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SEISMIC ANALYSIS OF NONSTRUCTURAL ELEMENTS

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Gracias a todos.

ABSTRACT

Nonstructural failures have accounted for the majority of earthquake damage in several recent earthquakes. Thus, it is critical to raise awareness of potential nonstructural risks, the costly consequences of nonstructural failures, and the opportunities that exist to limit future losses. Non-structural parts of a building have the potential to modify earthquake response of the primary structure in an unplanned way. This can lead to severe structural damage or even collapse. Failure of non-structural components may cause death or injury from: Falling panels, masonry or glass, collapsed ceiling components, falling fittings and fixtures, debris blocking exit ways, etc.

The configuration of a building could be called its seismic form. An obvious example of poor seismic configuration is a U or L shaped building on plan, if it is not structurally divided into simply shaped blocks. Such a building may suffer damage in an earthquake because the ‘free’ ends on plan will sway in a different way to the corner section, which is stiffer. Columns on re-entrant corners are particularly prone to damage because of the concentration of forces at such points. Therefore should be avoided in the design: irregular configuration, soft-storey, asymmetrical horizontal bracing, short pillars, etc. so as to properly distribute the stresses.

Of course, an important starting point in the study and evaluation of this topic is the consideration and the comparison of several local regulations and their approach to the seismic design of non-structural elements. The reason of the importance of considering and evaluating the possible differences among different approaches to the same problem is clearly understandable. These national regulations represent the expression of the “state of the art” about this topic of a specific country or region of the world. In this project analyzes regulations as: European regulation (Eurocode 8), American regulation (ASCE 7-10), New Zealand regulation (NZS 1170.5), Japan regulation and Spanish Regulation (NCSE-02).

Finally we focus on a specific non-structural element such as the facades. Facade systems can be categorized by three main types; infill panels, cladding and a combination of the two. In general, infill panels are constructed within the frame of the structure and cladding facades are attached externally to the primary structure. Each facade system will behave in a particular way when subjected to inter-storey drift. It is important to understand how each facade system behaves in order to determine which parameters are important for the performance based design. How a facade system is connected to the primary structure is the critical aspect in determining the interaction between the two systems. The current

practice in seismically active countries such as Japan, USA and New Zealand is to separate the facade system from the frame. For infill panels this is most commonly done using a seismic (or separation) gap between the wall and frame. Similarly to seismic gaps, the interaction between cladding systems and the frame can be minimized using movement connections. These connections commonly consist of a fixed and sliding connection which allows the cladding panels to move and rotate relative to the frame when undergoing seismic excitation. Another possible solution is use of a relative displacement between facade and structure. Facade systems can be integrated with energy dissipative connections that are designed to yield before the facade yields. Finally, having a complete integration of the facade system is often an effective strategy to reduce the drift of a structure because of the additional stiffness provided by the façade.

Finally, we analyze a masonry wall brick through a computer application that is based on the finite element method. We analyze a concrete masonry wall (the properties of each masonry wall vary depending on the individual parameters of the masonry and the connection between them) and adjust the model to obtain the stress distribution maps, compare the results with the Spanish regulation (CTE) and to draw conclusions. See as, masonry is a material designed to resist vertical loads, one of the principal factors must be considered in the design is the compressive strength. However, these structures are affected by other actions, such as earthquakes loads, which translate into horizontal forces. This requires considering the shear strength and tensile strength of the masonry.

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1.INTRODUCTION

Nonstructural failures have accounted for the majority of earthquake damage in several recent earthquakes. Thus, it is critical to raise awareness of potential nonstructural risks, the costly consequences of nonstructural failures, and the opportunities that exist to limit future losses. Nonstructural components of a building include all of those components that are not part of the structural system; that is, all of the architectural, mechanical, electrical, and plumbing systems, as well as furniture, fixtures, equipment, and contents. Windows, partitions, granite veneer, piping, ceilings, air conditioning ducts and equipment, elevators, computer and hospital equipment, file cabinets, and retail merchandise are all examples of nonstructural components that are vulnerable to earthquake damage.

Design of non-structural elements is important because:

Non-structural parts of a building have the potential to modify earthquake response of the primary structure in an unplanned way. This can lead to severe structural damage or even collapse.

Damage to non-structural elements themselves may prevent the building from functioning after an earthquake, or make it useless, even though the structure remains sound.

Failure of non-structural components may cause death or injury from:

- Falling panels, masonry or glass.
- Collapsed ceiling components.
- Falling fittings and fixtures.
- Debris blocking exit ways, etc.

Figure 1 Nonstructural failure in Lorca. 2011. (Informe del sismo en Lorca del 11 de mayo de 2011).

2. EARTHQUAKE

2.1 What is an earthquake?

An earthquake is a sudden and violent motion of the earth caused by volcanic eruption, plate tectonics, or man-made explosions which lasts for a short time, and within a very limited region. Most earthquakes last for less than a minute. The larger earthquakes are followed by a series of aftershocks which also may be dangerous.

2.2 Why do Earthquakes Happen?

Earthquakes can be caused by volcanic eruption, or by plate tectonics. Blasting, quarrying and mining can cause small earthquakes. Underground nuclear explosions are also man made earthquakes. But large majority of earthquakes and especially big earthquakes are invariably caused by plate tectonics.

The earth's crust is a rock layer of varying thickness ranging from a depth about 10 km under the ocean to 65 km under the continents. The crust is not one piece but consists of portions called plates, which vary in size from few hundred to many thousands of square kilometers *(Figure 2).* The theory of *plate tectonics* holds the plate ride upon the more mobile mantle, and they are driven by some yet unconfirmed mechanism, perhaps thermal convection currents. When the plates contact each other, stresses arise in the crust. These stresses may be classified according to the movement along the plate boundary:

- Pulling away from one another,
- Sliding sideways relative to each other and
- Pushing against each other.

The area of stress at plate boundaries, which releases accumulated energy by slipping or rupturing, is known as faults. A rupture occurs along the fault when accumulated stresses overpass the supporting capacity of rock mass and the rock rebounds under its own elastic stress until the stress is relieved. Usually the rock rebounds on both sides of the fault in opposite directions.

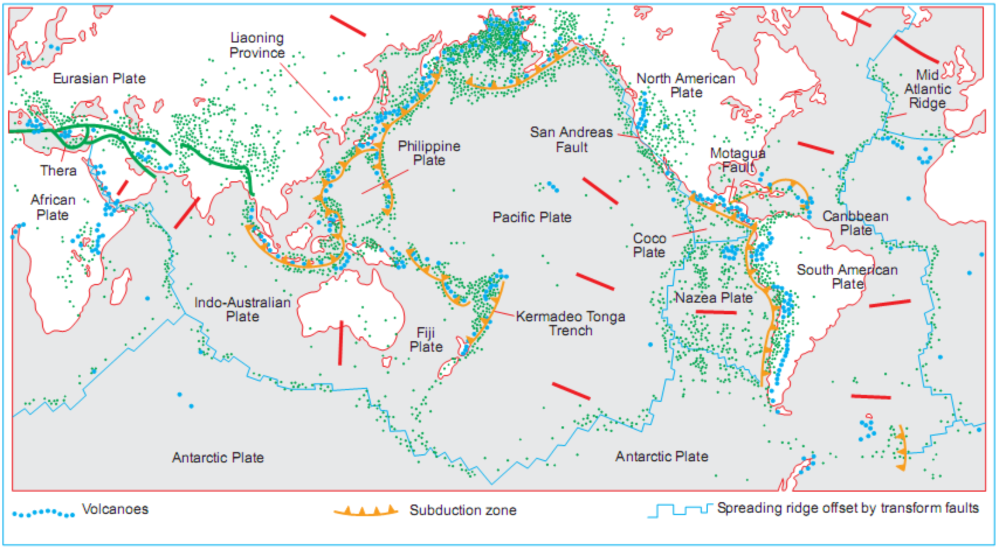


Figure 2 Different plates of earth (Protection of educational buildings against earthquakes. Jitendra Kumar Bothara, Ramesh Guragain, Amod Dixit).

The point of rupture is called the focus or hypocenter and may be located near the surface or deep below it. The point on the surface vertically above the focus is termed the epicenter of the earthquake *(Figure 3).* The fault rupture generates vibrations called seismic waves which radiate from the focus in all directions.

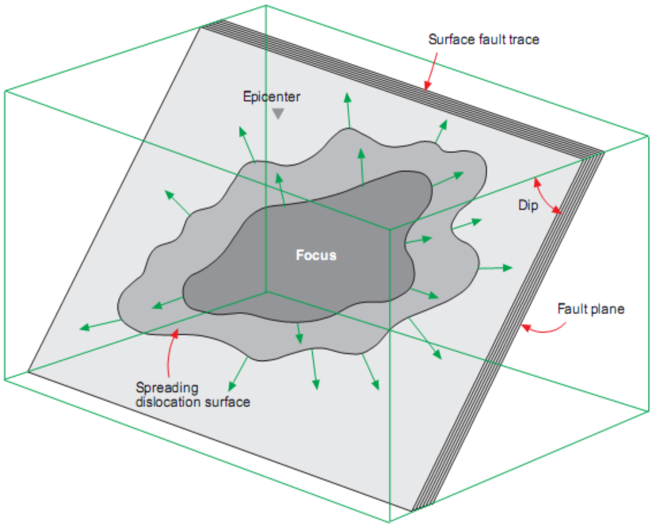


Figure 3 Epicenter and Focus (Protection of educational buildings against earthquakes. Jitendra Kumar Bothara, Ramesh Guragain, Amod Dixit).

2.3 Measurement of Earthquakes

2.3.1 Earthquake Magnitude

The magnitude of an earthquake is a measure of the amount of energy release at the source, the focal area. It is estimated from instrumental observations. The oldest and most popular measurement of an earthquake is the Richter scale, defined in 1936. Since this scale is logarithmic, an increase in one magnitude signifies a 10-fold increase in ground motion or roughly an increase in 30 times the energy release. Thus, an earthquake with a magnitude of 7.5 releases 30 times more energy than one with a 6.5 magnitude, and approximately 900 times that of a 5.5 magnitude earthquake. An earthquake of magnitude 3 is the smallest normally felt by humans. Largest earthquake that have been recorded under this system are from 8.8 to 8.9 in Magnitude.

2.3.2 Earthquake Intensity

The intensity is a measure of the felt effects of an earthquake. It is a measure of how severe the earthquake shaking was at any location. So it could differ from site to site. For any earthquake, the intensity is strongest close to the epicenter. A single event can have many intensities differing in the severity of ground shaking at different locations.

These two terms, earthquake Magnitude and earthquake Intensity are frequently confused in describing earthquakes and their effects. While Magnitude, expressed generally on the Richter scale, is a term applied to the amount of energy released of an earthquake as a whole, Intensity is a term applied to the effect of an earthquake at the affected site that determines the severity of the effect on a structure.

The most widely used scale for measuring earthquake Intensity has been the Modified Mercalli Intensity (MMI) scale that was first developed by Mercalli in 1902, later on modified by Wood and Neuman in 1931. It expresses the intensity of earthquake effects on people, structures and earth's surface in steps from I to XII. The further more detailed and explicit scale, the Medvedev-SponheuerKarnik (MSK) scale (1964) is now also commonly used. Both scales are very close to each other. A brief description of the different Intensities is given in *Table 1.*

Modelling a Structure as an Oscillator	Modelling a Structure as an Oscillator
I	Not noticeable
II	Scarcely noticeable (very slight)
III	Weak, partially observed only
IV	Largely observed
V	Awakening
VI	Frightening
VII	Damage considerable in poorly constructed buildings
VIII	Damage to masonary buildings
IX	Poorly built masons structures collapse
X	Most Masonary and frame structure destroyed
XI	Catastrophic danage to well built structures
XII	Total devastation with landscape changes

Table 1 MSK intensity scale in brief

3. STRUCTURAL DESIGN CONCEPTS

3.1 Simple Elastic Model of Building

To assess the likely demands on a building, a realistic representation or model of the building is required.

The only model that can accurately represent a structure and its behaviour accurately is one that is built full scale, with the same construction defects as the real structure, on the same foundation conditions as the real structure, and which is subject to identical loading over its life. Any other type of model is a crude approximation [1]. In reality, costs of producing accurate models are prohibitive, so all models used in engineering design are simple approximations. There are different levels of complexity of simple models and calibrations are carried out between numerical and physical models to ensure that the model used provides sufficiently accurate estimates of the key parameters.

Much of engineering design is based on the modelling of a realistic structure, such as that idealized in *Figure 4 (a),* as the single-degree-of-freedom (SDOF) oscillator shown in *Figure 4 (b).* This simple model can be represented by a **mass**, *m*, and **stiffness**, *k*. It is able to move only in one direction (horizontally). A structure of this type will vibrate with a ***period***, *T*.

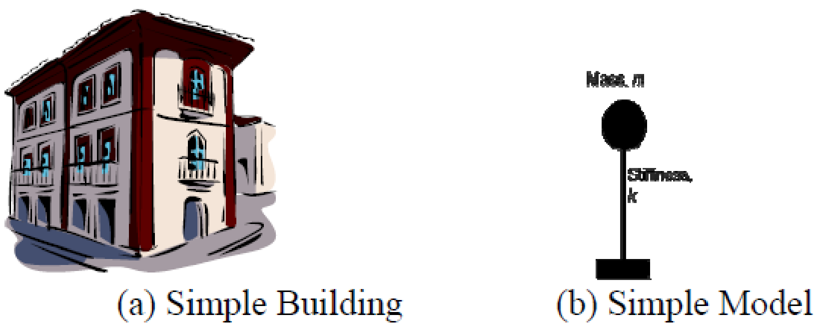


Figure 4 Modelling a Structure as an Oscillator (Review of NZ Building Codes of Practice. Gregory MacRae; Charles Clifton; Les Megget. 2011)

The ***period***, *T*, of the structure can be related to both the **mass**, *m*, and **stiffness**, *k*, using the equation in *Figure 5* Also, the period has physical significance. For an oscillator, pulled in one direction and suddenly released, it is the time taken for it to move in the other direction and then to return to its peak displacement in the direction from which it was released. Here, $x(t)$ is the displacement at any time, *t*. As the mass, *m*, increases, the period, *T*, increases. As stiffness, *k*, increases, the period, *T*, decreases. In general short (low rise) structures have much shorter periods than do multi-storey (high-rise) structures.

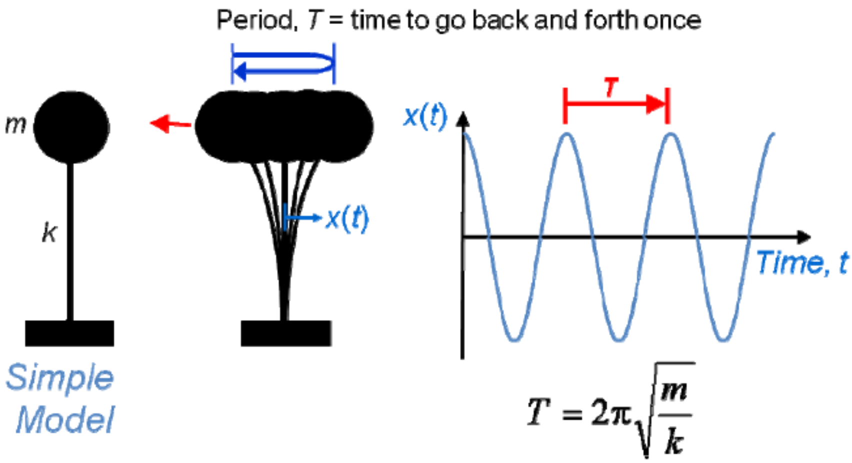


Figure 5 the Period, *T*, of a Simple Oscillator [1]

In practice, a building does not generally keep vibrating backwards and forwards to the same displacement in each direction, as shown in *Figure 5*. Instead, the displacement tends to decrease with time as shown in *Figure 6.* This decrease in response is generally referred to as “damping” and it accounts for a number of ways that energy may be removed from the system. These include energy dissipation as elements rub against each other and radiation of energy from the foundation. The actual “damping” value in real structures seems to be dependent on the magnitude of displacement.

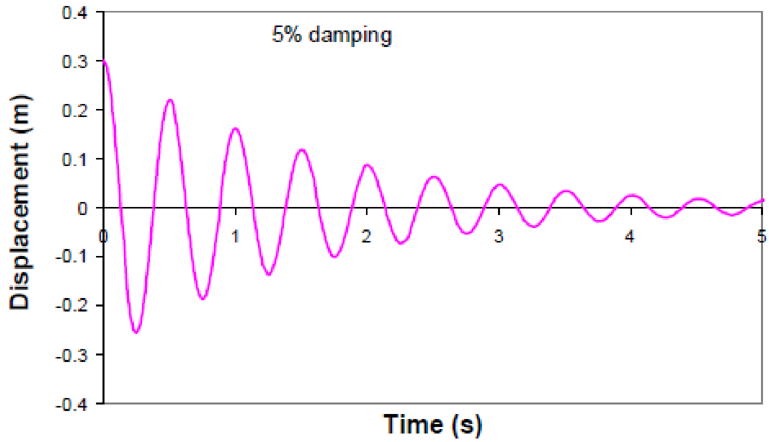


Figure 6 Effect of Damping on a Simple Oscillator [1]

As the period of the structure is dependent on the stiffness, realistic estimates of the structural stiffness are required. This stiffness is affected by:

- foundation flexibility,
- the presence of non-structural elements which may also carry some of the load, and
- the assumed member properties.

The influence of assumed member properties was not recognised in the earlier codes. It is especially significant for reinforced concrete members, which sustain flexural cracking during earthquake excitation. This cracking is expected. It is required if the benefits of the reinforcing bars are to be utilised. The flexural cracking also does not result in significant damage, but it can significantly reduce the stiffness and increase the frame's periods. For example, some beams in tests have been observed to have flexural stiffness as low as 10% of the gross stiffnesses. As codes have developed, they have included recommendations for stiffness.

3.2 Elastic Seismic Behaviour of Simple Structure

When a structure is subject to an earthquake the ground beneath the structure moves. As it moves in one direction, the mass of the structure has a natural resistance against moving due to its inertia.

However, after inertial force is applied to the mass for some time, it moves, and begins to oscillate. Additional ground shaking with time affects how it oscillates. *Figure 7* shows the displacement response versus time of 3 oscillators to a particular earthquake acceleration record. The short period structure can be considered to represent a short building, while the longer period oscillator is more likely to represent a tall building. Here, the periods, *T*, are 0.5s, 1.0s and 1.5s, and the damping ratio is 5%.

It may be seen that the response for each of these simple structures (oscillators) is quite different. In design, we generally consider the peak response over time. This can be taken from the peak response of each oscillator to obtain *earthquake response spectra*. This is shown for displacement in *Figure 7.* It can also be obtained for the acceleration of the mass in a similar way as shown in *Figure 8.*

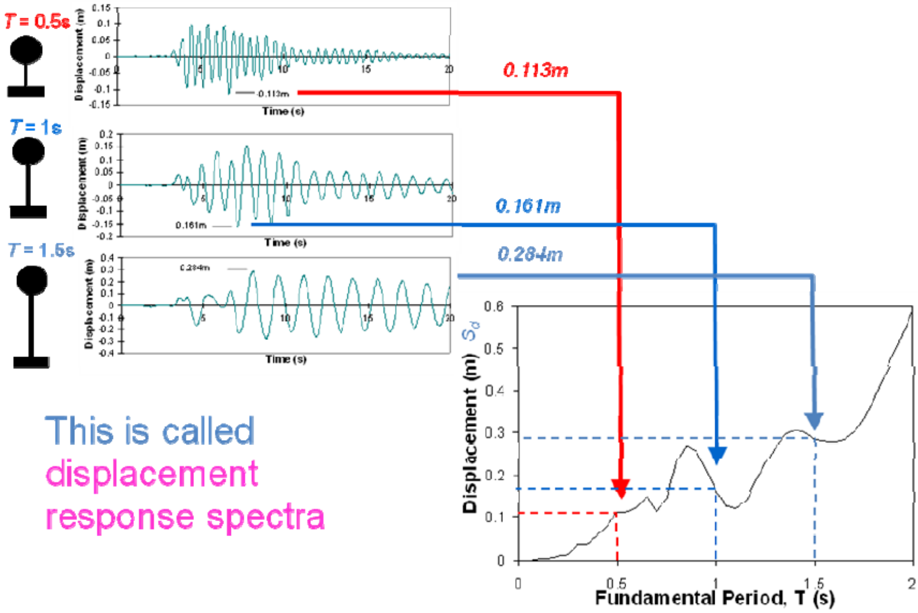


Figure 7 Development of displacement response spectra (Review of NZ Building Codes of Practice. Gregory MacRae; Charles Clifton; Les Megget. 2011)

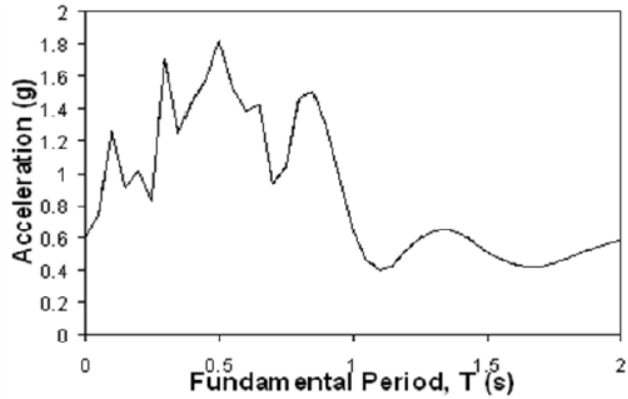


Figure 8 Example of Acceleration Response Spectra for One Earthquake Record (Review of NZ Building Codes of Practice. Gregory MacRae; Charles Clifton; Les Megget. 2011)

Because different earthquake records cause different peak accelerations at different periods, in general a smoothed acceleration response spectra is used such as the black line shown in *Figure 9.* The acceleration response spectra at a particular site depends also on the ground conditions at the site. For example, if the site has soft ground conditions, then a particular earthquake record is more likely to have a response spectrum with a greater level of shaking at longer periods, than at shorter periods as shown in *Figure 9.*

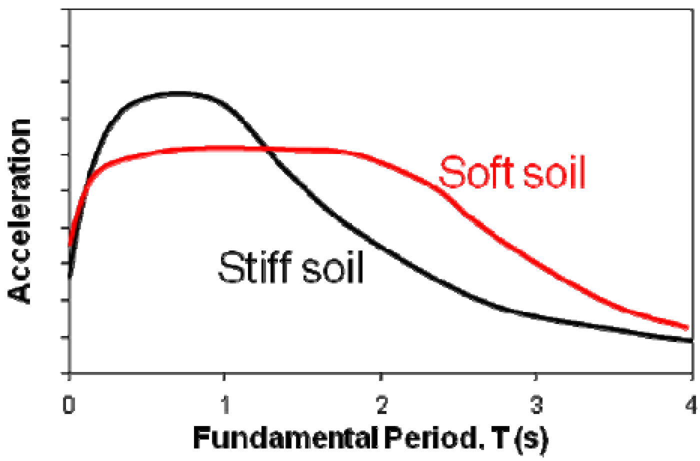


Figure 9 Idealization of Shape of Design Acceleration Response Spectra (Review of NZ Building Codes of Practice. Gregory MacRae; Charles Clifton; Les Megget. 2011)

The acceleration response is important because it is related to the level of force to which the building is subject.

Except for very short period structures, the acceleration decreases as the period of the structure increases. There is therefore an incentive for designers to use the longest period oscillators possible, because the forces are reduced. However, long period oscillators generally have larger displacements. Because of this, some sort of displacement limits often control the design of the members in the structure.

3.3 Elastic Seismic Behaviour of Multistorey Structure

In a more realistic model of a multi-storey structure, the structure does not have just one mass and one stiffness. Instead it has may have many locations of mass and different stiffnesses between these masses. Because of this it may also have multiple periods, *T*. Each period is associated with a specific mode shape of vibration, as shown for an idealized three storey structure in *Figure 10.* The fundamental period, which is associated with the first mode, generally (but not always), has the greatest accelerations and displacements. The higher modes tend to have shorter periods and hence vibrate faster than the fundamental mode. The contribution of the modes to the total response tends to decrease for higher modes.

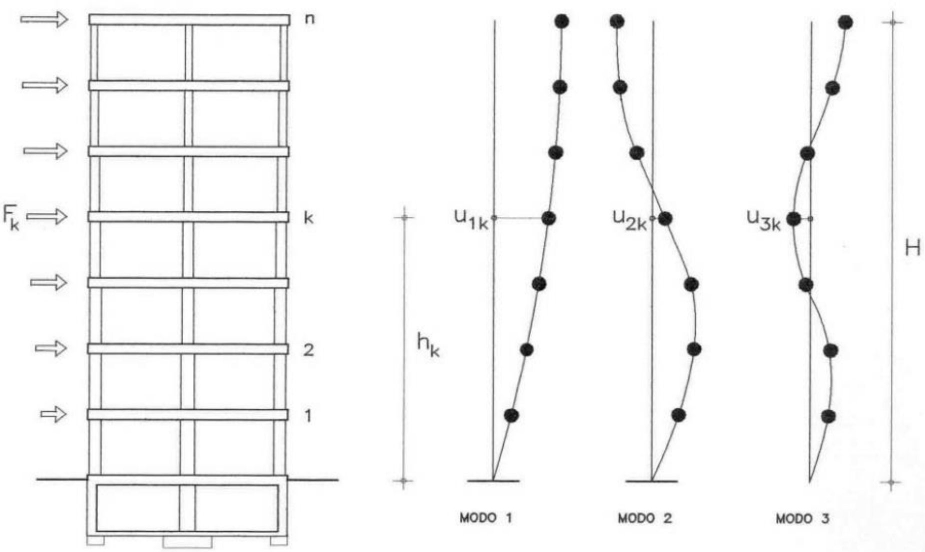


Figure 10 Modes in a Multistorey Structure (NCSE-02)

3.4 Inelastic Seismic Behaviour of Structure

While it may be possible to design for shaking associated with very strong earthquakes, this is not always done because to make the structure respond in an elastic way may result in very large member sizes and a high financial cost for the structure. Also, the behaviour of structures in previous earthquakes has indicated that they do not need to respond in an elastic way in order to remain standing and to preserve life. For these reasons, the strength provided may be much less than that expected for a design level earthquake. This is shown in *Figure 11,* where it may be seen that the level of acceleration used to provide strength to the structure in design may be much less than that expected if the structure is to remain elastic.

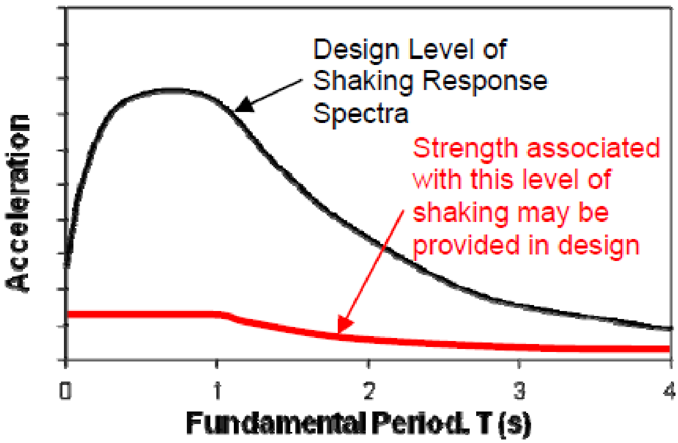


Figure 11 Design Level Shaking Expected and that used to Provide Strength (Review of NZ Building Codes of Practice. Gregory MacRae; Charles Clifton; Les Megget. 2011)

If a structure is brittle and is provided with strength lower that that expected from the design level shaking, then it is likely to collapse spectacularly during design level shaking. However, structures with significant ductility may yield a lot and have significant damage after the earthquake, but they should not collapse. An example of a brittle material is glass, which reaches its strength and breaks. An example of a ductile material is a steel wire which can sustain large deformations after it reaches its yield strength.

When a structure is required to behave in a *ductile* mode, then energy is generally dissipated. This energy dissipation may be referred to as "*controlled damage*" as the element which is undergoing the ductility may well be damaging itself in order to protect the rest of the structure. In traditional reinforced concrete and steel buildings, yielding of the steel is the method that has been used to provide the ductility and energy dissipation.

A building that has experienced significant shaking and has yielded, and used ductility, may have sustained significant damage. It may also not be vertical after the earthquake. As a result it may require major repair or it may need to be demolished. This is consistent with the current philosophy for design of ordinary structures that requires prevention of *life loss* after a design level earthquake, while damage may be acceptable. The behaviour is similar to that of a car in a crash. In a large crash, the car yields, crumples and sacrifices itself in a controlled way so that it protects the passengers, but the car itself is destroyed and cannot be used again. Providing ductility may be carried out by either:

- Providing all elements of the frame with ductility, or
- Providing some elements with ductility, and making sure that the brittle ones do not yield.

4. SEISMIC DESIGN BUILDINGS

The configuration of a building could be called its seismic form. Form to an architect means more than just shape and scale, but includes these qualities; so building configuration takes account of size and shape, but is also influenced by the location, size and nature of the structural elements, and of the non-structural elements as well.

An obvious example of poor seismic configuration is a U or L shaped building on plan, if it is not structurally divided into simply shaped blocks. Such a building may suffer damage in an earthquake because the ‘free’ ends on plan will sway in a different way to the corner section, which is stiffer. Columns on re-entrant corners are particularly prone to damage because of the concentration of forces at such points.

A building which has a simple plan form can nevertheless be badly damaged in an earthquake if it has abrupt changes in lateral stiffness, either on plan, or from one floor to the next.

Readers are referred to the book "Building Configuration and Seismic Design" by Arnold and Reitherman [2] for full treatment of the subject of configuration. It has many illustrations to clarify the subject matter. One diagram is reproduced in *Figure 12.* It shows a range of irregular structures, or framing systems, which are typical of the types of configurations seen to have performed badly in recent major earthquakes.

Elements which adversely affect a structure's seismic performance have, in practice, contributed to building failures in earthquake. Often the position, form and type of these elements are decided by the architect before any engineering analysis or detailed design is attempted. Alternatively, stiff architectural elements that affect the seismic response are added after the engineering concept has been determined. Bad structural forms and poor non-structural element configuration are often irrevocably decided at the architect's sketch design stage. Interaction between architect and engineer is required as the concept is developed.

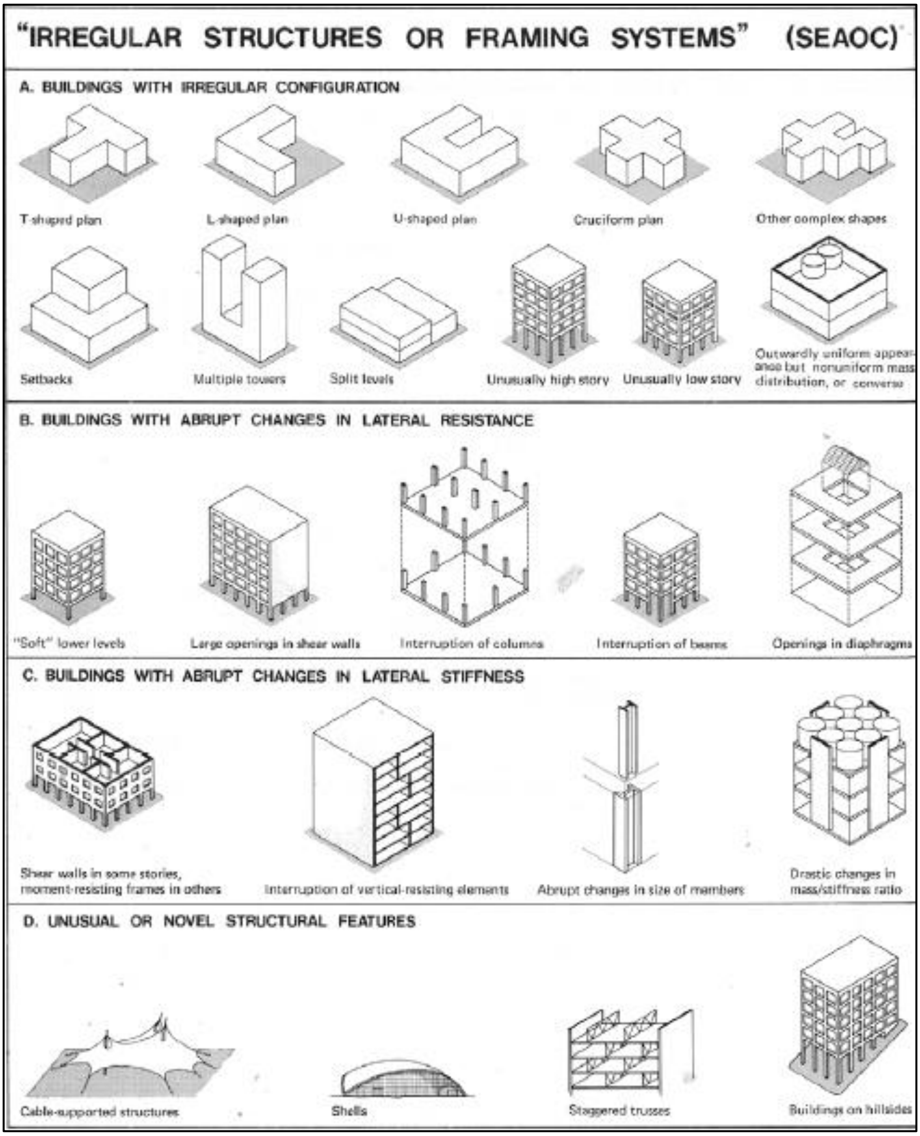


Figure 12 Graphic interpretation of irregular structures or framing systems from the Commentary to the SEAOC Recommended Lateral Force Requirements and Commentary ("Architectural Design for Earthquake". New Zealand Society for Earthquake Engineering. 2007)

4.1 Basic principles for the seismic design of buildings

4.1.1 Avoid irregular configuration

Irregular configurations are those with physical discontinuities important enough to result in stress concentrations that are problematic.

One way to achieve this is through irregular structures, which do not transmit continuously loads and can cause rotations at the ends. Therefore, it is usual to appear significant damage nonstructural elements, in the vertical structure even in the slabs. The most common solution consists in decomposing the complex structure in simple figures related by seismic joints, the width must be sufficient to prevent "*pouding*" between the structural units.

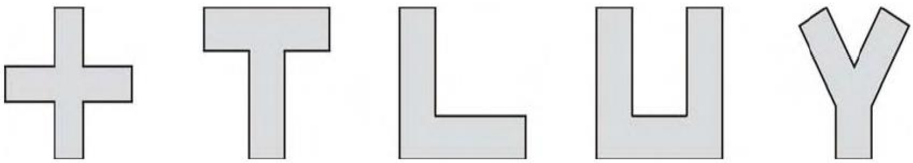


Figure 13 Irregular structures (Revista Zuncho. Nº30. Diciembre 2011. Especial Sismo)

4.1.2 Avoid soft-storey

A soft storey is one whose stiffness is significantly less that higher storeys, therefore, a storey becomes weak because its resistance is smaller than that of the next plant.

A soft storey can be given by the inclusion of a very tall plant *(Figure 14 a),* removing vertical resistant elements *(figure 14b),* shear walls interruption before reaching the foundation *(figure 14 c)* and the suppression of horizontal resistance elements *(figure 14 d).*

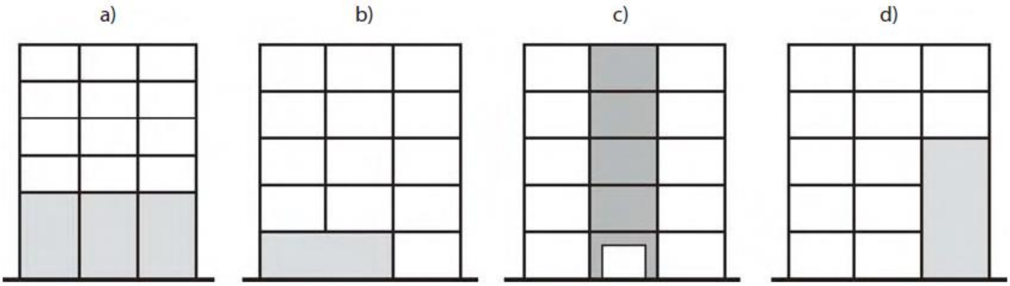


Figure 14 Soft storey (Revista Zuncho. Nº30. Diciembre 2011. Especial Sismo)

4.1.2.1 Soft-storey ground floors

Many building collapses during earthquakes may be attributed to the fact that the bracing elements, e.g. walls, which are available in the upper floors, are omitted in the ground floor and substituted by columns. Thus a ground floor that is soft in the horizontal direction is developed (soft storey). Often the columns are damaged by the cyclic displacements between the moving soil and the upper part of the building. The plastic deformations (plastic hinges) at the top and bottom end of the columns lead to a dangerous sway mechanism (storey mechanism) with a large concentration of the plastic deformations at the column ends. A collapse is often inevitable.

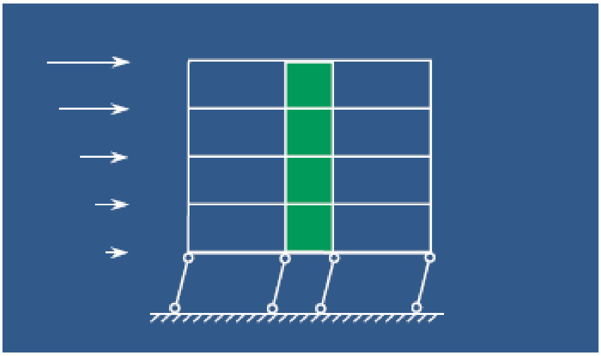


Figure 15 soft-storey ground floors (Hugo Bachmann. 2003)

Figure 16 example soft-storey ground floors [3]

Figure 17 damage at ends of pillars of soft-storey ground [4]

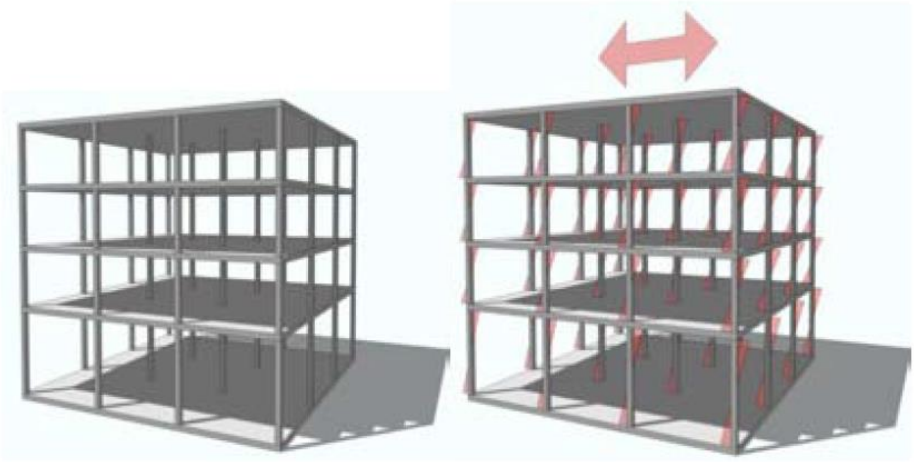


Figure 18 behavior of the building without the stiffening action of the masonry (Informe del Sismo de Lorca del 11 de Mayo de 2011)

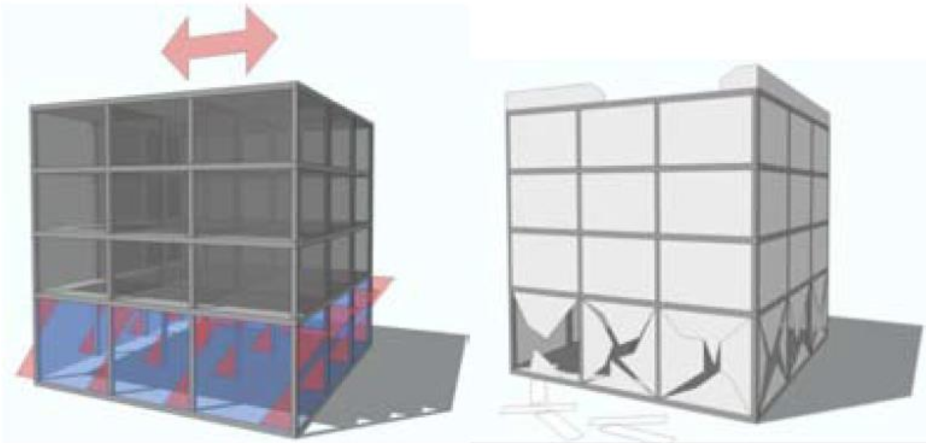


Figure 19 behavior of the building with the stiffening action of the masonry (Informe del Sismo de Lorca del 11 de Mayo de 2011)

4.1.2.2 Soft-storey upper floors

An upper storey can also be soft in comparison to the others if the lateral bracing is weakened or omitted, or if the horizontal resistance is strongly reduced above a certain floor. The consequence may again be a dangerous sway mechanism.

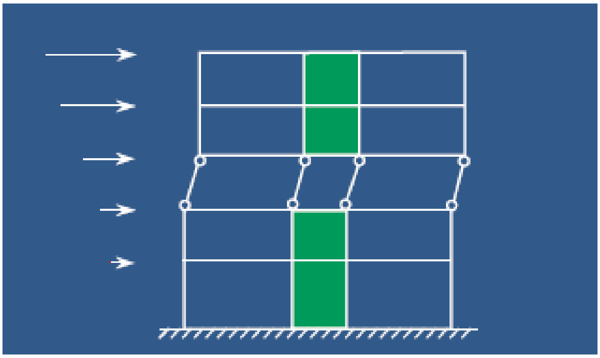


Figure 20 Soft Storey upper floors (Hugo Bachmann, 2003)

Figure 21 upper storey failed (Hugo Bachmann, 2003)

Figure 22 crushed upper floor of the office building (Kobe, Japan 1995).

4.1.3 Avoid asymmetrical horizontal bracing

Asymmetric bracing is a frequent cause of building collapses during earthquakes. In the two above sketches only the lateral bracing elements are represented (walls and trusses). The columns are not drawn because their frame action to resist horizontal forces and displacements is small. The columns, which «only» have to carry the gravity loads, should however be able to follow the horizontal displacements of the structure without loosing their load bearing capacity.

Each building in the sketch has a centre of mass M ("centre of gravity" of all the masses) through which the inertia forces are assumed to act, a centre of resistance W for horizontal forces and a centre of stiffness S (shear centre). The point W is the "centre of gravity" of the flexural and frame resistance of structural elements along the two major axes *(Figure 23).* If the centre of resistance and the centre of mass do not coincide, eccentricity and twisting occur. The building twists in the horizontal plane about the centre of stiffness. In particular, this torsion generates significant relative displacements between the bottom and top of the columns furthest away from the centre of stiffness and these often fail rapidly. Therefore the centre of resistance should coincide with, or be close to, the centre of mass, and sufficient torsional resistance should be available. This can be achieved with a symmetric arrangement of the lateral bracing elements. These should be placed, if possible, along the edges of building, or in any case sufficiently far away from the centre of mass.

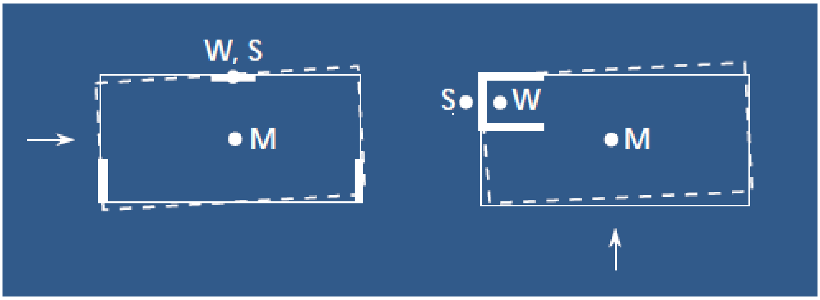


Figure 23 Asymmetric bracing (Hugo Bachmann, 2011)

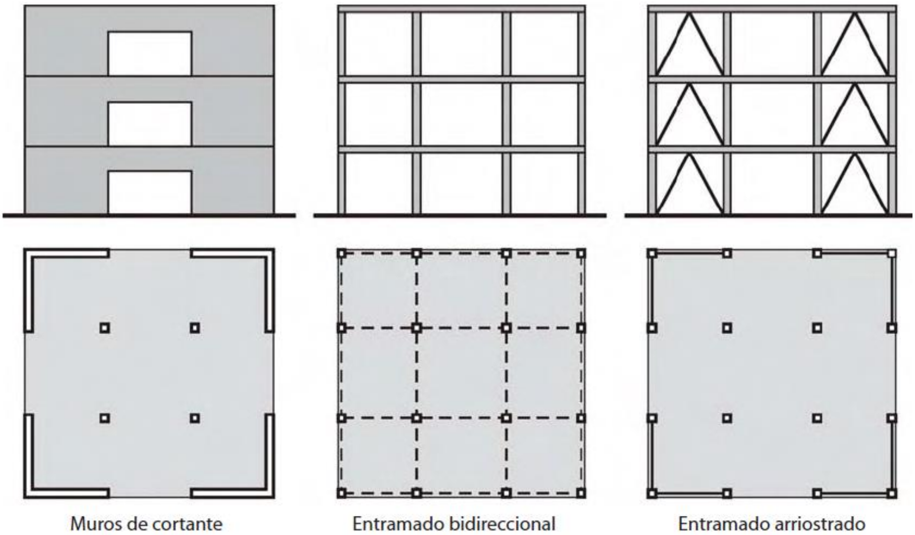


Figure 24 structural reinforcement ideal from the point of view seismic (Revista Zuncho. Nº30. Diciembre 2011. Especial Sismo)

Figure 25 torsion by the stiffening effect of the dividing walls in Lorca earthquake 2011 (Informe del Sismo de Lorca del 11 de Mayo de 2011)

4.1.4 Avoid short pillars

The seismic forces are considered to act dynamically on the oscillating mass of the building, considering the base of the building the ground plane of the ground from which emerges the structure. This usually coincides with the ground level when it is at ground level of the ground. In cases where the floor is raised over the ground, can still be considered as the base when you are swinging well stiffened. When the ground floor is elevated on pillars short, the building becomes a case of irregularity of stiffness in height.

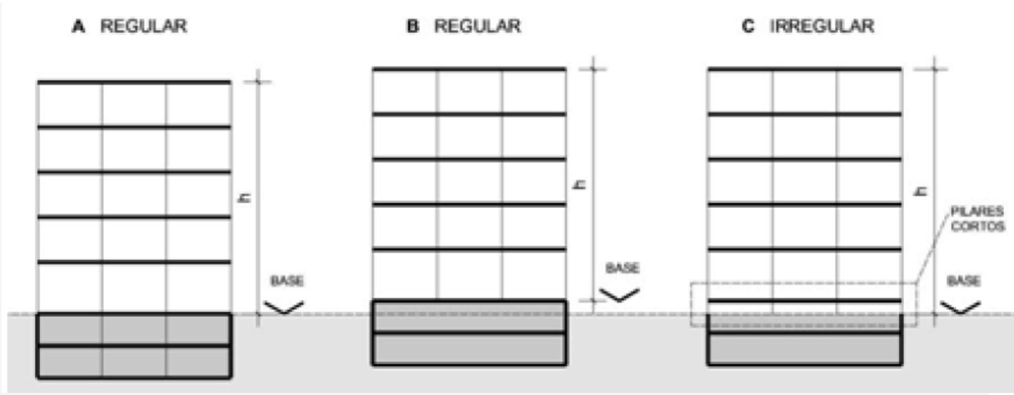


Figure 26 short pillars in the base of the building (Informe del Sismo de Lorca del 11 de Mayo de 2011)

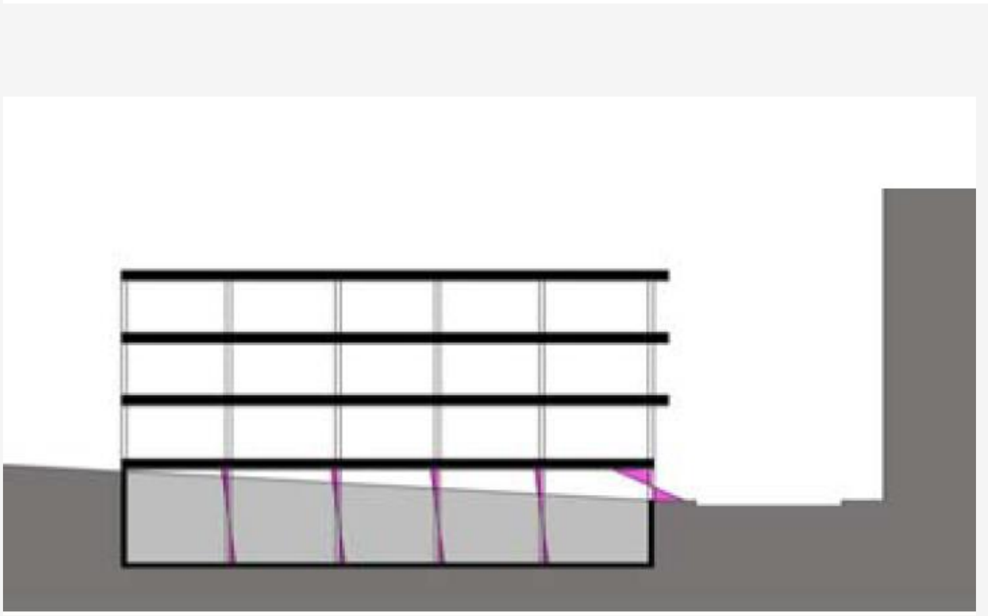


Figure 27 short pillars attract great part of the seismic load due to its high rigidity with respect to the other pillars (Informe del Sismo de Lorca del 11 de Mayo de 2011)

Figure 28 short pillars in Lorca earthquake 2011 (Tomas Espín, 2011)

4.1.5 Avoid "pounding"

Different structural units may clash repeatedly during a seismic event if there is no sufficient separation between them. This may alter and make the building response irregular and impose additional inertial forces to the structure. Damage can occur only if non-structural elements if the height of the two structures or their floors are similar, although the last structural unit may experience significant permanent displacements and even collapse *(figure 29 b)* [5].

Structural damage may be important in the following cases:

- Different height structures where the lower roof of the building impacts the central pillars of a middle floor of the tallest building *(figure 29 a)*
- Structures of different heights but similar floor height *(figure 29 c).*

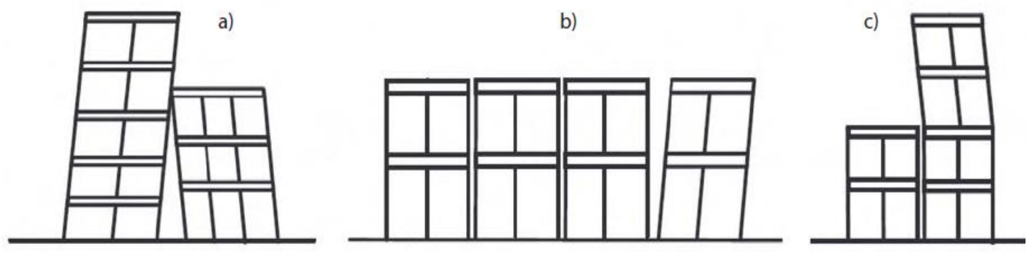


Figure 29 Examples of damage produced by pounding on the structure (Revista Zuncho. Nº30. Diciembre 2011. Especial Sismo)

Figure 30 Building damaged by pounding in Lorca earthquake (Informe del Sismo de Lorca del 11 de Mayo de 2011)

4.1.6 In general

In general the structures comply with regular configuration:

- Stability against tipping
- Reduced geometric slenderness
- Floor height equal
- Symmetry in plant on the two main axes
- Uniform elevations and sections
- Maximum torsion resistance in plant
- Direct load transmission

4.2 Behavior of nonstructural components

Effective seismic risk reduction strategies for nonstructural component damage begins by clearly understanding the scope and nature of nonstructural components in buildings, their behavior in earthquakes, and the consequences of damage.

4.2.1 Definitions

Buildings consist of both “structural” and “nonstructural” components. The distinction between the two types of building components is described below.

4.2.1.1 Structural components

The structural components of a building resist gravity, earthquake, wind, and other types of loads and typically include the following elements:

- Vertical supports such as columns, posts, pillars, and pilasters.
- Horizontal supports such as trusses, girders, beams, joists and purlins.
- Load-bearing walls that provide vertical support or lateral resistance.
- Diagonal elements such as braces.
- Floor and roof slabs, sheathing or decking.
- Foundation systems such as slabs on grade, mats, spread footings, or piles.

4.2.1.2 Nonstructural components

The nonstructural components of a building include all building parts and contents except for those previously described as structural. These components are generally specified by architects, mechanical engineers, electrical engineers, and interior designers. However, they may also be purchased and installed directly by owners or tenants after construction of a building has been completed. In commercial real estate, the architectural and mechanical, electrical, and plumbing systems may be considered a permanent part of the building and belong to the building owner; the furniture, fixtures, equipment and contents, by contrast, typically belong to the building occupants.

Nonstructural components are divided into three broad categories:

Architectural Elements

These are typically built-in nonstructural components that form part of the building. Examples include partitions and ceilings, windows, doors, lighting, interior or exterior ornamentation, exterior panels, veneer, and parapets.

Building Utility Systems

These are typically built-in nonstructural components that form part of the building. Examples include mechanical and electrical equipment and distribution systems, water, gas, electric, and sewerage piping and conduit, fire suppression systems, elevators or escalators, HVAC systems (heating, ventilating, and air conditioning), and roof-mounted solar panels.

Furniture and Contents

These are nonstructural components belonging to tenants or occupants. Examples include computer and communications equipment; cabinets and shelving for record and supply storage; library stacks; kitchen and laundry facilities; furniture; movable partitions; lockers; and vending machines. Not every conceivable item is included in these lists, so some judgment is needed to identify the critical items in a particular facility. In general, items that are taller, heavier, or important to operations, items that contain hazardous materials, and items that are more expensive should be included before items that are shorter, lighter, nonessential, inexpensive, and do not contain hazardous materials.

4.2.2 Factors affecting seismic behavior

The seismic risk for a particular nonstructural component at a particular facility is governed by a variety of factors, including:

- The regional seismicity.
- The proximity to an active fault.
- The local soil conditions.
- The dynamic characteristics of the building structure.
- The dynamic characteristics of the nonstructural element and its bracing to the structure.
- The location of the nonstructural component within the building.
- The function of the facility.
- The importance of the particular component to the operation of the facility.

4.2.3 Causes of structural damage

Earthquake ground shaking causes damage to nonstructural components in four principal ways:

- Inertial or shaking effects cause sliding, rocking or overturning.
- Building deformations damage interconnected nonstructural components.
- Separation or pounding between separate structures damage nonstructural components crossing between them.
- Interaction between adjacent nonstructural components.

4.2.3.1 Inertial forces

When a building shakes during an earthquake, the base of the building typically moves in unison with the ground. The entire building and its contents above the base experience inertial forces that push them back and forth in a direction opposite to the base excitation. In general, the earthquake inertial forces are greater if the mass of the building is greater, if the acceleration or severity of the shaking is greater, or if the location is higher than the base, where excitations are amplified. Thus, the earthquake forces experienced above the base of a building can be many times larger than those experienced at the base.

When unrestrained or marginally restrained items are shaken during an earthquake, inertial forces may cause them to slide, swing, rock, strike other objects, or overturn. File cabinets, emergency generators, suspended items, free-standing bookshelves, office equipment, and items stored on shelves or racks can all be damaged as they move and contact other items, fall, overturn or become disconnected from attached components. The shaking can also cause damage to internal components of equipment without any visible damage or movement from its original location.

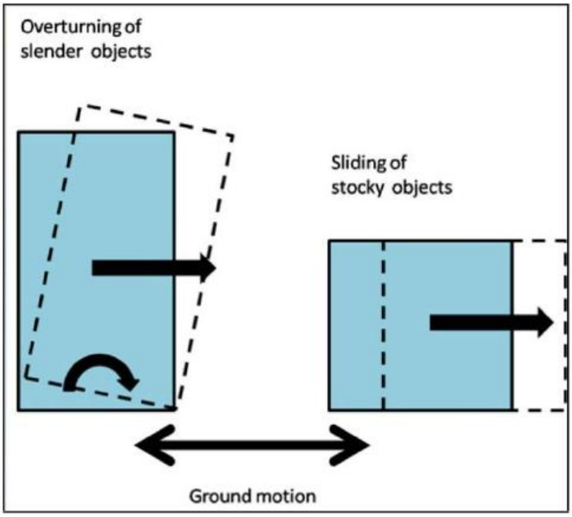


Figure 31 Sliding and overturning due to inertial forces (FEMA 74. 2011)

4.2.3.2 Building deformations

During an earthquake, structural members of buildings can deform, bend or stretch and compress in response to earthquake forces. For example, the top of a tall office tower may lean over a few feet in each direction during an earthquake. The horizontal deformation over the height of each story, known as the story drift, might range from a quarter of an inch to several inches between adjacent floors, depending on the size of the earthquake and the characteristics of the particular building structure and type of structural system. The concept of story drift is shown in *Figure 32.*

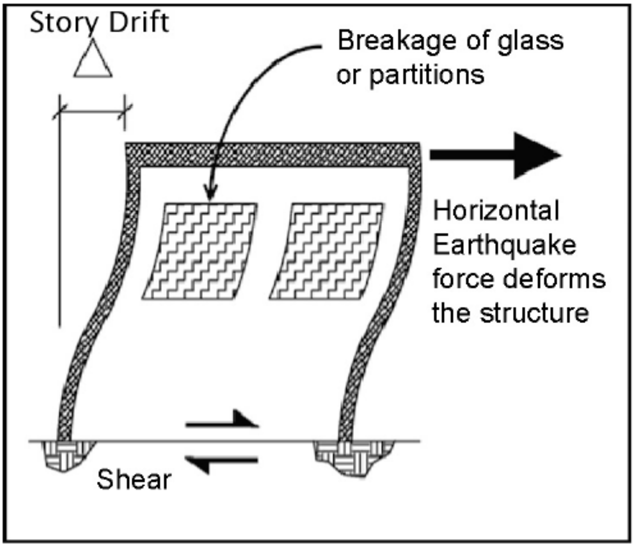


Figure 32 Nonstructural damage due to building deformation (FEMA 74. 2011)

When the building deforms, the columns or walls deform and become slightly out of square and thus, any windows or partitions rigidly attached to the structure must also deform or displace the same amount. Brittle materials like glass, plaster partitions, and masonry infill or veneer cannot tolerate any significant deformation and will crack when the space between stops or molding closes and the building structure pushes directly on the brittle elements. Once cracked, the inertial forces in the out-of-plane direction can cause portions of these architectural components to become dislodged and to fall far from their original location, possibly injuring passers-by underneath them.

4.2.3.3 Building separations

Another source of nonstructural damage involves pounding or movement across separation or expansion joints between adjacent structures or structurally independent portions of a building. A seismic joint is the separation or gap between two different building structures, often two wings of the same facility, which allows the structures to move independently of one another.

In order to provide functional continuity between adjacent structures or between structurally independent portions of a building, utilities must often extend across these building joints, and architectural finishes must be detailed to terminate on either side. The separation joint may be only an inch or two wide in older construction or a foot or more in some newer buildings, depending on the expected horizontal movement, or seismic drift between buildings. Flashing, piping, conduit, fire sprinkler lines, heating, ventilation, and air-conditioning (HVAC) ducts, partitions, and flooring all have to be detailed to accommodate the seismic movement expected at these locations when the two structures move closer together or further apart. Damage to items crossing seismic separation or expansion joints is a common type of earthquake damage. If the size of the gap is insufficient, pounding between adjacent structures may result, which can damage structural components but more often causes damage to nonstructural components, such as parapets, veneer, or cornices on the façades of older buildings.

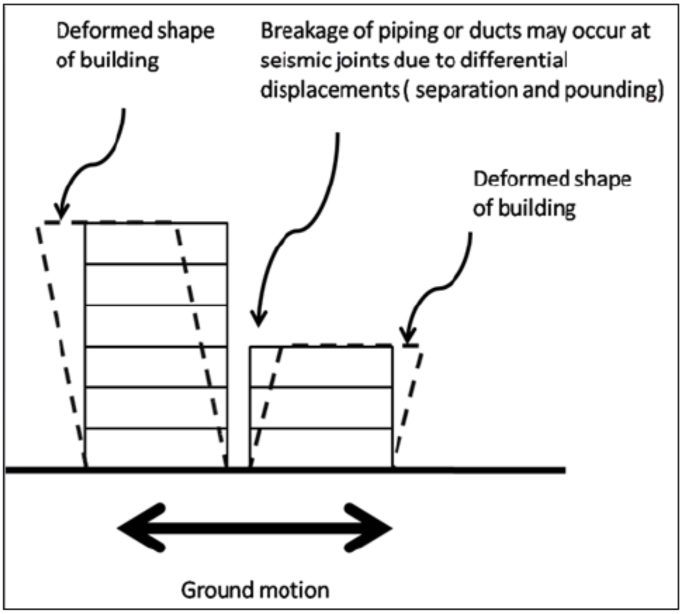


Figure 33 Nonstructural damage due to separation and pounding (FEMA 74. 2011)

4.2.3.4 Nonstructural interaction

An additional source of nonstructural damage is the interaction between adjacent nonstructural systems which move differently from one another. Many nonstructural components may share the same space in a ceiling plenum or pipe chase; these items may have different shapes, sizes, and dynamic characteristics, as well as different bracing requirements.

Some examples of damaging nonstructural interactions include:

- Sprinkler distribution lines interact with the ceiling causing the sprinkler heads to break and leak water into the room below.
- Adjacent pipes of differing shapes or sizes are unbraced and collide with one another or adjacent objects.
- Suspended mechanical equipment swings and impacts a window, louver, or partition.
- Ceiling components or equipment can fall, slide, or overturn blocking emergency exits.

5. IMPORTANCE OF NONSTRUCTURAL DAMAGE

Historically, earthquake engineers have focused on the performance of structural systems and ways to mitigate structural damage. As the earthquake engineering community moves toward more comprehensive earthquake standards and expectations of improved seismic performance, and as the public demands a higher level of earthquake protection, it is important to understand the significance of nonstructural damage.

The failures of nonstructural components during an earthquake may result in injuries or fatalities, cause costly property damage to buildings and their contents; and force the closure of residential, medical and manufacturing facilities, businesses, and government offices until appropriate repairs are completed. As stated previously, the largest investment in most buildings is in the nonstructural components and contents; the failures of these elements may be both dangerous and costly. The potential consequences of earthquake damage to nonstructural components are typically divided into three types of risk:

- Life Safety (LS): Could anyone be hurt by this component in an earthquake?
- Property Loss (PL): Could a large property loss result?
- Functional Loss (FL): Could the loss of this component cause an outage or interruption?

Damage to a particular nonstructural item may present differing degrees of risk in each of these three categories. In addition, damage to the item may result in direct injury or loss, or the injury or loss may be a secondary effect or a consequence of the failure of the item.

5.1 Types of risk

5.1.1 Life safety (LS)

The first type of risk is that people could be injured or killed by damaged or falling nonstructural components. Even seemingly harmless items can cause death if they fall on a victim. If a light fixture not properly fastened to the ceiling breaks loose during an earthquake and falls on someone's head, the potential for injury is great. Life safety can also be compromised if the damaged nonstructural components block safe exits in a building. Damage to life safety systems such as fire protection piping can also pose a safety concern should a fire start following an earthquake. Examples of potentially hazardous nonstructural damage that have occurred during past earthquakes include broken glass, overturned tall, heavy cabinets and shelves, falling ceilings and overhead light fixtures, ruptured gas lines and other piping containing hazardous materials, damaged friable asbestos materials, falling pieces of decorative brickwork and precast concrete panels, dislodged contents stored overhead, and collapsed masonry parapets, infill walls, chimneys, and fences.

The 2011 Lorca earthquake was a moderate magnitude 5.1 M_w earthquake that caused significant localized damage in the Region of Murcia, Spain. Centred at a very shallow depth of 1 km near the town of Lorca, it occurred at 18:47 on 11 May 2011, causing panic among locals and displacing many from their homes. The quake was preceded by a magnitude 4.4 M_w foreshock at 17:05 that inflicted substantial damage to many older structures in the area. Three people were killed by a falling cornice [6]. A total of nine deaths have been confirmed, while dozens are reported injured [7]. The earthquake was the worst to hit the region since a 5.0 M_w tremor struck west of Albolote, Granada in 1956 [8].

5.1.2 Property loss (PL)

As discussed previously, nonstructural components, such as mechanical and electrical equipment and distribution systems and architectural components, account for 75-85% of the original construction costs of a typical commercial building. Contents belonging to the building occupants, such as movable partitions, furniture, and office or medical equipment, represent a significant additional value at risk. For example, a high tech fabricating facility may have contents that are worth many times the value of the building and built-in components of the building. Immediate property losses attributable to contents alone are often estimated to be one third of the total earthquake losses [9].

Property losses may be the result of direct damage to a nonstructural item or of the consequences produced by its damage. If water pipes or fire sprinkler lines break, then the overall property losses will include the cost to repair the piping (a primary or direct loss), plus the cost to repair water damage to the facility (a secondary or indirect loss). If the gas supply line for a water heater ruptures and causes a fire, then clearly the property loss will be much greater than the cost of a new pipe fitting. Many offices and small businesses suffer losses as a result of nonstructural earthquake damage but may not keep track of these losses unless they have earthquake insurance that will help to cover the cleanup and repair costs.

5.1.3 Functional loss (FL)

In addition to life safety and property loss considerations, there is the additional possibility that nonstructural damage will make it difficult or impossible to carry out the functions that were normally accomplished in a facility. After life safety threats have been addressed, the potential for post earthquake downtime or reduced productivity is often the most important risk. For example, if a business loses the use of its computers, filing system or other instruments of service as a result of earthquake damage, then the money loss of replacing the damaged items may be relatively small, but the loss in revenue associated with downtime during recovery can be tremendous. In light of the global economy, loss of function can also translate to longer term loss of market share for some businesses as consumers find alternate suppliers for needed goods or services.

Many external factors may affect post earthquake operations, including power and water outages, damage to transportation systems, availability of materials and contractors to repair damage, civil disorder, police lines, and curfews. These effects are generally outside the control of building owners and tenants and beyond the scope of this discussion.

5.2 Common types of nonstructural earthquake damage

5.2.1 Heavy exterior cladding

Cladding is an architectural element used to provide the exterior skin for buildings. Often constructed of heavy precast concrete panels, these panels typically have four support points, two at the top of the panel connecting it to the beam above, and two at its base connected to the level below. Unless specifically designed to accommodate the anticipated inter-story drift and out-of-plane seismic forces, these supports can fail.

Figure 34 Failure Cladding of a building in Lorca Earthquake 2011 (Tomas Espín, 2011)

5.2.2 Heavy interior walls

Nonstructural walls in older buildings are often built of heavy, unreinforced masonry materials such as brick, concrete block, or hollow clay tile. These materials are advantageous for fire and sound proofing and thermal insulation, but are brittle since they do not have a grid of horizontal and vertical steel reinforcing bars embedded in them. Falling masonry in hallways and stairwells is a particular hazard for occupants attempting to exit buildings during an earthquake.

Figure 35 Damage interior walls. Lorca Earthquake 2011 (Tomas Espín, 2011)

5.2.3 Unbraced masonry parapets or other heavy building appendages

Unreinforced masonry parapets are a common feature of vintage commercial construction in many parts of the country. Parapets are the short walls around the perimeter of a roof, constructed to help prevent fire from jumping from one roof to the next, to provide guardrail protection for people on the roof, to hide roof-mounted equipment, or to provide an architectural effect of greater height. While some communities have enforced ordinances that require unreinforced masonry parapets to be braced or anchored, many jurisdictions have no such mandatory provisions. As these parapets often fail at the roofline and fall outwards onto the sidewalk, they represent a particular hazard for pedestrians and occupants attempting to exit damaged buildings.

Figure 36 Failure in the cornice of a building in Lorca Earthquake 2011 (Tomas Espín, 2011)

5.2.4 Unreinforced masonry chimneys

Residential chimneys are typically built of brittle unreinforced brick masonry that may be damaged even in relatively small earthquakes. This is also true of many commercial chimneys. Broken chimneys can fall through the roof and pose a safety risk to building occupants.

Figure 37 Collapse of a chimney on the roof of a building (Revista Zuncho. Nº30. Diciembre 2011. Especial Sismo)

5.2.5 Suspended lighting

Suspended overhead lighting is prone to damage in earthquakes, especially if the lights are supported solely by unbraced suspended ceilings, or if they interact with unbraced piping or other suspended components.

5.2.6 Large, heavy ceilings

Heavy suspended ceilings and soffits can be damaged during earthquakes, sometimes causing heavy and dangerous material to fall and injure people below.

Figure 38 Failure of suspended ceilings and light fixtures in Lorca earthquake 2011 (Tomas Espín, 2011)

Figure 39 Failure of office partitions, ceilings, and light fixtures in the 1994 Northridge Earthquake (FEMA 74, 1994).

5.2.7 Tall, slender, and heavy furniture such as bookcases and file cabinets

Tall slender shelving, bookcases, or file cabinets frequently overturn during earthquakes if they are unanchored or poorly anchored. These items are particularly hazardous if they are located adjacent to a desk or bed or located where they can jam doors or block corridors and exits.

Figure 40 Damage to overloaded racks during the 1994 magnitude-6.7 Northridge Earthquake (FEMA 460, 2005).

5.2.8 Heavy unanchored or poorly anchored contents, such as televisions, computer monitors, countertop laboratory equipment, and microwaves

Heavy contents situated above the floor level include a wide range of items that could become falling hazards in an earthquake. Many rooms have overhead wall- or ceiling-mounted televisions and monitors, offices have desktop computer monitors, or microwaves may be perched high on counters or shelves. Any of these items could cause injury if they fell and hit someone; damage to fallen items can add to property loss and downtime.

5.2.9 Glazing

Damage to storefront windows in older commercial buildings is common during earthquakes, often causing hazardous conditions on sidewalks in commercial areas.

Figure 41 Shards of window pane, 1994 Northridge Earthquake (Robert Reitherman, 2009)

Figure 42 Drift-caused damage to windows in the International Building in Mexico City in the M 8.1, September 19, 1985 Mexico Earthquake (Robert Reitherman, 2009)

5.2.10 Fire protection piping

Damage to suspended fire protection piping and other system components can render the system inoperable following an earthquake. The resultant loss of fire life safety protection can pose a serious risk to the life safety of building occupants.

5.2.11 Hazardous materials release

There have been a number of examples of hazardous materials release resulting from earthquake damage to piping, stored chemicals, commercial, medical, or educational laboratory facilities. Breakage of containers of chemicals can cause them to mix and lead to hazardous reactions. Exposure of asbestos materials due to earthquake activity has also resulted in the post earthquake evacuation of facilities that otherwise had little structural damage.

Figure 43 Overturned medical gas cylinders, 1971 San Fernando Earthquake (Scientific Service, Inc.)

5.2.12 Gas water heaters

Residential and small commercial water heaters have ignited fires following earthquakes, in instances where the gas supply line was damaged. As water heaters are typically tall and slender, the gas supply line can break if the water heater tips over.

5.2.13 Suspended piping for water or waste

Failures of suspended piping have lead to costly property loss in past earthquakes. While such failures are not often associated with life threatening injuries, they often result in costly property loss: both the cost to replace the damaged system and the cost to repair damage caused by the release of both clean and contaminated or hazardous fluids. Secondary damage due to fluid release is often a large component of nonstructural property losses.

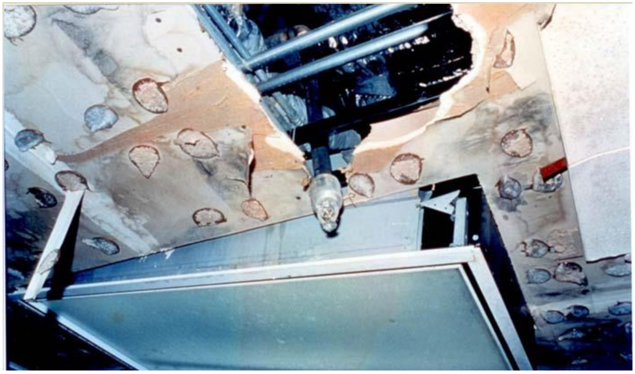


Figure 44 Broken sprinkler pipe at Olive View Hospital in Sylmar, California as a result of the 1994 Northridge, Earthquake. Pipe ruptured at the elbow joint due to differential motion of the pipe and ceiling (FEMA 74, 1994).

6. REGULATORY FRAMEWORKS

Of course, an important starting point in the study and evaluation of this topic is the consideration and the comparison of several local regulations and their approach to the seismic design of non-structural elements.

The reason of the importance of considering and evaluating the possible differences among different approaches to the same problem is clearly understandable. These national regulations represent the expression of the "state of the art" about this topic of a specific country or region of the world. Consequently a designer that has to deal with the project of a non-structural element must meet the requirements contained and expressed by these standards. Furthermore, an additional value is given by the importance of considering and comparing the approach to the same problem from extremely different points of view, such as those that can result from countries and regions of the world with deeply different traditions, building technologies and know-how.

Through the Tectonic Plate theory is possible to subdivide the whole earth crust in several huge rigid plates that slide and move one against the others [10]. As a consequence different high seismic regions are present around the world. In these geographical areas the probability to experience an earthquake in a specified period of time is much higher than everywhere else.

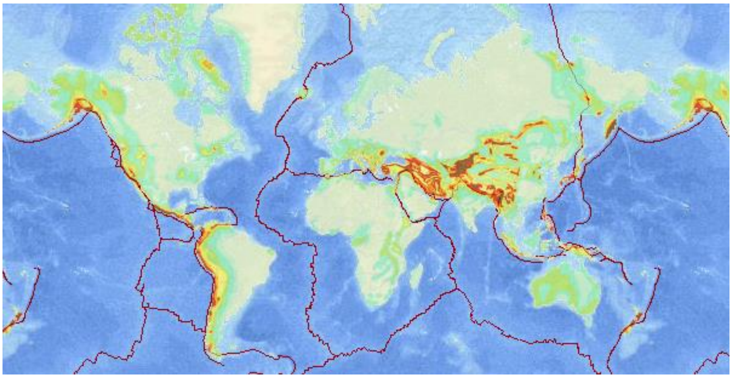


Figure 45 Real-time Earthquake Map (http://www.usgs.gov/)

California

It is the case of the whole western area of North America, especially located in California, where the Pacific Plate (on the west) meets with the North American Plate (on the east) along the famous San Andreas fault, 1100 km long, the principal element of the San Andreas fault system where concentrates a very high seismic activity.

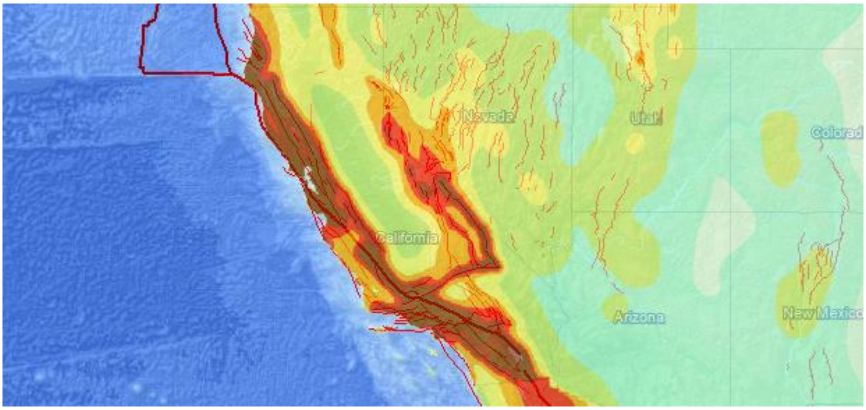


Figure 46 Real-time Earthquake Map. California (http://www.usgs.gov/)

Figure 47 San Andreas Fault zone, Carrizo Plains, central California. (http://www.usgs.gov/)

Southeast Asia

Another high seismic region is the Southeast Asia area [11], where several plates meet in the south region of Indonesia. The seismic activity is mainly focused in the so called “Sunda subduction zone” that is divided into four sections based on seismicity characteristics: Burma, Northern Sumatra-Andaman, Southern Sumatra and Java.

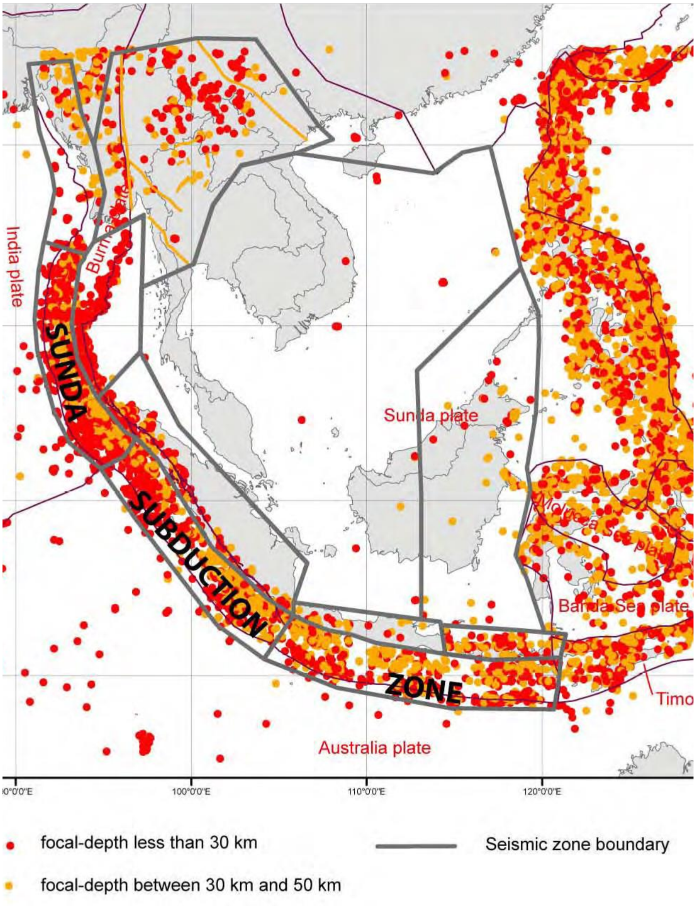


Figure 48 Map of shallow-depth earthquakes in the Southeast Asia. “Sunda Bubduction zone”. (U.S. Geological Survey, Documentation for the Southeast Asia Seismic Hazard Maps, Virginia 2007)

Japan

One of the other most active regions in the world is, of course, the Japanese area, where earthquakes are basically generated by the Pacific Plate moving westward and being subducted beneath the northern part of Japan, which is located on the Okhotsk Plate [12, 13].

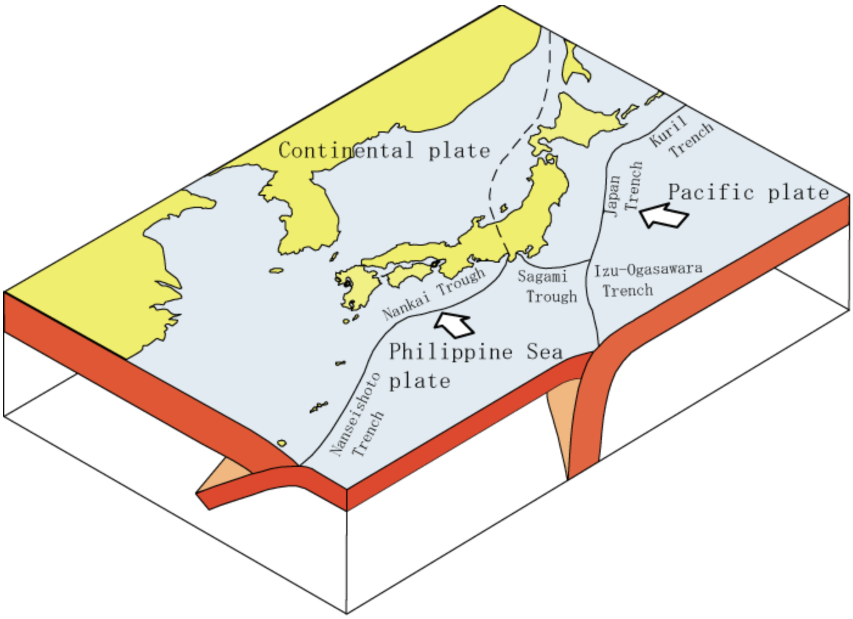


Figure 49 Tectonic plates in the region of the Japan Islands. (Earthquake Research Committee, National Seismic Hazard Maps for Japan, 2005)

These movements were able to cause the most powerful earthquake recorded in Japanese history, 8.9 magnitude, and the sixth largest earthquake in the world since 1900, when seismological records began. This event, besides the extraordinary magnitude, was also extremely devastating because of the tsunami that was triggered off by the subduction movement of the Pacific Plate that lifted up the Okhotsk Plate, around 200 km far from the Japanese coast, at a depth of 30 km.

New Zealand

Earthquakes in New Zealand are due to the country being part of the Pacific Ring of Fire, which is geologically active. About 20,000 earthquakes, most of them minor, are recorded each year [14]. About 200 of these are strong enough to be felt [15]. As a result, New Zealand has very stringent building regulations.

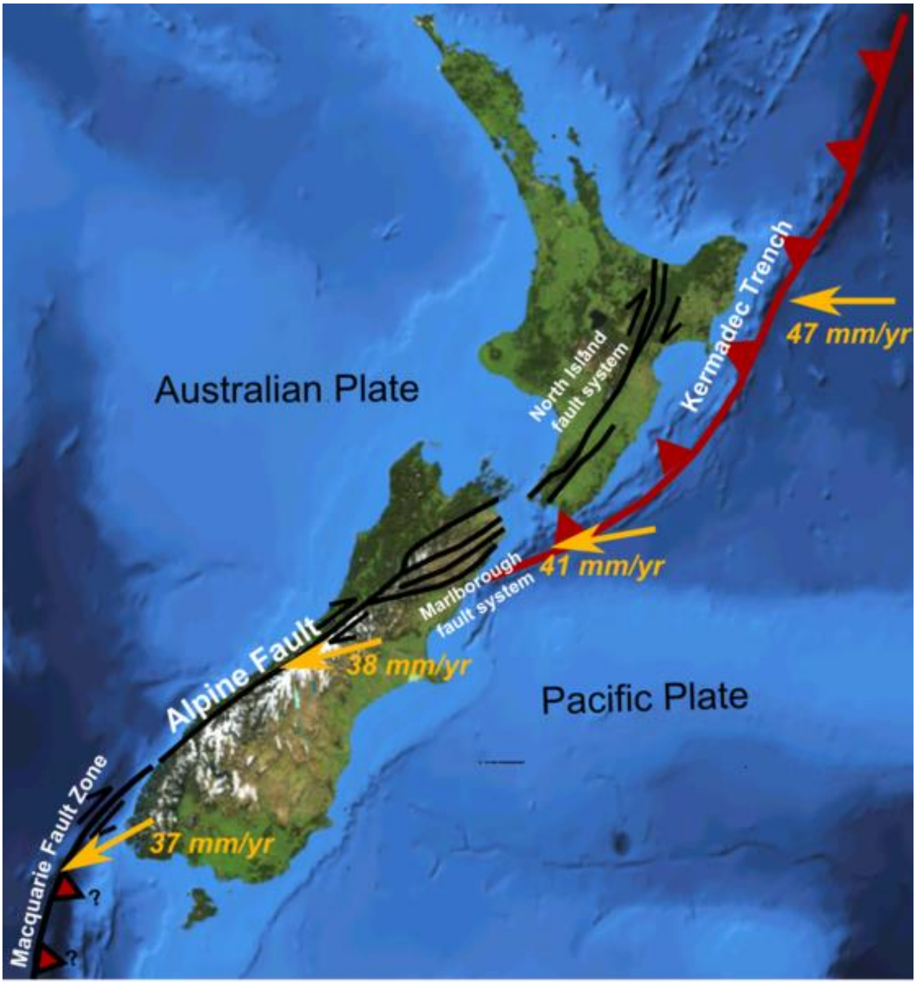


Figure 50 Fault zones associated with the plate boundary in New Zealand. (Mikenorton)

6.1 European regulation: Eurocode 8

Within the European Standard [16] Chapter 4.3 is dedicated to the structural analysis. Here is possible to find, at section 4.3.5, the Eurocode 8 requirements for building non-structural elements. Firstly, an interesting point to be analysed is how the standard classify non-structural elements. They are named as “appendages”, to clearly identify with just a word their main characteristic behaviour that is to be attached to structural elements. Then some examples are listed: “parapets, gables, antennae, mechanical appendages and equipment, curtain walls, partitions, railings”.

The very first general requirement of the standard requests that all these building elements “shall, together with their supports, be verified to resist the design seismic action”. Therefore it is possible to notice how a great importance is already given to the supporting and fastening system. This system in fact has a key role in the failure mechanism of the non-structural element.

Subsequently, before proceeding to the simplified method, it is specified that in case of “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.”

For all other cases a simplified method is described, consisting in the verification of a static seismic horizontal force Fa application to the considered non-structural element.

Considerations

Eurocode 8 underlines the importance of non-structural elements design for the general safety of persons and for the utilization of the building itself, stating that their failure could expose people to a serious hazard and affect building structure and facilities.

Moreover it concentrates upon the design of the fastening and supporting system, explicitly requiring its verification to resist the design seismic action. So it recognizes brackets and other fastening devices to be of great importance in the seismic resistance behaviour of the non-structural element.

Eurocode 8 utilizes a simplified method for determining the seismic action to be considered, calculating a static horizontal force that has to be applied to the non-structural element centre of mass. Within this seismic force calculation the standard uses different factors, such as the importance factor γa and the behaviour factor qa, to take into account the element importance and behaviour. Nevertheless there is not any consideration upon the building importance and behaviour or typology. Hence a non-structural element in a low-rise, normal or low-occupied and secondary important building will be subjected and verified for the application of the same seismic design action of a high-occupancy, medium to high-rise and maybe critical building, such as, for example, a hospital.

6.2 American regulation: ASCE 7 - 10

ASCE Standard 7-10 [17] provides requirements for general structural design and includes means for determining dead, live, soil, flood, wind, snow, rain, atmospheric ice, and earthquake loads, and their combinations that are suitable for inclusion in building codes and other documents. A detailed commentary containing explanatory and supplementary information to assist users of ASCE 7-10 is included with each chapter. ASCE 7-10 is an integral part of the building codes of the United States.

The current code requirements for nonstructural components are contained in ASCE/SEI 7-10 Section 13 which is adopted by reference in IBC 2012 (ICC, 2012) [18]. In recent years, engineers, researchers, and code committees have paid increasing attention to the issues of nonstructural performance. As a result, ASCE/SEI 7-10 now includes a 15-page chapter devoted to nonstructural components and contains design requirements for both force- and displacement-controlled nonstructural components. In contrast, the Uniform Building Code (ICBO, 1994) [19] covered the nonstructural requirements in less than two pages, where the focus of the requirements was primarily on position retention of the components. The requirements are now more detailed and include explicit provisions for more items that apply to facilities that require post earthquake functionality.

Moreover the National Earthquake Hazards Reduction Program (NEHRP) was established by the U.S. Congress when it passed the Earthquake Hazards Reduction Act of 1977, Public Law (PL) 95-124 [20]. In its initial NEHRP authorization in 1977, and in subsequent reauthorizations, U.S. Congress has recognized that several key Federal agencies can contribute to earthquake mitigation efforts. Today, there are four primary NEHRP agencies:

- Federal Emergency Management Agency (FEMA) of the Department of Homeland Security;
- National Institute of Standards and Technology (NIST) of the Department of Commerce (NIST is the lead NEHRP agency);
- National Science Foundation (NSF);
- United States Geological Survey (USGS) of the Department of the Interior.

One of the goals of the Department of Homeland Security's FEMA and the NEHRP is to encourage design and building practices that address the earthquake hazard and minimize the resulting risk of damage and injury. Highlights the content of the 2003 edition of the NEHRP Recommended Provisions for Seismic Regulation of New Buildings and Other Structures (FEMA 450-1/2003 Edition) [21], consisting in criteria and requirements for the design and verification of building subjected to earthquakes ground motion.

Considerations

This code provides a calculation method based on several coefficients to obtain the seismic force to be applied to the considered non-structural element centre of mass.

As well, some others requirements are provided for exterior non structural wall panels or elements, mainly about the possibility for the façade to accommodate and permit the inter-storey drift of the building structure without damages and specifically focusing on the fastening system design details.

6.3 New Zealand regulation: NZS 1170.5

The New Zealand Standard NZS 1170.5 [22] sets out procedures and criteria for establishing the earthquake actions to be used in the limit state design of structures and parts of structures within New Zealand. The section of the standard that has to be considered in this case is Section 8, "Requirements for parts and components", where it is required that all parts of structures, including permanent, non-structural components and their connection, and permanent services and equipment supported by structures, shall be designed for the earthquake actions specified. All these elements are recognised and named “parts” in the code.

Considerations

New Zealand Regulation requires the verification of non-structural elements, called “parts”, under the application of a seismic action to the centre of mass of the element. The equation provided and its coefficients are very similar to the calculation method prescribed by the European Regulation Eurocode 8. The seismic action to be applied is not only a horizontal force but also a vertical oriented one. In particular, NZS requires also a specific minimum ductility factor, equal to 1,25, of the considered part.

6.4 Japan regulation

Seismic design requirements for building mechanical equipment and nonstructural components are provided by two documents:

- Guideline for Aseismic Design and Construction of Building Equipment, published by the Building Center of Japan (1984).
- Guideline for Aseismic Design for Architectural Nonstructural Elements, published by Public Building Association (1987).

The coefficients used in determining the design seismic force are similar between these two guidelines when the Modified Seismic Coefficient Method is used as the design method [23, 24].

Considerations

Non Structural components and their attachment shall be designed to resist the total lateral force and vertical design seismic force and the seismic displacement requirement aren’t more specific than other building codes in prescribing the required seismic lateral displacement.

6.5 Spain regulation: NCSE – 02

The Technical Building Code (CTE) is the regulatory framework that establishes the requirements to be met by buildings in relation to the basic requirements of safety and habitability established by Law 38/1999 of November 5, Building Management (LOE) [25].

In the Basic Document (DB) Structural Safety - Actions Building (SE-AE), which is to establish rules and procedures to meet the basic requirements for structural safety found in paragraph 4.1 "Accidental actions earthquake" which leads the standard seismic resistant NCSE-02.

The standard NCSE-02 in Chapter 4, "Design rules and regulations in building construction" Section 4.2.4 Non-structural elements are, and also refer to it in section 4.7.

Considerations

The standard NCSE-02 says generally that the elements that can form the structure stiffness are taken into account in the calculation but does not say anything specific. It also mentions that the escape routes, especially stairs, must be provided with greater strength and ductility.

6.6 Comparison of code seismic design requirements

As can be seen previously, there are noticeable variations in code requirements for nonstructural building components, both in terms of level of detail in the requirements and in the calculation procedures. Some codes have more detailed descriptions of nonstructural building components and assign more specific coefficients to various components, while others are less specific in listing the applicable components. In such cases the seismic coefficients necessary for computing seismic lateral force and displacement requirements can only be estimated.

Some codes are more explicit than others, and the scope varies by country. *Table 2* shows generally an analysis of every scope according to the standard adopted.

Scope	Eurocode 8	ASCE 7-10	NZS 1170.5	Japan	NCSE-02
Seismic Design Force	✓	✓	✓	✓	x
Anchorages/ Connections	✓	✓	✓	x	x
Architectural Components	✓	✓	x	✓	✓
Mechanical/Electrical Components	x	✓	x	x	x

Table 2 Scope Building codes

6.6.1 Seismic design coefficients

We can see variations in seismic design requirements between codes, both in terms of seismic force and displacement calculations and in listings of nonstructural components and corresponding coefficients.

For the seismic force requirement, the building codes use three basic coefficients to account for the following factors in prescribing the design force:

- Seismicity of the region: where the building is located (seismic zone factor or effective peak-velocity acceleration, or component acceleration coefficient).

- Functionally of nonstructural component and buildings: in terms of life-safety importance (seismic importance factor, or component risk factor, or component performance criteria).
- Response characteristics of nonstructural components: to seismic lateral load (component seismic coefficient, or component horizontal force factor, or component response amplification factor).

Other factors not explicitly included in the seismic design requirements are:

- Response characteristics of the building: to seismic lateral load (building seismic coefficient).
- Site soil profile (building seismic coefficient).
- Component location: relative to building height (floor response amplification factor).

Table 3 summarizes the coefficients affecting the calculation of seismic lateral force of the codes reviewed.

Coefficients to account for…	Eurocode 8	ASCE 7-10	NZS 1170.5	Japan	NCSE-02
Seismicity of Region where buildings are located	Implied in S_a Depends on: S: soil factor α: ratio of the design ground acceleration on type A ground, a_g, to the acceleration of gravity g	Implied in S_{DS} *Ss* = the mapped MCE (Maximum Considered Earthquake) spectral response acceleration at short Periods	Hazard Factor: Z (0.13 – 0.60)	Seismic zone factor: Z (0.7 – 1.0)	-
Functionally of components in terms of life-safety importance	Importance factors: γ_a (1.0 - >1.5)	Component importance factor: I_p (1.0 – 1.5)	Part risk factor: R_p (0.9 – 2.0) Varies with the degree of affect on life-safety	Seismic importance reduction factor: I (1.0 or 2/3)	-
Component seismic response characteristics	Behaviour factor of the element	Component response modification	Part response factor:	Response amplification factor:	-

	q_a or 2.0)	factor: R_p (1.0 - 12.0) Depends on type of components Component amplification factor: a_p (1.5 – 2.5)	C_{ph} (0.45 – 1.0) through component ductility factor μp	K_2 (1.0 – 2.0)	
Building response characteristics and soil site profile	Implied in seismic coefficient S_a	Implied in component acceleration coefficient a_p	Implied in seismic coefficient C_{ph}	-	-
Component location relative to building height	Implied in seismic coefficient S_a	Implied in component acceleration coefficient a_p	Implied in seismic coefficient C_{ph}	Floor response amplification factor K_1 – 10/3) Varies with the height	-

Table 3 Coefficients Affecting Seismic Force Requirements in Various Building Codes

Finally the *Table 4* shows the formulas for nonstructural design according to each standard.

Eurocode 8	ASCE 7-10	NZS 1170.5	Japan
$Fa = \frac{Sa * Wa * \gamma a}{q_a}$	$Fp = \frac{0.4a_p S_{DS} W_p}{\left(\frac{R_p}{I_p}\right)}\left(1 + 2\frac{z}{h}\right)$	$Fpv = Cpv\ Cvd\ Rp\ Wp\ \leq 2.5\ Wp$ $Fph = Cp\ (Tp) Cph\ Rp\ Wp\ \leq 3.6\ Wp$	$K_H = ZIK_1K_2K_0$ $F_H = K_H W$ $F_v = K_v W$ $K_v = K_H/2$

Table 4 Seismic Force Requirement

6.6.2 Displacements:

For the seismic displacements requirement the ASCE 7-10 are more specifics than other building codes in prescribing the required seismic lateral displacement. It provides formulas for calculating the displacement at point of support for nonstructural components. The effects of seismic relative displacements shall be considered in combination with displacements caused by other loads as appropriate.

Japan is similar to other codes; seismic displacements requirements are not as specific as the seismic force requirements. The current building codes of Japan require that for pipes, vertical pipes shall be subjected to a maximum story drift of 1/200 radian times the story height. Pipes through expansion joints shall be designed for possible maximum relative displacement between two structures.

6.6.3 Anchorages/connections:

Attachment requirements of nonstructural components in the Eurocode 8 are not mentioned explicitly but taken into account in the calculation of horizontal seismic force with the importance factor (γa which shall not be less than 1,5 for anchorage elements of machinery and equipment required for life safety systems and tanks and vessels containing toxic or explosive substances considered to be hazardous to the safety of the general public) and the behaviour factor of the element (taking the value of 2.0 for exterior and interior walls, partitions and facades, anchorage elements for false ceilings and light fixtures, etc.)

Attachment requirements in ASCE 7-10 are divided in architectural components (for example: interior nonstructural walls, partitions, cantilever elements, ceilings, etc.) and mechanical and electrical components (elevators, piping, electrical conduit, etc.). This code is the most explicit. Each building component is separately.

NZS 1170.5 in the section 8.7 deals about connections. Non-ductile connections shall be designed for seismic actions corresponding to a ductility factor of the part μ_p = 1.25. Other connections may be designed for a greater value of μ_p where the specific detailing can be verified to sustain not less than 90% of their design action effects at a displacement greater than their yield displacement under reserved cyclic load.

NCSE-02 in the section 4.7 deals with general considerations for the binding of nonstructural elements to prevent the release of parts during earthquake situations. This section contains walls and partitions; sills, parapets, chimneys and fences; escape routes; exterior joineries; coatings and claddings and facilities and connections. Requires more if the acceleration is high seismic also taking into account the degree of ductility of the structure.

7. FACADES

The interaction between non-structural elements and bare structure can drastically alter the overall seismic response of the building, increasing strength and stiffness on one hand but also causing potential problems in terms of locals of global failure. Several researches have been done investigating the effect of infill panels (in particular unreinforced masonry infills) upon the seismic performance of reinforced concrete buildings. Soft storey behaviour is a particular concern in infilled structures especially but not limited to structures with open first floors. Soft storey mechanisms can in fact occur also at higher floors, due to the sudden failure of some infills at one floor level.

7.1 Facade types

In order to construct a performance-based seismic design framework for vertical non-structural elements/systems, a thorough overview and classification system of the available technologies is required. Facade systems can be categorized by three main types; infill panels, cladding and a combination of the two. In general, infill panels are constructed within the frame of the structure and cladding facades are attached externally to the primary structure.

7.1.1 Infill Panels

Infill panels have traditionally been made of heavy rigid materials, such as concrete or clay masonry blocks. However, more lightweight and flexible infill panel options are available in timber and steel frames. Masonry infill construction has a long history through much of Europe and is still one of the most popular choices today. In many European countries it is typical practice to use infill panels in the higher stories leaving the ground storey free from infill panel.

Figure 51 Infill panels (A. Palermo et al, 2010)

7.1.2 Cladding Panels

External cladding or curtain walls often incorporate stiff, brittle materials such as glass, concrete and stone. Cladding connections can be located on the columns, beams or a combination of the two which allows many variations in panel arrangements.

Stick systems are a popular option in modern multi-storey buildings. The stick system is a metal frame consisting of perpendicular transoms and mullions surrounding pieces of glass. Silicone sealant is usually used to allow the glass within the frame to move while keeping the building weather tight. Moreover, the double skin facade system consists of two layers of material which can create a sealed cavity to improve the thermal performance of a building. Double skin facade systems are being employed increasingly in high profile buildings, being touted as an exemplary 'green' building strategy.

Figure 52 Cladding panels (A. Palermo et al. 2010)

7.1.3 Mix Systems

It is also possible to have a combination of infill and cladding systems, commonly referred to as mix systems. Mix systems are common in Europe and are commonly employed for improved aesthetics. The most common mix system consists of masonry infill which provides support for cladding.

7.2 Classification of facade systems

With the facade technology categorised by three main types; infill panels, cladding and mixed systems, the next step is to classify each individual system. Each system also needs to be defined in terms of its modularity, the connection devices used to connect it to the primary structure and the modularity of the connections.

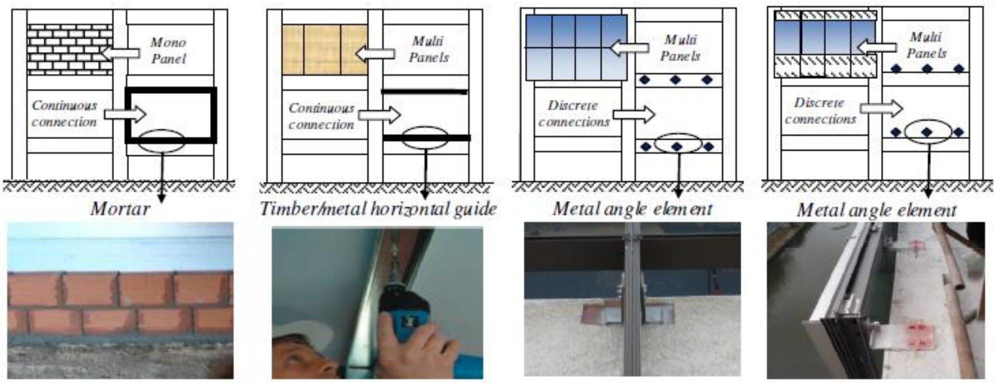


Figure 53 Classification of facade systems (A. Palermo et al, 2010)

The modularity describes the degree to which a system's panels/connections may be separated and recombined. Classification of the systems and their modularity is a crucial step in determining the seismic behaviour of each system. For example, a mono-panel will behave quite differently under seismic loading to a multi-panel system of the same material and different again if the connection modularity is different. Therefore it is important to define all such aspects for each system. The connections used in facade systems can be classified as either continuous or discrete connections. Continuous connections are more common in infill panel systems and include wet mortar connections and timber or metal horizontal guide connection. Discrete connections are more common in cladding panel systems and are typically metal connections.

7.3 Facade performance

The inter-storey drift of a structure during earthquake excitation typically dictates the behaviour and thus performance of most non-structural vertical elements or facade system. Even so, each facade system will behave in a particular way when subjected to inter-storey drift. This is dependent on all of the aspects covered in the previous section, e.g. system, connection, modularity. Some of the different failure mechanisms for different facade systems as a result of excessive inter-storey drift. It is important to understand how each facade system behaves in order to determine which parameters are important for the performance based design. Once this is fully understood, a priority can be established for the capacity design of the system. For example, for infill panels, it is usually the strength of the infill that governs failure and for cladding it is usually the connections that govern. However, if the connection is strong or designed in such a way to accommodate inter-storey drift, panel failure may again be the governing factor. Moreover this problem is complicated by the modularity of the facade and/or the connection which in some cases can be very influential in determining what type of failure will govern. The maximum permissible deformation for each facade system is taken into account in most seismic codes. However, the typical method used can be somewhat conservative as the treatment is identical for a range of facade systems.

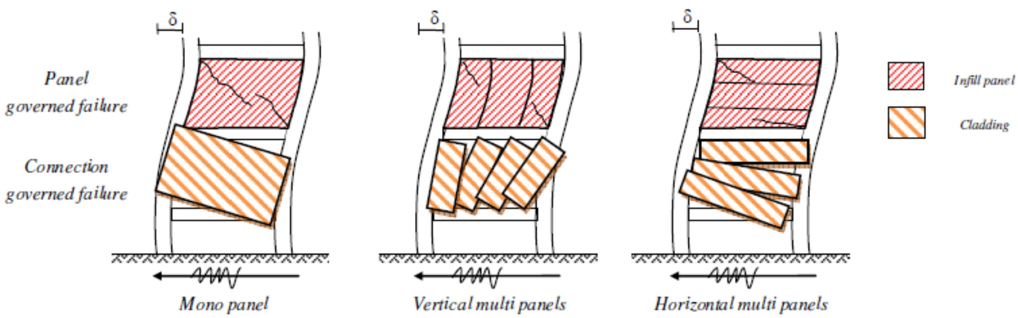


Figure 54 Facade performance (A. Palermo et al, 2010)

7.4 Performance indicators

7.4.1 Infill Panels

The definition of appropriate values for lower and upper bounds of each performance level represents the most critical and controversial phase of a reliable performance-based design. Suggestions have been made for the performance limits of masonry infill panels based on the strain limit states for infill panels which can be directly related to the inter-storey drift.

A common method used for modelling masonry infill panels in the equivalent strut model. Using the equivalent strut model, the monotonic stress-strain behaviour of the strut can be used to indicate the damage level of the infill. Predefined limit states, or performance levels, can thus be defined as a function of the axial deformation, εw. Basic geometric considerations can then be used to relate, for a given performance level, the axial deformation εw in the equivalent strut to the inter-storey drift, δ.

7.4.2 Cladding Panels

Cladding failure is expected to be typically governed by the design of the connection device. Thus the determination of values for the performance levels of cladding systems is dependent upon the connections. How well the cladding connections perform can be determined using the inter-storey deflection for the defined hazard. This is also dependent on whether the connections are fully or partially restrained. Cladding panels can be modelled using finite elements models. The equivalent strut model can be also be used for claddings if fixed rigid connections are used for panels connected at the beam-column joints

7.5 Damage reducing solutions

How a facade system is connected to the primary structure is the critical aspect in determining the interaction between the two systems. Because a structure is typically designed neglecting the façade system, the current approach is typically to connect the facade such that the interaction between the facade and the structure is minimized as much as possible. However, this means the facade system is simply a dead weight. More advanced systems can incorporate the stiffening and damping properties of the facade with the structure.

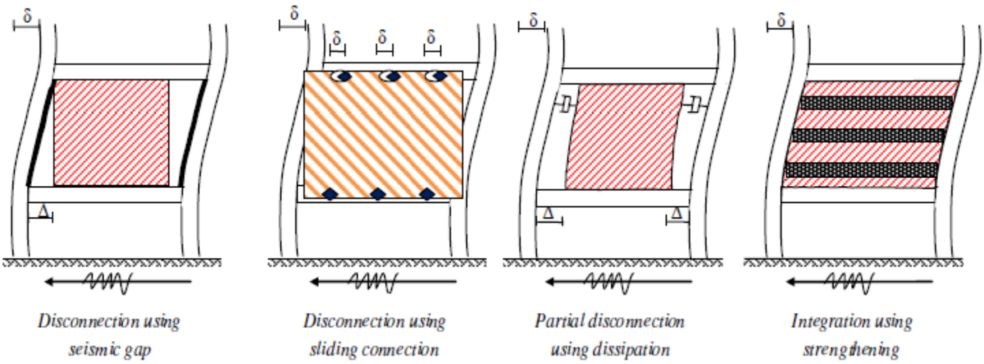


Figure 55 Damage reducing solutions (A. Palermo et al, 2010)

7.5.1 Disconnection from Primary Structure

Because a structure is often designed neglecting the facade system, the current practice in seismically active countries such as Japan, USA and New Zealand is to separate the facade system from the frame. For infill panels this is most commonly done using a seismic (or separation) gap between the wall and frame. Seismic gaps thus aim to prevent the infill panel from interacting with the frame. Seismic gaps present challenges regarding issues such as acoustic control, weather tightness, fire protection and aesthetic qualities that need to be addressed.

Similarly to seismic gaps, the interaction between cladding systems and the frame can be minimized using movement connections. These connections commonly consist of a fixed and sliding connection which allows the cladding panels to move and rotate relative to the frame when undergoing seismic excitation. An investigation into autoclaved lightweight aerated concrete (ALC) panel connections showed that these panels could be successfully isolated from the structure, even under a large inter-storey drift of 4%. The tests showed no visible damage to the panels and no contribution to the stiffness or strength of the structure.

7.5.2 Partial Disconnection with Dissipative Devices

The use of the facade as a passive control system for seismic behaviour of buildings makes more economic and dynamic sense because the facade is no longer simply a dead weight to the frame. Such a system makes use of a relative displacement between facade and structure. In practice this has proved difficult to achieve, and it has proved more economic (if not more performance effective) to use isolation.

Facade systems can be integrated with energy dissipative connections that are designed to yield before the facade yields. These connections utilize the interaction between the panels and the building structure to dissipate energy. At the same time, like other passive control devices, they provide additional lateral stiffness to the structure and alter its dynamic characteristics. Results show that energy dissipative cladding connections can reduce drift as well as provide the total hysteretic energy required of the structural system.

Another possible partial disconnection solution is the use of a seismic fuse device. Such a device is designed to allow infill panel and frame interaction under wind loading and minor to moderate earthquakes for reduced building drift but to disengage them under damaging events. The device acts as a sacrificial element just like a fuse to save the infill panel and frame from failure. The Seismic Infill Wall Isolator Subframe (SIWIS) is an example of such a system. It consists of two vertical and one horizontal sandwiched light-gauge steel plates with "rigid-brittle" elements in the vertical members. An experimental evaluation of the SIWIS system showed that the concept is a viable alternative but needs further performance evaluation under cyclic loading.

7.5.3 Full Interaction

Having a complete integration of the facade system is often an effective strategy to reduce the drift of a structure because of the additional stiffness provided by the facade. Often many structures have been built in this way, whether it was the intent of the designer or not to call upon this additional strength and stiffness. Therefore these solutions present the most likely possibilities as retrofit solutions to strengthen the fully integrated facades within existing structures.

Fibre-reinforced polymers (FRP) are seen as one of the most suitable retrofit solutions in strengthening unreinforced masonry infill panels in RC frames. Test results indicate that the use of glass FRP sheets as strengthening materials provide a degree of enhancement to infill panels, upgrading its strength and ductility as well as making the wall work as one unit.

As well as glass FRP sheets, FRP surface-mounted bars and Engineered Cementitious Composites (ECC) are other similar strengthening solutions for retrofitting masonry infill panels. ECC can be shot-creted onto masonry infill panels and provides a tensile strain capacity of several hundred times that of normal concrete. The fracture toughness of ECC is similar to that of aluminum alloys; furthermore, the material remains ductile even when subjected to high shear stresses.

8. INFILL WALLS

8.1 Problems associated with infill walls

Firstly, infill walls stiffen a building against horizontal forces. Additional stiffness reduces the natural period of vibration, which in turn leads to increased accelerations and inertia forces

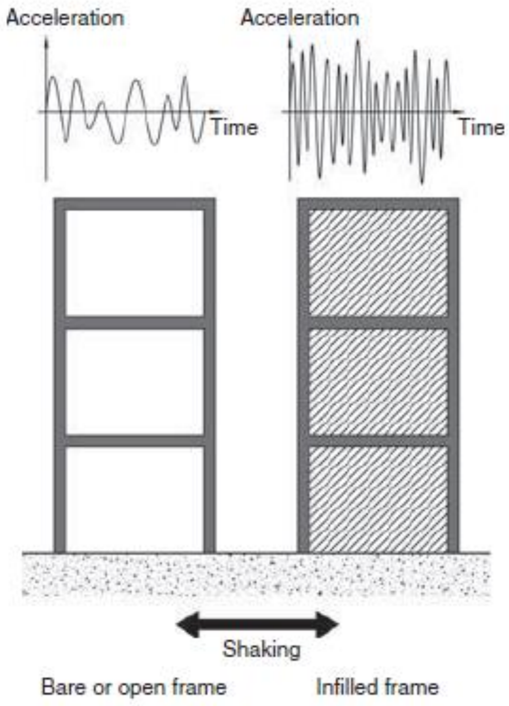


Figure 56 A comparison of roof-top accelerations of a bare or open frame with an infilled frame

As the level of seismic force increases, the greater the likelihood of non-structural as well as structural damage. To some degree, the force increase can be compensated for by the strength of the infills provided they are correctly designed to function as structural elements.

Secondly, an infill wall prevents a structural frame from freely deflecting sideways. In the process the infill suffers damage and may damage the surrounding frame. The in-plane stiffness of a masonry infill wall is usually far greater than that of its surrounding moment frame – by up to five to ten times! Without infill walls a bare frame deflects under horizontal forces by bending in its columns and beams. However, a masonry infill dominates the structural behaviour.

Rather than seismic forces being resisted by frame members, a diagonal compression strut forms within the plane of the infill, effectively transforming it into a compression bracing member. Simultaneously, a parallel diagonal tension crack opens up between the same two corners of the frame because of the tensile elongation along the opposite diagonal and the low tensile strength of the infill material. The infill panel geometry deforms into a parallelogram. After reversed cycles of earthquake force, ‘X’ pattern cracking occurs *(Figure 57).* The strength of the compression strut and the intensity of force it attracts concentrates forces at the junction of frame members. Shear failure may occur at the top of a column just under the beam soffit *(Figure 58).* Such a failure is brittle and leads to partial building collapse.

Figure 57 Damage to the tops of several columns due to infill wall compressive strut action. Mexico City, 1985 Mexico earthquake.

Figure 58 Typical infill wall diagonal crack pattern. 1999 Chi-chi, Taiwan earthquake.

During a damaging quake diagonal cracks and others, including those along the interface of infill and columns and the beam above, soften-up the infill. It becomes weaker and more flexible than a less severely damaged infill above it – in effect creating a soft-storey. Even if infill walls are continuous vertically from the foundations to roof, once ground floor infill walls are damaged a soft storey failure is possible.

Another danger facing a heavily cracked infill is its increased vulnerability to out-of-plane forces *(Figure 59).* The wall may become disconnected from surrounding structural members and collapse under out-of-plane forces. Due to their weight, infill walls pose a potential hazard to people unless intentionally and adequately restrained.

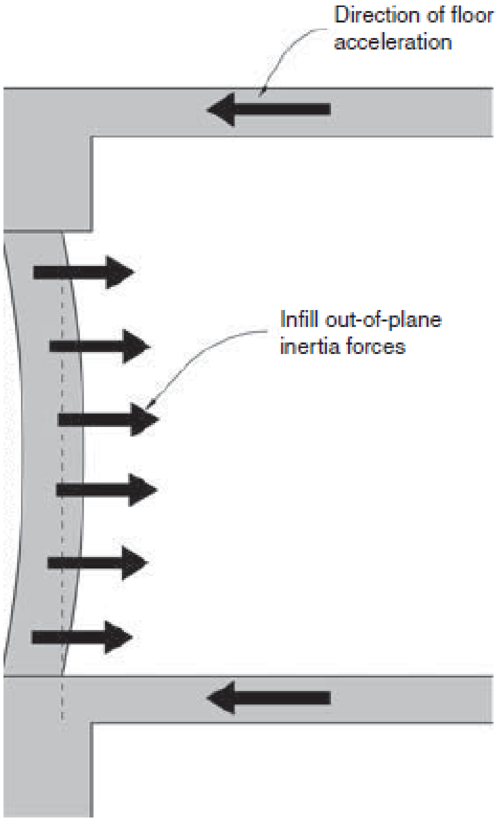


Figure 59 A section through two floors and an infill wall. Out-of-plane forces act on the infill which spans vertically between floors.

The final problem associated with the seismic performance and influence of infill walls is that of torsion. Unless infill walls are symmetrically placed in plan their high stiffness against seismic force changes the location of the Centre of Resistance (CoR). In *Figure 60 (a)* the CoR and Centre of Mass (CoM) are coincident; no significant torsion occurs. If infill walls are located as in *Figure 60 (b),* the CoR moves to the right and the subsequent large torsional eccentricity causes the building to twist when forced along the *y* axis *Figure 60 (c).* As one floor twists about the CoR relative to the floor beneath the columns furthest away from the CoR sustain large inter-storey drifts and damage. If the drifts are too large, those columns are unable to continue to support their gravity forces and their damage leads to that area of the building collapsing. In this example, the infill walls cause torsion during *y* direction shaking only.

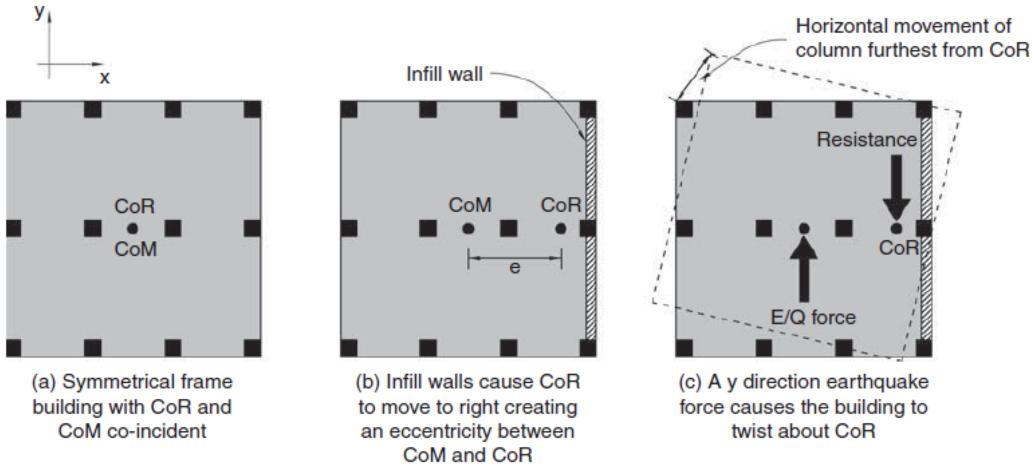


Figure 60 Asymmetrically placed infill walls cause building torsion that damages columns distant from the CoR.

8.2 Solutions to problems caused by infill walls

In steel and concrete moment frame construction, infilling some of the bays with walls made of masonry units is a common practice in many countries. Traditionally, such infill walls are specified by architects as interior or exterior partitions in such a way that they do not contribute to the vertical gravity load-bearing capacity of the structure. However, depending on their construction details, they can adversely influence the seismic behavior of the structure. In other words, under seismic lateral loads the infill walls can interact with the confining frames and take part in resisting lateral in-plane forces and lead to some damage to the wall or frame [26, 27]. Two methods of construction have been proposed and considered in the literature for infill walls [28, 29].

- The first method is to integrate the infill wall with the structural frame and basically turn it into a shear wall.

– The second approach is to isolate the infill wall from the structural frame by leaving gaps between them.

In the case of tight fit construction, depending on the details of their construction (partial vs. complete infill), the infill wall's interaction with the confining frames could possibly lead to premature column failure as a result of the short column effect or to increased levels of unaccounted ductility demand in columns. Furthermore, certain arrangement of the walls can influence the lateral stiffness distribution in plan and elevation, which may result in increased torsional effects and/or soft story situation. As masonry infill walls are usually much stiffer than the structural frame, with increased seismically induced story drift, the tight fit infill walls first experience damage in the form of cracks and separation from the structural frame. *Figures 61 and 62* shows some seismic induced failures in buildings as a result of the interaction of tight fit infill walls and frames.

Figure 61 Major damage to the unreinforced masonry infill walls and moderate damage to the columns of an R/C municipal building (Quindia, Colombia 1999 Earthquake, EERI).

Figure 62 Short column effect due to partial tight fit infill walls in an R/C building (Arequipa, Peru, June 2001 Earthquake, EERI).

Because of such failures, an alternative solution of separating the infill walls from the structural frame by leaving gaps between them is also available as shown in *Figure 63.* It is very important to ensure that the gaps, whose required widths are determined analytically, will not be accidentally filled with mortar or other stiff materials during construction procedure. According to Dowrick [29], there are two main performance problems with this approach, which need to be solved. The first issue is to provide convenient details for out-of-plane stability of the infill wall, while the second one is to address the acoustic and fire insulation requirements at the separation gap.

Figure 63 Isolated infill walls from R/C frame with gaps in a recent construction.

Two essential features of a seismically separated infill wall are: a clear vertical gap between the infill and columns (typically between 20 mm to 80 mm wide), and an approximately 25 mm wide horizontal gap between the top of the wall and the soffit of the beam above. This gap under a beam or floor slab must be greater than that element's expected long-term deflection, and also allow for the downwards bending deflection of a moment frame beam under seismic forces. Where provided, these gaps allow the floor above an infill to move horizontally to-and-fro without the infill wall offering any resistance in its plane.

Often the resolution of one problem creates another. Although an infill may be separated for in-plane movement, where it is separated on three sides it becomes extremely vulnerable to out-of-plane forces or face-loads as they are sometimes called. It must be stabilized against these forces acting in its weaker direction, yet at the same time allow unrestricted inter-storey drift along its length. One of several structural solutions is required.

The most obvious approach to stabilizing an infill wall against out-of-plane forces is to cantilever it from its base. But this is not usually feasible for two reasons. Firstly, the floor structure beneath may not be strong enough to resist the bending moments from the wall. Secondly, the infill wall itself may not be strong enough or may require excessive vertical reinforcing. On the upper floors of buildings, elements like infill walls are subject to very high horizontal accelerations well in excess of 1 g, the acceleration due to gravity.

The preferred option is to design an infill wall to resist out-of-plane forces by spanning vertically between floors. Through its own strength the wall transfers half of its inertia force to the floor beneath and the other half to structure above. Careful structural detailing at the top of the wall can provide sufficient strength to prevent out-of-plane collapse yet simultaneously accommodate inter-storey drift between the top of the wall and structure above. *Figure 64* illustrates some generic connections between reinforced masonry infill walls and concrete frames while *Figure 65* illustrates an as-built solution.

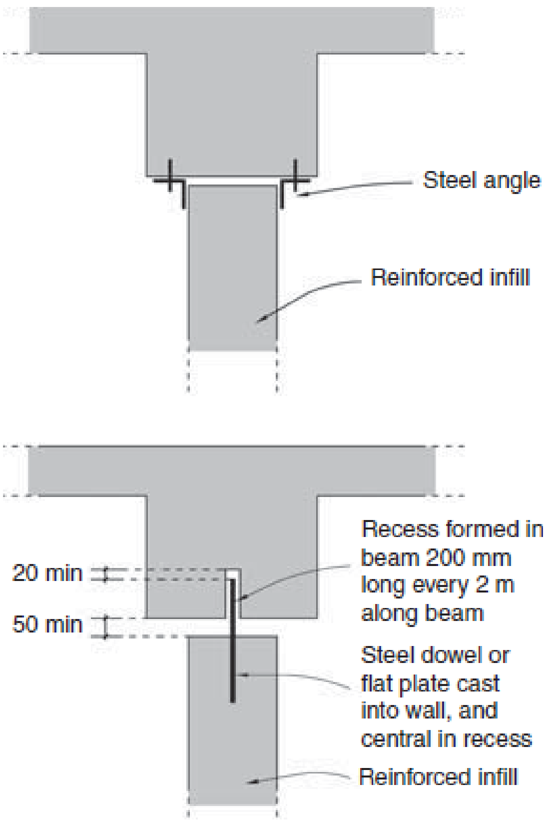


Figure 64 Two possible structural details that resist out-of-plane forces yet allow relative movement between an infill wall and structure above.

Figure 65 A reinforced masonry infill wall separated from surrounding structure.

Where infill walls are constructed from unreinforced masonry one approach is to provide small reinforced concrete columns within the wall thickness *(Figure 66).* Their function is not to support any vertical force but to stabilize the infill against out-of-plane forces. For a long panel, three or more intermediate “practical columns” (as they are sometimes called) may be designed by the structural engineer [30]. Support of this form is commonplace near the roof of a building where ground accelerations are amplified most strongly. Only the reinforcing bars that project vertically from these small columns connects to the underside to the beam. This detail, strong enough to resist out-of-plane forces, allows virtually unrestrained inter-storey drift in the plane of the wall. The ductile bending of the vertical practical columns bars will not provide significant resistance to that movement.

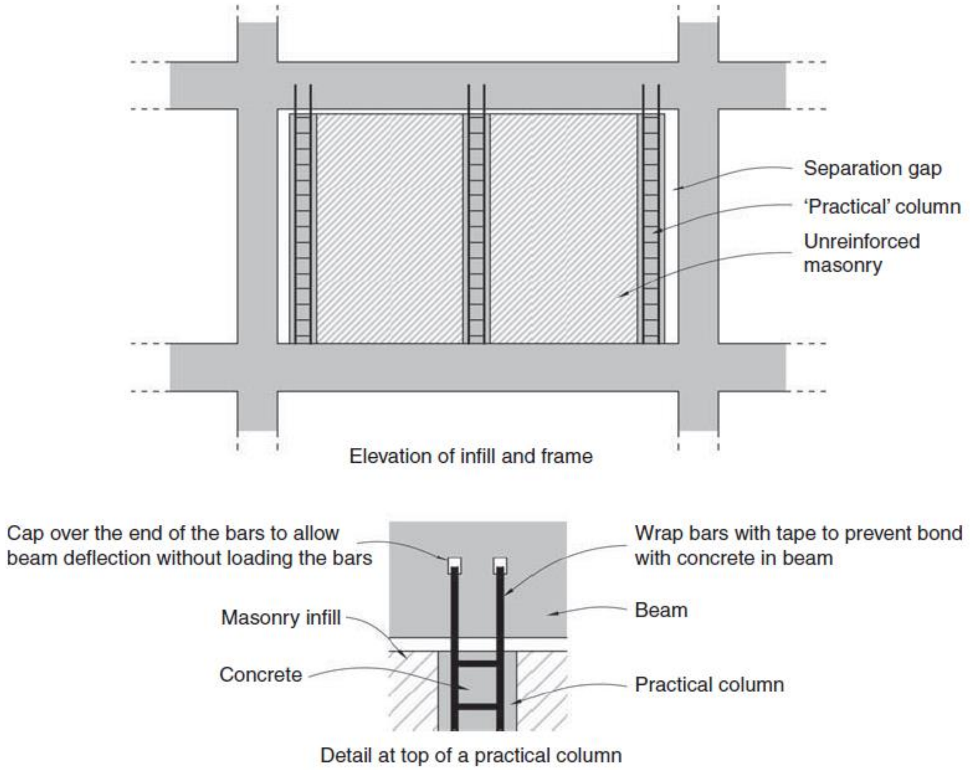


Figure 66 Separated unreinforced masonry infill wall with "practical columns" providing out-of-plane strength.

8.3 Design codes and international recommendations

Being aware of the importance of infill masonry elements in the behaviour of buildings in the last few years the new codes have included some provisions regarding the consideration of the infills and their influence on the structural response.

For example, the Eurocode 8 [16] in section 4.3.6 "Additional measures for masonry infilled frames", include general recommendation for non-structural elements, acknowledging that, in case of failure, are a risk to human life or affect the main structure of the building and should be verified to resist the design seismic action. Eurocode 8 [16] also refers to the safety verification of the non-structural elements, as well as their connections and attachments or anchorages, during the design, considering that the local transmission of actions to the structure by the fastening of non-structural elements and their influence on the structural behaviour should be taken into account. For the particular case of the infill masonry panels, in particular if the masonry infills are in contact with the frame (i.e. without special separation joints), but without structural connection to it (through ties, belts, posts or shear connectors) can affect the ductility class of the structure. In particular for panels that might be vulnerable to out-of-plane failure, the provision of ties can reduce the hazard of falling masonry. For the structural systems belonging to all ductility classes, DCL, M (medium ductility) or H (high ductility), appropriate measures should be taken to avoid brittle failure and premature disintegration of the infill walls (in particular of masonry panels with openings or of friable materials), as well as the partial or total out-of-plane collapse of slender masonry panels and particular attention should be paid to masonry panels with a slenderness ratio of greater than 15.

FEMA 310 [31] in the basic non-structural component checklist for the building evaluation define that non-structural components namely partitions masonry veneers, cladding and parapets, should respect the compliance criteria in accordance to seismic zoning (fixtures, spacing's and anchoring) and the evaluation procedure should be based on the forces and drift limits.

The definition of limit states for infill masonry panels can be directly related to the inter-storey drift demand. Based on the equivalent strut model, Magenes and Pampanin [32] have proposed drift values for the damage level of a masonry infill panel corresponding to a certain limit state, depending on the axial deformation. For example, an inter-storey drift value in the range of 0.4%-1.0% can be associated to the infill panel's failure.

The FEMA 306 [33] and FEMA 307 [34] documents provide also reference values of inter-storey drift ratios for RC buildings with infill masonry panels. The drift limits proposed differ with the type of masonry, from 1.5% for brick masonry to 2.5% for un-grouted concrete block masonry. In these documents are also indicated a drift reference value of 0.25% for the initiation of diagonal cracking [35]. Other authors recommended inter-storey drifts to be considered for the serviceability check ranging from 0.2% to 0.5%, depending on the type of partitions. Values around 0.2% are recommended for brick masonry infills in contact with the surrounding frame [36] whereas 0.5% is more appropriate for plywood, plaster, gypsum and similar light panels [37].

8.4 Specific solutions

8.4.1 SIWIS

The Seismic Infill Wall Isolator Subframe (SIWIS) system is developed for use in building frames with masonry infill walls in order to prevent damage to columns or infill walls and minimize life-safety hazards during potentially damaging earthquakes. The SIWIS system, which consists of two vertical and one horizontal sandwiched light-gauge steel studs with “rigid-brittle” elements in the vertical members, is designed to allow infill wall–frame interaction under wind loading and minor-to-moderate earthquakes for reduced building drift but to disengage them under damaging events. The SIWIS system acts as a “sacrificial” element just like a “fuse” to save the infill wall and frame from failure.

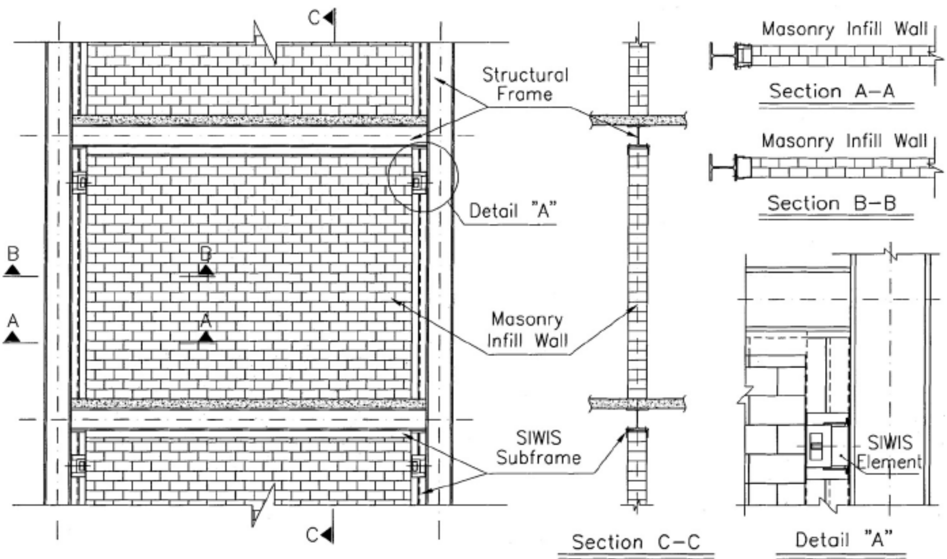


Figure 67 An example of SIWIS subframe system used in a building frame.

Figure 68 shows a section through a column (steel or reinforced concrete), where it can be seen that a vertical member of the subframe consists of two sandwiched light-gage steel studs with a “rigid-brittle” element between them. Within the subframe member at the top of the wall, which will not have rigid-brittle elements, and in open spaces of the subframe vertical elements, there can be a flexible filler material that will provide the needed sound insulation and fire-resistance. The out-of-plane stability of the infill wall will be provided through the top subframe member. The location of SIWIS elements shown near the top of the masonry wall panel in *Figure 67* is chosen because the frame will first contact the infill wall at that point under lateral drift and will tend to close the gap if there were no SIWIS elements.

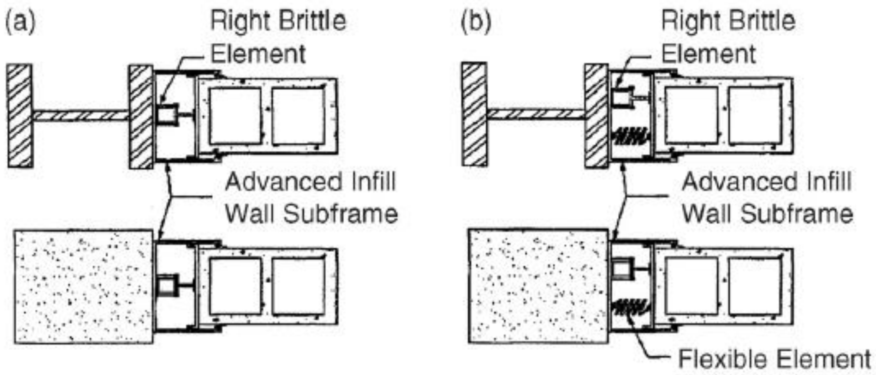


Figure 68 Details of SIWIS subframe system between infill wall and column from [6].

The SIWIS system is designed such that under small to moderate levels of structure frame lateral interface force, the SIWIS “element” will act as a rigid link and transfers the force to the infill wall for the benefit of its stiffness in reducing drift. At larger levels of lateral force (the designed threshold of the subframe), the brittle elements of the subframe break, just like a “fuse”, and as a result, the structural frame will be free to displace without transferring force to the infill wall. Different grades (e.g., low, medium, high) for the SIWIS element can be designed and specified according to the strength and stiffness quality of the infill wall. This way, if the infill wall is known to have poor quality or if much of the stiffness of the infill wall is simply not to be relied upon, the mild (low) grade of the SIWIS element will then be suitable. The SIWIS system can be used with many types of masonry infill walls including walls with or without openings, partial or full infills, and with masonry units ranging from high strength concrete masonry blocks and clay brick units to lower strength masonry such as thin wall hollow clay tile units and autoclaved aerated concrete blocks.

8.4.2 Fibre reinforced polymer (FRP)

A large number of unreinforced masonry (URM) buildings in the world are seismically vulnerable by today's standards. The failures and damages reported in recent earthquakes attest to the need for efficient strengthening procedures. The main structural elements that resist earthquakes in these buildings are the old unreinforced masonry walls which were designed to resist mainly gravity loads. Under seismic loading, unreinforced masonry walls have two possible failure mechanisms, namely in-plane and out-of-plane. In order to preserve these buildings, rehabilitation is often considered essential to maintain their capability and to increase safety. In the last decade, fiber reinforced polymer (FRP) composites have been used for strengthening URM walls of masonry buildings.

The advantage of FRP composites for masonry retrofitting include lower installation costs, improved corrosion resistance, flexibility of use, and minimum changes in member size (and in some cases appearance) after repair. Disturbance to occupants and loss of usable space are also minimal. Furthermore, for earthquake retrofits, seismic mass of the existing structure remains unchanged because there is little addition of weight.

Three types of fibers are commonly used in FRP composites for infrastructure applications: carbon, aramid and glass fibers. In the order listed, these fibers exhibit an ultimate strain ranging from 1 to 4%, with no yielding occurring prior to failure.

In many instances, glass FRP (GFRP) is preferred for strengthening of masonry. The lower elastic modulus of GFRP, as compared to carbon FRP (CFRP), is not as limiting in masonry strengthening applications as it might be in concrete structures because it is more compatible with the low elastic modulus of masonry. In addition, GFRP material costs are substantially less than carbon or aramid materials. Also, experimental data from shear-strengthening of masonry walls have shown that use of CFRP systems do not offer significant improvement in structural performance over similar GFRP systems.

Use of CFRP systems, however, is preferable for applications where masonry elements will be subjected to sustained stresses, such as in retaining walls. CFRP systems are more suitable for these applications since they have better resistance to creep than other fibers. Also, in exterior applications, CFRP is generally a better option because of its superior durability in moist environments compared to GFRP.

Aramid is not commonly used in masonry. The material properties for aramid are sensitive to moisture change, which is common in masonry construction.

a) first mortar layer   b) application of FRP mesh   c) second mortar layer

Figure 69 Typical installation FRP (Ciro Faella et al, 2004)

Various researchers have examined the use of various FRPs to enhance the in-plane performance of masonry walls under monotonic, cyclic or seismic loading. Large increases in both load and displacement capacity were observed, with the amounts depending on the quantity, type and layout of the FRP used. Also, different in-plane failure modes of FRP strengthened masonry walls were reported such as shear failure, flexural failure, FRP rupture, anchorage failure and debonding failure [39, 40].

Whereas extensive research has been conducted and reported for retrofitting of reinforced concrete structures using FRP, much less has been reported for retrofitting of URM walls [41]. Also, very small numbers of these researches on the subject of URM walls have been conducted on perforated URM walls [42, 43]. The response of masonry shear wall becomes quite complicated, if the wall has openings for functional requirements. Openings have a significant effect on the characteristics of masonry shear walls such as failure mode, strength and ductility [44, 45].

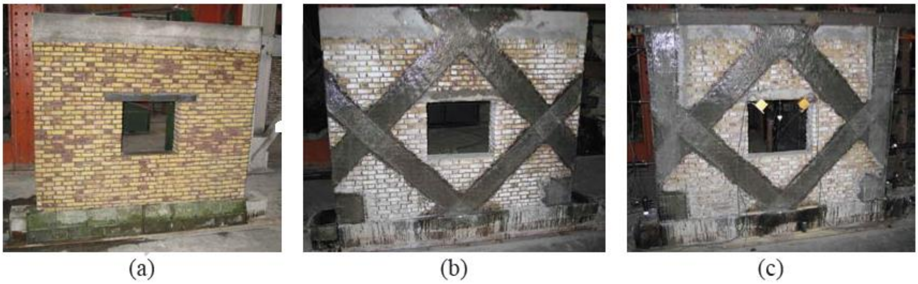


Figure 70 Openings in unreinforced masonry test under cyclic loading [46]

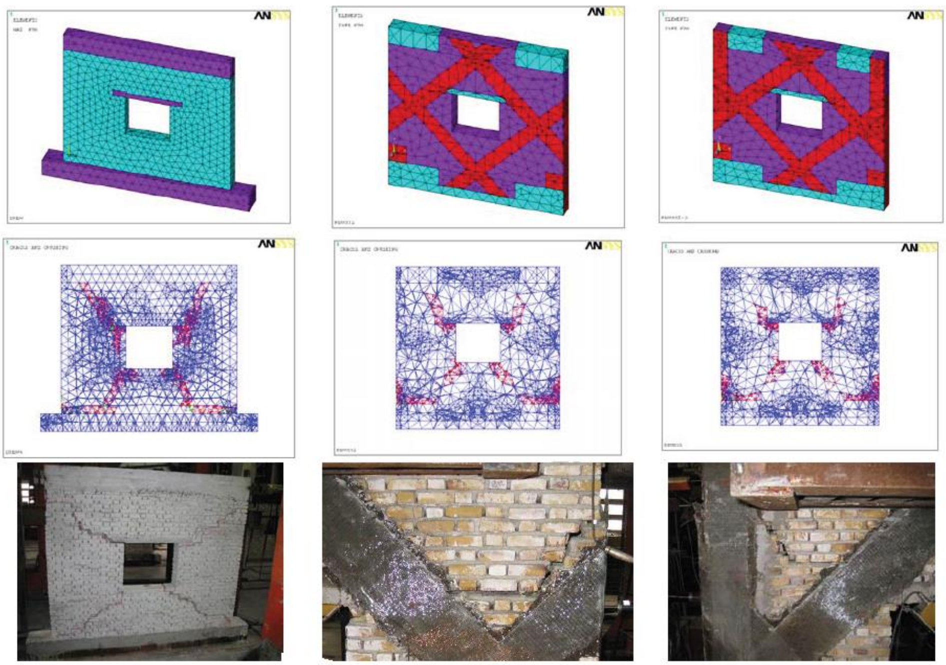


Figure 71 Result of finite element model and cracking pattern of the test walls [46]

9. CLADDING SYSTEMS

The effects of an earthquake on architectural precast cladding come from two actions. First, the inertia of the panels develops forces due to the acceleration of their mass. Second, the horizontal movement of the building structure from lateral drift imposes forces through the connections. The performance of cladding systems depends on the interaction between the cladding and building structural frame during a seismic event.

Cladding facades are generally considered as nonstructural elements and are not allowed to contribute any structural function to the building. Instead of trying to structurally isolate and protect facades by avoiding any interaction between the cladding panels and the structure, the present research explores ways to use this interaction to dissipate energy and thereby reduce building response. The basic concept of passive control is to dissipate the input seismic energy, reduce energy dissipation demand in the structural elements, and consequently, minimize potential damage to the structure. It will be shown that, when properly designed, ductile cladding connectors can be used to passively dissipate significant amounts of energy through inelastic hysteretic deformation driven by interstory drift. This concept was identified in earlier studies [47] and preliminary results showed feasible connector designs are capable of dissipating enough energy to reduce peak displacements by nearly 50% [48].

Figure 72 A Precast concrete panel (Medan Mehta et al, 2011)

9.1 Cladding connections

Although there are many different kinds of connection systems, all are generally composed of three main components:

- The anchor point, or insert, built into the precast panel, provides the panel anchorage;
- The connection body (often a steel angle), or connector, forms the structural connection between the cladding panel and the main structure; and,
- The anchor into the building structure (a second insert or an attachment to a steel member).

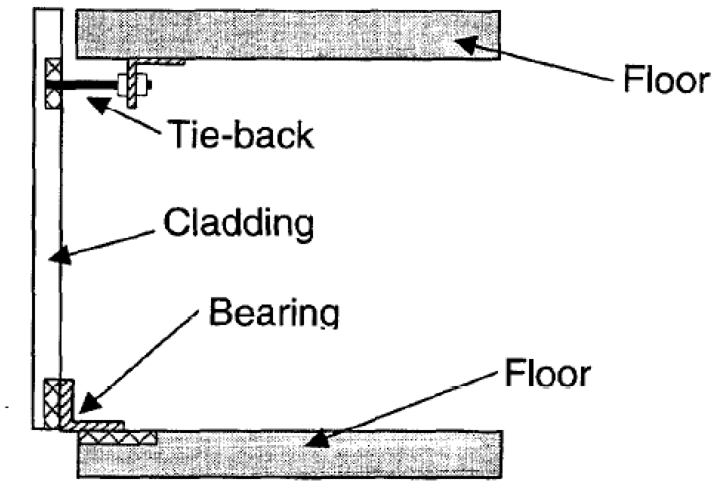


Figure 73 Schematic diagram and typical configuration of cladding system ("Ductile Cladding Connection Systems for Seismic Design". Barry J. Goodno and James I. Craig. Building and Fire Research Laboratory. National Institute of Technology. (1998))

There is considerable variation in the design of each of the three major components depending upon the function of the connection (bearing or tie-back), the type of connection (welded or bolted), the architectural requirements, and other considerations [49].

The connection anchor usually consists of a steel insert embedded in the concrete of the panel or the structure. Unlike the case of load-carrying structural panel connections, the anchorage of architectural cladding connections may be subjected, in addition to possible shear and pull-out, to torsional and bending moments due to the eccentricity of most of the connection designs. An experimental test program [50] has provided information on the behavior of cladding connection anchors when subjected to these combined shears and bending actions.

The data available from the tests showed that inserts embedded in concrete are not by themselves capable of providing the levels of ductility and damping required from an advanced connection without loss of strength and integrity due to extensive cracking of the concrete surrounding the insert. The conclusion is that in an advanced connection the energy dissipation must occur in the connector body if the integrity of the concrete panels is to be maintained, and the anchor must be kept in the linear elastic range. In other words, only one component in the connection system should yield, and this should be the connection body. The yielding of the connector therefore serves two purposes: it produces the necessary energy dissipation; and it protects the anchors by limiting the load that can be transferred through the connection.

The connector, then, becomes the focus of attention and must be designed to develop the needed energy dissipation while maintaining the structural integrity needed to insure that the cladding panels remain attached to the building structure. There are an almost unlimited number of devices that could be considered for such an application, although it is perhaps useful to consider the primary applied loads. A cladding connector must be capable of transmitting the following loads (listed in order of importance):

- Vertical or gravity load (the Weight of the panel).
- Normal load (force perpendicular to the vertical plane of the cladding).
- Transverse load (force in the plane of the cladding in the horizontal direction).

Load in these cases generally refers to a force, but it may also involve a moment if a “significant” rotational constraint is also present in the connector attachment. The vertical or gravity load may be carried by only a few of the cladding connectors used for a given panel while other connectors need not support any gravity load. This is often selected in order to provide a simpler statically determinate configuration for supporting gravity loads. As a result, connectors capable of supporting gravity loads are usually called “bearing” connectors. Normal loads arise from wind, environmental or seismic forces are usually carried by all kinds of connectors. Transverse loads usually arise from seismic forces or from interstory drift due to seismic or environmental forces. They can also arise due to thermal or weathering expansion or shrinkage.

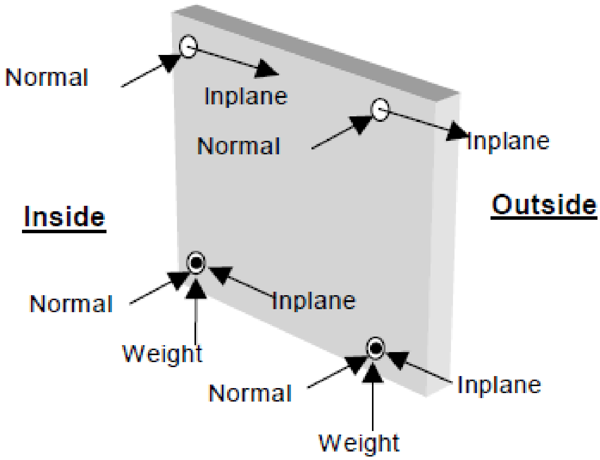


Figure 74 Schematic diagram of cladding panel loads (“Ductile Cladding Connection Systems for Seismic Design”. Barry J. Goodno and James I. Craig. Building and Fire Research Laboratory. National Institute of Technology. (1998))

In general, the accepted design practice is to use bearing connections to support the gravity loads in a statically determinate manner and to support normal loads at each connector (usually in a statically indeterminate fashion). The panel is then isolated from transverse loads by insuring that all but the bearing connectors are very flexible in the transverse direction. This provides resistance to seismic forces but isolates the panels from interstory drift and from environmental forces in the transverse direction. Such cladding connectors are frequently referred to as “tie-back” connectors because their primary role is to tie the panel back to the building structure and maintain the proper vertical alignment of the facade. (Note: while interstory drift can also result in relative displacement between top and bottom of a panel in a direction normal to the cladding plane, this is not normally assumed to give rise to interaction forces in the cladding because the connectors are usually very flexible in bending about a horizontal axis in the cladding plane).

The advanced connectors as defined in this report are assumed to function in every respect as conventional cladding connectors, BUT they are also allowed to transmit horizontal transverse loads (in the plane of the panel) that arise from interactions between the panel and the supporting structure due to interstory drift. In this respect the advanced connectors will allow the cladding to contribute to the interstory shear resistance of the building and at the same time to dissipate energy as a result of this interaction. Such action will introduce in plane shear forces into the cladding panel, and the panel must be capable of supporting such loads. However, by proper design, the connector can also be configured to limit the maximum level of shear force that can be introduced into the panel, thereby protecting not only the panel but also the panel insert and the building attachment.

There are a number of different ways to develop the desired properties in an advanced connector. Such a connector should provide about the same axial (normal) behavior as a conventional design but it should exhibit a finite stiffness in the transverse direction with a well-defined yield force and a generous and stable hysteresis loop (thereby assuring good hysteretic energy dissipation). The advanced connector should also exhibit good ductility and maintain its structural integrity during repeated cycles of transverse deflection. Of course the connector should also be simple to install and use to align the panel in the building facade.

With these characteristics in mind, *Figure 75* shows a number of conceptual sketches for an advanced connector. They are based on some of the ideas for structural dampers that have already appeared in the literature and they incorporate the constraints particular to a cladding connector. A useful taxonomy for such connectors is to characterize them by the principal kind of structural deformation that they employ. In this case, a suitable classification is:

- Simple axial deformation of a prismatic structural member,
- Flexural designs that involve transverse beam bending,
- Shear designs that employ shear deformation (in beams or other forms),
- Torsion designs that utilize torsional deformation of a shaft.

Axial deformation designs are the simplest in concept but they do not inherently make use of structural geometry beyond a simple cross sectional area. Flexural designs have received a great deal of attention due to their simplicity and familiarity. They make use of beam bending action in which the geometry of the beam, including its cross section as well as its taper, can profoundly affect the transverse deformation. Shear designs can utilize shear deformation in a number of ways ranging from simple shear webs to shear deformation in beam bending. Torsional designs are somewhat more complex because the rectilinear interstory drift must be converted into a rotation about the shaft of the torsion member. However, torsion designs, like some shear designs, can often be designed so that the stresses are developed more uniformly throughout the material than is the case for flexural designs. This means that the load transmitting material in the connector can be more effectively utilized to develop hysteretic energy dissipation.

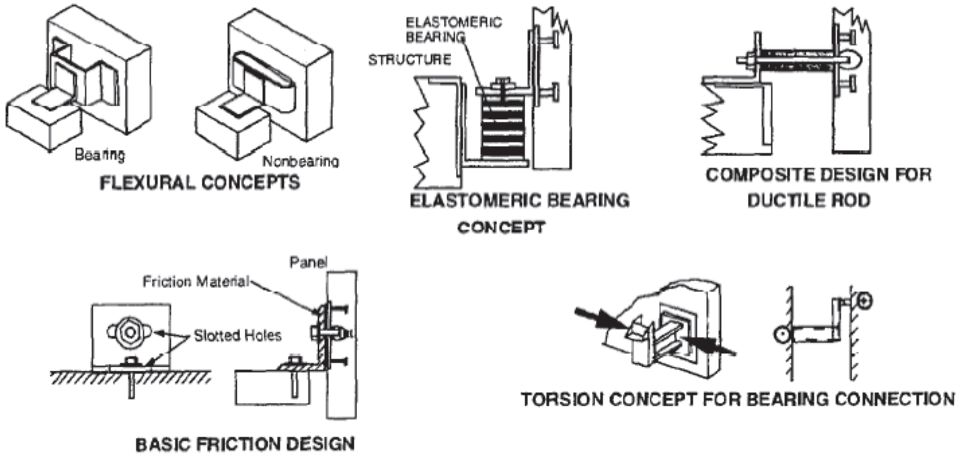


Figure 75 Conceptual designs for an advanced cladding connector ("Ductile Cladding Connection Systems for Seismic Design". Barry J. Goodno and James I. Craig. Building and Fire Research Laboratory. National Institute of Technology. (1998))

9.2 Precast panel configuration

For fabrication, handling, and erection economy, the use of the largest possible panels (subject to weight and transportation restrictions) is recommended. However, seismic requirements are often at odds with use of very large panels because of the accumulated deformations in the main structure that must be accommodated. While in non-seismic areas two- or three-story-height panels are used, the usual practice in higher seismic zones is to use panels that are limited to one story in height and seldom more than one horizontal bay in width.

Codes require that connections and panel joints allow for the story drift caused by relative seismic displacements. Connection details, and joint locations and sizes between cladding panels should be designed to accommodate any shrinkage, story drift, or other expected movement of the structure, such as sway in tall, slender structures. Panel geometry and joints must be configured so that panels do not collide with one another or with the supporting structure when it moves. If collisions occur, over-loading of the connections may result as well as damage to the body of the panel. Story drift must be considered when determining joint locations and sizes, as well as connection locations and their directions of resistance. If a connection can resist a force in a given direction then it can also cause panel motion in that direction.

Almost all non-structural (cladding) precast concrete panels are supported vertically at one floor only. This allows floors to deflect without transferring building gravity loads through the panel. The types of connections used to support the panel will ultimately determine the motion a panel will experience during a seismic event. Connection types are discussed in detail further on.

The way cladding panels behave in response to displacement of the supporting structure can be summarized as:

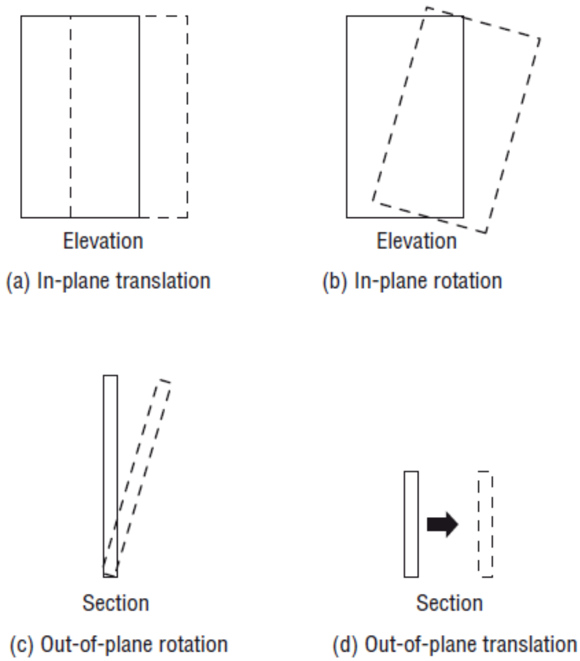


Figure 76 Modes of panel response to displacement ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

In-plane translation occurs when the panel is "fixed" in-plane to one level. The panel translates laterally with that level, remaining vertical. Spandrel panels and wall panels are typically designed to behave this way.

In-plane rotation, also known as "rocking", occurs when the panel is supported in-plane at two levels of framing. When the structure displaces, the lateral connections drag the panel laterally, causing it to rotate in plane and rest entirely on one bearing connection. This rotation requires bearing connections that allow lift-off. Narrow components such as column covers are often designed this way because of their aspect ratio (height to width) and the location of their connections.

Out-of-plane rotation is the tilting of a panel perpendicular to its face. This motion is common whenever a panel is connected to the structure at different levels of framing. The tie-back connections that support the panel for wind and seismic loads will also cause the panel to tilt out-of-plane during story drift. Bearing connections should be designed to accommodate this out-of-plane rotation, although it is generally so small that it is usually ignored with ductile connections.

Out-of-plane translation is common for spandrel panels that are attached to a single level of framing, since the movement is the same as the supporting beam to which it is attached.

9.2.1 Panels supported laterally at one floor only

Story drift is rarely an issue on spandrel panels because bearing connections and tie-back connections are located on the same floor beam. The tie-backs are not affected by story drift because the top and bottom of the floor beam move together:

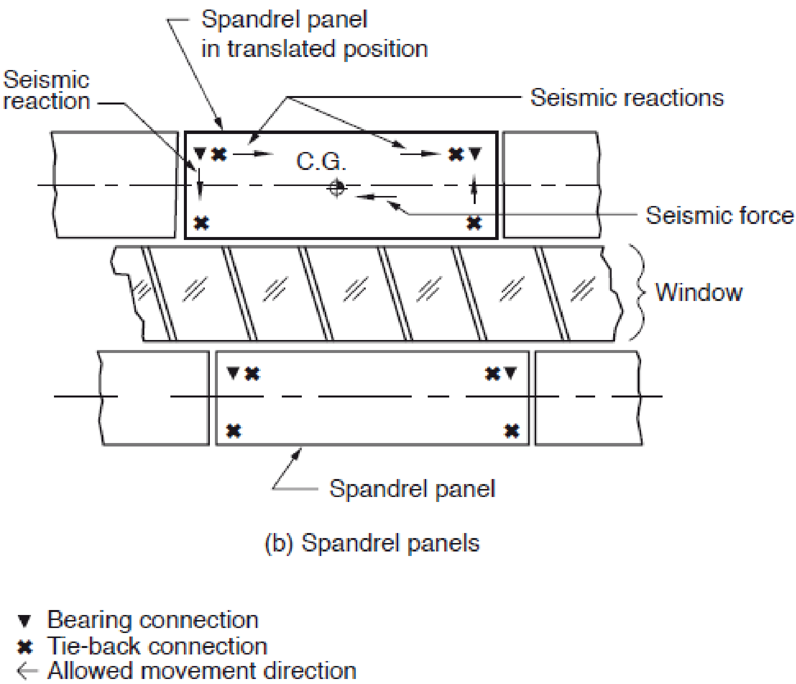


Figure 77 Cladding panel connection concepts–Seismic drift effect. Translating panels ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

Therefore, all panels connected to a given level will move with that level. The panels respond to building displacement because they are supported in-plane at one level only and because they are supported out-of-plane at one level only. Vertical panel joint widths can be kept to a minimum because there is no differential movement between panels and connections need only accommodate small movements from shrinkage or temperature changes.

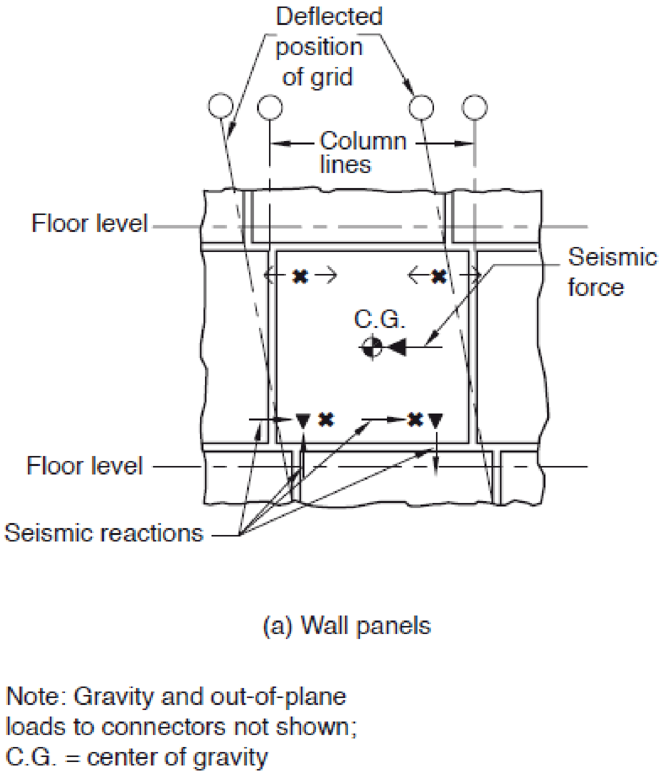


Figure 78 Cladding panel connection concepts–Seismic drift effect. Translating panels. ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

9.2.2 Panels supported at two levels of framing

When a panel is arranged such that it requires out-of-plane support from two levels of the structure, its connection system can make the panel rotate in-plane or translate without tipping or rocking.

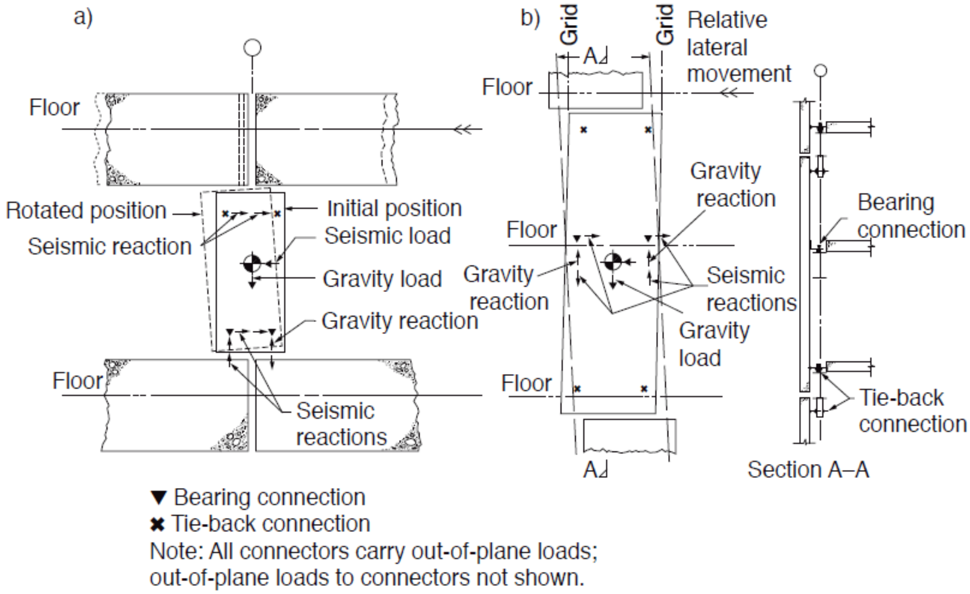


Figure 79 Tall/narrow units–high aspect ratios ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

It is essential that the potential movements be studied and coordinated with regard to the connection system and the joint locations and widths as well as adjacent construction. Such considerations often govern the connection design or the wall's joint locations and widths. The following discussions will address each type of motion in detail.

9.2.3 Panels connected out-of-plane at two levels and in-plane at one level (Translating)

Connections that resist imposed loads in all directions are referred to as rigid or fixed connections. Rigid bearing connections are generally used in panels that translate in-plane. Fixed bearing panels are vertical cantilevers in the in-plane direction. The two bearing connections resist the direct in-plane seismic force, as well as the resulting overturning moment. The moment is resisted by a couple formed by the bearing connections. When combined with panel self-weight, the tie-down forces may result in a net uplift on one connection and added downward force on the other. The bearing connections hold the panel down and prevent it from tipping.

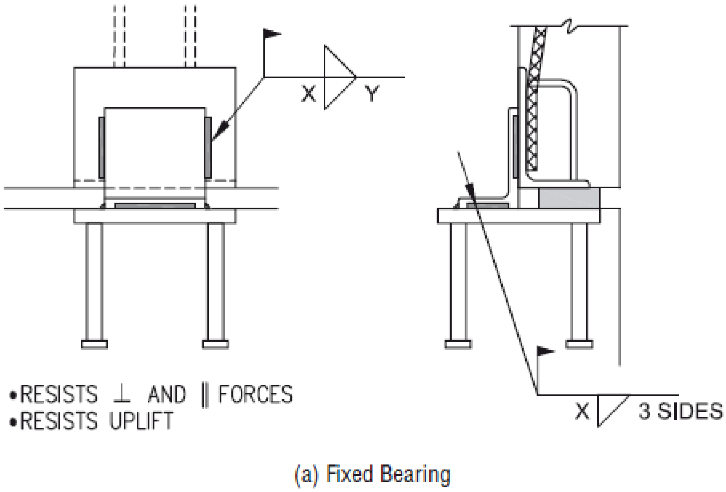


Figure 80 basic seismic connection types. Fixed Bearing ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

The upper tie-back connections, or "slip connections," that allow horizontal and vertical movement of the panel relative to the supporting structure, must only resist out-of-plane forces. If they were to resist in-plane forces, then they would also transmit in-plane movement. This would create a tug-of-war between the structure and the rigid bearing connections. These connections should be flexible or slotted in-plane to allow the structure to drift without over loading the connection. The panel will translate with the level of framing that the rigid bearing connections are attached to, and will remain vertical through this translation.

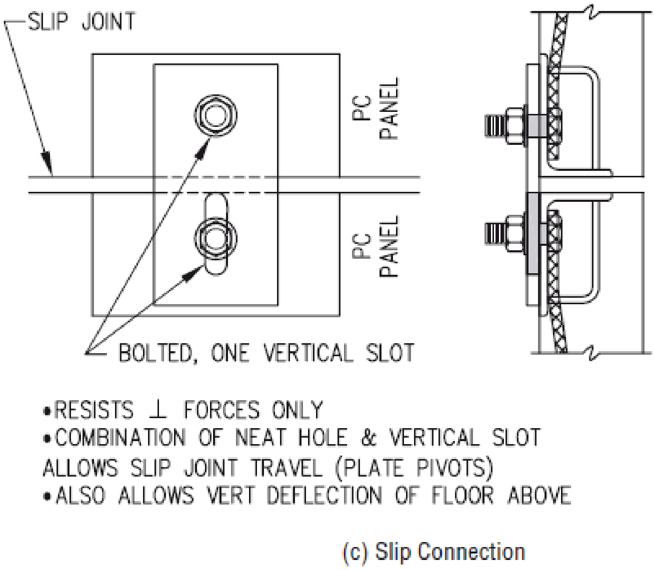
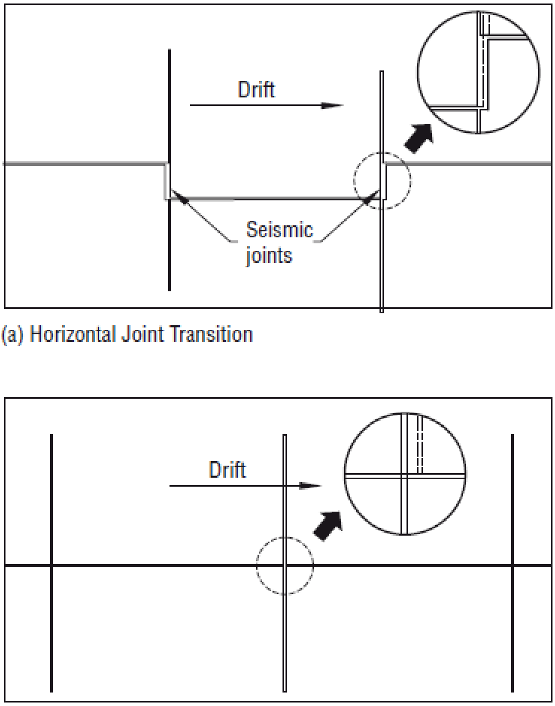


Figure 81 basic seismic connection types. Slip connection ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

Proper orientation and length of slotted inserts are necessary but not always sufficient to allow movement without binding. This is especially true if the connection parts are in compression against the connection body, or have high tensile forces that result in large friction forces against the fastener as slippage may be restricted. Corrosion protection of these sliding connections should also be considered to ensure their long-term performance so the sliding effect can occur without binding.

Flexible connections must have ample rod or plate length to truly bend and flex under drift without failing. All components of the connection system must be designed to allow either bending or sliding within the connection with slotted or oversized holes. The bottom connections will also have to be designed to handle the force that it takes to yield the upper connections. Careful installation and inspection are required to ensure that construction tolerances do not negate the available movement in a way to make the connection ineffective.

When panels are designed to translate, the horizontal joint at each level should remain at a constant elevation whenever possible, as it tracks around the perimeter of the building. This will permit the panels attached to one floor to move with that floor's drift relative to the panels above and below them. Elevation changes will require seismic joints at the transitions and detract from the aesthetics of the cladding.



(a) Horizontal Joint Transition

(b) Preferred Horizontal Joint

Figure 82 Joint elevation changes ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

A common way to avoid panel collisions is to increase the joint width, positioning adjacent panel beyond the limit of movement. A common case is where wall panels form the corner of the building. Wall panels are typically connected at two framing levels and consequently rotate out-of-plane in response to structure drift. In the case of the corner, building motion will be perpendicular to one panel while it is parallel to another, resulting in the joint between the two either opening or closing up. To avoid a collision, at outside corners the corner joint width must be increased relative to the magnitude of drift. Mitered panels may be used to reduce the width of the seismic joint required in this situation.

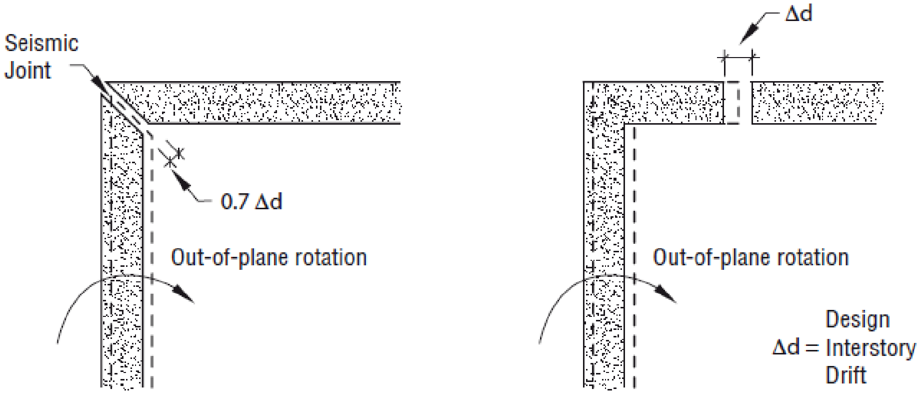


Figure 83 Corner joint made wider to avoid collision ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

9.2.4 Panels connected out-of-plane at two levels and in-plane at two levels (Rocking)

Bearing connections that allow vertical upward movement (lift-off) may be referred to as rocker connections. This type of connection would allow the panel to rotate in-plane. Rocking panels are vertical, in-plane simple spans. The upper connections must provide in-plane as well as out-of-plane support for the panel. The applied seismic force is resisted by horizontal reactions in the bearing connections and the upper lateral connections. The lower (bearing) connections must allow lift-off. The simple-pan reactions provide overturning stability, so there is no need for the bearings to resist tie-down forces (drift compatibility prohibits this).

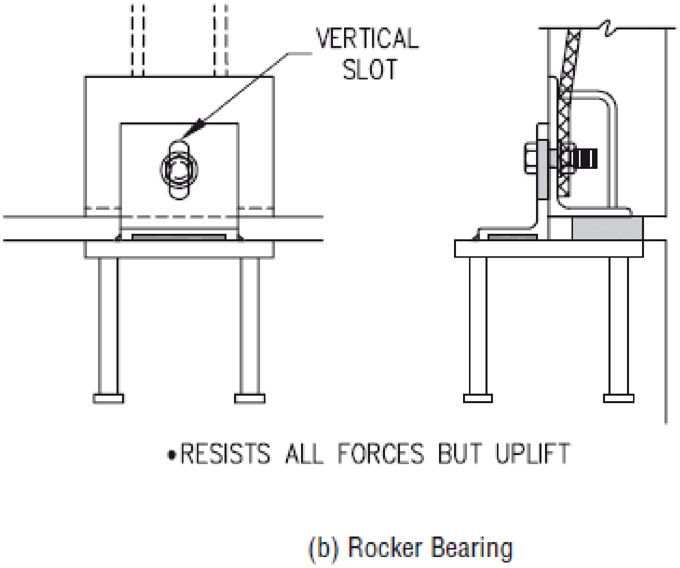


Figure 84 basic seismic connection types. Rocker Bearing ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

The bearing connections as well as the upper lateral connections must provide freedom for vertical motion of the panel as it rocks. The same rules apply as for the horizontal slots or yielding connections in the translating panels above, but in this case, the slots would be vertical.

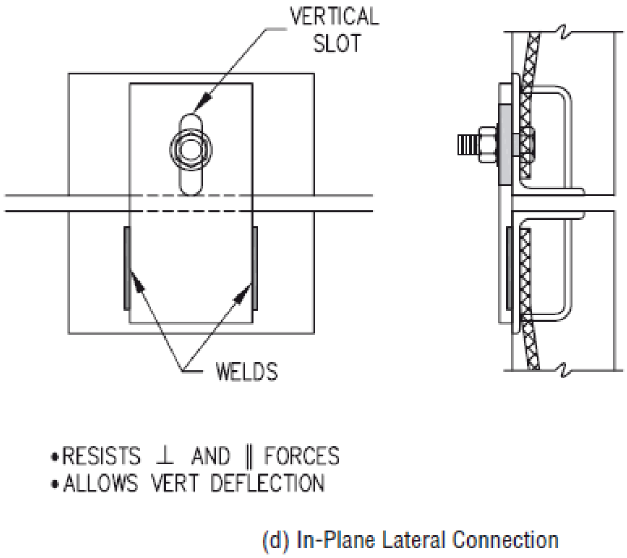


Figure 85 basic seismic connection types. In plane lateral connection ("Behavior of Architectural Precast Panels in Response to Drift". Designer´s notebook. Prestressed Concrete Institute. (2011))

The possible area for panel collisions now is no longer at the corners of the building because both panels are rocking in the same direction. The horizontal joint is at the top of the panel. It will open and close as the panel lifts up. One way to minimize vertical motion is to set the bearing connections closer together. Otherwise, the horizontal joint size may need to be increased.

9.2.5 How to choose between the two types of motion

A panel whose aspect ratio is small (height is similar to or significantly smaller than its width) is best designed for in-plane translation. If the panel were designed with rocking bearing connections and allowed to rotate, the upper horizontal joint would have to be sized to allow for large vertical movement of the panel. Depending on the specific geometry, this horizontal joint width could become quite large and affect the aesthetics of the cladding. More importantly, the force required to lift up the panel could easily become quite large. For these reasons, the panel should be designed to translate in-plane. The horizontal joint would be held to a nominal size and the overturning loads could reasonably be handled by taking advantage of the low aspect ratio of the panel.

Panel rotation (rocking) should be considered and rigid connections should be avoided in situations where the panels' aspect ratio is high (height significantly greater than width) because the resulting large overturning forces could become unmanageable. Instead of trying to resist the overturning force, rocker connections can be used to allow the panel to freely rotate to accommodate story drift. In this case, the bottom connections would be designed as rocker connections and the top connections would be designed as in-plane lateral connections.

9.3 Specific solutions

9.3.1 Seismic action parallel to the panel surface

Walls made with precast panels loaded to the ground are extremely rigid, therefore connections shall be designed in order to make it possible for the structure to move independently of the panels that remain fixed, so that the panel mass remains uninvolved during seismic effects.

If horizontal precast panels discharge their weight to pillars, their mass is involved but they must be implemented in a way so that the structure does not become more rigid.

9.3.2 Seismic action perpendicular to the panel surface

The panels do not offer rigidity and their connection to the structure must be sized according to their mass, with constraints that must be proportionate to the acceleration specified in the design.

Vertical panels placed on the ground FV/10 – FV/20

The FV/10 fisis connections for vertical panels permit a relative displacement of ±10 cm. The FV/20 fisis connections permit a relative displacement of ±20 cm: twice as much as the version described above. The Fisis connection for vertical panels may be anchored to the structure by wall anchors or with a template called backplate FVCP placed during concrete casting.

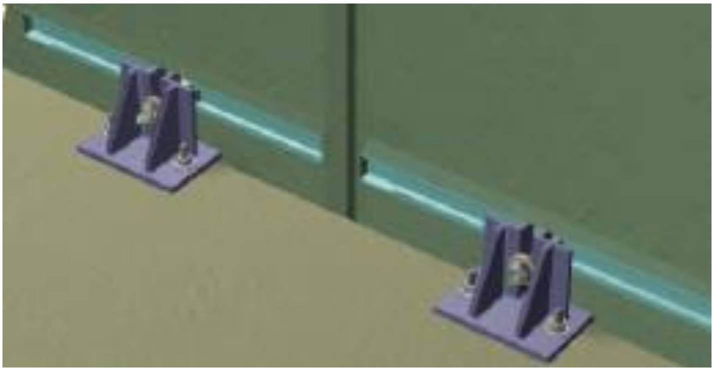


Figure 86 Vertical panels placed on the ground FV/10 – FV/20 (Ruredil. Construction chemicals and building technologies. Fisis Earthquake-proof system for fixing panels)

Horizontal panels placed on the ground FD/12

The fisis connections for horizontal panels that discharge their weight to the ground is called FD/12 and permits a relative displacement of ±12 cm between the pillar and the panel. This fisis connection uses the same retaining inserts as the vertical panels, but in this case they are inserted in the panels. A single guide is inserted in the pillar.

Figure 87 Horizontal panels placed on the ground FD/12 (Ruredil. Construction chemicals and building technologies. "Fisis Earthquake-proof system for fixing panels")

Horizontal panels placed on the ground FO/20

The FO/20 fisis connections for horizontal panels is used when the relative movement of ±12 cm permitted by the FD/12 connection is insufficient, and displacement up to ± 20 cm are required. This fisis connections remains exposed on the pillar, and so a specific protective box R120' is provided to meet fire resistance requirements.

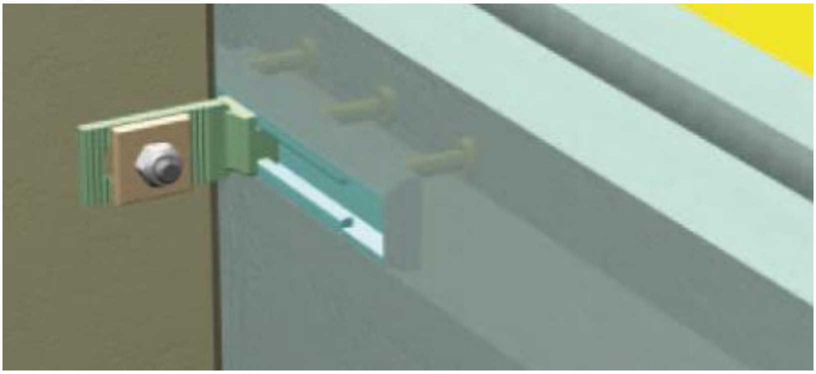


Figure 88 Horizontal panels placed on the ground FO/20 (Ruredil. Construction chemicals and building technologies. "Fisis Earthquake-proof system for fixing panels")

Horizontal panels suspended by pillars FO/00

If the panels are fastened to pillars (ex. on Girella or Tirella corbels), all rigidness should be eliminated with the fisis connections FO/00 that acts as a hinge, preventing relative displacement between the upper edge of the horizontal panel and the pillar.

The lower edge, must rest on 2 carriages in order to be able to move at least ±4 cm on the lower support corbels.

The two zeros show that for this use, there is no displacement and the guide permits ±10 cm of vertical tolerance in the installation. In the event of an earthquake parallel to the panel surface, the limit of the seismic action on the fisis connections FO/00 must be increased from 50 to 100 kN.

Figure 89 Horizontal panels suspended by pillars FO/00 (Ruredil. Construction chemicals and building technologies. "Fisis Earthquake-proof system for fixing panels")

9.3.3 Heavy cladding system

Heavy cladding systems consist typically of precast concrete: they may also have additional attached facing materials such as natural stone or ceramic tile.

Seismic codes require that heavy panels accommodate movement either by sliding or ductile connections. In high seismic zones sliding connections are rarely used, because of the possibility of incorrect adjustments when bolts are used, jamming or binding due to unwanted materials left after installation and jamming due to geometrical change of the structural frame under horizontal forces. Any of these causes may result in failure of the sliding faces that may be called upon to work instantaneously decades after their installation.

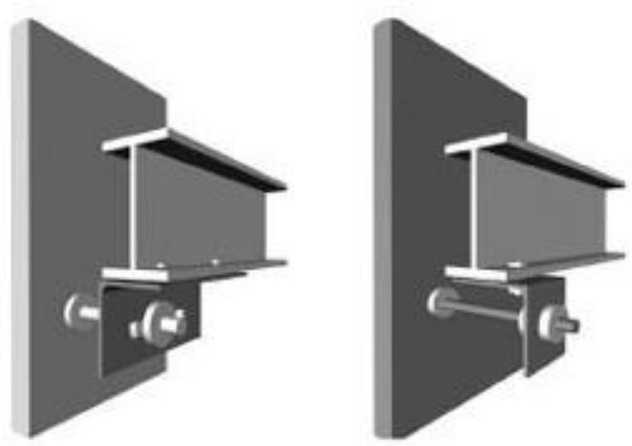


Figure 90 Examples of sliding (*left*) and push-pull (*right*) connections. (Based on Wang, 1971)

The need for disassociating the heavy panel from the frame has a major impact on connection detailing. As a result, a connection commonly termed "push-pull" has been developed, which provides, if properly engineered and installed, a simple and reliable method of de-coupling the panel from the structure. The generic connection method consists of supporting the panel by fixed bearing connections to a structural element at one floor to accommodate the gravity loads, and using ductile "tie-back" connections to a structural element at an adjoining floor.

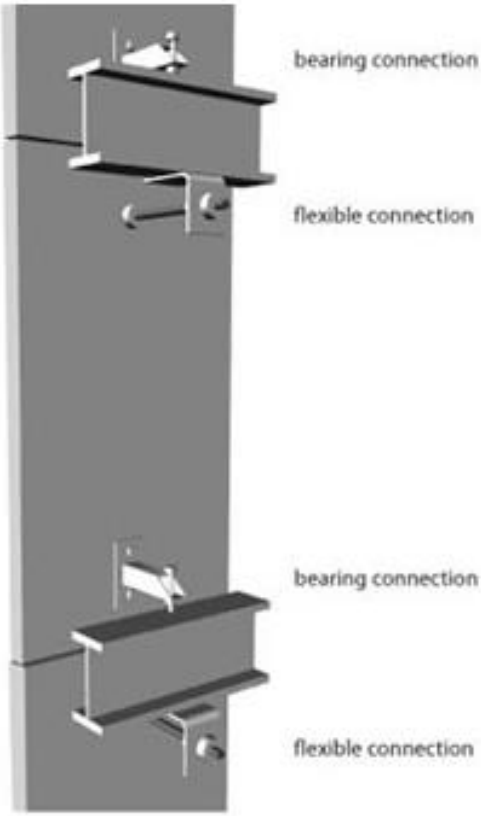


Figure 91 Typical floor-to-floor push-pull panel connections. Each beam has a bearing connection at the bottom of a panel and a flexible, or tie-back, connection for the panel below (Chris Arnold et al, 2009)

The tie-back connection is designed to deform under lateral forces and thus does not transmit racking forces to the panel. The tie-back must be capable of accommodating the out-of-plane forces on the panel, including wind.

Deep spandrel panels, that do not span floor to floor, are supported in the same way, with fixed and ductile connections, to accommodate possible deformation of the supporting under severe earthquake forces. The ductile connections also provide a simple means of aligning and leveling to panels. Narrow spandrel panels may have fixed connections to a structural member, and the inter-story drifts are then accommodated entirely by the glazing system.

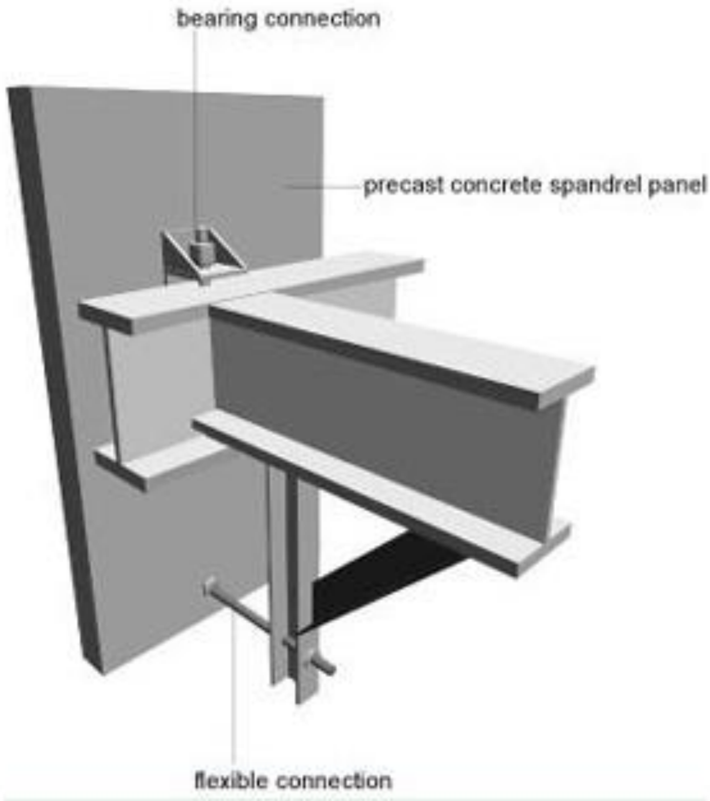


Figure 92 "Push-pull" connection for a deep pre-cast spandrel attached to a steel beam. Bearing connection at top, tieback connection at bottom (Chris Arnold et al, 2009)

The push-pull connection also represents one of the simplest ways of obtaining installation adjustments that are necessary for panel alignment, irrespective of seismic requirements. Since time is very costly in a competitive business environment, the need for connections that can be quickly installed is a major concern. Large pre-cast panels, using typical detailing, can be installed in less than 10 minutes per panel for low-rise construction and around 15-20 minutes for high-rise. This represents the time required for one cycle of panel installation, from lifting the panel from the ground (or truck), its placement, adjustment, and the removal of the lifting tackle.

Figure 93 Erection of precast concrete panels with push-pull connections (Chris Arnold et al, 2009)

Connections must be designed for safe temporary support while the panel is being adjusted, leveled vertically and horizontally and correctly spaced in relation to adjoining panels. Some installers complete the full panel installation, including welding, at the same time as the initial panel placement is done. In high-rise building, however, it is common to provide temporary placement for a large number of panels, before another crew returns to adjust and make final connections. The latter process maximizes use of the crane and operator. However, during final installation the panels are vulnerable to wind and seismic forces.

Casting, handling and erection economy mandates the use of the largest possible panels (subject to weight and transportation restrictions). The connections are expensive, both in construction and erection, and so the use of large panels reduces the number of connections. However, seismic requirements are in conflict with the use of very large panels because of the larger dimensional deformations in the main structure that must be accommodated. While in non-seismic areas two or three story height panels are used, normal practice in seismic zones is to use panels that are limited to one story in height and seldom more than one horizontal bay in width.

Since the panel connections, of which there may be hundreds or even thousands, are relatively expensive, designers sometimes reduce the number of connections necessary by varying to shape of the panels.

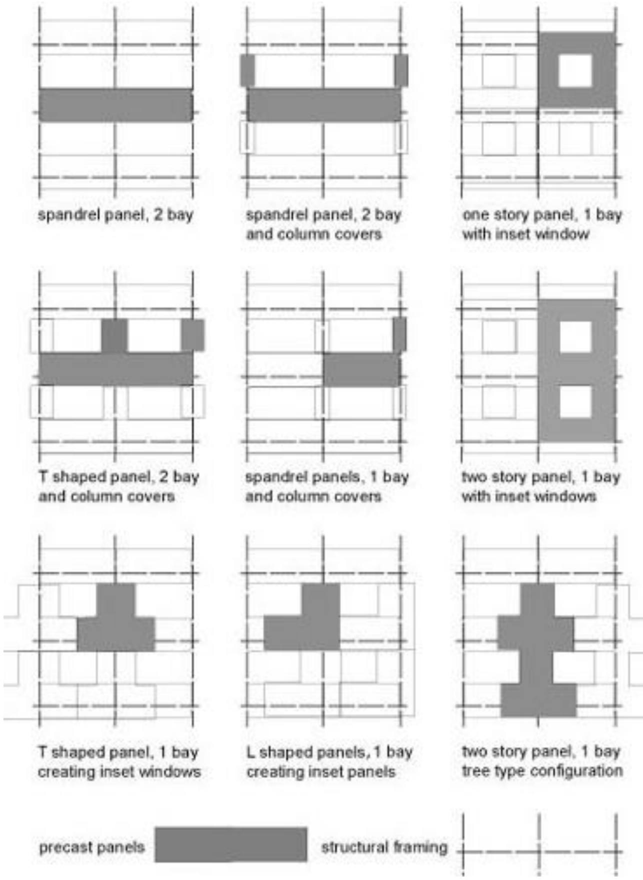


Figure 94 Panel façade layouts, including some designed to reduce the number of panels and connections (Chris Arnold et al, 2009)

Figure 95 shows an L-shape panel together with a parapet wall and its connections, and illustrates the complexity that may be necessary to achieve a satisfactory detail. The parapet panel is similar to a spandrel panel: the fixed connection is shown at the top and the ductile connection at the bottom.

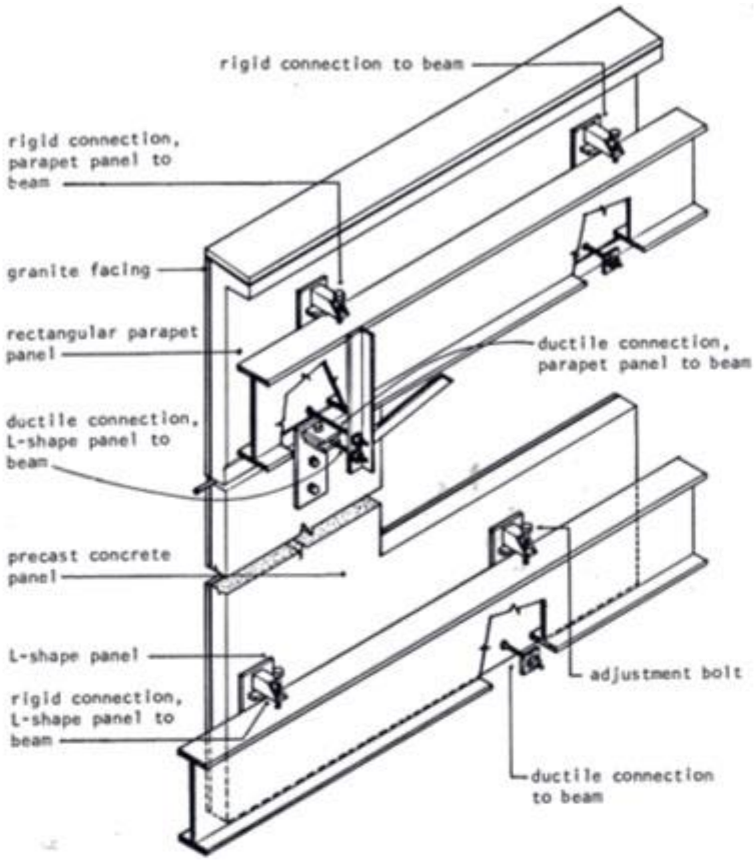


Figure 95 L-shape panel and its connections (Chris Arnold et al, 2009)

Another technique for reducing the number of panels and connection is to support a number of facing panels on a metal frame that is attached to the building structure. The frames can be shop welded: a number of facing panels are attached at the site and the entire assembly is then lifted up and attached to the structure. *Figure 96* shows a large spandrel panel assembly constructed in this way, using natural stone facing panels.

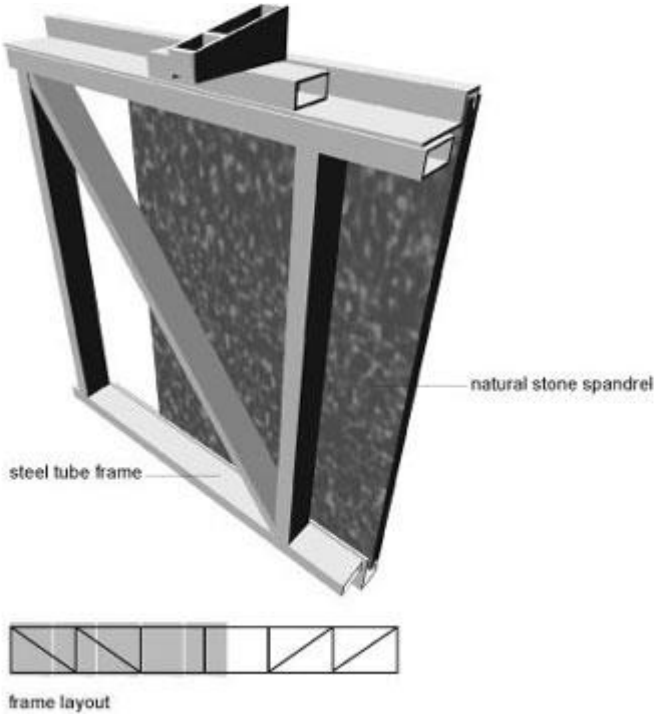


Figure 96 Spandrel assembly of natural stone mounted on a metal frame (Chris Arnold et al, 2009)

All these connection details are based on engineering principles and experience and represent a design response to accommodating the forces required by the seismic code. Vast numbers of these connections have been installed in the field, with a whole variety of configurations and details that have not been tested. Most engineers have developed typical details, but architectural requirements often require building-specific detailing. While the general concepts and execution of these details suggest that that they will perform safely in earthquakes there are a number of questions that might be answered by experimental test programs.

The typical connection detail is based on the concept that permits the structure to move independent of the panel. However, in successive floors, panels will move relative to one another. Slight dimensional variations or rotation of the beams may result in these panels touching with resultant undesirable interaction.

The typical detail is based on a two-dimensional concept that works for a limitless plane of panels. However, at outside and reentrant corners the problem becomes three dimensional and impact between the panels is possible under large drifts. *(Figure 97)* Use of larger gaps at reentrant corners and mitered panels at outside corners helps to relieve this possibility. *Figure 98* shows an alternative detail for solving the corner problem.

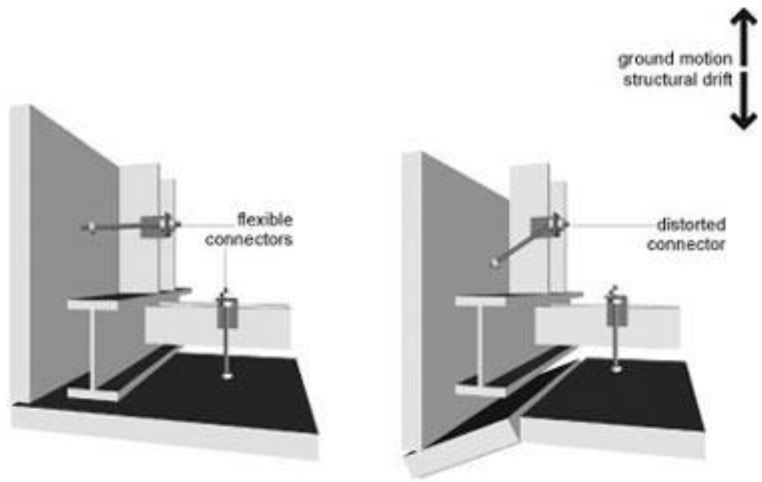


Figure 97 The cladding corner problem. Depending on the motion, panels may impact (Chris Arnold et al, 2009)

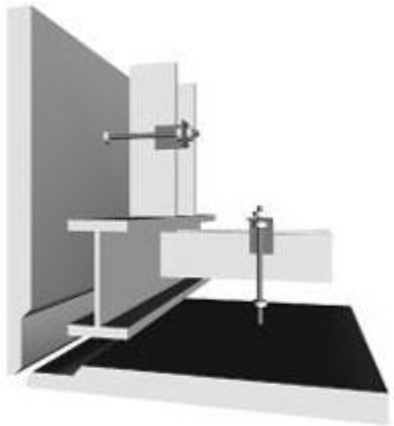


Figure 98 Typical detail for the attachment of corner panels showing mitered corner to permit panels to slide past one another with minimal damage (Chris Arnold et al, 2009)

Typical design of spandrel panels requires that the entire inter-story drift must be accommodated in the glazed space between the spandrels. Since this glass height is often less than one-half the floor-to-floor height the actual drift experienced by the glass may be more than double the design drift. While the detail may protect the panels, the exaggerated drift may difficult for the glazing to accommodate, particularly at reentrant corners. This condition is now accounted for in the seismic code.

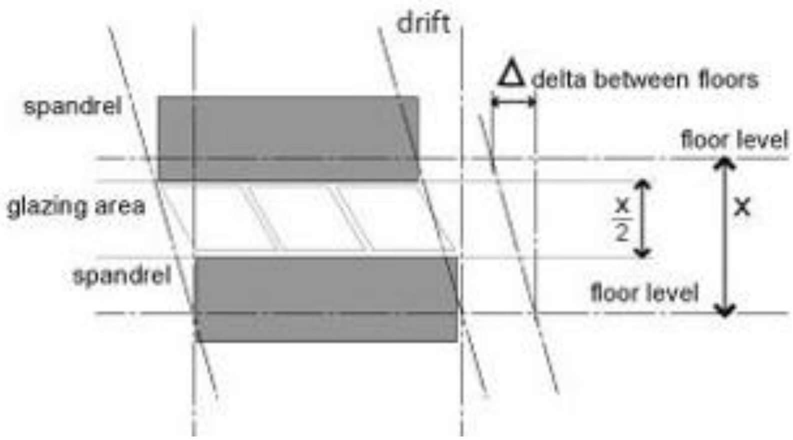


Figure 99 Diagram showing glass forced to accommodate the full height drift. Design interstory drift ratio is based on story height X. But connection of spandrel panels requires glazing to accommodate story drift in height X/2 or double the design story drift (Chris Arnold et al, 2009)

Large irregularities in vertical and horizontal configurations (high floors, set-backs, reentrant corners) can create serious problems of torsion and stress concentration that will result in distortion and drifts beyond those that the cladding can accommodate. Careful scrutiny of these conditions is necessary at an early design stage to ensure that the cladding detailer is not faced with conditions that may be almost impossible to make safe.

Weather protection of metal connections is an important long-term consideration, particularly in coastal regions with relatively salt-laden air. Current detailing methods generally result in a concealed connection once the building is finished, though bottom connections are sometimes revealed when ceiling panels are removed. Weather protection is accomplished by caulking butted joints: edge profiles that incorporate weather stops and drips are little used. This results in simpler forming and casting, with less likelihood of delicate edge sections fracturing upon formwork removal. However, prevention of moisture penetration is entirely dependent on the long-term integrity of the sealant.

9.3.4 Anchored Masonry Veneer

Brick veneer/steel stud walls must resist loads as prescribed by the governing building code(s). For exterior, nonbearing walls, these loads are typically due to wind and seismic events. Although a veneer is defined as a nonstructural facing, brick veneer does resist loads. Certainly the weight of the brick is supported by the brickwork itself. But brickwork also contributes to the resistance of out-of-plane loads generated by wind and seismic events. In addition, in-plane forces caused by the weight of the brickwork are also resisted internally, including in-plane loads generated by seismic events.

Anchored veneers are typically masonry, stone or stone slab units that are attached to the structure by mechanical means. These units and their connections must be designed to accommodate the anticipated seismic drift; otherwise they may pose a significant falling hazard.

Any typical causes of damage are:

- Anchored veneers and their connections may be damaged by inertial forces and by building distortion; units located at corners and around openings are particularly vulnerable.
- Rigid connections may distort or fracture if they do not have sufficient flexibility to accommodate the seismic drift; veneer units may crack, spall, or become completely dislodged and fall.
- Deterioration or corrosion of the mechanical connections is a significant concern; corroded connections may fail prematurely. Maintaining water-tightness at joints is important for the longevity of the anchors.

Seismic mitigation considerations are:

Conform to the requirements of the ACI 530 Building Code Requirements for Masonry Structures [51] for seismic zones. The code requires the veneer wall to have a continuous wire imbedded in the mortar joint in high seismic zones. This wire is then connected to the veneer anchor which is attached to the backup structure. The Seismic System allows greater stability; safeguards against problems caused by thermal expansion and contraction, and provide uniform distribution of the lateral forces.

As to the anchors specified in each of the model codes are similar. They are of steel and may be:

- Corrugated sheet of 0.76 mm by 22 mm
- Sheet metal of 1.5 mm by 22 mm
- Wire of minimum MW11

Anchors come in a variety of configurations *(figures 100 and 101).* Corrosion resistance is achieved with zinc or epoxy coatings or the use of stainless steel. Play in two piece adjustable anchors is limited to 1.6 mm.

Figure 100 Typical Unit Veneer Anchors

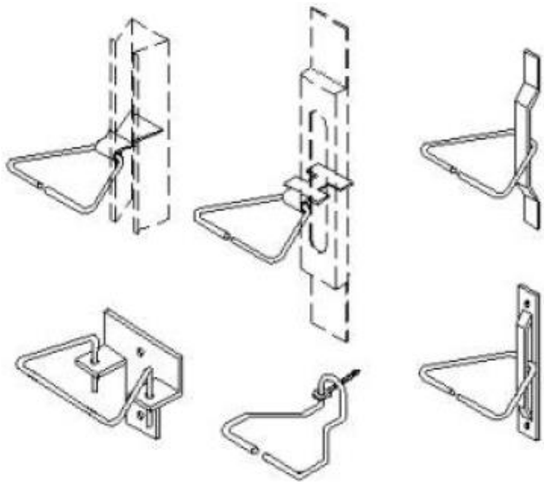


Figure 101 Typical Adjustable Anchors

In seismic regions, the use of seismic clips is recommended. A seismic clip engages continuous wire reinforcement in brick veneer. Both the seismic clip and the wire are embedded in the veneer's bed joint *(figure 102).*

An example of anchorages according to the requirements for the U.S. standard (Building Code Requirements for Masonry Structures [51]):

Anchors should provide the capacity to transfer loads applied to a maximum of 0.25 m^2 of wall area. Each anchor should be spaced a maximum of 457 mm on center vertically and a maximum 813 mm on center horizontally. They must be securely attached through the sheathing to the steel studs, not to the sheathing alone. Around the perimeter of openings, additional anchors should be installed at a maximum of 914mm on center within 305 mm of the opening.

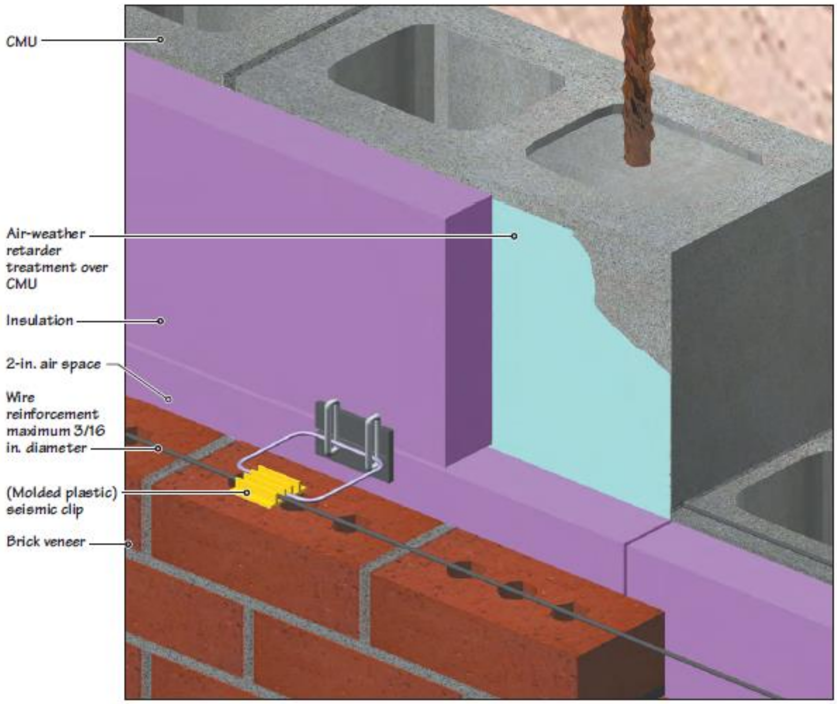


Figure 102 A typical seismic clip in brick veneer. The clip is embedded in the mortar along with wire reinforcement (Medan Mehta et al, 2011)

All anchors must be embedded at least 38 mm into the brick veneer with a minimum mortar cover of 15.9 mm to the outside face of the wall. Anchors in Seismic Design Categories E and F must be mechanically fastened to horizontal reinforcement in the brick veneer.

Some commercial examples that comply with U.S. regulations:

MATERIAL	ASTM STANDARD SPECIFICATIONS
Brass	ASTM B-16/B16M-05
Bolts and Anchor Bolts	A307-04 Grade A
Masonry Joint Reinforcement	A951-02 and UBC Standard 2106.1.1
PVC Control Joints	D2287-99 (2000)
Rubber Control Joints	D2000-05
Sheet Metal, Carbon Steel	A1008/A1008M-05b, A109/A109M-03 and A1011A-03
Sheet Metal, Hot Dipped Galvanized	A153/A153M-05 (1.50 oz/ft² - 458 g/m²)
Sheet Metal, Mill Galvanized	A653/A653M-05a (0.60 oz/ft² - 183 g/m²)
Sheet Metal, Stainless Steel	A167-99 (2004) - Type 304
Steel Plate, Carbon Steel	A36/A36M-05
Wire, Carbon Steel	A82/A82M-05a
Wire, Hot Dipped Galvanized	A153/A153M-05 (1.50 oz/ft² - 458 g/m²)
Wire, Mill Galvanized	A641/A641M-03 (0.10 oz/ft² - 30 g/m²)
Wire, Stainless Steel	A580/A580M-98 (2004) - Type 304

Table 5 products and corrosion protection specifications according to U.S. regulation

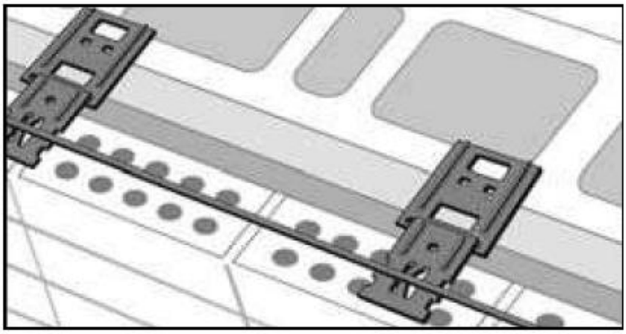


Figure 103 seismic joint reinforcement DA8706 pencil rod (DUR-O-WAL, 2007)

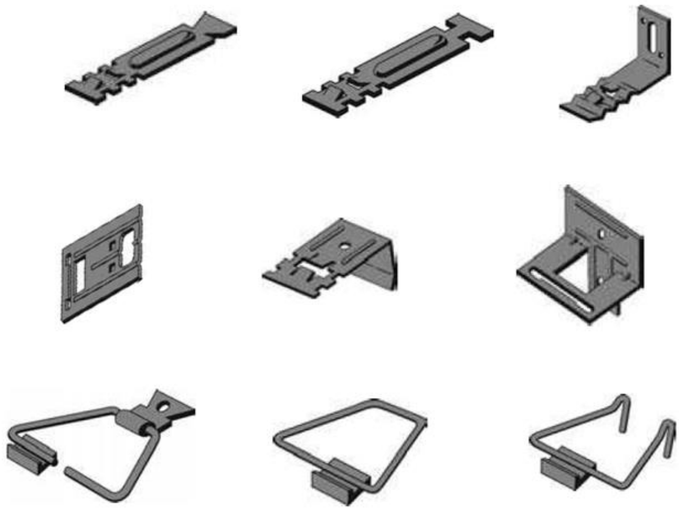


Figure 104 Seismic Ties and Anchors (DUR-O-WAL, 2007)

A typical section and elevation of a brick veneer – concrete block wall system is shown in *figure 105.* The shelf angle, which is attached to the floor system, carries the weight of the brick veneer, while the floor slab carries the weight of the concrete masonry backup wall. The two walls are attached to each other by means of lateral ties. Compressible filler is placed under the shelf angle so as to provide a horizontal control joint (a ‘soft joint’) [52]. Vertical and horizontal control joints are necessary to prevent stresses due to differential elastic, thermal, moisture, creep, and shrinkage deformations between the brick veneer, the backup wall and the rest of the building. Because of the potential for differential vertical deformation between the two wythes, the ties should have some flexibility with respect to in-plane differential movement between the two masonry wythes. However, the ties should restrain any out-of-plane movement.

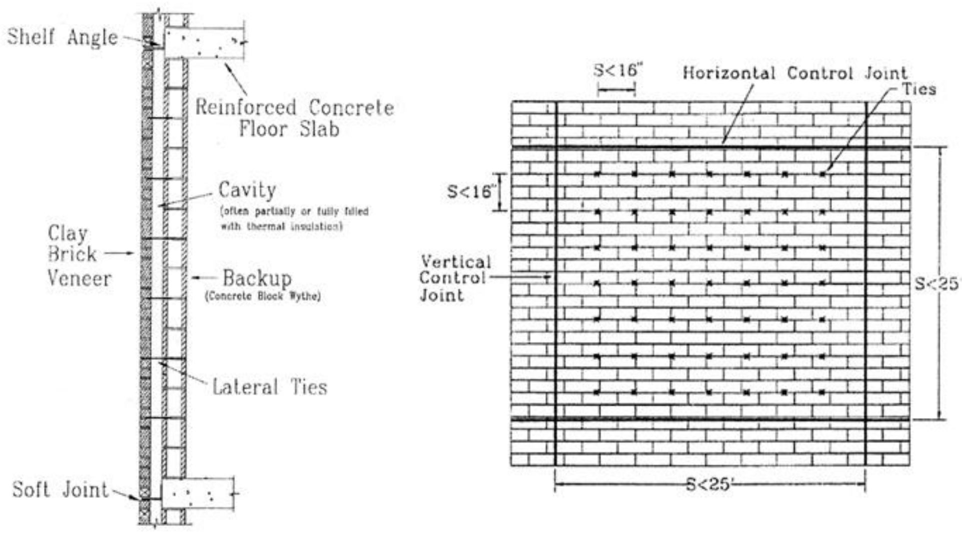


Figure 105 Section and elevation of a typical brick veneer wall system (Ali M. Memari et al, 2002)

10. CURTAIN WALLS

As we can see, the term “façade” is very wide and general. It can indicate and correspond to a huge variety of building typologies, just because it refers to the front of the building, but not to the technology or the way it has been built.

The term “curtain wall” instead, is much more specific and indicates a type of perimetric wall or enclosure real different from a normal and “traditional” one, because it is neither load-bearing the loads of the upper stories nor leaned and sustained by the underneath floor or beam. On the contrary it deals about a perimetric enclosuring wall that is completely outside the building and it is directly hung to the structural system, for the most to the beams or to the floors.

10.1 Curtain walling systems

Consequently, maintaining as a fixed point the outside position and the attachment to the structural elements of the building, we can find again a very huge variety of technologies for realize a curtain wall. Very briefly we can summarize the different typologies as follows:

- Stick
- Unitized and panellised
- Structurally sealed
- Structural glazing
- Single and double skin

10.1.1 Stick system curtain walling

Horizontal and vertical framing members (‘sticks’) are normally extruded aluminium profiles, protected by anodising or powder coating, but they may also be cold-rolled steel (for greater fire resistance) or aluminium clad with PVC-U. 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. Mullions are typically spaced between 1.0 and 1.8 m centres.

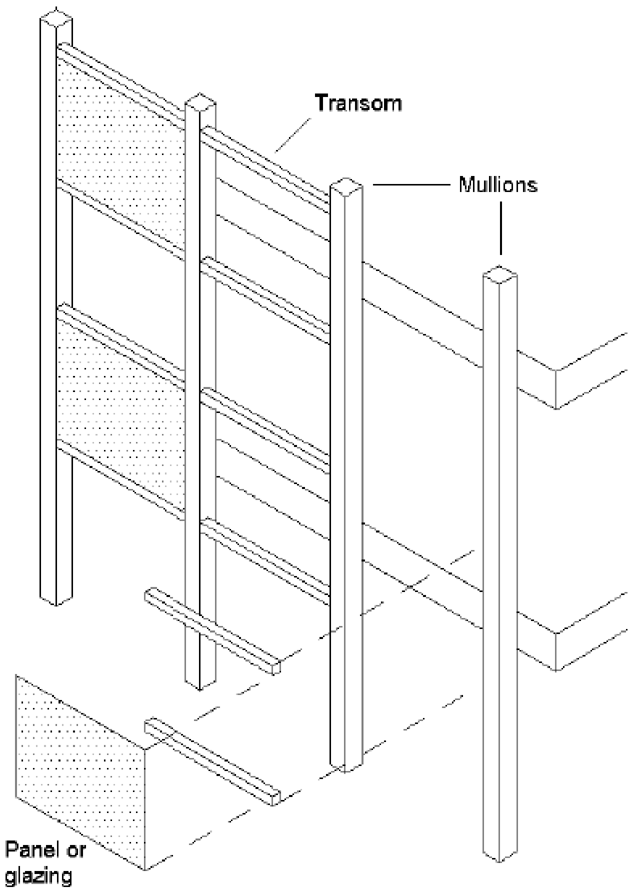


Figure 106 Stick system curtain walling: schematic drawing (Umberto GALLI, 2011)

10.1.2 Unitized and panellised system

Unitized systems comprise narrow, storey-height units of steel or aluminium framework, glazing and panels pre-assembled under controlled factory conditions. Mechanical handling is required to position, align and fix units onto pre-positioned brackets attached to the concrete floor slab or to the structural frame.

Unitized systems are more complex in terms of framing system, have higher direct costs and are less common than stick systems.

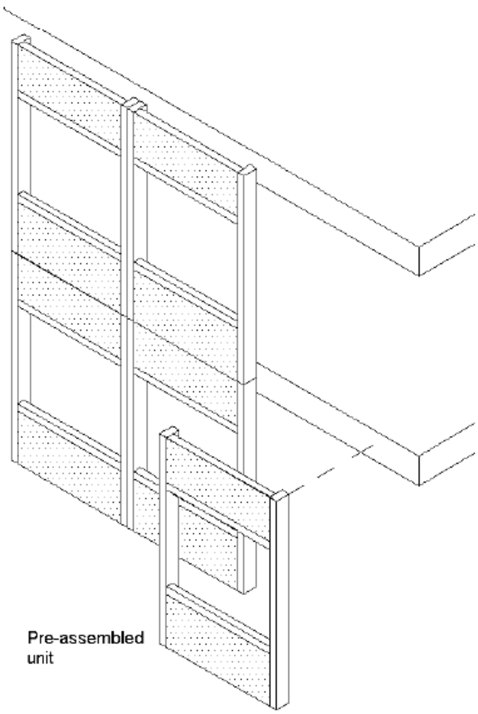


Figure 107 Unitized and panellized system curtain walling: schematic drawing (Umberto GALLI, 2011)

10.1.3 Structurally sealed system

Structural sealant glazing is a type of glazing that can be applied to stick unitised and panellized systems. Instead of mechanical means (i.e. a pressure plate or structural gasket), the glass infill panels are attached with a factory-applied structural sealant (usually 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.

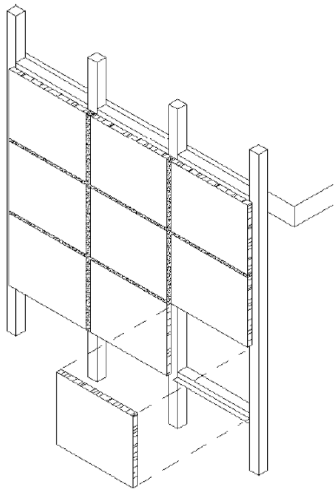


Figure 108 Structurally sealed system curtain walling: schematic drawing (Umberto GALLI, 2011)

10.1.4 Structural glazing system

Sheets of toughened glass are assembled with special bolts and brackets and supported by a secondary structure to create a near transparent facade or roof with a flush external surface.

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

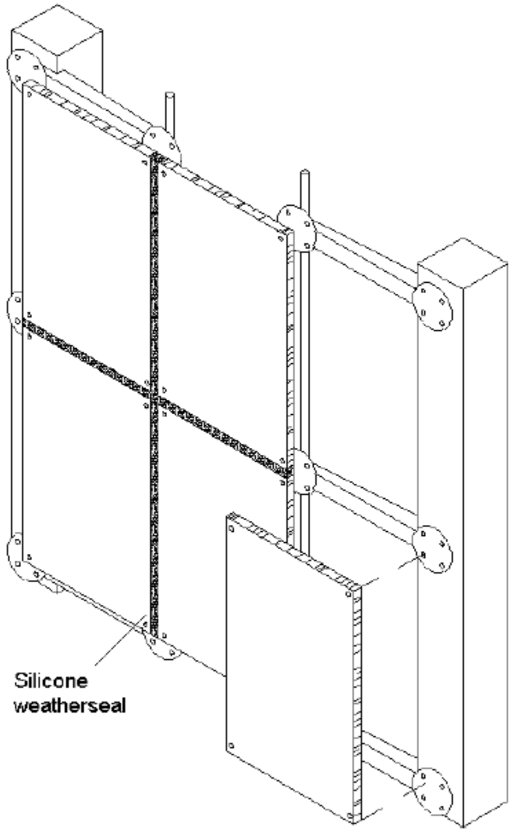


Figure 109 Structural glazing system: schematic drawing (Umberto GALLI, 2011)

Among all these curtain wall system, the unitized and panellized system is the most common one. This because of its great advantages compared to the others. One of the most important is that it is feasible also in case of very high building, because it doesn't need any scaffolding during the installation phase in the construction site. Every single unit in fact is pre-assembled in workshop, freighted to the construction site with trucks and finally mounted on the structure exploiting the available cranes.

Figure 110 Several moments during the installation phase of a unitized curtain walling system (Umberto GALLI, 2011)

So, every single critical phase of the façade construction is narrowed down to a more protected and controlled environment, such as a workshop or a factory where it is assembled. Once that a unit is ready, the only phase left is to move it to the construction site and to mount it in a very easy and fast way.

Another great advantage is the performance of this type of facades against the air and water permeability requirement. Also at very high height, where it's possible to have extremely strong wind pressure, it can ensure the necessary airtight and watertight sealing.

10.2 Fastening system

The unitized and panellized system is constituted of different modular units. Every unit, as we said before, has 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. Anyway the main requirements that have to be satisfied always remain the same:

- Horizontal tolerance
- Vertical tolerance
- Load-bearing capacity, against different types of loads, vertical and/or horizontal

Horizontal and vertical tolerance requirements are satisfied providing appropriate components or brackets. Brackets represent the fixing system and there are mainly two types of them: brackets fixing the facades to the main structure (steel or concrete) and brackets fixing facades components (e.g. vertical supports for glass, decorative elements). They usually are made of aluminium or steel. They must be designed to absorb tolerances, vertical and horizontal, of façade installation and the displacements of the building during its life.

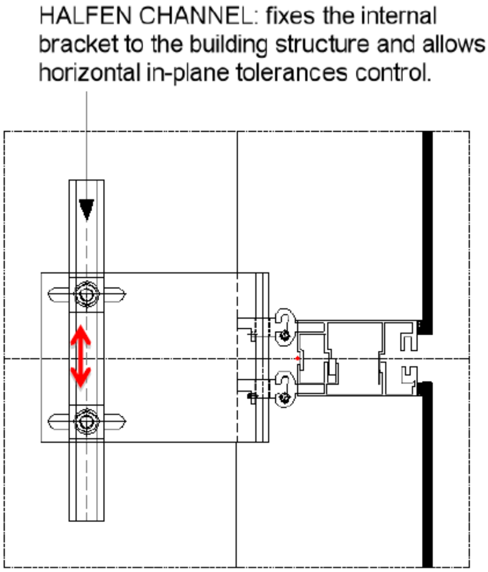


Figure 111 Horizontal section of the façade in correspondence of the fastening system (Umberto GALLI, 2011)

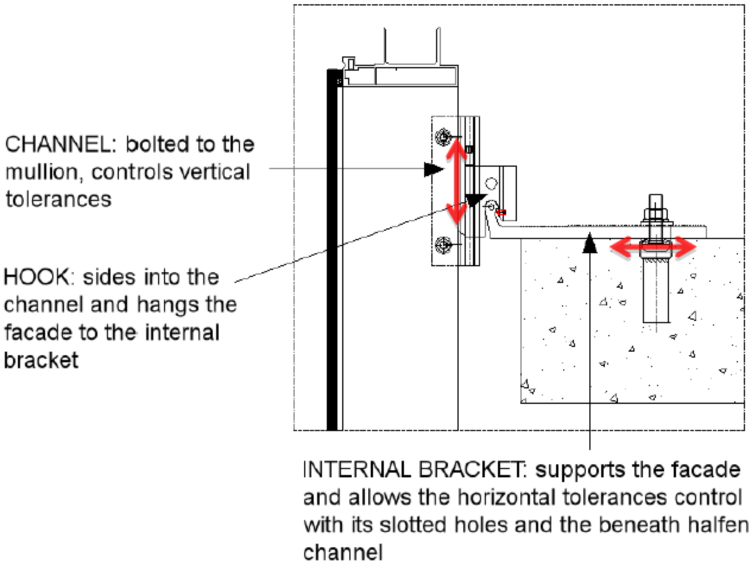


Figure 112 Vertical section of the façade in correspondence of the fastening system (Umberto GALLI, 2011)

Furthermore brackets and fixing devices must be designed and verified for different loads:

- The dead load coming from the self-weight of the unit itself;
- The wind load produced by the wind pressure on the façade;
- The additional load caused by people standing on the walkway in the gap, typically present in double skin façade. Usually for the calculation the limit state method is used.

10.3 Aluminium frame

The structure of the façade unit is constituted by different profiles, vertical and horizontal, that all together make up the frame of the unit itself. This is the element of the unit that actually resists to the wind pressure acting on the façade. In fact the different surfaces, glasses or spandrel elements, pick up the wind pressure and unload it to the frame. This has to resist to a bending stress and, at the same time, to unload the horizontal forces to the fasten system, already calculated and verified to resist to it.

The vertical profiles are called “mullions” and they are the most stressed elements of the frame, mainly because they cover the height of a storey and so they are also the longest profiles. The horizontal elements, instead, are called “transoms” and they 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.

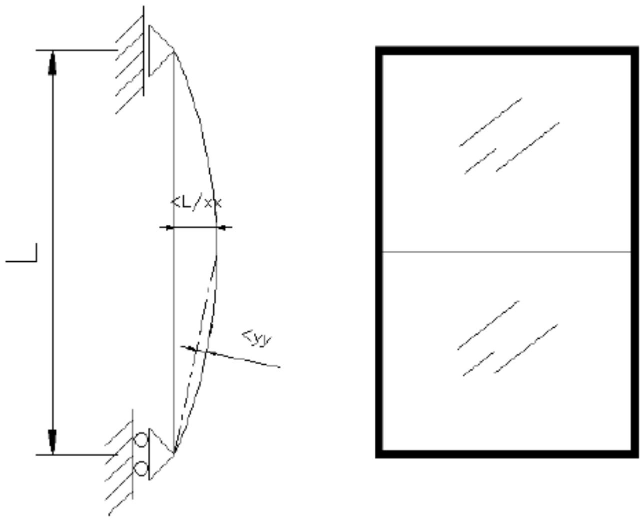


Figure 113 Schematic representation of the static scheme of a façade unit (Umberto GALLI, 2011)

10.4 Glazed curtain wall

The popularity of glazed curtain wall system is increasing due to a number of facts including aesthetics, increased natural light and sustainability considerations. Structural design of glazed curtain wall involves the analysis and design of the panels and connections to resist the out-of-plane wind pressures and to accommodate in-plane deflections resulting from wind induced building drift, long term floor deflections and thermal movement. In addition, the glazed curtain wall systems need to be able to resist earthquake loading without collapse. An important performance parameter is the ultimate drift capacity of the façade, particularly for glazed curtain wall systems. Severe earthquakes and wind cause damage to the main structural components of buildings as well as to the nonstructural elements such as window glass panels and curtain walls. There are two major concerns related to architectural glazing performance during and immediately following seismic event.

- Hazards to people from falling glass.
- Building down time and cost to repair.

The glazed curtain wall can be classified into two main types, namely; frame glazed curtain wall and frameless curtain wall.

10.4.1 Frame glazed curtain wall

Framed glazed curtain walls are typically designed with extruded aluminium members, although the early curtain walls were made of steel. The aluminium frame is typically in-filled with glass, which provides an architecturally pleasing envelope, as well as benefits such as natural day lighting. These curtain walls are designed to span multiple floors, and take into consideration design requirements such as, thermal expansion and contraction, building sway and movement, water diversion, and thermal efficiency for cost-effective heating, cooling, and lighting in the building.

10.4.2 Frameless glazed curtain wall

A growing number of architects are substituting the design of frame glazed curtain walls with frameless glass walls. With good design these unconventional glass walls can provide energy savings by allowing natural light to enter the buildings. This has a particular potential in architectural expression in terms of transparency, by removing the mullions and aluminium profiles from the pure planar nature of the glass surface, gives it a relief standing out from the transparent planes. Double glazing also can be fixed in this method as more energy efficient concept instead of a single sheet of glass. The two sheets are installed into a sash or frame with an air insulation gap between them creating a sealed unit. Many frameless glazed systems are now available for the use of glass in façades, all aimed at achieving maximum transparency by reducing the support structure such as,

- Point fixed glass on base supported steelwork. These are simple posts, trusses and fins. Trussed posts of various forms may be used to support glazing where the height of glazed walls exceeds 4.0 m.
- Point fixed glass on cable systems. Support structures can be constructed almost entirely from tension elements, such as rods or wires, and are therefore very light both physically and visually. Loads have to be transferred, at both ends of the cables, to boundary support structures. The weight of the vertical glazing is either supported by a tie rod hanger system or by each panel being suspended from the above panel.
- Fin walls comprising one-way spanning glazing supported on glass beams or fins. The glazing is either attached continuously to the fins using a soft silicone sealant or is connected intermittently using bolted connections. The structural model is that of a glass plate supported on its edges by glass beams.

Modern frameless glazing systems often bolt on steel support structures, which are important architectural elements and combine structure stability with aesthetic expression. The bolts provide point support to the glazing panels. Applications of bolted glazing systems range from simple structures, as shop windows and settlers, to more complex multi-storey buildings and large atria. Bolted fixings are commonly located towards the corners of the glazing panels, and additionally at intermediate points on long edges. They connect glazing panels to glazing support attachments, which in turn are connected to support structures.

Movement of the glazing panels and of the support structure as a result of thermal effects and under applied loads should be accommodated. If movement is resisted, stresses can be developed in the system. Provision is therefore usually made for bolted fixings, glazing attachments, or elsewhere to allow rotation and movement. Loads that arise as a result of the self-weight of the glazing system, including applied loads and load transfer effects that may occur when glazing panels are broken or removed.

The use of glass connections falls into two categories; such as a bolt through the glass that bears on the glass; or friction plates that are clamped on to the glass by bolts. Friction plates may be used to connect metal brackets to a piece of glass. They are also used to connect glass sheets by using patch plates that cover both pieces of glass. Metal plates are placed on both sides of the glass and clamped together to generate a normal force and a corresponding frictional load capacity in the plane of the glass. A suitable interface is required between the glass and the plates. A soft metal (pure aluminium) or fibre-reinforced plastics are normally used to provide the required coefficient of friction and accommodate any lack of flatness between the glass and the metal plate.

10.5 Glass

The most important element of the façade, either for its dimensions, its weight or for its frailty behaviour that imposes specific calculation methods, 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. As we said before, there are many different typologies of curtain wall and 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, so that the glass plate is tightened and fixed in the aluminium frame. Alternatively the restraint can be constituted by a structural silicon joint (Structural sealing façade) that, under appropriate verification and calculation method, 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".

A special attention should be taken for the design of the panel and glass behaviour under seismic action, represented by the drift between adjacent storeys for the façade, that could provoke glass rupture and potential fallout [53].

Glass breakage and fallout from the façade frame is one of the most common consequences for the curtain wall façade in case of earthquake. Even in case of lower magnitude seismic events, if the façade is not well designed, the failure of the glass components could occur, causing not only an immediate and serious hazard for people, but also the building to be unfit for use and declared inaccessible.

The hazard level represented by the glass rupture is mainly dependent on the glass type itself. The main glass typologies that it is possible to find in a curtain wall, or, more generally, in a building façade, are listed as follows [53, 54]:

<u>Annealed:</u> is the standard float glass product that has been slowly cooled after forming in the molten tin float bath. The slow, uniform cooling to the room temperature results in a relatively stress-free material that can be cut, drilled, edge worked, etc.

Heat-strengthened (HS): it is nominally twice as resistant to uniform wind loads as standard annealed glass and is produced in a similar way as following explained for the FT glass, as follows.

Fully tempered (FT): it is four times as resistant as annealed glass. The heat treatment processes for HS and FT involve heating the glass until it becomes soft and then uniformly quenching it on both sides with powerful air jets to cool and solidify the outer skin rapidly. The inner core of the glass then cools, shrinks, and puts the skin into a state of compression, with an equal and opposite tensile stress in the almost flawless middle core of the glass thickness. The quench process for HS glass is less vigorous than for FT glass and so creates less compressive stress on the exterior surfaces. Because glass breaks primarily under tensile stress, any wind load that causes bending must first overcome the built-in compressive stress of the heat treatment process, and so heat-treated glass is significantly stronger than annealed glass, which has essentially no built-in surface compressive stress. Because heat-treated glass (HS and FT) has had its temperature raised to the point where the glass becomes soft, it will not be as flat as annealed glass and will often show some visible distortion, especially in reflected images when viewed at longer distances, as compared to annealed glass. When broken HS glass will have a break pattern of relatively large pieces, similar to annealed glass, while FT glass shatters into myriad cubes each about the size of the glass thickness.

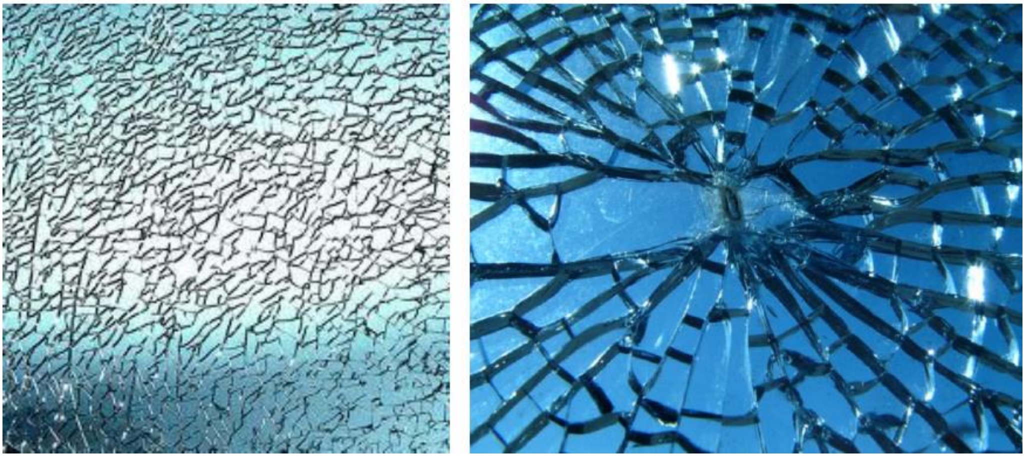


Figure 114 Typical example of fully-tempered (FT) glass breakage (Umberto GALLI, 2011)

Laminated: this glass is made by assembling a sandwich of two or more plies of equal or differing layers of glass with a transparent adhesive interlayer. This interlayer, usually polyvinylbutyral (PVB) or epoxy between two plies of glass, has nearly the same strength and stiffness as monolithic glass under short duration loads, but acts as a “safety glass” when broken by remaining in the frame and offers significant penetration resistance. The uniform load resistance is difficult to compute exactly. The plastic interlayer materials have a stiffness under short-term loads, especially at room temperatures and lower, which make the glass behave in a monolithic manner under short duration loads. For long duration loads or at high temperatures, a more conservative method is to use a layered approach, which assumes that each ply carries half the load (assuming they are of equal thickness) with no shear stress resistance offered by the interlayer. The main reason to use laminated glass is usually to supply protection to the building envelope against penetration and so the important variable then becomes the load resistance of the interlayer material itself after the glass plies have broken. If needed, this value is best obtained by full-scale testing.

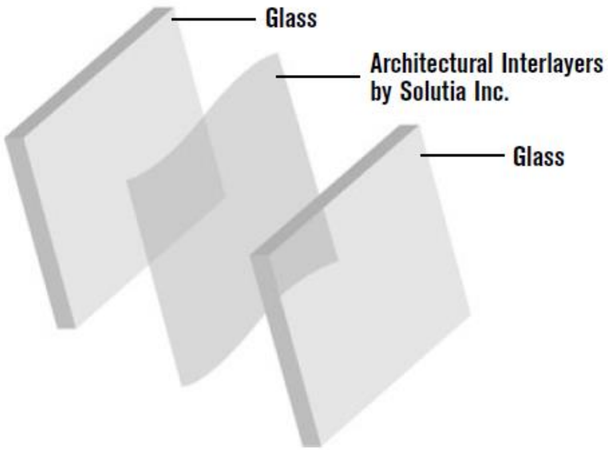


Figure 115 Laminated Glass Schematic (SAFLEX, 2007)

Consequently and dependently upon the different typology of glass we consider, influenced by the way it has been produced and manufactured, the consequence of its failure could really vary.

In terms of life safety, mainly of someone walking outside at the bottom of the façade, the principal hazard comes from the glass shards falling down from the façade.

Figure 116 Examples of damage to spider glazing systems (Andrew Baird et al, 2011)

The annealed type is probably the worst because it breaks in large and wide shards and it cannot stay in the frame once broken, so that all its shards fall down becoming an incredible hazard for someone walking under. The heat-strengthened behaves in a similar way, excepting the higher values of loads resistance.

Instead the fully tempered glass has a different behaviour caused by the uniform high compressive stress-state. In fact the glass breaks in small shards, approximately about plate thickness, that are much smaller and so less dangerous than those deriving from annealed glass rupture. In addition it’s also remarkable how the glass plate, when it breaks in a vertical position, doesn’t fall out from the frame excepting if it is charged with high values of horizontal load.

Finally 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).

10.6 Specific solutions

10.6.1 Allowing for Movement: Four Approaches

Four generic approaches are shown but it is possible for different methods to be used in one glazing system or in one building.

<u>*Seismic frame*</u>

The glazed frame moves in a seismic frame, which moves with the building. The glazing frame is usually fixed at the sill.

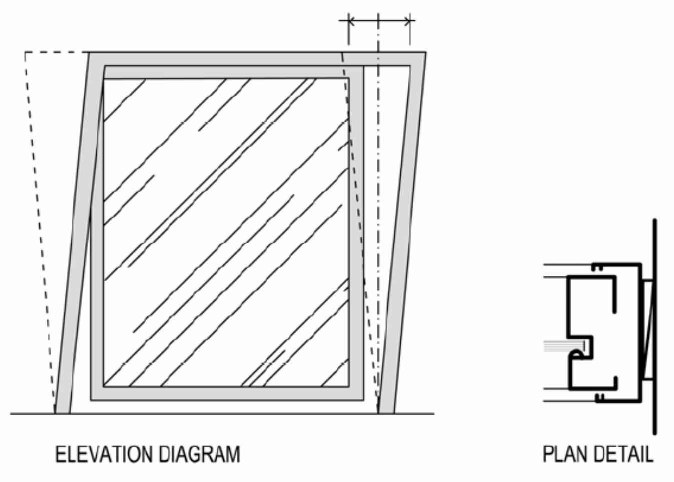


Figure 117 Seismic frame (Architectural Design for Earthquake, 2007)

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.

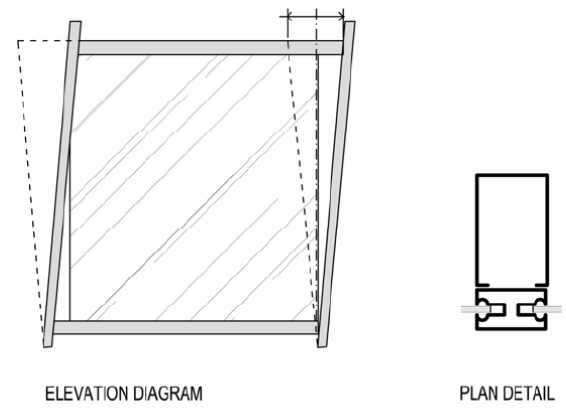


Figure 118 Glazing Pocket (Architectural Design for Earthquake, 2007)

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 work especially. Structural silicone is often used for fixing glass in unitized systems, but in this case the silicone itself is not required to accept deflections.

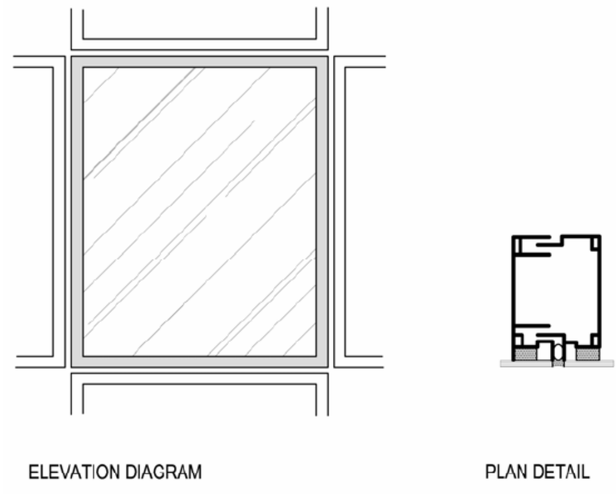


Figure 119 Unitized system (Architectural Design for Earthquake, 2007)

Structural silicone

Where the other approaches provide a positive gap, in this case movement depends on elasticity of the silicone. This approach is often used in conjunction with 'stick' systems.

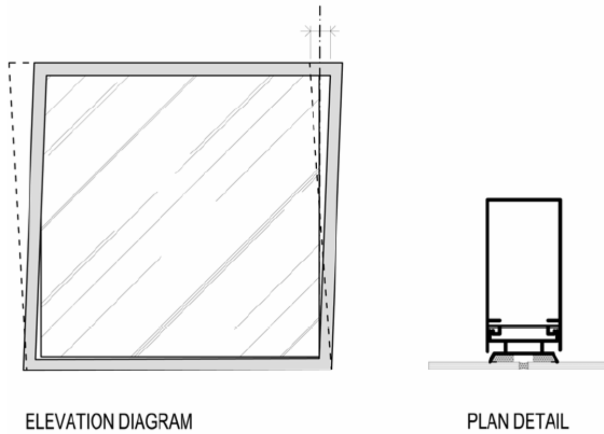


Figure 120 Structural silicone (Architectural Design for Earthquake, 2007)

10.6.2 Seismic Frame Designs

The design of glazing assemblies depends on the calculated inter-story drift for the building. Glazing generally performs better with stiffer structural systems that have lower inter-story drift or where larger edge clearances are provided at the mullions.

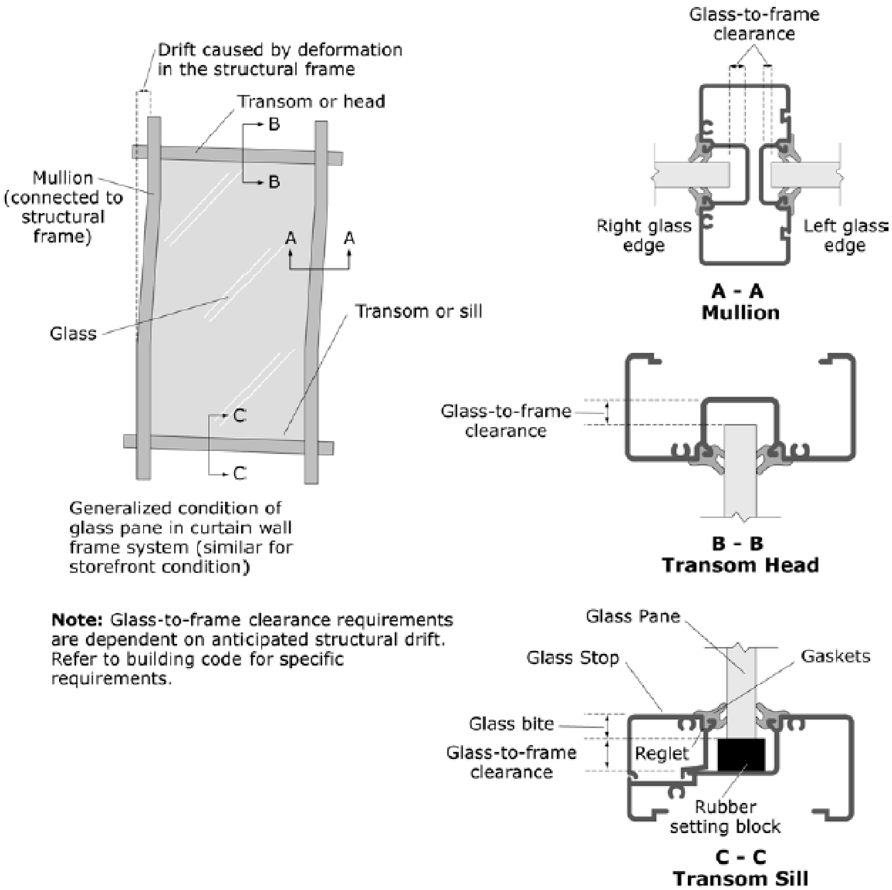


Figure 121 Glazed exterior wall system (FEMA 74, 2011)

Figure 122 illustrates a typical dry-glazed curtain wall system that could resist glass fallout effectively during seismic conditions. It contains laminated glass units with glass-to-aluminum clearances approaching 25 mm. The gaskets can also be replaced with a silicone glazing bead to minimize fallout potential. Actual clearances are determined from dynamic analysis of a specific building being designed, including calculated magnitudes of interstory drifts expected in the primary structural frame of the building under moderate and severe seismic conditions [55].

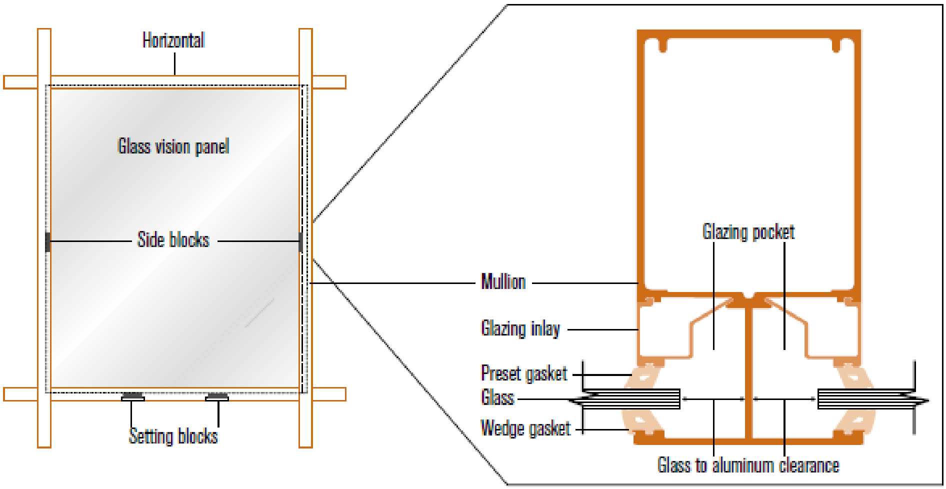


Figure 122 Dry-glazed System (SAFLEX, 2007)

Figures 123 and 124 illustrate a two-sided structural silicone glazing detail for both single layer and insulating glass systems. Test results indicate that glass fallout can also be minimized by the use of laminated glass units and structural silicone glazing [55].

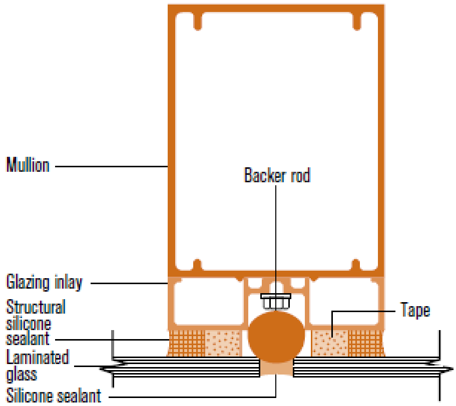


Figure 123 Typical Structural Silicone Glazing System with Laminated Glass (SAFLEX, 2007)

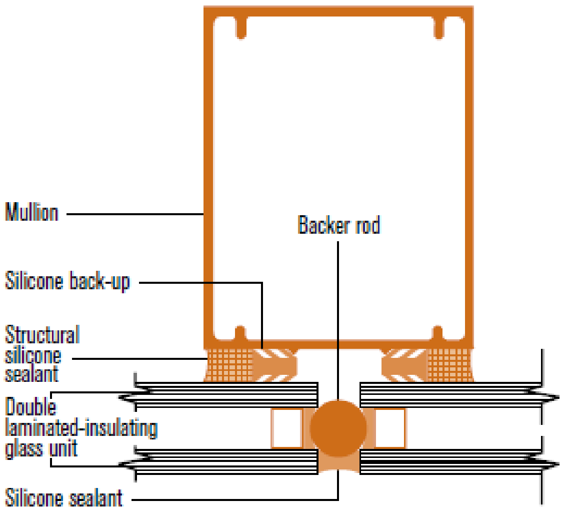


Figure 124 Typical Structural Silicone Glazing System with Double Laminated-Insulating Glass (SAFLEX, 2007)

10.6.3 Sealants and Structural Silicone

The ready availability of sealants such as silicones, polysulphides, urethanes and others, has led to their increasing use in glazing systems; either alone or in conjunction with preformed gaskets, tapes, etc.

The chemistry and mechanical properties of sealants is a complex subject. Manufacturer and specialist literature should be consulted. Silicones that are commonly used in glazing systems have a range of physical properties. Only some are suitable as structural silicone, i.e. to fasten glass or other materials to the framing system, as the primary means of supporting and restraining the glass. High modulus silicones are commonly used for this purpose. They are not recommended for weather sealing the non-structural joint between adjacent units, or panes of glass because of their relatively limited movement capability.

The application of silicone, especially for structural purposes, needs to be carried out under carefully controlled conditions. Factory glazing is much preferred to site glazing. Unitised window systems lend themselves to factory glazing.

10.6.4 Point Supported Glazing System: spider connections

The intent of this system is to isolate the glass from the primary building structure for in-plane deformations and loads while supporting it vertically and for out of plane loads. *Figure 125* illustrates the behavior of the glass panels with slotted spider connections. Building drift is accommodated proportionally through shear deformations at each horizontal glass joint, reducing the stress on the glass. This reduces the probability of breakage and therefore fallout, in an effort to preserve life safety in major seismic events.

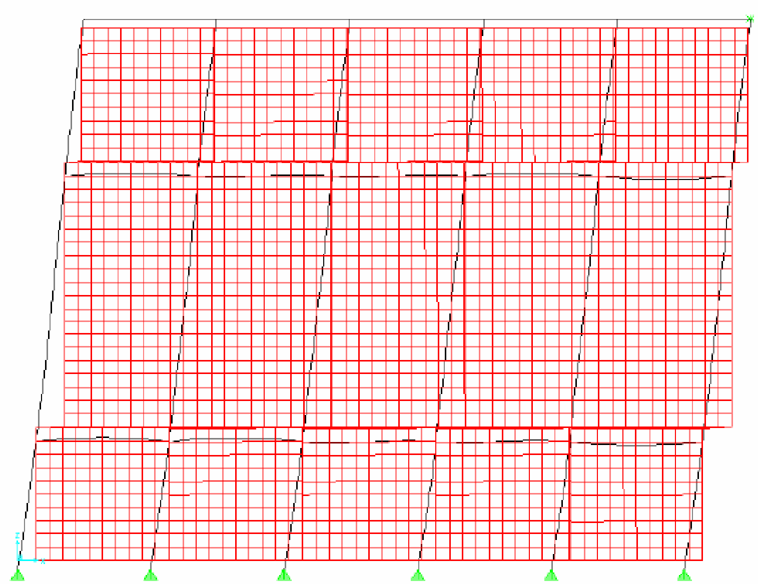


Figure 125 Elevation Illustrating Shearing Action as a Result of Seismic Drift (Prakash Desai et al, 2005)

Glass bolts connecting the glass to the spiders are also designed to minimize the possibility of failure in the glass. Holes are drilled in the glass to accept the bolt assembly with a malleable bushing against the glass to minimize stress concentrations around the holes. A ball and socket at this connection point prevents transmission of bending stresses in the glass *(figure 126).*

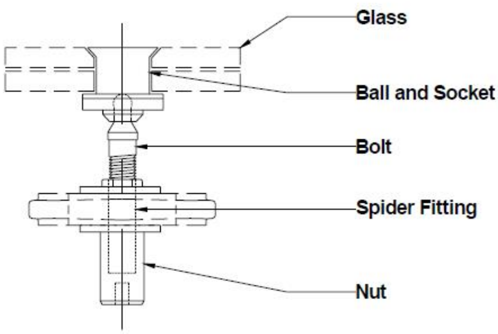


Figure 126 Glass Bolt Assembly Showing Glass and Spider Elements Prakash Desai et al, 2005)

Together, the bolts and spiders act like a pin connection for out-of-plane loads, and a roller connection for in-plane loads. Bushings for the bolts are inserted in the bottom slots of each spider to carry the glass dead load. Omitting the bushings in the top slots eliminates vertical support at that location, allowing the glass to expand due to thermal movements without inducing additional stresses in the glass.

The point supported glass panels are heat treated (heat strengthened or fully tempered) to resist local stresses at the holes, and can be laminated or monolithic based on the type of application. Low modulus silicones are used to create flexible joints that allow the glass panels to use the full travel provided in the spider slots. Perimeter closures must also be detailed to prevent the glass from contacting adjacent building elements or coming out of the joint under inelastic drift.

The framework supporting the spiders, bolts and glass is considered to be the backer support structure for the point supported glazing system. Because of the slotted holes in the spiders, the deformations of the backer structure do not need to coincide with that of the glazing system. Possible backer structures may include simple tube steel frames or more complex cable systems. The basic requirements of the framework are that it distributes drift over the height of the system and that it maintains structural integrity under the design load conditions.

10.7 Test methods

American Architectural Manufacturers Association (AAMA) provides a mock-up test guidelines for evaluating the behaviour of a storefront system subjected to the inter-storey drift and for determining the seismic drift causing glass fallout from a wall system, respectively AAMA 501.4-00 [56] and AAMA 501.6-01 [57] standards.

10.7.1 AAMA 501.4-00

In the first of these two documents is explained and described a "means of evaluating the performance of curtain walls and storefront wall systems when subjected to specified horizontal displacements in the plane of the wall". The method presented is not intended to test for dynamic, torsional or vertical movements. The relative slow, or "static", movements of this test method may not produce the same results as would be obtained in a dynamic movement test.

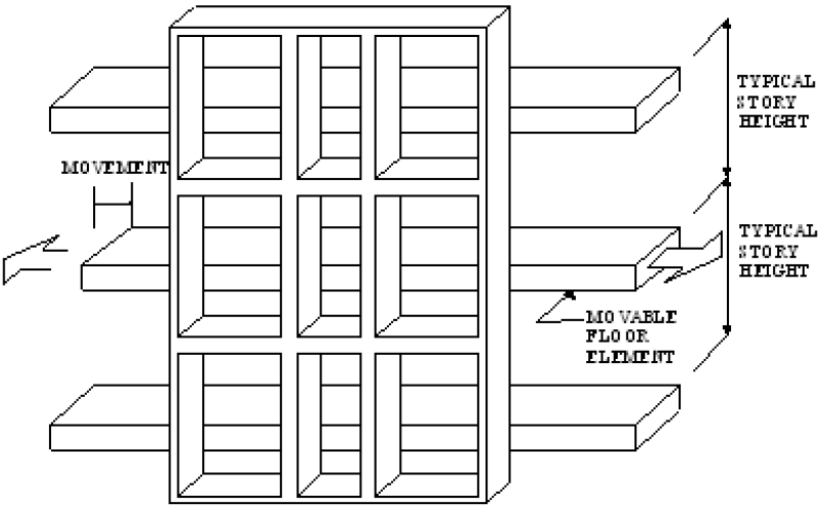


Figure 127 Typical test specimen configuration [56]

The declared scope of this test method is to primarily evaluate changes in serviceability of wall system specimens, for example air and water leakage rates, as a result of statically applied in-plane horizontal racking displacements. Thus, testing is conducted on a full-scale, multi-story mock-up to determine the ability of the curtain wall or storefront to withstand a specified design displacement. This seismic testing phase is preceded by a first complete series of system serviceability tests for air and water infiltration control, and then, after its completion, followed by another additional complete series of air and water leakage tests.

Accordingly it is possible to evaluate the serviceability of the curtain wall or storefront through the pass/fail criteria provided for three different types of facilities: essential, high-occupancy and standard occupancy. These three types of facilities are identified and described also through examples. Furthermore the several requirements for each of them are related with the different Performance Level identified in both the NEHRP Provisions [58] and in FEMA 273 [59].

In contrast is possible to underline that this standard requires the application at least of three cycles of displacements to the specimen, but it doesn't provide any information about how to determine or calculate the different amplitudes of movement to be applied. The "Test Agency", as named in the document, is charged of the determination of these amplitudes. Finally is also not provided any information or minimum requirements about the duration of the test, that again will be determine by the "Test Agency". The sole requirement is that the test must be conducted enough slowly to avoid any acceleration or deceleration.

10.7.2 AAMA 501.6-00

AAMA 501.6 [57] instead has the purpose of describing "a dynamic racking crescendo test for determining $\Delta_{fallout}$", defined as the "in-plane dynamic drift causing glass fallout from a glazed curtain wall panel, a glazed storefront panel or a glazed partition panel". This experimental determination is required by the National Earthquake Hazards Reduction Program (NEHRP) in the specific case that no sufficient clearance has been provided between glass edges and wall frame glazing pockets to prevent contact during seismic design displacement in the main structural system of the building. The following *Figure 128* shows how a dynamic racking test facility is structured.

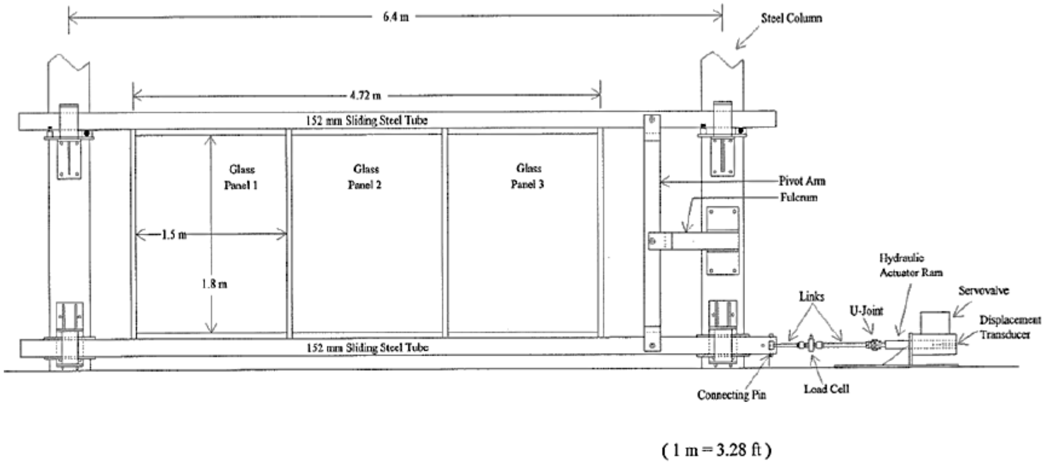


Figure 128 Dynamic racking test facility [57]

The “crescendo test”, named in this standard, consists of a concatenated series of “ramp up” intervals and “constant amplitude” intervals. Ramp up and constant amplitude intervals shall consist of four sinusoidal cycles each. Thus the glazed specimen is moved back and forth horizontally in sinusoidal motions at gradually and progressively higher racking amplitudes, exactly as in a musical crescendo. In the following *Figure 129* the entire drift time history of a crescendo test is reported.

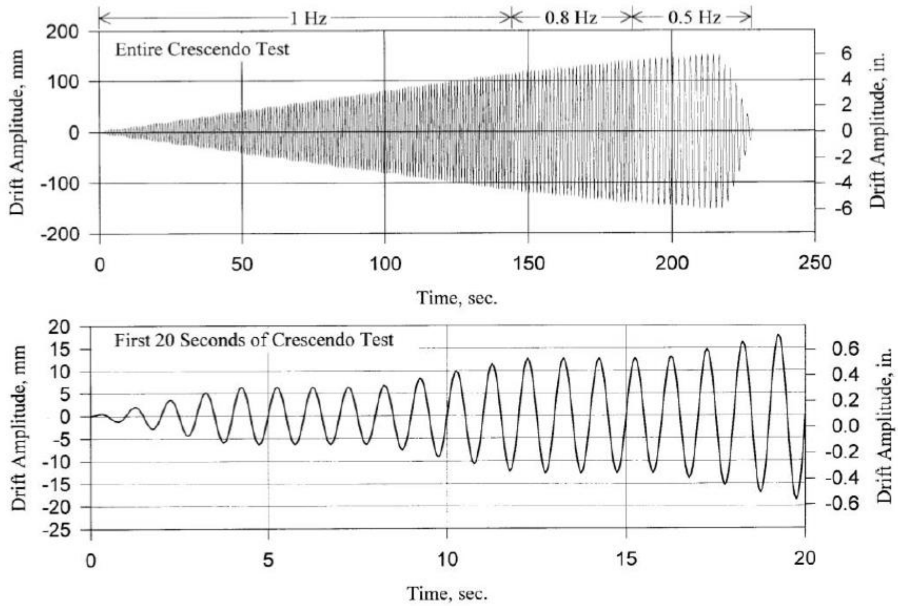


Figure 129 Drift time history in the crescendo test used for mid-rise architectural glass specimens [57]

Precise explanation of how this test has to be conducted is provided by the standard. Finally the lowest value of the racking displacement causing glass fallout for the three specimens that the standard requires to test is the reported value of $\Delta_{fallout}$ for that particular wall system glazing configuration.

10.7.3 Conclusions

In brief we could summarize that the AAMA 501.4-00 deals about the serviceability limit state, determining the performance, in terms of airtight and watertight sealing, of a curtain wall specimen or storefront for a predetermined and specified horizontal in-plane displacement. On the other hand the AAMA 501.6-01 deals about the ultimate limit state, determining the ultimate value of horizontal in-plane displacement next to that the specimen of the façade experiences the glass fallout from unit frame elements.

11. FINITE ELEMENT METHOD

The finite element method (FEM), sometimes referred to as finite element analysis (FEA), is a computational technique used to obtain approximate solutions of boundary value problems in engineering. Simply stated, a boundary value problem is a mathematical problem in which one or more dependent variables must satisfy a differential equation everywhere within a known domain of independent variables and satisfy specific conditions on the boundary of the domain. Boundary value problems are also sometimes called field problems. The field is the domain of interest and most often represents a physical structure. The field variables are the dependent variables of interest governed by the differential equation. The boundary conditions are the specified values of the field variables (or related variables such as derivatives) on the boundaries of the field. Depending on the type of physical problem being analyzed, the field variables may include physical displacement, temperature, heat flux, and fluid velocity to name only a few.

11.1 How does the finite element method work?

The general techniques and terminology of finite element analysis will be introduced with reference to *Figure 130.* The figure depicts a volume of some material or materials having known physical properties. The volume represents the domain of a boundary value problem to be solved. For simplicity, at this point, we assume a two-dimensional case with a single field variable $\Phi(x, y)$ to be determined at every point $P(x, y)$ such that a known governing equation (or equations) is satisfied exactly at every such point. Note that this implies an exact mathematical solution is obtained; that is, the solution is a closed-form algebraic expression of the independent variables. In practical problems, the domain may be geometrically complex as is, often, the governing equation and the likelihood of obtaining an exact closed-form solution is very low. Therefore, approximate solutions based on numerical techniques and digital computation is most often obtained in engineering analyses of complex problems. Finite element analysis is a powerful technique for obtaining such approximate solutions with good accuracy.

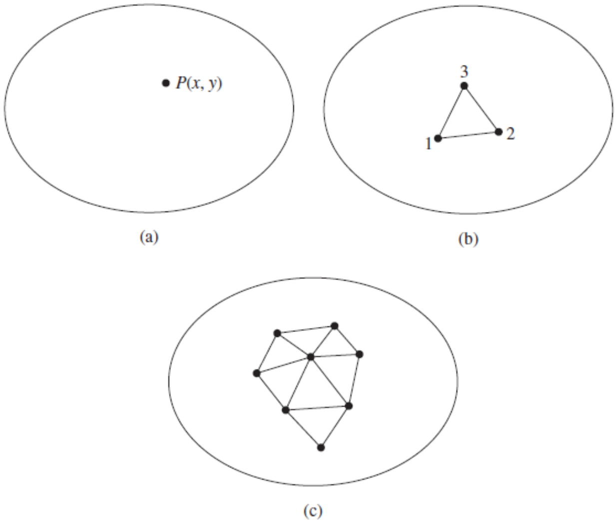


Figure 130 a) A general two-dimensional domain of field variable Φ(x, y). b) A three-node finite element defined in the domain. c) Additional elements showing a partial finite elements mesh of the domain.

A small triangular element that encloses a finite-sized sub-domain of the area of interest is shown in *Figure 130 b).* That this element is not a differential element of size dx x dy makes this a finite element. As we treat this example as a two dimensional problem, it is assumed that the thickness in the z direction is constant and z dependency is not indicated in the differential equation. The vertices of the triangular element are numbered to indicate that these points are nodes. A node is a specific point in the finite element at which the value of the field variable is to be explicitly calculated. Exterior nodes are located on the boundaries of the finite element and may be used to connect an element to adjacent finite elements. Nodes that do not lie on element boundaries are interior nodes and cannot be connected to any other element. The triangular element of *Figure 130 b)* has only exterior nodes.

If the values of the field variable are computed only at nodes, how are values obtained at other points within a finite element? The answer contains the crux of the finite element method: The values of the field variable computed at the nodes are used to approximate the values at non nodal points (that is, in the element interior) by *interpolation* of the nodal values. For the three-node triangle example, the nodes are all exterior and, at any other point within the element, the field variable is described by the approximate relation:

$$\emptyset(x,y) = N1(x,y)\emptyset 1 + N2(x,y)\emptyset 2 + N3(x,y)\emptyset 3$$

Where Φ1, Φ2 and Φ3 are the values of the field variable at the nodes, and N1, N2, and N3 are the interpolation functions, also known as shape functions or blending functions. In the finite element approach, the nodal values of the field variable are treated as unknown constants that are to be determined. The interpolation functions are most often polynomial forms of the independent variables, derived to satisfy certain required conditions at the nodes. The major point to be made here is that the interpolation functions are predetermined, known functions of the independent variables; and these functions describe the variation of the field variable within the finite element.

The triangular element described by the equation is said to have 3 degrees of freedom, as three nodal values of the field variable are required to describe the field variable everywhere in the element. This would be the case if the field variable represents a scalar field, such as temperature in a heat transfer problem. If the domain of *Figure 130* represents a thin, solid body subjected to plane stress, the field variable becomes the displacement vector and the values of two components must be computed at each node. In the latter case, the three-node triangular element has 6 degrees of freedom. In general, the number of degrees of freedom associated with a finite element is equal to the product of the number of nodes and the number of values of the field variable (and possibly its derivatives) that must be computed at each node.

How does this element-based approach work over the entire domain of interest? As depicted in *Figure 130 c),* every element is connected at its exterior nodes to other elements. The finite element equations are formulated such that, at the nodal connections, the value of the field variable at any connection is the same for each element connected to the node. Thus, continuity of the field variable at the nodes is ensured. In fact, finite element formulations are such that continuity of the field variable across inter-element boundaries is also ensured. This feature avoids the physically unacceptable possibility of gaps or voids occurring in the domain. In structural problems, such gaps would represent physical separation of the material. In heat transfer, a “gap” would manifest itself in the form of different temperatures at the same physical point.

Although continuity of the field variable from element to element is inherent to the finite element formulation, inter-element continuity of gradients (i.e., derivatives) of the field variable does not generally exist. This is a critical observation. In most cases, such derivatives are of more interest than are field variable values. For example, in structural problems, the field variable is displacement but the true interest is more often in strain and stress. As *strain* is defined in terms of first derivatives of displacement components, strain is not continuous across element boundaries. However, the magnitudes of discontinuities of derivatives can be used to assess solution accuracy and convergence as the number of elements is increased.

11.2 Comparison of finite element and exact solutions

The process of representing a physical domain with finite elements is referred to as *meshing,* and the resulting set of elements is known as the finite element *mesh*. As most of the commonly used element geometries have straight sides, it is generally impossible to include the entire physical domain in the element mesh if the domain includes curved boundaries. Such a situation is shown in *Figure 131 a),* where a curved-boundary domain is meshed (quite coarsely) using square elements. A refined mesh for the same domain is shown in *Figure 131 b),* using smaller, more numerous elements of the same type. Note that the refined mesh includes significantly more of the physical domain in the finite element representation and the curved boundaries are more closely approximated. (Triangular elements could approximate the boundaries even better.)

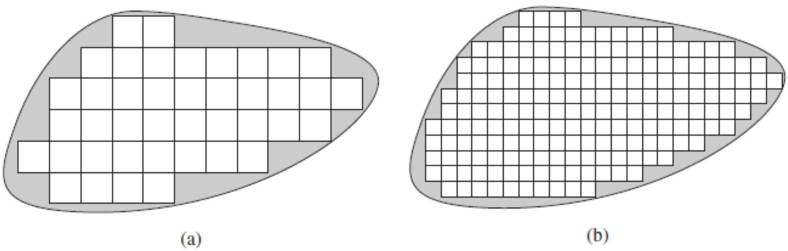


Figure 131 a) Arbitrary curved-boundary domain modeled using square elements. Stippled areas are not included in the model. A total of 41 elements is shown. b) Refined finite element mesh showing reduction of the area not included in the model. A total of 192 elements is shown.

If the interpolation functions satisfy certain mathematical requirements a finite element solution for a particular problem converges to the exact solution of the problem. That is, as the number of elements is increased and the physical dimensions of the elements are decreased, the finite element solution changes incrementally. The incremental changes decrease with the mesh refinement process and approach the exact solution asymptotically. To illustrate convergence, we consider a relatively simple problem that has a known solution. *Figure 132 a)* depicts a tapered, solid cylinder fixed at one end and subjected to a tensile load at the other end. Assuming the displacement at the point of load application to be of interest, a first approximation is obtained by considering the cylinder to be uniform, having a cross-sectional area equal to the average area of the cylinder, *Figure 132 b).* The uniform bar is a *link* or *bar* finite element, so our first approximation is a one-element, finite element model. The solution is obtained using the strength of materials theory. Next, we model the tapered cylinder as two uniform bars in series, as in *Figure 132 c).* In the two element model, each element is of length equal to half the total length of the cylinder and has a cross-sectional area equal to the average area of the corresponding half-length of the cylinder. The mesh refinement is continued using a four-element model, as in *Figure 132 d),* and so on. For this simple problem, the displacement of the end of the cylinder for each of the finite element models is as shown in *Figure 133 a),* where the dashed line represents the known solution. Convergence of the finite element solutions to the exact solution is clearly indicated.

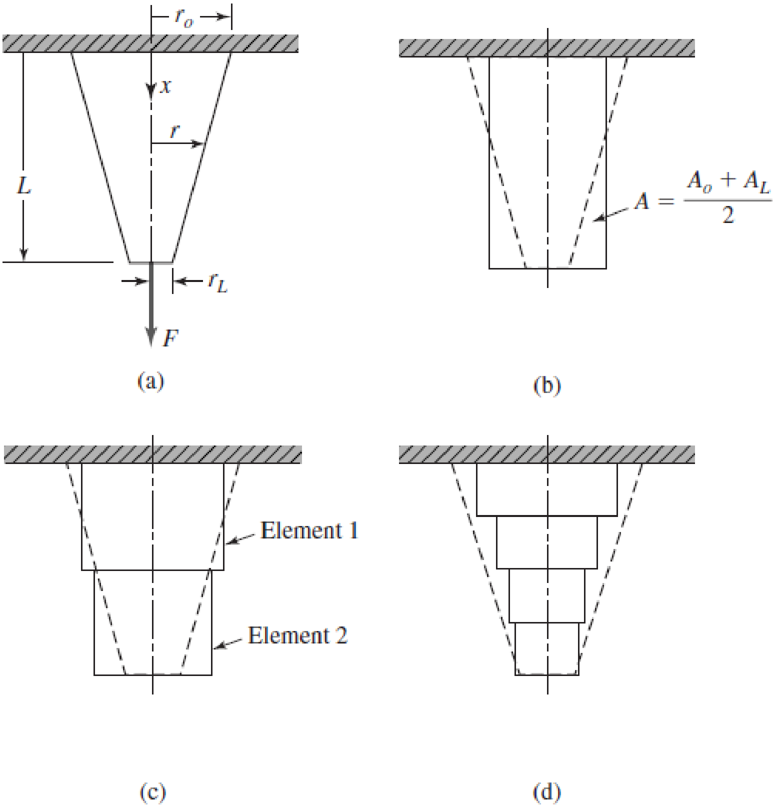


Figure 132 a) Tapered circular cylinder subjected to tensile loading. b) Tapered cylinder as a single axial (bar) element using an average area. c) Tapered cylinder modeled as two, equal-length, finite elements. The area of each element is average over the respective tapered cylinder length. d) Tapered circular cylinder modeled as four, equal-length finite elements.

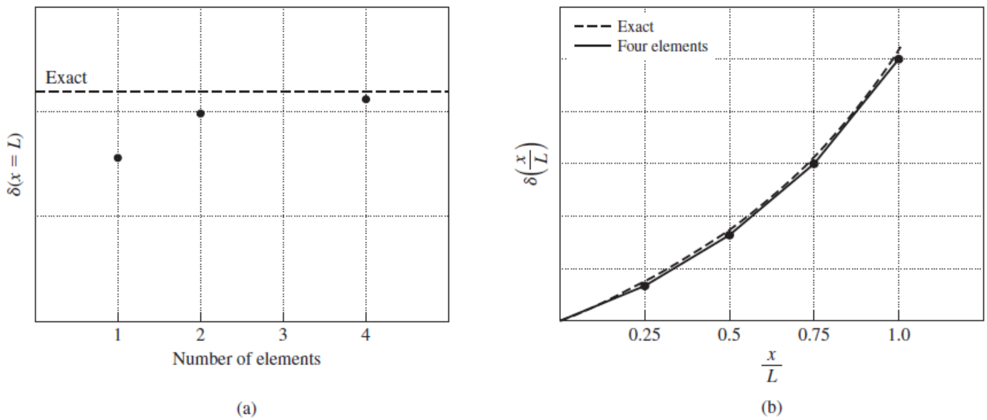


Figure 133 a) Displacement at x = L for tapered cylinder in tension of Figure 3. b) Comparison of the exact solution and the four-element solution for tapered cylinder in tension.

On the other hand, if we plot displacement as a function of position along the length of the cylinder, we can observe convergence as well as the approximate nature of the finite element solutions. *Figure 133 b)* depicts the exact strength of materials solution and the displacement solution for the four-element models. We note that the displacement variation in each element is a linear approximation to the true nonlinear solution. The linear variation is directly attributable to the fact that the interpolation functions for a bar element are linear. Second, we note that, as the mesh is refined, the displacement solution converges to the nonlinear solution at *every point* in the solution domain.

Our example shows how the finite element solution converges to a *known* exact solution (the exactness of the solution in this case is that of strength of materials theory). If we know the exact solution, we would not be applying the finite element method. So how do we assess the accuracy of a finite element solution for a problem with an unknown solution? The answer to this question is not simple. If we did not have the dashed line in *Figure 132* representing the exact solution, we could still discern convergence to *a* solution. Convergence of a numerical method (such as the finite element method) is by no means assurance that the convergence is to the correct solution. A person using the finite element analysis technique must examine the solution analytically in terms of:

1. Numerical convergence,
2. Reasonableness (does the result make sense?),
3. Whether the physical laws of the problem are satisfied (is the structure in equilibrium? Does the heat output balance with the heat input?),
4. Whether the discontinuities in value of derived variables across element boundaries are reasonable.

Many such questions must be posed and examined prior to accepting the results of a finite element analysis as representative of a correct solution useful for design purposes.

12. SIMULATION OF BRICK MASONRY WALL

12.1 Masonry elements in buildings

The masonry walls are not only used for structures, also for subdivisions of spaces, fire protection, thermal and acoustic insulation and esthetic appearance.

The strong earthquakes that have occurred to date have shown in most cases, the unreinforced masonry structures have been the most affected and have produced substantial loss of life, compared with other structural systems. The failure mode of these structures has shown a behavior with little ductility, because the collapse comes suddenly. In most cases, this type of failure has been associated with deficiencies characteristic of this type of construction, such as: bad connections, flexible floor diaphragms and mortars too sleazy, among others. Moreover, with less intense earthquakes, the structures have been affected only slightly without showing considerable damage. This could indicate that, for areas with low to moderate seismic hazard, using an adequate technique of reinforcement, securing a good performance of these structures, and therefore, reduce the risk to which they are exposed.

These walls are usually considered unsuitable in areas with high seismic hazard and, moreover, their seismic performance is quite variable, that is, in some cases has been catastrophic and others have performed excellently. The reason for this variability is explained from the stress-strain relationship: the masonry is initially rigid, showing little effect of load, but once it reaches the peak deformation, often fails so fragile. Considering that there is no type of reinforcement, the maximum stress redistribution is not possible and the local failure quickly becomes a global failure of the structure. Thus, the system loses strength and may collapse. However, when the response of the structure to an earthquake, falls within the first interval, that is, within the elastic range, its performance is quite good.

12.2 Mechanical properties of a masonry wall

Masonry wall is a complex material, not uniform, with different characteristics depending on their composition and the direction in which we want to analyze it and do not get the rules of elastic materials.

It is therefore necessary to get some values that allow us take rigidity and deformation parameters to be analyzed by ways of mathematical algorithms.

Its resistance varies between 15 and 30 MPa in compression brick and the low tensile 5 to 8% of this value, a tensile modulus between 5000 and 10000 MPa and a Poisson's ratio between 0.15 and 0.20. For the rock strength is between 30 and 100 MPa for 20 and sandstone and limestone to 200 MPa and 60 to 180 for granites. For bonding mortars with the proportions 1:3 to 1:5 (lime:sand) is about 2.5 MPa, and its tensile strength at 5%.

But there is little point to know the characteristics of the materials themselves if not consider them as part of the wall and analyze it together.

The resistance of the plant depends on the size of the pieces, the thickness of the boards and that the thickness of the bed joint resistance decreases tangential stresses. Moreover, the resistance also depends on the direction of the loads.

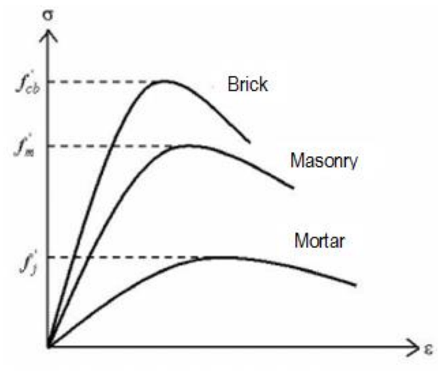


Figure 134 Stress-strain relationship for isolated units, mortar and masonry (Paulay and Priestley, 1992).

In general we cannot assume any given value even before we have to approximate data and obtain for each value by testing. For example [60]:

Properties	Values
Ex (MPa)	2460
Ey (MPa)	5460
Vxy	0,18
Gxy	1130

Table 6 Elastic properties of the composite masonry [60]

In other studies, based on the technical specifications to use, construction features, materials used, pathological studies made in some buildings, studies by other researchers [61] and, with the help of experienced engineers view on knowledge of the mechanical behavior of brick masonry unassembled, there has been a significant amount of numerical examples, which has helped define the mechanical parameters requiring definition of the macro elements and such data are obtained:

Properties	Values
E (MPa)	1800
Shear modulus (MPa)	300
Shear strength (MPa)	0,12
compressive strength (MPa)	4
specific gravity (kg/m3)	1800

Table 7 Mechanical properties of masonry walls [61]

These properties vary depending on the individual parameters of the masonry and the connection between them.

12.3 Analysis unidirectional

To get an understanding of the behavior of a masonry wall must be considered as an elastic material, consider the fact that the materials are deformed when they get a load, thus being able to use the concepts of stress and strain. The linear elastic study reveals possible states of balance by the distribution maps of maximum stresses of tension and compression. We can observe the working mechanism of the factory without cracking, giving an overview of areas of stress concentration.

To make this analysis we will be guided in real data taken from a seismic simulation of a particular masonry wall [62]. Simulate a wall of dimensions: 2000 mm long, 2000 mm high and 250 mm thick. Also include two vertical loads applied before the seismic lateral force of 160 kN (30.45 MPa) corresponding to axial loads that efforts would be met by a masonry wall on the ground floor with five upper floors.

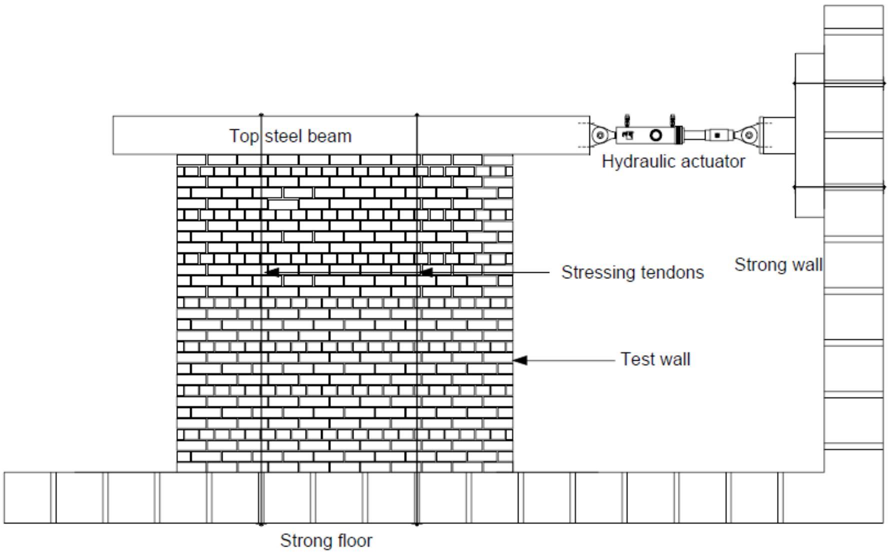
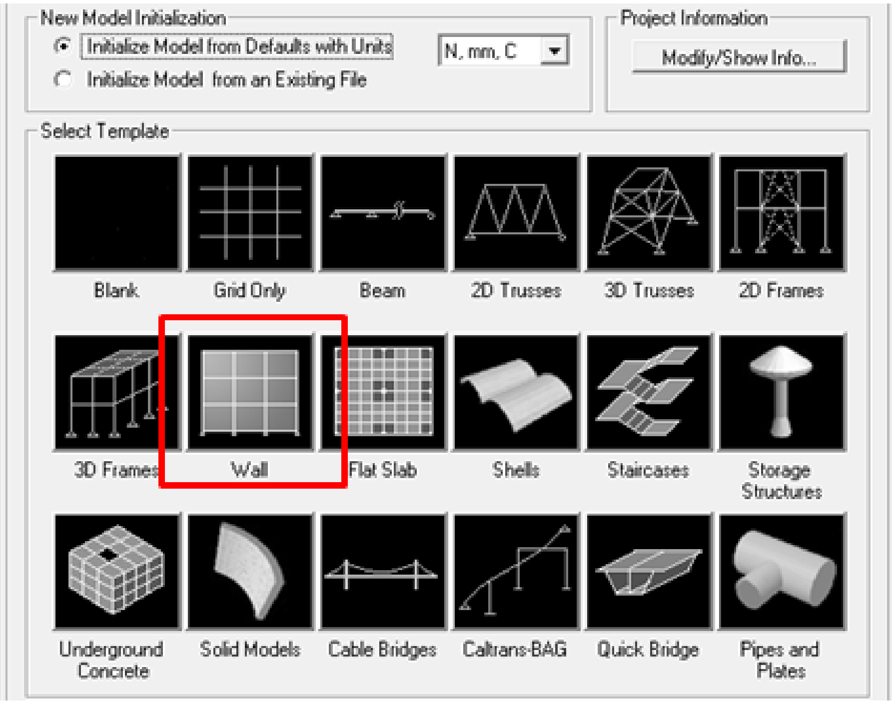
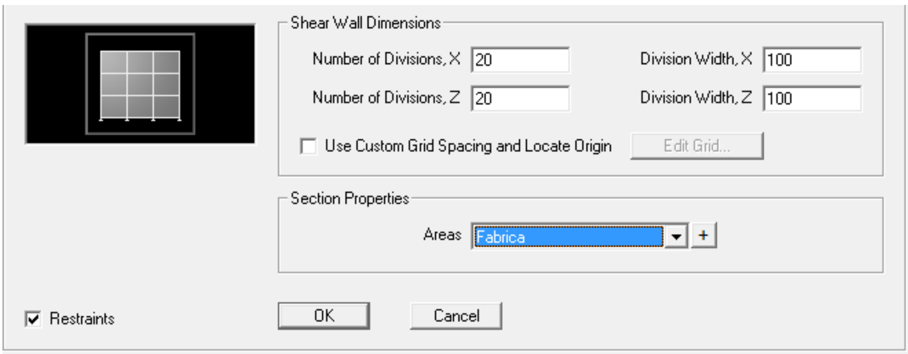


Figure 135 Test setup

For this analysis we base on the element software ***SAP2000*** using the finite element method (discussed in Chapter 11) and define:

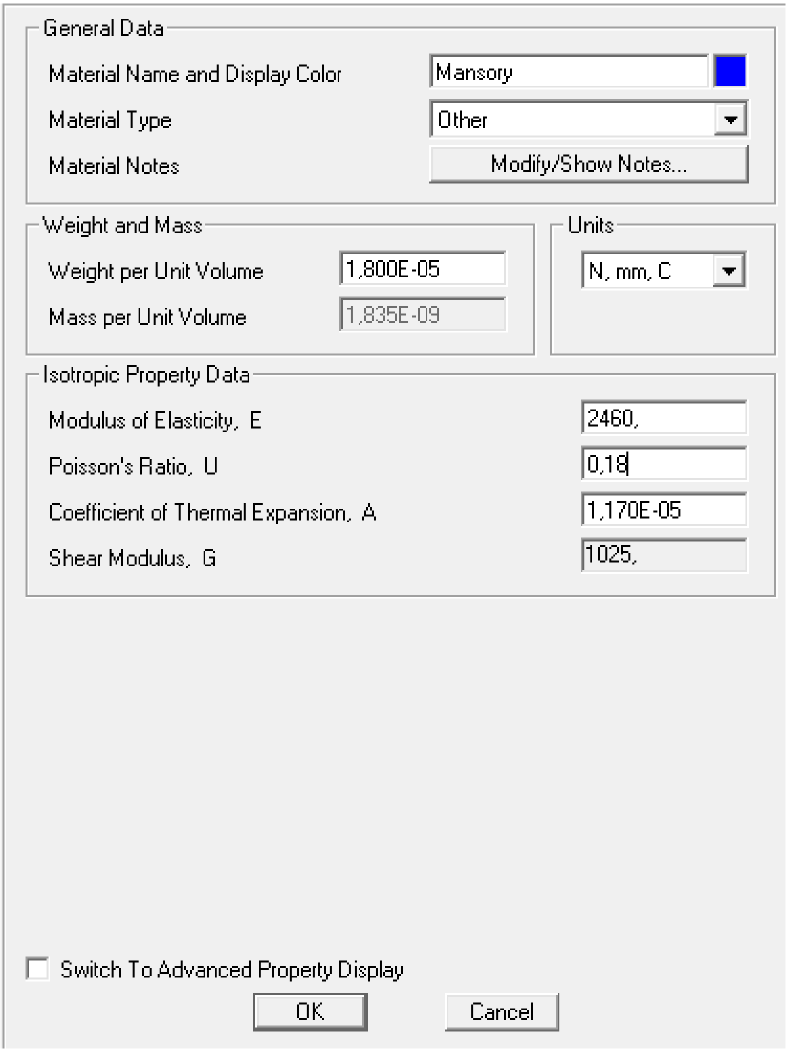
12.3.1 Geometry of the facade



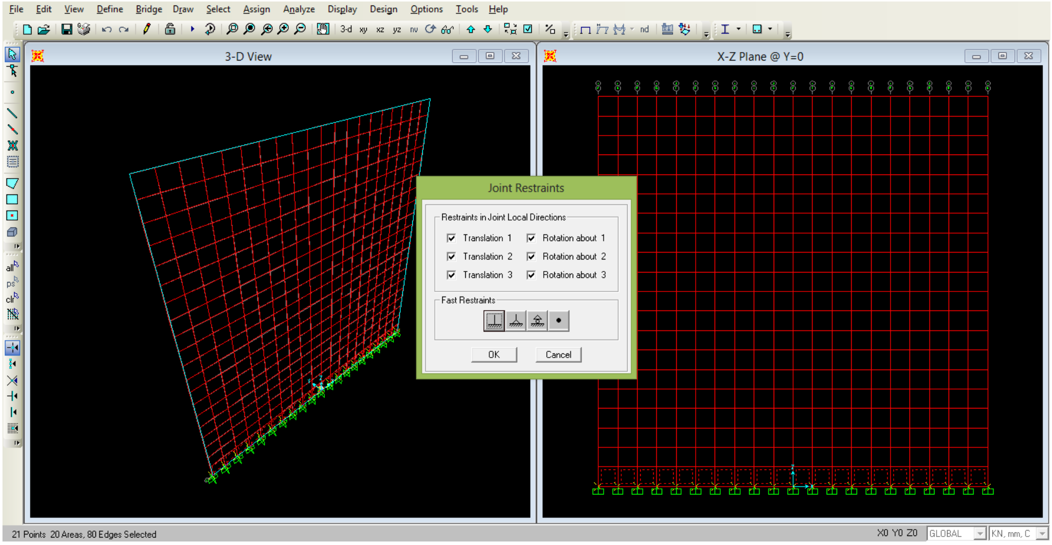


We see that for finite element analysis take 400 points.

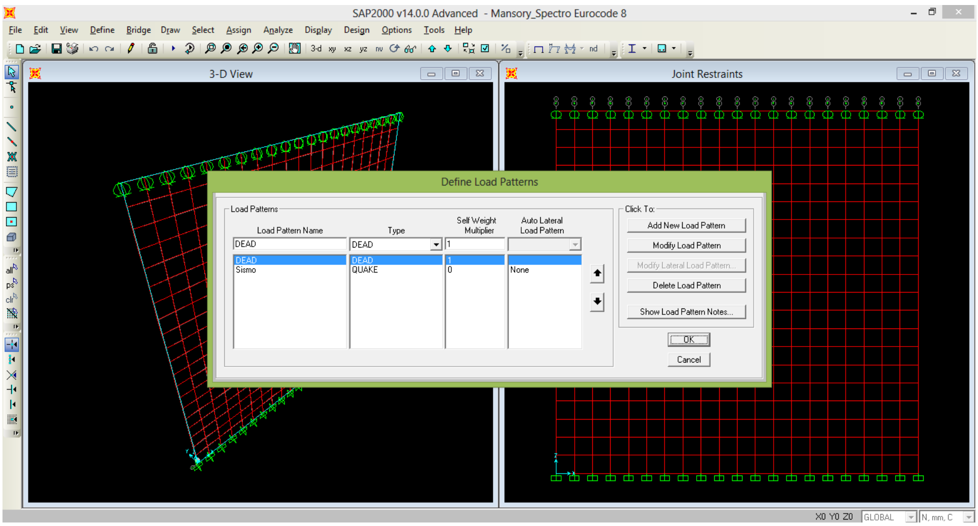
12.3.2 Material properties



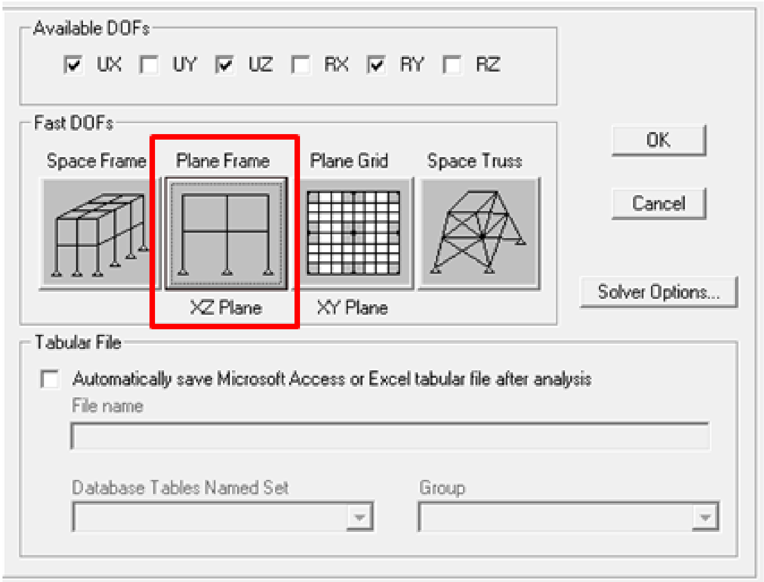
12.3.3 Definition of support conditions

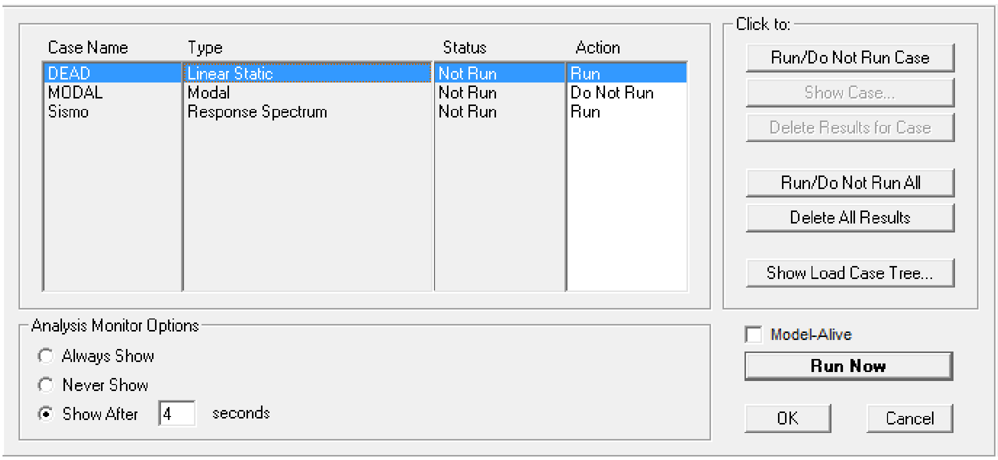


12.3.4 Definition of load patterns



12.3.5 Unidirectional analysis





12.4 Interpretation of results

In the actual test shows that the first break appears to a displacement of 2 mm equivalent to a horizontal force of 74 kN and rupture appears next to a displacement of 5 mm from the wall which completely lose their ductility and breaks [62].

To fit the model to actual results we change the wall material properties defined above and estimate a suitable elastic modulus. Since, with a modulus of 2460 MPa lower displacements get help not draw real conclusions (0.89 mm).

Thus we see that with a modulus of elasticity of 1000 MPa and obtain a displacement of 2.33 mm and can analyze the efforts that are presented on the wall.

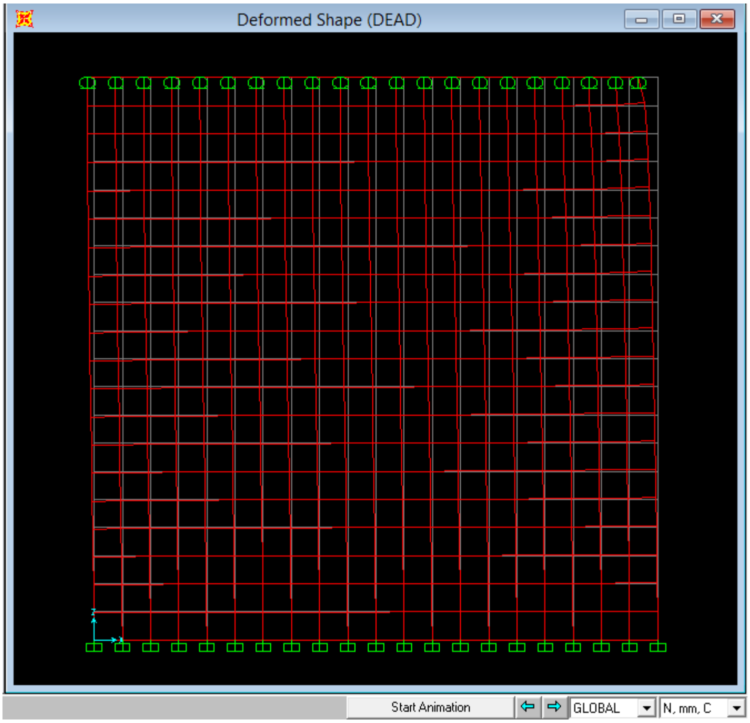


Figure 136 Deformed shape

To interpret the results we need to define a coordinate system. The six faces of a shell element are defined as the positive 1 face, negative 1 face, positive 2 face, negative 2 face, positive 3 face and negative 3 face as shown in the figure below. In this definition the numbers 1, 2 and 3 correspond to the local axes of the shell element. The positive 1 face of the element is the face that is perpendicular to the 1-axis of the element whose outward normal (pointing away from the element) is in the positive 1-axis direction. The negative 1 face of the element is a face that is perpendicular to the 1-axis of the element whose outward normal (pointing away from the element) is in the negative 1-axis direction. The other faces have similar definitions.

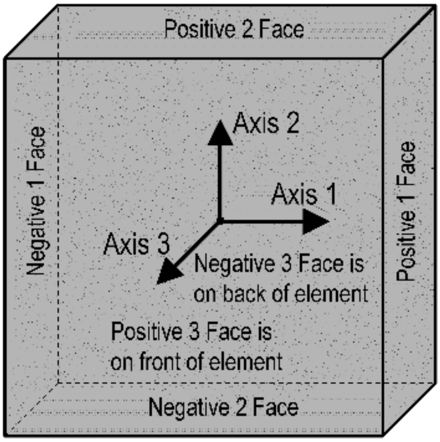


Figure 137 Shell element. Stresses Output Convention

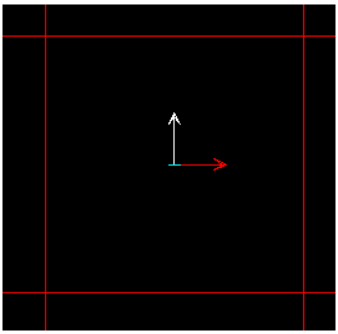


Figure 138 Local Axes. Red: local axis 1 and white: local axis 2

So we can say:

- S11: Force per unit area (in the face of the element) on the axis of the element 1
- S22: Force per unit area on the axis of the element 2
- S12: Shear stress

The level of response will be static linear displacement, mean stresses resulting from elastic behavior of the whole.

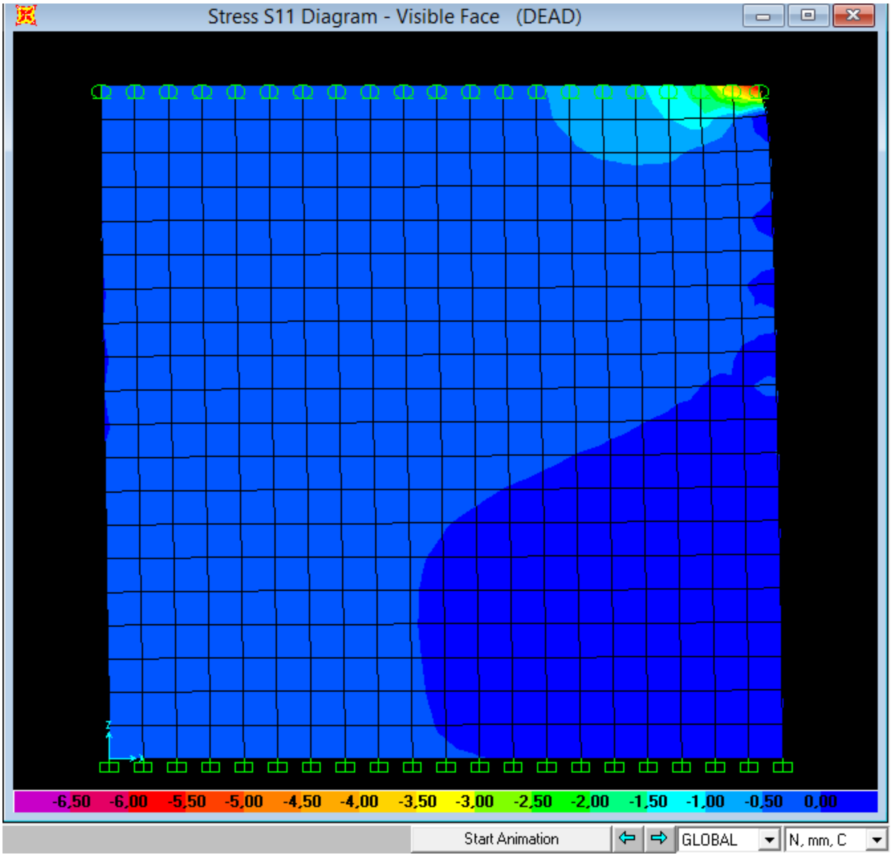


Figure 139 Stress S11 Diagram

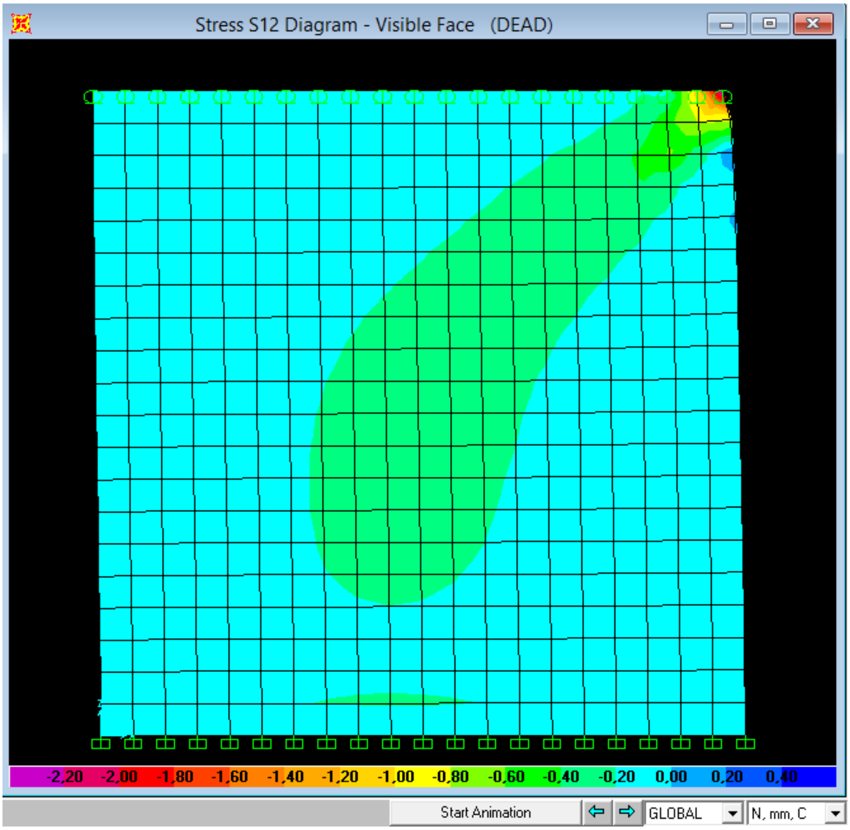


Figure 140 Stress S12 diagram

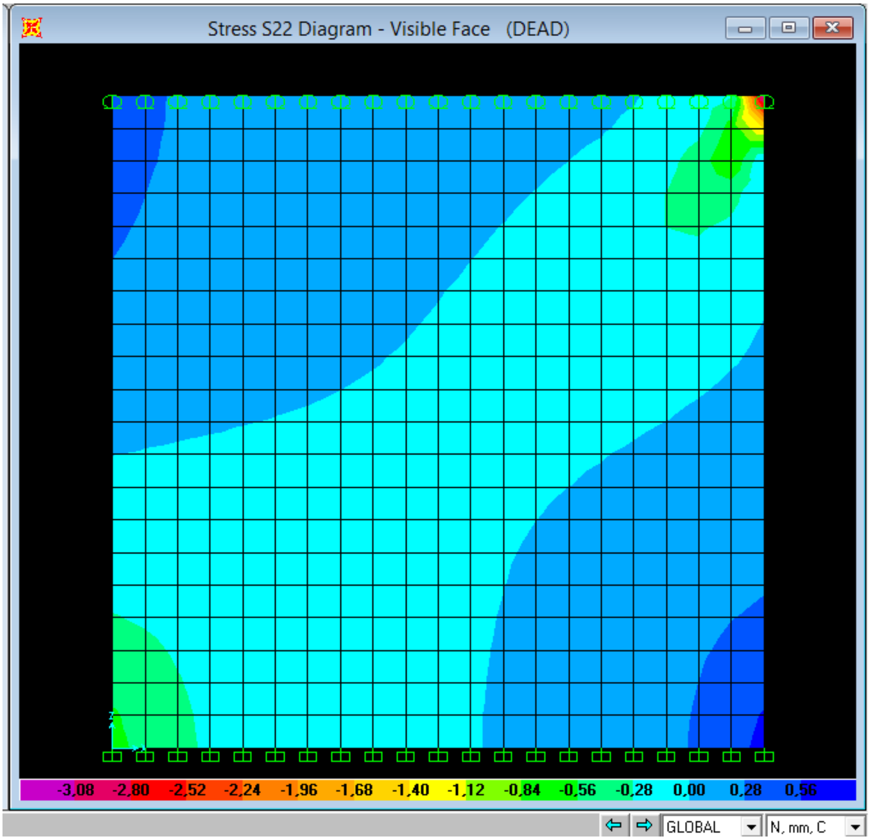


Figure 141 Stress S22 diagram

As seen in the x direction the focus is on the corner on the right with a compression of about 6 MPa. In axis 2 can be seen better than efforts concentrate on compression in the upper right and lower left him with average values of 0.8 MPa and tractions in opposite corners with values of 0.28 MPa. Therefore, in the third graph we can appreciate the shear that will occur in the wall with values of about 0.7 MPa, is in this area that appears typical earthquake break of the wall, and if we apply a cyclic load we have the typical break in "X".

Figure 142 Typical break in a wall due to earthquake

Compare the results with Spanish regulations and according to the CTE DB-SE-F have the masonry wall must have a compressive strength according to table:

Tabla 4.4 Resistencia característica a la compresión de fábricas usuales f_k (N/mm^2)

Resistencia normalizada de las piezas, f_b (N/mm^2)	**5**		**10**		**15**		**20**		**25**
Resistencia del mortero, f_m (N/mm^2)	2,5	3,5	5	7,5	7,5	10	10	15	15
Ladrillo macizo con junta delgada	-	-	3	3	3	3	3	3	3
Ladrillo macizo	2	2	4	4	6	6	8	8	10
Ladrillo perforado	2	2	4	4	5	6	7	8	9
Bloques aligerados	2	2	3	4	5	5	6	7	8
Bloques huecos	1	1	2	3	4	4	5	6	6

Table 8 Compressive strength according CTE DB SE-F

The test used a solid brick and mortar mix ASTM type “O” [62]. Taking an average value for the brick resistance 15 MPa and mortar strength 7.5 MPa, masonry wall must have strength of 6 MPa which is coherent with our simulation and can give as reliable our stress diagrams.

13. CONCLUSIONS AND FUTURE

The first objective of this project was to obtain the title of "Ingeniero de Edificación" and with this job I want to prove that I have the necessary competencies. Made a "state of art" of nonstructural elements earthquake and focusing on a particular element, such as the facades of masonry can be concluded:

The masonry is defined as the set of two phases of materials: units and mortar, to manually combine a regular or irregular. There are different types of units and mortars, which have a wide range of mechanical properties and geometrical. Masonry, presents a great variation in their characteristics and different behavior to the elements that constitute it.

Originally, masonry is a material designed to resist vertical loads, one of the principal factors must be considered in the design is the compressive strength. However, these structures are affected by other actions, such as wind loads and earthquakes, which translate into horizontal forces. This requires considering the shear strength and tensile strength of the masonry.

The brick masonry is a material having an optimum performance when subjected to compressive stresses. Their behavior and failure modes with axial loads, dependent interaction of parts and mortar: bricks and mortar have different stress-strain characteristics; therefore, when subjected to the same stress, an interaction occurs between them: less deformable material (brick) restricts the transverse deformations of deformable material (mortar) introducing compressive stresses in transverse direction. By contrast, in less deformable material transverse stresses are introduced in traction lowering its resistance compared to that obtained in the single compression test of the isolated material. Essentially there are three failure modes:

1) Shear failure along mortar joints for low compression forces, with cracks that are distributed in stages and through planes of weakness that make the unit-mortar interface.
2) Diagonal tensile failure for moderate compressive stresses, where cracks are distributed in an "X" crossing the units.

3) Compressive failure to high compressive stresses, with cracks propagating vertically traversing masonry units.

The shear strength of the masonry depends:

a) Bond strength
b) The frictional resistance between the mortar joints and units.
c) The level of compressive stress.

The behavior of the masonry tensile strength is controlled by the adherence. The tensile failure modes depend on the direction of load:

1) Tensile stress parallel to the horizontal joints, cracks propagate crossing the units vertically or staggered along the mortar joints.
2) Tensile stresses in the direction of the vertical joints, cracks are distributed horizontally crossing the boards or units.

Masonry structures can be simultaneously subjected to compressive loads and lateral loads, therefore, are generated and shear flexo-compression. The two most common types of failure that occur under these stress conditions are: shear failure and flexural failure. The first of them, the one that has been observed during surveys of damage after an earthquake occurred. The fault type is closely related to the aspect ratio (length / height) of masonry panels. So, usually to higher aspect ratios are shear failures and for flexural failure occurs with lower values.

Ultimately, the dynamic response of the unreinforced masonry structures depends on the strength, stiffness and ductility of the panels, such as the floor diaphragm, fittings and the magnitude of the vertical load.

As future work, we can make a real simulation and compare the results with those obtained by software simulation and adjust the model as possible. You can also perform a bidirectional analysis and make a more reliable performance of an earthquake. Moreover, one can study the behavior of a masonry wall with different compositions of mortars and units and see their behavior, ductility and response to an earthquake. Finally we can focus on other structural or other facade systems and study their behavior for different seismic loads, studying its stiffness, ductility, etc.

REFERENCES

[1] Hoedajanto, D. and MacRae, G., "Use of Pushover Analysis in Performance Based Design of Multistory Buildings", EASEC-9, Bali, Indonesia, 2003.

[2] Arnold, C. and Reitherman, R. (1982). *Building configuration and seismic design*, John Wiley & Sons, New York, 269 pp.

[3] Sezen, H.; Whittaker, A.S.; Elwood, K.J.; Mosalam, K.M. (2003). Performance of reinforced concre-te buildings during the August 17, 1999 Kocaeli, Turkey earthquake, and seismic design and cons-truction practise in Turkey. Engineering Structures (2003) 103-114.

[4] Elenas, A. (2003). Athens Earthquake of 7 September 1999: Intensity Measures and Observed Damages. ISET Journal of Earthquake Technology. Technical Note, Vol. 40, No. 1, marzo.

[5] Wibowo, A.; Kafle, B.; Mohyeddin, A.; wt al. Dama-ge in the 2008 China Earthquake. http://www. aees.org.au/Proceedings/2008_Papers/48_Wi-bowo.pdf

[6] "Dos terremotos sacuden Lorca y causan ocho muertos" ELPAÍS.com. 12 May 2011.

[7] "Spain Shocked by Deadly Earthquake". The Guardian. 12 May 2011.

[8] "El seísmo de Lorca, el que ha causado más víctimas desde 1956". ABC. 18 May 2011.

[9] FEMA, 1981.

[10] TAPAN, K.S., Fundamentals of seismic loadings on structures, John Wiley & Sons Ltd, 2009.

[11] U.S. Geological Survey, Documentation for the Southeast Asia Seismic Hazard Maps, Virginia 2007.

[12] Earthquake Research Committee, National Seismic Hazard Maps for Japan, 2005.

[13] HAYES, G., WALD, D., The 03/11/2011 Mw9.0 Tohoku, Japan earthquake, 2011, U.S. geological survey, National Information Center.

[14] Radio NZ news report on 2007. Gisborne earthquake.

[15] Earthquakes: The Short Story, "Te Ara Encyclopedia of New Zealand".

[16] Eurocode 8: Design of structures for earthquake resistance.

[17] ASCE, 2010, Minimum Design Loads for Buildings and Other Structures (ASCE/SEI 7-10), American Society of Civil Engineers, Reston, Virginia.

[18] ICC, 2000, 2003, 2006, 2009, 2012, International Building Code, International Code Council, Washington, D.C.

[19] ICBO, 1994, Uniform Building Code, International Conference of Building Officials, Whittier, California

[20] Public Law 95-124, 42 U.S.C. 7701, Eartquake Hazards Reduction Act of 1977, 1977.

[21] FEMA 450, Recommended provisions for seismic regulations for new buildings and other structures, FEMA, 2003.

[22] NZS 1170.5, Structural Design Actions – Part 5, ZEALAND, STANDARDS NEW, Wellington, NZ, 2004.

[23] International Association for Earthquake Engineering "Japan Standards for Aseismic Civil Engineering Construction", Earthqueake Resistant Regulations. A World List, Tokyo, Japan, 1992.

[24] Hirosawa, M.; Mizuno, H.; and Midorikawa, M., "State-of-the-Art Report on Seismic Design of Building Equipment and Nonstructural Component in Japan", Building Research Institute, Ministry of Construction, Japan, May 1991.

[25] LEY 38/1999, de 5 de noviembre, de Ordenación de la Edificación, España, 1999.

[26] Drysdale RG, Hamid AA, Baker LR. Masonry structures—behavior and design. 2nd ed. The Masonry Society; 1999.

[27] Paulay T, Priestley MJN. Seismic design of reinforced concrete and masonry buildings. Wiley; 1992.

[28] Tomazevic M. Earthquake-resistant design of masonry buildings. London: Imperial College Press; 1999.

[29] Dowrick DJ. Earthquake resistant design for engineers and architects. 2nd ed. Wiley; 1987.

[30] Hollings, J.P. et al. (1981). Indonesian Earthquake Study: Vol. 6, Manual for the design of normal reinforced concrete and reinforced masonry structures. Beca Carter Hollings and Ferner Ltd., Wellington (unpublished).

[31] FEMA 310 (1998). NEHRP handbook for the seismic evaluation of existing buildings – a Prestandard; Federal Emergency Management Agency, Washington D.C.

[32] Magenes, G. and Pampanin, S. (2004) 13th World Conference on Earthquake Engineering. Seismic response of gravity-load design frames with masonry infills Vancouver, B.C, Canada.

[33] FEMA 306 (1999). Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Basic Procedures Manual Federal Emergency Management Agency, Washington, D.C.

[34] FEMA 307 (1999). Evaluation of Earthquake Damaged Concrete and Masonry Wall Buildings - Technical Resources. Federal Emergency Management Agency, Washington, D.C.

[35] Bell, D. K. and Davidson, B. J. (2001) Evaluation of earthquake risk buildings with masonry infill panels. 2001 Technical Conference, Future Directions: A Vision for Earthquake Engineering in New Zealand New Zealand Society for Earthquake Engineering Taupo New Zealand.

[36] Valiasis, T. N. and Stylianidis, K. C. (1989) Masonry infilled R/C frames under horizontal loading — experimental results. European Earthquake Engineering 3:3, pp. 10-20.

[37] Freeman, S. A. (1977) Racking tests of high rise building partitions. Journal of Structural Division ASCE 103:ST8.

[38] Memari AM, Aliaari M. Seismic infill wall isolator subframe (SIWIS) system for use in buildings. In: Proceeding, ATC-17-2 seismic on response modification technologies for performance-based design. 2002.

[39] ElGawady MA, Lestuzzi P, Badoux M. Static cyclic response of masonry walls retrofitted with fiber-reinforced polymers. Journal of Composites for Construction, ASCE, 11(2007) 50−61.

[40] Santa-Maria H, Alcaino P. Repair of in-plane shear damaged masonry walls with external FRP. Construction and Building Materials, 25(2011) 1172−80.

[41] Shrive NG. The use of fibre reinforced polymers to improve seismic resistance of masonry. Construction and Building Materials, 20(2006) 269−77.

[42] Ascione L, Feo L, Fraternali F. Load carrying capacity of 2D FRP/strengthened masonry structures. Composites Part B: Engineering, 36(2005) 619−26.

[43] Moon FL, Yi T, Leon RT, Kahn LF. Testing of a full-scale unreinforced masonry building following seismic strengthening. Journal of Structural Engineering, ASCE, 133(2007) 1215−26.

[44] Shariq M, Abbas H, Irtaza H, Qamaruddin M. Influence of openings on seismic performance of masonry building walls. Building and Environment, 43(2008) 1232−40.

[45] Yi T, Moon FL, Leon RT, Kahn LF. Analyses of a two-story unreinforced masonry building. Journal of Structural Engineering, ASCE, 132(2006) 653−62.

[46] M.Z. Kabir and A. Kalali. In-Plane Numerical Modelling Of Strengthened Perforated Masonry Walls Using FRP Under Cyclic Loading. Asian Journal Of Civil Engineering (BHRC) Vol. 14, No. 1 (2013) Pages 161-179. May 2012.

[47] Palsson, H. (1982). "Influence of Nonstructural Cladding on Dynamic Properties and Performance of High-rise Buildings," Ph.D. Thesis, Civil Engineering, Georgia Institute of Technology, Atlanta, Georgia.

[48] Pinelli, J.-P., Craig, J. I., Goodno, B. J., and Hsu, C. C. (1993). "Passive Control of Building Response Using Energy Dissipating Cladding Connections," Earthquake Spectra, Vol. 9, No. 3, 529-546.

[49] PCI Manual on Design and Typical Details of Connections for Precast and Prestressed Concrete, Second Edition, 1988, Prestressed Concrete Institute, 175 W. Jackson Blvd., Chicago, Illinois 60604.

[50] Pinelli, Jean-Paul, Craig, J. I., and Goodno, B. J., 1990, "Development of Advanced Concepts for Precast Cladding," Proceedings,_ATC-29 Seminar: Seismic Design and Performance of Equipment and Nonstructural Elements in Buildings and Industrial Structures, held in Irvine, California, October 3-4, pp. 26.1-26.11.

[51] Building Code Requirements for Masonry Structures (ACI 530-05/ASCE 5-05/TMS 402-05) Reported by the Masonry Standards Joint Committee (MSJC).

[52] Drysdale RG, Hamid AA, Baker LR. Masonry structures. Englewood Cliffs, New Jersey: Prentice Hall, 1994.

[53] BEHR, R. A, *Architectural Glass to Resist Seismic and Extreme Climatic Events*, Cambridge, UK, Woodhead Publishing Limited, 2009.

[54] LE BOURHIS, E., Glass: Mechanics and Technology, Germany, Wiley-VCH, 2008.

[55] Glazing Systems: Performance under Seismic Conditions Laminated Glass with Solutia Architectural Glazing. SAFLEX. 2007

[56] AAMA 501.4-00, Recommended Static Test Method for Evaluating Curtain Wall and Storefront Systems Subjected to Seismic and Wind Induced Interstory Drifts, AAMA, Schaumburg, IL, USA, 2001.

[57] AAMA 501.6-01, Recommended Dynamic Test Method for Determining The Seismic Drift Causing Glass Fallout From A Wall System, AAMA, Schaumburg, IL, USA, 2001.

[58] FEMA 450, recommended provisions for seismic regulations for new buildings and other structures, FEMA, 2003.

[59] FEMA 273, NEHRP Guidelines for the Seismic Rehabilitation of Buildings, FEMA, Washington, D.C., 1997.

[60] "Continuum Model for Masonry: Parameter Estimation and Validation". Paulo B. Lourenço, Jan G. Rots and Johan Blaauwendraad. Jorunal of Structural Engineering. June 1998.

[61] "Metodología para la evaluación de la vulnerabilidad y riesgo sísmico de estructuras aplicando técnicas de simulación". Yepez, F. Tesis Doctoral. Universidad Politécnica de Cataluña, España, (1996).

[62] "Laboratory Testing Of Unreinforced Masonry Walls Retrofitted With Glass Frp Sheets" H. Mahmood, A. P. Russell, J. M. Ingham.

BIBLIOGRAPHY

"**Protection of Educational Buildings Against Earthquakes**". National Society for Earthquake Technology-Nepal (NSET-Nepal). Jitendra Kumar Bothara; Ramesh Guragain; Amod Dixit.

"**Review of NZ Building Codes of Practice**". Report to the Royal Commission of Inquiry into the Building Failure Caused by the Christchurch Earthquakes. Gregory MacRae; Charles Clifton; Les Megget. 2011.

"**Informe del Sismo de Lorca del 11 de Mayo de 2011**". Instituto Geográfico Nacional. Madrid. Julio 2011.

"**Architectural Design for Earthquake**". New Zealand Society for Earthquake Engineering. 2007**.**

"**Seismic Conceptual Design of Buildings – Basic principles for engineers, architects, building owners, and authorities**". Swiss Federal Office for Water and Geology Swiss Agency for Development and Cooperation. Hugo Bachmann. 2003.

Revista **"Zuncho"**. Nº30. Diciembre 2011. Especial Sismo.

"**Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide**". FEMA E-74. January 2011.

"**2011 Lorca earthquake**". Wikipedia.org

"**Primeras inspecciones aftershock**". Consideraciones y visión general de los daños ocasionados por el sismo de Lorca. Jornada de ingeniería sísmica. El terremoto de Lorca. Antonio Tomás Espín. 2011.

"**Nonstructural Earthquake Damage**". Consortium of Universities for Research in Earthquake Engineering. Robert Reitherman. 2009.

"**Focusing on reducing the earthquake damage to nonstructural components in buildings**". A. Palermo, S. Pampanin & A. Baird University of Canterbury, Christchurch, New Zealand P. Riccio Technical University of Milan, Milan, Italy. (2010).

"**Seismic design for architects**". Outwitting the quake. Andrew Charleson. First edition 2008.

"**Analysis of masonry infilled steel frames with seismic isolator subframes**". Mohammad Aliaari, Ali M. Memari. Department of Architectural Engineering, The Pennsylvania State University. United States. 2004.

"**Masonry Enclosure Walls: Lessons learnt from the recent Abruzzo Earthquake**" R. Vicente, H. Rodrigues, A. Costa, H. Varum, J.A.R Mendes da Silva. 2010

"**FRP Composites for Masonry Retrofitting**". Review of Engineering Issues, Limitations, and Practical Applications. Gustavo Tumialan, P.E., Ph.D., Milan Vatovec, P.E., Ph.D., and Paul L. Kelley, P.E. May, 2009.

"**In-Plane Numerical Modelling Of Strengthened Perforated Masonry Walls Using FRP Under Cyclic Loading**". M.Z. Kabir and A. Kalali. Asian Journal Of Civil Engineering (BHRC) Vol. 14, No. 1 (2013) Pages 161-179. May 2012.

"**Tuff Masonry Walls Strengthened With a New Kind Of CFRP Sheet: Experimental Tests and Analysis**". Ciro FAELLA, Enzo MARTINELLI, Emidio NIGRO, Sergio PACIELLO. 13th World Conference on Earthquake Engineering. Canada. 2004.

"**Behavior of Architectural Precast Panels in Response to Drift**". Designer´s notebook. Prestressed Concrete Institute. (2011).

“**Ductile Cladding Systems for Seismic Design**”. James I CRAIG, Barry J GOODNO, Peeranan TOWASHIRAPORN and Tumay DOGAN.

“**Ductile Cladding Connection Systems for Seismic Design**”. Barry J. Goodno and James I. Craig. Building and Fire Research Laboratory. National Institute of Technology. (1998).

“**Behavior of Architectural Precast Panels in Response to Drift**”. Designer´s notebook. Prestressed Concrete Institute. (2011).

“**Fisis Earthquake-proof system for fixing panels**”. Ruredil. Construction chemicals and building technologies.

“**Seismic Safety of the Building Envelope**”. Chris Arnold, FAIA, RIBA. Building Systems Development Inc. (2009).

“**Building Construction: Principles, Materials, & Systems**”. Edition 2. Medan Mehta Walter Scarborough, Diane Armpriest. Pearson Higher Ed USA. 2011

“**Building Code Requirements for Brick Veneer in Seismic Areas**”. J. Gregg BORCHELT. Brick Industry Association. 2004.

“**Seismic response of a new type of masonry tie used in brick veneer walls**” Ali M. Memari, Eric F.P. Burnett, Brian M. Kozy. University Park. USA. 2002.

“**Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide**”. FEMA E-74. January 2011.

“**Brick Veneer/Steel Stud Walls**” TECHNICAL NOTES on Brick Construction. The brick industry association. 28B. December 2005.

“**Masonry Product Catalog**” DUR-O-WAL. 2007.

“**Seismic Behaviour of Curtain Wall Facades**” A comparison between experimental mock-up test and finite element method analysis. Umberto GALLI. Politecnico di Milano. 2011.

“**Drift Performance of Façade Systems**” Sivanerupan S, Wilson JL, Gad EF, Lam NTK. AEES 2008 Annual Conference.

“**Glazing Systems: Performance under Seismic Conditions**” Laminated Glass with Solutia Architectural Glazing. SAFLEX. 2007

“**Facade damage assessment of multi-storey buildings in the 2011 Christchurch earthquake**” Andrew Baird, Alessandro Palermo and Stefano Pampanin. Bulletin of the New Zealand society for earthquake engineering, vol. 44, no. 4, December 2011

“**Reducing the Risks of Nonstructural Earthquake Damage – A Practical Guide**”. FEMA E-74. January 2011.

“**Architectural Design for Earthquake**”. New Zealand Society for Earthquake Engineering. 2007.

“**New Point Supported Glass Seismic System**”. Prakash (“DES”) Desai, S.E., Abbas Golmohammadi, P.E., Robert Garlipp and Bharath Gowda. 2005.

“**Fundamentals of Finite Element Analysis**”. David V. Hutton. McGraw-Hill. 2004.

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