Review of Design and Installation Practices

for

Non-Structural Components

Prepared for the Engineering Advisory Group by New Zealand Consultants,

Industry and Related Experts

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EXECUTIVE SUMMARY

Following the February 2011 Christchurch earthquake, the Engineering Advisory Group to the Department of Building and Housing requested that a document on the Review of Design and Installation Practices for Non-Structural Components be prepared. The document would highlight issues regarding the performance of the design and construction of non-structural elements. The non-structural issues to be addressed were related to ceilings, facades and partitions. This work was required to address technical issues only and was to describe any deficiencies in meeting current codes.

In response to this, a team from the University of Canterbury who was conducting research for the Ministry for Science and Innovation (MSI) under the Natural Hazards Research Platform was requested to coordinate this work.

In order to represent the issues of the key stakeholders in the process, the coordination team called three workshops/meetings at the University of Canterbury in which key stakeholders were invited. These occurred between May and August 2011. Stakeholders represented were consultants, researchers (e.g. University of Canterbury, BRANZ and GNS), manufacturers and installers of different systems, and representatives of related industry groups including the Association of Wall and Ceiling Industries (AWCI).

In the initial meetings it soon became recognised that the issues with non-structural elements were more than technical. Also, a number of techniques were becoming available, that at little extra cost could result in very little damage after an earthquake. This report goes above and beyond its initial scope of looking at technical issues, by describing in detail the key types of failure seen in the earthquakes, the regulatory/legislative issues which have resulted in, and continue to result in, poor performing non-structural elements, and some design/detailing techniques which can result in very low damage in an earthquake. In addition, schematics are included where possible to facilitate good communication of good structural details for fabricators and installers.

The report attempts to represent a consensus document that summarises the opinions of all participants. After a brief introduction on non-structural elements (prepared by Stephen Sawrey) the report includes three main sections on ceilings (led by Dr. Dhakal and Dr. MacRae), facades (led by Dr. Palermo and Baird) and partitions (led by Dr. Pampanin and Tasligedik). Each of these sections on ceilings, facades and partitions contains specific information on common systems, specifications used, issues with current procedures, summary of damage types, recommended practice, and best practice detailing guidelines.

Project Participants

The following people were informed about the progress of the project. Many attended the meetings and others provided email feedback.

Name	Organisation	Background
Chris Aubin	CFG Group	Distributor
Andrew Baird	UC	Research Engineer
Graeme Beattie	BRANZ	Research Engineer
Dave Brunsdon	EAG	Consulting Engineer
Paul Campbell	Opus	Consulting Engineer
Alistair Cattanach	Dunning Thornton	Consulting Engineer
Andrew Charleson	VUW	Research Engineer
Rajesh Dhakal	UC	Research Engineer
Greg Flemming	Bradfords Precast	Installer
Cavan Forde	CFG Concrete	Distributor
Hans Gerlich	Winstones	Distributor
John Hare	Holmes/EAG	Consulting Engineer
David Hayes	Thermosash	Facade Testing
Keith Hogg	Hush Interiors	Installer
John Keen	USG	Supplier
John Kitchen	Formans	Installer
Bruce Levy	Winstones	Distributor
Gregory MacRae	UC	Research Engineer
Robert Mannes	IRHACE	Consulting Engineer
John Marshall	Fulton Hogan	Installer
Scott Marshall	King Facades	Distributor
Ben McGill	CFG Concrete	Distributor
Martin Meyers	Meyers & Associates	Consulting Engineer
Murray Mitchell	Opus	Consulting Engineer
Alessandro Palermo	UC	Research Engineer
Stefano Pampanin	UC	Research Engineer
Dave Parkinson	Alutech	Distributor
Phil Parkinson	Alutech	Distributor
Dave Peck	USG	Supplier
Dennis Prout	AWCINZ/Formans	Installer
Arun Puthanpurayil	UC	Research Engineer
Vinod Sadashiva	UC	Research Engineer
Stephen Sawrey	Sawrey Consultants	Consulting Engineer
James Sefton	USG	Supplier
Clara Sumner	Winstones	Distributor
Sahin Tasligedik	UC	Research Engineer
SR Uma	GNS	Research Engineer
Jasper Van Der Lingen	Sheprout	Architect
Trevor Yeow	UC	Research Engineer

AWCINZ = Association of Wall and Ceiling Installers New Zealand
BRANZ = Building Research Association of New Zealand
EAG = Engineering Advisory Group
GNS = GNS Science
IRHACE = Institute of Refrigeration, Heating & Air Conditioning Engineers of New Zealand Inc.
UC = University of Canterbury
VUW = Victoria University, Wellington

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1 General Provisions for Non-Structural Elements

1.1 Introduction

Claddings, ceilings, partitions and plant are building elements that have structural requirements. Failure of such parts can be a life risk and can impair essential building functions so minimum requirements are set out in the New Zealand Building Code. Premature failure of parts that do not present a life risk has proven costly. Recommended industry practice, when followed, should minimise the possibility of life risk during a design level earthquake.

1.2 Minimum Requirements – The New Zealand Building Code B1.

The NZ Building Code is a mandatory performance based building code which includes building parts in its requirements. Compliance Documents (previously known as Approved Documents) provide one means of meeting the clauses of the Building Code. Other ways to follow the Building Code may involve a detailed product development and testing programme through a registered laboratory.

Buildings built to the method (Acceptable Solution or Verification Method) described in a Compliance Document are automatically deemed to comply with the Code. They prescribe a methodology for ensuring compliance. Verification Methods (VM) are by calculations or tests, usually by building professionals, to demonstrate Building Code Compliance.

Referring to B1/VM1 amendment 11, clauses 2.0 and 13.0 we can see that AS/NZS1170 and NZS 4219 respectively, both with modifications, are both cited as Compliance Documents.

NZS4203 (1976) used to be a compliance document until 1 December 2008.

The structural design of many building parts is required by B1/VM1 AS/NZS1170, specifically NZS1170.5 Section 8 - Parts. Ceilings, partitions, claddings and building plant are included in this section. Reference must also be made to AS/NZS1170.0 when evaluating risk and loading levels when designing to the Ultimate and/or Serviceability Limit States.

Building plant is also specified in NZS4219 so either compliance document may be used for the restraint of plant to meet VM1.

At present, except for heavy claddings, these requirements are seldom enforced by territorial authorities so it has been left to each industry sector to be self-regulating.

Designers often use commentary to AS1170.5, e.g. Table C8.1 for guidance examples of the necessary earthquake restraint of building parts. However, this table can be confusing for some cases when read alongside the actual standard that it is supposed to clarify. The standard itself should be used when attempting to comply with the compliance document.

Parts classifications in NZS1170.5 are inferred as follows:

- P1 – Claddings and glazing (ULS)
- P2 – Heavy plant, ceilings and heavy partitions in auditoriums (ULS)
- P3 – Heavy plant and partitions (ULS)

P4 – Egress stairs, partitions and ceilings etc whose failure would affect the function of emergency egress/lighting, life support systems and rescue systems. (ULS)

P5 – Ceilings, partitions, cladding, plant and other parts of structures with post disaster functions, medical emergency facilities etc (SLS2)

P6 - Parts for which the consequential damage caused by its failure are disproportionately great – e.g. pipework over valuable contents (SLS1)

P.7 – All other ceilings, partitions and other parts (SLS1)

The magnitude of the parts coefficient used to obtain the design load, is dependent upon the parts classification, as well as the building's importance level, the hazard class associated with its location, the subsoil class, the near fault factor (for long period structures), the building height, the height of the part within the structure, the natural period, the ductility of the element and its fixings, and its structural performance factor.

It should be noted that a ductility of 1.0 is adopted when assessing the serviceability limit state of a part. For the ultimate limit state, connections are typically designed for a ductility of 1.25.

Sometimes the full intent of VM1 has not been met. This may be a result of misunderstanding of the minimum requirements or because the lack of enforcement attracts poor compliance. Design guides (i.e. generic design guides or cookbooks) may not be written to take account of all code requirements. For example a ceiling design guide based on SLS1 may be wrongly adopted for a ceiling supporting emergency lighting.

1.3 Recommended Practice

Recommended practice for building elements considered is discussed further within the body of the report. However an overarching design recommendation is that a reasonable standard should adopt ULS 1 (in place of SLS1) as a minimum design action for all parts: ceilings, a/c plant, partitions etc. Alternatively, the SLS2 limit state may be appropriate for all buildings, not just IL4 buildings. When ultimate limit state design is to be used, the selection of the ductility of the part and its fixings to the structure should be done with care.

The Design Features Report contained in Appendix A of this document has a section on "parts and portions" which provides a check that the design of building parts of a new building has been covered. Appendix D provides guidelines for the design of suspended ceilings.

Currently there is a tendency for excluding the proper bracing of parts from construction projects. For example a design guide may cover SLS1 and exclude the ultimate limit state, even if it is required by NZS1170.5. Contractors often exclude the bracing of various parts from their tender (ceiling, a/c plant etc) and once excluded, bracing may not be reintroduced to the project.

2 Ceiling Systems

2.1 Background

Ceiling systems consist of the ceiling itself, and all the components that may interact with the ceilings. In NZ, while there is no restriction on the types of ceiling that may be used in different situations, small ceilings are commonly constructed with gypsum board. Moderate and large ceilings are more often suspended on a cold-formed steel grid, into which ceiling tiles sit as shown in Figure 2.1.1. Here, the tiles can move a few millimetres in different directions within the grids, so force must be transferred through the grid members, rather than directly between the neighbouring tiles. A number of manufacturers provide grid ceilings.

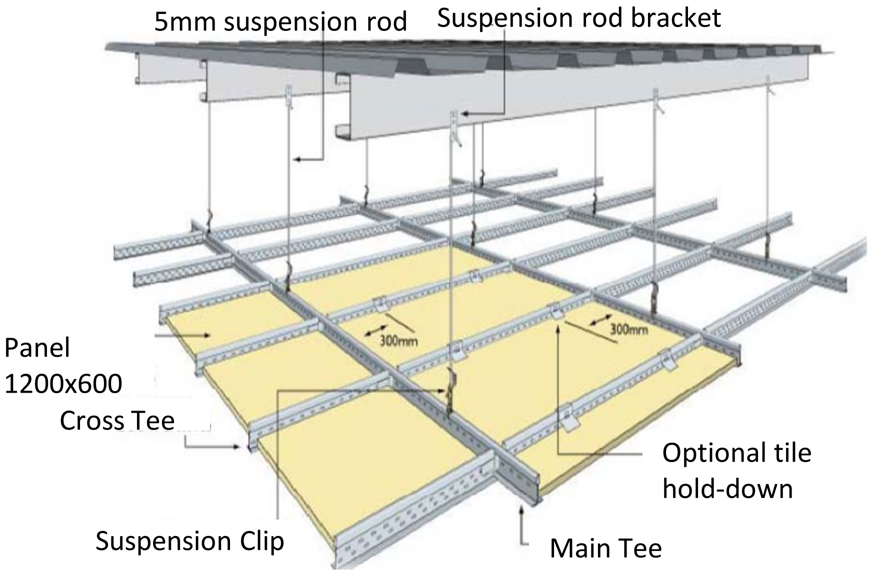
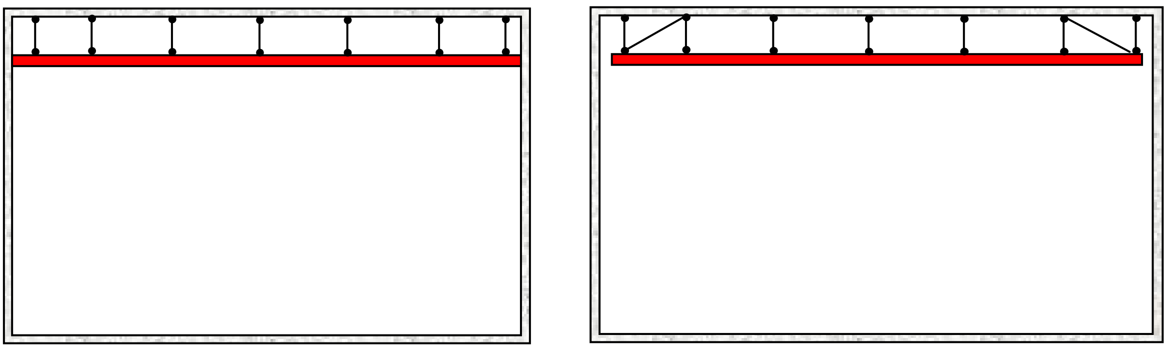


Figure 2.1.1 Typical ceiling configuration not showing seismic bracing (based on Rondo, 2009)

Suspended ceilings are commonly designed to be either fixed to the perimeter walls, or they may be floating, as shown in Figure 2.1.2. It may be seen in Figure 2.1.2a that it is possible to brace the ceiling off the walls. Floating ceilings (Figure 2.1.2b) must be braced to the floor or roof above to prevent large movements under service conditions, as well as to transfer horizontal earthquake induced inertia forces.



(a) Perimeter Braced (b) Floating

Figure 2.1.2 Typical ceiling configurations (Singh et al 2011)

When bracing the ceiling off the walls, as shown in Figure 2.1.2a, lateral inertial ceiling forces are taken axially through the tee rails in each direction, to the walls. One of three models is typically adopted. Firstly, the ‘compression only’ model, whereby the ceiling rails are modelled in compression only and bear horizontally on to the wall through the perimeter angle trim; secondly, the ‘tension only’ model, where the tee rails are in tension only and are fixed to the perimeter trim which is in turn, fixed to the wall structure. In the third ‘tension-compression’ model, one end of the ceiling tee rail structure is in compression (and bears on one wall) while the other end of the ceiling tee rail structure is in tension and is anchored to the opposite wall. Axial loads (tension and compression) are at their highest near the walls and at their lowest towards the centre of the room. Each tee rail will be in tension at one end and in compression at the opposite end.

Floating ceilings (Figure 2.1.2(b)) are braced to the structure above. Typically this system is adopted if tee lengths exceed the maximum length determined for perimeter fixed ceilings.

The ceiling system contains the ceiling itself as well as the interacting elements including partitions, bulk heads, heating, ventilating and air conditioning (HVAC) equipment, electrical equipment, and fire sprinklers. Excessive movement or failure of one of these can result in damage to, and/or collapse of, part of a ceiling.

Examples of these interactions are illustrated below in Fig 2.1.3.

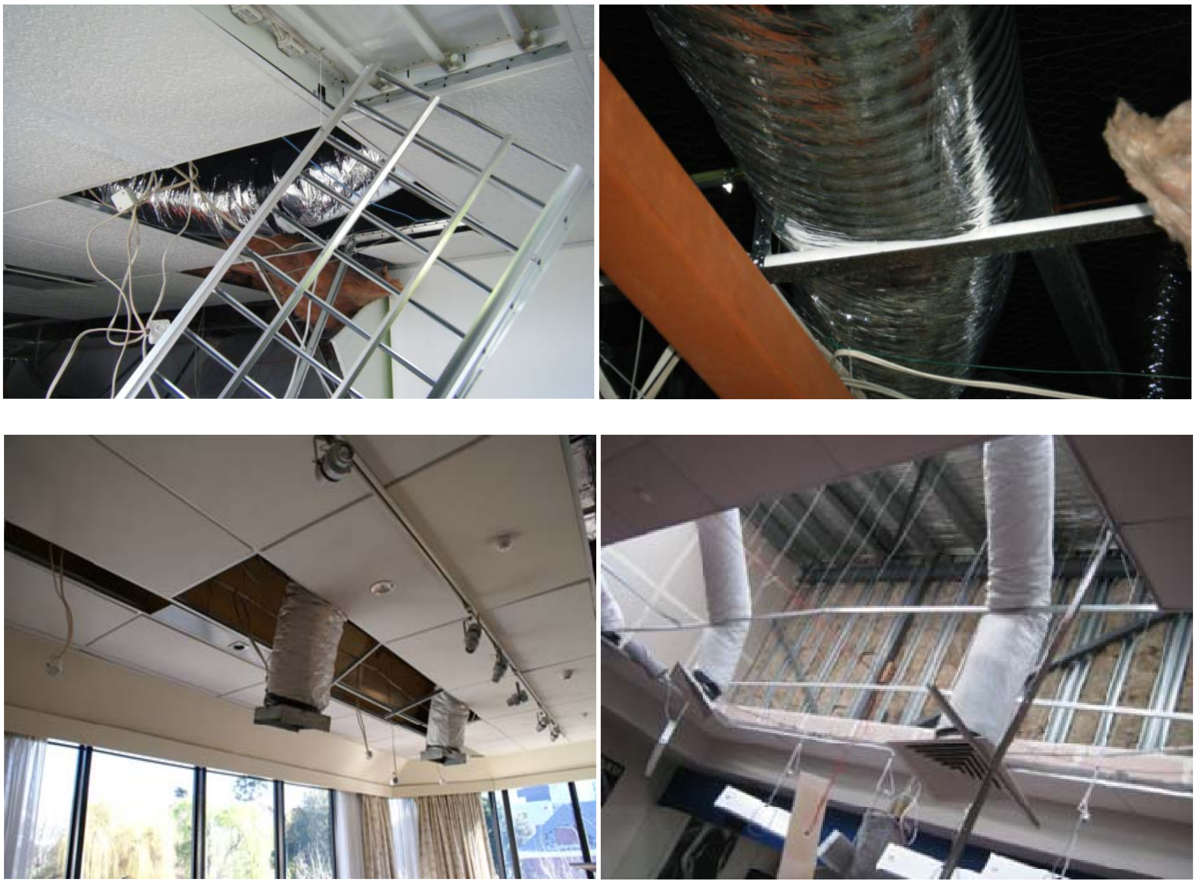


Figure 2.1.3 Interactions between ceilings and services in buildings in Christchurch (Photos: R Dhakal and K Hogg)

2.2 Current Design and Installation Specifications

The following documents are available for ceiling system design and installation in New Zealand.

- NZS 1170.5:2004 Structural design actions - Earthquake Actions
- AS/NZS 2785:2000 Suspended Ceilings: Design and Installation
- NZS 4219:2009 Seismic performance of engineering systems in a building.

A number of proprietary guides also exist. These generally follow loading standards older than NZS 1170.5 and none are current. Also international information is available from groups such as the US Federal Emergency Management Agency (FEMA).

Until July 2011, only AS/NZS 1170 was listed as a compliance document to the NZ Building Code as specified by the Department of Building and Housing (DBH). However, in 2011, the DBH also listed NZS 4219 as a compliance document for ceiling (DBH, 2011 Section B1/VM1).

Some key specifications from these documents are given below:

NZS1170.5:2004 Structural Design Actions Part 5: Earthquake actions – New Zealand covers ceilings in Section 8 "Requirements for Parts and Components".

- Clause 8.1.1 says: "Where required by Section 2, all parts of structures including permanent, non-structural components and their connections, and permanent services and equipment supported by structures, shall be designed for the earthquake actions specified in this Section." Note that Section 2 sets the performance criteria and states in clause 2.6 "All structure parts shall meet the requirements of Section 8."
- The commentary (in Appendix CA) says "Building part … an element that is either attached to, and supported by the structure but is not part of the structural system, would be cladding panels, machinery and architectural fitments …"
- In Clause 8.1.2, the buildings parts are divided into seven categories, among which ceilings feature in the following categories (specified in the Commentary Table C8.1):
    - Auditorium ceilings: P.2 (Part representing a hazard to a crowd of greater than 100 people within the building)
    - Light suspended ceilings: P.7 (All other parts)
- Auditorium ceilings (P.2) are designed for ultimate limit state (ULS) whereas suspended ceilings are designed for serviceability limit state (SLS1) (Clause 8.1.2).
- Clause 8.2 requires that for a ceiling at level *i* of a building, the design response coefficient is the horizontal acceleration coefficient $C_P(T_P)$ derived for that floor which supports the ceiling, and is calculated as:
    - $C_P(T_P) = C(0)\ C_{Hi}\ C_i(T_P)$
      where C(0) is the site hazard coefficient for T=0 determined using the values for the model spectrum and numerical time history methods, C_{Hi} is the floor height coefficient for level *i* calculated as a function of the height as specified in Clause 8.3 (it linearly increases from 1 at ground floor to 3 at a height of 20% of building height or 12m, whichever is less), T_P is the period of the ceiling and $C_i(T_P)$ is the spectral shape factor of the ceiling given in Clause 8.4 which is equal to 2.0 for periods less than or equal to 0.75 s, 0.5 for periods longer than or equal to 1.5 s, and varies linearly in between.

- The horizontal and vertical design forces are then calculated as a product of the horizontal/vertical design action coefficient, the risk factor R_P (which is 1.0 for ceilings), the ceiling horizontal/vertical response factor C_{PH}/C_{PV} which depends on the ductility and varies between 1 (for ductility equal to 1) and 0.45 (for ductility 3 or greater), and the weight of the ceiling W_P. The horizontal and vertical design forces are not to be greater than $3.6W_P$ and $2.5W_P$, respectively (Clause 8.5).
- Table C8.2 suggests that ceilings directly attached/framed to the structure/walls be designed for ductility 3 or less and a suspended ceiling shall not be designed for ductility greater than 2.

AS/NZS 2785:2000: Suspended Ceilings – Design and Installation

Performance objectives for suspended ceilings are stated as follows:

- The aim is to provide a ceiling system that has adequate strength and serviceability, is stable and durable, and satisfies other objectives such as economy and ease of construction (Clause 3.1.1).
- Ceiling systems shall be designed and installed in such a manner that the suspension and frame will remain structurally sound, without maintenance, for a period of 15 years (Clause 3.1.2).

The following limit states are recommended:

- Ultimate limit state (for life safety): A ceiling has adequate strength if the probability of failure of the system or components is acceptably low throughout its intended life (Clause 3.3.1).
- Serviceability limit state (for operational continuity and business interruption): A ceiling is serviceable if the probability of loss of serviceability of the system of the components is acceptably low and the ceiling maintains its intended performance level throughout its intended life (Clause 3.4.1).

The methods to design for the ultimate limit state are given as:

- The ultimate limit state is reached when the ceiling system or part thereof ruptures, becomes unstable, or loses equilibrium (Clause 3.3.1).
- The member or component shall be proportioned such that the design action effect S_u* (calculated for the specified loads/actions and load combinations) is not greater than the design ultimate strength NR_u (strength reduction factor times the nominal capacity obtained from the material standard or tests) (Clause 3.3.2).
- Ceiling hangers shall be proportioned such that the failure or removal of a single hanger does not trigger a progressive collapse of the ceiling system (Clause 3.3.2).

The method to design for the serviceability limit state is given in Clause 3.4.2 as:

- The total deflection of the ceiling shall take into consideration the deflection of the suspension system, and remain within the specified limits (between L/250 and L/600).

NZS4219:2009 - Seismic resistance of engineering systems in buildings:

Clause 5.13 "Suspended Ceilings, Equipment supported by the ceiling and equipment in ceiling voids" contains the following:

- Clause 5.13 "Suspended ceilings are outside the scope of this standard. Where service loads are greater than 3 kg/m^2, the ceiling designer should be advised."
- Clause C5.13 "Suspended ceilings should be designed and constructed in accordance with AS/NZS 2785. For importance level 4 buildings P5 may govern".
- "Equipment supported by the ceilings …not exceeding 10kg shall be positively fixed to the suspension system but not supported by the ceiling panels or tiles."
- A range of prescriptive installation details are specified for fixing elements to the ceiling within the void.

Clause 3.4: Earthquake load demand

$F = CW$

F = Earthquake load demand on a component

W = Operating weight of the component (Cl. 3.4.4)

C = lateral force coefficient = $2.7\ C_h Z C_p R_c$ not greater than 3.6

C_h = floor height coefficient; 3.0 above ground floor and 1.0 at or below ground floor (based on NZS 1170.5 but with conservative assumptions)

C_p = performance factor (Cl. 3.4.2)

R_c = component risk factor (Cl. 3.4.3)

This section is a simplification of NZS1170.5. Therefore, meeting these requirements also means that NZS1170.5 requirements are met.

Dead load considerations (Clause 5.13):

- Suspended ceilings, equipment in ceiling voids and supported by the ceiling are considered as dead load
- Service loads greater than 3 kg/m^2 (0.03 kPa) need special consideration
- Equipment with a mass less than 10 kg shall be supported by the grid of the suspended ceiling system.
- Equipment exceeding 10 kg mass shall be supported independently of the ceiling

Interference between ceilings and other equipment (Clause 5.13):

- Equipment supported independently of the ceiling shall have a clearance of 25 mm all round to allow independent movement between component and ceiling.
- Ceiling suspension components (hangers, braces, and so forth) shall be located with clearances as in Table 15. In this table, ceiling hangers and braces are considered to be restrained components.

- A minimum of 50 mm clearance in the vertical and horizontal direction is recommended between restrained components
- A minimum clearance of 150 mm in horizontal and 50 mm in vertical direction is recommended between unrestrained components and restrained components

Hanging of luminaries along with ceiling grid system (Clause 5.14):

- All fixings shall be positive, locking type to prevent disengagement
- Where luminaries are recessed or surface-mounted on suspended ceilings, they shall be positively clamped to the ceiling suspension systems
- Clamping shall be by means of screw and nuts or locking-type clamping devices

In addition to the abovementioned standards, the following standards are also relevant to ceiling design/installation and performance assessments:

- AS/NZS 1170.1:2002 - Structural design actions - Permanent, imposed and other actions
- AS/NZS 1530.3:1999 - Methods for fire test on building materials, components and structures - Simultaneous determination of ignitability, flame propagation, heat release and smoke release
- AS 2946:1991 - Suspended ceilings, recessed luminaries and air diffusers – Interface requirements for physical compatibility
- ISO 6308:1980 - Gypsum plasterboard - Specification
- ASTM C423-09A - Test method for sound absorption and sound absorption coefficients by the reverberation room method
- ASTM E1414-11 - Standard test method for airborne sound attenuation between rooms sharing a common ceiling plenum (two room method)
- In addition to the above documents, a number of proprietary guides exist in the industry. As these may not necessarily be current, it is the designer's responsibility to ensure that they are current before using these guides.

It should be noted that the previous loadings standards have also contained specifications for ceilings in their "parts" chapters. A summary of these is given below:

1) NZS 4203:1976 has specific provisions for ceilings as non-structural elements. Values for C_p, R_p or $C_{p,min}$ are provided in Clause C3.6.5.

   Clause 3.6.5.1 states that "The support system for suspended ceilings shall be designed and constructed so as to avoid sudden or incremental failure or excessive local or cumulative deformations that would release ceiling components likely to cause a hazard to the occupants".

   Clause 3.6.5.2 states that "suspended ceilings, including integral light fixtures, shall be designed with C_p be taken as 1.0 horizontally". That is, they should be designed to have a strength of 1g horizontally.

   Clause 3.6.5.3 limits spreading of grids to 5mm and Clause 3.6.5.4 states that elements need to be anchored against a net upward force equal to 1/3 their actual

weight. For very light tiles (< 2kg) which will not cause injury, they do not require the tiles to be fixed against earthquake forces.

2) NZS 4203:1982 takes C_p as 0.60 horizontally. Other clauses are similar to the 1976 code.

3) MOW PW/81/10/1, 1985
This document discusses both braced and unbraced (pendulum) ceilings. Pendulum ceilings are discouraged. The document discusses interaction of the ceiling with partitions and recommends a force from partitions of 200 N/m^2 of ceiling. Minimum strengths are specified for main runner and cross-runner members. Specific recommendations for vertical members are specified with design loads, the degree of verticality and the attachment to the slab above. They are not allowed to be attached to, or bent around, other services. The ceiling is required to carry the vertical forces even if one vertical member is removed. Specific requirements exist for bracing, perimeters and fixing. Items weighing more than 4 times that of the ceiling have to be supported independently of the ceiling. Other items have to be considered in the ceiling design, and those with a weight of more than 250N are required to restrained such that they cannot fall more than 100mm.

4) NZS 4203:1992 takes a different approach. For parts which could form a life hazard, or for which continuing function is important, Clause 4.12.1 indicates a risk factor, R_p, of 1.10, otherwise it is 1.0. The seismic floor coefficient is dependent on both the floor level and the seismic coefficient. It is given as the design force, if the structure were to remain elastic, at the level, divided by the weight of the level (Cl 4.12.2.2). A structural performance factor, S_p, is included in this calculation, where S_p is less than unity. This part coefficient is dependent on the period and ductility of the part. Different coefficients are used for the serviceability and ultimate limit states. A design example is provided. However, no specific considerations for suspended ceilings are listed, and it is not clear for designers and regulators what values should be taken for R_p, ductility, etc.

5) For parts which could fall resulting in loss of a life NZS1170.5 classifies parts into different categories to determine a part risk factor, Rp, in Table 8.1. These are required to be designed for ULS, SLS1 or SLS2 depending on the category selected. It is not clear to engineers what category should be selected for a specific suspended ceiling. The demands may be considerably different with different categories. Section 8.2 specifies the part coefficient as a function of the peak ground acceleration, a floor height coefficient, and part spectral acceleration shape value. The floor height coefficient is 3.0 in the upper 80% of stories of tall buildings. The part spectral acceleration shape value is 2.0 for parts with a period less than 0.75s. The part design force is then given as the part coefficient multiplied by the part weight, part risk factor, and part response factor representing the part ductility capacity. The part response factor may be as low as 0.45 for parts with ductility capacities greater than 3.0. The part ductility capacity must be determined by special study, if it is to be taken as less than unity. The maximum part design force is 3.6 times the part weight. Again, there is no clear guidance as to what category a ceiling should be. Section 8.7

states that non-ductile connections of some parts can be designed for a part ductility of 1.25.

Since suspended ceilings are generally developed by companies and are sold as proprietary systems, construction details, element capacities, as well as analysis methods to obtain element demands from global loads are generally provided by companies in their literature, or through discussion with their engineering staff.

2.3 Issues with Current NZ Design Procedures and Practice

A number of concerns have been expressed by designers, contractors and fabricators, etc. in the meetings of the ceilings groups. Both major and minor concerns are listed below.

Performance Objectives:

- Suspended ceilings are included in the least important category (Category 7 in the recommendations in NZS 1170.5 Clause C8.1) considering the lowest level of design demand (SLS1). The required design level for ceilings may be too low as evidenced by the fact that a number of ceilings were replaced several times in Christchurch during the Canterbury earthquake sequence.
- Life safety threat from ceilings is not fully acknowledged. There were fatalities resulting from ceiling damage in the 2011 Japan earthquake. It is fortunate/lucky that there was no life loss as a result of the several ceiling collapses in the 2010-2011 Christchurch earthquakes. It seems that ceilings should be designed to have no possibility of causing death/injury during ULS shaking to be consistent with the NZ Building Code.
- If performance objectives become too severe/conservative, this may mean that suspended ceilings will stop being a reasonable design option. This could have a significant adverse effect on the suspended ceiling industry.

Ceiling Standards:

- Some standards are not up-to-date and reference out-of-date standards (E.g. As/NZS 2815).
- The standards do not emphasize or state specific requirements to ensure good ceiling system performance, such as the gap-size between a floating ceiling and the wall, or how to consider interactions with specific service components.
- Floor acceleration profiles in current standards (NZS1170.5 and NZS4219) are the same up the building height for all building types. In reality, different building types have different accelerations at different heights, so this may result in some conservatism/non-conservatism.

Ceiling Capacities and Capacity Assessment for Design:

- Test protocols are available for ceilings. Many of these require shaking table testing.
- There is no method of relating test protocol results to design

Ceiling Design Practice Issues:

- No generic information is available on the principals involved in designing ceilings and providing bracing
- There is a lack of generic design examples, especially regarding lateral accelerations and bracing.
- Often ceiling systems are designed by engineers who are not the building designers and information necessary for ceiling design is not readily available. This includes the likely accelerations and drifts. This takes time and cost to estimate from an analysis of the building and it requires knowledge of such things as the soil conditions if it is done accurately.

- Ceiling examples are often not checked.
- The requirements for basic design criteria, drawings and documentation, and Interaction of Ceiling System and Building Services required for AS/NZS 2785 Appendix C is seldom provided at tender or consent time.

Installation Issues:

- There is a significant amount of poor installation.
- Installation issues include:
- The NZS 4219 recommended minimum clearances between the ceiling and equipment supported independently of ceilings are often not being adhered to in practice.
- Component connection is often inadequate
- Inappropriate connection of ceilings to suspended services (such as ducts), or
- A lack of ceiling support being provide around services
- Many installers have very limited training. No training is required for installation in general.
- There is a lack of training opportunity for installers
- While NZS 4219 requires all fixings between hangings of luminaries along with ceiling grid system to be positive, locking type to prevent disengagement there is a lack of examples and illustrative sketches leaving room for misinterpretation leading to faulty details.
- Often there is a lack of inspection
- Ceiling inspectors are often not trained

Political Issues:

- Ceiling design is generally considered as an afterthought, rather than as part of the major building contract.
- The tasks of building structure design/construction, service equipment design and placement, partition design/installation and ceiling design/installation are often conducted separately or with little interaction between these groups. Therefore coordination often does not occur to ensure that each of these parts acts as required.
- Often ceiling design has not been completed at the time of the tender for installation. Also, fees for ceiling design are seldom included in the tender. Responsible contractors, who pay for the ceiling design and pay the building design engineer therefore have a high contract price. Other installers, who do not perform the full design have lower contract prices and often win the contracts. As a result, the current system discourages rigorous design.

Anecdotal evidence suggests that the prices for ceilings in Christchurch are some of the lowest in NZ. Also, the quality of installation is correspondingly low.

Changes or building use or occupancy often results in changes to the building internal layout with movement of partitions and installation of different ceilings. This can result in poor seismic behavior if appropriate precautions are not made.

While there are a number of issues listed above that related to ceiling performance, this document will only concentrate on those related to the technical standards. Other issues need addressing with other methods.

2.4 Summary of Damage Types in Canterbury Earthquakes

The damage observed to suspended ceilings described below during the recent Canterbury earthquakes includes that sustained during the September 2010 Darfield earthquake (MacRae et al 2011) as well as the February 22nd and 13th June 2011 Christchurch earthquakes. This has been organised by damage type: grid damage; perimeter damage; interaction with other components; grid spreading; and combinations of different damage types.

2.4.1 Grid Damage

Grid damage results from excessive force on the grid members or connections. This then results in ceiling grid distortion under compression and subsequent buckling of the grid members or failure of the connections while the perimeter connections remain intact. Below are several examples of grid damage:

Figure 2.4.1 Damage resulting from the disconnection of the cross-tee from the main beam resulting in localized collapse of the grid and loss of tiles (Photos: Hush Interior Ltd)

Figure 2.4.2 Damage to grid members due to excessive compression force (Photos: K Hogg)

Figure 2.4.3 Damage resulting from main beam splice failure (left) and cross-tee disconnection with main beam (right) (Photos: Hush Interior Ltd)

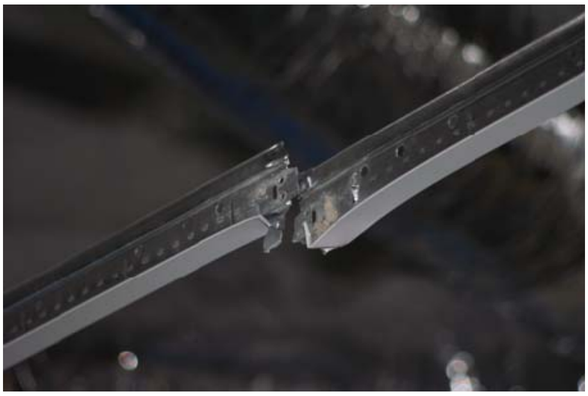


Figure 2.4.4 Damage resulting from main beam splice connection (left) and buckling of cross-tee connections and main beam splice failure (right) (Photos: Hush Interior Ltd)

The uniformly distributed mass (in the form of tiles) in a ceiling generates uniformly distributed inertial force, and the accumulated inertial force which causes axial compression in the grid members becomes greater near the supports (i.e. the perimeter) than in the middle (Paganotti et al 2011). In general, the observed compression damage of grid members was hence more severe near the perimeter, whereas failures due to tension and connection fracture did not follow a specific spatial pattern.

2.4.2 Perimeter Damage

Perimeter damage results from the main tee or cross-tee losing seating on the perimeter angle around the ceiling. Loss of seating can result due to a lack of a rivet to connect the grid member to the angle or failure of the rivet itself. This results in the grid members and tiles dropping from the ceiling. Edge perimeter hanging wires can prevent the member and tiles from falling, however, this can result in the tile and members being forced back into the angle causing damage to the tiles and members; see Figure 2.4.5 (right) below.

Figure 2.4.5 Damage caused by disconnection of main beams and cross-tees. Note damage to panels at right which have not dropped from the ceiling but have been forced back into the wall angle causing damage (Photos: Hush Interior Ltd)

Common current practice is to connect the grid members to the perimeter angle with centre single-size riveting which only connects to the face cap. Such a riveting system was found to be inadequate. In some cases, the inadequate rivets were observed to fail in tension, leaving only the aluminium cap to hold the system together. In other cases, the rivets were also found to have ripped through the steel wall angles and tee rails. Probably, the standards need to

provide detail specifications on the number, size and locations of rivets for such perimeter connections.

Figure 2.4.6 Damage caused by failure of the main beam/cross-tee to wall angle rivet (Photos: Hush Interior Ltd.)

In one case, during repair after the February earthquake damage, engineered tee connections were provided with an increased number and diameter of rivets. This detail survived the June 13 earthquake and the quakes afterwards. Only the trim suffered a small amount of distortion. A photo of this connection is shown in Figure 2.4.7.

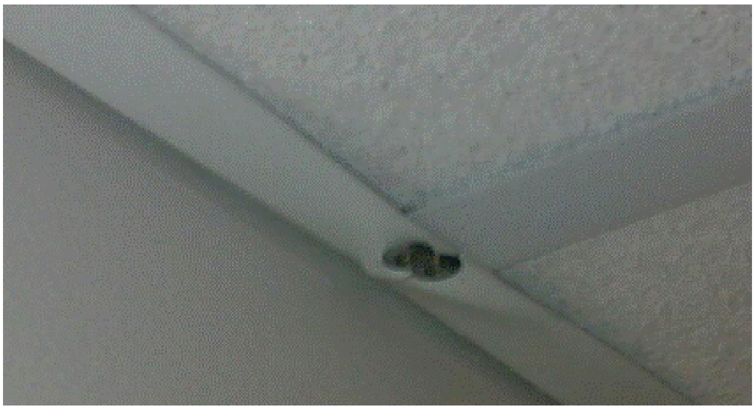


Figure 2.4.7 Perimeter connection with increased number/diameter of rivets (Photo: Hogg)

2.4.3 Interaction with Other Components and Equipment

Damage to suspended ceilings can result from force transferred from services above the ceiling into the ceiling itself. In a standard installation of a suspended ceiling the hanger wires are placed at 1200mm centres. However, the presence of services (such as HVAC)

above the ceiling can mean this is not possible. As a result suspended ceilings are sometimes partially hung from services within the ceiling (most commonly HVAC ducting and plant). As this plant is rarely secured properly, when it moves it imparts force into the ceiling, causing damage. Additionally, services above the ceiling moving during an earthquake can impact the hanging wires of the suspended ceiling, once again imparting force into the ceiling. Suspended ceilings are not designed to take the additional force from this plant. Grills within the ceiling plane were often observed to have fallen from the ceiling and localised loss of tiles often occurred around the location of these grills. Some instances of ceiling failures resulting from the interaction with the services are shown in the figures that follow.

Figure 2.4.8 Damage caused by interaction with services above the ceiling (Photos Hush Interiors Ltd.)

Figure 2.4.9 Damage caused by unbraced AC ducts swaying or causing damage (Photos: R Dhakal, K Hogg)

In some buildings the ducting sizes make it difficult to provide hangers at the required spacing and the service heights provided were too low to provide proper bridging under services. Figure 2.4.10 below shows cases like these.

Figure 2.4.10 Damaged ceiling with inadequate service height (Photos: K Hogg)

Interaction with partition walls also caused damage to some ceilings braced to the walls. Some internal walls extend to the floor above and are fixed either directly to the floor or beam or through inclined braces which are required to minimize buckling of slender walls. In some cases the ceiling may also be supported on these internal walls. The braces of these walls are close to the ceiling hangers and other equipment braces and are likely to interact with the system. It was found that the failure of the braces (some of which fell off) could easily have caused significant damage to the ceilings (see Fig 2.4.11 left).

Figure 2.4.11 Interaction between internal walls and ceiling (Photos: K Hogg)

In some cases, the partition walls stopped at the ceiling level and were braced by ceilings. Obviously, in such cases the ceiling needs to cater for the wall as well. Glazed partitions fall into this category, and due to small aluminium sections they are difficult to brace. In some buildings, such wall partitions were found to be out of plumb and in one glazed partition, glass jumped out of the top track (see Fig 2.4.11 right).

2.4.4 Grid Spreading

The suspended ceilings discussed in the previous sections are all two-way exposed grid systems. That is, they consist of a two-way grid of inverted 'T' shaped members hung from

the ceiling above. The panels are then dropped in and rest on the flanges of the inverted ‘T’ sections.

The damage shown in the photos below occurred to grids that are different to the two-way type grid discussed in the previous sections of this report. The grids below consist of main beams spanning one way and hung from the structure above. There are typically no transverse runners (except where the ceiling may have been retrofitted with these). The drop-in panels prevent the grids from spreading apart during an earthquake. However, if panels do drop out there are no members to stop the grids moving apart (spreading) and causing further panels to fall from the ceiling. Also, as opposed to the two-way systems discussed above, the panels are not supported on all four edges by the grid system. The panels are instead interlocked, so when one panel falls it leaves the next panel susceptible to falling

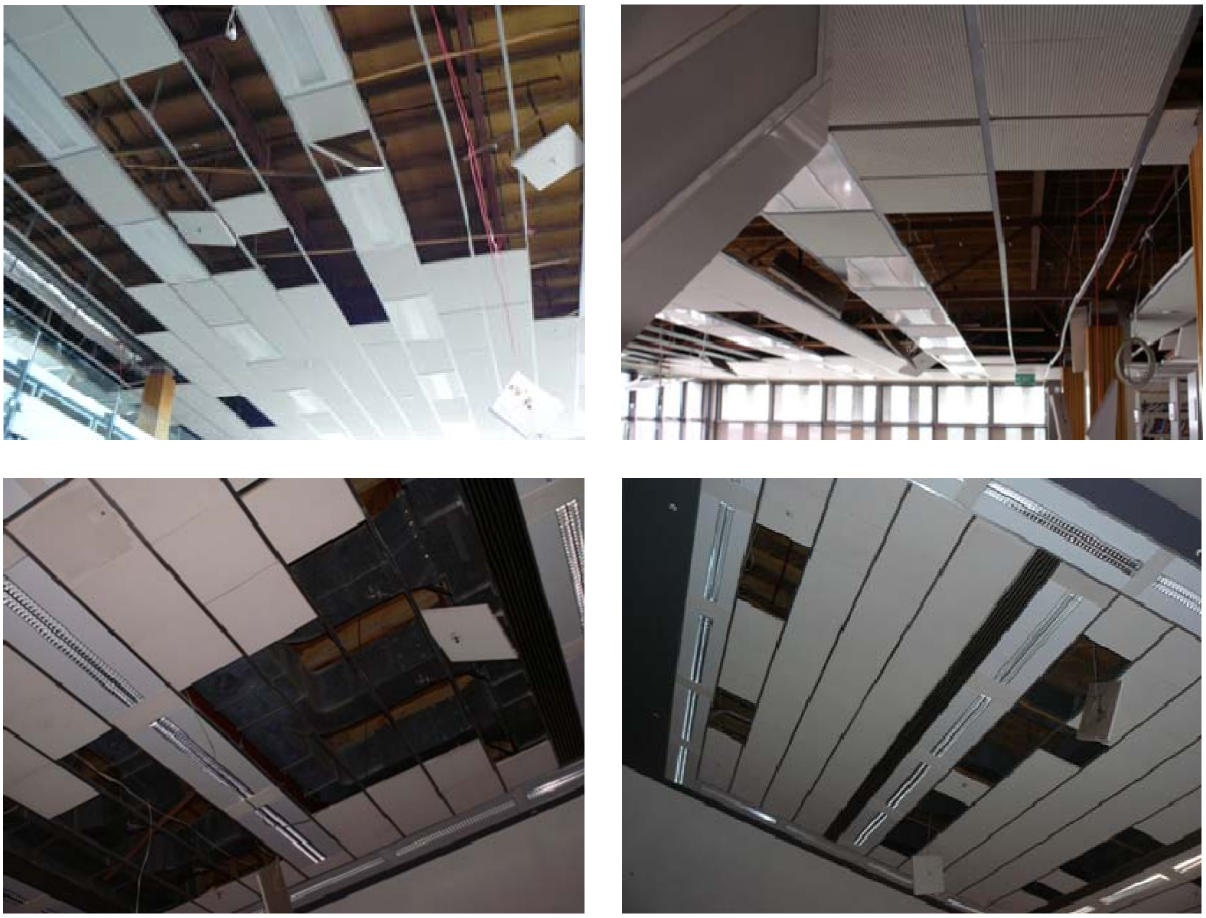


Figure 2.4.12 Damage caused by grid spreading causing the panels to fall from the ceiling (Photos: T Abu and Hush Interior Ltd)

2.4.5 Other Types of Damage

Damage to suspended ceilings can also result from elements that are connected to the ceiling but should be independent of it. Common examples of this are timber or steel framed bulkheads and partitions. Partitions, in particular, should not rely on lateral support during an earthquake from the suspended ceiling. Unless it is considered explicitly in design, this type of construction applies extra force into the ceiling and can result in damage (see Fig. 2.4.13 for example).

In some cases damages were attributable to the interaction between ceilings and bulkheads. Bulkheads hanging from tiled ceilings dropped when the ceiling perimeter detail failed. In a building it was found that during renovation, bulkheads were removed but no bracing was put in place to take the ceiling load. In such cases, the partition loads change during alterations and ceiling line load increases, hence it is advisable to provide bracing to take care of the increased load on ceiling.

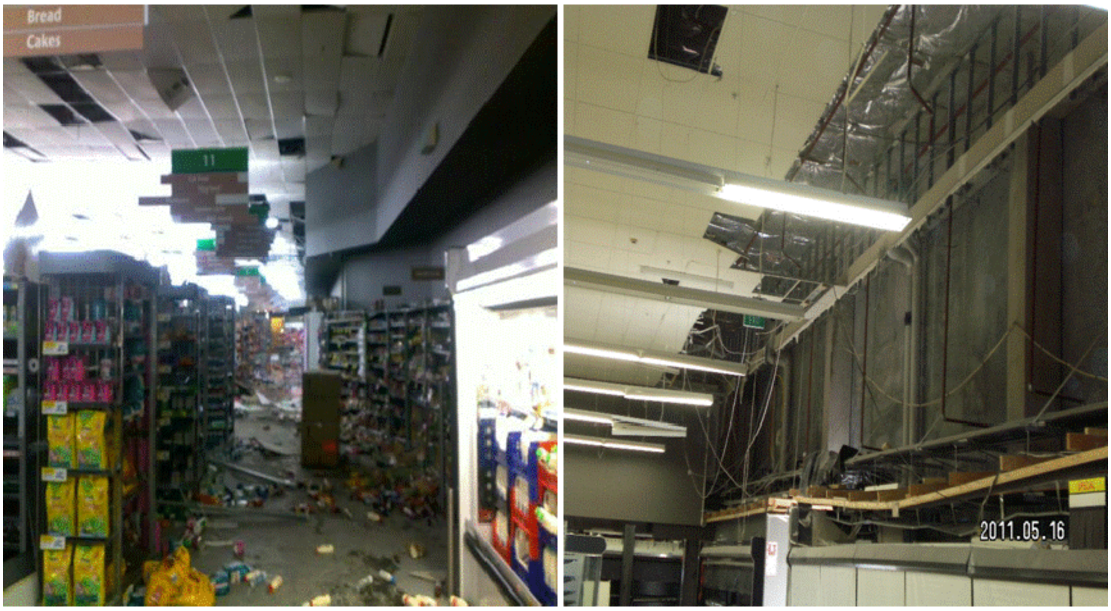


Figure 2.4.12 Damage due to interaction between bulkheads and ceiling (Photos: K Hogg)

The above forms of damage can occur simultaneously during an earthquake causing widespread damage to a single ceiling system. Some examples of this are shown in the figures below.

Figure 2.4.13 Widespread damage of a single ceiling (Photos: R Dhakal, T Abu and Hush Interior Ltd.)

Figure 2.4.14 Widespread damage of a single ceiling (Photos: R Dhakal, T Abu and Hush Interior Ltd.)

2.4.6 Summary of Observations

Some of the observations are:

- Compression failure generally occurred closer to the perimeter, than the middle, of two way grids.
- More compression failures, or more severe failures, were observed in larger ceilings.
- Ceilings with heavier tiles/panels were observed to undergo more severe damage compared to those of the same size with lighter tiles in the similar shaking region.
- The observed damage in ceilings was very severe in many cases and it was only a coincidence that nobody was killed due to ceiling failure in these earthquakes. As the recent Japan earthquake has proved, heavy ceiling tiles falling from several metres can easily be fatal. Even in rooms without heavy tiles, cross members bent down like skewers, causing a major hazard for anyone egressing from the building.
- Interactions with services above the ceilings, partition walls and bulkheads were found to cause many ceiling failures. This was most significant when services/walls were not constrained and interacted with the ceiling.
- Some ceilings were replaced several times during the earthquakes indicating that they were not able to sustain the levels of shaking imposed.
- Poor performance often resulted from poor system installation. That is, often the ceiling system was not installed to AS/NZS2815. An example of bad practice is given below where a seismically designed replacement ceiling is installed after the February earthquake. However, the partition contractor has braced the glazed wall to the A/C unit.

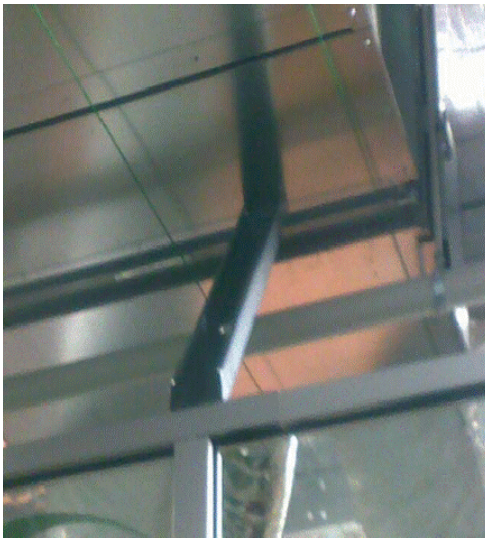


Figure 2.4.14 Poor practice: Glazed wall braced to the A/C unit (Photo: K Hogg)

2.5 Recommended Practice - Design and Installation Guidance

In relation to the issues raised in Section 2.3 and the observed ceiling damage described in Section 2.4, the following “recommended practice” guidelines are proposed for ceiling systems, which include the ceiling itself, and all interacting elements:

Technical recommendations:

1) All ceiling systems be designed and installed following appropriate standards.
    a. NZS 1170.5:2004: for calculating the seismic demand on ceiling systems
    b. NZS 4219:2009: for designing services and their interaction with the ceiling
    c. AS/NZS 2785:2000: for designing ceilings

2) While using NZS1170.5 Table 8.1 to determine the design limit state, ceilings shall be placed in the following categories:
    i. P2 – Auditorium ceilings
    ii. P3 – Suspended ceilings with one way or two way grid systems
    iii. P4 – Suspended ceilings with heavy (more than 10kg) tiles
    iv. P7 – All other ceilings

    This replaces the recommendations in NZS1170.5 Table C8.1.

Table 2.1. Summary of Related Categories from NZS1170.5 Table 8.5

Category	Criteria	Part Ris Factor	Limit State
P2	Part representing risk to a crowd of >100 people in structure	1.0	ULS
P3	Part representing risk to an individual life within structure	0.9	ULS
P4	Part necessary for evacuation and life safety systems	1.0	ULS
P7	All other parts	1.0	SLS1

3) If interstorey displacements are not available from the design engineer and they are not computed explicitly, then:
    a. the minimum ULS drift ratio demand shall be of 2.5% at all stories
    b. the minimum SLS1 drift ratio demand shall be one third of the ULS value above

4) If floor accelerations are not available from the design engineer and they are not computed explicitly, then:
    a. the minimum ULS acceleration demand shall be 3 times the zone factor, Z, at all levels
    b. the minimum SLS1 acceleration demand shall be one third of the ULS value above

5) Ceiling connections shall be designed for a ductility of 1.25 for the ULS unless it is shown that another value is satisfactory.

6) Equipment with mass less than 5 kg (per m^2 of ceiling area) may be supported by the grid of the suspended ceiling system, and those exceeding 5 kg mass shall be supported independently of the ceiling. This recommendation overrides the existing statement in Clause 5.13 in NZS 4219:2009.

Regulatory recommendations:

7) If the installer is required to provide the ceiling system design, the following is to be provided by the building design engineer in the tender documents:
   a. floor accelerations for SLS and ULS actions.
   b. interstorey displacements for SLS and ULS actions.

8) Design and detailing of ceilings and components interacting with ceilings (giving due consideration to the interactions between them) is to be undertaken under the supervision of a Chartered Professional Engineer.

9) Construction of ceilings and services is to be undertaken by a trained and experienced installer.

10) Construction review is to be undertaken by a suitably qualified engineer; preferably the designer.

11) Structural modifications to components interacting with ceiling systems shall consider the implication of the modification to the ceiling.

12) For quality control, the building consent documentation is to define who is responsible for the following design, installation and certification/review processes:
   a. Producer Statement 1 (PS1) – Design
   b. Producer Statement 3 (PS3) – Construction
   c. Producer Statement 4 (PS4) – Construction Review

   If the ceiling is a design and supply item the tender documents shall spell out the requirement for each of these.

13) AS/NZS 2785 needs to be updated in line with the seismic demand stated in NZS1170.5.

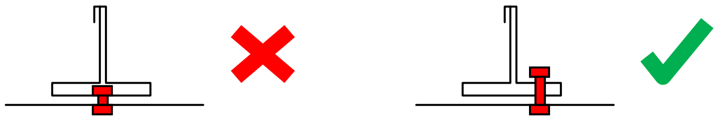
14) Training programmes and qualifications need to be made for installers.

It is likely that some clients will request buildings and non-structural components, such as ceilings, be designed for greater performance than that associated only with life safety. Also insurance companies may require this in Christchurch and elsewhere as a result of the significant damage resulting from the Canterbury earthquakes. Improved performance may be obtained by designing components greater levels of ground shaking, such as the NZS1170.5 Maximum Considered Event (MCE) which is defined as being 1.8 times the NZS1170.5 Ultimate Limit State (ULS) Event. Alternatively, it is possible to design for higher serviceability criteria. In addition damage may be minimized by using the Best Practice details in the following pages.

An example of a design of a regular ceiling follows. Using of P4 from Table 8.2 with R_p = 1.0, ULS design, NZS1170.5 is followed and the part connection ductility factor is 1.25. Generally the ceilings will be of short period, so the part spectral acceleration shape value is 2.0 and the floor height coefficient is 3.0. For example, if the peak ground acceleration, PGA, is 0.3g, then the response coefficient is approximately 0.3g * 3 (for height) * 2 (for part S_a) * 1.0 (for risk) * 0.8 (for ductility) * weight = 1.44 of the weight.

2.6 Best Practice Detailing Guidance

a) Rivets should be placed on the side of a T-section, to go through two, rather than one piece of steel which is the case if they are placed in the middle.



(a) Central rivet (b) Rivet in side

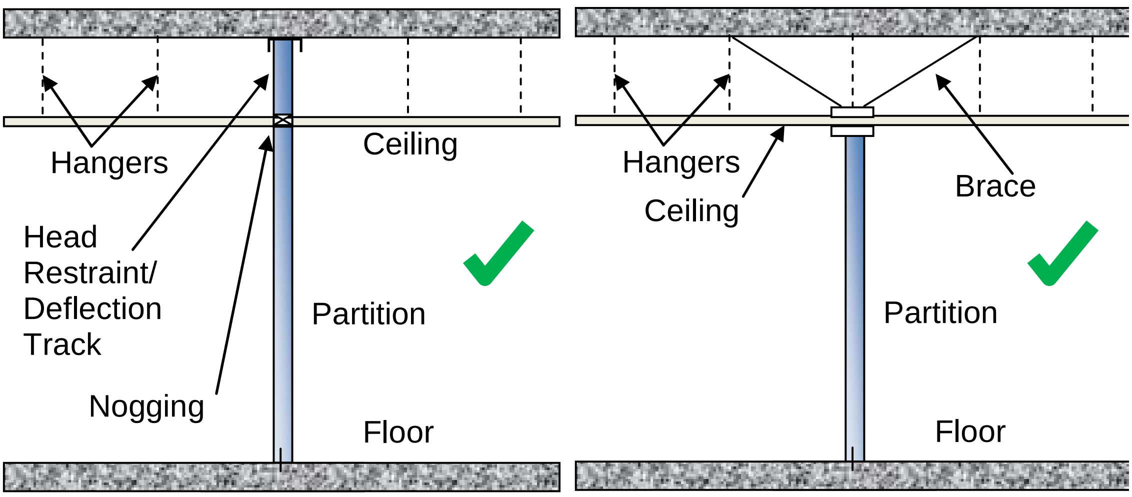
Figure 2.6.2. Locations of Rivets in T-Rails (elevation)

b) Rivets be placed with sufficient distance to the end of T-rails so they can develop their full strength

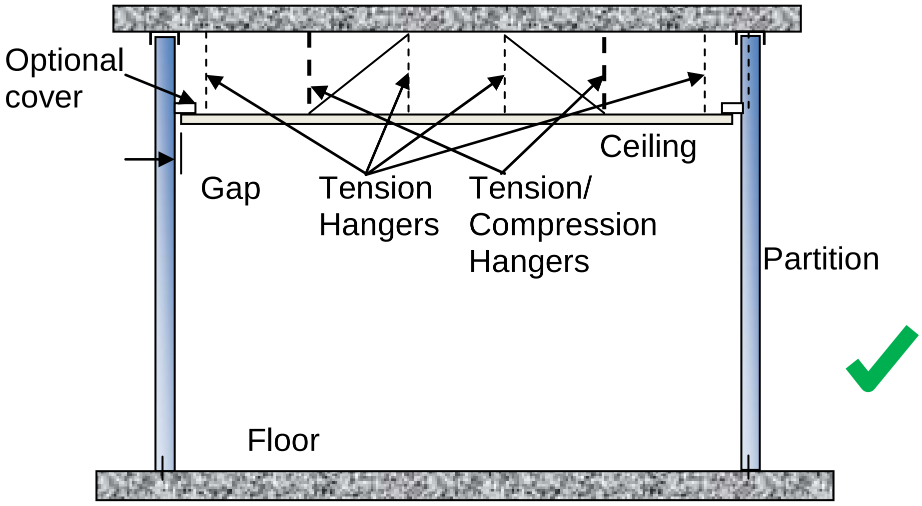
(a) Rivet too close to end (b) Rivet close to end

Figure 2.6.2. Locations of Rivets in T-Rails (plan)

c) It should be clear whether partitions are bracing the ceiling, ceiling is bracing the partitions, or whether they are independently braced as shown in the Figures below. A head-restraint can be used at the top of the partition. This restrains the partition in the out-of-plane direction. Not that hangers at which go to the bottom end of a vertical brace must resist both compression as well as tension, as shown in Figure 2.6.3, in order to prevent large ceiling swinging displacements.



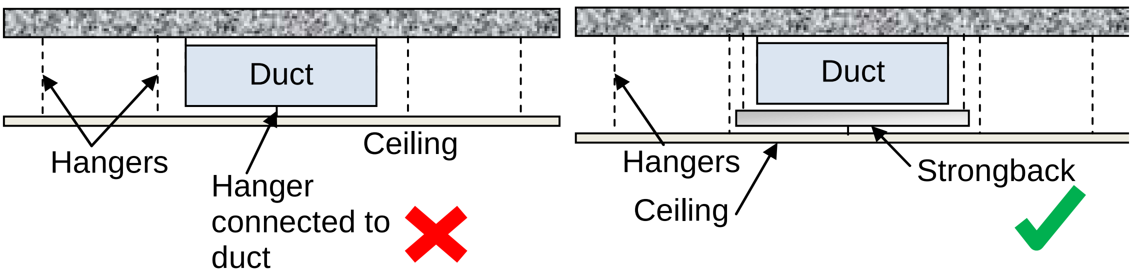
(a) Partition Bracing Ceiling (b) Ceiling Bracing Partition



(c) Ceiling and Partition Independently Braced (Floating ceiling)

Figure 2.6.3 Some Examples of Ceiling/Partition Configurations

d) Providing sufficient space for bridging or strongbacks under ducts



(a) From Duct (Undesirable)

(b) From Strongback (Desirable)

Figure 2.6.4 Suspension of Ceiling Hangers around Ducts

e) Using flexible chords for fire sprinklers, or provide sufficient gaps in the ceiling so that large forces are not imposed.

f) Lighter tiles should be used rather than heavy tiles as these ceiling systems are likely to sustain less earthquake damage.

2.7 References

ASTM International, 2009, ASTM C423-09A – Standard Test method for sound absorption and sound absorption coefficients by the reverberation room method, PA, USA.

ASTM International, 2011, ASTM E1414-11 - Standard test method for airborne sound attenuation between rooms sharing a common ceiling plenum, PA, USA.

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Standards Australia, 1999, AS/NZS 1530.3:1999 - Methods for fire test on building materials, components and structures - Simultaneous determination of ignitability, flame propagation, heat release and smoke release, Sydney, Australia.

Standards New Zealand, 1992, NZS 4203:1992 General Structural Design and Design Loadings for Buildings, Wellington, NZ.

Standards New Zealand, 2000, AS/NZS 2785:2000 Suspended Ceiling – Design and Installation, Wellington, NZ.

Standards New Zealand, 2002, NZS 1170.1:2004 Structural Design Actions Part 1: Permanent, Imposed and Other Actions, Wellington, NZ.

Standards New Zealand, 2004, NZS 1170.5 Supp 1:2004 - Structural Design Actions Part 5: Earthquake Actions – New Zealand: Commentary, Wellington, NZ.

Standards New Zealand, 2004, NZS 1170.5:2004 Structural Design Actions Part 5: Earthquake Actions – New Zealand, Wellington, NZ.

Standards New Zealand, 2009, NZS 4219:2009 Seismic Performance of Engineering Systems in Buildings, Wellington, NZ.

Additional references related to ceiling seismic design/performance:

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3 Facades and Cladding Systems

3.1 Background

Facade systems can be grouped by two main types; claddings and infills. Claddings are attached such that they are positioned outside the structure, whereas infills are constructed within the frame. The focus of this section is upon claddings. Infills are included within the partitions section.

Claddings often incorporate stiff, brittle materials such as glass and concrete. Precast concrete panels have been the most popular cladding material used in new non-residential buildings in New Zealand over the past decade (Page, 2008).

The weight of a cladding can be described as being light, medium or heavy according to NZS 3604. Light cladding is defined as not having a mass exceeding 30 kg/m^2. Medium cladding is defined as having a mass exceeding 30 kg/m^2, but not exceeding 80 kg/m^2. Heavy claddings can be defined as having a mass exceeding 80 kg/m^2. There is no equivalent definition for structures that are not timber-framed. However, in general, heavy systems can be thought of as precast concrete, stone or brick veneer cladding systems. The remaining systems can be considered as light-medium weight cladding.

Apart from weight, the most important difference between these two groups (heavy and light) is in how they are attached to the structure. The typical connection method for heavy cladding consists of a bearing and movement connection, as shown in Figure 3.1.1. The fixed bearing connections support the claddings gravity loads, while the movement connections allow relative movement between the cladding and the structure while accommodating the out-of-plane forces on the panel, including wind. This flexible connection could also accommodate movement by sliding or rotating.

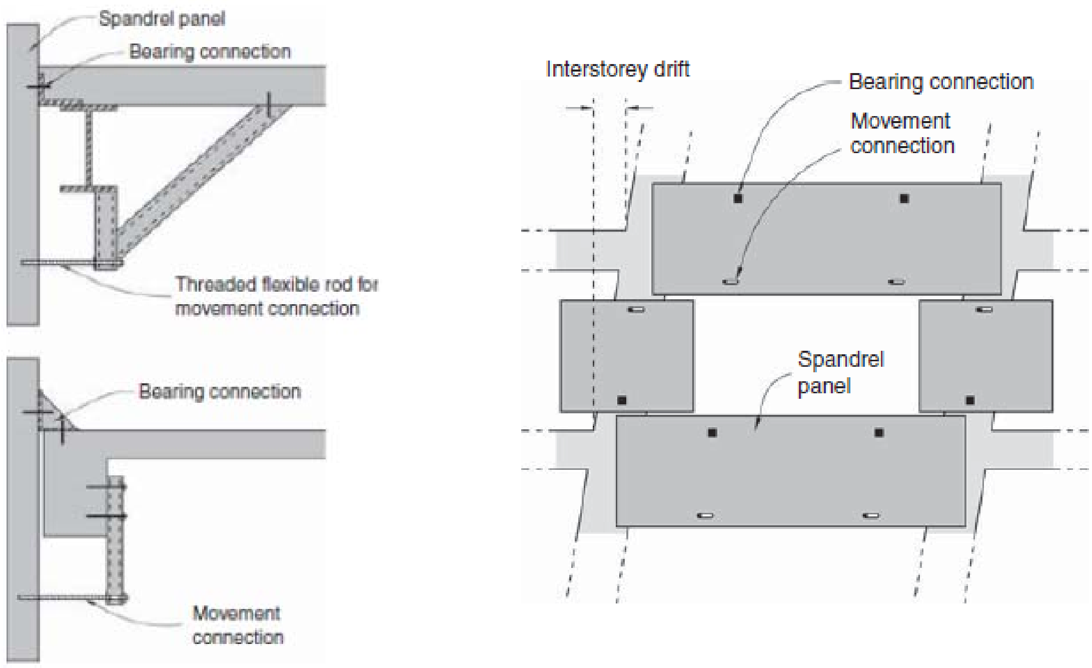


Figure 3.1.1 Spandrel panel showing bearing and movement connections when attached to steel and concrete frames (left), one approach to isolating spandrel and column panels from interstorey drift (right), (Charleson, 2008)

Lightweight claddings are generally fixed to the structure with connections that do not allow movement. Therefore, the cladding must be able to accommodate relative displacement within itself. This is typically achieved using the flexibility of the cladding and by providing gaps around glass or other stiff objects to allow movement. Rubber gaskets are used to hold the panes of glass in place and keep the system watertight, as shown in Figure 3.1.2.

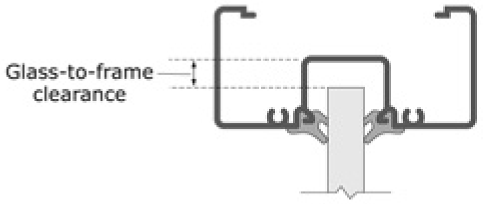


Figure 3.1.2 Rubber gaskets used to allow movement and keep system watertight

The responsibility for the design of a building's facade varies depending on the system used. Typically a Structural Engineer will not have responsibility for the facade as it is a secondary structural element and these often become the responsibility of the Architect. A Structural Engineer may be employed to design precast or specific secondary structural elements where this forms part of their brief and defined scope of work. This is more common for heavy claddings where the design of fixings requires particular attention and engineering calculations. If the design is the responsibility of the Architect then they usually transfer responsibility for design, performance and durability onto a successful Subcontractor through open tendering by Specification Conditions of Contract. This process is more common for light-medium weight systems.

Generally the supervision and inspection of the installation of the facade is done in-house. If no standard installation specifications exist then specifications are individually written for each project. Quality assurance that the installation meets the specifications is usually done in-house.

3.2 Current Design and Installation Specifications

In terms of the seismic design of cladding systems, the predominant document that is used is NZS 1170.5:2004 – Structural design actions - Earthquake Actions. The Standard requires all permanent, non-structural components and their connections to be considered as a 'Part'. A 'Part' must also weigh more than 10 kg and be able to fall more than 3 m onto a publicly accessible area to be considered. There is no distinction made between heavy and light facades, so their treatment is the same. Some key specifications from this document are given below:

NZS1170.5:2004 Structural Design Actions Part 5: Earthquake actions – New Zealand covers facades in Section 8 "Requirements for Parts and Components"

- Clause 8.1.1 says: "All parts of structures including permanent, non-structural components and their connections, and permanent services and equipment supported by structures, shall be designed for the earthquake actions in this Section."
- The commentary (in Appendix CA) says "Building part … an element that is either attached to, and supported by the structure but is not part of the structural system, would be cladding panels, machinery and architectural fitments …"
- In Clause 8.1.2, the buildings parts are divided into seven categories, among which facades feature in the following categories (specified in the Commentary Table C8.1):

  Cladding Panels: P.1 (Part representing a hazard to life outside the building)
- Cladding panels (P.1) are designed for ultimate limit state (ULS)(Table 8.1)
- Clause 8.2 infers that for a cladding at level i of a building, the design response coefficient is the horizontal acceleration coefficient $C_P(T_P)$ derived for that floor which supports the cladding, and is calculated as:

  $C_P(T_P) = C(0)\ C_{Hi}\ C_i(T_P)$

  where C(0) is the site hazard coefficient for T=0 determined using the values for the model spectrum and numerical time history methods, C_{Hi} is the floor height coefficient for level i calculated as a function of the height as specified in Clause 8.3 (it linearly increases from 1 at ground floor to 3 at a height of 20% of building height or 12m, whichever is less), T_P is the period of the cladding and $C_i(T_P)$ is the spectral shape factor of the cladding given in Clause 8.4 which is equal to 2.0 for periods less than or equal to 0.75 s, 0.5 for periods longer than or equal to 1.5 s and varies linearly in between.
- The horizontal and vertical design forces are then calculated as a product of the horizontal/vertical design action coefficient, the risk factor R_P (which is 1.0 for cladding), the cladding horizontal/vertical response factor C_{PH}/C_{PV} which depends on the ductility and varies between 1 (for ductility equal to 1) and 0.45 (for ductility 3 or greater), and the weight of the cladding, W_P. The horizontal and vertical design forces are not allowed to be greater than $3.6W_P$ and $2.5W_P$, respectively (Clause 8.5).
- Non-ductile connections for claddings shall be designed for seismic actions corresponding to a ductility of 1.25 (Clause 8.7.1). Non ductile connections include expansion anchors, shallow chemical anchors or shallow cast-in-place anchors and not engaged with the main reinforcement.
- Other connections may be designed for a greater ductility value where the specific detailing can be verified to sustain not less than 90% of their design action effects at

a displacement greater than twice their yield displacement under reversed cyclic loading (Clause 8.7.2).

- The following table shows suggested ductility and deformation limits for facades (Table C8.2)

Table 3.1 Suggested ductility and deformation limits

Cladding Type	**Ductility**	**Indicative deformation limits for onset of damage**
External wall or prefabricated cladding panel (lightweight – including metal faced, fibre-cement, tile)	3	H/200 (face loading), H/300 (in-plane)
External wall or cladding panel (precast concrete)	3	H/300 (face loading), H/400 (in-plane)
External wall or cladding (masonry – including glass blocks)	2	H/300 (face loading), H/600 (in-plane)
Masonry veneer attached to external wall	2	H/200 (face loading)
Curtain wall system (with framing elements)	2	H/150 (face loading), Clearance in frame (in-plane)
Structural glazing system	1	H/150 (face loading), Clearance (in-plane)

- Where H is in the height between supports. If H is the inter-storey height, which is a typical configuration, then the inter-storey drift can be found by letting H=1, e.g. H/200 = 0.5%
- The code states that since connections are frequently the “Archilles Heel” for building parts under earthquake actions; their ductility is taken to be 1.25, unless more rigorous detailing is employed.
- Connections for cladding required to accommodate inter-storey drift between points of attachment may be made using sliding connections, or by ductile bending of steel components. Detailing of connections shall be such as to preclude concrete fracture, anchor withdrawal, or brittle fracture at or near welds.
- Verification allowing the use of higher ductility values can most readily be achieved by testing, which has the added benefit of checking buildability issues on a prototype.
- Guidance on serviceability limits for the design of wall elements under the actions from earthquakes are given in Table 3.2 below (Table C7.1). This table identifies deflection limits related to earthquake actions with an annual probability of exceedance of 1/25 years beyond which problems have been observed.
- Clause 7.5.2 says that for Serviceability limit state (SLS) the design inter-storey horizontal deflections “shall not be greater than any separation provided to avoid contact between adjacent parts of the structure, or between the structure and its parts and shall be limited so as not to impair their function nor that of other structure components.”
- NZS 1170.5 also specifies that the interaction between the facade and the structure only needs to be taken into account if the mass of the facade is in excess of 20% of the combined mass of the facade and the primary structure. If this is the case then a special study needs to be undertaken.

Table 3.2 Acceptable serviceability limit state criteria for earthquakes

Element	Phenomenon controlled	Serviceability parameter	Element response
Walls – metal cladding (in plane)	Weathertightness (tearing at fasteners from inplane racking)	Relative residual deflection Top to bottom	Height/500
Walls – concrete masonry or brick (in plane)	Cracks – sufficiently visible to need repair Weathertightness(base/top cracks from out-of-plane rotation)	Relative racking deflection Top to bottom Relative residual deflection Top to bottom	Height/600 Height/600
Walls – concrete and masonry (face loading)	Weathertightness (base/top cracks from out-of-plane rotation)	Relative residual deflection Top to bottom	Height/400
Windows, facades, curtain walls, fixed glazing systems	Facade damage, broken glass	Relative racking deflection Top to bottom	Span/250 2 x glass clearance

The commercial cladding industry also completes rigorous structural, air infiltration, weathering and seismic laboratory testing to substantiate that their cladding solutions comply with New Zealand Standards or project specific higher performance requirements. This includes substantiation of performance after Serviceability Limit State (SLS) inter-storey seismic movement tests and accommodation of Ultimate Limit State (ULS) inter-storey movements without endangering human life. Some key specifications from this document are given below:

AS/NZS 4284:2008 Testing of building facades

- AS/NZS 4284 sets out a method for determining the performance of a representative building facade under simulated loading conditions. It may be applied to all types of facades, including low-rise, high-rise, commercial, industrial and residential buildings. Tests include;
    - Structural test at serviceability limit state
    - Air infiltration test
    - Water penetration test by static pressure
    - Water penetration test by cyclic pressure
    - Building Maintenance Unit (BMU) restraint test
    - Strength test at ultimate limit state
    - Seismic test
    - Seal degradation test
- The seismic test displaces the facade in plane using a racking apparatus. The number of cycles, period and displacement used in the test is based on the geographical region and is determined by the structural designer. It is common place for the testing order to first be serviceability limit state seismic test, then cyclic water penetration test and finally ultimate limit state seismic test. The test requires that the facade does not collapse in any way. Collapse of the facade can include: disengagement of any framing member or facade panel, failure of any fixings, repeated breakage of glass (glass may only be replaced once before facade is deemed to have collapsed).

The following standards also contain information relating to the design and installation of facade systems:

- NZS 3101 Concrete Structures
- NZS 3604 Timber Buildings
- NZS 4230 Design of reinforced masonry structures

In regards to quality assurance, the following producer statements are used to help establish compliance with the Building Act and Building Code.

PS1 - Design

- This is provided by the CPEng designer of the element. For elements designed by the producer, the producer provides the PS1 as the designer of the products. Otherwise, the project's engineer is the designer and therefore facade elements form part of the items listed with other specifically designed elements of the whole structure.

PS2 - Design peer review

- A requirement of more complex structures. Not usually required for standard product elements or non-complex structures.

PS3 - Manufacture or Construction

- Provided by the manufacturer of the product designed in the PS1 or the contractor/builder of the building. Not necessarily an engineer. Quite often a quality manager or supervisor.

PS4 - Construction review

- This document is provided by the project's CPEng engineer once the structure has been built and he/she is satisfied the structure, including the facade elements, has been built in accordance of the designer(s)

According to the DBH, the designer has the responsibility that the movement allowances are met on any project.

3.3 Issues with Current NZ Design Procedures and Practice

Several questions arise from the transfer of design responsibility under “specific design” tender specifications:

- Who is responsible to checking compliance of the design solution with codes and substantiation that the system selected by open tender actually complies, given the lack of Engineers employed by the subcontract cladding industry?
- Does the Architect have the engineering skills necessary to supervise the engineering industry?
- Does the Structural Engineer want to get involved for insurance/reinsurance liability and responsibility issues, weathering/leaky building liability, given IPENZ policy recommendation to their Members?

Specific design facade contracts are generally selected based on low price. There is often very little supervision by Main Contractors and Consultants that products supplied comply with minimum code requirements. Traditional tender/procurement methods are inherently flawed by this process. The overseas trend in commercial construction has seen a strong move towards design/build contracts, where there is a perhaps better understanding of buildability and risk management to prevent contractual dispute, and downstream guarantee/product in service performance liability.

It has been observed that the consulting industry has struggled to identify the transition point when a commercial facade solution is required over a domestic window system, when simplistically they look similar. Subsequently cheaper domestic window systems are often used in situations that require a commercial facade solution.

The engineering selection of suitable materials for facade application needs to be carefully considered in order to manage the risk to the public. Using laminated or toughened glass reduces the risk significantly. Currently it appears there is a bias towards the use of toughened glass in glazing codes.

NZS1170.5 states in the commentary that since connections are frequently the Achilles Heel for building parts under earthquake actions that their ductility is taken to be 1.25, unless more rigorous detailing is employed. However, it also states that the suggested ductility for cladding systems ranges from 1 to 3.

The more ductile system (ductility of 3 or greater) has a lower response factor and only needs to be designed to carry 45% of the load where the ductility is equal to 1. Additionally, non-ductile connections for claddings, typically the anchorage part of the connection, are to be designed with a ductility of 1.25. What ductility the designer is supposed to use in their design for is very confusing.

There is lack of technical understanding within much of the engineering industry of façade systems - what works and why. Without understanding what product features that are required to generate performance and compliance with codes, decisions revert to the easiest selection method (low cost). This is not helped by the difficulty in comparing different systems. Consequently the pressure on contractors to use the cheapest subcontractors to be competitive has resulted in many sub-standard facade systems.

3.4 Summary of Damage Types in Canterbury Earthquakes

For various facade typologies, a range of damage states were observed. The observed damage can be grouped in terms of the following facade typologies:

- Heavy claddings, e.g. precast concrete panels, stone panels
- Light-medium weight claddings, e.g. curtain wall, lightweight panels, stick curtain, stucco, spider glazing, brick veneer, double skin, shopfront glazing

3.4.1 Facade Performance Levels

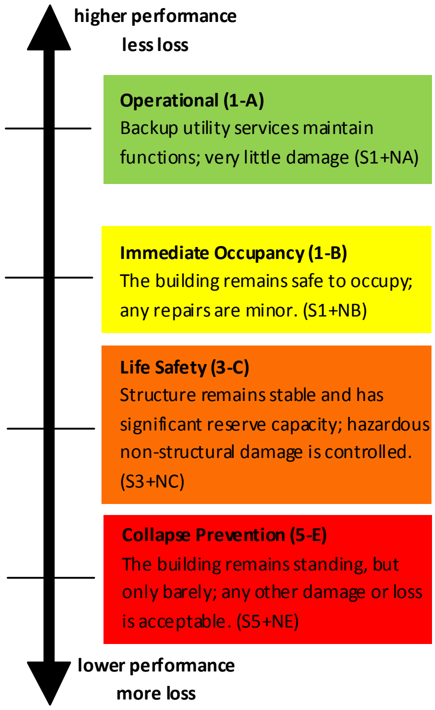
The facade performance levels (or damage states) suggested by FEMA 356 are the following: Operational, Immediate Occupancy, Life Safety and Hazards Reduced. One of the problems with using these performance levels as a means to assess damage is that they are intended for use in design. In particular, the hazards reduced level is aimed at preventing serious injury caused by large or heavy items falling. However, not all surveyed facades met this design criterion. In order to avoid the confusion, the hazard reduced performance level is herein re-named the 'High Hazard' performance level to accurately include any cases where there was a high risk of serious injury or fatality from facade damage. Figure 3.4.1 presents photographs and a graphic illustration of the different facade performance levels sustained during the Christchurch earthquake.

Figure 3.4.1 Facade Performance Levels (Baird, 2011)

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. Definitions of the performance levels that were used in the survey are described below and are based around those suggested by FEMA 356.

It is important to distinguish that the level of structural and non-structural damage can be different and hence the structural and non-structural performance levels are not necessarily the same. It is generally expected that the damage level of the non-structural components will be worse than the damage level of the structure. Shown in Figure 3.4.2 is the performance based design matrix that combines both structural and non-structural performance levels. A target building performance level consists of a selection of a structural

performance level and a non-structural performance level. The four highlighted squares represent the four target building performance levels suggested by FEMA 356.



Non-structural Performance Levels	Structural Performance Levels and Ranges: S-1 Immediate Occupancy	S-2 Damage Control Range	S-3 Life Safety	S-4 Limited Safety Range	S-5 Collapse Prevention
N-A Operational	1-A	2-A	N.R.	N.R.	N.R.
N-B Immediate Occupancy	1-B	2-B	3-B	N.R.	N.R.
N-C Life Safety	1-C	2-C	3-C	4-C	5-C
N-D Hazards Reduced	N.R.	2-D	3-D	4-D	5-D
N-E Not Considered	N.R.	N.R.	N.R.	4-E	5-E

N.R. = Not Recommended

Figure 3.4.2 Post-earthquake structural and non-structural building performance levels (FEMA 389)

Operational Performance Level

- The facade is able to support its pre-earthquake functions, although minor clean-up and repair may be required.

Immediate Occupancy Performance Level

- Damage to the facade is present but building access and life safety systems remain available and operable. Minor window breakage could occur. Presuming that the building is structurally safe, occupants could safely remain in the building, although normal use may be impaired and some clean-up required. The risk of life-threatening injury due to facade damage is very low.

Life Safety Performance Level

- Damage to the facade is present but the damage is non-life threatening. Potentially significant and costly damage has occurred to the facade but the majority of the system has not become dislodged and fallen, threatening life safety either inside or outside the building. Egress routes within the building are not extensively blocked, but may be impaired by lightweight debris. While injuries may occur during the earthquake from the failure of facade components, overall, the risk of life-threatening injury is very low. Restoration of the facade may take extensive effort.

High Hazard Performance Level

- Damage to the facade is present creating multiple falling hazards. Extensive damage has occurred to the facade with the potential to seriously threaten life safety outside the building. Widespread window breakage is likely and disconnection of components of the facade system from the structure is possible. Restoration of the facade is likely only possible with a complete replacement of the system.

The performance level of each facade system was determined according to the criteria discussed above. Overall, 64% of facade systems surveyed were deemed operational, 14% deemed Immediate Occupancy, 12% deemed Life Safety and 10% deemed High Hazard. This means that the performance of 37 facade systems was outside an acceptable level for even a very rare earthquake event as it posed a significant risk to life safety. Shown in Figure 3.4.3 is the performance composition of facades surveyed in the Christchurch CBD.

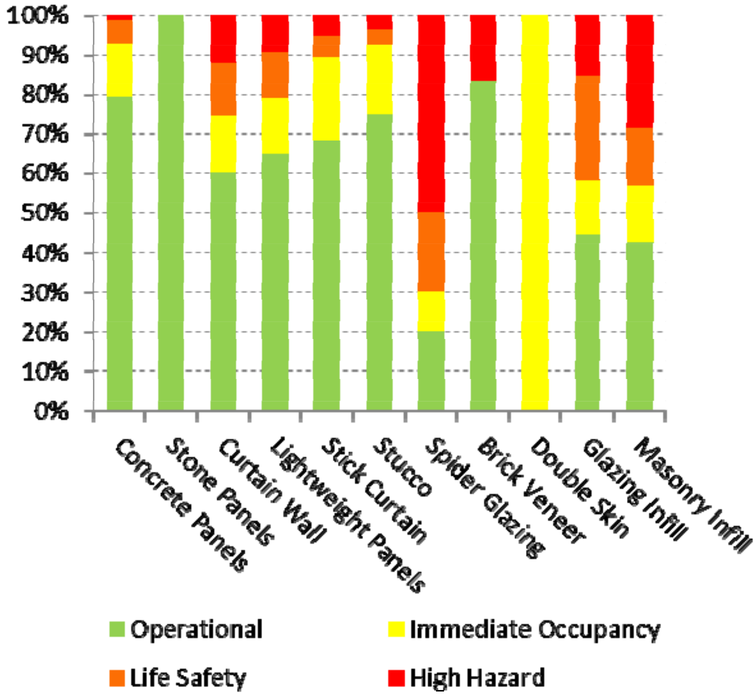


Figure 3.4.3 Facade typology composition in Christchurch CBD (left) and damage state composition (right) (Baird, 2011)

It could be concluded that heavy claddings performed better than most facade systems; with 94% of heavy claddings deemed either operational or immediate occupancy. However, it is possible a more thorough assessment of the connections from inside the buildings may lower this percentage. More importantly the possible consequence of heavy claddings falling is severe which means further attention towards their treatment is necessary.

The composition of performance levels for light-medium weight claddings varied greatly. Overall, 82% of lightweight claddings were deemed either operational or immediate occupancy, exhibiting either no damage or very minor damage such as ejected window seals or cracked glass.

A large number of high hazard cases were also observed. This was usually due to a significant portion of the glazing falling from the system. The glass damage was recorded for all lightweight cladding that contained glass. Nearly half of all glazed lightweight claddings had glazing damage and 39% presented a falling hazard. Only 60% of infill systems were deemed either operational or immediate occupancy. 17% were deemed high hazard, the highest of the facade groups.

3.4.2 Heavy Claddings

The majority of heavy claddings exhibited little to no damage. Where damage was present, it likely consisted of cracking or corner crushing. Corner crushing was most likely due to pounding with adjacent panels, as seen in Figure 3.4.4 (left). Minor damage was also observed in the form of panels having residual displacements and/or rotations. The ejection or rupture of sealing joints due to movement between panels was also common, as shown in Figure 3.4.4 (right).

Figure 3.4.4 Corner crushing of spandrel panels (left), torn polysulphide seal (right) (Photos: A. Baird)

Within the CBD only one case of panel disconnection was observed. It was the result of several spandrel panels shearing off their bolted connections and falling to the sidewalk below, as shown in Figure 3.4.5. Unfortunately the falling panels, each weighing approximately 6 tonnes, fell onto a car, killing its occupant, 57-year-old Linda Arnold.

The panels were attached to the structure by an angle which was fixed to the panel by a cast-in anchor. Horizontal slots were present in all metal angles to allow sliding of the bolt, however, upon inspection, many of these bolts had sheared off close to the bolt head.

The slotted connections should theoretically have prevented large in-plane forces being carried in the panels. However, it was observed that the bolt heads had not been able to move along the slots because their washers had been welded to the metal angle. This would have resulted in significant forces being transferred through the panels under in-plane deformation of the structure, likely leading to the shear failure.

Figure 3.4.5 Disconnected precast spandrel panels (Photos: A. Baird, J. Marshall, NZ Police)

Complete disconnections of large concrete panels were also observed in the magnitude 6.3 aftershock on June 13th 2011. The remaining connection is shown in Figure 3.4.6 (left). The connections would not have been able to provide any significant degree of movement allowance.

Frame elongation caused significant damage to the connections and panels in a multi-storey reinforced concrete perimeter frame building within the Christchurch CBD. Shown in Figure 3.4.6 (right) is a close up of the connection between the panel and the beam.

Figure 3.4.6 Connection of coffered precast panels that failed in June 13 aftershocks (left and centre), precast panel and connection damage due to beam elongation (right) (Photos: A. Baird)

Of great concern regarding the performance of heavy claddings is the difficulty in being able to observe damage to the cladding connections. The cladding can appear to be completely undamaged from outside as well as inside the building. However, once the internal linings are removed, it has been observed in several heavy claddings that the connections are

damaged or even broken. The damage is not always consistent either, making inspection of virtually every connection necessary.

3.4.3 Light-medium Weight Claddings

Light-medium weight cladding includes a broad range of facade systems. Each typology of light-medium weight cladding can also include a large range of systems. For example, the curtain wall typology includes numerous arrangements of extruded aluminium members infilled with glass or lightweight panels. Often light-medium weight cladding incorporates a large amount of glazing. They can therefore appear to look a lot more lightweight than they in fact are, with some systems (such as the double skin) containing a substantial amount of weight.

Lightweight claddings of all ages showed various levels of damage. Cracked or broken glass is usually the most obvious indicator of damage to light-medium weight cladding systems. Older systems normally provide less movement allowance for the glass and consequently were more likely to exhibit glazing damage, like that shown in Figure 3.4.7. Several buildings with older, non-seismic glazing frames were re-glazed between September and February, only to be damaged again in the February earthquake.

Figure 3.4.7 Damage to light-medium weight cladding systems (Photos: A. Baird)

Newer systems exhibited proportionately less likely moderate to severe damage. However, issues do still exist with current design and construction techniques since several lightweight cladding systems less than 20 years old were heavily damaged. For light-medium weight claddings, the difference between reaching SLS and ULS can be only a small step. This was evident by systems showing either negligible damage or significant damage with broken and fallen glass. Once the glass in the cladding is broken, SLS is surpassed and there is also a falling hazard. Managing the risk of falling glass is a difficult issue to deal with. Although most damage cases observed involved standard glass, one evident approach to try and reduce the risk of falling glass was the use of laminated and toughened glass. Using these types of glass had both positive and negative consequences.

The use of laminated glass aims to prevent the glass being able to break up and fall as sharp pieces. This was successful in most damaged laminated glass observed; however, some cases were also observed where the entire laminated pane fell from frame, presenting a significant falling hazard.

Toughened (tempered) glass is stronger than normal glass and when it is damaged it breaks into thousands of small glass fragments that present a much smaller falling hazard. Damage to toughened glass was typically observed as an empty frame and a pile of glass fragments on the footpath. Although the use of toughened glass involves accepting that the glass is going to fall if it is broken, it was clear the hazard of the falling fragments was lower than that of glass shards or entire panes.

Damage to the frame of light-medium weight claddings was difficult to distinguish from street level, so it is likely this type of damage was overlooked. However there were observed cases of frames being bent and warped, as well as one case where the glass has punctured through the frame itself. Failure of the frame was rare, with only one curtain wall system having a large-scale failure. This involved multiple sections of a curtain wall system completely detaching from the building, as shown in Figure 3.4.8. The entire aluminium frame and glazing along one side of the building at the second floor fell to the ground. Closer inspection showed that the aluminium frame was screwed into a wooden sub-frame and the failure was a result of the screws both shearing off and tearing out of the wood.

Figure 3.4.8 Disconnected cladding systems (Photos: A. Baird)

A lot of heavy damage was observed in spider glazing, as can be seen in Figure 3.4.9. Spider glazing is a reasonably modern system so it would be expected that it should have performed better than other systems, however this was not the case. It appeared that damage originated around the ‘spider’ that holds each glass pane, likely a result of the ‘spider’ creating stress concentrations in these regions due to the restraint of the connection to the structure.

One of the recently installed spider glazing systems was designed to allow ULS seismic inter-storey displacement of +/-50mm. The actual measured inter-storey displacement during the February 22 earthquake was 220mm, over four times the structure’s design level displacement. The amount of movement a spider glazing system can accommodate is not large (50 mm is near the limit of a spider aesthetic system) and this was apparent by the amount of damage observed.

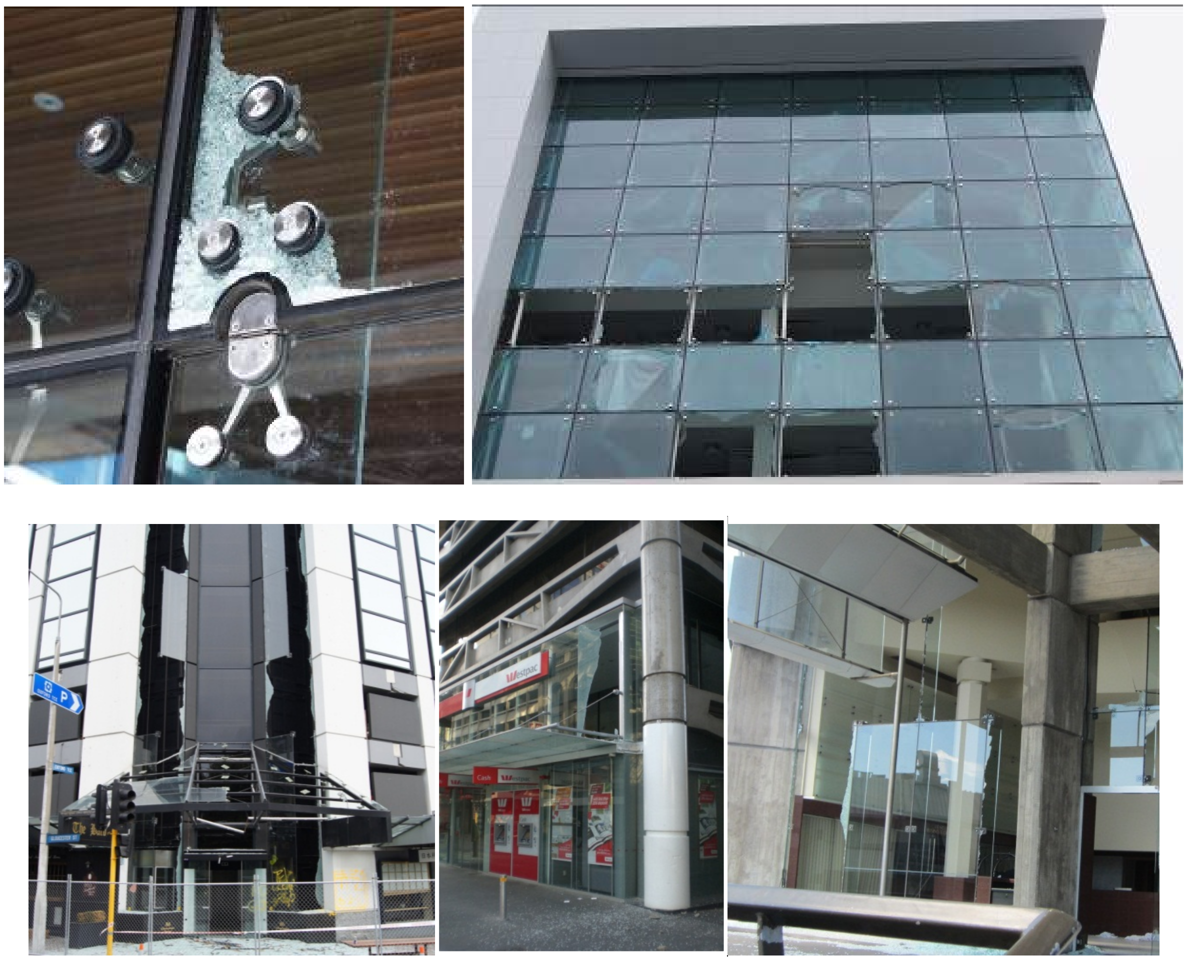


Figure 3.4.9 Spider glazing damage (Photos A. Baird)

Lightweight claddings of all ages showed various levels of damage, although older systems clearly were more likely to be heavily damaged. The majority of facade systems that were in undamaged lightweight facades were less than 20 years old. However, issues do still exist with current design and construction techniques since several facade systems installed in the past 24 months were heavily damaged due to lack of appropriate tolerances.

Inadequate movement allowances for glazing was likely the cause for the majority of glazing damage. Several buildings with non-seismic glazing frames were re-glazed between September and February, only to be damaged again.

Glazing infill is a common facade typology, particularly in older buildings. Since older glazing frames typically provide little movement allowance, a large percentage of glazing infill suffered heavy glass damage, as shown in Figure 3.4.10. Modern systems with seismic heads, mullions, etc. which are able to provide larger amounts of movement showed very little or no damage.

Figure 3.4.10 Infill glazing damage (Photos A. Baird)

Many glazed shopfronts performed very poorly. This typology consists of large self-supporting panes of glass connected to adjacent panes with sealant alone. The glass gains its out-of-plane strength through perpendicular flanges of glass which slot into top connections. However the connections are not securely fixed (possibly to allow for in-plane movement during an earthquake) and as such is able to warp or force its way out of the top connections, tipping onto the ground, as shown in Figure 3.4.11.

Figure 3.4.11 Shop-front glazing damage (Photos A. Baird)

3.5 Recommended Practice - Design and Installation Guidance

In relation to the issues raised in Section 3.3 and the observed facade damage described in Section 3.4, the following recommended guidelines are proposed for the design and installation of facade systems:

Technical guidelines:

- Ductility of connections must be proven
- Connection design considers hierarchy of strength so brittle failure mechanisms are avoided as shown in Figure 3.5.1
    - Anchorage/fixing should <u>never</u> be the weakest part of the connection
    - This needs to take into account the possible strain hardening of the connection
    - A level of design over-strength may be necessary to further avoid this failure mechanism
    - It is preferred that damage to the cladding panel occurs before disconnection of an undamaged panel

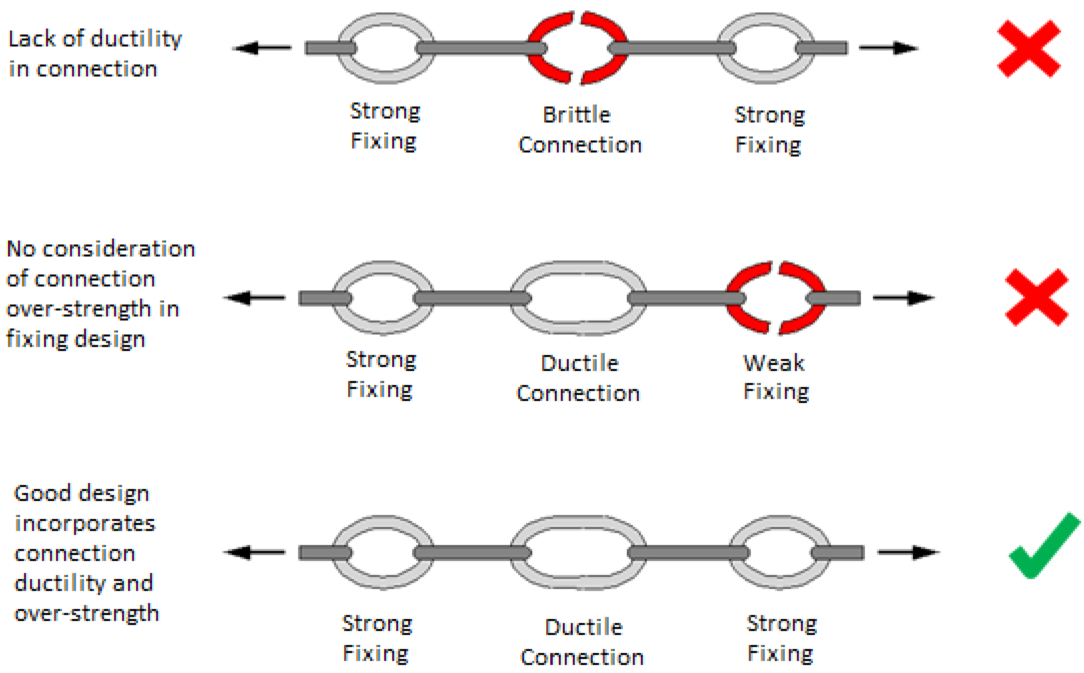


Figure 3.5.1 – Extension of capacity design or hierarchy of strength principles (“weakest link of the chain” concept) to the design of facade systems

- Redundancy incorporated, e.g. more than four connections
- If slotted connections are used then installation needs to be inspected to ensure slot can perform as intended
    - Not over tightened
    - Not welded
    - Vertical allowance for ‘squashing’ of frame and out-of-plane movement
    - Rod is in centre of slot after installation

- Slotted connections require testing of performance and ductility beyond slot limit to ensure a ductile failure mechanism or slot must be extended past ULS drift limit
- Shopfront glazing is required to be restrained out of plane by a connection which passes through the glass, similar to spider glazing.
- If possible, bearing connection should be located at top of panel
    - In this way if the tie-back connections fail the panel is far less likely to fall from the structure

Regulatory guidelines:

- Introduction of mandatory PS1/PS3 requirements signed by a Licensed Building Practitioner for cladding contracts where the design responsibility is transferred to a specialist design/build/guarantee contractor responsible for the installed end product
- Requirement for compulsory Professional Indemnity Insurance cover where design responsibility is transferred to design/build contractors to manage design risk is also required
- Working Drawings required on every job – these are rarely provided: a fundamental requirement
- Certification by qualified Engineer (Licensed Building Practitioner) – B1, B2, E2 for façades. Prescriptive legislation required, ie make PS1/2/3/4 requirements mandatory on all specific design façade projects. Certified Practitioners will also help to manage the risk of delivering projects which perform, signing off PS3 and PS4 (Construction).
- More stringent fixing requirements for cladding which faces street-side versus that which faces inside of street block
    - A risk rating system for possible consequences of falling facade could be adopted

3.6 Best Practice Detailing Guidance

a) Slotted connections should :

  i. Have adequate horizontal movement allowance
  ii. Have adequate vertical movement allowance
  iii. Be in the centre of the slot in installation
  iv. Not be welded or over tightened

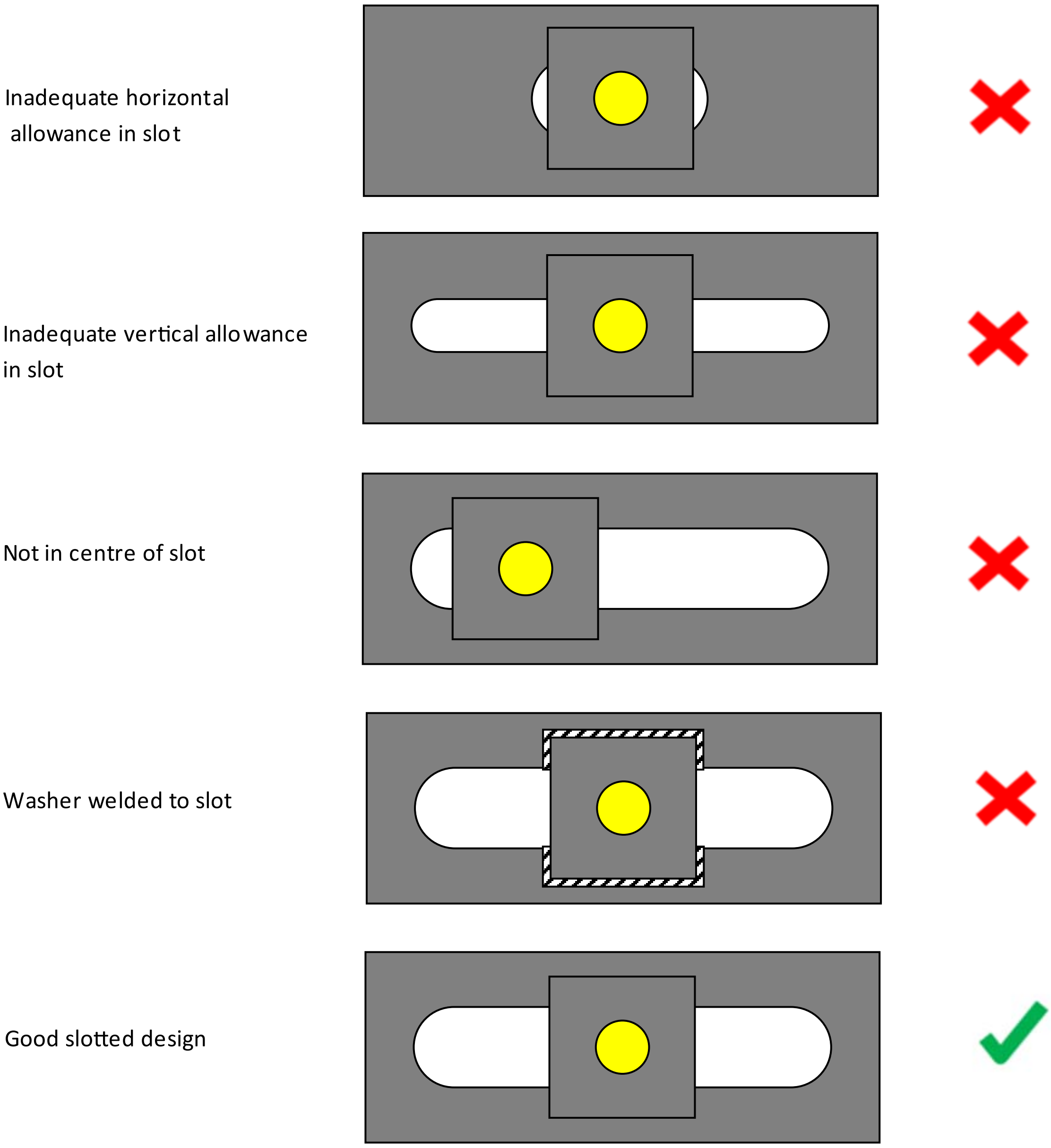


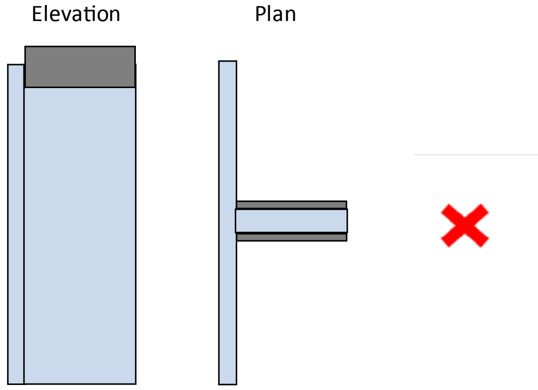
Figure 3.5.2 Slotted connection design

b) Tie-back connection should:

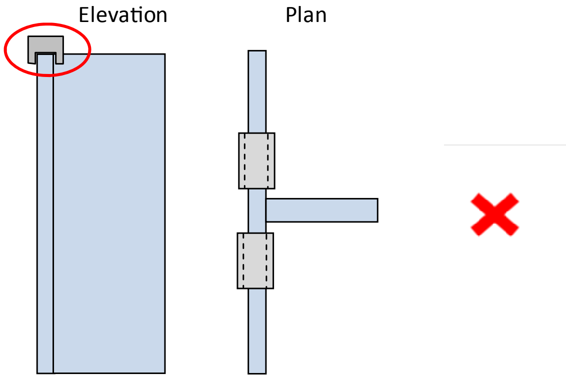
  i. Be as long as practical (less likely to face low-cycle fatigue problems, buckling check for out-of-plane may be necessary)
  ii. Be as thin as practical (4 D24's have better ductility than 2 D30's but similar axial capacity)
  iii. Have redundancy (More redundancy in 4 rods than 2)

c) Shop-front glazing should be provided adequate out-of-plane restraint

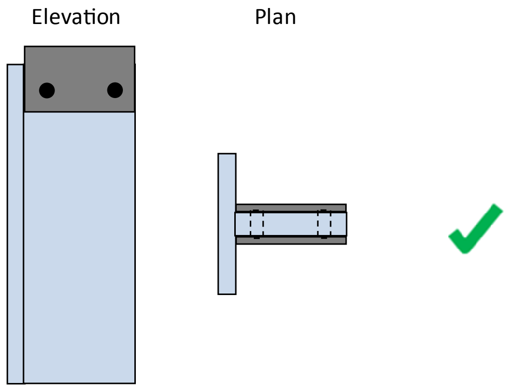
No out-of-plane fixity
at top of glazing



Inadequate out-of-plane
fixing
(Glass may easy force/
bend through short
aluminium frame)



Glass fixed to provide out-
of-plane restraint



Glass provided adequate
out-of-plane restraint

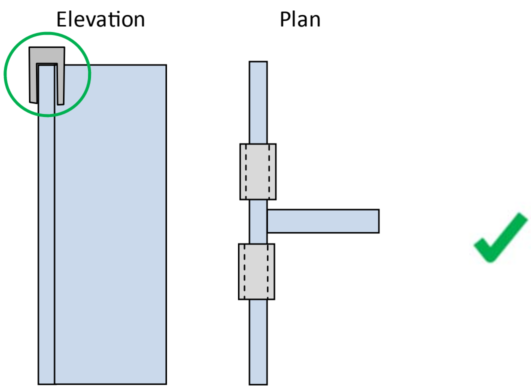


Figure 3.5.3 Shop-front glazing out-of-plane restraint

3.7 Preliminary Experimental Validation of Low-Damage Solutions

A detailed experimental programme assessing the likely damage and interaction of claddings with moment resisting frames during earthquake attack is currently underway at the University of Canterbury. Systems have been developed and tested that achieve no damage at 3.5% drift, as shown in Figure 5.2.1. The investigation aims to provide detailing recommendations to achieve nil-low damage systems. Experimental tests are also being conducted which include supplemental damping devices within the connections that do not cause any damage to the frame or cladding but utilise the differential movement between the cladding and the frame in order to absorb energy from the earthquake. In this way, the effect upon the overall structure of the earthquake is reduced also. The experimental campaign is still ongoing with plans to test light-medium weight systems and a full report with further details is under preparation (Baird et al., 2012).

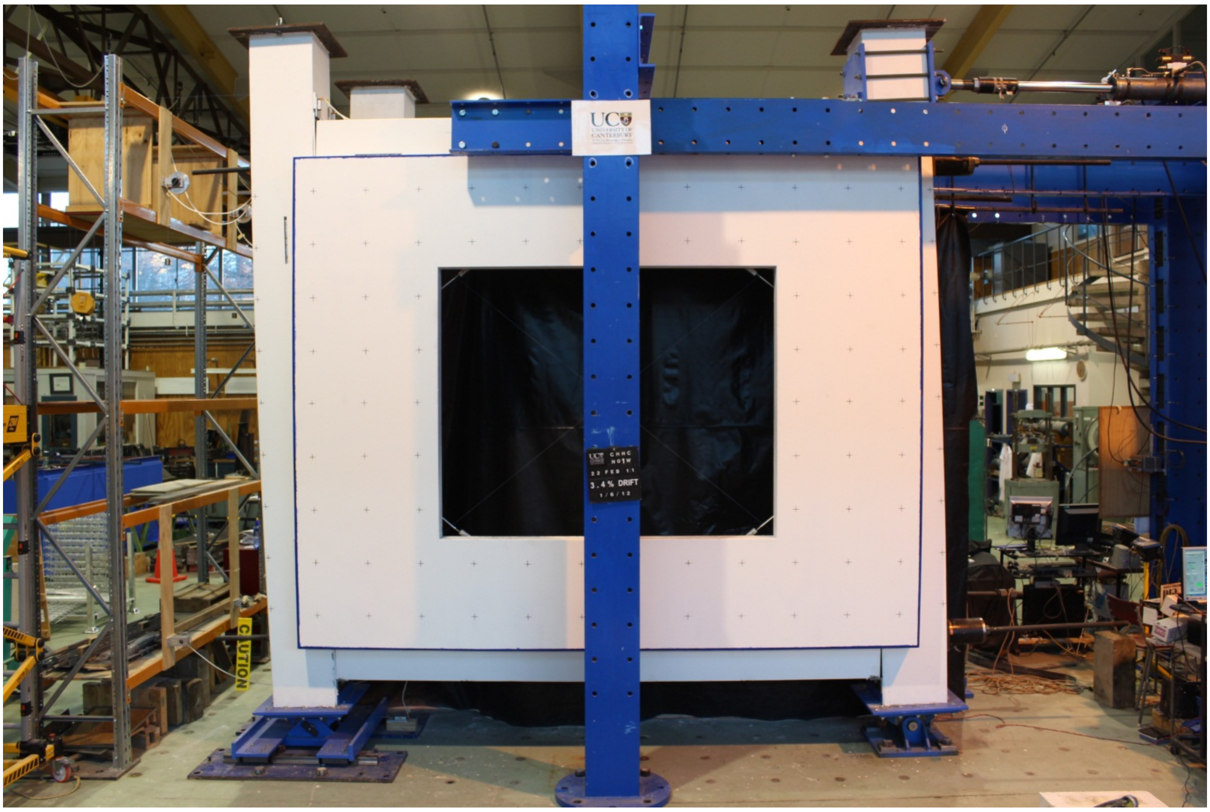


Figure 3.7.1 Precast concrete cladding at 3.5% drift level,
part of experimental testing at University of Canterbury (Baird et al. 2012)

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4 Partition Wall and Infill Systems

4.1 Background

Partition or infill walls are non-structural elements that can either form the external facade of the structure or be located within the building. They can be both constructed within the frame of the structure or as stand-alone elements. The terms “heavy” and “light” are commonly used in associating the possible impact due to the failure of each component:

- Heavy: Failure of which may threaten life safety of people. Therefore, heavy has been used in order to refer to such constitutive materials as concrete blocks, clay bricks, concrete claddings;
- Light: Failure of which may not necessarily cause life safety issues, but rather economic issues. Therefore, light refers to such constitutive materials as gypsum board drywalls

In this context, “medium” weight materials are apparently not contemplated.

The construction practice in New Zealand has been affected by the dramatic events of recent history. The Napier Earthquake in 1931 (Figure 4.1.1) showed the high seismic vulnerability of un-reinforced masonry (URM) buildings and resulted in development of the first construction standards, which favoured the use of reinforced concrete (RC) construction. Prior to the 1930s, clay brick (Figure 4.1.2a) was a common construction material both in URM (as part of the structure) and in RC buildings (as non-seismic resisting walls or infill walls). However, starting between the 1930s and 1950s, concrete blocks (Figure 4.1.2b) have been more commonly used as infills replacing clay bricks.

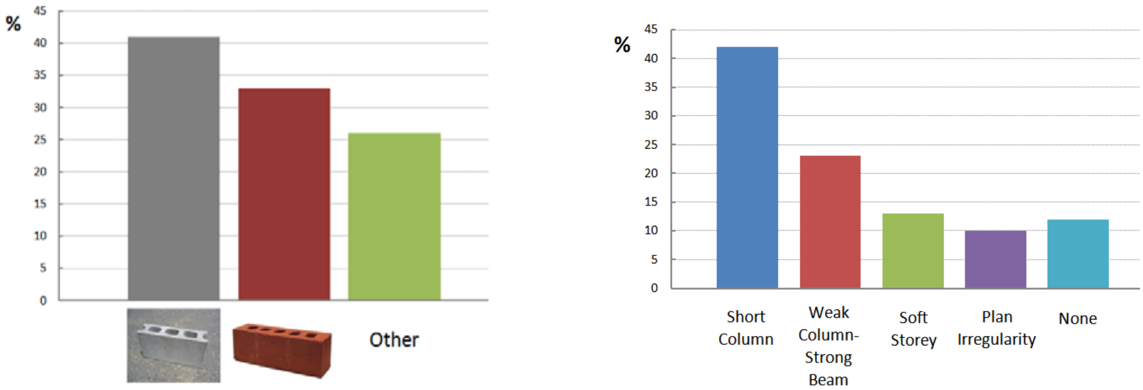
These two types of heavy infill solutions, blocks or bricks, have been mostly used as infilling walls in the external frames of buildings. For internal frames, light drywall construction methods (partition walls with light steel or timber framing, Figure 4.1.2c) have been mostly preferred since the introduction of gypsum plasterboard into New Zealand in 1927.

Figure 4.1.1 Napier Earthquake 1931 (Christchurch City Library)

a) Clay Brick (Heavy) b) Concrete Block (Heavy) c) Drywall (Light)

Figure 4.1.2 Partition (Infill) wall typologies

As part of a FRST-funded (Foundation for Research Science and Technology) research project on "Retrofit Solutions for NZ multi-storey Buildings" (www.retrofitsolutions.org.nz), an external-only structural inventory and assessment survey was carried out for critical RC buildings in the Christchurch CBD following the 4 Sept 2010 earthquake (Pampanin et al., 2010). A population of buildings, built between 1930 and 1970, was chosen to represent the common population of buildings available in CBD. According to the survey, the percentage of buildings with concrete blocks was 41% and 27% using clay bricks (Figure 4.1.3a). In Figure 4.1.3b, it can be noted that, amongst the potential structural/seismic weaknesses of infilled frames, the development of short or captive columns, due to interaction between surrounding partial-height infills and the columns was observed to be one of the highest risk.



a) Infill Wall Type b) Possible Deficiencies

Figure 4.1.3 Survey of external infill in Christchurch CBD

Materials for heavy infill walls are produced in various dimensions and shapes. Masonry units, which include both clay bricks and concrete blocks, must be produced in accordance to AS/NZS 4455.1:2008. The design of masonry walls must be in accordance to NZS 4230:2004 and their construction must comply with NZS 4210:2001.

On the other hand, specifications for drywall construction are commonly given by the manufacturers of lining materials. In the case of gypsum plasterboard, their application and finishing must comply with AS/NZS 2589 (Gypsum linings-Application and finishing).

The first comprehensive definitions of infill/partition wall practices can be found in NZS 1900:1964, and have been used since, after some improvements, in revised standards.

The original definitions given in NZS 1900:1964 were as follows;

- 'Infilling Panel' means any wall between beams, columns, or floors which by virtue of its position and construction is subject to induced and/or applied loadings (Figure 4.1.4a shows a double skinned clay brick wall as an example, but it can also be single skinned)
- 'Partition Wall' means a wall which by virtue of its position and construction does not contribute to the strength or rigidity of a structure (Figure 4.1.4b, a drywall partition with double linings have been shown as an example)
- 'Reinforced Grouted Brick Masonry' means a construction of two or more skins of brick between which reinforcing steel is embedded in grout (Figure 4.1.4c)
- 'Reinforced Hollow Masonry' means masonry of cellular units having reinforcement in filled cells (Figure 4.1.4d)
- 'Reinforced Masonry' means any masonry in which reinforcing steel is so bedded and bonded that the two materials act together in resisting forces
- 'Shear Wall' means a structural wall which, because of its position and shape, makes a major contribution to the rigidity and strength of a building

Examples of design Standards for each wall type are shown in Figure 4.1.4. Minimum dimensions given by the corresponding current design standards of each wall type are shown in Table 3.1.

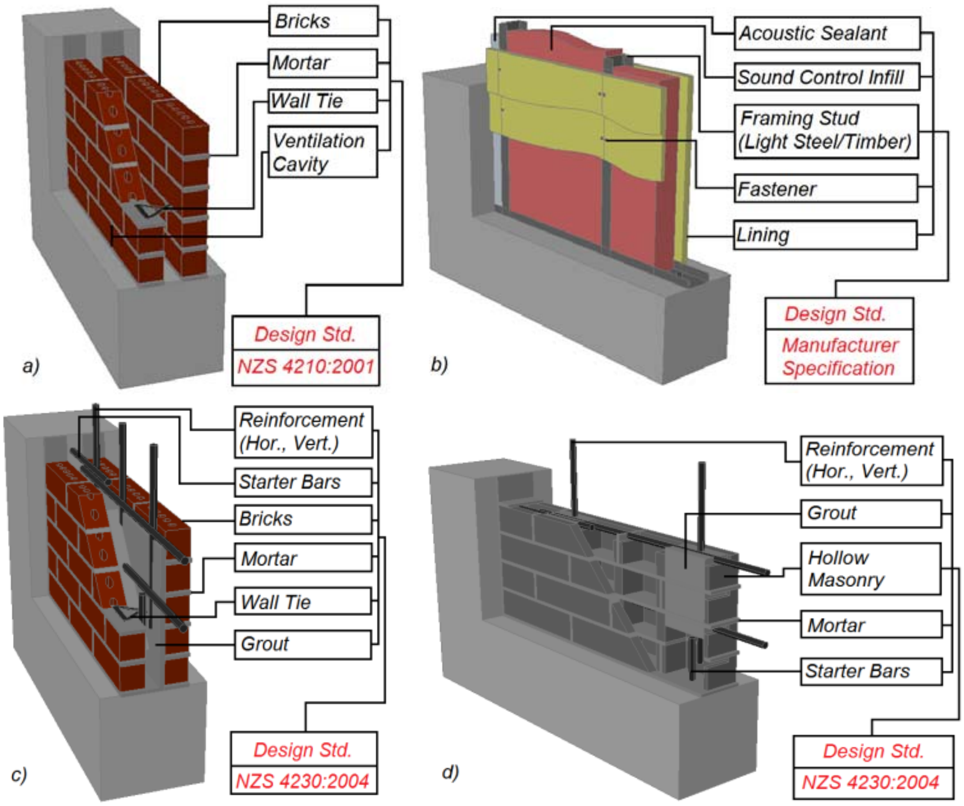


Figure 4.1.4 a) Unreinforced Masonry Infill, b) Light Steel/Timber Framed Partition Wall/Drywall, c) Reinforced Grouted Brick Masonry, d) Reinforced Hollow Masonry (from Tasligedik et al., 2011)

4.2 Current Design and Installation Specifications

The following documents are available for partition system design and installation in New Zealand.

• NZS 1170	Structural design actions
• AS/NZS 2589	Gypsum linings-Application and finishing
• NZS 4230:2004	Design of reinforced masonry structures
• AS/NZS 4455	Masonry units and segmental pavers
• NZS 4210	Masonry construction: materials and workmanship
• Manufacturer Spec.	Drywalls are manufactured using specifications given by the companies
• NZSEE 2006	Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquake

Some key specifications from these documents are given below:

AS/NZS 2589:2007: Gypsum Linings-Application and Finishing

Structural movement

- Partitions and gypsum linings shall be relieved from the building elements, particularly at columns, ceilings and intersections with dissimilar materials, where differential movement is likely to occur.
    - Most cracking of gypsum lining in buildings are caused by deflection, physical change in materials due to temperature and humidity changes, earthquake actions or a combination of these factors. Permanent and imposed actions in floor slabs cause deflections in partitions at the midpoint between supports. See paragraph D5 of Appendix D for further information.
    - Gypsum linings should not be assumed as a structural element unless verified by test or rational design methods.

Control Joints

- Control joints shall be provided at no more than 12m intervals in either direction for walls and ceilings. The control joint shall be capable of accommodating the magnitude of the expected movement. Figure 4.2.1 shows a separation of underlying frame to achieve an effective control joint in the gypsum lining which is aligned with the movement joint in the underlying structure. Figure 4.2.2 shows a control joint in the gypsum lining placed at the junction of two different substrates and Figure 4.2.3 shows a horizontal control joint at an internal mid-floor position such as a stairwell in a multi-storey building.
    - When correctly designed, a full height door, window, bulkhead or archway may perform the same function as a control joint. For further information, see Paragraph D5 and D6 of Appendix D.

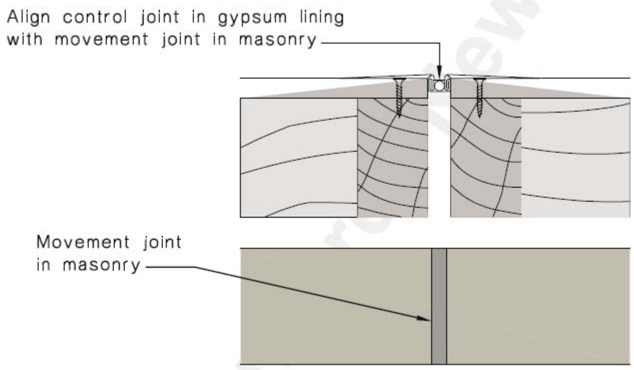


Figure 4.2.1 Alignment of control join in gypsum lining with movement joint (AS/NZS 2589)

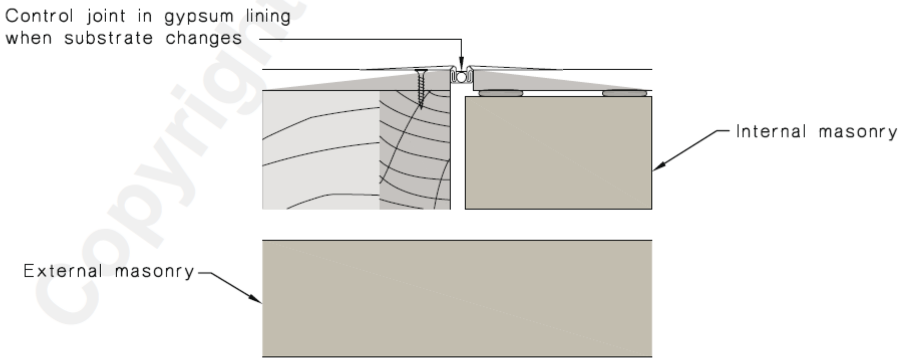


Figure 4.2.2 Control joint for substrate changes (AS/NZS 2589)

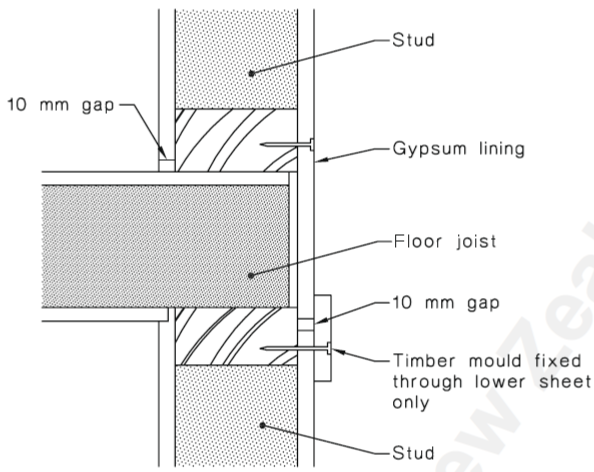


Figure 4.2.3 Horizontal control joint at an internal mid-floor position (AS/NZS 2589)

NZS 4230:2004: Design of Reinforced Concrete Masonry Structures

Separation approach

- It must be recognized that masonry in-fill panels modify the structural behaviour of the containing frame under lateral load, unless sufficient separation is provided at top and sides to allow free deformation of the frame to occur, in which case the panel must be designed as a partition in accordance with 12.6. It should be noted that even where sufficient separation is provided at top and ends of a panel, the panel will still

tend to stiffen the supporting beam considerably, concentrating frame potential plastic hinge regions in short hinge lengths at each end, or forcing migration of hinges into columns, with a breakdown of the weak-beam, strong-column concept.

Integration approach

- When in-fill panels are constructed without full separation from the frame, the composite action must be considered in analysis and designed accordingly (Figure 4.2.4).

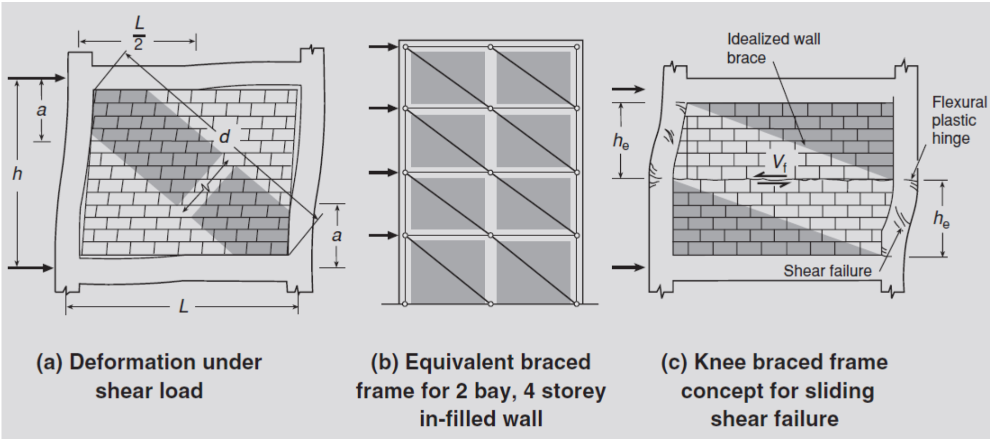


Figure 4.2.4 Equivalent diagonal bracing action of masonry infill (NZS4230)

NZS1170.5:2004 Structural Design Actions Part 5: Earthquake actions – New Zealand covers ceilings in Section 8 “Requirements for Parts and Components”.

- In-fill panels separated from the structural system such that the ultimate limit state inter-storey deflections calculated in accordance with NZS 1170.5 are accommodated shall be considered to be partitions and shall comply with the requirements of 12.6
- Partitions shall be separated from the structural system such that the ultimate limit state inter-storey deflections calculated in accordance with NZS 1170.5 are accommodated.
    - Although separation is advised by the standard, considering the separation approach in 12.5.1, no proper detailing is given as an example without forcing the plastic hinges to occur in the joints. Moreover, how to consider the infills in the design phase has been left to engineer without giving details (Rephrased in 3.2)

4.3 Issues with Current NZ Design Procedures and Practice

In most of international design codes, infill walls and partitions are typically considered as "non-structural elements" and thus tend to be mostly neglected in the structural design process. However, the observations made after major earthquakes (Duzce 1999, L'Aquila 2009, Christchurch 2011) have shown that even though infill walls might be considered to be "non-structural" elements, their interaction with the structural system response during seismic actions can modify the overall bare frame system response, potentially leading to unexpected and undesired failure mechanisms either at a local level (e.g. shear failure in captive columns, damage to joint region) or a global level affecting the structure's seismic response (e.g. soft storey mechanism). On the other hand, under moderate shaking intensity, infills can provide additional stiffness and strength to the building. The positive or negative effects of infills on the seismic response of a structure still represent a controversial and unsolved topic.

A typical approach in modern code provisions is to either require the engineer to consider and model the interaction of the infills in the overall seismic response in the design phase, or alternatively to provide adequate separation to minimize that interaction (see examples provided below, extract from NZS 4230:2004).

a) Duzce, Turkey 1999 b) L'Aquila, Italy 2009 c) Christchurch, NZ 2011

Figure 4.3.1 Damage to heavy (masonry) infill walls and interaction with the surrounding RC frame

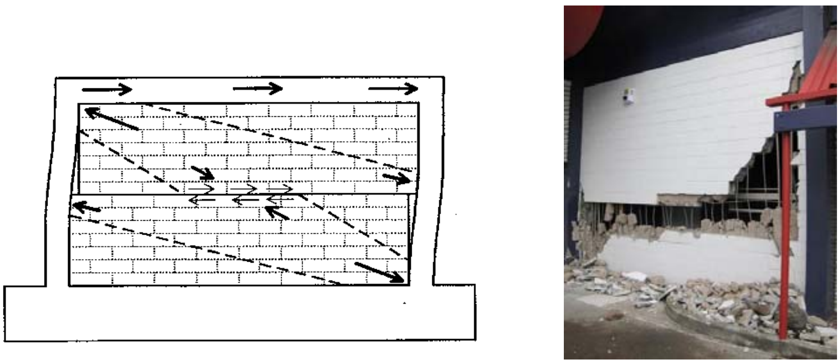


Figure 4.3.2 Potential Interaction between infills and surrounding frame (left). Example of reinforced concrete block infill damaged in Christchurch (right)

4.3.1 Performance-based objective and criteria considerations

From a performance-based design point of view, modern seismic codes prioritize life safety criteria in the seismic design of structural systems, which allows plastic hinging in certain structural members without causing a loss of global stability. However, even when the main structure has not collapsed in recent seismic events, severe structural damage was observed due to structure-infill interaction, in particular with heavy or brittle infills (concrete panels or masonry walls, consisting of clay bricks or concrete blocks). Particularly when dealing with heavy infills, their failure may very well be a significant threat to human life both inside and outside of the building.

Light infill-partitions might arguably be required to only satisfy a less demanding performance objective. However, it has been proven that the cost of their replacement is substantially higher than predicted. Furthermore, when considering their potential fire-protection contributions, cracking and damage might cause more severe consequences than expected.

Given the peculiar new seismicity in the Canterbury region, it is likely that more frequent earthquake events (likely to occur within the life-time of the structure) will actually have shaking intensity similar to what previously (prior to the increase of the Z factor) was considered an ultimate limit state (ULS) design level event. Thus, in such a unique risk scenario, the serviceability limit state (SLS) requirements for infills and partitions would become substantially more severe and demanding.

4.3.2 Code general design concept: integrated design or disconnection

Considering the above mentioned effects of ‘non-structural’ infill walls on life safety and global stability of the structure, code provisions are stated in NZS 4230:2004 (Section 12.5), as follows;

a) When infill panels are constructed without full separation from the frame, the composite action must be considered in analysis and designed accordingly.

b) It should be noted that even where sufficient separation is provided at top and ends of a panel, the panel will still tend to stiffen the supporting beam considerably, concentrating frame potential plastic hinge regions in short hinge lengths at each end, or forcing migration of hinges into columns, with a breakdown of the weak-beam, strong-column concept.

Although these two alternatives are contemplated in NZS 4230, there is in general a lack of comprehensive guidelines, including practical examples of construction details, to support their actual implementation.

4.3.3 Compliance issues with the interaction approach

Considering point a) in 4.3.2, interaction required to be accounted for, NZS 4230 does not clearly mention how to take infills into account, leaving engineers with many options, mostly derived from international literature. Quite recently (2006) the NZSEE ‘Guidelines for Assessment and Improvement of the Structural Performance of Buildings in Earthquake’ (NZSEE, 2006) provided some valuable references for the modelling of unreinforced masonry infills as an equivalent diagonal strut.

However, as these guidelines have been mostly prepared for assessment of the structural performance of existing buildings instead of new ones, other types of infills are not mentioned. It would be very useful to have tables specifying the values of parameters for design/assessment of different kinds of infill and partition walls (heavy and light).

Also, although “light” infills are more commonly used as partitions in modern buildings in New Zealand than brick walls, simple and quick rules to determine the external actions on light weight infills/or partitions are not available yet. This information, in the form of tables or design charts based on drift demand and actual internal forces, would be very useful in order to quickly calculate the likely damage level of any type of infill.

4.3.4 Compliance issue with the separation approach

On the other hand, for case b) in 4.3.2, separation required, while sufficient separation can eliminate the complexity of modelling infill/frame interactions, other issues are raised from a structural, architectural and a practical point of view:

- Separating the infills/partitions from the adjacent columns and possibly from the top beam, might activate out-of-plane failure mechanisms, which have been proven to be particularly weak, especially when dealing with heavy partitions and subjected to out-of-plan accelerations. A set of examples of details and techniques are required for a variety of situations, validated by adequate experimental testing.
- Evaluation of the required separation is a complex issue if lack of interaction between infills, partitions, and surrounding frames is to be maintained. In the light of new seismicity conditions in Christchurch, it is necessary to ***revisit performance criteria*** not only for the structural system but also for the non-structural components. The target drift (inter-storey relative displacement) corresponding to the SLS or ULS earthquake should be considered and specified in direct consultation with the client according to the desired level of acceptable performance or damage (for example fully operational after another aftershock as that occurred on 13 June 2011).
- In addition to the structural requirements and considerations, the most common obstacles to a wide implementation of infill/partition-frame separation are apparently related to the lack of information on practical examples and techniques to achieve such a separation while respecting or maintaining other performance parameters such as architectural features, weather tightness, fire and acoustic separation, and thermal insulation.

Current New Zealand Building Code provisions do not require nor enforce seismic separation detailing to minimise damage to infill walls and drywall partitions. Common practice and architectural preference is for a monolithic finish by creating flush finished painted surfaces and ‘square stopped’ or ‘square finished’ junctions at wall-to-wall, wall-to-frame, or wall-to-ceiling intersections. These details provide very little if any freedom to move when a building undergoes seismic deformations. Therefore, they are *susceptible to damage*, as demonstrated by the constant, repetitive failures and associated high cost of repair, which is observed following the main Christchurch earthquake events (4 Sept, 26 Dec, 22 Feb, 13 June).

Recommendations do exist for gypsum plasterboard control joints, but these are targeted at thermal and moisture movement of different building materials, and do not aim to address seismic movement. Expansion (or control joint) provisions for gypsum plasterboard linings

are typically required at 9 metre intervals for walls. The concept of temperature and humidity control joints can also be adapted to cope with higher displacement demands resulting from seismic activity.

4.4 Summary of Damage Types in Canterbury Earthquakes

In general, there are four typical failure patterns for *heavy infill walls* as summarized in Figure 4.4.1: ‘Crushing at the centre’, ‘Crushing at the corners’, ‘Sliding shear’, and ‘Diagonal cracking’;

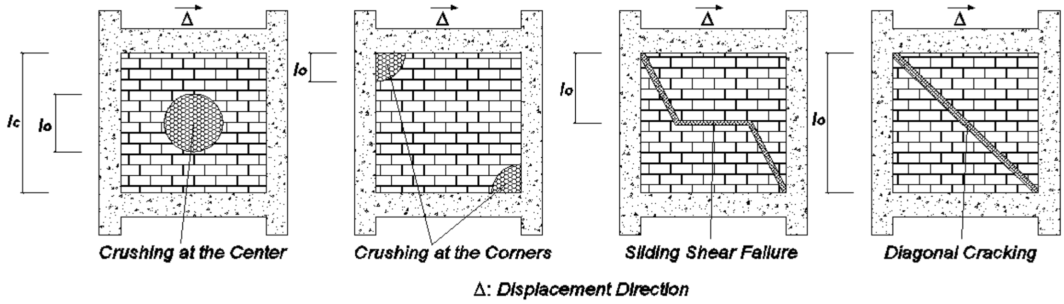


Figure 4.4.1 Typical Failure Patterns of Infill Walls/Partitions (from Tasligedik et al., 2010)

It should be noted that for masonry bricks and blocks, diagonal cracking can show itself as stepped cracks.

For *gypsum board drywalls*, imposed actions can cause damage to the surface linings and underlying light framing.
The most recurrent observed damages are:

- Steel studded partitions: bottom and top track movement, not securely fastened
- Steel studded partitions: studs coming out of bottom and/or top track
- High steel studded and timber framed partition walls: linings detaching from framing; more prone to damage with wall heights above 2m
- Sheets detaching from framing on large walls with timber and steel framing
- Sheet cracking around openings and penetrations
- Compression of plasterboard sheets where butted into structural members
- Steel partition walls detaching from structural framing
- Cracking to the perimeters of sheets fixed to steel studs and timber framing

After the 4 September and 22 February 2011 earthquakes in Christchurch, most of these damage modes were observed throughout the CBD. The following photos are only a small fraction of the general observations. The photos has been presented in four categories: unreinforced clay brick infill walls (heavy), reinforced hollow concrete masonry block infill walls (heavy), gypsum lined drywalls (light), and infill wall-frame interaction related damage observations.

4.4.1 Unreinforced Clay Brick Infill Walls

Figure 4.4.2 Diagonal cracking (Photos: A.S. Tasligedik)

Figure 4.4.3 Sliding shear and interaction with surrounding frame (Photos: A.S. Tasligedik)

Figure 4.4.4 Corner crushing and interface cracking (Photos: A.S. Tasligedik)

Figure 4.4.5 Spalling of bricks (left) and separation of infill from frame (right) (Photos: A.S. Tasligedik)

4.4.2 Reinforced Hollow Concrete Block Masonry Infill Walls

Figure 4.4.6 Corner crushing and revealed internal reinforcement of the infill wall (Photos: A.S. Tasligedik)

Figure 4.4.7 Splitting crack caused by corner compressive stresses Left) and stepped (or diagonal) cracking (right) (Photos: A.S. Tasligedik)

Figure 4.4.8 Crushing on top of the reinforced masonry infill wall (Photos: A.S. Tasligedik)

Note that the top layer of block masonry has not been grouted and not been fixed to the top beam to allow for seismic movement. Although this may be a recommended practice, it did not properly isolate the wall from the frame considering the suffered damage

4.4.3 *Gypsum Lined Drywalls*

Figure 4.4.9 Cracking at lining interfaces (Photos: A.S. Tasligedik)

Figure 4.4.10 Steel stud buckling (left), broken lining (centre) and cracked lining at interface and the corner of doors (right) (Photos: Hans Gerlich)

4.4.4 Infill Walls and Frame Interaction

Figure 4.4.11 Short column shear failure due to infill presence (Photos: A.S. Tasligedik)

4.5 Recommended Practice - Design and Installation Guidance

In relation to the issues raised in Section 4.3 and the observed partition and infill damage described in Section 4.4, the following “recommended practice” guidelines are proposed taking into account the two following ideologies:

- provide disconnection (practice example is given in Figure 4.5.1)
- or account for interaction in the design (not practical due to the variety of wall types and complexity in the behaviour of the used materials)

Simpler and improved way of calculating the interaction between any type of infill and frame may be recommended based on their stiffness and geometry. This could include evaluation of the required gap associated to the interstorey drift demand and/or evaluation of the internal forces acting onto the infill/partitions for a more accurate design.

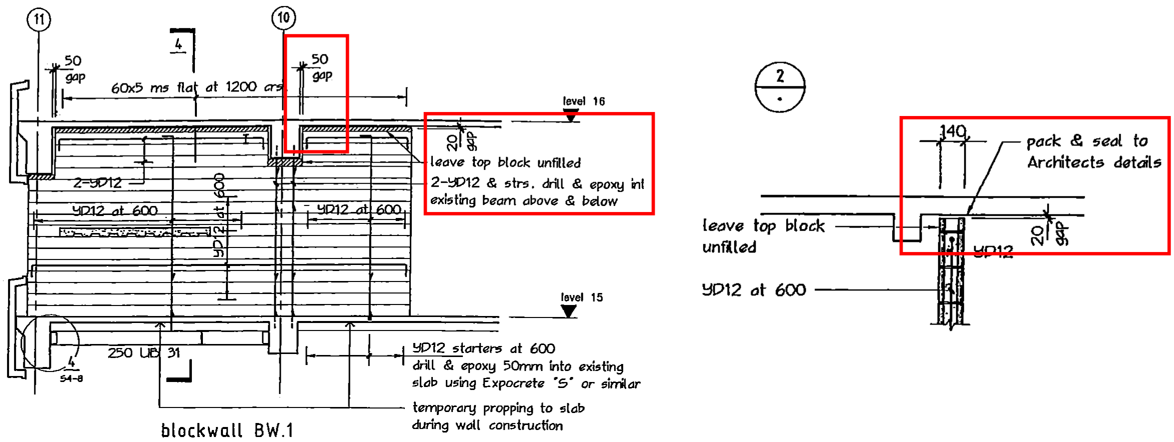


Figure 4.5.1, Example of recommended practice (Elevation and section view). Note that the gap widths may need to be reconsidered to properly isolate the infill wall from the frame (considering the damage observed after February earthquake, refer to concrete block masonry damage photos)

Suggestions for inter-storey drift damage limit states for infills have been proposed by Magenes & Pampanin, 2004 (Figure 4.5.2) for the performance evaluation of masonry infill panels based on the strain limit states for infill panels.

A compromise solution would be to leave a partial gap corresponding to 1 to 1.5% drift, such as 25 to 35mm, and then investigate the subsequent interaction between infills and frame such that it will not cause failure for a desired level of drift, such as 2-to-3% corresponding to 50-to-75mm of demand displacement.

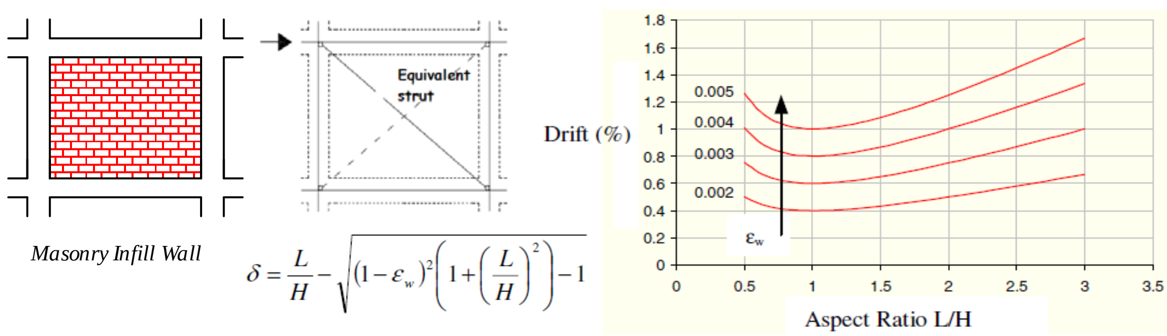


Figure 4.5.2 Evaluation of drift limit state for masonry infills using an equivalent diagonal strut model (Magenes and Pampanin 2004)

4.6 Best Practice Detailing Guidance

The NZS4230 code on “Design of reinforced masonry structures” recommends providing isolation gap around the masonry walls without specifying or showing typical detailing. The solutions are left for designers to develop. Further experimental tests are required to develop a few simple standard solutions. Due to the lack of guidelines and enforcement of the standard solutions for reducing damage to partitions, unfortunately, these types of solutions are very rarely used in practice. These details provide a degree of freedom to the walls during seismic movement and limit their damage. In the following figures, several poor and recommended practices are shown. However, it should be noted that these detailing are based on the observations made on the existing practice and a thorough verification of these details are necessary before they are adapted in practice.

Unreinforced Clay Brick

Referring to Figure 10, Infill wall and frame interaction can be reduced using a soft packing material at the sides of the wall. Here, an acoustic/thermal sealant can also be utilized for air tightness and thermal proofing. In addition to those, a light steel mesh can be placed in a cement based plaster layer on the surface for out of plane resistance. The wall should also incorporate a top gap where a detail similar to the sides can be applied. However, a C-channel must be placed at the top for out of plane resistance. For more details, please refer to Figure 10 on the next page.

Reinforced Hollow Masonry

Referring to Figure 11, it can be seen that the detailing required for this wall type is very similar to clay brick masonry. However, in this type, the reinforcement is grouted in the masonry block rather than at the surface. Also the top block layer should not be grouted since this level makes up the main part of the interaction. It should also be noted that there are cases where this type of walls are designed and constructed as structural walls, which is not the focus in this particular report. In this report, the reinforced hollow masonry as non-structural wall was focused.

Drywalls with Gypsum Linings

This wall type is dominantly used all around New Zealand and occupies the major portion of the so called partitions. Referring to Figure 12, this type of partitions can be improved by isolating the gypsum lining from the surrounding framing by a designed amount of gap such that the lining starts to suffer damage after a certain drift level of choice. The resulting gaps can be hidden using existing plastering techniques easily. Moreover, the expansion gap provision existing for steel studded drywalls can be revised and increased to allow for possible seismic deformations in vertical. Please refer to Figure 12 for details.

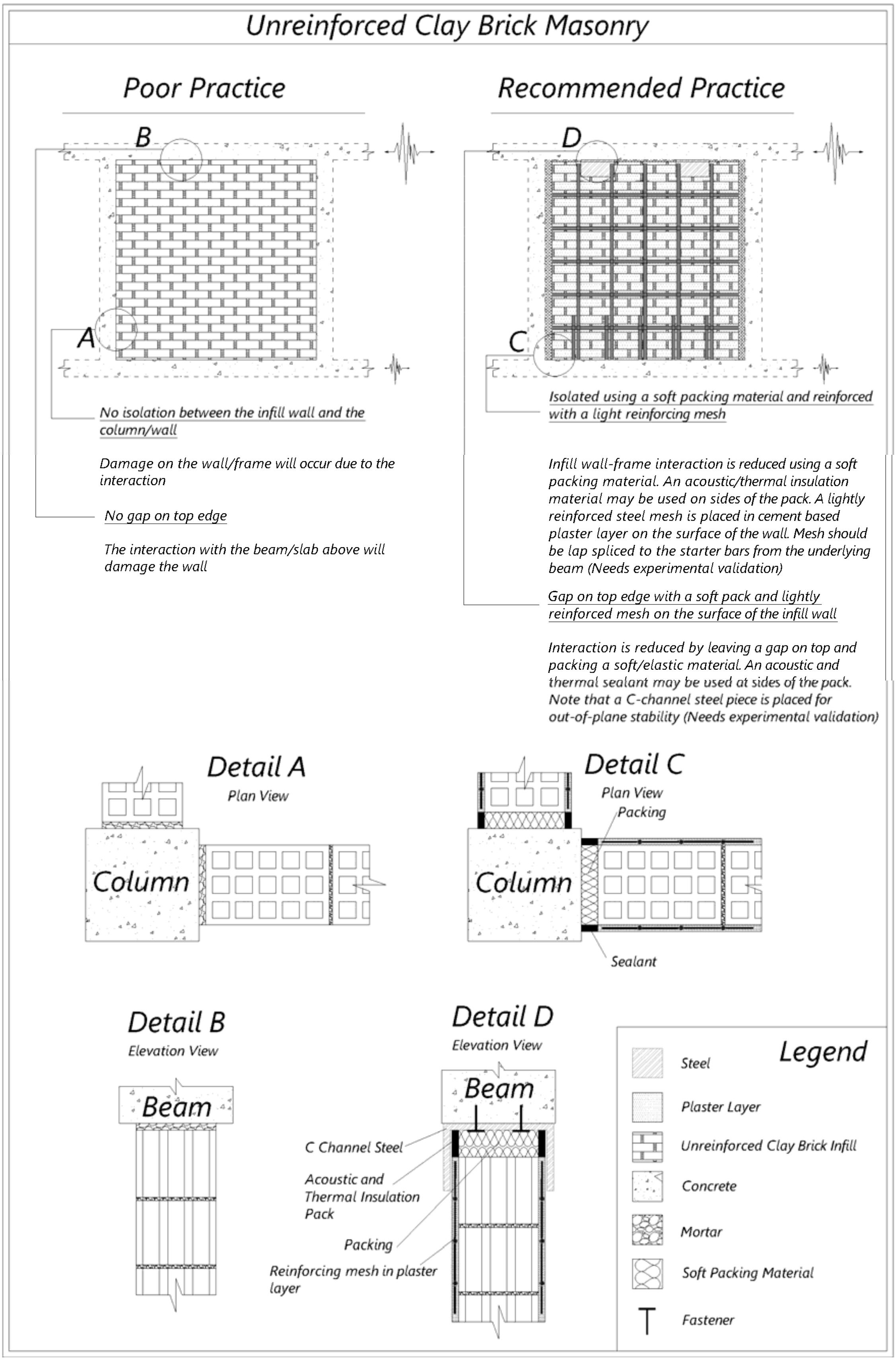


Figure 4.6.1 Unreinforced clay brick masonry recommended practice (Figure courtesy of A.S. Tasligedik)

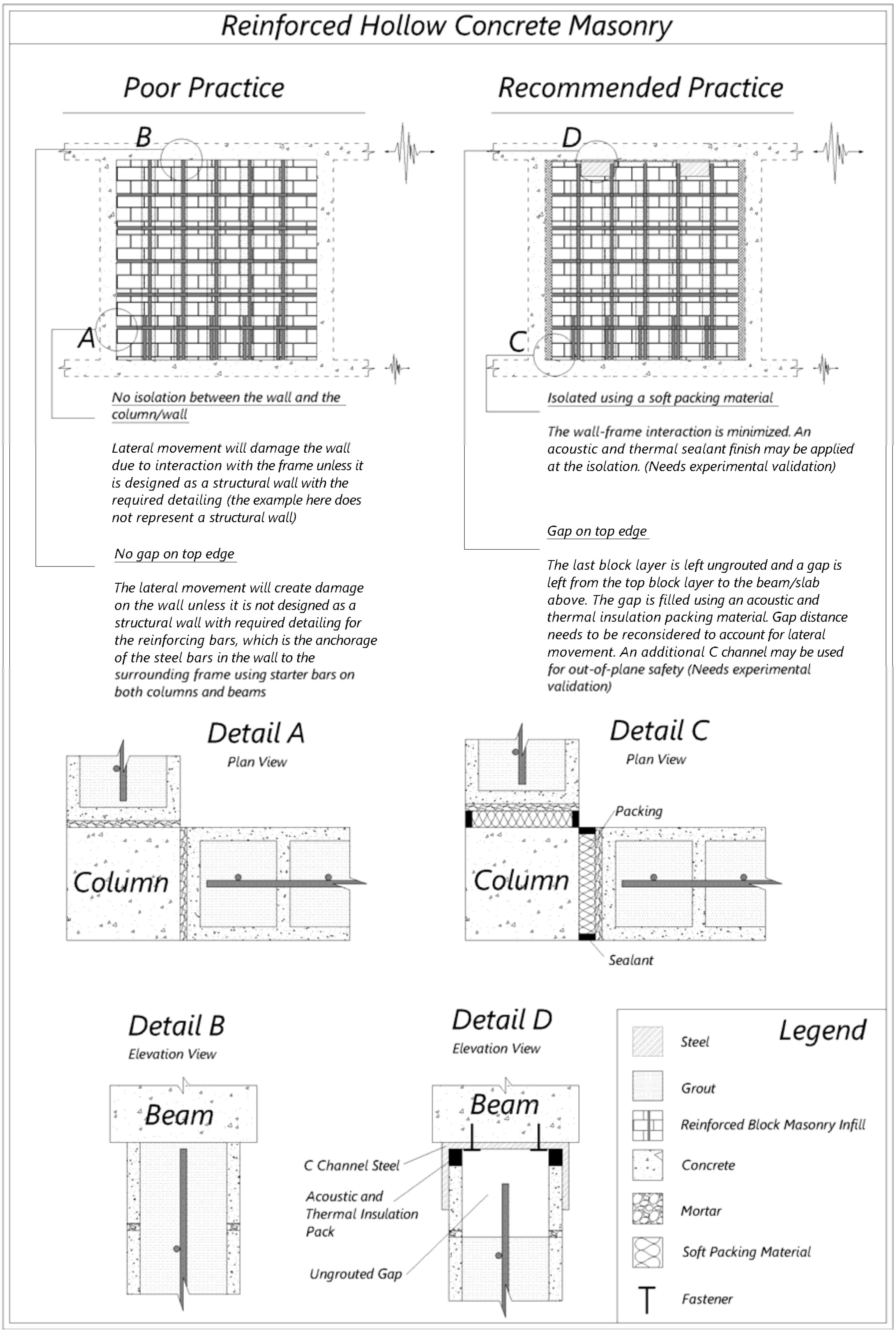


Figure 4.6.2 Reinforced hollow masonry recommended practice (Figure courtesy of A.S. Tasligedik)

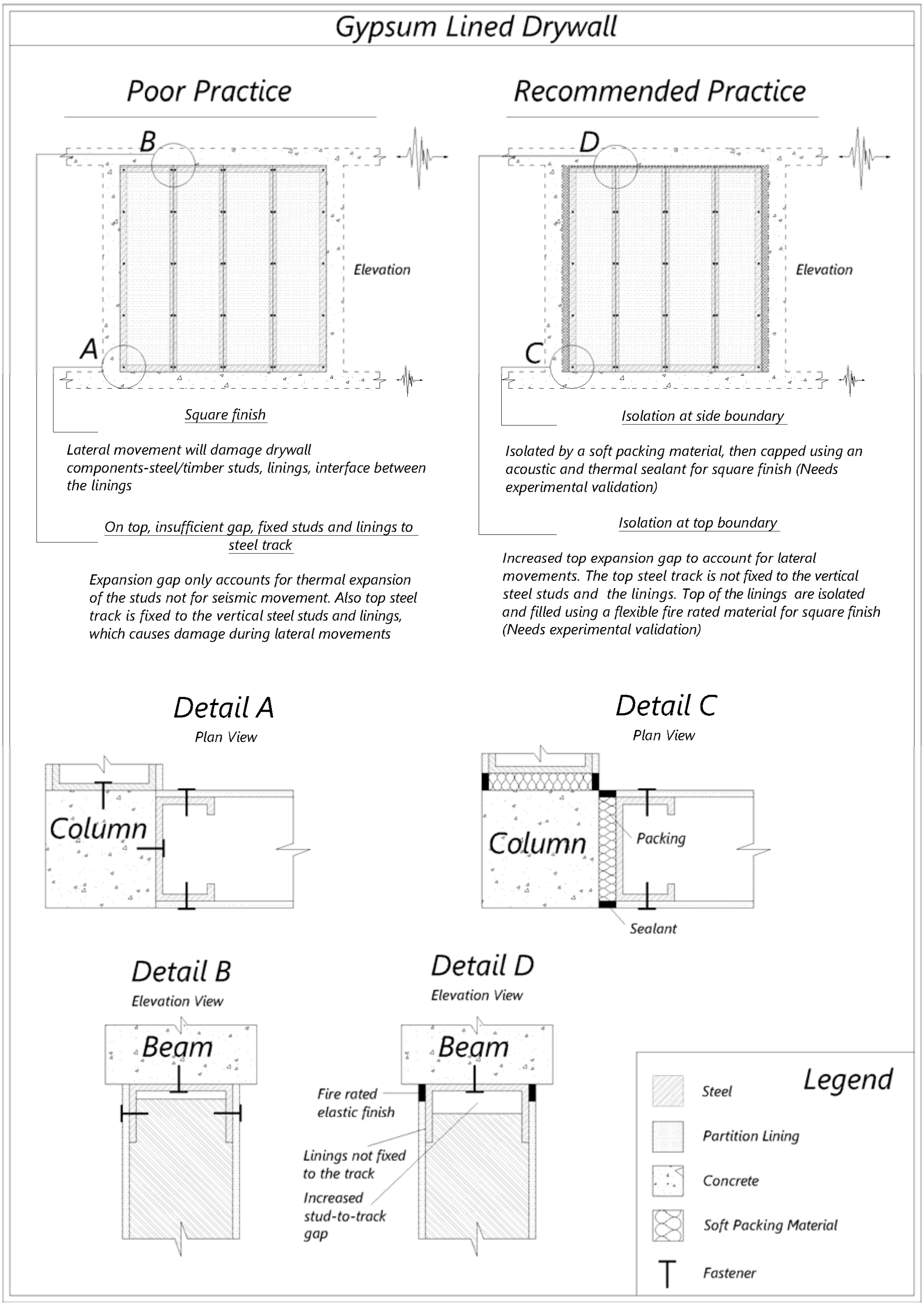


Figure 4.6.3 Gypsum lined drywall recommended practice (Figure courtesy of A.S. Tasligedik)

4.7 Preliminary Experimental Validation of Low-Damage Solutions

Recently, two “damage-free” (or, better, damage-control) drywall types have been developed and successfully tested at the University of Canterbury. The developed drywalls are able to achieve no-damage state until 2.0% drift level. At 2.5% drift level, the walls still remains serviceable with the only damage being at the side plasters (Figure 5.1.1). However, this drift level can be engineered by choosing different gap distances depending on the architectural requirements. The experimental campaign is still ongoing and a full report with further details is under preparation (Tasligedik et al., 2012)

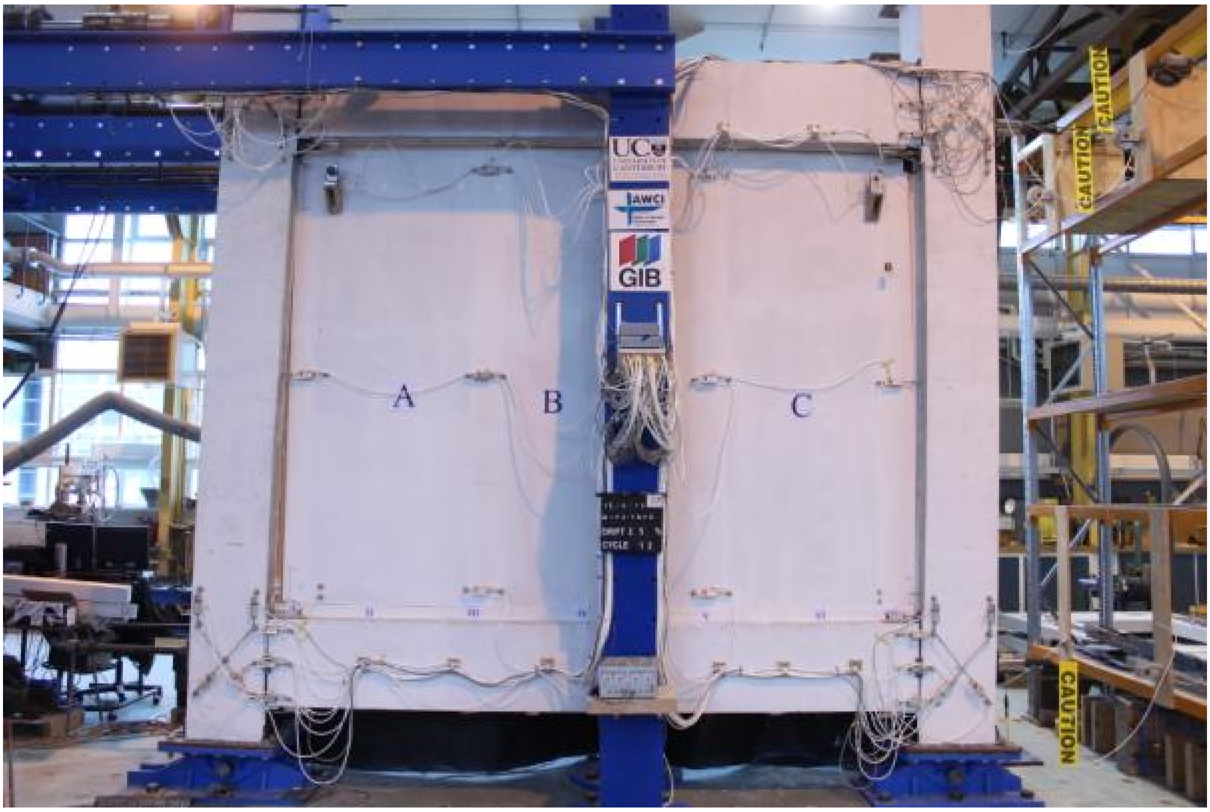


Figure 4.7.1 Damage resistant drywall solution at 2.5% drift level with only side plaster damage, still in serviceable condition. Experimental test campaign at University of Canterbury (Tasligedik et al., 2012)

4.8 References

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