FOCUSING ON REDUCING THE EARTHQUAKE DAMAGE TO FACADE SYSTEMS

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SUMMARY

Earthquake engineering is facing an extraordinarily challenging era. These challenges are driven by the increasing expectations of modern society to provide low-cost, architecturally appealing structures with high seismic performance. Modern structures need to be able to withstand a design level earthquake with limited or negligible damage such that disruption to business be minimised because of the economic consequences of such downtime.

Technological solutions for seismic resisting structural systems are emerging. However, within the goal of developing a seismic-resisting building, not only the structural skeleton of the building but the entire system must be fully protected from damage. This includes the non-structural components of the building such as the claddings, ceilings and contents. Substantial studies are still required to develop technological solutions and design methods capable of achieving such an earthquake resistance structure.

This paper presents a review of current technology for facades, including design guidelines for seismic-resistant non-structural components and the steps made towards a performance-based design framework. Alternative conceptual strategies and technical solutions to reduce the damage to non-structural elements will also be introduced.

INTRODUCTION

As the earthquake engineering community moves toward a higher expectation of seismic performance, and as the public demands a higher level of earthquake protection, reducing the damage to non-structural components has become a critical factor in building design. Non-structural elements are typically more vulnerable to seismic damage than structural elements. A study of the 66,000 buildings damaged by the 1994 Northridge earthquake showed that approximately three quarters of the buildings suffered damage to only non-structural elements [1]. Similar trends were exhibited in the September and February earthquake events in Christchurch, as can be seen in Figure 1. Moreover, the direct and indirect costs associated with the damage of non-structural components can be significantly more than the costs associated with the damage to the structure itself. A study showed that the non-structural investment costs (including contents) for a typical office are 82% and for hospitals, up to 92% [2].

The recent earthquake in Christchurch on the 22nd of February 2011, where many buildings within the CBD remain vacant due to non-structural damage, has shown that there is an urgent need to develop and propose practical and efficient solutions to reduce the damage to non-structural components during an earthquake event.

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 unexpected failure mechanisms. Significant research has 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 also occur at higher floors, due to the sudden failure of some infills at one floor level [3].

This paper aims to summarise the facade technology available in New Zealand and overseas with the intent to propose a classification framework for facade systems. The classification system will be used as the base to build performance-based seismic design philosophies for each facade typology. Performance-based design for facade systems requires the identification of performance objectives and performance indicators for each type of facade system, which will be introduced in this paper.

Finally, design philosophies and technical solutions capable of meeting the required objective of reducing damage to facade systems will be presented at a conceptual level.

Figure 1: Example of facade damage to masonry infill (left) and precast concrete panels (right).

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OVERVIEW OF FACADE TECHNOLOGY

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

Infill Panel

Infill panels have traditionally been made of heavy rigid materials, such as clay bricks or concrete masonry blocks. However, more lightweight infill panel options such as light steel/timber framed infill walls (or drywalls) are available. Masonry infill construction has a long history through much of Europe and South America and is still one of the most popular choices today. In many European countries it is typical practice to use infill panels in the building leaving the ground storey completely open due to architectural restraints [4], which hugely increases the risk of a soft-storey failure mechanism.

Unreinforced masonry infill construction has been avoided in New Zealand for several decades; primarily because of concerns over its poor seismic performance and the complexity of its interaction with the structure. Consequently, there are a growing number of cases where existing, undamaged masonry infills are being removed and substituted with a lightweight infill. Even so, masonry infill still occupies a large portion of the building stock in New Zealand, with reinforced masonry continuing to be popular in modern construction.

The use of timber framing in New Zealand is a very popular option, particularly in residential construction. It is often preferred because for many situations it is the cheapest option and it offers ease of construction. Steel framing is another alternative, offering several advantages including long life span, fire resistance, strength, durability and the potential to be re-cycled when the building reaches the end of its useful life. It is typical for an infill panel to be combined with a glazing infill system. Glazing infill consists of an aluminium frame attached directly to the infill panel or structure. The frame has rubber gaskets to hold the panes of glass in place and keep the system watertight whilst allowing some in-plane movement. This type of system is simple to construct and is particularly prevalent in low to mid-rise office structures. Infill panels are almost always clad both externally and internally to enhance thermal performance as well as improve aesthetics. Examples of infill technology are shown below in Figure 2.

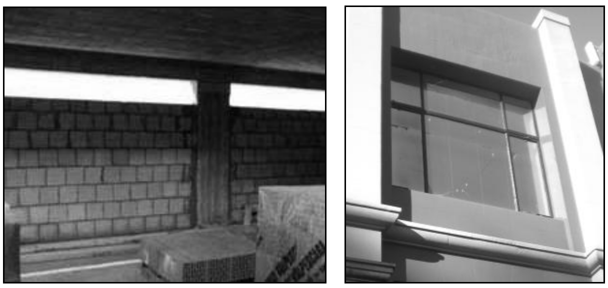


Figure 2: ***Infill technology; masonry infill (left), glazing infill (right).***

Cladding Panel

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. Precast and tilt-slab concrete panels have been one of the most popular cladding material in new non-residential buildings in New Zealand over the past decade [5]. Autoclaved Lightweight Concrete (ALC, also called Autoclaved Aerated Concrete) panels are also among the most widely used material for claddings in Japan [6].

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. These metal frames can be produced so that they provide in-plane movement in order to accommodate differential displacements between adjacent transoms or mullions. Silicone sealant is usually used to allow the glass within the frame to move while keeping the building weather tight.

One of the more recent variations of the stick system is the double skin facade system. Double skins consist of two layers of facade material (typically glass) 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. Examples of cladding technology are shown below in Figure 3.

Cladding panel systems can have many different configurations, compared with infill panels which are more limited. These configurations can be grouped into three primary groups based on the way they are assembled and installed and the type of structural sub-framing. These groups are: unit assemblies, grid assemblies and built-up assemblies [7].

- Unit assemblies: generally prefabricated. Examples include spandrel panels, floor-to-floor panels or multi floor panels.
- Grid assemblies: consist of continuous vertical and horizontal mullions and transoms. Typically built up on site.
- Built-up Assemblies: generally built up on site. Examples include brick or stone veneer.

Figure 3: ***Cladding panel technology; clockwise from top left: double skin, cladding panels, stick curtain.***

Mix System

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 to improve aesthetics. A common mix system consists of timber frame infill with varieties of lightweight cladding.

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 panel typology and modularity, the connection devices used to connect it to the primary structure and the modularity of the connections.

The proposed classification of individual systems is as follows:

Infill Panel Systems

- Masonry infill (clay brick, concrete/cinder block)
- Timber frame infill
- Steel frame infill
- Glazing infill

Cladding Systems

- Stick system
- Lightweight cladding panels e.g. zinc coated steel
- Heavy cladding panels e.g. precast concrete
- Spider glazing
- Double skin
- Monolithic cladding e.g. Exterior Insulation and Finish Systems (EIFS)

The modularity describes the degree to which a system's panels/connections may be separated and recombined. For example a mono-panel has no modularity as there is no way to separate or recombine the panel. A cladding system however may have a large degree of modularity in both the panels and the connections. For example, the panels may be storey-height panels, which may be continuously solid, or incorporate 'hole-in-the-wall' windows. There has been a return to this type of approach in recent years as architectural trends have changed. [1]. Another possibility is spandrel panels, often approximately half storey height, from window head to the sill of the next storey, but can be no more than a beam facing where more glass is used.

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 differently under seismic loading to a multi-panel system of the same material. It will behave differently again if the connection modularity is varied. Therefore it is important to define all such aspects for each system in order to determine the seismic performance of the facade. 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 connections as shown in Figure 4.

Discrete connections are more common in cladding panel systems and are generally metal angle elements. Cladding attachment for heavy systems such as precast concrete panels typically consists of two bearing connections and two lateral (or tieback) connections, as shown in Figure 5.

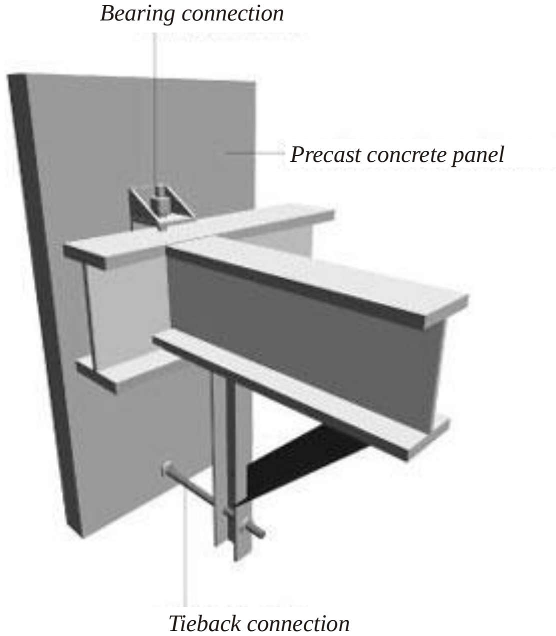


Figure 5: ***Cladding panel detail showing location of bearing and tieback connections.***

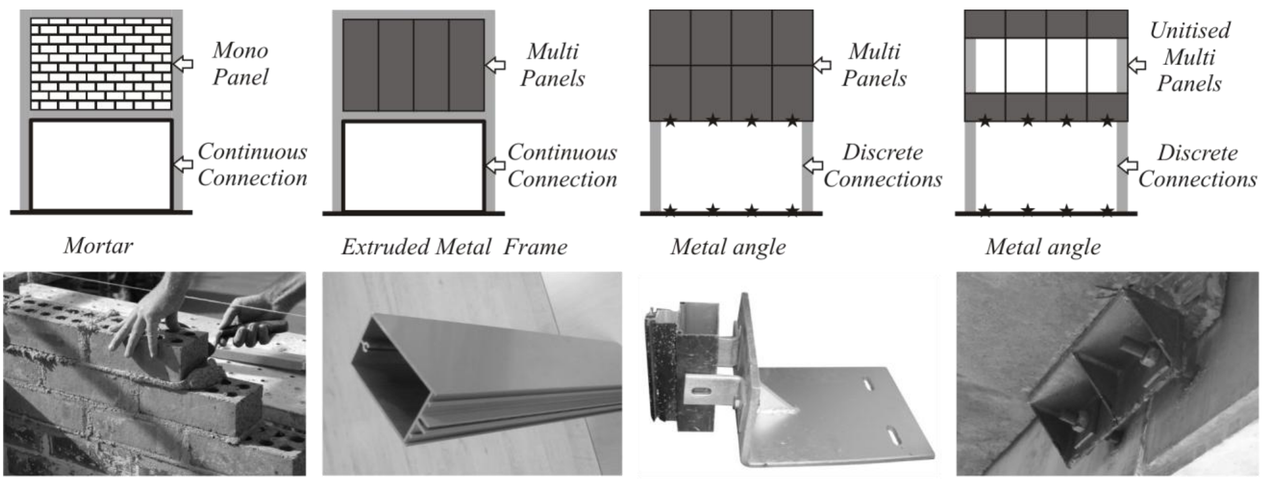


Figure 4: ***Examples of classifications of facade systems including various modularity and connection.***

Bearing connections are intended to transfer vertical loads to the frame or foundation. Tieback connections transfer out-of-plane forces to keep the panel vertical. Tieback connections are also typically designed to allow some deformation, allowing in-plane movement of the panel. Lightweight systems by definition do not have large gravity loads so bearing connections are not often as essential.

Many modern systems incorporate large amounts of glazing, typically held in place using extruded aluminium frames. It is possible to eliminate this frame with the use of 'tong' connections, providing a continuous glass surface which is aesthetically pleasing. This system is commonly called spider glazing. The spider framework supports the large glass panes, avoiding flexing or buckling which may happen if the panes were to rest on their bottom edge. It also accommodates movement of the building within the spider framework. Monolithic glass panes are suspended by means of tongs, which press both sides of glass, as shown in Figure 6. In New Zealand spider glazing is more commonly seen in lobbies or shop frontages, however overseas it has been used for entire tall building envelopes.

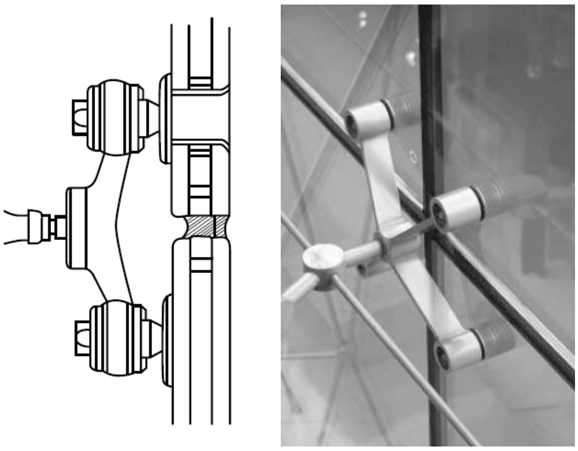


Figure 6: Spider glazing technology.

FACADE FUNCTION AND PERFORMANCE

Facade design and performance is a complex and broad structural research topic. One particular performance aspect cannot be examined without taking into account the numerous other functions of a building's facade at the same time. Therefore, while trying to define and ultimately improve the seismic performance of facade systems, it is very important not to neglect the other aspects of facade performance whilst doing so. The overall performance of the facade can be determined by taking into account all of the facade's functions. These can be grouped as primary and secondary functions. The primary functions are functions that the facade is principally responsible for, these include the following:

1. Define the aesthetic image of the building;
2. Keep water out of the building;
3. Prevent air leakage;
4. Control the passage of light and heat (radiation and conduction);
5. Control sound from the outside;
6. Avoid thermal bridges;

The primary functions are often categorised as the facade's architectural systems. The secondary functions are not the main responsibility of the facade system and include the following:

1. Adjust to movement in the building due to wind, earthquakes, creep etc.
2. Adjust to thermal expansions and contractions
3. Control the passage of water vapour
4. Resist fire
5. Resist structural movement from wind, earthquakes, creep and shrinkage.
6. Resist weather conditions gracefully (without streaking, oxidation, corrosion, freeze-thaw spalling)

Up until the late 1980's and early 1990's there was no standard procedure for assessing the seismic performance of facade systems [8]. Generally the inter-storey deflections were given to the manufacturers and the manufactures chose a system 'off-the-shelf' that they considered most appropriate. However, it was realised that this approach was not adequate since it did not take into account the deformation of individual components. The latter aspect is crucial since the ductility required of each particular component may differ significantly from the overall building ductility [8]. Therefore, the Building Research Association of New Zealand (BRANZ) developed a standardised procedure and rig for testing the racking resistance of cladding systems. The testing procedure simulates a building under earthquake loading by imposing inter-storey deflection. The rig subjects the cladding to racking displacements only, as shown in Figure 7. Inter-storey deflection is seen as the most important parameter so beam curvatures and column rotations are therefore ignored by using this test.

A racking test, similar to that developed by BRANZ has been adopted as a required test in the Australian/New Zealand Standard AS/NZS 4284 [9] which sets out a method for determining the performance of building facades. It includes a number of tests to determine the performance of various facade functions. These tests include wind deflection at serviceability and ultimate limit state, air infiltration, water penetration, seismic test, building maintenance unit restraint, strength and seal degradation tests.

The seismic test involves the in-plane, lateral displacement of the facade sample for a number of cycles at a given period. The parameters used for displacement, number of cycles and period are specified by the structural designer in accordance with the specified serviceability and ultimate limit states appropriate to the geographic region. For a design life of 50 years, NZS 1170.0 defines a Serviceability Limit State (SLS) event as having an annual probability of exceedance of 1/25, while an Ultimate Limit State (ULS) event corresponds to an annual probability of exceedance of 1/500 [10].

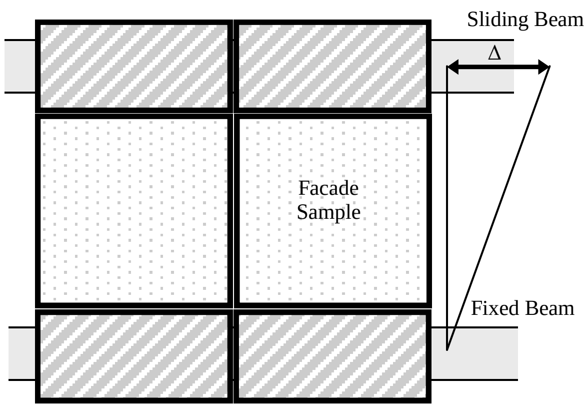


Figure 7: In-plane racking test used for assessing seismic performance of facades.

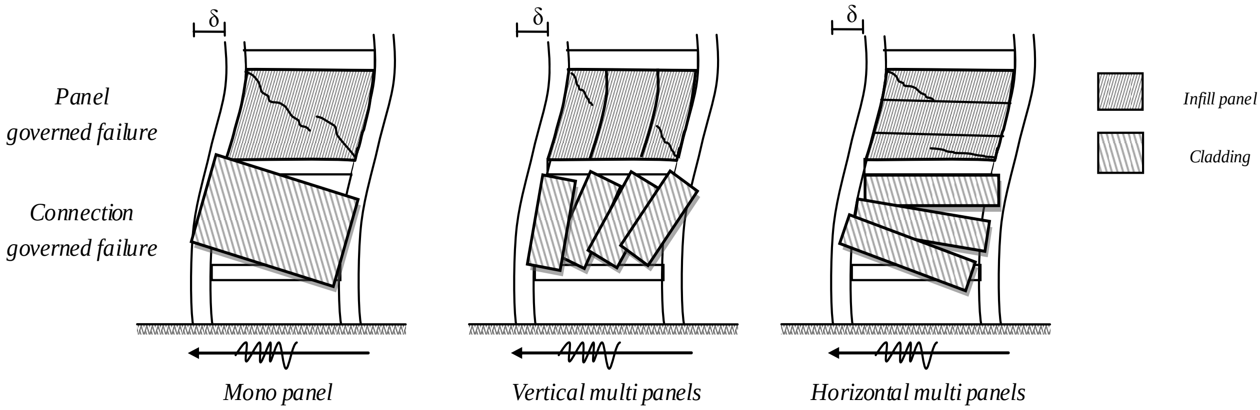


Figure 8: Fully-restrained panel governed failure versus partially restrained connection governed failures.

The SLS displacement test is first carried out, followed by a cyclic water penetration test to determine whether the facade has a reduced weather tightness performance after an SLS level earthquake.

The performance requirements for the test are straightforward and for a facade to meet the standard's requirements it must meet all the test performance requirements. After being subjected to a racking test to SLS the facade must not show any leaks from a cyclic water penetration test. At ULS there is to be no collapse of the test sample. This includes partial or full disconnection of any part of the facade.

Figure 9: A building with numerous different facade systems exacerbates the complicated job of assessing a structures facade performance.

The inter-storey drift of a structure during earthquake excitation typically dictates the behaviour and thus the seismic performance of most non-structural vertical elements or facade systems [11]. Even so, each facade system will behave differently when subjected to the same inter-storey drift level. This is dependent on all of the aspects covered in the previous section, e.g. system, connection, modularity. Figure 8 displays some of the different failure mechanisms for different facade systems as a result of excessive inter-storey drift. Obviously for a building with numerous different facade systems, like that shown in Figure 9, the task of assessing the structures overall facade performance becomes a complicated process.

It is important to understand how each facade system behaves in order to determine which parameter plays the primary role in 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, parameters which relate to the strength of the infill, e.g. mortar strength, are what govern failure, while for cladding it is usually the connections that govern [6]. 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 occur. 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.

DESIGN STANDARDS

The New Zealand Standard, NZS 1170.5 [12], specifies that non-structural elements must be detailed so that they do not contribute in an unplanned way to the buildings seismic response and that damage of non-structural elements is kept to an acceptable level. The result of this prescription is that stiff elements such as masonry infill panels typically need to be fully separated from the structure.

The Standard specifies acceptable serviceability limit state (SLS) criteria in terms of relative racking deflection for different non-structural elements. For example, the acceptable deflection for masonry walls in order to control cracking is the height/600 (e.g. 0.16% inter-storey drift). This is quite a strict requirement since design limits for a new structure are typically set as height/300 or height/250. Displacement Based Design (DBD) clearly makes determining whether such displacement criteria are met much simpler than when a Force Based Design (FBD) is used. The Standard also specifies that a "special study to determine the dynamic characteristics" must be taken out when the mass of a non-structural element is in excess of 20% of the combined mass of the non-structural element and the primary structure.

FEMA 450 [13] and Eurocode 8 [14] are also based on the specification of limits to the relative seismic displacement of non-structural elements. To take into account the varying behaviour of different facade systems, both codes incorporate a number of factors into the design equations. For example, the US code [13] defines a 'component importance factor' and 'response modification factor'. These represent the functionality of the component to the structure, and the energy absorption capability of the component and its attachments respectively. Eurocode 8 [14] also has a special section dedicated to the design of masonry infilled frames. This is designed to take into account the high uncertainties related to the behaviour of masonry infills as well as the possible adverse local effects due to frame-infill interaction.

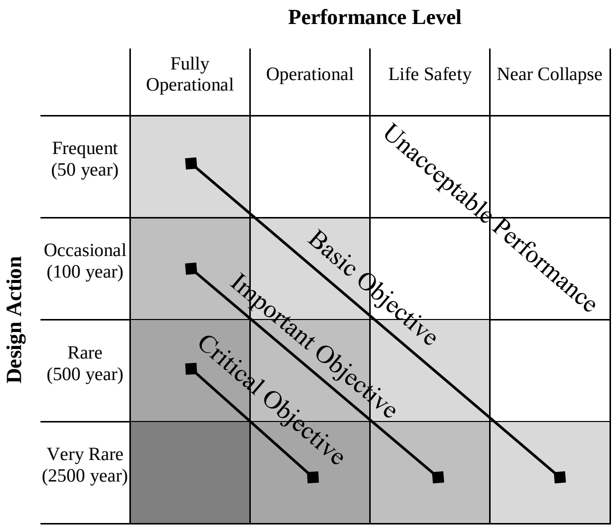
PERFORMANCE-BASED DESIGN

Performance Objectives

Performance-based engineering has become a standard norm for research, development and practice of earthquake engineering, particularly after the 1994 Northridge and 1995 Kobe earthquakes [6]. The primary function of performance-based seismic design is the ability to achieve, through analytical means, a building design that will reliably perform in a prescribed manner under different seismic hazard conditions [10]. The performance, or condition of the building as a whole, should be expressed through qualitative terms, intended to be meaningful to the general public. These terms should use general terminology and concepts describing the status of the facility (i.e. Fully Operational, Operational, Life Safety and Near Collapse) and be classified through appropriate technically-sound engineering terms and parameters [15]. These engineering parameters have to be able to assess the extent of damage (varying from negligible to minor, moderate and severe). Currently this is most commonly done using parameters that measure a structure's maximum deformation (i.e. inter-storey drift or ductility).

This methodology of performance-based engineering can be applied for individual structural members, non-structural elements as well as of the whole building system. Table 2 provides a generic performance matrix with four different performance levels and design actions.

Table 1: Seismic Performance Design Objective Matrix [16].



The basic requirements for setting facade performance objective levels are relatively simple. For example, the basic performance objective would be that a facade remains undamaged following frequent earthquakes and that it does not fail in large (very rare) earthquakes. However, this objective level means that the facade may be damaged to some degree in occasional earthquakes. If it was required that the facade remain undamaged in such earthquakes, a higher objective level would need to be set. This philosophy is similar to that used when determining a building's importance levels using NZS 1170.0 [10].

Performance Indicators

The definition of appropriate engineering parameters to characterise each performance level represents the most critical and controversial phase of performance-based design [15]. These engineering parameters (commonly called performance or damage indicators) need to accurately reflect the level of damage in the structure after an earthquake. Each performance indicator should also typically include appropriate upper and lower bounds. Using this proposed framework, expected or desired performance levels can be connected to levels of seismic hazard by performance design objectives as illustrated in Table 2.

Inter-storey drift or displacement is most commonly used as the performance indicator for determining the likely level of damage in facade systems. Inter-storey drift only requires minimal information about the building so computation is straightforward. However, defining the performance of a building's facade system by using only the maximum drift can be inadequate, just as it is for structural elements. The role of residual (or permanent) deformations has been more recently emphasised as a major additional and complementary damage indicator for both structural and non-structural components [15]. In regards to facade systems, residual deformations can result in increased cost of repair or replacement due to problems associated with the buildings rest position being altered, e.g. windows being jammed and compromised weather-tightness. The suggested performance levels in Table 2, taken from FEMA 356, specify drift levels as being either transient or permanent for masonry walls.

Table 2: Suggested Structural Performance Levels [11].

		Structural Performance Level		
Element	**Type**	**Collapse Prevention**	**Life Safety**	**Immediate Occupancy**
Masonry Walls	Damage	Crushing; extensive cracking. Some fallen units.	Extensive cracking (< 6mm) distributed throughout wall. Isolated crushing.	Minor (< 3mm width) cracking. No out-of-plane effects.
	Drift	1.5% transient or permanent	0.6% transient; 0.6% permanent	0.2% transient; 0.2% permanent
Precast Concrete Panels	Damage	Some connection failure but no elements dislodged.	Local crushing and spalling at connections but no gross failure.	Minor working at connections, crack width <1.5mm
Cladding	Damage	Severe damage to connections and cladding. Many panels loosened.	Severe distortion in connections. Distributed cracking, bending, crushing and spalling of cladding elements.	Connections yield; minor cracks (< 1.5 mm with) or bending in cladding.
Glazing	Damage	General shattered glass and distorted frame. Widespread falling hazards.	Extensive cracked glass; little broken glass.	Some cracked panes; none broken.

MODELLING

Infill

The most common and practical method used for macro modelling of masonry infill panels is the equivalent compression strut model. The model consists of two diagonal struts resisting only compression to represent the infill panel as shown in ***Figure 10***. The stress-strain behaviour of the strut can be used to indicate the damage level of the infill. This is achieved using previously identified limit states, or performance levels, defined as a function of the axial deformation of the diagonal strut, ε_w [17].

Basic geometric considerations can then be used to relate, for a given performance level, the axial deformation ε_w in the equivalent strut to the inter-storey drift, δ. As a result, a simple expression, shown by Equation 1, or supporting chart, as shown in ***Figure 10***, for discrete values of ε_w can be produced.

$$\delta = \frac{L}{H} = \sqrt{\left(1-\varepsilon_w\right)^2\left(1+\left(\frac{L}{H}\right)^2\right)-1} \qquad (1)$$

where δ = drift;
L = frame span;
H = inter-storey height;
ε_w = axial strain in diagonal strut.

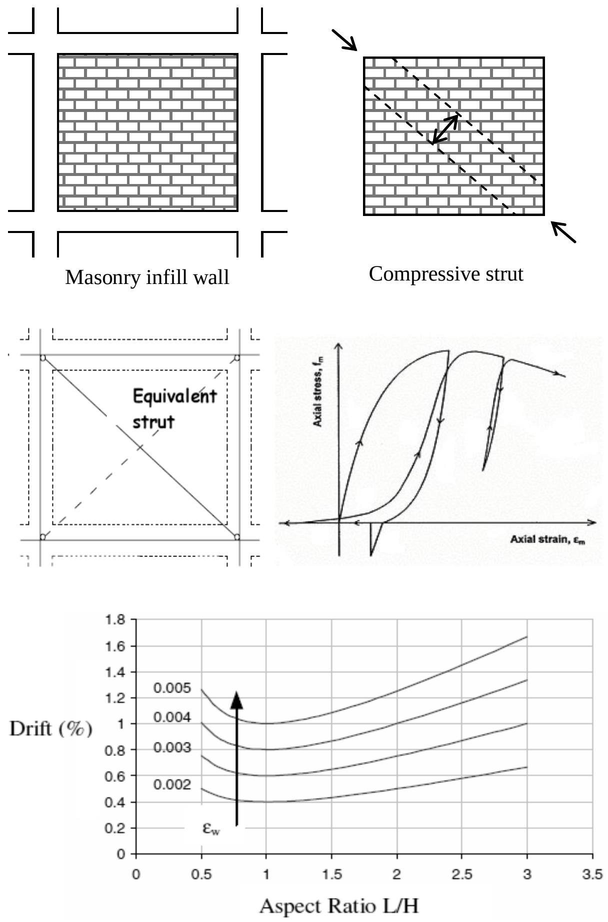


Figure 10: Equivalent diagonal strut model for masonry infill panels [17].

The equivalent diagonal strut can be represented using the hysteretic rule proposed by Crisafulli [18] to simulate the axial response of masonry. This model takes into account the non-linear response of the masonry in compression, including contact effects in the cracked material (pinching) and small cycle hysteresis. The Crisafulli model also provides the ability to take into account the variation of the strut's cross section as a function of the axial deformation experienced by the element. In this way it is possible to consider the loss of stiffness due to the shortening of the contact length between frame and panel as the lateral load increases [4]. The stress-strain relationship for the Crisafulli hysteretic model is shown in ***Figure 10***. By assigning the degree of damage to the level of axial strain, this modelling technique can be used to provide a simple relationship between drift and expected damage state.

Cladding

Cladding systems are typically connected to a structure by a number of discrete connections. As mentioned previously, cladding systems may have a large degree of modularity in both the panels and the connections. The connections may also be located in a variety of locations on the beams and/or the columns. This complicates the problem of attempting to model cladding panels. It also means defining performance levels is difficult without experimental testing to discover where critical weaknesses are in the cladding.

However, capacity design (hierarchy of strength) principles can be used to define a number of different scenarios. Assuming that the cladding systems is comprised of a structural frame member, a connector body and cladding panel, linked together with strong, stiff attachments, as shown in Figure 11, then the problem can be simplified in order to determine where failure is most likely to occur.

If the in-plane strength of the cladding panel is greater than that of the connector body, then the connector body is expected to govern the overall cladding failure mechanism. Conversely, if the connector body is stronger than the panel, then failure is governed by the panel strength. For the above two scenarios it is assumed that the attachment of the connector body is stronger than both the cladding and the connector body itself. This is typically the case designed for; however, errors have been made in the past where the attachment ends up being the weakest link in the system, as shown in Figure 12 (left) where the cast-in channel has torn out of precast concrete panels. When the attachment governs failure then the risk of falling panels is very high.

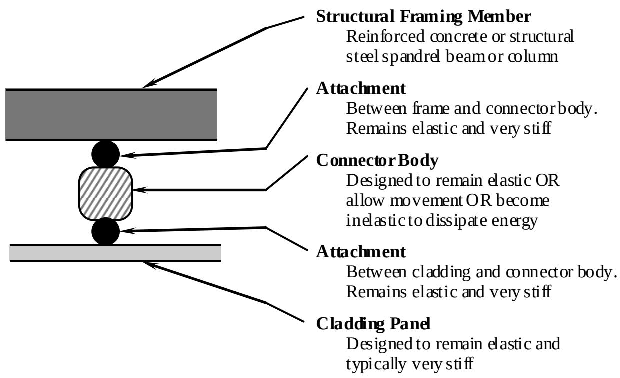


Figure 11: Structure-cladding system [19].

However, for many cladding types the failure mechanism is expected to be governed by the failure of the connection device, as shown in Figure 12 (right). Therefore, each performance level can be related to the performance of the connection alone. How well the cladding connections perform can commonly be determined using the inter-storey deflection as this is used to define the relative displacement between connections. Thus the expected relative displacement between connections shall be found for each hazard level in order to determine performance.

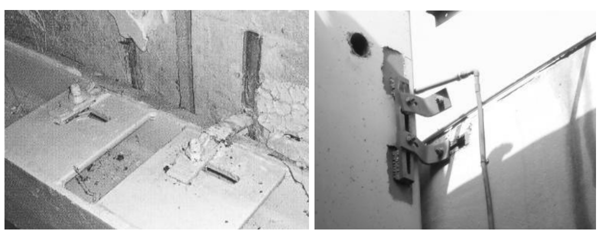


Figure 12: Failure of precast concrete panels; tear out of cast-in channel (L'Aquila, 2009), connection failure (Chile, 2010).

In order to determine the performance at different displacement levels, either experimental or numerical testing may be required. Numerical testing is commonly done looking at the local behaviour using refined finite element models (FEM) like that for a dissipative connection shown in Figure 13.

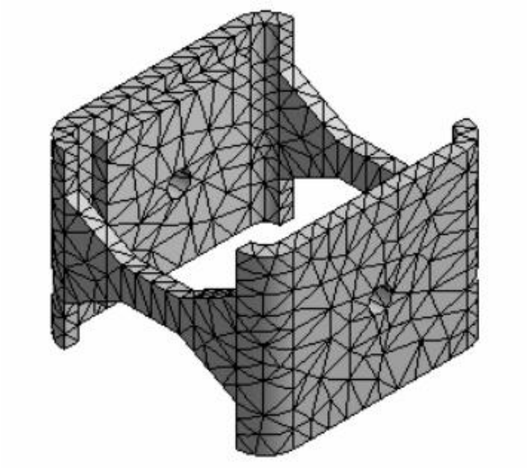


Figure 13: ***Finite element mesh of taped tube cladding connection [20].***

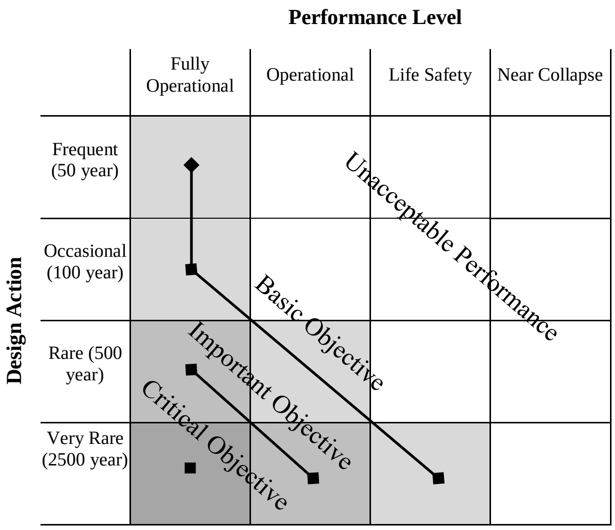
The performance is also dependent on whether the connections are fixed or allow some movements. For example, a connection that is slotted to allow ±25 mm can be assumed to not sustain any damage up to 25 mm relative displacement. However, past 25 mm it is very difficult to predict how the connection and therefore the entire building's cladding system will perform.

The connections can also be used in global models to capture the whole interaction between seismic resisting system and facade elements, by using macro-models based on multi-directional spring elements for the connection with cyclic force-deformation properties derived from test data. Results from such investigations have shown that the inclusion of cladding in the analytical model can significantly affect the seismic response of the building [21]

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. As shown in Table 3 within a Performance Based Objective Matrix, it is possible to show that the objectives can be shifted from that in Table 1 towards a better performance level for same intensity by implementing damage reducing solutions that improve the seismic performance [22].

Table 3: Shifted Seismic Performance Design Objective Matrix (modified after [22])



Because a structure is typically designed neglecting the facade system, the current approach is typically to connect the facade such that the interaction between the facade and the structure is minimised 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.

According to Arnold [23], for the possible contribution of cladding to the seismic resistance of a building, four levels of participation can be identified:

1. "Theoretical Detachment: the cladding, usually lying outside the structure, does not contribute to its lateral stiffness at all. In practice, this would very rarely be the case as in a building with hundreds of cladding panels it is likely that the detachment is not complete, and there is some transmission of forces from the structure to the panels and vice versa."

2. "Accidental Participation: this occurs with connections such as slotted connections and sliding joints in which, because of being or errors in installation, the separation between the cladding and structure in not effective. The result is uncontrolled participation."

3. "Controlled Stiffening or Damping: this involves the use of devices to connect the cladding to the structure in such a way that the damping of the structure is modified (usually increased) or the structure is stiffened."

4. "Full Structural Participation: the cladding and the structure become a new integrated composite structure in which each element performs an assigned role. The cladding may participate in vertical support, and definitely contributes to lateral resistance."

In theory the fourth level of participation makes the most economic and dynamic sense because the cladding is removed from its role of dead weight to one of integral support. In practice this level has proved to be difficult to achieve, and it has proved more economic (if not more performance effective) to adopt level one. Study of other structures in the dynamic environment, such as airplanes and automobiles, has shown a steady evolution from level one to level four. Today's building cladding compares to the doped fabric of a 1920s wood-structured airframe [23].

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 [1]. Such practices have not been as thoroughly adopted in seismically active European countries. 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 minimised using connections which allow lateral movement. Tie-back connections, mentioned previously, are one example which allow for such movement. The pairing of bearing connections with tieback connections mean that the cladding is rigidly fixed to the structure at the bearing connection and thus any relative displacement between the bearing and tieback connections has to be accommodated in flexure of the tieback connection [24].

Similarly to this system is that based on a fixed and sliding connection. In this case the lateral movement is accommodated in the sliding connection by a slot, like that shown in Figure 154. The slot allows the cladding panels to move and rotate relative to the frame when undergoing seismic excitation. An investigation using autoclaved lightweight aerated concrete (ALC) panels connected with fixed and sliding connections showed that under proper detailing, these panels could be successfully isolated from the structure, even under a large inter-storey drift of 4% [6]. The tests showed no visible damage to the panels and no contribution to the stiffness or strength of the structure.

Systems which allow relative movement between the facade and the structure, as shown by the left two diagrams in Figure 145, present challenges regarding issues such as acoustic control, weather tightness, fire protection and aesthetic qualities that need to be addressed. Seismic gaps also present the additional problem of out-of-plane weakness since the gaps means the facade is disconnected from the surrounding frame. If the disconnections are both vertical and horizontal, as shown in Figure 154, then the facade is effectively acting as a cantilever wall which is fixed to the top of the beam. This out-of-plane weakness can be resolved using bracket or slot details which will allow in-plane movement but provide out-of-plane restraint.

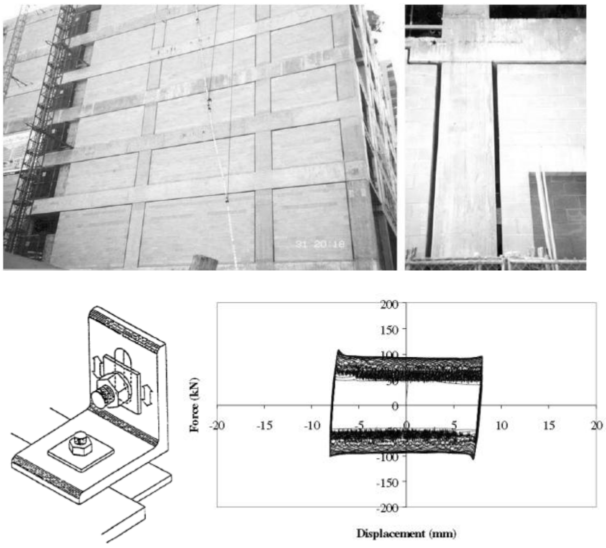


Figure 15: ***Seismic gap (top) and sliding bolted connection with friction-type hysteresis (bottom) [25].***

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 than complete disconnection. This system requires a significant shift in the conceptual design, as the focus on the attachment of the facade is to now benefit structurally from its presence, rather than reduce its possible structural influence. Such a system activates through a relative displacement between facade and structure. At this stage this type of system has proved difficult to achieve, and it has proved more economic (if not more performance effective) to isolate with supplemental damping elsewhere [26].

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 facade and the structure to dissipate energy, as depicted in the third diagram in Figure 145. When the connection is deformed beyond its elastic regime, yielding of the connections is activated. Once the connection has yielded, energy dissipation occurs within the connection which transfers load demands away from the structure. 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 [27].

Finding ways to control and generate energy dissipation in structures is a research field which has been growing steadily over several decades and in which there are constant new ideas being conceived. By controlling the damping in a structure using an energy dissipation device it is much easier to understand what level of damping a structure actually has and how this damping occurs, something which is currently not very well understood. One of the likely requirements of an energy dissipating system is the need to replace damaged/yielded parts after an earthquake. Therefore, having easy access to the damaged/yielded part of the system after an earthquake is an important factor in order to limit disturbance to the occupants of the building. Therefore, facade energy dissipation devices which are externally accessible and which do not detract from the aesthetics of the building can be very desirable as they can be replaced using a building maintenance unit without disturbing the occupants. Figure 16 shows some possible examples of energy dissipative connections.

Another possible solution based on partial disconnection is the use of a seismic fuse device. Such a device is designed to allow full interaction between infill panel and frame under wind loading as well as minor to moderate earthquakes for reduced building drift, but to disengage them under higher intensity and more damaging events.

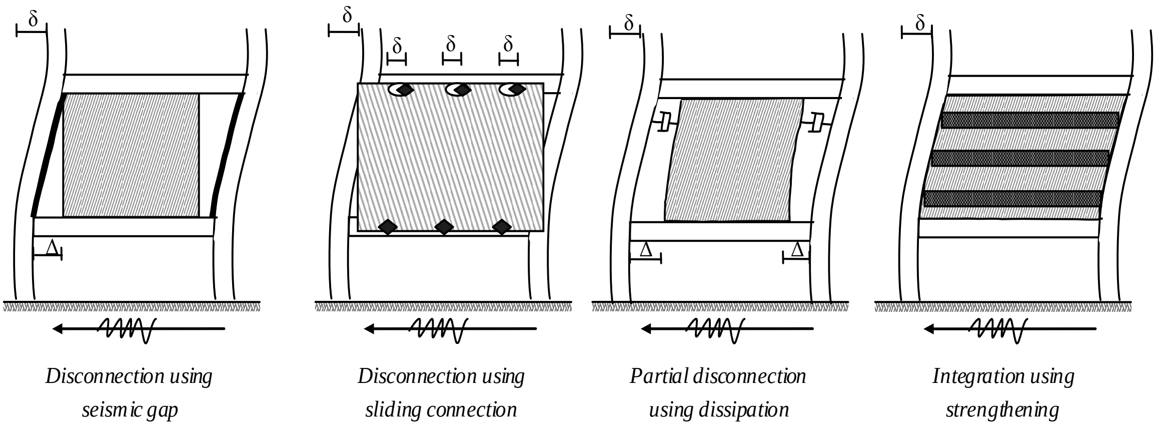


Figure 14: ***Damage reducing solutions for facade systems.***

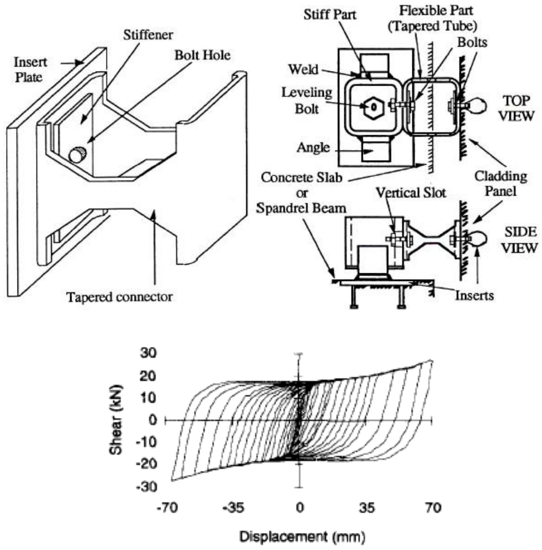


Figure 16: ***Energy dissipative connections, clockwise from top left; tapered connector, possible attachment, hysteretic loop [28].***

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), as shown in Figure 17, is an example of such a system [28]. It consists of two vertical and one horizontal sandwiched light-gauge steel plates with "rigid-brittle" elements in the vertical members. It 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.

An experimental evaluation of the SIWIS system was conducted using a series of lateral load tests on the two-bay three-storey steel frame. The tests showed that the concept of SIWIS system is a viable alternative, but it needs further experimental study for better understanding of the system performance under cyclic loading [29].

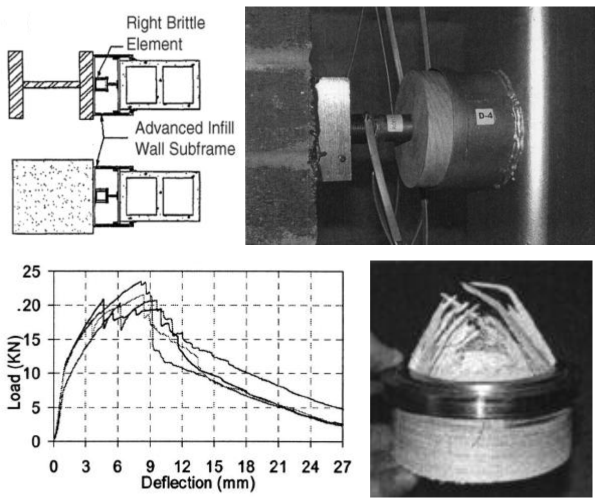


Figure 17: ***Seismic Infill Wall Isolator Subframe (SIWIS) system including force-displacement curve and failure mode [29].***

Full Interaction

Having a complete integration of the facade system is often an effective strategy to reduce the drift of a structure due to the additional stiffness provided by the facade. Often many structures have been built in this way, for example confined masonry infills, where the RC frame is cast after constructing the masonry infill is a common construction practice in South America and more recently has been proposed by the World Housing Encyclopaedia (WHE) to be more widely used than what is considered typical infilled frames [30]. Strengthening solutions for infill walls are typically very simple and straightforward to add to existing infill walls, as displayed in Figure 18. Therefore these solutions present the most likely possibilities as retrofit solutions. Once strengthened, the facade can be fully integrated within the existing structure. The end result of full integration is the transformation of the frame and facade into what is effectively a shear wall. This can be seen as desirable in buildings where stiffening of the frame is required.

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 [31].

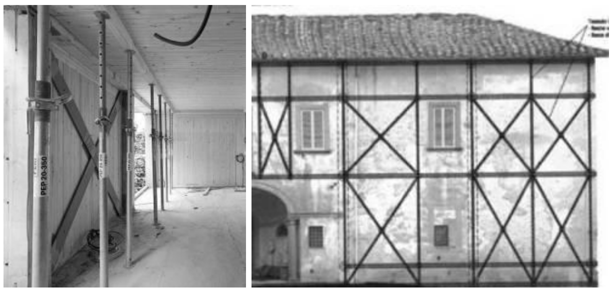


Figure 18: ***Full interaction strengthening solutions.***

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 shotcreted onto masonry infill panels and provides a tensile strain capacity of several hundred times that of normal concrete. Tests at UCSD showed the effectiveness of this type of treatment at preventing large scale damage to masonry infill in RC frames. [32] The fracture toughness of ECC is similar to that of aluminium alloys; furthermore, the material remains ductile even when subjected to high shear stresses [33].

Typically full interaction is a damage reducing solution employed with infill walls rather than cladding technology. Of the typical cladding technology used in modern construction, it would appear that only precast concrete panels have the strength and stiffness to realistically fulfil the role of a strengthening solution.

PROPOSED FUTURE RESEARCH

The authors intend to continue research in order to develop a performance-based seismic design framework for multi-storey buildings considering facade interaction. The research plans to follow the steps outlined below.

Classification of facade systems

Classification in terms of:

- Type of facade panel
- Panel modularity
- Connection type
- Connection modularity

Shown below in Figure 19 are example classifications for several different cladding typologies.

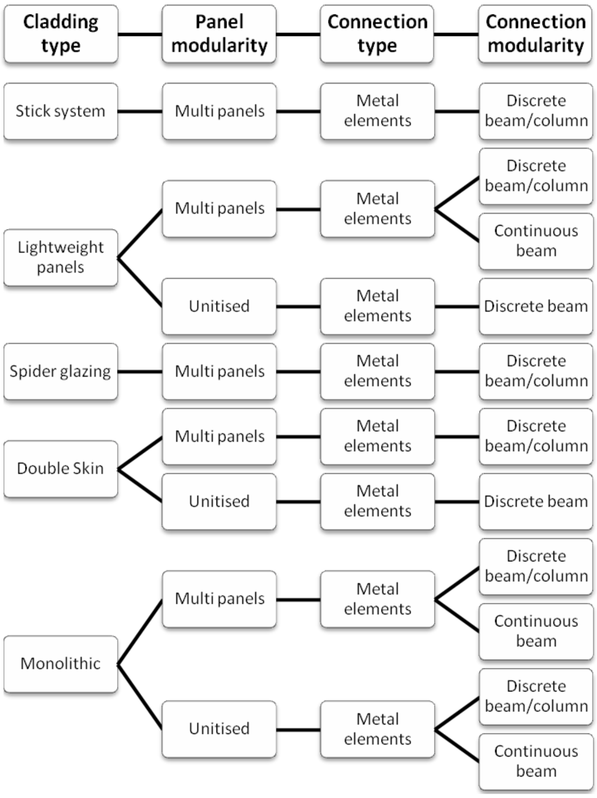


Figure 19: ***Example classifications for several cladding typologies.***

Definition of damage indicators

Damage indicators to identify performance levels will be defined for each facade typology. Because a building's facade is a multi-function system the damage indicators for each typology need to represent the performance of the facade as an entire system, e.g. decreased weather-tightness even though there may not be any actual damage represents an overall reduced performance level. This step is required before the experimental phase and will be used to develop fragility functions for later stages.

Experimental testing of various types of facade technology

As part of a more comprehensive research project on solutions to control and reduce the damage to non-structural elements, the cyclic behaviour of the most common typologies of facades and connections as identified from the classification stage will be investigated experimentally. This will be achieved using a test frame that represents a single-bay and single-storey of a reinforced concrete (RC) building, as shown in Figure 20.

The cyclic loading by imposed transverse displacement will define the progressive damage of the facade. Besides the usual strength and ductility criteria, the energy dissipation capacity (for each level of displacement/drift) will also be defined, based on the overall cyclic response.

The modularity of both the facade and connections will also be varied, as shown in Figure 21. The group of test specimens will provide a good representation of the current facade technology used in New Zealand as well as around the world.

Using the experimental test results, the drift values suggested for the various performance levels in FEMA 356 (see Table 2) can be improved upon. The permanent drift ratios and corresponding damage states (including description) from the experimental tests will be valuable information necessary for damage assessment purposes also.

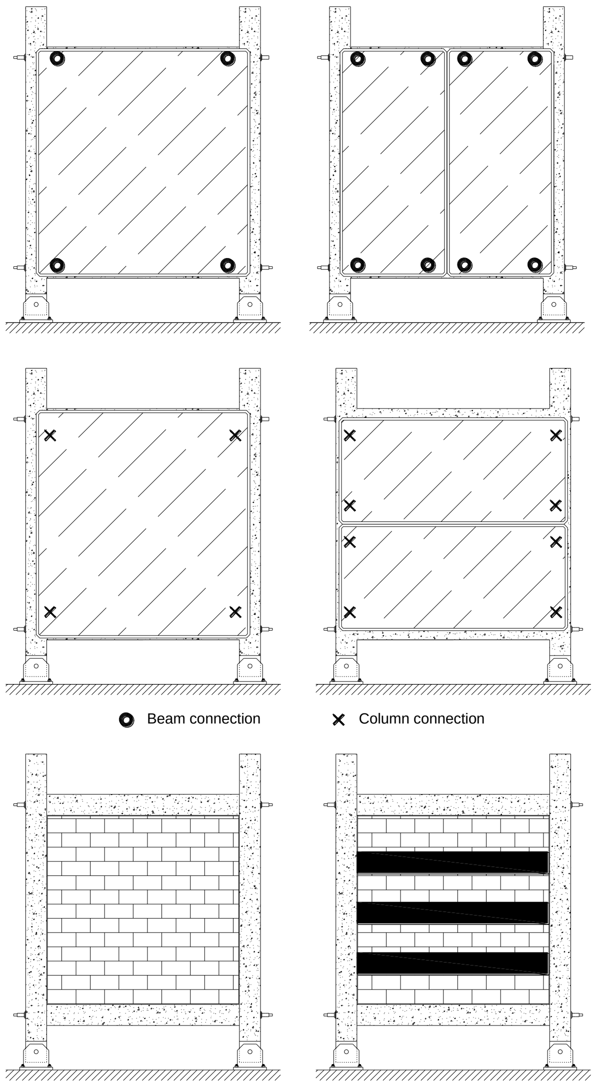


Figure 20: ***Cladding panels of various panel and connection modularity (top four), infill panels without and with strengthening (bottom two).***

Numerical multi-storey building analyses

The seismic behaviour of multi-storey buildings with different facade typologies will be investigated by means of non-linear dynamic analyses. This will be done by adopting simplified hysteretic rules from the experimental phase. This will allow correct characterisation of the seismic response of the building, taking into account the structural interaction of different facade and connection systems under earthquake conditions. The facade panel and connection modularity will be varied as well as the configuration of the panels on the building, as shown in Figure 21. Initially, importance will be given to the existing and most common facade systems in New Zealand buildings. Subsequently, future and new technological solutions will be investigated.

The analyses will be carried out on various multi-storey buildings, designed in accordance with NZ Standards, using nonlinear dynamic analysis with real/recorded earthquake motions. The force-deformation relationship of the facades on the overall response behaviour will be examined to determine whether, when determining facade performance, it is necessary to include facades explicitly in a nonlinear analysis or whether it is sufficient to conduct analysis with a frame only and then apply the recorded inter-storey drifts to determine facade damage.

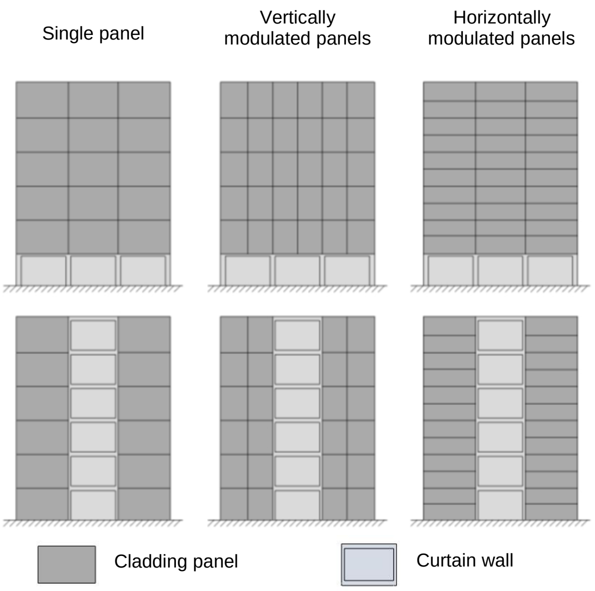


Figure 21: Various panel configurations with varying panel modularity.

Design guidelines

The results of the analyses will be applied to the development of a performance-based seismic design framework for multi-storey buildings considering facade interaction. An estimation of the likely damage to structural and facade systems will be derived using previously defined non-structural limit states and more conventional structural ones. Figure 22 below depicts a performance framework considering both structural and non-structural performance for increasing design actions.

The future research will also look into verification of the current design parameters suggested by FEMA 450 and Eurocode 8 to identify if improvements can be made.

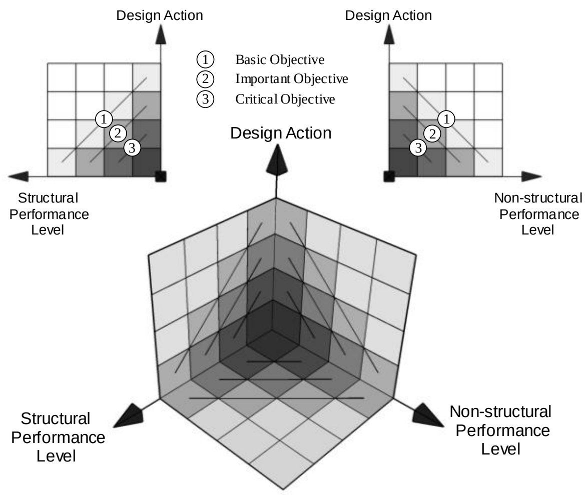


Figure 22: Multi-performance based seismic design objective framework

CONCLUSION

This paper presented a brief summary of the facade technology available in New Zealand and around the world. This was used to show the need for a classification framework of facade systems in terms of type, modularity of the panel, connection type and modularity of the connection. The concept of performance-based seismic design was introduced in relation to facade performance. This provides the foundation for further research work using the classification framework to determine specific performance based design limits for facade solutions. Finally, design philosophies and technical solutions that reduce the damage to non-structural components were presented at a conceptual level. The need to integrate seismic performance with architectural performance was also highlighted. The authors intend to investigate numerically and experimentally all the above mentioned concepts for a proper implementation of a performance based design procedure for non-structural components for both existing and new buildings.

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