PART C

Secondary Structural and Non-Structural Elements

C10

Document Status

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This version of the Guidelines is incorporated by reference in the methodology for identifying earthquake-prone buildings (the EPB methodology).

Document Access

This document may be downloaded from www.building.govt.nz in parts:

1 Part A – Assessment Objectives and Principles

2 Part B – Initial Seismic Assessment

3 Part C – Detailed Seismic Assessment

Document Management and Key Contact

This document is managed jointly by the Ministry of Business, Innovation and Employment, the Earthquake Commission, the New Zealand Society for Earthquake Engineering, the Structural Engineering Society and the New Zealand Geotechnical Society.

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Errata and other technical developments will be notified via www.building.govt.nz

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Contents

C10. Secondary Structural and Non-structural Elements

C10.1 General

C10.1.1 Scope and outline of this section

This section provides guidance for engineers carrying out Detailed Seismic Assessments (DSA) of building elements that are not part of the primary lateral or gravity structure within a building or section of a building. Such elements are referred to as secondary structural and non-structural (SSNS) elements and systems. These guidelines aim to provide a consistent approach to assessing these elements and systems, with a focus on scoring them on the basis of both their capacity and whether or not they are expected to be a significant life safety hazard or have the potential to damage adjacent property should their capacity be exceeded.

Note:

To be a life safety hazard, or to cause damage to other property, a building element must be able to fall to the extent that it is able to create the hazard. This could be direct or through impact with other building elements. To be classified as significant, a number of people need to be exposed to the danger.

These concepts are discussed further in Part A of these guidelines, which defines the types of building element that are expected to pose a significant life safety hazard and whether or not they would be expected to be considered in establishing the earthquake rating for the building as a whole.

An assembly of SSNS elements is often referred to as a system (e.g. a suspended ceiling system), but is considered as a single building element within these guidelines.

Damage to some types of building element can have a significant economic impact, particularly in terms of repair and business continuity costs, even in relatively small earthquakes. While these aspects are not life safety hazards, they may require specific consideration for other types of assessment brief.

During the Christchurch earthquake of 22 February 2011, a significant number of the injuries that people suffered within buildings were attributed to building contents. Assessment reports should also comment on the vulnerability of the building contents.

C10.1.2 Definitions and acronyms

Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable strength capacity has been reached. A brittle structure has little tendency to deform inelastically before it fractures.
Building element	Any structural or non-structural component and assembly incorporated into or associated with a building. Included are fixtures, services, drains, permanent mechanical installations for access, glazing, partitions, ceilings and temporary supports (from the New Zealand Building Code).
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
HSNO	Hazardous substances and new organisms
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Lateral load	Load acting in the horizontal direction, which can be due to wind or earthquake effects
Load path	A path through which vertical or seismic forces travel from the point of their origin to the foundation and, ultimately, to the supporting soil
Non-structural element	An element within the building that is not considered to be part of either the primary or secondary structure
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also be part of the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be part of the primary gravity structure.
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects, this is determined using probable material strengths. For geotechnical issues, the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
Secondary structural element	A structural element that is not part of the primary structure
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but nevertheless is required to transfer inertial and gravity loads for which assessment/design by a structural engineer would be expected. Included are precast concrete panels, curtain wall framing systems, stairs and supports for significant building services items.
Significant life safety hazard	A hazard resulting from the loss of gravity load support of a member/element of the primary or secondary structure, or of the supporting ground, or of non-structural items that would reasonably affect a number of people. When shelter under normally expected furniture is available and suitable, mitigation of the hazard below a significant status is assumed.
SSNS	Secondary structural and non-structural
Unreinforced masonry (URM)	A member or element comprising masonry units connected together with mortar and containing no steel, timber, cane or other reinforcement

Veneer	A skin or leaf of URM (typically a single wythe) that is reliant on intermediate support from other elements to resist face loads
Wythe	A continuous vertical section of masonry one unit in thickness. A wythe may be independent of, or interlocked with, adjoining wythe(s). A single wythe is also referred to as a veneer or leaf.

C10.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%NBS	Percentage of new building standard as assessed by application of these guidelines
b_{g}	Glass panel width
$C_{\mathrm{i}}(T_{\mathrm{p}})$	Spectral shape factor for a part in accordance with NZS 1170.5:2004
c	Clearance between glass panels and framing
h_{cw}	Curtain wall fixing height
h_{g}	Glass panel height
Q	Element performance factor
S_{p}	Structural performance factor of a building in accordance with NZS 1170.5:2004
T_{p}	Fundamental period of vibration for a part in accordance with NZS 1170.5:2004
$\delta_{\mathrm{capacity}}$	Deformation capacity of an SSNS element and its connections
δ_{demand}	Deformation demand on an SSNS element and its connections
δ_{i}	Lateral deformation of building floor level i at the location of an SSNS element
Δ_i	Lateral deformation of building floor level i at its centre of mass
δ_{panel}	Deformation capacity of an individual glass panel
μ_{p}	Ductility of a part in accordance with NZS 1170.5
ϕ	Strength reduction factor

C10.2 Observed Behaviour of Secondary Structural and Non-structural Elements in Past Earthquakes

C10.2.1 General performance

SSNS elements are often extensively damaged in moderate to large earthquakes. For example, a study of the 66,000 buildings that suffered damage in the 1994 Northridge earthquake revealed that three quarters of these sustained damage to building elements when there was no damage to their primary structure (Charleson, 2008).

Historically, fatality rates attributed to SSNS element failures have tended to be significantly lower than those attributed to failures of the primary structure. The low rate for SSNS failures is mainly because they fell rather than because they triggered a disproportionate collapse. However, injuries and damage to other property were relatively common, particularly heavy items which were particularly hazardous.

The consequences of the failure of a secondary structural element are illustrated in Figure C10.1, which shows the final and original locations of a precast concrete cladding panel.

Figure C10.1: Failure of a precast concrete panel (Baird et al., 2011)

C10.2.2 Performance in the Canterbury earthquakes

There was extensive damage to SSNS building elements during the 2010-11 Canterbury earthquake sequence. Damage was commonly observed for ceilings and in-ceiling services, as were failures of masonry veneer, stairs, precast cladding panels (SESOC, 2013) and plant support in some buildings.

There were a limited number of fatalities solely attributed to SSNS failures in non-residential buildings. However, SSNS were the cause of the majority of injuries caused directly by earthquake damage (Yeow et al., 2017).

Some examples of non-structural damage as a result of these earthquakes are shown in Figure C10.2.

Figure C10.2: Ceiling and other damage (Dhakal, 2010 and McGuigan)

C10.3 Historical Treatment and Factors Affecting Seismic Performance

The structural design of many potentially hazardous SSNS elements has historically received less consideration from a seismic perspective than the design of the primary structural elements. These building elements were generally designed after the primary structure – often, during the construction phase – and with little consideration of how the building in which they would be installed could influence their behaviour during an earthquake.

Many existing SNSS elements were proprietary systems and their design and construction were unlikely to have been overseen by a structural engineer.

There has also been a limited and variable degree of compliance with standards for building services (e.g. NZS 4219 Seismic Performance of Engineering Systems in Buildings). Engineering assessments should therefore always assume non-compliance with these standards until proven otherwise.

The capacity of elements and their connections to the primary structure is usually governed by their ability to accommodate:

- deformations within the structure that are generated by the earthquake
- internal inertial forces generated by the earthquake, and
- impact from any other building elements during the earthquake.

Connections can affect the performance of a building element and will often limit the earthquake score of the whole element, particularly where there is little redundancy or capacity to accommodate deformations.

Note:

Connections for precast concrete panels, curtain wall framing systems and stairs often have insufficient capacity to accommodate the seismic deformations of the primary lateral structural system.

Inertial forces are more likely to cause tall, slender building elements to fail in flexure or topple; shorter elements to rock, overturn or slide; and squat elements to rock or slide.

Failure of an inadequately restrained element (e.g. a suspended ceiling system) can lead to a consequential failure of another element (e.g. a fire sprinkler system).

C10.4 Assessing Secondary Structural and Non-structural Elements

C10.4.1 Approach and objectives

This section outlines the detailed assessment approach that, along with the extent of the assessment, will vary with the complexity of the element or system it forms, and the degree of certainty about element capacities.

The assessment should use the objectives outlined in Section C1 and the procedures outlined in that section and Section C2. In particular, refer to Sections C1 and C2 for guidance on:

- documentation that should be sourced to undertake the assessment
- inspections required to verify that the construction and installation are in accordance with the design documentation
- what, if any, intrusive testing should be considered, and
- the general assessment and analysis procedures that should be considered.

An Initial Seismic Assessment (ISA) should usually be performed before a DSA because this can identify high risk building elements such as heavy chimneys and unreinforced masonry (URM) parapets that could significantly reduce the earthquake rating for a building. Mitigation or replacement of these elements can increase the expected building performance and may avoid the need for a DSA.

Note:

Some elements (e.g. curtain wall framing) may require specialist advice to carry out the assessment. However, when this happens, the engineer carrying out the assessment should establish a holistic understanding of how the building, the elements and their connections interact based on this advice.

Engineers should consider particular vulnerabilities or weaknesses within building elements and use engineering judgement to consider the effects these would have on their behaviour during an earthquake.

C10.4.2 Inspection and investigation

During site inspections, engineers are expected to identify the systems, sub-systems, elements, members and connections, as described in Section C1. They should then use an appropriate level of inspection to determine the condition of the building elements and their connections, as these may have deteriorated since their original installation. This may require intrusive work in roof spaces, ceilings and wall cavities to investigate and record the relevant details. Element and connection damage, decay and corrosion should be recorded so that capacities are downgraded to account for this.

The assessment should also consider the effects of any alterations, particularly for older buildings. These buildings have often gone through multiple changes of occupancy. As a result, there may be a number of additions and modifications to building elements that may not have been well considered.

Some buildings will have large numbers of some types of building element. The inspection and investigation programme for these buildings will need to identify the more critical locations, where the elements are likely to be subjected to the highest demands (forces and deformations) and/or likely to be in the worst condition (e.g. due to corrosion etc.). A statistically valid sample of those more critical elements needs to be inspected, with the inspection quantity chosen to best represent the life safety hazard posed by that type of element within the building.

Note:

A condition assessment is recommended as part of a DSA for all secondary and non-structural elements, particularly those in older buildings. The focus should be on inspecting and assessing the condition of the following elements and their connections to the primary structure:

- URM parapets/chimneys
- precast concrete panel and curtain wall framing
- external heavy plant and HSNO vessel support frames that are attached to a building
- canopies, balconies, billboards and other appendages that are attached to a building.

C10.4.3 General assumptions and considerations

Assessments of SSNS building elements should:

- determine which elements are expected to either pose a significant life safety hazard or damage adjacent property, either directly or indirectly, if they are able to fall. Refer to Part A.
- consider whether a force based approach, a deformation-based approach or aspects of both approaches are appropriate for each element.
- consider how a force based approach used in one direction would influence or be influenced by a deformation-based approach in the other
- determine demand forces and deformations in accordance with Section C3.

Note:

The demands for these elements are likely to be those specified in Section 8 of NZS 1170.5:2004.

- carefully consider the appropriate part/component categories (P1 to P6) within tables 8.1 and C8.1 of NZS 1170.5:2004. More than one part/component category may apply to a particular building element, but only those relating to life safety are intended to be considered.
- consider whether an element could fall off supports such as ledges that are too short to accommodate the deformations of the primary lateral structure
- consider whether an element is or could become an unintended integral part of the primary structure that can both change the behaviour of the structure and overload the connections to the element, or cause it to buckle or crush

Note:

Heavy, non-loadbearing infill walls can have inadequate clearance between them and the structural fames that surround them. Refer to Section C7 for guidelines on assessing their effect on reinforced concrete frames.

SSNS elements such as stairs and precast panels that are often connected to more than one level of the building can also do this when they have insufficient capacity to accommodate the deformations of the primary lateral structure.

The effect these have on the primary structure, including the impact on plan regularity, also needs to be considered.

- consider how the orientation of the element relative to the primary structure will affect both the demand and the capacity
- identify the critical and controlling load paths, the strength hierarchy, and likely mechanisms of the system to assist with determining the available ductility capacity using a rational analysis (where possible)
- base the assessment on probable capacities (i.e. the strength reduction factor should be set at 1.0 and probable material strengths should be used), and
- assess element capacities using Section C10.6 assuming that the load path into and out of each element is complete and sufficient to transfer the required demands. This assumption should be confirmed.

A force based assessment approach (Section C10.4.4) may be sufficient for SSNS elements that are only connected to the primary structure at one horizontal elevation or floor level.

A deformation-based assessment approach (Section C10.4.5) is usually needed when:

- a relatively rigid SSNS building element is connected to the primary structure at more than one horizontal elevation, or
- there are multiple sub-systems of different configurations and materials.

Elements that are flexible in one direction may be able to accommodate deformations of the structure in that direction, so may only need to be assessed using a force based approach.

The capacity of some connections to the primary structure (Section C10.5) and SSNS elements may need to be assessed for simultaneously applied deformations and forces, particularly when the elements are heavy.

C10.4.4 Force-based assessment

Element demand forces should be determined in accordance with Section C3.

For a force-based assessment the engineer carrying out the assessment will need to determine the probable strength capacity of each member within the SSNS element using:

- material-specific guidance in Sections C1 to C9
- connection-specific guidance in Section C10.5
- element-specific guidance in Section C10.6, and
- other references as necessary.

In doing so, the potential failure mechanisms should be identified and, where relevant, deformation capacities and fixing capacities determined by rational analysis.

C10.4.5 Deformation-based assessment

Lateral displacements of the primary structure will normally be used to assess the impacts on SSNS building elements and to determine how earthquake actions could be accommodated. Structural analysis (refer to Section C2) will normally be used to predict inter-storey displacements at the location each element is attached to the primary structure.

Note:

The assessment needs to include all sources of lateral deflection (e.g. soil flexibility, ductility (where appropriate), torsion and P-delta) and other deformations (e.g. frame elongation and vertical deformations), including those that may not have been considered when the building was originally designed.

The earthquake score for an SSNS element is in terms of the deformation capacity of the element and its connections and the deformation demand imposed by the building:

$$\%NBS = \frac{Q \times \delta_{\text{capacity}}}{\delta_{\text{demand}}} \times 100\% \qquad \ldots\text{C10.1}$$

where:

δ_{demand} = deformation demand for the element and its connections

δ_{capacity} = deformation capacity of the element and its connections

Q = element performance factor; $Q = 1.0$ for most SSNS elements.

The structural analysis predicting lateral displacement, Δ_{i}, at the centre of mass at each level within the building needs to transform this displacement to the displacement, δ_{i}, at the location of each SSNS building element. Other forms of deformation (e.g. frame elongation and vertical deformations) will need to be added to deformations calculated from those displacements.

The deformation capacity of an SSNS element requires judgement because the structural elements within modern buildings are detailed to prevent building collapse at deformations well beyond ultimate limit state defined shaking levels.

Some elements, such as stairways supported on ledges, have a higher risk of falling and need $Q < 1$ to reflect that. Guidance is provided by Section 8 of NZS 1170.5:2004, which requires ledges providing gravity support for building elements to accommodate ultimate limit state deformations multiplied by a factor of $2/S_{\mathrm{p}}$, where S_{p} is the structural performance factor. The Q for elements supported on ledges would therefore be $Q = S_p/2$.

C10.5 Connections to the Primary Structure

C10.5.1 General considerations

The connections used to attach SSNS building elements to the primary structure are an important consideration and often determine the capacity of these elements. Connections typically have three independent parts that need to be assessed: a fixing into the body of the element, a connector or body of the connection, and a fixing to the structure.

It is important to consider how the seismic actions are distributed to each of the element’s connections. This may be more complex when the axes of the element are not aligned with those for the primary structure.

Aspects of connections that the engineer carrying out the assessment should consider include the following:

- the offset between the element’s centre of mass and each connection
- the stiffness (or flexibility) of the element relative to that of the structure to which it is affixed
- how inertial forces from the element are distributed between its connections
- continuity of the load path for internal forces through to its connections
- whether there could be unintended consequences due to eccentricity either within the connection or between the connection and the structure, and
- how relative horizontal and vertical movements of different fixing points into the structure are accommodated (e.g. points either side of a seismic gap as well as on two levels).

Note:

These guidelines focus on building elements that pose a significant life safety hazard or are likely to cause damage to other property should they fall. A portion of their connections or fixings may be able to fail without allowing the building element to fall, either because of their location (e.g. those at the bottom of a hanging panel) or because they have some redundancy. However, it will also be necessary to confirm if such a failure could lead to premature failure of other connections or cause other building elements to fall.

C10.5.2 Inspecting connections

Issues that have been observed when inspecting SSNS building element connections and that need careful attention during inspections include:

- corrosion, particularly of hidden fixings
- mechanical damage caused during construction or service
- substitution with another type of connection during installation
- insufficient strength and/or ductility and/or deformation capacity
- fasteners within regions of primary structure that are likely to suffer significant deformation, e.g. an active link zone within an eccentrically braced frame or plastic hinge regions within concrete beams
- the use of welds near areas of high stress concentration

- poor workmanship or design, particularly for anchors
- higher capacity connections that have no mechanical interlock with the element (or structural) reinforcement, and
- connections that rely on shallow embedded or drilled-in anchors.

Holes and slots in connections often have insufficient clearance to accommodate the cumulative effects of fabrication tolerances, installation tolerances and deformations between two levels or sections of the primary structure. Additional issues that affect connections with slotted holes include:

- slots that are too short to accommodate the deformation demands
- bolts that are too close to one end of the slot when the element is installed, and
- possible constraints to the free movement of bolts as washers are missing (which can lead to binding) or the bolts or nuts are more than snug tight.

Note:

Unless the bolts in a slotted connection can be left loose, they may not be able to provide the degree of separation necessary to prevent damage to the SSNS building element or avoid the SSNS element from participating as part of the primary lateral structure. Bushes can be used in the slots to provide confidence that the slot will behave as intended without binding if the bolts have been accidentally overtightened.

The type, size, spacing and condition of fixings such as bolts, rivets and screws will often be critical when determining their capacity. Although it may be difficult and impractical to confirm every fixing, checks should be made to confirm the general arrangements and condition. Consideration could be given to destructive or proof testing of one or more fixings if there is doubt about what is hidden beneath the surface of concrete or these are otherwise inaccessible.

Refer to the appropriate material sections (Sections C5 to C9) for additional guidance on inspecting the:

- material the element is formed from and any fixings into it
- material used for the body of the connection, and
- fixing attaching the connection body to the structure.

C10.5.3 Force-based connection assessments

For force-based assessment of SSNS element connections, the part ductility factor should be taken as μ_{p} = 1.0 unless it can be demonstrated that a higher factor is appropriate.

The probable connection capacity should be taken as the minimum of the capacity of the fixings into the component, the fixings into the structure and the connector strength. The earthquake score can be evaluated as the probable connection capacity divided by the critical connection demand.

The probable connection capacity should be determined in accordance with either the relevant material sections in Part C of these guidelines or other recognised sources such as technical literature from manufacturers. The probable capacity of fixings cast into concrete can be calculated in accordance with Section 17.5 in NZS 3101:2006.

Note:

The additional requirements for fixings in Section 17.5 of NZS 3101:2006 should be considered when assessing the probable connection capacity.

The capacity may be affected by structural deformations in another direction.

Forces are only likely to be uniformly distributed between connectors when the body of the element and the structure are both rigid relative to the connector stiffness. A tributary mass method may be more appropriate for heavy elements, with appropriate adjustments where there is little or no connector redundancy.

The probable seismic capacity of adhesive anchor systems is not covered by NZS 3101:2006 and may be assessed using ACI 355.4:2011 Qualification of Post-Installed Adhesive Anchors in Concrete.

C10.5.4 Deformation-based connection assessments

For deformation-based assessment of SSNS element connections, consideration should first be given to the consequences of failure. If the connection is the weakest link within a relatively rigid element and is therefore likely to fail when this element is subjected to the types of seismic deformation demands defined in Section C10.4.5, the earthquake score is evaluated using the probable deformation capacity of the connections.

Note:

The deformation capacity of a connection for a rigid building element will often be limited by the observed clearance or length of slot as noted in Section C10.5.2.

Flexible building elements that are able to accommodate some or all of the structural deformations may require a combination of force and deformation-based assessment.

C10.6 Specific Element Capacities

C10.6.1 General considerations

SSNS elements are often assemblies of members that collectively define the strength and deformation capacity of the element. Behaviour of the elements (including precast panels, stairs, ceilings, and bracing) is dictated by physical properties such as their area, material grade, thickness, depth/slenderness ratios, lateral torsional buckling resistance, and connection details. Connected members include sheet products, light gauge steel, bracing, stiffeners, struts, and frames.

The probable capacity of SSNS elements should be determined using the probable material strengths as outlined in Sections C5 to C9 depending on the type of material. No strength reduction factor needs to be applied (i.e. use a strength reduction factor of $\phi = 1.0$) and probable material strengths can be used.

The inspected physical dimensions of individual members/elements that are being relied on for load transfer should be used. Modifications to member capacities can be caused by notching and holes. The presence of decay or deformation should be noted and allowed for.

C10.6.2 URM parapets/chimneys

The seismic responses and earthquake scores of URM parapets and chimneys will often be governed by their aspect ratios, condition, construction details and any existing restraint details.

C10.6.2.1 Inspection

Common issues that have been identified in seismic assessments of these elements and that need careful attention during inspections include:

- insufficient or no restraint to prevent out-of-plane movement/toppling
- roof flashings that are often chased into masonry just above roof level, creating a potential weak point where rocking can occur
- out-of-plumb parapets or chimneys that have a reduced capacity to resist toppling in the out-of-plumb direction
- architectural features such as concrete strips or beyond-vertical facings that can create potential weak points or impose overturning forces
- poor condition of restraint members or the structure they are attached to, and
- elements that are able to fall onto and cause damage to adjacent property.

C10.6.2.2 Assessment

The probable capacities of URM parapets and URM chimneys should be calculated in accordance with the “vertical cantilevers” subsection within Section C8 of these guidelines.

The earthquake score of a rectangular URM chimney is the lower of the scores for each of the two principal axis directions.

The assessment needs to consider the potential life safety hazard and the potential for damage to adjacent property should a parapet or chimney fall.

Note:

Chimneys falling onto iron roofs, or through tiles into ceiling spaces with a mitigation measure such as a plywood deck, may not result in a significant life safety hazard and therefore may not need to have an earthquake score. Similarly, a chimney falling onto an iron roof and sliding off will only be a significant life safety hazard if the rubble can fall onto an area where a number of people could be at risk such as paths and entryways.

Assessments of URM parapets and URM chimneys with existing bracing members need to consider the following issues that were identified following the Canterbury earthquakes:

- any interaction between the response modes of the URM element and the support structure that the other end of the bracing member connects to
- the degree of deformation compatibility between bracing support points when a URM parapet/chimney is braced by more than one bracing member
- the strength of the bracing member, its fixings, and the structure it is fixed to, and
- early pull-out of adhesive anchors due to poor workmanship, poor design, or their being installed in URM flexural tension zones.

C10.6.3 Masonry veneers

Earthquake scores for masonry veneers are often governed by the locations and condition of their internal ties. Common issues that are identified in seismic assessments of masonry veneers and that should be considered by the engineer include:

- insufficient numbers of ties to restrain out-of-plane actions
- ties that can be rendered ineffective by inadequate anchorage, weak mortar or corrosion
- the capacity of veneers tied to a main wythe in cavity wall construction can depend on the capacity of the main wythe, and
- insufficient clearance to prevent in-plane loading from the primary structure.

Note:

The location of a masonry veneer may determine how it is to be considered in terms of a significant life safety hazard or its potential to damage other property if it were to fall. There is a greater life safety hazard when a veneer is situated above an egress path from the building or public thoroughfare. Other areas around the perimeter of the building are less critical and may not need to be considered.

C10.6.3.1 Inspection

Identify the high-hazard locations and determine the thickness of any veneers. A good indication of the thickness is often obtained by examining a corner intersection of the veneers. The interlocking bricks should have a brick end visible in every second course, which will identify the thickness. If it is impossible to identify the veneer thickness in this manner, it may be necessary to drill a small hole through the veneer to physically measure its thickness.

Tie locations can be identified using a metal detector from the outside surface of the veneer. This investigation will also provide information on the spacing of the ties. Once located, a hole can be drilled through the veneer and an endoscope inserted to establish the type and condition of the ties. Note that this requires specialist equipment and operators. An individual brick can also be removed (for later replacement) to gain access to the cavity.

The thickness of the veneer and the presence, condition and locations of ties are used to decide whether or not further assessment is required.

C10.6.3.2 Assessment

Brick masonry veneers and requirements for ties shall be assessed in accordance with Section C8 with the demands determined in accordance with Section C3.

Supporting structure shall be assessed in accordance with the requirements for the appropriate material (refer to Section C5 to C9).

C10.6.4 Heavy non-loadbearing partition walls

Heavy non-loadbearing partition walls usually consist of:

- unreinforced clay brick masonry
- hollow clay brick masonry (which can be filled or unfilled, reinforced or unreinforced)
- unreinforced concrete block masonry (which can be solid or hollow, unfilled, partially filled or fully filled), or
- reinforced concrete block masonry (which can be partially filled or fully filled).

Common issues that have been identified in seismic assessments of heavy non-loadbearing partition walls include:

- insufficient or missing restraint at the tops of the walls that is required to prevent out-of-plane movement, forcing the walls to act as cantilevers, and
- insufficient gaps between ends of walls and the main structure to allow for inter-storey drift, which can result in damage to the walls and/or the main structure that may not have otherwise occurred.

Earthquake scores for these elements will often be governed by their aspect ratios and connections.

The probable out-of-plane capacity of URM non-loadbearing partition walls should be calculated in accordance with Section C8 of these guidelines as “wall elements under face load”.

The probable in-plane capacity of URM non-loadbearing partition walls should be calculated in accordance with Section C8 of these guidelines as “walls under in-plane load”.

C10.6.5 Precast concrete panels

Precast concrete panels for the purposes of this section include all tilt-up and cladding/façade variants. Where these are an integral part of the primary structure, even if this is unintentional, they should be considered as primary structure.

Earthquake scores for precast concrete panels will often be governed by their connections, which should be inspected and assessed using the guidance in Section C10.5.

The assessment of precast concrete panels normally requires a combination of a force-based approach and a deformation-based approach.

C10.6.5.1 Inspection

Inspections of precast panels should aim to identify the following high-risk features:

- panels that do not have sufficient clearance with adjacent panels to accommodate inter-storey drift
- panels attached to two floor levels using slotted connections
- panels that may collide at building corners
- L-shaped panels that wrap around building corners, and
- panels that are supported by cantilevered concrete slabs, or with large eccentricities to the supporting structural element.

Note:

Following the February 22, 2011 Christchurch earthquake, several precast panel connections that were slotted to accommodate movement were observed to have been installed incorrectly, e.g. with washers welded in place or the bolt at one end of the slot, as shown in Figure C10.3 (Baird et al., 2011). These slotted connections were not able to accommodate movement as intended.

Figure C10.3: Precast connections with no remaining sliding movement allowance to accommodate inter-storey drift (Baird et al., 2011)

C10.6.5.2 Force-based assessment

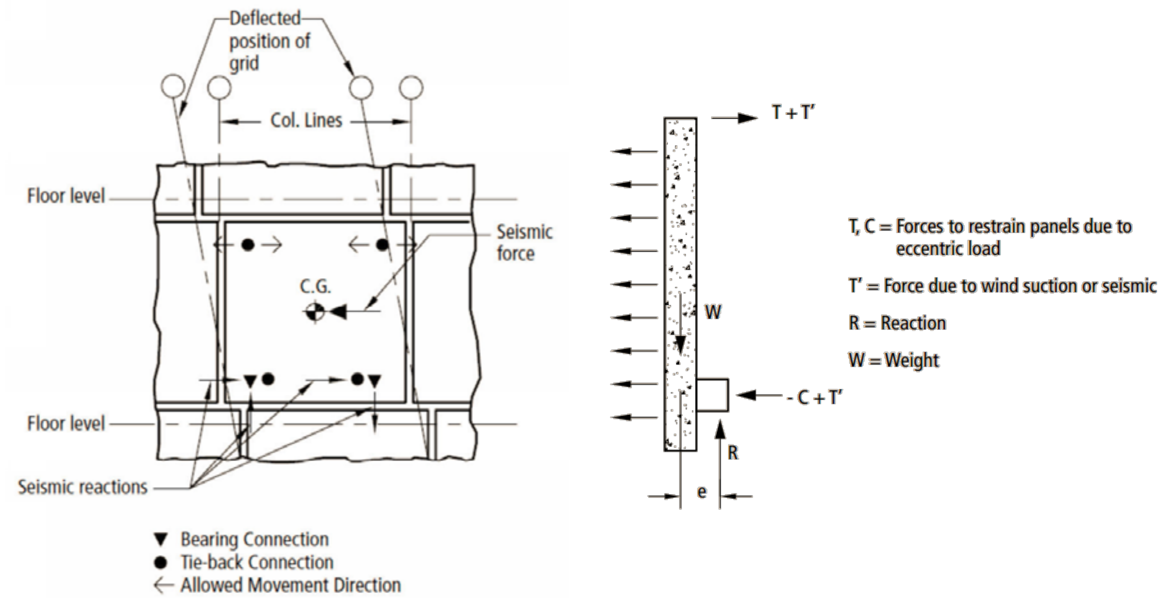
A force-based approach is required to assess the in-plane and out-of-plane demands and capacities of precast panels.

Note:

When calculating seismic demands, the NZS 1170.5 part spectral shape factor can be conservatively taken as $C_{\mathrm{i}}(T_{\mathrm{p}})$ = 2.0. The fundamental period of vibration for most precast panels, T_{p}, will be less than 0.75 seconds. Similarly, the NZS 1170.5 ductility of the part should be taken as $\mu_{\mathrm{p}} = 1.0$ unless the specific detailing of the connections can be verified as per Section C10.5.2.

It is important to consider how the seismic demands are distributed to each of the precast panel connections. For example, in-plane seismic demands for a single-storey panel are typically resisted by a pair of fixings at the base of the panel. These connections have to resist the total horizontal in-plane demands and the additional vertical demands induced by over-turning in the panel, as shown in Figure C10.4(a).

Out-of-plane seismic demands can often be considered as a uniformly distributed load that is resisted by all panel connections or that may be distributed using tributary areas. Additional out-of-plane demands will also be generated by the eccentricity between the panel and supporting primary structure, as shown in Figure C10.4(b).



(a) In-plane demands from deformation of primary structure relative to a precast panel

(b) Out-of-plane demands, including eccentric support by the primary structure

Figure C10.4: Simultaneous seismic demands on a precast concrete panel (PCI, 2007)

Section C10.5.3 provides guidance on evaluating the seismic demands and capacities for precast panel connections. If required, the out-of-plane capacity of the panel can be assessed in accordance with the BRANZ Design Guide – *Slender Precast Concrete Panels with Low Axial Load* (Beattie, 2007).

Note:

The capacity of the connection is typically more critical than the out-of-plane capacity of the panel. The out-of-plane capacity of the panel is more critical when the panel is very slender, such when the height-to-thickness ratio is greater than 60 (Beattie, 2007), or when reinforced using non-ductile mesh.

C10.6.5.3 Deformation-based assessment

A deformation-based approach is also required to assess the deformation capability of the precast panel and its connections. The lateral displacements of the primary structure can be used to determine the horizontal deformation demands for the panel and its connections.

The horizontal deformation demands are the drift between fixing points of the panel. For full storey height panels, this will be equivalent to the inter-storey deformation evaluated as described in Section C10.4.5. These demands should include any additional deformations, such as frame elongation and vertical deformations.

The high in-plane stiffness of precast panels normally requires the connections to accommodate all of the relative deformations between the primary structure and the panel during earthquake shaking, as shown in Figure C10.4.

A slotted or flexible connection is often used in modern structures to accommodate this relative deformation. The earthquake score can be evaluated as the lateral deformation capacity of the connections divided by the lateral deformation demands.

Older structures are not commonly detailed to accommodate these deformations; however, the panels and their connections may be able to deform and accommodate some lateral deformation. In such cases, loads will be introduced into the panel and the engineer should then consider the capacity the panel and connections have to transfer these loads when assessing the earthquake score. Potential failure mechanisms include failure of the connections (bolts shearing off, brackets yielding, and fixings pulling out) or shear/flexural damage to the panel.

Earthquake scores for precast panels should reflect the high-risk features identified during their inspection (refer to Section C10.1.1 earlier).

C10.6.6 Stairs and ramps

Stairs and ramps are part of the gravity structure. They are not generally elements within the primary lateral structure, but ramps may be designed to be an integral part of it.

Both stairs and ramps are required to support gravity loads and may be the only means of escape from the building following an earthquake. Collapse of one flight of stairs can lead to a pancaking effect as this progressively overloads each flight below it (as was observed in the Forsythe Barr building in the 22 February 2011 Christchurch earthquake). This cuts off the means of escape as well as being a significant life safety hazard.

C10.6.6.1 Inspection

The earthquake scores for stairs and ramps will often be governed by their connections. Therefore, the first step in an inspection is to identify how the stair or ramp element is connected to the primary structure and how it is likely to respond to movement of the upper floor relative to the lower floor. Straight stairs may be attached at both ends but are often free to slide longitudinally at one end (usually the bottom), or at an intermediate landing.

Identify the support provided for any mid-height landings – particularly for return stairs that are often supported along one edge of the landing – and how the landing and flights will be affected by lateral movements. Landings in straight stairs may only be supported by the flights.

Note:

There may be different degrees or types of connectivity in the longitudinal direction (along the flight) and transverse direction.

There may be unintentional connection (e.g. when a gap becomes plugged or closed) or disconnection (e.g. when a ledge has insufficient width) as the primary structure deforms.

Stairs with inadequate allowance for some primary structure deformations can become an unintended strut between floors and should be considered as an element within the primary lateral structure when assessing their earthquake score. Their potential effect on the primary structure, including the impact on plan regularity, also needs to be considered.

Straight stairs were shown to be more likely to pose a significant life safety hazard than return stairs in the Christchurch earthquake. However, damaged landings in return stairs may affect building evacuation.

Common issues that have been identified in seismic assessments of stairs and that need careful inspection include:

- slotted holes, sliding gaps, and ledge bearing dimensions, which are often insufficient to accommodate the cumulative effects of fabrication tolerances, installation tolerances and primary structure deformations
- ledge widths that can be reduced by spalling along the edge
- sliding gaps at the base of precast concrete stair units, which often become filled over time and could impair their function or lead to premature stair failure due to axial compression (refer to Figure C10.5), and
- mid-height landings and flights of return stairs that may warp (vertically) to accommodate horizontal deformations of one floor level relative to the other.

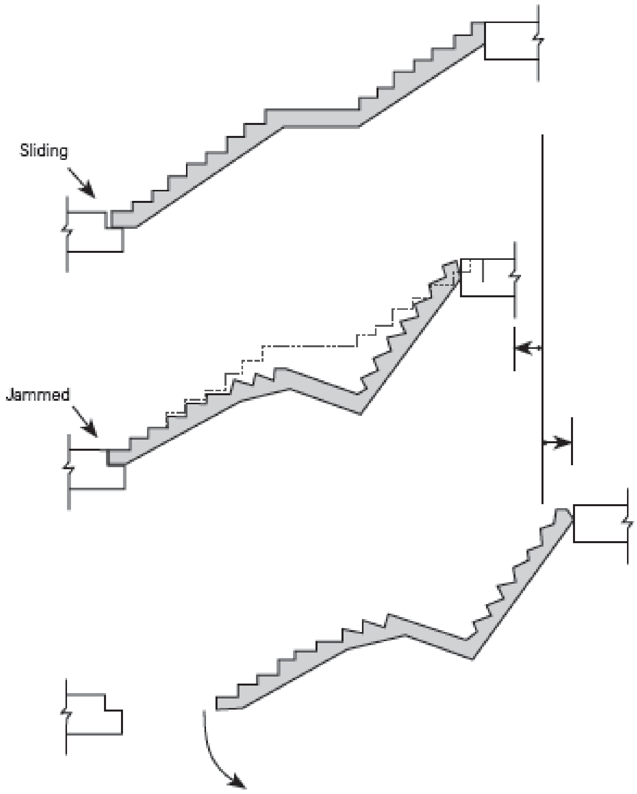


Figure C10.5: Straight stair failure as the upper floor moves left (middle) and then right (bottom) relative to the lower floor (Beca, 2011 and Bull, 2011)

Additional guidance is provided by DBH (2011), SESOC (2013) and FEMA E-74 (2012).

C10.6.6.2 Force-based assessment

Force-based assessment is generally required to verify that the connection capacity transverse to the ramp or stair flight(s) is capable of restraining the ramp or stairs. The transverse strength of the ramp or stair flights is unlikely to be of concern because the loading will be in plane.

C10.6.6.3 Deformation-based assessment

The deformation demands for ramps and stairs are calculated using Section C10.4.5.

The deformation capacity depends on the geometry of the ramp or stairs and how they are connected (or not) to the structure at the two floor levels. Sliding joints have three states: sliding (or free); jammed; and unseated (or falling), as illustrated for straight stairs in Figure C10.5. The undesirable jammed state induces similar compression forces in the element to those that would occur if it was connected at both ends. However, it may not pose the same life safety risk as the unseated or falling state. Stairs connected at both ends have a fourth state whereby the flight becomes a tie and is stretched.

Deformations that are transverse to straight stairs are less likely to pose a life safety hazard because there will usually be a portion of the seating supporting the element. However, for wider ramps, that could even be wide in comparison to their length or span, and could become unseated over sufficient length to jam as soon as the deformations reverse.

The earthquake score for sliding joints should be taken as the lesser of the following:

- the seating length (less reductions due to spalling, etc.), divided by the
- the available gap, divided by the required seating length from NZS 1170.5:2004, and
- the compressive capacity of the element divided by the difference between the deformation demand and the available gap (see below).

Steel stair flights may be capable of accommodating larger inter-storey deformations than reinforced concrete flights using slotted hole connections. However, if the bolts reach the ends of the slots, parts of the stair may yield or deform in a manner that does not result in collapse. If the slot length is insufficient to accommodate the expected inter-storey drift and there is the potential for ductile deformation of the steel elements, the bolt fixings should be checked to ensure that they are not weaker than the steel elements.

When a ramp or stairway is attached at both ends of the flight or is jammed, compressive loads will be induced in the stairs. Their earthquake scores will depend on their capacity to resist these loads (in which case they will influence the behaviour of the main structure) or, if they are not strong enough, on how they will fail. Potential failure mechanisms include failure of the end connections (shearing of bolts, yielding of rebar, fracture of timber connections), buckling or compressive failure of the stair flight, and shear failure of the stair flight.

Yielding steel may prevent collapse of a reinforced concrete flight that is being stretched, provided the steel is well anchored into both the floor and the flight. However, compression forces that develop when the motion reverses could lead to shear failure at one or both ends of the flight, and possibly even collapse. Understanding the detailing of reinforced concrete stairs is important for judging the likely earthquake behaviour.

Steel framed structures may have consequences that are similar to those for reinforced concrete structures. Again, the analysis of the main structure behaviour will influence the stair behaviour.

Stairs in timber structures that are fixed at both ends are more likely to influence the behaviour of the primary structure because the diagonal stair linking the floors may have a similar stiffness to the primary lateral structure. Large timber and steel stair stringers are likely to transfer significant forces without overloading the stair. However, the connections between the stair and the floor may fail, either by extracting the fixings from the floor or through local failure of the timber at the connection (refer to Figure C10.6). This could lead to collapse of the flight in an opening situation or the top of the flight could rise above the floor level, as shown in this figure.

Figure C10.6: Stair that has sheared connections to the floor on a timber framed structure and ridden up above the floor (BRANZ)

C10.6.7 Heavy plant, storage racking, and hazardous substances and new organisms (HSNO) vessels

Earthquake scores for heavy plant, storage racking and HSNO vessels will often be governed by the capacity of braced frames and the capacity of connections to the main structure.

Heavy plant includes air conditioning/handling units, pumps and chillers, although this list is not exhaustive. Minimum mass limits for heavy plant, depending on the location of the plant item and its contents, are given in Section 8 of NZS 1170.5:2004. Boilers have additional considerations that are addressed in the specific guidance for HSNO vessels below (Section C10.6.7.4).

Storage racking has a wide range of applications, ranging from low level shop racking (typically less than 2 m high for the public to pick items from stock) to 10 m or more in height for warehouse storage applications. This guidance is for high level storage racking (>2 m high) because lower racking generally only poses a low risk to life safety.

HSNO vessels are tanks or containers with substances that would pose a significant health hazard if the vessel or its restraints failed or the tank ruptured and the contents were spilled. The contents of some pressure vessels (e.g. boilers) are stored at high temperature and pose significant scalding and explosion hazards.

Note:

HSNO vessels may or may not be marked as containers of hazardous substances, depending on the type and volume of the substance. There is guidance on labelling in the *Approved Code of Practice for the Management of Substances Hazardous to Health in the Place of Work* (Department of Labour, 1997) and the *Stationary Container Systems Performance Standard* (Worksafe New Zealand, 2015).

For pressure vessels, AS/NZS 1200:2015 *Pressure Equipment* refers to NZS 1170.5:2004 and to IPENZ Practice Note 19 *Seismic Resistance of Pressure Equipment* (IPENZ, 2016) for the seismic design of boilers, pressure vessels and pressure piping.

C10.6.7.1 Inspection

Common issues that are identified in seismic assessments of heavy plant, storage racking and HSNO vessels that require consideration when inspecting them include:

- insufficient bracing and inadequately proportioned frame members
- insufficient connection capacity, particularly to the primary structure, and
- insufficient allowance for differential movement between plant and associated piping.

Note:

The contents of an unmarked vessel should be assumed to be hazardous unless proven otherwise.

C10.6.7.2 Assessing heavy plant

Heavy plant items may provide significant driving mass for the primary structure but do not otherwise influence the structure response. Plant items with a mass exceeding 20% of the combined mass of the plant item and the structure and a lowest translational period greater than 0.2 seconds are expected to affect the behaviour of the primary structure. Such situations require special consideration during the assessment of the primary structure and the dynamic characteristics of the item. This should be undertaken in accordance with the rest of Part C of these guidelines.

Likely failure mechanisms and their consequences with respect to life safety need to be identified for heavy plant items. The consequences of failure can be variable. The location will often determine its potential life safety hazard along with the consequences of its failure. For example, a container discharging a hazardous substance into an isolated location may pose minimal life safety risk but could still pose a serious health risk.

For those items that are considered to pose a significant life safety risk, the engineer should ascertain the method(s) of support to determine whether the plant item will be influenced by force or deformation. Floor mounted and ceiling supported plant are likely to be most influenced by lateral forces. However, ceiling mounted equipment may also require deformation considerations and the effects of impact against the main structure or other components. Floor mounted plant may also be connected to pipework and electrical trunking which is attached to the level above. Consideration should be given to the interaction between the plant item and these items, keeping in mind the potential to cause a significant life safety hazard.

Heavy plant items within a building are categorised as “P2 and P3” in accordance with NZS 1170.5:2004 Table 8.1. However, these items may also be categorised as:

- P5 when in importance level 4 (IL4) buildings or they are required to remain operational for the building to be occupied, or
- P6 if their failure could cause disproportionate damage (e.g. water loss above perishable goods).

It should be noted that neither the P5 nor the P6 cases are related to life safety as considered in these guidelines, notwithstanding that damage to perishable goods may become a health risk if they cannot be removed quickly.

The engineer should determine the demands on the plant item restraint system in accordance with Section C3.

The earthquake score can be evaluated as the probable capacity of the restraint system, including the connections divided by the demand on the restraint system.

C10.6.7.3 Assessing storage racking

High level racking systems (those over 2 m) are considered to be structures in their own right and are therefore subject to the requirements of the New Zealand Building Code (NZBC). However, there are many storage racks between 2 m and 4 m high in commercial premises that were unlikely to have been designed and installed to meet the NZBC.

Racks are normally only installed at the ground floor level of a structure to provide forklift access for loading and unloading. Therefore, no site ground motion amplification will be necessary.

However, some racking systems are installed on a suspended floor slab (e.g. supermarket racking in a store with parking underneath). In these cases, the engineer needs to consider the effect of the supporting structure on the rack performance when assessing the probable rack capacity. The strength of the supporting structure should also be considered to ensure that the seismic loads introduced to the floor can be adequately resisted (refer to Section C5).

The majority of racking systems will be proprietary products and, if it is possible to ascertain the brand of the rack, an approach to the supplying company is recommended to obtain design information. These companies may also be able to supply the design capacity using in-house design software.

High level storage racks are quite different from building structures in that they use simple principles in their vertical and lateral load-resisting systems and they generally carry large loads; normally much larger than the weight of the rack itself. Therefore, it is important to identify the characteristics of the loads stored on the racks. There is also little redundancy available in the system, so the failure of one element can lead to a domino effect on the rest of the rack.

Racks have an advantage over a building structure in that, as they are lightweight compared to their contents, their performance can be improved by strategically re-arranging the contents so the heavier items are near the floor and lighter items above. Even when there is no opportunity to re-arrange the stock to reduce demands on the rack, there is still an opportunity to partially unload it and set appropriate loading limits.

Note:

The engineer needs to be sure that any content stacking arrangements or loading limits can be managed on an ongoing basis if these are to reduce the demands on the racks and avoid life safety (or any other) consequences.

When assessing the seismic capacity of the racking system, the load-resisting mechanisms should be checked first. In the down-aisle direction, the critical factor will usually be the bending capacities of the connections between the uprights and the beams, because these members are normally stronger than these connections. In the cross-aisle direction, lateral load resistance is provided by braced transverse frames. Racks have minimal reliable diaphragm action at the beam (or shelf) levels, which is provided by friction forces between stored pallets and the beams.

The seismic demand should be assessed in accordance with NZS 1170.5:2004. For the cross-aisle direction, a maximum ductility factor of 1.25 should be used, unless cyclic test results are available for representative frames. In the down-aisle direction a ductility factor of 1.25 should also be used unless cyclic tests on connections demonstrate ductile behaviour. While not so critical in the down-aisle direction, the probable capacities and ductilities of the cross-aisle frame baseplates and anchor bolts should be estimated in order to determine the cross-aisle capacity. Anchor bolts are seldom cast-in-place so their capacity will generally be less than that of a cast-in anchor. In the absence of information on the anchor bolts, one of these should be removed to determine its properties. An assessment of the anchor capacity may be made using NZS 3101:2006 section 17.

The earthquake score can be evaluated as the probable capacity of the racking system and its connections to the main structure in both the down-aisle and cross-aisle directions divided by the critical demand on the rack and its restraint system. The structure to which the rack is attached should also be considered, and the minimum earthquake score will be the least of these values.

C10.6.7.4 Assessing HSNO vessels

HSNO vessels should be assessed using either a force-based or deformation-based assessment as appropriate.

Unmarked vessel contents should be assumed to be hazardous unless proven otherwise, and assigned a Part category of at least P2 or P3 in accordance with NZS 1170.5:2004 Table 8.1. Refer also to the notes beneath that table for further direction on inclusions and exclusions.

Forced-based assessment of HSNO vessels

HSNO vessels will normally be mounted on the floor of a building structure unless they are light enough or have a small enough volume to be mounted on a wall. In the absence of specific information on the contents and their degree of hazard, the engineer should judge the likely effects of failure given the position of the vessel and his/her assessment of the contents, and then use NZS 4219:2009 or NZS 1170.5:2004 to determine the demands on the vessel and its restraint system.

The earthquake score can be evaluated as the lowest of the probable vessel or restraint capacity, including the probable capacity of the connections, divided by the critical demand on the vessel or its restraints, respectively.

Deformation-based assessment of HSNO vessels

HSNO vessels will generally not be stand-alone items that are only connected to the structure through the restraint system. Supply and/or distribution pipes may also be connected to the vessel. The failure of these may be as catastrophic as failure of the vessel itself.

If the only pipework is on the same floor as the HSNO vessel, the restraints provided for this pipework should be assessed for their capacity against critical demands. The potential for stiffness incompatibility between the vessel and pipework restraints and the effect on the integrity of the system should be assessed. Flexible joints may be already in place or all connections may be rigid.

If the pipework is restrained at a floor level above the one on which the vessel is mounted, there will be a degree of inter-storey drift that should be accommodated.

The maximum demand drift may be calculated using Section C10.4.5. A judgement will be required as to the ability of the pipework and its connection to the vessel to accommodate this demand.

For pipework that spans between the vessel and the floor above, the earthquake score can be evaluated as the probable deformation capacity of the pipework divided by the demand deformation.

C10.6.8 Curtain wall framing systems

Earthquake scores for curtain wall framing will often be governed by the ability of the system to accommodate the lateral deformations of the primary structure. A force-based assessment is only required for heavy curtain wall framing systems; i.e. those that weigh more than 70 kg/m^2.

C10.6.8.1 Inspection

Common issues that have been identified in seismic assessments of curtain wall framing include:

- limited or no ability to accommodate inter-storey deformations, and
- in the event of failure, that significant areas of the glazing or curtain wall may fall.

Mitigation measures may be provided to limit the life safety risk should a curtain wall or its large or hazardous components fall. Where these are provided, they should be inspected first because further inspection and assessment may not be required.

Examples of mitigation measures include situations where the curtain wall is above an area that is inaccessible and low hazard glazing has been used; for example toughened glass (also known as tempered glass), laminated glass, or wire mesh glass (also known as wired glass).

Note:

Toughened (or tempered) glass greatly reduces the seismic hazard because the glass breaks into small dull fragments instead of large hazardous shards. Toughened glass can sometimes be identified by a small label etched in the corner of the glass. Look for words such as "TEMPGLASS", "TEMPERED", "TOUGHENED" or "SAFETY". If no label is present, toughened glass can also be identified by looking at a light source on the other side of the glass through polarised lenses. If the glass is toughened it will reveal lines or spots in the glass that are not normally visible.

Laminated glass has a film layer that does not allow for the glass to separate when broken. This reduces the risk of the glass falling out of the curtain wall and also prevents people or objects from falling through the glass. Laminated glass is commonly used where there is the possibility of impact by a person, such as next to a walkway. Laminated glass is more difficult to identify as it is less likely to have a label. It is possible to identify laminated glass by looking for multiple reflections when an object is placed next to the glass. Glass that is not laminated shows only two reflections from the two surfaces of the glass.

C10.6.8.2 Force-based assessment

The seismic demands on connections for these systems can be determined using Section C10.5.3.

Note:

The double-skin systems shown in Figure C10.7 are examples of curtain wall framing that would require a force-based assessment.

As most residential and commercial curtain wall systems are lighter than 50 kg/m^2, a force-based assessment is not often required.

Figure C10.7: Double skin curtain wall system

Note:

Wind actions typically govern the design of the connections and componentry of curtain wall systems because these systems are relatively lightweight. A force-based assessment will generally demonstrate that curtain wall systems are adequate to resist seismic actions and hence there is no need for further assessment. Observations of curtain wall framing damage during the 2010-2011 Canterbury earthquakes demonstrated that most failures were the result of deformation demands, rather than overloading of connections (Baird et al, 2011).

When calculating seismic demands, the NZS 1170.5:2004 part spectral shape factor, $C_{\mathrm{i}}(T_{\mathrm{p}})$, can be taken conservatively as equal to 2.0. Similarly, the NZS 1170.5 part ductility demand, μ_{p}, should be taken as equal to 1.0 unless it can be verified that the connections are specifically detailed to provide ductility as per Section C10.4.4.

Curtain wall connections are normally regularly spaced, so it is reasonable to consider that seismic demands will be distributed evenly between connections in both the in-plane and out-of-plane directions. Seismic demands should therefore be calculated using the tributary weight of the curtain wall multiplied by the parts coefficient.

Curtain wall connections typically comprise three components: fixing to the curtain wall transom or mullion, the connector body, and the fixing to structure. The probable connection capacity should be taken as the lowest of these three components. The earthquake score can be evaluated as the probable connection capacity divided by the critical connection demand.

The probable capacity of connections should be determined in accordance with the relevant sections within Part C of these guidelines or other recognised sources such as technical literature from manufacturers.

C10.6.8.3 Deformation-based assessment

If the engineer determines that the curtain wall itself presents a significant life safety risk if it fell from the building, then a deformation-based assessment is required. Deformation demands are evaluated using Section C2.

The deformation demands upon curtain wall framing are typically accommodated by clearances within the system between the stiff glass panes, and the more flexible extruded aluminium transoms and mullions, as shown in Figure C10.8. In higher seismic zones, or in more flexible buildings, more clearance is required than can be accommodated by the glass to frame clearance alone, so seismic mullions/transoms are sometimes provided. These have separate extruded aluminium sections that can move relative to each and accommodate the seismic movement.

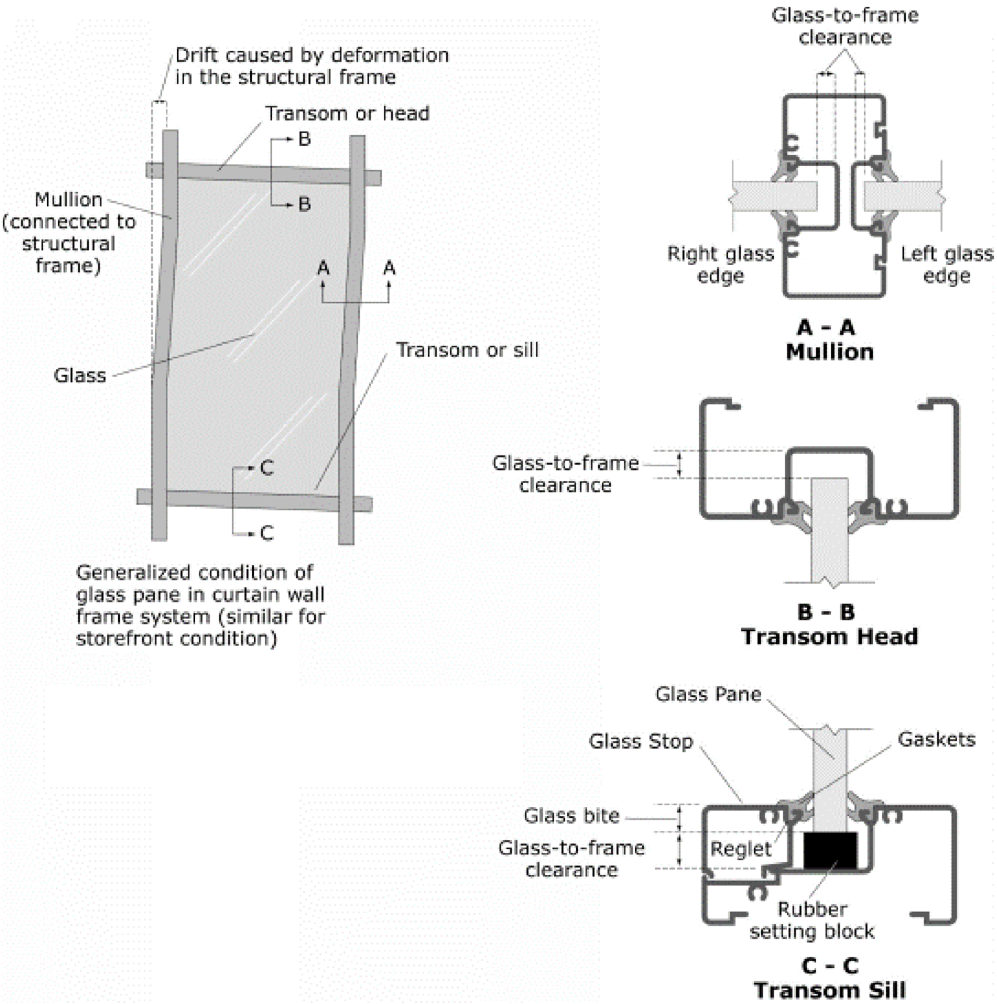


Figure C10.8: Typical curtain wall indicating glass-to-frame clearance (FEMA E-74, 2012)

The deformation demands for the individual glass panel are given by Equation C10.2223 below. The curtain wall fixing height is normally the inter-storey height. However, care is needed where curtain wall systems are fixed to precast concrete spandrel panels and the inter-storey drift is concentrated over less than the full storey height.

$$\delta_{\text{capacity}} = \frac{h_{\text{cw}}}{h_{\text{g}}} \times \delta_{\text{panel}} \qquad \ldots C10.2$$

where:

δ_{panel} =deformation capacity of an individual glass panel
h_{g} =glass panel height
h_{cw} =curtain wall fixing height.

Most of the deformation capacity in an individual panel is provided by the glass to frame clearance, or the movement capacity of mullions/transoms, which should be determined from drawings if possible. If no information is available and it is not possible to determine these by inspection of the glazing system, use the typical construction clearance of 10 mm between the glass and frame. The deformation capacity is calculated from the clearances using Equation C10.3334, which includes allowance for the glass being seated on setting blocks.

$$\delta_{\text{panel}} = c\left(2 + \frac{h_{\text{g}}}{b_{\text{g}}}\right) \qquad \text{…C10.3}$$

where:

c =clearance between sides and top of glass panel and framing
b_{g} =glass panel width.

Experimental testing has demonstrated that curtain wall systems are typically able to withstand deformations greater than the initial glass to frame contact before glazing falls out (Behr, 2009; King and Lim, 1991; and Wright, 1989). This is due to cracking of the glass, localised crushing at the contact location and deformation of the framing.

The risk of curtain wall failure is adjusted using the element performance factor, Q, with the coefficients given in Table C10.1.

Table C10.1: Element performance factor, Q, for curtain walls (adapted from NZS 4223.1:2008 and Behr 2009)

Curtain Wall Description	Q factor
Modern commercial curtain wall system	4
Pre-1980s curtain wall or residential curtain wall system	2
Frameless glazing or rigid framing	1.25

C10.6.9 Ceilings

Earthquake scores for suspended ceilings will often be governed by the spacing of bracing elements and the capacity of connections, particularly those to the primary structure.

C10.6.9.1 Inspection

Common issues that are identified in seismic assessments of suspended ceilings include:

- insufficient bracing to the primary structure above (this does not always apply to perimeter fixed suspended ceiling systems)
- insufficient capacity of connections, particularly those to the primary structure
- interconnected partition walls with inadequate separation to primary structure causing unintended restraint within ceiling systems

- insufficient clearance to in-ceiling services, and
- heavy and unrestrained ceiling tiles.

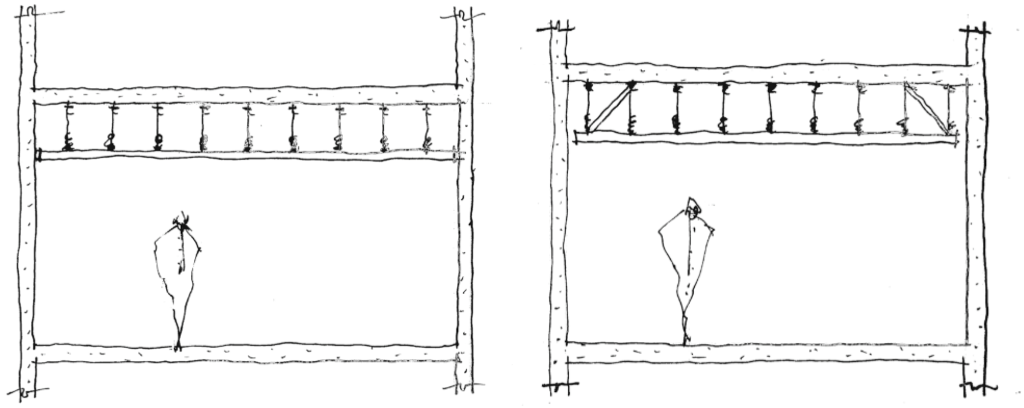
Common issues that are identified in seismic assessments of monolithic ceilings include the insufficient capacity of members and connections, particularly to the primary structure.

The assessment of ceilings should first consider whether the failure of the ceiling would create a significant life safety hazard if it were to fall, and that there is no practical mitigation available to those who would be affected by the ceiling falling (e.g. furniture under which to shelter). The assessment should aim to identify whether the following high-risk attributes are present:

- large, uninterrupted regions of ceiling with no (or very little) seismic restraint
- heavy ceiling tiles (e.g. vinyl-faced or plasterboard tiles weighing over 7.5 kg)
- partition walls fixed to the ceiling system
- building services supported directly by the ceiling
- very large plenum heights or ceiling heights
- braces installed at angles greater than 45 degrees
- no vertical stiffener present at diagonal braces.

C10.6.9.2 Force-based assessment

If a detailed assessment of the ceiling is deemed necessary, then a force-based approach is required to assess the capacity of ceiling systems. The assessment should identify what seismic restraint is provided (if any) in order to determine the component that will govern the ceiling capacity. Suspended ceilings are typically either perimeter fixed or back-braced, as shown in Figure C10.9.



(a) Perimeter fixed suspended ceiling **(b) Floating ceiling with back-braces**

Figure C10.9: Types of suspended ceiling (BRANZ, 2015)

The following steps outline the process for force-based assessment of a suspended ceiling:

Step 1

Inspect the ceiling to identify how it is restrained (perimeter fixed or back-braced) and to determine the maximum main and cross tee rail lengths in the ceiling system. Refer to

Figure C10.9 for typical ceiling restraint arrangements and to Figure C10.10 for typical back-bracing details.

Step 2

Determine the seismic weight of the ceiling per unit area, including the suspended services supported by the ceiling. In the absence of any material data, the seismic weight of the ceiling should be based on the weights provided in Table C10.2.

Step 3

Determine seismic demands per unit area of ceiling, assuming the ceiling is a part in accordance with Section C3.

Step 4

Determine seismic demands upon both the main and cross tees using the seismic demands per unit area from Step 3 and the spacing of the respective tee rails.

Step 5

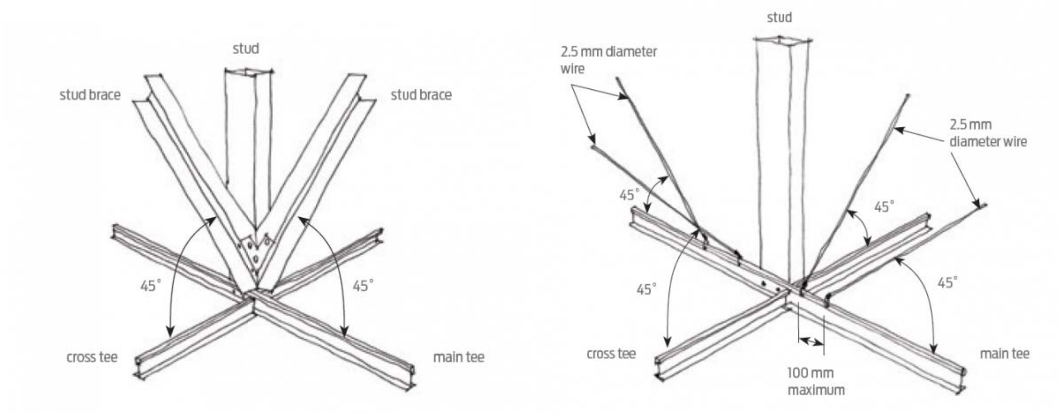
Calculate the theoretical maximum tee rail lengths by dividing the tee rail capacities by the seismic demands obtained in Step 4. In the absence of any product data, the tee tail capacities should be based on those provided in Table C10.3.

Step 6

Calculate the earthquake score by dividing the maximum theoretical tee rail lengths obtained in Step 5 by the actual lengths of tee rail identified in Step 1. If back-bracing is present, the distance between braces should be used in lieu of the actual tee rail lengths.

Step 7

If back-braces are present it is necessary to check the braces have adequate capacity for the area of ceiling they are restraining. In the absence of any product data, capacities of the components and their connections should be based on those provided in Table C10.3. For a back-braced system the earthquake score is calculated by dividing the brace capacity by the seismic demand of the area of ceiling being restrained.



(a) Strut bracing **(b) Wire bracing**

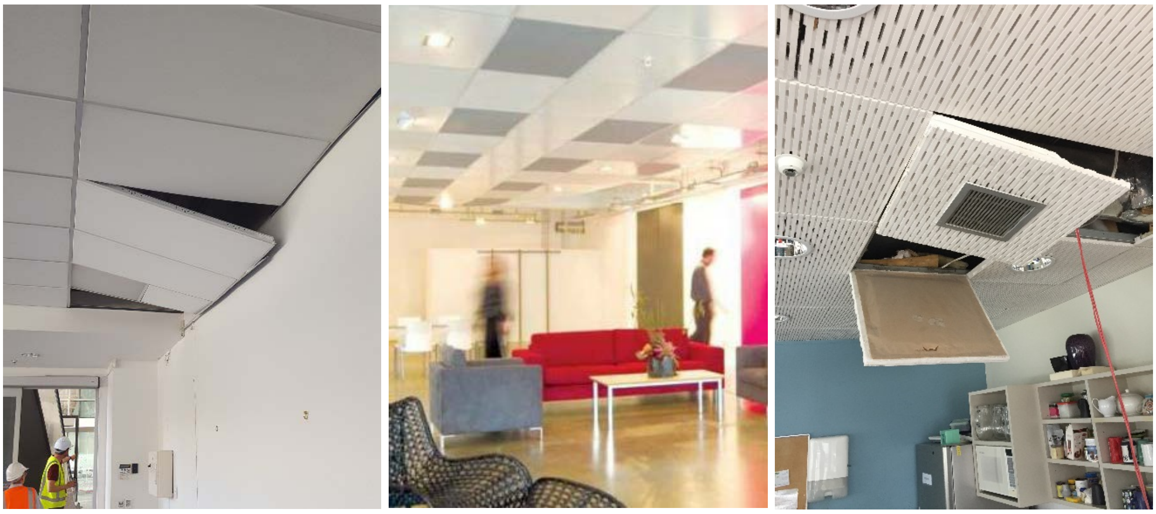
Figure C10.10: Back-braced suspended ceiling system (BRANZ, 2015)

Table C10.2: Typical ceiling weights

Ceiling type	Description	Weight (kg/m^2)
Light-weight suspended	Mineral fibre ceiling tiles, supporting typical suspended services	0.10
Medium-weight suspended	Vinyl-faced ceiling tiles, supporting typical suspended services	0.15
Heavy-weight suspended	Plaster ceiling tiles, or light-medium weight ceiling supporting heavy suspended services	0.20
Monolithic	Plasterboard lined steel stud, supporting typical suspended services	0.15

Table C10.3: Typical suspended ceiling component capacities (Armstrong, 2013)

Ceiling component	Capacity (kN)
Main tee	1.0
Cross tee	0.6
2.5 mm diameter wire bracing (at 45 degrees)	1.0



(a) Light-weight mineral fibre ceiling tile **(b) Medium-weight vinyl-faced ceiling tile** **© Heavy-weight plaster ceiling tile**

Figure C10.11: Illustrations of light, medium and heavy weight ceilings (BRANZ)

Note:

When calculating seismic demands, the NZS 1170.5:2004 part ductility demand, μ_p, can be taken as equal to 2.0 for suspended ceilings and equal to 3.0 for fixed ceilings, as per Table C8.2 in NZS 1170.5:2004.

The seismic weight of the suspended ceiling should include an allowance for a distributed service load of not less than 3 kg/m^2. This weight is included in Table C10.2 above.

In an Importance Level 4 building such as a hospital, it is recommended that under-braced suspended ceilings should be identified because the space beneath them would need to be evacuated, even though their failure would not pose a significant life safety hazard.

C10.6.9.3 Deformation-based assessment

Deformation-based assessments are only required when the suspended ceiling system is likely to impact the tops of non-structural walls that are only attached to the lower floor. This assessment will require consideration of:

- the deformation demand (refer to Section C10.4.5)
- the horizontal gap between the ceiling system and the wall
- the deformability and strength of the wall, and
- how any impacting would affect the ceiling system.

C10.6.10 Canopies, balconies, billboards and other appendages

The earthquake scores of canopies, balconies, billboards and appendages that are attached to buildings will often be governed by the capacity of their connections to the primary structure. Common issues that are identified in seismic assessments of these elements include:

- insufficient capacity of connections to the primary structure, and
- corrosion of hidden fixings.

Note:

Ratcheting should be considered on non-vertical cantilevering elements where this may affect the earthquake score of the element.

The assessment should begin by determining whether a force-based assessment, a deformation-based assessment, or both are applicable. Procedures for both assessment procedures are given below.

C10.6.10.1 Force-based assessment

The following steps outline the force-based assessment procedure for canopies, balconies, billboards and other appendages attached to buildings:

Step 1

Inspect the element to identify the properties of its structural members and how these members are connected to the building.

Step 2

Determine the seismic weight of the element.

Step 3

Determine seismic demands on the members, assuming the element is a part in accordance with Section C3.

Step 4

Calculate the theoretical maximum element capacities.

Step 5

Determine the earthquake score of each member and connection by dividing the capacity by the seismic demand

Step 6

Take the earthquake score for the element to be the lowest of the Step 5 scores.

C10.6.10.2 Deformation-based assessment

A deformation-based approach is required when it is necessary to assess the deformation capacity of the element and its connections. The lateral displacements of the primary structure derived from structural analysis can be used to determine the horizontal deformation demands on the element and its connections. The horizontal deformation demands should be taken as the drift between fixing points of the element. For full storey items, this will be equivalent to the inter-storey drift.

The deformations of the primary structure at the element connection points will need to be accommodated in the element, its connections, or both. Slotted or flexible connections are often provided in modern structures to accommodate relative deformations. The earthquake score can be evaluated as the total lateral deformation capacity of the element and connections, divided by the lateral demands calculated.

Note:

Where a slotted connection has been used, it should be checked to ensure that the slot can accommodate movement as intended.

In older structures, it is uncommon for connection movement to be accommodated. However, the element and its connections may be able to deform to accommodate some lateral deformation. In such cases, loads will be introduced into the element and the capacity the element and connections have to resist these loads is used to calculating the earthquake score. Potential failure mechanisms include failure of the connections (shearing of bolts, yielding of brackets, and pull-out of fixings) and shear/flexural damage to the element.

C10.7 Improving Seismic Performance

The process of conducting a DSA may identify potential seismic performance issues in building elements and their connections. These can be mitigated to improve their seismic performance.

Typical methods for improving seismic performance include:

- removing heavy elements such as URM parapets, URM chimneys, ceilings and heavy non-loadbearing partition walls
- improving cladding panel connections
- relocating heavy plant from high level installations to a basement or ground level
- replacing corroded bracing elements/connections
- adding supplementary bracing
- inserting joints into building elements that span across primary structure seismic joints
- improving stair bearing and sliding details
- internal post-tensioning, steel tube reinforcement, or concrete filling of URM chimneys
- adding supplementary connections and/or improving existing connections, and
- providing tethers to restrict the fall distance.

Note:

Tethers and other means of preventing a falling element becoming a significant life safety hazard need to be assessed on their ability to reliably arrest the element's fall. Refer to NZS 1170.5:2004 for guidance.

Some additional methods for improving the seismic performance of elements and their connections can be found in FEMA E-74, ASCE 41-06 (2006) and ASCE 41-13 (2014).

Note:

Some typical URM façade and URM parapet securing concepts have been published (MBIE 2017b). Designers may find these securing concepts useful when designing strengthening measures to improve the seismic performance of these elements.

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