POLITECNICO DI MILANO

VI Facoltà – Ingegneria Edile-Architettura

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Tesi di laurea magistrale

CURTAIN WALL DESIGN:

THE SEISMIC BEHAVIOUR OF GLAZING FAÇADES

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Sommario

Abstract

The present work is mainly the result of a an internship period in Belgium at Reynaers Aluminium, a company specialized, among the rest, in design, production and manufacturing of curtain walls. Thanks to its test institute facility and research&development offices, this study could have been carried out.

The aim of this study was to investigate the curtain walling façade behaviour when subjected to a seismic event, in the interest to find the best design solutions to face this problem.

The first part includes an introduction about curtain wall façades and seismic phenomena: great importance is given to the features to be considered in the evaluation of the seismic behaviour of "non-structural" elements in general and to glazed façades in particular. The worst load induced by an earthquake to a curtain wall is the "interstorey drift" (i.e. the relative displacement between two stories), if compared to forces and accelerations resulting from the same seismic event.

After this introduction the state of the art has been investigated through three different points of view: a theoretical one, an experimental one and a normative one. In particular great relevance is given to the comparison between European test standards and American ones, the most used worldwide, underlining how Europe increasing interest in this matter is expressed in the upcoming effectiveness of prEN13830.

The following section describes the experimental performance mock-up tests carried on by Reynaers Aluminium with my contribute. Their most used stick systems have been tested to evaluate their drift capacity and compare it to the theoretical estimated ones. The tests results show the ability of the system to accommodate interstorey drifts without engaging the glass in a way that will produce great damage like a fallout.

Finally the common approach in new tall buildings is presented through the case study of Isozaki Tower at CityLife, pointing out the preference to assign the seismic responsibility to the main structure without excessively involve the façade. In the end other alternative approaches are collected to encouraging the research in a field not yet well examined in depth.

Il presente elaborato è in larga parte risultato del periodo di tirocinio da me svolto a Duffel, in Belgio, presso Reynaers Aluminium, un'azienda specializzata nella progettazione e produzione facciate continue. Questo studio è stato elaborato grazie alle attrezzature del loro Test Institute e all'ufficio di ricerca e sviluppo. L'obiettivo primario è quello di studiare approfonditamente il comportamento al sisma delle facciate continue con lo scopo di trovare un modo più adatto di progettarle.

La scelta di questo argomento come tema della tesi di laurea magistrale risiede nel mio apprezzamento per le grandi superfici vetrate e nella loro scarsa diffusione in un territorio a forte rischio sismico come Messina, la mia città natale. Un altro scopo dello studio è infatti quello di dimostrare che realizzare facciate continue in territorio sismico è possibile, vincendo così il pregiudizio diffuso che le vede come un oggetto estremamente fragile e pericoloso. Nonostante sia molto vasta la conoscenza del comportamento sismico delle strutture durante un terremoto, storicamente, in effetti, sono state sottovalutate le conseguenze degli effetti di un sisma sugli elementi non strutturali in genere, ma negli ultimi anni la sensibilità verso il problema è aumentata, fornendo diverse indicazioni e soluzioni.

La prima parte di questo lavoro introduce I vari tipi di facciata continua e I fenomeni tellurici: particolare importanza è data al comportamento della non struttura e alle vetrate in particolare, evidenziando come il principale responsabile del danno sia lo spostamento "drift" di interpiano.

Dopo questa introduzione si è fatto il punto sullo stato dell'arte attraverso tre diversi punti di vista: uno matematico-teorico, uno sperimentale e uno legislativo. In particolare grande rilievo è dato al confronto tra la normativa in materia di prove prestazionali europea e quella americana, la più utilizzata, mostrando come il crescente interesse dell'Europa per questo argomento sia espressa nella prEN13830, che diverrà effettiva nel 2015.

La sezione seguente descrive I test sperimentali su mock-up condotti da Reynaers Aluminium con il mio contributo. I loro profili del sistema montanti e traversi più diffusi sono stati testati a spostamento laterale indotto e gli spostamenti sopportati sono stati registrati e confrontati con quelli teorici stimati. I risultati mostrano le ottime capacità del sistema di ospitare gli spostamenti di interpiano senza coinvolgere il vetro in modalità di rottura altamente dannose o che potrebbero portare al crollo.

Infine l'approccio comune nei nuovi edifici alti è rappresentato dal case study della Torre Isozaki a Citylife, segnalando la preferenza di assegnare tutta la responsabilità sismica alla struttura principale cercando di coinvolgere al minimo la facciata. Infine è stata fatta una carrellata di approcci alternativi al problema, con lo scopo di stimolare la ricerca in un settore ancora non abbastanza approfondito.

A tutti coloro
che hanno subito le conseguenze
di una catastrofe naturale

Section 1: Overview of the problem

1 Introduction

Earthquakes are the most powerful natural events on earth able to release more energy than thousands of atomic bombs in a few seconds. The effects can be severe damage and high loss of life through a series of destructive agents, the principal of which is the violent movement of the soil resulting in laying stress of building structures (buildings, bridges, etc..), often accompanied by other effects such as flooding (damburst), tsunami, subsidence of the ground (landslides, landslides or liquefaction), fires or spills of hazardous materials.

When a strong earthquake occurs, it can change one place's history forever and indelibly mark lives of people who experience this event. The consequences can be perceived for years or centuries, as it still happens in Messina, totally destroyed by the deadliest earthquake in Europe[1], back in 1908 and which greatest legacy is fear.

Figure 1-1: Messina after the earthquake of 1908, December 28. Magnitude 7.1, Mercalli XI

[1] It caused 123.000 dead people. (The world's worst natural disasters Calamities of the 20th and 21st centuries, CBC News. Retrieved October 29, 2010).

Tremors are not rare events, they are frequent in seismic areas, but traditionally this has not received proper attention in the common way of building. Just as a result of recent earthquakes all over the world (Sumatra, Japan, Haiti, L'Aquila), interest in the design of buildings to resist seismic loads and displacements has increased.

Figure 1-2: L'Aquila after the earthquake of 2009, April 6. Magnitude 5.8, Mercalli IX

Obviously, the main purpose is to prevent damage to people, but a reflection must be done also to the enormous financial costs for repairing and reconstructing damaged and destroyed buildings or restarting a business activity.

Although the interest and awareness in seismic structure is great, what is still underrated is the big and real danger caused by the failure of non-structural elements, such as façades, ceilings and equipment present inside or outside the building.

In particular, this master thesis studies the seismic behavior of curtain walls, increasingly common façade typology in buildings, which seismic dangerousness is basically connected to broken glass falling hazard. For this reason it is necessary to carefully design these glazed façades in order to make possible appropriate prevention measures so that they can remain functional and safe and allow, if necessary, an eventual postponed substitution.

Figure 1-3: 2010 Chile earthquake, Magnitude 8.8, Mercalli VIII

Figure 1-4: 2011 Christchurch (NZ) earthquake, Magnitude 6.3, Mercalli IX

The present work wants to give a global overview of this topic both from the theoretical and the application points of view and t is divided into four parts.

Section 1 is a survey about the two keywords of the study: curtain walls and earthquakes. It presents what a curtain wall is and the different typologies and synthetically explains how earthquakes work, can be measured and how to prevent their damages.

Section 2 makes the point on the state of the art by showing some theoretical mathematical models developed in the last decades and two experimental studies that gave the basis for the current approach to the matter. In the end international standards and guidelines about laboratory testing of curtain walls are widely analysed, then sum up in a comparison between American and European approach.

Section 3 is the central work, corresponding to the stage experience in the test centre of Reynaers Aluminium in Duffel, Belgium. Here static tests on two different profile curtain walls were carried on, with the aim of studying their behaviour and to compare it to the theoretical results. Despite some initial difficulties the tests have revealed successful.

Section 4 is a collection of solutions to the problem: a case study about Isozaki Tower at CityLife is presented at first, as a representative approach in current new buildings; in the end alternative approaches generally concerning the connection to the structure are analysed.

2 About curtain walls

A building envelope is the physical separators between the conditioned and unconditioned environment of a building, it includes all of the elements of the outer shell that contribute to create a security, weather and thermal barrier.

The term “curtain wall” particularly, is much more specific and indicates a type of outer covering of a building in which the outer walls are non-structural, i.e. they don’t carry any dead load weight from the building other than their own dead load weight and they are directly hung to the structural system, for the most to the beams or to the floors. The curtain wall transfers horizontal wind loads that are incident upon it to the main building structure through connections at floors or columns. Curtain wall systems are typically designed with extruded aluminium frames which are typically infilled with glass, which provides an architecturally pleasing building, as well as benefits such as daylighting.

2.1 Curtain walling systems

There is a great variety of technologies for realize a curtain wall, but they can be summarized as follows:

- Stick system
- Unitized and panelised system
- Structurally sealed system
- Structural glazing system

Stick system

Horizontal and vertical framing members (sticks) are normally extruded aluminium profiles, protected by anodizing or powder coating. Members are cut and machined in the factory prior their on-site assembly as a kit of parts: vertical mullions, which are fixed to the floor slab, are firstly erected, followed then by horizontal transoms, which are fixed in-between mullions, finally glass infilled.

Unitized and panelised system

Unitized systems consist of storey-height units of steel or aluminium framework, glazing and panels pre-assembled during factory fabrication. These completed units are hung on the building structure to form the building enclosure. Unitised systems are faster to install and have a superior quality control but having higher direct costs they are less common than stick systems.

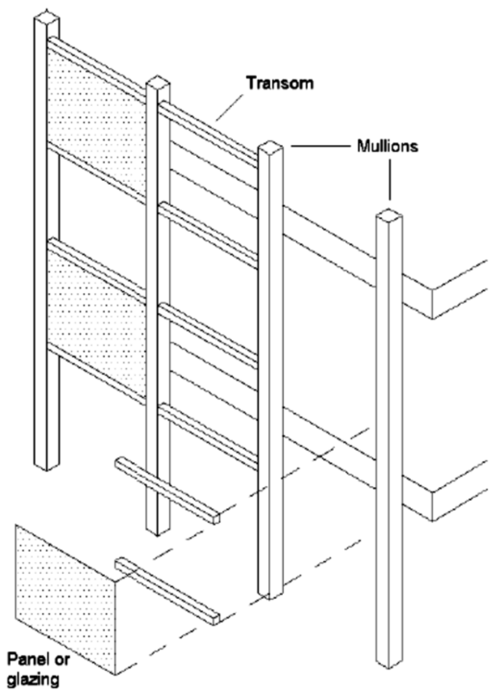

Figure 2-1: stick system

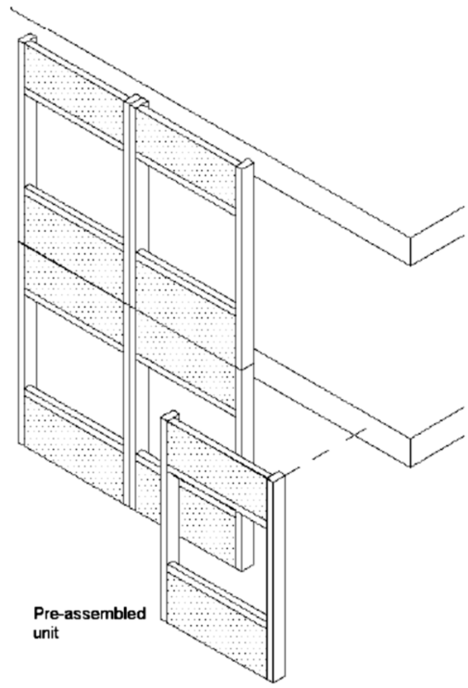

Figure 2-2: unitized system

Structurally sealed system

Structural sealant glazing is a type of glazing that can be applied to stick, unitized and panelised systems. Instead of mechanical means (i.e. a pressure plate or structural gasket), the glass panels are attached with a structural sealant (usually silicon) to metal carrier units that are then bolted into the framing grid on site. External joints are weather-sealed with a wet-applied sealant or a gasket.

Structural glazing system

Toughened glasses are assembled with special bolts and brackets and supported by a secondary structure to create a transparent facade with a continuous external surface.

The joints between adjacent panes/glass units are weather sealed on site with wet-applied sealant.

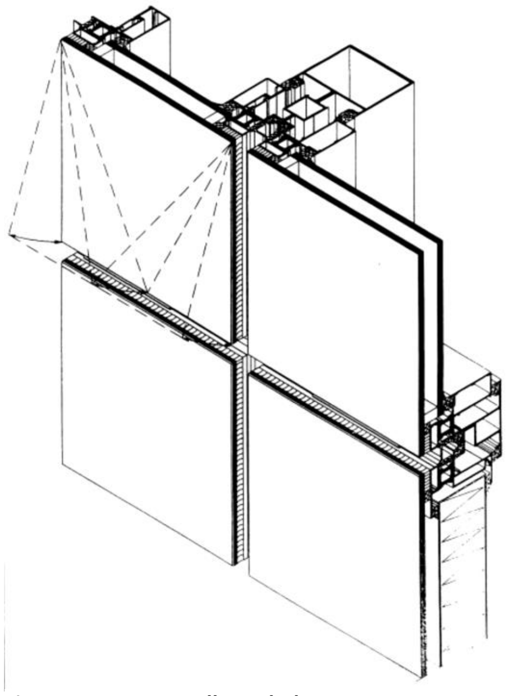

Figure 2-3: structurally sealed system

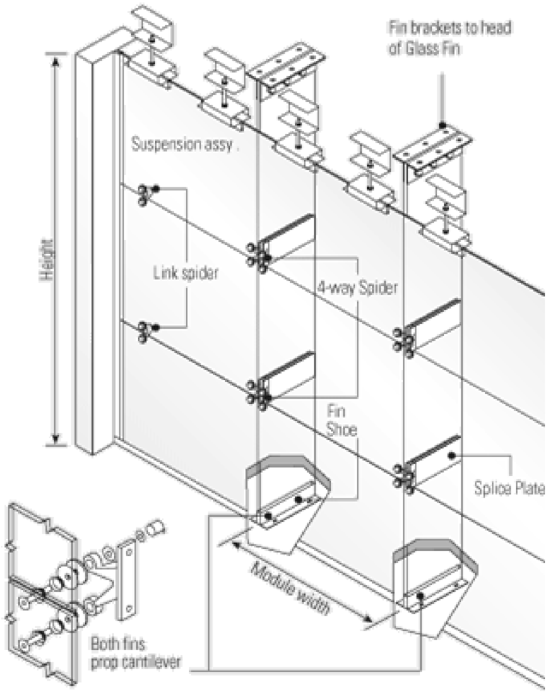

Figure 2-4: structurally glazing system

2.2 Curtain walling components

Fastening System

The unitized and panelised system is constituted of different modular units which have to be attached to the structure, usually to the concrete floor slab or to structural elements such as beams. There are many different ways to fasten the façade unit to the building structure, in order to obtain horizontal tolerance, vertical tolerance and loadbearing capacity, against different types of loads, vertical and/or horizontal.

Brackets represent the fixing system both for the facades to the main structure and façades components between each other. They usually are made of aluminium or steel. They must be designed to absorb vertical and horizontal tolerances of façade installation and the displacements of the building during its life. They must be designed and verified for the dead load coming from the self-weight and the wind load produced by the wind pressure on the by using the limit state method.

Aluminium frame

The façade is constituted by different profiles, usually made of aluminium, that build the structure that support all surfaces loads and that actually resists to the wind pressure acting on the façade. The aluminium frame unload the horizontal forces to the fasten system, already calculated and verified to resist to it.

The vertical profiles (mullions) are the most stressed elements of the frame, they cover the height of a storey and they are the longest profiles. The horizontal elements(transoms) pick up a part of the wind load collected by the glass, unloading it to the mullions, even if they have mostly to support the glass and to stiffen the whole facade unit.

Glass

The main element of the façade, either for its dimensions and its weight is the glass plate, fixed and sustained by the aluminium frame of the unit. Because of its huge dimensions it picks up high values of wind load but its behaviour under the acting loads is mainly influenced by the constraint system. Among many different typologies of curtain walls, some of them are characterized by the glass-to-frame restraint system. This can be mechanical, constituted by an outer element called “pressure plate” pressing all along the edge of the glass against the inner profile or it can be constituted by a structural silicon joint that retains the glass all along its edge, while the weight is supported by two elements under the glass plate, to reduce the sealant joint size, called “setting blocks”. Different types of glass can be used (i.e. annealed, heat-strengthened, fully tempered, laminated…) and dependently upon the different typology, influenced by the way it has been produced and manufactured, the consequence of glass failure could really vary.

3 About earthquake

3.1 What is it?

An earthquake is the result of a sudden release of energy in the Earth's crust that creates seismic waves. At the Earth's surface, earthquakes manifest themselves by shaking and sometimes displacing the ground. In its most generic sense, an earthquake can be a natural phenomenon or even an event caused by humans (mine blasts...).

Natural earthquakes usually occur along the boundaries of the tectonic plates which are induced to move reciprocally by convective movements inside the mantle layer. These plates, which are to be considered rigid, concentrate and store up all the energy inside their boundary contact region until a state limit is reached, and all the energy stored up is released. So energy propagates in a radial concentric way to the original breaking point that goes by the name of "hypocentre", which is the origin of an earthquake. The point on the surface, corresponding to hypocentre is called "epicentre".

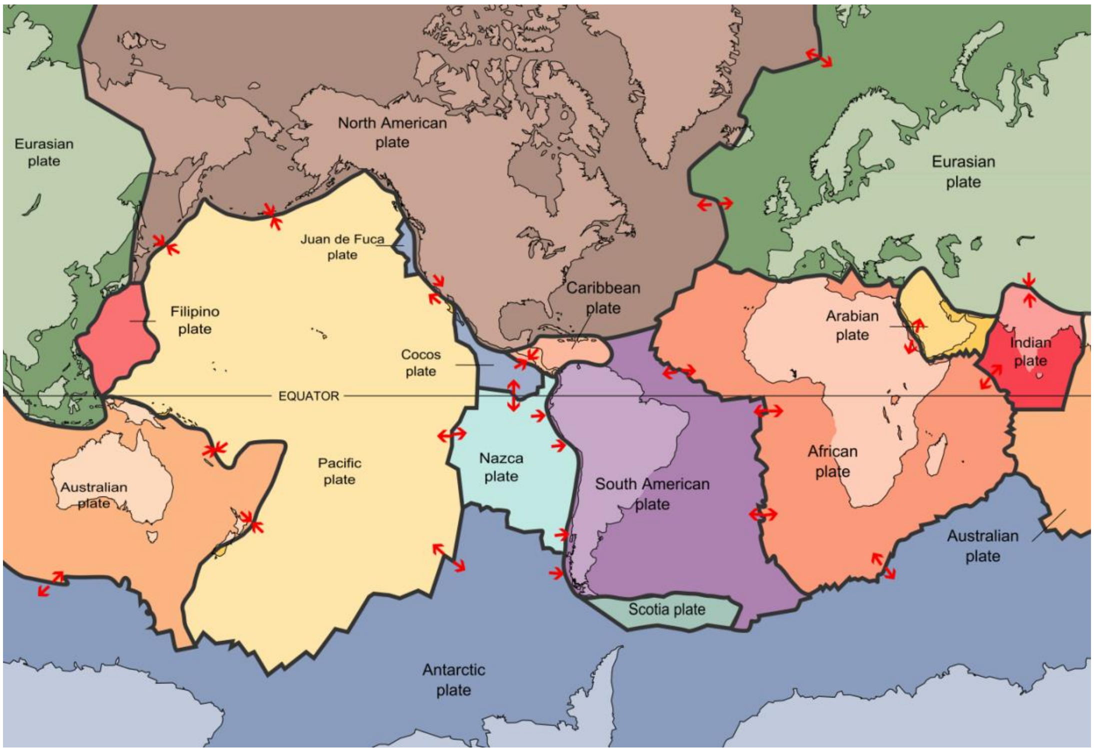

Figure 3-1: map of tectonic plates

Seismic waves generated by the energy released during an earthquake travel through the earth's layers from the hypocentre in every direction. It is possible to make a broad distinction between body waves and surface waves.

Body waves travel through the interior of the Earth. They create ray paths refracted by the varying density and stiffness of the Earth's interior. There are two types of body waves: primary waves and secondary waves. P-waves are compressional waves that are longitudinal in nature. P-waves are pressure waves that travel faster than other waves through the earth to arrive at seismograph stations first. These waves can travel through any type of material, including fluids. Typical speeds in solid rock are about 5-6 km/s. S-waves are shear waves that are transverse in nature and displace the ground perpendicular to the direction of propagation. S-waves can travel only through solids, as fluids do not support shear stresses. S-waves are slower than P-waves, and speeds are typically around 60% of that of P-waves in any given material.

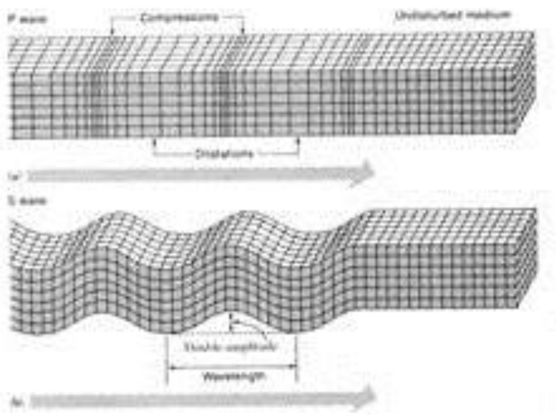

Figure 3-3: body waves are P-wave and S-wave

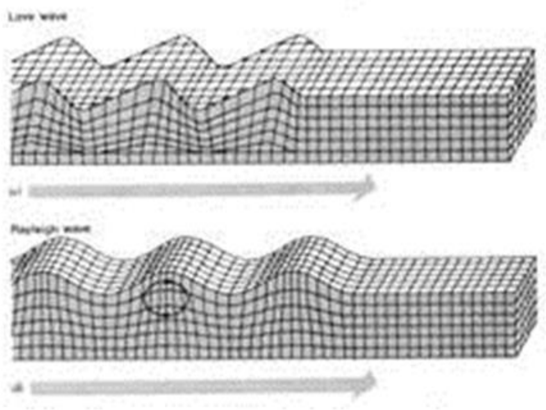

Figure 3-2: surface waves are L-wave and R-wave

Surface waves travel along the Earth's surface. Their velocity is lower than those of seismic body waves. Because of the long duration and large amplitude of the surface waves, they can be the most destructive type of seismic wave. The most important ones are Rayleigh waves and Love waves. R-waves, are surface waves that travel as ripples with motions that are similar to those of waves on the surface of water. L-waves are horizontally polarized shear waves. They usually travel as fast as Rayleigh waves, about 90% of the S wave velocity, and have the largest amplitude.

In the case of local or nearby earthquakes, the difference in the arrival times of the P, S and surface waves can be used to determine the distance to the epicentre.

3.2 How can it be measured?

There are two scales for measuring earthquake severity: intensity and magnitude.

Intensity scale is the historical one, it is based on the observation of damage of an earthquake (humans, objects of nature, and man-made structures) at a particular place and it classifies the degree of shaking on a descriptive scale from MM I (weak) to MM XII (catastrophic).

The magnitude of an earthquake is a measure of its size and relates to the amount of energy released, usually by rupture of the fault. Magnitude is based on the Richter scale. Every time the magnitude increases by one it represents a thirty-twofold increase in the size of the earthquake. By measuring magnitude through accelerometers in different stations it is possible to localize the epicentre.

For earthquake engineering the most important input parameter is the peak ground acceleration (PGA) that measure the earthquake acceleration on the ground. Unlike the Richter and moment magnitude scales, it is not a measure of the total energy (magnitude, or size) of an earthquake, but rather of how hard the earth shakes in a given geographic area (the intensity).

The peak horizontal acceleration (PHA) is the most commonly used type of ground acceleration in engineering applications, and is used to set building codes and design hazard risks. In an earthquake, damage to buildings and infrastructure is related more closely to ground motion, rather than the magnitude of the earthquake. For moderate earthquakes, PGA is the best determinate of damage; in severe earthquakes, damage is more often correlated with peak ground velocity or displacement.

3.3 Seismic design

The structural system, with all other non-structural systems, has its own vibration way that is essentially defined by the fundamental period of the building. Through this parameter it is possible to describe how the structure replies to excitations, like seismic activity or wind pressure.

At the base of a good seismic design and construction there's the concept of the building as a whole, considering structure, non-structure, plants and special furniture too, so that it is possible to foresee the failure mode. This should allow to dissipate a lot of energy before getting to collapse. The main goals of seismic design are:

- Protection of human lives
- Limitation of damages to constructions and whatever is inside
- Full functionality guaranteed to buildings with special functions (hospitals, bridges, nuclear power plants, museums…)

A seismic event causes the building structure to undergo various displacements producing relative interfloor deflection and interfloor story drift. The most important thing to do is to avoid the structural collapse and destruction of a building, which can be realized in several ways, either isolating the structure from the very beginning, or letting energy to come in and predisposing appropriate devices to dissipate it without damaging the structure, or to make the structure active and selecting the structural elements failure sequence according to the hierarchy of resistance.

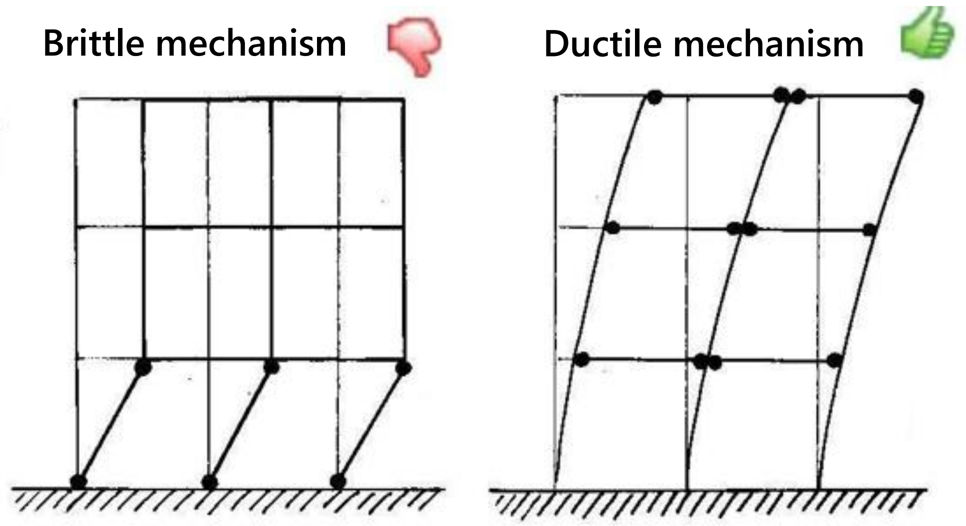

Figure 3-4: different approach of structures to an earthquake

A non-structural element, by its nature, is not necessary for the building to resist and not to collapse because, if it fails or not, it does not really affect loadbearing capacity of the structural system. On the other hand, when a seismic event happens one of the real issue is the danger caused by the failure of non-structural elements, such as masonry, ceilings, cladding façades, curtain walls and all the equipment present inside or outside the building. So, even when a building is well designed from the structural point of view, the non-structural elements issue has to be carefully considered. In case of normal and ordinary earthquake magnitude (not extraordinary events) non-structural elements must remain functional and safe and allow, if necessary, an eventual postponed substitution.

Glass has considerable in-plane strength and out-of plane flexibility, by the way it will be the most influenced and at risk component of the façade, just because of its frailty behaviour. Earthquake forces cause the structure to drift, and in a typical curtain wall the aluminium framing, which is rigidly attached to the structure, tends to follow easily the stories relative displacements trough either moving itself or elastically deforming its shape. On the other hand glass behaves like a rigid element only moving and without deforming and corners of the glass may impact the metal frame. This could cause a frail break and, in the worst case, also the completely fallout of the glass from the frame.

It is necessary to evaluate carefully a system or a non-structural element behaviour to understand the failure and collapse process that predominates, so that it would be possible to take appropriate prevention measures and to intervene during its design. Therefore, depending on the non-structural system and its characteristics, it will be necessary to evaluate which is (or are in the case there were more than one) the worst loading condition and proceed to verify it.

Usually the wind action is the predominant load condition that leads the design, above all when air pressure acts on high-rise building façades. This horizontal action can be both parallel and perpendicular to the plane of the façade itself and assume extremely high values in the most of the considered cases. The wind load, in fact, can even be an order of magnitude stronger that the other loads, such as seismic ones. As a result, normally glass, frame structure and fastening system verification under wind load also implies the satisfaction of the seismic load (considered as a force or an acceleration) verification.

So it is possible to forecast in this phase that it must be the relative displacement between two adjacent stories the main danger for the integrity of the several facade components, also because of the difference between masses and inertia compared to the structure. The curtain wall system must be designed to tolerate the seismic-induced building displacements in function of the seismic zone rating and of the building frame stiffness.

According to Italian guidelines NTC 2008, seismic actions in each building are evaluated in relation to:

- Nominal lifetime
- Importance class of use
- Reference period for the seismic section

<u>Nominal lifetime</u>

It is the number of years in which the construction must be able to be used for the purpose to which it is intended, under ordinary maintenance.

	Type of construction	**Nominal life V_n (y)**
1	Temporary works – In progress structures	≤10
2	Ordinary works, bridges, infrastructural works and dams of lower dimension or normal importance	≥50
3	Major works, bridges, infrastructural works and dams of higher dimension or strategic importance	≥100

Importance class of use

Importance class	Buildings
I	Bui1dings of minor importance for public safety, agricultural buildings, etc.
II	Ordinary buildings, not belonging in the other categories.
III	Buildings whose seismic resistance is of importance in view of the consequences associated with a collapse, e.g. schools, assembly halls, cultural institutions etc.
IV	Buildings whose integrity during earthquakes is of vital importance for civil protection, e.g. hospitals, fire stations, power plants, etc.

Reference period for the seismic section

V_R can be found with the following formula: $V_R = V_N \times C_U$

where V_N is the nominal lifetime and C_U is the coefficient of use, defined in relation to the class of use.

Class of use	I	II	III	IV
Coefficient C_U	0,7	1	1,5	2

Section 2: State of the art

4 Current design approaches

Seismic movement mechanisms require careful detailing to ensure that they are activated when required. Below, four common approaches are shown but it is possible for different methods to be used in one glazing system or in one building.

- **Seismic frame**: the glazed frame moves in a seismic frame, which moves with the building. The glazing frame is usually fixed at the sill.
- **Glazing pocket**: the glass is usually gasket glazed direct into the frame with pockets around the glass sufficiently deep to admit movement. This is a common approach in stick systems.
- **Unitized system**: individual units interlock, with provision for movement between each unit, both horizontally and vertically. This approach has become very common in multi-storey buildings especially.
- **Structural silicone**: where the other approaches provide a positive gap, in this case movement depends on the elasticity of the silicone. This approach is often used in conjunction with a stick system.

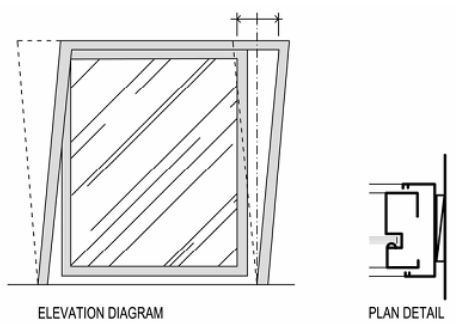

Figure 4-1: Seismic frame

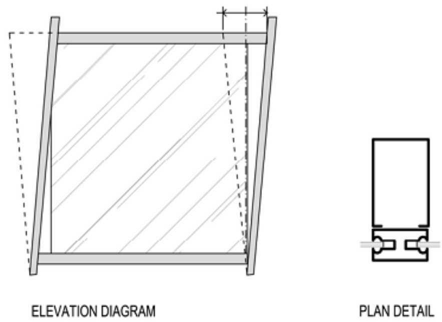

Figure 4-2: Glazing pocket

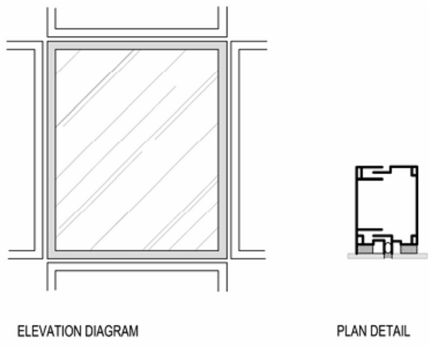

Figure 4-4: Unitized system

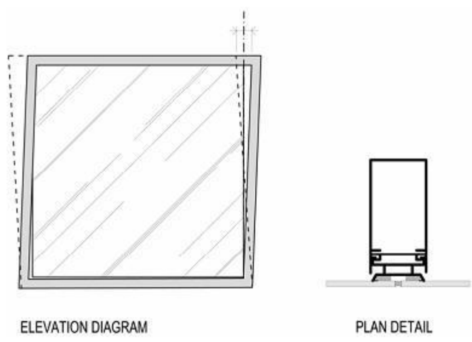

Figure 4-3: Structural silicone

5 Theoretical studies and mathematical formulations

5.1 Estimation of the drift capacity

As been observed by Bouwkamp, the in-plane deformation of window panels under lateral loading takes place in two phases: 1) the window frame deforms and the glass plate translates within the frame until contact occurs at two opposite corners of the glass panel; 2) the glass panel further rotates until its opposite corners coincide with the adjacent frame corners.

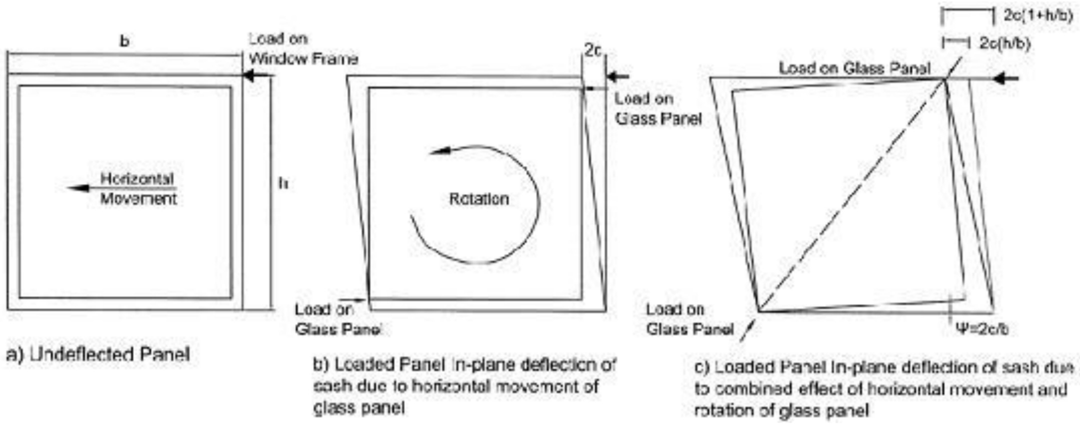

Figure 5-1: In-plane movement of window panel subjected to lateral loading

Sucuoglu and Vallabhan found that the total lateral deformation of the window panel due to rigid body motion of the glass panel in the window frame can be expressed in terms of the geometric properties of window panel components as:

$$\Delta = \mathbf{2}\boldsymbol{c}(\mathbf{1} + \frac{\boldsymbol{h}}{\boldsymbol{b}})$$

where *Δ* is the lateral drift capacity of the glass frame and *c*, *h* and *b* are physical dimensions as defined in the figure above.

For uneven clearances between vertical and horizontal glass edges and the frame, the equation can be modified as:

$$\Delta = \mathbf{2}\boldsymbol{c}_{\mathbf{1}}(\mathbf{1} + \frac{\boldsymbol{h}_{\boldsymbol{p}}\boldsymbol{c}_{\mathbf{2}}}{\boldsymbol{b}_{\boldsymbol{p}}\boldsymbol{c}_{\mathbf{1}}})$$

where: h_p = height of the rectangular glass panel, b_p = width of the rectangular glass panel, c_1 = clearance between the vertical glass edges and the frame, and c_2 = clearance between the horizontal glass edges and the frame.

This equation indicates that the in-plane drift capacity of the glazed frame, before glass breakage is only dependent on the edge clearance and the aspect ratio and that the in-plane drift capacity of the framed façades can be modified by increasing the edge clearance or aspect ratio, as shown in the following table referred to a 3600 mm high frame glazed curtain wall:

Height (h) (mm)	Width (b) (mm)	Aspect ratio (h/b)	In-plane drift capacity for typical edge clearances (mm)			
			c = 6	c = 8	c = 10	c = 12
3600	3000	1.2	26	35	44	53
3600	2400	1.5	30	40	50	60
3600	1800	2.0	36	48	60	72
3600	1200	3.0	48	64	80	96

Table 5-1: Typical in-plane drift capacity of framed glazed curtain walls

This expression is supposed to be valid when the glass panel is glazed with a soft sealant which permits the relative motion of the glass panel with respect to the window frame. Although the sealant hardens due to ageing, reducing the lateral drift capacity of the window panel, modern glazing systems which uses neoprene gaskets and other soft sealants possess sufficient resilience to accommodate the relative motion of glass panels in window frames.

Anyway, even a well-designed architectural glass curtain wall or window could potentially pose some seismic hazard after many years in service. That's why the existing architectural glass curtain walls or windows should be periodically inspected by a curtain wall professional, as an essential part of the evaluation process. The vulnerability can be expressed by a score in a suitable rating system for life-safety hazard.

5.2 Seismic rating system

A. M. Memari and A. Shirazi, from The Pennsylvania State University, presented a procedure for seismic evaluation in existing buildings of the class of nonstructural systems that includes architectural glass in curtain walls, storefronts, and windows, in order to develop a seismic rating methodology for architectural glass.

The overall score for the curtain wall depends on three major tasks:

- the story drift should be predicted by the use of building properties and seismic hazard maps as prescribed in building codes;
- computing the cracking initiation stress at the edge of the glass, which depends on the conditions of glass, frame, connections and glass panel boundary conditions;
- the relationship between the applied drift and the resulting stress in the glass panel should be developed.

Consequently, stresses due to input earthquake action and ultimate crack initiation stress can be compared and expressed as a score. This score would present the vulnerability of architectural glass for earthquake.

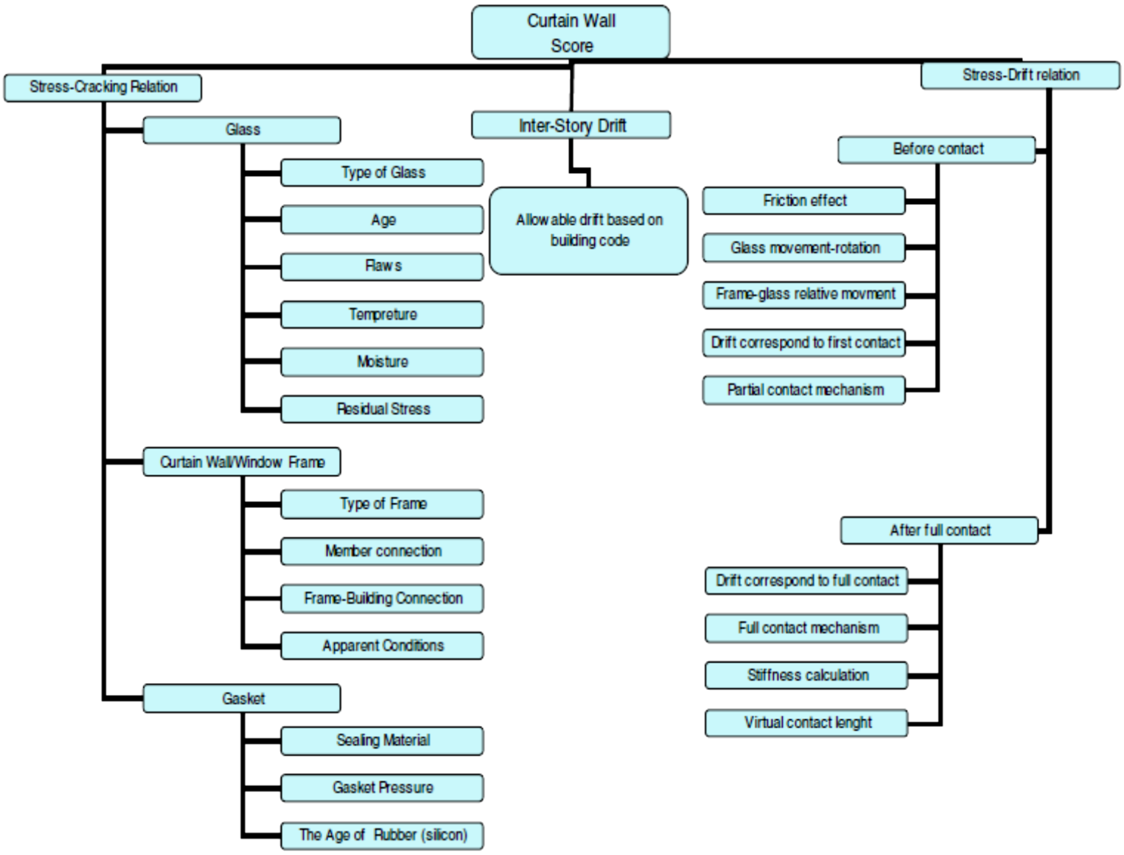

Figure 5-2: Parameters defined in a curtain wall glass rating system

In order to represent the conditions of the curtain wall glass by a score, parameters relevant to glass, frame, boundary conditions and building drift should be considered. The following flowchart illustrates the procedure for the score calculation.

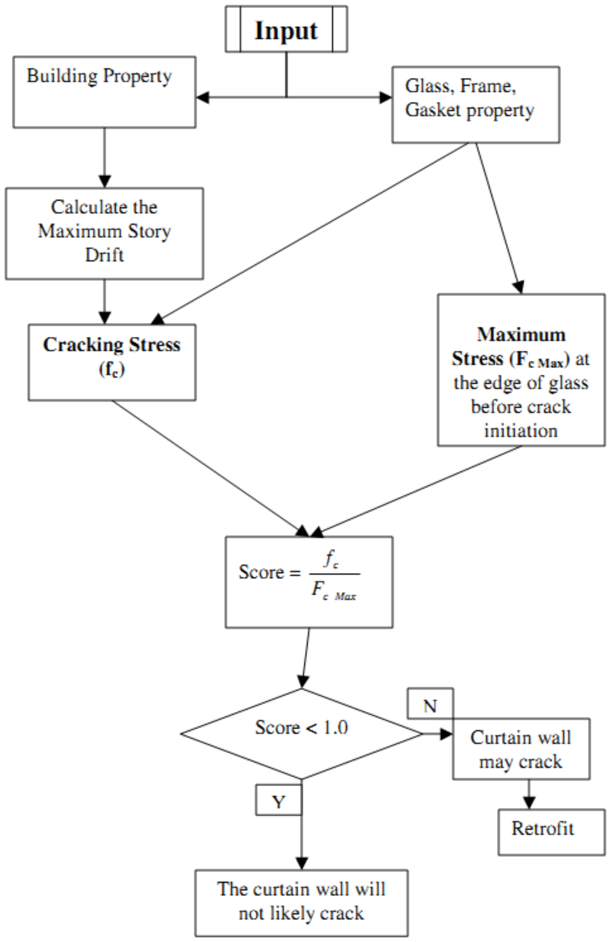

Figure 5-3: Flowchart for score calculation

Building properties are used to calculate the maximum interstorey drift. The stress in the glass panel induced in this drift can be computed by the use of displacement-stress relationships, which along with the equation for Fc Max are expressed in term of the gasket, frame and glass properties. The maximum crack initiation stress (Fc Max) is the stress corresponding to the crack initiation in the glass panel. The maximum crack initiation stress depends not only on crack initiation stress of manufactured glass but also on other conditions such as flaws and imperfections in the glass. By the use of ultimate manufactured glass stress and other curtain wall conditions, i.e., the conditions of the frame, glass panel, and the boundary conditions of the glass, the maximum stress that would correspond to crack initiation can be computed.

With the score defined in terms of stresses, the relationship between the lateral load applied to the glass panels and the associated displacement on the one hand, and the relationship between the displacement and the resulting stresses on the other hand should be established.

5.3 Vibration response of glass panels during earthquakes

There are two seismic response modes of window panels: in-plane deformation and out-of-plane vibration. Observations on the past earthquake damage indicate that in-plane deformation is the primary cause of window glass damage.

Seismic design codes tend to mitigate nonstructural damage in the out-of-plane vibration mode by designing for equivalent static seismic forces believing, perhaps, that glass panels are flexible enough to vibrate in bending by remaining within the low flexural stress levels. By the way, though this is true for certain cases, many glass failures, especially those on the storefront windows of commercial buildings during earthquakes are due to excessive out-of-plane vibrations.

In multistorey buildings which are relatively rigid, the seismic resistance of window glass panels due to out-of-plane vibration depends on the tensile strength of glass, that should exceed the developed tensile stresses expressed by equation proposed by Sucuoglu and Vallabhan:

$$\sigma_f = \frac{24\rho(1+\mu)}{\pi^4}\frac{a^2}{t}S_{af}$$

where ρ = mass density of the glass, μ = Poisson's ratio, a = glass panel dimension, t = thickness, S_{af} = absolute floor acceleration response spectra for the boundary excitation.

The stiffness of structural sealants in structural glazing systems has a negligible effect on the out-of-plane dynamic flexural response of the glass units. The effects of floor and response amplification should be taken into account realistically in the determination of lateral forces on window glass.

6 Experimental studies

There are two important studies, published respectively by A.B. King and S.J. Thurston from the Building Research Association of New Zealand (BRANZ) in 1992 and by R.A. Behr from the Pennsylvania State University in 1998, which have been the starting point and the main references for the International test standards and guidelines: the former is a static test and the latter a dynamic test.

6.1 Static interstorey sway test by A.B. King and S.J. Thurston

King and co-workers have performed a three year research programme which focused on the behaviour of curtain wall glazing systems when subjected to simulated interstory drift as may be expected to occur during the response of multi-storey buildings to earthquake attack. Four types of glazing systems were subjected to in-plane racking testing:

- Neoprene gasket dry-glazed system
- Unitized 4-sided structural silicone glazed system
- Two-sided silicone glazed system
- Mechanically fixed patch plate systems (with toughened glass)

Five different configurations were tested:

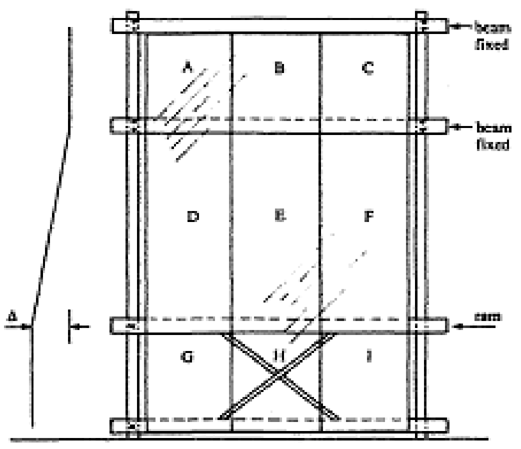

Figure 6-2: Single storey specimen with zero adjacent interstorey drift

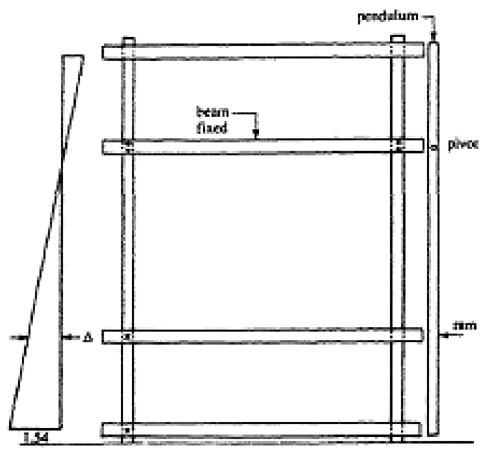

Figure 6-1: Single storey specimen with full adjacent interstorey drift

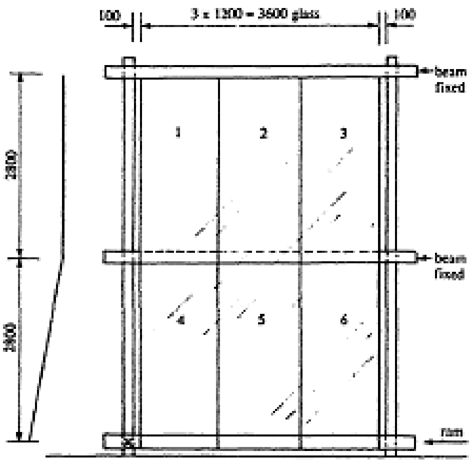

Figure 6-4: Double storey specimen with zero adjacent interstorey drift

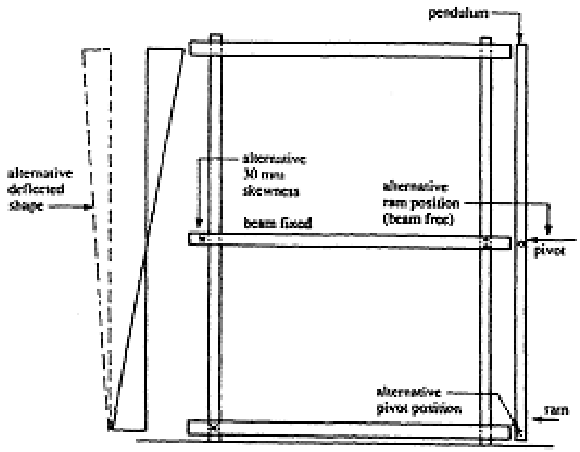

Figure 6-3: Double storey specimen with full adjacent interstorey drift

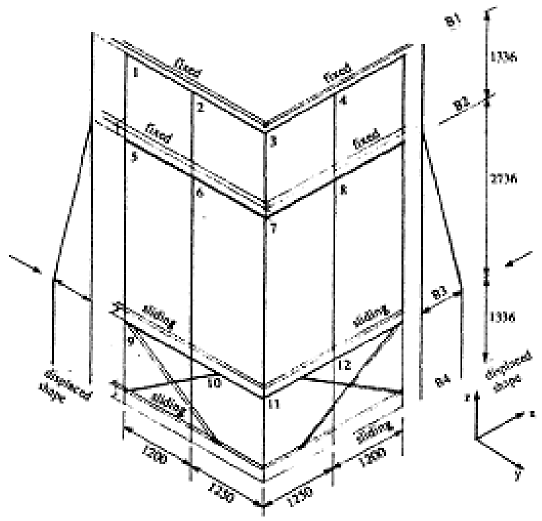

Figure 6-5: Corner specimen with zero adjacent interstorey drift

The procedure involved cyclically displacing the "floor" beam to a designed peak displacement which was increased by the appropriate increment each time. Movement was initiated using an hydraulic actuator operating under displacement control (two of these were used for the corner specimens).

One of the two theoretical mechanisms developed in all cases: either pane rotation within skewed frames (sometimes accompanied by mullion twisting), or a slip plane enabling frames to slide relative to each other.

Gasket glazed system

Interstorey displacements in excess of 100 mm were achieved without failure in all configurations. The panes were observed first to rotate within their frames. The aspect ratio of the panes tested was approximately 2.3:1 and the clearance was 17 mm. It was calculated that, for this geometry and clearance, contact between glass and framed would occur at an interstorey drift of approximately 140 mm.

The gaskets were observed to work loose from the frame during repeated cycling in excess of 15 mm interstorey drift. In each case failure was preceded by the loss of the gasket and the subsequent clashing of the glass and mullion because of misalignment. The glass typically developed a scallop shaped crack in one corner, which rapidly developed into multiple cracks and eventual (typically after a further one to three displacement exertions) the glass fell from the frame.

The corner specimen demonstrated similar pane rotation. Failure occurred in the corner pane at an in-plane interstorey displacement of 85 mm. Prior to this, the corner mullion cover plate separated and fell from the system. One significant difference observed between the planar and comer specimens, was that in the former tests, significant twisting along the axis of the mullion was noted and it was caused by eccentricities between the floor and the plane of the glass. This action was not present in the corner specimen because of the connections of the framing members at the corner mullion.

<u>Unitized four-sided silicone system</u>

The four-sided silicone panels were subjected to displacements in excess 80 mm without failure. The shear distortion was accommodated by a combination of slip between the units, and relaxation of the support brackets. At high drift levels, the complete panels were observed to rotate at which stage the panels disengaged from each other. One of the glass panes developed a diagonal crack as a result of out-of-plane distortion which occurred after this disengagement. By the way the glass remained attached to the frame through the silicone and therefore did not fulfill the failure criteria.

<u>Two-sided silicone system</u>

Different systems were used for the planar test specimen and the corner specimen.

In the planar specimen the glass was observed to distort the silicone joint relative to the mullions characteristic of vertical shear along this joint at 25 mm. The mullions were observed rotate about their splice points. At peak displacements of 60mm, the screws fastening the glazing bar to the mullion failed until fracture occurred so that glass failure could not be initiated.

In the corner specimen little movement was observed between the glass and frame when compared to the dry-glazed system. Most movement was accommodated by rotation of the complete unitized frame, particularly dominant in each corner pane, and by relaxation of the frame to floor connections. An initial crack was observed to develop in a panel at an interstorey displacement of 80 mm. A substantial portion of glass fell from this frame during the subsequent 100 mm interstorey displacement cycle.

Patch fitting system

The patch plates were detailed to allow joint rotation in an attempt to alleviate localized stress concentrations anticipated at the connection points. Both mechanisms of rotating and sliding were observed in combination and failure of the toughened glass occurred when the mechanism had displaced to the half of the available movement potential of the fixing. Failure occurred at an interstorey displacement of 40 mm, typically initiating at one fixing point, the toughened glass shatter pattern rapidly spread across the panel and the glass mass fell from the frame in coherent fragments measuring around 0.7 m for a side, which shattered onto smaller ones at the impact on the floor (around 10x10 mm).

In the end we can say that being the behaviour of the corner specimens consistent with the one of the planar specimens, it is reasonable to limit the standard tests to planar system only. The most appropriate configuration for testing is the single storey (with two half storey) one accompanied by the most severe limit of zero displacement of adjacent storeys.

6.2 Dynamic crescendo test by R.A. Behr

Behr conducted controlled laboratory tests to investigate the cracking resistance and fallout resistance of different types of architectural glass installed in storefronts and mid-rise wall systems. Effects of glass surface prestress, lamination, wall system type, and dry versus structural silicone glazing are discussed. Laboratory results revealed that distinct magnitudes of “drift” cause glass cracking and glass fallout in each glass type tested.

In-plane dynamic racking tests were performed using the facility shown in the figure below. Rectangular steel tubes at the top and bottom of the facility are supported on roller assemblies, which permit only horizontal motion of the tubes. The bottom steel tube is driven by a computer-controlled hydraulic ram, while the top tube is attached to the bottom tube by means of a fulcrum and pivot arm assembly. This mechanism causes the upper steel tube to displace the same amount as the lower steel tube, but in the opposite direction.

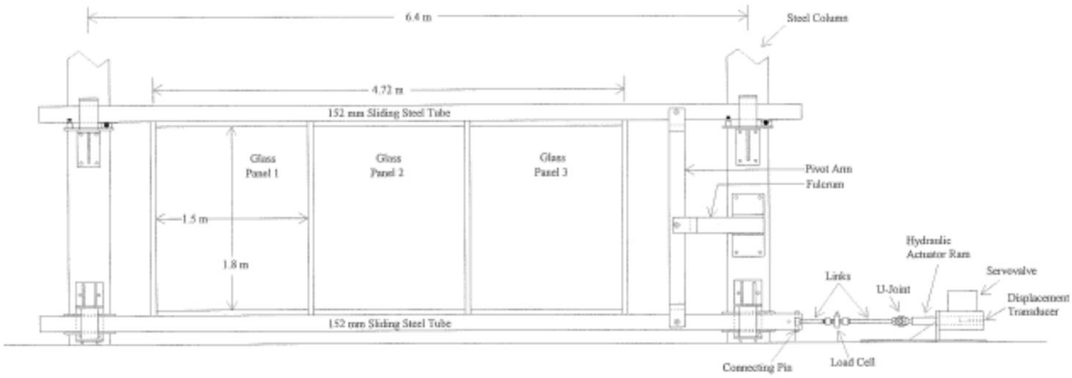

Figure 6-6: Dynamic racking test facility

All mid-rise glass types were tested using a dry-glazed wall system, which uses rubber gaskets between the glass edges and the curtain wall frame to secure each glass panel perimeter. In addition, three glass types were tested like two-sided structural silicone glazing systems. Six specimens of each glass type were tested. The glass types were:

- 6 mm Annealed Monolithic
- 6 mm Heat-Strengthened Monolithic
- 6 mm Fully Tempered Monolithic
- 6 mm Annealed Monolithic with 0.1 mm PET Film (film not anchored to wall system frame)
- 6 mm Annealed Laminated
- 6 mm Heat-Strengthened Laminated
- 6 mm Heat-Strengthened Monolithic Spandrel
- 25 mm Annealed Insulating Glass Units
- 25 mm Heat-Strengthened Insulating Glass Units

The crescendo test consisted of a series of alternating ramp-up and constant amplitude intervals, each containing four drift cycles. Each drift amplitude step was ± 6 mm. The entire crescendo test sequence lasted approximately 230 seconds. Crescendo tests on mid-rise glass specimens were conducted at different reducing frequencies for increasing dynamic racking amplitudes to avoid exceeding the capacity of the hydraulic actuator ram in the dynamic racking test facility.

The drift magnitude at which glass cracking was first observed was called the serviceability drift limit. The drift magnitude at which glass fallout occurred was called the ultimate drift limit. In addition the drift magnitude at which contact between the glass panel and the aluminum frame first occurred was recorded by using thin copper wires attached to each corner of the glass panel and to an electronics box. If the copper wire came into contact with the aluminum frame, an indicator light on the electronics box was actuated.

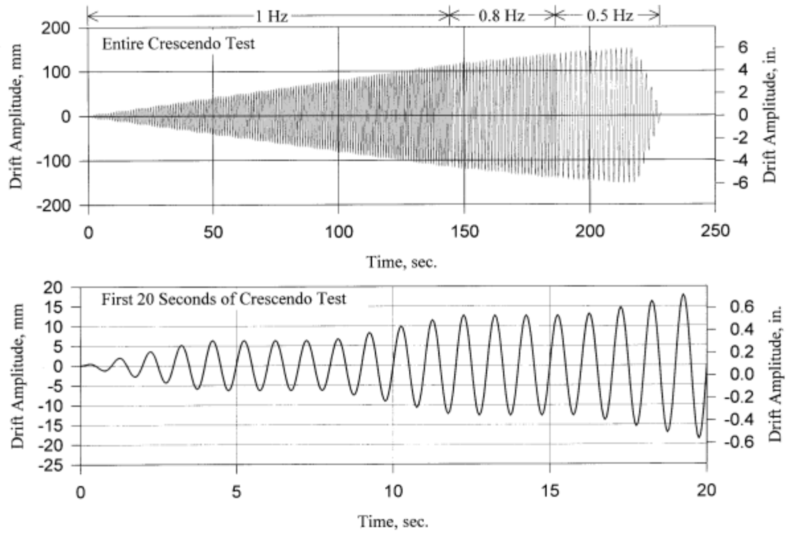

Figure 6-7: Drift time history in the crescendo test used for mid-rise architectural glass specimens

The results were:

- Annealed monolithic glass tended to fracture into sizeable shards, which then fell from the curtain wall frame.
- Heat-strengthened monolithic glass generally broke into smaller shards than annealed monolithic glass, with the average shard size being inversely proportional to the magnitude of surface compressive prestress in the glass.
- Fully tempered monolithic glass shattered into much smaller, cube-shaped fragments.
- Annealed monolithic glass with unanchored 0.1 mm PET film fractured into large shards, much like annealed monolithic glass without film, but the shards adhered to the film.

Dynamic drift amplitude, concerning initial contact with mullion, observable cracking and glass fallout, was put in relation with each type of glass to show different effects.

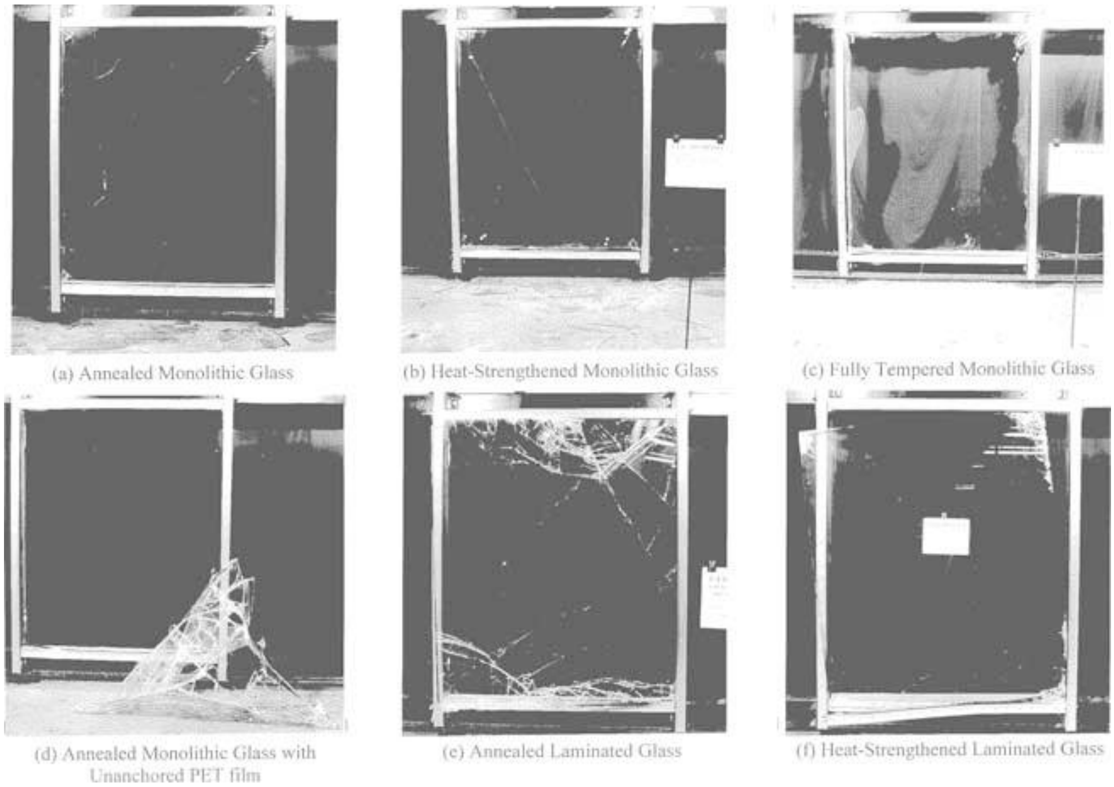

Figure 6-8: Typical failure patterns in various architectural glass types after in-plane dynamic racking tests

<u>Effects of glass surface prestress</u>

Slight increases in cracking and fallout drift limits can be seen for 6 mm monolithic glass as the level of glass surface prestress increases from annealed to heat-strengthened to fully tempered glass. However, effects of glass surface prestress on observed seismic drift limits were statistically significant only when comparing 6 mm fully tempered monolithic glass to 6 mm annealed monolithic glass. All six of the 6 mm fully tempered monolithic glass specimens shattered when initial cracking occurred, causing the entire glass panels to fall out. Similar behavior was observed in four of the six 6 mm heat-strengthened monolithic glass specimens. No appreciable differences in seismic drift limits existed between annealed and heat-strengthened 25 mm insulating glass units.

<u>Effects of lamination configuration</u>

Lamination had no appreciable effect on the drift magnitudes associated with first observable glass cracking but it had a pronounced effect on glass fallout resistance. All six annealed monolithic glass panels experienced glass fallout during the tests, five of six annealed monolithic glass specimens with unanchored 0.1 mm PET film experienced fallout and only one of six annealed laminated glass panels experienced fallout. All six heat-strengthened monolithic glass panels experienced fallout, while only four of six heat-strengthened laminated glass specimens fell out. Heat-strengthened laminated glass units tended to fall out in one large piece, instead of smaller shards like heat-strengthened monolithic glass.

Effects of wall system type

To investigate this parameter, results from the storefront wall system crescendo tests were compared to results from the mid-rise curtain wall crescendo tests. For all four glass types tested in both wall system types, the lighter, more flexible storefront frames allowed larger drift magnitudes before glass cracking or glass fallout than did the heavier, stiffer, mid-rise curtain wall frames. This observation held true for all glass types tested in both wall system types.

Effects of two-side structural silicone glazing

It increased the dynamic drift magnitudes associated with first observable glass cracking in both heat-strengthened monolithic glass and annealed insulating glass units. Architectural glass specimens with two-side structural silicone glazing exhibited higher resistance to glass fallout than comparable glass specimens that were dry-glazed.

Finally we can say that:

- the annealed type is probably the worst because it breaks in large and wide shards that fall down so that it's a very big hazard for someone walking under
- the heat-strengthened behaves in a similar way, excepting the higher values of loads resistance
- the fully tempered glass has a different behaviour caused by the uniform high compressive stress-state, it breaks in small shards, that are less dangerous than those deriving from annealed glass rupture
- the laminated glass has the additional value of being able to remain in the frame also after its rupture, because of the PVB keeping the shards stuck in the initial position. For this reason its use can be suggested or even required by the codes and standard local regulation for sloped glazing or even for vertical glazing of the façade in case of strong horizontal loads (for example seismic or wind loads).

7 International standards and guidelines

The most complete and important national standards concerning the seismic behavior of structural and non-structural elements are the ones of the highest seismic risk regions in the world. They represent the expression of the "state of the art" about this topic worldwide.

International standards

- ICC IBC 2009: International Building Code
- ISO 15822: Test method of doorset opening performance in diagonal deformation – seismic aspects
- ISO/NP 13033: Seismic actions on non-structural components for building applications (under development)
- ISO 3010: Basis for design of structures – seismic actions on structures

European standards

- EN 1998: Eurocode 8 – Design of structures for earthquake resistance
- prEN 13830: Curtain walling – product standard

Inside the European area we can find different standards for every country. In general they refer or absorb the prescription given by Eurocode 8, the only differences are in the mapping methods based on PGA (Peak Ground Acceleration). Here is a short list for some of the most sensible areas:

- NTC08: Building design technical regulation and Istruzioni CNR Vetro DT210 – Italy
- EAK2000: Greek antiseismic regulation – Greece
- NCSE-02: Antiseismic constructions regulation - Spain
- Arrêté du 22/10/10: Classification et regulation of antiseismic constructions - France

American standards/guidelines

FEMA: Federal Emergency Management Agency

- Residential
  - FEMA 232: Homebuilders' guide to earthquake resistant design and construction
- New buildings
  - FEMA 450: recommended provisions for seismic regulations for new buildings and other structures
  - FEMA 451: NEHRP (National Earthquake Hazards Reduction Program) recommended provisions: design examples
  - FEMA 454: Designing for earthquakes: a manual for architects
  - FEMA P-750: NEHRP recommended seismic provisions for new buildings and other structures (edition 2009)
- Existing buildings
  - FEMA P-420: Engineering guideline for incremental seismic rehabilitation

AAMA: American Architectural Manufacturers Association

- AAMA 501.4: Recommended static test method for evaluating curtain wall and storefront systems subjected to seismic and wind induced interstory drifts
- AAMA 501.6: Recommended dynamic test method for determining the seismic drift causing glass fallout from a wall system

ASCE: American Society of Civil Engineers

- ASCE 7-10: Minimum design loads for buildings and other structures

UFC: Unified Facilities Criteria

- UFC 3-310-04: Seismic design for buildings

ASTM: American Society for Testing and Materials

- ASTM E2026-07: Standard guide for seismic risk assessment of buildings

Australian standards

- AS 1170.4: Structural design actions – Earthquake actions in Australia
- AS/NZS 4284: Testing of building facades

New Zealand standards

- NZS 1170.5: Earthquake actions – New Zealand
- NZS 4219: Specification for seismic resistance of engineering systems in buildings
- NZS 4104: Seismic restraint of building content
- AS/NZS 4284: Testing of building facades

Chinese standards

- GB 50011-2001: Code for seismic design of buildings (mandatory)
- GB/T18250-2000: Test method for performance in plane deformation of curtain walls (voluntary)
- GB/T18575-2001: Shake table method of earthquake resistant performance for building curtain wall (voluntary)
- JGJ 102-2003: Technical code for glass curtain wall engineering (professional)

Indian standards

- IS 1893 (part 1): Criteria for earthquake resistant design of structures – General provisions and buildings
- IS 13935: Indian standard guidelines for repair and seismic strengthening of buildings

Japan standards

- JASS14: Japanese Architectural Standard Specification for Curtain Wall

7.1 Comparison between European and American Standards

This comparison wants to show the different approach to the topic by the European regulation and the American one.

The first one still doesn't give precise prescriptions about test methods to evaluate the seismic behaviour of curtain walls, only a standard under approval exists, prEN 13830 – Curtain walling – Product standard (DAV 2015-03), so we tend to refer to the American one, which is complete and it's a base for the European one.

The comparison is between EN 1998: Eurocode 8, Italian CNR DT210 concerning glass and FEMA 450, for what concerns calculation methods, and between prEN 13830 and AAMA 501.4/ AAMA 501.6 for what concerns testing methods.

EN 1998: Eurocode 8

Part 3 – Ground conditions and seismic action

National territories shall be subdivided by the National Authorities into seismic zones, depending on the local hazard. The hazard is described in terms of a single parameter, i.e. the value of the reference peak ground acceleration on type A ground, agR which derives from zonation maps found in National Annexes.

The reference peak ground acceleration corresponds to the reference return period TNCR of the seismic action for the no-collapse requirement chosen by the National Authorities. An importance factor *γI* equal to 1,0 is assigned to this reference return period. For return periods other than the reference, the design ground acceleration on type A ground *ag* is equal to *agR* times the importance factor *γI* (*ag = γI agR*).

In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used. In cases of very low seismicity, the provisions of EN 1998 need not to be observed.

Part 4.3.5 – Non-structural elements

Non-structural elements of buildings (e.g. curtain walls) that might, in case of failure, cause risks to persons or affect the main structure of the building or services of critical facilities, shall, together with their supports, be verified to resist the design seismic action.

For non-structural elements of great importance or of a particularly dangerous nature, the seismic analysis shall be based on a realistic model of the relevant structures and on the use of appropriate response spectra derived from the response of the supporting structural elements of the main seismic resisting system.

In all other cases the effect of the seismic action may be determined by applying to the non-structural element a horizontal force F_a which is defined as follows:

$$F_a = \frac{s_a w_a \gamma_a}{q_a}$$

where:

F_a is the horizontal seismic force, acting at the center of mass of the non-structural element in the most unfavorable direction;

W_a is the weight of the element;

S_a is the seismic coefficient applicable to non-structural elements;

γ_a is the importance factor of the element;

q_a is the behaviour factor of the element;

The seismic coefficient S_a may be calculated using the following expression:

$$S_a = \alpha S[\frac{3\left(1+\frac{z}{H}\right)}{(1+(1-\frac{T_a}{T_1})^{\,2}} - 0{,}5]$$

where:

α is the ratio of the design ground acceleration on type A ground, *ag*, to the acceleration of gravity *g*;

S is the soil factor (in National Annexes, it depends on the ground type);

T_a is the fundamental vibration period of the non-structural element;

T_1 is the fundamental vibration period of the building in the relevant direction[2] ;

z is the height of the non-structural element above the level of application of the seismic action (foundation or top of a rigid basement);

H is the building height measured from the foundation or from the top of a rigid basement.

The value of the seismic coefficient S_a may not be taken less than αS. The importance factor γ_a may be assumed to be γ_a = 1.0, while the behaviour factor q_a is assumed q_a= 2.0 for façades.

Part 4.4.3.2 – Damage limitation of interstorey drift

The "damage limitation requirement" is considered to have been satisfied, if, under a seismic action having a larger probability of occurrence than the design seismic action

[2] It is possible to calculate T_1 as prescribed in paragraph 4.3.3.2.2 of Eurocode 8. T_a is usually unknown, so it is possible to consider the ratio T_a/T_1 = 1 in favour of security. S_a will assume its maximum value, and so F_a too, as consequence.

corresponding to the "no-collapse requirement", the interstorey drifts are limited as follows[3]:

a) for buildings having non-structural elements of brittle materials attached to the structure:

$d_r v \leq 0{,}005h$;

b) for buildings having ductile non-structural elements:

$d_r v \leq 0{,}0075h$;

c) for buildings having non-structural elements fixed in a way so as not to interfere with structural deformations, or without non-structural elements:

$d_r v \leq 0{,}010\ h$

where:

d_r is the design interstorey drift evaluated as the difference of the average lateral displacements d_s (this is the displacement of a point of the structural system induced by the design seismic action) at the top and bottom of the storey under consideration;

h is the storey height;

v is the reduction factor which takes into account the lower return period of the seismic action associated with the damage limitation requirement (it may be found in National Annexes but in general the recommended values of v are 0,4 for importance classes III and IV and v = 0,5 for importance classes I and II).

<u>CNR-DT 210/2013 Instructions for Design, Construction and Control of Buildings with Glass Structural Elements</u>

Part 4.4 - Seismic action

4.4.1 - Introduction

From a seismic point of view, except in special cases, the structural elements of the glass elements can be considered non-structural, i.e. both the stiffness and the resistance of these elements do not affect significantly on the global response of the work. In fact, glass elements are designed with adequate play in connections that can

[3] Additional damage limitation verifications might be required in the case of buildings important for civil protection or containing sensitive equipment.

"isolate" them from the behavior of the main structure; else, being glass brittle, it must be assumed that they get fragmented under seismic action.

In the case in which it is required that the glass element is not damaged when subjected to seismic action, this must be suitably protected and seismically isolated from the structure to which it is connected. The support system must therefore ensure the glass panels to be able to move rigidly in their plane and out of it: the technical terminology international calls this ability clearance.

4.4.2 Definition of the design earthquake

The definition of the design earthquake is made according to the class of use of the building, of its service life and limit states that must be considered.

4.4.2.3 Evaluation of the capacity and performance levels required

In order to reduce the risk induced by damage and / or collapse of glass structural elements, the system, which is a set of glass elements and connection elements, must be designed and built in such a way as to provide adequate stability. The performances required are identified from four levels linked to four different limit states, as defined in the table below. The partial or total control of these levels depends on the class of use of the structure and the limit state that one wants to ensure to the structure itself.

Classification	**Description**
ND – No damage	It is assumed that the system is free from damages which require the replacement of the glass for the functionality of the building. In particular, the elements of the facade and roof must keep their requirements of impermeability to wind and precipitations
DL – Light damage	It is assumed that the system can suffer the loss of functionality of some elements, which rapid replacement does not involve any particular technical difficulties, remaining the building accessible. There is no risk for users linked to partial collapses.
DE – Heavy damage	The system is severely damaged, with high loss of functionality, with high charges for recovery, but there is no risk of falling material that may cause high risks as consequence.
C – Collapse	The system is severely damaged with possible extended slumps, too. Any glass fall out would cause risks comparable to other elements fall such as cornices and external cladding.

Table 7-1: Classification of required performaces

The performance requirements are given in the next table, that shows the level of performance required depending on the class of use of the structure for each defined four limit states. The level of performance is identified by the designation given in the previous table, accompanied by a subscript identifier of the return period. The value of the return period uniquely defines the accelerogram of the project.

Level	**Class of use**			
SL	**I**	**II**	**III**	**IV**
SLO	-	-	ND_{45}	ND_{60}
SLD	DL_{35}	DL_{50}	DL_{75}	DL_{100}
SLV	DE_{333}	DE_{475}	DE_{713}	DE_{950}
SLC	-	-	$C1_{463}$	$C1_{950}$

Table 7-2: Performance required in relation to limit states and class of use

4.4.4 Design displacement

Being local actions due to seismic acceleration usually minor compared to actions caused, for example, by the wind, the verification against local actions appears generally not significant.

The displacements of the building and especially the drift resulting from seismic action are essential parameters for the design of glass walls. In general, these come from the structural building analysis for the different limit states and performance levels required. The designer of the glass structures will refer to these data to design joints and connection systems of glazed elements to the rest of the structure.

For the only purpose of making a pre-sizing, or for preliminary assessments, the designer can refer to simplified evaluation shown in the Appendix 4:11.

4.4.5 Combination of the seismic action with other actions

To determine the combination of actions it is possible to refer to the information reported in the technical regulations in force at the national level [the NTC 2008]. For each limit state, the verifications must be carried out by combining the seismic action (E) with the action of permanent loads (G) and characteristic variable loads (Q_{kj}), in agreement with the following rule of combination, which refers to the combination coefficients (ψ_{2j}) reported in the following table.

Category variable action	ψ_{2j}
Category A: Residential	0.3
Category B: Office	0.3
Category C: Spaces susceptible to crowding	0.6
Category D: Commercial	0.6

Category E: Libraries, archives, warehouses and industrial	0.8
Category F: Garages and parking (for cars weighing ≤ 30 kN)	0.6
Category G: Garages and parking (for cars weighing > 30 kN)	0.3
Category H Roofing	0.0
Wind	0.0
Snow (altitude ≤ 1000 m a.s.l.)	0.0
Snow (altitude > 1000 m a.s.l.)	0.2
Thermal variations	0.0

Table 7-3: Combination coefficients

The effects of seismic action will be evaluated taking into account the masses associated with gravity loads:

$$G + \sum_j \Psi_{2j} Q_{kj}$$

The above described actions will be especially used to assess the movements of the points of attachment of the glazed elements, carrying out audits in accordance with the procedures outlined in Section 7.6.2.

4.11 Appendix. Simplified method for the evaluation of the capacity request in terms of displacement

The designer can refer to a simplified method for a preliminary pre-sizing of structural elements, suited exclusively to cases of multi-frame buildings with high flexibility. However, to design glass elements is recommended to use the displacements calculated by the designer of the supporting structures.

Defined the site, the geomorphology of the terrain and the class of use of the building, depending on the return period, the response spectra are calculated in terms of the pseudo-acceleration relative to each of the limit states SLO, SLD, SLV and SLC.

From the spectra in terms of pseudo-acceleration $Sa(T)$ is possible to pass to the spectra in terms of displacement $Sd(T)$ and finally find the maximum displacement of a one degree of freedom oscillator at the base, relative to SLC ($d$max,SLC). This can be assumed as a reference value for the other limit states, SLV, SLD, SLO by rescaling it appropriately according to the coefficients of the following table.

SLO	0.085
SLD	0.22
SLV	0.71
SLC	1

Table 7-4: coefficients for limit states to find the maximum displacement

Finally *d*max,MDOF at the top of the building can be found . Interstorey drift D_p is the ratio *d*max,MDOF / n where n is the number of storeys.

7.6.2 Test of of compatibility of displacement

The test of compatibility with the displacement of the bound points due to the deformation of the sismo-resistant structure represents the most important check for glass elements.

The fastening system of the glass element to the sustaining structure should be designed so as to guarantee the performance levels previously defined.

Defined the class of use of the building, the design accelerograms are evaluated on the basis of the return period defined. From the analysis of the load bearing structure of the building, conducted with the methods specified in the technical standards (linear or non-linear analysis, static or dynamic), drifts are evaluated at the points of attachment of the glazed elements for each of the 4 states limit.

The stress derives from the relative displacements of these points of attachment. The required capacity to the system is defined by the performance levels defined for each limit state.

In the case where the designer provides for the possibility of broken glass, you will still have to verify that the system (glass + connection) is designed to prevent the catastrophic fall of the element under the seismic action. In particular, it must be monitored the performance of the silicone joints.

<u>FEMA 450</u>

Part 3.3 – Ground motion – General procedure

The acceleration parameters S_S and S_1 shall be determined from the respective 0.2 sec and 1.0 sec spectral response accelerations showed on maps in the same document.

The maximum considered earthquake (MCE) spectral response acceleration parameters S_{MS} and S_{M1}, adjusted for site class effects, shall be determined as follows:

$S_{MS} = F_a S_S$ and $S_{M1} = F_v S_1$

where F_a and F_v are site coefficients defined in tables.

The design acceleration parameters S_{DS} and S_{D1} shall be determined as follows:

$S_{DS} = 2/3\ S_{MS}$ and $S_{D1} = 2/3\ S_{M1}$

Part 6.2 – Architectural, mechanical and electrical component design requirement – General design requirement

6.2.2 Component importance factor. All components shall be assigned a component importance factor as indicated in this section. The component importance factor, I_p, shall be taken as 1.5 if any of the following conditions apply:

1. The component is required to function after an earthquake,
2. The component contains hazardous materials, or
3. The component is in or attached to a Seismic Use Group III structure and it is needed for continued operation of the facility or its failure could impair the continued operation of the facility.

All other components shall be assigned a component importance factor, I_p, equal to 1.0.

6.2.6 Seismic forces. The seismic design force, F_p, applied in the horizontal direction shall be centered at the component's center of gravity and distributed relative to the component's mass distribution and shall be determined as follows:

$$F_P = \frac{0{,}4 a_P S_{DS} W_P}{R_P/I_P}\left(1 + 2\frac{z}{h}\right)$$

F_p is not required to be taken as greater than: $F_p = 1.6 S_{DS}\, I_p\, W_p$

and F_p shall not be taken as less than: $F_p = 0.3\ S_{DS}\, I_p\, W_p$

where: W_p is the operating weight of a non-structural component and a_p (component amplification factor) and R_p (component response modification factor) are determined in the following table:

Exterior non-structural wall elements and connections	a_p	R_p
Wall element	1.0	2.5
Body of wall-panel connections	1.0	2.5
Fasteners of the connecting system	1.25	1.0

Table 7-5: a_p and R_p for exterior non-structural wall elements and connections

The force F_p shall be independently applied in each of two orthogonal horizontal directions in combination with service loads. In addition, the non-structural component shall be designed for a concurrent vertical force $\pm 0.2 S_{DS} W_p$.

Where wind loads on non-structural exterior walls or building code horizontal loads on interior partitions exceed F_p, such loads shall govern the strength design, but the detailing requirements and limitations prescribed in this chapter shall apply.

6.2.7 Seismic relative displacements. The relative seismic displacements, D_p, for use in component design shall be determined as follows:

$D_p = \delta_{xA} - \delta_{yA}$

D_p is not required to be taken greater than:

$$D_P = (X - Y)\frac{\Delta_{aA}}{h_{sx}}$$

where: δ_{xA} is the deflection at level x of structure A, δ_{yA} is the deflection at level y of structure A, X is the height above the base of the upper support attachment, Y is the height above the base of the lower support attachment, Δ_{aA}/h_{sx} is the is the allowable drift index for structure A.

The effects of relative seismic displacement shall be considered in combination with displacement caused by other loads as appropriate.

6.3.2 Exterior non-structural wall elements and connections. Exterior non-structural wall panels or elements that are attached to or enclose the structure shall be designed to accommodate the seismic relative displacements defined in Sec. 6.2.7 and movements due to temperature changes. Such elements shall be supported by means of positive and direct structural supports or by mechanical connections and fasteners in accordance with the following requirements:

1. Connections and panel joints shall allow for a relative movement between stories of not less than the calculated story drift D_p or 1/2 in. (13 mm), whichever is greater.
2. Connections to permit movement in the plane of the panel for story drift shall be sliding connections using slotted or oversized holes, connections that permit movements by bending of steel, or other connections that provide equivalent sliding or ductile capacity.
3. Bodies of connectors shall have sufficient deformability and rotation capacity to preclude fracture of the concrete or low deformation failures at or near welds.
4. All fasteners in the connecting system such as bolts, inserts, welds, and dowels and the body of the connectors shall be designed for the seismic force F_p determined.
5. Where anchorage is achieved using flat straps embedded in concrete or masonry, such straps shall be attached to or hooked around reinforcing steel or otherwise terminated so as to effectively transfer forces to the reinforcing steel.

Glass in glazed curtain walls and storefronts shall be designed and installed meeting the relative displacement requirement:

$\Delta_{fallout} \geq 1.25\ I\ D_p$ or *0.5 in. (13mm)*, whichever is greater.

6.3.8 Seismic Drift Limits for Glass Components. $\Delta_{fallout}$, the drift causing glass fallout from the curtain wall, storefront or partition, shall be determined in accordance with AAMA 501.6, or by engineering analysis.

<u>AAMA 501.4 – Recommended static test method for evaluating curtain wall and storefront systems subjected to seismic and wind induced interstorey drift</u>

6.0 – Test specimens

The specimen width shall not be less than two typical units plus the connections and supporting elements at both sides, and shall be sufficient to provide full loading on at least one typical vertical joint or framing member, or both. The height shall not be less than the full building story height or the height of the unit, whichever is greater, and shall include at least one full horizontal joint, which accommodates vertical expansion. For multi-storey systems, the specimen height shall not be less than two full building stories plus the height necessary to include one full horizontal joint accommodating vertical expansion.

All parts of the curtain wall or storefront test specimen shall be full size, using the same materials, type of glass, details, methods of construction, and anchorage as those used on the actual building.

The test chamber structure shall simulate the main structural supports of the actual building. However, the test chamber support structure may differ from the actual buildings as required to perform the required displacement. For curtain wall mock-ups, the test chamber shall be constructed so that the anchorage at the simulated floor structure at an intermediate level of the test specimen is moveable in the horizontal direction(s). For single story mock-ups, the test chamber shall be constructed so that the anchorage at the top or bottom is moveable in the horizontal direction.

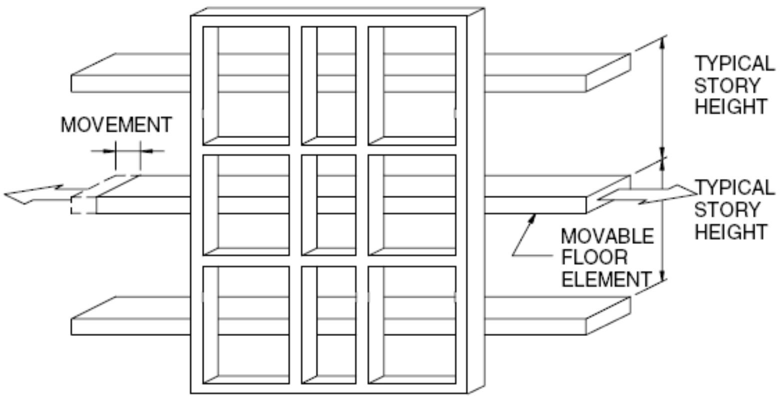

Figure 7-1: Test chamber structure

7.0 – Recommended test procedure

Test chamber elements representing the primary building structure shall be displaced to produce the specified movements. Each test shall consist of three full cycles (a cycle is defined as a full displacement in one direction, back to the originating point, full displacement in the opposite direction, and back to the originating point).

The Test Agency shall record all areas of visual distress, such as disengagement, metal distortion, sealant or glazing failure, or permanent deformation, defining the cause of an eventual glazing breakage.

The time duration of each cycle is not prescribed.

The design displacement shall be determined by the specifier according to the predicted interstorey movements of the subject building. For multi-story mock-ups, the displacement between levels may vary due to different story heights. Unless otherwise specified, the design displacement shall be 0.010 x the greater of the adjacent story height.

The displacement shall be measured at the movable floor element, not at the test specimen.

Serviceability tests for air leakage and water penetration may be conducted after the static displacement tests, following the default test sequence, unless otherwise specified:

- Air leakage (ASTM E283)
- Static water resistance (ASTM E331)
- Dynamic water resistance (AAMA 501.1) (optional)
- Structural performance @ design wind pressure (ASTM E330)
- Repeat air leakage (ASTM E283) (optional)
- Repeat static water resistance (ASTM E331) (optional)
- Seismic movement @ design displacement (AAMA 501.4)
- Repeat air leakage (ASTM E283)
- Repeat static water resistance (ASTM E331)
- Structural performance @ 1.5 x design wind pressure (ASTM E330)
- Seismic movement @ 1.5 x design displacement (AAMA 501.4)

11.0 – Pass/fail criteria

The project specifications shall state detailed pass/fail performance criteria for the curtain wall or storefront wall system. If detailed pass/fail performance are not in the project specification, the following criteria shall be utilized:

- A specimen subjected to the design displacement test shall be considered passing the interstory drift provisions if the applicable performance level (with certain building occupancy type) is achieved.
- A specimen subjected to the 1.5 x design displacement shall be considered passing if all glass is retained completely in the glazed opening with no glass fallout and no wall components fall off.

AAMA 501.6 – Recommended dynamic test method for determining the seismic drift causing glass fallout from a wall system

5.0 – Test specimens

Unless otherwise specified, $\Delta_{fallout}$ tests shall be conducted on test specimens that simulate closely the components of the overall wall system being evaluated.

For wall systems comprised of individual windows or punched openings, the test specimen shall consist of three individual units. For the case of individual units, it is permissible to install all three units together in the test apparatus (assuming the actuator capacity is not exceeded at $\Delta_{fallout}$) and test them simultaneously, or install each unit in the test apparatus individually, and test them one at a time.

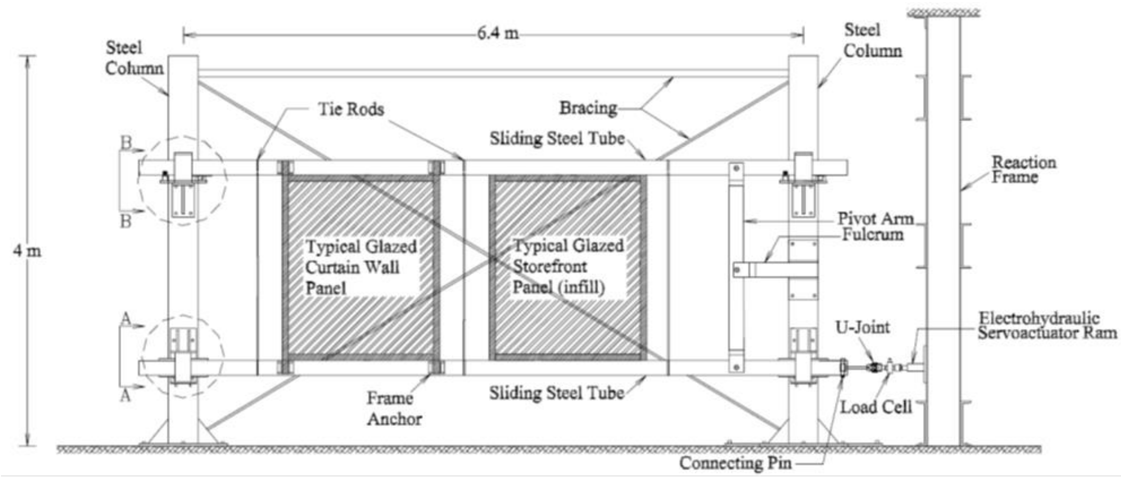

Figure 7-2: Dynamic Racking Test Facility at the Building Envelope Research Laboratory, Department of Architectural Engineering, The Pennsylvania State University, University Park, PA.

6.0 – Test procedures

"Crescendo tests" shall consist of a concatenated series of "ramp up" intervals and "constant amplitude" intervals. In-plane (horizontal) racking displacement steps between constant amplitude intervals shall be 6 mm. Ramp up intervals and constant amplitude intervals shall consist of four sinusoidal cycles each.

Each crescendo test shall be run continuously until completion. Each crescendo test shall proceed until the first of the following conditions exists: glass fallout, the drift index over the height of the glass panel is at least 0.10 (10%), a dynamic racking displacement of ±150 mm is applied to the test specimen.

Three test specimens of each glass panel configuration in the $\Delta_{fallout}$ test plan shall be subjected to the crescendo test. The dynamic racking amplitude associated with glass fallout, $\Delta_{fallout}$, shall be measured and recorded during each crescendo test. The lowest Δfallout value measured during the three crescendo tests shall be the controlling value reported for that set of specimens. Glass fallout is considered to have occurred when an individual glass fragment larger than 650 mm2 falls in any direction from the test panel glazed opening. If no glass fallout occurs by the end of the crescendo test, Δfallout for that specimen shall be reported as being "greater than" the maximum drift amplitude in mm imposed on the test specimen during the crescendo test.

prEN 13830 - "Curtain walling - Product standard"

4.10 – Requirements – Seismic Resistance

The curtain walling must withstand a seismic action having a larger probability of occurrence than the design seismic action, without risks to persons, the occurrence of damage and the associated limitations of use.

Serviceability is defined as the ability of the curtain walling to remain serviceable following the declared seismic action. When tested in accordance with 5.10 the watertightness and the air permeability classes shall confirm those measured.

Safety in use is defined as the ability of the curtain walling to: 1) resist the inertia forces caused by the declared seismic action, the fixings shall transfer the inertia forces to the supporting structure; 2) have movement accommodation to prevent failure of the infill panels, frame connections or fixings as a result of the declared seismic action; 3) no components of the curtain walling kit shall separate and fall from the curtain walling kit as a result of the declared seismic safety limit, unless it has been specifically evaluated that it is safe for them to do so.

5.10 – Testing, assessment and sampling methods– Seismic Resistance

For testing serviceability, the curtain walling kit shall be assessed by imposing in-plane movements as shown in Annex E.4 prior to retesting for air permeability and watertightness.

The maximum racking movement that the specimen can undergo and still retain its acceptable air permeability and watertightness performance shall be recorded. Seismic serviceability limit is expressed as angular rotation of a mullion from the vertical (in-plane).

When testing safety in use, in accordance with Annex E, the maximum racking movement and maximum inertia forces that the curtain walling kit can undergo without becoming unsafe shall be recorded. Seismic safety limit shall be expressed as both angular rotation of a mullion from the vertical (in-plane) and acceleration out of plane.

E.2 Assessment of seismic serviceability limit. The serviceability limit of the curtain walling kit shall be assessed by imposing in-plane movements as reported in E.4 prior to re-testing for air permeability and watertightness. The test specimen should be subjected to three cycles of movement (a cycle is defined as a movement to one extreme position, a movement to the other extreme position, the return to the original position).

The extreme position should be the displacement at the seismic serviceability limit. The rate at which the displacements are applied are decided by the manufacturer.

The positive difference between the air permeability measured at maximum pressure before and after the seismic movement should not differ by more than 0,6 m3/h.m2 (0,2 m3/h.m length of joint).

E.3 Assessment of seismic safety limit. Most curtain walling kit are sufficiently lightweight that the out-of-plane seismic inertia forces are less than the design wind load. The movement accommodation of the curtain walling kit may be assessed by imposing in-plane movements as reported in E.4. The test specimen should be subjected to one cycle of movement.

The extreme position should be the displacement at the seismic safety limit. If the curtain walling kit remains in a safe condition following the seismic movement regime the sequence of movements may be repeated at a higher magnitude.

The curtain wall shall safely withstand the seismic movement regime and shall retain its integrity in fulfilling the following criteria: 1) no parts shall fall down; 2) any holing shall not occur; 3) any infilling panel shall remain in its position and come off only when removed; 4) any permanent deformation of curtain wall component shall be accepted.

E.4 Seismic movement regime. For stick construction the test specimen shall be subjected to the movements shown in figure 6-3 (fixed base). It is acceptable to restrain the head of the curtain walling kit against movement and apply the in-plane horizontal movement at the base of the curtain walling kit. The height h shall represent the intended construction.

It may be easier on larger specimens of stick construction to achieve the required movements by using the arrangement shown in figure 6-4 (top and base are fixed, the middle is moved).

For unitized construction the test specimen shall be subjected to the movement shown in figure 6-5. The height h shall represent the intended construction. The specimen should contain at least two panels in the width and two panels in the height.

The movement *Δ* shall be reported as the angle of rotation *arctangent(γ)*, where

γ = Δ/h.

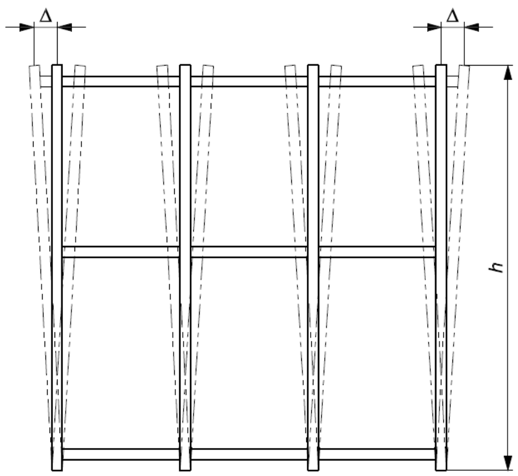

Figure 7-4: Test specimen for stick construction one storey height

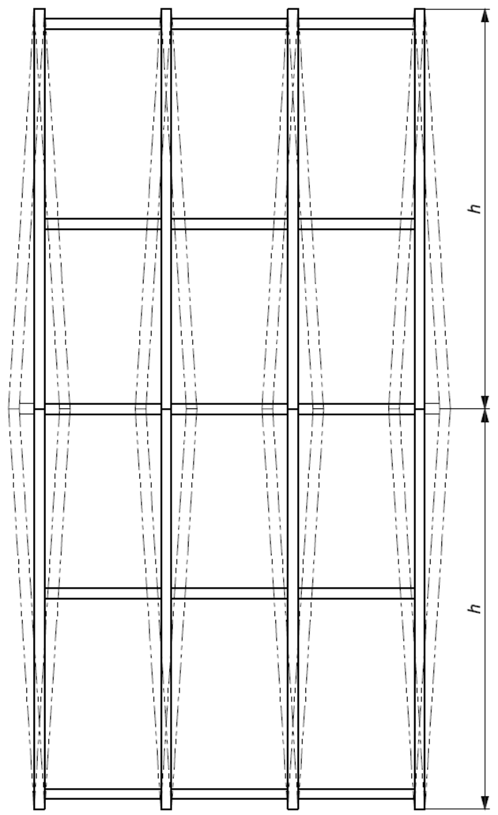

Figure 7-5: : Test specimen for stick construction two storeys height

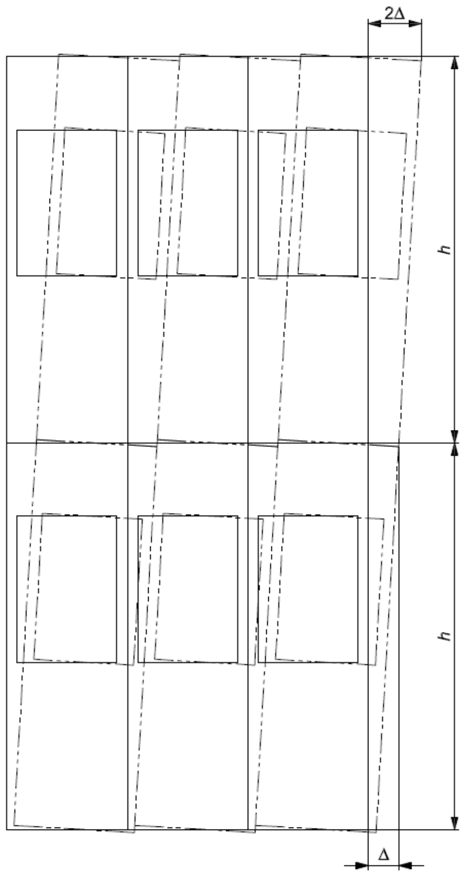

Figure 7-3: Test specimen for unitized construction two storeys height

Proposal: Annex XX , Part 10– Response of curtain wall to seismic action – Testing

The test specimen shall be selected in such a way that it is representative of the curtain wall system. The specimen shall be mounted on movable supports so that the top of the wall may be moved horizontally in the plane of the wall. It shall be possible to conduct air permeability and watertightness tests on the specimen as described in EN 13830.

Tests shall be performed in the following sequence:

a) Air permeability – for classification
b) Watertightness under static pressure – for classification
c) Resistance to windload – serviceability
d) Air permeability – repeat to confirm wind resistance classification
e) Water tightness – repeat to confirm wind resistance classification
f) Seismic movement regime – serviceability
g) Air permeability – repeat to confirm seismic serviceability classification
h) Water tightness – repeat to confirm seismic serviceability classification
i) Resistance to wind load, increased wind resistance test – safety
j) Seismic movement regime – safety

The person requesting the test for serviceability shall state the displacements to be applied to the top of the specimen but they shall not be less than 10mm. The top of the wall shall be moved gradually as follows: 1) the top of the wall shall be moved to the required displacement in one direction and held in that position for 5 minutes; 2) it shall then be moved to the other extreme of movement and held in that position for 5 minutes; 3) the wall shall be returned to its undisplaced position; 4) a period of at least 5 minutes shall elapse before any further tests are conducted.

The person requesting the test for safety shall state the increments of displacement to be applied to the top of the specimen. The top of the wall shall be displaced in each direction at each increment of displacement. The top of the wall shall be moved gradually as follows, a typical movement sequence might be: 1) Left, 20mm; 2) Right 20mm; 3) Left 30mm; 4) Right 30mm.

Throughout the test any damage shall be noted and the displacement at which the damage occurred shall be recorded.

Here is a table which sums up and compares the two regulations.

Seismic behaviour of curtain wall façades by an International Standards point of view	
EUROPE	AMERICA
Calculation methods	
EN1998: Eurocode 8 - Design of structures for earthquake resistance	FEMA 450: recommended provisions for seismic regulations for new buildings and other structures
Ground acceleration parameters defined by each National Authority	Acceleration parameters defined in the same document for all USA
Referred to non-structural elements in general	Referred to architectural elements in general at first and to external non-structural wall elements and connections later
Seismic force applied in the centre of mass in the most unfavourable direction (horizontal): $\boldsymbol{F_a} = \frac{S_a W_a \gamma_a}{q_a}$	Seismic force applied in the centre of mass in the horizontal direction: $F_P = \frac{0{,}4 a_P S_{DS} W_P}{R_P/I_P}\left(1+2\frac{z}{h}\right)$
Damage limitation if interstory drift shall be observed for different types of non-structural elements	Connections and panel joints shall allow for a relative movement between stories of not less than the calculated relative seismic displacement
No specifications or references for curtain walls	Reference to AAMA 501.6 for glass displacement in glazed curtain walls and storefronts
Testing methods	
Only a standard under approval and a proposal exist: prEN13830 and Annex XX	Two different standards exist: AAMA 501.4 and AAMA 501.6
prEN 13830: Curtain walling – Product standard (same document for all requirements and tests for curtain walling)	AAMA 501.4: Recommended static test method for evaluating curtain wall and storefront systems subjected to seismic and wind induced interstory drift
Annex E: 2 tests (serviceability and safety); 3 seismic movement regimes for different systems	Accurate description of the test specimen and the test chamber structure
Serviceability test: 3 cycles at the same displacement (seismic serviceability limit); pressure differences at air permeability between after and before prescribed. Safety test: 1 cycle at the seismic safety limit displacement	3 cycles (time frame not prescribed);test sequence (safety test: 1.5 x design displacement) detailed pass-fail criteria for different building occupancy types
	AAMA 501.6: Recommended dynamic test method for determining the seismic drift causing glass fallout from a wall system
Annex XX: Response of curtain walls to seismic action – testing. Test sequence	Detailed description of dynamic racking test facility and specimen (3 individual units)
Serviceability: defined time frame between each displacement	Crescendo test: concatenated series of ramp up intervals and constant amplitude intervals (4 sinusoidal cycles each); detailed pass-fail criteria
Safety: continuous gradual increment of displacement	

Section 3: Static tests at Reynaers Aluminium

8 Experimental performance static tests

For the purpose of investigating the seismic behaviour of curtain walls, I gave my contribute to a study carried out by Reynaers Aluminium, a leading European specialist in the development and marketing of innovative and sustainable aluminium solutions for windows, doors, curtain walls, sliding systems, sun screening and conservatories, in its test center of the headquarter in Duffel, Belgium.

The aim was to investigate the drift capacity of two most popular stick systems of the company, CW50 and CW60, and to compare it to the theoretical estimated ones.

In order to do that, a full-scale mock-up test according to the American test standard AAMA 501.4 has been conducted.

These non-standardized tests are intended to evaluate the ability of the system to accommodate interstorey drifts without engaging the glass in a way that will produce breakage.

8.1 CW50 static test

8.1.1 Description of the mock-up

AAMA 501.4 recommends that for curtain walls the specimen width shall be not less than two typical units plus the connections and supporting elements at both sides and that, for multi-storey systems, the specimen height shall not be less than two full building stories plus the height necessary to include one full horizontal joint accommodating vertical expansion. The specimen used is made by 4 mullions and 9 transoms of standard stick curtain wall system CW50 series in order to obtain 6 glazed units as it’s shown in the following scheme.

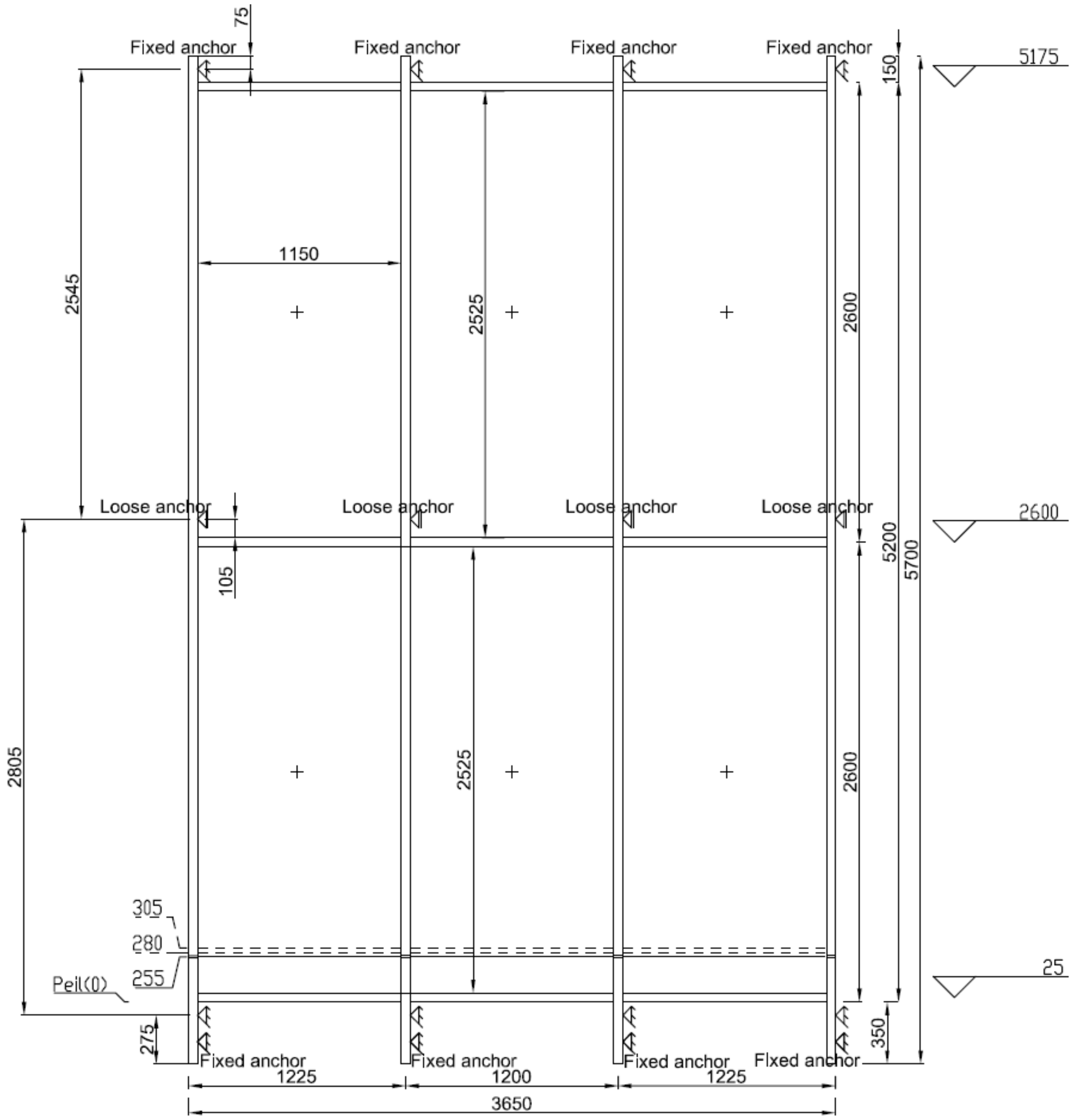

Figure 8-1: installation scheme

At the bottom there is an interruption of the mullion to allow dilatation; the bottom anchors are longer than the others to simulate the curtain wall continuity; no glass panels are present under the lower transoms and above the top transoms.

The specimen is connected to the structure by fix anchors on the top and on the bottom and by loose anchors at the middle.

A double glass with composition 44.2/12/44.2 is used in the test. The inside as well as the outside glass pane is laminated. This type of glass increases the safety during the test and increases the fall out resistance.

The structure is made by 4 steel tubes: one is on the top, one is on the bottom and two are in the middle for simulating the interstorey drift by a reciprocal movement. The displacement is made possible by a manually controlled hydraulic cylinder.

In order to allow a good visualization of what happens in the corners it has been decided to let them uncovered for 200 mm on each side by the pressure plate.

The following figures show the whole structure through a **Figure 8-2: frontal view**, a **Figure 8-3: back view** and a **Figure 8-4: lateral view**; two pictures are also present (**Figure 8-5: picture of back view, Figure 8-6: picture of frontal view**).

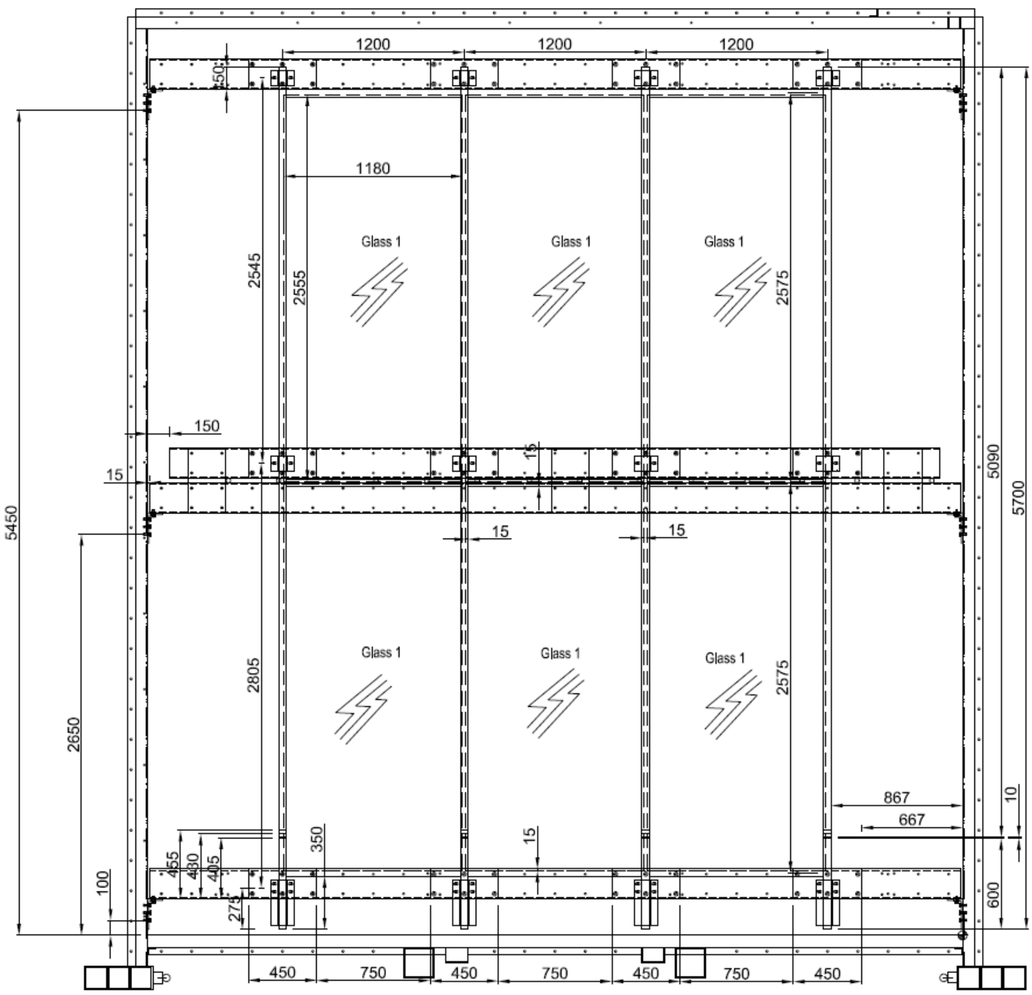

Figure 8-2: frontal view

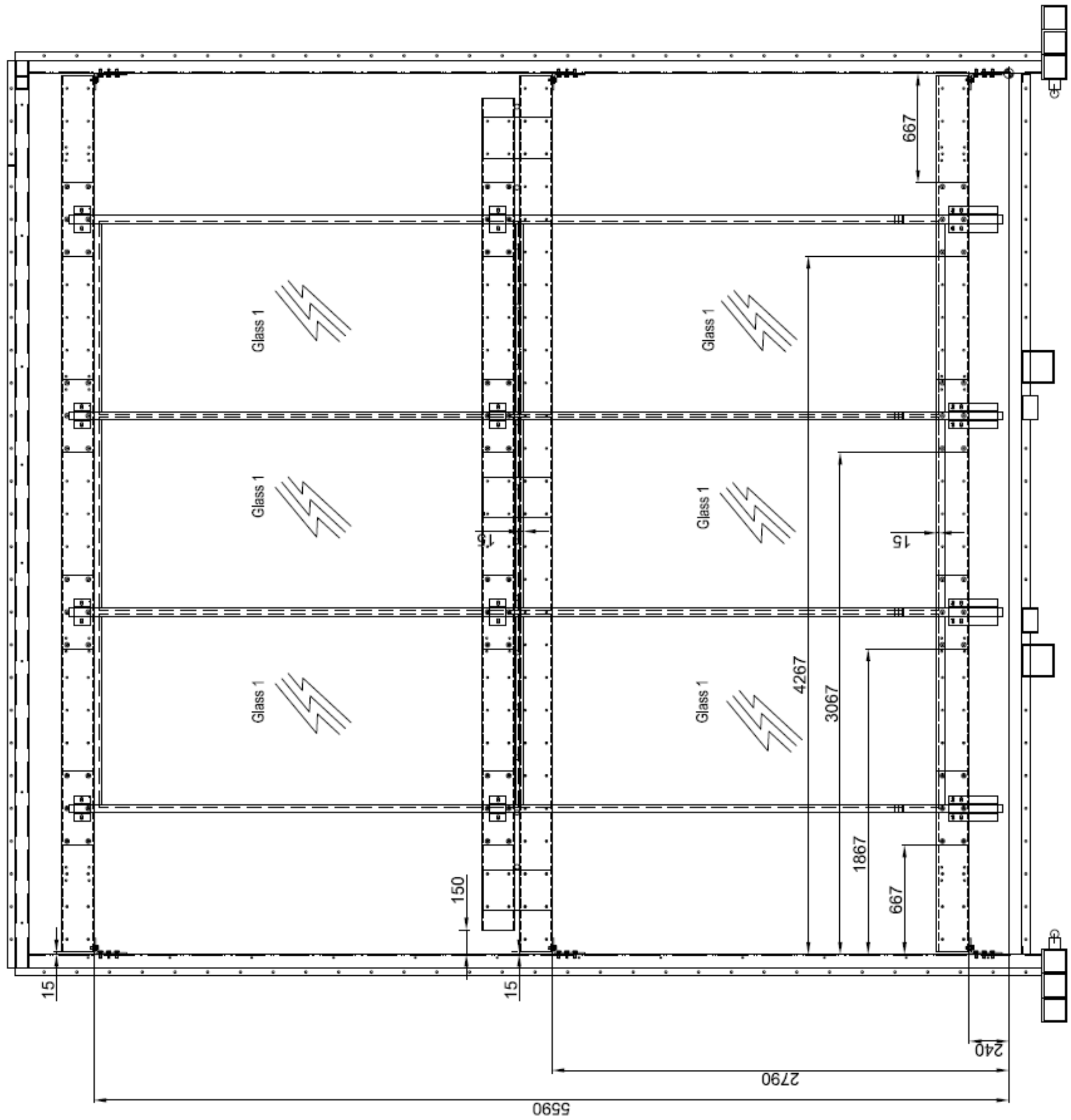

Figure 8-3: back view

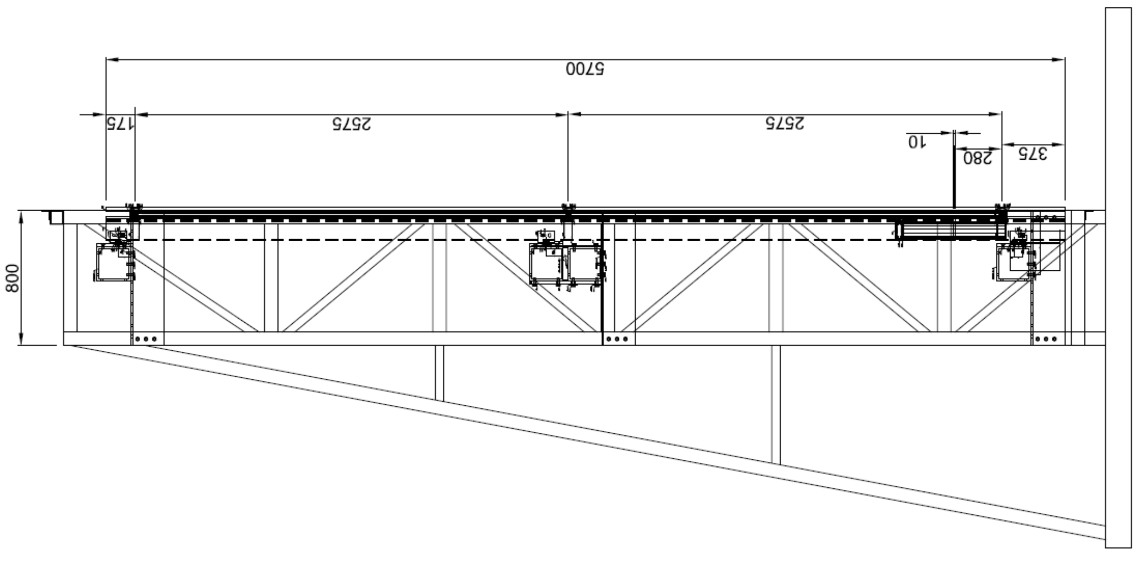

Figure 8-4: lateral view

Figure 8-5: picture of back view

Figure 8-6: picture of frontal view

The horizontal and vertical sections of the profiles (**Figure 8-7: horizontal section of mullions** and **Figure 8-8: vertical section of transoms**) show the gap between glasses and aluminium frame.

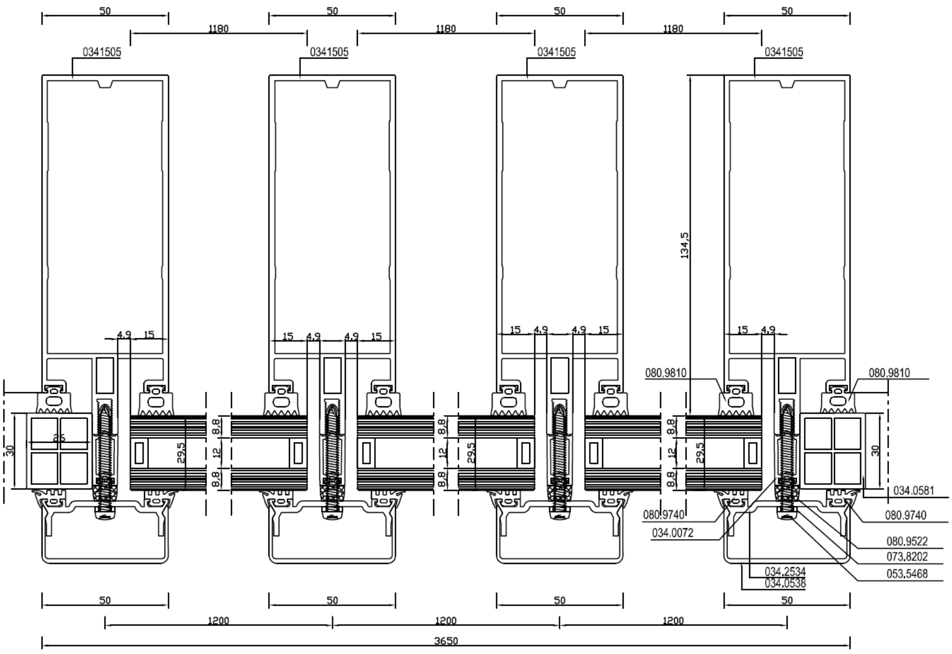

Figure 8-7: horizontal section of mullions

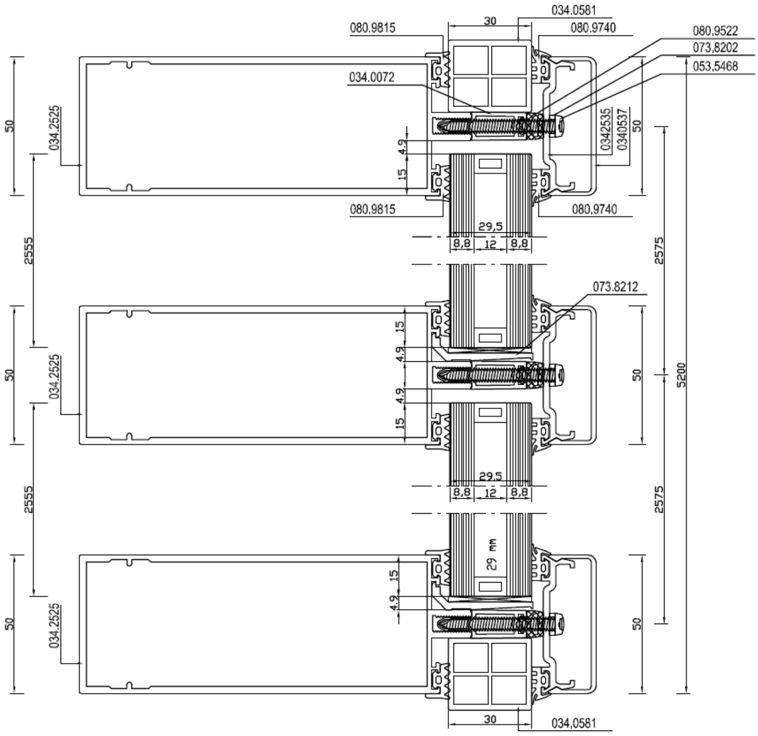

Figure 8-8: vertical section of transoms

Here are details of the anchorage to the steel structure, at the top (Figure 8-9), at the center (Figure 8-10, Figure 8-11), at the bottom where it's possible to see the horizontal joint for thermal dilatation (**Figure 8-12, Figure 8-13, Figure 8-14**).

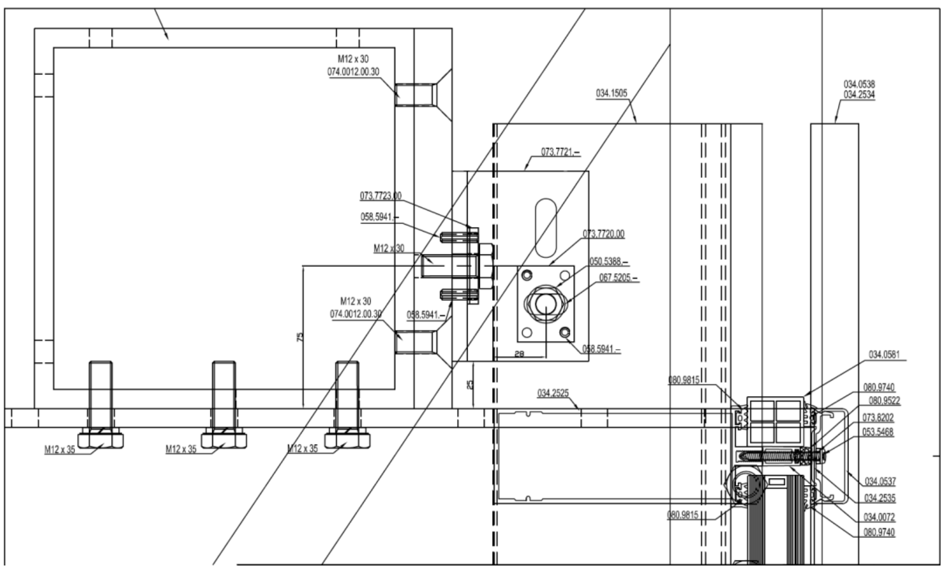

Figure 8-9: top anchorage to the steel structure (fixed anchor)

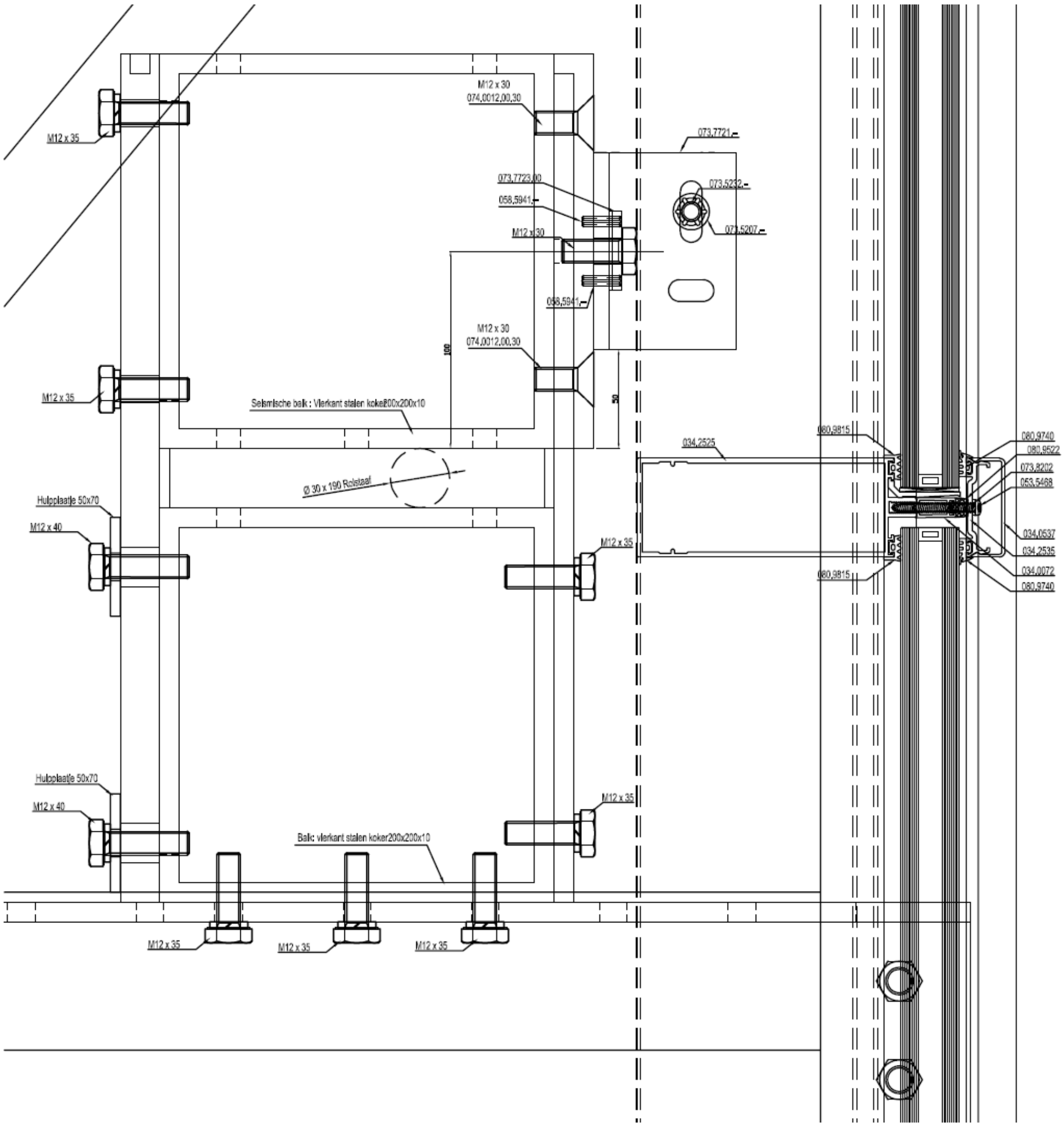

Figure 8-10: middle anchorage to the steel structure (loose anchor)

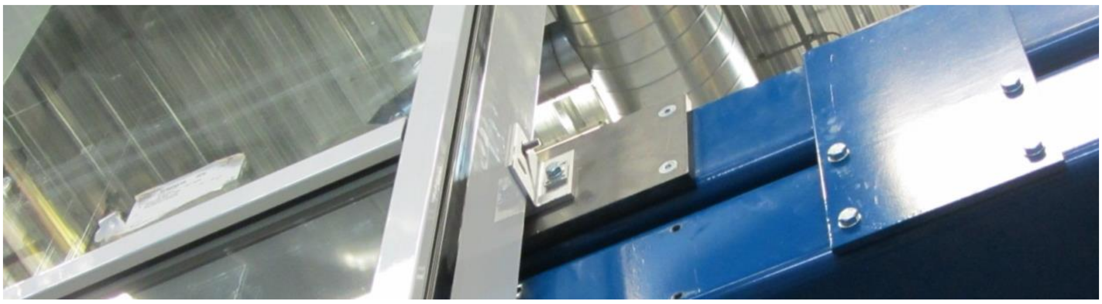

Figure 8-11: picture of a loose anchor

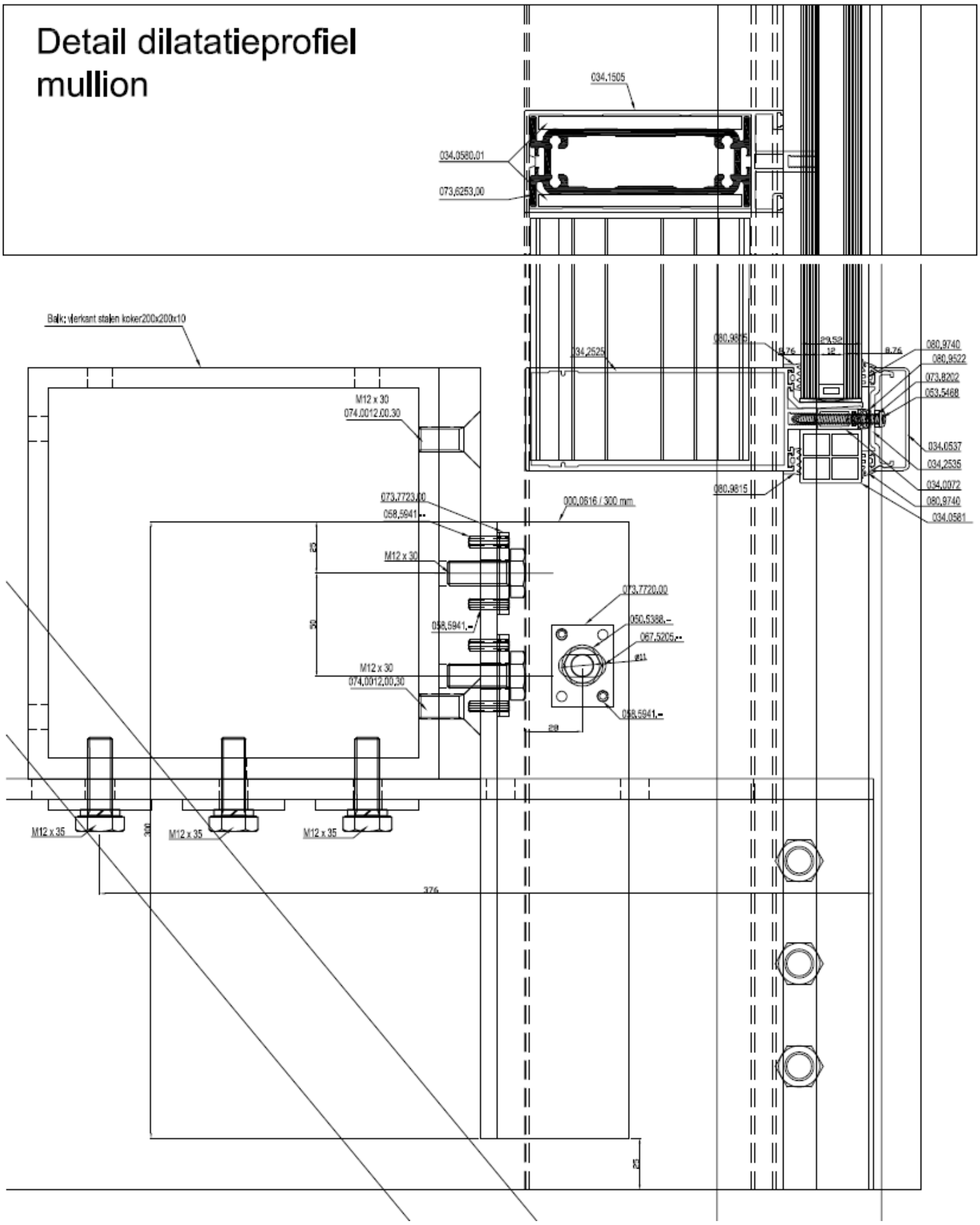

Figure 8-12: bottom anchorage to the steel structure (fixed anchor) with dilatation joint

Figure 8-13: : picture of a fixed anchor

Figure 8-14: : picture of a dilatation joint

The mechanism used to impose design displacements is composed by a motor with a hydraulic pump connected to a hydraulic cylinder and a displacement measuring device (**Figure 8-17, Figure 8-16, Figure 8-15**).

Figure 8-17: picture of the system of displacement

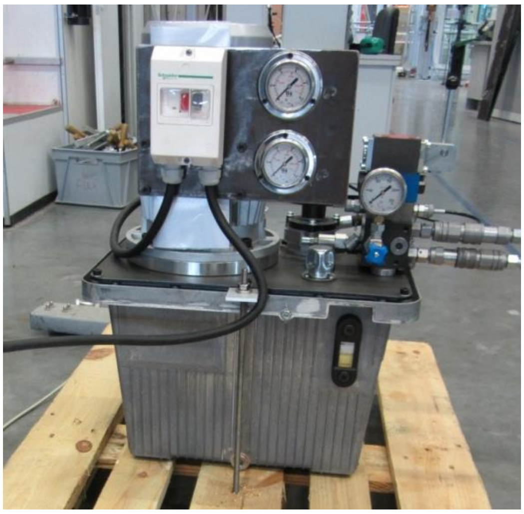

Figure 8-16: picture of the motor with the hydraulic pump

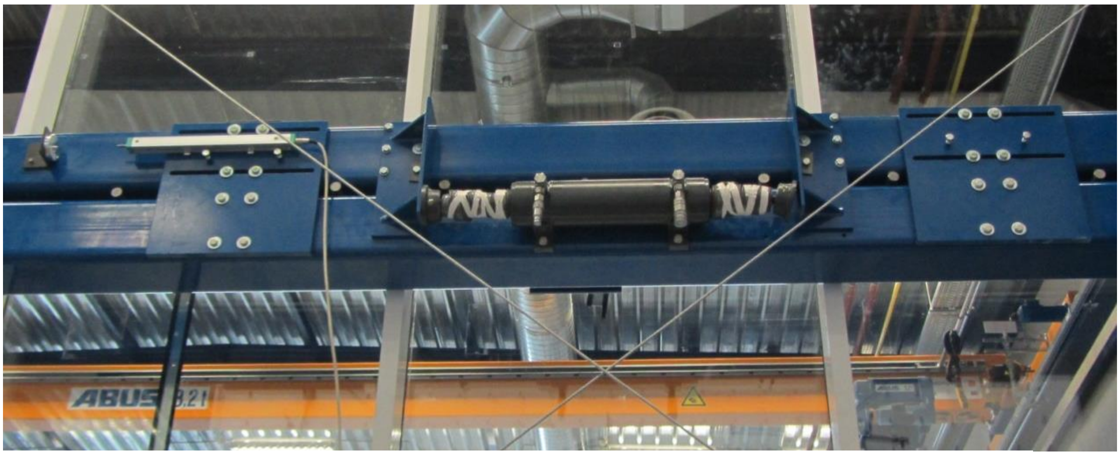

Figure 8-15: picture of the hydraulic cylinder with the displacement measuring device

8.1.2 Theoretical results

Before performing the test, the drift capacity of CW50 has been calculated, both for the designed model and the real one, through the formula found by Sucuoglu and Vallabhan, which gives the total lateral deformation of the window panel due to rigid body motion of the glass panel in the window frame:

$$\Delta = 2c\,(1 + h/b)$$

where Δ is the lateral drift capacity of the glass frame and c, h and b are physical dimensions as defined in the figure above.

For the designed model the lateral drift capacity is:

Clearance between vertical glass edges and frame	c	4.9	mm
Smallest width of the rectangular glass panel	b	1180	mm
Height of the glass panel	h	2555	mm
Minimal drift capacity glass panel	$\boldsymbol{\Delta_{cap}}$	**31.0**	**mm**

Table 8-1: lateral drift capacity for CW50

As in the real model, clearance between vertical or horizontal glass edges and frame was uneven, a modified formula has been used:

$$\Delta = 2c_1\,(1 + h_p c_2 / b_p c_1)$$

where: h_p = height of the rectangular glass panel, b_p = width of the rectangular glass panel, c_1 = clearance between the vertical glass edges and the frame, and c_2 = clearance between the horizontal glass edges and the frame.

For the real model the range of drift capacity for each panel is sum up in the following table:

	Minimum (mm)	Maximum (mm)	Average (mm)
Glass1	**24**	**37**	30.5
Glass2	**21**	**40**	30.5
Glass3	**19**	**43**	31
Glass4	**23**	**40**	31.5
Glass5	**22**	**36**	29
Glass6	**26**	**41**	33.5

Table 8-2: range of drift capacity of the real model for CW50

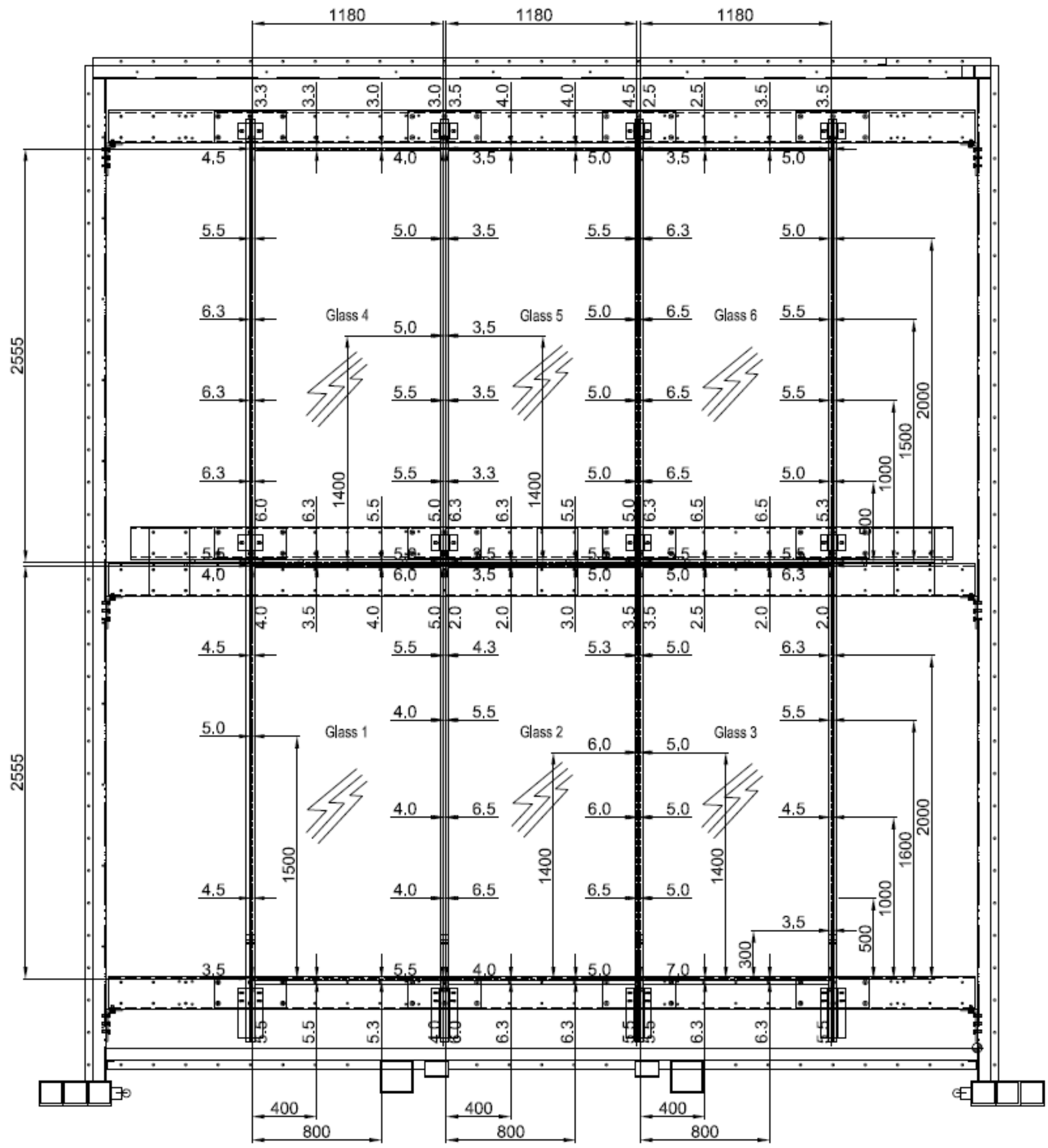

Figure 8-18: scheme of different clearances between glass edges and the frame

8.1.3 The test

In order to fulfil the American Standard, elements representing the primary building structure are displaced to produce the specified movements. Each test consists of 3 full cycles, i.e. a full displacement in one direction, back to the originating point, full displacement in the opposite direction, and back to the originating point. Being the specimen height equal to two full building stories, the design displacement is imposed at the center, while the base and the top stay fixed. After each 3 cycles of one specific horizontal drift, a visual inspection of the mock-up for evidences of failure takes place. Everything is recorded through pictures and videos (one for the global structure and one for the central corners).

By analyzing the calculated drift capacity it has been decided to start with a 15 mm and to proceed by incrementing it of 10 mm each time. Looking at the video it is possible to notice that, during the first cycle, the displacement in the opposite direction has gone further than 15 mm (approximately 100 mm). The result is that some problems already occurred at the first test, such as a start of glazing failure and frame deformation with screws coming out in the lower part. According to Standard, since neither the true displacement nor the causes of the start of glass failure are determined, the glass must be replaced and the test repeated. Anyway it has been decided to continue the tests to see how much drift capacity the system can have.

The following displacements have been 30 mm, 40 mm, 50 mm until 60 mm when the visual distress was so evident to lead to the decision to stop testing further, in fact besides the glasses also transoms, screws and anchorages have been damaged.

To repeat the test the entire curtain wall must be remade. After the dismounting all failures occurred are clearer.

Here is a table which shows the remarks for every drift of the test sequence.

Drift (mm)	Remarks	Pictures of reference
15 (and >15)	Glass3 start of breakage, frame deformation and screws coming out in the connection transom-mullion under glass3	Figure 8-19: deformed frame and start of glass 3 breakage (15 mm), Figure 8-20: screw coming out under glass 3 (15 mm)
30	---	
40	It was possible to hear some crackling noise	
50	More crackling noise	
60	Even more crackling noise, corner or edge “exfoliation” of all lower glazing, distorted transoms (out-of-plane rotation), screws deformation and coming out. Neither glass falling out nor breakage. After dismounting it was remarkable that the same facts also occurred to the upper panels and transoms, and that the middle anchorages to the structure were distorted too.	Figure 8-21: edge "exfoliation" in glass 1 (60 mm), Figure 8-22: out-of-plane rotation of transoms (60 mm), Figure 8-23: deformed screw (after dismounting), Figure 8-24: distorted transoms (after dismounting), Figure 8-25: screw coming out in connection transom-mullion (after dismounting), Figure 8-26: fragmented glass in the corner (after dismounting), Figure 8-27: deformed anchor to the structure in the middle (after dismounting), Figure 8-28: deformed anchor to the structure (after dismounting)

8.1.4 Conclusions

The system is still safe even after a 60 mm displacement, while its serviceability is supposed to be ended up at 30 mm. Since the first displacement has been of about 100 mm, another, more careful test must be repeated to have more precise information

The falling off of some glass fragments at the edges and corners is due to the contact between glass and its aluminium support: in most cases, except for the right and left bottom panels, glass won’t be replaced because these facts are not remarkable due to the presence of the cap.

The most damaged elements are transoms in their connection to mullions through screws while mullions don’t seem to be damaged at all.

Distortion in transoms would probably have been lower if there had been continuous pressure plates instead of interrupted in corners ones.

8.1.5 Pictures

Figure 8-19: deformed frame and start of glass 3 breakage (15 mm)

Figure 8-20: screw coming out under glass 3 (15 mm)

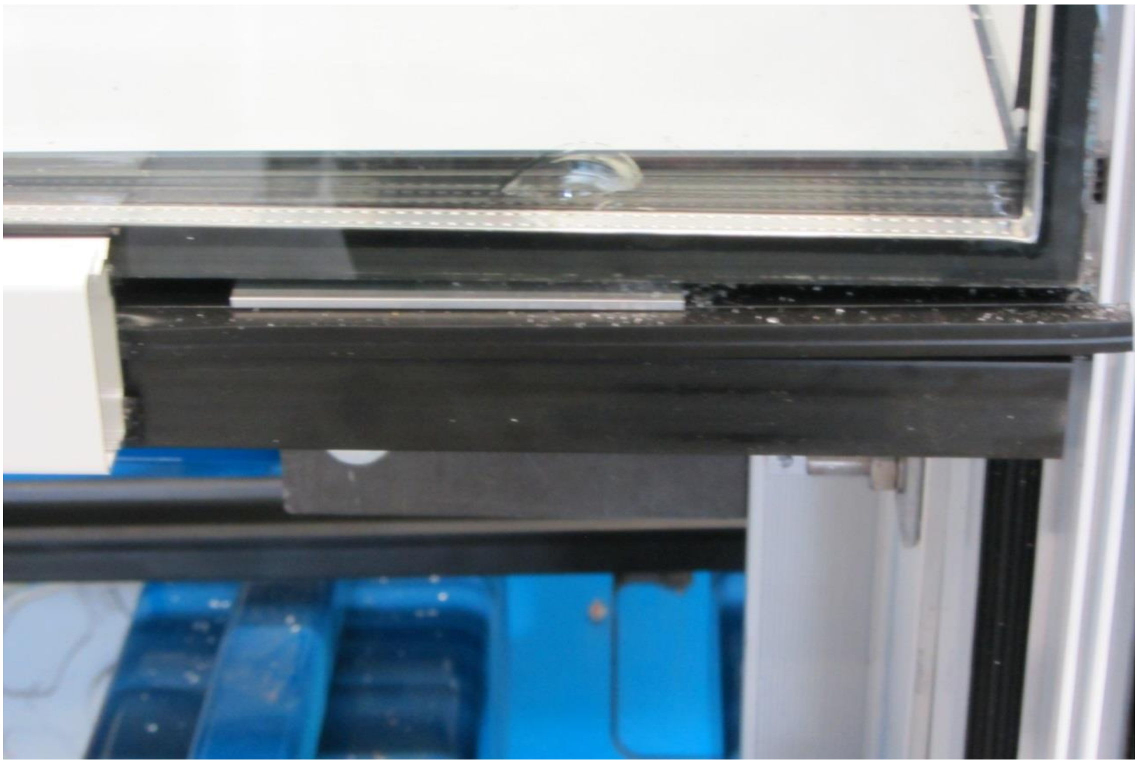

Figure 8-21: edge "exfoliation" in glass 1 (60 mm)

Figure 8-22: out-of-plane rotation of transoms (60 mm)

Figure 8-26: fragmented glass in the corner (after dismounting)

Figure 8-25: screw coming out in connection transom-mullion (after dismounting)

Figure 8-24: distorted transoms (after dismounting)

Figure 8-23: deformed screw (after dismounting)

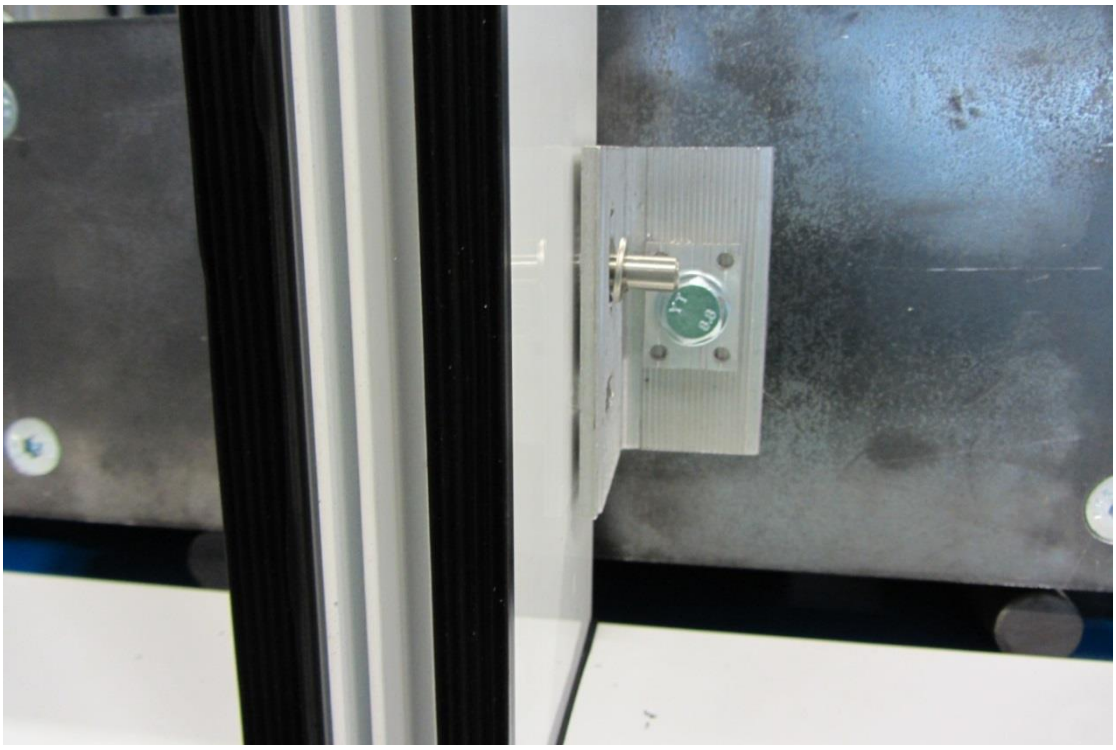

Figure 8-27: deformed anchor to the structure in the middle (after dismounting)

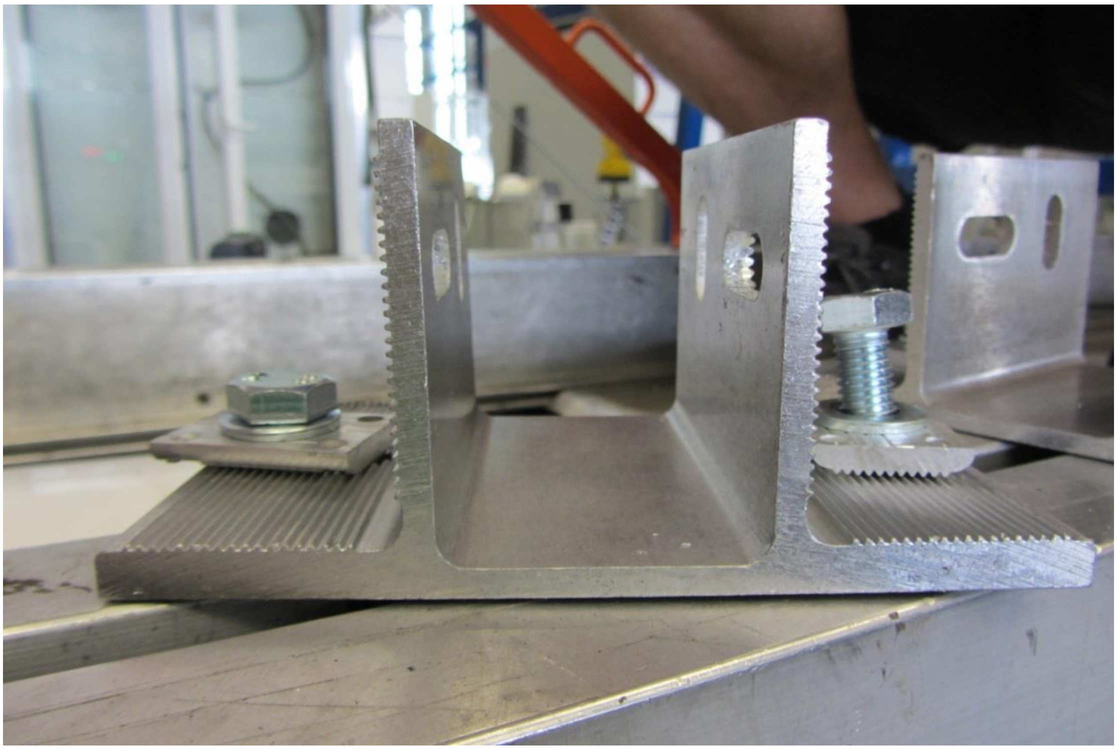

Figure 8-28: deformed anchor to the structure (after dismounting)

8.2 CW60 static test

8.2.1 Description of the mock-up

For the CW60 system too, the specimen used is made by 4 mullions and 9 transoms in order to obtain 6 glazed units as it's shown in the following scheme.

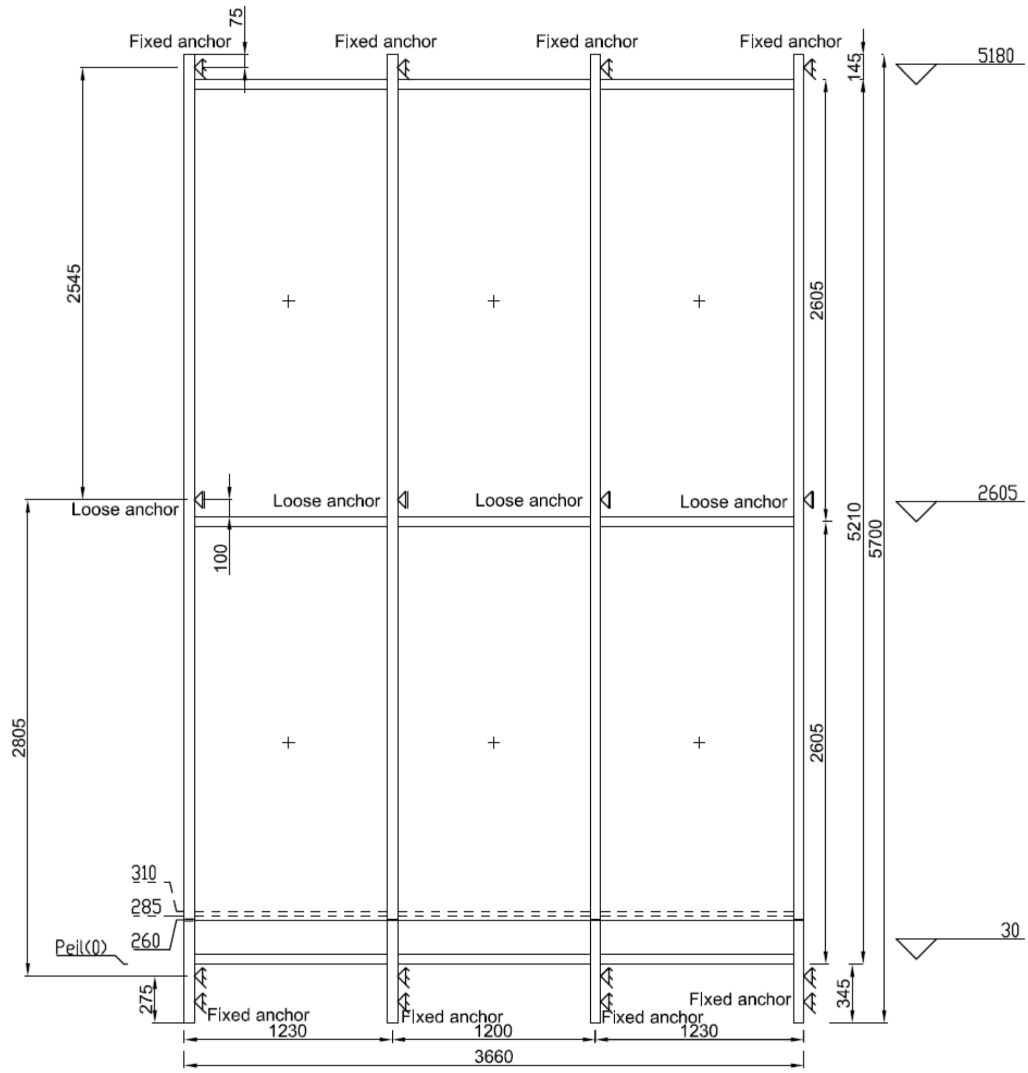

Figure 8-29: installation scheme

As for the CW50, at the bottom of the specimen there is a dilatation joint, the bottom anchors are longer than the others to simulate the curtain wall continuity and no glass panels are present under the lower transoms and above the top transoms.

The specimen is connected to the structure by fix anchors (taking wind and weight load) on the top and on the bottom and by loose anchors (taking only wind load) at the middle.

The structure is made by 4 steel tubes: one is on the top, one is on the bottom and two are in the middle for simulating the interstorey drift by a reciprocal movement.

The displacement is made possible by the same system, based on a manually controlled hydraulic cylinder.

Since in the CW50 case the excessive distortion in transoms had been attributed to the interruption of pressure plates in corners, it has been decided to let them uncovered only for 50 mm on each side, just to allow the visualization in those points.

The following figures show the whole structure.

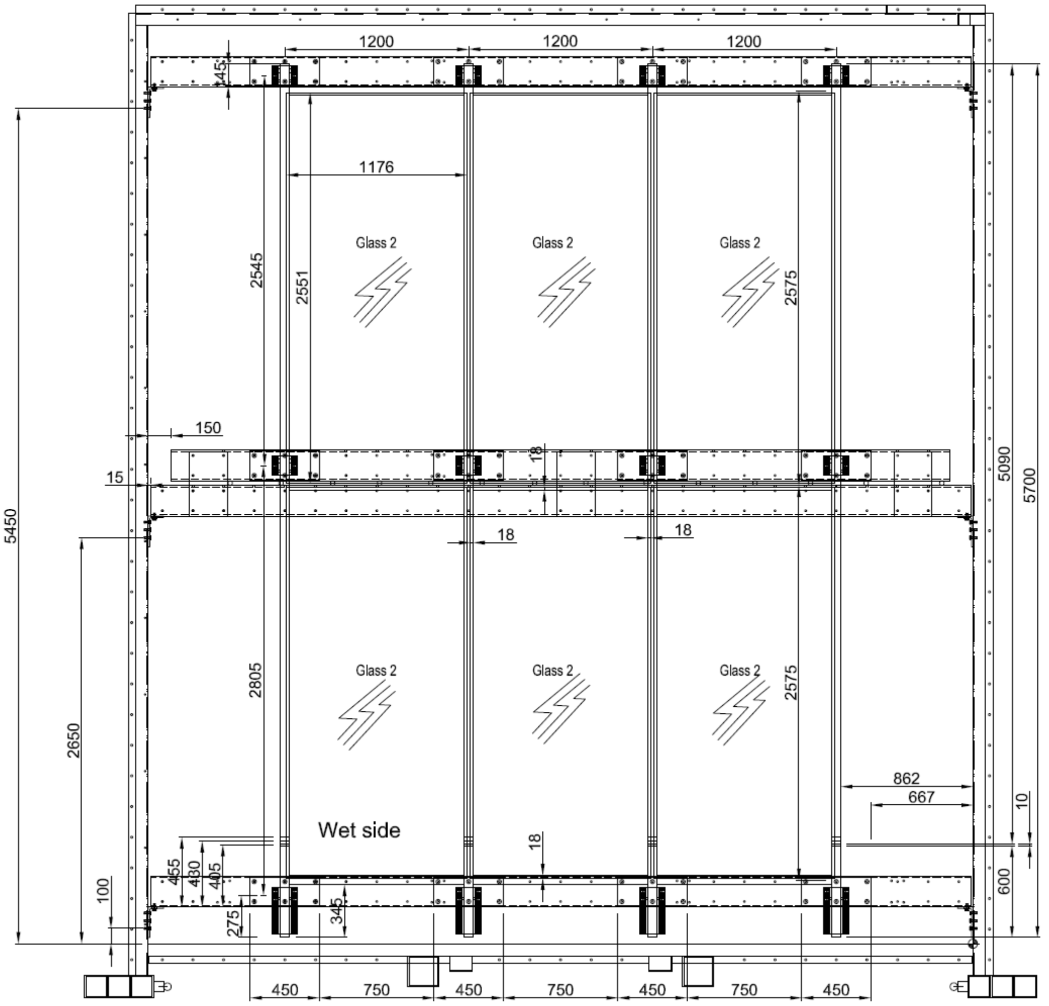

Figure 8-30: frontal view

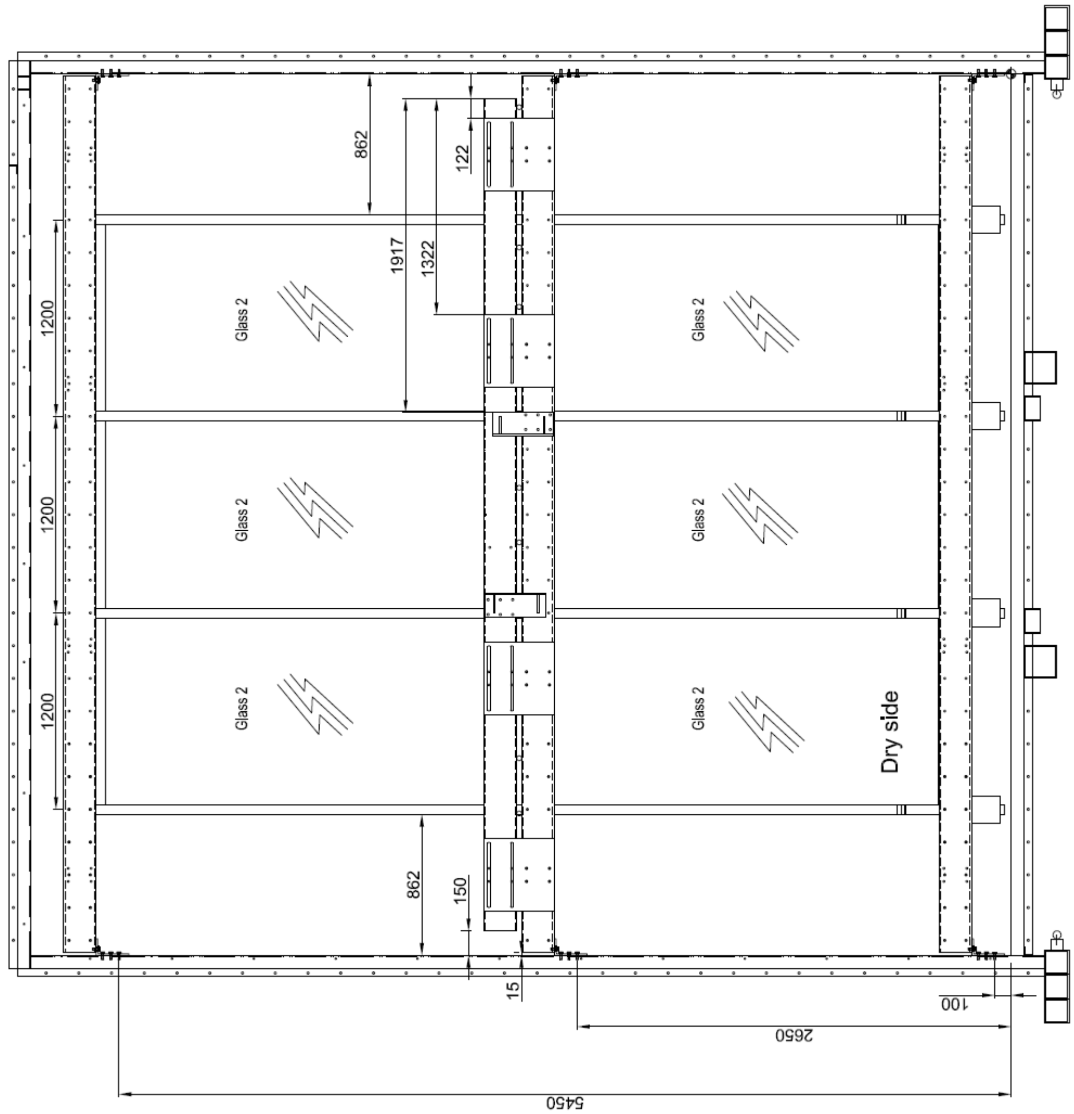

Figure 8-32: back view

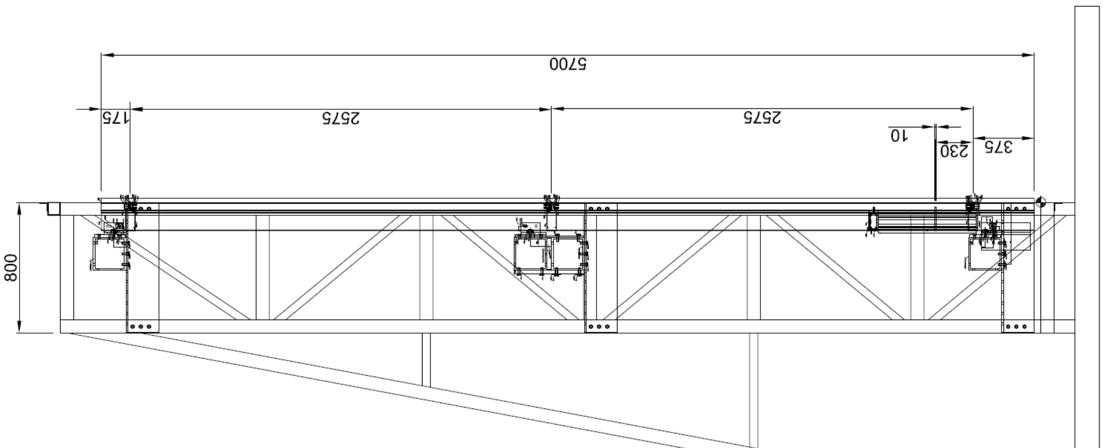

Figure 8-31: lateral view

Figure 8-33: picture of frontal view

Figure 8-34: picture of back view

The horizontal and vertical sections of the profiles (**Figure 8-35** and **Figure 8-36**) show the gap between glasses and aluminium frame.

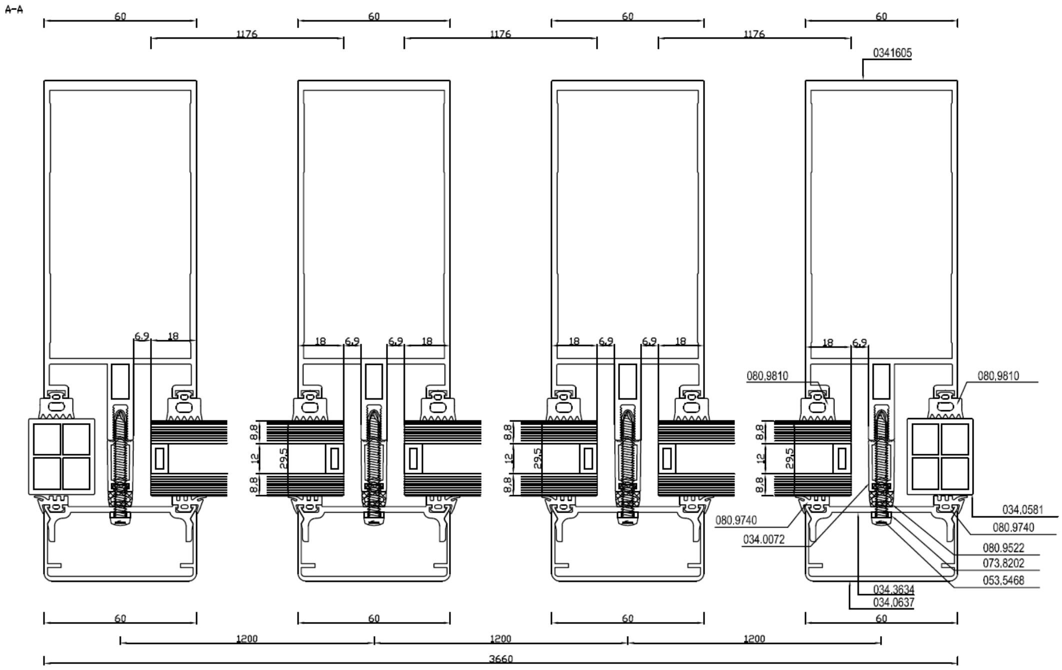

Figure 8-35: vertical section of transoms

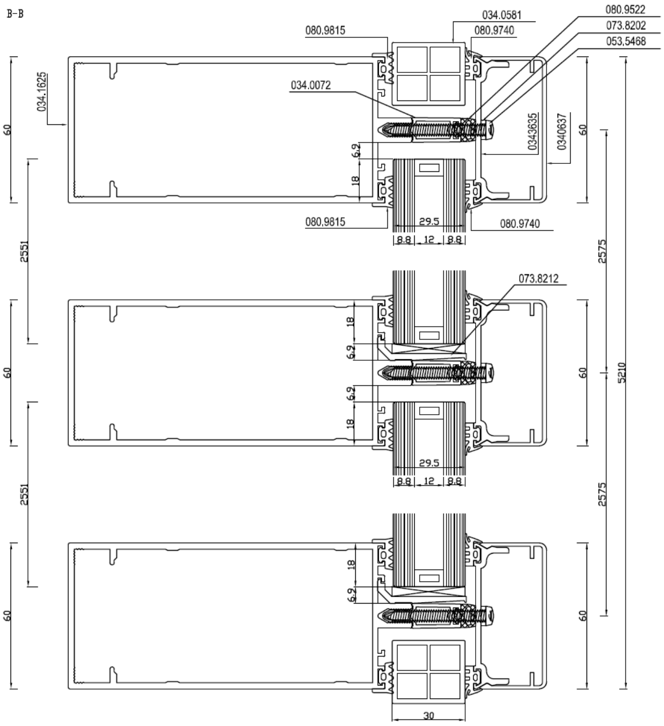

Figure 8-36: horizontal section of mullions

Details of the anchorage at the top, at the center and at the bottom, where it's possible to see the horizontal joint for thermal dilatation, are quite similar to the CW50 ones.

8.2.2 Theoretical results

Before performing the test, the drift capacity of CW60 has been calculated, both for the designed model and the real one.

For the designed model the lateral drift capacity has been calculated by using Sucuoglu and Vallabhan formula, already used:

$\Delta = 2c\,(1 + h/b)$.

The result is:

Clearance between vertical glass edges and frame	c	6.9	mm
Smallest width of the rectangular glass panel	b	1176	mm
Height of the glass panel	h	2551	mm
Minimal drift capacity glass panel	Δ_{cap}	**43.7**	**mm**

Table 8-3: lateral drift capacity for CW60

As in the real model, clearance between vertical or horizontal glass edges and frame was uneven, the modified formula has been used:

$\Delta = 2c_1\,(1 + h_p c_2 / b_p c_1)$.

For the real model the range of drift capacity for each panel is sum up in the following table:

	Minimum (mm)	Maximum (mm)	Average (mm)
Glass1	**39.5**	**50.6**	45.1
Glass2	**33**	**47.5**	40.3
Glass3	**39.5**	**48.1**	43.8
Glass4	**29.9**	**50.3**	40.1
Glass5	**34.9**	**50.7**	42.8
Glass6	**34.3**	**45.9**	40.1

Table 8-4: range of drift capacity of the real model for CW60

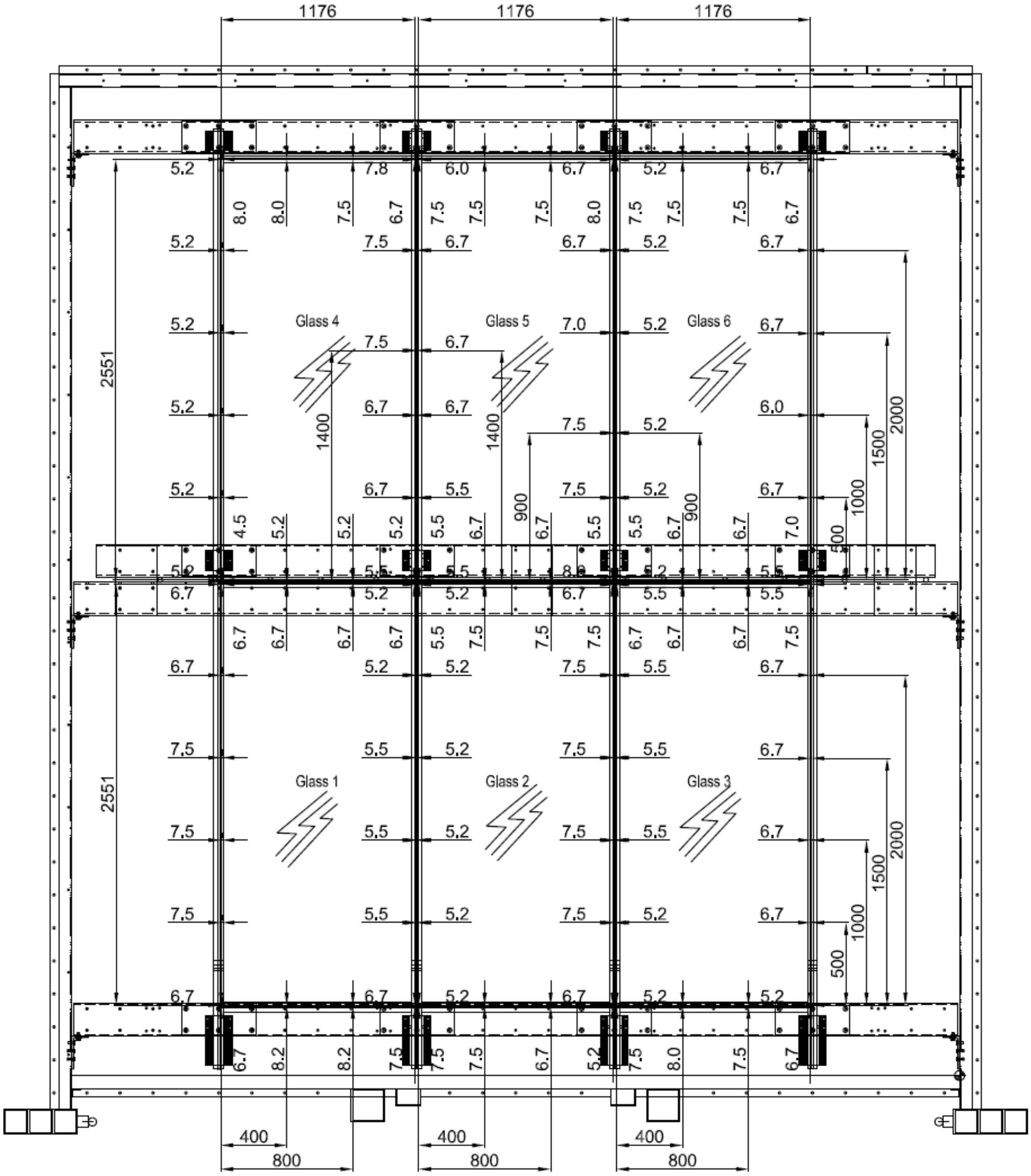

Figure 8-37: scheme of different clearances between glass edges and the frame

8.2.3 The test

The test procedure is the same described for the CW50 system.

Everything has been recorded through pictures and videos (one for the global structure, one for the central corners and one for the bottom corners).

By analyzing the calculated drift capacity it has been decided to start with a 20 mm displacement and to proceed by incrementing it of 5 mm each time.

No problems have been noticed, so the test has been continued imposing the following displacements: 25 mm, 30 mm, 35 mm, 40 mm, 45 mm, 50 mm until 55 mm.

- At 30 mm the clearance between the vertical glass side and the aluminium in the right bottom corner of the central bottom glazing has decreased but it stabilized during the following displacements;
- at 40 mm glasses have started to vibrate;
- at 45 mm the gasket between the pressure plate on the right mullion and the central low glass has started to slide down (Figure 7-43) and it has been realized that the glass vibration is due to a bending of the steel structure;
- at 55 mm, even though there were no problems with the curtain wall, the test had to be stopped because of excessive bending of the steel structure.

The curtain wall can be still used for continuing the test but the steel structure has to be fixed up to prevent more problems.

Figure 8-38: gasket slide down

8.2.4 Conclusions

No visual distress has been noticed so we can tell that the system is still safe and probably serviceable even after a 55 mm, which is a value higher than the expected one. The test can be continued after the fixation of the steel structure to know the exact limit of serviceability and safety.

8.2.5 Repeating the tests

Since the tests were not valid, later they were both repeated. In the new mock-up two building storeys have been created with each a height of 2.6 m. The steel structure was reinforced with steel cables held by manual winches as shown in Figure 7-45. Four corners of the middle bottom glass are left open on the outside, over a distance of 100 mm from the center of the mullion, to be able to investigate the movement of the glass during the test.

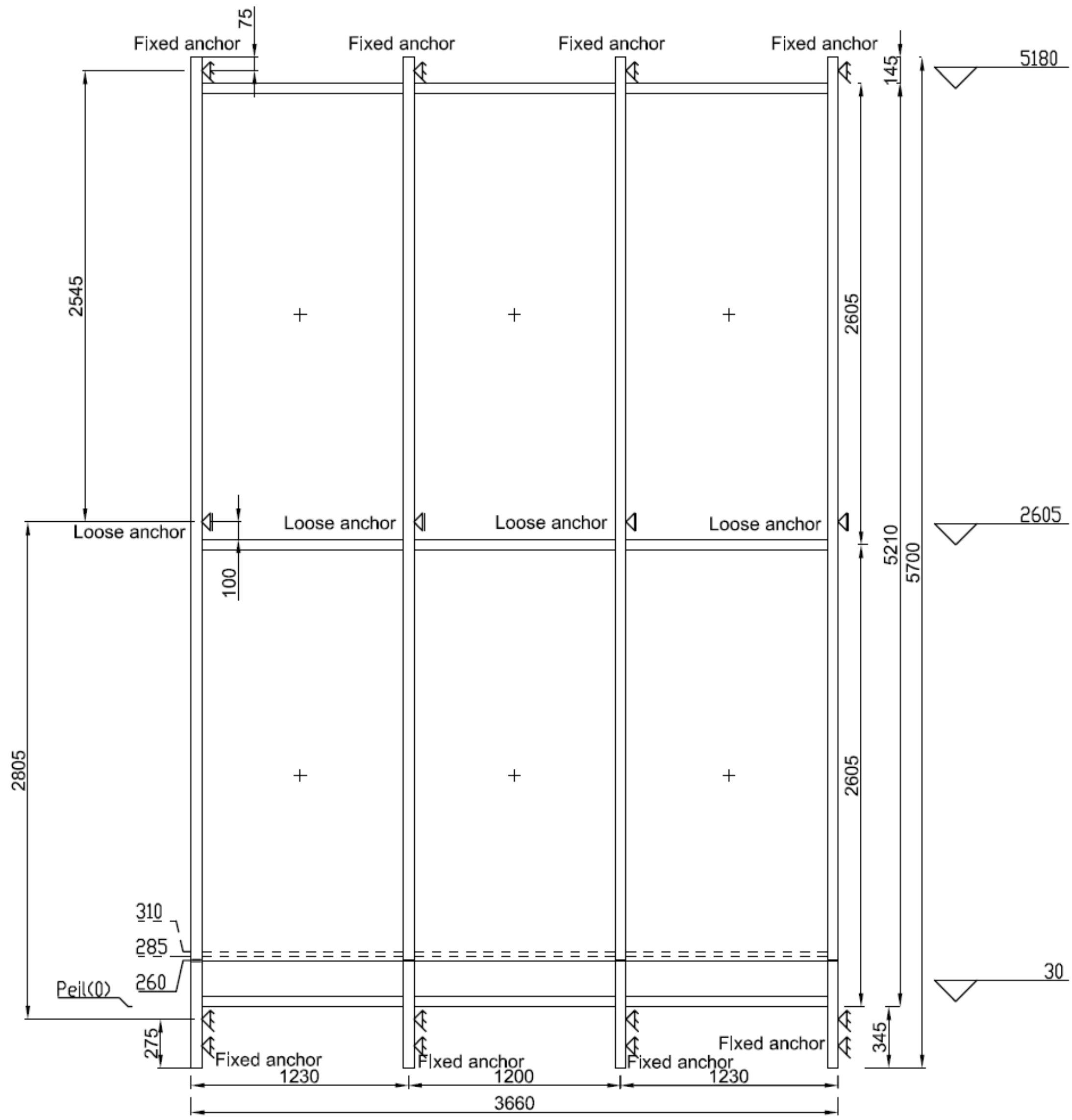

Figure 8-39: Test mock-up

Figure 8-40: Picture of front view with the new double x reinforcement

8.2.6 Test results and conclusions

The test method was exactly the same. The tests were started at the drift of 20 mm with an increase of 5 mm each time. After each 3 cycles of one specific horizontal drift, a visual inspection of the mock-up for evidences of failure takes place.

The results were very good for both CW50 and CW60 systems: the tested drift corresponding to first glass cracks and consequent loss of serviceability were superior to the theoretical calculated ones.

In CW50 system Δ_{test} =55 mm>Δ_{cap} =31 mm and in CW60 system Δ_{test} =90 mm>Δ_{cap} =43,7 mm**.**

Here are two tables describing what happened at every tested drift, the former concerning CW50 system and the latter concerning CW60 system.

CW50		
Tested drift (mm)	**Remarks/Observations**	**Acceptable**
20	/	Yes
25	Very slight rotation of bottom transoms Small displacement of left support block	Yes
30	More supporting blocks have shifted	Yes
35	One supporting block touches nose mullion	Yes
40	Again rotation of bottom transoms Making video of one corner is started	Yes
45	Crackling noise, but visual nothing specific seen	Yes
50	Often crackling noise More rotation of bottom transoms	Yes
55	Continuous crackling noise Serious rotation of bottom transoms	No
60	Bottom middle glass shows small crack	No
70	Enormous crackling noise of glass Bottom glass plates have lowered so much that they are barely clamped by the middle pressure plates	No

CW60		
Tested drift (mm)	**Remarks/Observations**	**Acceptable**
20	/	Yes
25	/	Yes
30	Clearance between vertical glass side and aluminium of the central bottom glazing has decreased but it stabilized during the next displacements	Yes
35	/	Yes
40	Glass is vibrating	Yes
45	Glass is vibrating due to bending of the steel structure One glazing gasket has slide down over a few centimeters	Yes
55	No problems with curtain wall element Test stopped due to excessive bending of steel structure	Yes
Reinforcement of steel structure		
60	/	Yes
65	/	Yes
70	/	Yes
75	Crackling noise No further remarks	Yes
80	/	Yes
85	/	Yes
90	/	Yes
95	Rotation of bottom transoms Almost no rotation of top and middle transoms Bottom right and top left glass plate show small crack	No

Section 4: Solutions to the problem

9 Case study: Isozaky Tower at CityLife

This case study wants to show how architects and engineers face the problem of the seismic behaviour of curtain walls when they design a new building.

The studied building is Isozaki Tower at CityLife, a residential and business district under construction in Milan, Italy.

Figure 9-1: render of Isozaki Tower at CityLife

Designed by Arata Isozaki and Andrea Maffei, the tower is 207 m tall and includes 50 floors of offices with an interstorey of 3,9 m each, placed on an open space entrance lobby on two levels.

The structure consists in two load-bearing cores at the sides in the internal space and some columns in the middle which give great space flexibility to each floor. At level 24 and at level 50 two belt beams, the first one in steel and the other one in reinforced concrete C60/70 with metal diagonals, are put to make the two load-bearing cores act together. Moreover four slanted struts, which characterize the architecture, go from the ground floor to level 11 and are connected directly to the core through the façade, help to support the tower and reducing, among other things, the bulk of the load-bearing structures in the internal space.

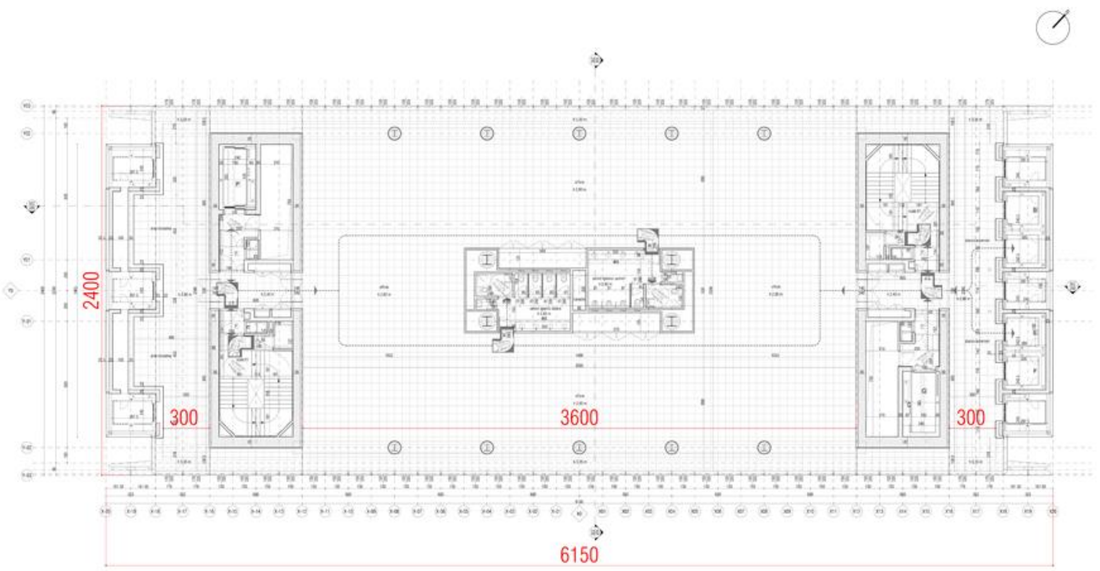

Figure 9-2: typical floor plan shows the main structure.

The building is characterized by a curved main façade, marked by 8 wavy sectors, each of 6 plans, clad with a double-glazed glass skin. The standard module is 1.5 m x 3.9 m. In the main façade two end panels cantilever are supported by steel beams. The side façades are partly glazed and display the structure of the panoramic elevators that lead to the various floors of the building.

In total there are 10 different façade types but only the main one is studied in this context, being the most relevant one in the whole building.

The aim of the case study is to show how the curtain wall is designed to respond to the interstorey displacements due to horizontal actions such as wind and seismic action resulting from the dynamic tests carried on by Colombo Costruzioni spa.

9.1 Report on the calculation of the displacements of the building in serviceability and earthquake conditions

Colombo Costruzioni has drawn up a report presenting the main results of the assessments conducted on the deformability of the structure to horizontal and vertical actions in significant points on the perimeter, in order to provide elements of assessment to the supplier of facade systems. Only the behaviour at horizontal actions are studied in this context.

The structural analysis and relative checks of safety measures are set with reference to the geometric characteristics, actions, properties of materials and to the sequences defined in the executive project.

Materials used in the structure are:

- C40/50 concrete for floors, walls, partitions and r.c. columns;
- C50/60 concrete for r.c. columns and composite steel-concrete columns
- C60/75 concrete for reinforced concrete belt beams
- C70/85 concrete for r.c. columns
- B450C steel bars for reinforced concrete
- S355 steel for metal columns and the two belt beams

In general terms, the actions have been combined to ultimate and serviceability limit states in accordance with the requirements set out in Chapter 2.5.3 and Section 3.2.4 of the DM 14/01/2008.

Structural analysis were carried out with the aid of software Midas Gen V. 2.1.

Structural analyzes were conducted with reference to computational models for finite element fully representative of the structure under examination, focusing on issues related to global structural response actions for instantaneous horizontal and vertical actions and long-term vertical actions. To examine the behavior of the building in relation to horizontal stiffness and occupant comfort, evaluations were carried out on three-dimensional models in which the elements are of linear-elastic type, consistent with the parameters specific to each material defined in the project. Seismic analysis, carried out within the linear dynamic structure factor equal to 2.88, provided the effects on belt beam elements module lower than those derived from static analysis to the Ultimate Limit State with wind action taken as dominant variable.

9.1.1 Dynamic features of the building

Here are the most significant computational geometry together with the results of dynamic modal analysis with which the characteristics of the building have been identified for the calculation of the wind action.

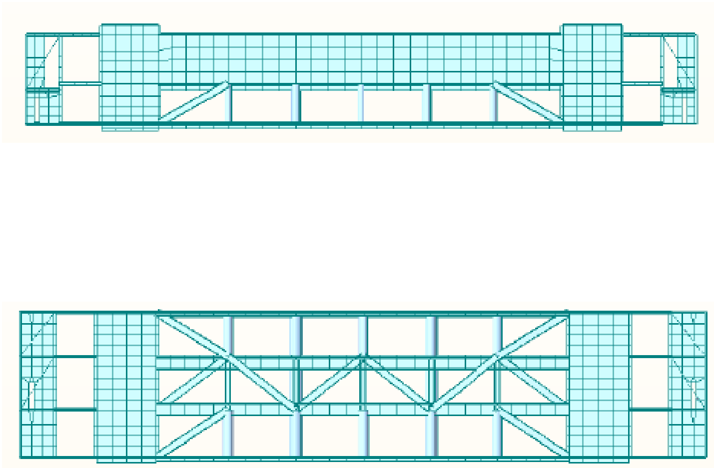

Figure 9-3: detail of the central steel belt beam and top reinforced concrete belt beam

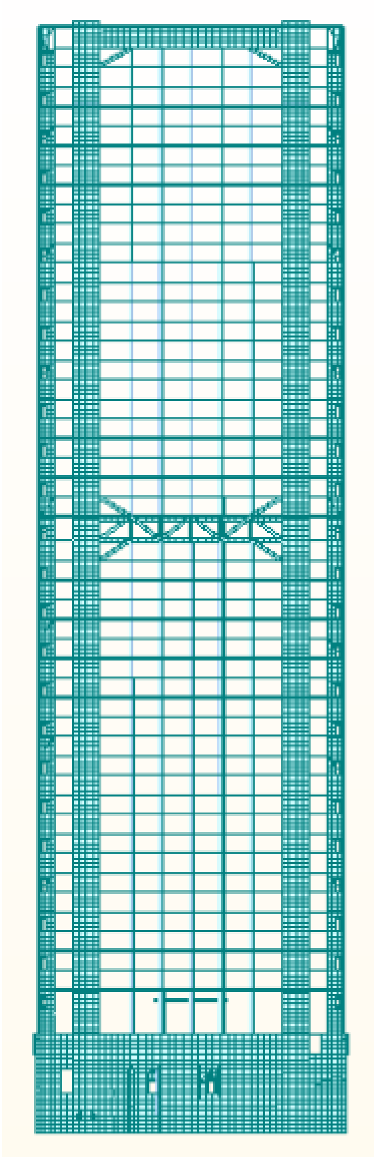

Figure 9-4: section of the tower - global structure

The numeric results of the modal analysis are separatly shown in Appendix A.

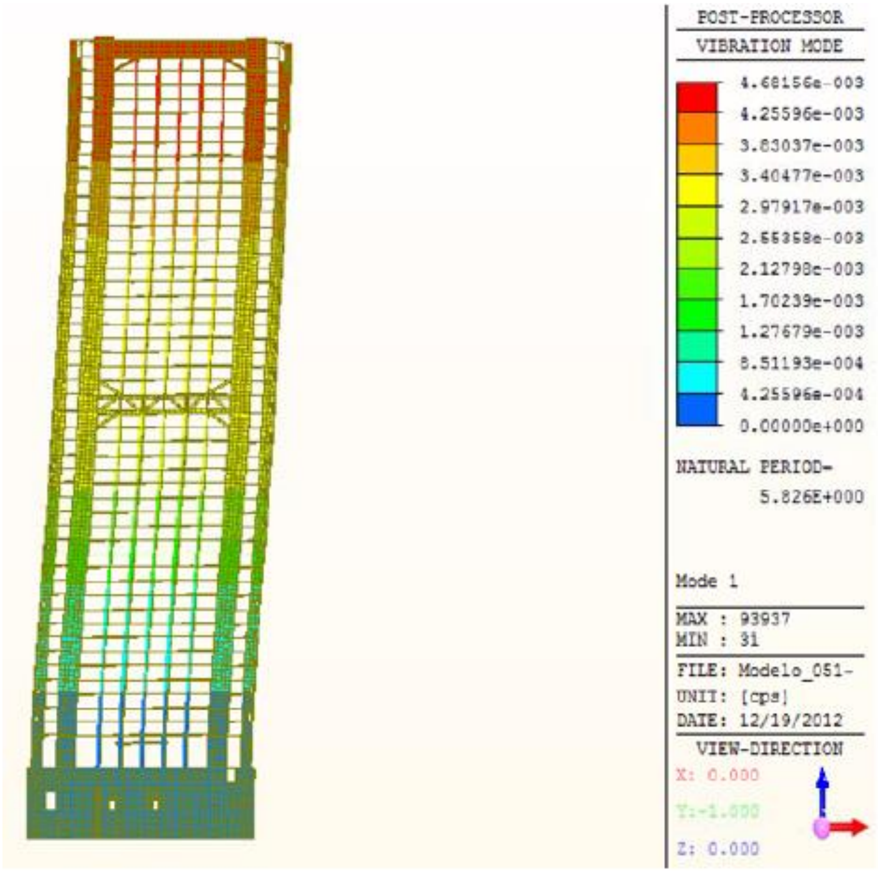

Figure 9-6: Vibration mode 1

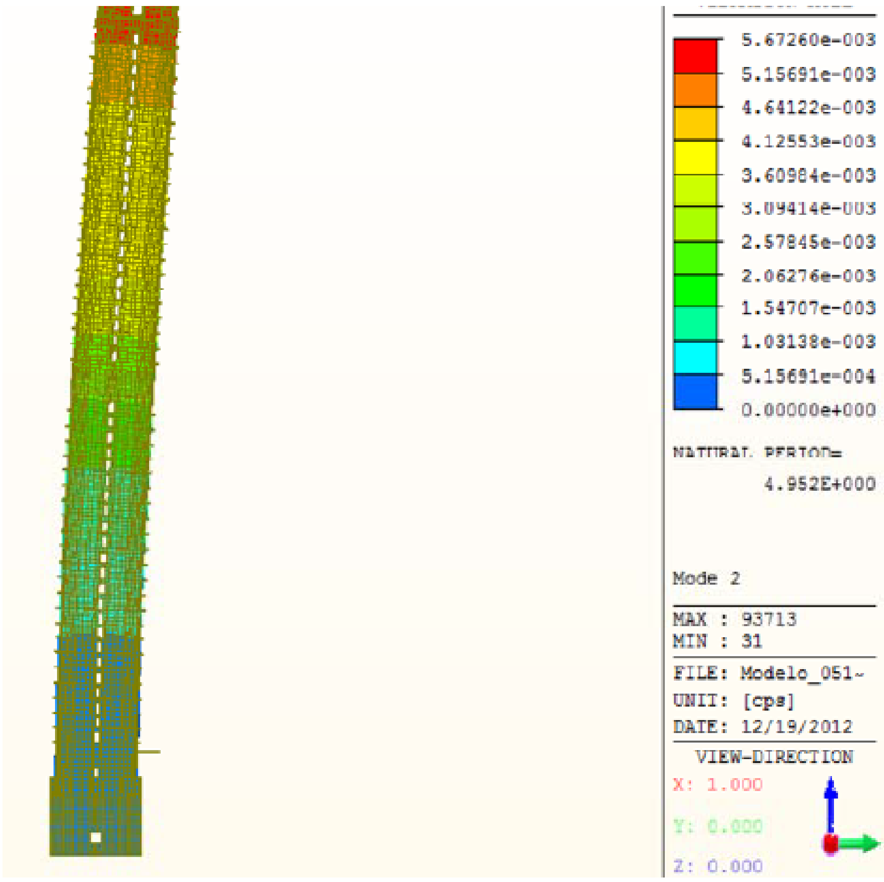

Figure 9-5: Vibration mode 2

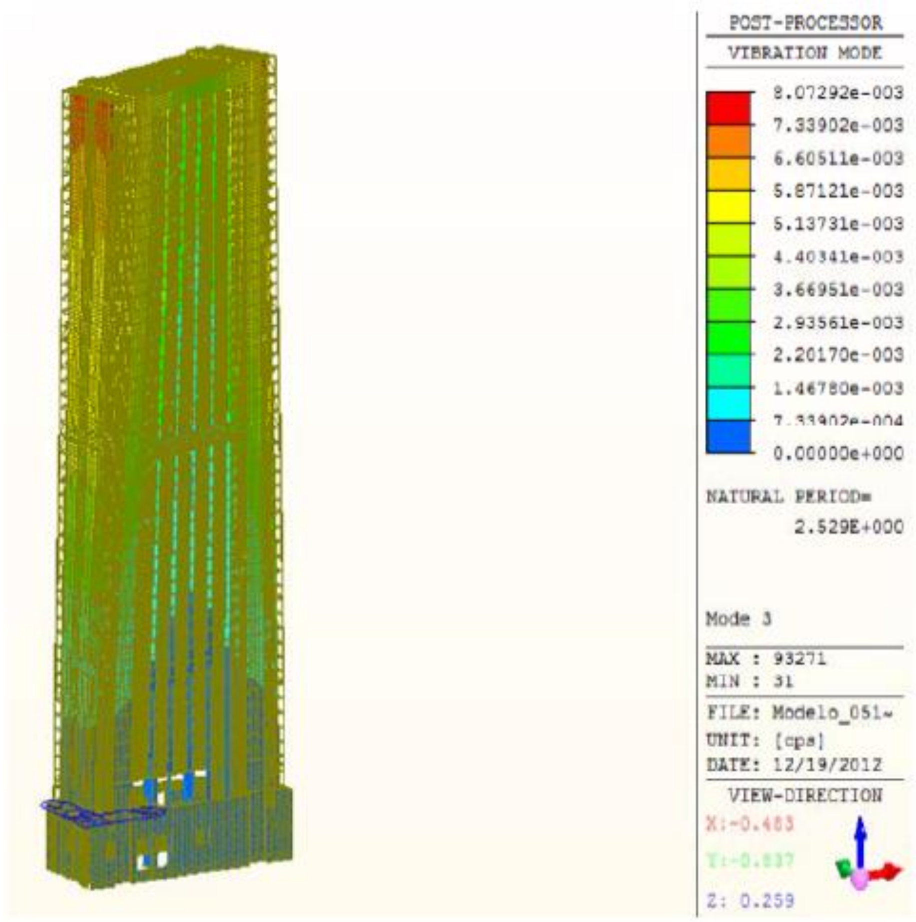

Figure 9-7: Vibration mode 3

9.1.2 Lateral drifts caused by horizontal actions

Following are reported the maximum interstorey displacements of points placed in established positions. Since only the main façades are studied relative to the in-plane displacements, only the drifts in direction x are analysed. The following figure shows the read points located on the boundary for each floor of the building. The AMSL is referred to level P0 (+128.80).

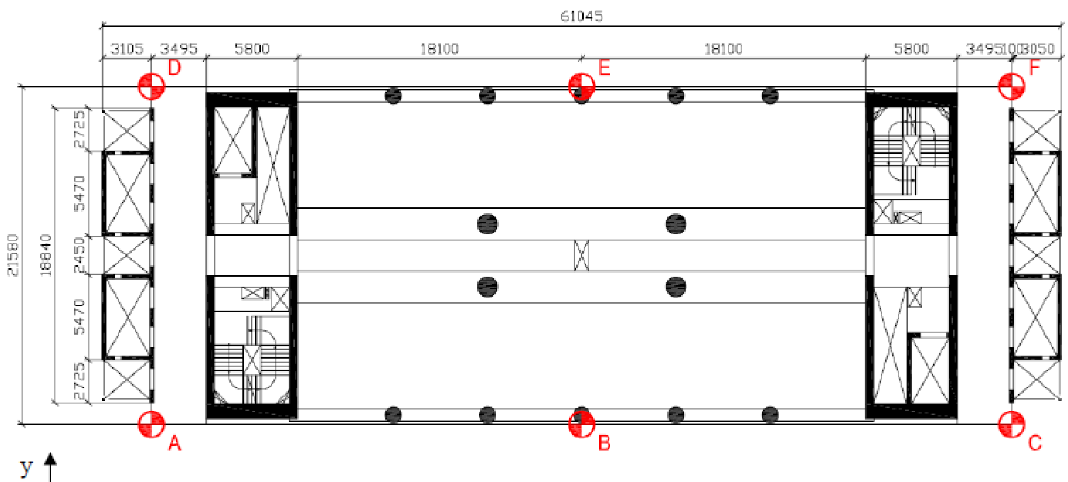

Figure 9-8: read points location on the boundary for each floor

Lateral displacements induced by the characteristic combination of wind load with return time of 50 years old and wind actions defined according to the CNR-DT 207/2008.

Point A

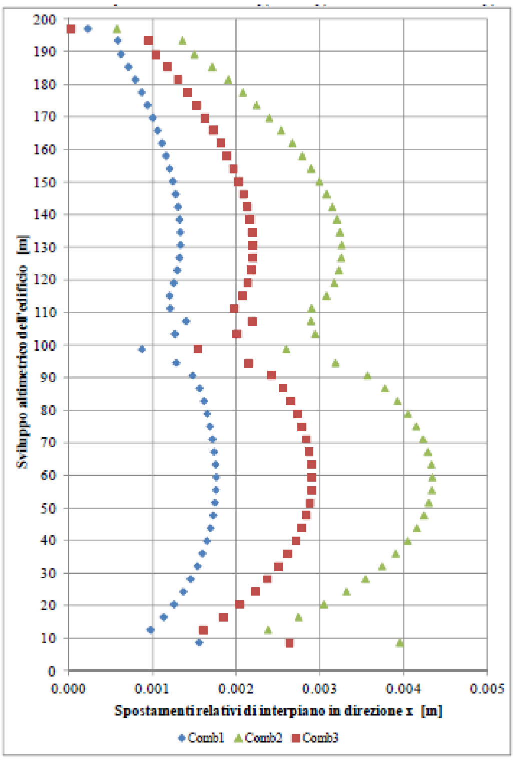

Figure 9-12: Point A - Drag component orthogonal to long side

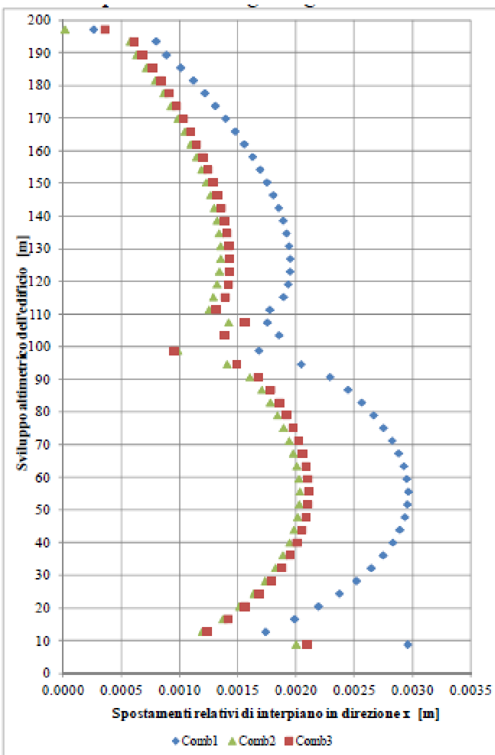

Figure 9-9: Point A - Drag component orthogonal to short side

Point B

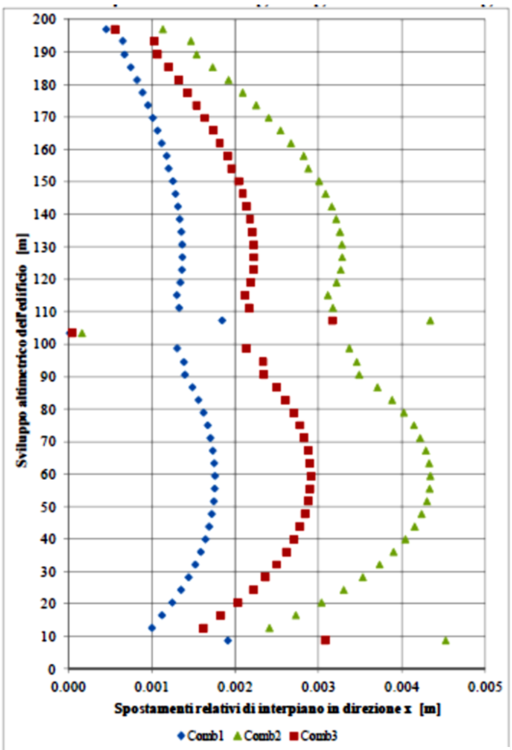

Figure 9-10: Point B - Drag component orthogonal to long side

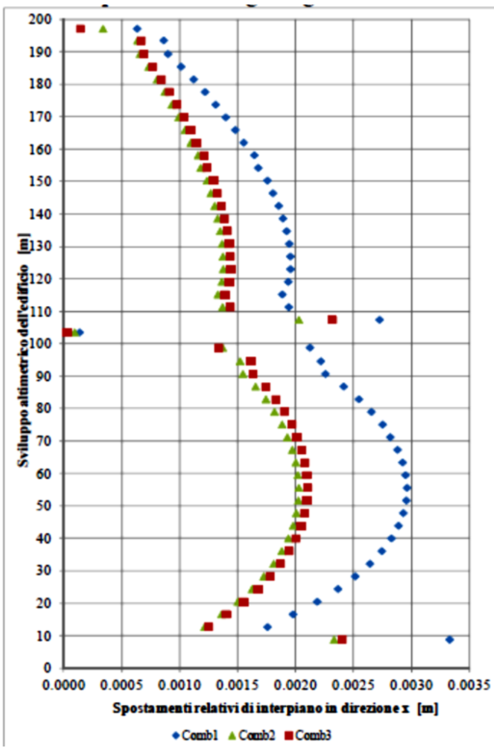

Figure 9-11: Point B - Drag component orthogonal to short side

Point C

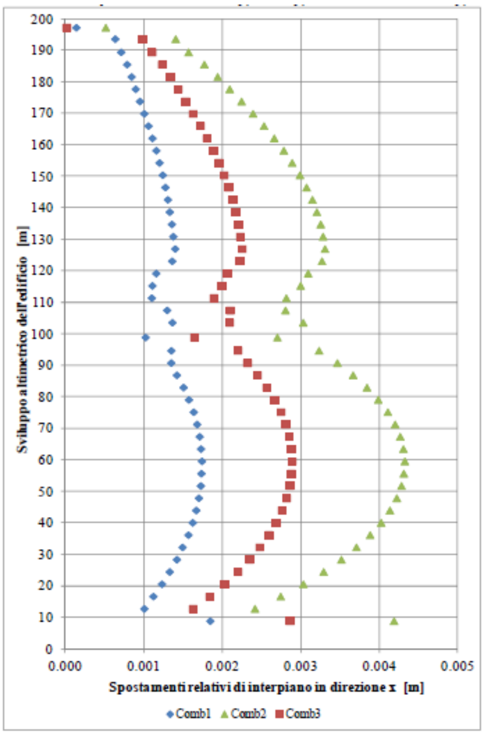

Figure 9-14: Point C - Drag component orthogonal to long side

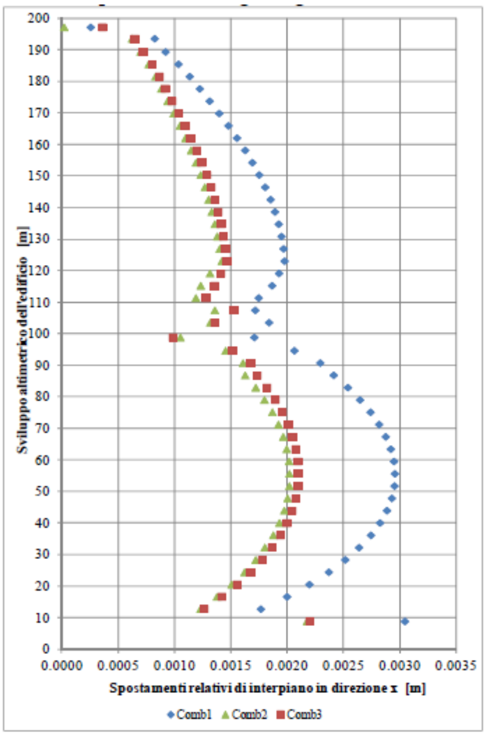

Figure 9-13: Point C - Drag component orthogonal to short side

Point D

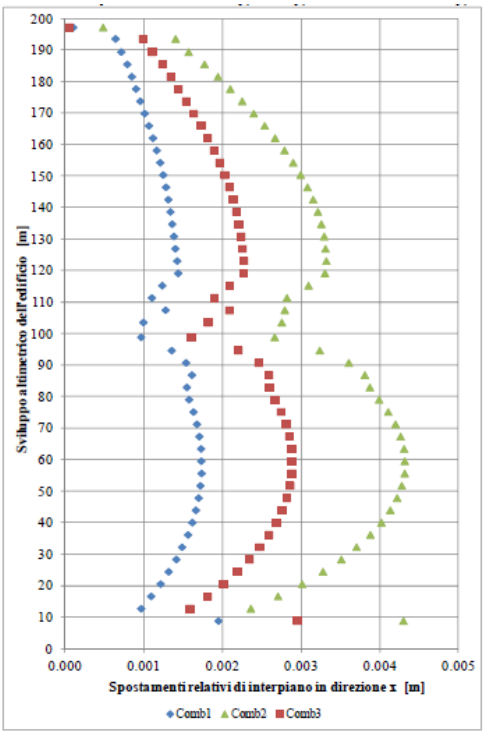

Figure 9-15: Point D - Drag component orthogonal to long side

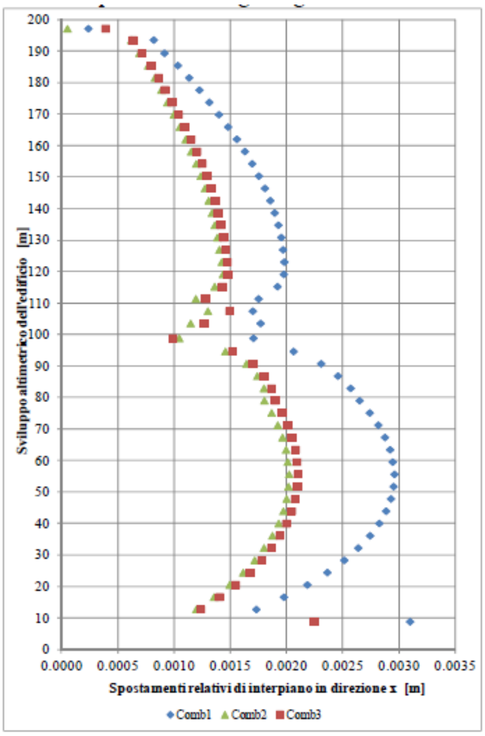

Figure 9-16: Point D - Drag component orthogonal to short side

Point E

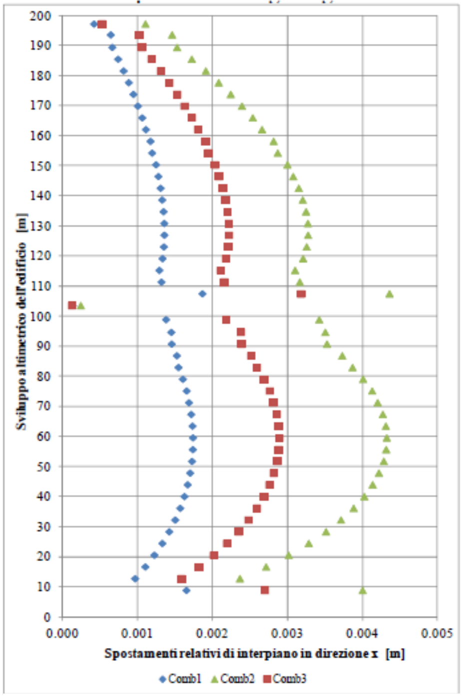

Figure 9-18: Point E - Drag component orthogonal to long side

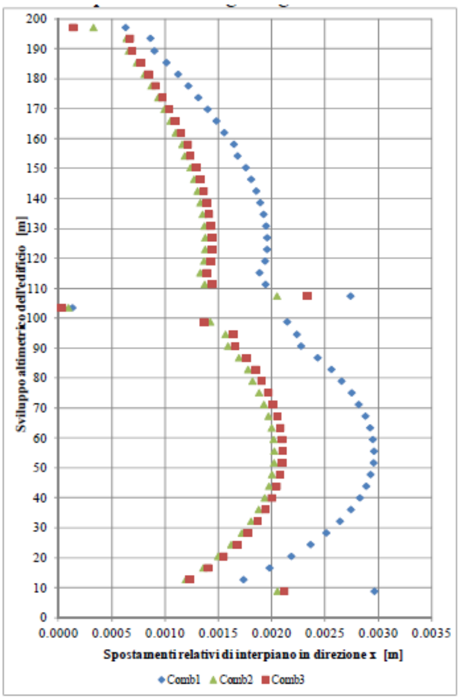

Figure 9-17: Point E - Drag component orthogonal to short side

Point F

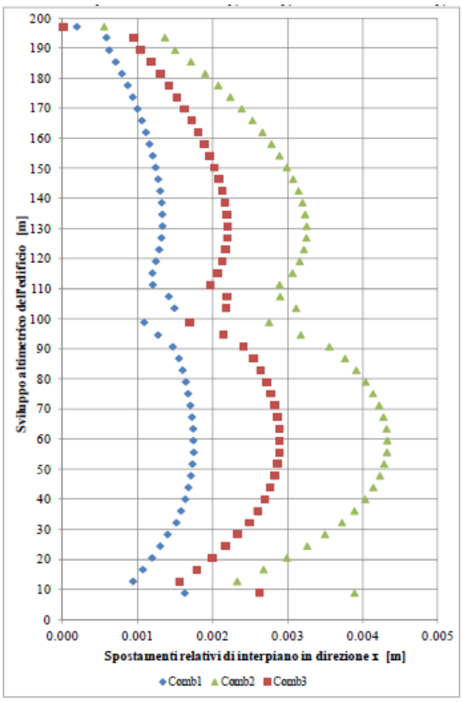

Figure 9-20: Point F - Drag component orthogonal to long side

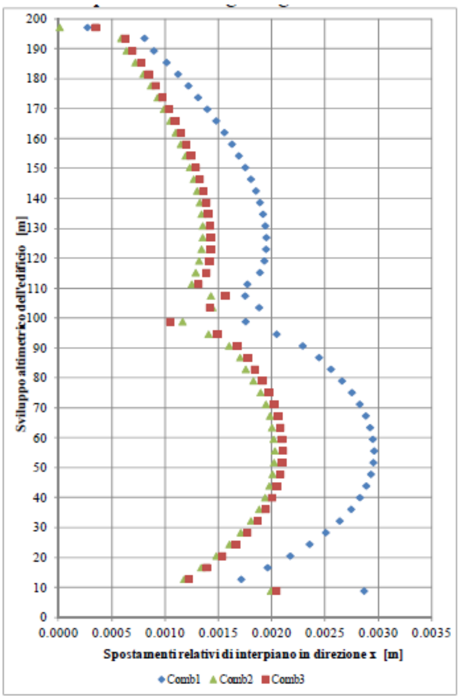

Figure 9-19: Point F - Drag component orthogonal to short side

Lateral displacements induced by the earthquake SLD

Point A

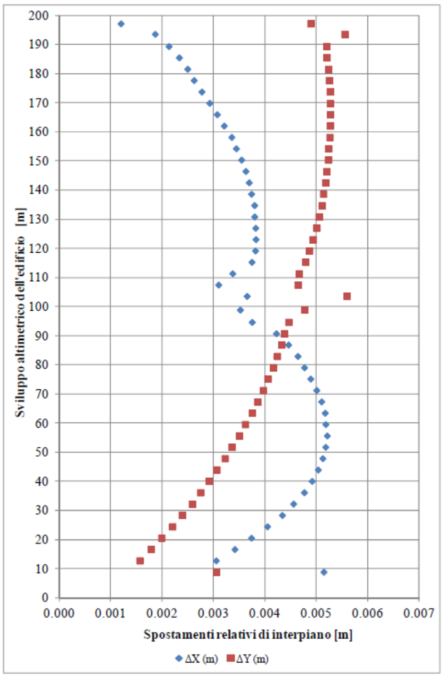

Figure 9-22: x & y displacement induced by earthquake in point A

Point B

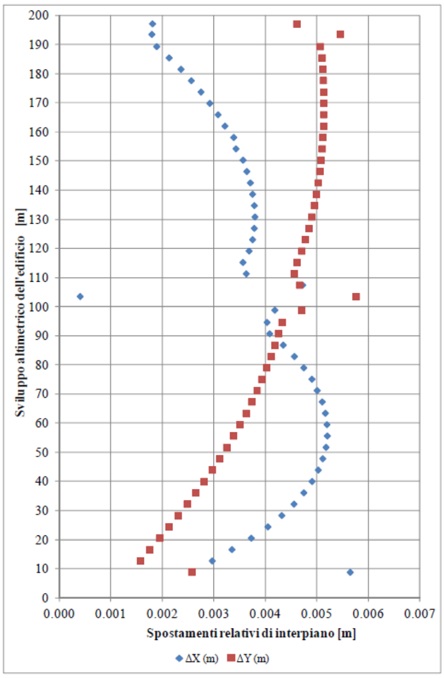

Figure 9-21: x & y displacement induced by earthquake in point B

Point C

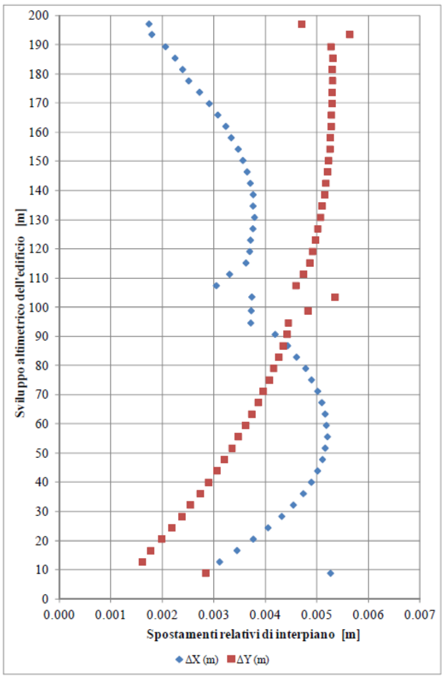

Figure 9-23: x & y displacement induced by earthquake in point C

Point D

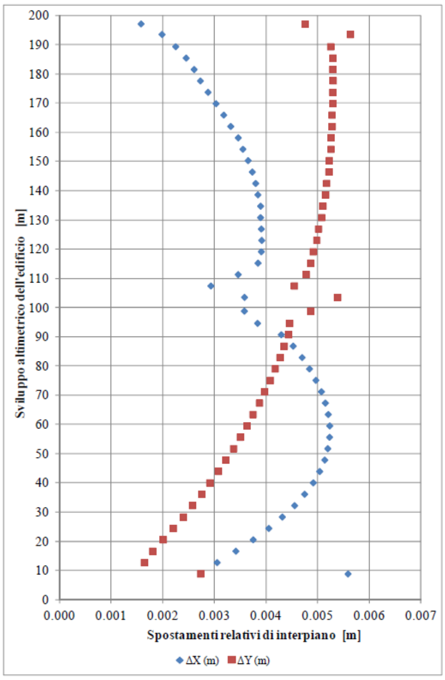

Figure 9-24: x & y displacement induced by earthquake in point D

Point E

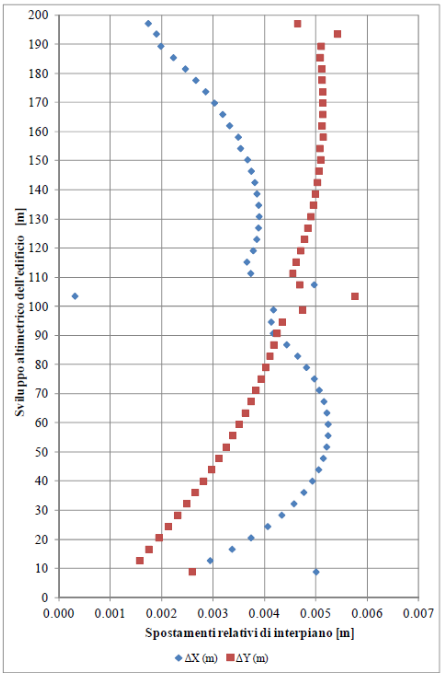

Figure 9-26: x & y displacement induced by earthquake in point E

Point F

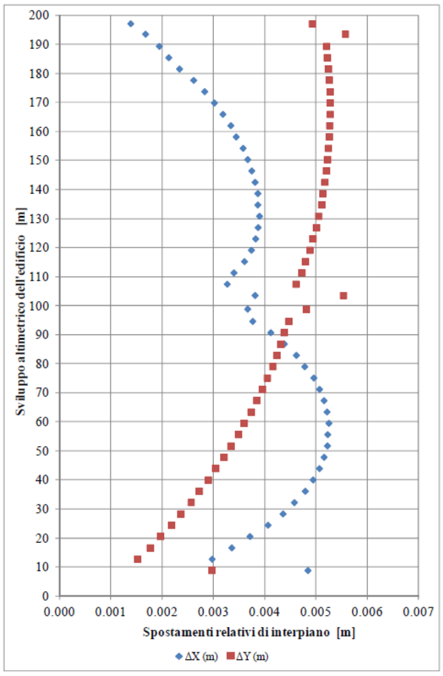

Figure 9-25: x & y displacement induced by earthquake in point F

The prevoius diagrams are relative to:

- Comb1 – SSL characteristic combination
  $G_1 + G_2 + P + Q_{k1} + \sum_{i=2}^{n}(\Psi_{0,1} \cdot Q_{k,i})$
- Comb2 – SSL frequent combination
  $G_1 + G_2 + P + \Psi_{1,1} \cdot Q_{k1} + \sum_{i=2}^{n}(\Psi_{2,1} \cdot Q_{k,i})$
- Comb3 – SSL quasi-permanent combination
  $G_1 + G_2 + P + \sum_{i=2}^{n}(\Psi_{2,1} \cdot Q_{k,i})$

for wind load action, and to:

- SLD $G_1 + G_2 + P + E + \sum_{j=2}^{n}(\Psi_{2,1} \cdot Q_{k,j})$

for the seismic action.

These diagrams show the high stiffness of the whole structure. The most stressed points are B and E, at the centre of the main façades, both for wind action and seismic action. In particular the widest displacements are at the 1st floor (no slab in the entrance hall) and at the 24th floor, above the central belt beam. Seismic action results giving a wider displacement than the wind but, in the end, the maximum absolute displacement recorded is just 5,7 mm, and the maximum interstorey drift is 4,8 mm for the point E at the 24th floor.

9.2 Triple-glazed curtain wall design

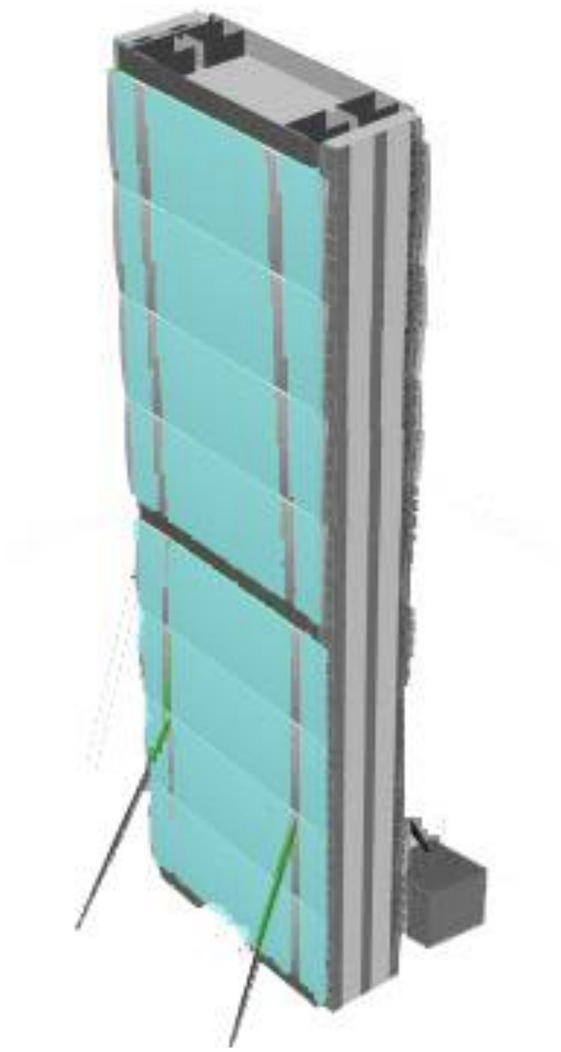

Figure 9-27: the studied façade of Isozaki Tower

It is used an unitized system with thermal break and aluminum frame. The double-glazing is curved and fixed with structural silicone on all 4 edges, the horizontal edges show just silicone, while outer gaskets are put along the vertical frames. The size of a module is 3.9 x 1.5 m. The curvature of the glass has a radius of 86m.

Mullions are curved as the glass, while transoms are simply rotated of about 8° in relation to the façade. The curtain wall is connected to the main structure with brackets.

The glazing is composed by: fully tempered laminated exterior glass with SGP interlayer with magnetron coating / air space 16 mm / central monolithic toughened glass / air space 16 mm / internal monolithic tempered glass.

The main displacements of the structure are taken up by horizontal and vertical joints between the glass panels. The movements caused by seismic events are offset by movements in the expansion joints integrated into the structure of façades, in order to minimize any damage. The total displacement of the building is less than H/500 and the intersorey drift is less than H/200.

9.3 Conclusions

The design strategy adopted to face the problem of seismic behavior of the façade in Isozaki Tower is quite clear. The structure is very rigid so that the induced interstorey drifts by horizontal actions are very small and are not dangerous for the façade. Moreover it is designed by using the unitized system, which decouples each storey from the adjacent others, minimizing wall system damage, and can rely on expansion joints to accommodate movements.

10 Proposed design alternative approaches

During past years researchers and designers tried to give solutions to problems caused to curtain walls by seismic-induced interstorey drift, by approaching in different ways. The main object of study has been the façade connection to the structure. In the following paragraphs three approaches are shown:

- Pendulum approach by Chamebel
- Decoupling approach by Wulfert
- Energy dissipation connections approach

10.1 Pendulum approach by Chamebel: the Panoflex system

In the 1960’s the Belgian façade supplier company Chamebel developed a pendulum-like anchoring system named Panoflex.

Chamebel noticed that even the simplest type of curtain wall underwent various effects when subjected to earthquake displacements, such as:

- permanent random displacement of the infill elements (glass and panels) on their supports with consequent possible failure and loss of weather tightness.
- displacement of the vertical and horizontal expansion joints beyond the allowable clearance with consequent excessive buckling stress in mullions and transoms, causing permanent deformation
- displacement or damage caused to sealing gaskets, requiring extensive repairs after the earthquake in order to make the façade air and watertight again

In order to prevent these difficulties, Chamebel developed a special curtain wall based on the principle of single rigid units mounted on special fixings providing a large degree of independence of movement of the infill elements relative to the framework of the building. Each frame holds a spandrel panel and a glazed part.

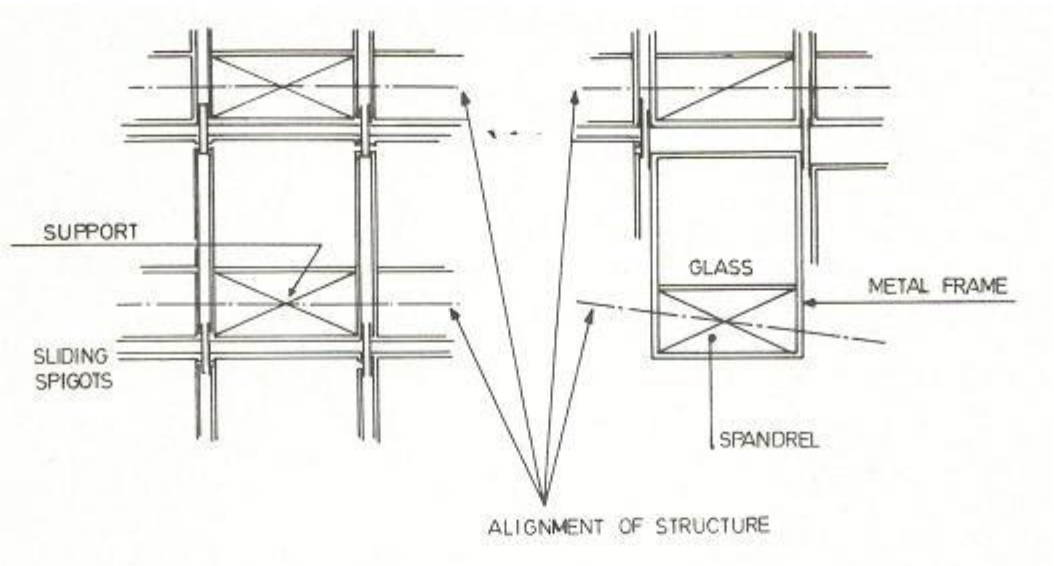

Figure 10-1: Principle of the Chamebel curtain wall

The original features of Chamebel's system are the method by which the frames are fixed to the main structure and the design of the gaskets between frames. They are fitted with a balance arm (2) having a central dead load fixing (3) and two lateral wind load fixings (3).

The frames are jointed to each other in two ways: for little displacements in the structure, jointing is ensured by spigots sliding in their sockets at the junction between mullions and for large displacements in the structure, jointing is by male/female blocks, fixed on the horizontal transoms of adjacent frames.

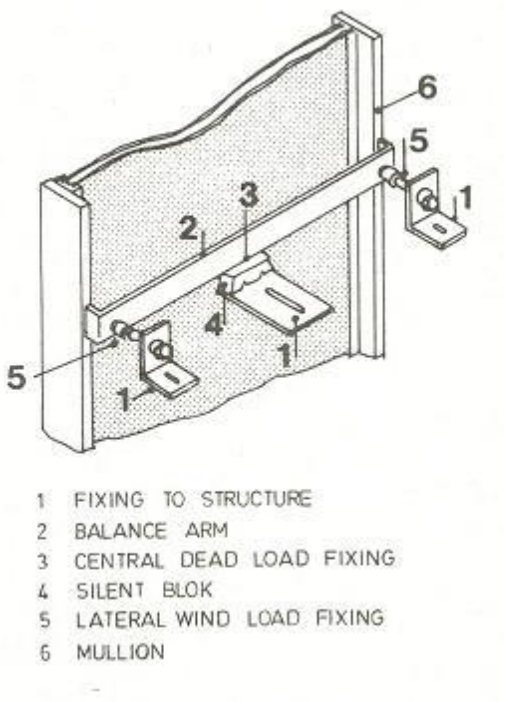

Figure 10-2: Fixation of the frames to the main structure

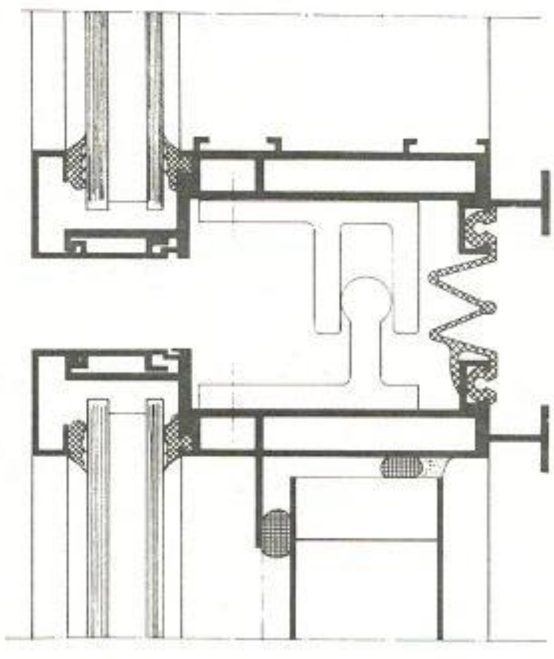

Figure 10-3: Male/female blocks in adjacent frames

In the plane of the façade, these blocks permit horizontal displacement of the frames relative to each other, moreover the joint assembly acts as a hinge, allowing a certain amount of rotation around it. The fixing (3) located on the vertical axis through the center of the frame transmits the frame's dead load to the building framework, while the lateral fixings (5) take up only the wind pressure. The lateral fixings don’t transmit any force in the plane of the façade. The in-plane vertical and horizontal seismic movements of the main structure in the plane of the curtain wall are transmitted to the curtain wall by the main fixings, while the out-of-plane horizontal movements of the facade are taken up by all the fixings (3) and (5). In this way it’s possible to prevent deformation into parallelograms of the sections of the frame, so that the infill elements are not subject to any pushing forces in their seating.

Finally, the sealing interface between frames is made of flexible neoprene gaskets with fully vulcanized cross or butt joints. The flexible joints are then not affected by relative displacements of frames due to earthquakes and sealing remains fully intact after the earthquake.

10.2 Decoupling approach by Wulfert: the earthquake-immune curtain wall system

Although curtain wall systems are normally considered to be "non-structural" parts of a building because they don’t help a building stand erect, they must have the ability to withstand structural loads imposed by natural phenomena such as earthquakes and severe windstorms. By considering that the primary factors causing earthquake-induced damage of conventional curtain wall systems are movements of the building's primary structural frame in response to earthquake ground movements and the fact that mullions in conventional curtain wall systems are connected structurally to more than one floor of the primary structural frame, in 2005, Wulfert et al. developed an earthquake-immune curtain wall system by decoupling each storey from the adjacent others so that they are all structurally isolated, minimizing wall system damage and the attendant risks of falling debris (in the forms of broken glass, concrete, etc.) during an earthquake.

The earthquake-immune curtain Wall system achieves structural isolation of each storey by employing a newly developed “seismic decoupler joint” between each storey and a newly developed structural support system for vertical mullions in the wall system frame. As a result, relative movements between adjacent stories in the building frame transfer no significant forces between adjacent stores in the curtain wall frame. This invention embodies a curtain wall system that is essentially “immune” from the effects of earthquake-induced building frame motions.

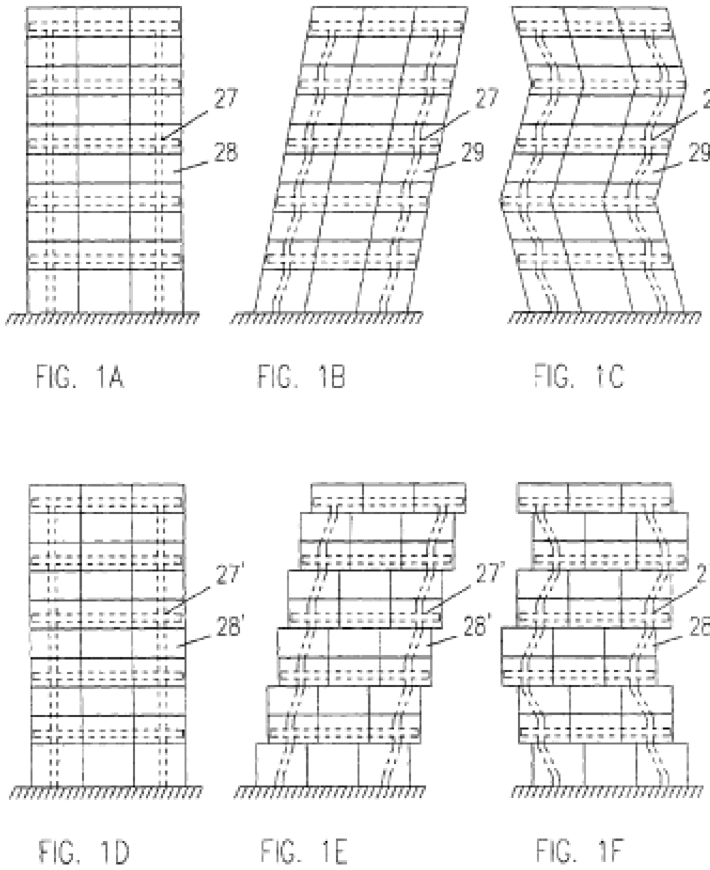

Figure 10-4: 1A—1C schematic displacement response of a typical building frame having a conventional curtain wall system to earthquake-induced ground motions; 1D—1F schematic displacement response of a building frame having an earthquake immune curtain wall system

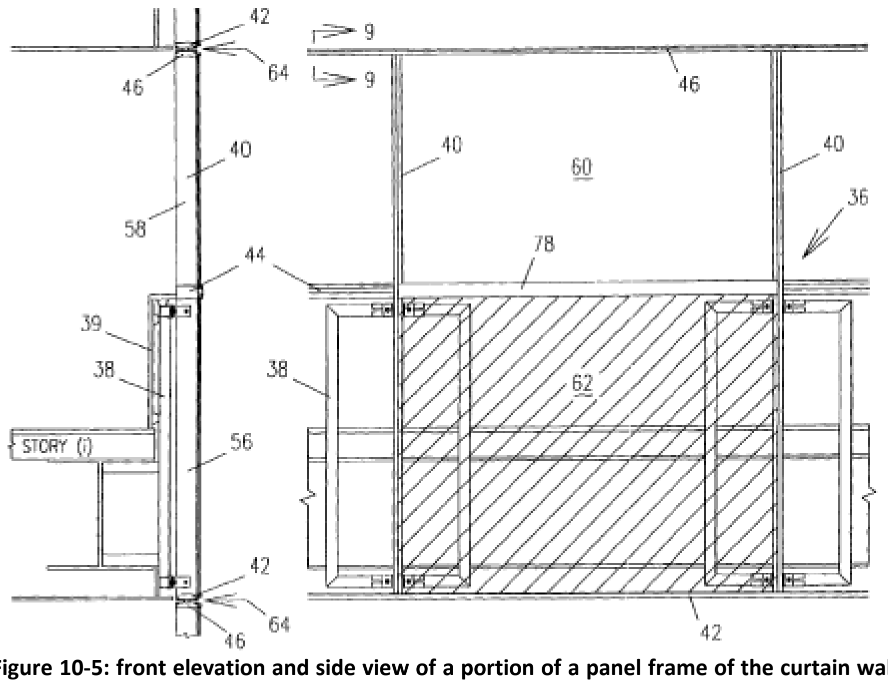

Figure 10-5: front elevation and side view of a portion of a panel frame of the curtain wall system according to the invention including vision panels and spandrel panels

In the following figure the anchoring system is illustrated in detail. Each steel frame 38 is connected to a spandrel beam at each story level in the main building structure using connection bars secured as necessary to the spandrel beam at two locations to provide stability against rotation about X, Y, or Z orthogonal axes. Each anchor frame is typically constructed of horizontal and vertical tubular steel members, respectively, in a rectangular configuration with sufficiently large cross sections to provide adequate strength and bending stiffness to resist design wind loads.

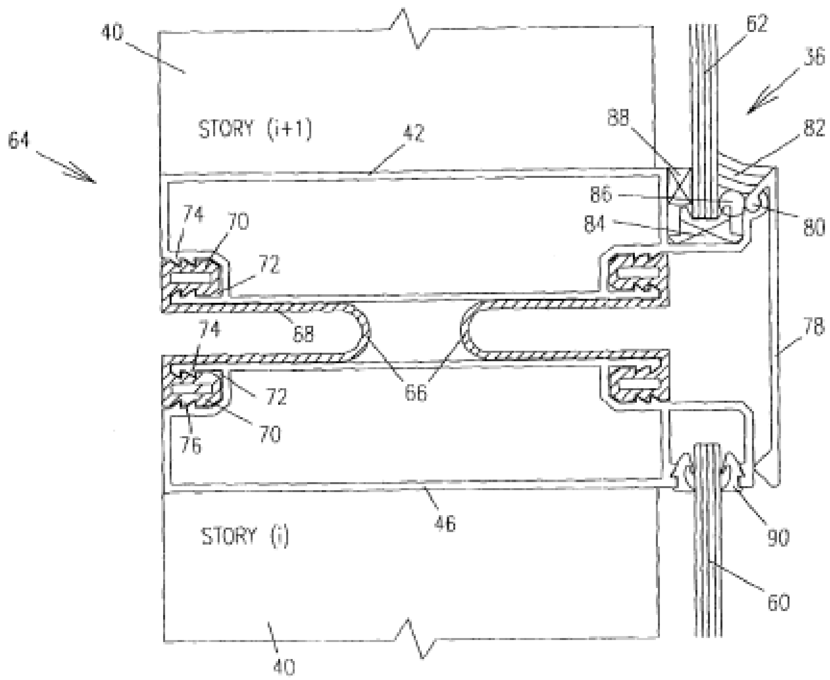

Figure 10-6: Vertical section of the seismic decoupler joint through line 9-9 of Figure 9.5

The decoupler joint uses a pair of continuous, flexible gaskets 66 made of polymeric material that accommodates in-plane, out-of-plane, and vertical movements between adjacent stories of the main building frame under earthquake conditions. Each gasket 66 is made of an elongate, extruded flexible material that may span the entire width of a floor. In cross section, each gasket includes a central portion 68 connected between locking end portions 70. The central portion 68 is originally flat. When installed, the central portion is rolled into position and assumes a U-shape. The locking end portions 70 are force-fit into channels 72 provided in the lower horizontal mullions 42 and upper horizontal mullions 46. The channels 72, in cross section, include teeth 74 for lockable engaging corresponding notches 76 in each locking end portion 70. As shown, a flexible gasket is placed at both the front and rear of adjacent lower horizontal mullions 42 and upper horizontal mullions 46. As a result, the central portions 68 extend inwardly between the lower horizontal mullion 42 of Story (i+1) and the adjacent upper horizontal mullion 46 of Story (i). The seismic decoupler joint 64 also includes a rotation accommodating face cap 78 that accommodates movement by means of a face cap hinge 80 and the use of a bead 82 of glazing sealant, e.g., structural silicone or other appropriate material, that has high deformation capability.

In summary, the seismic decoupler joint 64 accommodates interstorey movements in all directions, transfers no significant loads between adjacent stories and provides an effective thermal insulation and weather seal between adjacent stories in an earthquake-immune curtain wall system.[4]

10.3 Energy dissipation connections approach: advanced façade connectors

To improve façade performance under seismic loads, some researchers developed advanced connections with the idea to reassign a structural role to the architectural façade. The advanced façade connection can provide a better uniformly distributed energy dissipation over the height of the building and, in order to protect the façade panels, the connections limit the forces transmitted into the panel.

10.3.1 Friction damping connectors

A friction mechanism is the basis for many different proposed connection designs. One of the predominant benefits of the friction mechanism is its capability to dissipate a huge amount of energy through friction because of its inelastic functioning. The friction mechanism also has some defects, for example it experiences corrosion, and in addition, as in conventional tie-back connections, an insufficient length of the slot could reduce the effectiveness of the friction mechanism.

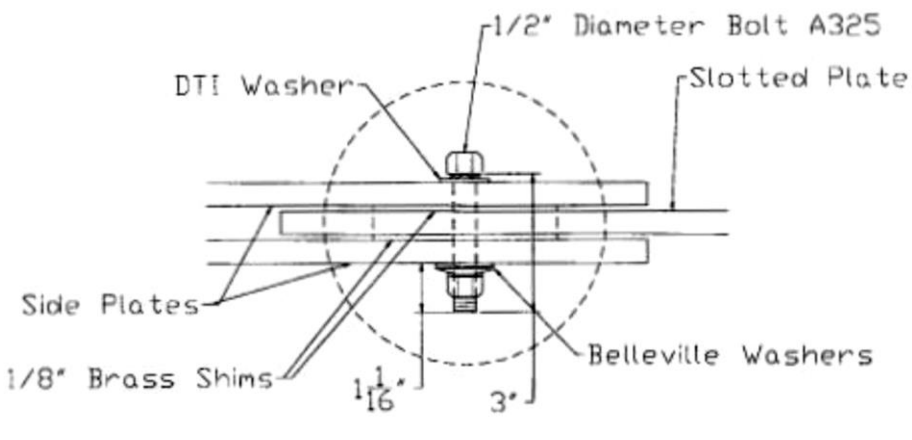

Figure 10-7: Typical slotted bolted connection by Grigorian et al.

4 Patent Wulfert et al., Earthquake-immune curtain wall system, USPNO 6,598,359

10.3.2 Viscoelastic dampers

Viscoelastic dampers consist of plates separated by inert polymer materials that dissipate energy by shear deformation of the copolymer. Force-displacement characteristics of VE dampers are influenced by the function of either the relative velocity between the ends of the damper or the frequency of motion. However, the response of these devices may also be a function of relative displacement. One defect of VE dampers is the fact that they are temperature sensitive, which could create special challenges for external fittings on structures.

Research and development of VE dampers for seismic application was started in the early 1990s and in the following years a 50% reduction in the seismic response of the frame equipped with VE dampers have been confirmed by the ones developed by Showa and Shimizu corporations.

In the work by Rahila Wardak Hareer from the Queensland University of Technology, 3-storeys, 6-storeys, 12-storeys and 18-storeys building facade systems with and without energy absorbing connections were investigated under three different earthquake records. The result is that the VE damping connections in the majority of cases were able to produce remarkably high improvement and reduced the seismic effect on facades at all levels of the structure. In some cases (under the Kobe earthquake excitations) the VE damping connections in the upper storeys were not effective at all, as in those storey levels, an increase in the magnitude of the parameters were noticed.

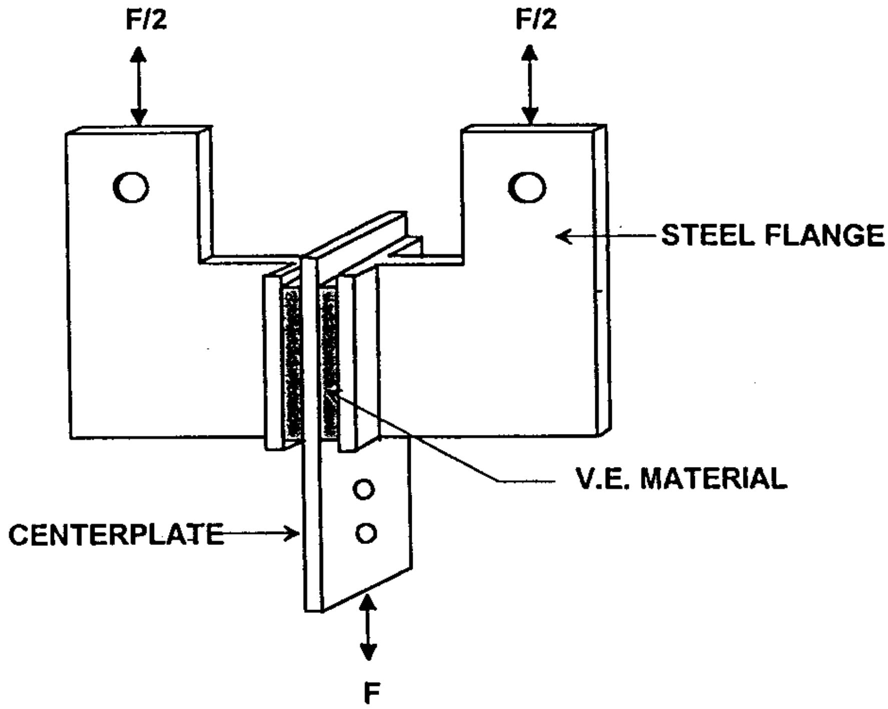

Figure 10-8: Layout of viscoelastic dampers (Chang, Lin, Lai)

Conclusions and future developments

As a result of the previous chapters some conclusions have been reached and here are summarized for a final and global consideration about the carried out work and potential future developments.

The aim of this study was to investigate the curtain walling façade behaviour when subjected to a seismic event, in the interest to find the best design solutions to face the problem.

In order to do this, Section 1, after presenting the different typologies of curtain wall, explored the earthquake fenomena and how buildings react. Particular attention was given to the effects on non-structural elements, pointing out that the most serious hazard type of load on façades is recognized to be the in-plane relative displacement between two adjacent stories, the so called interstory drift.

Section 2 made the point on the state of the art from three different points of view:

- by giving the mathematical instruments for evaluating the lateral drift capacity of window panels under horizontal loadings;
- by showing the results of two relevant experimental studies that gave the basis for the current approach to the matter;
- by examining the international standards and guidelines about laboratory testing of curtain walls, then summed up in a comparison between American and European approach.

This section is very important because it gives the instrument for the right designing of glazed façades: mathematical formulas allow to design the system so to accommodate the interstorey drift, the experimental studies suggest the best system configurations and which glass types are recommended or not (for example it is way better to use fully tempered laminated glass than monolithic one) and international standards represent a guide to the designing, testing and verificating curtain walls.

In Section 3 the testing approach is analysed very closely. In fact here is described the stage experience in the test centre of Reynaers Aluminium in Duffel where static tests on two different profile of stick system curtain walls were carried on with the aim of studying their behaviour and to compare it to the theoretical results. Being the European standard not effective yet, the American one AAMA 501.4 was used as reference. First tests were not valid due to some human errors and a not perfectly designed steel structure. By the way tests were repeated and revealed a great response of the system: the façades was perfectly able to respect all the discussed requirements and were still serviceable even after imposed displacements superior to the theretical calculated ones.

When subjected to extreme interstorey drift (100 mm) it was observed rotation of transoms, deformation of screws and some cracking lines and exfoliations in glass corners, the most stressed points. It's possible to state that this is an adequate way to behave, both in the safety and economic points of view. No glass fallout occurred and maintenance intervention would concern only some elements and would not be very urgent allowing people not to undergo any damages.

Section 4 is a collection of practical examples of curtain walls design in relation to seismic action. As a representative approach in current new buildings a case study was analysed: Isozaki Tower at CityLife. It's evident how one of the most common approaches for tall buildings in Italy is to well design the structure, so that it is very rigid and does not present wide interstorey drifts when subjected to horizontal in-plane actions. To make the façade even more reliable a unitized system with module as high as the interstorey height is used, even in order to minimize the effects of interstorey drift, which is absorbed by expansion joints into the structure.

In the end other alternative approaches generally concerning the connection to the structure are analysed:

- The option to decouple vertical load transmission from horizontal one;
- The option to decouple each storey from the adjacent others so that they are all structurally isolated by using a decoupler joint;
- The option to use connections to the structure that dissipate seismic energy through a friction system or a viscoelastic damper system.

This work fits in a really wide but still not enough well-known research field.

What emerges from this study is that some good base instrument to evaluate the seismic behaviour of curtain walls are already present, but they have not completely been absorbed by the common way of building.

This happens especially in Europe where the test standards are not official yet and where the sensibility to the matter has grown only recently, with the globalization and the increasing interest of big aluminium frames producers in exporting their systems outside their countries, frequently in highly seismic areas such as Middle East.

The experimental mock-up tests are a fundamental evaluating tool for the real façade behaviour and proved that the stick system used by Reynaers potentially behaves in an optimal way. I say “potentially” because it was not actually tested to air permeability, water tightness and resistance to wind load as prescribed by AAMA 501.4, so its serviceability is only supposed, even if apparently fulfilled.

It is evident how, being this just a recent challenge, there are still wide margins of development for the experimental methods of testing.

For what concern the approaches to face the problem of interstorey drift in real buildings, it seems that designers feel safer if they design the structure very capable to respond to seismic action so not to involve too much the façade in the reaction. This is surely a good way, even better than excessively count on drift accomodation of the frame, but other systems could be considered such as dampers that allow to better distribute energy dissipation, giving a sort of structural role to the façade.

These kind of connections are still not perfectly defined and probably they have a high cost, by the way they result to be the most studied because they seem to provide better performance over time and a great degree of safety.

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Appendix A

Here are reported the numerical results of the modal analysis with which the characteristics of the Isozaki Tower have been identified for the calculation of the wind action.

Base reactions:

Load	FX (kN)	FY (kN)	FZ (kN)
PP	0.000	0.000	958591
G	-0.001	0.000	216816
Q	-0.002	0.001	211953
GI	0.000	0.000	31630

Total mass:

Mtot [kN/g]
129306.87

Dynamic charateristics:

Mode No	Frequency		Period	Toleranc
	(rad/sec)	(cycle/sec)	(sec)	
1	1.08	0.17	5.83	0.00
2	1.27	0.20	4.95	0.00
3	2.48	0.40	2.53	0.00
4	3.62	0.58	1.74	0.00
5	6.25	1.00	1.00	0.00
6	8.22	1.31	0.76	0.00
7	9.18	1.46	0.68	0.00
8	13.15	2.09	0.48	0.00
9	14.83	2.36	0.42	0.00
10	19.27	3.07	0.33	0.00
11	19.50	3.10	0.32	0.00
12	19.54	3.11	0.32	0.00
13	23.07	3.67	0.27	0.00
14	24.28	3.86	0.26	0.00
15	25.94	4.13	0.24	0.00
16	26.69	4.25	0.24	0.00
17	30.27	4.82	0.21	0.00
18	30.27	4.82	0.21	0.00
19	31.25	4.97	0.20	0.00
20	32.37	5.15	0.19	0.00

Mode No	TRAN-X		TRAN-Y		TRAN-Z		ROTN-X		ROTN-Y		ROTN-Z	
	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)	MASS(%)	SUM(%)
1	66.14	66.14	0.01	0.01	0.00	0.00	0.01	0.01	32.78	32.78	0.00	0.00
2	0.01	66.15	57.40	57.41	0.00	0.00	41.70	41.71	0.00	32.78	0.00	0.00
3	0.00	66.15	0.00	57.41	0.00	0.00	0.00	41.71	0.00	32.78	62.33	62.33
4	9.84	75.99	0.00	57.41	0.00	0.00	0.00	41.71	19.22	52.00	0.00	62.33
5	0.00	75.99	18.98	76.39	0.00	0.00	12.35	54.06	0.00	52.00	0.00	62.33
6	5.54	81.53	0.00	76.39	0.00	0.00	0.00	54.06	7.19	59.19	0.00	62.34
7	0.00	81.53	0.00	76.39	0.00	0.00	0.00	54.06	0.00	59.19	12.93	75.26
8	2.36	83.89	0.00	76.39	0.00	0.00	0.00	54.07	3.33	62.52	0.00	75.26
9	0.00	83.89	6.64	83.03	0.00	0.00	9.84	63.91	0.00	62.52	0.00	75.27
10	0.01	83.90	0.00	83.03	60.37	60.37	0.00	63.91	0.01	62.53	0.01	75.28
11	2.08	85.98	0.00	83.04	0.15	60.52	0.00	63.91	1.59	64.12	0.27	75.55
12	0.10	86.09	0.00	83.04	0.22	60.74	0.00	63.91	0.08	64.20	5.56	81.11
13	0.43	86.52	0.00	83.04	0.00	60.74	0.00	63.91	6.10	70.30	0.00	81.11
14	0.00	86.52	3.51	86.55	0.00	60.74	4.74	68.65	0.00	70.30	0.00	81.11
15	0.00	86.52	0.00	86.55	12.45	73.19	0.00	68.65	0.00	70.30	0.00	81.11
16	1.20	87.72	0.00	86.55	0.00	73.19	0.00	68.65	0.88	71.18	0.00	81.11
17	0.00	87.72	0.00	86.55	0.00	73.19	0.00	68.65	0.00	71.18	0.00	81.11
18	0.00	87.72	0.00	86.55	0.00	73.19	0.00	68.65	0.00	71.18	0.00	81.11
19	0.00	87.72	0.00	86.55	0.00	73.19	0.00	68.66	0.00	71.18	3.22	84.34
20	0.03	87.76	0.00	86.55	0.00	73.19	0.00	68.66	0.04	71.21	0.00	84.34

Mode No	TRAN-X		TRAN-Y		TRAN-Z		ROTN-X		ROTN-Y		ROTN-Z	
	MASS	SUM	MASS	SUM	MASS	SUM	MASS	SUM	MASS	SUM	MASS	SUM
1	85522	85522	10	10	0	0	38725	38725	193417888	193417888	277	277
2	16	85538	74223	74233	0	0	233604286	233643011	16920	193434808	833	1111
3	2	85540	1	74234	0	0	3273	233646284	147	193434955	30873178	30874288
4	12721	98261	0	74234	0	0	3750	233650033	113395935	306830890	786	30875074
5	0	98261	24542	98776	0	0	69212225	302862258	5124	306836015	8	30875082
6	7168	105429	1	98777	0	0	1542	302863800	42441857	349277872	592	30875674
7	0	105430	1	98778	0	0	2951	302866751	4761	349282633	6403723	37279397
8	3049	108479	2	98781	0	0	17919	302884670	19652782	368935415	0	37279397
9	1	108480	8589	107370	0	0	55135299	358019968	2246	368937661	377	37279774
10	10	108490	0	107370	58848	58849	128	358020096	29418	368967079	6944	37286718
11	2693	111183	0	107370	150	58999	1084	358021181	9410138	378377218	134511	37421229
12	132	111315	1	107371	214	59213	7113	358028294	444485	378821703	2753827	40175056
13	560	111875	0	107371	0	59213	1465	358029759	36006298	414828001	12	40175069
14	0	111875	4539	111910	1	59214	26578114	384607872	123	414828124	96	40175165
15	0	111875	0	111910	12136	71350	274	384608146	4810	414832934	0	40175165
16	1557	113432	0	111910	0	71350	305	384608451	5171504	420004438	24	40175190
17	0	113432	0	111910	0	71350	217	384608668	62	420004500	3	40175193
18	0	113432	0	111910	1	71351	4	384608672	3	420004503	77	40175270
19	0	113432	2	111913	0	71351	15148	384623820	336	420004838	1596849	41772119
20	45	113477	0	111913	0	71352	482	384624302	221569	420226407	356	41772474

Index of figures

Index of tables

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