Index of change proposals

All Sections

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C0-**1**	*Section numbering*	Referencing sections of the guidelines is challenging because Level 5 and Level 6 headings are not numbered.	Add numbering to Level 5 and Level 6 headings	Editorial to improve useability.
C0-**2**	*Appendix numbering*	Referencing sections of the guideline appendices is challenging because Level 4 headings are not numbered.	Add numbering to appendix Level 4 headings	Editorial to improve useability.

Section C1 (2017)

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C1-**1**	*C1.4*	Reference under Step 4 indicates SSWs are discussed in C1.1.1, which is not correct.	Point to C1.5.3 instead	Editorial
C1-**2**	*C1.5.1*	Equation C1.1 does not give a correct definition of %NBS if gravity actions exist in an element.	Add new equation later in section $$S_{prob} = G + \psi_E Q + \psi_{NBS} E_u$$	The ψ_{NBS} notation is used in the ReCast floors overview paper. Could alternatively present as "%NBS.E_u" but this seems clumsy.
C1-**3**	*C1.5.1*	Guidelines currently state in numerous locations that deformation capacities are "probable" capacities – which is commonly not the case.	Add note explaining the background to basis of defining capacities in the guidelines. See guidelines for wording.	Guidelines currently fail to clearly explain the intent of deformation limits. Providing this background will improve consistency of future updates to other parts by clarifying what is intended. Suggest also remove last line of Section A6.3 which is too definitive on basis of definition of deformation capacities.
C1-**4**	*C1.5.3*	Definition of a non-ductile axial force as $0.5A_g f_c'$ is inconsistent with reduction of this limit to $0.2A_g f_c'$ that was made in 2018 C5.	Change text to $0.2A_g f_c'$	Aligning C1 with C5-2018
C1-**5a**	*C1.5.3*	Flat slab SSW specifies that it applies where lateral capacity is reliant on the slab-column connections. This is inconsistent with C2.	Remove reference to "lateral capacity is reliant" Change reference to cast in-situ concrete buildings so that it refers to cast in-situ slab. Specify that SSW only applies when there is no shear reinforcement.	Makes this consistent with C2. Reference to cast in-situ concrete is unnecessarily restrictive – could conceivably apply to concrete slab fixed to steel columns.
C1-**5b**	*C1.5.3*	Note at end of C1.5.3.1 is unclear where it states "e.g. ledge and gap stairs"	Change to "e.g. ledge/sliding supports"	Editorial
C1-**6**	*C1.5.3*	Wall axial "through-the-thickness" is described in C5-2018 as an SSW, but does not feature in C1.5.3 currently.	Add new item to end of list: *Wall through-the thickness failure, meaning sudden axial failure featuring crushing and shifting in the out-of-plane direction across the*	Aligning C1 with C5-2018.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			length of the entire wall (refer to Section C5 for definition of susceptible walls).	
C1-**7**	*C1.6.2*	Guidelines currently point specifically to ASCE 41-13, which is now well outdated.	Change section heading to generic "ASCE 41" with addition to note under heading C1.6.2 stating that the most current version should be used. Also change reference to ASCE 41-17 (unless 41-23 is released before finalisation).	Updates of ASCE 41 are subject to a rigorous balloting system by subject matter experts. There is no reason not to bring these updates into NZ adoption of that document automatically.
C1-**8**	*C1.6.2*	Reference under 8th bullet indicates SSWs are discussed in C1.1.1, which is not correct.	Point to C1.5.3 instead	Editorial
C1-**9**	*C1.6.2 Table C1.1*	Use of a collapse prevention scale factor of 1.8 has been shown to result in ASCE 41 based assessments being systemically more conservative than those that follow the general provisions of the guidelines (Thompson & Oliver 2019)	Change factor to 1.5. Alter note to refer to Thompson & Oliver 2019 as basis. Change following note to remove reference to return periods.	Change will result in more consistent assessment outcomes. Refer to Thompson, A. J., and Oliver, S. (2019). "Reviewing Expected Margins to Collapse in the Assessment of Existing Buildings in New Zealand." Proc. SESOC Conference, Structural Engineering Society of New Zealand, Auckland, New Zealand.
C1-**10**	*Note under Table C1.1*	Fifth, sixth, and seventh and ninth paragraphs of note are not specific to the ASCE 41 NLTHA approach	Move paragraphs to C2C.1	

Section C2 (2017)

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**1a**	*Notation*	μ_{hy} listed as part ductility, which is also (correctly) denoted as μ_p	Delete μ_{hy} and definition	Editorial
C2-**1**b	*Notation*	μ_{sys} definition refers to achievable ductility, rather than the ductility that actually develops at the level of demand considered. This is inconsistent with Table C2D.1, and does not make sense.	Change definition to refer to ductility that develops.	Editorial
C2-**2**	*C2.4.2 Step D3*	Note states that for low rise structures the effective mass “should” be taken as the full mass. This is unnecessary.	Change to “can conservatively be taken as…”	Editorial
C2-**3**	*C2.4.3/C2.4.1*	Second paragraph is a general overview on applicability that would seem more appropriately located in C2.4.1 Should point users to NLTHA for ductile irregular/higher mode driven structures Does not provide guidance on mathematical approaches to estimating yield displacement.	Move second paragraph. Alter paragraph to read: *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. If NLPSPA is used for such structures elastic modal response spectrum analysis should be undertaken as well (refer to Section C2.8.2.4).* Add reference to ASCE 41 procedures for estimating yield displacement.	The combination of NLSPA and modal response spectrum analysis is less applicable to complex ductile structures than NLTHA.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**4**	*C2.5.1*	Note needs to refer back to C1.5 for new note on how capacities should be defined	Add to end of note: *Refer to Section C1.5.1 for guidance on how deformation capacities should be defined for use in these guidelines.*	Editorial – pending acceptance of C1.5.1 change.
C2-**5**	*C2.5.3*	Last paragraph of note uses "low ductile gravity systems" – should be "low ductility"	Change to "low ductility"	Editorial
C2-**6a**	*C2.5.7*	Discussions at Joint Committee suggest specification of ±10% accidental eccentricity is punitive for existing buildings.	Alter to ±5% accidental eccentricity in line with ASCE 41	Considered consistent with the philosophy that assessment targets the expected capacity.
C2-**6b**	*C2.5.7*	Note adds nothing to the document – and is bordering on contradictory to the content of the section.	Delete note. Add replacement line: *Background information regarding the basis of the accidental eccentricity required to be considered by these guidelines can be found in Elms (1976).*	Editorial Refer to *Elms, D. G. (1976). "Seismic Torsional Effects on Buildings." Bulletin of the New Zealand National Society for Earthquake Engineering, 9(1), pp.79–83.*
C2-**7**	*C2.5.8*	Section is confusing and suggests actions that do not make logical sense. Appears to use "torsional amplification" where "torsional response" would make more sense. Suggests "amplifying forces according to NZS 1170.5" – but this approach is not one taken by NZS 1170.5. Same concepts are better explained in Appendix C2F.	Change section to refer to "torsional response". Delete most of section and instead point user to Appendix C2F for the details of how to approach torsion. Leave equation numbers as placeholders to avoid flow on effects. See marked copy for full changes.	Simplifies and clarifies document.
C2-**8**	*C2.5.9*	3rd paragraph - No mention of other examples where bi-axial effects could be important.	Add new concluding sentence: *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.*	Editorial

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			Correction of typo at end of fourth paragraph so that combination reads 30%X + 100%Y rather than 30%Y + 100%Y.	
C2-**9**	*C2.5.10*	Statement that higher modes are likely to be significant where a building's fundamental period exceeds one second is too blunt – would be dependent on height of structure for instance.	Move statement to new commentary note.	Simplifies and clarifies document.
C2-**10**	*C2.5.10*	Statement that "higher modes are not likely to be significant if 60% or more of the mass participates in the first mode in a particular direction" is vague and probably often contradictory to more specific guidance about mode shapes given above.	Move statement to new commentary note.	Simplifies and clarifies document.
C2-**11**	*C2.5.10*	Current approach to dynamic amplification: • is too focussed on concrete, • does not apply for shorter walls (less than 6 stories) • Only requires consideration for $\mu \geq 3.0$ • Has some strange wording ("If there is a shear force" – presumably there is always a shear force) • Is likely too conservative for taller buildings	Remove reference to specific material. Remove exclusion of shorter structures. Extend consideration to include structures with $\mu \geq 1.25$. Improve and simplify language. Change so that wall factor applies for any height of structure, but caps at 30% increase rather than 80%.	Editorial and to improve overall consistency with guidelines philosophy.
C2-**12**	*C2.5.10*	Document currently makes no reference to drift modification factor, k_{dm}.	Add new sub-section heading "Dynamic amplification" after fourth paragraph. Add new sub-section before C2.5.11 covering "Drift modification" that requires use of a modified form of NZS 1170.5 k_{dm} equation for elastic analysis of ductile structures other than wall buildings.	Clarifies that k_{dm} or equivalent adjustment needs to be included in assessments.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			Add definition for k_{dm}^{*} in notation with * added to emphasise difference from NZS 1170.5.	
C2-**13**a	*C2.6.2*	Note is unclear and does not add to the document. %NBS is better defined in A and C1, so explanation here is unnecessary.	Remove note.	Editorial.
C2-**13b**	*C2.6.2 Table C2.1*	Currently repeats definitions of when higher modes are critical Table currently specifies that two load patterns are required for NLSPA only when higher modes are critical, whereas C2.8.2.3 always requires two patterns. Minor typo – double bracket in MRSA section of table.	Refer back to Section C2.5.10 Remove reference to when two load shapes are required as this is always the case. Correct minor typo.	Improving consistency of document.
C2-**14**	*C2.8.2.1*	Fourth paragraph is vague and generally incorrect. *Most NLSPA computer programs cannot deal with negative structural stiffness (so-called "falling branch behaviour")…*	Delete paragraph	Editorial
C2-**15**	*C2.8.2.4*	Basis for assessing significance of higher modes differs from earlier section C2.5.10	Remove reference to 60% participation and instead reference back to C2.5.10. Add extra sentence noting that NLTHA is preferable to the combination of NLSPA and MRSA.	Improving consistency of document.
C2-**16**	*C2.8.2.4*	Last paragraph is not consistent with international guidance on NLSPA such as FEMA 440	Delete paragraph.	Simplifying document and making more consistent with international practice.
C2-**17**	*C2.8.3*	Specific reference made to outdated version of ASCE 41	Change to generic reference as for section C1	Editorial

Appendix C2A

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**18a**	*Figure C2A.4*	Numbering skips out C2A.3	Renumber figure to C2A.3 Also corrects currently incorrect reference in notation that already refers to C2A.3.	Editorial
C2-**18**b	*C2C.1*	Information currently in C1.6.2 is generic to all time history analysis rather than just to analysis using ASCE41	Move paragraphs from C1.6.2 to note in C2C.1	

Appendix C2C

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**19**	*C2C.2*	Reference to 1170.5 clauses for scaling likely to become outdated with release of work by current industry working group.	Delete clause references and leave as a pointer to C3.	Editorial/future proofing.
*C2-**20***	*C2C.6 - Structural Performance Factor, Sp*	The document currently recommends using S_p = 1.0 when average response from NLTHA are used. This recommendation was originally included in the document due to concerns adopting an average response might be unduly unconservative if the target spectra was reduced by (1+Sp)/2 as per NZS 1170.5. This is now believed to be unnecessarily conservative provided a rationale method is adopted to scale ground motion records used for the NLTHA. NZS 1170.5 no longer reflects industry best practice as set out in draft SESOC/NZSEE/NZGS guidance on NLTHA.	Delete the last paragraph that reads: *In the scenario where a large number of ground motion records and average responses from NLTHA are used, there is an argument that S_p as per NZS 1170.5:2004 may be unconservative. For scenarios where average responses from NLTHA are used, S_p= 1.0 should be adopted.* Add "or other industry consensus guidance after references to NZS 1170.5 in remaining two bullets.	Experience since 2017 suggests this is unlikely to be a significant issue when ground motions are scaled to the target spectrum using average spectrum i.e. similar to that recommended in ASCE 41-17. Industry Guidance for NLTHA is currently being drafted by SESOC & NZSEE. It is anticipated this document will not recommend the use of Sp = 1.0 when average response is used i.e. will recommend (1+Sp)/2. It is anticipated further research on this topic will be undertaken to validate how Sp should be applied to NLTHA. For the interim period, until the NLTHA provisions in NZS 1170.5 are updated, it

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
				is suggested the recommendation to use S_p = 1.0 be deleted form this section.

Appendix C2D

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**21**	*Table C2D.1*	Note 2 currently refers to μ_{hy} when the table uses (correctly) μ_{sys}	Change to μ_{sys}	Editorial

Appendix C2E

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**22**	*C2E.3*	Reference to Section 4.4 of NZS 3101 is wrong.	Not entirely clear what this is meant to refer to: Could be Clause 13.4 – but suggest delete paragraph as redundant given broader reference to Chapter 13 of NZS 3101 in preceding paragraph	Correcting error in cross references.
C2-**23**	*C2E.4*	Current order is illogical – building overstrength is discussed before pESA envelope	Move to be a subsection of C2E.5 (now C2E.4) – Pseudo-Equivalent Static Analysis Rename as *pESA Envelope* as now not focussed on overstrength factors.	Editorial
C2-**24**	*C2E.5 (Now C2E.4)*	Contains unnecessary background information that really pertains to the development of pESA rather than its application Insufficient guidance provided about what to do for buildings taller than 9 storeys.	Delete first to third, and sixth paragraphs Add note pointing to alternative methods, and noting that pESA can be applied to taller buildings but may be conservative where PGA part of envelope governs over too much of height.	Editorial. Editorial/consensus of balloting group.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**25**	*C2E.5 (Now C2E.4)*	pESA envelope is more easily defined for assessment based on overstrength forces rather than overstrength factor, because the only strength increase above the probable strength is due to strain hardening.	Replace notations on Figure C2E.2 with reference to forces rather than overstrength factors	Simplifying application of the document.
C2-**26**	*C2E.5 (Now C2E.4)*	Document currently silent on accidental eccentricity and concurrency for diaphragms.	Confirm that accidental eccentricity does not need to be considered, but that concurrency does. Text added to C2E4	Position on necessity of these agreed at workshop.
C2-**27a**	*C2E.4 (Now C2E.4.1)*	Current content of section is confusingly presented and erroneous in places (equation C2E.2)	Replace section with newly proposed alternative – see marked copy for detail.	Largely editorial. Minor reduction of 100%NBS demands for non-overstrength case to make S_p value consistent with use of μ = 1.25.

Appendix C2F

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**27**b	*C2F.2*	Currently references 10% accidental eccentricity, which is inconsistent with change C2-**6**a	Reword to state 5% and reference back to guidelines section C2.5.7.	Editorial to align with change C2-**6**a

Appendix C2G

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C2-**28**	*C2G.2*	Definition of a non-ductile axial force as $0.5A_g f_c'$ is inconsistent with reduction of this limit to $0.2A_g f_c'$ that was made in 2018 C5.	Change text to $0.2A_g f_c'$	Aligning C2 with C5-2018
C2-**29**	*C2G.8 (proposed new section)*	Wall axial “through-the-thickness” is described in C5-2018 as an SSW, but does not feature in C2G currently.	Add new Section C2G.8: *C2G.8 Wall through-the thickness failure*	Aligning C2 with C5-2018.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			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 same or slightly larger drift level compared with walls suffering 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.*	

Section C3 (2017)

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C3-**1**	*C3.1.3 Notation*	Definition for Kξ refers to "Section 0"	Change reference to "Section 3.3"	Editorial.
C3-**2**	*C3.1.3 Notation*	Definition given for π simply points to equations it is used in.	Does not require definition - delete from notation list	Remove circular reference.
C3-**3**	*C3.2*	Note states "*The basis for setting the ULS seismic demand for determining %NBS generally is the demand determined in accordance with the versions of the above documents that are current at the time the assessment is completed*." This is inconsistent with intended practice, which is to keep using 2004 actions for foreseeable future.	Delete first paragraph of note.	Aligning guidelines with intended practice
C3-**4**	*C3.4*	First paragraph refers to "Section 0"	Change reference to "Section 3.3"	Editorial.
C3-**5**	*C3.4 Equation C3.2*	Equation is currently specified to give answer in mm – better to present generically.	Change to $g.(T/2\pi)^2$	Editorial
C3-**6**	*C3.5*	Equation C3.3 suggests using total mass as default – this is unduly conservative, and inconsistent with other parts of the guidelines that (correctly) specify effective mass.	Change default to be based on effective mass as per section C2.4.2.	Editorial/improving technical accuracy and consistency.
C3-**7**	*C3.6*	Document is slightly ambiguous about whether the 2004 or 2016 vertical spectra should be used, with these differing significantly.	Clarify by adding note specifying use of 2004 spectra	Confirmed that this is the intent.
C3-**8**	*C3.7*	Specification to use 1170.5 scaling procedure is inconsistent with C2 which notes that other documents are more advanced.	Add sentence "*Alternative scaling procedures may also be employed provided their application is consistent with the intent of these guidelines*"	Reflects wide industry practice.
C3-**9**	*C3.7*	Direction to always apply the vertical earthquake component is liable to cause problems – most structural models are not appropriately	Change third paragraph to read: *All three components of any ground motion records should be ~~used where all components are~~*	Matches common practice.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
		conditioned to give realistic vertical shaking response.	*scaled by the same factor which is determined separately for each direction of application of the principal component. The two horizontal components should be applied simultaneously. The vertical ground motion component should additionally be implied if it is expected to significantly affect the analysis outcome.* Add note identifying challenge of modelling response to vertical earthquake motion. Point to (forthcoming) SESOC guidance on NLTHA as source of additional information.	
C3-**10**	*C3.10.2*	It is not clear how Sp should be applied for slama or NLSPA.	Note that Sp for NLSPA should be as per 1170.5.	Based on discussions with sub-group of reviewers. May be adjusted by deliberations of that group.

Section C5 (2018)

C5.1 including definitions and notation

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**1**	*C5.1.1*	First note is outdated and doesn't really add to document.	Delete note	
C5-**2**	*Non-ductile column*	Definition was not updated when non-ductile column provisions in C5.3.2 were improved in 2018.	Replace definition with: *Lightly reinforced concrete columns and/or beam-column joints that* • *Are at risk of non-linear response in flexure or shear* • *Have axial loads greater than 20% of the gross column capacity (i.e. $N^* \geq 0.2A_g f'_c$)* • *Are lightly reinforced as defined by equations C5.1 – C5.3, and* • *Have potential to lead to progressive collapse of the entire storey if they failed.*	Updated to match already agreed definition as amended by Change C5-**17**a.
C5-**3**	A_{co}	Notation is inconsistent with NZS 3101	Change to A_{oc}	Editorial – related to change for Equation C5E.12
C5-**4**	*c – Depth of compression zone*	Effectively duplicates following definition of neutral axis depth.	Delete	See related change to Figure C5.22 and Equation C5.68
C5-**5**	f_{st}	Notation no longer used	Delete	Used in C5-2017 Appendix C5H.1 which was removed in 2018 version
C5-**6**	k_{lb}	Notation no longer used	Delete	Used in C5-2017 Appendix C5H.1 which was removed in 2018 version
C5-**7**	Δ_u	Notation no longer used	Delete	

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**8**	α (first use)	Redundant – replicates α_1 later in notation	Delete	
C5-**9**	α (second use)	Ambiguous	Change to α_{col} to match figures C5.13 and C5.22	
C5-**10**	α_{sh} (new notation)	Notation for α in concrete contribution to shear (eq C5.65) is ambiguous and not listed	Change equation to use α_{sh} Add definition: *Parameter accounting for the influence of shear span on concrete contribution to shear strength.*	
C5-**11**	β_{sh} (new notation)	Notation for β in concrete contribution to shear (eq C5.65) is ambiguous and not listed	Change equation to use β_{sh} Add definition: *Parameter accounting for the influence of longitudinal reinforcement on concrete contribution to shear strength.*	
C5-**12**	γ_{sh} (new notation)	Notation for γ in concrete contribution to shear (eq C5.65) is ambiguous and not listed	Change equation to use γ_{sh} Add definition: *Parameter accounting for the influence of plastic deformation on concrete contribution to shear strength.*	
C5-**13**	ρ_ℓ	Notation used more broadly than just for beams, and used as total reinforcement ratio (note that ρ is used for tension reinforcement ratio).	Replace definition with: *Longitudinal reinforcement ratio, equal to* A_{st}/A_g.	
C5-**14**	ρ_t	Existing definition does not match usage in C5-2018	Replace definition with: *Transverse reinforcement ratio, =* $A_v/b_c s$	

Section C5.2 and C5.3

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**15**	*C5.2*	Number for section C5.2 is missing and has an extraneous carriage return preceding the heading.	Delete return, add number.	Editorial
C5-**16**	*Figure C5.2*	Preceding note box is missing reference to the figure.	Add reference to figure	Editorial
C5-**17a**	*C5.3.2*	Potentially unclear that all four conditions need fulfilling for column to be an SSW. Wording "not protected from flexural yielding" is a double negative, and confusing to some. Wording neglects that non-linear shear behaviour is equally problematic. Stating "failure would lead to progressive collapse" is problematic due to difficulty of predicting progressive collapse.	Add "all of" before conditions, and "and" at end of each bullet. Change wording to "Are at risk of non-linear response in flexure or shear" Change to "Failure has potential to lead to progressive collapse.	Editorial
C5-**17b**	*C5.3.2 – second note*	Currently confusing where it states "…is not intended to be applied to the probable lateral flexural…" Also warrants clarification that joints are only an SSW if the column is not capacity protected – i.e. that the joint can be weaker than the beam or column and still not be an SSW.	Change note to read *The SSW requirements for non-ductile columns are intended to identify and significantly penalise lightly reinforced columns in situations where gravity loads cannot be redistributed and that are susceptible to axial failure and loss of gravity load support. The penalty factor of 2 is intended to be applied to the probable deformation at onset of loss of gravity, Δ_f/L_c, as defined in section C5.5.4. It is not intended to be applied in cases where in the event of column failure gravity loads can be redistributed to other parts of the structure. Nor is it intended to be applied to the probable deformation capacity. Δ_{cap}/L_c, calculated in accordance with Section C5.5.3, although the probable deformation capacity for SSW columns may be reduced because the probable deformation capacity should not be*	Previous discussions with Rob Jury have confirmed the intent of how the SSW is to be applied to beam-column joints, which is consistent with the wording of the provisions but not currently clear.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			taken as greater than the deformation at onset of loss of gravity load carrying capacity including the reduction from the penalty factor. *Equations C5.2 and C5.3 are based on 50% of the requirement for confining reinforcement specified in NZS 3101:2006 for columns not required to exhibit ductility.* *Columns should be considered at risk of flexural yielding unless the joint they connect to is expected to form a beam sway (strong column/weak beam) mechanism.* *The non-ductile column SSW applies to beam-column joints only when flexural yielding of the adjacent columns is not precluded. The joint itself does not need to be capacity protected, i.e. to have a shear strength in excess of the demands that could develop.*	
C5-**17c**	*C5.3.3*	Failure mechanisms for shear walls does not refer to through-the-thickness failure.	Add new sentence to end of paragraph: *An additional failure mechanism not shown in Figure C5.3, through-the-thickness failure (Zhang et al. 2018), also requires consideration using the methods set out in Section C5.5.4.4.*	Editorial

Section C5.4

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**18**	*Table C5.3*	No guidance provided on default concrete strength prior to 1970. Guidance on how to handle strength of poor quality concrete is limited.	Change so that first row ($f_c' = 20MPa$) applies to 1930 to 1981. Add content to note following table noting that pre-1930s concrete is likely to require testing, and that anecdotal evidence shows that low	Numerous studies show that older concrete structures generally have actual concrete strengths of 30 MPa or greater. Prior to 1930 there is little data about expected strength, but anecdotal

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			strengths (e.g. 10 MPa) are commonly encountered. Add words "or other signs that the concrete is of poor quality" to sentence discussing use of lower concrete strength if poor compaction is observed. Move note regarding use of f_c' notation so that it follows first paragraph of C5.4.2.2	evidence that poor concrete with low strength is common.
C5-**19**	*C5.4.3*	Some references to reinforcement "grade" are not capitalised. Two sequential notes without content between	Capitalise "Grade" in two places. Merge notes	Editorial
C5-**20**	*C5.4.3*	Guidelines do not currently address corroded structures	Add content to note after Table C5.4 pointing to Natraj et al. as a source of guidance on how corrosion affects the strength and ductility of reinforcement.	Paper provides a specific focus on corrosion in the context of NZ assessment guidelines: Nataraj, S., Hogan, L., Scott, A., and Ingham, J. (2022). "Simplified Mechanics-Based Approach for the Seismic Assessment of Corroded Reinforced Concrete Structures." Journal of Structural Engineering, 148(3), pp.04021296.
C5-**21**a	*C5.4.4 Equation C5.6*	Currently ambiguous as to whether L_d (required development length) should be calculated using nominal or probable yield strength.	Add text stating that probable yield strength should be used to calculate L_d	Editorial (the equation gives practically the same answer irrespective of whether nominal or probable strength is used, provided that this is consistent between f_y and L_d used in the equation)
C5-**21**b	*C5.4.4*	Note currently implies that elements with plain bars may fail before developing yield strength.	Add comment linking to Section C5.5.2.2 on flexural strength, and to Direct Rotation limits	Editorial/consensus view of balloting committee.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**22**	*C5.4.4*	Guidelines do not currently address corroded structures	Add content to note under equation C5.6 pointing to Natraj et al. as a source of guidance on how corrosion affects bond strength. Adapt Table C6.1-4 from fib Model Code 2010 to give more direct source of information than is provided in Nataraj et al. Include rationalisation of this table to remove odd steps in the fib data where the same crack width gives two different bond reductions depending on corrosion depth.	Paper provides a specific focus on corrosion in the context of NZ assessment guidelines: Nataraj, S., Hogan, L., Scott, A., and Ingham, J. (2022). “Simplified Mechanics-Based Approach for the Seismic Assessment of Corroded Reinforced Concrete Structures.” Journal of Structural Engineering, 148(3), pp.04021296.
C5-**23**	*C5.4.4 Equation C5.7*	Approach of reducing effective splice length is incompatible with proposed approach for calculating rotation capacity of splice-controlled columns. Leaving the equation in creates confusion.	Delete equation C5.7 and related text.	Accepted Opabola & Elwood change proposal for ACI 369 that also defines new approach for splice-controlled columns as discussed in changes to C5.5.3 and C5.5.4.
~~C5-**24**~~	~~*Equation C5.7*~~	~~Current document does not clarify that equation C5.7 should not be applied to plain round reinforcement.~~	~~Add note:~~ ~~*The flexural strength of columns reinforced with spliced plain bars should be assessed based on the tension stress, f_splice, calculated according to equation C5.6. No reduction of effective splice length (equation C5.7) need be considered.*~~	Change redundant given deletion of equation
C5-**25**	*C5.4.5.2 Table C5.6*	Table indicates grout sleeve behaviour that does not match available experimental data. Note 3 erroneously refers to tensile strength when it should refer to yield strength.	Change both NMB and Reid grout sleeve capacity to read: *Probable tensile strength with allowance for construction inadequacies*[4] Add new note 4 stating: *Unpublished testing of specimens extracted from existing structures has shown that, when properly constructed, grout sleeves are able to sustain the tensile strength of coupled reinforcing bars. However, the testing also shows that a high proportion of groutsleeves are inadequately constructed, either through insufficient insertion*	Refer to accompanying note 99003.05 setting out method used to consider grout sleeve test results. Results were for Reid grout sleeves. However, there is no reason to think NMB would perform differently given behaviour is construction/QA related. Reference is to Brooke, N. J. (2024). “Updating New Zealand’s Guidance for Seismic Assessment of Existing Concrete Buildings.” Proc. NZSEE Conference, New Zealand Society for Earthquake

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			of bars or inadequate grouting. Both types of deficient construction result in failure prior to achievement of the bar yield strength. *Unless specific investigation is undertaken, it is recommended that 50% of grout sleeves in an element are considered ineffective. The ineffective grout sleeves may be assumed to be approximately uniformly distributed. Where it is necessary to consider the overstrength capacity, all grout sleeves at a section should be considered effective.* *Testing of grout sleeves is complex and expensive. There are no obvious non-destructive methods for considering adequacy of construction. Caution should be exercised in relying on the assumption that 50% of grout sleeves are effective when the grout sleeves are critical to the capacity of a load path. This is particularly the case when failure of a few (1-3) grout sleeves could significantly affect assessment outcomes.* *Further detail can be found in Brooke (2024).* Change note 3 to refer to yield strength (same change also applies to note 2 under table C5.7).	Engineering, Wellington, New Zealand, 15p.
C5-**26**	*Note under Table C5.6*	Extensive new material published looking at Drossbach ducts	Add reference to SESOC task group papers: *Additional guidance on assessment of Drossbach ducts can be found in the outputs from a recent SESOC working group (Henry 2022; Holliss and Traegar 2022). Henry's paper (2022) is focussed on recommendations for new construction, but may be of assistance in guiding assessments.*	Henry, R. S. (2022). Grouted Connections and Drossbachs - GD#3: Design Basis and Worked Examples (Draft for Public Comment). SESOC, Auckland, New Zealand, 40p. Holliss, B., and Traegar, D. (2022). Grouted Connections and Drossbachs - GD#1: Existing (Legacy) Buildings - Investigation and Remediation (Draft for Public Comment). SESOC, Auckland, New Zealand, 30p.

Section C5.5

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**27**	*Equation C5.8 (now Equation C5.12)*	May be unclear how equation should be applied for SSW columns	Add clarification of how the factor of 2 penalty should be included when checking equation C5.12 (Equation moved to new note after Δ_f is defined as discussed in change below)	Based on presumption that the factor of 2 penalty includes allowance for "greater than ULS shaking" and that this should not be cumulative with the 2/3 factor.
C5-**28**	*Equation C5.8*	It is not clear whether Equation C5.8 would be applied to elements (such as secondary columns or flat slab connections) that are not relied on to resist lateral forces. This could lead to the full drift limit at axial failure being compared to a ULS drift demand. While some calibration is built into plastic rotation values (see below) this could lead to there being very little margin for scenarios (e.g. heavily loaded slabs or columns) where there is little or no plastic deformation capacity before axial failure is expected. There is also double counting of how Δ_f is reduced to give a reasonable value for ULS. Currently,. θ_a values are reduced relative to ASCE CP acceptance criteria to account for the use of ULS rather than MCE demands, as well as providing the 2/3 factor in the current Eq C5.8.	Move Equation C5.8 to after definition of Δ_f (i.e. close to current equation C5.12). Define new notation $\Delta_{\mathrm{f,ULS}}$ as the displacement at axial failure appropriate to be compared to ULS demands. Set $\Delta_{f,ULS} = \frac{2}{3}\Delta_f$ Make other changes in Section C5.5.4 so that θ_a values are equivalent to ASCE CP acceptance criteria.	Proposed approach is equivalent to checking ASCE 41 CP acceptance criteria against demands that are 1.5 times ULS values.
C5-**29**a	*C5.5*	Text discussing onset of loss of gravity describes Δ_f as "axial failure deformation"	Replace with "deformation at the onset of loss of gravity load capacity". Other language changes to improve readability	Editorial
C5-**29**b	*C5.5.1.4 Strength reduction factors*	Text regarding inclusion of factor in shear capacity equations is outdated due to change C5-**54**a	Change to "where considered necessary, a factor to provide a safety margin against undesirable failure <u>should be</u> included…" Add note stating *In previous editions of Part C5, factors to provide a safety margin against undesirable failures were*	Consistency with

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			included in the derivation of shear capacity equations. These have been removed as a general requirement so that the result of the calculation is a probable value in accordance with the philosophy of these guidelines. Sufficient margin against undesirable behaviour is generally achieved by consideration of factors such as overstrength and dynamic amplification of shear forces as recommended elsewhere in these guidelines. *It remains appropriate to apply reduction factors to shear strength where only elastic analysis without a mechanism check is undertaken, though such limited analysis is not recommended by these guidelines.*	
C5-**30**a	*C5.5.1.7 Effective stiffness*	There is currently inconsistency between NZS 3101 I_e values and the elastic deflections given by C5.5.3.1. This is largely due to the β_v factor. C5.5.1.7 also does not acknowledge that stiffness values could be derived from yield curvature given in C5.5.1.7.	Alter C5.5.1.7 to clarify that effective stiffnesses may be based on NZS 3101, or derived from the probable yield curvature as defined in C5.5.3.1. Add comment that NZS 3101 values for beams and columns should be adjusted by β_v (NZS 3101 already includes β_v for squat walls. Modify note to remove statement that stiffnesses based on effective yield curvature should not be used.	The β_v factor was calibrated against experimental data so can be viewed as more accurate than 3101 table. Application of β_v to fixed I_e values is also consistent with content from Opabola & Elwood 2023: E A Opabola and K J Elwood, 'Flexure-Axial-Shear Interaction of Ductile Beams with Single-Crack Plastic Hinge Behaviour', Earthquake Engineering & Structural Dynamics, 14 March 2023
C5-**30**b	*C5.5.2.2 - Lap splices of deformed bars or hooked plain bars*	Section does not consider walls, for which direct rotation limits are proposed to be added.	Add walls to first bullet point	Consensus view of balloting committee.
C5-**30**c	*C5.5.2.2 – plain bar lap splices*	Section does not consider walls, for which direct rotation limits are effectively identical to columns.	Add "and walls" to bullet relating to $24d_b$ splice length	Consensus view of balloting committee.
C5-**31**	*C5.5.3.1 Probable yield curvature*	Currently no consideration of beams.	Add language to state that beams should be treated as for columns with no axial load.	Consistent with Opabola and Elwood (2023) as referenced above.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
		Application of β_v may underestimate stiffness of elements where yielding is not expected, e.g. columns in a strong column weak beam frame.	Add line indicating that β_v should be taken as 1.0 for elements not expected to yield based on the mechanism of the structure. Add paragraphs to note stating *The factor β_v accounts for non-flexural deformations that significantly reduce the stiffness of members subjected to large deformations. These include shear deformations and bar slip. Background on the β_v factor can be found in Opabola & Elwood (2020).* *Application of β_v to elements that are not expected to yield is likely to underestimate the stiffness of a structure. Consideration of whether elements are expected to yield should be based on the mechanism predicted to form in the structure. For example, in a strong-column/weak-beam frame the columns could be assumed not to yield. It is not intended that iterative checking should be undertaken to adjust the expected stiffness of the structure based on the demands predicted for particular elements at a particular level of earthquake demand.*	
C5-**32**	*C5.5.3.1 Probable yield curvature*	Confusion can arise with reference to "effective height" for walls.	Clarify that this reference is to the ratio of moment to shear demand on the specific wall.	Editorial
C5-**33**	*Equation C5.24*	Text preceding the equation states it relates to probable rotation capacity (i.e. elastic plus plastic) when it should only be the plastic component.	Change text to read "*On this basis, the <u>plastic</u> rotation capacity…"*	Editorial.
C5-**34**	*Equation C5.24*	Notation erroneously requires that yield strain not be taken as greater than 0.0002	Change to 0.002	Existing value is a typo
C5-**35**	~~*Changed reverted*~~	~~Change reverted~~	~~Change reverted~~	

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**36**	*C5.5.3.3 – Beams reinforced with deformed bars.*	Text just above equation C5.30 stating "*For beams that comply with the limited ductile or ductile detailing requirements of NZS 3101:2006*" is vague.	Change to *For beams that have longitudinal and transverse reinforcement detailing that complies with the limited ductile or ductile requirements of NZS 3101:2006 1:2006 for:* *• Longitudinal reinforcement quantity, and* *• Transverse reinforcement quantity, and* *• Transverse reinforcement spacing along the beam*	Editorial clarification
C5-**37**	*Equation C5.30*	Notation erroneously requires that yield strain not be taken as greater than 0.0002	Change to 0.002	Existing value is a typo
C5-**38**	*C5.5.3.3 – Beams reinforced with deformed bars.*	ACI 369 now provide separate equation for beams controlled by inadequate splices.	Add new section "Beams reinforced with deformed bars controlled by inadequate splices"	ACI 369.1-22 is a consensus Standard.
C5-**39a**	*C5.5.3.3 – Probable rotation capacity of columns*	Note in this section is redundant if earlier change (C5-28) is adopted.	Delete note.	Editorial if earlier change (C5-28) is adopted
C5-**39b**	*C5.5.3.3 – Columns with deformed bars not controlled by inadequate splices*	The equation provided is conservative for well confined circular columns.	Add paragraph to note stating: *The rotation capacity of modern, well confined circular columns may be underestimated by Equation C5.34. If this is important to the outcome of an assessment it is recommended to calculate the rotation capacity using NZS 3101 instead of Equation C5.34.*	
C5-**40**	*C5.5.3.3 – Columns with deformed bars controlled by inadequate splices*	Current provisions for rotation capacity of columns controlled by inadequate splices do not account for influence of axial load or splice length.	Implement approach outlined in accepted change proposal for ACI 369 developed by Opabola and Elwood.	ACI 369 is a consensus Standard. Approach also outlined in Opabola & Elwood 2021: Opabola, E. A., and Elwood, K. J. (2021). "Seismic Assessment of Reinforced Concrete Columns with Short Lap

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
				Splices." Earthquake Spectra, 37(3), pp.1726–1757.
C5-**41**	*Equation C5.37*	Use of α is ambiguous	Change to α_{col} in equation and notation Also add formula giving the value of α_{col}	Editorial
C5-**42a**	*C5.5.3.3 – Probable rotation capacity of walls*	Section provides no guidance for connections of singly reinforced walls subjected to out-of-plane demands.	Add new sub-section, along with intro note in opening of section.	Limit proposed based on Hogan, L. S., Henry, R. S., and Ingham, J. M. (2023). "Out-of-Plane behaviour of dowel type precast Panel-to-Foundation connections." Structures, 58, pp.105447.
C5-**42**b	*C5.5.3.3*	Section not clear on what the in-plane rotation capacity of singly reinforced walls should be taken as. Leads to over-conservative reference to NZS 3101 in some assessments.	Add note clarifying that rotation limits apply equally to singly and doubly reinforced walls. Emphasise also that singly reinforced walls can sustain non-linear deformation despite what is indicated by NZS 3101 design provisions.	Consensus view of the balloting committee, and source material does not distinguish between singly and doubly reinforced walls.
C5-**42c**	*Figure C5.13*	Existing use of c as notation for compression zone depth is confusing	Change equation, notation, and related figure to refer to stress block depth, a	Priestley et al. (2007) states "the axial force is…applied…through the centre of flexural compression". This is more consistent with the stress block depth, a, than the neutral axis depth, c.
C5-**43a**	*Equation C5.42*	Notation erroneously requires that yield strain not be taken as greater than 0.0002	Change to 0.002	Existing value is a typo
C5-**43**b	*Equation C5.42*	Recent University of Canterbury research (Pollalis et al.) shows that even staggered lap splices reduce the deformation capacity of wall plastic hinges.	Add comment box noting that lap splices may reduce rotation capacity and point to Pollalis paper..	Refer Pollalis, W., Kerby, C., and Pujol, S. (Submitted for publication). "On Estimating the Drift Capacity of Reinforced Concrete Walls with Lap Splices at their Bases." Bulletin of Earthquake Engineering.
C5-**44**	*Table C5.8 and Table C5.9*	Heading of third column states $V_s \geq 0.75V_y$ where it should have $V_s \leq 0.75V_y$	Change to $V_s \leq 0.75V_y$	Change matches ASCE 41-17 definition of conforming reinforcement per note d to Table 10-19

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**45**	*Strain penetration length*	Current value of $L_{sp} = 0.022 d_b f_y$ does not account for important variables that affect the strain penetration length [1]	Adopted proposed value from Opabola and Elwood[1]: $$L_{\mathrm{sp}} = \frac{\left(\frac{f_{\mathrm{u}}}{f_{\mathrm{y}}} - 1\right)}{4\sqrt{f_c'}} f_{\mathrm{y}} d_{\mathrm{b}}$$ Define only at equation C5.51 and then reference other uses to that equation.	See referenced paper. Also considered proposals by Engstrom as referenced by Davey and Blaikie[2] and Goodnight et al.[3]. Engstrom's proposal may not be calibrated for cyclic loading. Goodnight et al. include shear span in their proposal which may double count with β_v factor.
C5-**46**	*C5.5.4 – Note at start of section.*	No link back to requirements set out at the beginning of section C5.5 regarding comparison of drift at onset of loss of gravity load capacity to ULS demands.	Expand note with new first paragraph that states: *The deformation capacities calculated using the methods presented in this section are not appropriate for direct comparison with ULS demands that are used as the basis of %NBS earthquake scores. Instead, they should be used to derive the deformation at the onset of loss of gravity load capacity for comparison against ULS demands, $\Delta_{f,ULS}$, using equation C5.12.*	Editorial provided earlier change (C5-28) is adopted.
C5-**47**	*C5.5.4.1*	There is currently double counting in reducing loss of gravity deformations from ASCE CP values to values comparable to ULS demands as discussed in relation to change C5-28	Alter formulae and accompanying text so that values given are equal to ASCE b value. Add reference back to equation C5.12b to end of note to ensure clarity.	Provides a consistent way of comparing ASCE based values to ULS demands.
C5-**48**	*C5.5.4.2 – Columns with deformed bars controlled by inadequate splices*	Current provisions for rotation capacity of columns controlled by inadequate splices do not account for influence of axial load or splice length.	Implement approach outlined in accepted change proposal for ACI 369 developed by Opabola and Elwood. Similar recalibration as for the above change to remove double counting.	ACI 369 is a consensus Standard. Approach also outlined in Opabola & Elwood 2021: Opabola, E. A., and Elwood, K. J. (2021). "Seismic Assessment of Reinforced

[1] E A Opabola and K J Elwood, 'Flexure-Axial-Shear Interaction of Ductile Beams with Single-Crack Plastic Hinge Behaviour', *Earthquake Engineering & Structural Dynamics*, 2023, eqe.3873.
[2] R Davey and E L Blaikie, 'On the Flexural Ductility of Very Lightly Reinforced Concrete Sections', in *Proceedings of the NZSEE Conference* (NZSEE Conference, Wairakei, New Zealand: New Zealand Society for Earthquake Engineering, 2005), 6.
[3] Jason C. Goodnight, Mervyn J. Kowalsky, and James M. Nau, 'Modified Plastic-Hinge Method for Circular RC Bridge Columns', *Journal of Structural Engineering* 142, no. 11 (November 2016): 04016103, https://doi.org/10.1061/(ASCE)ST.1943-541X.0001570.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
				Concrete Columns with Short Lap Splices." Earthquake Spectra, 37(3), pp.1726–1757.
C5-**49**	*C5.5.4.3 – Columns with plain bars*	Current specification that $\theta_a = 2\theta_p$ is non-conservative compared to Opabola and Elwood change proposal accepted for ACI 369	Change so that ratio of $\theta_a/\theta_p$is dependent on axial load as per change proposal. Add note identifying that columns with plain bars could be an SSW.	ACI 369 is a consensus Standard.
C5-**50**	*C5.5.4.4*	Currently little guidance on how to assess drift at loss of gravity load carrying capacity for walls.	For flexural walls, implement a version of the approach adopted in ACI 369-22 but improved to remove the need for two-variable interpolation, and to remove an ambiguity about how to deal with lightly reinforced walls. For walls controlled by shear – ACI 369 procedure not adopted as not improved from ASCE 41-17, which was not considered appropriate at last revision of C5.	ACI 369 is a consensus Standard. Proposed equation gives values that are range between 96% and 102% of the values tabulated in ACI 369. Awaiting feedback from authors of ACI procedure regarding the lightly reinforced wall ambiguity.
C5-**51**	*C5.5.4.4*	Second equation currently states $$0.08 < \frac{(A_s - A_s')f_y + N^*}{t_w l_w f_c'} \ll 0.3$$ The "<< 0.3" is extraneous and confusing as failure should be identified for any case greater than 0.08. Method can currently catch walls with very low axial load.	Remove "<< 0.3" Change presentation to put values in a table as current approach is unclear. Add limit that axial load ratio must exceed $0.01A_g f_c'$	No logic to including the proposed deletion. Confirmed with Ken Elwood/Rick Henry. Added following balloting committee discussions
C5-**52**	*C5.5.4.4 note*	Currently unclear that failure drift is to be taken as the drift at loss of axial capacity.	Move content in note to explicit statement regarding how to calculate drift at loss of gravity load capacity.	Editorial
C5-**53**	*C5.5.4.5*	There is currently double counting in reducing loss of gravity deformations from ASCE CP values to values comparable to ULS demands as discussed in relation to change C5-28	Alter formulae and accompanying text so that values given are equal to ASCE b value.	Provides a consistent way of comparing ASCE based values to ULS demands.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			Add reference back to equation C5.12b to end of note to ensure clarity.	
C5-**54**a	*C5.5.5.1/Equation C5.62 (now C5.64)*	0.85 reduction factor on shear strength results in unintended consequences such as reducing likelihood of predicting actual behaviour.	Replace 0.85 factors applied to shear strength with new variable ϕ_v, equal to 1.0 unless elastic analysis without a mechanism check is undertaken in which case $\phi_v = 0.85$. Add content to note stating that 0.85 factor should be applied if no mechanism check is undertaken.	Consensus view of balloting committee.
C5-**54**b	*Equation C5.63 (now C5.65)*	Use of $0.8A_g$ as shear area is not correct if element has large flanges or boundary elements.	Change to b_wd, and note that d can be taken as 0.8 times section depth.	More appropriate measure of shear area.
C5-**55**	*Equation C5.63 (now C5.65)*	Notation used is ambiguous	Add subscript "sh" to α, β, and γ in the equation, notation, and figure.	Editorial
C5-**56**	*Equation C5.69*	Use of α is ambiguous	Change to α_{col} in equation and notation	Editorial
C5-**57**	*Figure C5.22 and Equation C5.70*	Existing use of c as notation for compression zone depth is confusing	Change equation, notation, and related figure to refer to stress block depth, a	Priestley et al. (2007) states "the axial force is…applied…through the centre of flexural compression". This is more consistent with the stress block depth, a, than the neutral axis depth, c.
C5-**58**	*C5.5.6*	Guidelines do not currently address corroded structures	Add brief section C5.5.6 pointing to Nataraj et al. (2022) and include their flow chart setting out how to adjust the guidelines to cover corrosion.	Paper provides a specific focus on corrosion in the context of NZ assessment guidelines: Nataraj, S., Hogan, L., Scott, A., and Ingham, J. (2022). "Simplified Mechanics-Based Approach for the Seismic Assessment of Corroded Reinforced Concrete Structures." Journal of Structural Engineering, 148(3), pp.04021296.

Section C5.6

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**59**	*C5.6.1*	Use of Φ = 0.75 for strut-and-tie is inconsistent with other aspects of the guidelines where effectively Φ = 0.85 is used for shear.	Change so that second paragraph concludes: *However, the strength reduction factor shall be taken as:* $\phi = 1.0$ *where actions applied to the strut-and-tie model have been derived based on capacity design principles* $\phi = 1.0$ *for diaphragms, in accordance with Section 5.6, and* $\phi = 0.85$ *for all other cases.*	Consistent with approach to strength reduction factor for shear.
C5-**60**a	*Equation C5.79 and Equation C5.85*	0.85 reduction factor on shear strength results in unintended consequences such as reducing likelihood of predicting actual behaviour.	Replace 0.85 factors applied to shear strength with new variable ϕ_v, equal to 1.0 unless elastic analysis without a mechanism check is undertaken in which case $\phi_v = 0.85$. Add content to note stating that 0.85 factor should be applied if no mechanism check is undertaken.	Consensus view of balloting committee.
C5-**60b**	*Note below equation C5.81*	Fourth paragraph starts with an incomplete sentence – hangover from earlier edits.	Delete *as well as* ρ_t *versus drift presented in literature and based on extensive experimental tests.*	Editorial
C5-**61**	*Note below equation C5.81*	Joint deformation values are currently ambiguous for some types of joint.	Move values to new table and reword descriptions for clarity.	Editorial. Questions have been raised about some of the values given. However they appear consistent with available literature, and no advice has been received on why they should be changed.
C5-**62**	*Equation C5.91*	Equation erroneously uses beam height, h_b, in the denominator	Replace with h_c	Error correction

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
~~C5-**63**~~	~~*Equation C5.92 and C5.93*~~	~~The equations are currently unfactored best fit values taken from the underpinning research.~~ ~~Equations should have 0.85 factor applied in common with other approaches to joint shear strength in the guidelines~~	~~Add 0.85 factor to both.~~	Change deleted due to removal of 0.85 factors on shear. ϕ_v factor not required for these equations because you cannot determine the plastic rotation without a mechanism check.
C5-**64**	*Equation C5.96*	Equation erroneously uses beam height, h_b, in the denominator	Replace with h_c	Error correction
C5-**65**	*Equation C5.99*	Equation erroneously uses beam height, h_b, in the denominator	Replace with h_c	Error correction
C5-**66**	*C5.6.3.1*	Section only recommends strut-and-tie analysis as method of determining strength. Other techniques that also consider non-linear tension/compression behaviour of concrete are also valid.	Add references to "*other equivalent non-linear analysis in which the tensile behaviour of concrete is reasonably represented*" to the text and note.	Makes text consistent with accepted industry practice
C5-**67**	*C5.6.3.3 Step 4*	Description is too specifically focussed on grillage method.	Replace text with: *Develop appropriate analysis model for the diaphragm, which may comprise a strut-and-tie model, a grillage model, or other appropriate non-linear analysis (for example an implementation of compression field theory/rotating strut methods).*	Makes text consistent with accepted industry practice
C5-**68**	*C5.6.3.3 Step 5*	Reference to building overstrength factor is inconsistent with proposed changes to C2	Change to read: *Develop pESA envelope in accordance with Section C2 and use it to calculate diaphragm force at each level.*	
C5-**69**	*C5.6.3.3 Step 6*	Description is too specifically focussed on grillage method.	Replace "grillage" with "diaphragm analysis". Clarify that forces come from the pESA analysis.	Makes text consistent with accepted industry practice

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**70**	*C5.6.3.3 Step 7*	Description is too specifically focussed on grillage method.	Replace “grillage” with “diaphragm analysis”. Clarify that forces come from the pESA analysis.	Makes text consistent with accepted industry practice
C5-**71**	*C5.6.3.3 Step 11*	Refers to diaphragm inertia forces, when in fact the forces referred to are a combination of inertia and transfer.	Change reference to “pESA envelope and hence diaphragm forces”	Editorial
C5-**72**	*C5.6.3.3 Step 8*	Description is too specifically focussed on grillage method.	Delete “the grillage model”	Makes text consistent with accepted industry practice
C5-**73**	*Step 12*	Basis of current equation is unclear, and perceived as creating an unjustified step function.	Change approach to direct comparison of pESA envelope associated with the capacity of the diaphragm to the overstrength and upper bound envelopes as defined in change proposal for C2. Add note explaining that the step function is justified, and akin to the step functions that arise where minor changes of transverse reinforcement change behaviour of element from ductile flexure to shear failure.	Outcome of proposed approach should be similar/identical to current, but more transparent.
C5-**74**	*Step 12*	Queries have been raised regarding whether K_{dia} applies to ductile collector elements	Following discussions, redefine K_{dia} as: 2.0 for diaphragm collector elements within diaphragms that are an SSW in accordance with Section C2G.5, 1.5 for diaphragm collector elements that connect to a vertical element that resists more than 25% of the storey shear force at the storey above or below the diaphragm, 1.0 for diaphragm elements not included in either of the preceding categories.	Aim is to focus on collector elements that are critical to the performance of the diaphragm.
C5-**75**	*Steps 15-17*	These steps refer to “%NBS values” rather than Earthquake Scores.	Change references to “Earthquake Score”	Editorial to align with language specified elsewhere in the guidelines.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**76**	*C5.6.3.5*	Clarification needed about how to account for interaction of moment and axial forces in diaphragm elements.	Add note identifying that axial and flexural forces should be considered together – e.g. analyse as a column, or assume compression reinforcement is available as chord.	Agreed approach at workshop in March.
C5-**77**	*C5.6.3.6*	Current provisions for extent of diaphragm cracking are overly conservative, and unnecessarily cross reference to C5E. Reference to C5E.3 is unduly inconvenient. Should have the provisions complete in the clause. The specification that the "wide crack"/elongation zone is very commonly L/2 at either end of a beam is hard to credit, and makes determination of a diaphragm load path almost completely impossible for many 80s/90s buildings. At a minimum – wide crack development should be linked to drift.	Reconfigure wide crack provisions based on Mike Parr research. • Reduce lengths to 1-2 h_b depending on position/configuration • No impact for drifts less than yield • Where crossed by deformed bars – pro-rata from there to full disconnection at 3% drift.	Largely based on Parr, M. (2023). Retrofit Solutions for New Zealand Hollow-Core Floors and Investigation of Reliable Diaphragm Load-Paths in Earthquakes (PhD Thesis). The University of Canterbury, Christchurch, New Zealand, 636p.
C5-**78a**	*C5.6.3.6*	Parts (b) and (c) of figure above heading for Section C5.6.4 are not referred to, and potentially confuse users.	Delete parts (b) and (c) Remove (a) sub-caption Change caption to *Plan on a part floor showing location of cracks and areas where shear can be transferred to perimeter frames* Change references to just point to Figure 5.40 – i.e. no (a)	Editorial clarification
C5-**78b**	*C5.6.4*	Section does not provide guidance on rotation capacity of panel connections.	Edit note to link back to new content added in Section C5.5.3.3 (Change C5-**42**a)	Editorial

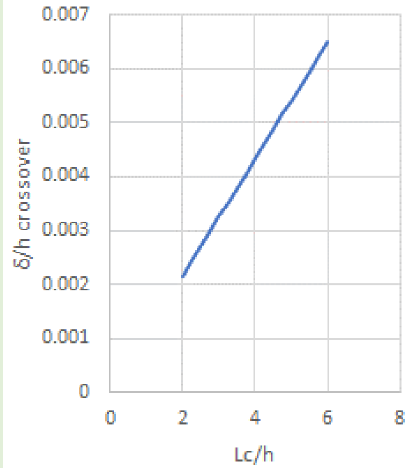
References and Bibliography

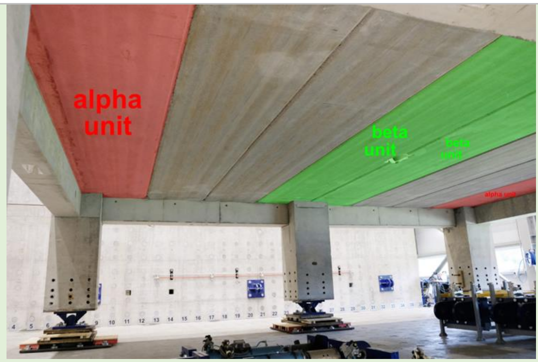
#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**79**	*Various*	Some references are repeated	Remove repeated references	Editorial

Appendix C5E

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**80**	*C5E.1*	Reference to “modern support detailing” is inconsistent with current advice that there is no good detail for support of hollow-core.	Delete “*, unless modern support detailing is provided (post-NZS 3101:2006 detailing with low friction bearing strip)*”	SESOC/NZSEE/ENZ position on hollow-core support, and change made to B1/VM1.
C5-**81**	*C5E.1*	“Factor of 2” is currently: 1- Lacking prominence given its importance to the outcomes 2- Somewhat ambiguous in the reference to “brittle failure modes” and listing some but not necessarily all modes. 3- Excessively conservative when compared to	Add new subheading. Reword to increase prominence. Explain basis of procedures is to estimate loss of reliable gravity load path, and use of 2 mm drop metric. Clarify language to emphasise that factor applies to all failure types. Change factor from 2 to 1.5 (hollow-core/double tee) and 1.25 (flat slab/rib-infill) and add commentary explaining background to this. Delete paragraph referring to assessment in accordance with NZS3101 – this seems unnecessary and potentially confusing. Move note on background of material to before the new subheading See marked copy of C5E for detailed changes.	Refer to Brooke, N. J. (2024). “Updating New Zealand’s Guidance for Seismic Assessment of Existing Concrete Buildings.” Proc. NZSEE Conference, New Zealand Society for Earthquake Engineering, Wellington, New Zealand, 15p.
C5-**82**	*C5E.2.1*	Last sentence of second bullet is repeated	Delete “*Inspection for paperclip must be done when the seating is identified to be less than 20mm*”	Editorial

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C5-**83**	*C5E.2.1*	Unresolved 2018 editorial suggestion to note that seating length variation arises due to creep and shrinkage as well as poor construction.	Add "and post-installation creep and shrinkage of units" to end of final bullet point	Editorial
C5-**84**	*C5E3.1*	Wording about lightly restrained R2 hinges is unclear.	Change wording to: *The restraint for R2 hinges relies on the continuity bars spanning over the adjacent transverse beam. Consequently the R2 hinge should be considered unrestained if any of the following are true:* • *There is no continuity reinforcement over the transverse beam, or* • *The continuity bars are light (i.e. D12@600 or lighter), or* • *Continuity bars are too short to be adequately developed.*	Addition of case where continuity bars are too short is done on logical grounds that undeveloped bars cannot provide restraint.
C5-**85**	*C5E.3.1*	Suggestion in note box that the minimum elongation should be taken as $0.005h_b$ irrespective of frame demand is punitive for low drift structures with small seating lengths.	Change Equation C5E.2 to use total rotation instead of plastic rotation. Add requirement that elongation for reversing plastic hinges not be taken as less than geometric elongation, i.e. $\frac{\theta_{\mathrm{m}}}{2}(d-d')$. Change note to explain basis of changes: *Unidirectional plastic hinges experience geometric elongation, i.e. elongation that occurs due to the fact that tension strains induced by flexure of reinforced concrete elements are larger than the corresponding compressive strains. Equation C5E.2 provides an estimate of geometric elongation, and is the same as the equation used in NZS 3101:2006 (A3) to estimate the elongation of unidirectional plastic hinges.* *Greater elongation is expected in reversing plastic hinges as is suggested by Equation C5E.1. Plastic*	Change of equation C5E.2 required for consistency with new minimum elongation. There appears to be no background basis for using plastic rotation in this equation anyway. Approach has been validated by Frank Büker based on data from the ReCast super-assembly tests. For interest – the point where reversing hinge elongation crosses over with geometric elongation depends on the shear span to depth ration. For spans likely to be parallel to precast units the crossover is in the range $0.005h_b$ to $0.007\ h_b$ as shown below.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			rotations are used here to estimate beam elongation given good agreement with experimental data (Marder et al., 2018). Note, that NZS 3101:2006 (A3) uses the same equation but with total rotation, instead of plastic rotation, which provides a more conservative assessment of beam elongation. The requirement of equation C5E.1 that the elongation not be taken as less than the geometric elongation reflects the fact that elongation occurs prior to the occurrence of yielding and plastic rotation. This requirement replaces the previous specification of a minimum elongation equal to 0.5% of beam depth which was unduly punitive for situations where low drifts were considered with short seating lengths.	
C5-**86**	*Location of plastic hinge elongation*	Third paragraph of note is ambiguous – it implies that typical conditions give an elongation zone of L/4, which is not correct for some unrestrained plastic hinges.	Rewrite as below to clarify intent: *Elongation zones of $l_e = L/4$ recommended in Figure C5E.5 are based on certain typical conditions. Where these do not apply, the recommendation of $l_e = L/4$ should be replaced by $l_e = L/2$. Such situations are:*	Editorial
C5-**87**	*C5E.4.1*	Change deleted		
C5-**88**	*C5E.4.1*	No discussion of alpha and beta units, or support on materials other than monolithic concrete	Add brief discussion in new note.	Discussion on alpha and beta units adapted from ReCast paper. Basis for accepting shell beams is calcs considering pull off force in comparison to adhesion between face shell and core in conjunction with bending at the web-flange junction. These show that even for heavy long span hollow-core the strength should be sufficient.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
				
C5-**89**	*C5E.5.1*	General content repeats a lot of information about PMF that is in the note	Reorganise and simplify intro text – see tracked change version. Content on effectiveness of R16 bars in infilled cells moved to note in C5E.5.2.	Editorial
~~C5-**90**~~	~~*C5E.5.1*~~	~~Detailing/bearing strips are not sufficient to reliably preclude web cracking related failures~~	~~Alter language so that web cracking failure is excluded.~~ ~~*Two criteria are used to define whether a unit is expected to experience positive moment failure, namely wide opening of a transverse soffit crack proximate to the support or presence of a transverse soffit crack along with web cracking. The latter is sometimes referred to (Brooke et al. 2022) as web splitting failure (WSF). It is not necessary to check positive moment failure caused by wide opening of a transverse soffit crack if:*~~ • ~~*The seating for the hollow-core unit uses low friction bearing strips as required by Amendment 3 to NZS 3101:1995 (published in April 2004), or*~~ • ~~*Anchorage detailing, as given in Clause 18.6.7 of NZS 3101:2006 A3 is provided, including R16 bars in two (but no more) filled cells.*~~	~~Damage to units in the BNZ Building shows that modern hollow-core detailing does not preclude web cracking.~~ Change has become redundant with new approach to PMF (change C5-**94**) – text largely deleted.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			~~*The presence of the detailing described above is not deemed sufficient to preclude positive moment failure due to web cracking.*~~	
C5-**91**	*Figure C5E.8*	Figure is not consistent with proposed new approach for PMF (see change C5-**94**).	Update figure, and remove second bullet underneath that identifies that PMF does not need checking if bearing strips are provided.	Editorial provided change C5-**94** is accepted
C5-**92**	*Note under Figure C5E.8*	Reference to using guidelines to determine seating length for retrofit is inconsistent with recommended practice	Remove reference and instead point to relevant guidance on required length: *Guidance on the seating length that should be provided during retrofit can be found in other guidance (Brooke et al. 2022, Büker et al. 2022b).*	For consistency with ReCast recommendations to use NZS 3101 for retrofit.
C5-**93**	*C5E.5.3*	Content regarding impact of supplemental seating on NMF is outdated compared to findings of ReCast floors project	Delete Figure C5E.18, following paragraph, and note immediately prior to heading C5E.5.4. Add comment in note above Figure C5E.17 pointing to Büker et al. 2022 for guidance on considering influence of seating on NMF:	Technical justification is Büker et al 2022. As a relatively rare issue, makes sense to provide reference rather than incorporating content in C5.
C5-**94**	*C5E.5.4*	Current methods for addressing PMF and WSF is laborious, does not address beta units, and overall does not match experimental data well.	Adopt "category 1-4" tabulated limit approach developed by Ken Elwood/Frank Büker. Would require additional change to note in C5E.4.1 to remove statement that no provisions are included for beta units (see Change C5-88).	New approach is dramatically quicker to apply, covers beta units, and gives better match to experimental data.
~~C5-**95**~~	~~*C5E.5.5*~~	~~Should be a subsection of C5E.5.4~~	~~Demote heading~~	~~Editorial~~ Change has become redundant with new approach to PMF (change C5-**94**) – text largely deleted.
~~C5-**96**~~	~~*Equation C5E.9*~~	~~Numerator states:~~ $\cancel{L/2 - s - 0.9h_b}$	~~Change numerator to:~~ $\cancel{L/2 - h_c/2 - 0.9h_b}$	Change has become redundant with new approach to PMF (change C5-**94**) – text largely deleted.

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
		~~which is inconsistent with note that states the critical section is from the column face.~~		
~~C5-**97**~~	~~*Equation C5E.11*~~	~~Uses f_{dt} whereas it should be f_{ct} both outside and under the radical~~	~~Changed to~~ $$\cancel{v_{tn} = \min\left\{f_{ct}\sqrt{1+\frac{f_{pc}}{f_{ct}}};\ 0.2f'_c;\ 10\text{MPa}\right\}}$$	~~Editorial~~ Change has become redundant with new approach to PMF (change C5-**94**) – text largely deleted.
~~C5-**98**~~	~~*Equation C5E.12*~~	~~Should use A_{oc} rather than A_{co} for consistency with NZS 3101~~	~~Change equation and notation to A_{oc}~~	~~Editorial~~ Change has become redundant with new approach to PMF (change C5-**94**) – text largely deleted.
C5-**99**	*Note below Figure C5E.29*	Reference to "a further mechanism involving the mechanical action of the loop bar" seems unwise given that these have twice been observed to simply fold out of the way.	Delete "*A further mechanism involving the mechanical action of the loop-bar may also be available in some situations*".	
C5-**100**	*Note below Figure C5E.29*	Provide reference to CNZ seminar notes as a means of demonstrating how the capacity can be calculated.	Add paragraph: *Further discussion of the difficulty of demonstrating that loop bar hangers have reliable capacity can be found in notes for the Concrete New Zealand Learned Society seminars on assessment of precast concrete floors (Elwood et al. 2018).*	Reference is to: Elwood, K. J., Bull, D. K., Poland, C., and Ashby, C. (2018). Assessment of Existing Precast Concrete Floors (TR72). Concrete New Zealand Learned Society, Wellington, New Zealand, 168p.
C5-**101**	*C5E.7.1*	Second paragraph below note over emphasises the possibility of secondary load paths given that conforming shear reinforcement is uncommon in reality.	Alter paragraph as below: *There is typically no reliable tension load path between ribs and the insitu slab. Thus it is generally inappropriate to rely on the insitu slab to support the ribs after the occurrence of a failure, just as it is inappropriate to rely on the topping concrete to support hollowcore or double tee units. Less commonly, the presence of shear reinforcement between the ribs and insitu slab may ~~also~~ allow for a reliable secondary load path after failure of a limited number of ribs has*	

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
			initiated (refer to Figure C5E.31). These secondary load paths include the catenary action of kinked starter bars bearing on the rib stirrups and load sharing between adjacent ribs. For these load paths to be reliable it is essential that longitudinal reinforcement (starters or saddle bars for instance) run under the stirrups from the ribs. This configuration is rare, and verification by site investigation is strongly recommended before relying on the secondary load paths.	
C5-**102**	*Figure C5E.31*	Figure does not reflect simplified approach to PMF	Update figure	Editorial
C5-**103**	*C5E.7.3*	PMF approach for hollow-core has now been changed.	Edit section to remove reference to critical crack widths, and instead to state a limiting drift of 2.0% per Cat 3 hollow-core PMF unless a bearing strip is present.	For consistency with approach to hollow-core.
C5-**104**	*Figure C5E.35*	Figure does not reflect simplified approach to PMF	Update figure	Editorial
C5-**105**	*C5E.8.4*	Title refers to rib failure. PMF approach for hollow-core has now been changed.	Remove word “rib” from title. Edit section to remove reference to critical crack widths, and instead to state a limiting drift of 2.0% per Cat 3 hollow-core PMF unless a bearing strip is present.	Editorial For consistency with approach to hollow-core.

Section C7 (2017)

#	Guideline reference	Summary of current issue	Proposed change	Justification for proposed change
C7-**1**	*C7.6.2*	Current recommendation of specific part ductility values are overly conservative.	Change text to permit user discretion based on table 8.2 of NZS 1170.5	reflects knowledge gained since 2017 including that the parts ductility in NZS 1170.5 is intended to account for nonlinearity from sources as rocking, bolt slip, sliding etc in addition to the traditional yielding mechanisms used in primary structure design.
C7-**2**	*Equation C7.14*	Equation C7.14 currently states $$P = \left(\Delta/h_{\mathrm{inf}}\right)^2 \theta_{\mathrm{e}}^2 t L_{\mathrm{inf}} E_{\mathrm{m}}$$ Both Δ/h_{inf} and θ_e represent interstorey drift, which is consequently double counted.	Delete θ_e^2 from equation	Email exchange provided to JCSAEB by Stuart Oliver (developer of original Section C7) confirms editorial error.
C7-**3**	*C7.8.7.2*	Not clear how the altered crack angle should be applied with updated column shear model.	Clarify that altered crack angle is no longer relevant.	Note that ASCE 41 does not apply any similar provision to the same column shear strength model.