Part C - Detailed Seismic Assessment

Part C2 – Assessment Procedures and Analysis Techniques

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Contents

C2. Assessment Procedures and Analysis Techniques

C2.1 General

C2.1.1 Scope and outline of this section

This section sets out the elastic and nonlinear assessment procedures that can be used in the Detailed Seismic Assessment (DSA) Steps 4 to 11 (outlined in Section C1). It includes recommendations for selecting the most appropriate procedures and associated analysis techniques, details of these, and guidance on specific issues.

The procedures presented include a first principles, mechanism-based method based on either a force or displacement-based approach. A significant change from the previous edition of these guidelines (NZSEE, 2006) is the emphasis on understanding the nonlinear behaviour of the structural systems present, even when elastic-based procedures are being used.

For this reason, these guidelines recommend using the Simple Lateral Mechanism Analysis (SLaMA) procedure as a first step in any assessment. While SLaMA is essentially an analysis technique, it enables engineers to investigate (and present in a simple form) the potential contribution and interaction of a number of structural elements and their likely effect on the building's global capacity.

In some cases, the results of a SLaMA will only be indicative. However, it is expected that its use should help engineers achieve a more reliable outcome than if they only carried out a detailed analysis, especially if that analysis is limited to the elastic range. The objective is not to rely on sophisticated techniques without first developing an understanding of how the building resists seismic loads and identifying the various critical load paths and how the various systems might interact.

Note:

The previous edition (NZSEE, 2006) presented both force-based and displacement-based assessment approaches.

The force-based assessment was based on a first principles approach developed by Priestley and Calvi, 1991 and by Park, 1996 for reinforced concrete frames. This was developed further by Priestley in 1996 but in the form of a displacement-based approach (Priestley, 1996) and has now been significantly expanded to clarify the procedures and consider some practical implementation issues (Kam et al. 2013).

It has been observed that the traditional force-based assessment approach is often misused by engineers who attempt to link it too closely to the design process adopted for a new building. This has been neither straightforward nor successful, as current capacity design procedures are deterministic in nature and rarely achievable in older buildings.

While these guidelines emphasise the understanding of the nonlinear behaviour of structures and the governing inelastic/collapse mechanism, it is acknowledged that most designers are currently more familiar with force-based procedures. Therefore, these procedures have been retained but with additional requirements.

These guidelines shift the assessment focus away from an overreliance on detailed analysis and, instead, encourage this as just one way to better understand the way in which the building resists seismic loads.

These guidelines also stress that structural analysis is just one part of the assessment procedure and that identifying potential modes of behaviour (mechanisms), potential structural weaknesses (SWs) and severe structural weaknesses (SSWs) is crucial. To do this satisfactorily, even for relatively simple buildings, requires judgement and experience so that the analysis techniques chosen allow the engineer to appropriately investigate and quantify the relevant issues.

Note:

It is apparent that many previous DSAs have been approached using a detailed analysis as the principal – and often only – part of the assessment procedure. This approach has largely been driven by the adherence to conventional structural design processes, which rely on an elastic analysis as the principal means of deriving internal actions for member sizing. This is rarely appropriate for seismic assessment.

This section provides guidance on the:

- assessment procedures supported by these guidelines and issues that might arise in following them (Sections C2.2 to C2.5), and
- analysis techniques that can be adopted (Sections C2.6 to C2.9).

Note:

A significant amount of material on specific subjects has been moved to the appendices to improve flow and readability. Much of this material, when relevant, is essential to these guidelines and forms an integral part of them.

C2.1.2 Definitions and acronyms

ADRS	Acceleration-displacement response spectrum (spectra)
Brittle	A brittle material or structure is one that fractures or breaks suddenly once its probable strength capacity has been reached. A brittle structure has little tendency to deform before it fractures.
Critical structural weakness (CSW)	The lowest scoring structural weakness determined from a DSA. For an ISA all structural weaknesses are considered to be potential critical structural weaknesses.
Damping	The value of equivalent viscous damping corresponding to the energy dissipated by the structure, or its systems and elements, during the earthquake. It is generally used in nonlinear assessment procedures. For elastic procedures, a constant 5% damping as per NZS 1170.5:2004 is used.
DBA	Displacement-based assessment
DDBD	Direct displacement-based design
Design level/ULS earthquake (shaking)	Design level earthquake shaking or loading is taken to be the seismic load level corresponding to the ULS seismic load for the building at the site as defined by NZS 1170.5:2004 (refer to Section C3)
Detailed Seismic Assessment (DSA)	A quantitative seismic assessment carried out in accordance with Part C of these guidelines
Diaphragm	A horizontal structural element (usually a suspended floor or ceiling or a braced roof structure) that is connected to the vertical elements around it and that distributes earthquake lateral forces to vertical elements, such as walls, of the primary lateral system. Diaphragms can be classified as flexible or rigid.
Ductile/ductility	Describes the ability of a structure to sustain its load carrying capacity and dissipate energy when it is subjected to cyclic inelastic displacements during an earthquake
Elastic analysis	Structural analysis technique that relies on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material nonlinearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis.
Equivalent static analysis (ESA)	Equivalent static analysis as prescribed in NZS 1170.5
Flexible diaphragm	A diaphragm which for practical purposes is considered so flexible that it is unable to transfer the earthquake loads to shear walls even if the floors/roof are well connected to the walls. Floors and roofs constructed of timber, and/or steel bracing in a URM building, or precast concrete without reinforced concrete topping fall in this category. A diaphragm with a maximum horizontal deformation along its length that is greater than or equal to twice the average inter-storey drift. In a URM building a diaphragm constructed of timber and/or steel bracing.
Initial Seismic Assessment (ISA)	A seismic assessment carried out in accordance with Part B of these guidelines. An ISA is a recommended first qualitative step in the overall assessment process.
Irregular building	A building that has an irregularity that could potentially affect the way in which it responds to earthquake shaking. A building that has a sudden change in its plan shape is considered to have a horizontal irregularity. A building that changes shape up its height (such as one with setbacks or overhangs) or that is missing significant load-bearing elements is considered to have a vertical irregularity. Structural irregularity is as defined in NZS 1170.5:2004.

MRSA	Modal response spectrum analysis
NLSPA	Nonlinear static pushover analysis
Nonlinear analysis	Structural analysis technique that incorporates the material nonlinearity (strength, stiffness and hysteretic behaviour) as part of the analysis. Includes nonlinear static (pushover) analysis and nonlinear time history dynamic analysis.
Nonlinear time history analysis (NLTHA)	An analysis of the building using strong motion earthquake records and modelling the nonlinear behaviour of the structure (also referred to as nonlinear response history analysis)
Non-structural element	An element within the building that is not considered to be part of either the primary or secondary structure
PGA	Peak ground acceleration
Primary gravity structure	Portion of the main building structural system identified as carrying the gravity loads through to the ground. Also required to carry vertical earthquake induced accelerations through to the ground. May also be part of the primary lateral structure.
Primary lateral structure	Portion of the main building structural system identified as carrying the lateral seismic loads through to the ground. May also be part of the primary gravity structure
Probable capacity	The expected or estimated mean capacity (strength and deformation) of a member, an element, a structure as a whole, or foundation soils. For structural aspects this is determined using probable material strengths. For geotechnical issues the probable resistance is typically taken as the ultimate geotechnical resistance/strength that would be assumed for design.
pseudo-Equivalent static analysis (pESA)	Loading for rigid diaphragm assessment. Refer to Section C2.9.3 and the broader definition in NZS 1170.5:2004.
Rigid diaphragm	A diaphragm that is not a flexible diaphragm
Secondary structure	Portion of the structure that is not part of either the primary lateral or primary gravity structure but, nevertheless, is required to transfer inertial and vertical loads for which assessment/design by a structural engineer would be expected. Includes precast panels, curtain wall framing systems, stairs and supports to significant building services items
Serviceability limit state (SLS)	Limit state as defined in AS/NZS 1170.0:2002 (or NZS 4203:1992) being the point at which the structure can no longer be used as originally intended without repair
Severe structural weakness (SSW)	A defined structural weakness that is potentially associated with catastrophic collapse and for which the capacity may not be reliably assessed based on current knowledge
SFSI	Soil-structure-foundation-interaction (refer to Section C2.9.2)
Simple Lateral Mechanism Analysis (SLaMA)	An analysis involving the combination of simple strength to deformation representations of identified mechanisms to determine the strength to deformation (push-over) relationship for the building as a whole
Single-degree-of-freedom (SDOF)	A simple inverted pendulum system with a single mass
Soil-structure-foundation-interaction (SFSI)	Interaction between the structure of the building and the foundation and soil surrounding it. Synonymous with SSI
SRSS	Square root sums of squares method of combining variables
SSI	Interaction between the structure (including foundations) and the soil surrounding the foundation

Structural element	Combinations of structural members that can be considered to work together; e.g. the piers and spandrels in a penetrated wall, or beams and columns in a moment resisting frame
Structural member	Individual items of a building structure, e.g. beams, columns, beam/column joints, walls, spandrels, piers
Structural sub-system	Combination of structural elements that form a recognisable means of lateral or gravity load support for a portion of the building: e.g. moment resisting frame, frame/wall. The combination of all of the sub-systems creates the structural system.
Structural system	Combinations of structural elements that form a recognisable means of lateral or gravity load support; e.g. moment resisting frame, frame/wall. Also used to describe the way in which support/restraint is provided by the foundation soils.
Structural weakness (SW)	An aspect of the building structure and/or the foundation soils that scores less than 100*%NBS.* Note that an aspect of the building structure scoring less than 100*%NBS* but greater than or equal to 67*%NBS* is still considered to be a structural weakness even though it is considered to represent an acceptable risk.
Ultimate capacity (seismic)	A term defined in regulations that describes the limiting capacity of a building for it to be determined to be an earthquake-prone building. This is typically taken as the probable capacity but with the additional requirement that exceeding the probable capacity must be associated with the loss of gravity support (i.e. creates a significant life safety hazard).
Ultimate limit state (ULS)	A limit state defined in the New Zealand loadings standard NZS 1170.5:2004 for the design of new buildings
Unreinforced masonry (URM)	A member or element comprising masonry units connected together with mortar and not containing any steel, timber, cane or other reinforcement
(XXX)*%NBS*	The ratio of the ultimate capacity of a building as a whole or of an individual member/element and the ULS shaking demand for a similar new building on the same site, expressed as a percentage. Intended to reflect the expected seismic performance of a building relative to the minimum life safety standard required for a similar new building on the same site by Clause B1 of the New Zealand Building Code.
(XXX)%ULS shaking (demand)	Percentage of the ULS shaking demand (loading or displacement) defined for the ULS design of a new building and/or its members/elements for the same site. For general assessments 100%ULS shaking demand for the structure is defined in the version of NZS 1170.5 (version current at the time of the assessment) and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016. For engineering assessments undertaken in accordance with the EPB methodology, 100%ULS shaking demand for the structure is defined in NZS 1170.5:2004 and for the foundation soils in NZGS/MBIE Module 1 of the Geotechnical Earthquake Engineering Practice series dated March 2016 (with appropriate adjustments to reflect the required use of NZS 1170.5:2004). Refer also to Section C3.

C2.1.3 Notation, symbols and abbreviations

Symbol	Meaning
%*NBS*	Percentage of new building standard as assessed by application of these guidelines
$ADRS_{\mathrm{ULS}}$	Refer to Figure C2.7
$A_{\mathrm{c}}/A_{\mathrm{g}}$	Low core-to-gross concrete area - refer to Section C2G.2
$A_{\mathrm{g}}f'_{\mathrm{c}}$	Refer to Section C2G.2 & C2G.3
A_{χ}	Torsional amplification multiplier
C	Refer to Table C2A.1
$C(T_1)$	Ordinate of elastic site hazard spectrum for first mode period T_1 and relevant site subsoil type as defined in NZS 1170.5:2004
C_{vx}	Vertical load distribution as defined in Equation C2.18
El	Refer to Figure C2A.1
$e_{\mathrm{stiffness}}$	Distance between centre of mass and centre of rigidity/stiffness
e_{strength}	Distance between centre of mass and centre of strength
e_{vx}	Strength eccentricity - refer to Figure C2F.2 and Section C2F.4
e_{vy}	Refer to Figure C2F.2
F	Generalised term for lateral force
f'_{c}	Refer to Figure C2A.1
F_{y}	Generalised term for yield force
g	Acceleration of gravity (9.81m/s^2)
G_{j}	Joint rotational springs – refer to Figure C2C.2
H	Refer to Table C2A.1
h_{eff}	Effective height of the equivalent SDOF system
$h_{\mathrm{eff,beam}\ sidesway}$	Refer to Table C2A.1
$h_{\mathrm{eff,}col\ sidesway}$	Refer to Table C2A.1
$h_{\mathrm{effmixed\ sidesway}}$	Refer to Table C2A.1
h_{i}	Height (in m) from the base to floor level i
h_{n}	Height from the base of the building to the top of the primary lateral structure
h_{sx}	Refer to Table C2A.1
h_{x}	Height (in m) from base to floor level x
h_{w}	Height of the walls – refer to Table C2A.1
k	Refer to Equation C2.16
k^*_{dm}	Modified drift modification factor, adapted from NZS 1170.5:2004 as defined in Section C2.5.10.2.
k_{eff}	Effective stiffness of the equivalent SDOF system

Symbol	Meaning
k_{s}	Refer to Figure C2A.3
k_{μ}	Inelastic spectrum scaling factor based on the achievable ductility of the building, μ_{sys} ,in accordance with NZS 1170.5:2004
L	Distance between the centroids of the lateral load resisting lines in the orthogonal direction
L_{w}	Length of wall
l_{w}	Refer to Figure C2F.2
l_{wt}	Refer to Figure C2F.2
Mb	Beam moment – refer to Figure C2A.1
M_{bl}, M_{br}	Beam expected maximum flexural strengths at the left and right of the joint, respectively, at the joint centroid
M_{c}	Column moment – refer to Figure C2A.1
M_{ca}, M_{cb}	Minimum expected column flexural strengths above and below the joint, respectively, at the centroid of the joint
m_{eff}	Effective mass of the equivalent SDOF system
m_{i}	Lumped mass at level i
m_{n}	Refer to Figure C2.5 a)
M_{wp}	Refer to Table C2A.1
n	Number of levels above the base of the primary lateral structure
N^{*}_{g}	Refer to Section C2G.3
N_{f}	Refer to Table C2A.1
n_{t}	Number of storeys
OTM	Refer to Table C2A.1
p'_{t}	Refer to Figure C2A.1
$P - delta$	The destabilising effects from a compression force on a column or building acting through the lateral end displacement of a column or building drift
$P/A_{g}f'_{c}$	High axial load demand - refer to Section C2G.2
Q_{u}	Dead and reduced live load – refer to Section C2G.3
r	Generalised post-yield stiffness
r_{b}	Beam plastic hinge – refer to Figure C2C.2
r_{c}	Column plastic hinge – refer to Figure C2C.2
r_{j}	Joint rotational springs – refer to Figure C2C.2
R_{v}	Refer to Section C2E.4
S	Separation between adjacent buildings at any level (for pounding assessment). Refer to Appendix C2B.
s	Spacing – refer to Section C2G.2
S_{a}	Pseudo spectral acceleration

Symbol	Meaning
S_{d}	Pseudo spectral displacement
S_{i}	Sway index
S_{p}	Structural performance factor associated with the detailing and assessed ductility of the system. Determined in accordance with NZS 1170.5:2004. Refer to Section C3.
$S_{\mathrm{p,nltha}}$	Refer to Section C2C.6
T	Refer to Table C2A.1
T_1	Fundamental period of the building which has the greatest mass participation
T_{eff}	Effective period of the equivalent SDOF structure
V	Total lateral seismic force. Refer to Figure C2.11 in C2.7.3, Figure C2A.4 and Table C2A.1
V_{b}	Refer to Table C2A.1
V_{base}	Horizontal seismic base shear
$(V_{\mathrm{base}})_{\mathrm{i}}$	Horizontal seismic base shear associated with sub-system i
v_{c}	Refer to Section C2G.4
V_{E}	Refer to Section C2E.5
V_{e}	Plastic base shear corresponding to M_{e}
V_{f}	Refer to Table C2A.1
$V_{\mathrm{o/s}}$	Horizontal seismic shear consistent with the overstrength capacity of the structure
V_{prob}	Probable horizontal seismic capacity of the global structural system
$\left(V_{\mathrm{prob}}\right)_{\mathrm{i}}$	Probable horizontal seismic base shear associated with sub-system i
v_{s}	Refer to Section C2G.4
V_{shear}	Refer to Table C2A.1
V_{u}	Refer to Figure C2.11 in C2.7.3 and Table C2A.1
V_{wp}	Refer to Figure C2F.2
V_{ℓ}	Refer to Figure C2.11 in C2.7.3
W_{t}	Total seismic weight of structure
w_{i}	Portion of total building weight W on floor level i
w_{x}	Portion of total building weight W on floor level x
α	Refer to Section C2.9.4 and Figure C2C.2
β	Refer to Figure C2C.2
Δ	Lateral displacement at centre of action of lateral seismic forces. Refer to Figure C2.11 in C2.7.3
Δ_{bX}	Refer to Table C2A.1
Δ_{cap}	Global system displacement capacity – refer to Section C2.5.11

Symbol	Meaning
$(\Delta_{\text{elastic}})_{\text{i}}$	Elastic displacement at level i $\leq (\Delta_y)_i$
Δ_{fy}	Refer to Figure C2A.3
Δ_{i}	Refer to Figure C2.5 a), Step D3 and Equation C2A.2
Δ_{plastic}	Plastic displacement (or rotation)
$(\Delta_{\text{plastic}})_{\text{i}}$	Plastic lateral displacement at level i
Δ_{prob}	Probable lateral displacement capacity of the global structural system
$(\Delta_{\text{prob}})_{\text{i}}$	Probable lateral displacement capacity of the global structural system at level i
$(\Delta_{\text{prob}})_{\text{top}}$	Probable lateral displacement capacity of the global structural system measured at the top of the primary lateral structure
Δ_{s}	Refer to Table C2A.1
Δ_{u}	Global roof displacement – refer to Figure C2.9 in C2.5.11 and Figure C2A.3
$\Delta_{\text{u,sys}}$	Refer to Figure C2.9 in C2.5.11
Δ_{ULS}	100%ULS demand deflection on the equivalent inelastic SDOF system (i.e. for T_{eff} , ξ_{sys})
ΔV_{t}	Shear force increase in the lateral load resisting members in the orthogonal direction
Δ_{wy}	Refer to Figure C2A.3
Δ_{X}	Refer to Table C2A.1
Δ_{y}	Yield displacement (or rotation)
$(\Delta_{\text{y}})_{\text{i}}$	Lateral yield displacement at level i
$\Delta_{\text{y,sys}}$	Refer to Figure C2.9 in C2.5.11
$(\Delta_{\text{y}})_{\text{top}}$	Refer to Section C2.3.1 Step F1 and Section C2.A.2 Step 5 & 6
δ_1	Estimated lateral deflection of Building 1 relative to ground under the loads used for the assessment
δ_2	Estimated lateral deflection of Building 2 relative to ground under two-thirds of the loads used in the assessment
δ_{average}	Refer to Equation C2.14 in C2.5.8
δ_{i}	Displacement profile for the primary structural system normalised to the top level displacement
δ_{max}	Refer to Equation C2.14 in C2.5.8
ε_{c}	Refer to Figure C2A.1
ε_{y}	Yield strain of steel reinforcement
θ	Refer to Table C2A.1
θ_{b}	Beam hinge rotation – refer to Figure C2A.2
θ_{c}	Column hinge rotation – refer to Figure C2A.2
θ_{j}	Joint rotation – refer to Figure C2A.2

Symbol	Meaning
θ_{p}	Plastic rotation at the base of a wall
μ	Structural ductility in accordance with NZS 1170.5:2004
μ_{f}	Ductility available from the frame. Refer to Figure C2A.3
μ_{p}	Ductility of part in accordance with NZS 1170.5:2004
μ_{sys}	Displacement ductility for the system at the level of demand considered
μ_{w}	Ductility available from the wall. Refer to Figure C2A.3
ξ_{hy}	Available hysteretic damping for the structural system
ξ_0	Available Inherent equivalent viscous damping
ξ_{d}	Added damping due to supplemental viscous dampers – refer to Equation C2D.1
ξ_i	Refer to Equation C2.11 in C2.4.2 Step D3 and Equation C2A.3 in C2A.2 Step 6
ξ_{sys}	Achievable equivalent viscous damping for the global structural system
$\sum_{\mathrm{i}} M_{\mathrm{coli}}$	Refer to Table C2A.1
$\sum_{\mathrm{n}} V_{\mathrm{end\ beam.n}}$	Refer to Table C2A.1
$\sum_{\mathrm{x}} V_{\mathrm{end\ beam.x}}$	Refer to Table C2A.1
$\sum V_{\mathrm{wpi}}$	Refer to Figure C2F.2
ϕ	Strength reduction factor
ϕ_{ob}	Overall building overstrength factor (for diaphragm assessment)
χ	Ratio of the maximum displacement at any point on the level x diaphragm to the average displacement $\chi = \delta_{\mathrm{max}}/\delta_{\mathrm{average}}$
ω_{v}	Dynamic magnification factor for shear demand

C2.2 Choosing an Assessment Procedure

C2.2.1 Key objectives of the DSA

The fundamental objective of a DSA should be to understand the structural load paths and likely behaviour of the building in earthquake shaking in sufficient detail to allow quantification of the behaviour and the earthquake rating. A good understanding of the governing nonlinear load path generally leads to a consistent assessment outcome.

The most suitable procedure will depend on the circumstances, and many buildings will not require – or justify – the use of lengthy and detailed analysis. However, the focus in all cases should be on determining the displacement of the structure and the governing inelastic lateral and loss-of-gravity support mechanisms during “severe” earthquakes. Internal actions generated, such as shear, moment and axial load, should be considered as consequences of this deformation, not the cause of it.

Note:

This is the essence of the displacement-based procedures covered in these guidelines. As noted earlier, force-based procedures are still included but engineers are expected to adopt displacement-based thinking when using them, especially when mixed mode systems are present.

These guidelines also recommend that the capacity of a building should be considered independently from the demands (imposed inertial loads and displacements) placed on it, bringing both together in the final step of the assessment.

The extent to which the structure is modelled and the length to which other analysis needs to be carried out requires careful thought. A qualitative overview of the structure (such as might be obtained by applying the ISA procedures) will help to identify structural weaknesses and/or particularly vulnerable elements.

Note:

Engineers should remind themselves that the objective of the earthquake-prone building legislation is to reduce seismic risk. It may be better for a relatively crude but effective strengthening measure to be carried out than to postpone strengthening work while the owner saves up to pay for an unnecessarily expensive analysis.

Sophisticated analyses should be complemented with various preliminary “throw-away” studies to gain insights into the complex structural response and its sensitivity to various input parameters.

More focus on the assessment of the loss of gravity load support and “brittle” inelastic mechanisms is also recommended, noting that both of these are challenging to model explicitly using commercially available analysis packages. It may be necessary to carry out a series of analyses using various techniques, in conjunction with post-processing assessment of the gravity load path, to confirm the capacity of the building.

Note:

Any analysis requires a level of judgement, experience and expertise, but this is especially so when using more sophisticated procedures. The dangers of relying on "black box" analysis, where the impact of the assumptions made or included within the analysis are not always obvious to the engineer, should be apparent.

The uncertainties, precision and reliability remain a function of the level of checking and rigour of the analysis (e.g. the number of runs, sensitivity analysis and well-defined analysis parameters). Even the most advanced analysis should always be "tested" using simpler models and rational methods.

C2.2.2 Determining a suitable approach

This section divides the assessment procedures into two categories, each with a corresponding selection of analysis techniques. These are:

- **Elastic-based assessment procedures** (force-based), which employ analysis techniques such as:
  - simplified nonlinear pushover analysis using SLaMA (an essential step)
  - equivalent static analysis (linear static)
  - modal response spectrum analysis (linear dynamic)
  - pseudo-nonlinear static pushover analysis

  Note:

  A SLaMA should be carried out as part of the elastic-based procedure because of the insights this is expected to bring into the expected seismic behaviour of the building, particularly as it goes into the nonlinear range. This is discussed below.

- **Nonlinear assessment procedures** (force-based and displacement-based), which employ analysis techniques such as:
  - simplified nonlinear pushover analysis using SLaMA
  - nonlinear static pushover analysis
  - nonlinear time history analysis (nonlinear dynamic).

These assessment procedures are considered applicable for all lateral force-resisting elements and materials. However, they may require some modification depending on the circumstances. The necessary adaptation for particular materials and structural forms is indicated in the specific material sections (Sections C5 to C9).

Note:

These guidelines distinguish between assessment procedures and the structural analysis techniques used within these procedures. While these concepts have previously been interchangeable, it has been recognised that too much focus on the analysis – particularly on computer-based modelling – can affect the validity of the DSA outcome. The emphasis is therefore on the assessment procedure as a whole, which uses detailed structural analysis as one of the techniques to gain a better understanding of the building's seismic behaviour.

SLaMA is recommended as a starting point whether the engineer selects an elastic or nonlinear based assessment procedure. It is also recommended that engineers use a range of analysis techniques to support their assessment as this is often necessary to address all of the issues. For example, the use of modal response spectrum analysis by itself may be of little use for mixed-ductility systems, but the linear dynamic analysis result can give some information about the higher mode behaviour and potential to affect the response.

Note:

While there are no specific restrictions on which assessment procedure an engineer can use, Section C2.6 outlines some limitations with the associated analysis techniques which may inform this choice.

Engineers should start simply and gradually increase the sophistication of their assessment and associated analyses as appropriate. In practice, engineers may adopt an iterative process of selecting the appropriate techniques as the assessment progresses, as implied by the DSA flowchart in Section C1.

Figure C2.1 summarises the common assessment procedures and serves as a reminder that they all involve simplifications and assumptions regarding earthquake shaking, building characteristics, analysis and the likely performance of structural elements.

This figure also illustrates that force-based and displacement-based assessments are two ways of looking at the same issue:

- In the linear/force-based approach, the behaviour of the various components is assessed by examining the forces in critical elements and using rules to assess the limits of integrity of the structural members.
- In the nonlinear/displacement-based approach, the response of the building structure is considered from the outset on the basis of the structural displacements resulting from the ground shaking. These are then used to examine the effect on the structural elements, again using rules to measure the limits of integrity and performance.

In relation to Figure C2.1:

- **Modelling of the earthquake shaking** – this will vary according to the assessment procedure and analysis techniques used. For example, the NZS 1170.5:2004 design hazard spectra may be used for force-based equivalent static or modal analysis but a site specific set of earthquake records (scaled as required by NZS 1170.5:2004) will be needed for nonlinear time history analyses. For displacement-based methods, displacement spectra can be generated using the NZS 1170.5:2004 pseudo acceleration spectra. Refer to Section C3 for further information.
- **Modelling of the structure –** numerous assumptions are necessary in relation to member properties and boundary conditions, and to the nonlinearity that should be considered and/or modelled.
- **Considering of geotechnical effects and soil-structure-interaction** – soil-structure interaction (SSI) is a key boundary condition for any structure analysis. Refer to Section C2.9.2 on SSI and to Section C4 for detailed guidance on consideration of geotechnical effects.

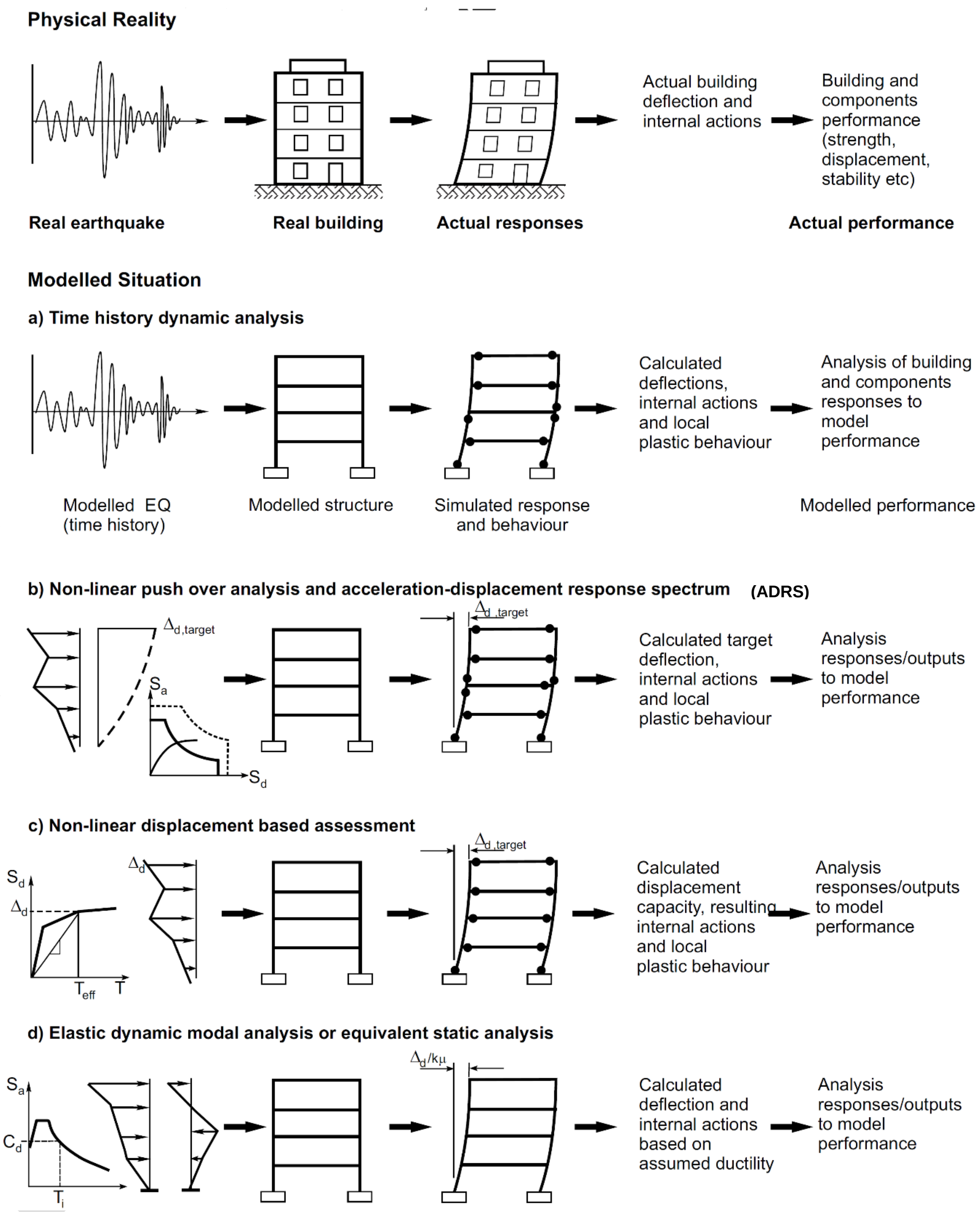


Figure C2.1: Real and modelled responses of buildings to earthquakes

- **Choice of analysis techniques** – this may depend on the building's structural configuration and level of complexity of the problem to be analysed. This choice will also drive the inputs and assumptions required, as well as the details of response derived (i.e. internal actions or deformations).
- **Modelling of the capacity of structural elements** – this process is significantly different from that used in the design of new buildings. For new buildings there are prescribed details (e.g. stirrup spacing) which will achieve the global ductility assumed. For existing buildings the ability of elements to deform plastically will depend on their detailing. A first principles approach to assess member ductility is likely to be required.

- **Comparison of demand and ultimate capacity –** irrespective of the analysis technique, a consistent measure of the probable capacity of the building should be derived. It is intended that this is always expressed in terms of the earthquake rating of the building (i.e. *%NBS*).

Note:

Figure C2.1 illustrates a range of options to characterise the ground shaking for the different assessment procedures and analysis techniques. Regardless of how the seismic demand is characterised, this should be correlated with the ultimate limit state (ULS) uniform risk design hazard spectra as given in NZS 1170.5:2004. The ULS hazard provides a benchmark for comparison of assessments using different procedures and analysis techniques.

Engineers should always be aware that all approaches – including the design hazard spectra, site-specific probabilistic uniform risk hazard spectra or deterministic scenario spectra – are predictors of long recurrence period events based on highly incomplete information.

These guidelines emphasise the need for the engineer to carry out appropriate quantitative analyses to form an understanding of the structural response and behaviour across a range of ground shaking levels rather than specifically for a single level of hazard.

C2.2.3 Treatment of uncertainties

Research in the past decade has improved the ability to identify and quantify the sources of uncertainties within the seismic assessment process. These uncertainties include:

- source seismicity and hazard – e.g. what earthquake magnitudes contribute to the hazard?
- ground motion attenuation/site effects – given an earthquake for a given tectonic mechanism at a specific distance and magnitude, what is the peak ground acceleration for a given site?
- structural responses analysis/modelling – given the ground shaking, what is the structural response in the linear and nonlinear range for a particular building?
- material properties and as-built conditions – what is the actual reinforcing detail or mortar strength?
- structural element nonlinear behaviour and capacities – given the structural response (drift or internal actions), what are the local mechanisms for the columns and beams?

While targeted in-depth or more advanced assessment (e.g. material testing, intrusive inspection, site-specific hazard analysis and nonlinear dynamic analysis) may reduce some of these uncertainties, the underlying uncertainties and the limitation of current analysis techniques remain and it is practically impossible to predict performance accurately. Conceptually, one can carry out multiple analyses with varying parameters and ground motions – i.e. the “Monte Carlo simulation” (FEMA P-58, 2012) – to get an idea of the average trends and behaviour, and this is assuming all mechanisms and failure modes can be modelled.

However, while this type of analysis may provide some insights into general trends relating to a building’s seismic performance to inform code development, it is not necessarily very meaningful for the assessment of individual buildings at this stage.

These guidelines promote the use of nonlinear assessment procedures with simple analysis techniques (e.g. SLaMA or nonlinear pushover) as the preferred assessment method, as this provides a balance between the uncertainties in the input parameters and the inherent uncertainties in these simpler forms of analysis.

In practice, uncertainties can be treated in a more pragmatic way in which reasonable upper and lower bounds of key variables are assessed as part of a sensitivity analysis to form a view of the range of expected seismic performance. For example, these could be the effects of:

- variation in critical material properties such as concrete compressive strength for unreinforced beam-column joint shear strength calculation
- achievable structural ductility (μ and S_{p})
- seismic loading and the associated input parameters such as site subsoil class.

The upper and lower bounds can give a range to the seismic assessment result (*%NBS*) which reflects the uncertainties in the inputs and assumptions. This also allows the engineer to determine which assumptions can be refined via further analysis, intrusive inspection, material testing, etc. in order to reduce uncertainties in the assessment result in the best manner. However, care and engineering judgement are needed so the engineer does not report a range of results that is no longer relevant to the client or end user.

Note:

The engineer should always bear in mind that the objective of assessment is not to predict the response of the building in a particular level of earthquake shaking. Rather it is to gain a better understanding of how the building might respond in earthquakes in general. Therefore, it is not considered critical to fully understand the degree of variance in material strengths (for example) other than when this might change the way in which the building will behave, i.e. affect the hierarchy of yield if this is being relied on for the assessment outcome.

C2.2.4 Role of SLaMA

As noted earlier, these guidelines recommend SLaMA as a starting point for any DSA. This is a simplified technique for determining the probable inelastic deformation mechanisms and their lateral strength and displacement capacity by examining load paths, the hierarchy of strength along critical load paths, the available ductility/displacement capacity of the identified mechanisms and the manner in which various mechanisms might work together.

Put another way, SLaMA is a method to determine the global nonlinear pushover capacity curve by summation of simple representations of the capacities of the individual members/elements/subsystems. It is expected that these summations are completed by hand.

SLaMA involves a degree of simplification and some assumptions of the structural response and capacity. For example:

- First mode response is dominant and higher mode amplification is negligible. For low-rise structures this will almost always be the case.

- The hierarchy of strength of interconnecting elements can be established by comparison of comparable “internal actions”; e.g. relative probable capacity in flexure and shear for beams.
- The governing mechanism at local element level can be extrapolated to global building behaviour by assuming either that load redistribution is possible (ductile response) or it is not (brittle global response). For example:
  - Beam flexural mechanism at different levels of a multi-storey frame building is assumed to be “ductile” locally such that sufficient load redistribution is possible to mobilise all beam flexural hinges to achieve a beam-sway global mechanism. Localised limitations on member behaviour (e.g. shear in a member) can then be superimposed based on deformation capacities if required.
  - Conversely, a weld failure in a diagonal bracing connection is a relatively brittle local mechanism. The analysis would either proceed assuming the diagonal bracing does not contribute to the lateral load resisting system beyond the deformation consistent with the connection failure, or the analysis would terminate with the connection failure being the governing global mechanism.
- A complex structural configuration can be simplified to stick models of individual bracing lines and ultimately to an equivalent single-degree-of-freedom (SDOF) oscillator.

Note:

Even for SLaMA it should not be forgotten that the focus is on assessing the capacity of the mechanism at the point a significant life safety hazard is created. This may not be at the point the capacity of the first member or element is reached.

In some ways, SLaMA is a kind of a reverse form of capacity design used for modern seismic design. By comparing the probable capacities, the hierarchy of strength and governing mechanism at each level can be determined for:

- individual members (e.g. beam, column or joint)
- interconnecting members forming structural elements (beam-column joints sub-assembly or perforated walls)
- interconnecting elements forming a system per bracing line (full height moment frame and foundation), and
- multi-systems together providing lateral resistance to the global building (summation of different systems).

This method is well developed for reinforced concrete structures (Priestley, 1996; Park, 1996; Priestley and Calvi, 1991; NZSEE, 2006 Appendix 4E.10; and Kam et al., 2013).

SLaMA’s main weakness is that the sequence of development of inelastic action between different members of the structure may not be identified. For structures with low member ductility capacity there may be a tendency to overestimate the load distribution and thus also the global strength and displacement capacity.

Figure C2.2 illustrates the SLaMA assessment pathway and gives an example for a moment resisting system. Appendix C2A sets out key steps for carrying out a SLaMA.

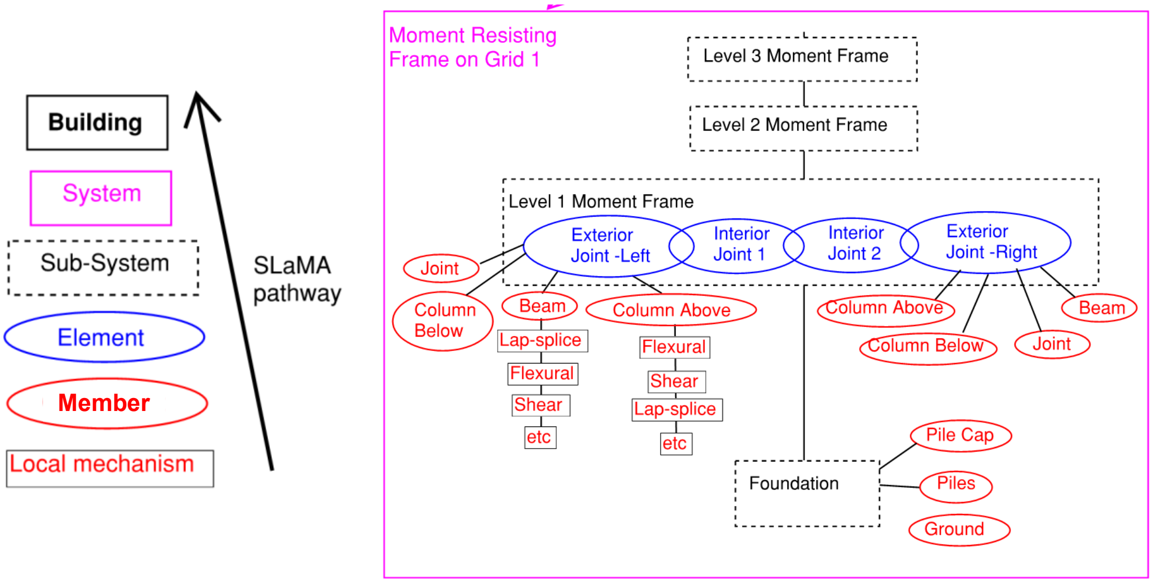


Figure C2.2: SLaMA assessment pathway and example for a moment resisting system

Note:

For many typical low to mid-rise buildings, the assessment can proceed solely on the basis of the SLaMA results as computer modelling is unlikely to provide any additional insights on the nonlinear behaviour of the building.

For more complex structures, SLaMA can inform the next level of analysis. For structures with brittle or no ductile capability, a force-based elastic analysis may suffice. Similarly, a force-based elastic analysis would suffice for structures in low seismicity regions with limited ductility demand on structural elements. SLaMA has identified the critical structural weakness and the inelastic mechanism of the critical elements/systems: therefore, a more appropriate structural ductility factor can be used within the force-based assessment.

Even if sophisticated nonlinear analyses are to be used, SLaMA provides information on the members/elements and nonlinearity that need to be modelled in detail and which mechanisms can likely be ignored in the nonlinear analysis.

C2.3 Assessment Procedure – Elastic

C2.3.1 Force-based procedure and key steps

The force-based assessment procedure outlined in these guidelines relies on the engineer having a good understanding of the probable inelastic mechanism of the structure and an indirect assessment of the local and global ductility capacities. Nonlinear behaviour is considered in the first step (SLaMA) and also indirectly in the analysis (e.g. reducing the stiffness of damaged/hinged elements).

The key steps of this procedure are modified from the recommendation of Park, 1996 and previous versions of these guidelines (NZSEE, 2006). The sequencing of, and interaction between, these steps is shown as a flowchart in Figure C2.3. More detail of each step follows.

Note:

The conventional force-based assessment procedure is based on the design for new building methodology in which the design engineer selects a structural system and a structural ductility and then calculates the required seismic base shear and internal actions for the lateral load resisting system. An elastic analysis with seismic loads (reduced for the assumed ductility) is then used to distribute the forces.

The problem with this approach is that often the structural ductility factor that is assumed will not be appropriate as the underlying assumptions will not be met. As a result, there can be an inadequate assessment of mixed ductility systems or of systems where a concentration of ductility demand occurs, e.g. presence of soft or weak storeys or potential for torsional twist, especially once the lateral system experiences nonlinear behaviour.

These guidelines recommend carrying out a SLaMA as part of the force-based assessment procedure to encourage direct determination of the ductility available. This can then be compared against the original assumption of what might have been considered possible.

The use of force-based procedures for the assessment of mixed ductility systems is problematical, just as it is for design. In such cases the engineer will not be able to avoid explicit consideration of deformation compatibility, particularly post yield, if undue conservatism is to be avoided.

Note:

Determination of global ductility from local ductilities needs to be undertaken with care. For a mixed ductility system, the global structural ductility capacity can be estimated by considering the probable yield displacement and probable deformation capacity of the various systems and limiting the assumed ductile capability to the system with the lowest deformation capacity (i.e. a displacement-based consideration is required even if a full displacement-based procedure is not followed). Refer also to Section C2.5.11.

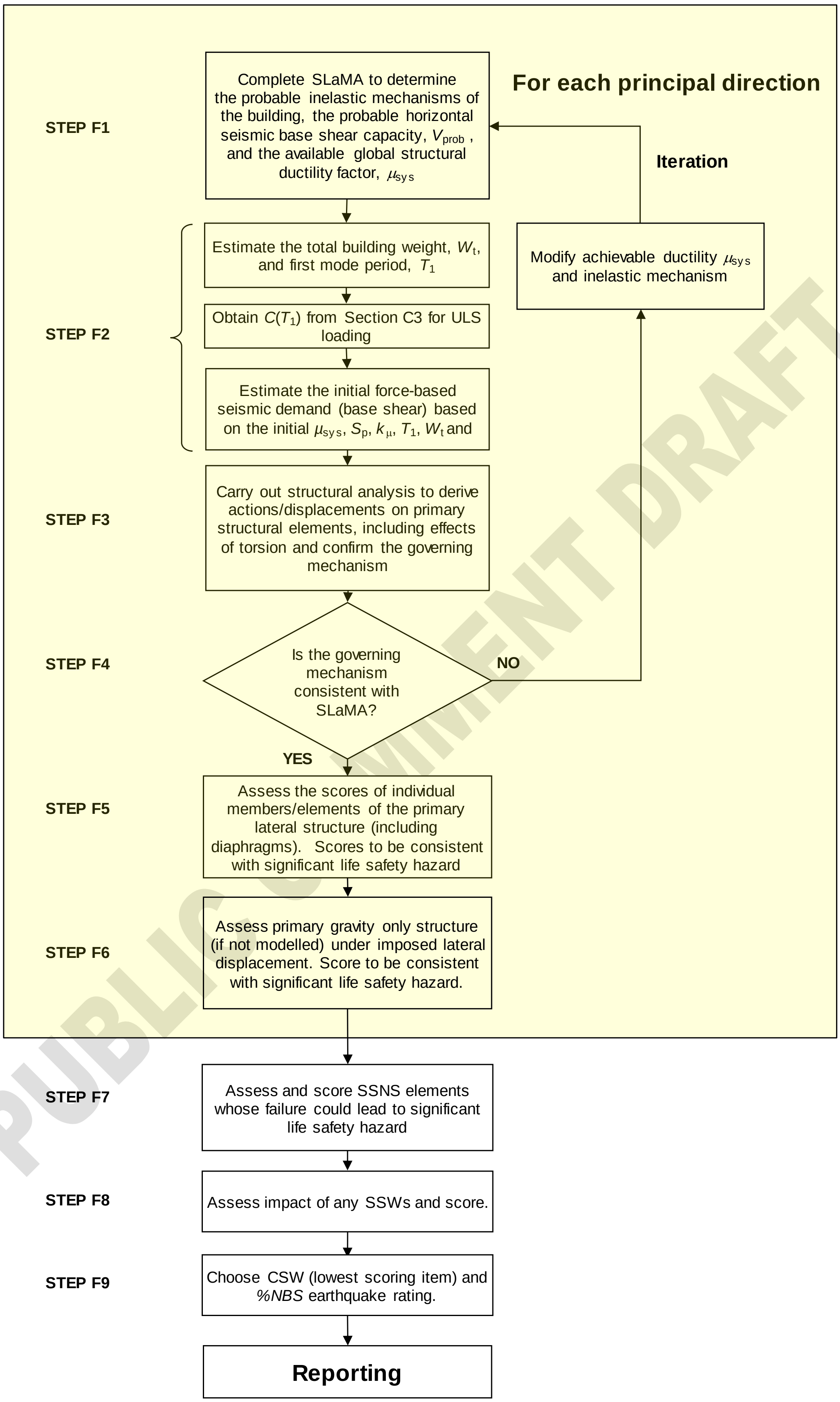


Figure C2.3: Summary of force-based assessment procedure

Step F1

Determine the inelastic sub-system mechanisms within the building that are likely to occur during seismic loading and from these calculate the probable horizontal seismic base shear capacity of the structure, V_{prob} , $(\Delta_{\text{prob}})_{\text{top}}$, and $(\Delta_{\text{y}})_{\text{top}}$ using the SLaMA technique outlined in Section C2.2.4 and Appendix C2A.

Note:

It is recommended that the SLaMA technique is extended through to calculating the global earthquake score, recognising that this may make the following steps redundant for a simple building.

This step will require the assessment of the member/element/system probable capacities and also the interaction between modes of behaviour, e.g. degrading of the ductile capacity of concrete members in the presence of shear. It provides a simplistic representation of the probable strength capacity of the primary lateral structure and also the probable lateral displacement capacity measured at the top of the primary lateral structure.

Estimate the global structural ductility factor, μ_{sys} as the ratio $(\Delta_{\text{prob}})_{\text{top}}/(\Delta_{\text{y}})_{\text{top}}$.

For multiple sub-systems, $(\Delta_{\text{y}})_{\text{top}}$ can be determined graphically as illustrated in Figure C2.7.

The global structural ductility capacity is generally governed by the dominant inelastic mechanism that contributes most to the plastic deformation of the system.

For structural systems with well distributed and defined plastic hinges of similar ductility capacity, the local structural ductility can be taken as the global ductility capacity. For example, if the local ductilities for a 1980s designed moment frame system with a ductile beam-sway mechanism is assessed as adequate for a displacement ductility of 4, the global ductility can also be taken as 4 by assuming sufficient moment redistribution can occur across the various levels.

Step F2

Estimate the fundamental period of vibration, T_1, and calculate the total seismic weight, W_{t}, of the structure.

Obtain the seismic coefficient $C(T_1)$ for ULS loading from Section C3.

Calculate the initial force-based ULS seismic demand (V_{base}) based on the estimated, μ_{sys}, and values for S_{p}, k_{μ}, T_1, W_{t} and $C(T_1)$.

$$V_{\text{base}} = \left(\frac{C(T_1)S_{\text{p}}W_{\text{t}}}{k_{\mu}}\right) \quad \text{...C2.1}$$

where:

$C(T_1)$	=	ordinate of the elastic site hazard spectrum for T_1 and for the site subsoil type, calculated in accordance with Section C3
W_t	=	total seismic weight of the structure
S_p	=	structural performance factor (a function of μ_{sys})
k_μ	=	initial inelastic spectrum scaling factor based on the achievable structural ductility of the building, μ_{sys}.

Step F3

Carry out appropriate structural analysis to determine the internal actions (demands) for the members/elements of the primary structural system (lateral and/or gravity) and the relationship between displacement/drift and lateral load. Either an equivalent static analysis or modal response spectrum analysis can be carried out, depending on the criteria set out in Section C2.6.

If appropriate, the elastic analysis model can be modified to reflect the actual nonlinear response of the building to avoid unnecessarily penalising the assessment. For example:

- The stiffness of members identified to respond nonlinearly can be modified to reflect the secant stiffness under the ULS loading.
- Members/elements considered not to present a significant life safety hazard can either be removed from the model or made to be "fully hinged".

Where appropriate, dynamic magnification factors due to higher mode and nonlinear behaviours should be included (refer to Section C2.5.10). Refer to Section C2.7.3 for guidance on the use of a pseudo-nonlinear analysis.

The effects of torsion should be included as outlined in Appendix C2F.

Calculate the lateral displacement and inter-storey drift demand of the structure under ULS loading. The displacement and drift output from the elastic analysis should be scaled appropriately in accordance with Section 6 of NZS 1170.5:2004.

Step F4

Review the structural behaviour based on the analysis results, and if the governing mechanism is not consistent with the results from SLaMA, review the SLaMA and/or the analysis and adjust the value of μ_{sys} as appropriate and repeat from Step F2.

Step F5

Assess the scores of the members/elements of the primary lateral system by dividing their probable strength capacity by the demands determined in Step F3. Determine the lowest scoring member/element that is consistent with the development of a significant life safety hazard.

If the failure of a member/element is unlikely to present a significant life safety hazard the assessment should be reiterated removing this member/element or reducing its capacity to its residual value until the appropriate lowest scoring member/element is identified.

Note:

It is important to realise that the failure or exceedance of the capacity-to-demand ratio of one structural member does not necessarily constitute global capacity exceedance. In order for the exceedance to be relevant in this context the failure of the element or member must create a significant life safety hazard as outlined in Part A.

Also score the calculated displacements/drifts against the design limits given for the ULS in NZS 1170.5:2004. Consider carefully if the design displacement/drift limits are appropriate for the particular building and, if not, adjust the score.

Note:

For relatively lightweight existing buildings (e.g. light timber buildings and light steel industrial buildings) the drift limit of 2.5% may be too severe a constraint when there is confidence that additional drift can be accommodated without compromising either the lateral or vertical load carrying capacity of the building. To recognise this, the limits prescribed in NZS 1170.5:2004 for this type of building may be exceeded in an assessment provided that it can be shown that the capacity of the building is not compromised.

The lowest score from consideration of the strength capacity of the primary structure and the deformation limits will provide the score for the global capacity of the primary lateral structure.

Step F6

Assess the primary gravity structure, if this has not been included in the assessment of the primary lateral structure, under the imposed lateral displacement and inter-storey drift demands determined in Step F4. The primary gravity structure must be able to accommodate the imposed deformations without exceeding its probable gravity load carrying capacity; i.e. it must be able to "go along for the ride".

Score the primary gravity structure.

If the score for the primary gravity structure is less than that for the primary lateral structure, then it will be the limiting score for the primary structure of the building.

Note:

Steps F5 and F6 are both required.

Step F7

Score any secondary structural systems or critical non-structural items in the building whose failure could lead to a significant life safety hazard. The demands on these elements/items should be typically determined in accordance with Section 8 of NZS 1170.5:2004.

Refer also to Part A and Section C10 for further guidance.

Step F8

Assess the impact of any severe structural weaknesses (SSWs) identified in the building and score these. Refer to Section C1.

Note:

SSWs are specific inelastic mechanisms that have the potential to result in disproportionate loss of gravity support and catastrophic collapse. Refer to Appendix C2G for guidance on assessing and scoring these.

Step F9

The lowest scoring item will be the critical structural weakness (CSW) and its score will become the *%NBS* earthquake rating for the building, in accordance with Section C1.

C2.4 Assessment Procedures – Nonlinear

C2.4.1 Available approaches

There are a number of ways to complete a nonlinear assessment procedure. The main ones outlined in these guidelines are:

- **Nonlinear static pushover procedures**
  - displacement-based assessment, extending the SLaMA (Priestley, 1996; Kam et al., 2013)
  - pseudo-nonlinear using iterative elastic analysis (also commonly known as elastic-plastic analysis for steel plastic design)
  - explicit nonlinear modelling
- **Nonlinear time history procedures.**

Nonlinear static pushover analysis (NLSPA, described in detail in Sections C2.4.3 and C2.8.2) is generally applicable for the assessment of low to medium rise regular buildings, where the response is dominated by the fundamental (first) mode of vibration. NLSPA is less suitable for taller, slender or irregular buildings, where multiple vibration modes affect the behaviour. Nonlinear time history analysis (NLTHA, refer sections C2.4.4 and C2.8.3) may be preferable if dynamic response and higher modes are considered to be significant (refer to section C2.5.10). If NLSPA is used for such structures elastic modal response spectrum analysis should be undertaken as well (refer to Section C2.8.2.4).

Alternative rational assessment procedures based on fundamental principles of engineering, mechanics and dynamics (e.g. using finite element analysis) may be appropriate but are not covered specifically by these guidelines and therefore cannot be used to determine whether or not a building is earthquake prone. If other procedures are to be used for other purposes it is recommended that they be shown to meet the objectives described in Part A and Section C1 and deliver an earthquake rating in the same form as outlined in Part A. Alternative procedures should be reviewed by a suitably qualified and independent engineer experienced in the procedure used, with particular emphasis on how well these aspects have been met.

C2.4.2 Displacement-based procedure and key steps

Displacement-based assessment (DBA) procedures focus on establishing the probable displacement capacity of the primary lateral system (Priestley, 1996; Priestley et al., 2007; Sullivan and Calvi, 2011; Kam et al., 2013; Grant, 2016). DBA utilises displacement spectra which can more readily and directly represent the response of a building in earthquake shaking.

Displacement-based methods use the same methods as force-based assessment to determine the force-displacement response of the structure. However, the expected displacement demand is based on the structural characteristics (effective stiffness and equivalent viscous damping) at the assessed displacements rather than on initial elastic characteristics. Displacement spectra set for different levels of elastic damping or ductility are used rather than the acceleration spectra reduced for ductility that are used for force-based design.

The displacement-based approach enables degrading strength and the influence of poor hysteretic response characteristics to be incorporated in the analysis. Similarly, the concepts can be extended to seismic retrofit design (Priestley et al., 2007; Kam and Pampanin, 2011).

The key steps of a DBA procedure are explained below. The sequencing of, and interaction between, these steps is described in the flowchart in Figure C2.4. The changes from the force-based procedure (described in Section C2.3.1) are shown in blue.

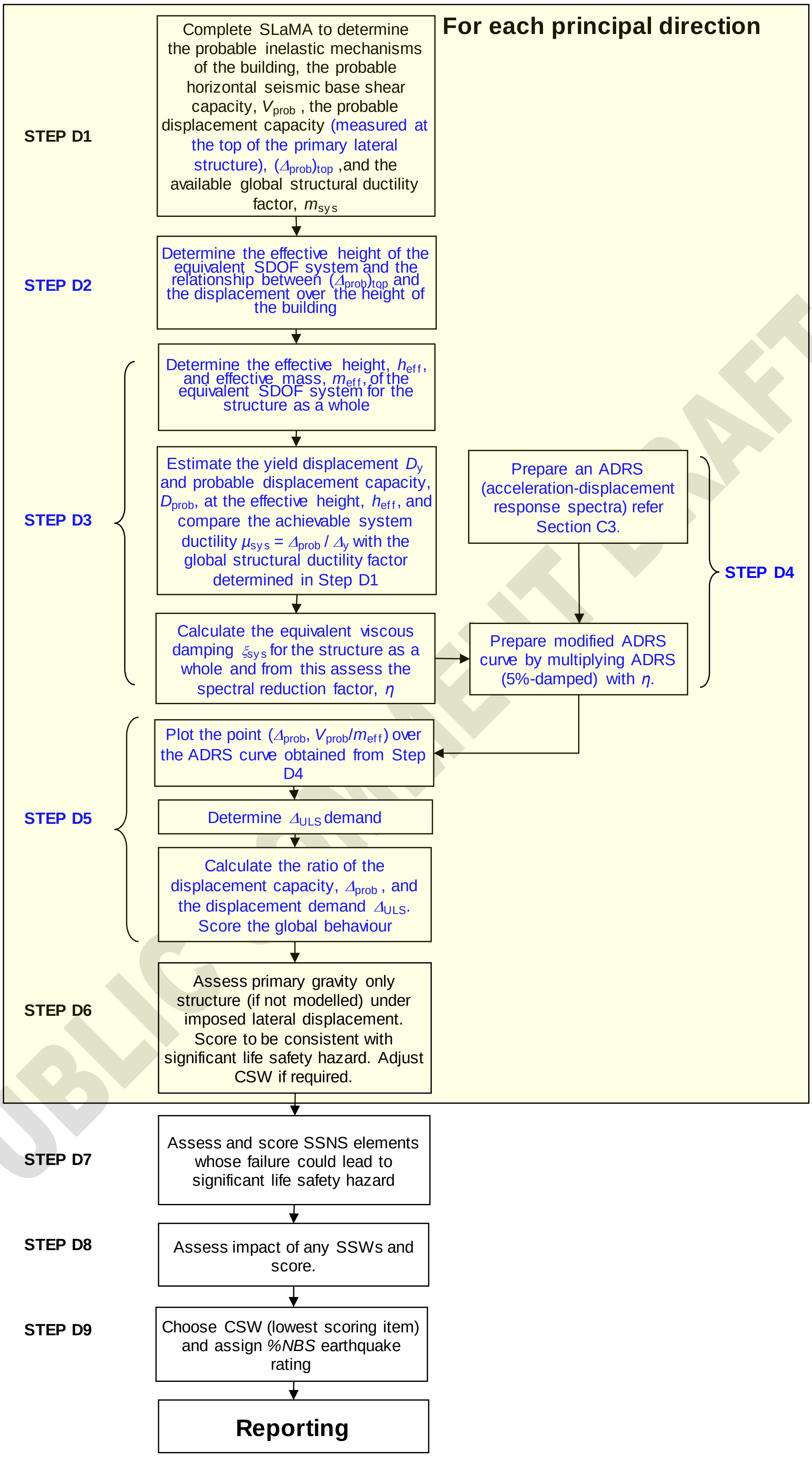


Figure C2.4: Summary of displacement-based assessment procedure (changes from the force-based procedure shown in blue)

Step D1 (similar to Step F1 for the force-based procedure)

Determine the inelastic sub-system mechanisms within the building that are likely to occur during seismic loading. From these calculate the probable horizontal seismic base shear capacity of the structure, V_{prob}, $(\Delta_{\text{prob}})_{\text{top}}$, and $(\Delta_{\text{y}})_{\text{top}}$ using the SLaMA technique outlined in Section C2.2.4 and Appendix C2A.

This step will require the assessment of the member/element/system probable capacities and also the interaction between modes of behaviour, e.g. degrading of the ductile capacity of concrete members in the presence of shear. It provides a simplistic representation of the probable strength capacity of the primary lateral structure and also the probable lateral displacement capacity measured at the top of the primary lateral structure.

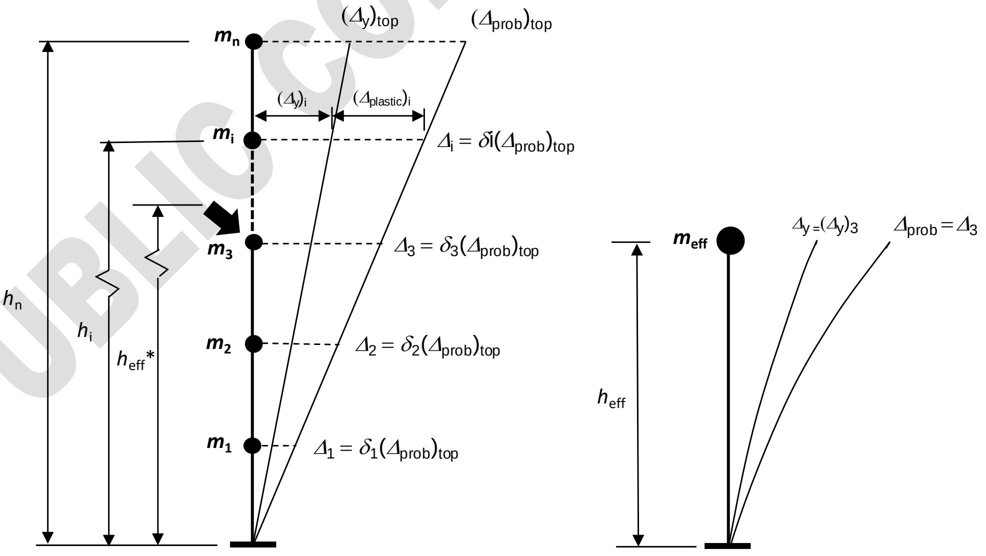
Estimate the global structural ductility factor, μ_{sys} as the ratio $(\Delta_{\text{prob}})_{\text{top}}/(\Delta_{\text{y}})_{\text{top}}$. For multiple sub-systems, $(\Delta_{\text{y}})_{\text{top}}$ can be determined graphically as illustrated in Figure C2.7.

Step D2

Identify the governing inelastic mechanism in the building to determine an appropriate deflection profile over the height of the building.

The global structural ductility capacity is generally governed by the dominant inelastic mechanism that contributes most to the plastic deformation of the system. The choice may not be immediately obvious and several options may need to be trialled.

A generic deflection profile is shown in Figure C2.5(a).



* by calculation then lower to nearest lumped mass

a) Deflection profile **b) Equivalent SDOF system**

Figure C2.5: Deflection profile and equivalent SDOF system for a generic multi-degree of freedom system

The deflection at the top of the building is taken as $\left(\Delta_{\mathrm{prob}}\right)_{\mathrm{top}}$ and the deflections at the assumed positions of the lumped masses are taken as $\delta_{\mathrm{i}}\left(\Delta_{\mathrm{prob}}\right)_{\mathrm{top}}$ where δ_{i} represents the normalised deflection profile at level i.

Deflection profiles are indicated below for various well developed inelastic mechanisms. These profiles include both the elastic and plastic proportion of the deflection.

- For a frame structure with beam-sway mechanism:

$$\delta_{\mathrm{i}} = \frac{h_{\mathrm{i}}}{h_{\mathrm{n}}} \text{ for } n \leq 4 \qquad \text{...C2.2}$$

$$\delta_{\mathrm{i}} = \frac{4}{3}\frac{h_{\mathrm{i}}}{h_{\mathrm{n}}}.\left(1 - \frac{h_{\mathrm{i}}}{4h_{\mathrm{n}}}\right) \text{ for } n > 4 \qquad \text{...C2.3}$$

- For a (moment or braced) frame structure with a potential ground floor column-sway soft-storey mechanism:

$$\delta_{\mathrm{i}} = 0.8 \text{ for } n = 1 \qquad \text{...C2.4}$$

$$\delta_{\mathrm{i}} = 0.8 + \frac{h_{\mathrm{i}}}{h_{\mathrm{n}}}(0.2) \text{ for } n > 1 \qquad \text{...C2.5}$$

- For cantilevered walls with a flexural mechanism governed by curvature ductility:

$$\delta_{\mathrm{i}} = \left(\Delta_{\mathrm{y}} + \Delta_{\mathrm{plastic}}\right)_{\mathrm{i}} = \frac{\varepsilon_{\mathrm{y}} h_{\mathrm{i}}^{2}}{L_{\mathrm{w}}}\left(1 - \frac{h_{\mathrm{i}}}{3h_{\mathrm{n}}}\right) + \theta_{\mathrm{p}} h_{\mathrm{i}} \qquad \text{...C2.6}$$

  Equation C2.6 above can be used as a lower bound envelope deformation profile, as wall systems generally have a linear deformation profile.

- For a steel braced frame with distributed plasticity mechanism:

$$\delta_{\mathrm{i}} = \frac{h_{\mathrm{i}}}{h_{\mathrm{n}}} \text{ (linear profile)} \qquad \text{...C2.7}$$

If a steel braced frame is susceptible to a soft-storey mechanism because the storey sway mechanism is not suppressed by capacity design considerations, then Equations C2.4 and C2.5 above can be used.

Guidance on various deformed shape profiles can also be found in literature (Priestley et al., 2007; Sullivan and Calvi, 2011) and the material sections (Sections C5 to C9) of these guidelines. Further research is required to extend the range of inelastic displacement profiles for other inelastic mechanisms and construction forms but, in the interim, it is considered reasonable to derive a representative deflection profile by reference to those that are available.

Note:

The dominant mechanism may not involve a well-developed ductile mechanism as outlined above and Δ_{prob} achievable for the equivalent SDOF system may not be above the flexural yield displacement. Notwithstanding these issues, it is expected that it will be possible to estimate appropriate values for deflections up the height of the structure based on the estimates of deflection capability at the top of the primary lateral structure determined from SLaMA.

Step D3

Determine the effective height, h_{eff}, and effective mass, m_{eff}, for the equivalent SDOF representation for the primary lateral system as follows:

$$h_{\mathrm{eff}} = \frac{\sum m_{\mathrm{i}}\Delta_{\mathrm{i}}h_{\mathrm{i}}}{\sum m_{\mathrm{i}}\Delta_{\mathrm{i}}} \qquad \text{...C2.8}$$

$$m_{\mathrm{eff}} = \frac{(\sum m_{\mathrm{i}}\Delta_{\mathrm{i}})^2}{\sum m_{\mathrm{i}}\Delta_{\mathrm{i}}^2} \qquad \text{...C2.9}$$

where:

Δ_{i} = $\delta_{\mathrm{i}}\left(\Delta_{\mathrm{prob}}\right)_{\mathrm{top}}$ determined from Step D2
m_{i} = lumped mass at level i
δ_{i} = generalised displacement at level i of the deformed shape profile.

These are shown diagrammatically in Figure C2.5.

Estimate the yield displacement, Δ_{y}, and the probable displacement capacity, Δ_{prob}, at the effective height, h_{eff}, from the deflection profile determined in Step D2. In practical terms, this will be the deflection determined for the level immediately below h_{eff}. Refer to Figure C2.5(b) which indicates what is intended.

Note:

For buildings less than or equal to five storeys (i.e. $n \leq 4$), h_{eff} can be assumed (to sufficient accuracy) to be the height to the level below $0.67h_{\mathrm{n}}$ and m_{eff} can conservatively be taken as the total mass of the building.

m_{eff} is associated with the effective mass in the principal inelastic deformed shape, which is typically first modal shape. Therefore m_{eff} should be calculated as per Equation C2.9 above and the engineer should not use the modal mass from elastic modal analysis.

Calculate the achievable system ductility, μ_{sys}, as follows:

$$\mu_{\mathrm{sys}} = \frac{\Delta_{\mathrm{prob}}}{\Delta_{\mathrm{y}}} \qquad \text{...C2.10}$$

Compare with the estimate obtained from SLaMA in Step D1, rationalise any significant differences and adjust if necessary.

Estimate the equivalent viscous damping, ξ_{sys}, available from the structure as a whole as set out in Appendix C2D.

The equivalent viscous damping of the system may be taken (Priestley et al., 2007) as the weighted average of the effective viscous damping values for each sub-system (identified in Step D1) based on the probable base shear capacity of each, i.e.:

$$\xi_{\mathrm{sys}} = \frac{\sum (V_{\mathrm{base}})_{\mathrm{i}}\xi_{\mathrm{i}}}{\sum (V_{\mathrm{base}})_{\mathrm{i}}} \qquad \text{...C2.11}$$

where: $(V_{\mathrm{base}})_{\mathrm{i}}$ and ξ_{i} are respectively the lateral base shear capacity and effective viscous damping for each sub-system i.

Step D4

Prepare the 100%ULS acceleration-displacement response spectrum (ADRS) for 5% damping and modify for the estimated effective viscous damping, ξ_{sys}, as described in Section C3.

Step D5

Plot the point $(\Delta_{\text{prob}}, V_{\text{prob}}/m_{\text{eff}})$ over the modified inelastic ADRS curve.

Determine the Δ_{ULS} demand by extending a line radiating out from the origin of this plot through the point $(\Delta_{\text{prob}}, V_{\text{prob}}/m_{\text{eff}})$ to intersect with the ADRS curve.

The *%NBS* earthquake score for global behaviour is the ratio of Δ_{ULS}, and Δ_{prob}.

Step D5 can be completed numerically using the principle of substitute structure (Priestley et al, 2007) as follows:

- Calculate the effective stiffness and effective period of the equivalent SDOF system

$$k_{\text{eff}} = \frac{V_{\text{prob}}}{\Delta_{\text{cap}}} \qquad \text{C2.12}$$

$$T_{\text{eff}} = 2\pi\sqrt{\frac{m_{\text{eff}}}{g.k_{\text{eff}}}} \qquad \text{C2.13}$$

- Plot T_{eff} line on the ADRS plot and read off the displacement demand Δ_{ULS} at the intersection of the T_{eff} and the modified inelastic ADRS demand curve.

Figure C2.6 shows a system involving several sub-systems and indicates the intent of Step D5.

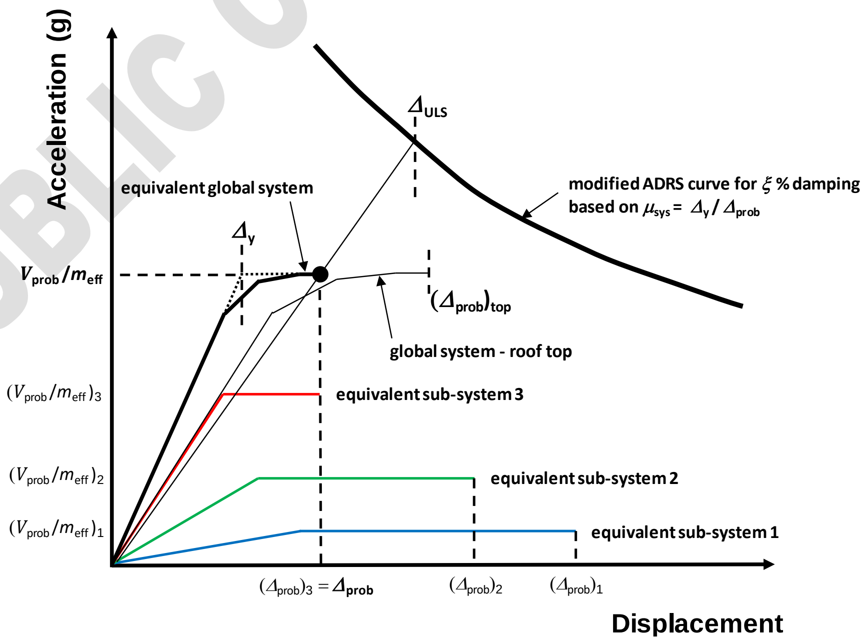


Figure C2.6: Derivation of earthquake score using SLaMA

Note:

Having plotted the capacity and the demand as shown in Figure C2.6 the effect of the various decisions that have been made are readily apparent. For example, if on reflection it is determined that exceeding the capacity of sub-system 3 does not lead to a life safety hazard (say the failure of the bolts in the brace connections of a braced frame) equivalent sub-system 3 can be ignored. This will lower Δ_{prob} but μ_{sys} will increase, bringing the ADRS curve closer to the origin. It is quite possible that the resulting global *%NBS* earthquake score will not be significantly different from when sub-system 3 was included. However, the additional score for the same strength capacity is achieved at higher expected deflections, which may have an impact on secondary structural and non-structural items.

Similarly, if sub-system 3 is retrofitted to achieve a higher displacement capacity the effect is twofold. Firstly the point $(\Delta_{\text{prob}}, V_{\text{prob}}/m_{\text{eff}})$ moves to the right and secondly the ADRS curve moves to the left with the higher μ_{sys}. The effect can be significant. It is also apparent that there may be little point in improving the displacement capacity of sub-system 3 beyond that available from sub-system 2 which may then govern.

Clearly, this representation provides an appreciation of the effect of decisions that may not be readily available from the results of more detailed analyses.

Also score the calculated displacements/drifts against the design limits given for the ULS in NZS 1170.5:2004. Consider carefully if the design displacement/drift limits are appropriate for the particular building and, if not, adjust the score.

Note:

For some buildings the deformation limits provided for the ULS in NZS 1170.5:2004 may be too severe to constitute a significant life safety hazard. For example, the 2.5% drift limit for a portal frame in an industrial building where the axial loads in the portal leg are low is unlikely to be of concern from the perspective of a significant life safety hazard.

The lowest score from consideration of the strength capacity of the primary structure and the deformation limits will provide the score for the global capacity of the primary lateral structure.

The effects of torsion should now be considered as outlined in Section C2.9.3 and Appendix C2F.

Steps D6 - D9

These steps are identical to Steps F6 through to F9 of the force-based procedure provided in Section C2.3.1.

C2.4.3 Nonlinear pushover procedure

This assessment procedure uses a nonlinear static pushover analysis (NLSPA) to determine the probable global displacement capacity of the building. This is then compared with the demand.

When using the NLSPA the lateral seismic forces acting on the frame are gradually increased until a mechanism forms. The behaviour of the structure is in the elastic range until the first plastic hinge forms, and then the post-elastic deformations at the plastic hinges need to be taken into account. The number of plastic hinges forming increases with an increase in lateral force until a mechanism develops, giving the probable lateral force capacity.

The nonlinear capacity curve can be generated by nonlinear modelling of the building structure, or a pseudo-nonlinear analysis using iterative elastic analyses, or a simplified method such as SLaMA. Refer to Section C2.6 for further guidance on the various analysis techniques available to generate the nonlinear pushover capacity curve.

The flow chart in Figure C2.8 summarises the generic assessment procedure using nonlinear static pushover analysis. The steps are similar to those for the displacement-based procedure described in Section C2.4.2 (changes from the force-based procedure described in Section C2.3.1 are in blue, and from the general displacement-based procedure are in red). Figure C2.7 illustrates the use of acceleration-displacement response spectrum (ADRS) with nonlinear pushover analysis to determine the *%NBS* earthquake score from the performance point ($\Delta_{\text{prob}}, V_{\text{prob}}$).

There are various ways to estimate Δ_{y} from the graphical representation. In Figure C2.7 the approach has been to approximate (by eye) the areas between the pushover curve and the assumed stylised initial stiffness (with cracked stiffness) and post mechanism stiffness (along the plateau) representations. More rigorously, the mathematical approach described in Clause 7.4.3.2.5 of ASCE 41 (2024) may be adopted.

Note:

The theoretical basis for the approach used is the equivalent linearisation approach for capacity spectrum assessment (ATC 40, 1996) and the substitute structure approach used in direct displacement-based design (DDBD) (Priestley et al., 2007).

Other approaches such as the coefficient methods presented in FEMA 440 (2005) and ASCE 41 (202) are available but the work has not been done to correlate them to the objectives of these guidelines. Therefore, they are not specifically covered by these guidelines.

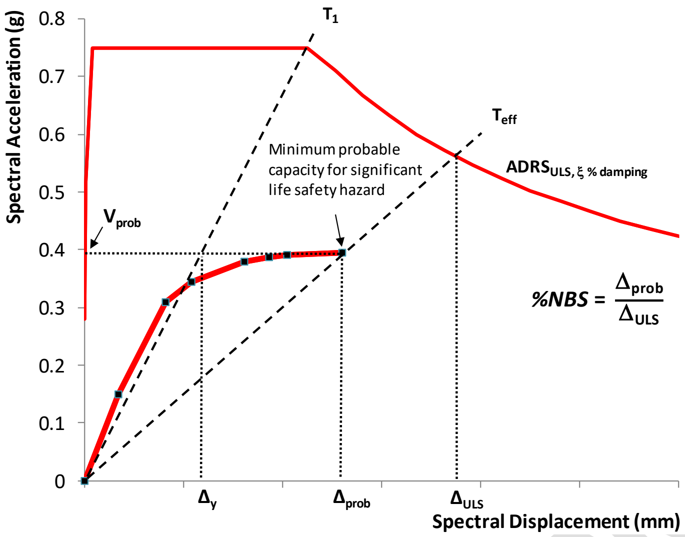


Figure C2.7: Nonlinear static pushover assessment using the ADRS plot

C2.4.4 Nonlinear time history procedure

This procedure uses a series of nonlinear time history analysis (NLTHA) to evaluate the level of earthquake motion that generates actions in the building less than or equal to those necessary to generate the probable deformation capacity in elements/members and/or the probable global deformation capacity of the building.

Refer to Section C2.8.3 and Appendix C2C for further guidance on NLTHA.

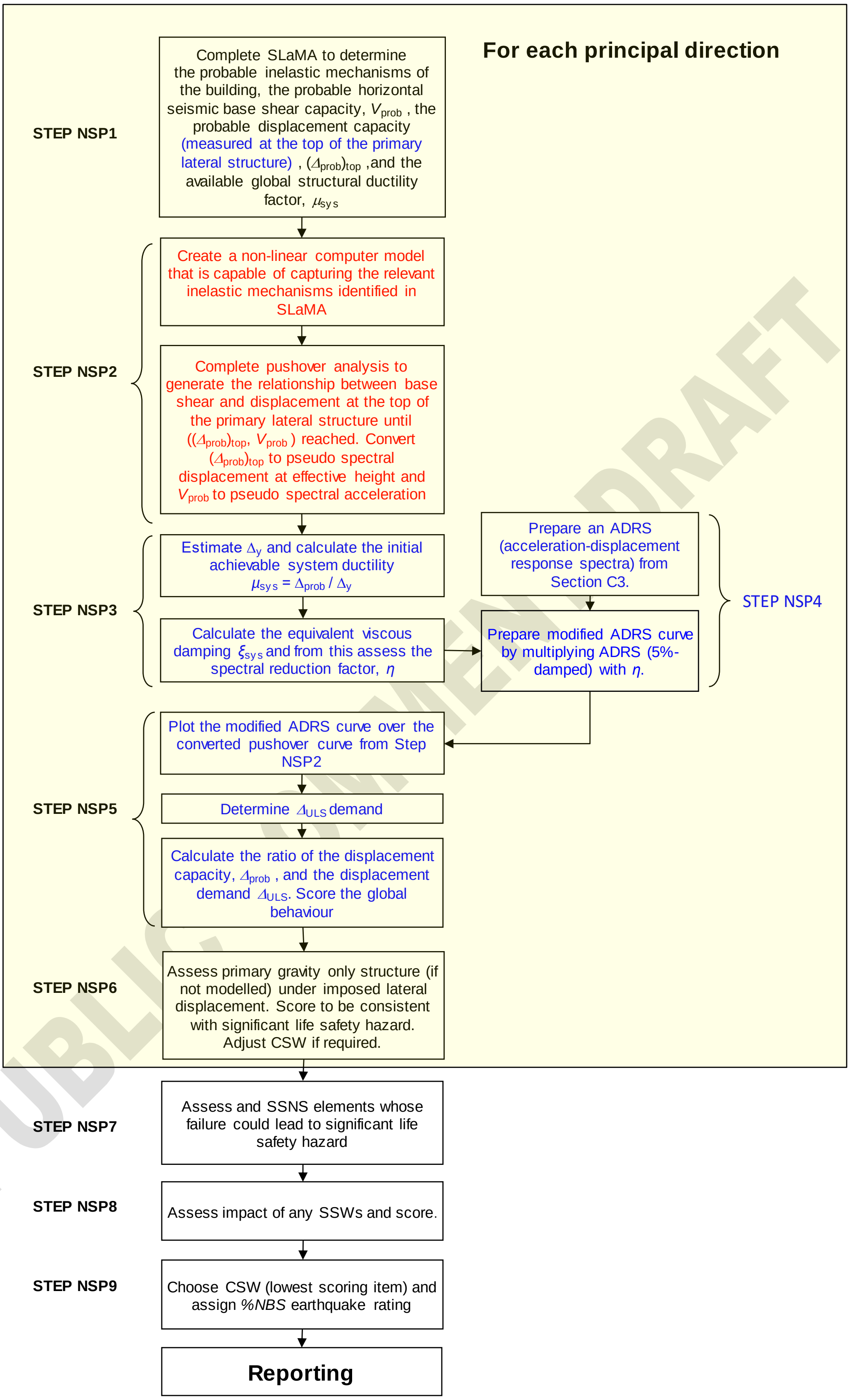


Figure C2.8: Assessment procedure using nonlinear static pushover analysis (changes from the force-based procedure are shown in in blue, and from the general displacement-based procedure in red)

C2.5 Assessment Procedures – Specific Issues

C2.5.1 Strength reduction factors

The probable capacities determined by these guidelines are typically calculated setting the strength reduction factor, ϕ, equal to 1.0. This is in addition to adopting probable material strengths.

For elements/members that are expected to behave in a particularly brittle fashion once the probable capacity is exceeded, the calculated probable capacities should be factored down to provide a margin against the brittle behaviour. This is particularly important when the brittle behaviour is intended to be avoided by a hierarchical load limiting approach (e.g. where a full ductile mechanism is intended to limit actions on brittle elements/ members).

The materials sections (Sections C5 to C9) build this factor into the assessment of probable capacity. In such cases, this is noted in the appropriate section (e.g. shear in reinforced concrete members or ground anchors failing in tension). The material sections also provide other specific adjustments to the reduction factor if it is considered that the calculated probable capacities are subject to high uncertainties and the consequence of exceeding these is significant.

Note:

Care needs to be taken when extracting probable capacities from other sources that capacities reflecting brittle behaviour are appropriately factored down to provide the margins intended in the other provisions of these guidelines. Refer to Section C1.5.1 for guidance on how deformation capacities should be defined for use in these guidelines.

C2.5.2 Characterising earthquake demand

The earthquake demand is determined in accordance with Section C3.

C2.5.3 Primary gravity structure

While seismic assessment should always include adequate consideration of the full primary structure, a high level of attention should be placed on the gravity system and on how well protected it is by the primary lateral system.

Note:

In the past, many engineers have expended a significant amount of effort in assessing the ability of the primary lateral system to resist the required lateral loads while paying, at best, only scant attention to other building elements (Kam and Jury, 2015).

It is clear that collapses are due to a failure of gravity support and catastrophic collapses due to a significant loss of gravity support. Catastrophic collapses typically occur because the primary lateral system has provided inadequate protection to the gravity system, particularly when the lateral and vertical resisting systems are separate and the gravity systems are heavily loaded and lack ductile capability.

While properly detailed and configured primary lateral systems have rarely failed due to a lack of strength capacity, there are numerous instances of gravity failures due to inadequate deformation capacity in the gravity system to sustain the applied deformations.

Often, the applied deformations have significantly exceeded those that would have been estimated during the design process. This can be due to the size of the earthquake but is often due to unexpected behaviour of the lateral system once it goes inelastic, particularly when this also causes torsional behaviour due to unexpected eccentricities.

Engineers need to recognise that earthquake induced deformations are not constrained in reality to levels assumed in design. Care needs to be taken when assessing low ductility gravity systems and adequately reflecting this in the analyses completed - and finally in the scoring of the primary gravity system. Refer also to Appendix C2G regarding the assessment of SSWs.

C2.5.4 Global versus component assessment

C2.5.4.1 Local component and element mechanisms

It is very important to recognise that the determination of member capacity, overall structural capacity and demand are not entirely separable and there is considerable interaction between these.

An obvious example is the need to know the strength of beam and column cross sections before carrying out an inelastic time history analysis or pushover analysis in order to ensure the correct mechanisms are identified. Another example is the need to correctly assess stiffness of members and the structure when doing modal analysis. Initial assumptions of member properties/capacities will have a bearing on the calculation of structural displacement. This in turn will affect the calculated demand on structural elements.

In the face of this, engineers will need to assess the implications carefully before choosing the most appropriate method of analysis (refer to Section C2.6). Considerable judgement will be needed to achieve a credible assessment to match the circumstances and available budget. For example, it may be possible to quickly identify which members/frames/walls are critical and restrict the analysis to those elements.

C2.5.4.2 Critical and non-critical element deficiencies

While the assessment of a building is focused on the members and elements level, it is important to appreciate that the overall building system performance is quantified by the consequence of the members and elements exceeding their probable capacities. It will generally be too conservative to rate a building on the basis of the first member or element exceeding its probable capacity. As set out in Part A, it is a crucial aspect of the assessment process outlined in these guidelines that the CSW identified must relate to behaviour that would lead to a significant life safety hazard as defined in Part A. In this regard it is recommended that that this test be applied whenever scoring the various aspects of the building.

C2.5.5 Horizontal and vertical irregularity

Horizontal and vertical irregularities, when they are present, can have a significant effect on the behaviour of a building. They can lead to the non-uniform development of ductility demands in a building's primary lateral structure and high displacement demands on the primary gravity structure.

When significant irregularities are present it is vital that the structural analysis models the effects appropriately and that adequate account is taken of the dynamic amplification effects that irregularities can have on the response once yield occurs in the structure.

Therefore, for buildings with identified horizontal and vertical irregularities it is recommended that nonlinear analysis – even if this is limited to SLaMA and DBA – is carried out to quantify the effects.

C2.5.6 Severe structural weaknesses (SSWs)

SSWs are not readily amenable to reliable assessment using the usual methods, which assume the probable capacity of the system can be determined from the probable capacities of the individual elements/members of that system.

Appendix C2G describes the aspects that need to be assessed as SSWs in a DSA and how to assess their capacity.

C2.5.7 Accidental displacement of the centre of mass

The consideration of an "accidental" displacement of the centre of mass for new building design is used to reflect uncertainties and variations in permanent structure densities, distribution of live loads and superimposed dead loads at the time of the earthquake, member/element/system stiffnesses and strengths, spatial shaking differences, etc.

While an argument can be made that some of these effects can be allowed for explicitly in the seismic assessment of existing buildings, it is difficult to determine the contribution that each makes to the design allowance.

The approach taken in these guidelines is that the allowance for accidental displacement of the centre of mass in a seismic assessment should be half that taken for design; i.e. ± 5% of the width of the building at right angles to the assumed direction of loading.

Note:

Background information regarding the basis of the . ± 10% accidental eccentricity required to be considered during design can be found in Elms (1976). It is considered that a less conservative accidental eccentricity is appropriate for assessment, which is also consistent with ASCE 41.

C2.5.8 Effects of torsion

Accidental eccentricity, plan irregularity and other irregularities results in torsional response, both in the elastic and nonlinear range of behaviour. The effects of torsion should be considered in the assessment using one of the approaches outlined in Appendix C2F.

The torsional response due to plan and structural irregularities is additive to the accidental mass eccentricity (refer to Section C2.5.7). The accidental mass eccentricity should always be applied irrespective of the analysis procedures used to considered torsional effects.

Equation deleted ...C2.14

Equation deleted ...C2.15

C2.5.9 Concurrent/bi-axial effects

The procedures outlined in Section C2.4 imply that the assessment procedure can be conducted for each principal direction separately to understand the governing inelastic mechanism and probable capacity of the systems. This is similar to the NZS 1170.5:2004 approach for ductile and limited ductile systems. By relaxing the concurrent effects in the analysis, a simpler assessment process can be achieved.

Concurrency effects should be considered for elements and systems where bi-axial effects would significantly change their response and performance.

For example, corner columns and their foundations which form part of two-way moment or braced frames should be assessed for bi-axial effects. This includes assessing the capacity-to-demand ratio from bidirectional or 45 degree diagonal loadings. It is also important to assess bi-axial effects for walls and foundations where the wall has significant flanges, for example T, C, I, and L shaped walls.

Sections C5 to C9 provide further guidance on elements and components that should be assessed for bidirectional effects. If concurrency actions are to be considered, the NZS 1170.5:2004 provisions for elastic and nominally ductile responding structure should be used (100%X + 30%Y; 30%X + 100%Y).

For NLTHA, the concurrency effect is modelled explicitly in the analysis by having pairs of horizontal input ground motions and bi-axial properties for elements and components that are affected by concurrency effects (refer to Section C2.8.3 for more on NLTHA).

C2.5.10 Higher mode effects

Higher mode effects can be assumed to be influential in structures if shear in any one storey, calculated from a modal analysis considering sufficient modes to achieve at least 90% mass participation, exceeds 130% of the corresponding storey shear resulting from a second analysis considering only the first mode participation.

For structures where higher modes are influential it is recommended that either linear or nonlinear dynamic analyses are carried out in conjunction with other analyses. For example, modal response spectrum analysis (MRSA) can be carried out in conjunction with SLaMA to give a better insight to the building's seismic performance.

Refer to Section C2.8.2 for information about the consideration of high modes for nonlinear static analyses.

Note:

Higher modes are more likely to be significant if a building's fundamental period exceeds approximately one second or if less than 60% of the mass participates in the first mode in a particular direction.

C2.5.10.1 Dynamic amplification

For cantilevered walls responding in a flexural ductile mechanism and for columns in ductile moment resisting frames, the shear force demand up the building is amplified by higher mode effects. A shear amplification factor, ω_{v}, should be applied as follows when considering these systems:

- For cantilevered shear walls responding in a flexural ductile mechanism ($\mu \geq 1.25$) the amplification factor should be taken as:

$$\omega_{\mathrm{v}} = 0.9 + \frac{n_{\mathrm{t}}}{10} \leq 1.3 \qquad \text{...C2.16}$$

where:

n_{t} = number of storeys.

- For moment resisting frames responding in a flexural ductile mechanism ($\mu \geq 1.25$) the amplification factor should be taken as:

$$\omega_{\mathrm{v}} = 1.3 \qquad \text{...C2.17a}$$

Note:

Column moment dynamic amplification (as per NZS 3101.2006 Appendix D) is not specifically required.

C2.5.10.2 Drift modification

Elastic-based methods of analysis can underestimate the critical inter-storey deflections of ductile structures. This discrepancy increases with the height of the building and the structural ductility factor. Similar discrepancies also occur where elastic time history analysis is used to assess inter-storey drifts.

To account for these effects a modified form of the drift modification factor from NZS 1170.5:2004, denoted here as k_{dm}^{*}, should be applied as outlined below when elastic methods are used to analyse a structure unless one or more of the following following statements apply:

- The expected displacement ductility does not exceed $\mu = 1.25$, or
- The structure being considered is a wall building.

The value of k_{dm}^{*} should be calculated using Equation C2.17b

$$1.0 \leq k_{dm}^{*} = 1.2 + 0.02(h - 15)) \leq 1.5 \qquad \text{...C2.17b}$$

The drift modification factor does not need to applied to results of a non-linear analysis, e.g. NLSPA, NLTHA, or SLAMA.

C2.5.11 Mixed ductility systems

Existing buildings generally have a mixed lateral load resisting system. In the same building there can be two or more structural systems providing lateral bracing but each with its own lateral stiffness, strength and deformation capacities.

When more than one lateral load mechanism is present or when there are components of varying strengths and stiffness, a displacement-based approach is considered essential to ensure that displacement compatibility is achieved and the global capacity is not overstated.

In general, mixed ductility and strength systems cannot be modelled adequately with only elastic analysis. The results can also be misleading, as the stiffer elements generally "attract" more loads while the displacement and ductility demands may be concentrated at the more flexible elements.

Figure C2.9 illustrates this effect: the deformation capacities of elements W3 and W4 cannot be utilised when considering the global deformation capacity as they both exceed the deformation capacity of element W7.

The global system displacement capacity, Δ_{cap}, and global ductility factor, μ_{sys}, can be determined by considering the sums of the force-displacement capacity curves, as also illustrated in Figure C2.9.

Note:

A common mistake is to assess a building with different deformation capacities allocated to different elements, without considering issues of displacement compatibility.

For example, in the case of a building containing a stiff masonry wall and flexible portal steel frames, the masonry wall would need to exceed its displacement capacity in order to yield the steel frame. Even if the initial stiffness of the two elements are relatively close together, there are still significant issues if one element yields or reaches its deformation capacity prematurely before the other element generates sufficient deformation to justify the higher global structural ductility factor.

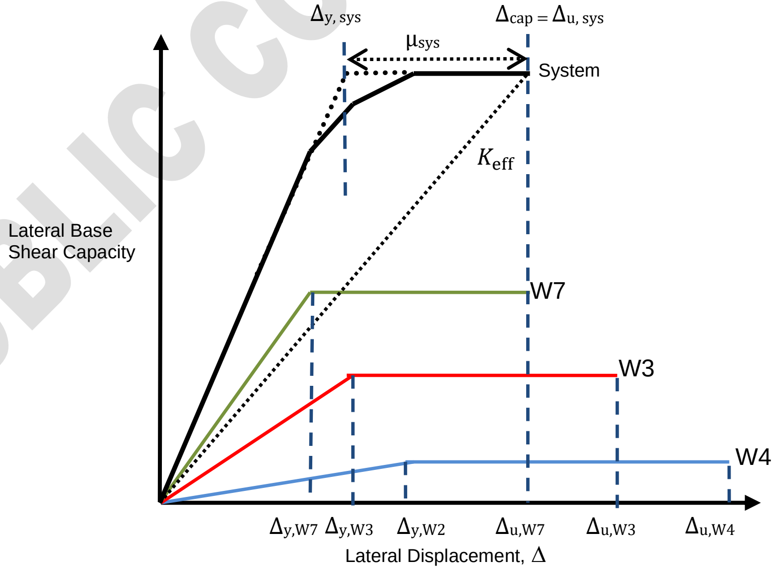


Figure C2.9: Displacement compatibility and mixed ductility

C2.5.12 Damping

The consideration of inherent damping and energy dissipation (hysteretic damping) varies depending on the assessment procedures and associated analysis techniques used:

- For DBA and nonlinear static procedures, the assessment of the equivalent viscous damping, ξ_{sys}, is important as the results are quite sensitive to the choices made.
- For force-based assessments, the capability for energy dissipation is captured within the structural ductility factor, μ, that is assumed and the inherent ductility is included in the assessment of the demand (typically 5%).
- For NLTHA, hysteretic damping and supplementary damping must be explicitly considered and modelled in the analysis.

Refer to Appendix C2D for detailed guidance on the assessment of ξ_{sys}.

C2.5.13 Secondary structural and non-structural (SSNS) elements

SSNS elements should be analysed as individual elements and assessed for imposed deformations in accordance with Section C10.

C2.6 Choosing Suitable Analysis Techniques

C2.6.1 Key principles

Assessing the seismic capacity of an existing building is fundamentally different from designing a new building for seismic actions. Seismic assessment requires a clear understanding and reliable evaluation of the existing load paths, the probable inelastic deformation mechanisms, the probable "collapse mechanism", and the available ductility/displacement capacity of the structure.

This relies on an understanding of the hierarchy of strength and sequence of failure of a structure in an earthquake, which can be gained by undertaking simplified hand calculations such as SLaMA (described in Section C2.2.4 and Appendix C2A). The comparison of capacities of various mechanisms (e.g. flexural versus shear) and within connected elements (e.g. wall to foundation) generally provides an indication of the hierarchy of strength and the likely post-elastic behaviour of a building in a severe earthquake.

Some considerations when selecting the most appropriate analysis techniques for seismic assessment follow:

- **For relatively simple structures** that conform with certain established criteria where complex analysis may not be warranted, the calculated demand and capacity of the building may be modified with appropriate factors based on the identified governing inelastic mechanism; e.g. the use of different allowable μ/S_{p}.
- **For complex/more significant structures** (e.g. with regard to occupancy, consequence of failure), and/or where greater levels of reliability of assessment are sought, the engineer is expected to assess, both qualitatively and quantitatively, the seismic behaviour of the building across a range of seismic shaking and to consider the inelastic behaviour and eventual governing mode(s) of failure, using appropriate methods.

Whatever analysis techniques are used, the consideration of nonlinear behaviour is fundamental for the assessment procedures given in these guidelines. The engineer can mobilise all available inelastic mechanisms – i.e. *%NBS* is not governed by the first element exceeding its strength/strain capacity – provided that local behaviour does not lead to loss of gravity support and/or lateral instability that could reasonably lead to a significant life safety issue.

Some key principles of structural analysis also apply:

- The structural model (computer or hand analysis) should consider and include the appropriate boundary conditions. In particular, the foundation system and the soil-structure flexibility should be modelled if this is deemed to have a significant influence on the behaviour of the building (refer to Section C2.9.2).
- Diaphragm flexibility or the presence of any diaphragm actions should be assessed prior to applying any diaphragm constraint in the model.
- A seismic assessment procedure itself is not tied to a specific analysis technique and the assessment should be informed by a number of analyses. Specific analysis techniques will have their own particular ways of characterising the earthquake demand and specific ways to predict the structure response and ultimate capacity. However, the assessment

procedure used to derive the *%NBS* needs to be holistic and include some aspects of engineering judgement.

- Analysis results, especially those of advanced techniques such as NLTHA, are highly dependent on the input parameters. Each of these is subject to judgement, probabilistic outcome and potential errors (e.g. ground motion records, nonlinear parameters, soil-structure modelling, etc.). There is a fine balance between accuracy, reliability (or precision), cost and complexity in structural analysis, as illustrated in Figure C2.10.

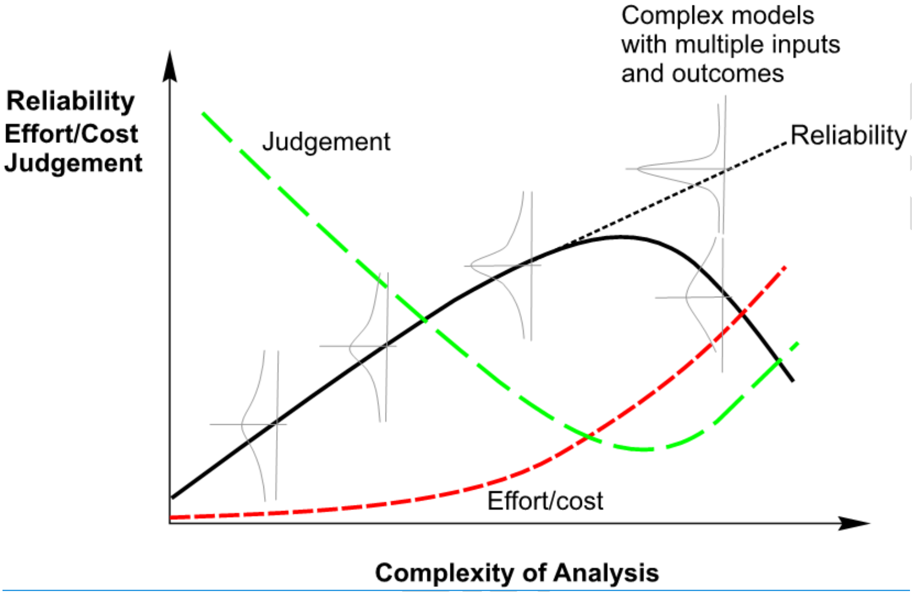


Figure C2.10: Trade-off between reliability, engineering judgement, cost and complexity of structural analysis (modified from Kam and Jury, 2015)

C2.6.2 Recommended techniques, their application and limitations

There are a number of analysis techniques that can be used in the seismic assessment of existing buildings to determine the distribution of member actions due to lateral seismic forces and gravity loading. The primary decision is whether to use a nonlinear analysis technique over the conventional elastic analysis that is more commonly used for designing new buildings.

In general, linear elastic analyses, including equivalent static analysis and modal response spectrum procedures, are applicable when the structure is expected to respond in an elastic or nominally ductile manner at the level of shaking consistent with the *%NBS* earthquake rating or when the nonlinear response and ductility demand is generally uniform throughout the structure (e.g. beam sway of frame structure).

The advantage of conventional elastic analysis techniques is that engineers are familiar and comfortable with these and the simplified analysis techniques often allow for analysis inputs to be minimised.

However, the results of linear analyses can be very misleading when applied to buildings with some of the following characteristics:

- highly irregular structural systems, either on plan/horizontal or vertical irregularity
- displacement incompatibility and mixed ductility responses (e.g. ductile steel frames in parallel with unreinforced masonry (URM)), or
- a complex sequence of failure mechanisms and inability of the structure to redistribute the lateral loads without compromising the gravity load path (e.g. premature shear failure of coupling beams which leads to softening of the coupled wall system, so the overall building has higher lateral displacement demand).

The effect of these characteristics may be to concentrate ductility demands to a greater extent than assumed when setting a global ductility factor. As a result the ductility demands in some parts of the building may well exceed the inherent assumptions that have been made.

A more sophisticated analysis does not, by itself, ensure improved accuracy of the building assessment. For example, it may not be possible to model degrading strength characteristics in an explicit nonlinear model; so that the improved precision implied by use of static pushover or time history analysis may be no more reliable than achieved by conducting a simple DBA with SLaMA.

Linear static and dynamic analysis techniques supplemented with SLaMA to consider the governing nonlinear mechanisms are likely to be sufficient for most buildings.

For some complex circumstances where even explicit nonlinear analysis is unable to predict the behaviour well, specific post-processing checks are required to complement the analysis techniques used (e.g. torsional instability of irregular and ductile systems).

Recommendations on the application and limitation of the various analysis techniques are summarised in Table C2.1 and covered in more detail in the following sections.

Table C2.1: Recommended analysis techniques: application and limitations

Analysis techniques	Comments
Elastic analysis	
Equivalent static analysis (linear static)	Useful for preliminary assessment, low-rise assessment and quick calculation • building height not exceeding 30 metres • no significant vertical stiffness or mass irregularly present • no significant torsional stiffness irregularity present, and • orthogonal lateral force-resisting systems present. Either: • elastic responding under 100%ULS shaking, or • low ductility demand/capacity (μ ≤ 2.0) under 100%ULS shaking and the following are satisfied: – no in-plane or out-of-plane discontinuities present in primary lateral force-resisting system – no significant weak storey irregularity present – no significant torsional strength irregularity present in any storey.

Analysis techniques	Comments
Modal response spectrum analysis (MRSA) (linear dynamic)	Useful for assessing higher modes and dynamic behaviour prior to nonlinear responses; and useful to assess modern buildings designed using modal response spectrum analysis. This method should not be used for systems with mixed ductility unless appropriate nonlinear assessment (e.g. SLaMA) has been undertaken in addition to the MRSA. Either: • elastic responding under 100%ULS shaking, or • low ductility demand/capacity ($\mu \leq 2.0$) under 100%ULS shaking and the following are satisfied: – no in-plane or out-of-plane discontinuities present in primary lateral force-resisting system – no significant weak storey irregularity present. While MRSA would consider the elastic torsion responses, it is recommended that inelastic torsion is also assessed via the procedure outlined in Section C2.5.8.
Pseudo-nonlinear using elastic analysis software	Uses elastic analysis but allowance for some non-consequential inelastic behaviour. This procedure is suitable for nonlinear analysis of relatively simple and small structures where the inelastic mechanism can be identified with a high degree of confidence. • Plastic hinge stiffness is removed/reduced artificially – iterative analysis – with plastic moment capacity applied as a moment at locations of plastic hinges. • This method relies on the local inelastic mechanism being well identified and ductile. The method assumes load redistribution with formation of plastic hinges. This condition needs to be confirmed, i.e. ductile hinges are formed. • To be used in conjunction with equivalent static and modal response spectrum analysis.
Nonlinear analysis	
SLaMA (pushover by hand)	SLaMA is recommended to be the first step of any assessment to determine the global inelastic mechanisms. • No significant torsional stiffness irregularity. If torsional irregularity is present then additional inelastic torsional check (Appendix C2F) must be undertaken. • Higher mode effects not critical. Linear dynamic analysis must be used in parallel if higher mode effect is influential. • Uses ADRS or displacement spectra as demand. • DBA is generally not suitable to be used by itself for large complex structures and should be complement by appropriate analysis that would allow the investigation of load redistribution and complex inelastic displacement profiles.
Nonlinear static pushover analysis (NLSPA)	• No significant torsional stiffness irregularity. If torsional irregularity is present then additional inelastic torsional check (Appendix C2F) must be undertaken. • Higher mode effects not critical. Linear dynamic analysis must be used in parallel if higher mode effect is influential (refer to Section C2.5.10).

Analysis techniques	Comments
Nonlinear time history analysis (NLTHA) (nonlinear dynamic)	Useful as either a final verification/analysis or in-depth analysis with multiple input ground motions and sensitivity study (research). • Suitable for highly irregular structures and buildings with significant higher mode effects. • Requires suitable nonlinearity models for the identified potential inelastic behaviour. • Preliminary nonlinear assessment using SLaMA, nonlinear pushover or similar analysis is required. • No limitation in terms of structural configuration. • High expertise and independent peer review are required.

C2.6.3 Use of other rational analysis techniques

The use of other alternative analysis techniques (e.g. energy-based methods, finite element model) that are rational and based on sound engineering principles are not precluded but cannot be considered to be specifically covered by these guidelines. If these methods are to be used it is highly recommended that other techniques as outlined in these guidelines are also used for comparison purposes and that alternative techniques (results and method) are peer reviewed by an independent engineer with relevant expertise.

C2.7 Elastic Analysis Techniques

C2.7.1 Equivalent static analysis

Under the equivalent static method, the lateral seismic forces are assumed to be distributed over the building height in accordance with the requirements of Section 6 of NZS 1170.5:2004 and the corresponding internal forces and building displacements are determined using a linear elastic static analysis.

Note:

It will be necessary to carefully consider any vertical irregularities as described in NZS 1170.5:2004. A common issue is the lightweight penthouse or upper storey on a building. This may present a vertical irregularity by virtue of the change in storey mass and/or storey stiffness. The issue of the irregularity may be avoided by considering the upper storey as secondary structure and lumping its mass at the top of the main part of the structure, which is then considered as the primary lateral structure. The actions on the secondary structure are then evaluated in accordance with Section 8 of NZS 1170.5:2004 and scored accordingly.

In this analysis the lateral seismic forces (distributed as above) are varied until the probable strength capacity is reached in a structural element/member that would be considered to be a significant life safety hazard if it were to fail. This is a lower bound estimate of the probable capacity of the primary lateral structure.

Some limited account may be taken of the post-elastic deformation capacity of the structure, to allow use of a system structural (displacement) ductility factor of $\mu > 1.0$. Assessment of this factor should be based on the expected displacement ductility capability of the weaker link components in the structure, but should not be taken greater than $\mu = 2$ unless the engineer is convinced (and can justify) that reliable mechanisms, preferably with a reliable hierarchy of hinge formation, are present.

Note:

Use of equivalent elastic analyses is not generally recommended but may be appropriate in circumstances where there is significant strength capacity to achieve the target *%NBS*.

C2.7.2 Modal response spectrum analysis (MRSA)

Modal response spectrum analysis (MRSA) is a linear dynamic analysis and is commonly used for new building design.

This technique is appropriate for use with structures that are expected to respond elastically to the input seismic action. In addition, MRSA is also suited for structures with well-defined and distributed plastic mechanisms, such as ductile frames, or for assessing recently designed structures (i.e. that meet capacity design or other modern seismic design requirements). It is generally only in such circumstances that an assumed initial level of global ductility will be able to be relied on to limit element/member ductilities to the required limits.

Note:

MRSA is typically used to assess existing multi-storey buildings designed post-1976 in which the original designer may have used MRSA as a basis of design. Engineers should check the appropriate "ductile" mechanism is achievable and that the ductility and inelastic mechanism are consistent with the initial assumption.

Accordingly, it is recommended that a SLaMA is completed before the 3D modelling and MRSA.

MRSA is generally **not appropriate** on its own for mixed-ductility systems. However, it can be a useful method to complement nonlinear static pushover analysis or SLaMA as it enables some consideration of higher mode effects.

The use of MRSA as a nonlinear technique to account for anticipated nonlinear response is generally inappropriate for the assessment of existing structures for the following reasons:

- There is no simple way of assessing the expected inelastic deformations from an MRSA. Common methods, which tend to assume that structure and member ductility levels are identical, are not necessarily correct unless no irregularities are present and the expected behaviour of any nonlinear mechanisms are well understood and ductility demands are well distributed.
- MRSAs underestimate the force levels and local ductility demand associated with higher mode response when member force levels are scaled back to inelastic mechanism strength. This is due to the different between inelastic deformed shapes and the elastic mode shapes used in MRSA (Carr, 1994). Conversely, MRSAs may overestimate torsional response levels for most buildings that respond inelastically.
- MRSAs cannot consider the influence that seismic axial force variations in members may have on their flexural stiffness. This can result in inaccurate estimates of the point at which inelastic action develops in reinforced concrete frame members. The influence of seismic force on member stiffness can be included directly in nonlinear methods.

Accordingly, apart from structural steel and timber structures, and concrete structures that are expected to respond elastically to seismic action, MRSAs should not be used as the sole means of analysis to assess existing structures unless special modifications are made to allow consideration of the above issues.

If MRSA is to be used, the modal response analysis should be carried out in accordance with NZS 1170.5:2004 and good engineering practice (e.g. Carr, 1994). MRSAs are carried out using linearly elastic response spectra, with the resulting forces generally scaled to match the lateral force used in the equivalent static procedure and the components evaluated in the elastic range of strength and serviceability. For any output from the MRSA, the aspect required should be found for each mode before statistical combination methods are applied. The post elastic deformation capacity of the structure is addressed in the same way as for the equivalent static method.

The earthquake demand should be in the form of response spectra derived as required by Section C3.

C2.7.3 Pseudo-nonlinear static pushover analysis

Pseudo-nonlinear static pushover analysis is a technique in which a nonlinear static pushover analysis is completed using a series of progressive elastic analyses.

Using this technique, the equivalent static earthquake forces are increased from zero until the first plastic hinge forms. The lateral seismic force corresponding to the development of the first plastic hinge gives a lower bound to the probable lateral force capacity of the structure, as shown in Figure C2.11. This lower bound estimate will always be less than or equal to the actual lateral force capacity. In reality, moment redistribution will permit higher lateral seismic forces to be resisted while further plastic hinges form until a mechanism develops or a member capacity is exceeded locally.

The lateral load and displacement at the point of the first plastic hinge formation is recorded and plotted on the pushover capacity curve.

The elastic analysis model is updated by releasing the member fixity at the point of the first hinge formation (e.g. the end of the beam or column) and by assigning an external moment (equivalent to the overstrength flexural capacity of the hinge). This will allow any additional moment to redistribute and the overall building softens. The elastic analysis is then re-run with increased force or displacement vectors until the formation of the next plastic hinge.

This sequential analysis is continued until a significant life safety hazard is identified. This marks the end of the pseudo-nonlinear static pushover analysis.

This analysis technique does not automatically track the actual ductility demand at individual hinges and assumes all hinges can sustain some level of ductility. If the hinge is non-ductile in nature (e.g. a shear mechanism on coupling beams) the engineer can elect to release the member fixity without assigning any external moment. This is appropriate as long as the gravity load carrying capacity is not compromised by the local inelastic mechanism.

Note:

This is a manual approach to undertake nonlinear pushover that can be particularly useful for practitioners unfamiliar with software packages capable of nonlinear static pushover analysis.

It can also be used to modify an elastic analysis model for force-based assessment procedures. In particular, the elastic analysis model can be modified to reflect the actual nonlinear response of the building. For example, beams exceeding their flexural capacity can be assigned "hinged" properties to release the moment and allow moment redistribution in the following elastic analysis. Multiple elastic models and iterative analysis to identify various "secondary" inconsequential mechanisms will be required.

When a mechanism can form, the method should yield the same result as the SLaMA.

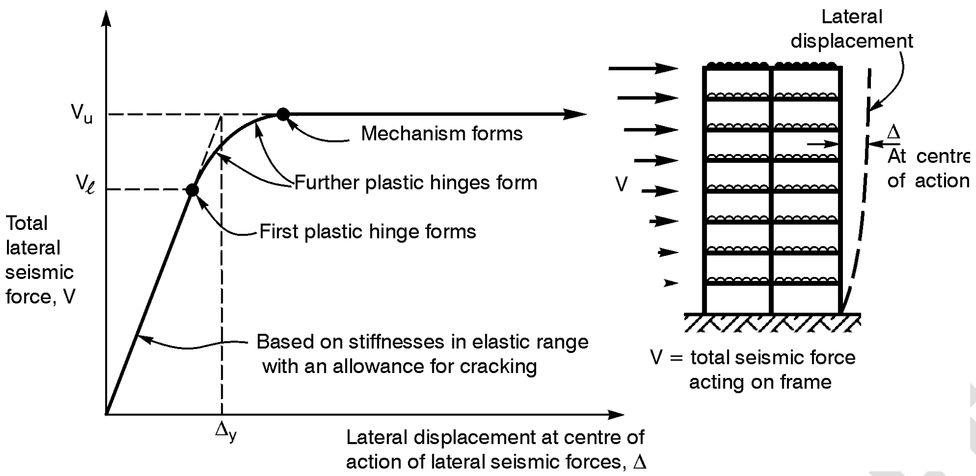


Figure C2.11: Pseudo-nonlinear static pushover analysis with iterative elastic analysis

C2.8 Nonlinear Analysis Techniques

C2.8.1 General

Nonlinear analyses involve significantly more effort to perform and should be approached with specific objectives in mind. Nonlinear analyses require a clear understanding of the probable inelastic behaviour and building response that will depend on both lateral forces and deformation.

The nonlinear models also require the definition of member analytical models that can capture the force-deformation response of these members. In some cases, there are no robust member analytical models available to capture a specific mechanism using commercially available software.

Nonlinear structural analysis models can vary significantly depending on a number of factors that:

- can be controlled (the objective of the analysis, outputs required, the level of structural nonlinearity that is modeled, the level of resources available, and the simplification adopted), and
- cannot be controlled (such as the ability of available analytical models to capture specific mechanisms).

A SLaMA is considered an essential initial stage for any nonlinear modelling to help identify which areas may require more focus and which are unlikely to undergo any inelastic deformation. A SLaMA will also help to provide an appreciation of how the various elements of the building are likely to interact.

A nonlinear analysis technique is appropriate for buildings which contain irregularities and when high levels of nonlinear behaviour are anticipated. If nonlinear pushover analyses are used, the engineer should include appropriate allowances in the analysis for anticipated cyclic strength and stiffness degradation.

Nonlinear time history analyses (NLTHAs), which are dynamic analyses, should be used with care. They require specific expert inputs to account explicitly for such factors as cyclic strength, stiffness degradation, higher modes, and inelastic dynamic behaviour (e.g. torsion).

Note:

The need to model strength and stiffness degradation is intended to pick up the detrimental aspects of these. Where these factors result in a beneficial effect they should be carefully appraised and be subjected to objective peer review.

However, NLTHAs are very complex and their results can be very sensitive to the input parameters, which may be associated with significant variability including within the modelling approach, uncertainties in input ground motions, input hysteresis models, etc. Sensitivity analyses will likely be warranted to “test” the effects of this variability.

Recent research (Krawinkler et al., 2011; Deierlein et al., 2010) has shown that a combination of NLTHA and NLSPA is better for the overall understanding and quantification of a structure’s seismic performance than either technique used by itself.

Therefore, it is advisable to employ a combination of both methods to understand seismic performance and quantify important engineering demand parameters. Both techniques are explained below.

C2.8.2 Nonlinear static pushover analysis (NLSPA)

C2.8.2.1 Description

NLSPA is essentially a refinement of the SLaMA approach which relies on explicit modelling of nonlinear parameters and load distribution within a computer programme.

An incremental inelastic lateral analysis of the structure is carried out under a lateral vector of floor forces, the magnitude of which is gradually increased. The onset of inelastic action of each member can thus be identified and the inelastic deformation of critical members tracked directly. This identifies the structure's probable capacity more reliably than is possible using linear elastic techniques.

NLSPA results in a simplified force-displacement response which can be used with a nonlinear assessment procedure (refer to Section C2.4) to determine *%NBS*.

The value of NLSPA is that it allows a detailed inspection of response and is a relatively simple tool for identifying critical regions of a structural system and inelastic mechanism.

The choice of the shape of the lateral force vector will affect the results; possibly including the location and type of inelastic action. Most engineers are familiar with the inverted triangle distribution of floor forces, but a structure developing a soft-storey sway mechanism should have a force vector essentially uniform with height.

As it is difficult to incorporate higher mode effects into NLSPA, in most cases it is still essentially a single mode approach and collapse mechanisms associated with higher modes may be missed.

Note:

As the structural model is being "pushed over", elements/members may experience nonlinear behaviour. The demands on the building (e.g. drifts) and the elements (curvature/rotations) are compared with the probable capacities (e.g. maximum curvature ductility, maximum inter-storey drift) at various steps of the pushover analysis. The governing condition occurs when the probable capacity is exceeded; provided that exceeding this capacity generates a significant life safety hazard.

Note that some elements/members can be allowed to exceed their probable capacity as long as the gravity load carrying capacity is maintained throughout the earthquake.

The base-shear versus centre-of-mass (or roof) displacement – i.e. the pushover capacity curve – is then analysed with a seismic spectral acceleration-displacement demand curve in order to determine the performance points. The assessment using a capacity-spectrum framework assumes that complex multi-degree-of-freedom models can be simplified into equivalent SDOF systems.

The seismic performance in terms of *%NBS* can be estimated by reducing the percentage of seismic demand such that the response demand parameters do not exceed the acceptable performance criteria.

C2.8.2.2 Modelling and analysis requirements

When carrying out an NLSPA:

- The reference point should be located at the centre of mass at the roof of a building. For buildings with a penthouse, the floor of the penthouse should be regarded as the level of the reference point. The displacement of the reference point in the mathematical model should be determined for the specified lateral loads.
- The relationship between base shear force and lateral displacement of the reference point should be established for reference point displacements ranging between zero and the displacement at which a significant life safety hazard is determined to occur.
- The component gravity loads should be included in the mathematical model for combination with lateral loads as specified in AS/NZS 1170.0:2002. The lateral loads should be applied in both the positive and negative directions.
- The analysis model is discretised to represent the load-deformation response of each member along its length to identify locations of inelastic action. All lateral-force-resisting elements should be included in the model.
- The force-displacement behaviour of all elements can be explicitly included in the model using full backbone curves that include strength degradation and residual strength, if any.
- Alternatively, a simplified analysis can be used. In such an analysis only primary lateral force-resisting elements are modelled, the force-displacement characteristics of such elements are bilinear, and the degrading portion of the backbone curve is not explicitly modelled. Elements not meeting the acceptance criteria but which do not represent a significant life safety hazard can be removed from the mathematical model.

Note:

When using the simplified analysis care should be taken to make sure that the removal of degraded elements from the model does not result in changes in the regularity of the structure that could potentially significantly alter the dynamic response. The simplified analysis does not automatically capture changes in the dynamic characteristics of the structure as yielding and degradation take place.

In order to explicitly evaluate deformation demands on elements that are to be excluded from the model, the engineer may consider including them in the model but with negligible stiffness to obtain deformation demands without significantly affecting the overall response.

C2.8.2.3 Lateral load vector/inelastic deformed shape profile

Lateral loads are applied to the mathematical model in proportion to the distribution of inertia forces in the plane of each floor diaphragm. For all analyses, at least two vertical distributions of lateral load should be applied. One pattern should be selected from each of the following two groups:

- A modal pattern selected from one of the following:
  - A vertical distribution of lateral load proportional to the values of C_{vx} given in Equation C2.18 below. Use of this distribution should be made only when more than 75% of the total mass participates in the fundamental mode in the direction under consideration, and the uniform distribution is also used.

$$C_{\mathrm{vx}} = \frac{w_{\mathrm{x}} h_{\mathrm{x}}^{\mathrm{k}}}{\sum_{\mathrm{i}=1}^{\mathrm{n}} w_{\mathrm{i}} h_{\mathrm{i}}^{\mathrm{k}}} \qquad \ldots\text{C2.18}$$

where:

C_{vx}	=	load distribution factor
w_{i}	=	portion of total building weight W on floor level i
w_{x}	=	portion of total building weight W on floor level x
h_{i}	=	height (in m) from base to floor level i
h_{x}	=	height (in m) from base to floor level x
k	=	2.0 for $T_1 \geq 2.5$ seconds and 1.0 for $T_1 \leq 0.5$ seconds. Linear interpolation is to be used for intermediate values of T_1.

- A vertical distribution of lateral load proportional to the shape of the fundamental mode in the direction under consideration. Use of this distribution should be used only when more than 75% of the total mass participates in this mode.
- A vertical distribution of lateral load proportional to the storey shear distribution calculated by combining modal responses from a response spectrum analysis of the building, including sufficient modes to capture at least 90% of the total building mass, and using the appropriate seismic demand spectrum from Section C3. This distribution should be used when the period of the fundamental mode exceeds 1.0 second.

• A second pattern selected from one of the following:
  - A uniform lateral load distribution consisting of lateral forces at each level proportional to the total mass at each level.
  - A lateral load distribution that changes as the structure is displaced. The load distribution is modified from the original load distribution using a procedure that considers the properties of the yielded structure.

Note:

A difficulty with the pushover analysis is that a static representation of the distribution of the seismic forces acting on the frame is required. Conventionally, an inverted triangular distribution of lateral seismic forces up to the height of the frame could be assumed, but this distribution takes no account of higher mode effects or changes in displaced shape post yield.

A sensitivity bound analysis is recommended to assess the differences in lateral force capacity of the frame arising from different distributions of seismic load; for example, uniform up the height.

In lieu of using the uniform distribution to bound the solution, it is possible to use a pushover analysis (Satyarno, 1999; Antoniou and Pinho, 2004) whereby the loading vector is updated at each analysis step to reflect the inelastic deformed shape and the associated redistribution of loading due to stiffness change and progressive damage accumulation in the structure. The changes in the distribution of lateral inertial forces are captured explicitly.

Procedures for developing adaptive load patterns include the use of storey forces proportional to the deflected shape of the structure (Fajfar and Fischinger, 1989), the use of load patterns proportional to the storey shear resistance at each step (Bracci et al., 1997), adaptive load pattern based on modal analysis result (Gupta and Kunnath, 2000) and displacement-based adaptive pushover (Antoniou and Pinho, 2004).

FEMA 440 (2005) and Pinho et al. (2006) provide a good summary of the various pushover techniques of this type. However, at this stage, only several commercially available software packages are capable of such analysis (e.g. Carr, 2007; Seismosoft, 2013; McKenna et al., 2004).

C2.8.2.4 Higher mode effects

Linear dynamic analysis should be used in parallel with NLSPA if higher mode effects are likely to be influential. If the higher modes are deemed to be significant according to Section C2.5.10 the assessment should include checks of internal actions from MRSA (scaled to the achievable base shear from NLSPA). Alternatively, it may be preferable to instead adopt NLTHA for the assessment (see Section C2.8.3).

The MRSA should have corresponding global ductility and stiffness modifiers to reflect the inelastic mechanism assessed from NLSPA. The MRSA base shear should be scaled to the achievable base shear as determined by NLSPA capacity curve. The MRSA results will be used to assess the displacement and internal actions of elements susceptible to higher mode effects, e.g. upper floors structure in a multi-storey building.

C2.8.3 Nonlinear time history analysis (NLTHA)

NLTHA is a form of dynamic analysis that, in principle, offers the most realistic prediction of seismic response.

The most important value of NLTHA is as an investigative tool to improve the understanding of the overall nonlinear mechanism trend and mean responses. NLTHA offers the ability to track the onset of inelastic response that is obtained from the nonlinear static pushover methods, while at the same time including higher mode effects in a realistic way as well as the manner in which they might vary as the structure becomes nonlinear.

As structural engineers become increasingly familiar with NLTHA and the relevant software becomes more readily available, this technique is expected to become a more popular analysis technique for structural assessment; particularly for more important structures. Even so, it requires considerable judgement, and the NLTHA results and model should be peer reviewed from a holistic view point by an independent engineer with appropriate expertise.

Refer to Appendix C2C for more about NLTHA and guidance on its use. Further guidance on the use of the ASCE 41 assessment approach using NLTHAs is provided in Section C1.

Note:

A number of guidance documents have been published on NLTHA for performance based seismic design and assessment (e.g. Deierlein et al., 2010; ASCE 41-23, 2023; FEMA 440, 2005). A number of software programs for NLTHA are also commercially available. As is the case for all analyses using proprietary computer programs the user must have a good understanding of methodologies adopted and the inherent limitations of the assumptions that are incorporated.

C2.9 Analysis Techniques – Specific Issues

C2.9.1 Primary, secondary structural and non-structural elements/members

Primary structural elements/members should be checked for earthquake induced forces and for deformations in combination with gravity load effects. Secondary structural elements/members should be checked for deformations imposed by the primary lateral structure in combination with gravity load effects and for seismic loads assuming the element/member is a part in accordance with Section 8 of NZS 1170.5:2004.

Primary structural elements/members that are considered to be part of the primary lateral load resisting system should be modelled in the lateral load analysis. Primary elements/ members that are identified as part of the primary gravity system only can be omitted from the model but should be checked for imposed displacements using post-processor techniques. Judgement will need to be exercised to decide which elements/members should be modelled explicitly.

Refer also to Section C10 for the treatment of SSNS elements.

The displacement capability of SSNS elements may limit the earthquake rating of the building as a whole.

C2.9.2 Soil-structure interaction (SSI) modelling

Close collaboration between structural and geotechnical engineers is needed to clarify the potential soil-structure interaction (SSI) behaviour (also known as soil-structure-foundation-interaction, or SFSI). Of critical consideration for both is the potential impact of geotechnical issues on the building structure in terms of life safety. A critical geotechnical weakness that does not in turn create a significant life safety hazard for the building will not be a potential critical structural weakness for the building and therefore it will not influence the building's earthquake rating.

The degree of SSI analysis and modelling sophistication will vary depending on the potential sensitivity of the superstructure and foundation to the overall SSI system. However, it is expected that a structural engineer assessing the building would consider such aspects as foundation flexibility and whether any step change behaviour is anticipated. If there is any indication that geotechnical issues could influence the behaviour of the building or where there is any doubt about this, a geotechnical engineer should be consulted.

Refer to Section C4 for more information on SSI, its likely influence on the earthquake rating and how to model SSI effects.

Note:

In the past, structural engineers have typically adopted a fixed base model for the interface between the structure and the ground on the basis that, for responses dominated by the first mode, this has been considered to be a conservative assumption. However, foundation flexibility often has a significant effect on the formation of mechanisms and also on the deformation capacity of the building, which can significantly affect the assessment rating determined for the building (e.g. Mylonakis and Gazetas, 2000). Fixed base assumptions may represent a conservative approach but this should be carefully reviewed before adoption.

C2.9.3 Diaphragm modelling and torsion effects

C2.9.3.1 General approach

Diaphragms are typically suspended floors or roof structures that are relatively thin horizontal structural systems capable of resisting and distributing lateral forces. Diaphragms transfer inertial forces from themselves and connected elements, such as stairs and services connected to them, to the lateral force-resisting structural systems. They may also resist differential in-plane movement of the lateral force-resisting structural systems.

A diaphragm can be classified as flexible or rigid:

- **Flexible diaphragm**: a diaphragm for which the maximum horizontal deformation of the diaphragm along its length is more than twice the average inter-storey drift of the vertical lateral force-resisting elements of the storey immediately below the diaphragm. For diaphragms supported by basement walls, the average inter-storey drift of the storey above the diaphragm should be used. In a URM building it is a diaphragm constructed of timber and/or steel bracing.
- **Rigid diaphragm**: a diaphragm that is not flexible. For assessment purposes, the structural model can assume that the storey mass and storey lateral shear force are concentrated at the centre of mass (including accidental allowance), and a coupled torsion moment is applied at the centre of rigidity.

Figure C2.12 illustrates some of the terminology used for diaphragms and wall displacements.

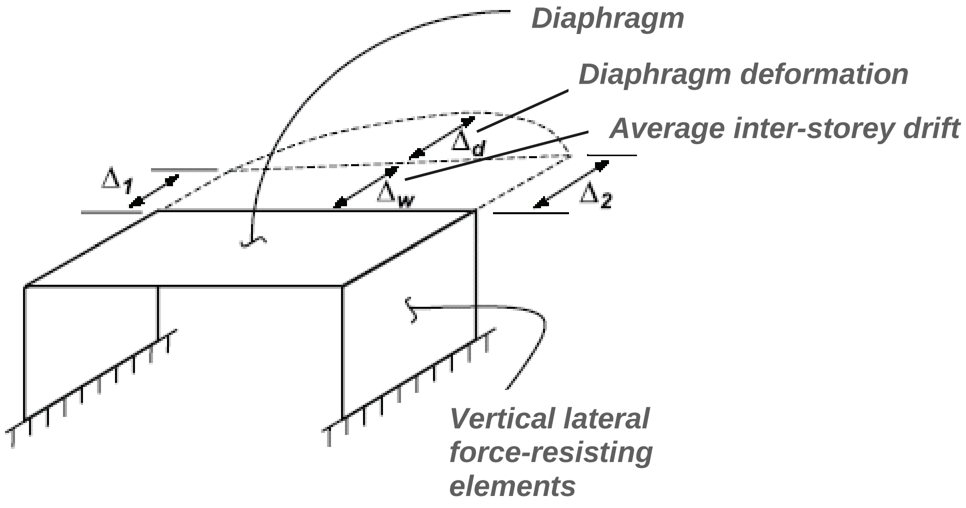


Figure C2.12: Diaphragm and wall displacement terminology

Note:

For the purpose of classifying diaphragms, the inter-storey drift and diaphragm deformations should be calculated using relevant diaphragm inertia loads. The in-plane deflection of the diaphragm should be calculated for an in-plane distribution of lateral forces consistent with the distribution of mass, and including all in-plane lateral forces associated with offsets in the vertical seismic framing at that diaphragm level.

In modern computer analysis package, the use and definition of flexible diaphragm is straight-forward and is not very demanding computational. Therefore, it is generally recommended to model as flexible diaphragm with the appropriate in-plane stiffness of the diaphragm system.

For flexible diaphragms, each lateral load resisting system can be assessed independently, with seismic mass assigned on the basis of the tributary area. The structural model can assume load distribution by tributary area. The engineer will need to check the displacement compatibility of the overall system and induced transfer forces within the diaphragm to ensure that the diaphragm, even though it is flexible, remains intact.

If the building has flexible diaphragms at each floor level each lateral force-resisting elements in a vertical plane can be assessed independently, with seismic masses assigned on the basis of tributary area. Although the centre of mass should be displaced between the lines of lateral support to reflect the accidental allowance, this will rarely prove to be significant.

For buildings with rigid diaphragms it will be necessary to consider the torsional amplification effect arising from the demand and resistance eccentricities (centre of mass and the location of the centre of stiffness or strength as appropriate).

Please refer to:

- Appendix C2E for more detailed guidance on diaphragm modelling and analysis
- Appendix C2F for more on the torsional amplification effects (for buildings with rigid diaphragms) and ways to assess this.

C2.9.3.2 Influence of infill walls

The potentially detrimental effect on the torsional response of non-uniform loss of infill (due either to in-plane or out-of-plane actions) at one or more storeys or on one or more lateral resistance lines of action should be considered, although any residual capacity of the bounding frames may also be taken when evaluating the lateral and torsional capacity of the building.

Note:

The engineer should recognise that the loss of the infill in all frames at the same storey may not occur and, therefore, an assumption that the residual capacity of the frames alone is available across any storey should not be relied upon.

C2.9.4 P-delta effects

Buildings should be checked for P-delta effects as set out in Section 6.5 of NZS 1170.5:2004.

Note:

P-delta effects are caused by gravity loads acting through the laterally deformed structure and result in increased lateral displacements.

A negative post-yield stiffness may significantly increase inter-storey drift and the displacement demand. Dynamic P-delta effects are introduced to consider this additional drift. The degree by which dynamic P-delta effects increase displacements depends on the:

- ratio α of the negative post-yield stiffness to the effective elastic stiffness
- fundamental period of the building
- structural ductility demand, μ, which is the ratio of the yield displacement to the ultimate displacement
- hysteretic load-deformation relations for each storey
- frequency characteristics of the ground motion, and
- duration of the strong ground motion.

C2.9.5 Seismic pounding

Many existing buildings do not comply with the current requirements for building separation. With insufficient building separation there is a high risk that seismic pounding (building to building impact) will occur, potentially affecting the performance of both structures. However, pounding is not usually an issue for adjacent buildings that are of the same height, have similar configuration and have aligning intermediate floors.

The effects of seismic pounding should be included in the building assessment (refer to Appendix C2B for details).

Note:

Appendix C2B also contains information on how to mitigate the effects of seismic pounding. However, in many cases, resolving pounding issues can be difficult given the different ownership of adjacent buildings.

References

Alizadeh, A. M. (2018). Diaphragm Performance in Steel Frame Structures Under Lateral Loading (PhD Thesis). The University of Canterbury, Christchurch, New Zealand, 701p.

Anagnostopoulos, S.A. and Karamaneas, C.E. (2008). *Collision shear walls to mitigate seismic pounding of adjacent buildings*, Proceedings of the 14th World Conference in Earthquake Engineering, Beijing, 12-17 October 2008.

Antoniou, S. and Pinho, R. (2004). *Advantages and limitations of adaptive and non-adaptive force-based pushover procedures*, Journal of Earthquake Engineering, Vol. 8(4), 497-522.

AS/NZS 1170.0:2002. *Structural Design Actions – Part 0: General Principles*, Standards Australia/Standards New Zealand.

ASCE 7-22 (2022). Minimum Design Loads and Associated Criteria for Buildings and Other Structures (ASCE/SEI 7-22). American Society of Civil Engineers, Reston, Virginia, 1046p.

ASCE 41-17 (2017). *Seismic Evaluation and Retrofit of Existing Buildings*, American Society of Civil Engineers, and Structural Engineering Institute, Reston, Virginia, USA.

ASCE 41-23 (2023). Seismic Evaluation and Retrofit of Existing Buildings (ASCE/SEI 41-23). American Society of Civil Engineers and Structural Engineering Institute, Reston, Virginia, 563p.

ATC 40 (1996). *Seismic evaluation and retrofit of concrete buildings*, Applied Technology Council, Redwood City, California, USA, Vol. 1 & 2, Report SSC 96-01, November 1996.

ATC 72 (2010) *Modelling and acceptance criteria for seismic design and analysis of tall buildings*, Applied Technology Council, Redwood City, California, USA. PEER/ATC 72-1 report, October 2010.

Baker, J.W. and Cornell, C.A. (2006). *Spectral shape, epsilon and record selection*, Earthquake Engineering and Structural Dynamics, 35 (9), 1077-1095.

Bertero, V.V. (1986).*Observations on structural pounding in the Mexico Earthquakes—1985: Factors Involved and Lessons Learned—1986*, Mexico City, Mexico: ASCE, New York, NY, USA, 1986.

Beyer, K. and Bommer, J.K. (2007). *Selection and Scaling of Real Accelerograms for Bi-Directional Loading: A Review of Current Practice and Code Provisions*, Journal of Earthquake Engineering, 11 (S1), 13-45.

Boys, A., Bull, D.K. and Pampanin, S. (2008). *Seismic performance of concrete columns with inadequate transverse reinforcement*, Proceedings of the 2008 New Zealand Concrete Industry Conference, Rotorua, New Zealand, 4-6 Oct 2008.

Bracci, J.M., Kunnath, S.K. and Reinhorn, A.M. (1997). *Seismic performance and retrofit evaluation of reinforced concrete structures*, Journal of Structural Engineering ASCE, Vol. 123, 3-10.

Bradley, B.A. (2010). *A generalized conditional intensity measure approach and holistic ground-motion selection*, Earthquake Engineering & Structural Dynamics, 39(12):1321-1342.

Bradley, B.A., Burks, L.S. and Baker, J.W. (2015). *Ground motion selection for simulation-based seismic hazard and structural reliability assessment*, Earthquake Engineering & Structural Dynamics, 44(13). 2321-2340.

Carr, A.J. (1994). *Dynamic analysis of structure*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol 27 (2). June 1994.

Carr, A.J. (2007)

Carr, A.J. and Moss, P.J. (1994). *Impact between buildings during earthquakes*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 27, No. 2, June 1994.

Castillo, R., Paulay, T. and Carr, A.J. (2002). *Design Concepts for Ductile Single Mass Asymmetric Systems*, Proceedings of the 3rd European Workshop on the Seismic Behaviour of Irregular and Complex Structures, Firenze, Italy.

Charney, F.A. (2008). *Unintended consequences of modeling damping in structures*, ASCE Journal of Structural Engineering, American Society of Civil Engineers, 134 (4), 581-592.

Cole, G.L., Dhakal, R.P., Carr, A.J. and Bull, D.K. (2010). *Building pounding state of the art: Identifying structures vulnerable to pounding damage*, Proceedings of the 2010 New Zealand Society for Earthquake Engineering Conference, Wellington, New Zealand, March 2010, Paper Number 11. 9 pages.

Cole, G.L., Dhakal, R.P., Carr, A.J. and Bull, D.K. (2011). *Case studies of observed pounding damage during the 2010 Darfield earthquake*, Proceedings of the Ninth Pacific Conference on Earthquake Engineering: Building an Earthquake-Resilient Society, Auckland, New Zealand, 14-16 April 2011, Paper Number 173.

DBH (2009). *Hollowcore guideline Seismic Performance of Hollow Core Floor Systems Guidelines for Design Assessment and Retrofit: Preliminary Draft. April 2009*, Department of Building and Housing (DBH). Available online: http://www.building.govt.nz/UserFiles/File/Consulting/pdf/2009/Seismic-Performance-of-Hollow-Core-Floor-Systems.pdf.

Deierlein, G.G., Reinhorn, A.M. and Willford, M.R. (2010). *Nonlinear structural analysis for seismic design, NEHRP Seismic Design Technical Brief No. 4*, National Institute of Standards and Technology (NIST), Gaithersburg, MD, NIST GCR 10-917.

EAG (2012). *Guidance on detailed engineering evaluation of earthquake affected non-residential buildings in Canterbury*, Part 3.9 - Reinforced Concrete Wall Buildings. Revision 0.5, Engineering Advisory Group (EAG) - Commercial, Christchurch.

Elwood, K.J. and Moehle, J.P. (2005). *Axial Capacity Model for Shear-Damaged Columns*, ACI Structural Journal. 102:4, 578-587.

Elms, D. G. (1976). "Seismic Torsional Effects on Buildings." Bulletin of the New Zealand National Society for Earthquake Engineering, 9(1), pp.79–83.

Fajfar, P. and Fischinger, M. (1989). *N2 – A method for nonlinear seismic analysis of regular buildings*, Proceedings of the 9th World Conference on Earthquake Engineering, Tokyo, Kyoto, 1988, Maruzen, Tokyo, Vol. V, 111-116.

FEMA 440 (2005). *Improvement of nonlinear static seismic analysis procedures*, Federal Emergency Management Agency, FEMA Report 440, Washington, D.C.

FEMA 440a (2009a). *Effects of strength and stiffness degradation on the seismic response of structural systems*, Federal Emergency Management Agency, FEMA Report 440a, Washington, D.C.

FEMA P-58 (2012). *Seismic Performance Assessment of Buildings, Methodology and Implementation*, September 2012. Federal Emergency Management Agency, FEMA Report P-58, Washington, DC. Three volumes.

Gardiner, D.R. (2011). *Design recommendations and methods for reinforced concrete floor diaphragms subjected to seismic forces*, PhD Thesis, Department of Civil and Natural Resources Engineering. University of Canterbury, 2011.

Goel, R.K. and Chopra, A.K. (2004). *Extension of Modal Pushover Analysis to Compute Member Forces*, Earthquake Spectra, 21:1, 125-139.

Grant, M.L. (2016). *Displacement based assessment and improvement of a typical New Zealand building by an average engineer*, Proceedings of the 2016 New Zealand Society for Earthquake Engineering Conference, Christchurch, 1-3 April 2016, Paper No. O-17.

Gupta, B. and Kunnath, S.K. (2000). *Adaptive spectra-based pushover procedure for seismic evaluation of structures*, Earthquake Spectra, 16:2, 367-392.

Hancock, J., Bommer, J.J. and Stafford, P.J. (2008). *Numbers of scaled and matched accelerograms required for inelastic dynamic analyses*, Earthquake Engineering & Structural Dynamics 37(14): 1585-1607.

Ibarra, L., Medina, R. and Krawinkler, H. (2005). *Hysteretic models that incorporate strength and stiffness deterioration*, Earthquake Engineering and Structural Dynamics, 34 (12), pp 1489-1511.

Jeng, V. and Tzeng, W.L. (2000). *Assessment of seismic pounding hazard for Taipei City*, Engineering Structures, 2000. 22(5): pp 459-471.

Kalkan, E. and Chopra, A.K. (2010). *Practical guideline to select and scale earthquake records for nonlinear response history analysis of structures*.

Kam, W.Y. (2010). *Selective weakening and posttensioning for the seismic retrofit of non-ductile RC frames*, PhD Dissertation, Department of Civil and Natural Resources Engineering, University of Canterbury. November 2011. 2 volumes.

Kam, W.Y., Akguzel, U., Jury, R. and Pampanin, S. (2013). *Displacement-based seismic assessment: Practical considerations*, Proceedings of the 2013 New Zealand Society for Earthquake Engineering Conference, Wellington, 26-28 April 2013, Paper No. 45.

Kam, W.Y. and Jury, R. (2015). *Performance-based seismic assessment: Myths and Fallacies*, Proceedings of the 2015 New Zealand Society for Earthquake Engineering Conference, Rotorua. 10-12 April 2015, Paper No. S-03.

Kam, W.Y. and Pampanin, S. (2011). *Displacement-based seismic retrofit design for non-ductile RC frame structures using local retrofit interventions at beam-column joints*, Proceedings of the 9th Pacific Conf. on Earthquake Engineering, Auckland, New Zealand, 14-16 April 2011. Paper No. 192.

Kam, W.Y., Pampanin, S. and Elwood, K. (2011). *Seismic performance of reinforced concrete buildings in the 22 February Christchurch (Lyttelton) Earthquake*, Bulletin of New Zealand Society for Earthquake Engineering, Vol. 44, No. 4, 239-278, December 2011.

Kang, T.H.K. and Wallace, J.W. (2006). *Punching of Reinforced and Post-Tensioned Concrete Slab-Column Connections*, ACI Structural Journal, July/Aug 2006, Vol. 103 Issue 4, 531.

Karayannis, C.G. and Favvata, M.J. (2005). *Earthquake-induced interaction between adjacent reinforced concrete structures with non-equal heights*, Earthquake Engineering & Structural Dynamics. 34(1), 1-20.

Kasai, K., Jeng, V., Patel, P.C., Munshi, J.A. and Maison, B.F. (1992). *Seismic Pounding Effects – Survey and Analysis*, Proceedings of the 10th World Conference on Earthquake Engineering. Balkerna Rotterdam, pp 3893-3898.

Kasai, K., Maison, B.F. and Patel, D.J. (1990). *An Earthquake Analysis for Buildings Subjected to a Type of Pounding*, Proceedings of the Fourth US National Conference on Earthquake Engineering, EERI, Oakland, CA.

Khatiwada, S., Chouw, N. and Butterworth, J.W. (2011). *Development of pounding model for adjacent structures in earthquakes*, Proceedings of the 9th Pacific Conference on Earthquake Engineering, Auckland, New Zealand, 14-16 April 2011, Paper No. 080.

Krawinkler, H., Lignos, D. and Putman, C. (2011). *Prediction of Nonlinear Response – Pushover Analysis versus Simplified Nonlinear Response History Analysis*, Structures Congress 2011, pp 2228-2239.

McKenna, F., Fenves, G.L. and Scott, M.H. (2004). *OpenSees: Open system for earthquake engineering simulation*, Pacific Earthquake Engineering Research Center, University of California, Berkeley, CA; p. Inelastic Analysis Finite Element program.

Mylonakis, G. and Gazetas, G. (2000). *Seismic soil-structure interaction: beneficial or detrimental?* Journal of Earthquake Engineering, Vol 4(3), 277-301.

NIST (2013). *Review of Past Performance and Further Development of Modeling Techniques for Collapse Assessment of Existing Reinforced Concrete Buildings*, NIST-Report NIST GCR 14-917-28. National Institute of Standards and Technology (NIST), Gaithersburg, Maryland. 201 pages.

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

NZS 1170.5 inc. A1. *Structural design actions, Part 5: Earthquake Actions - New Zealand* (NZS 1170.5 inc. A1). Standards New Zealand, Wellington, New Zealand, 88p.

NZS 3101:2006. *Concrete structures standard*. NZS 3101:2006. Standards New Zealand, Wellington, NZ.

NZS 3604:2011. *Timber-framed buildings.* NZS 3604:2011. Standards New Zealand, Wellington, NZ.

NZSEE (2006). *Assessment and improvement of the structural performance of buildings in earthquakes. Incl. Corrigenda 1 & 2*, New Zealand Society for Earthquake Engineering (NZSEE), Wellington, NZ.

O'Reilly, G.J. and Sullivan, T.J. (2015). *Direct Displacement-Based Seismic Design of Eccentrically Braced Steel Frames*, Journal of Earthquake Engineering, DOI 10.1080/13632469.2015.1061465.

Pampanin, S., Magenes, G. and Carr, A. (2003). *Modelling of shear hinge mechanism in poorly detailed RC beam-column joints*, Proceedings of the Concrete Structures in Seismic Region: fib 2003 Symposium, Federation International du Beton, Athens, Greece, Paper No. 171.

Park, R. (1996). *A static force-based procedure for the seismic assessment of existing reinforced concrete moment resisting frames*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 30, No. 3.

Park, R and Paulay. T. (1975). T. *Reinforced Concrete Structures*, John Wiley and Sons, New York.

Paulay, T. (2000a). *Principles of displacement compatibility*, Journal of the Structural Engineering Society, New Zealand. Vol. 13(2):14-21.

Paulay, T. (2000b). *Understanding torsional phenomena in ductile systems*, Bulletin of New Zealand Society for Earthquake Engineering, Vol.33, No.4, 403-420.

Paulay, T. (2001). *Some Design Principles Relevant to Torsional Phenomena in Ductile Buildings*, Journal of Earthquake Engineering. Vol.5, No.3, 273-300.

Pekcan, G., Mander, J.B. and Chen, S.S. (1999). *Fundamental considerations for the design of nonlinear viscous dampers*, Earthquake Engineering Structural Dynamics 28, 1405-1425, 1999.

Pinho, R., Antoniou, S. and Pietra, D. (2006). *A displacement-based adaptive pushover for seismic assessment of steel and reinforced concrete buildings*, Proceedings of the 8th US National Conference on Earthquake Engineering, San Francisco, USA, Paper No. 1701.

Priestley, M.J.N. (1996). *Displacement-based seismic assessment of existing reinforced concrete buildings*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol. 29, No. 4, December 1996.

Priestley, M.J.N. and Calvi, G.M. (1991). *Towards a capacity-driven assessment procedure for reinforced concrete frames*, Earthquake Spectra 7(3): 413–37.

Priestley, M.J.N., Calvi, G.M. and Kolwasky, M.J. (2007). *Displacement-based seismic design of structures*, IUSSS Press. Pavia, Italy.

Robertson, I.N. and Johnson, G.P. (2004). *Non-ductile slab-column connections subjected to cyclic lateral loading*, 13th World Conference on Earthquake Engineering Vancouver, B.C., Canada August 1-6, 2004 Paper No. 143.

Satyarno, I. (1999). *Pushover Analysis for the Seismic Assessment of Reinforced Concrete Buildings*, Ph.D. Thesis, Department of Civil Engineering, University of Canterbury, March 1999.

Seismosoft (2013)

Stafford, P., Sullivan, T.J. and Pennucci, D. (2016). *Empirical correlation between inelastic and elastic spectral displacement demands*, Earthquake Spectra, Aug 2016, Vol. 32 (3), 1419-1448.

Stirrat, A.T., Gebreyohaness, A.S., Jury, R.D. and Kam, W.Y. (2014). *Seismic performance assessment of non-ductile columns*, Proceedings of the 2014 New Zealand Society for Earthquake Engineering Conference, Auckland, New Zealand, 21-23 March 2014, Paper O1. 17 pages.

Sullivan, T.J. (2016). *Guidance on the use of equivalent viscous damping for seismic assessment*, Proceedings of the 2016 New Zealand Society for Earthquake Engineering Conference, Christchurch, 1-3 April 2016, Paper No. O-27.

Sullivan, T. and Calvi, G.M. (2011). *Considerations for the seismic assessment of buildings using the direct displacement-based assessment approach*, Proceedings of Italian Annual National Conference (ANIDIS 2011). Online: http://ww2.integer.it/Web_1/database_locale/ATTI%20ANIDIS_2011/Papers/SS04/298%20996-2044-1-RV.pdf.

Sullivan, T.J., Pennucci, D., Piazza, A., Manieri, S., Welch, D.P. and Calvi, G.M. (2013). *General aspects of the displacement based assessment approach*, in Developments in the Field of Displacement-Based Seismic Assessment, Edited by Sullivan, T.J., Calvi, G.M., IUSS Press, Pavia, Italy, ISBN; 978-88-6198-090-7.

Sullivan, T.J., Priestley, M.J.N. and Calvi G.M. Editors (2012). *A Model Code for the Displacement-Based Seismic Design of Structures*, DBD12, IUSS Press, Pavia, ISBN 978-88-6198-072-3, 105 pages.

ULIEGE (2007). *Analysis of hammering problems. Stage 2*, Viewable at: http://www.lessloss.org/main/index.php?option=com_docman&task=doc_details&gid=309.

Uma, S. R., Holden, T., King, A. B., and Bell, D. K. (2009). Acceptable Inter-Storey Drift Limits for Buildings at Ultimate Limit States (GNS Science Consultancy Report 2009/16). GNS Science, Lower Hutt, New Zealand, 41p.

Uma, S. R., King, A. B., and Holden, T. (2012). “Inter-storey Drift Limits for Buildings at Ultimate Limit States.” Proceedings of the NZSEE Conference, Christchurch, New Zealand, 10p.

Wijesundara, K.K., Nascimbene, R. and Sullivan, T.J. (2011). *Equivalent Viscous Damping for Steel Concentrically Braced Frame Structures*, Bulletin of Earthquake Engineering, Vol.9, Issue 5, 1535-1558.

Suggested Reading

Bradley, B.A. (2015). *Benefits of site-specific hazard analyses for seismic design in New Zealand*, Bulletin of the New Zealand Society for Earthquake Engineering, Vol 48, No. 2 92-99.

Bradley, B.A. (2015). *Period Dependence of Response Spectrum Damping Modification Factors due to Source and Site-Specific Effects*, Earthquake Spectra, Vol. 31 (2): 74e5759.

Brook, R.A., Kelly, T.E. and Mackenzie, C.S.M. (2007). *Performance based assessment and design policy recommendations*, Proceedings of the 2007 New Zealand Society for Earthquake Engineering Conference, Palmerston North, New Zealand, March 2007.

Bull, D.K. (2011). *Stairs and Access Ramps between Floors in Multi-storey Buildings,* Report to the Royal Commission*.*

CERC. (2012). *CERC Final report - Volume 2: The performance of Christchurch CBD Buildings*, Canterbury Earthquakes Royal Commission (CERC). August 2012. 236 pages.

Clayton, P., Kam, W.Y. and Beer, A. (2014). *Interaction of geotechnical and structural engineering in the seismic assessment of existing buildings*, Proceedings of the 2014 New Zealand Society for Earthquake Engineering Conference, Auckland, 21-23 March 2014, Paper Number O39. http://db.nzsee.org.nz/2014/oral/ 39_Kam.pdf

Cole, G.L. (2012). *The effects of detailed analysis on the prediction of seismic building pounding performance*, PhD dissertation. Department of Civil and Natural Resources Engineering, University of Canterbury. 487 pages.

DBH (2011). *Practice Advisory 13 Egress Stairs: Earthquake checks needed for some*, Wellington, New Zealand, September 2011.

DZ 1170.5 (2014). *Public draft of NZS 1170.5 Structural Design Actions – Part 5: Earthquake actions – New Zealand – Standard Amendment 1*, Standards New Zealand, Wellington, NZ.

EN 1993-1-1:2005. *Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings*, European Committee for Standardization (CEN).

Fenwick, R., Bull, D. and Gardiner, D. (2010). *Assessment of hollow-core floors for seismic performance*, Research Report 2101-2, Civil and Natural Resources Engineering, University of Canterbury, 2010, 151.

Kong, C., and Kowalsky, M.J. (2016). *Impact of Damping Scaling Factors on Direct Displacement-Based Design*, Earthquake Spectra, May 2016, Vol. 32 (2), 843-859.

Kwong, N.S. and Chopra, A.K. (2015). *Selection and Scaling of Ground Motions for Nonlinear Response History Analysis of Buildings in Performance-Based Earthquake Engineering*, PEER 2015/11 Report.

Mahaney, J.A., Paret, T.F., Kehoe, B.E. and Freeman, S.A. (1993). *The Capacity Spectrum Method for Evaluating Structural Response During the Loma Prieta Earthquake*, Proceedings of the 1993 National Earthquake Conference, Earthquake Hazard Reduction in the Central and Eastern United States: A Time for Examination and Action, , Memphis, Tennessee, 2-5 May 1993, Vol. II, 1993.

Millen, M.D.L., Pampanin, S., Cubrinovski, M. and Carr, A. (2016). *A performance assessment procedure for existing buildings considering foundation deformations*, Proceedings of the 2016 New Zealand Society for Earthquake Engineering Conference, Christchurch, 1-3 April 2016, Paper No. O-02. http://www.nzsee.org.nz/db/2016/Papers/O-02%20Millen.pdf.

Moehle, J. P., Hooper, J. D., Kelly, D. J. and Meyer T. R. (2010). *Seismic design of cast-in-place concrete diaphragms, chords, and collectors*, NEHRP Seismic Design Technical Brief No. 3., NIST GCR 10-917-4, National Institute of Standards and Technology, Gaithersburg, MD.

Oliver, S.J., Boys, A.G. and Marriot, D.J. (2012) *Nonlinear Analysis Acceptance Criteria for the Seismic Performance of Existing Reinforced Concrete Buildings*, Proceedings of the 2012 New Zealand Society for Earthquake Engineering Conference, Christchurch, NZ 13-15 April 2015, Paper No. 42. 57 pages.

Pennucci, D., Sullivan, T.J. and Calvi, G.M. (2011). *Displacement Reduction Factors for the Design of Medium and Long Period Structures*, Journal of Earthquake Engineering, Vol. 15, Supplement 1, 1-29.

Rezaeian, S., Bozorgnia, Y., Idriss, I.M., Abrahamson, N.A., Campbell, K.W. and Silva, W.J. (2014). *Damping scaling factors for vertical elastic response spectra for shallow crustal earthquakes in active tectonic regions: "average" horizontal component*, Earthquake Spectra Vol. 30, 939-963.

Shibata, A. and Sozen, M.A. (1976). *Substitute structure method for seismic design of reinforced concrete*, Journal of the Structural Engineering, American Society of Civil Engineers 102(1), 1-18.

Appendix C2A: Simple Lateral Mechanism Analysis (SLaMA)

C2A.1 General

The SLaMA is a simple nonlinear analysis technique that provides an estimate of the global probable capacity of the primary lateral structure of the building as the summation of the probable capacities of the identified individual mechanism/systems. The capacities of the individual mechanisms/systems are typically represented in elasto-plastic form (although post yield stiffness and strength changes can be incorporated) with strength and maximum deformation equal to the assessed probable strength and maximum deformation capacity respectively.

The SLaMA is considered to be a relatively easy way of obtaining an estimate of the nonlinear pushover relationship (strength vs deformation) of reasonably complex structures comprising multiple nonlinear systems of varying ductile capacity. For this reason SLaMA is recommended as the first step in all of the assessment procedures presented in these guidelines.

Although SLaMA is a simplistic process it provides the engineer with the opportunity of observing the contribution that each individual member/element/system has on the capacity of the whole system. Often, the clarity of the simplistic representation will prove more useful in understanding the seismic behaviour of the building than more sophisticated analyses, where the available detail may cloud the individual aspects of the behaviour.

As a SLaMA will not typically allow incorporation of torsional effects these need to be addressed using other techniques.

The steps for completing a SLaMA are outlined in this appendix.

C2A.2 Key Steps

The key steps for a SLaMA are:

Step 1 Assess the structural configuration and load paths to identify key structural elements, potential structural weaknesses (SWs) and severe structural weaknesses (SSWs).

Step 2 Calculate the relevant probable strength and deformation capacities for the individual members.

Step 3 Determine probable inelastic behaviour of elements by comparing probable member capacities and evaluating the hierarchy of strength.

Step 4 Assess the sub-system inelastic mechanisms by extending local to global behaviour.

Step 5 Form a view of the potential governing mechanism for the global building by combining the various individual mechanisms and calculate the probable base shear and global displacement capacity measured at the top of the primary lateral structure. The global displacement capacity will typically be limited to that for the system with the lowest displacement capacity.

Step 6 Determine equivalent SDOF system, seismic demand and *%NBS*.

Step 1 Assess the structural configuration and load paths to identify key structural elements, potential SWs and SSWs

Review the structural drawings and collected as-built structural data thoroughly to understand the structural configuration and lateral load paths.

Separate out the structural members and elements that are part of the primary lateral load resisting system and those that are part of the primary gravity load resisting system. Gain an understanding of when these systems are combined and when they are separate. The primary gravity systems, when not part of the primary lateral system, need to be assessed to ensure they can continue to "go along for the ride" with the primary lateral system.

Step 2 Calculate the probable strength and deformation capacities for individual members

Calculate the relevant probable strength and deformation capacities for individual members with reference to the material sections of these guidelines (Sections C5 to C9) or relevant literature (e.g. EAG, 2012). For example, for members within reinforced concrete moment resisting frames it would be necessary to calculate the flexural and shear capacities for the beams and columns, joint shear capacities and anchorage/lap-splice capacities, if applicable.

Note:

The devil is in the detail! The seismic behaviour of a non-ductile structural system is often governed by the detailing and failure mechanisms not considered by either the original designer or by the code/standard of the day.

In many cases, the absolute strength capacity of the structural member is not necessarily critical. The ability to respond nonlinearly in a ductile manner (i.e. having sufficient deformation and ductility capacity) is more important as it allows load redistribution and mobilisation of other structural elements within the system.

While progress has been made in providing quantitative procedures to calculate the deformation capacity of various non-ductile mechanisms, in many cases the assessment of the achievable local ductility is qualitative in nature and requires significant engineering judgement and understanding of the basis of the detailing requirements in the current standard.

For example, the transverse reinforcement detailing for reinforced concrete columns plays a significant role in their ductile capacity. Some buildings have column tie detailing that does not even satisfy the minimum requirement of transverse steel reinforcement or maximum tie spacing. Engineers need to apply necessary judgement to the quantitative procedures set out in the material sections of these guidelines to estimate the achievable inelastic deformation capacities.

It is often easier and more informative to evaluate the general capacity relationship for a member and then superimpose other deformation limiting issues over the top of the general relationship (e.g. the limiting effect of a reducing shear capacity with increasing ductile flexural behaviour in members within a ductile concrete moment resisting frame). This allows the effect of each aspect on the capacity of the member to be readily observed and has the additional benefit of clearly indicating the effect of undertaking retrofit to address individual aspects.

The capacity of individual URM wall members (piers and spandrels) requires consideration of each of the behavioural modes described in Section C8.

Step 3 Determine probable inelastic behaviour of elements by comparing probable member capacities and evaluating the hierarchy of strength

Determine the potential inelastic behaviour of each element in the critical bracing line by checking the hierarchy of strength of the interconnected members/components.

Figure C2A.1shows an example of the hierarchy of strength assessment of a reinforced concrete beam-column joint. The capacities of the individual elements of beams, columns and joints are assessed separately before an equivalent comparison is made to identify the governing inelastic mechanism within the beam-column joint subassembly.

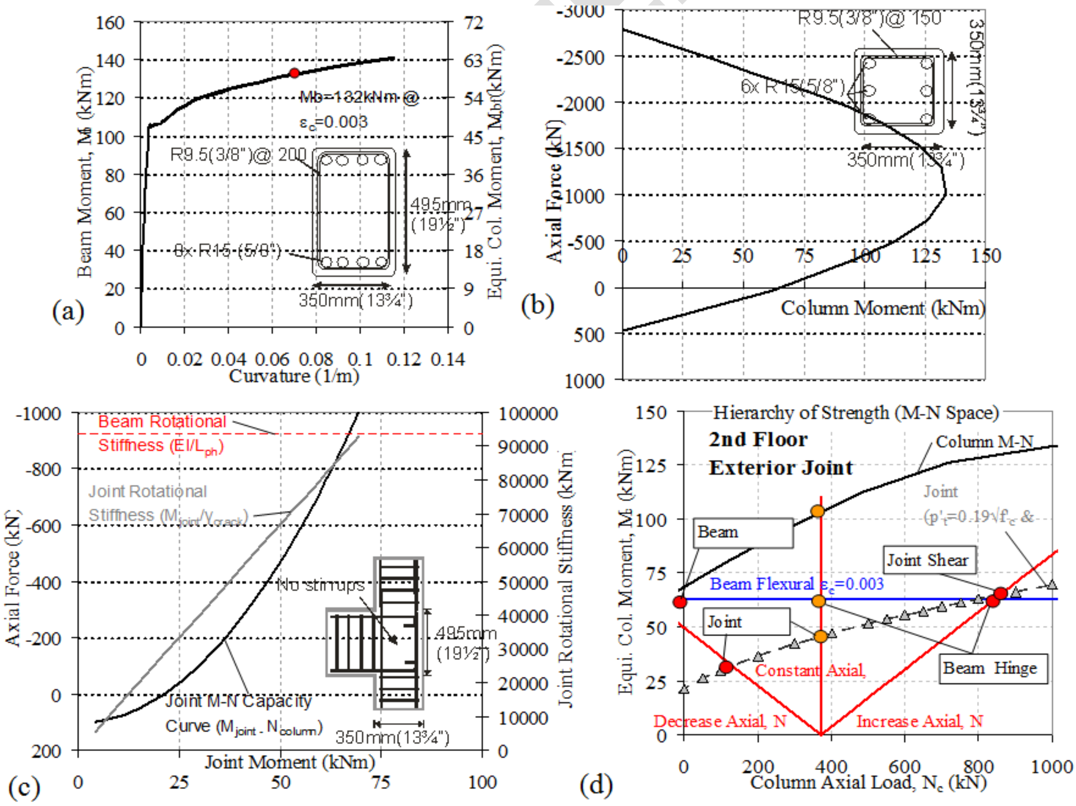


Figure C2A.1: Example of evaluation of element capacity and hierarchy of strength for a non-ductile exterior beam-column joint element as part of a reinforced concrete frame system (Kam, 2010)

For URM wall sub-systems determine the hierarchical behaviour of spandrels and piers. For example, does rocking of the piers between openings occur before the capacity of the spandrels is reached?

Step 4 Assess the sub-system inelastic mechanisms by extending local to global behaviour

Establish the relationship between the local and the global behaviour based on some assumptions of deformed shape profiles and ability to redistribute forces after the formation of “hinges” to determine the limiting mechanism and probable strength and deflection capacity (measured at the top) of each subsystem. The method will depend on the structural configuration and the identified local mechanism:

- For moment resisting frames use the Sway Index (refer to note below) to investigate the likely hierarchy of plastic hinge formation. The inelastic deformed shape profile will depend on the hierarchy of plastic hinge formation as shown in Figure C2A.2.

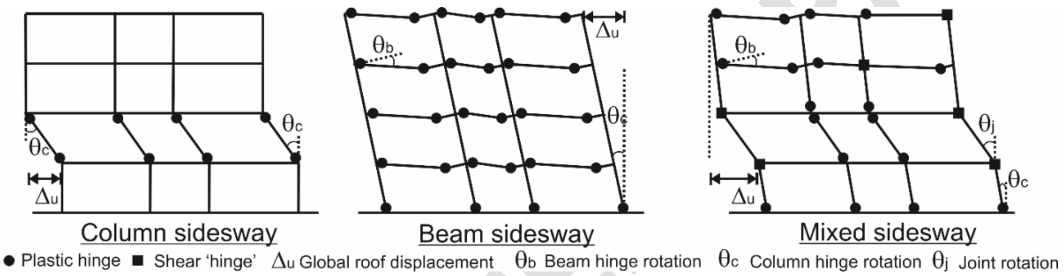


Figure C2A.2: Possible mechanisms of post-elastic deformation of moment resisting frames

Note: Potential inelastic mechanisms in moment frames using Sway Index

At this stage in the analysis, it is important to identify the probable location of post-elastic deformations due to severe earthquake forces and hence to determine the critical mechanism of post-elastic deformation.

This will involve determining whether flexural plastic hinges occur in the beams or the columns at each beam-column joint and/or whether shear failure occurs in the members or joints. The imposed shear forces on members should be those associated with the plastic hinge (flexural) mechanism. The imposed horizontal shear forces on beam-column joint cores should be those associated with the adjacent plastic hinges. The horizontal joint shear force is given conventionally by the sum of the tensile forces in the top and bottom longitudinal beam reinforcement minus the column shear force. Comparisons of these calculated imposed shear forces and the expected shear strengths will determine whether or not shear failures occur before the flexural strengths are reached.

To assess the likely inelastic mechanism the Sway Index, S_{i}, is used. S_{i} compares the overstrength beam flexural capacity to the probable column flexural capacity at the beam-column joint:

$$S_{\mathrm{i}} = \frac{\sum(M_{\mathrm{bl}}+M_{\mathrm{br}})}{\sum(M_{\mathrm{ca}}+M_{\mathrm{cb}})} \qquad \text{…C2A.1}$$

where:

M_{bl}, and M_{br} = beam expected maximum flexural strengths at the left and right of the joint, respectively, at the joint centroid

M_{ca} and M_{cb} = minimum expected column flexural strengths above and below the joint, respectively, at the centroid of the joint.

These are summed for all the joints in the same line at that horizontal level.

The lateral seismic force capacity associated with the critical mechanism of post-elastic deformation can then be calculated.

For a building frame, the critical mechanism is often not simply a beam sidesway mechanism or a column sidesway mechanism (see Figure C2A. above), but is a mixed mechanism involving flexural plastic hinges at some locations combined with shear failures of members and/or joints at other locations.

When $S_{\mathrm{i}} > 1$, column plastic hinges may be expected to form (Sullivan and Calvi, 2011). However, to include the effects of higher modes of vibration, and a possible overestimation of column flexural strength, it is suggested (Priestley, 1996) that column plastic hinges are assumed to form if $S_{\mathrm{i}} > 0.85$. Accordingly, the dynamic magnification factor, ω_{v}, does not need to be applied in this procedure.

The use of the dynamic magnification factor in the capacity design of new columns is intended to significantly reduce the possibility of column hinge formation. Less conservative measures are appropriate if individual column hinging can be accepted, provided that a full storey column sidesway mechanism does not develop.

- For cantilevered wall systems it can generally be assumed the capacity of the base will govern. For mid to high-rise wall buildings it will be necessary to assess the shear demand at the upper levels as well, as this can be amplified due to higher mode responses and the termination of shear reinforcement with height will not always match the demand requirements to ensure the base governs.
- The various mechanisms for penetrated URM walls can be considered in a similar fashion to that outlined for frames above.

It is important to assess the whole load path as some mechanisms may be limited by another more deficient member/element. For example, the connection capacity often governs the overall lateral load capacity of steel braced frame systems designed pre-1980s. Before the introduction of modern capacity design philosophies for steel structures the connections would have rarely been designed to yield the braces.

Some specific mechanisms can be discounted by inspection based on past experience and understanding of typically observed mechanisms.

Note:

Engineers need to consider whether or not the item/aspect identified as limiting the capacity of a mechanism is likely to present a significant life safety hazard if the capacity of the particular aspect was exceeded (i.e. would this cause loss of gravity load support?). If it is determined that it would not, that aspect is not necessarily material to the assessment and the probable global capacity of the building. Engineers should either remove it from the analysis or reduce its capacity to its residual value and repeat the analysis of the mechanism.

The potential displacement at every level is:

$$\Delta_i = (\Delta_{elastic})_i + (\Delta_{plastic})_i \qquad \text{...C2A.2}$$

where:

$(\Delta_{elastic})_i$ = elastic displacement at level i $\leq (\Delta_y)_i$

$(\Delta_{plastic})_i$ = plastic displacement at level i.

The elastic component of the displacement capacity can be significant for flexible structures and should be accounted for.

The elastic component of the displacement capacity may also be important in regions of low seismicity. For example, for steel portal frames or light timber frames, Δ_{prob} may be less than the yield displacement of the system, indicating the system will remain elastic.

Step 5 Form a view of potential governing mechanism and calculate probable base shear and global displacement capacity

Having identified the mechanisms for the various sub-systems, the next step is to determine how these mechanisms work together to provide the global inelastic mechanism for the building.

It is intended that this step is done by hand and follows the following procedure for a particular considered direction:

- Determine the lowest available deformation capacity of any of the linked sub-systems. This is the available probable global lateral deformation capacity, $(\Delta_{prob})_{top}$.
- Determine the probable base shear capacity of each sub-system at the global deformation capacity determined above and add. The sum is the probable global base shear capacity, V_{prob}.

Note:

It will be apparent that the procedure outlined above requires the deflection of each sub-system to be determined at the same level, the top of the primary lateral structure. It is possible that some sub-systems will not extend to through to this level. In such cases the deflections at the top of the sub-system can be assumed to extend through to the top of the structure.

Figure C2A. illustrates a combination of probable force-displacement capacity curves of a dual system (refer to Section C5 for further description of this). Table C2A.1 below illustrates some examples of the derivation of global overturning and lateral base shear capacities for different global systems.

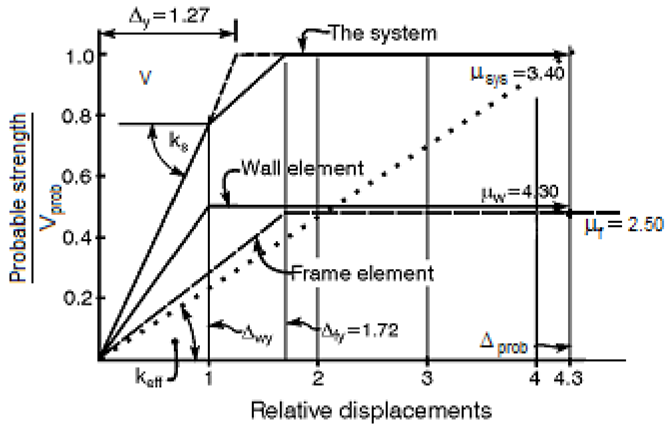


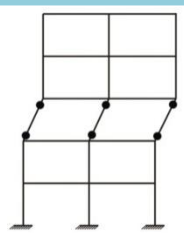
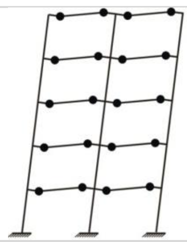
Figure C2A.3: Combination of force-displacement probable capacity curves of a dual system

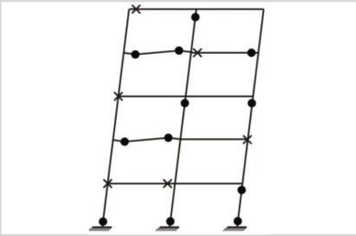
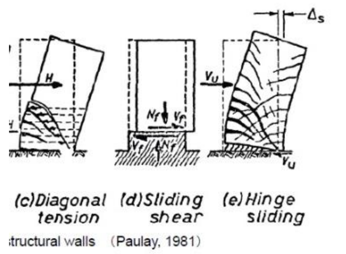
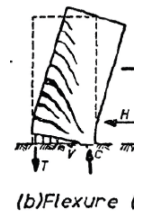
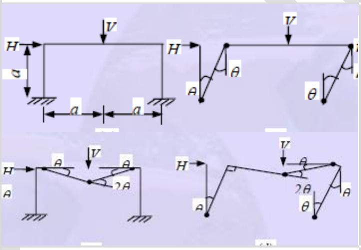
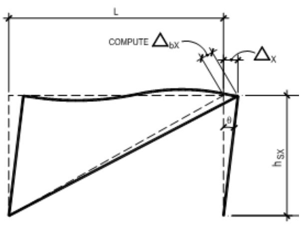
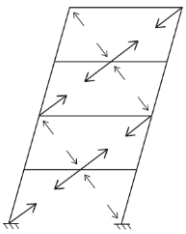
Note:

One of the weaknesses of SLaMA is the potential for overestimating the global capacity by missing the mechanism that has a lower strength and displacement capacity. In some ways, SLaMA has the same underlying principles as the plastic method for steel design, where the lower bound plastic mechanism is used to estimate the upper bound global capacity.

Therefore it is very important that the correct inelastic or "collapse" mechanism is identified so the lateral force capacity is not overestimated. The mechanism that has given the least lateral force capacity is the correct one and must be sought.

Table C2A.1: Calculation of lateral load capacity based on the mechanism

System	Mechanism		Calculation/equation
Frame (steel or concrete)	**Soft storey – column sway** • Concrete: column flexural, lap splice or shear failure • Steel: axial-flexural buckling, web buckling • URM: pier mechanism		$OTM = \sum_{i} M_{\text{coli}}$ $V_{\text{b}} = \frac{OTM}{h_{\text{eff,col sidesway}}}$
	Beam-sway – distributed • Concrete: beam hinging, joint hinging • URM: spandrel mechanism		$OTM = \sum_{i} M_{\text{coli}} + \left(\sum_{n} V_{\text{end beam.n}}\right) L$ $V_{\text{b}} = \frac{OTM}{h_{\text{eff,beam sidesway}}}$

	Mixed mechanism		$OTM = \sum_i M_{\mathrm{col}i} + \left(\sum_x V_{\mathrm{end\ beam.x}}\right) L$ $V_{\mathrm{b}} = \dfrac{OTM}{h_{\mathrm{eff,mixed\ sidesway}}}$
Wall	**Brittle mechanism** • Global overturning/ instability • Concrete: shear failure , global buckling		$V_{\mathrm{prob}} = 0.85V_{\mathrm{shear}}$ where: $0.85V_{\mathrm{shear}}$ = probable base shear capacity corresponding to the brittle mechanism
	Ductile/limited ductile mechanism • Concrete: flexural yielding, rocking/uplift • Timber: nail/ply shear		$V_{\mathrm{prob}} = 1.5\Sigma M_{\mathrm{wp}}/h_{\mathrm{w}}$ where: h_{w} = height of the walls, which is assumed here to be the same as the height of the building
Steel Portal Frame	**Portal frame mechanisms**		Refer to Section C6 for steel
Braced Frame	**Concentrated brace failure**		Refer to Section C6 for steel
	Distribute brace failure		Refer to Section C6 for steel
URM	**Refer to Section C8 for unreinforced masonry**		

Step 5 provides μ_{sys} , $(\Delta_{\mathrm{y}})_{\mathrm{top}}$ and $(\Delta_{\mathrm{prob}})_{\mathrm{top}}$.

Step 6 Determine equivalent SDOF system, seismic demand and *%NBS*

The procedure for this step is described in Section C2.4.2. It is completed for each direction and can be summarised as follows:

- For each sub-system estimate the relationship between $\left(\Delta_{\mathrm{y}}\right)_{\mathrm{top}}$ and $\left(\Delta_{\mathrm{prob}}\right)_{\mathrm{top}}$ (found from Step 5) and the displacement over the height of the structure from the assumed lateral load distribution with height.
- Estimate the effective height for the equivalent SDOF oscillator for the structure as a whole and determine the simplistic pushover force displacement curve at the effective height. The effective mass, m_{eff}, can be taken as the total mass of the structure. The probable strength is taken as V_{prob}, and the probable displacement capacity, Δ_{prob}, is taken as the lowest value of displacement for all sub-systems calculated at the effective height for the structure as a whole.
- Plot the point $(\Delta_{\mathrm{prob}}, V_{\mathrm{prob}}/m_{\mathrm{eff}})$ over the ADRS curve for 100%ULS shaking and the system viscous damping taken (Priestley et al, 2007) as the weighted average (based on the probable shear capacity) of the sub-system effective viscous damping values, i.e.:

$$\xi_{\mathrm{sys}} = \frac{\Sigma(V_{\mathrm{base}})_{\mathrm{i}}\xi_{\mathrm{i}}}{\Sigma(V_{\mathrm{base}})_{\mathrm{i}}} \qquad \text{...C2A.3}$$

where $(V_{\mathrm{base}})_{\mathrm{i}}$ and ξ_{i} are the lateral shear capacity and effective viscous damping for each sub-system i.

- Extend a line from the origin through the point $\Delta_{\mathrm{prob}}, V_{\mathrm{prob}}/m_{\mathrm{eff}}$ to intersect with the ADRS curve.
- The *%NBS* earthquake score based on SLaMA is the ratio of the spectral displacement at the intersection with the ADRS curve, Δ_{ULS}, and Δ_{prob}.

Appendix C2B: Assessment of Seismic Pounding

C2B.1 Introduction

This appendix provides discussion and guidance on:

- general observations on seismic pounding (Section C2B.2)
- an overall approach to assessment (Section C2B.3)
- qualitative screening for the potential for seismic pounding with significant consequences for a building's seismic performance (Section C2B.4), and
- quantitative assessment for various building configurations (Section C2B.5).

It also lists some alternative mitigation (retrofit) approaches (Section C2B.6).

C2B.2 General Observations

Older buildings have often been built up to property boundary lines, with little or no separation from adjacent buildings. As a result, buildings with inadequate separation may impact on each other or pound during an earthquake. Such impacts will transmit short duration, high amplitude forces to the impacting buildings at any level where pounding occurs. This has the following consequential effects:

- High accelerations within the building in the form of short duration spikes.
- Modification to the dynamic response of the impacting buildings, the pattern and magnitude of inertial demands, and deformations induced on both structures. Response may be amplified or de-amplified and is dependent on the relative dynamic characteristics of the buildings including their relative heights, masses and stiffness, as well as ground conditions that may give rise to soil-structure interaction and the magnitude and direction of travel of the earthquake motions.
- Local degradation of strength and/or stiffness of impacting members.

Numerous pounding damage surveys and numerical and analytical pounding studies have been undertaken in recent years, especially after the 1985 Mexico City earthquake (Bertero, 1986) which caused an unusually large number of building failures. In the 2011 Lyttelton (Canterbury) earthquake, seismic pounding was also observed to cause significant damage in a number of URM buildings (Cole et al., 2011).

It is clear that pounding is a complex problem and can occur in a number of circumstances. The results of studies undertaken to date are sensitive to the many parameters related to the building structures (and their numerical modelling) in addition to the prevalent soil conditions and the characteristics and direction of seismic attack. However, based on these studies and evidence from past earthquakes, it is possible to draw the following general conclusions:

- Where buildings are significantly different in height, period and mass, large increases in response from pounding can be expected.
- Differences in height, particularly between neighbouring buildings, can result in significant pounding effects and produce large response increases in the upper part of the taller building (refer to Figure C2B.1(a)). The shears in the impact-side columns for the taller building can be up to 50-70% higher than in the no-pounding case at the levels

immediately above the lower building, and 25-30% at levels higher up. This is because the shorter building acts as a buttress to the taller building. In soft ground conditions where soil-structure interaction and through-soil coupling occurs, the impact-side shears can be enhanced by a further 25-50%.

- For buildings of similar height, mass and stiffness, in most cases the effects of pounding will be limited to some local damage (mostly non-structural and nominally structural), and to higher in-building accelerations in the form of short duration spikes. In such conditions, from a practical viewpoint the effects of pounding on global responses can be considered insignificant.
- Where building floors are at different elevations, the floor slabs of one structure can impact at the mid-storey of the columns of the others, damaging the columns and initiating partial or total collapse (refer to Figure C2B.1(b)). Buildings that are particularly susceptible to such action are those overtopping a shorter neighbouring building whose columns may be impacted at mid-storey by the uppermost level of the shorter building.
- The local high amplitude and short duration accelerations induced by colliding buildings will increase the anchoring requirements for the contents of the buildings as well as for architectural elements.

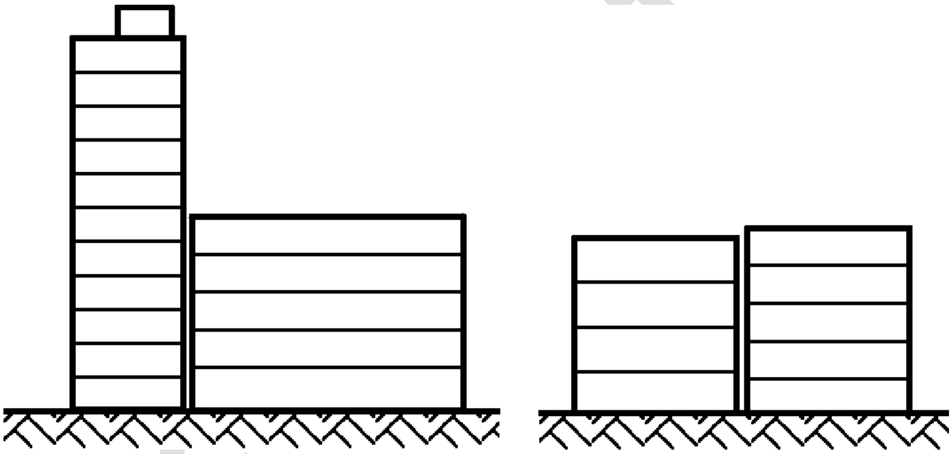

(a) Buildings of unequal height (b) Buildings of equal height

Figure C2B.1: Example of differing floor elevations in adjacent buildings

Note:

When adjacent buildings are of similar height and mass and have matching or similar floor levels, it is not expected that engineers need to account for the effects of pounding, irrespective of the provided separation clearances. The exception is if a building is on the end of a row of buildings without separation (as per item 4 in Table C2B.1).

Similarly, experience from past earthquakes has indicated that solid boundary walls can mitigate the effects of seismic pounding between two buildings with similar stiffness and mass (Anagnostopoulos and Karamaneas, 2008; Kam et al., 2011).

C2B.3 Overall Approach

When pounding is found to be a potential issue it is recommended that the building is first assessed by assuming that pounding does not occur. The next step is to consider any mitigating effects as outlined in Section C2B.4, and then to quantify any remaining issues in accordance with the recommendations in Section C2B.5.

Note:

The quantification of the effects of impact due to pounding is very difficult and is associated with considerable uncertainty. Adjacent buildings of different height and local effects can be scored as outlined in Section C2B.5, but precision should not be assumed.

C2B.4 Screening for Potential for Consequential Seismic Pounding

While seismic pounding between two adjacent buildings in earthquakes is a complex physical phenomenon, it is generally accepted that its effects are more critical for some building configurations than for others. It is also recognised that, in many cases, seismic pounding may only result in localised damage and that the likelihood of pounding is subject to the complex dynamic phasing of two separate structures in an earthquake.

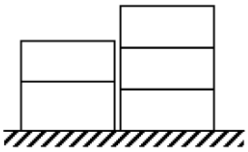
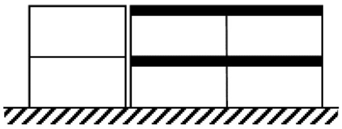
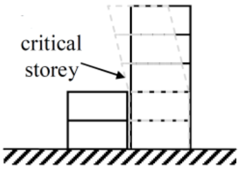
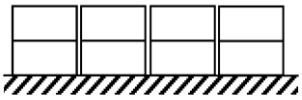
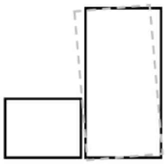
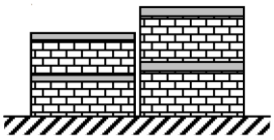
Damage to buildings from seismic pounding can be divided into two categories:

- **local damage** (damage resulting from the magnitude of the force applied during physical contact), and
- **global damage** (damage due to the change in dynamic building properties resulting from momentum transfer during collision). Global damage can increase the lateral response of a stiffer building while reducing the lateral response of a more flexible building, when compared to a standalone structure not affected by pounding.

Local and global damage effects are found to be fundamentally different consequences of collision, with the two categories responding differently to changes in the modelled system.

From observations of earthquake damage, six key building configurations have been identified as vulnerable to seismic pounding (Jeng and Tzeng, 2000; Cole et al., 2011; Kasai et al., 1992). Table C2B.1 describes these configurations and includes notes on their assessment (covered in more detail in the following Section C2B.5).

Table C2B.1: Evaluation of potential pounding vulnerabilities

Scenario		Illustration	Comment	Assessment
1	Column-to-floor		Columns resisting the floor collision are subject to very high shear forces (Karayannis and Favvata, 2005).	Refer scenario 1 in Sections C2B.5.2 and C2B.5.3
2	Floor-to-floor with greatly different masses		The lighter building is more susceptible to collapse. If the lighter building can sustain the imposed drift demand (e.g. a timber building) the pounding effects may be negligible.	Refer scenario 2 in Section C2B.5.2
3	Different building heights		An increase in shear and ductility demands is expected in the taller building in the storey immediately above the top floor of the shorter building.	Refer scenario 3 in Sections C2B.5.2 and C2B.5.3
4	Row of buildings without separation (URM only)		The end building suffers increased damage due to the momentum transfer from the interior buildings, in particular for URM buildings.	For URM only – refer to Section C8 Unreinforced Masonry Buildings *No assessment required for other construction systems*
5	Plan irregularity and pounding		Building configurations can excite torsional response which can lead to amplified local demand.	Depending on the relative stiffness of the buildings, bound the analysis (e.g. assume the long building is being propped by the square building).
6	Pounding of brittle materials, i.e. URM		URM buildings are very susceptible to pounding, which results in amplified lateral demands to the adjacent building. Refer to Cole et al. (2011).	This is generally only critical for the front and rear façade elevations of URM buildings. Refer to Section C8 Unreinforced Masonry Buildings
Note: Figures are adopted from Cole et al. (2011).				

The effects of seismic pounding need to be considered when both of the following criteria apply:

- **Any** of the following conditions exist:
  - adjacent buildings are of different heights and the height difference exceeds two storeys or 20% of the height of the taller building, whichever is the greater, or
  - floor elevations of adjacent buildings differ by more than 20% of the storey height of either building, or

- no boundary reinforced concrete walls are present that would allow transmission and distribution of the localised pounding forces

AND

- Separation between an adjacent building at any height is less than a distance given by:

$$S = \sqrt{\delta_1^2 + \delta_2^2} \quad \text{...C2B.1}$$

where:

δ_1 = estimated lateral deflection of Building 1 relative to ground under the loads used for the assessment

δ_2 = estimated lateral deflection of Building 2 relative to ground under *two-thirds* of the loads used in the assessment.

However, the value of S calculated above does not need to exceed 0.03 times the height of the building at the possible point(s) of impact.

The engineer should calculate S assuming that the building being assessed can be either Building 1 or Building 2.

Note:

The potential or likelihood of pounding needs to be evaluated using calculated drifts for both buildings. The Square Root Sums of Squares (SRSS) combination of structural lateral deflections of both buildings is proposed, as adopted in ASCE 41-17 (2017), to check the adequacy of building separation. This approach has been adopted to account for the low probability of maximum drifts occurring simultaneously in both buildings while they respond completely out of phase. It is not intended for detailed analysis or modelling to be undertaken to determine building drifts; rather, that general estimates are used.

C2B.5 Quantitative Assessment of Pounding Effects

C2B.5.1 Recommended approaches

The effects of pounding effects can be considered using either:

- simplified checks, or
- an approximate approach, or
- a detailed analytical approach.

Note:

Analytical methods have been proposed for assessing the effects of pounding, including time history analyses and elastic response spectrum analyses (Kasai et al., 1990; Cole et al., 2010). However, the use of such approaches may prove impractical for many buildings or may not be within the capability of many design practitioners (Cole et al., 2010).

An alternative simplified approach has been proposed, based on simple factoring of earthquake design forces applicable to the building, to ensure some account of pounding effects is made. Both moment/shear capacities and P-delta effects need to be considered. A number of studies (Kasai et al., 1990; Jeng and Tzeng, 2000; Carr and Moss, 1994; Karayannis and Favvata, 2005) have shown that column and storey shears in the taller building above the pounding level can be increased by anywhere up to or exceeding 100%. The level of increase is dependent on many factors including initial separation distances and the relative mass and stiffness of the adjacent buildings. A mid-range increase in design shear has been adopted for the simplified approach at this stage.

While it is recognised that this approximate approach is relatively crude, it has the benefit of being easy to apply and does not need the use of, or familiarity with, sophisticated analysis tools. As further research on seismic pounding is undertaken, it is expected that more appropriate and practical means to evaluate and mitigate pounding will become available.

Irrespective of the approach adopted, the *%NBS* score determined for pounding will be based on the %ULS shaking that leads to a significant life safety hazard due to a loss of gravity support (based on probable member/element capacities).

C2B.5.2 Simplified checks

Simplified checks can be performed to estimate the upper and lower bound responses if seismic pounding occurs. Some examples follow. Scenario numbers correspond to Table C2B.1.

Scenario 1 – Misaligned floors and column-to-floor pounding

Assume the columns in collision with the floor have failed in shear. Check if the gravity load path is maintained by the secondary load path (e.g. floor beams or slab cantilevered back to the building or boundary walls exist). If a reliable scenario load path is available (either existing or through seismic retrofit), no further assessment is required.

Scenario 2 – Aligned floors but with mass difference

Assume the stiffer building will "prop" the more flexible building. Assess the stiffer building with 20% or more seismic mass from the adjacent building. If the lighter/less stiff building does not have a rigid diaphragm, the additional seismic inertia to be resisted by the stiffer building can be estimated based on the tributary area.

Scenario 3 – Aligned floors but with building height difference

Carry out an initial assessment of the taller building by assuming its building height is truncated by the shorter building (which would decrease its fundamental period and, therefore, increase its seismic loading). If the shorter building is of concern, assess this against a 20% storey shear from the adjacent building applied at the point of impact.

C2B.5.3 Approximate approach

Scenario 1 – Misaligned floors and column-to-floor pounding

If the floor elevations of adjacent buildings differ and there is potential for mid-storey hammering of each building, the impact-side columns of the building(s) which may be impacted between storeys should have sufficient strength to resist design actions resulting from imposition of a displacement on the columns, at the point of impact, corresponding to one half of the value of S derived from Equation C2B.1 in Section C2B.4.

The imposed displacements only need to be applied at any one level. However, critical design actions should be derived considering application of the imposed displacements at any level over the building height where impact could occur.

In addition, if the buildings are of unequal heights, in accordance with Section C2B.4 the requirements of Scenario 3 below also apply.

Scenario 3 – Aligned floors but with building height difference

If two buildings are of unequal height but their floor elevations align, the impact-side columns of the taller building should have sufficient strength to resist the following design actions:

- 175% of the column design actions (shear, flexural and axial) occurring in the taller building under the application of the seismic lateral loading in accordance with Section C3, assuming the building is free standing, applied above the height of the shorter building
- 125% of the column design actions occurring in the taller building under the application of the seismic lateral loading in accordance with Section C3, assuming the building is free standing, applied over the height of the shorter building, and
- all other columns remote from the building side suffering impact should have sufficient strength to resist 115% of the column design actions occurring under the application of the seismic lateral loading in accordance with Section C3, assuming the building is free standing, over the full height of the building.

C2B.5.4 Detailed analytical approach

Detailed modelling of the seismic pounding phenomena requires consideration of the transfer of momentum and energy between the buildings as they impact, both in terms of local contact damage and of global building response changes. Possible approaches for a variety of pounding situations and varying levels of model detail are available in e.g. Cole et al., 2011; and Khatiwada et al., 2011.

NLTHA with simplified mass and stiffness and appropriate contact elements appears to be the only appropriate detailed quantitative assessment of pounding between two buildings (Cole et al., 2010).

These guidelines recommend the simplified and approximate checks as outlined in preceding sections in preference to a detailed analytical approach. The limitations of NLTHA and pounding modelling mean this method is not necessarily viable for practitioners.

C2B.6 Mitigation

In some circumstances, rather than carrying out a complex analysis of the seismic pounding phenomenon it may be more cost effective to accept the seismic pounding risks and undertake steps to mitigate its effects.

Retrofit options to mitigate the risk of seismic pounding include:

- tieing adjacent buildings together. This approach may prove practical for a row or block of buildings of similar height and configuration
- providing additional structural members/elements away from the points of impact to compensate for/replace members/elements that may be severely damaged due to impact
- improving individual buildings to reduce displacement or increase resilience to pounding and additional seismic inertia from the adjacent building
- providing robust boundary shear walls to act as buffer elements to protect the rest of the building (Anagnostopoulas and Karamaneas, 2008). The use of collision shear walls would prevent mid-storey impact to columns of adjacent buildings, reducing potential for local damage and partial or total collapse, and/or
- linking adjacent buildings with energy dissipating devices to reduce the severity of pounding and collisions (ULIEGE, 2007).

Appendix C2C: Nonlinear Time History Analysis

C2C.1 General

Nonlinear Time History Analysis (NLTHA) or Response Analysis is a highly specialised analysis technique that provides a real time "snapshot" prediction of the seismic response of a building under earthquake actions. It is particularly important as an investigative tool to improve the understanding of "what happened": i.e. the overall nonlinear mechanism trend and mean responses.

Advanced and sophisticated analyses such as NLTHA are useful in understanding the nonlinear and dynamic behaviour of the building. However, they require significant effort and engineering judgement to ensure the validity of the outputs. While the accuracy may have increased with the use of complex and sophisticated analysis, the uncertainties, precision and reliability remain a function of the level of checking and rigour of the analysis (number of runs, sensitivity analysis and well-defined analysis parameters).

Note:

A number of guidance documents have been published on the use of NLTHA for seismic assessment (e.g. Deierlein et al., 2010; ASCE 41-17, 2017; FEMA 440, 2005; ATC 72, 2010). There are also a number of software programs for NLTHA that are now commercially available.

It is important to recognise that any NLTHA output is only a representation of the building's response to one particular earthquake record and is highly dependent on the ability to model the nonlinear element behaviour adequately. The performance in an actual earthquake is contingent on a number of other variables that may or may not be modelled (NIST, 2013).

Note:

Engineers should resist the temptation to believe that NLTHAs reliably predict the performance of a building in a particular earthquake. The whole assessment approach is based around rating a building's performance against that of a similar new building. Therefore, care should be taken not to overcomplicate a NLTHA in the pursuit of unattainable accuracy; especially if loss of clarity of the behavioural issues in the building is the result.

When running the analyses, care should be taken to ensure that the element that is limiting the level of shaking that can be sustained does represent a significant life safety hazard. If it is determined that the critical element does not then the level of shaking should be increased until the critical member/element is considered to represent a significant life safety hazard. URM spandrels supported on lintels, for example, may be damaged to the extent that they can no longer participate in the lateral load resistance of the building but may have a low risk of collapse until the actions are fully redistributed in the building and the deformations become high enough that collapse is expected. This is because the structure reconfigures the way in which it is resisting the shaking until the spandrels sustain a level of deformation beyond which they can no longer be assumed to remain reliably in place. It is only at this point that a significant life safety hazard develops.

Ignoring this ability to redistribute could result in a *%NBS* rating significantly less than intended by the methods set out elsewhere in these guidelines.

NZS 1170.5:2004 specifically requires shear deformations to be included in the modelling for time history analyses but also has provisions for new structures that are intended to reliably prevent the shear capacity of the primary structure from being exceeded. These same prevention methods are not always available in existing buildings, especially when traditional capacity design methods were not used in their original design. Accordingly, for shear dominated systems the modelling of shear deformations when assessing an existing building needs to be approached with care, especially when these effects are beneficial (i.e. limit the shear actions). These concerns are not present when the system is flexurally dominated. Therefore, when any shear critical primary seismic structural elements are present in significant buildings (i.e. with more than six storeys) it is recommended that nonlinear shear behaviour is not relied on to limit the actions in the building for the purposes of the assessment unless a deformation margin (to that required to reach the maximum predicted shear (strength) capacity of these elements) of at least two can be shown to be present. This behaviour can be treated similarly to an SSW. An alternative may be to model the shear as linear elastic in such situations or treat it as force controlled in applying ASCE 41.

NLTHAs should never be approached on the basis that they provide a “black box” assessment procedure. These are highly sophisticated analyses requiring particular skills and experience. While the results of such analyses may not be replicated by more simple methods it is not considered appropriate to complete such analyses in the absence of at least some confirmatory analyses at a more basic level. Simply relying on the NLTHA to deliver a *%NBS* without careful consideration of the results and the behaviour they imply is considered inappropriate. Peer review, including the modelling, the analysis and the calculation of *%NBS*, is considered essential when such methods are used.

It is recommended that an NLTHA should not be the sole analysis technique used for a structural assessment but should be supported by the results of simplified approaches. This is for the following reasons:

- Individual results from individual runs are highly dependent on the characteristics of the ground motion and its interaction with the nonlinear characteristics of the building. As such, NLTHA is a poor predictor of the exact performance or the exact magnitude of response for any given earthquake input motion.
- Special care and skill is required to select appropriate modelling approximations. For example, the definition of elastic damping needs careful consideration, as inappropriate definition will result an incorrect estimate of response.
- Typically, the interactions between flexure, shear and axial load are not modelled in NLTHA programs, making it impossible to model the onset of shear failure reliably. Similarly, few NLTHA programs include the influence of axial force in columns on their stiffness. This can influence predictions of onset of inelastic response, and can be critical for structures with brittle failure modes.
- Some NLTHA programs cannot model degrading strength characteristics, and few have special elements representing the strength and degradation characteristics of beam-column joints in concrete or steel structures.
- The refinements of an NLTHA may also be inappropriate when the uncertainty associated with the seismic intensity is considered. The seismic intensity is typically represented by the shape of the response spectrum for the earthquake record but will also

be affected by other factors such as ground conditions, site source distance and path, and magnitude and duration of shaking, as discussed below. When NLTHAs are carried out, it is usually necessary to run several analyses with different records representing the design intensity. This is to improve the chance that all potential inelastic mechanisms are identified and appropriately "tested". When it is necessary to determine the actual level of intensity corresponding to a given level of earthquake shaking rather than assessing a pass/fail result for a reference intensity, multiple analyses will be required, scaling the intensity of the records until the required level of shaking is reached.

C2C.2 Input Ground Motions

Where an inelastic time history analysis is carried out, the model representation of the building structure should be subjected to earthquake shaking represented by ground motion time histories in accordance with Section C3.

Note:

Research has shown that consideration of different ground motions is ESSENTIAL to the application of nonlinear response history analyses. The calculated response can be highly sensitive to the characteristics of individual ground motions.

More recent research (Baker and Cornell, 2006; Hancock et al., 2008; Bradley, 2010; Beyer and Bommer, 2007; Kalkan and Chopra, 2010) has indicated that the NZS 1170.5:2004 requirements for input ground motions may need to be updated. These include the minimum number of ground motion records that should be analysed and the method which should be used to assess the results. This is summarised in Table C2C.1.

As this is an area of active research, it is recommended that engineers review the latest literature (e.g. Bradley et al., 2015; Kwong and Chopra, 2015). ASCE 41 also provides more up-to-date guidance than available from NZS 1170.5:2004. At this stage, it is recommended that the selection and scaling of input ground motion is independently reviewed.

Table C2C.1: Suggested number of ground motion acceleration history records

Condition	Method of computing results	Number of ground motion records
Far-field (>5 km/3 mi)	Average	Record pairs ≥10
Far-field (>5 km/3 mi)	Maximum	**3 ≤** record pairs ≤ 9
Near-fault (≤5 km/3 mi)	Average	Near-fault record pairs ≥ 5; total number of record pairs ≥ 10
Near-fault (≤5 km/3 mi)	Maximum	**3 ≤** near-fault record pairs ≤ 9

Vertical ground motion should be included in the NLTHA, particularly if the structure has any element or component that is sensitive to the amplification of axial and gravity loadings (such as a cantilevered transfer structure). Similarly, this should be included if the structure's lateral load carrying capacity is largely dependent on the gravity restoring forces (e.g. URM rocking piers).

Note:

Care needs to be taken to ensure that the additional complexity of including the vertical acceleration component is warranted, as each increase in the complexity of the analysis has the potential to cloud the behaviour. In many instances the maximum vertical accelerations have dissipated before the maximum lateral shaking occurs.

The use of record pairs, applied in both directions in the NLTHA, should adequately account for the concurrency effects.

C2C.3 Modelling of Nonlinearity

Inelastic structural element models can be differentiated by the way in which plasticity is distributed through the member cross sections and along its length. For example, Figure C2C.1 shows a comparison of five idealised model types for simulating the inelastic response of beam-columns, ranging from a lump plasticity rotational spring model to a detailed continuum finite element model. All models are empirical as the models are calibrated to experimental results either at the macro level or micro-material level.

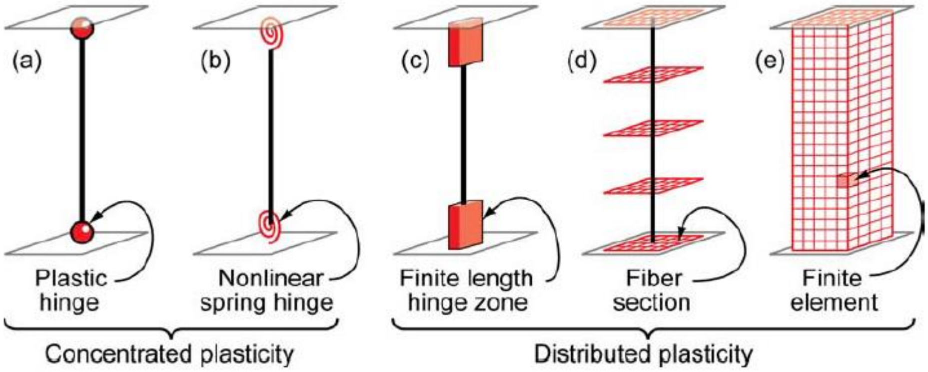


Figure C2C.1: Idealised nonlinearity model (from Deierlein et al., 2010)

It is important to understand the limitations of the modelling type and approximations inherent for each modelling assumption (e.g. whether shear failure is modelled). Engineers should appreciate the trade-off between different nonlinear modelling approaches and apply judgement as appropriate.

In general, lumped plasticity models are recommended for nonlinear analyses of large buildings as these simplify the number of inputs required and can be used to pinpoint the governing local inelastic mechanism. However, effects such as the interaction between axial, flexure, and shear failure in concrete members are difficult to capture using lumped plasticity models. Figures C2C.1 and C2C.2 show an example of a 2D nonlinear lumped-plasticity model for a non-ductile reinforced concrete frame (Kam, 2010).

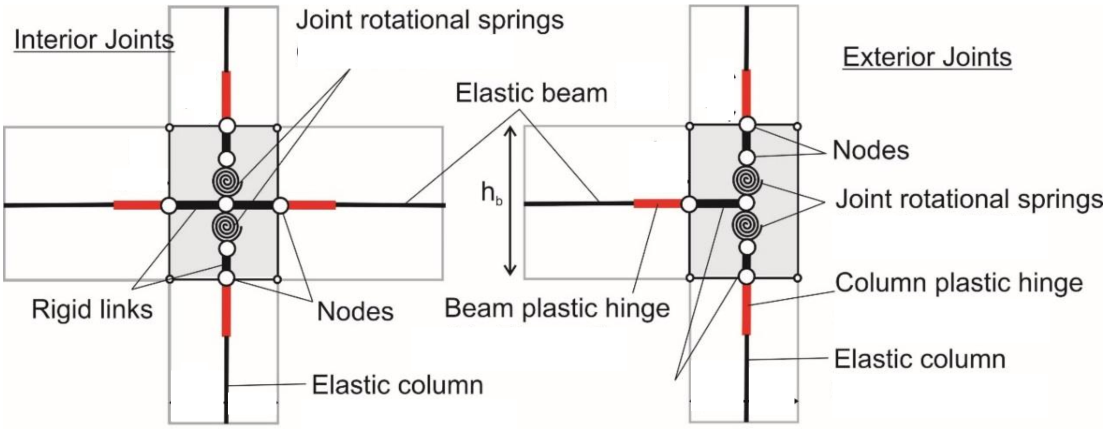


Figure C2C.2: Rotational joint spring model for non-ductile beam-column joint element (adapted from Kam, 2010)

For nonlinear procedures, a connection should be modelled explicitly if the connection is weaker, has less ductility than the connected components, or the flexibility of the connection results in a change in the connection forces or deformations greater than 10% (as found from sensitivity analyses).

There are a number of sources for guidance on the appropriate hysteretic modelling parameters (Carr, 2007; ASCE 41-17, 2017; Deierlein et al., 2010; FEMA 440, 2005).

C2C.4 Nonlinear Hysteretic Model Parameters

Irrespective of whether a concentrated plasticity or distributed plastic modelling approach is adopted, the definition and selection of the parameters for the nonlinear hysteresis model for either lumped plasticity or constitutive fibre/finite element elements is a very important step in nonlinear analysis.

In particular, the selected nonlinear hysteretic parameter should match as closely as possible to empirical data/evidence. This is particularly important for elements with strength and stiffness degradation (e.g. non-ductile reinforced concrete joints or columns) or pinching hysteresis (e.g. debonding and failure of lap splice of reinforcing bars).

Specialist knowledge of the appropriate type of hysteresis model and parameters is required in order to define the nonlinear hysteresis model appropriately. For example, Ibarra et al., 2005 and Pampanin et al., 2003 provide suitable strength and stiffness degrading models for non-ductile beam-column joints.

Based on available supporting literature (e.g. Carr, 2007; Seismosoft, 2013; McKenna et al., 2004; FEMA 440a, 2009a) the engineer should calibrate appropriate hysteresis parameters with appropriate experimental test data of similar structural sub-assemblies/members.

It is important to exercise engineering judgement in selecting the appropriate hysteretic parameters. Sensitivity analyses of key parameters are recommended.

C2C.5 Damping

Refer to Section C2D.4 for guidance on damping in an NLTHA.

C2C.6 Structural Performance Factor, S_p

There are two approaches when S_p is applied to NLTHA:

- **Spectrum reduction method**: S_p is applied to the seismic hazard demand curve in which the input ground motions are scaled to as per clause 5.5.2 of NZS 1170.5:2004 or other industry consensus document.
- **Base shear reduction method:** A 5%-damped elastic spectrum is used with $S_{p,nltha}$ = 1.0 for the initial scaling of the input ground motions. The NLTHA is implemented with the probable capacities. The resulting governing capacity-to-demand ratio and therefore *%NBS* are multiplied by $1/S_p$ (effectively increasing *%NBS*). The same S_p as per NZS 1170.5:2004 or other industry consensus document can be applied directly.

C2C.7 Interpretation of NLTHA Results and *%NBS*

It is expected that *%NBS* will be evaluated from NLTHAs as follows:

- Run analyses for the required number of earthquake strong motion records scaled to represent ULS earthquake shaking as defined in NZS 1170.5:2004.
- Check that all members/elements satisfy the deformation limits defined for the particular material types in Sections C5 to C9 in these guidelines. If they do, the analyses indicate that the building achieves at least 100*%NBS*.
- If they do not, scale all of the records using the same scale factor and re-run the analyses until acceptance is just achieved. The scale factor applied at this point is related to *%NBS* as shown in Equation C2C.1.

$$\%NBS = \text{Scale factor x } 100 \qquad \text{...C2C.1}$$

C2C.8 Peer Review

As NLTHA results are highly sensitive to the input parameters, modelling assumptions and input ground motions, the results and model of an NLTHA should be peer reviewed by an independent engineer with a good knowledge and experience of running this type of analysis. Peer review solely of inputs and outputs by an engineer with little understanding of the limitations of NLTHA will rarely provide the degree of overview expected or required.

Appendix C2D: Damping

C2D.1 Introduction

Assessing the level of damping available is a critical aspect of the assessment procedures outlined in these guidelines.

The treatment of damping and how it is incorporated varies depending on the analysis technique adopted. Damping can be allowed for explicitly (e.g. NLTHA, NLSPA, SLaMA) or implicitly (force-based procedures). Guidance is provided on the intended approach for each analysis technique in the following sections of this appendix.

C2D.2 Force-based Assessment Procedures and Elastic Analysis Techniques

For force-based assessment, energy dissipation and damping is captured as an inherent ductility of 5% in the defined demands and in the defined structural ductility factor, μ, and to some extent, the structural performance factor, S_{p}. These are defined in NZS 1170.5:2004 and relate to the inherent capability of the seismic resisting systems to sustain the ductility demand and dissipate energy. They assume that the system mechanisms will be fully developed, which is not always the situation with existing buildings where the response is invariably limited by deficiencies that would not be present in a new building.

For modal response analysis it is expected that 5%-damped spectra from NZS 1170.5:2004 should be used.

For mixed ductility systems, the appropriate structural ductility factor for the total system needs to be assessed to account for the governing inelastic mechanism and the actual achievable ductility in the system.

If additional damping is present (e.g. viscous dampers) a nonlinear procedure/analysis technique should be used, as the effectiveness of the dampers will be a nonlinear function of the deformations sustained.

C2D.3 Displacement-based Assessment Procedures and Nonlinear Analysis Techniques

C2D.3.1 General

For displacement-based assessments and nonlinear static procedures, these guidelines include damping in the form of an effective system viscous damping, ξ_{sys}. The derivation of ξ_{sys} is presented in Section C2D.3.2.

The intended method of derivation of ξ_{sys} for mixed inelastic systems is given in Section C2D.3.3.

C2D.3.2 Effective system viscous damping, ξ_{sys}

The effective viscous damping for the system, ξ_{sys}, is defined as follows:

$$\xi_{sys} = \xi_0 + \xi_{hy} + \xi_d \qquad \text{...C2D.1}$$

where:

ξ_0	=	the inherent damping
ξ_{hy}	=	the hysteretic damping
ξ_d	=	added damping due to supplemental viscous dampers. This is taken as zero if there are no dampers present.

The inherent damping, ξ_0, present is likely to be in the range of 0.02 (2%) and 0.05 (5%) damping. For the methods outlined in these guidelines ξ_0 may be taken as 0.05 (5%).

Typical values for ξ_{hy} (expressed as a % of critical damping) are shown in Table C2D.1. The assessment of ξ_{hy} is intended to proceed as follows:

- Identify the type of structural system present.
- Evaluate the level of hysteric energy dissipation expected to be available.
- Determine the level of displacement ductility achieved at the displacement when a significant life safety hazard develops in the building, μ_{sys} .
- Obtain the value of ξ_{hy} from Table C2D.1 for the system under consideration.
- Deal with multiple systems with different values of ξ_{hy} in accordance with Section C2D.3.3 to obtain the combined effective system viscous damping, ξ_{sys} , for the building.

Note:

The evaluation of the effective hysteretic damping factor for inclusion in the calculation of ξ_{sys} will necessarily be based on judgement, interpolation of the values provided in Table C2D.1 and use of values for specific structural types from available literature. Recent research has been carried out (e.g. Wijesundara et al., 2011; Sullivan et al., 2012; Sullivan et al., 2013; O'Reilly and Sullivan, 2015) that provides expressions for a wide range of systems. This is still an area of active research (Sullivan, 2016).

Table C2D.1: Typical values of ξ_{hy} for various structural types, materials and levels of hysteretic energy dissipation

Assessed level of hysteretic energy dissipation available	μ_{sys} [2]	ξ_{hy} [1, 2] (%)	Applicable structural systems/ mechanisms
H (High)	6	15	• Concrete frame (deformed bars) • Steel frame with rigid connections
	3	12	
	2	10	
	1.25	3	
	≤1	0	
M-H (Medium to High)	6	12	• Concrete wall • Light weight timber frame • Shallow footing rocking systems • Steel EBFs
	3	10	
	2	7	
	1.25	3	
	≤1	0	
M (Medium)	6	10	• Steel frame with flexible connections • Steel CBFs • Concrete frame (plain bars) • URM system[3]
	3	8	
	2	6	
	1.25	2	
	≤1	0	
L (Low)	6	5	• Hybrid prestressed concrete frame • Rocking system
	3	4	
	2	3	
	1.25	2	
	≤1	0	

Note:

1. After Pekcan et al. (1999), Priestley et al. (2007), Sullivan (2016) and NZSEE (2006).
2. The value of μ_{sys} in the table relates to the displacement ductility experienced at the level of demand considered. Thus, even though a structure may be detailed to achieve $\mu = 6$, the value of ξ_{hy} should be chosen assuming $\mu_{sys} = 3$ if the structure is only loaded to, say, half capacity. Generally, engineers will be interested in performance at the displacement consistent with a significant life safety hazard so only one value of ξ_{hy} will need to be assessed.
3. For unreinforced rocking masonry walls the concept of hysteretic damping is not appropriate. However, it is considered reasonable to use $\xi_{sys} = 0.15$ (15%) for walls loaded in-plane and $\xi_{sys} = 0.05$ (5%) for face loaded walls (refer to Section C8) when applying the procedures and analysis techniques set out in these guidelines. These values of damping should be used irrespective of the level of displacement expected.

C2D.3.3 Mixed inelastic systems

These guidelines adopt a weighted approach (based on the proportion of strength capacity contributing to the lateral resistance at the level of displacement being considered) to compute the achieved effective system damping for the building (as a whole) containing mixed-inelastic mechanisms (e.g. a steel braced frame coupled with concrete shear walls).

Note:

It is difficult to extrapolate the local ductility capacity to the global ductility capacity for mixed-inelastic mechanisms. The approach outlined here represents a pragmatic solution to the issue (refer to Section C2.5.11).

C2D.4 Nonlinear Time History Analysis (NLTHA)

The viscous damping in a NLTHA is associated with the reduction in seismic response through energy dissipation other than that modelled explicitly by the nonlinear hysteresis. This inherent damping occurs principally in:

- structural and non-structural items that are treated as elastic or inconsequential but which, as a whole, may contribute to not insignificant damping, and
- foundation radiant damping.

Supplementary energy dissipation devices (e.g. viscous, hysteretic or friction dampers) should be modelled explicitly in the nonlinear model.

In absence of any substantive data, an inherent viscous damping ratio of 0.05 (5%) as per Clause 6.4.6 of NZS 1170.5:2004 should be adopted across all modes of vibration.

The modelling of inherent damping in computer programs varies. Engineers should become very familiar with the method in the particular program being used and ensure that the response is appropriately damped.

Note:

The application of an elastic damping model in the NLTHA itself is a challenging topic. The use of a Rayleigh damping model or damping based on initial stiffness can be problematic and potentially unconservative (Priestley et al., 2007; Charney, 2008). A standard Rayleigh damping model will generate unrealistically high damping force post yield, although the majority of these non-modelled damping sources do not increase post elastic.

In the Rayleigh damping formulation the damping matrix is calculated as a linear combination of mass- and stiffness-proportional damping. Alternatively, a modal damping formulation can be used where percentages of critical damping are assigned to specific periods/elastic modes.

As an existing structure is very likely to experience inelastic behaviour and strength/stiffness degradation, a tangent-stiffness damping model is generally preferred to an initial-stiffness damping model (Carr, 2007; Priestley et al., 2007).

Further information on damping is available in literature (e.g. Carr, 2007; Deierlein et al., 2010).

Appendix C2E: Diaphragm Modelling and Analysis

C2E.1 Modelling

Mathematical modelling of buildings with flexible diaphragms should account for the effects of that flexibility by modelling the diaphragm as an element with an in-plane stiffness consistent with the structural characteristics of the diaphragm system.

Modelling of buildings with rigid diaphragms should account for the effects of horizontal torsion (refer to Appendix C2F).

C2E.2 Flexible Diaphragm Analysis

For buildings with flexible diaphragms at each floor level, each lateral force resisting element in a vertical plane may be permitted to be designed independently, with seismic masses assigned on the basis of tributary area.

Note:

Whether or not a diaphragm is considered to be flexible will depend on the relative flexibility of the diaphragm compared with the lateral stiffness of the members/elements being supported/interconnected.

Figure shows a flexible roof diaphragm arrangement commonly encountered in New Zealand low-rise buildings. In this example the diaphragm will be flexible compared with the connected walls.

Figure C2E.1: Example of a flexible diaphragm

C2E.3 Rigid Diaphragm Analysis

Seismic demands on rigid diaphragms should be determined using an equivalent static analysis.

Note:

Use of modal analysis is not appropriate for determining inertia forces to be resisted by a diaphragm. This analysis technique provides envelopes of maxima actions that will not provide relevant information for assessing load paths across diaphragms. These maxima do not provide actions that occur together at the same point in time. Therefore, they are not in equilibrium and do not produce a vector sense for the action (the outputs are all of one sign).

Actions within the diaphragms should account for higher mode effects and the influence of overstrength actions generated from within the structure as a whole.

Research and calibration of the recommended pseudo-Equivalent Static Analysis (pESA) method (Gardiner, 2011), described in Section C2E.4, account for the overstrength actions generated in the building and for dynamic higher mode effects.

Once the seismic demands from the global analysis is calculated, internal design actions on rigid diaphragms should be determined using a strut and tie analysis.

For concrete diaphragms, refer to the provisions of Section 13 of NZS 3101:2006 and Clause 6.1.4 of NZS 1170.5:2004. Note the added complication of transfer diaphragms, particularly when not originally designed as such.

C2E.4 Pseudo-Equivalent Static Analysis (pESA)

Figure C2E.2 illustrates the static floor forces used for the pESA. The pESA floor forces are determined in accordance with Section 6.2 of NZS 1170.5:2004 except as modified in Section C2E.4. As illustrated in Figure C2E.2, in the lower levels of the building the floor accelerations should not be taken lower than the peak ground acceleration (PGA).

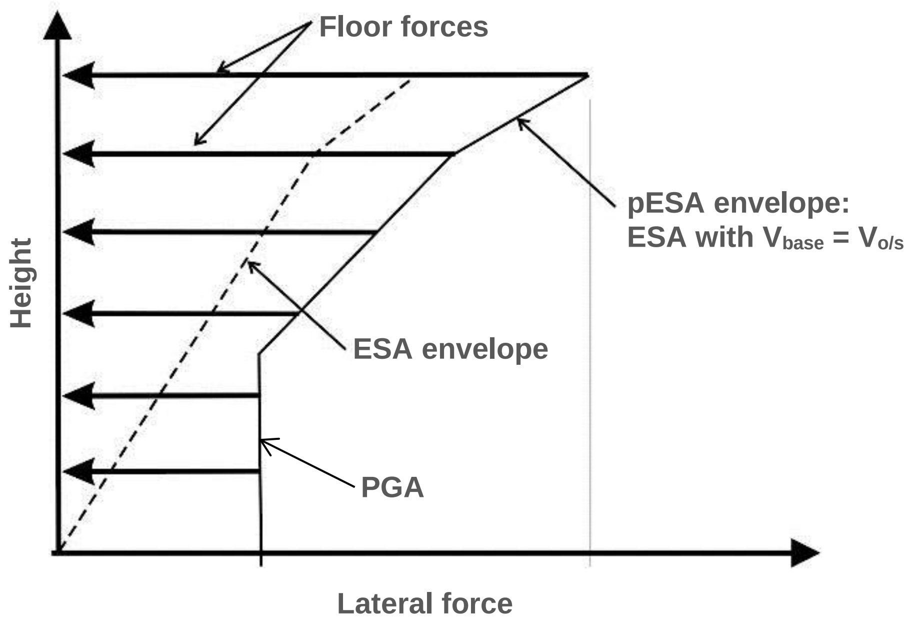


Figure C2E.2: Static forces for ESA and pESA envelopes (adapted from NZS 1170.5:2004)

Note:

Background on the pESA method can be found in NZS 1170.5 Amendment 1, along with in Gardiner (2011) and Alizadeh (2018)

NZS 1170.5 states that the pESA method is currently limited to buildings of up to nine storeys (NZS 1170.5:2004). This limit does not need to be adhered to for assessment purposes. However, pESA is expected to be conservative in situations where the part of the envelope defined by PGA extends beyond approximately the 5th storey.

When pESA is unsuitable, other rational methods can be used to assess diaphragm demands. NLTHA can be used as the basis for these demands. Alternatively, the method outlined in ASCE 7 (2022) can be used though it is noted that it there are questions around the way this method should be applied (Alizadeh 2018).

The line of action for the floor forces should be taken through the centre of mass of each floor. It is not necessary to consider accidental eccentricity during the pESA. It is required to consider concurrency/skewed earthquake actions. These should be included as follows:

- For ductile structures, it should be assumed that earthquake actions can occur on any axis and that all lateral systems develop their overstrength capacity.
- For nominally ductile or brittle structures, actions may be applied in accordance with NZS 1170.5:2004A1 Clause 5.3.1.2. That is,
  - For structures with lateral systems aligned on two perpendicular axes, actions may be applied separately in each perpendicular direction (100% on the first axis with 30% on the second axis, and then 30% on the first axis and 100% on the second axis).
  - For structures where lateral systems are not aligned on two perpendicular axes, the action set should be applied in sufficient directions so as to produce the most unfavourable effect in the diaphragm.

C2E.4.1 pESA Envelope

The diaphragm floor forces (effectively the inertia of each floor) should be determined based on those specified for diaphragm design in NZS 1170.5:2004A1 but with appropriate allowances made as outlined below to reflect the generally less conservative approach adopted for assessment.

The strength of a diaphragm need not exceed that required to resist the demands arising when the overstrength capacity of the building, $V_{o/s}$, develops. To determine these demands the pESA envelope should be defined using the value of $V_{o/s}$ for the building which can be calculated as:

$$V_{\mathrm{o/s}} = \phi_{\mathrm{ob}} V_{\mathrm{prob}} \qquad \text{...C2E.1}$$

where ϕ_{ob} is the building overstrength factor and V_{prob} is the probable lateral strength of the as-built structure.

In situations where the diaphragm is not strong enough to resist the demands arising from the overstrength capacity of the building, the value used to define the pESA envelope should be taken as:

$$V_{\mathrm{o/s,100\%NBS}} = V_{E,\mu=1.25} \qquad \text{...C2E.2}$$

where $V_{E,\mu=1.25}$ is the ULS base shear corresponding to nominally ductile demands calculated taking the structural performance factor as $S_p = 0.925$ in accordance with NZS 1170.5:2004.

Note:

Whether based on building overstrength or nominally ductile demands, the pESA envelope should not be taken as less than PGA as shown in Figure C2E.2.

In an assessment both $V_{o/s}$ and V_{prob} should be based on development of an inelastic mechanism, with the difference between these strengths due only to strain hardening. Therefore the ratio of $V_{\mathrm{o/s}}/V_{\mathrm{prob}}$ will be equal to the average material overstrength factor for the building, i.e. typically approximately 1.25.

Account should be made for the likelihood of some of the plastic hinge zone not forming (for example, using the R_{v} factor in NZS 3101:2006).

Appendix C2F: Torsion

C2F.1 General Approach

For buildings with rigid diaphragms it will be necessary to consider the torsional amplification effect arising from the demand and resistance eccentricities (centre of mass including accidental displacement allowance) and the location of the centre of stiffness or strength as appropriate).

Torsion does not need to be considered in buildings with flexible diaphragms.

There can be several types of torsion response:

- **accidental torsion** arising from the effects of the rotational component of the ground motion, differences between computed and actual stiffnesses, and unfavourable distributions of dead and live load masses (assumed to be covered by the allowance for accidental displacement of the centre of mass)
- **inelastic torsion** arising from the effects of nonlinear behaviour and interaction between systems with different post-yield stiffness, strength degradation and ductility capacity, and
- **torsional amplification** arising in a deteriorating system resulting in plan irregularity; e.g. premature damage of infill panels at one elevation may lead to inelastic torsion response.

Torsional response can lead to amplification of either internal actions or displacement responses, depending on several factors:

- degree of plan stiffness irregularity:

  $e_{\text{stiffness}}$ = distance between centre of mass and centre of rigidity/stiffness

- degree of plan strength irregularity:

  e_{strength} = distance between centre of mass and centre of strength (linear or nonlinear)

- torsionally restrained (TR) systems with torsional resistance from lateral bracing elements in the orthogonal direction or torsional stiffness of the overall lateral load resisting system
- nonlinear behaviour and interaction of the lateral load resisting systems. Systems with low post-yield, high differential strength degradation and ductility capacities are shown to have higher inelastic torsion amplification.
- rotational mass inertia. Torsionally unrestrained (TU) systems can have significant displacement/ductility demand amplification due to rotational mass inertia (Paulay, 2000b).

This appendix provides three assessment methods. The first, Method A, is for systems expected to respond elastically or with nominally ductile demand at the lowest loading that leads to a significant life safety hazard.

However, for most existing structures with nonlinear response – and especially for those with mixed ductility response – an inelastic torsional assessment should be carried out using either:

- Method B: Inelastic torsion response with ductile responding systems (in both directions), or
- Method C: Removing strength eccentricity in inelastic response.

C2F.2 Method A: Elastic Torsion Response

For assessment using the elastic force-based procedure and linear analysis techniques (equivalent static or modal response analysis), only the consideration of accidental torsion will be required. A 5% accidental mass eccentricity assumption as per Section C2.5.7 should be adopted. This method is intended for systems that are expected to respond elastically or with nominally ductile demand at the lowest loading that leads to a significant life safety hazard.

C2F.3 Method B: Inelastic Torsion Response with Ductile Systems in Both Directions

This approach relies on the lateral load resisting system in the orthogonal direction to provide torsional resistance.

In this method, the shear demand in the orthogonal direction is amplified to account for the torsional demand (as outlined below). Alternatively, the engineer could calculate the torsional moment of inertia and amplify the displacement demand corresponding to the torsional demand.

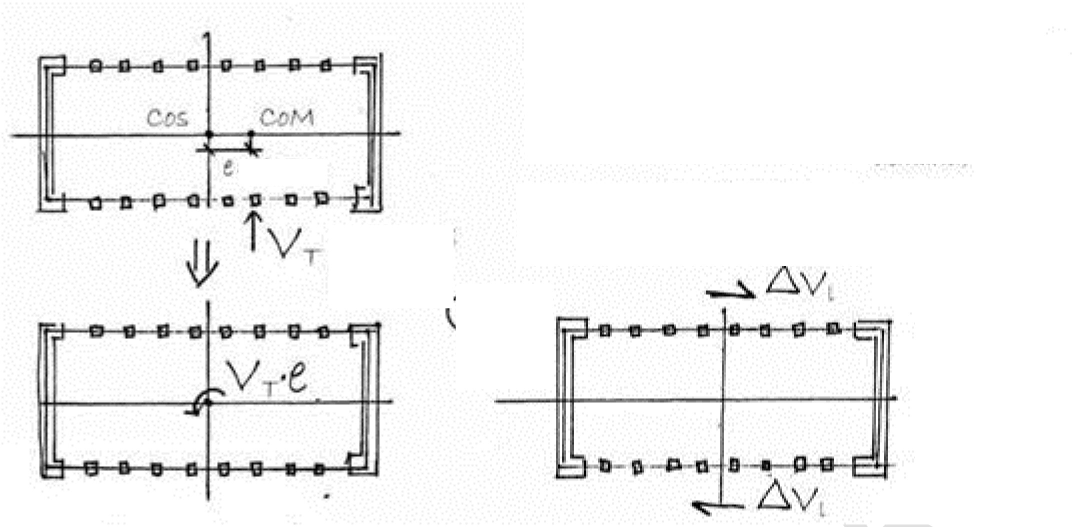
If the lines of the primary lateral system in the direction being considered have some ductility capacity ($\mu \geq 2$) it is considered acceptable to resist the torque resulting from the eccentricities solely by the couple available from the lines of the primary lateral system perpendicular to the direction of loading (refer to Figure C2F.1). If this approach is followed the centre of strength rather than the centre of stiffness should be used when evaluating the eccentricities.

The increase in shear demand in the orthogonal direction can be calculated assuming any torsional demand is resisted by a torsional couple formed by the lateral load resisting members in the orthogonal direction. Therefore, for the example shown in Figure C2F.1(b):

$$\Delta V_{\mathrm{t}} = V_{\mathrm{base}} \text{ x } e_{\mathrm{strength}} / L \qquad \text{…C2F.1}$$

where:

ΔV_{t}	=	the shear force increase in the lateral load resisting members in the orthogonal direction
L	=	the distance between the centroids of the lateral load resisting lines in the orthogonal direction.



(a) Torsional demand **(b) Resistance provided by orthogonal wall lines**

Figure C2F.1: Relationship between demand and resistance for a building with rigid diaphragms

Note:

For this method to be valid for nonlinear responding systems, the lateral load resisting systems should be ductile ($\mu \geq 2$) and the elements in the orthogonal wall lines should not become nonlinear under these actions. In addition, this method requires that the diaphragm maintains its structural integrity in order to mobilise the lateral system in the orthogonal direction to provide torsional resistance.

In a building assessed following the Christchurch earthquake sequence (Kam et al., 2011) the lateral load resisting systems were severely compromised due to the combination of inelastic torsion response and diaphragm failure. In this building, the lateral load resisting system comprised perimeter ductile moment resisting frames with mesh-reinforced concrete topping on precast double tees acting as a diaphragm.

In the 4 September 2010 earthquake, the ductile perimeter moment frames in the north-south direction yielded, resulting in a nonlinear system with low post-yield stiffness. In the 22 February 2011 earthquake, the loss of the diaphragm as well as relatively low stiffness of the moment frames in the orthogonal direction resulted in limited to no torsional resistance being provided by the moment resisting frames in the east-west direction.

C2F.4 Method C: Absence of Strength Eccentricity

In this approach, the achievable lateral capacity (e.g. base shear) is reduced to eliminate the strength eccentricity and therefore any inelastic torsion amplification. This method is intended for torsionally unrestrained systems or systems with limited torsional redundancy.

If the strength eccentricity exceeds 2.5% of the relevant lateral dimension of the plan, the probable strength of the system should be revised by reducing the probable strengths of those elements which are responsible for the strength eccentricity until the strength eccentricity is eliminated. Therefore, the global base shear capacity is artificially reduced to eliminate the strength eccentricity and mitigate any inelastic torsional response.

This procedure is based on the assumption that, in the absence of the strength eccentricity, the response of the system may be considered to be governed primary by translatory displacements (Paulay, 2001; Paulay, 2000a). In terms of ductile response, effects of stiffness eccentricity may be ignored.

This method may provide a conservative estimate of the lateral capacity of the building as it seeks to minimise inelastic torsion response. Castillo et al. (2002) indicates that this method is most suitable for torsionally unrestrained systems for which other methods may be un-conservative.

Note:

Figure C2F.2 illustrates an example where it is found that the relative probable translatory strengths of elements (1), (2) and (3) are 46%, 18% and 36% respectively. These result in a negative strength eccentricity of $e_{vx} \approx 0.10A > 0.025A$.

By only relying on 30% and 13% strength contribution to elements (1) and (2) respectively, the overall strength eccentricity, e_{vx} can be eliminated. This reduces the achievable probable strength to 79%.

The expected displacement ductility demand on the system may then be based on this reduced system strength. Under these circumstances displacement demands on element (3) due to system translations and rotations, while developing 100% of the probable system strength, will not be critical.

In traditional design procedures which were based on elastic structural behaviour, strengths of elements were assigned in proportion to their assumed stiffness. Subsequently, strength redistribution (NZS 3101:2006) within a 30% limit was permitted to be used, provided that the total seismic strength of the building was not reduced.

However, this restriction on the allocation of seismic strength to elements is now considered to be unnecessary. Therefore, reliance on the probable strengths of elements, as constructed, may be made without recourse to analysis of the elastic structure when evaluating the probable global base shear of the system.

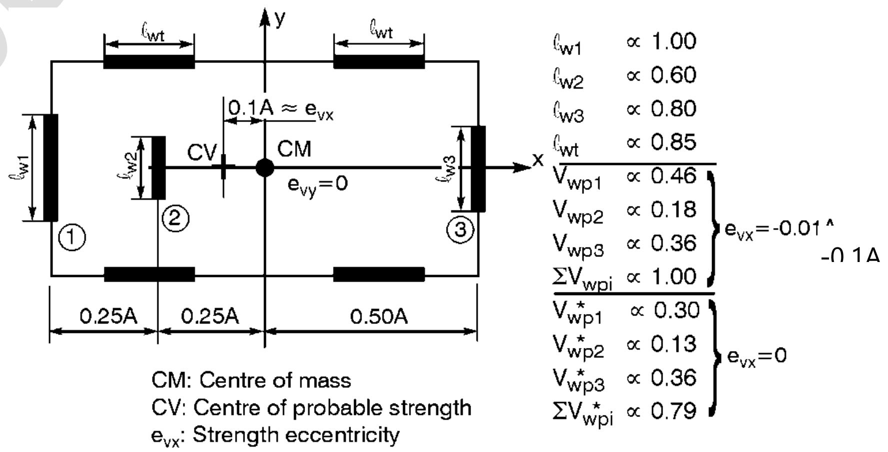


Figure C2F.2: Torsional effects in walled buildings (after Paulay, 2001)

Appendix C2G: Severe Structural Weaknesses

C2G.1 Approach to Assessing SSWs

Aspects that should be evaluated as SSWs in a DSA to meet the requirements of these guidelines and the quantitative means for assessing them are described in Section C1.

> **Note:**
>
> Refer to Section C1 for more about the rationale for assessing SSWs and the criteria used to define these.

The typical approach followed in these guidelines is to determine the capacity of SSWs as follows:

- Assess the probable capacity of the elements/members comprising the system that is considered to be the SSW using the methods outlined in Sections C4 to C9.
- Assess the probable capacity of the system using the methods outlined in this section.
- The capacity (strength and deformation) of the SSW is then taken as one half of the probable capacity.

For the SSWs involving geohazards it may be more convenient to determine the level of shaking that would lead to the levels of deformation in the structure that would create a significant life safety hazard, in accordance with Section C4. The capacity of the SSW is then taken as half of this level of shaking.

C2G.2 Non-ductile Columns with Axial-shear Failure

These are lightly reinforced concrete columns and/or beam-column joints (refer to Section C5 for definition) with axial loads greater than $0.2\ A_g f'_c$ which are part of the primary structural system (typically the gravity system) of buildings where multiple fatalities would be possible if one or more storeys were to suffer full collapse. To be a SSW the failure of a column and/or beam/column joint would need to lead to a progressive collapse scenario for the entire storey.

The capacity of any such non-ductile reinforced concrete columns susceptible to axial-shear failure should be taken as **one half of the probable lateral drift capacity determined in accordance with Section C5**.

> **Note:**
>
> The axial-shear failure of key vertical load carrying members such as primary columns and beam-column joints has been shown to be a critical deficiency that has led to catastrophic collapse of reinforced concrete buildings (Elwood and Moehle, 2005; Boys et al., 2008; Kam et al., 2011).
>
> Some structural systems have the ability to redistribute the axial load for localised axial-load failure. In other cases, progressive collapse can be initiated with the failure of one column or joint. The severity of the axial-shear failure of a particular vertical load carrying member should be assessed.

It is vital that both the overall analysis and the assessment of ductility demand take proper account of the characteristics of short columns. Displacements generated in the structure can have a severe effect on the integrity of these elements by driving up shear forces beyond the capability of the sections.

The most critical aspect of the detailing of reinforced concrete columns for flexural ductility capacity is the amount of transverse reinforcement provided; in particular, the spacing between adjacent reinforcement sets (Park and Paulay, 1975). The transverse reinforcement provides confinement to the core concrete and prevents the longitudinal bars from buckling. In general, the lower the transverse reinforcement ratio the less ductile the column will be under lateral displacements such as those experienced during an earthquake.

The performance of non-ductile reinforced concrete columns with low quantities of transverse reinforcement has been covered extensively in literature (in particular, refer to Boys et al., 2008; Elwood and Moehle, 2005; Kam et al., 2011).

In addition to low quantities of transverse reinforcement, several other characteristics of a column can contribute to its vulnerability in an earthquake. The following is a list of indices (Stirrat et al., 2014) that may suggest the columns are susceptible to non-ductile behaviour. Note that the suggested limits are based on available literature and experience, and have not been extensively tested. The nomenclature is that used in NZS 3101:2006.

- Low or inadequate quantities of transverse reinforcement – spacing, $s > d/2$
- High axial load demand – $P/A_g f'_c > 0.2$
- Low core-to-gross concrete area – $A_c/A_g < 0.7$
- High inelastic inter-storey drift demand – > 1.5% drift
- Detailing – inadequate lap-splice length, lap-splice located in potential plastic hinge zone, poor detailing of transverse reinforcement anchorage (e.g. 90 degree bends), welded detailing, lack of support to longitudinal bars
- Location of column – in location prone to inelastic torsional amplification of displacements (e.g. corner column or column on opposite face to eccentric shear core).

C2G.3 Non-ductile Shear Wall Without Redundancy

This SSW is a shear wall system meeting the following criteria:

- it supports a significant level of axial load where $N^*_g \geq 0.15 A_g f'_c$, where N^*_g is the axial load under dead and reduced live load (Q_u), and
- it has shear-failure dominated force-controlled mechanism (i.e. not flexural governed behaviour), and
- it is a group of interconnected walls acting as a single unit (single core wall) which supports more than 60% of the seismic lateral demand, and
- multiple fatalities would be possible if the building were to suffer full collapse.

The shear capacity for these critical walls should be taken **as one half the probable shear capacity determined in accordance with Section C5**. The shear capacity should be appropriately modified accounting for axial and flexural interaction.

C2G.4 Flat Slab Floor System Susceptible to Punching Shear Failure

This SSW is a flat slab system in a cast insitu concrete floor without shear reinforcement in the slab and with gravity-only shear demand exceeding 40% of the probable shear capacity (v_c+v_s) at the critical shear interface, and multiple fatalities would be possible if one or more storeys were to suffer full collapse..

The capacity of this SSW is taken as **one half of the probable drift capacity of the axial-shear mechanism determined in accordance with Section C5**. The intent is indicated in Figure C2G.1.

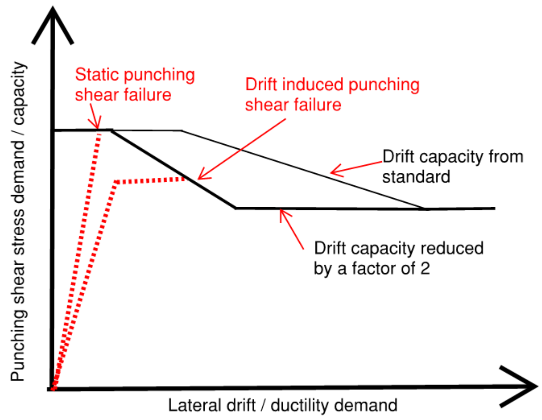


Figure C2G.1: Shear demand versus drift relationship for non-ductile flat slab-column system

Note:

Refer to Section C5 for the assessment of punching shear failure of a non-ductile flat slab by considering both gravity load and drift-induced punching shear demand.

The flat slab-column system is generally used with a rigid lateral load resisting structural system, such as shear walls or moment frames. Irrespective of the primary lateral load mechanism, the slab-column system must maintain its gravity load capacity based on displacement compatibility. As the flat slab system sways laterally, the unbalanced bending moment in the slab-column connection results in increased punching shear demand.

Flat slabs, particularly those with discontinuous bottom reinforcement or that are lightly reinforced, are susceptible to progressive collapse if punching shear failure occurs at a connection (e.g. Robertson and Johnson, 2004; Kang and Wallace, 2006). Many such failures have occurred in past earthquakes and led to significant loss of life. An example of this type of failure is shown in Figure C2G.2.

Figure C2G.2: Collapse of flat-slab system observed in Christchurch (from Kam et al., 2011)

C2G.5 Diaphragm Without Redundant Load Path

This SSW is a concrete diaphragm (most likely to be precast concrete) in systems where, if there is a loss of diaphragm connection, there is no ability to redistribute seismic actions through other means (e.g. a core wall building). This can also lead to undesirable inelastic torsional instability as described in Appendix C2F.

This SSW is only intended to apply to a diaphragm system that meets the following criteria:

- there is lack of ductile connection to the vertical lateral load elements; i.e. the failure plane of the diaphragm is either unreinforced or is only reinforced with brittle cold-drawn mesh reinforcement, and
- the diaphragm transfers all seismic inertia into no more than two vertical lateral load elements in a particular direction (i.e. there are no more than two bracing lines), and
- multiple fatalities would be possible if one or more storeys were to suffer full collapse.

The capacity of such a diaphragm is taken as **one half of the probable capacity determined in accordance with Section C5.**

Note:

Refer to Section C4, DBH (2009) draft guidelines and Fenwick et al. (2010) for further information.

C2G.6 Loss of Support Due to Complex Slope Failure

This SSW is a complex slope failure resulting in complete loss of the building platform and support. It applies where more than 50% of the building platform would be affected by slope failure; i.e. where the building is on a slope or cliff edge.

The capacity of the slope is taken as **one half of the peak ground acceleration causing a slope failure that would result in the complete loss of at least 50% of the building platform**. This is calculated using probable soil parameters in accordance with Section C4.

Note:

Refer to Section C4 for guidance on the assessment of slope failure.

C2G.7 Poorly Tied Together Multi-Storey URM Structure on Liquefiable Ground

This SSW applies to a poorly tied together building that:

- is on liquefiable ground, and
- where multiple fatalities would be possible if one or more storeys were to suffer full collapse.

A poorly tied together building is one where the ties within the building score less than the assigned *%NBS* for the SSW.

The capacity of this SSW is taken as **half of the peak ground acceleration that would result in wide spread liquefaction with potential for settlement beyond which support of the building cannot be assured.**

Foundation stiffness needs to be modelled and considered. For example, a URM building on well-tied together reinforced concrete strip footing is better able to sustain differential settlement than a URM building on discrete pile foundations.

Note:

Refer to Section C8 for guidance on the assessment of URM structures and Section C4 for guidance on the assessment of liquefaction potential.

C2G.8 Wall through-the thickness failure

Walls subjected to a high axial load may be susceptible to sudden axial failure featuring crushing and shifting in the out-of-plane direction across the length of the entire wall. This behaviour is referred to as through-the-thickness crushing failure, where a diagonal failure plane is developed through the thickness.

Walls exhibiting axial failure typically fail at, or just slightly beyond, the drift at lateral load failure. The capacity of walls susceptible to axial through-the-thickness failure should be taken as **one half of the probable lateral drift capacity determined in accordance with Section C5**.