire height of building structures. The outer skin is moveable and can act as a multi tuned mass dampers (MTMDs) that move and dissipate energy during strong earthquake motions. In this paper, using a three dimensional 10-storey building structure as the example, it is shown that with optimal choice of materials for stiffness and damping of brackets connecting the two skins, a substantial portion of earthquake induced vibration energy can be dissipated which leads to avoiding expensive ductile seismic designs. It is shown that the engineering demand parameters (EDPs) for a low-rise building structures subjected to moderate to severe earthquakes can be substantially reduced by introduction of a smart designed double skin system.

Keywords: façade; structural control; vibration; damper; earthquake

1. Introduction

Passive energy dissipation systems utilize different type of damping devices to dissipate the applied energy to a structure and reduce earthquake-induced building motions. One of the most popular passive systems is tuned mass damper which is a secondary mass connected to a generally much larger/heavier primary mass, to affect the dynamic response of the primary mass. The Tuned Mass Damper (TMD) was initially proposed by Frahm in 1909 (Den Hartog 1956) and later investigated by many other researchers to decrease the vibration of the primary system by tuning the TMD stiffness and damping coefficients to the specific natural frequencies of the main system (Aldemir 2003). Typically, a TMD system reduces response of the primary system in a narrow

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frequency band and consequently is not desirable for other frequency ranges (Behr 1998). Non-structural components play a significant role in adding stiffness to the building; they might also be able to dissipate some of the energy from applied earthquakes if they have the ability to move back and forth (Hareer 2007). However, the role played by the so-called ‘non-structural components’ is not considered in either Australian or international standards, and is consequently not considered in the current structural design process. Curtain wall systems are employed widely for low-, mid-, and high-rise buildings, but a research gap currently exists with regard to overall performance and capability of the building envelope and their effect on building behaviour during Earthquake-induced building motions (Fu and Johnson 2010). It should be mentioned that, replacement of conventional energy absorbing elements, integrated with the main structure, would be difficult after a strong earthquake event, but this issue readily resolved as the proposed elements are within the cavity between outer and inner skin of double skin façade system. In order to have a better understanding of the role of the façade panel in affecting structural performance, it is necessary to analyse and evaluate the contribution of each façade panel to the overall lateral response and energy dissipation capability of buildings (Baird *et al*. 2011). Because of TMD limitations in dealing with wide frequency ranges, the multiple tuned mass damper (MTMD) systems have been proposed to increase robustness by tuning multiple dampers to a wider range of building natural frequency bands. In this paper, a new type of façade system is studied that will incorporate the concept of multiple distributed mass damper system into their design (Behr and Belarb 1996). As life safety is the most crucial matter for a building structure subjected to earthquake forces, the primary intention of this system is to reduce vibrations caused by seismic activities to prevent structural damage (De Matteis 2005).

In the proposed system, the outer skin of Double Skin Façade (DSF) system is movable in each floor and acts as tuned mass damper (TMDs) that moves and dissipates energy during strong motions, such as those experienced during earthquakes (Hunt 2010). Because the outer skin of the DSF system is positioned along the height of a building and their weight are distributed throughout the building, the dampers are placed on each floor instead of concentrating them in one or in a few places like conventional TMDs (De Matteis 2005). This distributed mass damper (DMD) system is more challenging for engineers to design because of the very large number of individual dampers but can be more favourable to architects because no massive damper is located at the top of or elsewhere in the building to interrupt their design. Massive savings in premium space is just one advantage of movable/smart double skin façade system (SDSF) in comparison to traditional TMDs (Bupp *et al*. 2000). Parameters such as mass, and stiffness and damping coefficients should be optimized to achieve to an effective and robust structural control system (Fu and Johnson 2013). The proposed façade system can be useful as an environmental control systems as well as a source of energy dissipation (Yankelevsky 2011). The purpose of this paper is to systematically evaluate movable/smart building envelope system that not only resist earthquakes effectively but can also enhance the seismic performance of the primary structure (Goodno *et al*. 1996). The stiffness and energy dissipation contribution of the façade system is quantified as well. Building associations, designers, specifiers, building owners and glazing system manufacturers need to be aware of these notable changes in the way façade panels and their connecting components are to be designed and can be enabled to reduce part of the applied earthquake energy. Discussions and conclusions on the thermal performance of the proposed system and its effects on the building structure are also provided (Li *et al*. 2011). This paper briefly presents the movable dissipative façade system and the outcomes of a study that analytically evaluates the appropriateness of using such a system as supplementary energy dissipating devices for seismic design of certain types of buildings.

2. Details of analytical studies

ANSYS 14.0 software was utilized in this study to conduct the finite element analyses. This paper introduces finite element analyses of 10 storey three dimensional building structure under two selected earthquake excitations. To simplify and generalize the analyses, some assumptions have been made as listed below:

• To decrease the computational time and to simplify the structural modelling, one-dimensional frame elements were selected for beams and columns and two-dimensional plane stress elements were chosen for facade panels in this program.

• The beams connecting the two columns of the equivalent bay are modelled as elastic elements with rotational springs at both ends.

• Rotational springs are utilised to model the equivalent strength and stiffness of all beam/column connections. Each spring represents the cumulative strength of half the number of simple connections.

3. Selected earthquakes

The chosen earthquake acceleration records were applied in two directions (X and Z directions) in the 3-D structural models, at the base of the structure. The supports at the base of the structure were modelled as a rigid joint, restrained against translation and rotation in x, y and z directions. The vertical gravity loading on the structure was in the form lumped masses applied to the beams. The dominating frequencies of the ground motions varied over a wide range from 0.2 to 2.5 Hz, which concur with the natural frequencies (0.16 to 1.16 Hz) of the structures under consideration (Ji *et al*. 2005).

Fig. 1 shows that most of the earthquake energy lies between 1 and 10 Hz which coincides with natural frequencies of most building structures, except super tall buildings, and hence the chance of encountering resonant conditions in such structures during earthquake excitations is increased. Two earthquake records as shown in Table 1 with different frequency content and duration have been selected to conduct a comprehensive evaluation of the performance of the proposed system.

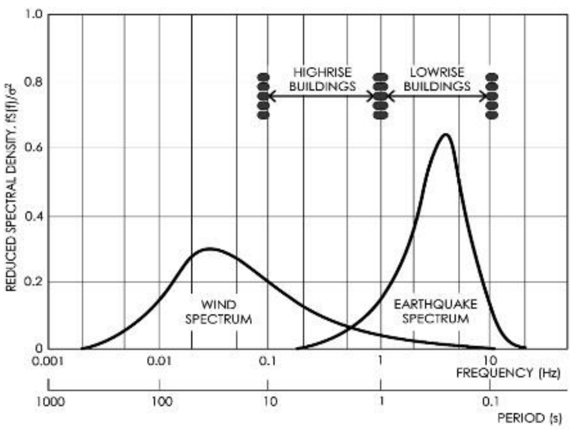


Fig. 1 Seismic and wind hazard versus excitation frequency (or period)

Table 1 Characteristics of selected earthquake records

Earthquake Record	Duration of strong motion (seconds)	Range of dominant frequencies (Hz)
El Centro (1940) - far field	1.5 - 5.5	0.09 - 0.69
Kobe (1995) – near field	7.5 - 12.5	0.21 - 1.22

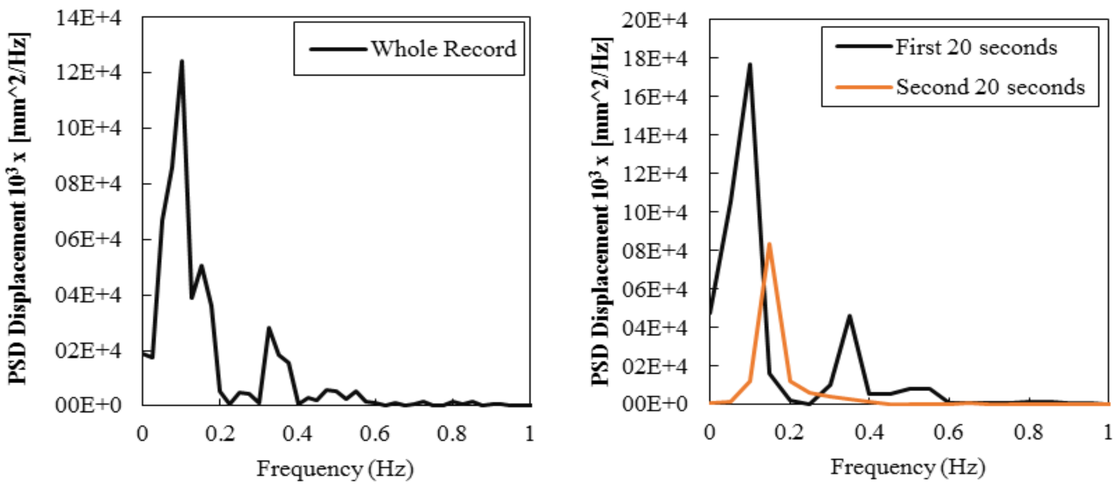


Fig. 2 Displacement Power Spectral Density for 1940 El-Centro earthquake

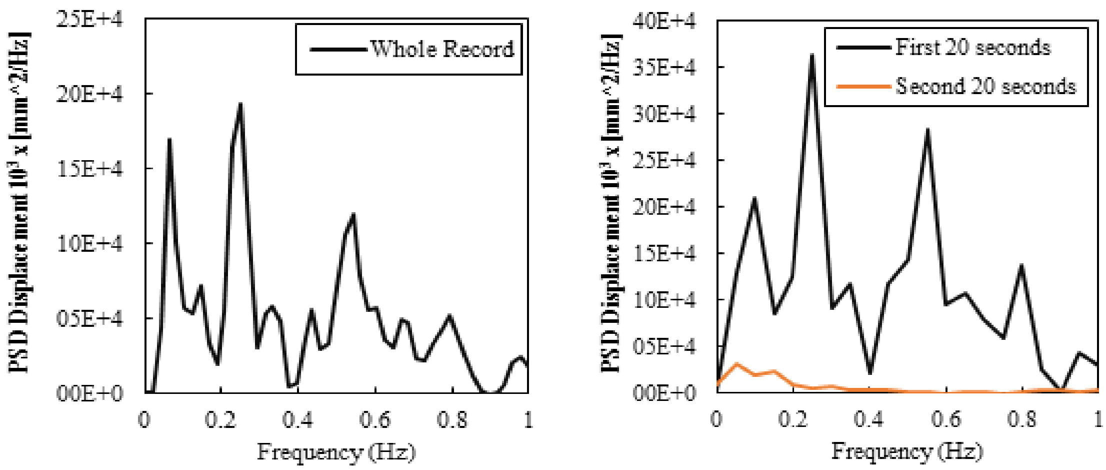


Fig. 3 Displacement Power Spectral Density for 1995 Kobe earthquake

It is important to note that evaluation of Power Spectral Density (PSD) of displacement response is necessary for designing and selecting the best facade bracket stiffness for each specific earthquake. Then, a series of response PSD evaluations have been performed using acceleration time-history of the earthquakes mentioned and the results are shown in Fig. 2 and Fig. 3, respectively. Frequency content of the records spread over a range of frequencies during excitation. In order to have a better understanding of excitations frequency content, each case is evaluated and interpreted individually as below. The frequency of façade columns is set up and tuned for each earthquake acceleration record in order to get the best results.

The range of dominant frequencies in 1940 El-Centro record is between 0.09 and 0.69 Hz which is close to frequency of the 10-storey structural model. The dominant frequencies that contain significant seismic energy are concentrated in the first 20 seconds of the record, but the rest of the record also affects the dynamic behaviour of the structural model to some extent. So, design of the bracket stiffness needs to be considered and tuned based on the frequency content of the whole record. The maximum value of PSD for 1940 El Centro record is around 130,000 mm^2/Hz at frequency of around 0.1 Hz.

4. Modelling details of the proposed system

4.1 Primary structure

Defining the reinforced concrete material with all its details and modelling of structural interaction between the steel bars and the concrete material is beyond the scope of this computer modelling. Hence, a smeared model is used combining the properties of concrete and steel in the complex structural model in the 3-D 10-storey structure. The multi-storey system involved in this analysis is a building structure with a storey height of 3.6 m over 10 storeys, hence a total height of 36 meters. The mesh discretization must balance the need for fine mesh to yield an accurate stress distribution and reasonable analysis time, so the beams and columns are divided into three sections along their length for meshing in all numerical modellings.

Sectional dimensions which are used for the model are listed in Table 2. The view of the frame structure used for the 3-D models in this section is shown in Fig. 4.

The model has four bays in X and three bays in Z directions, respectively. The span length of each bay is considered as 4 meters for both directions. It is assumed that building mass is distributed equally at each floor along the height of the structure. Fig. 5 shows beams and columns layout in the building structure plan.

Table 2 Structural sections for beam and column elements

Moment-resisting frame		
Storey/Floor	Square Column (mm×mm))	Rectangular Beam (mm×mm))
10	400×400	400×500
9	400×400	400×600
8	400×400	400×600
7	400×400	400×600
6	500×500	400×600
5	550×550	400×600
4	600×600	400×600
3	650×650	400×600
2	700×700	400×600
1	700×700	400×600

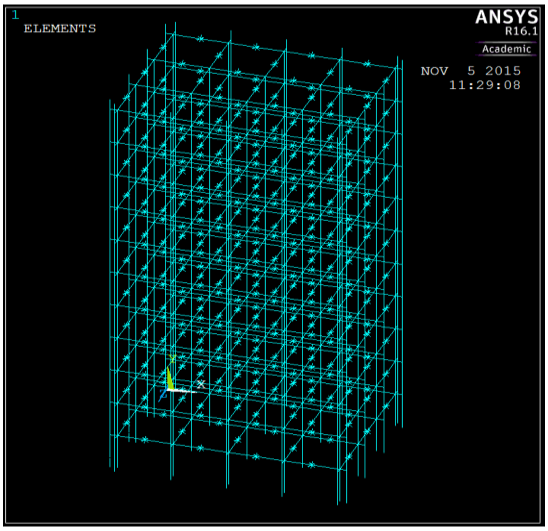


Fig. 4 Front view of exterior elevation of the 3-D frame model

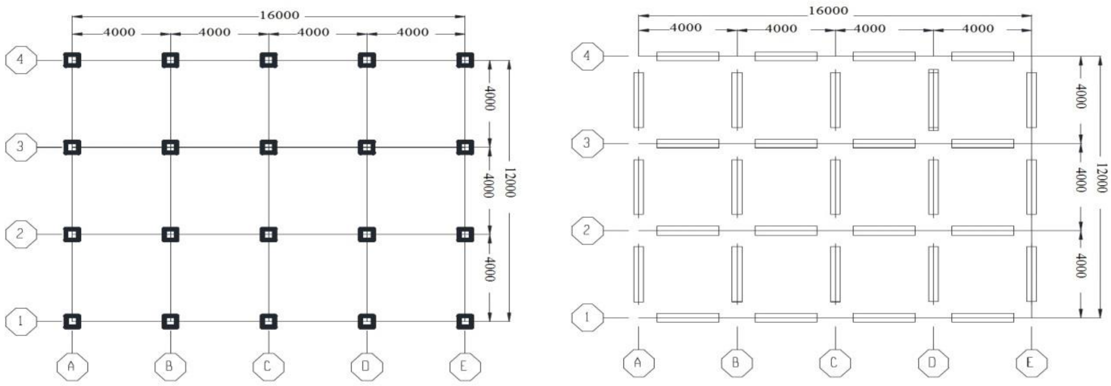


Fig. 5 Plan view of the 3-D frame model (dimensions are in mm)

Table 3 Selected concrete properties

Criteria	Value
Compressive strength, f'_c (MPa)	32
Young's modulus, E (MPa)	30,000
Density(kg/m^3)	2400
Poisson's ratio, ν	0.2

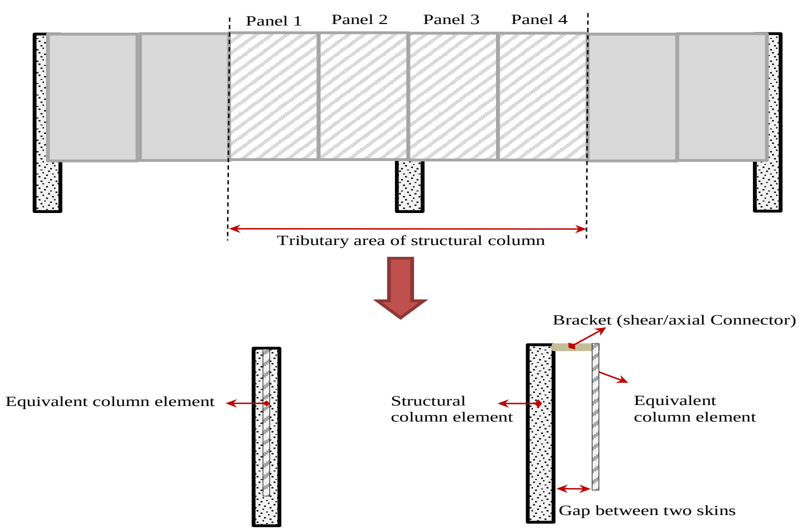
The concrete materials selected for the study are listed in Table 3. Equivalent sections with equivalent material properties were used for numerical modelling of the main structure, as reinforced concrete material is hard to model in ANSYS. The properties used for the equivalent material are the same as concrete except for modulus of elasticity.

Equivalent sections are used in order to simplify the ANSYS finite element models. Calculation of an equivalent section is very simple and can be found in many concrete design handbooks.

4.2 Façade system and bracket element

Double-Skin Facade (DSF') or "airflow" facade is assumed for the structural models. As compared to conventional facade systems, DSF's can reduce energy consumption by 30% (Pinelli *et al*. 1995). They can provide natural ventilation to buildings, and provide valuable noise reduction. They also create a visually transparent architecture that is impossible with conventional curtain wall facades with similar thermal properties. Generally, glass types are selected depending on their location in the building but in this research, all glasses are assumed to have the same dimensions and material properties; and dimensions of window panes are 180 cm in height and 150 cm in width. It is assumed that the insulating glass unit (IGU) panes consist of two 6 mm glass panes with a spacer of 12 mm in diameter. It should be noted that 25 mm IGUs are typically used where safety is not a concern, and heat-strengthened IGUs are used when the panes are located within 45 cm of the ground or within 120 cm of a doorway (Moon 2009). In order to have more accurate results and to model the façade panels closer to reality, the stack joint which connects two consecutive façade columns need to be defined and modelled (Maneetes and Memari 2014).

The panels are modelled as BEAM188 elements with specific material properties to represent four of the façade panels in reality. In order to define the joint in ANSYS, one small element was defined with a length of 100 mm. All three displacements in the *X*, *Y* and *Z* directions are fully restrained at both ends of the element, but rotations at both ends are allowed which is exactly how an ideal joint behaves. The façade load bearing elements are assumed rigid, and they are modelled with two-dimensional frames comprising rigid elastic beam elements. Fig. 6 illustrates how successive façade panels are modelled as one single beam element in each storey in ANSYS modelling. It should be noted that the façade column elements represent four façade panels in reality in terms of weight and dynamic response (frequency of movement) in all modellings. Claddings consist of full storey-height panels and are attached with horizontal bearing connection



(a) Front view (b) Side view

Fig. 6 Schematic view of facade column element and its configuration in each floor

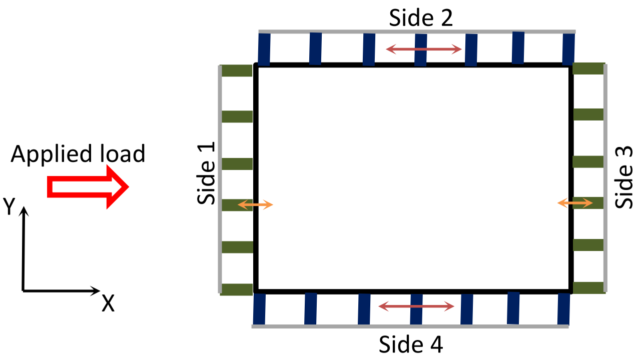


Fig. 7 Plan view of Configuration of shear and axial behaviour of façade brackets in building structure during earthquake

and flexible lateral connection to the slab structure. As the façade column is modelled as a linear beam element, then mesh size of facade column is assumed to be 10 cm in all numerical models. A code for various stiffness configurations that are modelled in this study is defined. Building structures with the conventional bracket system; bracket system similar to those on sides 1 and 3 in Fig. 7 and the bracket system similar to sides 2 and 4 on the same figure are called "Rigid", "Axial" and "Shear", respectively.

In building structures it is assumed that the façade panels are attached to the main structure on all four sides. When applied earthquake is in *X* direction (see Fig. 7), bracket façade elements on Side 1 and side 3 move in direction to yellow arrows and most of the bracket forces are axially induced. Bracket façade elements on side 2 and side 4 move in direction of red arrows and most of the bracket forces are shear induced. Respectively, if earthquake applies in *Y* direction, then behaviour of brackets on sides 1 and 3 will be changed to shear and sides 2 and 4 to axial. Moreover, the number which comes after each term represents the value of the stiffness used in connection with the bracket system. For example, "Shear-10" represents a bracket connection that is defined to move perpendicular to the applied earthquake and has stiffness value of 10 N/mm. It should be noted that the seismic energy absorption at the facade level is due to yielding of brackets and not any dashpots and hence the shear and axial stiffness of these brackets are the main design parameter rather than the damping ratio of any dashpot element between the two skins of the double skin facade.

5. Results and discussions

The global behaviour of the concrete moment resisting frame, due to earthquake excitation, is typically described by maximum deformation, inter-storey drift ratios, residual (permanent) drifts, floor accelerations, forces in bracket connections and base shear force. These engineering demand parameters (EDPs) were determined during the time history analyses of the analytical models and investigated in this study (Thambiratnam 2010). Trends for maximum values of the engineering demand parameters, typical of multi-storey concrete moment resisting frames, are discussed.

5.1 Top lateral displacement

Effects of a particular earthquake on a building structure are usually evaluated by maximum values of displacement at the top level of the building structure. The response time history of top-level displacement of the three-dimensional 10- storey building structure is presented. The fundamental frequency of the bare frame is 0.78 Hz (fundamental period of 1.28 sec) and it increases to 0.97 Hz (fundamental period of 1.03 sec) with double skin facades. The mode shapes are generally not affected by the inclusion of double skin facades.

Top floor displacement in case of “Rigid”, “Axial” and “Shear” connections are extracted and compared. Comparison of responses for the structure with rigid bracket facade and structure with axial bracket facade showed that the proposed connectors were not able to reduce the peak values of top floor displacement as seen in Fig. 8. Fig. 8 also shows the efficiency of the flexible connections for 1940 El-Centro earthquake. After approximately few seconds of the El Centro earthquake, the structure with Shear-50 connections began to significantly reduce response of the

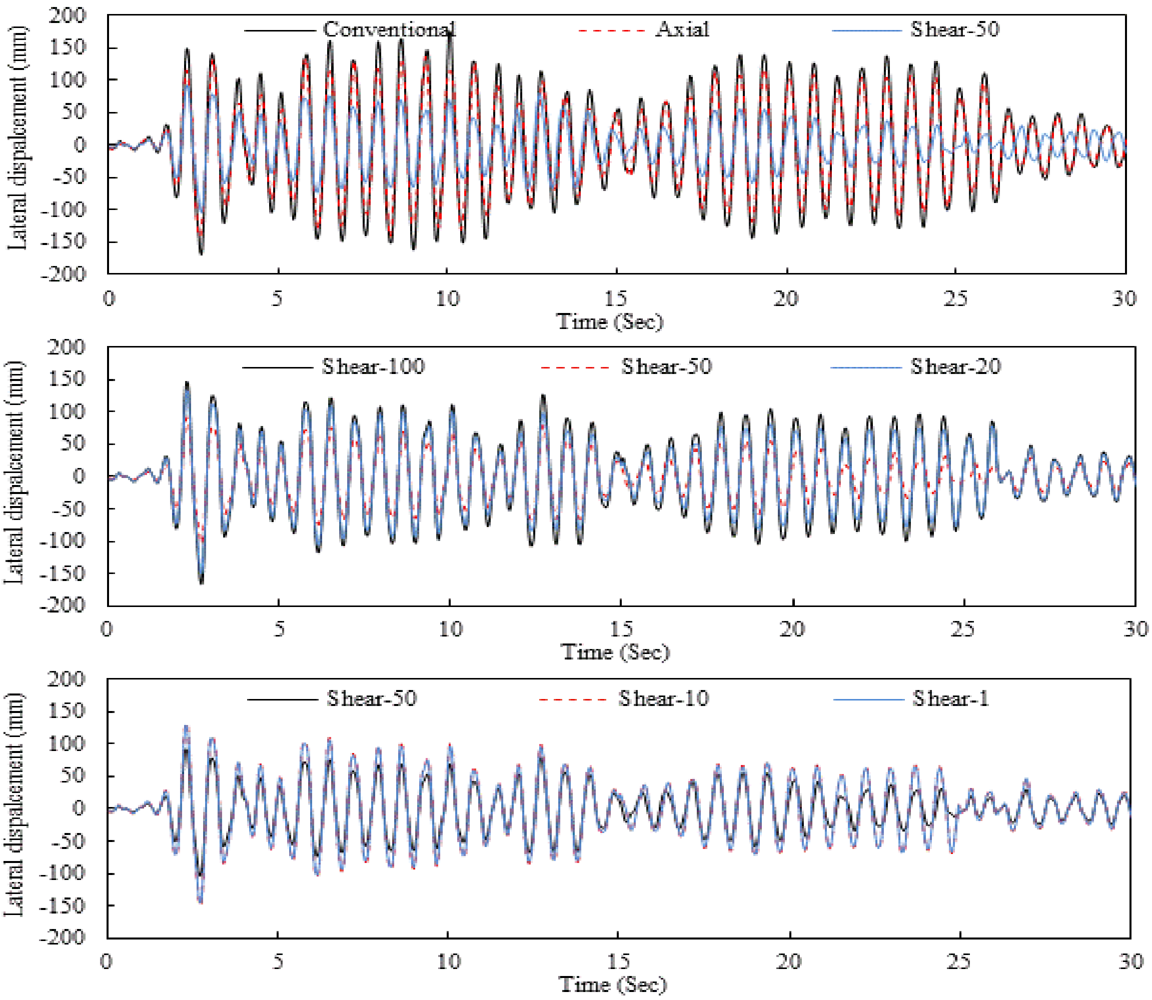


Fig. 8 Top displacements of primary structure coupled with DSFs with different bracket connector stiffness during 1940 El-Centro Earthquake

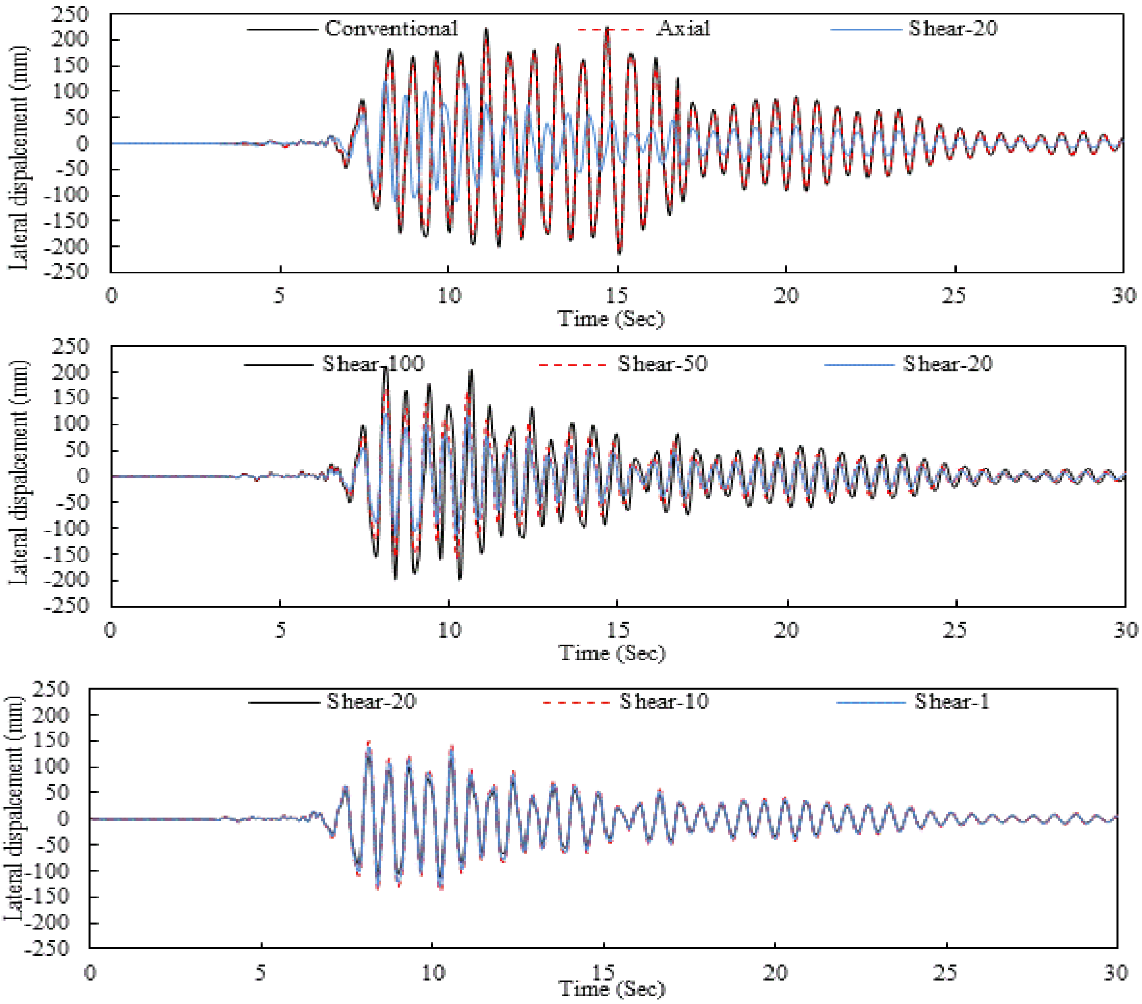


Fig. 9 Top displacements of primary structure coupled with DSFs with different bracket connector stiffness during 1995 Kobe Earthquake

main structure. The reduction continued up to about 30 seconds. During 1940 El-Centro excitation, bracket elements with shear stiffness of 50 N/mm have similar frequency to dominant frequency of the applied record. Additionally, it can be seen from Fig. 8 that this connection can achieve the highest reduction of top displacement of the main structure among other shear connections. It is seen that values of 1, 10 and 20 N/mm have less effect on response reduction compared to the value of 50 N/mm. To be more precise, it has been concluded that the optimum stiffness range is values between 30 N/mm to 60 N/mm for the selected earthquake record following analyses using a wide range of stiffness values.

Fig. 9 shows that the incorporation of shear connections to the structure façade has significantly changed the effects of the seismic loading from 1995 Kobe earthquake on the behaviour of the building system and produced desirable results. Bracket elements with shear stiffness of 20 N/mm has similar frequency to dominant frequency of Kobe excitation and it can be seen from Fig. 8 that this connection can be responsible for the highest reduction of top displacement of the main structure among other shear connections. It is seen that values of 1 and 10 N/mm have promising effect on response reduction of primary structure but less effective than the optimum value of 20 N/mm. To be more precise, it has been concluded that the optimum stiffness range is from 15 N/mm to 25 N/mm for the selected earthquake record. From the results above, it appears that the consideration of movable cladding reduces the top floor displacement of the main frame. According to these results, it is seen that by selecting optimum value of shear stiffness brackets, the overall lateral displacement of the primary structure subjected to seismic loads is decreased. Table 4 shows comparison between effects of different shear connections on the top displacement reduction of the primary structure. Top displacement can be reduced in the range of 30%-50% in comparison to "Rigid" and "Axial" bracket connections. In general, the results of the investigation of the proposed damping system have demonstrated an ability to reduce the seismic response of buildings by placement of the damping devices within the building facade system.

5.2 Structural inter-storey drift

Another very important engineering demand parameter (EDP) for multistorey concrete frame buildings is the evaluation of storey drifts.

The interstorey drift ratio is critical because it helps to describe global damage to drift sensitive components of the building such as structural framing, interior partitions, exterior cladding, and window glazing. In this section, dynamic time-history analyses of the model were performed to determine the maximum interstorey drift demand in each storey. The interstorey drift ratios in all

Table 4 Comparison between maximum top floor displacements (in mm) of primary structure coupled with DSFs with different bracket connector stiffness during the earthquakes

Earthquake	Type of bracket system used in double skin façade system						
	Rigid	Axial	Shear				
			100	50	20	10	1
El-Centro	175	142	147	103	132	132	128
Kobe	226	208	197	156	119	148	138

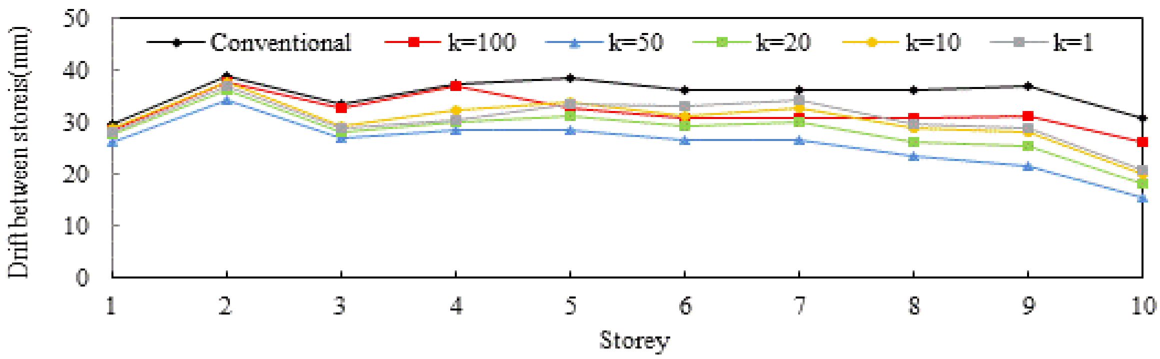


Fig. 10 Drift for primary structure with different stiffness for shear bracket façade elements during 1940 1940 El-Centro earthquake

stories are plotted for the 1940 El-Centro ground motion in Fig. 10. It can be seen that the maximum reduction of interstorey drifts is achieved with connection “Shear-50” on 10th floor.

The results of the time-history show that the interstorey drifts in the moment resisting frame are reduced significantly by the flexible shear connectors. These connections are the most effective ones because of their stiffness and strength. They do deform or become significantly damaged to absorb as much as the applied seismic energy as possible. The interstorey drift ratios in all stories are plotted for the 1995 Kobe ground motions in Fig. 11. It can be seen that the maximum reduction of interstorey drifts is achieved with connection “Shear-20” on 10th floor. It is concluded from the above figures that façade panels with energy absorbing connections have a favourable effect on the overall structural behaviour and are able to reduce interstorey drifts. Results of above figures are listed in Table 5 and Table 6 for better understanding of damper effects.

Absolute maximum values of inter-storey drifts for each of the earthquakes are compared for each bracket case in Table 7. It should be noted that for this example all interstorey drifts are within the allowable limit of 54 mm (1.5% of the storey height) in accordance with Australian Earthquake code As1170.4 (2007).

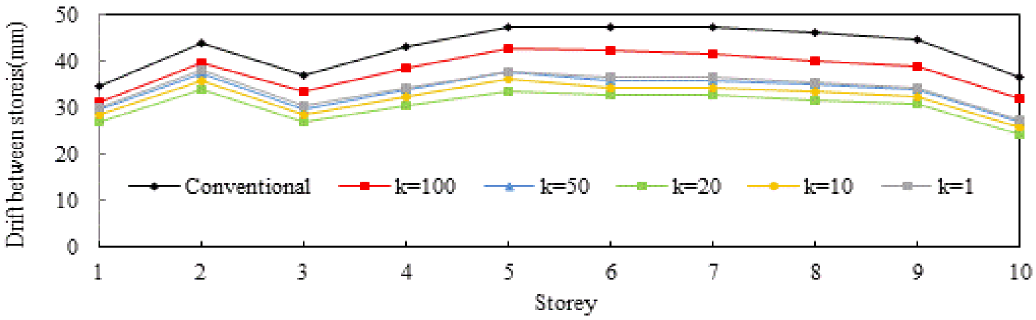


Fig. 11 Drift for primary structure with different stiffness for shear bracket façade elements during 1995 Kobe earthquake

Table 5 Comparison of storey drift with different bracket stiffness during 1940 El-Centro earthquake

Storey Level	Inter storey drift (mm)					
	Rigid	Shear stiffness (N/mm)				
		100	50	20	10	1
10	30	29	26	28	29	28
9	39	38	35	36	38	37
8	34	33	27	28	30	29
7	38	37	28	30	32	31
6	39	33	28	31	34	34
5	36	31	26	30	31	33
4	36	31	27	30	32	34
3	36	31	24	26	29	30
2	36	31	22	26	28	29
1	31	26	15	18	20	21

Table 6 Comparison of storey drift with different bracket stiffness during 1995 Kobe earthquake

Storey Level	Storey drift (mm)					
	Rigid	Shear stiffness (N/mm)				
		100	50	20	10	1
10	35	31	30	27	29	30
9	44	40	37	34	36	38
8	37	34	30	27	29	30
7	43	39	34	31	33	34
6	48	43	38	34	36	38
5	47	43	36	33	34	37
4	47	42	36	33	34	37
3	46	40	35	32	34	36
2	45	39	34	31	33	35
1	37	32	27	25	26	28

Table 7 Comparison between maximum drift of primary structure coupled with DSFs with different bracket connector stiffness during the earthquakes

Earthquake	Type of bracket system used in double skin façade system						
	Rigid	Axial	Shear stiffness (N/mm)				
			100	50	20	10	1
El-Centro	39	39	38	35	36	38	37
Kobe	48	44	43	38	34	36	38

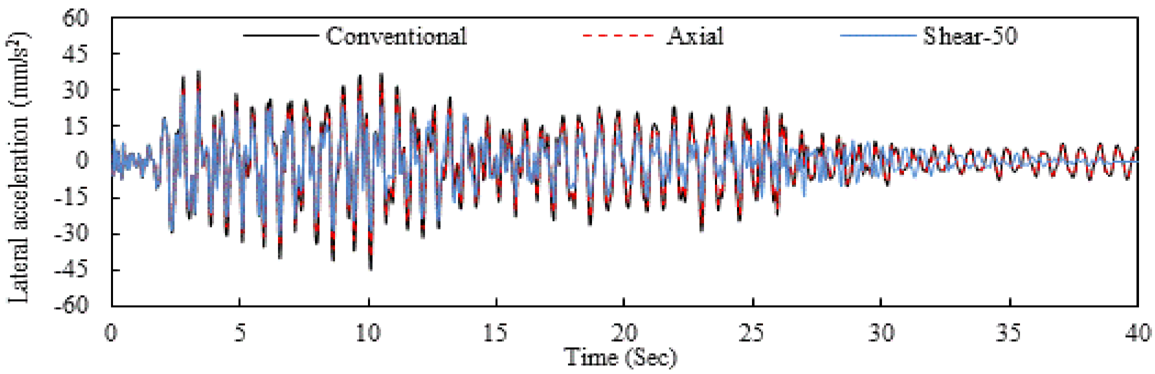


Fig. 12 Time-history top floor accelerations (mm/sec^2) of primary structure coupled with DSFs with optimal bracket connector stiffness during 1940 El-Centro Earthquake

5.3 Top lateral acceleration

The other global engineering demand parameter considered in this study is the maximum floor acceleration. Floor accelerations are used to predict the damage to acceleration sensitive components in the building, such as ceiling systems, chimneys, and mechanical and electrical equipment and IT and phone services. Effectiveness of the facade system, with various stiffness of the connections for the two earthquake records, is studied here. Top floor acceleration in case of rigid, axial and shear connections are extracted and compared in below figures and tables in this section. Comparison of responses for the structure with rigid bracket facade and structure with axial bracket facade showed that the proposed axial connectors were not able to reduce the peak values of upper floor acceleration. However, comparison of responses for the structure with rigid bracket facade and structure with flexible shear bracket facade showed that the advanced connectors were able to reduce peak values of upper floor acceleration.

Fig. 12 again shows the efficiency of the flexible connections. After approximately three seconds of the El-Centro earthquake, the structure with Shear-50 connections began to reduce the response of the main structure. The reduction continued up to end of the excitation.

Table 8 shows high efficiency of the flexible shear connections in the upper storey in terms of reduction of top lateral acceleration.

Table 8 Comparison between top floor accelerations (mm/sec^2) of primary structure coupled with DSFs with optimal bracket connector stiffness during 1940 El-Centro Earthquake

Storey level	Type of bracket system used in double skin façade system						
	Rigid	Axial	Shear stiffness (N/mm)				
			100	50	20	10	1
10	45	40	36	30	31	31	34
9	38	34	31	27	28	28	31
8	28	26	23	21	22	22	24
7	25	2	21	20	21	22	23
6	25	23	21	22	22	23	25
5	24	22	20	20	21	21	23
4	20	18	16	16	17	17	19
3	17	16	14	14	15	15	16
2	13	13	12	12	13	13	13
1	10	9	9	9	9	9	9

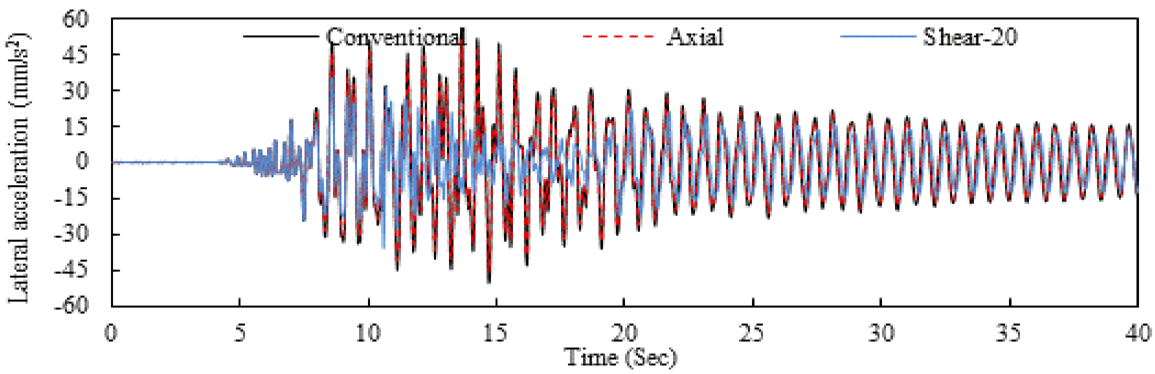


Fig. 13 Time-history top floor accelerations (mm/sec^2) of primary structure coupled with DSFs with optimal bracket connector stiffness during 1995 Kobe Earthquake

Table 9 Comparison between top floor accelerations (mm/sec^2) of primary structure coupled with DSFs with optimal bracket connector stiffness during 1995 Kobe Earthquake

Storey level	Type of bracket system used in double skin façade system						
	Rigid	Axial	Shear stiffness (N/mm)				
			100	50	20	10	1
10	56	51	49	37	36	38	41
9	43	39	37	31	30	32	31
8	42	38	36	32	31	33	30
7	37	34	32	30	28	30	27
6	33	30	29	27	26	27	24
5	31	28	26	25	24	26	25
4	25	23	22	22	21	22	20
3	24	21	20	21	20	21	20
2	18	16	15	16	16	17	16
1	10	10	10	10	9	10	10

Table 10 Comparison between maximum base shear of primary structure in kN coupled with DSFs with different bracket connector stiffness during the two earthquakes

Earthquake	Type of bracket system used in double skin façade system						
	Rigid	Axial	Shear stiffness (N/mm)				
			100	50	20	10	1
El-Centro	2,323	2,292	1,920	1,411	1,599	1,674	1,870
Kobe	3,001	2,989	2,801	2,450	1,597	1,611	1,801

Fig. 13 shows that the incorporation of shear connections to the structural façade has changed the effect of the seismic loading on the behaviour of the building system and produced more desirable results. The accelerations during intense shaking portion (between about 10 and 20 seconds) is clearly reduced while reductions after 20 seconds is marginal.

Table 9 shows that top floor acceleration of main structure can be reduced by using appropriate shear bracket in façade connections. It can be concluded that the flexible shear damping connections achieved good reductions of top acceleration for both earthquake excitations with the reductions being slightly higher for the Kobe earthquake excitation.

5.4 Base shear

Base shear is an estimate of the maximum anticipated lateral force that happens due to seismic ground motion at the base of a structure. Base shear directly depends on the input seismic acceleration and its value (V) depends on the following factors according to most seismic codes:

• Soil conditions at the site
• Proximity to potential sources of seismic activity (such as geological faults)
• Probability of significant seismic ground motion
• Level of ductility and over strength associated with various structural configurations and the

total weight of the structure

• Fundamental (natural) period of vibration of the structure when subjected to dynamic loading

For this example building, the base shear forces are directly obtained from the numerical analyses. The resulting base shear forces of the 3-D structural model are compared and tabulated in Table 10.

6. Conclusions

This paper proposes a novel system for structural building control through integrating façade system and dissipative mass dampers. This study has developed an innovative design concept that improves both architectural and structural performance by integrating a novel dissipative system into building structures. The resulting system can significantly reduce structural motions when subjected to earthquake excitation. The proposed system is intended to give the structural/façade designer an alternative technique to improve the earthquake resistance of a building structure. A substantial number of nonlinear time-history analyses related to general dynamic behaviour of the proposed system have been carried out to evaluate the nonlinear response of the system. With respect to the analysis and scope of this paper, the following summary and findings can be stated:

• The study on the 10-storey building demonstrated that by incorporating bracket elements with multi linear nonlinear behaviour into a moment resisting frame, it is possible to achieve a reduction in building response comparable to conventional façade system.

• Based on the results of the nonlinear time-history analyses performed on the ten-storey example building, one can conclude that the proposed system is able to significantly reduce inter-storey through the combined action of added stiffness and energy dissipation. This was achieved through careful selection of the shear connections. Therefore, the selection of bracket stiffness is a crucial design parameter for SDSF (movable/smart double skin façade) system.

• It is clear that with the variation of the façade type, the stiffening effect of the façade system on the structural system also varies.

• Building codes, all over the world, do not provide any direct design provisions for the seismic design of movable architectural glass elements, nor do standard laboratory test methods presently exist for evaluating the seismic performance of architectural glazing systems.

• The dissipative dampers would be located in the cavity space between the back face of the outer skin and front face of the inner skin. Although double skin facade (DSF) system design incorporates some sort of ventilation mechanism to dry the moisture in the cavity space, but high humidity will probably exist in the cavity due to rain or snow. It is highly recommended that, the damper unit be protected against corrosion through sealing the holes and gaps (along the edges).

• It is expected that, structural columns on which the SDSFs (movable/smart double skin façade) are attached are subjected to concentrated loads along their heights during earthquake excitations. But, as this load is resisted by the structural frame, it should not create any major issues during or after moderate earthquakes.

• It has been concluded that if values between 20-50 N/mm is selected for shear stiffness, the highest performance will be achieved in response reduction of primary structure. The range of 20-50 N/mm found as optimum for the 10 storey example building found to be effective for a 30 storey building as well and hence this value can be used as the initial value for the design and analysis, subject to further optimisation if required.

• The findings of this paper will be validated experimentally using a shake table and a double skin facade component system as part of future studies.

The adaptability of the movable/smart façade system to different types of excitations was addressed. A cost/ benefit analysis of the system has been performed to account for the structural effects on the building during its life cycle. The results of this part will be published in future articles.

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