

High-Risk Deficiencies in Category B Buildings

EPB Legislation and Methodology Guidance Annex 1





Ministry of Business, Innovation and Employment (MBIE) Hīkina Whakatutuki – Lifting to make successful

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ISSN (online) 978-1-997308-78-2

October 2026

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Document status

This resource is version 0.1 of MBIE’s EPB Legislation and Methodology Guidance Annex 1.

Document history		
Status	Date	Description
Version 0.1	October 2026	Draft for Consultation

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A1.1 Evaluating Whether a Category B Building is Earthquake-prone

A1.1.1 Process for Evaluating Whether a Category B Building is Earthquake-prone

The territorial authority determines whether a *Category B building* is an *earthquake-prone building (EPB)* in accordance with the *EPB Methodology*. The *EPB Methodology* sets out how the findings of an *HRD review*, a previous assessment and the status of an existing EPB are used in that determination, including under the identify after statutory timeframes pathway.

HRD reviews on Category B buildings must be carried out in accordance with this Annex. Category B buildings can be earthquake-prone if they are confirmed to have one or more *high-risk deficiencies*. They can also be earthquake-prone if the territorial authority is proceeding on the basis that no such evaluation has been made. Figure 1 shows this process.

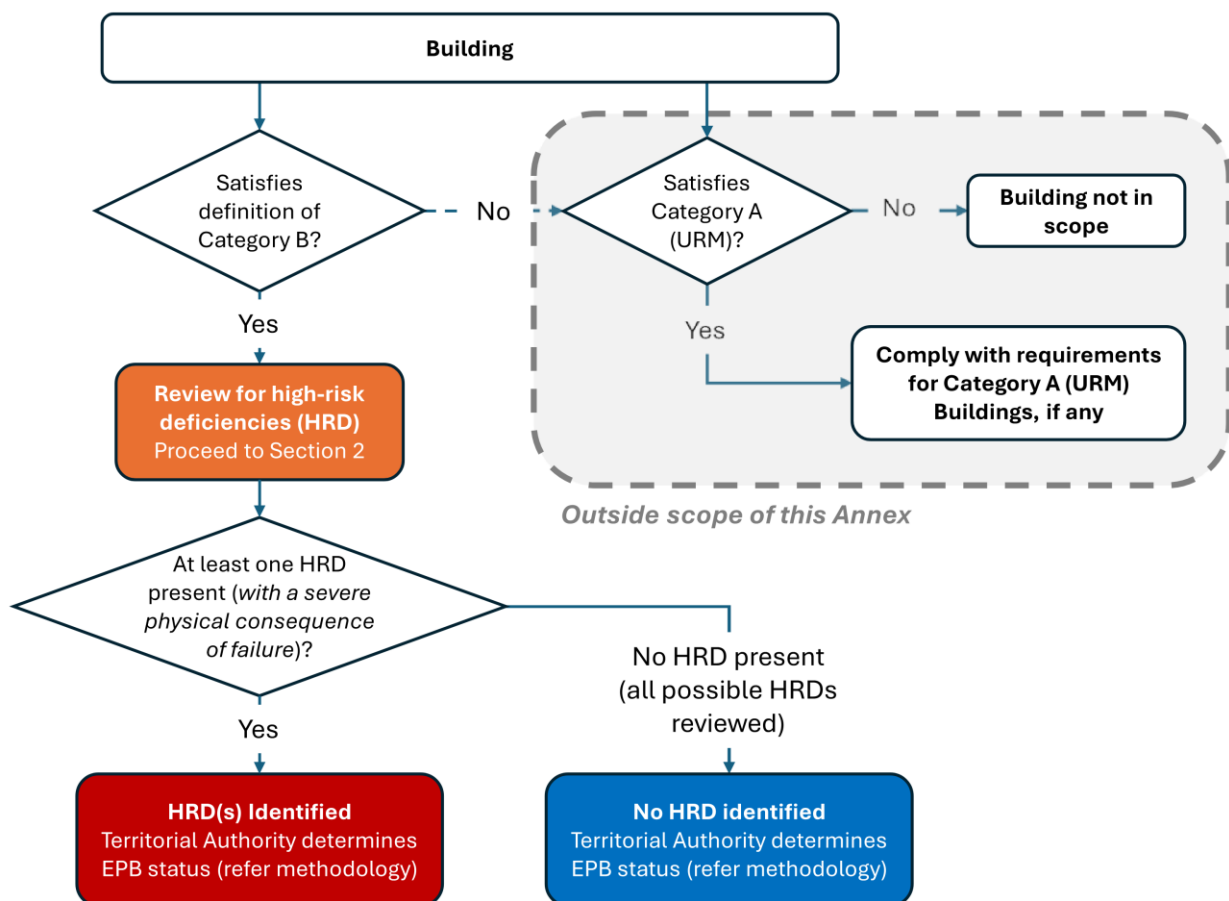


Figure 1: Process for using an HRD review to determine whether a Category B building could be Earthquake-prone

Where a *Category B building* has *unreinforced masonry (URM)* elements, it must also be checked against the criteria for *Category A buildings* (refer to the *EPB Methodology*). Additional *Category A* requirements, if applicable, must also be met and are outside the scope of this Annex. This Annex addresses *URM* elements only where they affect the stability of the building (*HRD A-04*, refer 4.5).

A1.1.2 What Defines a High-Risk Deficiency?

A *high-risk deficiency* (an *HRD*) is a specific building feature (or combination of features) that causes elevated risk resulting from partial or full building collapse in earthquake. The presence of such features is used to determine whether a *Category B building* is *Earthquake-prone*. An *HRD* can comprise any of:

1. **High-risk component-only deficiencies:** Where the risk arises from high vulnerability in the component alone.
2. **High-risk building configurations:** Exceptionally weak frames with low inherent strength and stability
3. **High-risk compounding component and configurational deficiencies:** Where the risk arises from a building *configurational vulnerability* that can amplify demands on *vulnerable components*.

Section A1.2 explains how to review for *HRDs*. Only the building features specifically listed in this Annex (Table 4, Section A1.3.2) can be *HRDs*, and there must be a *severe physical consequence of failure*.

A1.1.3 Terminology and Notation

A1.1.3.1 Defined Terms and Acronyms

Table 1: Defined terms and acronyms used in this Annex

Term	Definition
Benchmark seismic hazard	The reference level of earthquake shaking set out in Annex 2, 2.1.1 and used by sections of this Annex to identify the general magnitude of earthquake demand in a selection of components so that their vulnerability can be appropriately classified in <i>HRD review</i> .
Boundary area	Boundary area of a wall as used in provisions for assessing susceptibility to through-the-thickness axial failure (see section A1.6.6.2).
Category A building	An unreinforced masonry (URM) building that either has at least one <i>façade requiring risk mitigation</i> or has three or more storeys and is in an urban centre. See the <i>EPB Methodology</i>
Category B building	A Category B1 or B2 building, as defined in the <i>EPB Methodology</i> .
Column Index, CI	See <i>Column Wall Index, CWI</i>
Column Wall Index, CWI	The ratio of effective cross-sectional area of vertical structure within a storey (columns, walls, and braces) to the total gross floor area supported above. Details are defined specifically in section A1.8.1 of this Annex. The <i>CWI</i> is computed along each building axis and at

	sufficient storeys to confirm the critical (lowest) value for each direction and the building as a whole.
CoM	Centre of mass
Complete steel frame	A building where steel-only columns connected to steel beams support all the gravity floor load and roof load and where the <i>vertical LFRS</i> is comprised solely of structural steel elements.
Configurational vulnerability	A feature of a building's configuration that causes the building components within its influence to be more vulnerable (because it locally amplifies earthquake demands).
CoR	Centre of resistance
Core	An assemblage or grouping of structural walls (structurally coupled or uncoupled), usually associated with or encompassing an area of vertical transportation or reticulation (stairs, lifts and risers) or amenities. See also <i>side-core</i> .
EPB	Earthquake-prone building: as defined in the <i>EPB Methodology</i> .
EPB Methodology	The methodology to identify, evaluate and remediate earthquake-prone buildings, set by MBIE's Chief Executive under section 133AZ of the Building (Earthquake-prone Buildings) Amendment Act 2026.
Heavy floors	Floors or floor areas constructed substantially in reinforced or unreinforced concrete (including as part of composite floor construction and floor construction incorporating masonry elements of any type).
Heavy materials	Means floors or roofs constructed substantially in reinforced or unreinforced concrete (including as part of composite floor construction and floor construction incorporating masonry elements of any type); or framing elements supporting and bracing the building (such as walls, columns, beams or braces) constructed in reinforced or unreinforced concrete.
High-risk deficiency (HRD)	High-risk deficiency: a specific building feature of a <i>Category B building</i> (or combination of features) that causes elevated risk of partial or full building collapse in earthquake.
HRD review	<i>High-risk deficiency</i> review: as defined in the <i>EPB Methodology</i> , carried out in accordance with this Annex.
LFRS	Lateral force resisting system: an assembly of structural members that act together to resist lateral actions in the direction of loading under consideration, including vertical systems, diaphragms and foundation systems that transfer lateral actions to the ground.
Poor anchorage	See joints.
Poor founding conditions	Foundation conditions that can result in high susceptibility to differential settlement for certain structural types due to liquefaction related effects or other known geotechnical hazards (see section A1.5.6 for complete definition).
PPHZ	Potential plastic hinge zone

RC	Reinforced concrete
RCM	Reinforced concrete masonry
Seismic Assessment Guidelines	<i>The Seismic Assessment of Existing Buildings: Technical Guidelines for Engineering Assessments (see A1.1.4 for referenced version)</i>
Severe physical consequence	A consequence of failure that meets the criteria set out in 2.3.
Side-core	A wall or assemblage of walls (<i>core</i>) located to the exterior of a building (and not completely enclosed by a substantial slab with a continuous enclosing tie of fully developed reinforcing).
Significant transfer diaphragm	A structural floor diaphragm constructed in heavy materials (see <i>heavy floors</i>), associated with the introduction or termination of walls and a significant horizontal transfer in storey shear force. See also section A1.2.2.
Storey strength index	The effective cross-sectional area of vertical elements (e.g. columns and walls) at a storey, as defined in sections A1.8.1 and A1.8.2.
Storey-sway potential	Where the strength hierarchy in a structure allows the potential for storey-sway mechanism to form in earthquake shaking, irrespective of whether a <i>weak storey</i> vertical irregularity is present. For the purposes of this Annex, storey-sway potential is defined in section A1.5.5 See also <i>weak storey</i> .
Torsional Displacement Index, TDI	An index that estimates how vulnerable a component is to drift amplification by torsional building displacements. This index is similar to the NZS 1170.5:2004 torsional displacement ratio γ , but it uses the effective cross-sectional area of vertical elements (e.g. columns and walls) to estimate strength and stiffness, to rapidly identify potentially <i>vulnerable components</i> .
Torsional Strength Index, TSI or T_{DI} / T_{CI}	A relative measure of how much torsional strength a building has to provide stability when, loaded through the <i>CoM</i> , it reaches its translational strength in a given direction. This index is based on the similar torsional strength ratio in FEMA P-2018, but it uses the effective cross-sectional area of vertical elements (e.g. columns and walls) to estimate strength, to rapidly identify potentially vulnerable configurations.
1RM	Unreinforced masonry
URM building	A building with <i>URM</i> materials in or on at least one of its façades. (refer to the <i>EPB Methodology</i>)
Vertical LFRS	Vertical or near <i>vertical LFRS</i> , such as such as moment-resisting frames, braced frames, structural walls and coupled walls.
Vulnerability	In non-italicised usage within this annex, this is a common term used to describe features of buildings that can be vulnerable to poorer performance in earthquake shaking (compared with modern well-configured buildings).

Vulnerable component	The italicised usage of this term has a specific meaning in this Annex. It is a component that contributes to the existence of an <i>HRD</i> , either Due solely to its own characteristics (as specifically defined in section A1.4 of this Annex), or Due to its characteristics and because it is within the influence of a configurational vulnerability (as specifically defined in sections A1.5, A1.6 and A1.7 of this Annex).
Vulnerable to lateral failure (describing a vulnerable component)	In this Annex, an element vulnerable to loss in lateral capacity <i>only</i> (should be assumed to continue to carry gravity loads after loss of lateral capacity).
Vulnerable to gravity failure (describing a vulnerable component)	In this Annex, an element vulnerable to loss in both lateral capacity <i>and</i> vertical (gravity) load carrying capacity. Interchangeably referred to as axial failure for columns and walls.
Wall Index, WI	See <i>Column Wall Index, CWI</i>
Wall Pier	A vertical wall segment within a structural wall, bounded horizontally by two openings or by an opening and an edge, with a ratio of clear height to horizontal length greater than 1.0, and either: singly reinforced; or, (where doubly reinforced) having a ratio of horizontal length to wall thickness greater than or equal to 4. This includes <i>wall piers</i> incorporating columns/pilasters to the interior or to one end only of their section.
Weak storey	A <i>weak storey</i> vertical irregularity, where the storey shear strength is notably lower than that of the storey above. For the purposes of this Annex, <i>weak storey</i> is defined in section A1.5.4. See also <i>storey-sway potential</i> .

A1.1.3.2 Notation

A_c	Area of concrete core of section measured to outside of peripheral spiral or hoop, mm ² .
A_{eff}	Effective area of vertical structural elements used in assessing torsional sensitivity. Values depend on the direction of loading being considered (denoted by $A_{eff,X}$ or $A_{eff,Y}$) and are given by $C_k \cdot A_{RCcol}$, A_{RCwall} , $A_{RCMwall}$, $A_{Masonrywall}$ in section A1.8 as appropriate (including adjustments under A1.8.2), mm ² .
A_g	Gross cross-sectional area of a column or <i>wall pier</i> , mm ²
A_j	Gross area of beam-column joint section $h_c b_j$, mm ²
A_{jh}	Area of joint shear reinforcement (located between top and bottom beam bars) parallel to the direction being considered, mm ² .
$A_{overlap}$	Overlapped cross-sectional area of a wall or wall pier supporting a discontinuous wall, as defined in section A1.6.3.3, mm ² .

$A_{MasonryWall}$	Gross cross-sectional area of <i>URM</i> wall, bounded by building frame elements or otherwise effectively engaged by imposed building deformations (see A1.8.1), mm ²
A_{RCCol}	Gross cross-sectional area of column, mm ² .
$A_{RCMWall}$	Gross cross-sectional area of reinforced masonry wall, effectively engaged to transmit lateral storey shear forces (see A1.8.1), mm ²
A_{RCWall}	Gross cross-sectional area of RC structural wall (see A1.8.1), mm ² .
A_s	For assessing walls for susceptibility to through-the-thickness axial failure, area of tension reinforcing, which should be taken as the tension zone force (under section analysis for M_p at N^*) divided by f_y , mm ² . For slab-column joints, the area of bottom reinforcing steel that is effectively continuous through the joint in one direction, mm ² .
A_s'	Area of compression zone reinforcing, which should be taken as the total area of longitudinal reinforcing steel contained within the compression zone, mm ² .
$A_{s,bottom}$	For slab-column joints, the area of effectively continuous bottom reinforcing passing through a column in a single direction.
b_c	Width of column, mm.
b_j	Effective width of joint as defined by the <i>Seismic Assessment Guidelines</i> section 2025-C5.6.2.1, mm.
B	For building aspect ratio, the width of the building, or width of a footing (as applicable) per section A1.5.6. mm. For determining the aspect ratio (H/B) of discontinuous walls or braced frames, the lever-arm distance between supporting elements providing overturning resistance (see A1.6.3), mm.
c_j	Adjustment factor for column area, used in computing the <i>column index CI</i> (and <i>column wall index CWI</i>), defined in section A1.8.1.
c_k	Adjustment factor for column area, used when computing the <i>storey strength index</i> for identifying weak storeys, and torsional indices for identifying severe torsional sensitivity. Defined in section A1.8.2.3.
d_b	Diameter of longitudinal reinforcing bar, mm.
e_{RXi}, e_{RYi}	The eccentricity of the <i>CoR</i> relative to the <i>CoM</i> used in evaluating torsional sensitivity, mm.
E_u	Earthquake actions at <i>benchmark seismic hazard</i> (see section A1.9 of this Annex, and Annex 2).
f'_c	Probable strength of concrete, as given by the <i>Seismic Assessment Guidelines</i> section 2025-C5.4.2.2, MPa.
f_{splice}	Stress that can be developed by a spliced reinforcing bar, as determined by the <i>Seismic Assessment Guidelines</i> section 2025-C5.4.4, MPa.
f_y	Probable yield strength of steel reinforcement, MPa.
h	Least cross-sectional dimension of structural element, mm.
h_b	Height of beam, mm.

h_c	Depth of column parallel to direction of loading, mm.
h_i	Storey height (total interstorey height), mm.
H	For building aspect ratio, the maximum height of a building or <i>vertical LFRS</i> (as applicable) above the point at which base shear is transmitted to the ground, mm. See A1.5.6 for details. For piers (A1.6.8), the maximum clear height of pier that can be intersected by a diagonal crack in either direction of loading (between floors, beams or spandrel elements), mm. For columns, the clear height of a column, mm. For determining the aspect ratio (H/B) of discontinuous walls or braced frames, the height of the wall or frame above the discontinuity (see A1.6.3), mm.
H_{eff}	Shear span M/V of wall, used in computing an aspect ratio adjustment for effective wall area in assessing torsional sensitivity (see A1.8.2.2), mm.
l_w	Wall length used in assessing susceptibility to through-the-thickness axial failure, mm. See section A1.6.6.2 for application to multiplanar walls.
L_c	Shear span M/V of column, wall or wall pier at critical section, mm.
$L_{d,prov}$	Development length (or lap length) provided, mm.
$L_{dh,prov}$	Provided hook bar development length, mm.
L_w	Length of wall segment or pier, mm.
M_p	Probable flexural strength of an element, calculated in accordance with the Seismic Assessment Guidelines section 2025-C5.5.2.2, Nmm.
N_E	Axial load in element (column, wall or <i>wall pier</i>) under earthquake actions E_u , N.
N_G	Permanent component of axial load in element (column, wall or <i>wall pier</i>) under seismic conditions based on gravity actions $[G + \psi_E Q]$ and any other permanent actions (compression taken as positive), N.
N^*	Compressive force (combined gravity and seismic) acting on a wall cross section for assessing susceptibility to through-the-thickness axial failure, N.
N_c^*	Axial load (maximum compression, taken as positive) including earthquake actions E_u (<i>benchmark seismic hazard</i>) acting in conjunction with gravity and permanent actions, as estimated according to section A1.9, N.
N_t^*	Axial load (maximum tension, taken as positive) including earthquake actions E_u (<i>benchmark seismic hazard</i>) acting in conjunction with gravity and permanent actions, as estimated according to section A1.9, N.
N_o	Overstrength derived axial load (including gravity and permanent actions), N.
p_{wall}	Perimeter of area enclosing <i>side-core</i> , mm.
s	Spacing of transverse reinforcing (stirrups, hoops or cross ties), mm
t	Thickness of structural element (e.g. wall segment or <i>wall pier</i>), mm
t_w	Wall thickness, mm.

$t_{w,eff}$	Effective thickness of wall compression zone used when assessing susceptibility to through-the-thickness axial failure, as defined in section A1.6.6.2, mm.
V_g	Perimeter shear demand acting on the critical perimeter section of a slab-column joint under N_G , N.
V_{oe}	Shear force at the face of a column or wall induced by end moments in a beam when overstrength moments act in a beam, N. For shear-controlled coupling beams, the overstrength shear.
V_p	Probable shear strength at the critical perimeter section of a slab-column joint (determined in accordance with NZS 3101:2006 and the Seismic Assessment Guidelines section 2025-C5.5.4.5), N.
$V_{p,col0}$	The undegraded shear strength of a column, determined in accordance with the Seismic Assessment Guidelines section 2025-C5.5.5.2., N.
V_y	The shear demand coincident with flexural yielding, V_y , based on the development of flexural hinges with strength M_p at both ends of the column's clear span, N.
$\alpha_{v,col}$	Dimensionless parameter for evaluating the effectiveness of transverse reinforcement in resisting shear, as given by section A1.6.2.3.
ρ_t	Transverse reinforcing ratio $A_v / (b_c s)$.

A1.1.4 Document References

Normative document references (by section or clause references within this annex) include the following documents:

- *Seismic Assessment Guidelines* which refers to the *Technical Guidelines for Engineering Assessments, Part C: Detailed Seismic Assessment* (2027 Edition).
- NZS 1170.5:2004 Structural design actions Part 5: Earthquake actions – New Zealand as adopted and amended by B1/VM1 as at the date of commencement of the Building (Earthquake-prone Buildings) Amendment Act 2026.
- NZS 3101:2006 Inc. Amd. 1-3 *Concrete Structures Standard*.

Other document references are informative.

Future released versions of the Seismic Assessment Guidelines should not be used for remediation of EPBs unless explicitly incorporated by reference into the EPB Methodology (by a future update to the EPB Methodology).

The following versions are assumed and are the basis for document and clause and equation number references made within:

C1-2025
C2-2025
C3-2025
C4-2027 (not yet released)
C5-2025
C6-2027
C7-2025
C8-2027 (not yet released)

C9-2017
C10-2017
C11-2027 (*not yet released*)

A1.2 Overview of the High-Risk Deficiency Review Process

A1.2.1 Process Outline

Category B buildings with a high cross-sectional area of vertical structure (for example, columns and walls) relative to their gross floor area are less inherently vulnerable in earthquake. This is measured by the *Column-Wall Index (CWI)*. Section A1.2.3 explains this index, and why it is used.

Category B buildings satisfying with a *CWI* that exceeds a threshold value of 0.30% or comprise a *complete steel frame* do not require a full *HRD review*. However, they must still be reviewed for a potential vulnerability in the connection between floors and vertical bracing elements (diaphragm connection).

Figure 2 shows this process. The following subsection lists specific exclusions for building elements (that shall not be considered to contribute to *HRDs*).

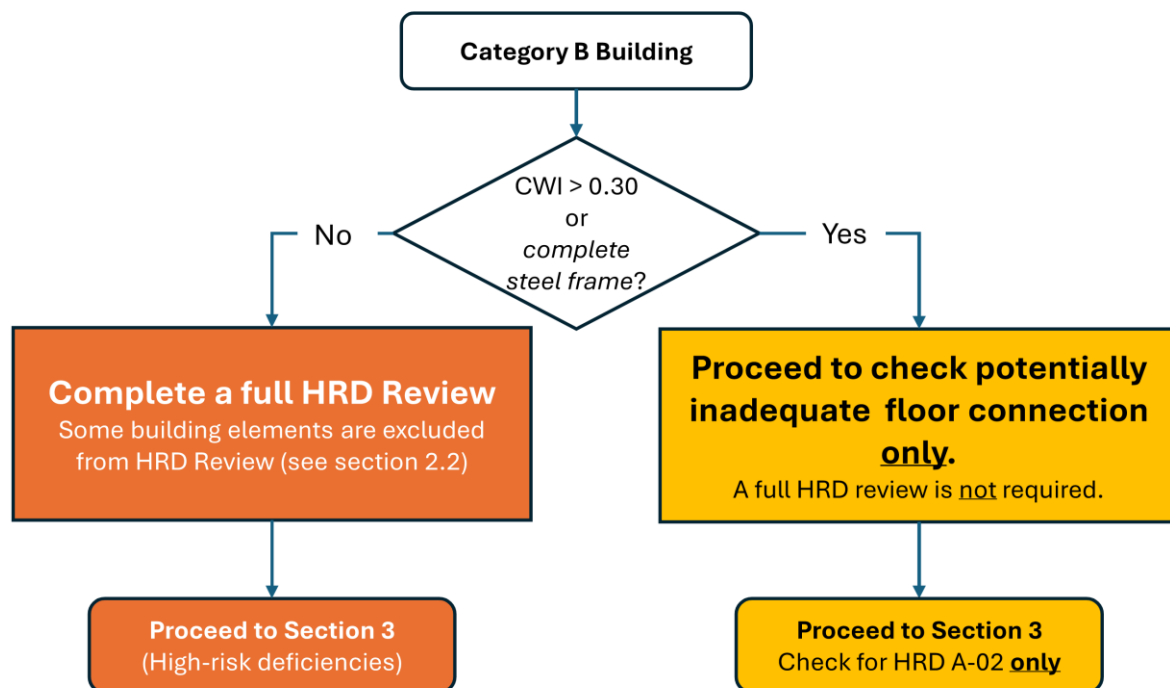


Figure 2: Determining whether or not an HRD review is required

A1.2.2 Building Element Exclusions

The building elements listed in Table 2 shall be excluded from consideration as an *HRD*.

Table 2: Building elements that are excluded from consideration as an *HRD*

Building Element	When they can be excluded from <i>HRD</i> Review
Lightweight roofs	Concrete elements supporting lightweight roofs.
Minor Penthouse Structures	Rooftop plant or penthouse structures of heavy construction situated above floors constructed with in-situ RC (excluding flat slabs with vulnerable slab-column connections), and with mass less than 20% of the combined mass of the appendage structure and the floor below. Includes minor structures located over a setback in floor plan.

A1.2.3 Explaining Indices Used in this Annex (Column-Wall Index, Torsional Indices)

A1.2.3.1 General

This Annex uses several geometric indices to rapidly identify potentially vulnerable building configurations. They are simple properties of a building's geometry that can be calculated based on site measure or drawing review.

They are introduced here because they are fundamental to the methodology in this Annex. Details of the calculation are in sections A1.5 and A1.8.

A1.2.3.2 Column Wall Index

The column wall index (CWI) is the ratio of effective cross-sectional area of vertical structure within a storey (columns, walls, and braces) to the total gross floor area supported above. Element cross sectional areas are adjusted based on the element type to obtain their effective area for the axis direction being considered (see section A1.8.1 for complete details).

The *CWI* is computed along each building axis and at sufficient storeys to confirm the critical (lowest) value for each direction and the building as a whole (Figure 3).

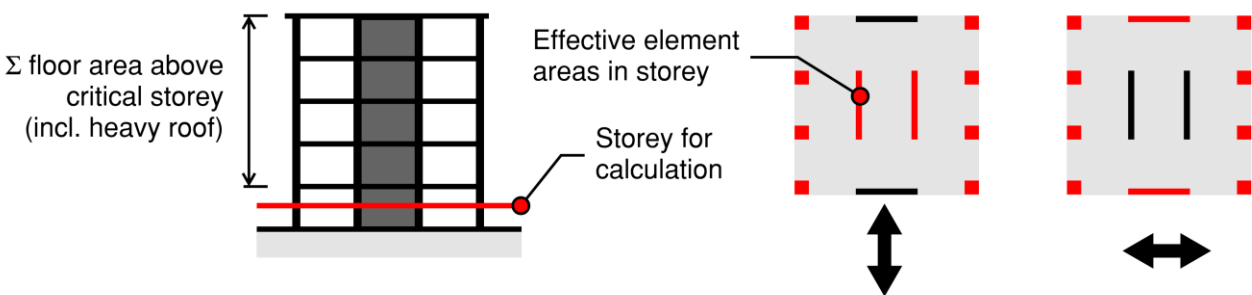


Figure 3: Illustrating the column wall index (see section A1.8.1 for complete details)

There is a significant body of post-earthquake reconnaissance data correlating this index with observations of building performance. This makes it a powerful tool in identifying buildings that are a higher earthquake risk.

The *CWI* is similar to the index originally developed by Hassan and Sozen¹, with adaptations appropriate to New Zealand's building inventory. Its usage in this Annex is similar to the 'NCREC index' used by seismic safety programs in Taiwan, and the 'seismic capacity index' I_s index used for similar purposes in Japan².

A1.2.3.3 Torsional Strength Index, TSI

The torsional strength index (TSI) provides a relative measure of how much torsional strength a building has to provide stability when, loaded through the CoM, it reaches its translational strength in a given direction (Figure 4). It is an important measure of a building's overall susceptibility to torsional effects.

The torsional strength index is based on the similar 'torsional ratio' (TR) in FEMA P-2018. However, like the *CWI*, the *torsional strength index* in this annex uses the effective cross-sectional area of vertical elements (e.g. columns and walls) to estimate strength, to rapidly identify potentially vulnerable configurations.

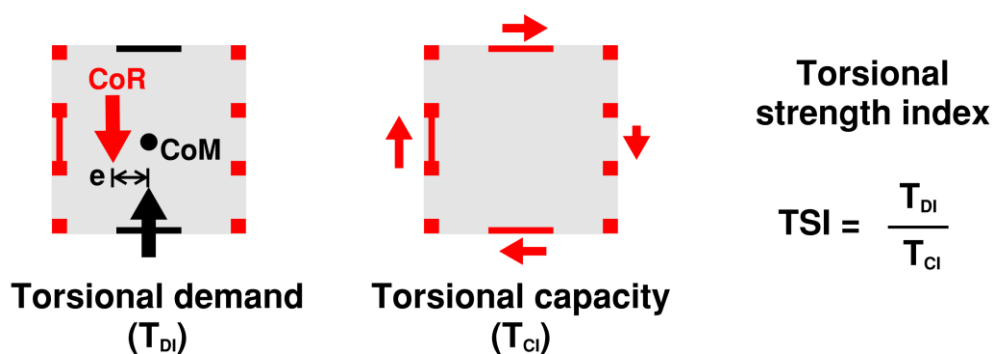


Figure 4: Illustration of torsional strength index (see section A1.5.3 and A1.8.2 for complete details)

A1.2.3.4 Torsional Displacement Index, TDI

This index estimates how vulnerable a component is to drift amplification from torsional building displacements (rotation). It is similar to the NZS 1170.5:2004 torsional displacement ratio γ . However, it uses the effective cross-sectional area of vertical elements (e.g. columns and walls) to estimate strength and stiffness, to rapidly identify potentially vulnerable components. The NZS 1170.5:2004 torsional displacement ratio requires structural analysis, but can be used instead of the TDI if that information is available.

Once the *torsional strength index* has identified a severe torsional sensitivity for the building overall, the *torsional displacement index* is useful for identifying which components are most vulnerable.

¹ Hassan, A. F., Sozen, M. A. (1997). *Seismic Vulnerability Assessment of Low-Rise Buildings in Regions with Infrequent Earthquakes*, ACI Structural Journal 94(4)

² Building Research Institute (2001). *Standard, Guidelines and Technical Manual for the Seismic Evaluation and Retrofit of Existing Reinforced Concrete Buildings*, Japan Building Disaster Prevention Association

A1.2.4 Severe Physical Consequence of Failure

For a *vulnerable component* to contribute towards the classification of an *HRD*, failure (if it occurred) must have a severe consequence for the building overall or a significant proportion thereof.

Table 3 defines a *severe physical consequence* of failure. It can relate either to gravity support, or lateral building stability.

Table 3: Defining severe physical consequence of failure

Physical Consequence	Definition
Loss of vertical support of a significant area of heavy floor	Where the failure of component(s) with <i>vulnerability to gravity failure</i> (as specifically identified in this Annex) would directly result in a loss of support to a significant area of <i>heavy floor</i> (a full structural bay), then a <i>severe physical consequence</i> is present. Where it can be demonstrated that a loss in gravity carrying capacity of the <i>vulnerable component(s)</i> identified in this Annex is not likely to lead to a loss of support to a significant area of <i>heavy floor</i> (due to structural redundancy), then a <i>severe physical consequence</i> of failure need not be assumed. Components that are identified by this Annex as <i>vulnerable to lateral failure</i> only should <u>not</u> be considered to lose gravity support.
Loss of significant proportion of lateral building stability	For floor-wall connections, any configuration meeting the <i>HRD</i> criteria for inadequate and non-redundant floor-wall connection is deemed to have <i>severe physical consequence</i> of failure. For all other cases, where the sum contribution of all <i>vulnerable components</i> relating to a configurational vulnerability amounts to more than 40% to a building's lateral stability in the direction under consideration (or more than 20% where $CWI < 0.15\%$), or where their failure would compromise torsional stability³ a <i>severe physical consequence</i> is present. If necessary, structural analysis can be used to help in deciding whether the lateral contribution of vulnerable component(s) is significant (see section A1.9). For linear elastic analysis subject to seismic actions (an ESA force distribution applied through the <i>CoM</i>) acting in conjunction with gravity loads, a significant contribution means either: - The sum of shear forces acting in all <i>vulnerable components</i> in any storey (and in the direction of the applied earthquake actions) exceeds 40% of the total storey shear demand at that storey (or 20% where $CWI < 0.15\%$); or, - The loss of lateral capacity of <i>vulnerable components</i> (and vertical capacity only where <i>vulnerability to gravity failure</i> is also present), increases the elastic lateral deflection at any location on the uppermost floor (uppermost principal seismic mass) by more than 60% in the direction of applied earthquake actions.

³ For example, if the omission of the vulnerable elements results in a *torsional strength index*, $T_{DI} / T_{CI} > 0.8$

Refer to section A1.9 for guidance on using structural analysis to assist with identifying HRDs. An analysis is not required where it is assumed that the criteria would be clearly met.

Where multiple configurational vulnerabilities are present in the building, each of these shall be considered individually in turn (consider only the components affected by the configurational vulnerability being reviewed, along with any component-only deficiencies).

A1.2.5 Geotechnical Considerations in HRD Reviews

The criteria applied in setting the list of potential *high-risk deficiencies* in this Annex focusses on high building-risk as outlined in section A1.1.2. For geotechnical aspects, only specific situations involve consideration of founding conditions, and these HRDs with ID 'F-' in Table 4 of section A1.3.

As a minimum, geotechnical engineers contributing to HRD reviews should be familiar with sections A1.1 to A1.3, A1.5.6 for details on the relevant HRD(s) requiring geotechnical consideration, and the standard building information tables and HRD review checklists.

Specific foundation assessment (or use of existing geotechnical or foundation assessment information) is not always required. However section A1.9.4 provides information regarding such assessments, if needed.

A1.3 High-Risk Deficiencies in Category B Buildings

A1.3.1 General

The specific features of *Category B buildings* that can be *HRDs* (i.e. can result in a building being *Earthquake-prone*) are listed in Table 4 with references to their detailed descriptions in sections A1.4 to A1.7. Only the building features specifically listed in Table 4 can be *HRDs*, and there must be a *severe physical consequence of failure*.

Note the combination of component deficiency and configurational deficiency that applies to several of the *HRDs*. In these cases, both conditions must be met for the deficiency to classify as an *HRD*.

For a building with $CWI > 0.30\%$ or a complete steel frame, only check for *HRD A-03* (Inadequate and non-redundant floor-wall connection).

For a building with a column wall index $CWI \leq 0.30\%$ that is not a complete steel frame, check the complete list of *HRDs* in Table 4 to find if any are present. The flow chart in Figure 5 can be followed when a full *HRD* review is required.

An HRD Review checklist is provided in section A1.10. This will help assessors to apply HRD reviews correctly, and should be used to record HRD review outcomes. Sections A1.4 to A1.7 must be followed for the complete definitions. The criteria prompts in the HRD review checklist tables do not include all necessary information.

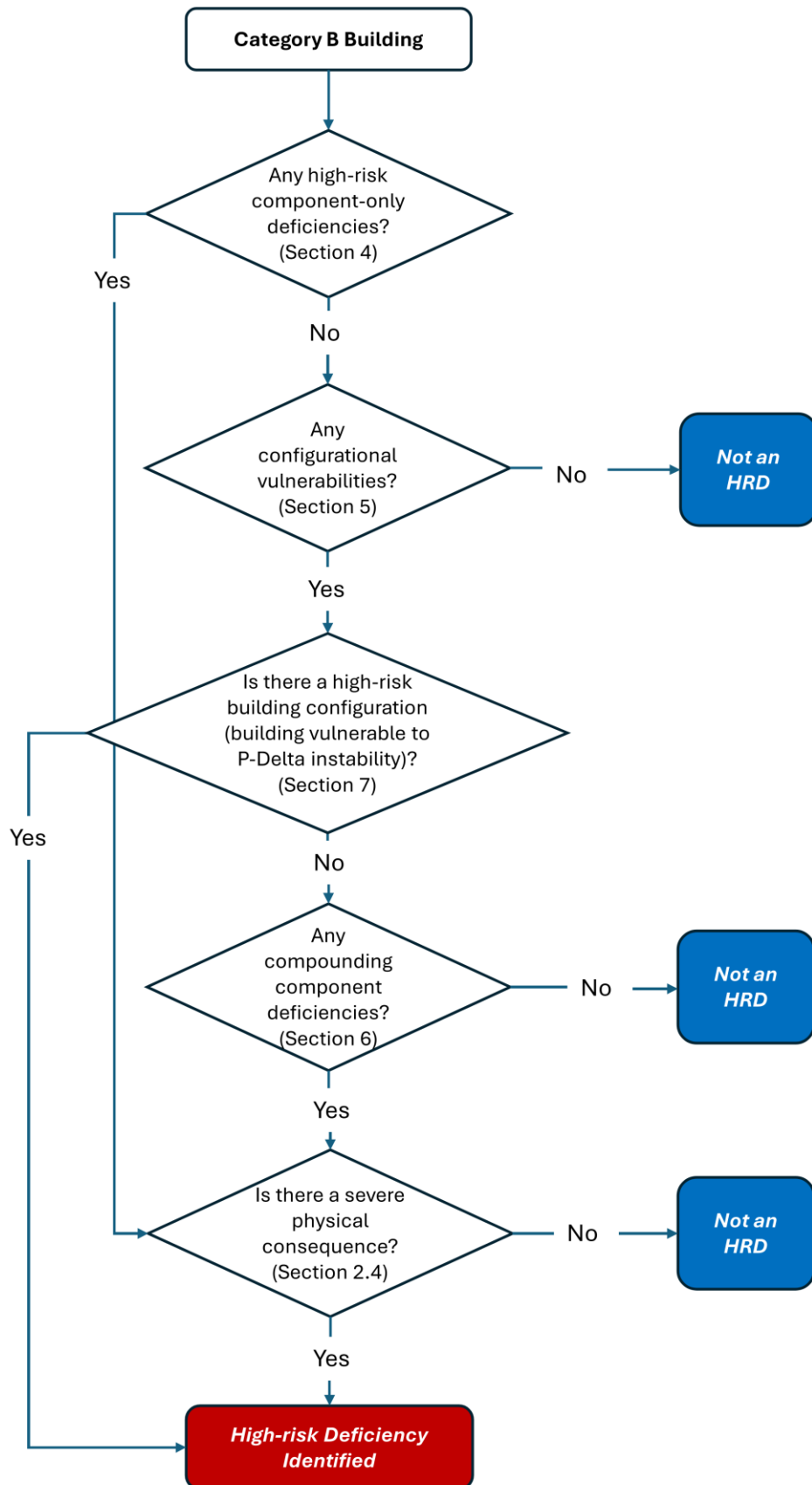


Figure 5: Process for identifying high-risk deficiencies in Category B Buildings

A1.3.2 List of Possible High-Risk Deficiencies

Table 4: List of possible *high-risk deficiencies* identified by this Annex

HRD ID	High Risk Deficiency (several can apply)			
Component Only Deficiencies				Ref.
A-01	Column with exceptionally high axial load			A1.4.2
A-02	Inadequate and non-redundant floor-wall connection			A1.4.3
A-03	Slab-column joint(s) with high shear demand			A1.4.4
A-04	URM elements contributing significantly to vertical or lateral stability			A1.4.5
Compounding Configurational and Component Deficiencies ² (both must be present to be an HRD)				
	Configurational Vulnerability	Ref.	Component vulnerability ¹	Ref.
B-01	Overall weak vertical LFRS	A1.5.2	Vulnerable column	A1.6.2
B-02			Vulnerable beam column joint	A1.6.4
B-03			Vulnerable slab-column joint	A1.6.5
B-04			Wall pier vulnerable to axial failure	A1.6.8
B-05			Wall vulnerable to axial failure	A1.6.6
B-06			Wall vulnerable to lateral strength loss	A1.6.7
C-01	Severe Torsional Sensitivity	A1.5.3	Vulnerable column	A1.6.2
C-02			Vulnerable beam column joint	A1.6.4
C-03			Vulnerable slab-column joint	A1.6.5
C-04			Wall pier vulnerable to axial failure	A1.6.8
C-05			Wall vulnerable to axial failure	A1.6.6
C-06			Wall vulnerable to lateral strength loss	A1.6.7
D-01	Weak storey (soft or weak storey vertical irregularity)	A1.5.4	Vulnerable column	A1.6.2
D-02			Vulnerable beam column joint	A1.6.4
D-03			Vulnerable slab-column joint	A1.6.5
D-04			Wall pier vulnerable to axial failure	A1.6.8
E-01	Storey-sway potential	A1.5.5	Vulnerable column	A1.6.2
E-02			Vulnerable beam column joint	A1.6.4
E-03			Vulnerable slab-column joint	A1.6.5
E-04			Wall pier vulnerable to axial failure	A1.6.8

F-01	Slender <i>vertical LFRS</i> with poor founding conditions	A1.5.6	Vulnerable column	A1.6.2
F-02			Vulnerable beam column joint	A1.6.4
F-03			Vulnerable slab-column joint	A1.6.5
G-01	Discontinuous wall or braced frame	A1.5.7	A supporting column, pier or wall overlap with excessive axial demand.	A1.6.3
G-02	(horizontal offset or vertical discontinuity irregularity)		A supporting column or pier vulnerable to drift-induced axial failure.	A1.6.2-A1.6.3
High-risk building Configurations				
H-01	Exceptionally weak frames (vulnerable to P-Delta instability)			A1.7.2

Notes:

- [1] Applies only to components that are within the influence of the configurational vulnerability.
- [2] For compounding configurational and component deficiencies, the component deficiency can only classify as an *HRD* where the compounding configurational vulnerability listed alongside *also* exists, and would amplify demands on the component.

A1.4 Component-Only Deficiencies

A1.4.1 General

The *HRDs* described within this section arise from high vulnerability in the component alone. The *vulnerable component(s)* identified in this section are an *HRD* when there is a *severe physical consequence of failure*.

A1.4.2 Columns with high axial load (vulnerable to axial failure)

Columns with high permanent axial load, $N_G > 0.4 A_g f'_c$ shall be classified as *vulnerable to axial (gravity) failure*.

In determining whether an *HRD* is present, consider the consequence of failure if all such columns experience axial/gravity failure.

A1.4.3 Vulnerable and Non-Redundant Floor-Wall Connection

A1.4.3.1 For general floors

Vulnerable floor connections are those connecting *heavy floors* to walls, assemblages of walls (*core*) or steel braced frames located to the building exterior⁴, and where any of the following criteria (a) to (d) apply:

For exterior concrete walls (or *side-core*) or steel braced frames

- Eq. (1) Where the distributed floor reinforcing over the length of the critical section (weakest reinforcing section near the connection to the wall or *side-core*) comprises non-ductile mesh reinforcing (with lower bound tensile strain capacity $\epsilon_{su} < 5\%$), or where anchorage to the wall employs shallow embedded anchorage instead of fully developed anchorage that engages wall reinforcing.
- Eq. (2) For any wall segment (including individual wall segments in a *side-core*), where less than 50% of the wall segment length engages the diaphragm (Figure 6).
- Eq. (3) For *side-core*, where the length of the critical section is less than 50% of the core perimeter, p_w (Figure 6).
- Eq. (4) For any floor type, where the plan configuration is severely torsional (e.g. due to inertial loads being resisted substantially by a single offset *side-core*) and resulting in a *torsional strength index*, $TSI > 0.8$ (computed based on the *CoM* of the storey under consideration).

⁴ A wall or assemblage of walls shall be considered exterior where it is not completely enclosed by a substantial slab with a continuous enclosing tie of fully developed reinforcing.

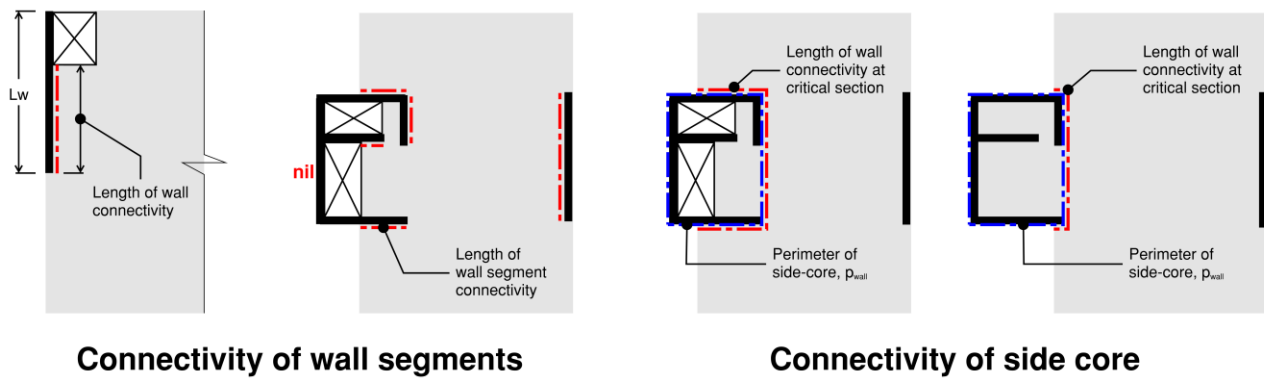


Figure 6: Connectivity of wall segments, and side-core.

Exceptions:

The following exceptions apply:

- Walls acting parallel to the building perimeter, constructed integrally with continuously reinforced beam extensions engaging the floor diaphragm, and sufficiently restrained to the floor diaphragm out-of-plane (see Figure 8 for examples). Such walls don't need to be considered to have vulnerable connections.
- Where sufficient redundancy exists in the building, as demonstrated along each building axis by the following criteria, computed **excluding all** exterior bracing elements with vulnerable connections (braces, walls or side-core acting in the direction under consideration, retaining only wall segments that can be engaged by a direct compressive load path):
 - Where the *Column-Wall Index* CW_I , is greater than 0.15%; and,
 - Where the *Torsional Strength Index* T_{DI}/T_{CI} , computed **excluding any single** exterior bracing element or wall assemblage acting in the direction under consideration is less than 0.8.

When checking redundancy, all walls forming part of an interlinked side-core should be considered exterior and part of a single wall assemblage (see Figure 7).

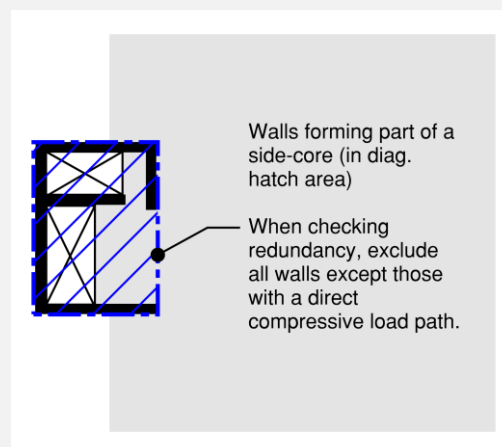


Figure 7: How to check redundancy for a side-core.

Figure 8 gives two examples of exterior walls constructed integrally with continuously reinforced beam extensions engaging the floor diaphragm, and sufficiently restrained to the floor diaphragm out-of-plane. Such configurations would not be required to be considered as potential HRDs.

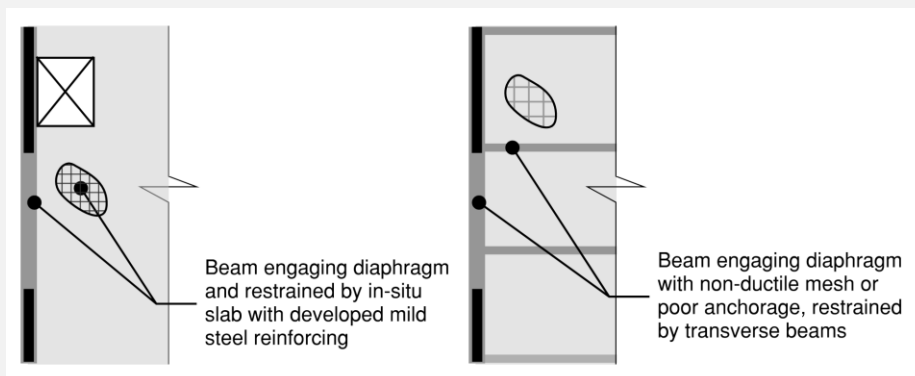


Figure 8: Examples of integral beams engaging the floor diaphragm and sufficiently restrained out-of-plane

A1.4.3.2 For significant transfer diaphragms

Significant transfer diaphragms are those associated with the introduction or termination of walls and a significant horizontal transfer in storey shear force.

The criteria for identifying vulnerable bracing elements to *significant transfer diaphragms* are similar to those for general floors A1.4.3.1 (a) and (b) but applied to all walls or *cores*, interior and exterior. Similar exceptions apply. The criteria do not apply to slabs on grade.

A1.4.4 Slab-Column Joint(s) with high shear demands

A1.4.4.1 Slab column-connections that contribute significant lateral resistance

For slab-column connections contributing significantly to the building's lateral stability⁵, vulnerable slab column connections are defined by the following criteria:

1. For slab-column connections with effectively continuous bottom reinforcement passing through the column in each direction with cross sectional area $A_{s,bottom} \geq 0.6 V_g / f_y$;
Where $V_g/V_p > 0.4$
2. In all other cases (**without** continuous bottom reinforcing meeting the above requirements):
Where $V_g/V_p > 0.2$

A1.4.4.2 Other slab-column connections

Any slab-column connection **without** effectively continuous bottom reinforcement passing through the column in each direction with cross sectional area $A_{s,bottom} \geq 0.6 V_g / f_y$ is defined as vulnerable when the gravity demand ratio is greater than $V_g/V_p > 0.4$.

A1.4.5 Unreinforced masonry elements contributing significantly to vertical or lateral stability

URM elements (including infill masonry) acting as part of the primary gravity or a significant part of the vertical LFRS (for the vertical support and stability of heavy concrete floors) should be considered vulnerable to loss in lateral strength and axial failure (as appropriate depending on their function).

URM walls that are tied into the primary structure (for example archaic party walls that have been built into concrete structures, or infill masonry) should be included in Column-Wall Index. Their contribution to lateral stability need only be considered significant in the rare circumstances where their sum contribution to the CWI is greater than 40%.

⁵ This can be based on the sum of affected columns having contribution to the Column-Wall Index $CI_{slab-col} / (CI + WI) > 40\%$.

A1.5 Configurational Vulnerabilities

A1.5.1 General

The *configurational vulnerabilities* described in this section can cause earthquake demands (forces and displacements) in certain parts of a buildings structure to be amplified compared with regular structures.

The presence of any compounding component deficiencies in these areas need to be reviewed according to section A1.6. Refer back to the list of *HRDs* in section A1.3.2 (Table 4). It states what compounding component deficiencies need reviewing when a configurational deficiency is identified.

A1.5.2 Overall Weak Vertical Lateral Force Resisting System

Buildings with an overall weak *vertical LFRS* are classified as those with a *Column-Wall Index*, $CWI \leq 0.15\%$ in the direction under consideration (computed in accordance with A1.8.1).

A1.5.3 Severe Torsional Sensitivity

A1.5.3.1 General

Severe torsional sensitivity is considered present when the following criteria are met.

1. The *Torsional Strength Index (TSI)* for the building as a whole (as estimated by effective element areas at the most critical level and direction of application) is greater than 0.4; and either:
 1. In the evaluation of vulnerable *core walls* or potentially inadequate wall-floor connections, the majority of lateral and torsional resistance is provided by a single *core*; or,
 2. The susceptibility of any component under consideration (such as a vulnerable column, joint or wall) to torsional amplification, as estimated by the *Torsional Displacement Index (TDI)* is greater than 1.4.

A1.5.3.2 Torsional Strength Index, TSI

The area-based *torsional strength index (TSI)* is given as follows, assessed independently for each level and each primary building axis with $CWI \leq 0.30\%$, sufficient to determine the single critical (highest) value for the building as a whole. The equations are presented for Z-direction earthquake loading.

$$\text{Torsional Demand Index, } T_{DI} \quad T_{DI} = (|e_{RX}| + 0.05B) \sum A_{eff,Y} \quad 1.$$

$$\text{Torsional Capacity Index, } T_{CI} \quad T_{CI} = \sum_1^n A_{eff,Yi} |x_i - e_{RX}| + \sum_1^n A_{eff,Xi} |z_i - e_{RY}| \quad 2.$$

$$\text{Torsional Strength Index} \quad TSI = \frac{T_{DI}}{T_{CI}} \quad 3.$$

Note: Accidental eccentricity of 5% of the overall floor plan dimension is included in the equation for Torsional Demand Index. The equations are presented for loading in the Y axis direction (Figure 9); X and Y variables should be interchanged for loading in the X axis direction.

Where (considering Figure 9 and earthquake loading in the Z direction):

$A_{eff,X}$	The effective area of vertical structural elements (acting in the axis perpendicular to the direction of earthquake loading) as given by $c_k A_{RCcol}$, A_{RCwall} , $A_{RCMwall}$, $A_{Masonrywall}$ in section A1.8 as appropriate (including adjustments under A1.8.2)
$A_{eff,Y}$	As for $A_{eff,X}$ but for elements acting in the direction of earthquake loading.
e_{RXi} , e_{RYi}	The eccentricity of the CoR relative to the CoM.
x_i , y_i	The coordinate location of elements on plan (centroid of their area A_{eff}) relative to the CoM.
CoR	Centre of resistance, taken as the centroid of effective element areas A_{eff} that act in each respective direction of earthquake loading. That is, the X coordinate of the CoR is based on the centroid of all $A_{eff,Y}$ and vice versa.
CoM	The centre of mass of all structure above the plane at which the TSI or TDI is being computed.

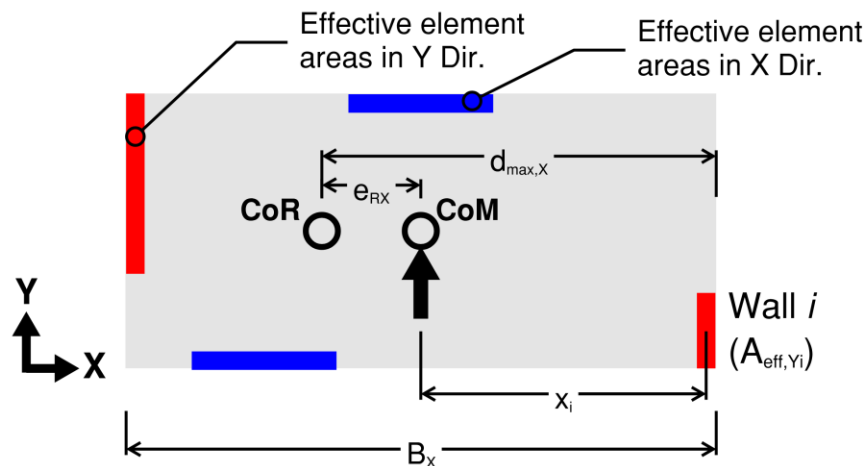


Figure 9: Definition of variables in calculation of TSI and TDI

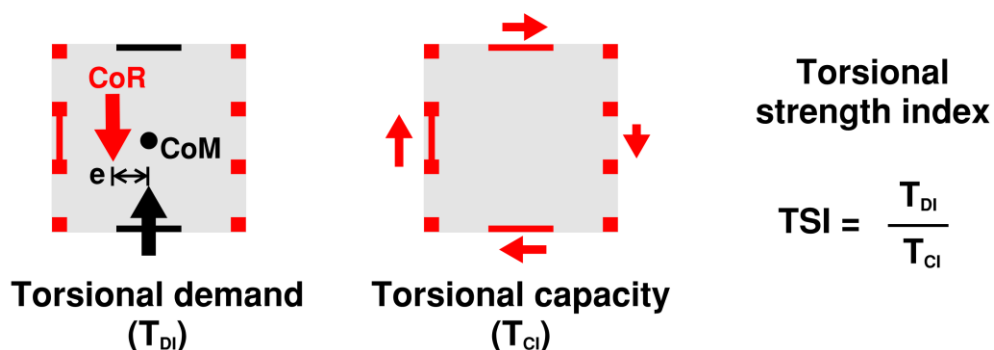


Figure 10: Illustration of torsional strength index

The torsional strength index is a relative measure of how much torsional strength a building has to provide stability compared to the torsional demand arising when it reaches its translational strength in a given direction (when loaded through the CoM). See Figure 10 for illustration.

This index is based on the similar torsional strength ratio in FEMA P-2018, but it uses the effective cross-sectional area of vertical elements (e.g. columns and wall) to estimate strength, to rapidly identify potentially vulnerable configurations.

The torsional strength index is generally only suitable for application above the upper-most grade plane or foundation plane providing a substantial contribution to base shear resistance.

A1.5.3.3 Torsional Displacement Index, TDI

The area-based *torsional displacement index (TDI)* as defined below, assessed independently for each level and each direction where $CWI \leq 0.30\%$. The equation is presented for Z-direction earthquake loading.

$$\text{Torsional Displacement Index} \quad TDI = 1 + \frac{e_{RX} d_{max,X} \sum A_{eff,Z}}{JI} \quad 4.$$

$$\text{Rotational Stiffness Index, } JI \quad JI = \sum_1^n A_{eff,Zi} (x_i - e_{RXi})^2 + \sum_1^n A_{eff,Xi} (z_i - e_{RZi})^2 \quad 5.$$

Note: Accidental eccentricity of 5% of the overall floor plan dimension should be added to the computed e_{RX} for the Torsional Irregularity Index.

Where all variables are as defined for the *torsional strength index*, and:

$d_{max,X}$ The maximum distance in metres from the CoR to the location of any vulnerable components under consideration alongside the configurational deficiency, perpendicular to the direction of loading.

Alternatively, it is permitted to use the NZS 1170.5:2004 torsional displacement ratio γ , in lieu of the *torsional displacement index*.

The torsional displacement index estimates how vulnerable a component is to drift amplification by torsional building displacements. It is similar to the NZS 1170.5:2004 torsional displacement ratio γ , but it uses the effective cross-sectional area of vertical elements (e.g. columns and wall) to estimate strength and stiffness, to rapidly identify potentially vulnerable components.

A different value for the torsional displacement index can be computed when considering different vulnerable components, as its purpose is to relatively assess the amount of displacement amplification that could occur in different areas of the floor plate.

The torsional displacement index is generally only suitable for application above the upper-most grade plane or foundation plane providing a substantial contribution to base shear resistance.

A1.5.4 Weak storey (weak storey vertical irregularity)

A *weak storey* vertical irregularity shall be considered to exist where the *storey strength index* computed for each storey, i (with storey height h_i) is less than 90% of the corresponding value for the

storey above, $i+1$. Soft storey need only be considered where the *Column-Wall Index* computed for the storey and direction under consideration $CWI \leq 0.30\%$.

$$\text{Storey Strength Index: } \sum A_{RCwall} + 0.5 \sum A_{RCMwall} + 0.1 \sum A_{Masonrywall} + \frac{2 \cdot \sum (c_k A_{RCcol,k})}{h_i} \geq 6.$$

Where:

A_{RCwall} Gross cross-sectional area of RC structural wall (see A1.8.1).

$A_{RCMwall}$ Gross cross-sectional area of reinforced masonry wall, effectively engaged to transmit lateral storey shear forces (see A1.8.1).

$A_{Masonrywall}$ Gross cross-sectional area of URM wall, bounded by building frame elements or otherwise effectively engaged by imposed building deformations (see A1.8.1)

$A_{RCcol,k}$ Gross cross-sectional area of concrete column k. See A1.8.1.5 for treatment of columns that are not RC (for example steel or composite columns).

c_k Adjustment for column k, based on whether the column is an ordinary frame column, a non-frame column, or a short column (as defined by A1.8.2.3).

h_i Total interstorey height.

Element areas (including allowances for non-concrete components) are clarified in A1.8.1.

Alternatively, it shall be permitted to apply the provisions of NZS 1170.5:2004 for *discontinuity in capacity – weak storey* (Cl. 4.5.1.3). Where this clause identifies no irregularity, then no *weak storey* vertical irregularity exists for the purposes of this section.

Figure 11 gives examples of configurations that can lead to weak storey.

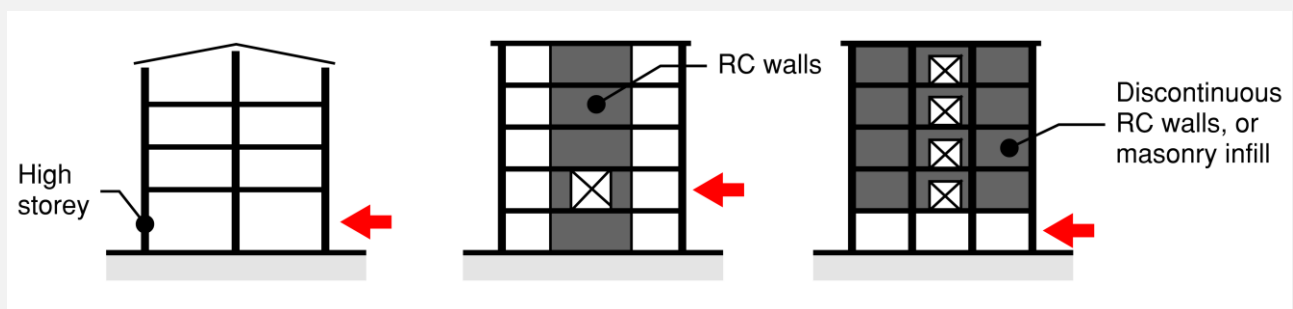


Figure 11: Example of building configurations that can result in weak storey (marked by red arrow)

Openings in walls (such as Figure 11, centre) reduce the storey strength index in that storey, and could indicate a weak storey. Considering the NZS 1170.5:2004 vertical irregularity provisions, this would not result in a weak storey where it can be shown that the storey-shear strength of a shear/sway mechanism across the openings is greater than the storey shear demand in that storey when the wall develops a plastic hinge at its base.

A1.5.5 Storey-sway potential

Storey-sway potential in frames, (including pier-spandrel frames) is considered to exist when identified by applying either of the following methods (the appropriate method should be selected by

the assessor). Storey-sway potential need only be considered where the *Column-Wall Index* computed for the storey and direction under consideration $CWI \leq 0.30\%$.

Storey way potential need only be considered in the lower half of regular buildings (or where there is a significant vertical discontinuity or setback, the lower half of the frame above that point).

1. Based on comparison of mechanism strengths

Based on the comparison of the storey shear strengths associated with the development of plausible mechanisms, including:

1. Beam sway or mixed mechanism, computed for overstrength.
2. Storey sway mechanism, computed at probable strength.

If the storey sway mechanism strength is less than the beam- or mixed-sway mechanism, then *storey-sway potential* is present.

Refer to the seismic assessment guidelines Appendix 2025-C2A: Simple Lateral Mechanism Analysis (SLaMA) for information on assessing hierarchy of inelastic mechanisms. FEMA P-2018 also contains methods for the estimation of mechanism strength (Figure 12).

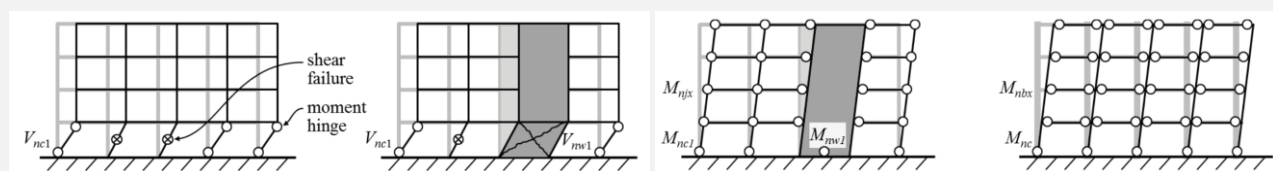


Figure 12: Mechanism strength calculations for storey sway mechanisms (left); and beam sway or mixed mechanisms (right). Diagrams reproduced from FEMA P-2018.

3. Based on the sway index for frames (not including pier-spandrel frames)

1. Where weak storey-sway is indicated based on the sway index method in the *Seismic Assessment Guidelines* (sections 2025-Appendix C2A and 2025-C5.7.1); and,
2. In the absence of walls with a contribution to the *Column-Wall Index* in the level and direction under consideration $WI / (WI + CI) \geq 20\%$.

Torsional storey-sway potential

In applying either method to buildings with high torsional sensitivity (for example open-front or corner buildings), calculations should be computed for the susceptible frames individually (as these may dominate response, per Figure 13). Section A1.5.3 defines severe torsional sensitivity. For buildings with low torsional sensitivity ($TSI < 0.4$ or $TDI < 1.4$) and tied by floor diaphragms, calculations can be made for the building as a whole (aggregated for all frame lines and elements acting in the direction under consideration).

Figure 13 gives examples of the torsional storey-sway behaviour addressed by this clause.

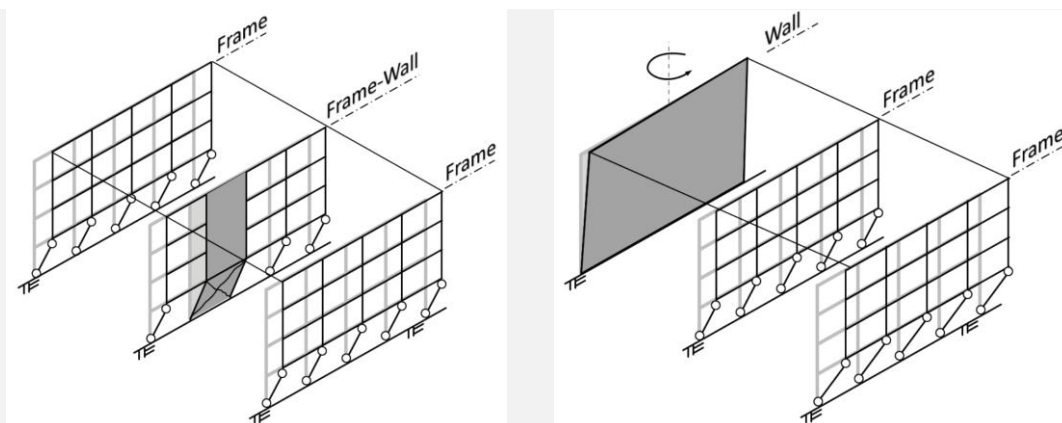


Figure 13: The right figure shows and examples of a building where torsional sensitivity requires the *storey-sway potential* of frames to be considered on frames individually (rather than the building as a whole). Diagrams reproduced from FEMA P-2018.

A1.5.6 Slender Vertical Lateral Force Resisting System with Poor Founding Conditions

A1.5.6.1 General

Buildings with a slender *vertical LFRS* and can be susceptible to differential settlement or tilting effects when there are *poor founding conditions*. Some structural types can be vulnerable to these effects. These specific types are listed below.

They relate to situations where frame components (columns, and beam/slab-column joints) can be more vulnerable to axial/gravity failure because they are exposed to high strain from differential settlement, or high drift demand because of *vertical LFRS* tilting.

If these structural types occur in conjunction with *poor founding conditions* (defined below), then the frame components should be considered for vulnerability to axial/gravity failure **only**.

A1.5.6.2 Potentially vulnerable structural types that are susceptible to differential settlement or tilting effects

Table 5: Building structures that can be vulnerable with *poor founding conditions*

System Type	Definition (applies along each primary building axis)	Schematic
Slender building frames vulnerable to settlement instability	Buildings with isolated shallow foundations under frame columns that not connected by ground beams of equal or greater section proportions (A_g) to the first storey frame section dimensions; and, The overall aspect ratio in the direction under consideration $H / B > 1.5$	

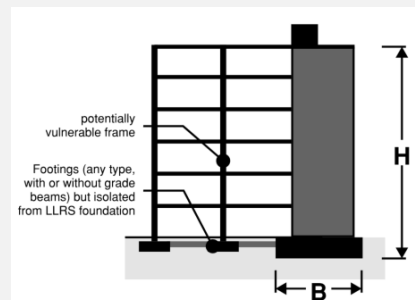
Rotation of slender vertical LFRS affecting associated frame columns not sharing the same foundation

Buildings where more than 60% of the lateral resistance (based on contribution to *CWI*) is from isolated bracing elements (cantilevered wall, core or braced frame); and,

The overall aspect ratio of the isolated vertical LFRS under consideration $H / B > 2.0$; and,

Potentially vulnerable frame components are founded on separate footings or pilecaps.

Does not include situations where the piles of the LFRS extend to found on rock.



Where:

H For a building frame, the overall height of the building frame in metres from the uppermost heavy floor above the point at which base shears are transmitted into the soils. Excludes penthouse structures constructed in heavy materials with a mass of less than 20% of the combined mass of the penthouse and floor below.

For a concentrated bracing system, the overall height in metres above the base of the footing.

B For a building frame, the total width of the building frame on centreline in the direction under consideration (for buildings that are irregularly shaped in plan, the greatest frame dimension should be used).

For a concentrated bracing system with shallow foundations, the width of footing in the direction under consideration. For piled foundations, the centreline between the outermost piles engaged by the wall foundation.

A1.5.6.3 Poor founding conditions

Poor founding conditions should be assumed when any of the structural types listed in Table 5 are encountered on sites with the following site subsoil categorisations, unless specific foundation assessment information is available that shows that poor founding conditions are not likely to exist. It does not include situations where the piles of the vertical LFRS extend to found on rock.

1. NZS 1170.5:2004, site subsoil class C, D or E.

Section A1.9.4 has guidance on how existing or new specific foundation assessment information can be used to demonstrate that poor founding conditions are unlikely.

A1.5.7 Horizontal Offset or Vertical Discontinuity Irregularity

A horizontal offset irregularity exists where a wall or braced frame is not continuous to ground, and is supported either by:

1. A column or pier; or,

2. A wall overlap (such as an overlap of the discontinuous wall to a wall segment below that is either in the same or different plane to the discontinuous wall) as clarified in figure REF; or,
3. A transfer beam.

The elements supporting the discontinuous elements should be considered as potential *HRDs* in accordance with A1.6.3.

A vertical discontinuity can also contribute to a vulnerability in the wall itself (as opposed to the element(s) supporting it) if it also results in lack of sufficient anchorage. This is addressed by the provisions for potentially vulnerable walls in section A1.6.7.4.

A1.6 Compounding Component and Configurational Deficiencies

A1.6.1 General

This section describes the component deficiencies which can be vulnerable when compounded by configurational issues. The *vulnerable component(s)* identified in this section are an *HRD* when there is a *severe physical consequence of failure*.

Use the flow chart and list of *HRDs* in section A1.3. It lists what qualifies as an *HRD*.

The provisions in this section are specific about when components should be considered *vulnerable to axial/gravity failure*, or *vulnerable to lateral failure only*. This is important when considering if there is a *severe physical consequence*. If not, there is no *HRD*.

A1.6.2 Vulnerable RC column(s)

A1.6.2.1 General

The vulnerability of reinforced concrete (RC) columns should be classified based on the process shown in Figure 14 and the provisions in the following subsections.

Only consider columns affected by configurational vulnerabilities

Only columns whose demands (force or displacement) would be affected (amplified) by a configurational deficiency should be considered for potential vulnerability, as part of this section.

Classify columns as flexure or shear limited

Columns should be evaluated as being limited by flexure, or shear behaviour. Columns not so evaluated shall be assumed to be limited by shear. Evaluation shall be based on:

- 1) Probable flexural strength M_p , based on section analysis in accordance with the *Seismic Assessment Guidelines* section 2025-C5.5.2.2, computed under the permanent component of axial load, N_G .
- 2) Undegraded shear strength of the column, $V_{p,col0}$ computed in accordance with the *Seismic Assessment Guidelines* section 2025-C5.5.2.
- 3) The shear demand coincident with flexural yielding, V_y , based on the development of flexural hinges with strength M_p at both ends of the column's clear height (with consideration of potential captive column conditions arising from non-structural elements).
- 4) Alternatively, if (based on a strength hierarchy assessment) the column is expected to hinge at only one end, V_y can be determined based on the probable flexural strength at the critical section divided by the minimum shear span L_c associated with plausible frame mechanism(s).
- 5) Permanent axial load, N_G is the axial compressive load for the permanent action only (generally gravity actions comprising $G + \psi_E Q$) and should include the effect of other permanent actions, such as post-tensioning (for example).

Column behaviour shall be considered limited by shear when the following relationship is satisfied:

$$\frac{V_y}{V_{p,Col0}} > 0.7$$

4.

Columns protected from hinging or shear failure

If it can be shown by a joint hierarchy analysis (considering overstrength), or other rational capacity design method, that a column has a high degree of protection against flexural hinging or shear failure when the building fully develops a lateral plastic mechanism (considering all plausible mechanisms), then such columns need not be classified as vulnerable.

How to evaluate potentially vulnerable columns

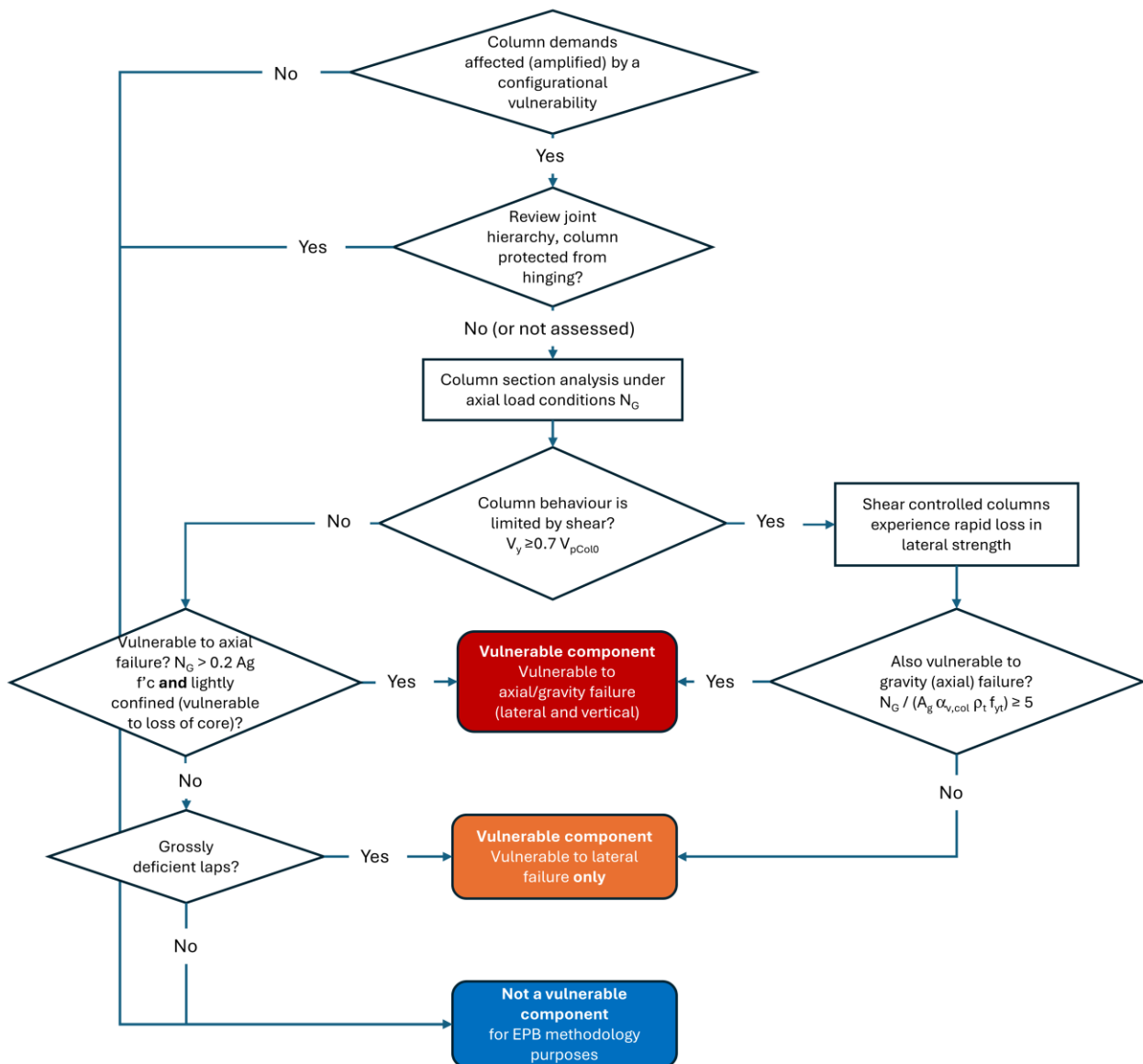


Figure 14: Flow chart for identification of vulnerable column(s) affected by a configurational deficiency.

A1.6.2.2 Considering vulnerability to loss in lateral column strength

Reinforced concrete columns that meet either of the following criteria shall be classified as *vulnerable to lateral failure*:

1. Column(s) whose behaviour is limited by shear ($V_y / V_{p,Col0} > 0.7$) shall be classified as *vulnerable to lateral failure*.
2. Columns where reinforcing is lapped within the PPHZ and where:
 1. For plain round reinforcing $L_{d,prov} \leq 24d_b$
 2. For deformed reinforcing $L_{d,prov} \leq 15d_b$

A1.6.2.3 Considering vulnerability to gravity (axial) failure

Columns that meet the criteria under 1) or 2) (as applicable) shall be classified as *vulnerable to axial/gravity failure* (with low tolerance to building drift).

1. **For shear limited columns only** (columns with $V_y/V_{pCol,0} > 0.7$):
Columns that satisfy the following criterion shall be classified as vulnerable to axial/gravity failure:

$$\frac{N_G}{A_g \alpha_{v,col} \rho_t f_{yt}} > 5 \quad 2.$$

Where:

- | | |
|------------------|---|
| N_G | Permanent component of axial load in element (column, wall or <i>wall pier</i>) under seismic conditions based on gravity actions [$G + \psi_E Q$] and any other permanent actions (compression taken as positive) |
| A_g | Gross area of column |
| $\alpha_{v,col}$ | Dimensionless parameter for evaluating the effectiveness of transverse reinforcement in resisting shear determined as follows: $0 \leq \alpha_{v,col} = 4 - 4^s/d \leq 1.0$ |
| s | Spacing of transverse reinforcement |
| d | Effective depth of column, taken as $0.8h_c$, where h_c is the dimension of the column in the direction of the shear. |
| ρ_t | Transverse reinforcement ratio $A_v / b_c s$ |

The value of $\alpha_{v,col} \rho_t$ shall not be taken as greater than 0.0175, nor greater than 0.0075 when ties are not adequately anchored into the core. For columns supporting discontinuous walls or braced frames, N^*_c shall be used in place of N_G (see A1.6.3).

3. **All columns:**
Columns with *high permanent axial load*, $N_G > 0.2 A_g f'_c$ and which are also *lightly confined* shall be classified as vulnerable to axial/gravity failure. Columns are considered to be *lightly confined* where they satisfy either of the following criteria:

Widely spaced stirrups:

$$s > \frac{h}{2}$$

Or,

Low confining steel ratio:

$$\rho_t < 0.2\%$$

Or,

Low core area ratio:

$$\frac{A_g}{A_c} > 1.5$$

5.

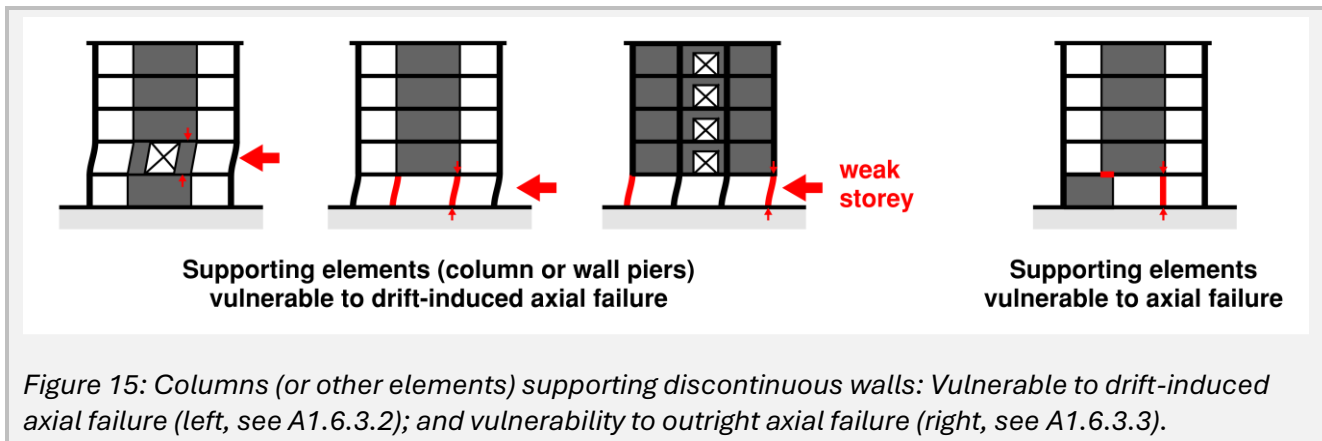
A1.6.3 Vulnerable columns (or other elements) supporting discontinuous walls or braced frames

A1.6.3.1 General

Elements supporting discontinuous walls or braced frames can have the following vulnerabilities:

1. Vulnerability of the columns to drift in the discontinuous storey, if the discontinuity also results in a *weak storey*, or torsional vulnerability. If there are other walls in the discontinuous storey (e.g. wall is offset transferred to another location in plan) then this vulnerability may not be present.
2. Vulnerability of the columns (or supporting elements) supporting the wall to an outright axial failure, due to high overturning demands in conjunction with load path discontinuities or detailing vulnerabilities.

These two distinct vulnerabilities are illustrated in Figure 15, and classified in the following two subsections.



A1.6.3.2 Supporting elements (columns or wall piers) vulnerable to drift-induced axial failure

This column or pier vulnerability is classified only when a drift-amplifying configurational vulnerability exists other than the vertical discontinuity itself. For clarity, this means one of the following must also be present in the discontinuous storey:

1. Overall weak *vertical LFRS* (A1.5.2)
2. Severe torsional sensitivity (A1.5.3)

3. Weak storey (A1.5.4)

Consider overturning forces on the columns or piers

For columns or piers supporting walls or braced frames, overturning action on the columns resulting from lateral earthquake loads shall be considered when the wall or braced frame over has an overall aspect ratio $H/B > 1$ (see Figure 16).

When one of the above listed configurational vulnerabilities is present, all provisions of A1.6.2 and A1.6.8 shall apply to respectively classify the drift-induced vulnerability of columns and piers, **with the exception that N^*_c shall be used in place of N_g when the aspect ratio of the wall or frame over satisfies $H/B > 1$** . See A1.6.3.3 for definition of N^*_c (which includes earthquake actions).

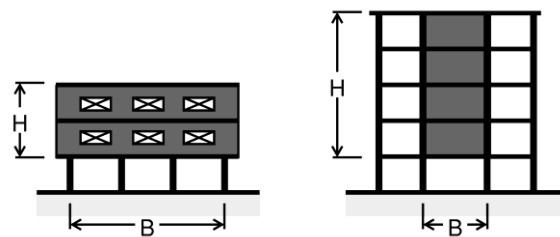


Figure 16: Aspect ratio of wall or braced frame supported on columns (B is lever-arm distance between supporting elements providing overturning resistance).

A1.6.3.3 Supporting elements (columns, wall piers or wall overlaps) vulnerable to axial failure

Columns or *wall piers* supporting discontinuous walls or braced frames are evaluated as *vulnerable to gravity/axial failure* where they meet the following criteria:

1. **For compressive actions:**
 1. For all *wall piers*, wall overlaps and for columns that are *lightly confined* as defined by A1.6.2.3 (3, where $N^*_c > 0.5 A_g f'_c$.
 2. For any column, where $N^*_c > 0.7 A_g f'_c$.
2. **For tensile actions:**
 1. Where $N^*_t > A_g f_{splice}$, unless the conditions in (b) are met:
 2. For columns only, the tensile criteria can be ignored where all of the following criteria are met:
 1. Deformed bar reinforcing is used, and the section tension capacity is not limited by reinforcing splicing or anchorage; and,
 2. Stirrup ties are adequately anchored into the core; and,
 3. All longitudinal reinforcing bars are effectively enclosed by stirrup ties or hoops with spacing $s \leq 10d_b$.

Where:

N^*_c = The axial compressive load from analysis under the earthquake actions prescribed in A1.9.2 acting in conjunction with permanent actions/gravity loads; and,

N^*_t = The tension demand (if present) under the same combination of actions.

f_{splice} = Stress that can be developed by a spliced reinforcing bar, as determined by the *Seismic Assessment Guidelines* section 2025-C5.4.4.

A_{overlap} = Overlapped cross-sectional area of a wall or wall pier supporting a discontinuous wall, as illustrated in Figure 17.

Section A1.9.2 permits N_c^* to be no greater than the overstrength derived axial load N_o . For a regular configuration of a wall supported on a column at each end (for example), this may mean N_o plus the tension capacity of the other supporting column.

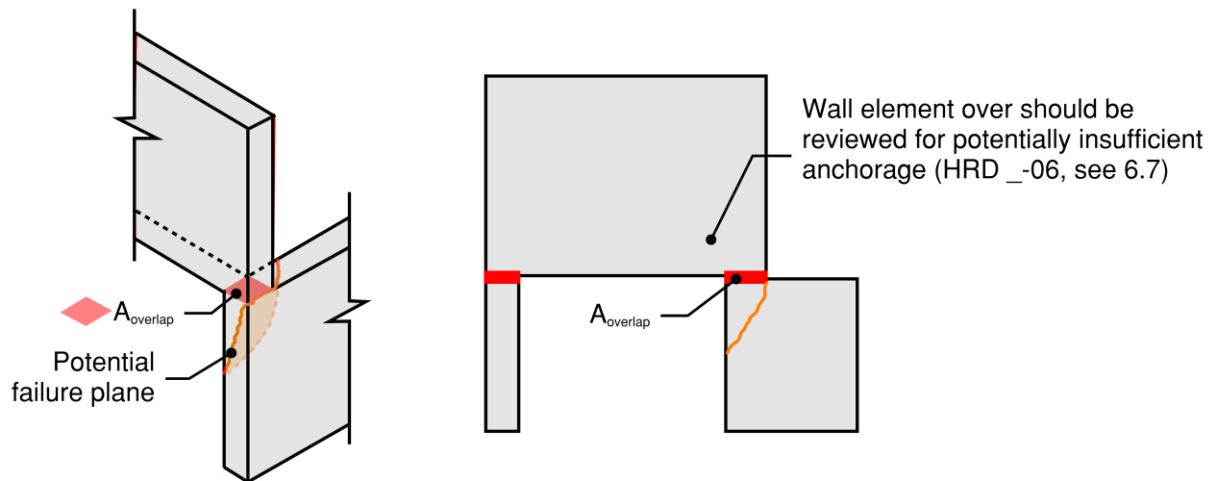


Figure 17: Overlapped wall components (pier or wall overlaps) supporting discontinuous walls, and definition of overlap area A_{overlap} .

A1.6.3.4 Potentially vulnerable transfer beams supporting discontinuous walls

Discontinuous walls supported on beams (excluding beams on grade) should be considered vulnerable to vertical (gravity) failure, unless it can be shown that the supporting beams have adequate capacity to resist actions imposed by the wall based on a rational capacity design procedure.

This criterion only applies to significant discontinuities below cantilevered shear walls or braced frames. It does not apply to minor irregularities or vertical discontinuities within punched wall or pier systems (such as a wall pier supported on spandrel, spanning over a wider wall opening below).

A1.6.4 Vulnerable beam-column joints

A1.6.4.1 Vulnerability to shear failure

Exterior or corner joints should be considered *vulnerable to lateral failure* (i.e. can contribute to loss of lateral building stability) where:

- Article I. Hoops are spaced at $> h_c / 2$ within the joint; or,
- Article II. In the direction under consideration, the joint aspect ratio $h_b / h_c > 1$ and beam bars have *poor anchorage*, meaning either:
 - Section 2.01 terminated according to case 2 or 3 (Figure 18).
 - Section 2.02 Terminated according to case 1, but with development $L_{\text{dh,prov}} < 0.7 h_c$.

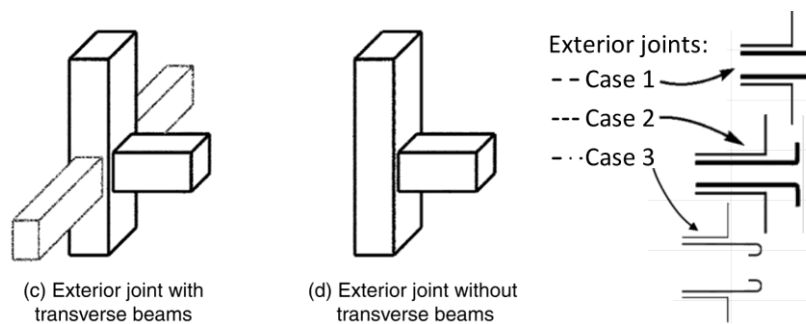


Figure 18: Example of exterior joint configurations including corner joints that can be vulnerable when lightly confined.

A1.6.4.2 Vulnerability to shear and axial failure

Exterior or corner joints identified as vulnerable to shear/lateral failure (to A1.6.4.1) should be considered also **vulnerable to axial/gravity failure** where they also meet the following criteria:

1. An axial load ratio on the effective joint area (as defined in the *Seismic Assessment Guidelines*) $N_G > 0.3 A_j f'_c$; and,
2. Joints are lightly confined, either having fewer than two stirrups within the joint or a transverse reinforcement ratio $A_{jh}/(b_j h_b) < 0.2\%$.

A1.6.5 Slab-column joints without continuity steel (vulnerable to loss in gravity support)

Any slab-column connection without effectively continuous bottom reinforcement passing through the column in each direction having cross sectional area $A_{s,bottom} \geq 0.6 V_g / f_y$ should be considered vulnerable to punching shear failure and loss in gravity support of the floor when the following criteria is also met:

1. A gravity demand ratio greater than $V_g/V_p > 0.2$

A1.6.6 Wall(s) vulnerable to axial failure

A1.6.6.1 General

The provisions of this section focus on walls or *wall piers* which are vulnerable to abrupt compression failures.

Fewer configurational vulnerabilities apply to walls

Walls shall only be classified as vulnerable in accordance with this section when compounded with the following (as specifically listed in A1.3.2: *List of High-Risk Deficiencies*):

1. Overall weak *vertical LFRS* (HRD B-05).
2. Severe torsional sensitivity (HRD C-05).

A1.6.6.2 Walls susceptible to through-the-thickness axial failure

Walls or interlinked (strongly coupled) wall complexes shall be considered vulnerable to axial/gravity failure when found to be susceptible to through-the-thickness axial failure. Such walls are classified

as those with axial load parameters exceeding the following criteria (but only when compressive action under gravity and permanent actions $N_G > 0.01 A_g f'_c$).

The selection of the appropriate criteria shall be based on the minimum amount of regularly spaced tie reinforcement present over the full extent of the *boundary area* as computed for all relevant directions of loading.

Table 6: Axial load and reinforcement configuration parameter for testing susceptibility to through-the-thickness axial failure

Wall reinforcement configuration	Limiting value of $\frac{(A_s + A'_s)f_y + N^*}{t_w l_w f'_c}$
$\frac{s}{t_w} \leq 1$	0.30
$\frac{s}{t_w} > 1$	0.08

Where:

A_s Area of tension reinforcing, which should be taken as the tension zone force (under section analysis for M_p at N^*).

A'_s Area of compression zone reinforcing which should be taken as the total area of longitudinal reinforcing steel contained within the compression zone.

f_y probable yield strength of longitudinal reinforcing

t_w Thickness of wall within the compression zone.

l_w Wall length (see clarification below for application to multiplanar walls).

s vertical spacing of regularly spaced closed hoops or cross ties in the *boundary area*.

The *boundary area* shall be taken as the area within $0.75c$ of the extreme compression fibre for general cases. Where the boundary region dimensions comply with NZS 3101:2006 Inc. Amd. 1-3 Eq. 11-20, the *boundary area* can be taken as the area within $0.1l_w$ of the extreme compression fibre.

c Distance of neutral axis from extreme compression zone fibre of the wall section.

N^* Axial load, taken as N^*_c as defined in section (A1.9.3.3).

Application to non-planar walls

For multiplanar walls, the following clarifications apply:

1. Actions on orthogonal axes as well as off-axis actions should be considered. This can be considered by section analysis with rotation applied along the orthogonal axes (parallel and perpendicular to the primary wall segments); and then additional checks to a set of axes rotated by 45 degrees.
2. The wall length, l_w should be based on the total projected wall length in the direction being considered.
3. Where the compression zone comprises multiple wall segments, the t_w parameter should be taken as equal to an effective compression zone wall thickness, $t_{w,eff}$ computed as the total area of the compression zone, A_{comp} (at M_p , computed for N^*), divided by the compression

zone depth, c (where c is the projected compression zone depth in the direction under consideration).

Application to coupled walls (assemblages of walls)

For coupled walls, the walls can be evaluated for susceptibility to through-the-thickness axial failure by either of the following three methods:

1. By section analysis at M_p under the axial load conditions N^* as determined by analysis subject to the *benchmark seismic hazard*.
2. Where walls are weakly coupled, by section analysis for M_p under the axial load conditions N_o consistent with the development of a plastic sway mechanism (developing $\sum V_{oe}$ in coupling beams). That is, taking $N^*_c = N_o$.
3. Where walls are strongly coupled, by section analysis for M_p as a total combined section (assuming plane sections remain plane), subject to N_G .

Figure 19 and Figure 20 show how the through-the-thickness axial failure criteria can be applied to various wall configurations.

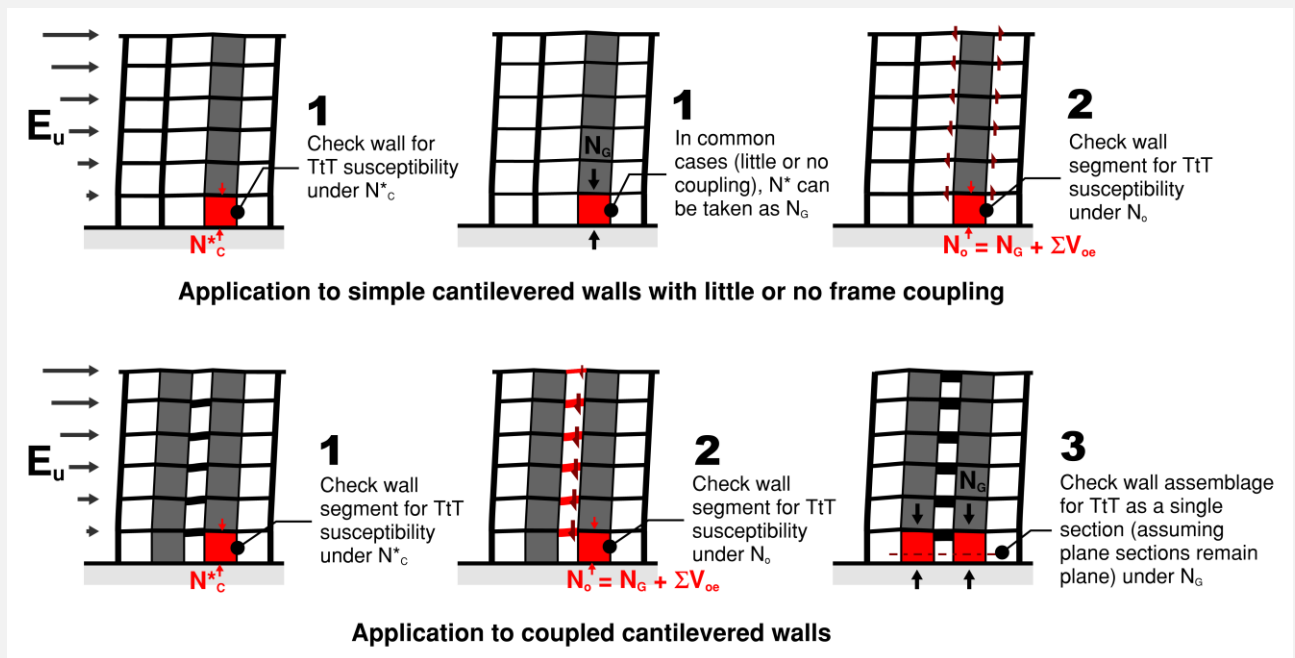


Figure 19: Examples of application for cantilevered walls, coupled or uncoupled.

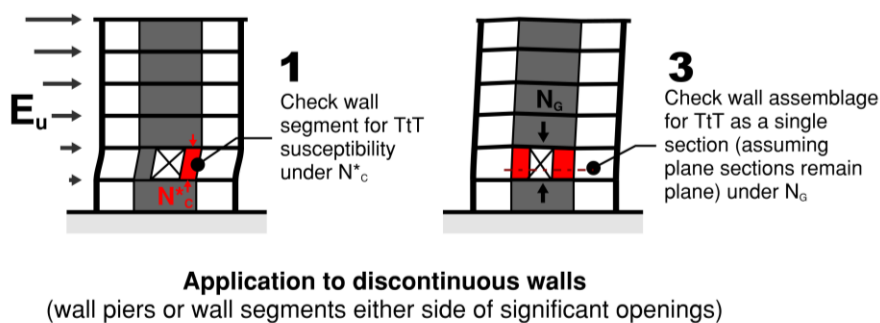


Figure 20: Example of checking wall segments (or wall piers) for through-the-thickness (TtT) axial failure susceptibility as individual wall segments (left); or a complete wall section (right).

A1.6.6.3 Multi-planar wall (lightly reinforced) susceptible to axial instability failure

Multi-planar wall sections in slender walls (with aspect ratio $L_c / L_w > 2$) shall be considered vulnerable and susceptible to axial instability failure (through an unstable compression flange or boundary element) when both of the following conditions (1 and 2) exist:

1. Compression forces are carried by potentially unstable flange or boundary elements. Such elements are defined as:
 1. Wall flanges that are singly reinforced⁶, or are without regularly spaced cross ties with $s / t \leq 1$.
2. One or a number of wall segments resisting shear forces (in the direction under consideration) are susceptible to sliding shear, which shall be assumed where (a) and (b) apply together:
 1. The wall segment(s) are part of a multiplanar wall which is overall susceptible to the concentration of deformation in a single crack, including:
 1. Walls with deformed bar and for which distributed cracking is not able to be assumed in accordance with the *Seismic Assessment Guidelines* section 2025-C5.5.1.6; or,
 2. Walls with plain round reinforcing.
 2. Where the web segment (excluding any overlap with flange or boundary elements) is entirely in tension. That is, the compression zone (under section analysis for M_p ⁷) is contained solely within a potentially unstable flange element.

A1.6.6.4 Walls acting out-of-plane

Walls shall be considered vulnerable where moment and shear capacity out-of-plane contributes to the lateral stability of *heavy floors*, and without regular crossties at a vertical spacing of $< t/2$.

⁶ This does not include walls which are doubly reinforced but have a singly reinforced joint between wall panels.

⁷ For section analysis to M_p , the same options for axial load conditions shall apply as for testing susceptibility to through-the-thickness axial failure in A1.6.6.2.

A1.6.7 Walls vulnerable to significant lateral strength loss (high seismic zone only)

A1.6.7.1 General

For high seismic zones, walls meeting any of the following classifications shall also be classified as vulnerable to loss in lateral strength only (they should not be considered as vulnerable to axial failure unless classified as such elsewhere in A1.6.6.2 to A1.6.6.4):

A1.6.7.2 Singly reinforced walls

Singly reinforced walls with aspect ratio $L_c / L_w > 2$ above the critical section, and in buildings with three or more suspended floors of heavy construction, and without enlarged boundary elements or piers.

A1.6.7.3 Lightly reinforced walls

Either of the following, in slender walls with aspect ratio $L_c / L_w > 2$:

1. Walls with deformed bar reinforcing and for which distributed cracking is not able to be assumed in accordance with the *Seismic Assessment Guidelines* section 2025-C5.5.1.6.
2. Lightly horizontally reinforced with $\rho_h < 0.25\%$.

A1.6.7.4 Walls with a lack of reinforcing anchorage in PPHZ

Either of the following in slender walls with aspect ratio $L_c / L_w > 2$:

1. Lap splices concentrated in a single storey within *PPHZ* where:
 1. For plain round reinforcing $L_{d,prov} \leq 24d_b$.
 2. For deformed reinforcing $L_{d,prov} \leq 15d_b$.
2. Vertical reinforcing not anchored into foundation or supporting element with equal or greater reinforcement ratio as the wall, except where the anchorage configuration is demonstrated capable of resisting demands arising from the lesser of a), b) or c):
 1. 70% of the wall probable section strength at the critical section computed assuming full anchorage of all wall reinforcing.
 2. The probable tension capacity of the supporting element (or in the case of shallow foundations, the capacity to resist uplift).
 3. Flexural action at the critical section under the earthquake actions prescribed in A1.9.2 acting in conjunction with permanent actions/gravity loads.

The criteria in 2) can apply to walls that are inadequately anchored into supporting piles, or similarly to discontinuous walls supported on columns, transfer beams or wall overlaps. Where the connection is stronger than the supporting elements as in 2(b), the provisions for elements supporting discontinuous walls will govern (section A1.6.3.3).

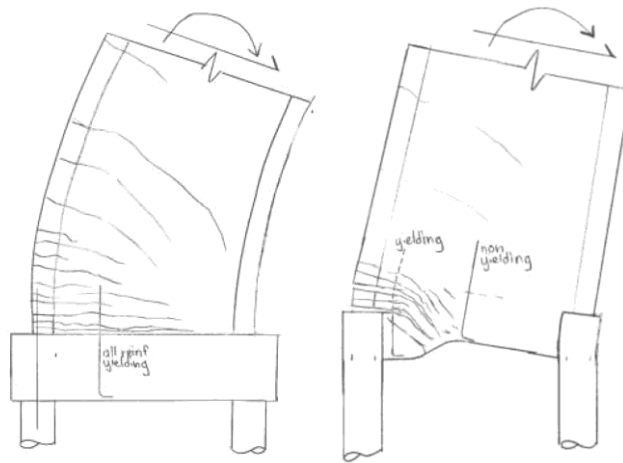


Figure 21: Example of conditions that can lead to a lack of reinforcing anchorage in PPHZ (right)

A1.6.8 Wall piers vulnerable to axial failure

A1.6.8.1 Defining wall piers

For application of this section, *wall piers* are defined as a vertical wall segment within a structural wall, bounded horizontally by two openings or by an opening and an edge, with a ratio of clear height H to horizontal length L_{wall} greater than 1.0, and either:

1. singly reinforced; or,
2. (where doubly reinforced) having a ratio of horizontal length to wall thickness L_w/t_w greater than or equal to 4.

This includes *wall piers* incorporating columns/pilasters to the interior or to one end only of their section. Clear height H means the maximum clear height of pier that can be intersected by a diagonal crack in either direction of loading (between floors, beams or spandrel elements).

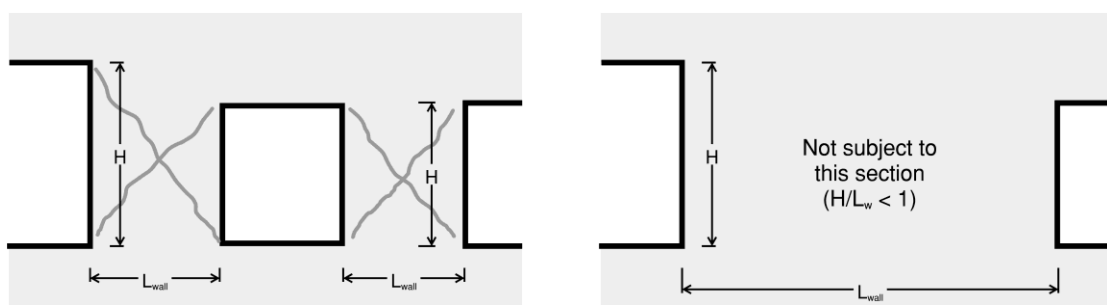


Figure 22: Schematic of potentially vulnerable wall piers. Shear controlled piers with higher aspect ratios are more vulnerable to axial (gravity) failure.

These provisions for piers focus on frame-like behaviour of pier-spandrel systems, or other situations where there is frame-type action. Figure 23 illustrates what is meant by frame-like behaviour. Shear-limited piers can be vulnerable to sliding off steep cracks when they have higher aspect ratios (within stories), and higher axial loads.

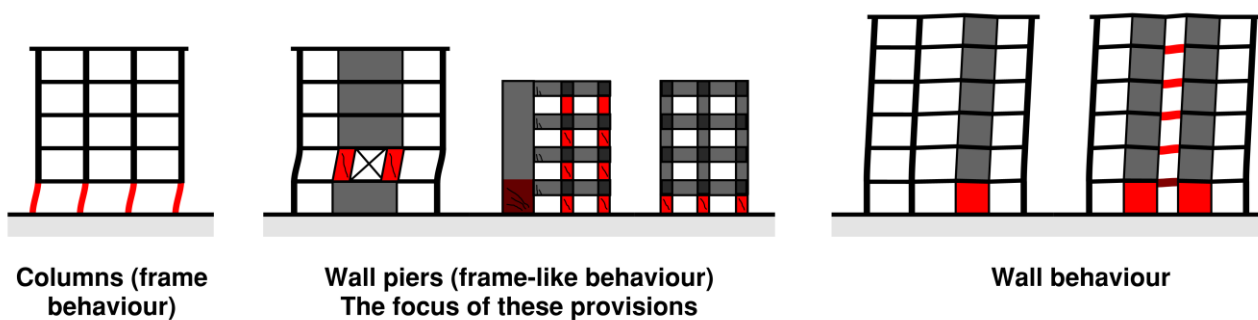


Figure 23: Schematics illustrating the behaviour of columns in frames and the similar frame-like behaviour of pier systems.

A1.6.8.2 Wall piers vulnerable to axial failure

Flexure controlled *wall piers* are classified as *vulnerable to axial/gravity failure* if they are found to be susceptible to through-the-thickness axial failure in accordance with section A1.6.6.2.

Shear-controlled *wall piers* (or any pier not confirmed as flexure controlled) are classified as *vulnerable to axial/gravity failure* (with low tolerance to building drift) when the following criteria are met:

1. The aspect ratio of the pier H/L_w is greater than 1.5; and,
2. The axial load ratio, $N_G / A_g f'_c > 0.05$.

Where:

- N_G Permanent actions, as defined in A1.6.2.1. N_G Shall be substituted for N^*_c for piers supporting discontinuous walls (where required by A1.6.3).
- H The maximum clear height of pier that can be intersected by a diagonal crack in either direction of loading (between floors, beams or spandrel elements)
- L_w Length of pier
- A_g Gross area of pier (including any pilaster elements)

Application to wall piers incorporating pilasters or columns

For *wall piers* incorporating pilasters or columns with closed hoops, consider whether a *severe physical consequence* exists. Where a pilaster evaluated as a column to section A1.6.2 is found not *vulnerable to axial/gravity failure* when subject to the full pier axial force, then *severe physical consequence of failure* need not be assumed.

A1.7 High-Risk Configurational Deficiencies

A1.7.1 General

Some buildings can be high-risk solely due to configuration and strength, irrespective of whether there are component deficiencies. This Annex defines exceptionally weak frames (vulnerable to P-Delta instability) as an *HRD*.

A1.7.2 Exceptionally weak frame (vulnerable to P-Delta instability)

Frames satisfying the following criteria classify as *HRDs*, as they are vulnerable to P-Delta instability:

1. Either of the following *configurational vulnerabilities* are identified:
 1. Weak storey (section A1.5.4); or,
 2. Storey-sway potential (section A1.5.5).
- And,
2. Either:
 1. The *column wall index CWI* in any affected storey is less than 0.08%; or
 2. The *torsional strength index* $T_{DI}/T_{CI} > 0.8$.

By definition, buildings satisfying these criteria have a *severe physical consequence* of failure.

In determining remediation requirements (per Annex 2), the *vulnerable components* are those affected by the *weak-storey* or *storey-sway potential configurational vulnerabilities*. For *storey-sway potential*, this applies only in the lower half of the building, and in storeys either side of a vertical irregularity as identified by NZS 1170.5:2004.

See Figure 13 (section A1.5.5) for an example of 2(b).

Note that modern taller buildings can have low CWI due to a combination of their longer period and selected design ductility. However, if capacity designed appropriately, it is unlikely that such buildings would meet criterion 1) as they should not have a weak storey or storey-sway potential.

A1.8 Area-Based Indices used in HRD Reviews

A1.8.1 Column-Wall Index (CWI)

A1.8.1.1 General

The *Column-Wall Index (CWI)* is calculated as follows, and elements of the calculation are further clarified in the subsequent subsections.

1. The *Column-Wall Index* should be calculated for each primary building axis at sufficient storeys to determine the critical storey (lowest value of CWI) for the building.
2. The single lowest value is taken as the *Column-Wall Index (CWI)* for the building as a whole.
3. The value computed at the storey and/or direction under consideration is used in other parts of this Annex (in the detailed definition of component and configurational irregularities).

Depending on irregularities in building massing and/or variations in column/wall geometry, the critical storey may not be the lowermost storey.

$$CI = \frac{\sum (c_j A_{RC\,col,j})}{\sum A_{floor}} \quad 4.$$

$$WI = \frac{\sum A_{RCwall} + 0.5 \sum A_{RCMwall} + 0.1 \sum A_{Masonrywall}}{\sum A_{floor}} \quad 5.$$

$$Column-Wall\ Index\ (CWI) = WI + CI \quad 6.$$

Where A1.8.1.2 and A1.8.1.3 clarify the calculation of WI and CI respectively, and:

$A_{RC\,col,j}$	Gross cross-sectional area of RC column j. See A1.8.1.5 for treatment of columns that are not RC (for example steel or composite columns).
c_j	Column effectiveness factor for column j, taken as: 0.5 for columns that are part of a frame in the direction under consideration (columns have frame action with spandrels or beams) 0.25 for columns that are not part of a frame in the direction under consideration.
A_{RCwall}	Gross cross-sectional area of RC structural wall.
$A_{RCMwall}$	Gross cross-sectional area of reinforced masonry wall, effectively engaged to transmit lateral storey shear forces in the direction under consideration.
$A_{Masonrywall}$	Gross cross-sectional area of URM wall, bounded by building frame elements or otherwise effectively engaged by imposed building deformations in the direction under consideration.
$\sum A_{floor}$	The sum of gross floor area (GFA) above the storey, typically calculated on the gross floor plate outline and without subtracting voids (stairs, lifts and risers and the like). Includes all floors, irrespective of construction materials. Includes heavy (concrete) roofs, but excludes lightweight roofs.

The above definitions are subject to the following subsections which clarify their application.

A1.8.1.2 Calculation of Wall Index, WI

RC or RCM wall area should be based on the web segment of any wall acting in the direction under consideration, including its projection into an enlarged boundary or flange (Figure 24). The remaining area of flanges or enlarged boundaries should be excluded from WI.

Effective RCM (contributing to $A_{RCMwall}$) should be limited to fully grouted RCM walls with reinforcing connectivity to floors at the top and base of panels. Otherwise, the area shall instead be taken as contributing to $A_{Masonrywall}$ (based on the gross area).

The area contribution of angled wall segments should be factored by the cosine of the angle between the axis of the wall segment and the primary building axis under consideration.

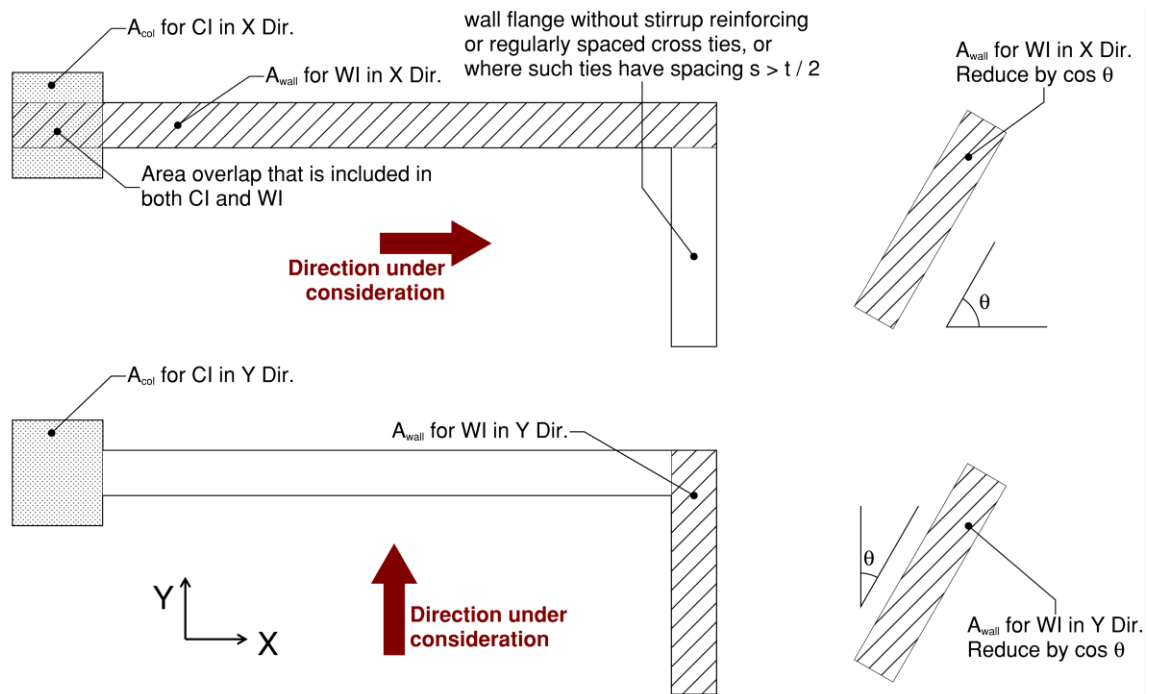


Figure 24: Clarifying contribution to Wall Index WI and Column Index CI

A1.8.1.3 Calculation Column Index, CI

For the *Column-Wall Index* used in this Annex (for New Zealand buildings), column area effectiveness is reduced by $c_j = 0.5$ for columns that are part of frames, and $c_j = 0.25$ where there is no frame beams considered effective in the direction under consideration. Where an effective beam of any dimensions (including a concrete secondary beam) frames into the column at an angle of up to 45 degrees from the direction under consideration, c_j should be taken as 0.5 (the column can be considered part of a frame).

Note the following additional clarifications:

1. Concrete beams with intentional flexural release shall be considered ineffective.
2. For hybrid structures, a steel beam with simple end connection (not effective at resisting flexure) shall be considered ineffective.
3. For slab-column configurations (with or without capitals or drop panels), use $c_j = 0.25$.

For use of C_I (or effective column areas) when assessing configurational irregularities, see A1.8.2.

Area contributing to W_I or C_I

Elements with the following member dimensions (in the direction under consideration) should be excluded from C_I and instead counted as wall area (contributing to W_I):

4. Aspect ratio within a storey (h_i / L_w) < 2.0, where h_i is the total interstorey height; or,
5. Shear span to depth ratio L_c / L_w < 1.0 (such as piers)

Enlarged boundary elements with stirrup reinforcement

The gross area of enlarged boundary elements (including the overlapped area that is also contributing to W_I) should be included as column area (contributing to C_I) where enlarged boundaries contain confining transverse reinforcing (hoops or ties) at a spacing of less than half of the relevant cross section dimension of the boundary element (Figure 24).

Otherwise, wall segments (including wall flanges) acting out-of-plane should not be included as column area.

A1.8.1.4 Accounting for brace elements present in reinforced concrete buildings

Braces present or retrofitted in RC buildings should be considered by conversion of the horizontal component of the assessed probable capacity of the brace system (including consideration of member capacity and connections) to an equivalent wall area under a 2MPa shear stress. The resulting area should be included in W_I .

A1.8.1.5 Application of Column-Wall Index to composite or hybrid structures

In hybrid structures (where vertical structure is combination of structural steel and concrete), the contribution of structural steel columns or braced frames can be excluded, or otherwise approximated by the following:

For columns that are part of the building's primary structural frame

1. For bare structural steel columns, by taking an equivalent area of A_{RCCol} based on 10 times the cross-sectional area of the steel section, A_s .
2. For concrete filled steel tube sections and concrete encased steel sections, as for an RC column (by the unmodified area of total cross section, A_g).
3. The contribution of cast or wrought iron should be excluded.
4. The contribution of bare structural steel columns with no webs parallel to the direction under consideration should be assigned $c_j = c_k = 0.25$.

For braced frames

5. For braced frames, in accordance with A1.8.1.4 (and for torsional sensitivity only, further subject to the aspect ratio adjustment given in A1.8.2.2 applied to the frame overall).

A1.8.2 Area-based methods for assessing configuration irregularities

A1.8.2.1 General

For assessing configuration irregularities only (soft or *weak storey* vertical irregularity, or torsional irregularity), effective areas of elements shall be based on the factors applicable to *WI* and *CI*, further modified as follows.

A1.8.2.2 Effective area of walls for assessing torsional sensitivity

For walls (including *wall piers*), when establishing A_{eff} for assessing torsional sensitivity only, the following aspect ratio adjustment shall apply to wall area in addition to those ordinarily applicable to the calculation of *WI* based on the wall type (see A1.8.1.2):

$$\text{Aspect Ratio Adjustment Factor} = 0.25 < \frac{H_{w,eff}}{L_w} < 1.0 \quad 6.$$

$$H_{w,eff} = M/V \quad 7.$$

The effective wall height H_{eff} approximates the shear span of the wall, based on M/V at the critical section. This can be determined by simple analysis under a nominal distribution of equivalent lateral forces, and for simple/common cases should be estimated as follows:

1. 70% of the wall height for cantilevered uncoupled walls.
2. For *wall piers* acting in conjunction with strong spandrels (punched wall or pier/spandrel system), half of the clear height (generally resulting in an adjustment factor of 1.0).

Full form of equation and clearer definition of variables in due course. Consider consolidating into torsional sensitivity.

A1.8.2.3 Effective area for columns for assessing torsional sensitivity or weak storey

For columns, when establishing A_{eff} for assessing torsional sensitivity or when assessing for soft storey (soft or *weak storey* vertical irregularity), the c_j factor in the equation for *CI* shall be replaced by the c_k as follows:

$$\sum A_{eff} = \sum c_k A_{RCcol,k}$$

Where:

- | | |
|-------|--|
| c_k | 1.0 for columns that are short columns (as defined below) in the direction under consideration. |
| c_k | 0.5 for other columns that are part of a frame in the direction under consideration (act in conjunction with spandrels or beams) |
| c_k | 0.25 for columns that are not part of a frame in the direction under consideration |

Short columns

Short (captive) columns are defined as follows (and do not include columns that are captive by full height concrete or masonry infill):

1. With clear height H less than 70% of the clear height of typical columns at that level; and,
2. Where $(h_c / H)^2$ is less than < 70% of the typical value for columns at that level.

Where:

h_c Depth of column

H Clear height of column

A1.9 Analysis Methods used in the Identification of High-Risk Deficiencies

A1.9.1 General

Most high-risk deficiencies do not require global structural analysis, as their identification is based on the presence of specific building features rather than detailed engineering assessment.

However, limited structural analysis can still be useful (or necessary) to assist in the identification of these features, for example in the following situations:

1. Helping to estimate gravity load N_G on elements where this parameter is required (as an alternative to estimating N_G on a simpler tributary or hand-estimated basis.
2. Helping to estimate earthquake overturning loads (N^*_e and N^*_t) when evaluating walls, or elements supporting discontinuous walls. Note that axial loads in walls, and elements supporting discontinuous walls, are the only aspects requiring consideration of earthquake actions E_u in HRD evaluation.
3. In limited situations, to help assess whether there is a *severe physical consequence* of failure. For example where it is not obvious if the criteria listed in section A1.2.4 are met (Table 3: Defining severe physical consequence of failure).
4. If desired, to help in evaluating whether a configurational vulnerability exists, or to assist in establishing strength hierarchy at a joint or global level.

For earthquake induced axial loads in (2), the purpose of structural analysis in HRD review (using computer analysis, where appropriate) is to estimate the general magnitude of variable (earthquake-induced) axial force demands (at the benchmark seismic hazard level), so that their vulnerability of elements can be appropriately classified by the criteria. It is not intended to be an accurate assessment. Therefore, appropriate simplifications are permitted.

A1.9.2 Earthquake demand

For the sole purpose of assessing whether *HRDs* exist, the elastic site spectrum for horizontal loading shall be the *benchmark seismic hazard* E_u as defined in Annex 2.

A1.9.3 Structural analysis

A1.9.3.1 General

For the sole purpose of assessing whether *HRDs* exist, structural analysis (if used) shall be undertaken in accordance with this section.

A1.9.3.2 Earthquake actions and structural analysis method

For earthquake actions, the equivalent static method of analysis shall be used. Earthquake actions shall be based on the elastic site spectrum for horizontal loading E_u given by A1.9.2 with horizontal design action coefficient for the equivalent static method $C_d(T_1)$ based on $k_\mu = \mu = 1.0$ and $S_p = 0.9$.

Actions should be applied through the centre of mass (excluding accidental mass eccentricity), with direction of action and concurrency in accordance with NZS 1170.5:2004 Cl. 5.3.1.2 (or alternatively considering at 45-degree increments).

If reviewing element strength hierarchy and/or development of plastic lateral mechanism(s), the load vector should be based on the same equivalent static method.

A1.9.3.3 Determining Axial Loads

Where gravity and permanent actions only are to be considered, N_G is determined as follows:

N_G Permanent component of axial compressive force carried by column, wall or *wall pier* under seismic conditions based on gravity actions $[G + \psi_E Q]$ and any other permanent actions, N .

Where earthquake actions are to be considered, N^*_c and N^*_t shall be determined considering earthquake actions E_u along with gravity and other permanent actions:

N_E Axial load in element (column, wall or *wall pier*) under earthquake actions E_u .

N^*_c Maximum compression force (taken as positive) under earthquake actions E_u acting in conjunction with gravity and permanent actions $N^*_c = N_G + N_E, > 0$.

N^*_t Maximum tension force (taken as positive) under earthquake actions E_u acting in conjunction with gravity and permanent actions $N^*_t = N_G + N_E, > 0$.

N^*_c and N^*_t need not be greater in magnitude than the overstrength derived axial load N_o consistent with the development of all plausible plastic lateral mechanisms in conjunction with gravity and permanent actions, as determined by the *Seismic Assessment Guidelines*.

Axial load in walls and elements supporting discontinuous walls are the only aspects requiring consideration of earthquake actions E_u in HRD evaluation. Other elements only require estimate of gravity and permanent actions.

The purpose of structural analysis in HRD review, is to estimate the order-of-magnitude action effects from gravity and earthquake actions in components that are part of HRD review (primarily axial forces in vertical primary elements). It may also be used to help identify configurational irregularities.

For simple, regular structures, simple calculation-based methods for estimating axial loads are acceptable. This includes estimating gravity loads via tributary basis, and estimating earthquake loads by simple distribution of equivalent lateral forces. FEMA P-2018 contains examples of such methods. In less regular configurations, simple computer analysis may be more practical (see A1.9.3.4).

A1.9.3.4 Analysis modelling for high-risk deficiency review

The effects of variable axial load on element deformation capacities are highly uncertain. Therefore, if computer analysis is used as part of *HRD review*, high levels of accuracy in modelling are not required and a simple linear-elastic model is sufficient.

The following simplifications can be made at the soil-structure interface:

- Vertical fixity can be provided at the soil/structure interface (except see following for piles supporting slender *vertical LFRS*).
- For piled foundations supporting slender *vertical LFRS* (high aspect ratio walls or braced frames), vertical fixity should be provided at the base of piles, and the structural axial stiffness of piles included on an effective cross-sectional area $0.5 A_g$ times the modulus of elasticity of concrete.
- Rigid rotational supports at the soil/structure interface can be used to represent foundation elements providing a significant degree of rotational rigidity to vertical components (piles, foundation beams and large foundation pads). Otherwise ‘pin’ supports can be used.
- Translational fixity to the building can be provided at a common ground plane (simple judgements may be necessary to avoid over-constraint to stepped ground planes that artificially over-restrained vertical LFRS).
- Subterranean basements can generally be excluded unless the ‘backstay’ force path is significant for the *vertical LFRS* (e.g. core wall) and where this is material to the axial load obtained in the *vertical LFRS*, or where there are potentially vulnerable elements in the basement requiring evaluation using the outputs of the analysis.

Appropriate adjustments for cracked stiffness of elements should be made, consistent with building performance at *ultimate capacity*.

A1.9.4 Specific Foundation Assessment (confirming whether poor founding conditions exist)

If the criteria in section A1.5.6 indicate poor founding conditions, a specific foundation assessment can be undertaken (or existing available assessment information reviewed) to explore whether poor founding conditions can be considered unlikely (and thus assumed not present).

Assessment should be undertaken by a Chartered Professional Engineer with a geotechnical practice field, in collaboration with the structural engineer. The assessment should consider the soils, foundations and structure, and whether “poor founding conditions” and susceptibility to significant differential settlement and tilting effects as defined in Section A1.5.6 are expected to exist, or whether they are unlikely. The assessment should consider earthquake demands from benchmark seismic hazard as given in Annex 2. In general cases it is expected that a desktop and qualitative assessment will be adequate. Quantitative soil foundation structural interaction (SFSI) analysis is only expected to be required in more complex cases.

High-risk deficiencies in relation to poor foundation conditions consider risks from vertical deformations. Therefore, specific foundation assessment should be limited to considering vertical stiffness and capacity of the foundations supporting the vertical LFRS only. Lateral deformation (including lateral spread and lateral inertial or kinematic effects on piles) does not need to be considered.

The objective is to identify conditions which could allow large (>100mm) of vertical differential deformation of the foundations under the earthquake actions (shaking and structural actions). Examples could include:

Liquefiable soils being present within 3B below a shallow foundation, where B is the foundation width. Liquefiable soils being present within 3B above and/or 5B below the base of an end bearing pile, where B is the base width of the pile.

Where shaft resistance of a pile is relied on for a pile's tension or compression capacity, liquefaction effects potentially reducing that shaft resistance by more than say 25%.

Desktop assessment for poor founding conditions that either identifies adverse geotechnical conditions, or finds them unlikely to be present, should normally suffice. The assessment should consider the benchmark seismic hazard as given in Annex 2, without application of a geotechnical step change factor.

Quantitative assessment of a foundation's vertical capacity is not expected as part of HRD reviews, but could be undertaken if desktop assessment is unable to rule out an HRD (see Annex 2).

A1.10 Example Template Forms and High-risk Deficiency Review Checklists – Draft Only

A1.10.1 Building Overview Standard Form of Table

Table 7: Example Only - Building information and review outcome (EPB system scope and HRD review)

Building information and review outcome (EPB system scope and HRD review)		
Location		Wellington
Seismic Zone		High
Year of design		Early 1960s
Building Use		Multi-unit residential
Importance Level (NZS 1170.0:2002)		IL2
No. storeys (description)		10 storeys (slab on grade, 9 suspended occupied floors, lightweight roof over with heavy lift penthouse structures)
Current EPB status		EPB
Basis for EPB notice If applicable/known		As if not assessed. A 2010 assessment report exists.
Construction	Floors	In-situ RC two-way slabs
	Gravity system	Slabs supported on interior and exterior walls, with point support from some interior columns
	Lateral stability system	RC walls. Some walls discontinuous at ground floor.
	Foundations	Piled
	Roof construction	Lightweight roof, concrete lift overrun structure
Categorisation	Medium or High Seismic Zone	Yes
	Heavy Materials	Yes

Building information and review outcome (EPB system scope and HRD review)			
To EPB Methodology Annex 4	3+ Stories (or >12m)	Yes	
	Building is a Category B building in scope	Yes	
HRD Review If applicable	HRDs Identified	A-01	Columns with exceptionally high axial load Grid X52 column
		E-01	Storey-sway potential, with vulnerable columns Shear-controlled columns on grids X and Y
			List as appropriate...
	Complete or partial HRD Review¹	Complete (all potential HRDs reviewed)	
	Outcome (action)	Building remains EPB Targeted retrofit to address HRDs	

Notes:

- [1] Partial HRD review may mean that whilst at least one HRD has been identified, not all potential HRDs have been reviewed. Buildings with partial HRD reviews must be assumed to be EPBs. Full reviews are recommended for EPBs to clearly define the mitigation requirement.

A1.10.2 Checking if a full HRD review is required

General exclusions based on the Column-Wall Index (CWI) and the check for potential diaphragm vulnerability are listed below. The torsional strength index (and torsional displacement index) are also included.

Table 8: Example Only - Area-based indices at critical level, and general exclusions

Parameter	X Direction	Y Direction
Column Index, CI	0.00%	0.00%
Wall Index, WI	0.22%	0.21%
Column-Wall Index, CWI (CI + WI)	0.23%	0.21%
Complete steel frame (Yes/No)	No	
Torsional Strength Index, TSI (T_{DI} / T_{CI})	0.13	0.13
Torsional Displacement Index, TDI	1.13	1.18
Concentrated bracing elements (walls or braced frames) to building exterior?	Yes	Yes
CWI, computed excluding all wall elements located to the building exterior	0.17%	0.16%
Torsional Strength Index, computed excluding all wall elements located to the building exterior	0.34	0.23
Significant elevated transfer diaphragm?	No	
Potential diaphragm collector vulnerability?	No	
Building requires HRD review?	Yes	

A1.10.3 High-Risk Deficiency Review Checklist

Table 9: Example Only - Component-only deficiencies

Component Deficiency					Severe Physical Consequence				HRD Identified?	
Component deficiency	Criteria		Y/N	Comment	Criteria	Value	Y/N	Comment	Y/N	HRD ID
Columns with exceptionally high axial load	$\frac{N_G}{A_g f'_c} > 0.4$		☒	Example only	Loss of support of significant area of heavy floor		☒	Example only	☒	A-01
Vulnerable and non-redundant floor-wall (or braced frame) connection	At significant transfer diaphragm(s)	Significant transfer diaphragm And, Non-ductile floor reinforcing at critical section, or low connectivity (<50% L _w) And, Absence of integral (continuously reinforced) collector beam	☐ ☐ ☐	Example only	Column wall index CWI excluding all such walls ² ≤ 0.15% Or Torsional strength index TSI excluding any single such bracing element or wall assemblage ² ≥ 0.8	State value (%) or “not computed” State TSI or “not computed”	☐		☐	A-02
	At general Floors	Exterior walls or side core And, Non-ductile floor reinforcing at critical section, or low connectivity to floor slab (<50% L _w for wall segments, <50% p _{wall} for side-core) And, Absence of integral (continuously reinforced) collector beam sufficiently restrained to floor plate	☒ ☐ ☐		Column wall index CWI excluding exterior walls ² ≤ 0.15% Or Torsional strength index TSI excluding any single exterior bracing element or wall assemblage ² ≥ 0.8	State value (%) or “not computed” State TSI or “not computed”	☐ ☐			
Slab-column joint(s) with high shear demand	Where joints contribute significant lateral resistance ¹	For any joints, where $V_g/V_p > 0.4$	☐		Where joints contribute significant lateral resistance ¹		☐		☐	A-03
		Absence of continuous bottom reinforcement in each direction meeting: $A_{s,bottom} > 0.6 V_g/f_y$ And, $V_g/V_p > 0.2$	☐ ☐							
	Any slab-column joint	Absence of continuous bottom reinforcement in each direction meeting: $A_{s,bottom} > 0.6 V_g/f_y$ And, $V_g/V_p > 0.4$	☐ ☐		Loss of support of significant area of heavy floor		☐			
URM Elements contributing significantly to vertical or lateral resistance		URM Elements present as part of primary structure	☐		Sum contribution to CWI > 40% Or Loss of support of significant area of heavy floor		☐ ☐		☐	A-04

Notes:

[1] Where the sum CI of columns with slab-column joints contribute > 40% of the building’s Column Wall Index (CWI).

[2] Unless those bracing element(s) or wall(s)/wall assemblage(s) can engage direct compressive load path in direction under consideration.

Table 10: Example Only - Configurational vulnerabilities

Configurational Deficiency	Criteria		Value		Criteria Satisfied?		Comment	Outcome	
			X	Y					
Overall Weak LFRS	Column Wall Index $CWI \leq 0.15\%$		0.23%	0.21%	☐	☐ B_	Example only	If any configurational vulnerability has been identified, Go to Table 11	
Severe Torsional Sensitivity	Torsional strength index, $TSI T_{Di}/T_{Ci}>0.4$ (in a storey with $CWI \leq 0.30\%$)		0.13	0.13	☐	☐ C_	Example only		
	And Torsional displacement index ¹ , $TDI>1.4$ (at vulnerable component) or for walls, where the majority of lateral and torsional resistance is provided by a single core.		1.13	1.18	☐		Example only		
Weak storey	Storey strength index $< 90\%$ of storey over ²		$\geq 100\%$	$\geq 100\%$	☐	☐ D_	Example only		
	And Column Wall Index $CWI \leq 0.30\%$ in the storey (and direction) under consideration		0.23%	0.21%	☒		Example only		
Storey-sway potential	Either of the following identified	Identified based on comparison of storey-sway and beam/mixed sway mechanism strengths; or,	☒	☐	☒	☒ E_	Example only		
	(in conjunction with low CWI, as defined below)	Identified based on the sway index method for frames	☐	☐					
	Column Wall Index $CWI \leq 0.30\%$ in the storey (and direction) under consideration ⁵ .		0.23%	0.21%	☒				
Slender vertical LFRS with poor founding conditions	One of the following structural types that are vulnerable to differential settlement or tilting effects (in conjunction with poor founding conditions, as defined below)	Buildings with isolated shallow foundations under frame columns ³	State H/B value (or n/a)		☐	☐	☐ F_		
		And Overall aspect ratio ⁴ $H/B > 1.5$			☐				
		More than 60% lateral resistance (based on CWI) is from isolated bracing elements (cantilevered wall, core or braced frame).	State H/B value (or n/a)		☐				
		And Overall aspect ratio ⁴ of lateral system is $H/B > 2.0$			☐				
		And Potentially vulnerable frame components are founded on separate footings	State H/B value (or n/a)		☐				
					☐				
	Poor founding conditions should be assumed for NZS 1170.5:2004 site subsoil class is C, D or E, but does <u>not</u> include situations where the lateral system piles extend to found on rock. Poor founding conditions need not apply where specific foundation assessment information available showing poor founding conditions are not likely to exist (note source in comments).		NZS 1170.5:2004 Site Class C		☒	☒	Example only		
			Specific assessment?		☐				
Discontinuous wall or braced frame	Discontinuous wall or braced frame (horizontal offset or vertical discontinuity irregularity): A wall or braced frame that is not continuous to ground, and is supported by a column or pier, wall overlap, or transfer beam.				☐	☐ G_			

Notes:

[1] Or torsional sensitivity, g computed in accordance with NZS 1170.5:2004 Cl. 4.5.2.3 (state in comments if this approach is used).

[2] Or assessed using provisions of NZS 1170.5:2004 for vertical stiffness irregularity and discontinuity in capacity – weak storey (Cl. 4.5.1.2 and 4.5.1.3 respectively). State in comments if this approach is used.

[3] Applies only to frames where foundations are not connected by ground beams of equal or greater section proportions (A_g) to the first storey frame section dimensions.[4] For a shallow foundation, B represents the width of the footing. For a piled foundation (excludes those with rock sockets), B is the centreline distance between outermost piles engaged by the wall or braced frame foundation.

[5] Storey-sway potential need only be considered in the lower half of regular buildings (or where there is a significant vertical discontinuity or setback, the lower half of the frame above that point).

Table 11: Example Only - Compounding component and configurational deficiencies (columns, joints and wall piers)

Component Deficiency								Severe Physical Consequence			HRD Identified ?
Component Deficiency	Criteria		Y/N	Comment	Component Vulnerability			Criteria	Y/N	Comment	
					Vuln. to Axial (Gravity) Failure	Vuln. to Loss in Lateral strength	Affected by config. vulnerability?				
Vulnerable column(s) (For columns supporting discontinuous walls, use N* _c instead of N _G)	Shear-limited(s): V _y /V _{pCol0} ≥0.7	Any shear-limited column	☒	Example only		☒	☐ B_ Overall weak LFRS ☐ C_ Severe tors. sens. ☐ D_ Weak storey ☒ E_ Storey-sway potential ☐ F_ Slender LFRS w/ poor founding ☐ G_ Disc. Wall/Braced Frame (in conjunction with B, D or E)	Loss of support of significant area of heavy floor Or Loss of significant proportion of lateral building stability	☒	Example only	☐ B-01 ☐ C-01 ☐ D-01 ☒ E-01 ☐ F-01 ☐ G-02
		$\frac{N_G}{A_g \alpha_{V,col} \rho_t f_{yt}} > 5$	☒	Example only		☒					
	Any column	$\frac{N_G}{A_g f'_c} > 0.2$ And, lightly confined, meaning either: Tie spacing, s > h/2, or ρ _t < 0.2%, or A _g /A _c > 1.5	☐ ☐ ☐	☒		☐					
Vulnerable beam-column joint(s)	Ext. joints vulnerable to shear failure	Hoop spacing > h _c / 2	☐			☐	☐ B_ Overall weak LFRS ☐ C_ Severe tors. sens. ☐ D_ Weak storey ☐ E_ Storey-sway potential ☐ F_ Slender LFRS w/ poor founding	Loss of support of significant area of heavy floor Or Loss of significant proportion of lateral building stability	☐		☐ B-02 ☐ C-02 ☐ D-02 ☐ E-02 ☐ F-02
		Joint aspect ratio h _b / h _c >1 and poor hook anchorage ¹	☐	☒		☐					
	Ext. joints vulnerable to axial failure	Vulnerable to shear failure (as above), also having both $\frac{N_G}{A_j f'_c} > 0.3$ And lightly confined ¹ .	☐ ☐ ☐	☒		☐					
Vulnerable slab-column joints	Absence of continuous bottom reinforcing in each direction meeting $A_{s,bottom} > 0.6 V_g / f_y$ And V _g / V _p > 0.2		☐ ☐	☒		☐	☐ B_ Overall weak LFRS ☐ C_ Severe tors. sens. ☐ D_ Weak storey ☐ E_ Storey-sway potential ☐ F_ Slender LFRS w/ poor founding	Loss of support of significant area of heavy floor	☐		☐ B-03 ☐ C-03 ☐ D-03 ☐ E-03 ☐ F-03
Wall piers vulnerable to axial failure	Satisfy wall piers definition And Aspect ratio H/L _w >1.5 And Axial load N _G / A _g f' _c > 0.05		☐ ☐ ☐	☒		☐	☐ B_ Overall weak LFRS ☐ C_ Severe tors. sens. ☐ D_ Weak storey ☐ E_ Storey-sway potential ☐ G_ Disc. Wall/Braced Frame (with B, C or D)	Loss of support of significant area of heavy floor Or Loss of lateral building stability	☐ ☐		☐ B-04 ☐ C-04 ☐ D-04 ☐ E-04 ☐ G-02

Notes:
[1] Refer to Annex 1 for definition of *poor hook anchorage* and *lightly confined* (for beam column joints).

Table 11 (cont’d): Example Only - Compounding component and configurational deficiencies (elements supporting discontinuous walls or braced frames)

Component Deficiency							Severe Physical Consequence			HRD Identified?	
Component Deficiency	Criteria		Y/N	Comment	Component Vulnerability			Criteria	Y/N		Comment
					Vuln. to Axial (Gravity) Failure	Vuln. to Loss in Lateral strength (of wall over)	Affected by config. vulnerability?				
Elements supporting discontinuous walls or braced frames¹ Vulnerability to axial failure from excessive axial demand	Compressive actions in columns, piers or wall overlaps	For any column: $\frac{N_c *}{A_g f'_c} > 0.7$	☐		☐		☐ G_ Disc. Wall/Braced Frame	Loss of support of significant area of heavy floor Or Loss of significant proportion of lateral building stability	☐		
		For columns that are lightly confined, and for any wall pier or wall overlap: $\frac{N_c *}{A_g f'_c} > 0.5$ <i>(use A_{overlap} in place of A_g for wall overlaps)</i>	☐ ☐	☐							
	Tensile actions in columns, piers or wall overlaps	For tensile actions in columns, piers or wall overlaps: $N_t * > A_s f_{splice}$	☐	☐		☐					
		Unless certain conditions met ² (tick if met and add comment)	☐								
	Beams supporting walls (excl. grade beams)	Unable to develop wall or braced frame strength <i>(or not checked)</i>	☐		☐						

Notes:
[1] N_c^* and N_t^* include earthquake actions acting in conjunction with permanent (gravity) actions. See Annex 1 for details.
[2] For columns only, the tensile criteria can be ignored when all of the following apply: deformed bar reinforcing is used, the section tension capacity is not limited by reinforcing splicing or anchorage, stirrup ties are adequately anchored into the core, and all longitudinal reinforcing bars are effectively enclosed by stirrup ties or hoops with spacing $s \leq 10d_b$. See Annex 1 for details.

Table 11 (cont'd): Example Only - Compounding component and configurational deficiencies (walls)

Component Deficiency							Severe Physical Consequence			HRD Identified?	
Component Deficiency	Criteria		Y/N	Comment	Component Vulnerability			Criteria	Y/N		Comment
					Vuln. to Axial (Gravity) Failure	Vuln. to Loss in Lateral strength	Affected by config. vulnerability?				
Walls vulnerable to axial failure	Susceptible to axial ‘through-the-thickness’ failure:	$\frac{(A_s - A'_s)f_y + N}{t_w l_w f'_c} > \lim.$ Where <i>lim.</i> Is 0.30 for walls with regularly spaced tie reinforcing over the extent of the compression zone with spacing $s/t_w > 1$, and 0.08 in other cases ³ .	☒	Example only	☐		☐ B_ Overall weak LFRS ☐ C_ Severe tors. sens.	Loss of support of significant area of heavy floor Or Loss of significant proportion of lateral building stability	☒	Example only	☐ B-05 ☐ C-05
	Multi-planar wall susceptible to axial instability failure	Compression zone comprises potentially unstable flange or boundary elements ¹ And Wall segments resisting shear are susceptible to sliding shear ²	☐ ☐		☐			☐			
	Walls acting out-of-plane	Req’d to act out-of-plane for building stability and without regular cross ties satisfying $s \leq t_w/2$	☐		☐			Loss of significant proportion of lateral building stability	☐		
Walls vulnerable to significant lateral strength loss (high seismic zone only)	☐ Slender walls $L_c/L_w > 2$	Singly reinforced walls	☐		☐	☐	☐ B_ Overall weak LFRS ☐ C_ Severe tors. sens.	Loss of significant proportion of lateral building stability	☐		☐ B-06 ☐ C-06
Lightly vertically reinforced (deformed bar and susceptible to concentration of deformation in a single crack)		☐	☐		☐						
Lightly horizontally reinforced $\rho_h \leq 0.25\%$		☐	☐		☐						
Walls with lack of anchorage in PPHZ ⁴		☐	☐		☐						

Notes:

- [1] Unstable flange or boundary elements are walls that are either singly reinforced, or without regularly spaced cross ties with spacing $s / t \leq 1.0$. See Annex 1 for details.
- [2] Walls susceptible to sliding shear are those where the wall section overall is susceptible to concentration of deformation in a single crack, and where the web segment (segment resisting shear) is entirely in tension from section analysis at M_p . See Annex 1 for details.
- [3] This criterion applies only to walls with compression actions under gravity $N_G > 0.01 A_g f'_c$. N^* includes earthquake actions acting in conjunction with permanent (gravity) actions. See Annex 1 for details including how to apply for planar, non-planar, and coupled walls (assemblages of walls).
- [4] Applies to grossly deficiency lap splices, or insufficient anchorage of wall reinforcing into a foundation or supporting element. See Annex 1 for details.

Table 12: Example Only - High-risk configurational deficiencies (exceptionally weak frames susceptible to P-Delta instability)

Configurational Deficiency									Severe Physical Consequence	HRD Identified?	
	Susceptibility to P-Delta					Configurational Vulnerability					
	Criteria		Value	Y/N	Comment	Criteria (See Table 10)		Comment			
Exceptionally weak frames susceptible to P-Delta instability	Either of the following in storeys affected by weak storey or storey-sway potential <i>(in the direction under consideration)</i>	Exceptionally weak frames:	X Dir: _	☐	☐	Examples only	Frames having either of the following: (in the direction under consideration)	☐ D_ Weak storey ☒ E_ Storey-sway potential	Example only	A severe physical consequence (lateral sway failure) is deemed to exist if this configuration is identified.	☐ H-01
		<i>Column wall index</i> ≤ 0.08%	Y Dir: _	☐							
		Severe torsional sensitivity:	X Dir: _	☐							
		<i>Torsional strength index</i> T _{DI} /T _{CI} > 0.8	Y Dir: _	☐							