Politecnico di Milano

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IMPROVING THE SEISMIC BEHAVIOR OF ARCHITECTURAL GLAZING USING FRICTION DAMPING CONNECTORS

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Abstract

Architectural glazing has been a widely practiced envelope system since the introduction of the modern architecture. This adaptation is basically driven by transparency and natural illumination which can be provided using glazed systems. High efficient energy behavior of double skin façade is one of the major reasons of increased interest among architects for applying glass skins over their buildings. Aesthetical features of glass have also made this material often used in the envelopes of prestigious and highly invested-in buildings.

The post-earthquake surveys have shown that, although having the buildings designed according to the most contemporary seismic design codes will protect the structure of the building during an earthquake, these provisions are hardly sufficient for avoiding damage to the nonstructural elements of the building. Among all the nonstructural elements of a building the glass envelopes and store-front windows are the most vulnerable ones to damage during an earthquake. That is mainly due to the high rigidity and stiffness of these systems in the in-plane direction which results in attracting forces, combined with fragility and delicacy of these systems and its components with respect to structural members. It is also shown that the deflections and displacements which occur in the structure of the building during severe situations like earthquakes are likely to be the main cause of damage to glazed envelope systems.

Application of advanced connection devices to protect the glazed envelopes is studied in this research as an alternative approach to the typical provisions of glass protection which is to provide clearance between the glass panel edges and the supporting frames of the envelope. Among different advanced connectors introduced in the literature, the friction damping connections are selected to provide a desired level of isolation between the envelope system and the structure of the building. This selection is based on the simplicity of their mechanisms and their ability to confine the transferred forces and moments to limited values.

A specific friction damping connector, named the friction moment rod, is introduced by the author that has the ability to be adapted in almost all of the glazed envelope systems with complex geometries and high aesthetical demand over composing elements.

Analytical approach is presented as a basis for tuning the friction connecting devices in glazed systems which is based on the mechanical strength of the glass panels connected with friction connectors and their behavior during earthquake. And finally with the help of numerical simulation, the analytical approach is evaluated and also other effects of the friction connectors on glazed envelopes are investigated, in order to find properties contributing to the tuning of friction connection devices.

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1. Introduction

1.1. Statement of the problem

Architectural glass exterior systems are considered to be one of the largest systems, as the entire building skin or part of its envelope, contributing to the proper function of the building. Building envelope, especially the façade system over high-rise buildings and structures, approximately cost over 20% or more of the total construction budget (Anonymous2009) and is considered to be an economically significant portion of the building. With the exception of few guidelines in building design codes, currently there is a lack of design approaches provided for the designers and engineers in proper selection of the glazing details effectively in order to resist earthquakes.

The existing design guidelines for architectural glazing are limited to very typical glazed systems in terms of geometry and technology. And also in many cases it limits the freedom of designers to from realizing their desired forms. In order to protect the architectural glazing against seismic actions the concept of offering mechanical compatibility between the structure of the building and its envelope is brought on in this research, which is in contrast with the common practice of offering clearance between the elements of the envelope system.

The main cause of damage to glass elements, during an earthquake, are the in-plane deformations within the glazed system generally caused by the deflections and displacements in the structure of the building, figure (1). Using the advanced connection devices it is possible to avoid these displacements to be transferred to the envelope, while still managing to keep the structure of the building as its main support system.

Friction damping devices, often used to reduce the seismic response of structures, are proposed in this research, to control the forces applied by the structure one the envelope system. With properly adjusting the friction connecting devices it is possible to maintain a desired level of isolation between the envelope and the building structure, and provide mechanical compatibility between the two systems.

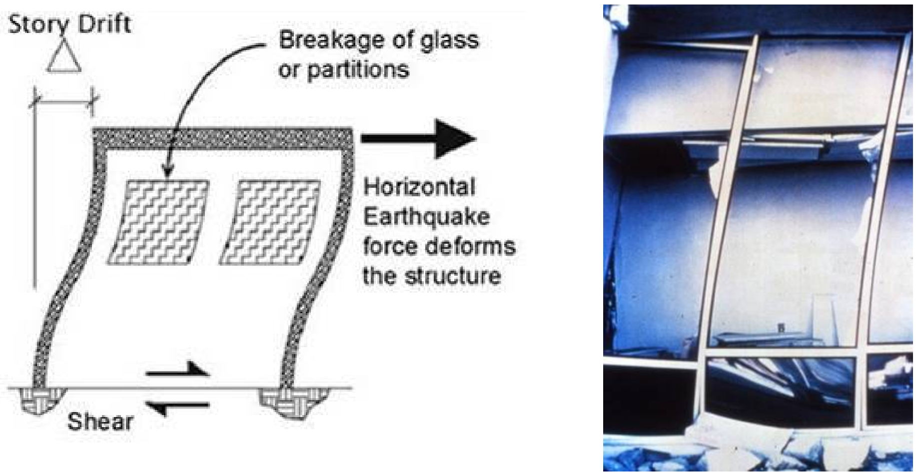


Figure 1: Seismic lateral drifts causing damage to envelope systems

1.2. Objectives

The ultimate goal of this research is to propose connection systems which result in compatible mechanical behavior between the main structure and the envelope of the building during severe loading conditions. The applicability and adaptability to different glazing systems are considered to be important factors for the proposed system. The factors below are considered to be major aspects of applicability and adaptability for the connection devices:

Manufacturing costs

Simplicity of production and installation

Ability to be connected to different elements in various positions and orientations

Having proper size, geometry and aesthetical characteristics

The other objective of this research is to provide reliable instructions to designers for tuning the friction connections to perform in the expected behavior. This will be done by an analytical study of the glass panels subjected to the forces applied by the friction damping devices. The results of the analytical studies of the glass panels will be later evaluated by numerical simulations. Aside from guaranteeing the safety of the glazed system, serviceability and functionality of the envelope would need to be further investigated in terms of air and water tightness. This will be assured by avoiding failure in the pre-formed or the silicon sealant gaskets of the envelope system.

2. State of the art

2.1. Architectural glazing

There can hardly be found materials that match the popularity of glass among engineers and architects. The great impotence that glass has gathered among other materials is associated with its ability to transmit light and providing a transparent environment.

Being able to use the transparent environment and capturing the warmth and brightness of the sun inside the building was a major problem up to the beginning of the twentieth century. In the first half of the twentieth century due to introduction of new materials to the building industry, like steel and concrete, and more complex structural solutions, the adaptation of glass into the building envelope was more and more made possible and changed its role from small openings in the façade to considerably large surfaces covering most of the building envelope.

(a) (b)

Figure 2: Dematerialization of the façade: a) 'Auditorium Building' Sullivan and Adler, 1889, b) Stary Browar shopping center, Poznań, Poland

From wall to skin:

The adaptation of the glazed elements in the building exterior is parallel to the dematerialization of the building façade, which is also coupled with relieving the façade from its load-bearing function and introducing the frame elements which transferred the load to the building foundation. Ultimately the dematerialization of the façades led to them being a skin boundary around the building separating the inside and outside environments. The boundary that we now refer to as the building envelope. (UNI 2005)(UNI 2005)

Although before the middle of the twentieth century there has been numerous examples of glass construction, mostly for coverings of railway stations, green houses, passages and early modernism of 20’s and 30’s that resulted in large private glass houses, the real breakthrough in the glass architecture came after the world war II when economic, technological and aesthetic factors all together forced the rapid spread of utilizing glass as building materials. It was at this time that technical advances of glass manufacturing, along with the sensation of modern characteristics embodied in it, made the glazed envelope the symbol of modern architecture. Towering glazed office buildings that were used as the headquarters of giant multinational companies represented growth, confidence and development within those companies as they still do. Even within city scale a silhouette of high-rise glazed skyscrapers, sharply reflecting the sun during the day and illuminating light as a sign of livelihood during night became signatures of wealth and prosperity.

Figure 3: New York skyline

The leading examples of modern glass architecture were first realized in the United States. Since in contrast with Europe that was dealing with problems of a post war environment, America was considered of having a reasonable economic growth suitable for investments, and at the same time it became home of many of the avant-garde immigrants during the years of third Reich like Mies van der Rohe and Walter Gropius. (Institut fu\0308r Internationale Architektur-Dokumentation. 2007)

Being the head of the Illinois Institute of Technology in Chicago and given the task to design the new university campus, Mies van der Rohe had the chance to reinterpret the idea of curtain wall which later became a distinguished aspect of the high rise buildings he designed. One of the first implementations of the glazed curtain wall where the façade of his buildings constructed during the 80's along the lake shore drive, in Chicago.

(a) (b)

Figure 4: a) '860–880 Lake Shore Drive' apartments, Mies van der Rohe; b) 'Pacific design center' by Cesar Pelli, 1975

Later the advances in load bearing silicon (structural silicon glazing) and other fixing techniques made possible the cladding of rooftops and more complex shapes with glass and having a smooth firmly skin eliminating the panel frames and maximizing the glazing of the envelope. Pacific design center designed by Cesar Pelli In 1975 Is one of the first structures benefiting from an all glazed envelope.

Other leading examples of modern glass architecture following the Second World War, that were realized in the US are: the skyscrapers 'Lever Building', 1951-52, by the architectural office SOM, the 'Seagram Building', 1954-58 by Mies van der Rohe, both in New York, and the 'Hancock Tower' in Boston, 1967-76 by I.M. Pei and Partners in collaboration with H.N. Cobb. Figure (5).

(a) (b) (c)

Figure 5: a) 'Lever Building', 1951-52, by the architectural office SOM; b) 'Seagram Building', 1954-58 by Mies van der Rohe; c) 'Hancock Tower' in Boston, 1967-76 by I.M. Pei and Partners

Disadvantageous energy loss in winter and overheating in the summer were combated passively with tinted glass and actively using mainly energy intensive mechanical air conditioning. After the energy crisis of the 70's double glazing against energy loss and reflective glazing against overheating were increasingly employed. In the meantime the glass industry was able to put double glazing with excellent thermal values on the market, thereby extensively reducing the significance of heat loss. However, undesirable heat gains due to solar radiation continue to pose problems.

Curtain walls are a possible solution to this problem. They are characterized by the addition of a single glazed skin in front of the double glazed building façade. The shading device can be located in the cavity between the two façades to control the heat gain due to solar radiation.

In the early 90's high quality façade constructions, in particular double skin building envelopes, were developed and often promoted as the definitive solution to energy loss. Forerunners of double skin constructions are the projects by Le Corbusier for the not awarded competition design for the People's Palace in Geneva, 1927, and for the building of the Centro-Soyus in Moscow, 1929. The impetus for this solution probably came from traditional double windows which Le Corbusier knew from his country Switzerland. Double skin façades had however already been built prior to this but had been recognized as works of civil engineering and not works of architecture. The production hall of the Steiff Company, Giengen on the Brenz, 1903 by R. Steiff, and the 'Hallidie Building', San Francisco, 1915-17 by W. Polk, are early examples of double skin glass façades. Figure (6)

(a) (b)

Figure 6. a) Production Hall Steiff, Giengen on the Brenz, 1903, Architect R. Steiff; b) 'Hallidie Building', San Francisco, 1915-17

Double façades are still a very interesting point of discussion in the building envelope in terms of materials, ventilation, shading, translucency and compatibility with the building structure. Where in this research there will be a great deal of attention given to the mechanical behavior of such systems.

2.2. Curtain walls

A curtain wall is defined as an exterior wall on a building, which does not support the roof or floor loads but is connected to the structural frame, and is an element of the larger building envelope system. While a CW encompasses systems that use various material cladding such as metal panels and stone, one of the most popular configurations is a metal frame assembly glazed with architectural glass. They ate usually consists of vertical and horizontal structural members, connected together and anchored to the supporting structure of the building and infilled, to form a lightweight, space enclosing continuous skin, which provides, by itself or in conjunction with the building construction, all the normal functions of an external wall, but does not take on any of the load bearing characteristics of the building structure (UNI 2005). Glass CW systems have become a common building component as mass load bearing wall systems slowly transitioned and were replaced with cavity wall systems during the twentieth century and lightweight wall system options were needed. Curtain walls have many various functions, some of which are harder to achieve effectively than others. These wall systems have requirements which include structural load transfer and resistance, water infiltration protection, air infiltration control, condensation prevention, energy management, sound attenuation, safety, maintainability, constructability, durability, aesthetics, and economic viability (Curtis 1987)

Description of curtain walling types:

The classification of types of curtain walling varies but the following terms are commonly used:

-Stick

-Unitized

-Panelized

-Spandrel panel ribbon glazing

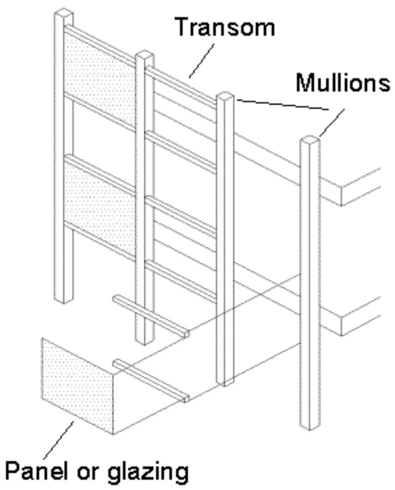
-Structural sealant glazing

-Structural glazing - bolted assembly & suspended assembly

Stick system curtain walling

The general arrangement of a stick system curtain wall is shown in Figure (7). Horizontal and vertical framing members ('sticks') are normally extruded aluminum protected by anodizing or powder coating, but may be cold-rolled steel (for greater fire resistance) or aluminum clad with PVC-U. Members are cut to length and machined in the factory prior to assembly on site as a kit of parts: vertical mullions, which are fixed to the floor slab, are erected first followed by horizontal transoms, which are fixed in-between mullions. Into the framework are fitted infill units, which may comprise a mixture of fixed and opening glazing and insulated panels (which may have metal, glass or stone facings). These units are typically sealed with gaskets and retained with a pressure plate.

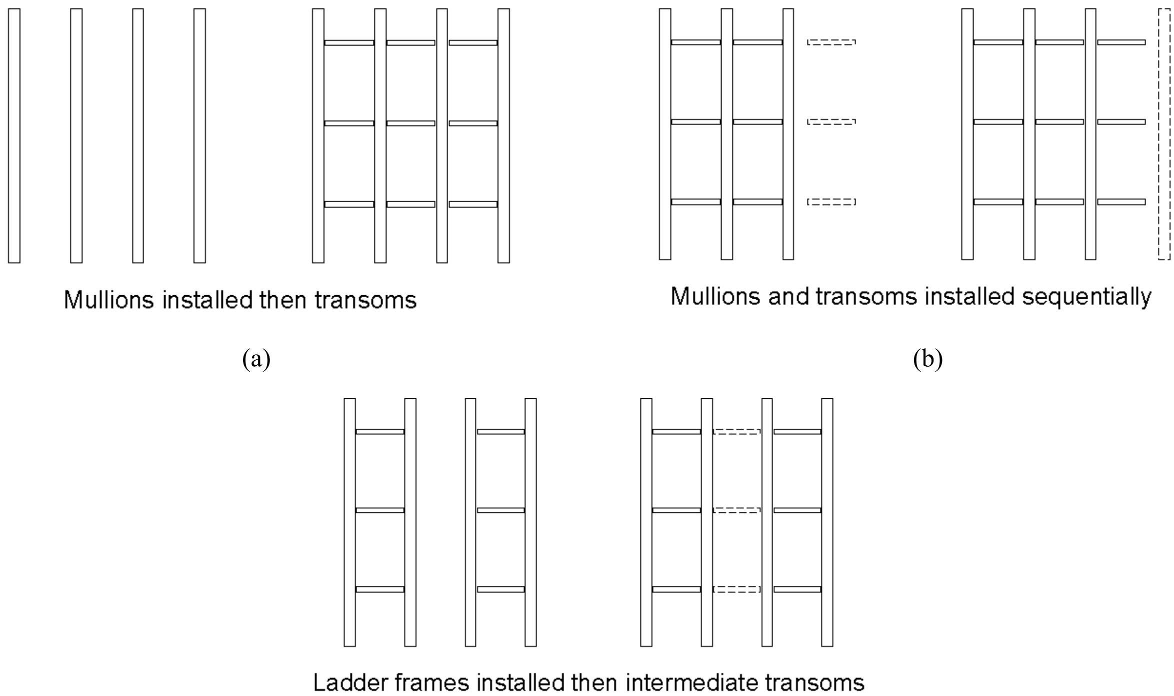
Stick curtain walling is very common and versatile and can be used for anything from 'glass towers' tens of storey high to single storey shop fronts. Because of the number of joints in stick curtain walling it is generally very good at accommodating variability and movement in the building frame. It is also suitable for irregular shaped buildings. Assembly is slow compared with pre-assembled systems and performance (e.g. weather tightness) is dependent on knowledgeable installers



(a) (b)

Figure 7: a) Stick system curtain walling; b) IIT Crown hall Mies van der Rohe, 1940

Many manufacturers (systems suppliers) produce standard stick systems. Insulated panels, usually designed for the project, may be faced with anything from aluminum or steel sheet, to glass or expensive stone composites Stick system curtain walling may be erected in one of three sequences shown in Figure (8).



(c)
Figure 8: Different construction techniques for stick curtain walls

Unitized curtain walling

Unitized systems comprise narrow, storey-height units of steel or aluminum framework, glazing and panels pre-assembled under controlled, factory conditions, Figure (9). Mechanical handling is required to position, align and fix units onto pre-positioned brackets attached to the concrete floor slab or the structural frame. Unitized systems are more complex in terms of framing system, have higher direct costs but The smaller number of site-sealed joints in unitized curtain walling simplifies and hastens enclosure of the building, requires fewer site staff and can make such systems cost effective. If construction joints interlock consideration must be given to how damaged units could be removed and replaced. The reduced number of site-made joints compared with stick systems, generally leads to a reduction in air and water leakage resulting from poor installation.

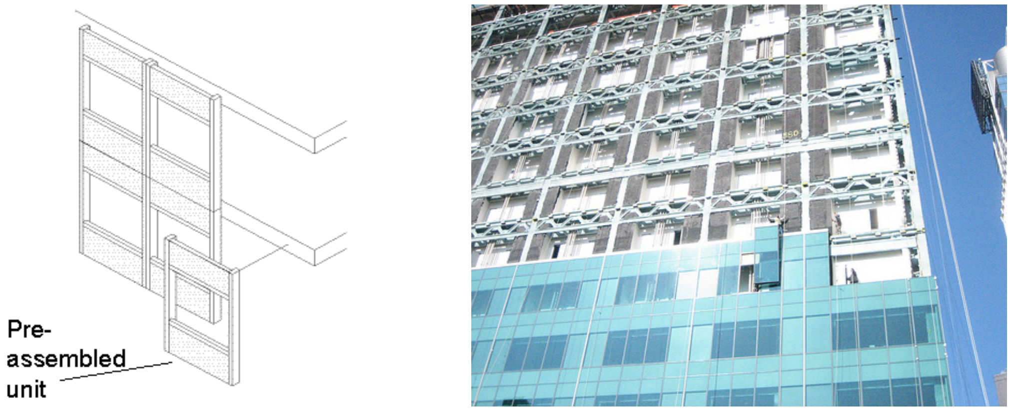


Figure 9: Unitized curtain wall systems

Panelized curtain walling

Panelized curtain walling consists of large prefabricated panels of bay width and storey height, which connect back to the primary structural columns or to the floor slabs close to the primary structure, Figure (9). Fixing the panels close to the columns reduces problems due to deflection of the slab at mid span, which affect stick and unitized systems.

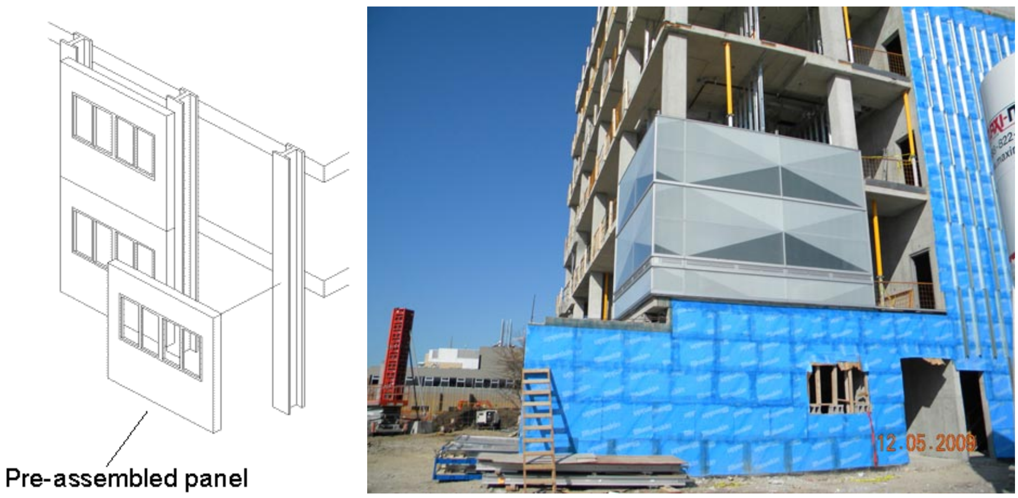


Figure 10: Panelized curtain wall systems

Panels may be of precast concrete or comprise a structural steel framework, which can be used to support most cladding materials (e.g. stone, metal and masonry). Structural steel panelized walls are known as ‘truss walls’ in North America. Aluminum or galvanized steel skins are generally fixed to the frame with insulation in the cavity. The wall construction is then completed by a plasterboard lining and external cladding.

Some do not differentiate between unitized and panelized systems, the main different between the two systems are the weight and size of the systems which result in fundamental differences in manufacturing and assembly. Usually panelized construction may have significant internal steel structure to support the extra weight, or may consist of precast concrete panels with openings for windows.

Spandrel panel ribbon glazing

Spandrel panel ribbon glazing is a long or continuous run of vision units fixed between spandrel panels supported by vertical columns or the floor slabs, Figure (11).

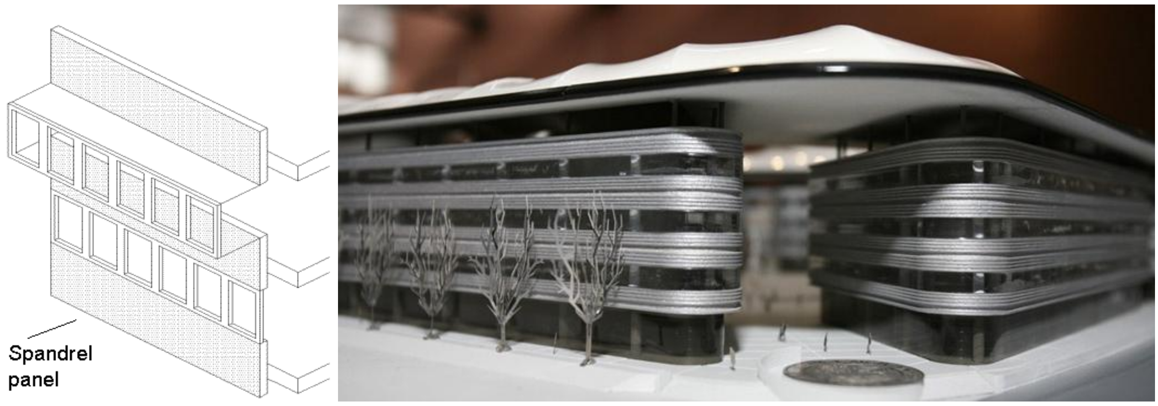


Figure 11: Spandrel ribbon curtain wall systems

Glazed areas may comprise: Several standard windows fixed together on site by joining mullions, Pre-glazed, bay width, factory-assembled frames, or Individual framing sections and glass infill panels which are site assembled.

Ribbon glazing is often used in conjunction with spandrel panels, that is, horizontally spanning prefabricated or precast concrete units. It may also be used with spandrels comprising up stand walls faced with rain screen panels. Ribbon glazing/spandrel panel construction generally results in buildings having a horizontal banded or strip appearance.

Structural sealant glazing

Structural sealant glazing is a form of glazing that can be applied to stick curtain wall systems and windows, particularly ribbon glazing. However it can also be used in unitized and panelized systems. Instead of mechanical means (i.e. a pressure plate or structural gasket), the glass infill panels are attached with a factory-applied structural sealant (usually silicone) to metal carrier units which are then bolted into the framing grid on site. External joints are weather sealed with a wet-applied sealant or a gasket, Figure (12).

(a) (b)

Figure 12: a) Structural sealant glazing; b) 'Quay West' building, Manchester by The Ratcliff Partnership Ltd

These walls are attractive to architects as they offer a smooth or semi-smooth facade. Structural sealant glazing has been used in the USA for around 30 years where it was initially site applied direct to the framing. However, this is no longer acceptable due to difficulties of application and replacement and all structural silicon joints are now made in a factory.

Structural sealant glazing systems can have sealant on two opposite sides or on all four-sides with or without the weight of glass supported mechanically. Generally, the glass is mechanically supported to reduce the size of the sealant bead.

Structural sealant glazing can be used to create a building exterior that is free from protrusions, but the framing system will be visible at night when backlit. Structural sealant glazing is more widely used on 'prestige' buildings and may be produced as a standard system, or on a project-by-project customized basis. The framing members are often more widely spaced than for traditional stick systems.

Any of the previous types of curtain walling and ribbon glazing could incorporate structural silicon glazed elements.

Structural glazing - bolted assembly & suspended assembly

These two systems are usually referred to as spider-glass curtain walls. In bolted assembly, sheets of toughened glass are assembled with special bolts and brackets and supported by a secondary structure, Figure (13), to create a near transparent facade or roof with a flush external surface.

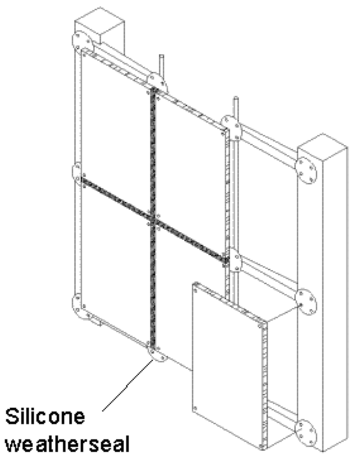


Figure 13: Bolted structural glazing

A multitude of discreet or prominent secondary structures can be designed (e.g. space frame, rigging or a series of mullions) which support the glazing through special brackets. The joints between adjacent panes/glass units are weather sealed on site with wet-applied sealant.

In case of suspended assembly the glass is fixed together with corner, rectangular or cross shaped patch plates and the whole assembly is then either suspended from the top or stacked from the ground and wet-sealed on site, Figure (14).

Figure 14: suspended structural glazing

Suspended glazing systems utilize the minimum amount of framing for a given glass area and are used as glazing features on prestige buildings, but also for prestige atria on otherwise simple buildings.

Glass fins may be used to brace the assembly. In some designs a light truss stabilizes the wall and transfers wind loading, while the weight of the glass is transferred through the corner plates and suspension system

Many systems are now available for the use of glass in facades, all aimed at achieving maximum transparency by reducing the non-support structure. In close collaboration with the glass industry, many architects have worked on the possibilities of glass for use in buildings. The two most common systems include the Patch Plate Glazing and the Dot Point Glazing.

The patch plate suspended glazing system is the earliest system. It comprises a series of specially processed and toughened glass lites bolted together at their corners by small metal patch fittings. Pane-to-pane joints are sealed with a silicon building sealant, and toughened glass stabilizers or steel plates are used at each vertical joint to provide lateral stiffness against wind loading. The assembly is suspended from the building structure by hangers bolted to its top edge and is sealed to the building in peripheral channels. The concept of the design ensures that the façade is at all times, “floating” in the peripheral channeling and the problems which might arise due to differential movement between components are eliminated. Sealing is carried out at all joints in the façade using either the structural or weather- proof sealant depending on the load transfer intent of the system. The principle behind the design of the fittings for a suspended glazing assembly is that all in-plane forces transferred between components are resisted by friction developed at the metal/gasket/glass interfaces arising from the tension of the fixing bolts. The friction grip is of particular importance in the design of the splice joints and root support of the stabilizers where the bearing strength of the holes are unlikely to resist the turning moments generated in the stabilizers which is caused due to the wind forces. If required, the coefficient of friction at the metal/gasket/glass interfaces can be enhanced by applying a suitable adhesive. The façade lites resist lateral wind forces through the small metal patch plates supporting the four corners of adjacent lites off the stabilizers, These metal patch plates clamp the glass at the corners of each pane, developing significant stress concentrations at the edges of the plate and around the bolt holes. To safely design this system, it is essential to have detailed knowledge of the stresses generated and also the knowledge of the strength of toughened glass.

One of the features of a patch plate suspended glazing assembly was that it could not be used in conjunction with sealed insulating glass units (IGU) or any non vertical applications, such as the sloped glazing. The Dot Point Glazing System (DPS), sometimes called the "Spider" glazing can be used for both cases. It has been increasingly popular to support glass using bolted fixings directly connected to the glass. These fixings allow improved transparency and offer architectural opportunities in detailing the bolted connections and fittings. It is capable of fixing either the monolithic (single lite) or insulating toughened glass to any structure. In some cases, glass mullions are used to form part of the substructure to which the glass is attached. This system is used for vertical or sloped glazing and can be incorporated as a complete cladding system. The fittings are designed to support the weight of the glass by direct bearing of the bolt through the bushing on the hole in the glass. The fitting is also designed to give minimal clamping by attaching the fixing bolt which is flexible and allow rotation of the glass. Recent designs on the fixing bolts also have provision for articulation or ball-type joints. The overall effect is to significantly reduce the stresses developed in the glass in the region of the planar fitting compared to those developed around the patch plates. It allows the panes of glass to move in relation to their supporting structure while maintaining a smooth outer surface appearance. When large piece of glass bend under wind loads, a high load concentration will occur in the area of the hole if the fixing bolt is firmly fixed to the supporting structure. In brief, the above underlines the following design issues:

- A supporting system at point locations causes high load concentrations;

- A hole in the glass is very sensitive, especially when it is countersunk;

- Enlarging the hole can reduce the load in the bearing surface;

- Articulated assemblies allow the differential movements between the glass and the structure to be absorbed.

Because frictional forces and hence the clamping forces are not important in the design of the planar fixing, its design is suitable for use in the insulating glass version. In this, the outer lite provides the main load bearing capability. Careful research into the stresses, especially around the hole is necessary in the design of this type of system. Because the glass lites are individually mounted to the structure, there is no restriction on the height of the buildings which can be glazed.

Most standard typical fittings consist of two or four fixed arms, depending on the location of glass, whether glass is adjacent to the concrete structure or to the glass itself. Considering a typical 4-legged fitting with an M10 bolt at each arm, enlarged holes usually diameter 22mm are normally used at the end of the nodes on both the top two arms. These enlarged holes allow for both horizontal and vertical adjustment during installation. A horizontally elongated hole and a nominal hole, usually 11 X 22mm and diameter 11mm respectively are provided at the bottom two arms. As most glass wall systems are suspended, these arrangements are necessary for proper transfer of loads.

2.3. Building structure and envelope compatibility

There are for categories defined in literature as building systems (Schwartz 2001). These four systems are:

-Structure

-Envelope

-Interior parts

-Mechanical systems and services

There are other classifications in other references but almost all of them share the ones mentioned above. Each one of these categories is subject of research and design in different academic and technical fields. Yet another subject of great interest is the relationship between these systems and the influence of one of them on the others and their level of integration.

In order to better understand the interaction between the structural system and the envelope it is necessary to define integration levels between these two systems. Five levels of integration are introduced as:

Remote:

The case in which the two systems are completely physically separated from one another and they are just coordinated in terms of functionality. Examples of this type of integration are noise barrier walls protecting the buildings in highly noise polluted areas –as in airports and highways- having their own supporting structure

Touching:

In this case the envelope of the building has its own supporting structure for carrying the weight of the envelope but rests on the main structure of the building, relying on it mostly for lateral loads like wind and earthquake. Examples of this type of integration are the double façades having their own structure which are connected to the main structure of the building with horizontal elements to transfer the lateral loads.

Connected:

In this one system is mechanically fastened and dependent on the other. Examples of this type are glazed and non-glazed claddings which do not have a support system of their own and are connected to the main structure of the building with brackets and adjusting frames.

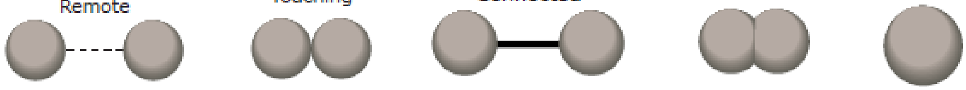
Meshed:

In this case the two systems are completely attached to each other and they nearly occupy the same space and the envelope system does not have any supporting structure, it could be said that the two systems are glued together. Examples of this type of integration are buildings that have stones, ceramics and bricks as their exterior envelope which is glued to the main structure of the building.

Unified:

In this case the systems share the same physical components and there are no more two separate systems and there is just one physical system which acts both as the building structure and its envelope. Massive concrete structures which benefit from a concrete exterior that is also the main structure of the building are the examples of this type.

Table 1: integration diagram



Integrating level	Remote	Touching	Connected	Meshed	Unified
Physical connectivity	No connectivity	Relying on	Fastened with connections	glued	united

It is obvious that in the first and the last type, the mechanical compatibility of the structure with the envelope is meaningless. But in the three middle cases it is very important for the two systems to be well adjusted in two cases of displacement tolerances and deflections.

Usually the problems with displacement tolerances are the ones that occur during and before production and construction phases. The displacement tolerances that are accepted during construction of the structural system often tend to centimeters. While considering the great deal of importance given to the exterior envelope of the building in terms of visual appearance, water and air permeability and etc. the intensity of tolerances in the envelope is of the order of millimeters. This issue is often tackled using connection details having brackets with ellipsoid-shape holes that permit the installation to adjust these tolerances in all of the three dimensions. Other than connection brackets there are a lot of other connection details that allow the installers to justify the placement of the envelope in the right position.

Dealing with the problems regarding the deflection compatibility between the structure and the envelope is much more complicated than justifying the installation tolerances, mainly because they usually occur after the final fixing of the connections between the two systems and cause stresses and forces in the envelope elements. These deflections that appear in the structure of the building and then transferred to the envelope are of two types, static and dynamic. The static ones are often the result of added live loads to the structure like furnishings or sometimes due to gradual sinking of the building foundation. The dynamic ones are the drifts in the structure due to dynamic loads like earthquake and wind forces.

Another problem of adjusting the two systems for deflection displacements is that usually these displacements are not very easy to define. although structural design codes have various guidelines and restrictions for structural designers from the point of view of deflection, trying to minimize these deflections and to prevent them from increasing above some values but almost all of these restrictions are applied to keep the behavior of the structure under the theoretical designing assumptions and to keep the structure in an operable state, and They are far behind the limitations required to maintain the properties of the envelope. Of course it is reasonable that in extreme situations like earthquakes the center of attention be on the safety of the structure and its functioning but in some expensive projects that the cost of the exterior of the structure exceeds thousands of Euros for meter square, there must be some considerations to protect these large investments.

2.4. Architectural glass provisions in seismic codes

The ASCE 7-02 code (Rush, American Institute of Architects. 1986) in section 9.6.2.10 instructs the designers of glazed curtain-walls, store-fronts and glazed partitions to provide enough clearance between the glass panel edges and the frames of such systems to avoid damage to the glass. The following sections (9.6.2.4.2–9.6.2.10.2) are the verbatim seismic design provisions for architectural glass included in ASCE 7-02. Actual section, equation, table, and reference numbers are cited below for accuracy, and for ease when referring to the ASCE 7-02 document:

1. **9.6.2.4.2 Glass**. Glass in glazed curtain walls and storefronts shall be designed and installed in accordance with sec.9.6.2.10 (see below).

2. **9.6.2.8.2 Glass**. Glass in glazed partitions shall be designed and installed in accordance with Sec. 9.6.2.10 (see below).

9.6.2.10 Glass in Glazed Curtain Walls, Glazed Storefronts, and Glazed Partitions

9.6.2.10.1 General. Glass in glazed curtain walls, glazed storefronts and glazed partitions shall meet the relative displacement requirement of Eq. (9.6.2.10.1-1)

$$\Delta_{fallout} \geq 1.25\, I D_P \qquad (9.6.2.10.1\text{-}1)$$

Or *13 mm*, whichever is greater

Where: $\Delta_{fallout}$ is the relative seismic displacement (drift) causing glass fallout from the curtain wall, storefront or partition; D_p is the relative seismic displacement that the component must be designed to accommodate (Eq. 9.6.1.4-1); D_p shall be applied over the height of the glass component under consideration; and I is the occupancy importance factor (Table 9.1.4).

Exceptions:

1. Glass with sufficient clearances from its frame such that physical contact between the glass and frame will not occur at the design drift, as demonstrated by Eqs. (9.6.2.10.1-2), shall be exempted from the provisions of Eq. (9.6.2.10.1-1)

$$D_{clear} \geq 1.25\, I D_P \qquad (9.6.2.10.1\text{-}2\text{-}a)$$

$$D_{clear} = 2c_1\left(1 + \frac{h_p c_2}{b_p c_1}\right) \qquad (9.6.2.10.1\text{-}2\text{-}b)$$

Where h_p is the height of the rectangular glass; b_p is the width of the rectangular glass, c_1 the clearance (gap) between the vertical glass edges and the frame; and c_2 the clearance (gap) between the horizontal glass edges and the frame.

2. Fully tempered monolithic glass in Seismic Use Groups *I* and *II* located no more than *3 m* above a walking surface shall be exempted from the provisions of Eq. (9.6.2.10.1-1).

3. Annealed or heat-strengthened laminated glass in single thickness with interlayer no less than *0.76 mm* that is captured mechanically in a wall system glazing pocket, and whose

perimeter is secured to the frame by a wet glazed, gunable curing elastomeric sealant perimeter bead of *13 mm* minimum glass contact width, or other approved anchorage system, shall be exempted from the provisions of Eq. (9.6.2.10.1-1).

Discussion of seismic design provisions:

In essence, Eq. (9.6.2.10.1-1) requires that the resistance to glass fallout of an individual glass panel be greater than the relative seismic displacement demand the component must accommodate as a result of being attached to the primary structural system of the building. Thus, the resistance to glass fallout is the capacity of a given glazing system. *In the absence of special drift accommodating connections between the main building frame and the curtain wall framing members, this relative seismic displacement demand is governed by the calculated seismic inter-story drifts for the specific building being designed for earthquake loading conditions.*

2.5. Energy dissipation and mechanical isolation

The concepts of energy dissipation and mechanical isolation are leading examples of developing innovative concepts to better protect the structure against environmental forces like wind and earthquake or unexpected occurrences like blasts and etc.

The design concept of energy dissipation and mechanical isolation is to absorb or consume a portion of input energy within specific deices in the structural system thus minimizing the energy that acts upon structural members, or providing a desired level of isolation between some elements and avoiding some members to impose forces on others. This is in contrast with the traditional approach that is to provide a combination of strength and ductility to resist the imposed loads.

These design strategies have been thoroughly studied and practiced in the structural design of many buildings usually against seismic actions. Having considered the change of approach in seismic design of buildings from safety criterion to serviceability criterion, more and more attention is now given to protection of nonstructural elements during an earthquake, among them the building envelope. Below there is a brief study of previous researches in adaptation of energy dissipation and system isolation technologies in building cladding and curtain walls.

2.5.1. Cladding Research

Only a few researchers have conducted experimental research on the performance of currently used cladding systems.

Wang (1987) conducted research on the seismic behavior of curtain wall cladding elements on a full scale test frame. Both slotted hole and push-pull systems were tested. Rihal (1988a), 1988b, conducted studies of the seismic behavior and design of pre-cast façades/cladding and push-pull connections in low/medium rise steel frame buildings, including experimental testing.

A significant continuing direction in heavy cladding research is that of the study of panel frame interaction: an allied field of study is that of the use of cladding as an integral part of a lateral bracing system.

Thiel et.(1986) published a feasibility study on seismic energy absorbing cladding systems. Henry and Roll, (1986) conducted analytical studies of cladding frame interaction. Sack et al (1989) conducted experimental studies on cladding /frame interaction. Pall (1989) researched and developed a friction-damped connection for pre-cast concrete cladding. The connection has been used in several projects.

The current main researcher in this area is Goodno and his team at Georgia Institute of Technology, for example Goodno et al. 1989, and Pinelli et al. 1993. Goodno et al 1998 describes the analytical and experimental studies of several developments of "advanced" cladding connectors. Results showed that either up to 41% reduction in peak displacement response could be achieved from the baseline (as-built) configuration by retrofitting advanced cladding connectors, or else as much as a 27% reduction in structural weight (in the longitudinal direction) could be achieved for the same baseline response level. Figure (15)

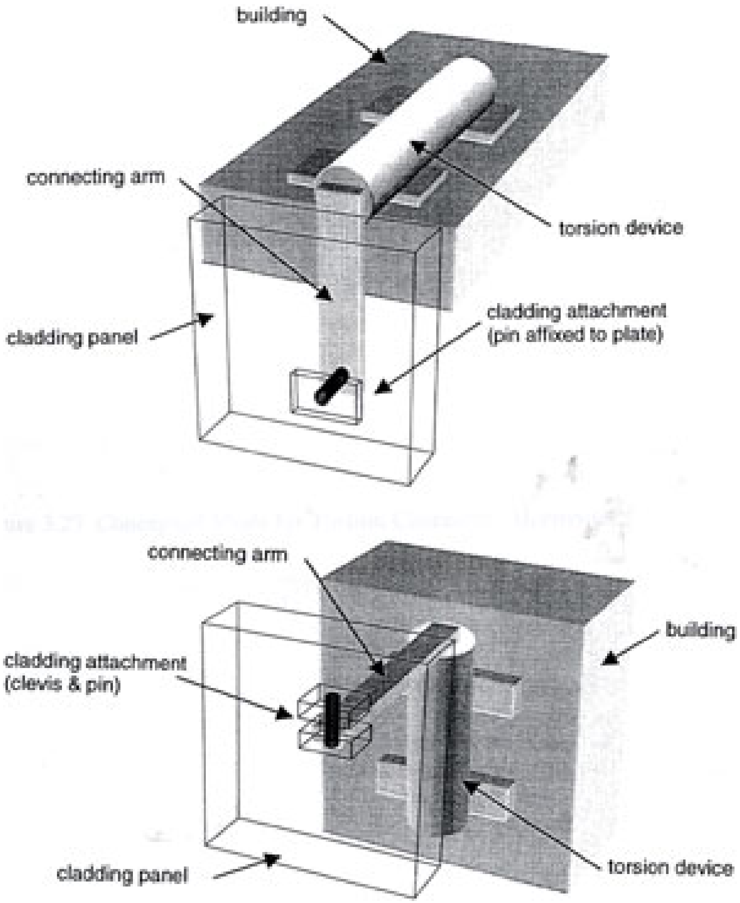


Figure 15: Torsion damping connection Top, Horizontal; Botton Vertical.
(Goodno et al, 1998)

This advanced connection is capable of developing good energy dissipation qualities through torsion deformation, in a similar manner to the torsion bars in an automobile suspension. The torsion device consists of circular torsion element mounted inside a concentric tube and attached to a vertical surface of a building structure (in the bottom figure above). The outer tube supports the torsion element and fixes it at the lower end, providing rotational bearing support at the upper end of this figure. An arm connected to the torsion element is used to convert the inter story drift (left-right in the illustration) into rotation. The arm is attached to the cladding panel (shown in wireframe mode) with a pin and clevis. The upper figure shows a horizontal application.

2.5.2. Glazing and Curtain Wall Research

Study of the behavior of glazing under seismic conditions has also been limited so far. Bouwkamp and Meehan (1960) investigated the performance of window panels subjected to racking loads. Cupples, (1985) performed racking tests on a Robertson-Cupples curtain wall system to evaluate the overall performance of the wall system and evaluate the glass-to-frame connection details. Lim and King (1991) investigated the seismic performance of curtain wall systems at the Building research Association of New Zealand, including in-plane dynamic racking tests on full-scale glass and aluminum curtain wall assemblies.

In the early 1990s Richard Behr and a team at the University of Missouri, Rolla and later at Pennsylvania State University, University Park, began a long-term program of experimental testing of the seismic behavior of a number of glazing systems. These included store-front glazing, curtain walls with a variety of glass types and glazing techniques, and glazing with applied film. This work led to a number of recommended revisions to the *NEHRP Recommended Provisions for the Seismic Regulations for New Buildings and Other Structures* (FEMA 302) which were published in the *2000 NEHRP Provisions (FEMA 368)*. Members of the team participated in developing a recommended *Dynamic Test Method for Determining the Seismic Drift Causing Glass Fallout from a Wall System*, published as AAMA 505.6-01 and referenced as *Recommended Static Test Method for Evaluating Curtain Wall and Storefront Systems Subjected to Seismic and Wind Induced Inrterstory Drifts* in the *2000 NEHRP Provisions*.

The new NEHRP seismic design provisions for glass and the new AAMA seismic test method for glass have been adopted (in a slightly modified format) in American Society of Civil Engineers (ASCE) 7-02 *Minimum Design Loads for Buildings and Other Structures*, which is referenced in the *International Building Code* and *NFPA 5000 Building Construction and Safety Code*.

Current research by the team includes a study of the use of rounded corners on glass panels to reduce damage. Tests have revealed significant gains in accommodation of drift. (Memari 1992)

Another investigation has resulted in the development of an "Earthquake-Isolated Curtain Wall System (EICWS)" that de-couples each story level of the system structurally from adjacent floor levels Figure (16) shows the response of the isolated curtain wall frames to a number of modes of vibration. Figure (17) shows how a seismic "decoupler" joint is able to accommodate relative inter story movements while still maintaining a building envelope weather seal. In-plane and out-of-plane movements are accompanied by horizontally continuous, flexible, elastomeric gasket loops that act as weather seals between stories.

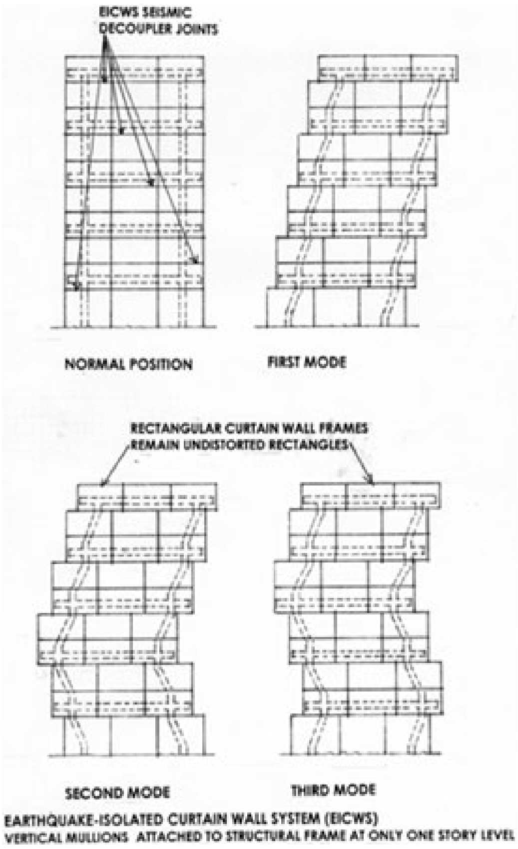


Figure 16: Earthquake-isolated curtain wall system schematic, showing diagrammatic response at first, second and third modes of vibration.

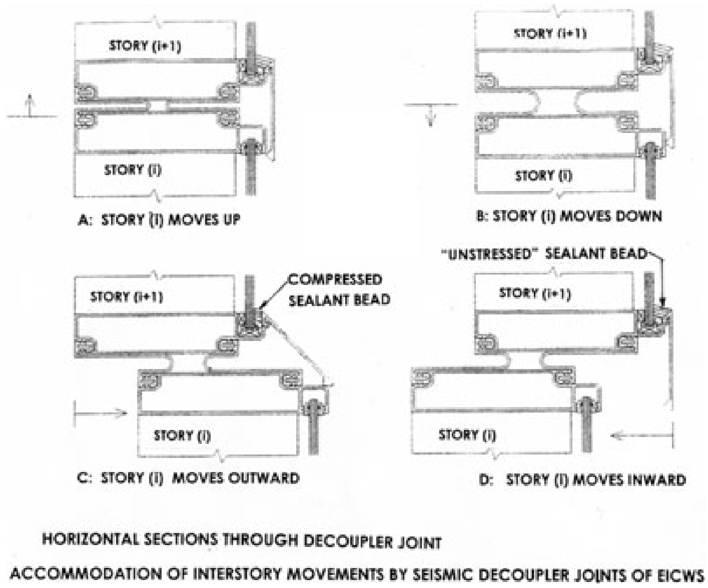


Figure 17: The Earthquake-Isolated Curtain Wall System (EICWS), detail of the decoupler joint. (Brueggman et al. 2000)

3. Seismic behavior of glass curtain walls

As it was mentioned earlier the basic cause of the damage to the nonstructural elements of the building are the deflections and displacements within the structural elements. In order to protect the nonstructural elements of buildings seismic design codes provide limitations on the story drift during the structural design phase (British Standards Institution. 1996, International Code Council 2000). The story drift is provided in two main directions of the story plane and in each direction is defined as the relative displacement between the top and bottom of the story divided by the story height. These limitations are mostly based on psychological comfort of the inhabitants during severe situations (avoiding large swinging in floors) and serviceability of typical construction technologies (avoiding failure in partition walls and mechanical appliances). These limitations are hardly enough for the glass façades and other considerations must be made.

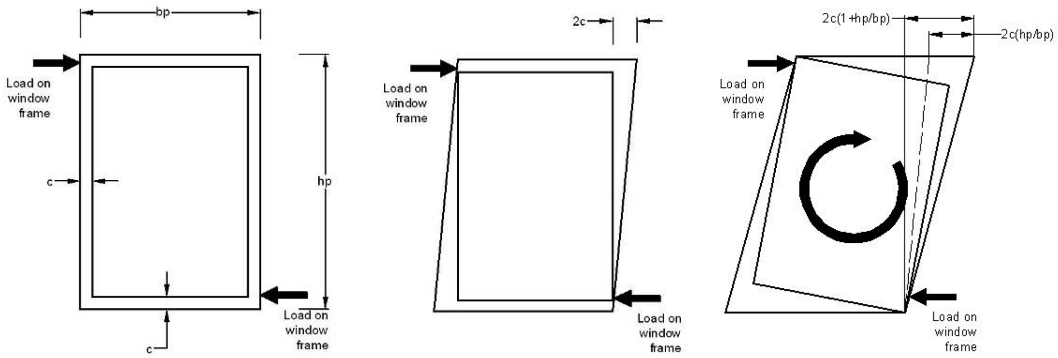
During a design process, in order to investigate the effect of the deflections within the structure of the building on the glass facades and envelopes of the building, it is advised that the structural engineer provide a list of these deformations and drifts for different load cases and combinations. However in case of absence of such data the maximum allowable drift ratios in the seismic code used for the structural design will be used as input data. In this research – in accordance with most seismic design codes – *0.02* is considered as maximum allowable drift ratio for the structural design, And a value of *0.01* is considered as a usually witnessed drift ratio in buildings in the effect of an earthquake.

3.1. Dry glazed systems with edge clearance

The behavior of glass panels remains in elastic range both during the in-plane and out-of-plane deformations. The main causes of damage to glass panels during earthquake are the in plane deformations which occur in the curtain wall system. This is due to the significant stiffness of the glass panels in that direction. Vallabhan (Sucuoğlu, Vallabhan 1997) argues that the in-plane deformation response of a glazed curtain wall having clearance provided between the glass panel edges and the frame can be described in two phases: First phase during which the glass panel undergoes a rigid body motion within its supporting frame, without having any force exerted on it. And the second phase in which the two opposite corners of the glass panel start to experience a diagonal pressure from the supporting frame.

3.1.1. Deformation due to rigid body motion

Figure (18-a) schematizes a glass panel and its supporting frame having c as clearance between the panel edge and the frame. It can be observed in Figures (18-b) and (18-c) that the panel can withstand a relative displacement of Dr according to the clearance length and the dimensions of the panel without having exerted any force on the glass panel; the displacements of the glass are only due to rigid body motion.



(a) (b) (c)

Figure 18: In-plane deformation within a curtain wall system due to rigid body motion

Equation (3.1.1-1) shows the relation between allowable panel deformation (D_r) with the panel size (h_p and b_p) and clearance(c).

$$D_r = 2c\left(1+\frac{h_p}{b_p}\right) \qquad (3.1.1\text{-}1)$$

The above concept is more generalized in ASCE 7-02 (American Society of Civil Engineers 2010) as it was previously described in section 5.4. It can be observed that within this approach the mechanical strength of the glass panels, to withstand load, is not brought into consideration and the design objective has been to avoid contact between the glass edges and the frames. The requirement described by ASCE 7-02 section 9.6.2.10 is either mentioned or referred to in numerous building design codes. Table (2) shows the in-plane displacements that can be endured within the window panel having edge clearances equal to *0.5* and *1* centimeters for different glass panel sizes it is assumed that both vertical and horizontal clearances have the same value. The drift values associated to every glass pane is derived by the endured displacements divided by the height of the glass pane. These values are not dependant on the thickness of the glass pane and only relate to its dimensions.

Table 2: Rigid motion allowable displacement for different glass panel sizes and clearances

Height	Width	thickness	c	Dr	Drifts
m	m	mm	cm	cm	
1	1		0.5	2	0.02
			1	4	0.04
1.3	.85		0.5	2.53	0.019
			1	5.06	0.039
1.5	1.2		0.5	2.25	0.015
			1	4.5	0.03
2	1		0.5	3	0.015
			1	6	0.03
2.6	1.7		0.5	2.53	0.01
			1	5.06	0.019

3.1.2. Deformation due to pressure on the glass

What was previously presented regards one part of the deformation capacity of a glazed system which is due to the rigid body motion of the glass inside the frame and provided by the clearance between the glass edges and window frame. There is also another deformation component, in addition to the rigid body component expressed previously, that contributes to the in plane deformation capacity of window glass.

When the glazed system reaches the ultimate deformation that can be achieved with rigid body motion (already expressed by D_{clear}), Figure (18-c) the two opposite corners of the glass panel coincide with the adjacent corners of the window frame and the glass plate will start to receive diagonal compression. At this stage, it tends to deflect in a diagonal buckling mode accompanied by a shortening in the loaded diagonal direction and further rotation together with the window frame as shown in Figure (19). A lateral deflection D_r results due to this shortening/rotation mechanism, which can be related to the diagonal shortening $\Delta d = d - d'$ through simple geometric relationships by assuming $Dr \ll d$

$$\Delta d = \frac{b}{d} D_r \tag{3.1.2-1}$$

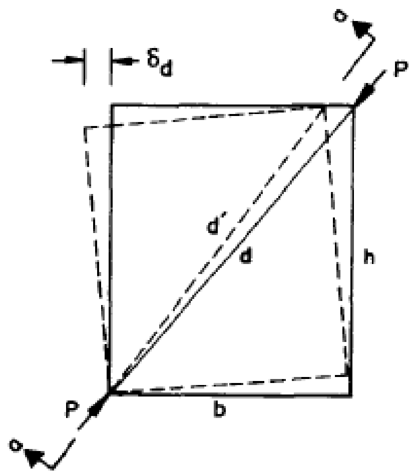


Figure 19: In-plane deformation within a curtain wall system due to glass panel deformation

The shortening in the glass panel is the result of an out of plane buckling deformation in diagonal direction which is expressed in Figure (20). If we consider that deformed shape function of the glass panel follows a sinusoid shape, as in the buckling deflection of long columns under longitudinal loads we have:

$$y = a \sin \frac{\pi x}{d} \tag{3.1.2-2}$$

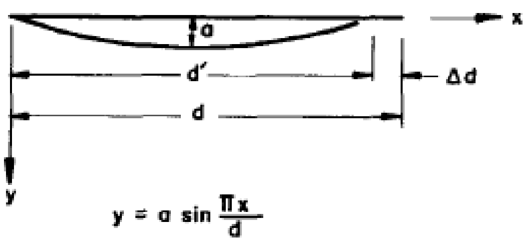


Figure 20: Glass panel deformation along the diagonal of the plate (section a-a in Figure 19)

Where a is the maximum amplitude at the centre, and Δd the diagonal shortening of the glass panel along the applied forces.

Sucuoglu and Vallabhan (Sucuoğlu, Vallabhan 1997) had defined the in-plane deformation in the curtain wall panel caused by diagonal shortening of the glass panel with the assumption that the glass will reach its maximum allowable stress in the middle of the panel by Equation (3.1.2-3)

$$D_r = \frac{1}{b}\left(\frac{\sigma_{all} d^2}{\pi E t}\right)^2 \qquad (3.1.2\text{-}3)$$

Table (3) shoes the values for D_r for different panel sizes and thicknesses based on Equation (3.1.2-3)

The elastic modulus (E) equal to 70 GPa and allowable stress (σ_{all}) equal to 50 GPa has been considered as the mechanical properties of glass panels.

Table 3: Allowable displacement by mechanical deflection for different glass panel sizes and thicknesses

Height	Width	thickness	Dr	Drifts
m	m	mm	cm	
1.00	1.00	6	0.574966	0.00575
1.00	1.00	10	0.206988	0.00207
1.30	0.85	6	0.984233	0.007571
1.30	0.85	10	0.354324	0.002726
1.50	1.20	6	1.630999	0.010873
1.50	1.20	8	0.917437	0.006116
1.50	1.20	12	0.40775	0.002718
2.00	1.00	8	2.021365	0.010107
2.00	1.00	16	0.505341	0.002527
2.60	1.70	8	4.429048	0.017035
2.60	1.70	10	2.834591	0.010902
2.60	1.70	16	1.107262	0.004259

3.2. Unitized and Panelized systems

Unitized and panelized systems are curtain wall systems that are mostly manufactured in factories and then assembled and adjusted onsite. Although due to more complex framing and added components these systems have higher costs, but the increase in the construction speed and also better sealing and enclosure performance of these systems, due to higher quality control over production achieved in factories, has made them a more often practiced solution for curtain walling compared with stick framing systems. Being produced in a better controlled environment – a factory – makes the adaptation of structural silicon as both the sealant and holding components of these systems trouble-free. Hence the structural siliconis commonly used in these systems to fasten the glass panes to the framing which is then attached to the main structure of the building.

Having the glass pane fully fastened to the framing system by the structural silicon makes it impossible to provide necessary clearance between the glass pane and the framing system. On the other hand having supported all the edges of the glass pane evenly, provides an even distribution of the loads applied on the glass and prevents the effect of localized stresses.

The seismic provisions considered for these types of curtain walls, which are not always put into practice, are basically based on the concept of separating them from the building structure I the horizontal in-plane direction. Non-dissipating connections (introduced in chapter 6) are usually used in these systems for protecting the curtain wall against lateral drifts. In the event of absence of isolating connectors or improper behavior of them these systems may experience three types of damage;

1-Frame distortions

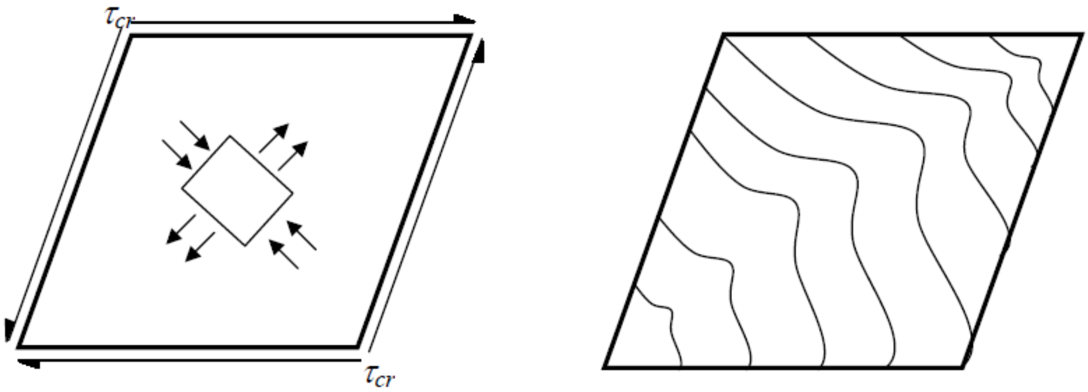
2- Structural silicon rapture

3- Glass pane failure

The first two types of damage are related to the serviceability of the curtain wall but the third type relates to safety measures unless safety glass (laminated or tempered) are used as composing glass panes. The effects of of the seismic forces over the structural silicon and the framing system can only be investigated by performing experimental tests on real-scale curtain wall mock-ups, and the results of such tests can highly differ according to the materials used and details of the composing elements. But for the glass panes subjected to shear forces around the edges it is possible to use the theory of buckling of plates to achieve the maximum load bearing capacity of the glass within the system.

Plate buckling for glass panes subjected to shear

In the structural silicon glazing, drift deformations in the curtain wall system do not cause compression forces on opposite corners of the glass panels as it is described by Vallabhan for dry glazed systems. Instead, due to the continuity of the connections in the glass panels these deformations act as shear forces on their edges. So in order to investigate the behavior of the glass panels in a structural silicon curtain wall the behavior of plates subjected to shear force along the sides needs to be studied.



(a) (b)

Figure 21: plate subjected to shear loading; a) principal stresses in the center of plate, b) deformation in the plate

Considering the large dimensions of glass panels compared with thickness, the most probable failure mode of a glass panel subjected to shear force along its edges is the shear buckling.

Having the governing differential equation of plates as below:

$$\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2 \partial y^2}+\frac{\partial^4 w}{\partial y^4}=\frac{1}{D}\left(q+N_x\frac{\partial^2 w}{\partial x^2}+2N_{xy}\frac{\partial^2 w}{\partial x \partial y}+N_y\frac{\partial^2 w}{\partial y^2}\right)$$
$$D=\frac{Et^3}{12\left(1-\nu^2\right)} \qquad (3.1.3\text{-}2)$$

Where: w is the out-of-plane deflection, q is the out-of-plane pressure acting on the plate , t is the thickness of the panel, E and v are respectively the modulus of elasticity and Poisson ratio.

The governing differential equation of plates is used as the basis for determining the critical shear force acting on the glass edges which causes the glass panel to buckle. The only difference is that in the governing differential equation (Equation (3.1.3-2)) we only consider the N_{xy} and the rest of the external forces to be zero. Hence the Equation (3.1.3-2) will be simplified to:

$$\frac{\partial^4 w}{\partial x^4}+2\frac{\partial^4 w}{\partial x^2 \partial y^2}+\frac{\partial^4 w}{\partial y^4}=\frac{2}{D}\left(N_{xy}\frac{\partial^2 w}{\partial x \partial y}\right) \tag{3.2-1}$$

We can assume w to be in the form of:

$$w=\sum_m \sum_n w_{mn}\sin\frac{m\pi x}{a}\sin\frac{n\pi y}{b} \tag{3.1.3-4}$$

This satisfies the boundary condition of simply supported plates. Now in order to find the critical values for N_{xy} we need to substitute w from Equation (3.1.3-4) to Equation (3.1.3-3) and solve the resulting differential equation for its obvious solution which gives us the unstable situation of the equation and in our case buckling. The following is the obvious solution to the resulting differential equation:

$$\left(N_{xy}\right)_{cr}=-\frac{abD}{32}\frac{\sum_m \sum_n a_{mn}{}^2\left(\frac{m^2\pi^2}{a_2}+\frac{n^2\pi^2}{b^2}\right)^2}{\sum_m \sum_n \sum_p \sum_q a_{mn}a_{pq}\frac{mnpq}{\left(m^2-p^2\right)\left(q^2-n^2\right)}} \tag{3.2-2}$$

It is necessary to select such a system of constants a_{mn} to make N_{xy} a minimum. This will be done by equating the derivatives of the Equation (3.2-2) with respect to a_{mn} using the proper notation we can have:

$$\left(N_{xy}\right)_{cr}=k\frac{\pi^2 D}{b^2}\ \rightarrow \tau_{cr}=k\frac{\pi^2 D}{b^2 t} \tag{3.2-3}$$

Where k is a constant depending on the ratio $a/b=\beta$ and has to be numerically calculated in order to minimize Equation (3.2-3). Limiting the number of considered mode shapes to 5 the values of k are given in table (4)

Table 4: Values of factor k in the Equation (4.2-3).

a/b	1.0	1.2	1.4	1.5	1.6	1.8	2	2.5	3	4
k	9.4	8.0	7.3	7.1	7.0	6.8	6.6	6.1	5.9	5.7

The value of k for the case of simply supported plates subjected to shear force, can also be estimated with accuracy of more than 90% with the below formulas;

$$\begin{aligned} k &= 5.34 + \frac{4}{(h/b)^2}; \qquad (h/b) \geq 1 \\ k &= 4 + \frac{5.34}{(h/b)^2}; \qquad (h/b) \leq 1 \end{aligned} \tag{3.2-3}$$

It is now possible to determine the shear capacity of the glass panels and the forces that can be applied on them through the connection devices. Table (5) shows the values of the shear capacity for different sizes and thicknesses of glass panels.

Table 5: Values of critical shear force for different window panel sizes and thicknesses

Height	Width	thickness	k	τ_{cr}	N_{xy}
m	m	cm		kg/cm2	Kg/cm
1.00	1.00	0.60	9.35	204.4063	122.6438
1.30	0.85	0.60	7.060059	91.32809	54.79686
1.50	1.20	0.60	7.91	76.85581	46.11349
1.50	1.20	0.80	7.91	136.6326	109.306
2.00	1.00	0.8	6.35	61.69841	49.35873
2.60	1.70	0.8	7.060059	40.59026	32.47221
2.60	1.70	1	7.060059	63.42229	63.42229

As it is obvious from the table above, the glass panes while supported with structural silicon and subjected to shear forces can demonstrate a considerable amount of strength against buckling and therefore failure. It is much more probable that the damage will be imposed on the silicon patches or the framing system of the unitized and panelized glazing systems before the glass panes start to experience damage. Although the effects of localized stresses and local buckling which also have contributions to the failure mode of the glass panel, especially the region near the connection devices, need to be further investigated.

3.3. structural Glazing systems

The idea behind structural glazing systems, either bolted assembly or suspended assembly, is a high aesthetical demand for an all glass envelope from the outside view and maximum transparency from inside. The fact that a continues and smooth glazed surface is a design objective and requirement for these curtain wall systems eliminates any possibility of providing isolations between the glass panes in horizontal and vertical directions. Also the fact that the glass panes, in these systems, is fully fastened to the supporting structure and will be directly subjected to the damaging loads in the appearance of lateral drifts in the main structure highlights the necessity to separately investigate the glass pane behavior with respect to applied forces and displacements.

Glass pane buckling for structural glazing

Again considering the small values of glass thickness compared to its height and width the theory of plates is used for analyzing the behavior of the glass panes in structural glazing, and as well as most structural plate members subjected to in-plane loads the case of buckling is the most probable form of failure, and we need to make sure that the imposed forces do not exceed the critical values for plate buckling.

For such an analysis first we consider that the glass pane is connected at its four corners to the substructure. It is possible to exert the effect of lateral movements and drifts with a pair of transversal loads at the top corners of the plate which results in the reaction forces at the bottom corners of the plate as it is shown in Figure (22)

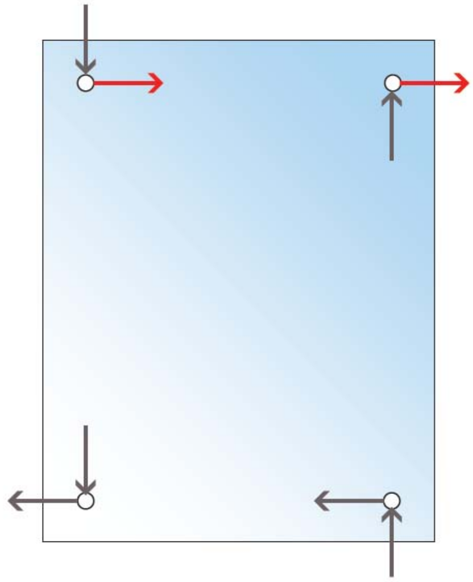


Figure 22: the exerting and reaction forces applied by fixings of the panel caused by lateral drifts

In this case the effect of the acting forces on the glass pane is analogues to the case of glass pane subjected to shear, but instead of having the shear equally distributed along the edges of the plate it is applied at two points at the two ends of plate edges.

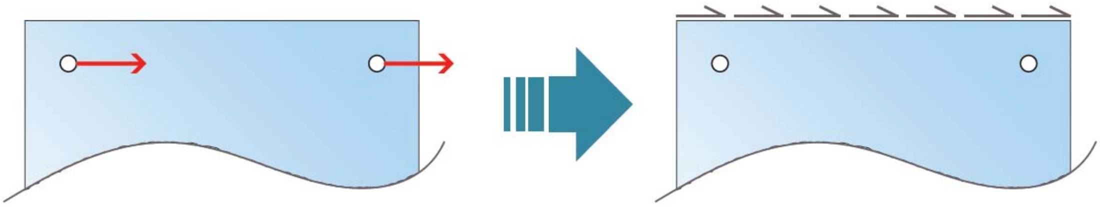


Figure 23: replacing the point loads on the fixing holes of glass panel with equivalent distributed shear on the edge

It should be noted that the total amount of the distributed shear force, which is the shear intensity multiplied by the length of the glass edge and its thickness, is equal to the sum of the point forces applied at the glass corners.

One major difference in this case with the case of plate subjected to distributed shear forces, earlier discussed in section (3-2), is that in this case the boundary conditions of the plate are free, contrary to the plate subjected to distributed shear forces where the boundary conditions where supposedly considered to be simply supported. This change in the boundary conditions will result in considerable differences in the value of *k* and it is not possible to adapt the values of plate buckling coefficient from table (4). in this case the value of *k* is considered to be equal to *0.43*, which is an all time minimum value for buckling coefficient of plates and is used when the boundary conditions of the plate do not play any role in resisting the buckling of the plate. This value will be used for all sizes and thicknesses of glass panes. The values of this analysis on the buckling of glass are presented in table (6)

Table 6: Values of critical point loads on the fixings of spider glazing glass panes with different sizes and thicknesse

Height	Width	thickness	k	τ_{cr}	F_h	F_v
m	m	cm		kg/cm2	Kg	Kg
1.00	1.00	0.60	0.43	9.400506	282.0152	282.0152
1.00	1.00	1.00	0.43	26.11252	1305.626	1305.626
1.30	0.85	0.60	0.43	5.562429	141.8419	216.9347
1.30	0.85	1.00	0.43	15.45119	656.6757	1004.328
1.50	1.20	0.60	0.43	4.178002	150.4081	188.0101
1.50	1.20	0.80	0.43	7.42756	356.5229	445.6536
1.50	1.20	1.2	0.43	16.71201	1203.265	1504.081
2.00	1.00	0.8	0.43	4.178002	167.1201	334.2402
2.00	1.00	1.6	0.43	16.71201	1336.961	2673.922
2.60	1.70	0.8	0.43	2.472191	168.109	257.1078
2.60	1.70	1	0.43	3.862798	328.3378	502.1638
2.60	1.70	1.6	0.43	9.888763	1344.872	2056.863

In the table above; *k* is the buckling coefficient, τ is the imaginary distributed shear, F_h and F_v are the horizontal and vertical components of the maximum allowable force on the glass pane corners and are based on below formulations:

$$
\tau_{cr} = k\frac{\pi^2 D}{b^2 t}
$$

$$
F_h = \frac{\tau_{cr} \times b \times t}{2} \qquad (3.3\text{-}1)
$$

$$
F_v = \frac{\tau_{cr} \times b \times t}{2} \times \frac{h}{b}
$$

Another approach for analytically demonstrating the effect of the applied forces over structural glazing glass panels is to investigate the case where a slightest rotation occurs in the panel due to lateral drifts in the structure, and in this case the glass pane encounters a diagonal pressure imposed over two of its opposing corners as in Figure (24).

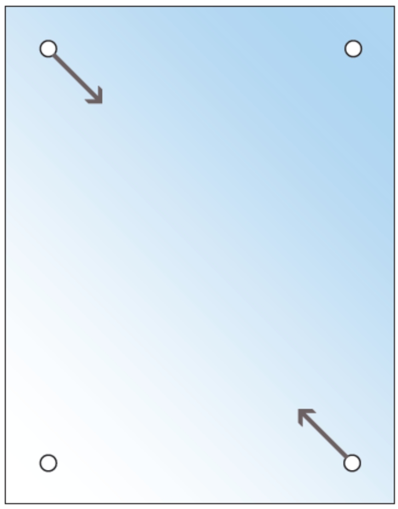


Figure 24: glass pane subjected to diagonal loading from oppsite corner fixings

Here it can be shown that the stress state resulting in a plate from such assumption is very much similar to the case of glass pane subjected to distributed shear but with free edge conditions. Figure (25)

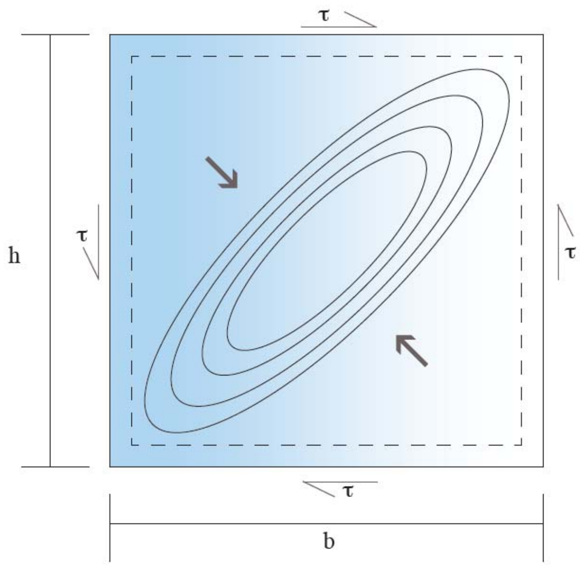


Figure 25: similarity between plates subjected to shear loading and diagonal pressure

Since there is no closed form solution for plates under diagonal pressure, it is assumed that the glass panel in question is part of an imaginary rectangular plate parallel to its diagonal, as is demonstrated in Figure (26). It is also assumed that the force *p* is constantly distributed along the edges perpendicular to the diagonal under compression with intensity N_p. having these assumptions it is now possible to calculate the maximum allowable force *P* causing the compression buckling in a glass panel.

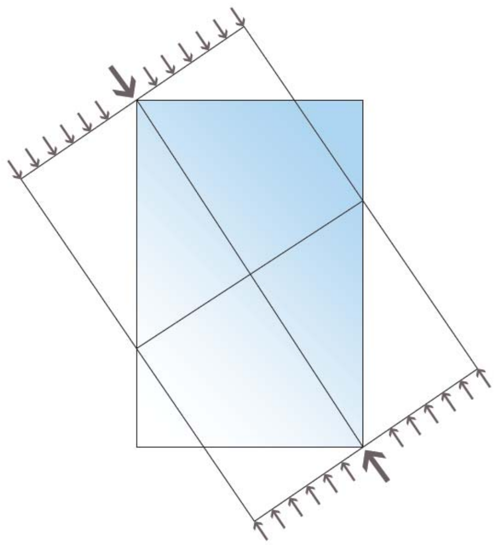


Figure 26: imaginary plate-beam subjected to unirofm pressure replacing the initial diagonaly loaded glass plate

Another approach is then presented here for calculating the critical buckling forces which is the buckling of column plates subjected to pressure. This is the case when a rectangular plate is free over two edges and simply supported on the other two where a uniform pressure is applied on the plate. Figure (27)

Figure 27: plate-column with uniform pressure loading and simply supported

According to the well-known Euler theory, the critical buckling load for any column becomes:

$$P_{cr} = \frac{\pi^2.EI}{h^2} \tag{4.2-3}$$

For adjusting this expression for relatively large width in relation to the buckling length of the strut we need to add a quotient $1/(1-\upsilon^2)$, which is due to the free strain deformation in the transverse direction in the center part, in relation to constraints at the loaded edges. And we get:

$$P_{cr} = \frac{\pi^2.EI}{h^2}.\frac{1}{\left(1-\upsilon^2\right)} \tag{3.3-2}$$

Now transforming the critical buckling load (P_{cr}) to an equivalent buckling stress (σ_{cr}) we get the formulas below:

$$\sigma_{cr} = \frac{P_{cr}}{b.t} \;\; \& \;\; I = \frac{b.t^3}{12} \quad \rightarrow \quad \sigma_{cr} = \frac{\pi^2.E}{12.\left(1-\upsilon^2\right).\left(\frac{h}{t}\right)^2} \tag{3.3-3}$$

Since our plate is diagonally loaded the imaginary plate is now assumed with a height equal to the diagonal of our plate and the width equal to the cross section of our plate prependecular to its diagonal. Based on the geometry of the initial glass panel the dimensions of the imaginary rectangle are defined below:

$$h' = \sqrt{b^2+h^2} \qquad , \qquad b' = \sqrt{b^2+h^2}.\frac{b}{h} \tag{3.3-4}$$

It should be noted that since the solution to the newly proposed problem is an upper bound to the initial problem the data calculated can be conservatively used in a design process, however the effect of local stresses and local buckling on the edges of the glass panel shall be separately considered in a numerical modeling.

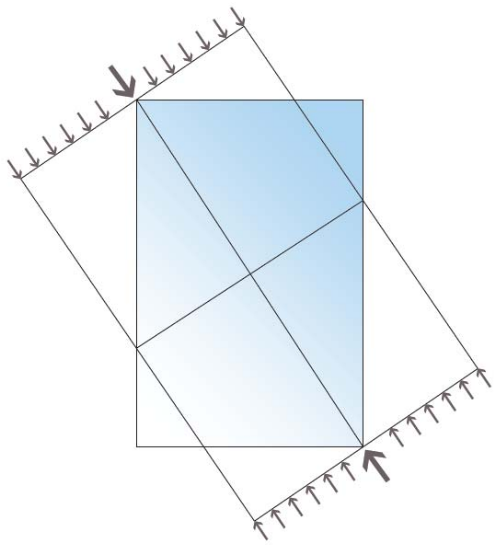


Figure 28: imaginary plate-beam subjected to unirofm pressure replacing the initial diagonaly loaded glass plate

The results of this analysis are presented in table (7) for different sizes and thicknesses of glass panes and then compared to the results of the first approach.

Table 7: Values of critical diagonal pressure on the oppsosing corners of spider glazing glass panes

Height	Width	thickness	h’	b’	P	F_h	F_v
m	m	cm		kg/cm2	Kg	Kg	Kg
1.00	1.00	0.60	1.41	1.41	927.51	655.85	655.85
1.00	1.00	1.00	1.41	1.41	4294.03	3036.34	3036.34
1.30	0.85	0.60	1.55	1.02	552.17	302.18	462.15
1.30	0.85	1.00	1.55	1.02	2556.36	1398.97	2139.60
1.50	1.20	0.60	1.92	1.54	546.27	341.25	426.57
1.50	1.20	0.80	1.92	1.54	1294.87	808.90	1011.13
1.50	1.20	1.2	1.92	1.54	4370.19	2730.04	3412.55
2.00	1.00	0.8	2.24	1.12	695.24	310.92	621.84
2.00	1.00	1.6	2.24	1.12	5561.93	2487.37	4974.74
2.60	1.70	0.8	3.11	2.03	654.43	358.14	547.74
2.60	1.70	1	3.11	2.03	1278.18	699.48	1069.80
2.60	1.70	1.6	3.11	2.03	5235.43	2865.08	4381.89

Knowing that the horizontal component of the force P is resulted from the drifts in the structure and is the origin of the damage to the glass, it can be seen that this amount shows good proximity to the sum of the horizontal components of the first approach, over the top and bottom edges of the glass. The same is also true for the vertical components of both approaches.

3.4. Laminated glass corrections

All the calculations above has been made under the assumption that glass shows homogeneous characteristics and also behaves with respect to the Bernoulli theory for bending, which indicates that during the course of bending the sections of the beam, column, plate and etc. remain flat plates. This is not entirely true for the case of laminated glass where we have an interlayer of PVB between two layers of glass Figure (29).

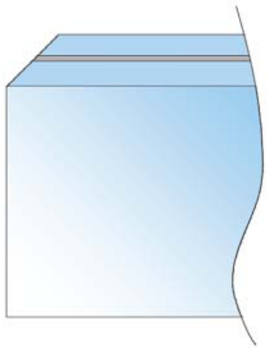


Figure 29: schematic figure of laminated glass section

In this case the ability of the middle section of the glass to completely transfer the shear stresses developed parallel to the interlayer will highly drop and results in a much less bending stiffness in the glass panel. One way to overcome this issue is by describing the load carrying behavior of the glass according to the sandwich theory (Dweib et al. 2004, Bedon, Amadio 2012). This way the critical buckling pressure for the case of plate-column glass with a two layer sandwich becomes:

$$
\begin{aligned}
N_{cr} &= \frac{\pi^2\left(1+\alpha+\pi^2\alpha\beta\right)}{1+\pi^2\beta} \cdot \frac{EI_s}{h^2} \\
\alpha &= \frac{I_1+I_2}{I_s} \\
\beta &= \frac{t_{PVB}}{G_{PVB}.b\left(z_1+z_2\right)^2} \cdot \frac{EI_s}{h^2} \\
I_i &= \frac{bt_i^{\,3}}{12} \\
I_s &= b\left(t_1 z_1^{\,2} + t_2 z_2^{\,2}\right)
\end{aligned}
\tag{3.4-1}
$$

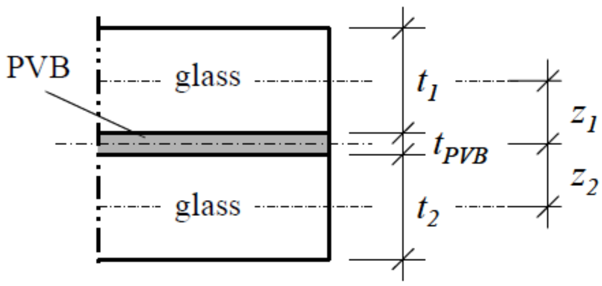


Figure 30: laminated glass section and proportions

Although being analytically very accurate the problem with the above approach is that it is limited to the problem of plates with uniaxial loading having two free and two simply supported edges. Another way to include the effect of layered glasses in the buckling analysis previously made, is to come up with an equivalent thickness for the laminated glass panels.

Assuming that the PVB interlayer does not provide any shear resistance in between the glass layers of the same thickness, we observe that the overall boundary stiffness of the glass will drop by 75% comparing to the case of having a single layer glass with same thickness.

$$
I_1 = 2\frac{b.t_1^{\,3}}{12}\,,\; I_2 = \frac{b.t_2^{\,3}}{12} = 8\frac{b.t_1^{\,3}}{12} \;\;\rightarrow\;\; I_1 = 0.25 I_2 \tag{3.4-2}
$$

Now if we assume a single layered glass with the same bending stiffness of the double layered glass the thickness of that single layered glass will be the equivalent thickness of our glass pane:

$$I_1 = \frac{b.t_{eq}^{\ 3}}{12} = 2\frac{b.t_1^{\ 3}}{12}$$
$$\rightarrow t_{eq} = \sqrt[3]{2}t_1 = 1.26t_1\,, t_{eq} = \sqrt[3]{0.25}\,t_2 = 0.63t_2 \tag{3.4-3}$$

In cases where shear stress is developed in laminated glass parallel with the interlayer, the interlayer can be considered as having some shear resistance. This can be taken into account in evaluating resistance to bending of the laminated glass using a suitable engineering formula in combination with the shear resistance of the interlayer.

The following approach, using the concept of 'effective thickness' can be used.

The effective thickness for calculating bending deflection is:

$$h_{ef;w} = \sqrt[3]{(1-\varpi)\sum_i h_i^3 + \varpi\left(\sum_i h_i\right)^3} \tag{3.4-4}$$

And the effective thickness for calculating the stress of glass ply number j is:

$$h_{ef;\sigma;j} = \sqrt{\frac{\left(h_{ef;w}\right)^3}{\left(h_j + 2\varpi h_{m;j}\right)}} \tag{3.4-5}$$

Where ϖ is a coefficient between 0 and 1 representing no shear transfer (0) and full shear transfer (1),

h_i, h_j are the thicknesses of the glass plies, Figure (31), and

$h_{m;j}$ is the distance of the mid-plane of the glass ply j from the mid-plane of the laminated glass, ignoring the thickness of the interlayers, Figure (31)

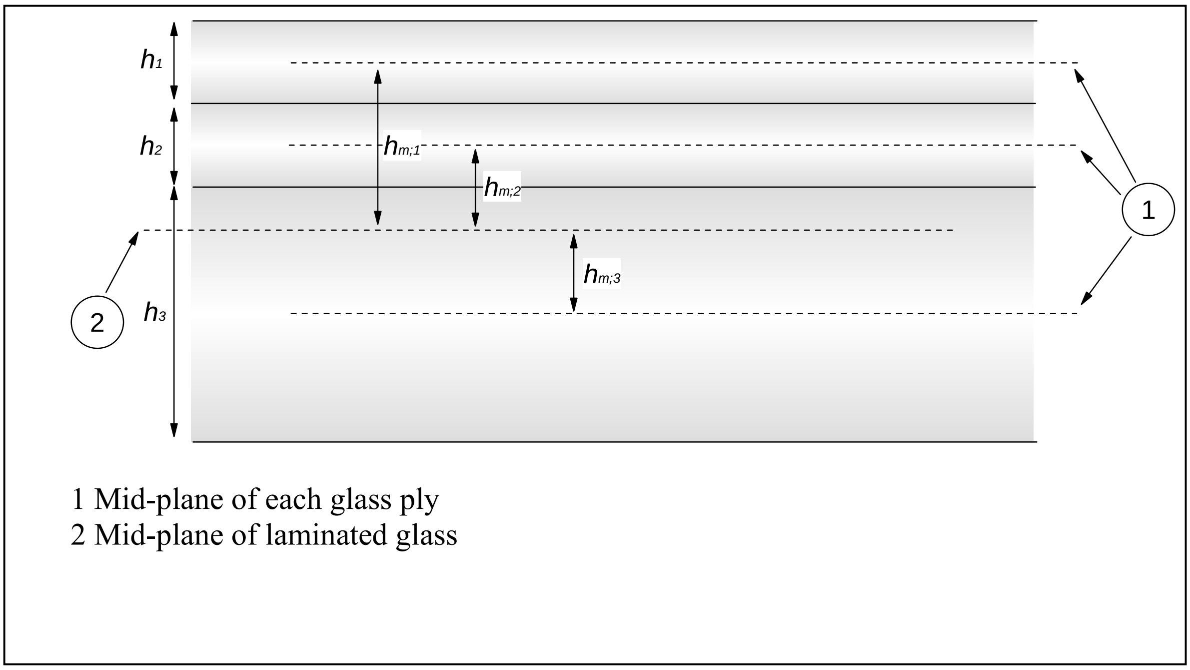


Figure 31: Example of 3 layers laminated glass thickness dimensions

The effective thicknesses for calculating stresses and deflection in laminated glass comprising two plies of the same thickness using a value of $\varpi = 0.25$ are given in table (8).

Table 8: Effective thicknesses of laminated glass with two plies of the same thickness and $\varpi = 0.25$

Glass thickness	Short duration loads ($\varpi = 0.25$)		Long duration loads ($\varpi = 0.05$)	
	$h_{ef;w}$	$h_{ef;\sigma;j}$	$h_{ef;w}$	$h_{ef;\sigma;j}$
3 + 3	4.55	5.02	3.96	4.44
4 + 4	6.07	6.69	5.28	5.92
5 + 5	7.59	8.37	6.60	7.40
6 + 6	9.11	10.04	7.92	8.88
8 + 8	12.15	13.39	10.56	11.84
10 + 10	15.18	16.73	13.20	14.80

Determination of ϖ

The value of ϖ to be used for a specific interlayer and a particular load case depends on the interlayer stiffness family to which the interlayer belongs for that particular load case. The interlayer stiffness families and the equivalent values of ϖ are given in table (9).

Table 9: Value of ϖ associated with interlayer stiffness family

Interlayer stiffness family	Value of ϖ
4	0.7
3	0.5
2	0.25
1	0.1
0	0

Based on the corrections on the laminated glass thickness the values of the tables presented in sections above for critical buckling forces of glass panels are presented in tables below:

Table 10: Values of critical point loads on the fixings of spider glazing glass panes with different sizes and thicknesse (laminated glass)

Height	Width	thickness	τ_{cr}	F_h	F_v
m	m	cm	kg/cm2	Kg	Kg
1.00	1.00	0.60	5.57356	128.7492	128.7492
1.00	1.00	1.00	15.48211	596.0612	596.0612
1.30	0.85	0.60	3.297964	64.75553	99.03787
1.30	0.85	1.00	9.161012	299.7941	458.5087
1.50	1.20	0.60	2.477138	68.66626	85.83282
1.50	1.20	0.80	4.4038	162.7645	203.4556
1.50	1.20	1.2	9.908551	549.33	686.6626
2.00	1.00	0.8	2.477138	76.29584	152.5917
2.00	1.00	1.6	9.908551	610.3667	1220.733
2.60	1.70	0.8	1.465762	76.74729	117.3782
2.60	1.70	1	2.290253	149.8971	229.2543
2.60	1.70	1.6	5.863048	613.9784	939.0257

Table 11: Values of critical diagonal pressure on the oppsosing corners of spider glazing glass panes (laminated glass)

Height	Width	thickness	P	Fh	F_v
m	m	cm	Kg	Kg	Kg
1.00	1.00	0.60	423.44	299.42	299.42
1.00	1.00	1.00	1960.37	1386.19	1386.19
1.30	0.85	0.60	252.09	137.95	210.99
1.30	0.85	1.00	1167.06	638.67	976.80
1.50	1.20	0.60	249.39	155.79	194.74
1.50	1.20	0.80	591.15	369.29	461.61
1.50	1.20	1.2	1995.14	1246.35	1557.94
2.00	1.00	0.8	317.40	141.95	283.89
2.00	1.00	1.6	2539.20	1135.57	2271.13
2.60	1.70	0.8	298.77	163.50	250.06
2.60	1.70	1	583.53	319.34	488.40
2.60	1.70	1.6	2390.15	1308.01	2000.48

4. Advanced connectors

The use of advanced connectors in cladding systems was proposed by many scholars and designers after post-earthquake surveys and laboratory tests had shown that fixed elements of a cladding system are vulnerable to damage during an earthquake due to deformation accruing in the structure of buildings. The idea of using advanced connectors was to provide isolation between the envelope system and the structure and to dissipate seismic energy. Since light weight cladding systems do not affect the dynamic behavior of the building giving very little contribution to it, it is obvious that the energy dissipating approach on a building scale can only be carried out in heavy cladding systems. Barry J. Gondo and James I. craig (Goodno et al. 1998) provide a detailed study of different dissipating connection systems. But since energy dissipating mechanisms can also be used as a means of controlling the forces regarding displacements they still have the potential for being used in the light cladding systems in order to provide a desirable level of isolation. Due to their simplicity, both in terms of analytical study and practical use, friction damping connectors are proposed in this research as connecting devices between the glazed envelope and the structure of the building.

Although advanced connectors that are used in cladding systems have considerable differences based on the systems they are used in, and the functions which are expected of them, there are some primary applied conditions that they all face and some general characteristics in which they all need to have. Having the focus on the mechanical aspects of these connectors, the main applied condition for all different types of cladding connectors are the applied loads. Cladding connectors must be able to transmit the following sets of applied loads:

- Vertical loads
- Normal loads
- Transversal loads

The vertical loads, quite obviously, are the ones which act in the vertical direction over the connections and they are usually caused by the weight of the cladding elements, but they may also be the result of thermal expansions within the cladding panels or other occurrences in the cladding system. Transmitting the vertical load is the main function of a connection in cladding systems and therefore it is the most important force applied on it. The normal loads are the ones that are applied in the direction normal to the surface of the envelope and they are mostly caused by wind and sometimes by the acceleration effect of the movement of the cladding elements during earthquakes. Transmitting the normal forces will keep the cladding systems in place. The transversal loads are the ones that are acting in the plane of the cladding system in the horizontal direction and they are mainly caused by the displacements of inter-story drifts during earthquakes, although they can also be caused by thermal expansion in the envelope surface, or in case of heavy weight cladding by acceleration nature of the seismic forces. The main function of advanced connectors for cladding systems is to isolate the panels from transversal loads by insuring that all the connectors are flexible or energy dissipating in the transversal direction. This will guarantee that although the cladding system is being intact with the structure in normal conditions, isolation can be provided in transversal direction between the two systems to protect the envelope against inter-story drift and if possible dissipate seismic energy in the building during an earthquake.

Applied loads over connection systems is not necessarily confined to the forces acting on the connection systems and in the case of presence of significant rotational constraints, these forces will result in moments involved in the applied loads over connection elements.

Advanced connectors are assumed to function in every aspect the same as conventional cladding connectors, as they will also be able to transmit transversal loads between the two systems as much as the panels can support, however with the proper design of the connectors it should be possible to limit the maximum applied in-plane force that is introduced to the panel in order to protect the panel and its attachments. However different the connectors may be, they are generally composed of three main components:

- Anchor to the envelope
- Connection body
- Anchor to the structure

Based on the type of the cladding used as the building envelope, the structural system of the building (and in some cases the secondary structure of the envelope system), the architectural requirements and the function of the connection the components of the connection device have considerable design variations. Usually the energy dissipating mechanism within a connection device happens in the connection body.

4.1. Different mechanisms of advanced connectors

Providing the desired properties of energy dissipation and transversal isolation, that are the main purposes of advanced connectors, can be achieved by adapting different mechanical approaches. Maintaining the structural integrity of the connection device during repeated cycles of transverse deflections, having a well defined force transmitting value and a bulky while stable hysteresis loop are some major characteristics that should be taken into account for every approach taken and materials selected. It is only by satisfying the conditions above that an advanced connector can perform in a predictable behavior which guarantees the protection of the envelope system.

In general there are four types of advanced mechanical approaches used in advanced connectors to satisfy the conditions introduced above:

Yield damping connectors

The inelastic deformation of the metallic substances is one of the most effective mechanisms available for seismic energy dissipation in structures. In these connection devices the post yield ductility of the metallic members provides the required energy dissipation in one of the three mechanisms of torsional bending, flexural beam bending and U-strip flexural bending. Figure (32) demonstrates the concept of the three introduced mechanisms.

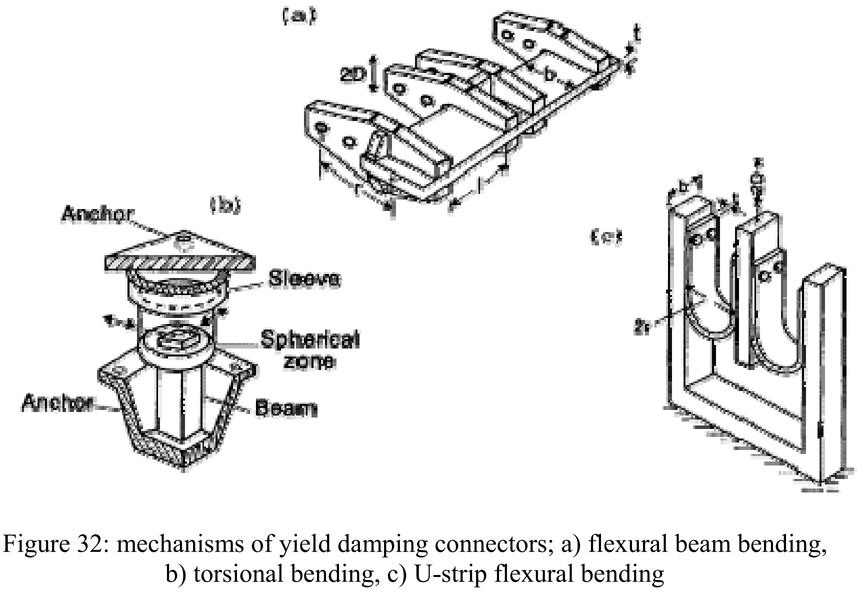


Figure 32: mechanisms of yield damping connectors; a) flexural beam bending, b) torsional bending, c) U-strip flexural bending

These systems (specially the flexural bending mechanism) have already been as seismic energy damping mechanisms in the bracings of steel structure. During the ensuing years, considerable progress has been made in the development of metallic dampers. For example, many new designs have been proposed, including the X-shaped and triangular plate dampers displayed in Figure (33). Alternative materials, such as lead and shape-memory alloys, have been evaluated. Numerous experimental investigations have been conducted to determine performance characteristics of individual devices and laboratory test structures. As a result of this ongoing research program, several commercial products have been developed and implemented in both new and retrofit construction projects. In particular, a number of existing structures in New Zealand, Mexico, Japan, Italy, and the United States now include metallic dampers as a means for obtaining improved seismic resistance.

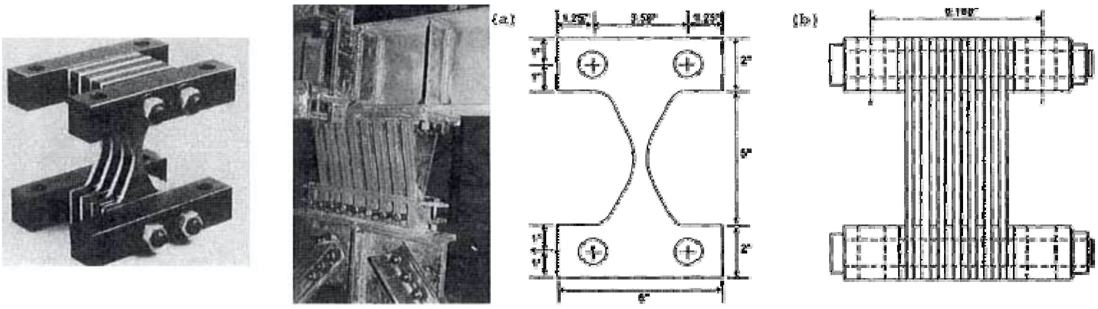


Figure 33: Metallic Dampers; X-shaped Plate and Triangular Plate Damper

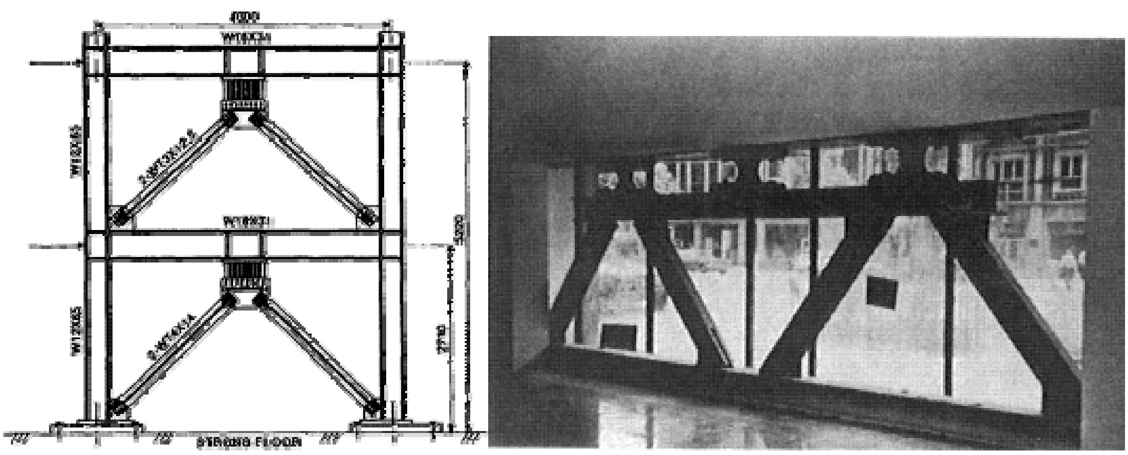


Figure 34: Triangular Plate Damper Within Structural Frame, Brace-damper Assembly

With some adjustments in geometry, shape and especially size, the connections with yield-damping concept that are utilized in the passive dynamic control of the structures can be modified to be used as advanced connection devices for claddings. Figure (35) shows the modified yield-damping connections to be used as advanced connection devices in building cladding.

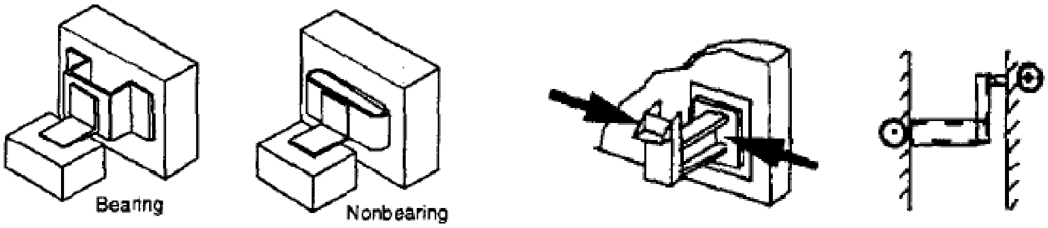


Figure 35: yield damping connectors for cladding systems

The adaptation of yield-damping connections in the building cladding is limited to heavy cladding systems like concrete claddings or terracotta cladding panes, where the yielding of the metallic plates can be easily achieved. In light cladding systems in order to adapt the yield-damping connectors the thickness and the dimensions of the metallic components must become so small that will endanger the structural integrity of the connection device.

Visco-elastic Shear based connectors

The visco-elastic dampers were first utilized in vibration control of aircrafts back in the 1950s in order to reduce the members fatigue caused by the vibrations in the system. Their first applications in the building sector dates back to 1969 when they 10’000 visco-elastic dampers where used in late twin towers of the world trade center to help resisting the wind loads followed by a number of similar adaptations to tall buildings against wind loads. Adaptation of these systems for seismic purposes has a more recent origin since due to their frequency dependent behavior it requires a much more effective use of these mechanisms.

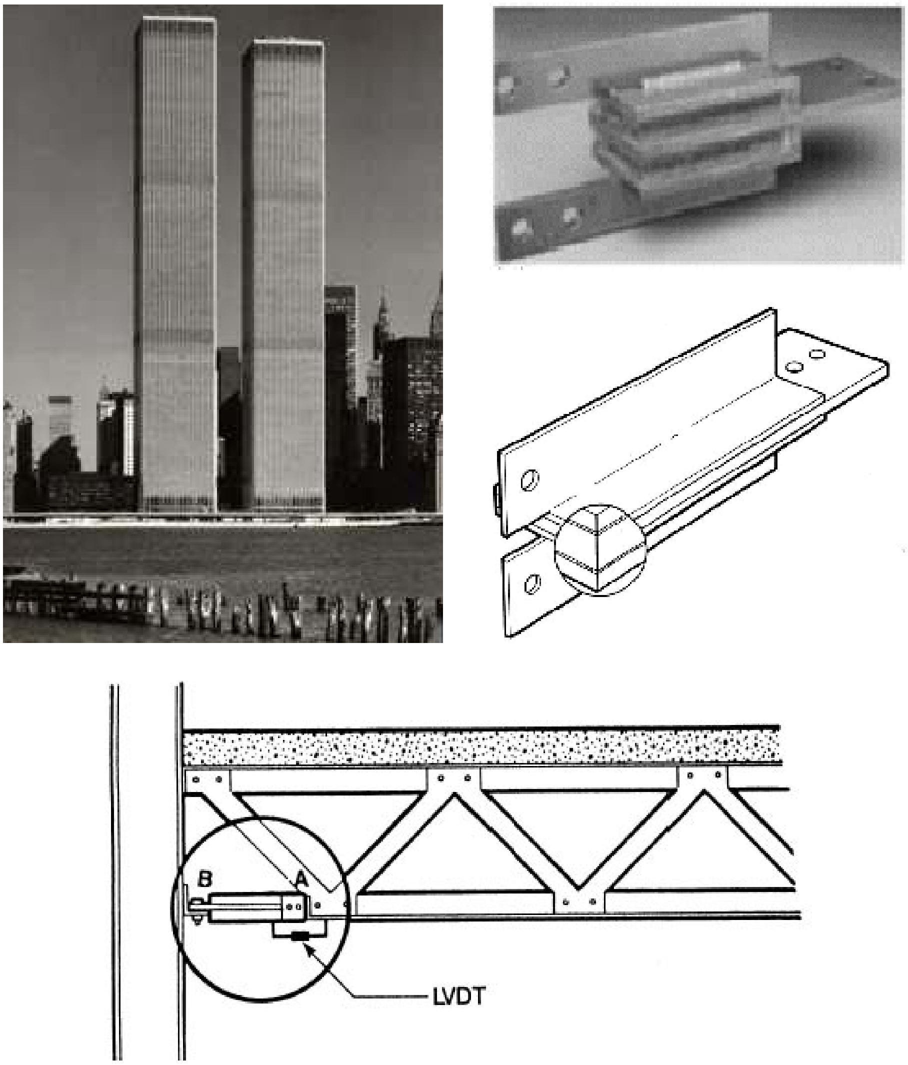


Figure 36: visco-elastic shear connectors in WTC twin towers

Figure (37) shows a typical version of VE connections in which a viscoelastic layer in bounded between steel plates, shear deformation accompanied by energy dissipation occurs when structural drifts induce a relative deformation between the adjacent steel plates. Copolymers or glassy substances that dissipate energy when subjected to shear deformations are usually materials used as viscoelastic layers.

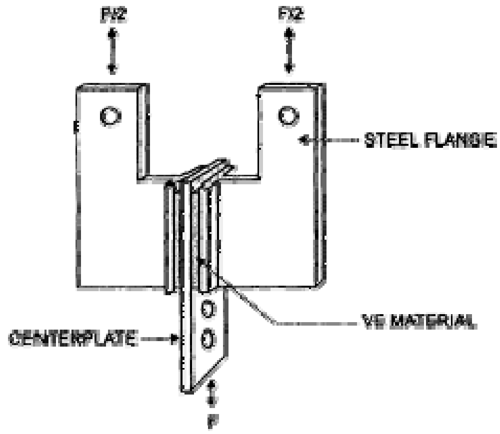


Figure 37: schematic demonstration of a typical VE damping connector

Studying the basic behavior and the mechanical governing equations for these systems shows that the shear storage modulus and the loss modulus of viscoelastic materials are generally dependent on excitation frequency, ambient temperature, shear strain and the material temperature. Having that said it can be concluded that adaptation of these connection devices in building envelopes for seismic considerations will result in loss of adequate predictability due to the varying ambient temperature – which is in direct contact with the building envelope – and the uncertain behavior during an earthquake. However based on the visco-elastic shear connections that are typically utilized for base isolation in bridge supports, some connection devices have also been introduced for the cladding systems.

The design objective of these connections has been to develop a load bearing connector that provides panel alignment and maintain high levels of ductility in the lateral direction while supporting the panel weight. For this, it is required to have high levels of stiffness in two directions and a lower level with hysteresis behavior in the transverse direction. To that end elastonomer neoprene bearing pads laminated with steel plates are used in these connections. They are very rigid in the direction of bearing but capable of sustaining large displacements perpendicular to it. The steel laminates prevent the elastomeric material from bulging under large compressive forces, but play no role in the shear stiffness of the connection which depends only on the elastomeric material and pad dimensions. Figure (38) demonstrates a typical scheme of these connection devices.

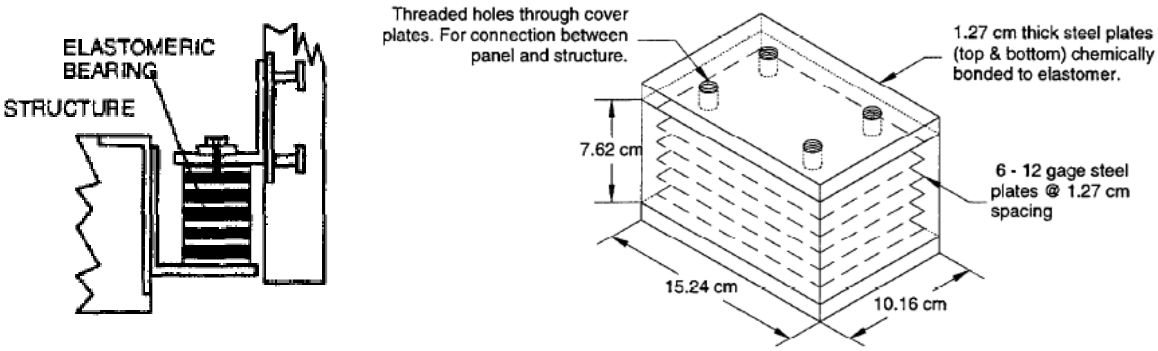


Figure 38: visco elastic shear damping connectors for cladding systems

Friction damping connectors

In the previous connection mechanisms the energy dissipating behavior which that had been considered as the basis for the design of the connections occurred in the bulk of material of the composing elements. In the friction connection mechanism the energy dissipating action which occurs between the surfaces of two contacting surfaces is used as the concept for designing the connection device.

Friction is one of the most influencing physical properties of materials and has been employed in many natural and also engineering processes. So it is of no surprise, adapting this phenomenon for solving a variety of engineering problems. Many studies have been performed on investigating the friction between two contacting surfaces, both on microscopic and macroscopic levels, but the coulomb model for friction has shown to be an adequate explanation of the trend in most of the engineering problems at hand.

The friction mechanism is the basis of a great number of energy dissipating devices in structural engineering. A well known example of these devices is the slotted bolted connection introduced by FitzGerald at. al. shown in Figure (39)

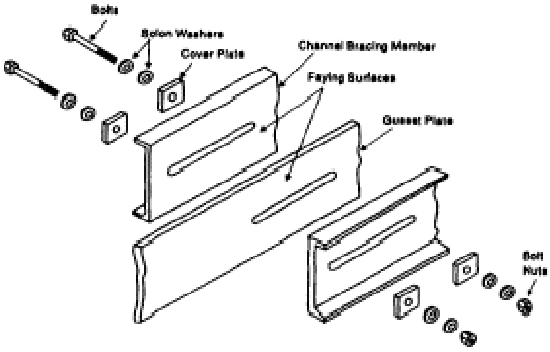


Figure 39: Slotted Bolted Connection (FitzGerald et al., 1989)

Another major difference between the friction connections and advanced connectors previously discussed is that unlike the yielding and visco-elastic connections, the friction connector's force displacement diagram does not follow a smooth curve but changes the behavior quite rapidly. Unless the applied forces on the connection reach a maximum value force the connection device will remain as a rigid connection point, but when the forces reach the maximum value the forces transferred through the connection will remain constant and equal to the maximum value force and the connection will begin to experience relative movements between its two ends. Figure (40) demonstrates a typical scheme of the friction connection device used in cladding systems. More detailed information on friction connection devices is presented in the next section of this chapter.

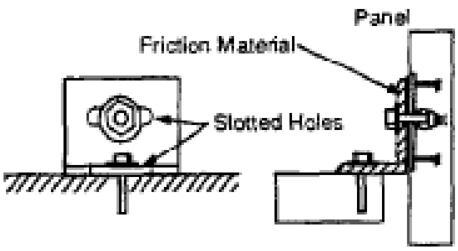


Figure 40: friction damping connector for cladding systems

Non-dissipating isolators

One of the highly practices connection mechanisms to protect the building envelope during seismic actions are the ones that do not rely on the energy dissipating behavior of the connections and focus the attention on the isolating feature of the connection. Since, as it was already mentioned earlier, light weight cladding systems do not play a significant role in the overall dynamic seismic response of the building in case of an earthquake, the adaptation of non-dissipating isolators has been more limited to these envelope systems. But in some heavy cladding systems considerations are made in order to separate the cladding from the building structure¸ both for protecting the cladding system during an earthquake and for avoiding unwanted and uncalculated effects of the cladding system over the building structure.

There are two main categories of non-dissipating isolating connection devices in building cladding systems; rocking connections and swaying connections. In the rocking connection systems the connection devices are managed in a way that the cladding panels can endorse a rotating movement in case lateral drift occurs in main structure, Figure(41-a). but in the swaying connection systems the cladding connections are managed so that the panel can freely experience translation in the horizontal direction, Figure (41-b). in both cases usually the lower connection provide resting points for the panel while the upper connections provide resistance only against out of plane movements, but this not necessarily always this way and sometimes the situation in vice versa, and the panel are hanging from their top connections.

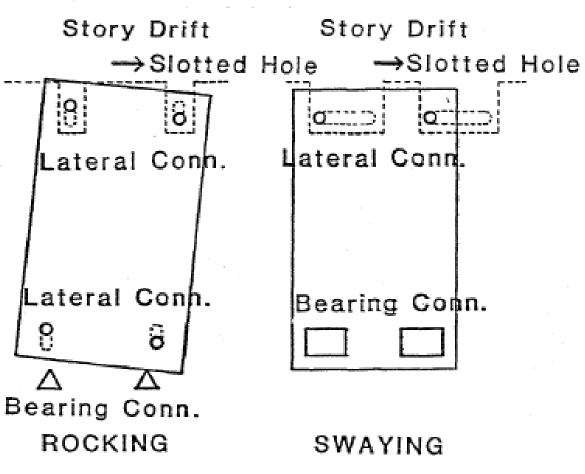


Figure 41: mechanisms of drift accommodation in building cladding systems

In rocking connection systems tie back connections are typically used for minimum stiffness against lateral movement and high resistance against out of plane movements while the panel is resting on or hanged from load bearing supports. In the sway connection systems slotted hole connection systems replace the tie back connections in order to restrain the movements only to transversal directions.

Sway connection systems are highly preferred in glass curtain wall, since rotation in the glass panels can result in distortions in the curtain wall system and an equal horizontal displacements at story level is much easier to mechanically deal with in such systems. Figures (42) and (43) demonstrates an advanced sway connection system for curtain walls (Brueggeman et al. 2000), introduced by Wulfert et al. which the panels are only connected to one floor level at every story and decoupling joints are used to connect panels at different story levels to maintain the air-tightness and water-tightness of the system.

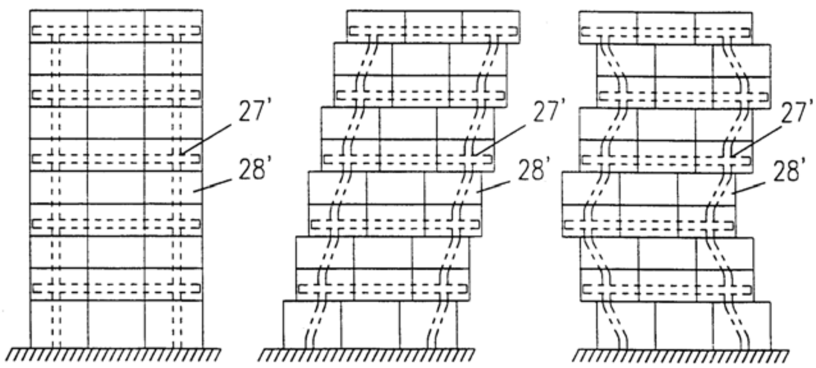


Figure 42: dipiction of earthquake isolated curtain wall system; vertical mullions are attached to building frame at only one story level

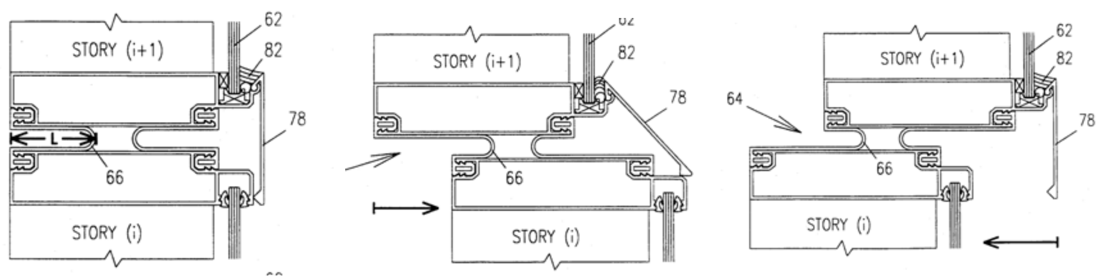


Figure 43: decoupler joints accommodating building frame inter-story movements (Wulfert et al. 2003)

Sliding connections are another type of sway connection usually used in unitized and panelized curtain wall systems. In sliding connection both the load bearing and horizontal isolating behavior are combined, and they are used both at the bottom and top of curtain wall panels, Figures (44) and (45) show the details of two sliding connections.

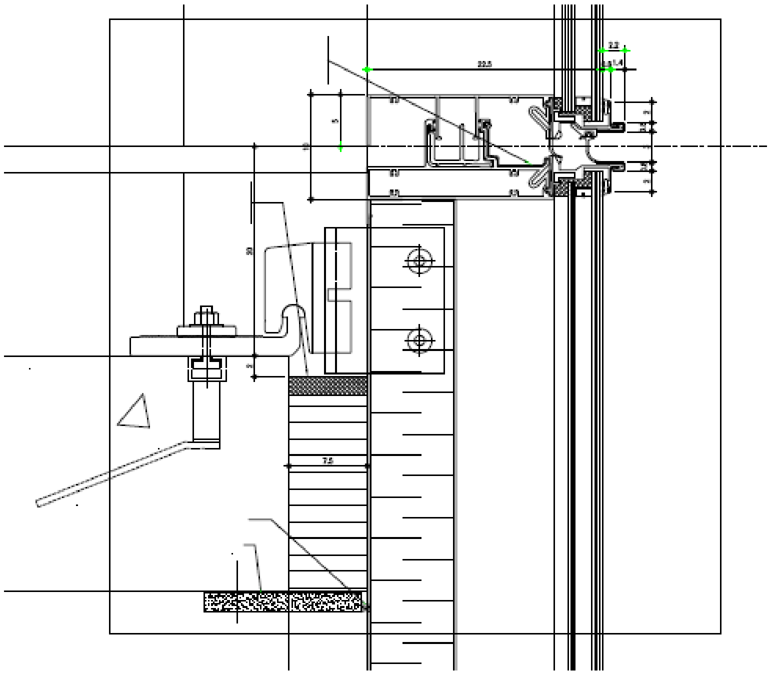


Figure 44: sliding connection details for unitized curtain wall systems

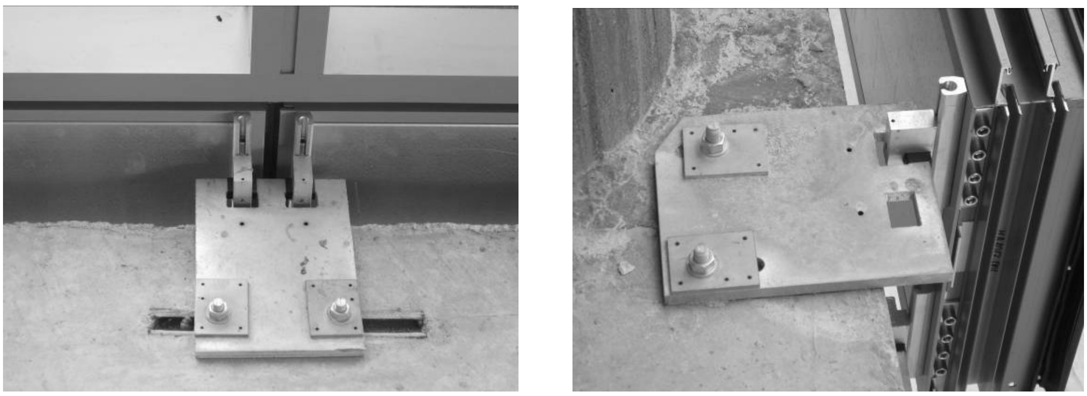


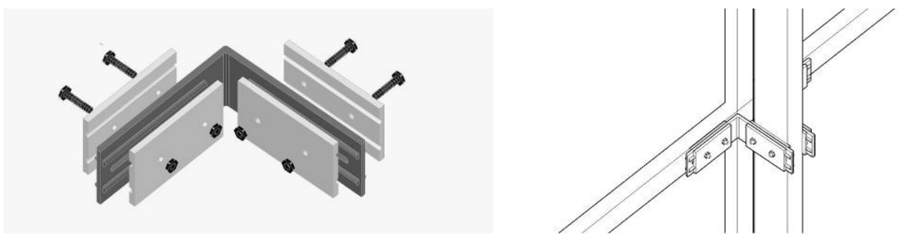
Figure 45: Equivalent panel under uniform pressure

4.2. Friction damping connectors

Friction damping connectors are mechanisms that use friction – usually between two sliding surfaces – as the basis of energy dissipation (Soong, Dargush 1997). The load bearing capacity (friction behavior) of the device will be controlled by the friction coefficient between the two sliding surfaces and the force perpendicular to the surfaces attaching them together. They were first introduced to the building sector based primarily upon an analogy to the dissipation of kinetic energy in automotive brakes with the objective of reducing the seismic motion of the building by "braking" rather than "breaking".

One advantage of this system is that it can be tuned in order to transmit a limited amount of force between the two connected systems and if the force exceeds a certain limit the device will no longer transmit the exceeding force and will only experience displacement in the direction of the exerted force until it reaches its maximum displacement capacity. How to find the limit force for a certain glass panel is described later in the chapter devoted to tuning the connection.

The other advantage of the friction damping devices is that they are rather simple devices, easy to manufacture and very easy to use in the construction of curtain walls. They can be applied in place of most of the existing brackets with very little or no modification. Figures (46-a) and (46-b) schematically show the friction connection bracket which is the simplest form of a friction connector. More intricate friction connectors can be used for different and more complex details of the curtain wall and the supporting structure, like the friction rod discussed later in this research, but they all follow the same mechanical behavior, that is, to let go when the force exceeds a predefined limit and keep the pressure on the curtain wall at a constant rate.



(a) (b)

Figure 46: Friction connector bracket

As shown in the Figures, the connection device is almost the same as a connection bracket with the only difference that the bracket in use for a friction damping connector is provided with a long slotted hole in the middle and accompanied by two sliding plates on each wing of the bracket. These allow the bracket to slide in two directions between the enclosing plates. The sliding of the bracket between the enclosing plates will be controlled by the pressure of the two plates on the bracket wings. In unitized systems it is considered that the glass panes have a framing attached to them using structural silicon bonding (primary framing) which is again connected to a set of vertical mullions (secondary framing). The vertical mullions are fully fastened to the structure of the building. The placing of the connection is between the primary and the secondary framing systems. Aside from the friction bracket introduced above different geometries like U, Z and other shape brackets can be used as friction connecting devices to connect different framing systems and accommodating isolation in desired directions.

It is essential for the materials used in friction connectors to present the two characteristics below:

- To have similar friction coefficients in both cases of static and dynamic frictions between the sliding surfaces.
- To have considerable resistance against environmental attacks, since corrosion and other changes which may happen to the surface of the plates can dramatically change the behavior of the connection device.

Incorporating brake lining pads that are used in automotive brakes as the enclosing brakes over sand blasted steel plates will result in a consistent force-displacement response and an almost perfect square hysteresis loop while satisfying the conditions above for friction connectors.

Figure 47: Brake lining pads

The friction coefficient between the brake lining pads and blasted steel disks used in automobile brake systems in most cases is somewhere in-between *0.3* and *0.4*.

4.3. Basis and behavior

The scientific study of solid friction has a long history dating back to da Vinci, Amontons and Coulomb. The basic theory is funded upon the following hypotheses, which were initially inferred from physical experiments with planar sliding of rectangular blocks:

1-The total friction force that can be developed is independent of the apparent surface area of contact.

2-The total frictional force that can be developed is proportional to the total normal force acting across the interface.

3-For the case of sliding with low relative velocities, the total frictional force is independent of that velocity.

As a result of these assumptions we have:

$$F = \mu N \qquad (4.3\text{-}1)$$

Where F and N represent the frictional and normal forces and, respectively, and μ is the coefficient of friction. Since it is frequently observed that the coefficient of friction is somewhat higher when the slippage is pending than it is during sliding, separate coefficients for static (μ_s) and kinetic (μ_k) frictions are introduced. But in both cases the friction force acts tangential to the interfacial planes and with a direction opposite to the motion, or the impending motion.

In order to extend the theory to more general conditions, involving non-uniform distributions or non-planar surfaces, these basic assumptions are abstracted to infinitesimal limit. Thus total forces are replaced by surface tractions, the generalization of Equation (5.2-1) becomes:

$$\tau_t = \mu \tau_n \qquad (4.3\text{-}2)$$

In term of normal traction τ_n and tangential traction τ_t. This form is useful for determining the normal contact stresses that are often required for proper design. The above generalization will be later used for determining the overall characteristic of the friction rod, but for planar surfaces Equation (5.2-1) is used as basis of the frictional behavior.

The concept of Coulomb friction, as described above, provides the theoretical basis for most of the work that has been done concerning friction damping. However it should be noted that the Coulomb model is an approximate modeling of the friction and a very good one for simple mechanical problems but does not consider all of the influencing parameters, for example the friction coefficient that is assumed to be constant for a pair of sliding materials may also depend on the surface treatments between the two, and the environmental parameters like temperature. Anyway for the purpose of this research there is no need for the identification of modern theories of friction and the microscopic mechanisms involved with the interfacial bonding and the simple Coulomb theory for infinitesimal surfaces is satisfactory.

5. Rotational friction connection

In typical friction connection devices the horizontal loads that are applied due to lateral drifts in the structure, are directly the acting forces which result in the frictional behavior. In this research an innovative friction device is introduced that does not directly use the transversal applied forces on the friction surfaces, but instead other effects that the horizontal loads will have on the connection are used as a mean to trigger the friction mechanism. More specifically, the moments that are resulted from the out of plane distance (outward length of the connection) between the applied loads and their reactions, is used for creating the friction energy dissipation, Figure (48). For this reason the connection is called a rotational friction connection or the Friction Moment Rod.

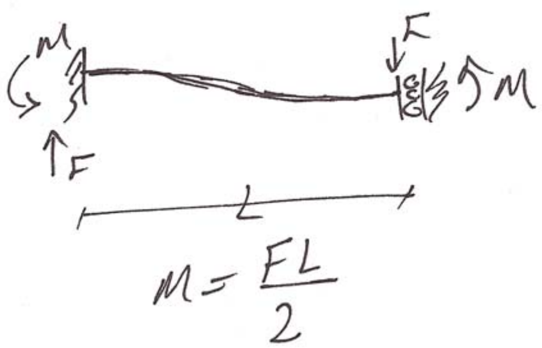


Figure 48: the resulting moments from apllied loads at the ends of connection device

Considering the rotational stiffness for the two ends of the connection the relation between the applied forces and the resulting moments becomes:

$$M_1 + M_2 = FL \ \rightarrow \ F = \frac{M_1 + M_2}{L} \tag{5-1}$$

Where M_1 and M_2 are the moments resulted at the two ends of the connection, one connected to the structure the other to the glazed system. Usually the connection is designed in way to eliminate or minimize the moments applied on the curtain wall system.

Although the energy dissipating ability of the friction connection devices (or any other advanced connection device) may not play a significant role in the dynamic response of the entire building in the case of light weight cladding systems – as in glass curtain walls –, but this property can play an essential role in the dynamic behavior of the envelope system itself. The energy dissipation, with minimizing the racking of the envelope system during and after a seismic event, will highly reduce the probability of damage occurring in the envelope system and its attachments.

. In this research the friction connection rod is introduced as an innovative connecting element that can almost be incorporated in all of the glazed envelope systems without disturbing their visual characteristics and at the same time protecting them against undesired forces caused by deflections within the actual structure. Two types of rotational friction devices are designed for unitized and suspended glazing systems. The basis of the behavior for both systems in mainly the same but due to geometrical, shape and attachments of these systems adjustments are made for these connections so that they can be easily adopted in the two different systems without much change and modification in the existing elements and attachments of these systems.

5.1. Development of the idea of rotational friction connectors

The idea of the rotation friction damping device rose from an attempt to satisfy the two main objectives below:

- To introduce a connection device than can handle relatively large displacements in as much degrees of freedom possible in rotation and translation.
- Maintaining the geometrical integrity of the envelope system and managing the connection device to behave in a controllable manner,

Typically to satisfy the first objective, adaptation of elastic or visco-elastic material seems to be the first candidate for the isolation mechanism, the same as the what is typically used in the bolting elements of the spider glazing systems to eliminate local stresses and reduce local moments imposed on the glass cornets, Figure (49). But these mechanisms have the disadvantage of undergoing some undesired and unaccounted for displacements. Such behavior would conflict with the second objective when dealing with relatively large displacements, this is specially true in cases where a moderate level of geometrical complexity is present or when then geometrical assembly tolerances are very low.

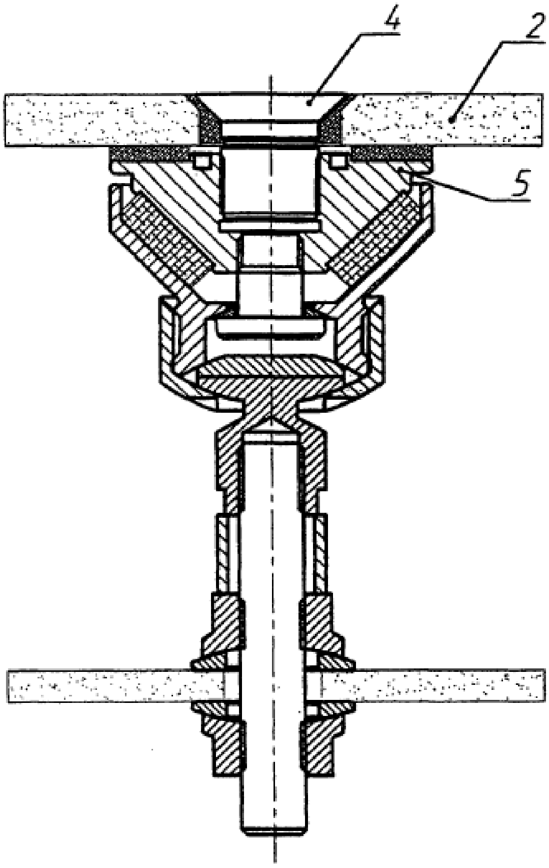
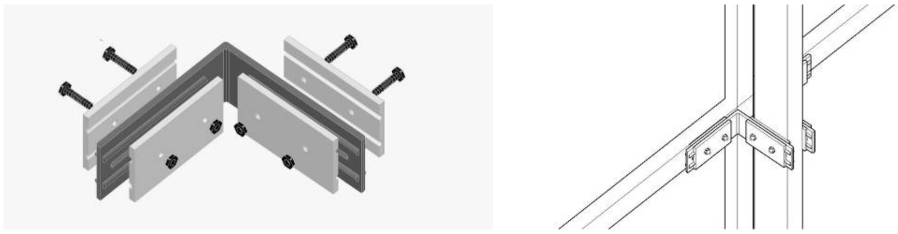


Figure 49: bolted attachement devices using elastic dampers

On the other hand among different mechanisms that could be adopted for satisfying the second objective mentions above, the friction mechanism was found to be the most appropriate. This is because basically friction connectors behave the same as rigid connections prior to the slippage and unless the forces and displacement applied on the envelope exceed a damaging limit they act as rigid connections hence maximizing the mechanical integrity of the system. But the problem with general types of friction connectors is that they usually can provide only one degree of freedom in transversal direction, also integrating them in geometrically complex systems is somehow challenging, Figure (50).



(a) (b)

Figure 50: Friction connector bracket

So it was decided to collect the advantages of these tow isolating mechanisms in a single connection device. Figure (51) demonstrates the first draft of the rotation friction device where instead of flat surfaces the friction occurs between two cylindrical surfaces and providing degrees of freedom in both translation and rotation.

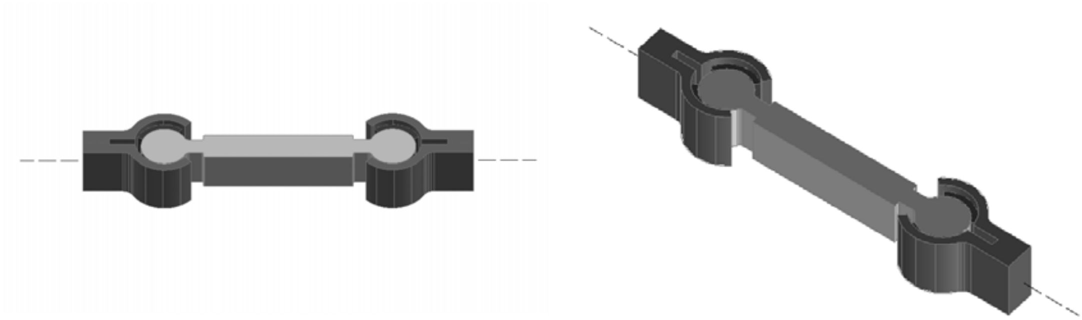


Figure 51: cylidrical rotational friction connector

Later the cylindrical components of the device where replaced by spherical elements to increase the degrees of freedom to both horizontal and vertical directions and increasing the geometrical adaptability of the device for more complex systems, Figure (52)

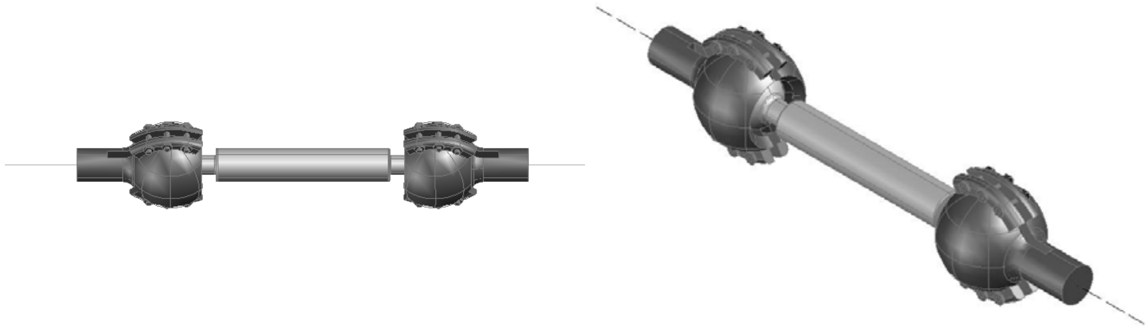


Figure 52: spherical rotational friction connector

Finally changes were made to the relative dimensions of the two ends of the connection device based on their anchor points. Due to relative delicacy and fragility of the envelope systems with respect to the supporting structure the elements on the side that was to be connected to the structure were considered to be larger in size than the ones connected to the envelope system. Figure (53). aside from directing the forces towards the larger and stiffer side this made the visual characteristics of the device more compelling and increased the aesthetical values of the device in exposed systems.

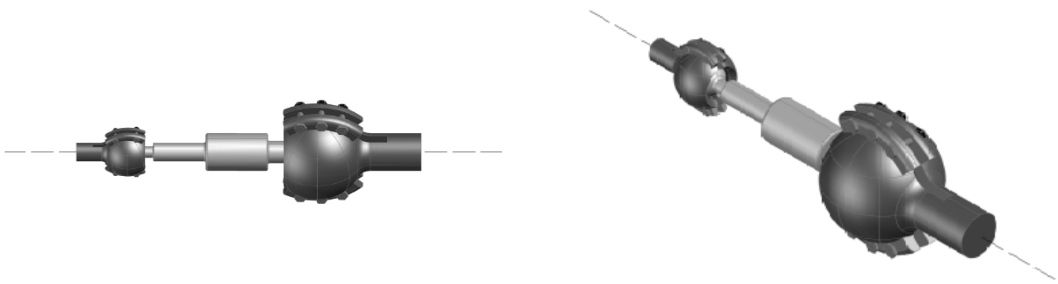


Figure 53: the Friction Moment Rod (FMR)

5.2. For suspended systems (Friction Moment Rod)

Ever since the introduction of friction damping connections to the building industry scholars and designers have introduced many devices to connect different structural systems or elements within one structural system together. Some installed between structure trusses, some to connect floor panels and some even to connect heavy concrete claddings to the main structure of the building, and they all have been used with the concept of maximizing the energy dissipation within the structure. The forces which acted upon them where in the level of structural forces and they were almost always hidden within the structure of the building and not a part of its outlook. Utilizing the friction connectors in a delicate system such as a curtain wall calls for great attention to many other aspects not usually considered during the design of these types of devices. Among these aspects are: aesthetical features of the device, the delicacy of the device and its applicability for different curtain wall manufacturing and construction techniques.

The friction brackets, replacing the common connecting elements for a typical curtain wall system, was already discussed, in general, in this research as one of the most simple friction connecting devices that can be cheaply and easily manufactured and used as curtain wall connecting devices. But it can only be used in a very limited group of glazed envelopes and special conditions for framing geometry. The possibility of benefiting from more geometrically and technically complex envelope systems with the ever improving technical advances within this field, along with the great tendency among architects and designers for adapting them in their buildings, calls for connecting devices compatible with these envelope systems.

One group of the envelope systems, which has gathered the attention of architects especially in the trend of modern architecture are the suspended glazing systems. But due to high exposure of these systems and high aesthetical demand over their composing elements the use of common dissipating or isolating connectors is impossible within them, and there exists a lack of proper connection devices that can provide isolation, if required, in these systems. And considering the facts that every individual glass pane in these systems is fully fastened to their supporting structure and there is no clearance between the composing panels, these systems have became highly susceptible to damage in the event of an earthquake. The friction moment rod is a connection device especially designed for these envelope systems in order to fill in the lack of proper connection devices for these relatively expensive envelope systems.

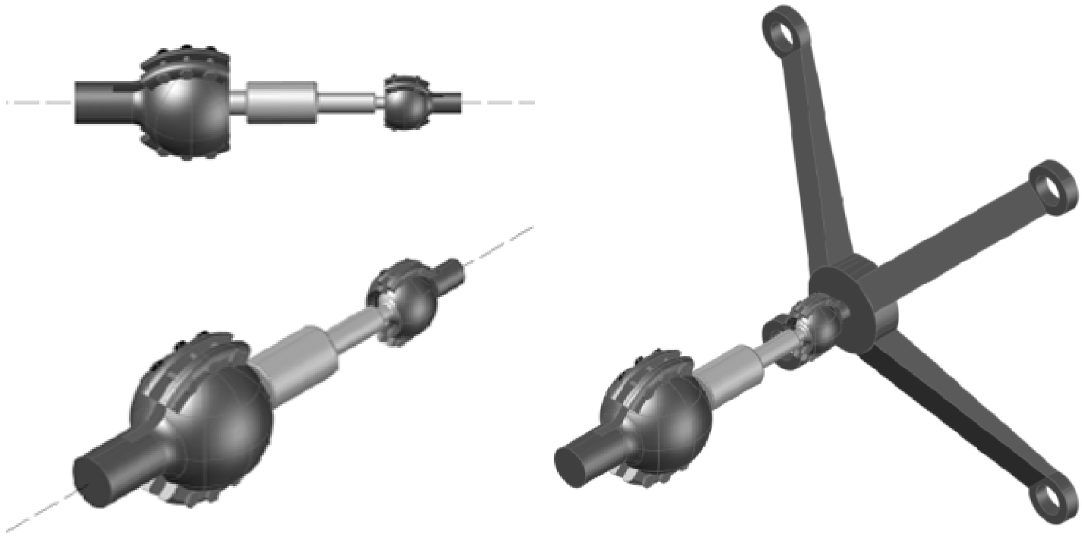
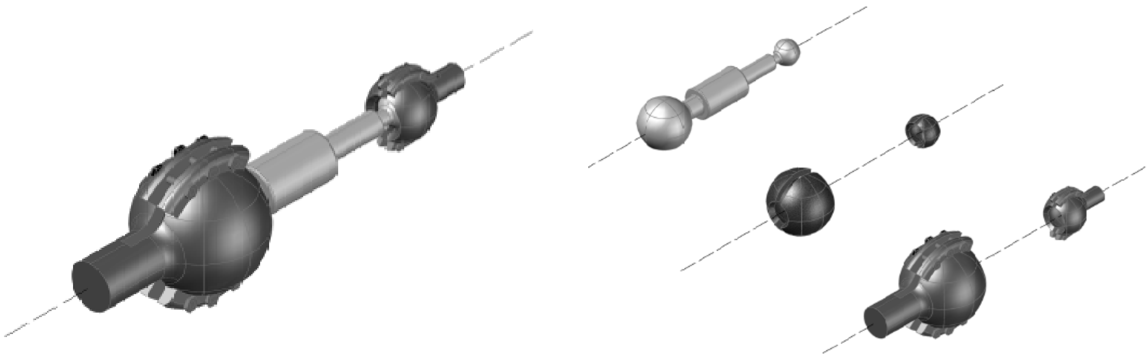


Figure 54: the Friction Moment Rod (FMR)

The friction rod is consisted of a tubular steel rod which is accompanied by two steel spherical balls at its two ends and a pair of supporting emptied spheres, over the steel balls attached to the rod, as it is shown in Figure (55). The connection of the spherical supports to the glazed envelope or to the structure of the building can vary depending on the requirements and details of the two systems. They can either be welded or screwed to the two system or they may already be attached to the elements used in the two systems during manufacturing, Figure (55-b). The spherical shapes of the two ends of the rod will give us the advantage of providing controllable isolation in four degrees of freedom, which are horizontal and vertical translations in the plane perpendicular to the axis of the rod and rotation over these two directions. This also makes the adjustments of the two systems quite easy during the construction phase.



(a) (b)

Figure 55: the Friction Moment Rod (FMR) components

As it is shown in Figure (55) the supporting spheres have an opening gap that divides them in two parts and a set of bolts are used over the opening gap to control the normal traction applied over the inner sphere that causes the tangential friction traction over the surface of the inner sphere. In order to guarantee a consistent and predictable frictional behavior the inner surface of the supporting spheres are covered with *2* or *3 mm* of brake lining material. The forces applied perpendicular to the direction of the middle rod will result in a torque at the two ends of the connecting device which triggers the frictional rotation of the inner sphere inside, these relations are described more in detail below.

5.3. For unitized and panelized systems

Based on the same concept that led to the design of Friction Moment Rod for suspended glazing systems, another rotational friction connection device is also presented in this research which is specially designed for unitized, panelized and stick curtain wall systems. Instead of having Sphere shaped balls at the two ends of the FMR, this device has two steel cylindrical attachments at the ends of the connection body which limit the rotation of the device only over vertical axis, and therefore it can only accommodate mechanical isolation in the horizontal direction. Other than the connection body there are two other composing elements in this connection; one which is anchored to the structure and the other to the curtain wall. The connection of this device to the structure is placed at the concrete floor deck with a steel plate that is embodied in the floor concrete and anchored to it, above the plate a triangular section attached to semi-cylindrical brackets is situated and the brackets will cover the cylindrical end of the connection body. Between the cylindrical surfaces again a brake lining pad material is used for consistent frictional behavior. A pair of pressure bolts control the pressure applied over the cylindrical tube and control the friction behavior of the connection device. At the other end, the connection body is pined to a U bracket attached to either vertical or horizontal mullions of the curtain wall, at this point no frictional behavior is expected and the bracket can freely rotate over the vertical axis around the pin. Schematic shape and details of this connection device is presented in Figures (56) and the details of the connection to the structure and the unitized curtain wall is demonstrated in Figure (57).

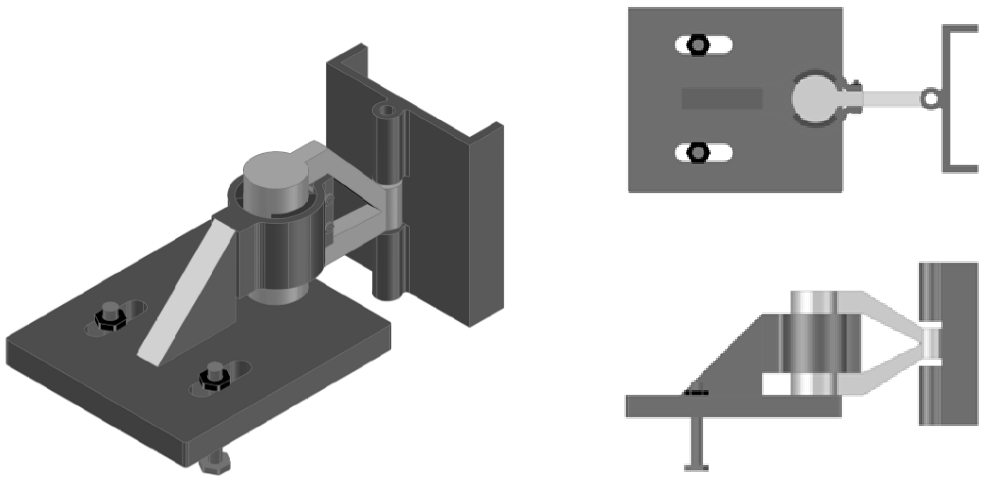


Figure 56: rotational friction connection for unitized and stick curtain wall systems

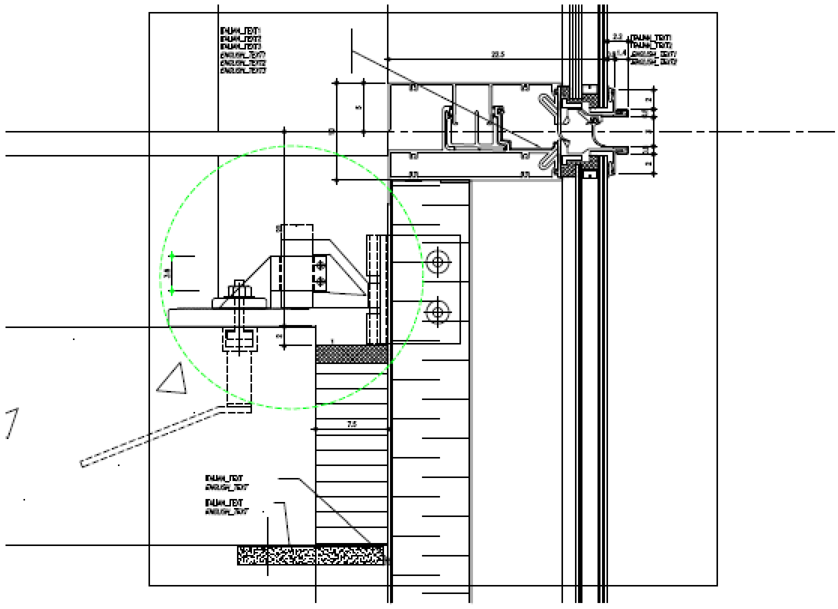
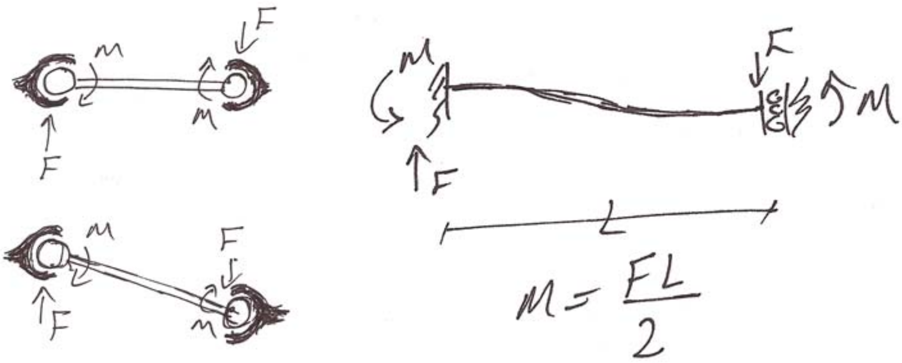


Figure 57: rotational friction connection for unitized and stick curtain wall systems

Unlike the suspended glazing systems, there are many solutions for providing mechanical isolation in unitized glass systems other than the rotational friction connection, mostely discussed in advanced connection devices in chapter 5 of this research. But one advantage of the rotational friction connection is that it is highly controllable and unless subjected to damaging forces – damaging to the envelope system – it acts exactly the same as a rigid connection, this will eliminate unaccounted for displacements in the curtain wall system and increase its overall reliability.

5.4. The behavior of the rotational friction connector

Figure (58) demonstrates the schematic behavior of the rotational friction connector for both unitized and suspended glazing systems.



(a) (b)

Figure 58: Structural model of the Friction Moment Rod

Before the slippage occurs between the spherical surfaces the structural model of the connection device is a clamped beam with displacement degree of freedom perpendicular to the axis of the beam as in Figure (59). And the resulting moment over the steel ball equals:

$$M_1 + M_2 = FL \;\rightarrow\; F = \frac{M_1 + M_2}{L} \tag{5.4-1}$$

Where F is the force cause by the lateral displacements of connection ends and L is the distance between the connection supports. Even after that the interacting surfaces of the spheres begin to slide over each other this relation will stay valid if the characteristics of the two ends are considered to be the same and the slippage starts to occur in both of the two ends at the same time.

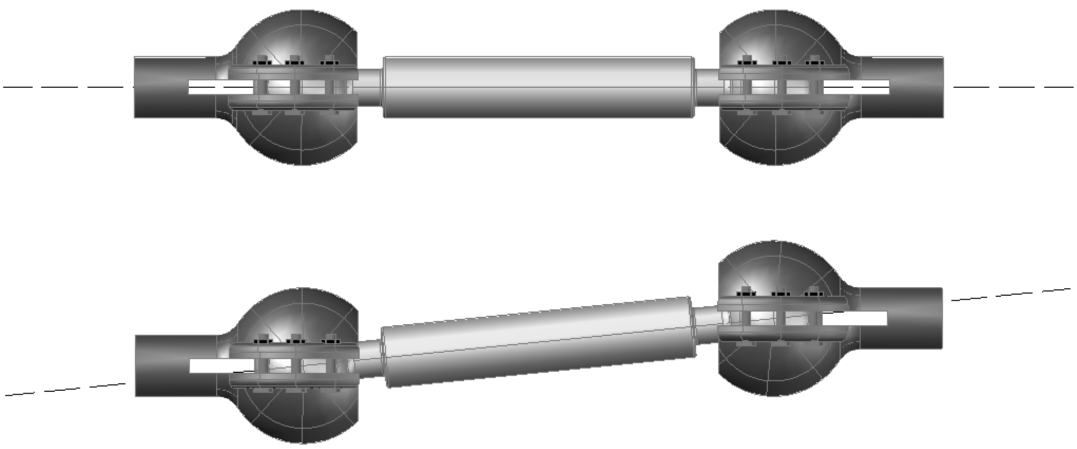


Figure 59: Slipage occurring in the Friction Moment Rod

It is now possible to control the lateral force transferred through the connection body by controlling the resulting moment at its two ends.

Formulations for the Friction Moment Rod (suspended systems)

On the other hand it can be shown that the torques at the two ends of the rod are the results of the friction tractions that happen between the outer surface of the steel ball and the inner surface of the supporting sphere.

Using the theory of solid friction described in section 7.2 for non planar surfaces and assuming that the normal traction over the surface of the steel ball caused by the joining bolts over the support is constant all over the surface we can find the relation between the torque *M* and the sum of the forces in the joining bolts.

Integrating the component of the normal tractions in the direction of the joining bolt over one half of the steel ball which corresponds to one side of the supporting sphere will result in:

$$N = \int_0^{\pi}\int_0^{\pi} R^2 \tau_n \sin^2(\theta)\sin(\varphi)\, d\theta d\varphi \tag{5.4-2}$$

$$N = \pi R^2 \tau_n \tag{5.4-3}$$

Where N is the force applied by the joining bolts through the supporting hemispheres and τ_n is the normal traction on the surface of the steel ball.

At the same time integrating the moment of the tangential friction tractions over the surface of the steel sphere gives us the amount of the acting torque M with respect to the tangential friction tractions. It should be noted that the tangential tractions over the surface of the steel ball oppose the direction of the slippage.

$$M = \int_0^{2\pi}\int_0^{\pi} R^3 \tau_t \sin^2(\theta)\sin^2(\varphi)\, d\theta d\varphi \tag{5.4-4}$$

$$M = \frac{\pi^2}{2} R^3 \tau_t \tag{5.4-5}$$

Having considered that for the infinitesimal surfaces we have:

$$\tau_t = \mu \tau_n$$

Determining τ_n from Equations (5.2-2) and (5.3.2-3) and putting into Equation (5.3.2-5) will give us the relation between the force applied by the bolt and the torque that it can handle, for each steel ball based on the pressure of the bolts and the radius of the sphere we have:

$$M = \frac{\pi}{2} \mu.R.N \tag{5.4-6}$$

Having in mind the Equation (5.3.2-1) relating the lateral force on the friction rod to its torque we can find the relation between the lateral forces which can be transferred through a friction rod, with respect to the pressing force applied on its ending steel balls with the joining bolts:

$$F_L = \frac{\pi.\mu}{2L}.\left(R_1 N_1 + R_2 N_2\right) \tag{5.4-7}$$

Formulations for the rotational friction connection for unitized systems

Again using the theory of solid friction described in section 7.2 for non planar surfaces, and assuming that the normal traction over the surface of the steel ball is constant all over the surface we can find the relation between the torque *M* and the sum of the forces in the joining bolts. The only difference here with the formulation of the FMR is that the integration are applied on cylindrical surfaces instead of spherical ones.

Integrating the component of the normal tractions in the direction of the joining bolt over one half of the steel cylinder which corresponds to one side of the supporting brackets will result in:

$$N = \int_0^h \int_0^\pi R\,\tau_n \sin\left(\theta\right) d\theta dz \tag{5.4-8}$$

$$N = 2R\,h\,\tau_n \tag{5.4-8}$$

Where *N* is the force applied by the joining bolts through the supporting hemispheres and τ_n is the normal traction on the surface of the steel cylender. Here *h* represents the height of the cylindrical steel that is covered inside the supporting brackets and is in contact with brake lining pads.

At the same time integrating the moment of the tangential friction tractions over the surface of the steel cylender gives us the amount of the acting torque M with respect to the tangential friction tractions. It should be noted that the tangential tractions over the surface of the steel cylender oppose the direction of the slippage.

$$M = \int_0^h \int_0^{2\pi} R^2 \tau_t d\theta dz \quad (5.4\text{-}10)$$

$$M = 2\pi R^2 h \tau_t \quad (5.4\text{-}11)$$

Having considered that for the infinitesimal surfaces we have:

$$\tau_t = \mu \tau_n$$

Determining τ_n from Equations (5.2-2) and (5.3.2-3) and putting into Equation (5.3.2-5) will give us the relation between the force applied by the bolt and the torque that it can handle:

$$M = \pi.\mu.R.N \quad (5.4\text{-}12)$$

Having in mind the Equation (5.3.2-1) relating the lateral force on the friction rod to its torque we can find the relation between the lateral forces which can be transferred through a friction rod with respect to the pressing force applied on its ending steel balls with the joining bolts:

$$F_L = \frac{\pi.\mu.R}{L} N \quad (5.4\text{-}13)$$

Adjusting the internal forces of the pressure bolts is the way to control the limit force of the connection device and now that we have the relation between pressure bolts and the limit force for both of the rotational friction connectors it is time to set limit force values for different glazing systems and sizes. This will be done in the next chapter dedicated to tuning the friction connection devices.

5.5. Friction lining material

Having discussed the basis of the behavior of the friction damping connectors it is now required to define materials and mechanisms that behave consistent to the theory in order to avoid unforeseen results. Pall et al. (Pall, Marsh & Fazio 1980) conducted static and dynamic tests on a variety of simple sliding elements having different surface treatments in order to find a system showing a consistent predictable response. Figure (60) shows the result of hysteretic behavior under displacement controlled cyclic loading.

As is apparent from the figure the system containing brake lining pads between steel plates provides a consistent predictable response. It is perhaps not surprising that brake lining materials would perform well in the dynamic tests, after all these materials have been specially developed over time in automotive industry to provide reliable frictional response.

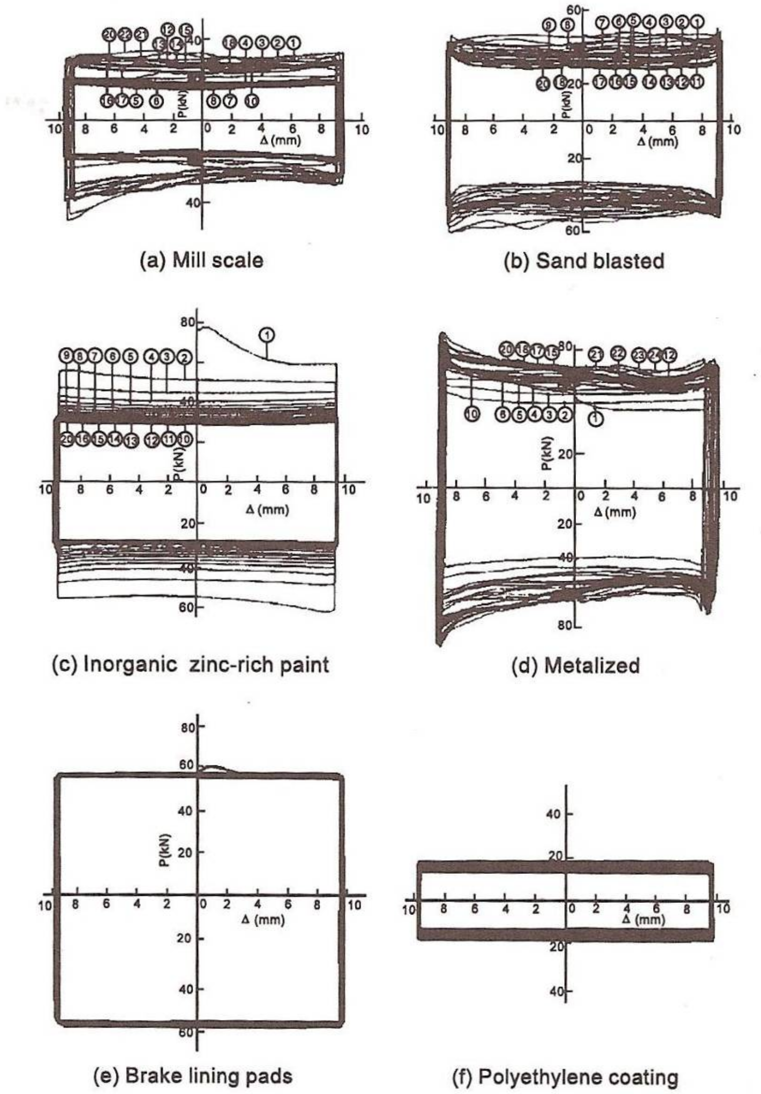


Figure 60: Hysteresis loops of limited slip Bolted Joints (Pall et al. 1980)

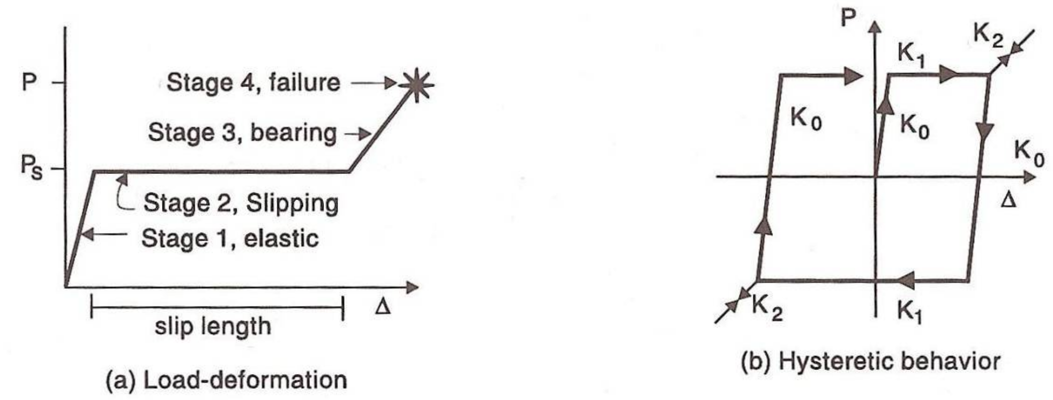


Figure 61: Force-displacement diagram and Hysteresis loops for lmited slip friction dampers with brake lining pad

Adaptation of brake lining material within the friction connector results in the below hysteretic behavior similar to an elastic-perfectly plastic model. Within the geometrical limits of the device the behavior of the friction connector will be demonstrated by Figure (62).

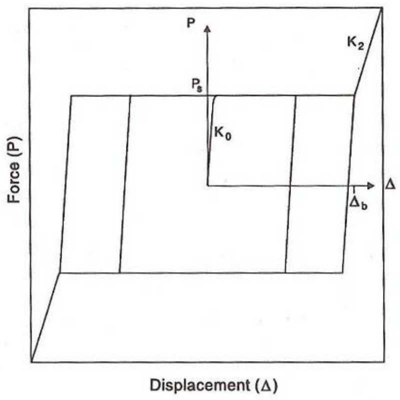


Figure 62: Macroscopic model for lmited slip friction dampers with brake lining pad

Where F_L is the limit force value of the connection device, Δ_L is the displacement which in each direction the friction connector can move freely and depends on the geometry of the connection and K_0 represents the mechanical stiffness of the connection device in the direction of the applied loads prior to the slippage.

6. Tuning the connector

Usually the devices which use solid friction phenomenon as their dissipative mechanism are tuned in order to maximize the energy dissipation by maximizing the area confined within their hysteresis loop. But the philosophy behind using the friction damping devices in connections of glazed systems to the main structure is rather different. These connection devices can be used in order to provide a desired level of isolation between the two systems. Although still energy dissipation will happen within the connection, considering the lightness of these envelope systems and their slenderness with respect to the elements of the main structure, it cannot have a significant effect on the overall dynamic behavior of the structure even if it is tuned for maximized energy dissipation.

Providing a desired level of isolation in mechanical terms means to limit the forces and moments acting from one system over the other. These limit forces will be based on the properties of the glazed envelope system and its load bearing capacity. This evaluation needs to be done in two parts. First within the safety criterion which is to protect the glass panels and its components from failure during or after a severe situation like an earthquake, and second within the serviceability criterion which is, in general, to maintain the desired behavior of the envelope system after the incident in terms of air-tightness and water-tightness and prevent distortions in the envelope surface.

To satisfy the safety conditions of a glazed system it is necessary to tune the friction damping device in a way that the forces which are transferred through it are kept below the maximum allowable forces which can be handled by the glass panel based on the positions of the connection devices. It is both necessary to study this effect both over a single glass panel and within a group of connected glass panels. This study will be limited to the silicon glazing systems where the use of advanced connection devices is of greater importance. An analytical approach is presented for tuning the friction based on the behavior of one panel window glasses. Then in order to check the results of the analytical solutions a numerical modeling will be performed. With the help of numerical modeling it is also possible to investigate more complex glass panel shapes, the effect of local stresses near the places of the friction damping devices and finally different placement of the connection device over the boundaries of the glass panel.

6.1. Analytical

What is here referred to as the analytical approach for tuning the connection device, is rather a decision making phase in the preliminary design stages of the connection device, rather than purely analytical solution for tuning the device. At the beginning of a mechanical design process it is necessary to have an understanding of the nature, order and magnitude of the applied forces, in order to make decision on the dimensions and the behavior of the device that is to be designed.

The results provided in chapter 4 of this research, on the behavior of glass panels in different glazed envelope systems are considered as the basis for determining the required information for the analytical study of the forces to design the connection device. Since it was already concluded that the buckling of glass pane is not the main reason of failure in unitized systems, the results for buckling of glass panes in suspended glazing systems will be investigated in this section for FMR connection, and the analysis of the design forces required for friction connections in unitized systems will be left over to the results of experimental tests.

Based on the discussions of chapter 4 of this research about the behavior of window glass panels during earthquakes, and assuming that the friction connection devices are situated at the four corners of the glass panel, the friction damping devices will be tuned based on the properties of the glass panel. By tuning the friction connector in this section we simply refer to deriving the limit forces to be transferred through the connection device.

Since the lateral displacements in a constant elevation are all of the same magnitude and direction the forces applied by the friction connectors will be also of the same magnitude (that is, of course, if the connectors are identical). Using the data of critical buckling stresses for glass panels of different sized and thicknesses it is possible to calculate an estimate for the limit forces of the friction connections. Having known that the horizontal displacements caused by the lateral drifts is the main source of damage to the envelope, only the horizontal component of the applied forces is assumed to be applied by the connection device. Hence, the sum of the limit forces of the connection device on one edge of the panel is assumed to be equal to the sum of horizontal components of the critical buckling forces. The results of the limit forces for two cases of buckling earlier discussed in chapter 4 are presented in table (12). The values of $F_{L,\tau}$ are the limit force values of the connection based on the distributed shear analysis and the value of $F_{L,P}$ are the limit force value based on diagonal pressure analysis.

Table 12: Values of tuning forces of friction damping connections for different glass panel sizes and thicknesses

Height	Width	thickness	$F_{L,\tau}$	$F_{L,P}$
m	m	cm	Kg	Kg
1.00	1.00	0.60	128.7492	149.708407
1.00	1.00	1.00	596.0612	693.094476
1.30	0.85	0.60	64.75553	68.9768507
1.30	0.85	1.00	299.7941	319.337272
1.50	1.20	0.60	68.66626	77.8970572
1.50	1.20	0.80	162.7645	184.644876
1.50	1.20	1.2	549.33	623.176458
2.00	1.00	0.8	76.29584	70.9728744
2.00	1.00	1.6	610.3667	567.782995
2.60	1.70	0.8	76.74729	81.7503415
2.60	1.70	1	149.8971	159.668636
2.60	1.70	1.6	613.9784	654.002732

These values are purely analytical values and they have to be evaluated with the help of the numerical modeling of the glass panels to provide safety factors for the actual design process.

6.2. Numerical

Determining the limit force of the friction connectors with the analytical process earlier described (based on the mechanical strength of the glass panels) is made possible with a group of simplifying assumptions for modeling the behavior of glass panels. The most important of these simplifying assumptions are: having shear buckling as the main cause of failure, uniform distribution of shear force along the edges of the glass panel and considering the glass pane to be part of an imaginary plate and performing the analysis on the imaginary plate. Although these assumptions make analytical modeling possible but we need to make sure they do not put the calculations distant from what happens in reality. One way to achieve more realistic results is to numerically model the behavior of the glazed system under applied loads and with the friction connections. Even though numerical modeling does not have the generality of the closed-form equations of analytical models, they can be used as a tool to evaluate the accuracy and authenticity of them. On the other hand with the numerical modeling it is possible to investigate the effect of other expected aspects in the systems that are not represented in the analytical model. Among these aspects are: the dynamic nature of the applied forces, the nonlinear behavior of the connection device, laminated glass properties, the local stresses in the glass panel near the connections and the interaction between the adjacent glass panels and its cumulative behavior.

If the results of the numerical and analytical models show considerable differences in value or the behavior of the system under same loading conditions, it can be concluded that the analytical model does not realistically represent the system in question and we need to generate more realistic models. This is true also in the case that the properties not included in the analytical modeling, like the dynamic nature of the applied forces, show significant effects during the numerical investigation of the system.

The numerical model is divided in two parts, where first the behavior of one panel with the connection devices is investigated and then the behavior of a group of connected panels.

First software, which is used for numerically investigating the system, is SAP2000. In the software the Thin-Shell elements are used for the glass panels and Link elements with an elastic-perfectly plastic behavior in the isolating directions are used for friction connection devices in nonlinear analysis. The modeling of the supporting structure is according to Eurocode 8 (British Standards Institution. 1996) when needed. Along with SAP2000 which is a general mechanical simulation software in structural engineering the SJ-MEPLA software is also used which is a commercial structural analysis developed specifically for glass structures and glass components of structural systems.

Considering the fact that newly introduced connection device named FMR (Friction Moment Rod) is highly suitable for exposed bolted glass systems and in order to reduce the effect of simplifications added to the model a curtain wall system of suspended glazing with bolted assembly (spider glazing) has been considered for the numerical simulations and experimental tests. This decision has been made under the influence of the fact that due to the somehow uniform texture of these systems the results of the numerical simulations will more likely be in close proximity to the real-time behavior of the glazing system. Also a great lack of study in the behavior of the spider glazing systems during and after earthquakes has provided sufficient motive to focus the attention on these glass envelope systems.

6.2.1. Behavior over one panel

The first step towards numerically analyzing the earthquake effect on a glass panels is to neglect the effect of other adjacent panels and concentrate on one individual glass pane subjected to forces that are caused by building drifts and applied through connection devices. Although to represent the true nature of the problem a dynamic simulation is required but first to evaluate the analytical procedures introduced above as the basis for tuning the friction connection devices. In order to evaluate the analytical model, in this part at first a single glass pane is subjected to a set of static forces, representing the limit forces of the friction connectors, in a linear static model.

Later a single panel with a set of connection devices connected to an arbitrary structure is modeled and analyzed using Ritz-vector nonlinear time-history analysis. The arbitrary structure is modeled in a way to show the allowable relative displacements similar to what occurs along a building story during an earthquake.

Static numerical simulation with SAP2000

At this stage, as described before, a single glass pane is subjected static loads which are applied through the connections of the glass panel to the structure of the envelope system. Figure (63) is a schematic structural demonstration of the simulation.

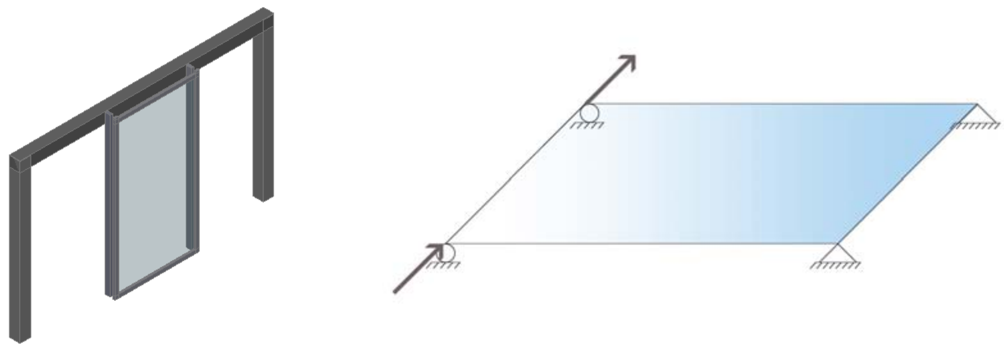


Figure 63: schematic demonstration of the numerical modeling

As shown above all the four resting points of the glass pane are provided with simply supported supports, resisting displacements in the normal direction to the surface of the glass pane, but allowing rotation over the supporting nodes for glass elements in all three dimensions, this is to avoid unnecessary stresses is the glass and mitigate the real-time behavior of the system. In addition to that the two bottom supports resist vertical and transversal movements of the glass ensuring the structural stability of the numerical model which is, of course, a necessity for the simulation. The effects of the transmitted lateral movements of structure are expressed with a set of two acting forces in the transversal direction, over the upper resting points of the glass pane. Again, in order to avoid singularity of forces and unwanted local stress fields in the glass, both the forces and the supports are distributed over the element nodes that correspond to the edges of the bolt holes in the glass pane.

In order to produce an accurate simulation of the stress fields and being able to generate a smooth deformation field in the glass pane, 4 node *thin shell elements* with a maximum dimension of 2.5cm in every direction are selected for the finite element analysis of the panel.

As for the mechanical characteristics of the glass, due to the common practice tempered glass and laminated glass as the components of the suspended glazing systems – due to post-failure safety reasons – the properties of these glass materials have been considered for the simulations, but as it turned out the elastic and pre-failure properties of these two types of safety glass materials are somehow the same as the monolithic glass and their major difference is related to the post-failure features and their shattering modes. Table (13) shows the mechanical properties typically considered for glass materials which are used in the simulations. These data have been extracted from material library of the CES database (Ashby 2010).

Table 13: glass table of proprties

Mechanical property	symbol	unit	value
Mass per unit volume	M	Kg/m^3	2400
Modulus of elasticity (young's modulus)	E	Pa	7E+10
Poisson's ratio	υ	-	0.2
Shear modulus	G	Pa	2.9E+10

Finally a buckling analysis is performed on the glass panes subjected to static forces and the results of the critical buckling forces derived from the SAP2000 software are presented in table

(14). The near buckling Von Misses stress state within the glass pane surface and the buckling deformation diagram is presented in Figures (64) through (75), in order to make comparisons with the results of the SJ-MEPLA software.

Table 14: critical buckling horizontal loads for glass panes (SAP2000)

Height	**Width**	**thickness**	**F_h**
m	m	cm	Kg
1.00	1.00	0.60	270
1.00	1.00	1.00	1200
1.30	0.85	0.60	115
1.30	0.85	1.00	515
1.50	1.20	0.60	120
1.50	1.20	0.80	270
1.50	1.20	1.2	910
2.00	1.00	0.8	110
2.00	1.00	1.6	880
2.60	1.70	0.8	110
2.60	1.70	1	210
2.60	1.70	1.6	860

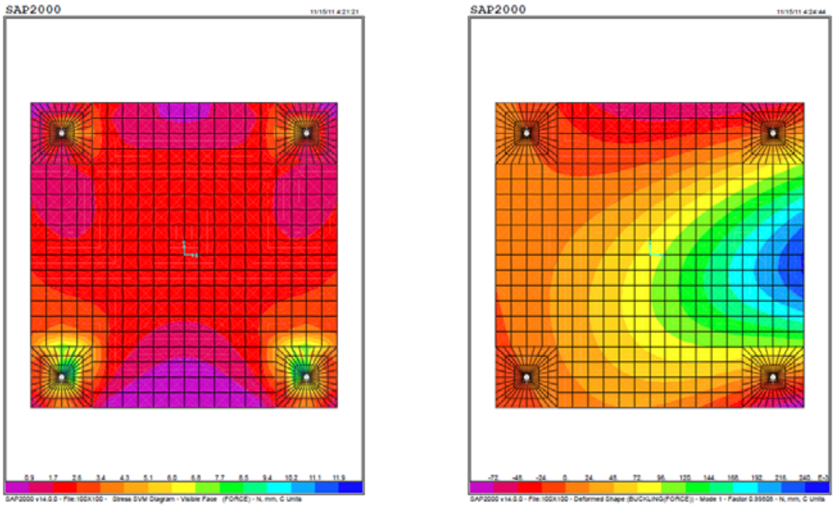


Figure 64: Von Misses stress state and buckling deformation diagram (panel 100X100X0.6)

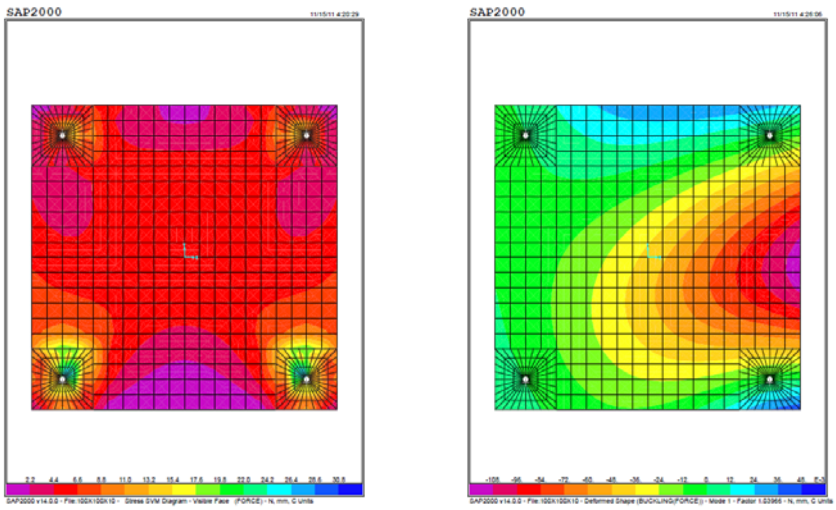


Figure 65: Von Misses stress state and buckling deformation diagram (panel 100X100X0.8)

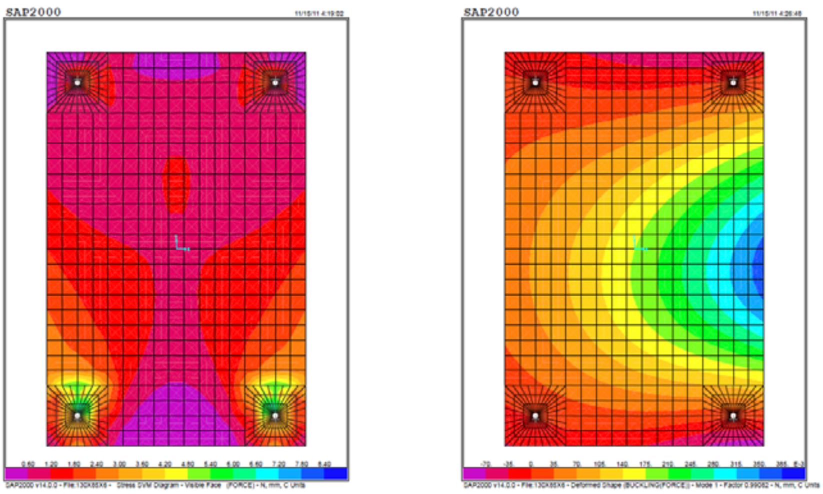


Figure 66: Von Misses stress state and buckling deformation diagram (panel 130X85X0.6)

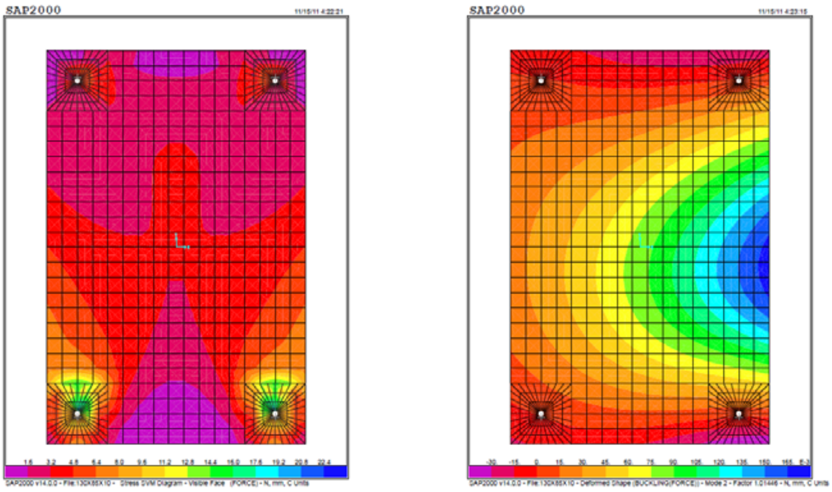


Figure 67: Von Misses stress state and buckling deformation diagram (panel 130X85X0.8)

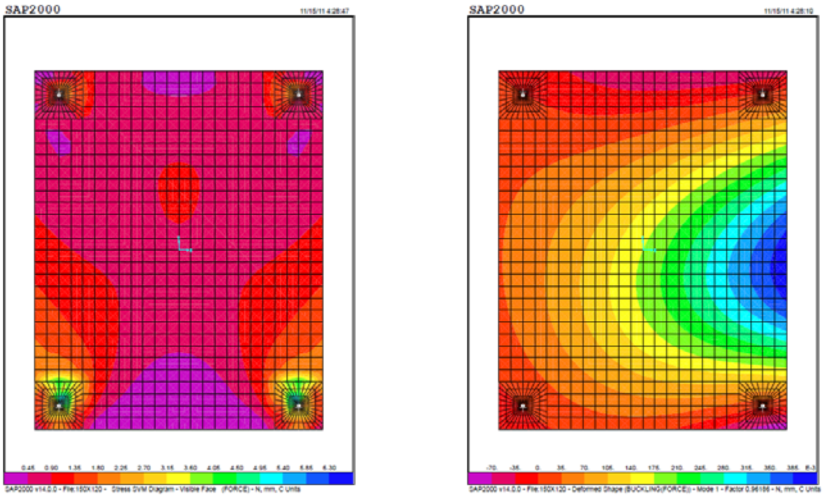


Figure 68: Von Misses stress state and buckling deformation diagram (panel 150X120X0.6)

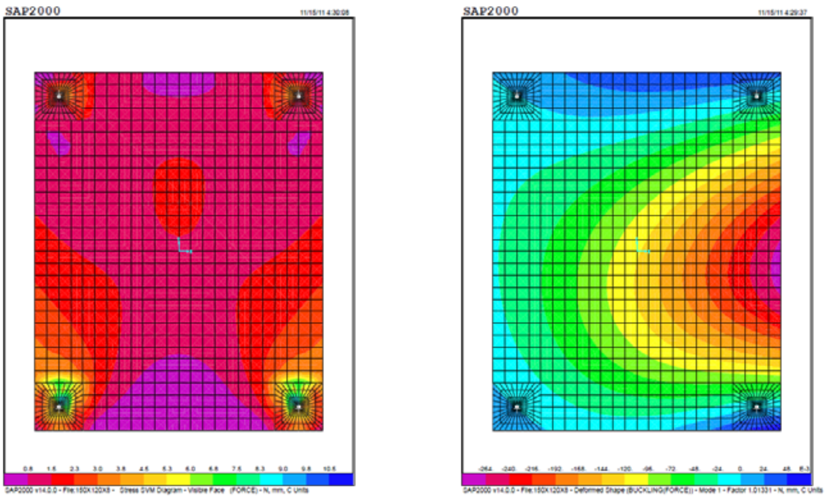


Figure 69: Von Misses stress state and buckling deformation diagram (panel 150X120X0.8)

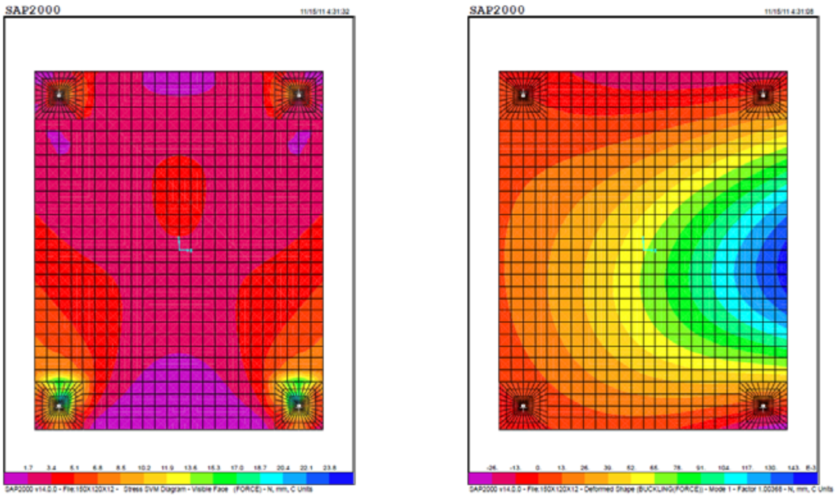


Figure 70: Von Misses stress state and buckling deformation diagram (panel 150X120X1.2)

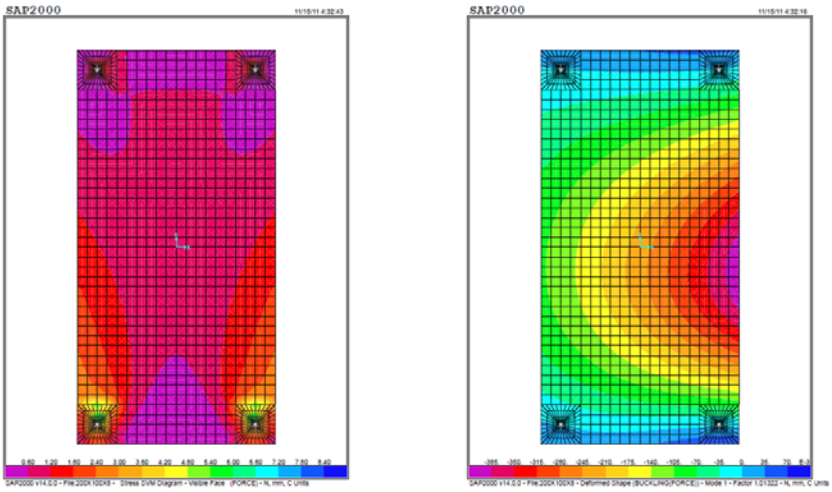


Figure 71: Von Misses stress state and buckling deformation diagram (panel 200X100X0.8)

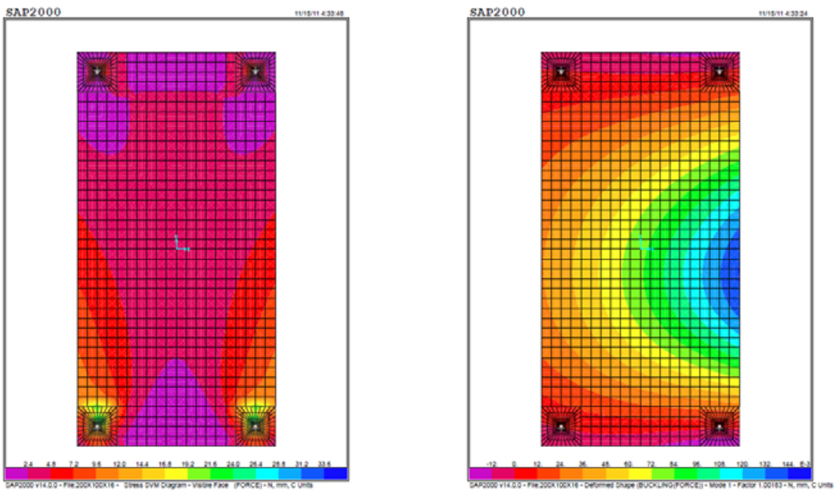


Figure 72: Von Misses stress state and buckling deformation diagram (panel 200X100X1.6)

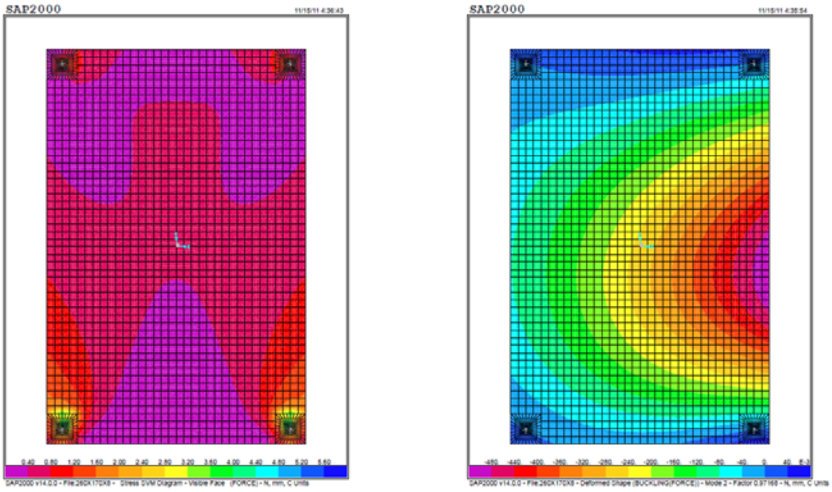


Figure 73: Von Misses stress state and buckling deformation diagram (panel 260X170X0.8)

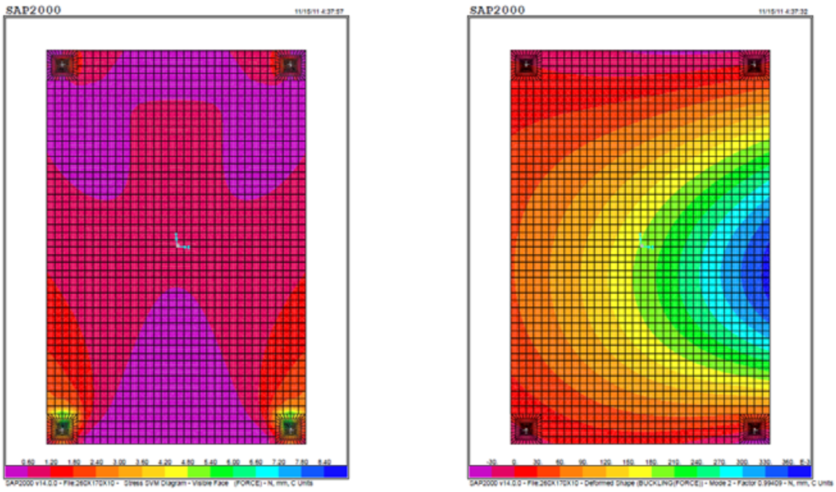


Figure 74: Von Misses stress state and buckling deformation diagram (panel 260X170X1.0)

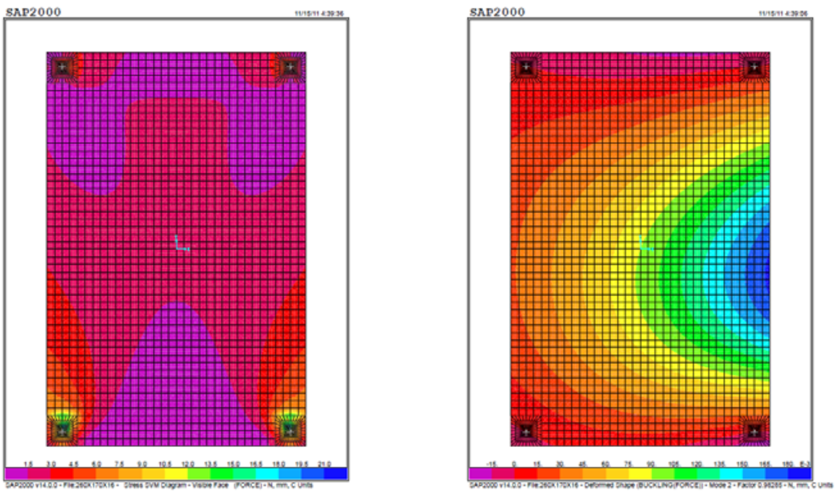


Figure 75: Von Misses stress state and buckling deformation diagram (panel 260X170X1.6)

Static numerical simulation with SJ-MEPLA

Although being a very powerful mechanical analysis software for nonlinear static and dynamic simulations, the SAP2000 software does not have the ability to model sandwich shell or plate elements, and for including the characteristics of laminated glass e bending stiffness reduction equal to *0.3* was inserted in the mechanical properties of the glass shell elements in SAP2000. This reduction on the stiffness of the shell elements is based on the corrections in the thickness of laminated glass elements, earlier discussed in chapter 4.

In order to double check the results of SAP another numerical simulating software bye the name SJ-MEPLA has been used. SJ-MEPLA is commercial finite element software, that is specifically developed to perform static and dynamic analysis, for glass structures and glass elements within a structure. With MEPLA it is possible to simulate any shape and details of glass components including; laminated glass, point glass fixings, framing supports around glass panels and etc. every small detail that is within the glass pane or its typical connections can be included in the simulations by MEPLA.

As mentioned earlier SJ-MEPLA is software that performs the calculations based on a finite element model. Contrary to SAP which uses *4 node shell elements* in MEPLA *9 node shell elements* are used for finite elements, so it is possible to it is possible to have a mesh composed of bigger elements than the elements of SAP file but with the same accuracy. The *9 node shell element* can be demonstrated as a rectangle having second order parabolic curves instead of strait lines between the corner edges. Figure (76)

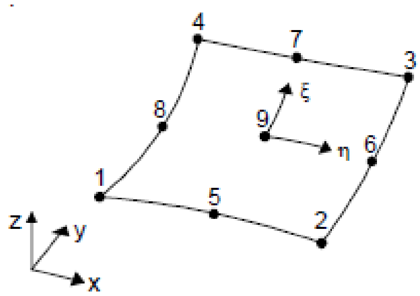


Figure 76: the 9 node finite element used in SJ-MEPLA

Using an automatic mesh producing algorithm the MEPLA will divide the glass pane into appropriate shell elements. It is only necessary to indicate the maximum element dimensions.

Based on the suggestions in the software the maximum shell dimensions varies between 12 to 20 centimeters for different glass dimensions smaller shell dimensions will be used automatically by the software where they are needed, for example near bolted corners of the glass, Figure (77)

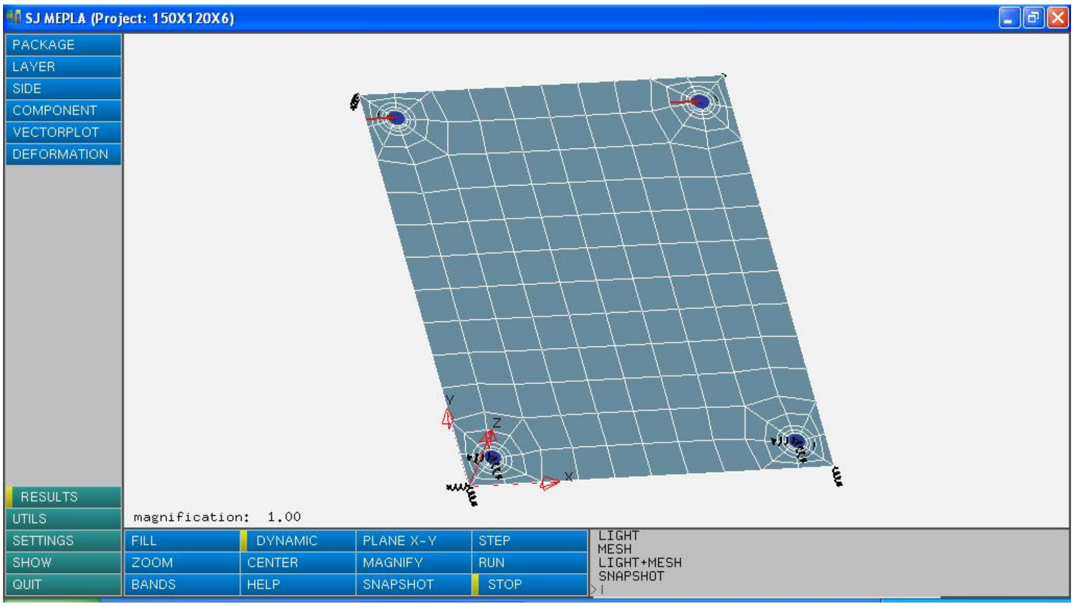


Figure 77: the automatic meshing of the glass panel

Inputs to MEPLA:

The geometry of the glass panes investigated in MEPLA are the same as the ones earlier modeled in SAP, but the thicknesses of the glass panes is associated in a layered system. For every thickness of the glass pane table (15) shows the details of the laminated glass used in MEPLA software.

Table 15: laminated glass thicknesses used in SJ-MEPLA

Glass thickness	t_{glass}	t_{PVB}
mm	mm	mm
6	3	0.38
8	4	0.38
10	5	0.38
12	6	0.76
16	8	0.76

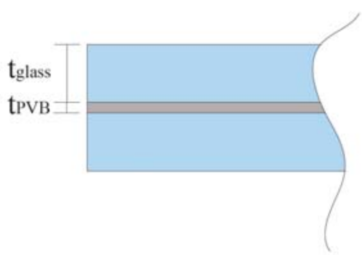


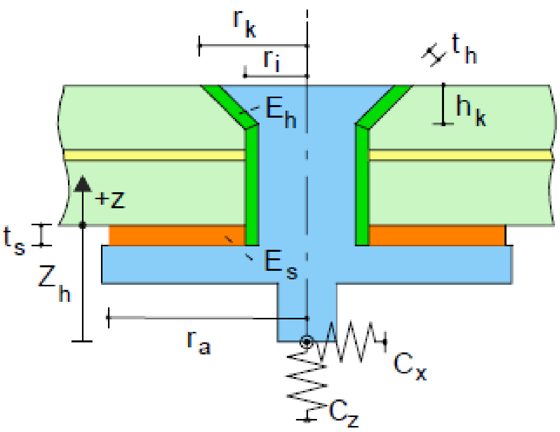
Figure 78: laminated glass

The material properties associated to glass and the PVB interlayer are the same as in SAP simulations. For the properties that are not present is SAP files the suggestions of the software has been used for the simulations. Table (16) shows the values of the properties used in simulations.

Table 16: table of properties for glass and PVB (SJ-MEPLA)

Mechanical property		symbol	unit	value
Glass	Mass per unit volume	M_G	Kg/m^3	2400
	Modulus of elasticity (young's modulus)	E_G	Pa	7E+10
	Poisson's ratio	υ	-	0.2
PVB	Mass per unit volume	M_{PVB}	Kg/m^3	1000
	Modulus of elasticity (young's modulus)	E_{PVB}	Pa	3E+4
	Poisson's ratio	υ	-	0.5

The bolted fixings of glass pane are considered to be countersunk fixings shown in Figure (79) and the dimensions of the countersunk fixings used in the simulations are presented in Figure (79). The two fixings at bottom of the glass pane are assumed to be fixed to the supports while the damaging forces will be applied on the top fixings of the glass pane.



r_i	r_a	E_s	E_h	t_s	t_h	h_k	r_k
18	35	60.	500	3	2	3	28

Figure 79: details of the countersunk fixing supports used in the simulations

the maximum bearing forces on the glass panes which result in the buckling of glass, based on the outputs of MEPLA are presented in table (17), also the stress state and the out of plane deformations of the glass panes near the buckling state are demonstrated in Figures (80) to (92).

Table 17: critical buckling horizontal loads for glass panes (SJ-MEPLA)

Height	Width	thickness	F_h
m	m	cm	Kg
1.00	1.00	0.60	270
1.00	1.00	1.00	900
1.30	0.85	0.60	130
1.30	0.85	1.00	450
1.50	1.20	0.60	120
1.50	1.20	0.80	250
1.50	1.20	1.2	700
2.00	1.00	0.8	110
2.00	1.00	1.6	750
2.60	1.70	0.8	110
2.60	1.70	1	200
2.60	1.70	1.6	740

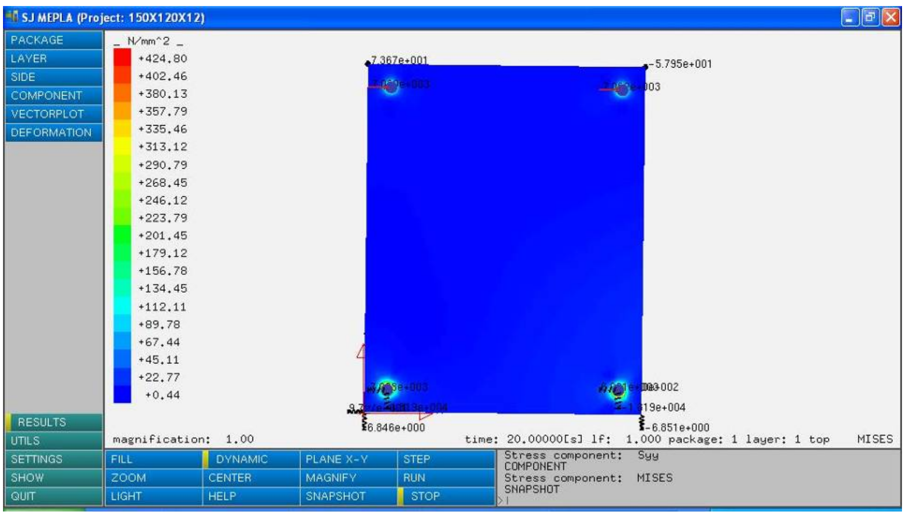


Figure 80: Von Misses stress state (panel 150X120X0.6)

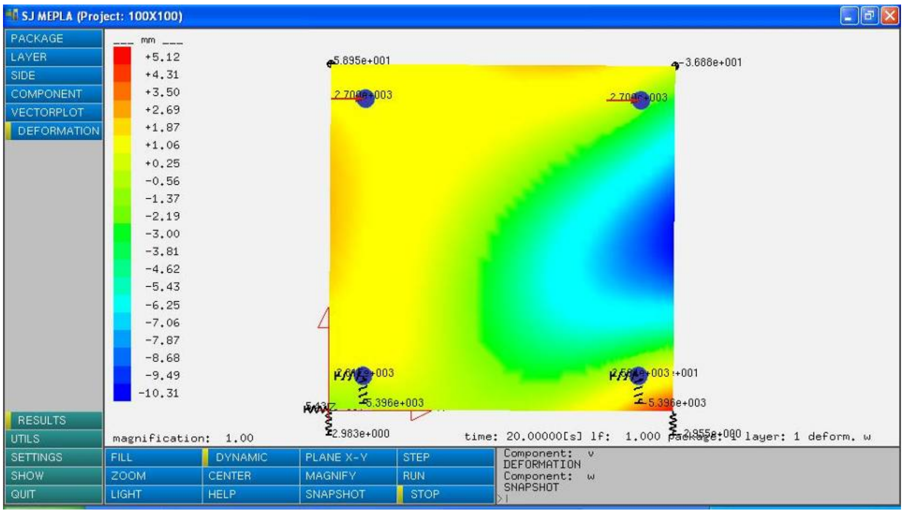


Figure 81: out of plate deformation near buckling state (panel 100X100X0.6)

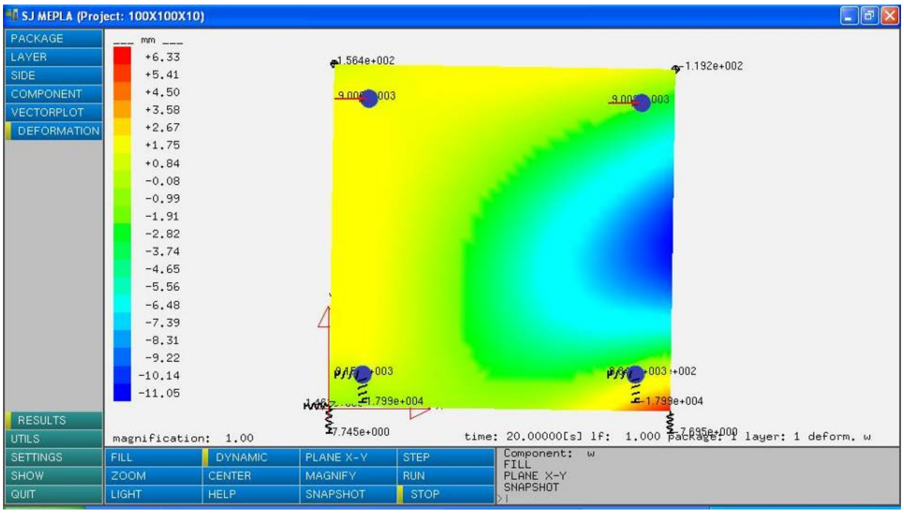


Figure 82: out of plate deformation near buckling state (panel 100X100X0.8)

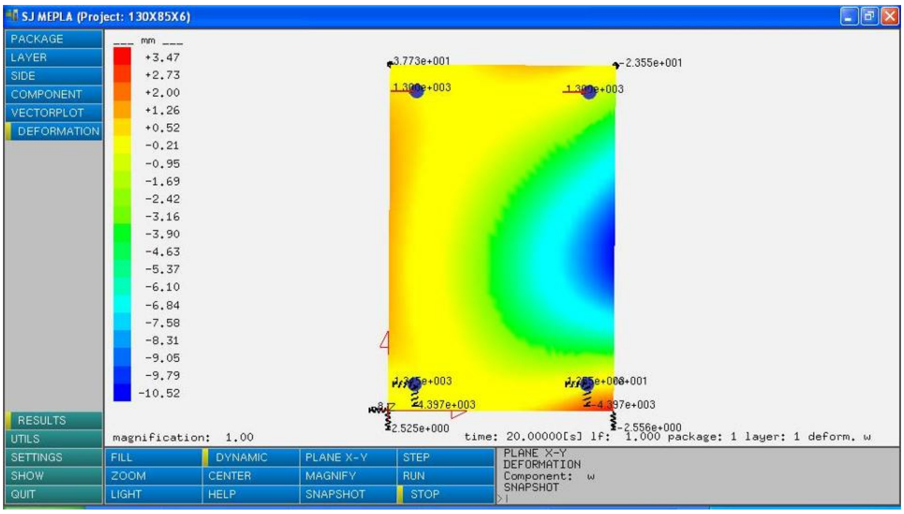


Figure 83: out of plate deformation near buckling state (panel 130X85X0.6)

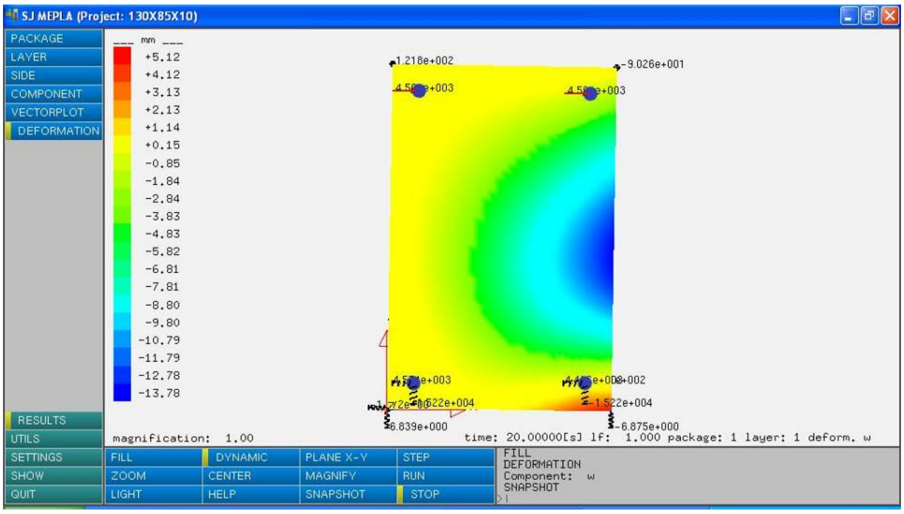


Figure 84: out of plate deformation near buckling state (panel 130X85X0.8)

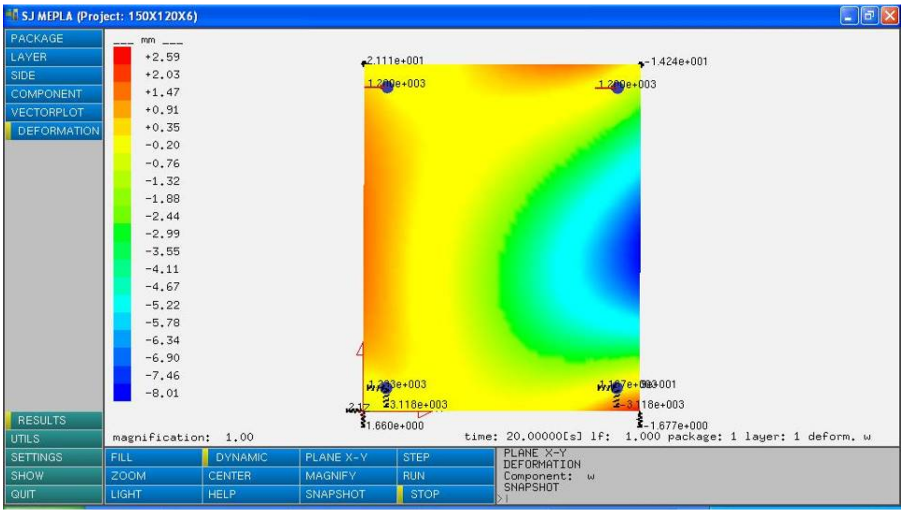


Figure 85: out of plate deformation near buckling state (panel 120X150X0.6)

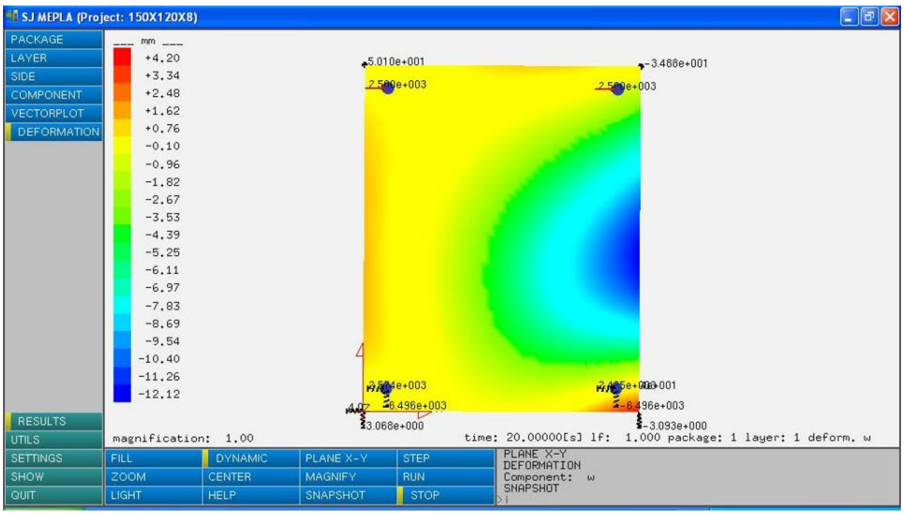


Figure 86: out of plate deformation near buckling state (panel 120X150X0.8)

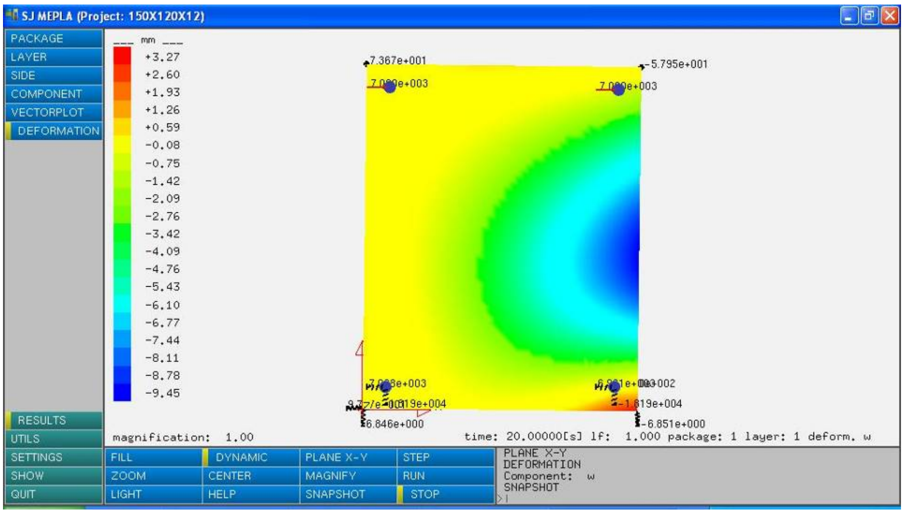


Figure 87: out of plate deformation near buckling state (panel 120X150X1.2)

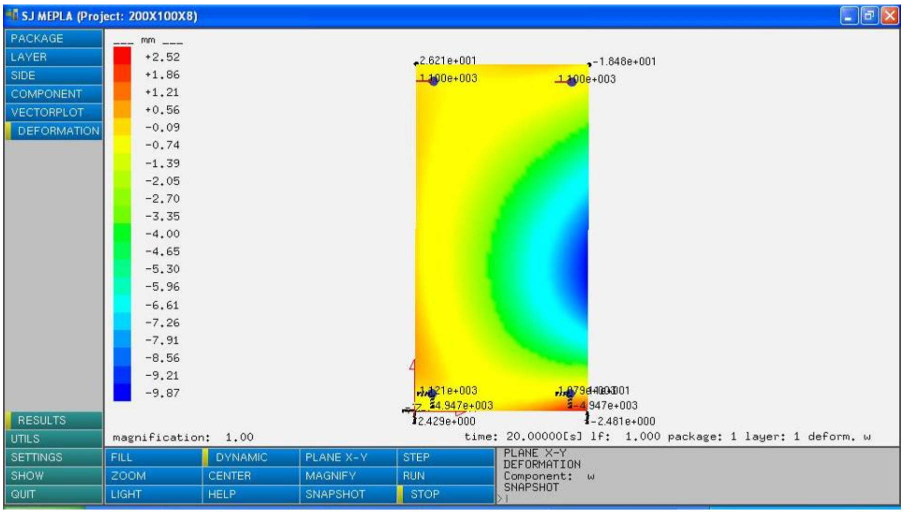


Figure 88: out of plate deformation near buckling state (panel 200X100X0.8)

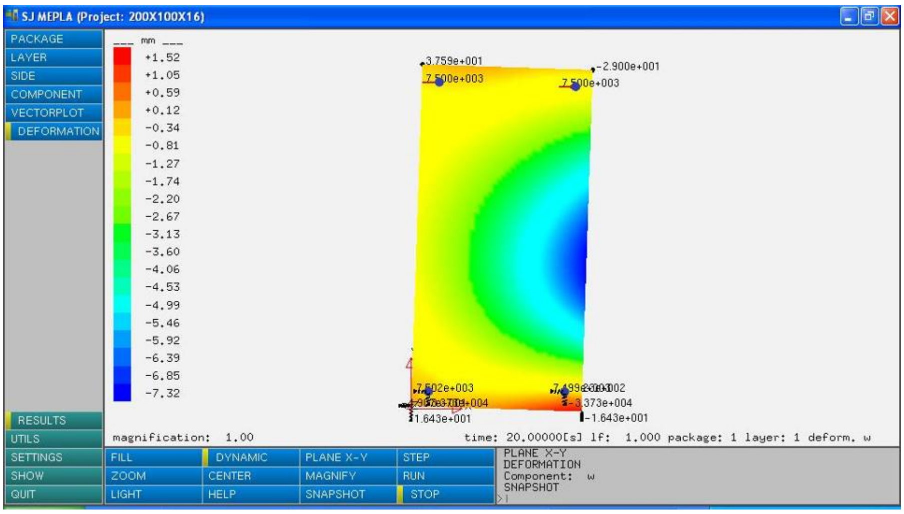


Figure 89: out of plate deformation near buckling state (panel 200X100X1.2)

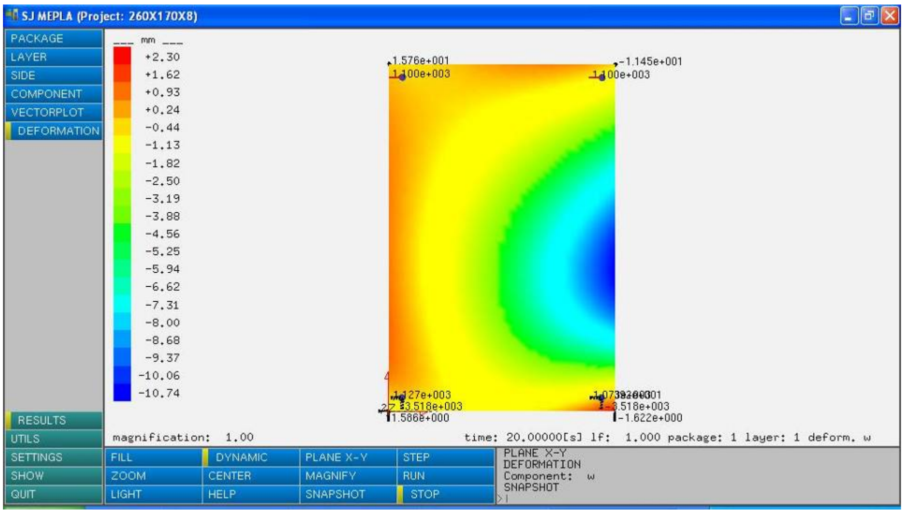


Figure 90: out of plate deformation near buckling state (panel 260X170X0.8)

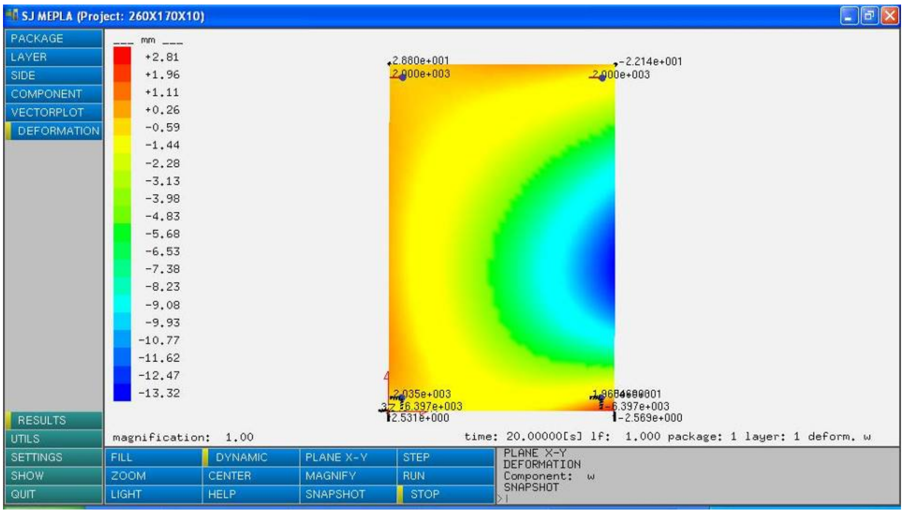


Figure 91: out of plate deformation near buckling state (panel 260X170X1.0)

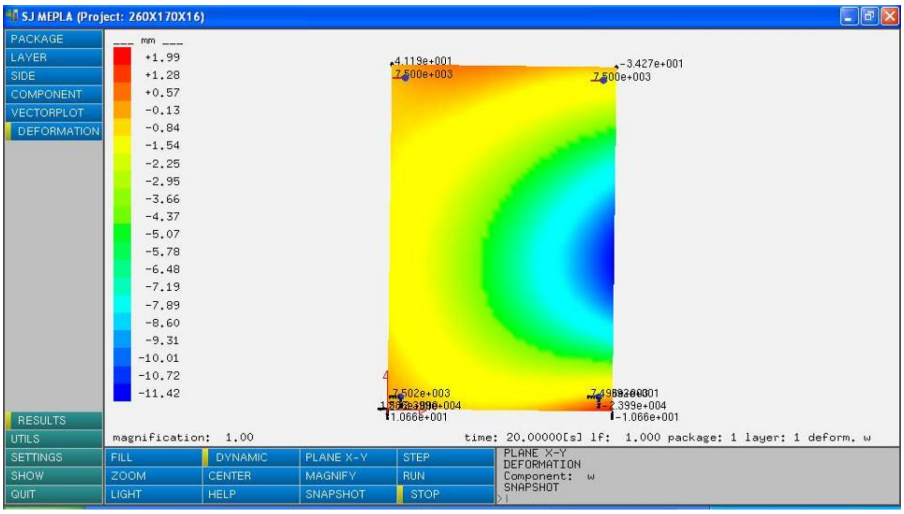


Figure 92: out of plate deformation near buckling state (panel 260X170X1.6)

It can be seen that for thin glass panels the results of the two software appear to be very close but for the thicker glass panes the results start to show variations in a range of 10 to 20 percents. This is probably due to the fact that in the thicker glass panels which have an interlayer of *0.76mm* the effect of PVB interlayer is more influencing. The buckling deformation diagrams of the SAP are almost the as the out of plane deformations in MEPLA and they both suggest the same mode of buckling for the glass panes and the place where the buckling starts. In both cases the buckling will start a little below the middle of the vertical edge that is in the direction of loading.

Result comparison

Table 18: result of the four approaches considered for analyzing the mechanical buckling of glass panes

glass pane dimensions	shear buckling scenario	diagonal pressure scenario	SJ-MEPLA	SAP2000
	kg	kg	kg	kg
100X100X0.6	128.75	149.71	270	270
100X100X0.8	596.06	693.09	900	1200
130X85X0.6	64.76	68.98	130	115
130X85X0.8	299.79	319.34	450	515
150X120X0.6	68.67	77.90	120	120
150X120X0.8	162.76	184.64	250	270
150X120X1.2	549.33	623.18	700	910
200X100X0.8	76.30	70.97	110	110
200X100X1.6	610.37	567.78	750	880
260X170X0.8	76.75	81.75	110	110
260X170X1.0	149.90	159.67	200	210
260X170X1.6	613.98	654.00	740	860

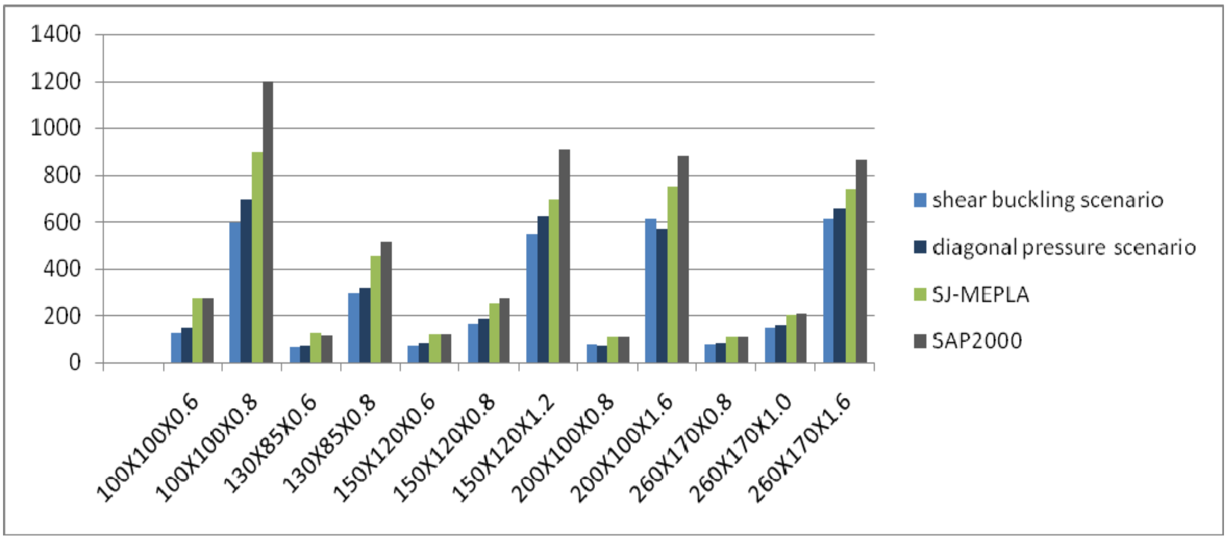


Figure 93: comparison chart

6.2.2. Dynamic modeling

The dynamic features of MEPLA is limited to a vibration analysis for the case of pendulum impact – a load case for determining the glass fallout – and it is not possible to perform the desired dynamic analysis with MEPLA, and the dynamic simulations will only be done in SAP2000 software.

All the previous numerical modelings were by imposing forces at the connection points on the glass pane, but in reality the forces that are applied on the glass are the product of displacements that occur in the structure during an earthquake. So in the dynamic modeling of the systems, instead of having the glass pane subjected to acting forces, displacements are assigned to the glass supports and a time-history dynamic nonlinear analysis is performed. Since the imposed displacements are assumed to be equivalent to the lateral movements in the structure, they are considered to be governed by a sinusoidal function of time which best describes the lateral drifts in the structure during earthquakes. Although the time period of the sine function strongly depends on the building natural frequency and its natural period, a time period equal to *1 sec.,* which is close to natural vibration period of regular midrise buildings, is considered for the sine function. The maximum allowable drift in the structural design of the buildings, in most building codes, is considered to be equal to *0.02* but in this research a drift value of *0.01* has been considered sufficient to observe the dynamic behavior of the structure. So the magnitude of the sine function for loading displacements is considered *0.01* multiplied by the height of the glass and three periods of loading in imposed on the glass for every simulation, Figure (94).

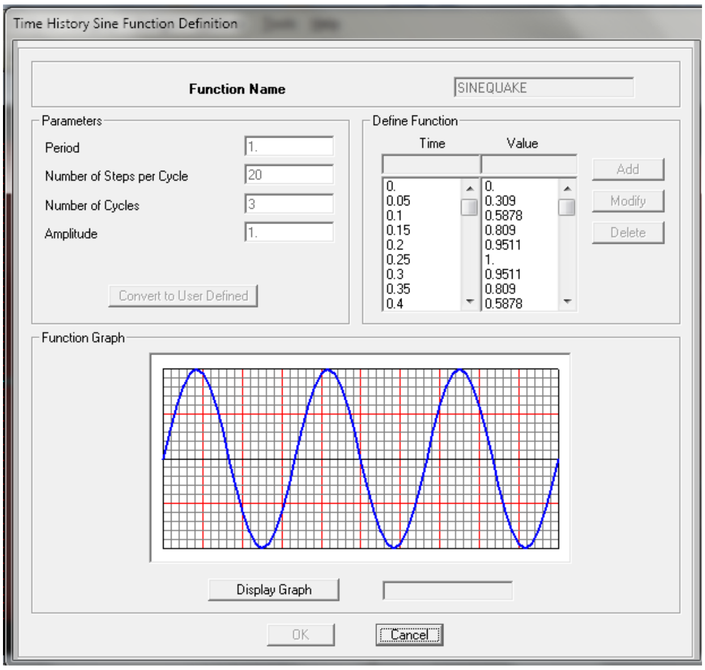


Figure 94: sinusoidal displacement function representing the lateral drifts of the main structure

At this point two cases of connection to the main structure are considered; first when there are rigid element connecting the glass to the supports and second when FMR (Friction Moment Rod) is placed between the glass and the supports.

In order to simulate the FMR in the model, a Link element with multi-linear deformation with a perfectly plastic limit equal to the limit force of the FMR –based on the buckling analysis is used in the place of the connections, Figure (95).

Figures (96) show the stress states at the peak of horizontal displacements of the supports, in the two conditions of using rigid connections and FMR. Animations from the dynamic simulation on glass panels can be found in the CD attached to this document containing numerical simulation files and other outputs that cannot be imprinted in the report.

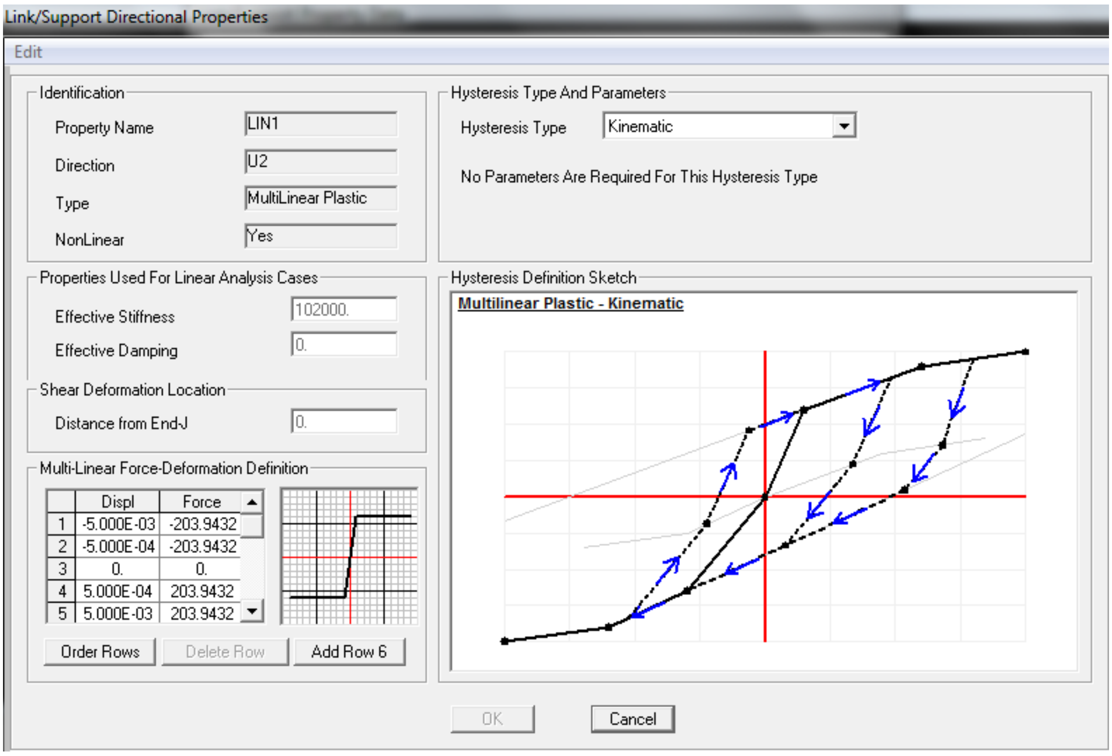
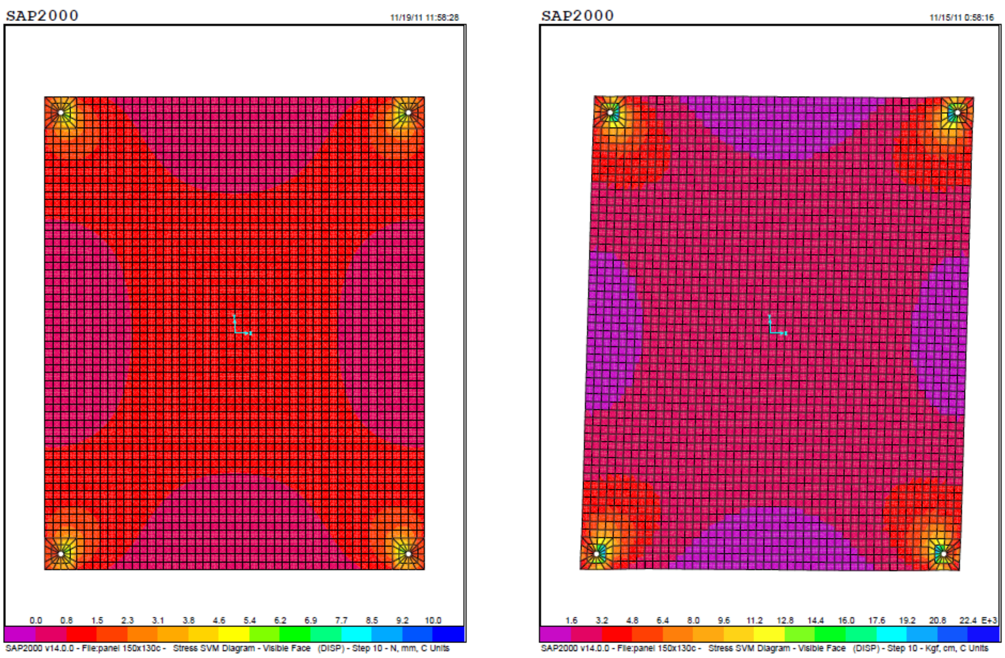


Figure 95: Elastic-perfectly plastic Link elemets properties for connecting the glass panel to the structure



(a) (b)

Figure 96. Von Misses stress states at the peak of horizontal displacements of the supports a) connected with FMR; b) connected with rigid connections

Comparing the legend of the two figures above it is obvious that glass panel without connection devices has far passed the stresses that can be handled by glass materials. The lateral displacement of the top left corner of the two cases during and after loading is also compared in Figures (97) and (98)

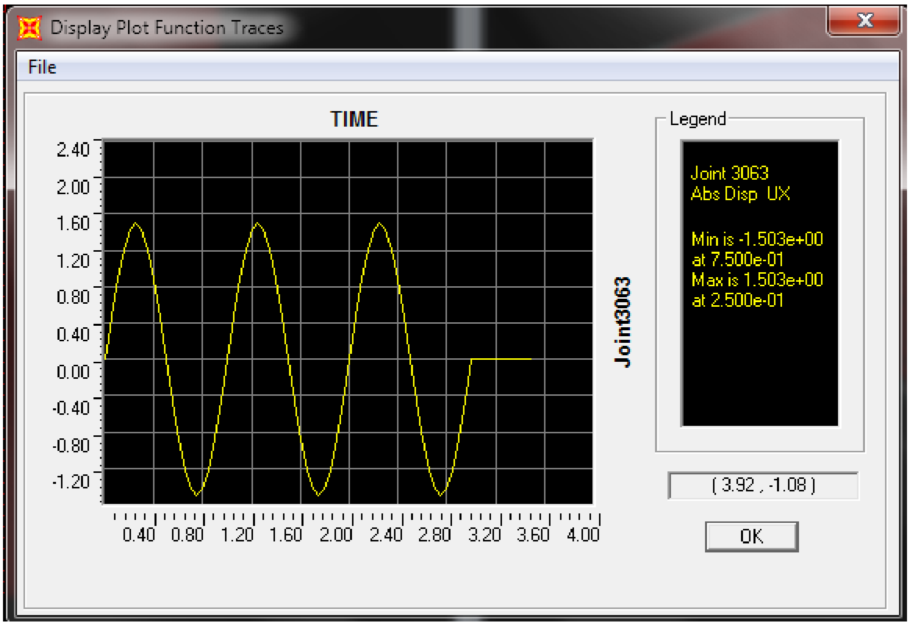


Figure 97. lateral displacement of the top left corner of the panel connected with rigid connections

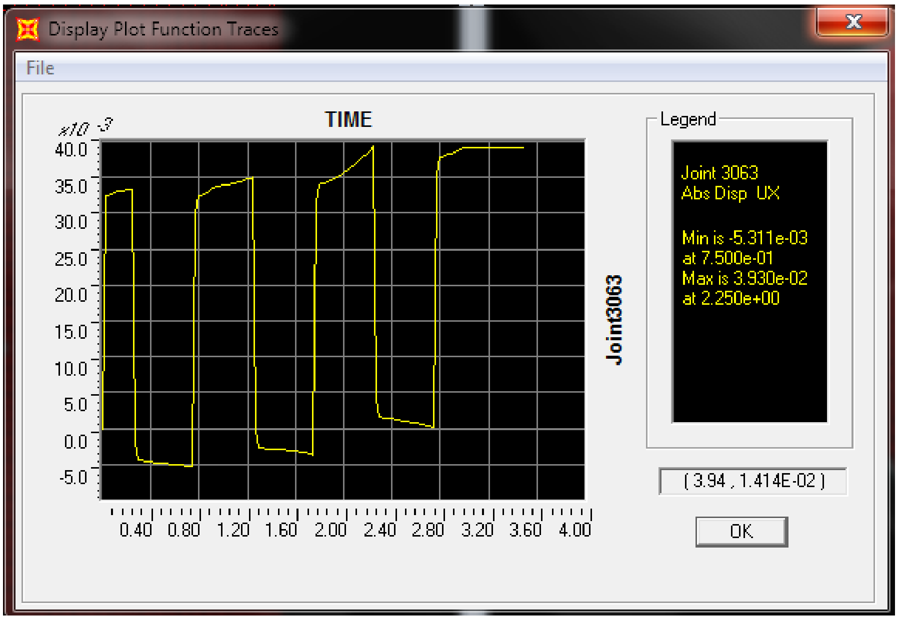
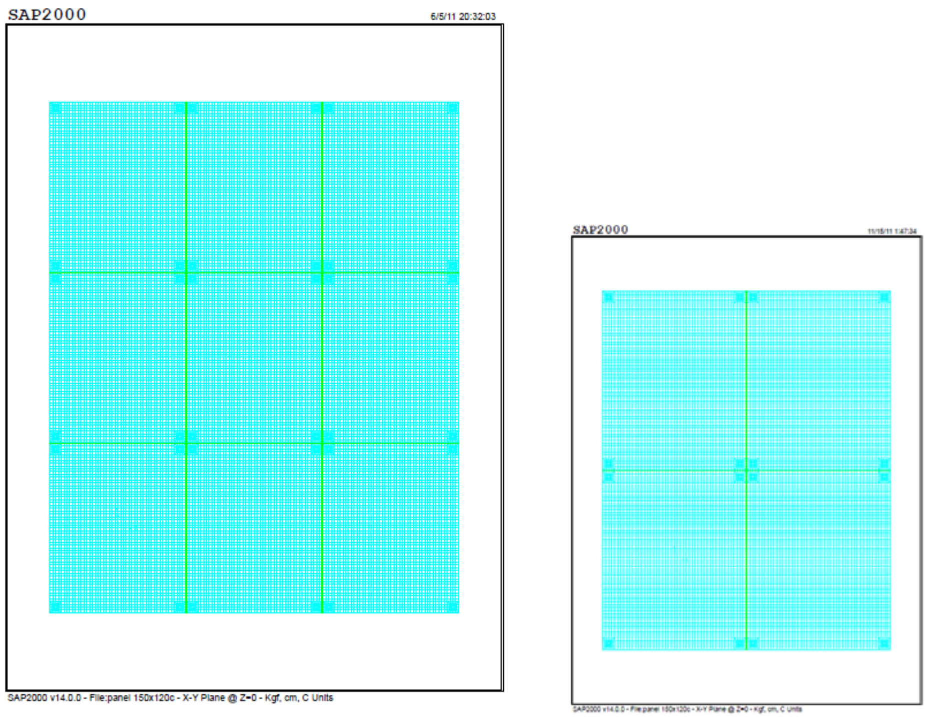


Figure 98. lateral displacement of the top left corner of the panel connected with FMR

It can be seen that after every cycle of loading the final position of the panel will have a very small disposition to the right direction that is because the imposed displacements on the supports at the beginning of the simulation where from left to right the same effect but in the opposite direction if the initial imposed displacements where from right to left. The reason of this disposition is that after every cycle of loading the friction connector will not return to the zero stress condition and as soon as the applied forces on the become less than the limit force of the connection the connection device will become like a rigid connection and at that point the remaining stresses and displacements will be kept in the system, these remaining stresses will also be present in the glass panes after the cyclic loading.

6.2.3. Group of connected panels

Having investigated the effects of lateral forces and displacements over a single glass panel, it is time to observe the effects of the lateral drifts over a group of adjacent glass panels. To achieve this, a matrix of glass panels is modeled in SAP, Figures (99-a) and (99-b).



(a) (b)

Figure 99. 2 sets of connected glass panels modeled in SAP

Neighboring edges of the glass panes is filled with silicon elastonomer material with a width equal to *1cm* and a depth equal to the thickness of the glass panes. For having the maximum possible consistency between the model and a real spider glazing system the nodes associated to the neighboring holes of the glass fixings, which are supposed to be connected to the same spider bracket, are constrained to undergo movements as a rigid body (meaning no relative displacements will happen between them). This will increase the overall reliability of the system, but decreases the accuracy of the results related to the silicon patches of the corners of the glass and between glass fixings. All the connections used in these simulations are FMR connections modeled Link elements discussed in the former section. Again the dynamic effect of the lateral displacements is imposed with a sinusoidal function with a period of *1 sec.,* the magnitude of the sine function for every support differs based on its elevation and is equal to the vertical distance between the support and the bottom of the model multiplied by *0.01*, which is the anticipated drift ratio.

The results of modeling a group of connected panels can be used for two cases below:

First, to investigate the cumulative effects that the neighboring panels might have on each other. Aside from the loads applied on the glass pane by the fixings, due to displacements in the structure, the stress state occurring in the glass panel is also related to the forces applied by other panels connected to it. This can be caused by the relative displacements or rotations that may occur between the connected glass panels. Figure (100) shows the Von Misses stress state of the glass panel at the peak of the horizontal displacement. Considerable effects of this type were not observed during the simulations of a set of panels and only a small increase to the stress values of the glass panes at the bottom of the set was detected. This effect needs to be further investigated in future research.

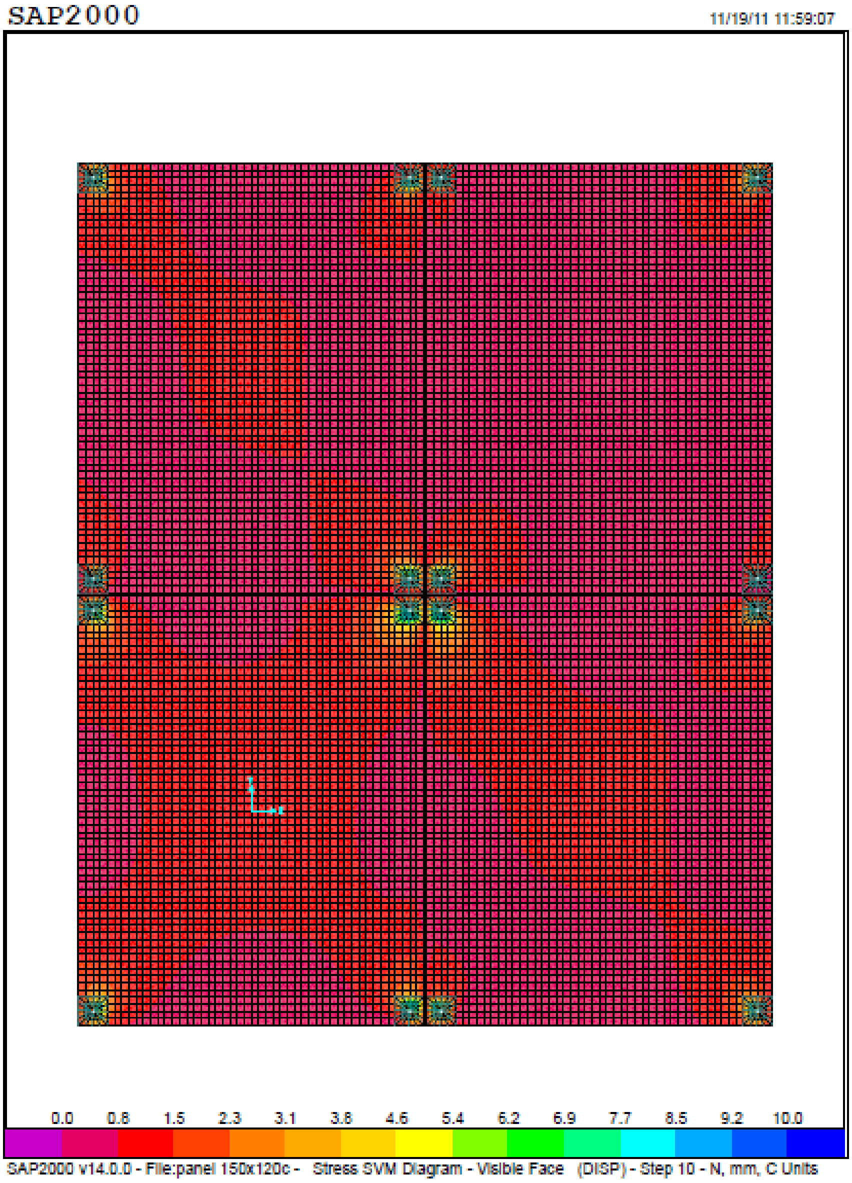


Figure 100. Von Misses stress states at the peak of horizontal displacements of the supports

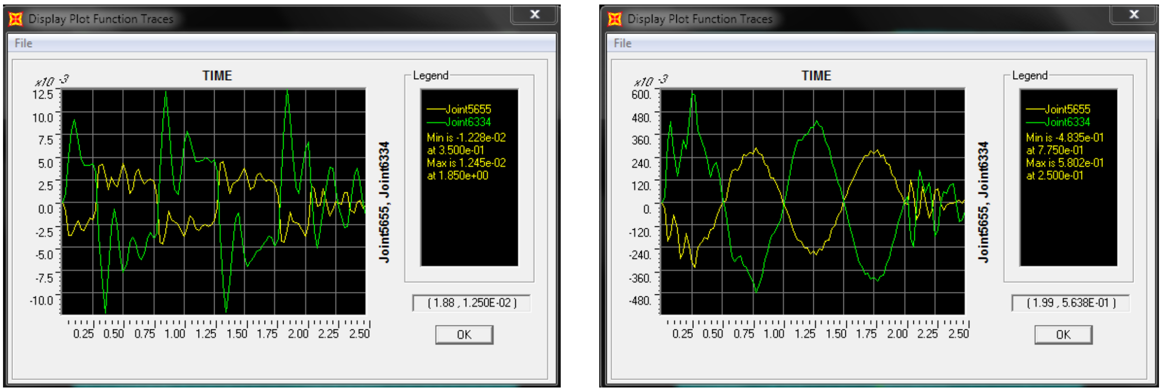
Second, to investigate the displacements and stresses in the silicon elastonomer, filling the spaces between the glass panels. With this approach it will be possible to obtain information on whether the silicon elastonomer will maintain its air-tightness and water-tightness properties after a seismic event. There are three points of interest for such observations in the system:

1- In the middle of two vertical edges

2- In the middle of two horizontal edges

3- In between the glass fixings

Figure (101) shows the absolute relative displacements between two sides of the silicon patch, each connected to a different glass panel.



(a) (b)

Figure 101. the absolute relative displacements between two sides of the silicon patch a) connected with FMR; b) connected with rigid connections

The safety criterion has been considered in order to derive the limit force values of the friction connector. It is clear that when the safety of the façade system has been assured, the serviceability of the system after earthquake needs to be further investigated. Air tightness and water tightness of the façade are the main features they compromised during an earthquake, which might be caused by breaking of the gaskets and failure in the structural silicon. Depending on the details of the façade system, maintaining the functionality of the façade may ask for further reduction in the limit force values of the friction connectors.

It must be noted that a great number of technical details are added to the system when the behavior of a set of connected panels (a mock-up) is under study, and it is not possible to include all these small details within a numerical simulation. So although the results of simulating a group of connected panels can be used for having a better understanding of the overall behavior of the system, they are not considered to be highly reliable and experimental studies are very much required for validating and evaluating this part of the numerical simulations.

7. Recommendations on experimental tests

Although experimental studies have not been performed within the scope of this research, in this chapter a brief discussion is presented on the experimental tests that are suggested for studying the behavior of the proposed connection devices and their effect on curtain wall systems during seismic events. The aim of these tests will be first to study the behavior of the connection device itself and later to see the effect that it will have on envelope systems subjected to seismic actions. In order to study the mechanical behavior of the connection device a laboratory test apparatus is discussed that is especially conceived for the study of cladding connections. And for the study of the effects of the connection device based on the recommendations of American Architectural Manufacturing Associations (AAMA)(Association American Architectural Manufacturers 2001), a test facility is proposed to perform two types of static and dynamic experiments on a mockup.

7.1. Behavior of the connection device

The theoretical behavior of the rotational friction connectors, presented in this research, was earlier discussed in section 5.3 of this research. In order to make sure the relations between the parameters of the system is realistically achieved it is necessary to have the connection devices subjected to lateral displacements, such that will happen when connecting the curtain wall to the building structure, and measure the parameters for limit force values and energy dissipation, accordance to different values for the applied pressure by the pressure bolts. The laboratory testing machines especially developed for connections in the cladding systems, in earlier works by Pinelli (Pinelli et al. 1996) is described here for running the tests on FMR. The primary objective behind the development of the test apparatus was the simulation of the behavior of an advanced cladding connector subjected to inter-story drifts, and to that end the machine must have:

- The ability to isolate and monitor the behavior of the connector elements.
- The ability to reproduce the actual service loads and deformations to which a connector is subjected during and earthquake

- The ability to accommodate different conditions of fixity for the connector ends.

In order to achieve these objectives, the machine must be capable of applying a number of specific types of loads to a connector. It is clear that the connector should be arranged in the machine in the same orientation that it would be used to support a vertical curtain wall. In this case the loads and constraints on the connection are:

- Horizontal forces or displacements

- Moment fixity at both ends

- Shear release in vertical direction

- Axial release (normal to the vertical façade plane)

- Gravity loads if required

The horizontal forces or displacements are on the connection device are provided with hydraulic actuators and the gravity load, if needed, is applied by putting weight on the connection. Figure (102) provides an schematic figure of the test apparatus and its fixture which is composed of the components below:

- A building anchor, which is a thick vertical steel plate attaches to a rigid box, the axial degree of freedom is releases with the help of roller bearings in axial direction under the rigid box

- The panel anchor, which is also consisted of a thick plate connected to a rigid support with long rods in order to prevent axial movements but allow movements in horizontal and vertical directions

- Horizontal load applying system, which is one or two hydraulic actuators attached to the edge of the panel anchor

- Hand operated spring actuators attached to the bottom edge of the panel anchor to apply gravity loads.

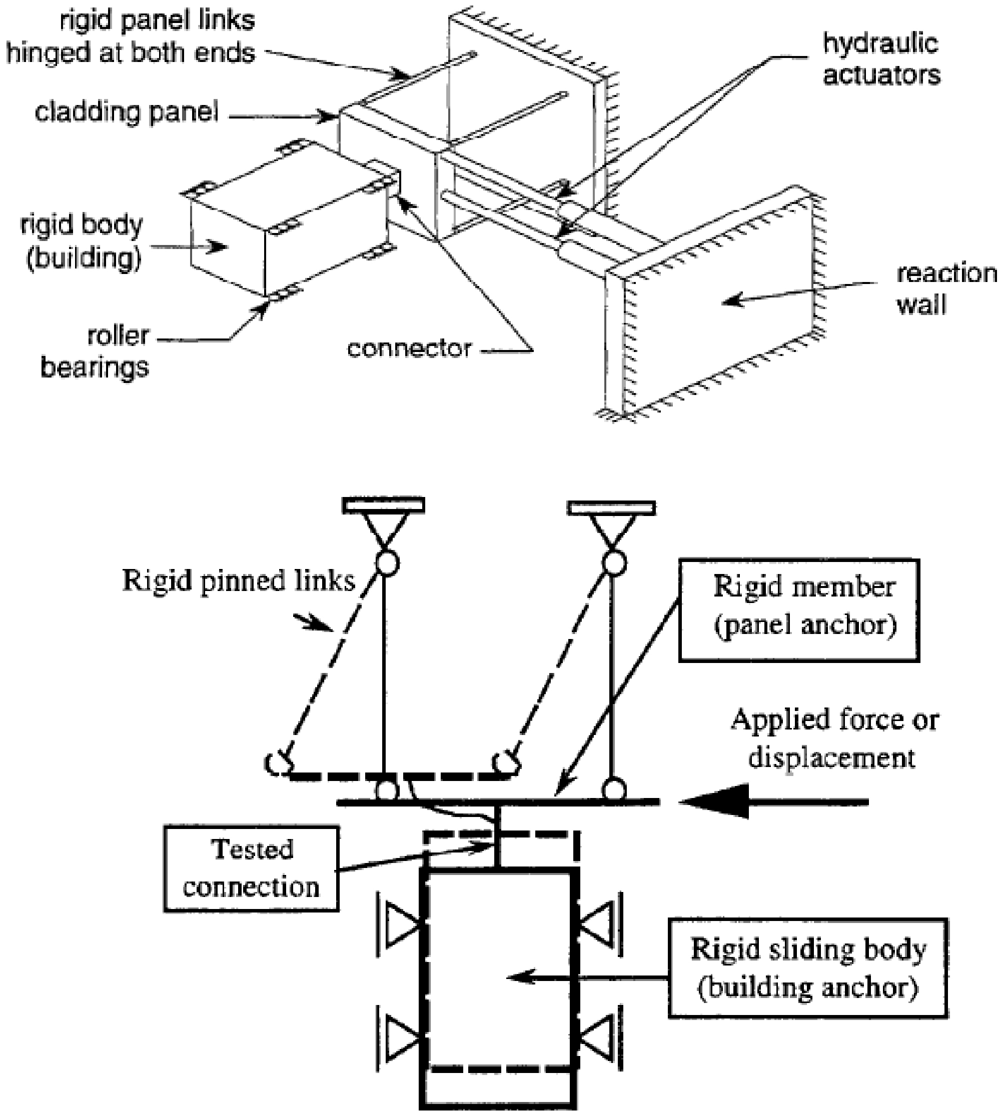


Figure 102. overall schematic figure of connection test fixture

More complete details of the test apparatus showing the kinematics of the test fixture are presented in Figures (103)

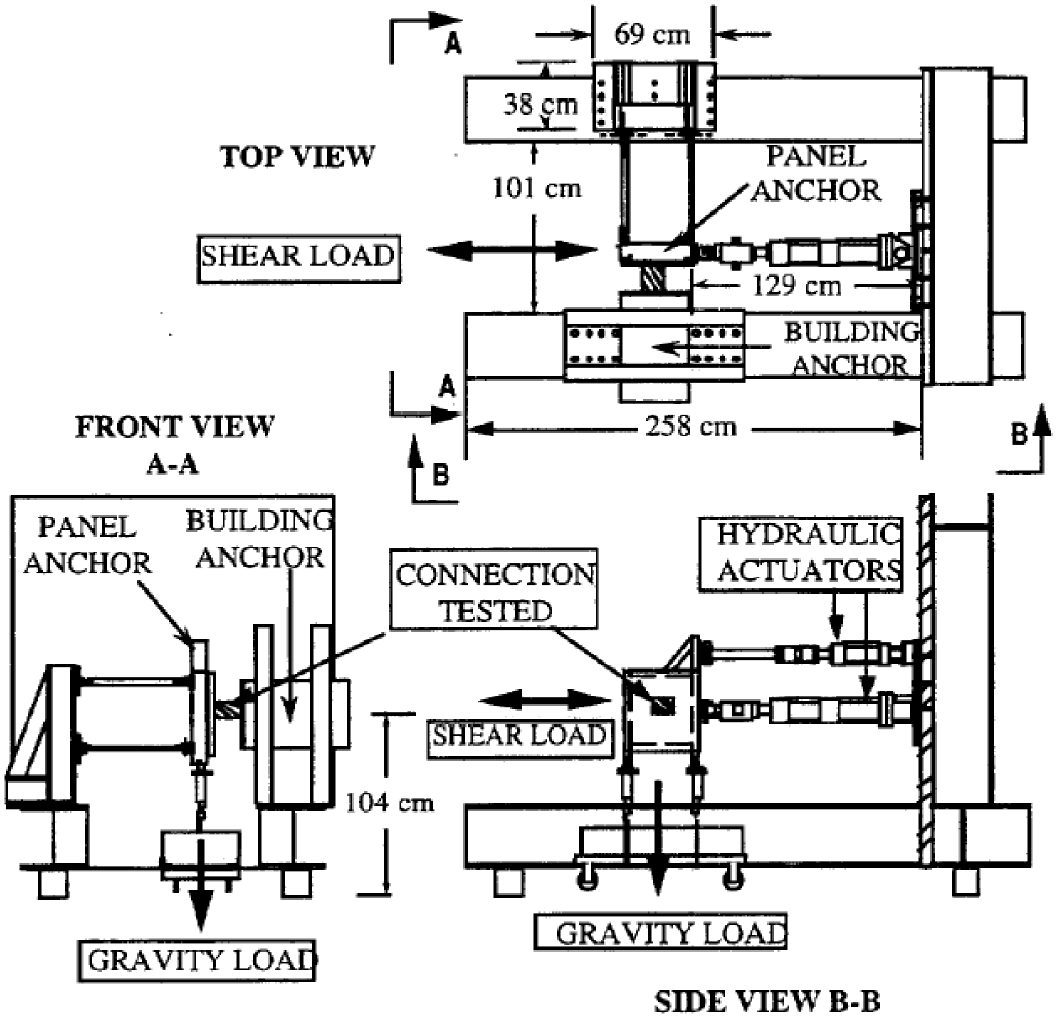


Figure 103. top, front and side view of the test fixture for claddign connection devices

The testing procedure is that cycles of increasing displacement are applied in small steps increment and at each step displacements and forces are recorded and corresponding hysteresis cycles – horizontal forces versus transverse displacements – are plotted. The objectives of the test may include the following:

- Evaluate stiffness, ductility and energy dissipation characteristics of the connection
- Evaluate the relations between pressure bolt forces and the behavior of the connection
- Evaluate the consistency of the behavior of the connection after multiple cycles of loading
- Investigate the effect of vertical loading on the connection

7.2. Test mockup

For evaluating the seismic behavior of a spider glazed curtain wall with connection device and subjected to seismic forces, and also to compare the results with the case that there is no friction connectors attached, a test facility is presented in this section which is based on the recommendation of AAMA documents 501.4 and 501.6 for evaluating curtain wall and storefront systems subjected to seismic and wind induced inter story drifts. To separate test procedures are considered to be performed on the system, a dynamic racking test for determining the ultimate seismic state of architectural glass and a statically applied load test with primary focus on serviceability of curtain wall system specimens. Same test facility is proposed for both test procedures which is composed of a spider curtain wall system with full size specimens and components using the same materials, type of glass, details, method of construction and anchorage as those used in actual building, supported by a test chamber structure that simulate the main structural supports of the actual building. However the test chamber support structure may differ from the actual building as it is required to impose the required displacements. The test procedures will differ in static and dynamic cases these procedures are directly quoted from AAMA 501.4 for static and AAMA 501.6 for dynamic tests(Association American Architectural Manufacturers 2001, Association 2000).

The main part of the test chamber structure for simulating the seismic behavior of the actual building is composed of a primary frame with beams and columns hinged for providing required lateral drift. Secondary beams are attached to the main frame of the test apparatus providing supports for connection elements and a hydraulic actuator is attached to the top corner of the main frame for controlling the lateral displacements required for the test. A full size spider glazed curtain wall is attached to the secondary beams of the test chamber. The test will be repeated in both cases of adapting the FMR connection elements and without them providing enough observations to categorize the advantages of adapting FMR connections in the system.

The composing elements of the test apparatus are:

- External framing system, composed of steel H members connected at the ends with hinged connections.
- Secondary beam elements, connected to the vertical columns of the main framing system

again with hinged connections and providing supports for the connections of the curtain wall system.

- Hydraulic actuator, connected to the top beam of the main frame and providing the desired forces and displacements.

- The curtain wall mockup, containing all the elements of a full sized spider glazing system and constructed exactly as those used in actual building

- FMR connection element, that are placed between the spider brackets of the curtain wall system and the secondary beams of the test chamber

Figure (104) demonstrates a schematic shape of the test apparatus and the connections between the curtain wall mockup and the test chamber structure

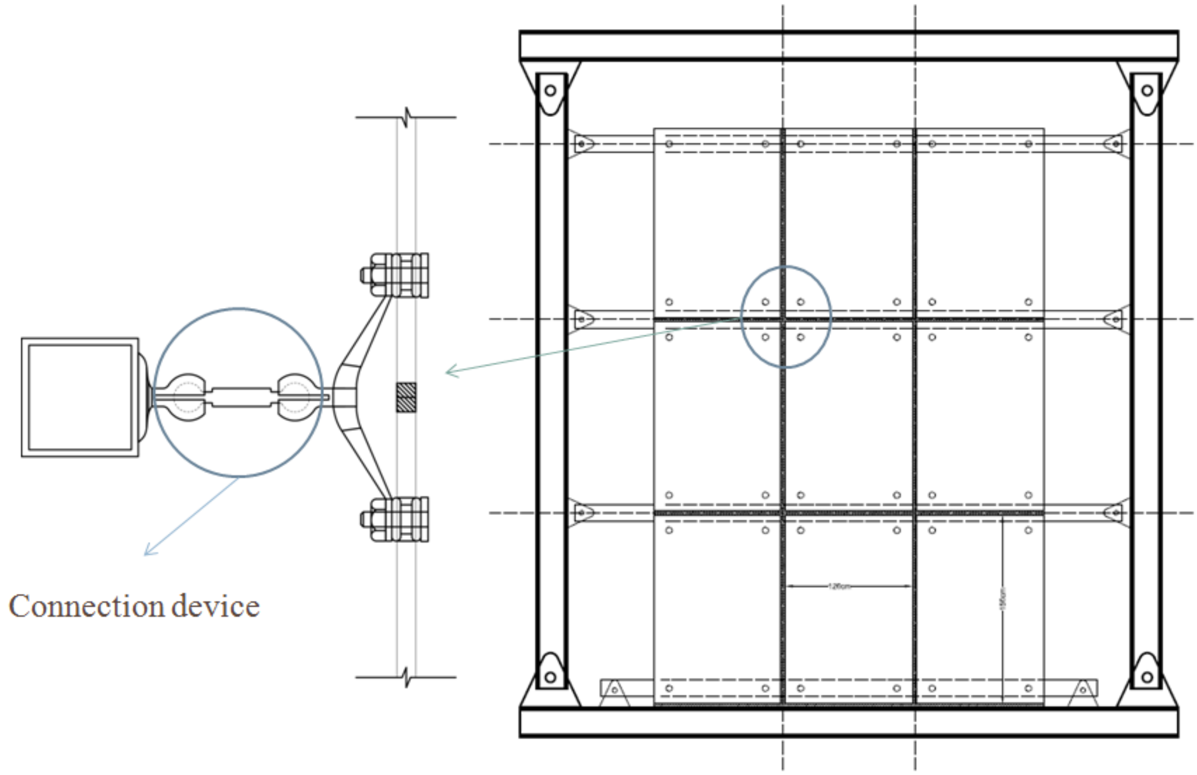


Figure 104. test frame and curtain wall mockup

Performing the dynamic and statics tests, in two cases of with and without the connection devices will provide quantify, and information on, the advantages of adapting the FMR connection devices on the both safety criterion – preventing mechanical failure in the glass panes – and serviceability criterion – maintaining the air tightness and water tightness functionality of the silicon patches – in-between the connection elements of the spider glazing systems.

8. Summary and conclusions

The goals of this research has been set to first define and propose connection systems which result in a compatible mechanical behavior between the building structure and its envelope during earthquake, and second to provide reliable instructions for tuning and adjusting the proposed connection devices. So that when adopted within the curtain wall system they will perform in the expected manner.

Study has been done on different mechanism and approaches that are already used in the design of advanced connectors and other energy dissipating techniques which are incorporated in the structural design of the building, and among these methods friction connection devices have been selected as a proper technical approach to be used for an energy dissipating and isolating connection. This decision has been made based on:

1. Highly predictable behavior
2. Rigid behavior of the connection prior to slippage
3. Ability to control the forces transferred through the connection
4. Simplicity of the manufacturing and installation

Among all the advantages of a friction connection device mechanism, the most important one is to sharply confine the forces that are transferred through the connection device.

Design of a connection device:

Among different types of curtain wall glazing systems studied in this research, a great deal of attraction was given to structural glazing systems. The reason to that is the high vulnerability of these systems against seismic actions, and also the lack of proper provisions to protect these systems against seismic effects. For that a special friction connector device was proposed specifically for these systems to satisfy the main design objectives described below:

- Applicability to the implemented in Complex geometrical situations
- Satisfy a higher aesthetical demand than the usual advanced connectors in practice.
- Ability to control the moments applied on the curtain wall system to avoid out of plane deformations

To satisfy the objectives above, instead of using the transversal forces, that one caused by the displacements in the main structure to trigger the frictional behavior, the resulting moments at the two ends of the Connection device are used to trigger the frictional behavior and based on this concept the device was named the Friction Moment Rod (FMR). Based on the concepts of applying rotational friction in connection devices, resulting in design of the FMR, another rotational friction device was also designed to be used in other types of curtain wall systems such as stick or unitized systems. This connection device is designed to easily replace the existing isolating connections currently used in these systems.

After having designed the connection device, it is necessary to obtain the mechanical characteristics and adjust the behavior of the connection in a desired way. This was done by investigating the mechanical behavior of different curtain wall systems during earthquake

Studying the behavior of glass panels during earthquakes:

The source of seismic loads that act upon the components of a structure is the acceleration nature of a seismic event, which multiplied by the mess of the structure result in force between the composing elements. but unlike structural members the source of the forces applied on light envelope systems, due to their comparably low mass, is not the accelerating nature of a seismic event, and the displacement which happen within a structure are the cause of damaging forces. That is why using connection devices that result in compatible behavior between the two systems is of great importance. at this point the mechanical behavior of different curtain wall systems was investigated based on drifts which will happen in a structure. Considering the significantly small dimensions of glass thickness compared to its height and width, and the in-plane nature of the applied forced, the buckling of glass pane was set to be analyzed as the most portable mode of failure in glass components during an earthquake.

The theory of buckling of thin plates subjected to distributed shear force was used for analyzing unitized systems and the result of the analysis implied that; considering the amounts of shear at the edges of the glass plate the buckling failure in these systems will be proceeded by failure in silicon patches and frame distortions. The high values of the critical shear forces in this case were associated to the edge supports of the glass preventing buckling. This type of lateral support is not present in the cases of structural glazing.

For the case of structural glazing systems, due to unavailability of a closed form theoretical solution to the problem at hand, two sets of simplifying assumptions where used for obtaining results. The absence of theoretical solutions that completely match the circumstances of the problem, are first because of existence of point loads in the model, and second due to the free edge boundary conditions of the problem.

The first simplified model was based on an analogy between the system at hand with a plate subjected to shear forces, and the second with a uniaxialy loaded plate. Both cases were considered to be having free lateral boundary conditions.

It is necessary to indicate that in comparison with the exact solution to the actual problem, both models were suggested in a way to give upper hand solutions to the problem so that the results can be reliable for the design of the connection device. The results of both simplified models have shown to be consistent with each other.

Numerical simulations:

In order to verify and evaluate the accuracy of the simplified models produced, a set of numerical analysis have also been performed in this research. At first the two softwares of SAP2000 and SJ-MEPLA were used to perform a static numerical simulation of the problem and a buckling analysis. Since it was not possible to realistically model sandwich shell elements in SAP2000 the recommended corrections of the European standards prEN 13474-3 (European Standards 2009) for thickness of laminated glass components, were used for glass pane inputs to the software. On the other hand being a commercial Finite Element software specifically developed for structural analysis of glass, the SJ-MEPLA had the ability to completely simulate all the details of a structural glazing system including laminated properties and fixing details.

The results of the two numerical simulations have showed very good correspondence for thinner glass elements with one interlayer of PVB (0.38 mm). But for thicker values of glass, with a double interlayer equal to 0.76mm, the result of SJ-MEPAL for critical buckling forces where between 10 to 25 percent less than the ones of SAP2000. This would suggest a further reduction required in the bending stiffness of laminated glass panes with more than one PVB interlayer. Comparing the results of the analytical solutions with the numerical solutions had shown that the results of the analytical solutions can be used as reference loads to determine the limit force value for the design of the Friction Moment Rod with an average safety factor equal to 1.5.

After having determined the limit force values (tuning forces) for the connection device, non-linear time history analysis where performed on a single glass panel connected to supports with friction damping connectors. This type of numerical simulation was possible only with SAP2000 software. In order to demonstrate the effect of lateral drifts on the glass panel a sinusoidal displacement was imposed on the support of the glass panel and link elements with elastic-perfectly-plastic behavior (representing the FMR devices), where located between the glass and the supports. Comparison between the maximum values of the stress fields in the dynamic simulation with near buckling stress states, made clear that adopting friction connection devices with the adjustments based on analytical models, will result in protection of the glass panels against lateral displacement within the structure.

Finally in order to investigate the effects that a set of adjacent structural glazing panels may have on each other, a group of panels connected with silicon elastonomer material where modeled in SAP2000 and again subjected to support displacements based on a function uniformly increasing with the elevation of the supports. In this case only minor and somehow negligible increase in the stress state of the lower glass panels was witnessed.

Further suggestions for research

- Select materials instead of lining pads more often used in building sector
- Performing numerical simulations in ANSYS or ABAQUS to have both the advantages of SAP and MEPLA at the same time
- Investigating the behavior of silicon patches between glass panes in the dynamic model (for serviceability criterion) from models with a group of panels
- More thorough investigation on the cumulative behavior of the glass panels

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