NZS 3101:Part 1:1995

CONCRETE STRUCTURES STANDARD
PART 1
THE DESIGN OF CONCRETE STRUCTURES

AMENDMENT NO. 3

March 2004

REVISED TEXT

EXPLANATORY NOTE
NZS 3101:1995 is amended as follows to provide for reinforcing steel manufactured to AS/NZS 4671:2001 and to include altered provisions for the seating of hollow-core units.

APPROVAL
Amendment No. 3 was approved on 26 March 2004 by the Standards Council to be an amendment to NZS 3101:Part 1:1995.

RELATED DOCUMENTS (Page 6)

Delete "NZS 3402 Steel bars for the reinforcement of concrete"

Delete "NZS 4702:1982 Metal-arc welding of grade 275 reinforcing bar"

In a new section headed "JOINT AUSTRALIAN/NEW ZEALAND STANDARDS" **add**:
"AS/NZS 4671:2001 Steel reinforcing materials" and
"AS/NZS 1554.3:2002 Welding of reinforcing steel".

(Amendment No.3, March 2004)

2.1 DEFINITIONS

DEFORMED REINFORCEMENT (page 13)
Delete "NZS 3402" and **substitute** "AS/NZS 4671".

HOLLOW CORE SLAB OR WALL (page 14)
Add to the end of the definition "Hollow core units have no shear reinforcement."

PLAIN REINFORCEMENT (page 15)
Delete "NZS 3402" and **substitute** "AS/NZS 4671".

(Amendment No.3, March 2004)

3.4.4.3 (Page 27)

Delete existing clause and **substitute**:

"Assessment of structural deflections for the ultimate limit state involving seismic forces shall make due allowance for the anticipated levels of concrete cracking, post-elastic effects and reinforcement grade".

(Amendment No.3, March 2004)

Single User PDF Terms & Conditions

You have material which is subject to strict conditions of use. Copyright in this material is owned by the New Zealand Standards Executive. Please read these terms and conditions carefully, as in addition to the usual range of civil remedies available to Standards New Zealand on behalf of the New Zealand Standards Executive for infringement of copyright, under New Zealand law every person who infringes copyright may be liable to a fine of up to $10,000 for every infringing copy or imprisonment of up to 5 years, or a fine of up to $150,000 or imprisonment not exceeding 5 years.

You have access to a single-user licence to read this non-revisable Adobe Acrobat PDF file and print out and retain ONE printed copy only.

We retain title and ownership of the copyright in this PDF file and the corresponding permitted printed copy at all times.

Under this license use of both the PDF file and the single permitted printed copy of this PDF file you may make are restricted to you. Under no circumstances are you permitted to save, sell, transfer, or copy this PDF file, the one permitted printed copy of this PDF file, or any part of either of them.

You undertake that you will not modify, adapt, translate, reverse engineer, decompile, disassemble or create derivative works based on any of the downloaded PDF file, nor will you merge it with any other software or document, even for internal use within your organization.

Under no circumstances may this PDF file be placed on a network of any sort without our express permission.

You are solely responsible for the selection of this PDF file and any advice or recommendation given by us about any aspect of this PDF file is intended for guidance only and is followed or acted upon entirely at your own risk.

We are not aware of any inherent risk of viruses in this PDF file at the time that it is accessed. We have exercised due diligence to ensure, so far as practicable, that this file does not contain such viruses.

No warranty of any form is given by us or by any party associated with us with regard to this PDF file, and you accept and acknowledge that we will not be liable in any way to you or any to other person in respect of any loss or damage however caused which may be suffered or incurred or which may arise directly or indirectly through any use of this PDF file.

Regardless of where you were when you received this PDF file you accept and acknowledge that to the fullest extent possible you submit to New Zealand law with regard to this licence and to your use of this PDF file.

4.3.6.4(b)(i) (page 36)
Delete:

"For solid and hollow-core slabs .... 50 mm

For beams or ribbed members .... 75 mm"

and **substitute**:

"For solid slabs .... 50 mm

For hollow-core slabs, beams or ribbed members .... 75 mm"

(Amendment No.3, March 2004)

4.3.6.4 (page 36)
Add new paragraphs at the end of this clause:

"(c) Where hollow-core units supported on a seating are used in buildings, they shall be mounted at both ends on continuous low friction bearing strips with a coefficient of friction of less than 0.7 and a minimum width of 50 mm.

The support of hollow-core flooring shall satisfy one of the three sets of conditions, (i), (ii) or (iii), set out below:

(i) *Use of soft packing behind hollow-core unit*

(A) Soft packing shall be placed against the end face of the hollow-core unit to allow relative rotation to develop between it and the supporting member, and
(B) The thickness of the soft packing shall be between 3 % and 4 % of the depth of the hollow-core unit, and
(C) Reinforcement in the topping perpendicular to and above the soft packing shall:
- Comprise Grade 300 above the soft packing; and
- Be anchored in the supporting beam or the adjacent span: and
- Bar diameter shall not be greater than $^{1}/_{5}$ the topping thickness; and
- Reinforcement up to a maximum ultimate strength of 113 kN/m width shall extend a minimum distance into the span beyond the soft packing by the greater of (L_d + 400) mm or 0.2 times the hollow-core span.
- Where the capacity of the reinforcement exceeds 113 kN/m, that portion in excess of this limit shall extend the entire span of the hollow-core.

or,

(ii) *Use of filled cells and reinforcement*

(A) Use of this detail shall be limited to structures with inter-storey deflections calculated in accordance with 4.7.4 of NZS 4203 of less than 1.2 % of the storey height.
(B) Cells shall be filled at not more than 600 mm centres with a maximum of 50 % of cells filled per hollow-core unit. The cells shall be filled and reinforced in accordance with (D) below for a minimum distance from the support of the greater of 800 mm or 3 times the hollow-core unit depth, and
(C) Each of these cells shall be filled with the same concrete at the same time that the topping concrete is cast, and
(D) Each filled cell shall contain Grade 300, plain round reinforcement near the bottom of the cell with an ultimate strength of 60 kN. This reinforcement shall be anchored by standard hooks at each end into the concrete core and the supporting beam, and

(E) Reinforcement in the topping passing over the end of the hollow-core unit shall:

- Comprise Grade 300 reinforcing, and
- Be anchored in the supporting beam or the adjacent span, and
- Bar diameter shall not be greater than $^1/_5$ the topping thickness, and
- Reinforcement up to a maximum ultimate strength of 113 kN/m width shall extend a minimum distance into the span beyond the end of the hollow-core unit by the greater of (L_d + 400) mm or 0.2 times the hollow-core span.
- Where the capacity of the reinforcement exceeds 113 kN/m, that portion in excess of this limit shall extend the entire span of the hollow-core.

or,

(iii) *Rational design*

Design may be based on a rational solution based on calculation or on methods proved through testing.

(d) The seating requirements provided under 4.3.6.4(b)(i) may be reduced by 15 mm where armoured edges are utilized in the supporting member and adequate support will continue to be provided following plastic hinge elongation.

(e) In the plastic hinge regions of ductile structures it shall be assumed that the cover concrete spalls."

(Amendment No.3, March 2004)

(Page 37)

Add new clause following 4.3.6.7:

" **4.3.6.8** *Deformation compatibility of hollow-core flooring systems*

Where a hollow-core flooring runs parallel to an adjacent beam, then either:

a) The hollow-core unit shall be placed no closer than 600 mm to parallel beams, and be linked to the beam by the reinforced topping only; or

b) Calculations shall be conducted to demonstrate that deformation incompatibility between the beam and floor at the ultimate limit state will not cause failure of the hollow-core unit."

(Amendment No.3, March 2004)

4.4.1.2 (Page 38)

Add paragraphs at the end of this clause:

"(a) For reinforcement that complies with AS/NZS 4671, the overstrength actions in potential plastic hinge regions shall be determined as set out below.

Where Grade 300E reinforcement is used the overstrength bending moment for:

(i) Beams shall be taken as 1.25 times the nominal strength;

(ii) Columns shall be calculated assuming a concrete strength of (f'_c + 15) MPa and a reinforcement strength of 1.25 f_y its design strength, modified, where required by (b) below.

For Grade 500E reinforcement the overstrength bending moment for:

(iii) Beams shall be taken as 1.4 times the nominal strength;

(iv) Columns shall be calculated assuming a concrete strength of (f'_c + 15) MPa and a reinforcement strength of 1.35 f_y, modified, where required by (b) below.

(b) For potential plastic hinge regions in a column, which can form against a base slab or other member that effectively confines the compression zone, the overstrength bending moment shall be calculated from equation A-8 in Appendix A. However, in no case shall the overstrength moment be taken as less than the value defined in (a) above."

(Amendment No.3, March 2004)

7.3.1.2 (page 73)

Delete existing clause and **substitute**:

"Reinforcement bars shall conform to AS/NZS 4671. Grade 500 bars shall be manufactured using the microalloy process unless the conditions for the use of the in-line quenched and tempered process are satisfied.

Reinforcement bars manufactured by the in-line quenched and tempered process shall only be used where welding, galvanizing, or threading of bars does not occur.

Reinforcement bars shall be ductility class E unless the conditions for use of class N are satisfied. Ductility class L reinforcement bars shall not be used.

Class N reinforcement may be used only where either condition (i) or (ii) is satisfied:

(i) Where a member is not subjected to seismic actions and the strain sustained at the ultimate limit-state does not exceed 0.033 when allowance is made for:
- Strains associated with stage by stage construction and shake down effects
- Strains associated with cracking arising from heat of hydration movements, differential temperature effects and creep and shrinkage movements.

(ii) Where a member is subjected to seismic actions but the strain in the ultimate limit state does not exceed a value of 0.02 when allowance is made for:
- Strains induced in plastic hinge regions due to rotation and elongation
- Strains in diaphragms due to deformation caused by elongation of beams/walls, and structural actions due to the transfer of forces between lateral force resisting elements
- Structural actions arising from dynamic magnification effects.

Welded wire fabric shall have a uniform elongation, as defined by AS/NZS 4671, of at least 10 %.

Lesser ductile welded fabric may be used where:

(a) The yielding of reinforcement will not occur at the ultimate limit state; or

(b) The consequences of yielding or rupture does not affect the structural integrity of the structure.

The provisions of NZS 3109 including its amendments shall apply to the welding, bending and re-bending of reinforcing bars."

(Amendment No.3, March 2004)

7.3.3 (page 74)

Delete "NZS 3402" and **substitute** "AS/NZS 4671".

(Amendment No.3, March 2004)

7.3.4.1 (page 74)

Delete "NZS 3402" and **substitute** "AS/NZS 4671".

(Amendment No.3, March 2004)

Table 7.1 (page 74)

Delete “430” and **substitute** “500”.

(Amendment No.3, March 2004)

Table 7.2 (page 74)

Delete “430” **and substitute** “500”
Add note to bottom of table:
“NOTE –
Where deformed bars are galvanized before or after bending, the minimum bend diameter shall be:

(i) $5d_b$ for bar diameters of 16 mm or less
(ii) $8d_b$ for bar diameters of 20 mm or greater.”

(Amendment No.3, March 2004)

Clause 7.3.16.1 (page 79)

Delete “NZS 4702” **and substitute** “AS/NZS 1554.3”

(Amendment No.3, March 2004)

7.3.16.2 (page 79)

Delete the first paragraph and **substitute:**
“In the design and execution of welding of reinforcing bars manufactured to AS/NZS 4671, appropriate account shall be taken of the process of manufacture. Welding of bars that have been manufactured by the in-line quenched and tempered process shall not be permitted.”

(Amendment No.3, March 2004)

7.3.16.5 (page 80)

Delete item (a) and **substitute:**

“(a) A full strength welded splice is one in which the bars are butt welded to develop in tension the breaking strength of the bar. Full strength welded splices shall not be used for reinforcement with a design yield stress of more than 450 MPa unless:

(i) Yielding of the reinforcement shall not occur

(ii) Proof testing using a portion of the actual bar to be welded and the selected welding procedure, demonstrates that failure of the bar occurs away from the weld.”

(Amendment No.3, March 2004)

7.3.16.5 (page 80)

Insert a new sub-clause:

“(c) Welding, including tack welding, of bars that have been manufactured by the in-line quenched and tempered process shall not be permitted.”

(Amendment No.3, March 2004)

7.5.2.5 (page 88)

Delete the first two lines of this clause and **substitute**:

"The maximum diameter of Grades 300 and 500 longitudinal beam bars passing through an interior joint shall be computed from either (a) or (b) below provided one of the conditions, (i) to (iv), given below is satisfied:

(i) Grade 300 reinforcement is used;

(ii) Inter-storey deflections are calculated using the time history method and satisfy the limits of 2.5.4.5 of NZS 4203;

(iii) The inter-storey deflections divided by the storey height at the ultimate limit state does not exceed 1.2 % when calculated using the equivalent static or modal response spectrum methods;

(iv) The beam column joint zone is protected from plastic hinge formation at the faces of the column (as illustrated in figure C7.19);

(v) The plastic hinge rotation at either face of the column does not exceed 0.006 radians.

If none of these conditions is satisfied the permissible diameter of Grade 500 beam reinforcement passing through an interior joint shall be determined by multiplying the diameter given by (a) or (b) below by γ.

$$\gamma = \left(2.2 - 1.5\frac{\delta_c}{\delta_m}\right), \text{ but not greater than 1.0}$$

Where:

δ_c = Calculated inter-storey deflections given by 4.7 of NZS 4203.

δ_m = Maximum permissible inter-storey drift given by 2.5.4.5 of NZS 4203."

(Amendment No.3, March 2004)

NZS 3101:Part 2:1995

CONCRETE STRUCTURES STANDARD
Part 2
COMMENTARY ON THE DESIGN OF CONCRETE STRUCTURES

AMENDMENT NO. 3

March 2004

APPROVAL
Amendment No. 3 was approved on 26 March 2004 by the Standards Council to be an amendment to NZS 3101:Part 2:1995.

C1.3.3 (page 10)

Add to the end of the fourth paragraph of the existing clause:

"Such specialized work may include any operation involving bending (including re-bending) or welding of reinforcement on site. Although these operations are permitted, subject to meeting the requirements specified in this Standard and in NZS 3109, sufficient supervision needs to be provided on site to ensure they are performed correctly."

(Amendment No.3, March 2004)

C3.4.4.3 (page 24)

Add new paragraphs at the end of the clause:

"Generally the effective area of wall and column cross-sections may be taken as the gross area. While flexural cracking reduces the effective area of a wall or column for resisting stresses due to axial loads, elongation associated with flexural cracking tends to negate any axial shortening calculated allowing for the reduction in effective cross-section.

The table gives an indication of the effective second moment of area of individual members in a structure for purposes of analysis for earthquake actions. The effective stiffness of reinforced concrete members is influenced by many factors. These include:

(a) The amount and distribution of reinforcement, particularly the reinforcement in the tension zone of the member;

(b) The extent of cracking, which affects the magnitude of tension stiffening;

(c) The tensile strength of the concrete;

(d) The initial conditions in the member before the structural actions are applied. For example, shrinkage and creep of the concrete places reinforcement in compression, and as a consequence at cracking the reinforcement can be sustaining appreciable compression. This increases the strains in the reinforcement and hence the curvature to a predetermined tensile stress level in the reinforcement. As a result the effective stiffness of a member can be reduced below that assumed in calculations which ignore these actions.

Not all the factors described above can be included in a table such as C3.1. Consequently the values given are indicative of those values, which might be expected in buildings with typical member dimensions and reinforcement proportions and initial stresses induced due to creep and shrinkage."

(Amendment No.3, March 2004)

Table C3.1 (page 25)

Delete “existing table C3.1“ and **substitute**:

“Table C3.1 – Effective section property, I_e

Type of Member	Ultimate limit state		Serviceability limit state		
	f_y 300MPa	f_y 500Mpa	$\mu = 1.25$	$\mu = 3$	$\mu = 6$
1 Beams					
(a) rectangular	$0.40\ I_g$	$0.32\ I_g$	I_g	$0.7\ I_g$	$0.40\ I_g$
(b) T and L beams	$0.35\ I_g$	$0.27\ I_g$	I_g	$0.6\ I_g$	$0.35\ I_g$
2 Columns					
(a) $N^*/A_g f'_c > 0.5$	$0.80\ I_g\ (1.0\ I_g)^\dagger$	$0.80\ I_g\ (1.0\ I_g)^\dagger$	I_g	$1.0\ I_g$	As for the ultimate limit state values in brackets
(b) $N^*/A_g f'_c = 0.2$	$0.55\ I_g\ (0.66\ I_g)^\dagger$	$0.50\ I_g\ (0.66\ I_g)^\dagger$	I_g	$0.8\ I_g$	
(c) $N^*/A_g f'_c = 0.0$	$0.40\ I_g\ (0.45\ I_g)^\dagger$	$0.30\ I_g\ (0.35\ I_g)^\dagger$	I_g	$0.7\ I_g$	
3 Walls					
(a) $N^*/A_g f'_c = 0.2$	$0.48\ I_g$	$0.42\ I_g$	I_g	$0.7\ I_g$	As for the ultimate limit state values
(b) $N^*/A_g f'_c = 0.1$	$0.40\ I_g$	$0.33\ I_g$	I_g	$0.6\ I_g$	
(c) $N^*/A_g f'_c = 0.0$	$0.32\ I_g$	$0.25\ I_g$	I_g	$0.5\ I_g$	
4 Coupling beams **					
(a) Diagonally reinforced	$\dfrac{0.40 I_g}{1.7+2.7\left(\dfrac{h}{L}\right)^2}$		$\dfrac{I_g}{1.7+1.3\left(\dfrac{h}{L}\right)^2}$	$\dfrac{0.70 I_g}{1.7+2.7\left(\dfrac{h}{L}\right)^2}$	$\dfrac{0.40 I_g}{1.7+2.7\left(\dfrac{h}{L}\right)^2}$
(b) Conventionally reinforced	$\dfrac{0.40 I_g}{1+8\left(\dfrac{h}{L}\right)^2}$		$\dfrac{I_g}{1+5\left(\dfrac{h}{L}\right)^2}$	$\dfrac{0.70 I_g}{1+8\left(\dfrac{h}{L}\right)^2}$	$\dfrac{0.40 I_g}{1+8\left(\dfrac{h}{L}\right)^2}$

† The values in brackets apply to the regions of columns, which have a high level of protection against plastic hinge formation in the ultimate limit state.
** The effects of shear deformations and strain penetration into walls along beam bars have been included.”

(Amendment No.3, March 2004)

C4.3.6.4 (page 41)

Add new paragraphs to the end of the existing clause:

“Hollow-core units are brittle in character and as such care is required to ensure satisfactory performance can be obtained in situations where either high diaphragm shear stresses are induced or relative displacements may be imposed between the hollow-core units and the supporting structure. Tests have shown that relative rotation between hollow-core floor units and the supporting structure, combined with elongation due to the formation of plastic hinges, can result in collapse of the floor at drift levels below the design limit

The seismic performance can be improved by using the details described in 4.3.6.4 c(i) and (ii) (as amended by this amendment). However, if detail c(i) is adopted the designer needs to consider:

1. The floor slab is simply supported and therefore less stiff than options that provide some continuity at the support. This does not normally affect floor vibration calculations as these normally assume node points at the supports;

2. Expansion of the flooring unit is restrained only by the topping. Under fire loading the impact of this on the fire resistance rating should be considered.

The two proposed details for the support of hollow-core flooring units are shown in figure C4.3/2.

The “compressible backing” material shown in figure C4.3/2 (a) may be extruded hollow plastic signage sheet or other material that has a similar compressive strength and is sufficiently robust to maintain its integrity and prevent concrete entering the voids during the pouring of the infill. The thickness of the infill is to be between 3 % and 4 % of the depth of the hollow-core section to provide adequate movement to accommodate the anticipated ultimate limit state deformations but allow contact at the lower surface to enable arch action to develop during fire exposure.

Research into the seismic performance of hollow-core flooring systems is in its infancy and understanding improves on a daily basis. The details described in this clause are considered probable best practice, based upon the information on hand at preparation of the amendment. Modification of the requirements may occur as more research data becomes available. The drift limits imposed on detail 4.3.6.4 (c)(ii) (as amended by this amendment), have been applied as the performance of this detail has been extrapolated from tests on similar but not identical details. The drift limitation may be relaxed in the future.

Elongation of plastic hinge regions in beams and/or relative rotation between the supporting structure and the precast floor units, can lead to the formation of wide cracks at the support zones. This cracking can induce high strains in any reinforcement that ties the precast units and their *in situ* topping concrete to the supports. As a result high axial forces and negative bending moments can be induced into the end of the units, which are not designed to sustain these actions.

To prevent brittle failure, the capacity at the floor/beam interface should have a lower capacity than the composite hollow-core and topping. This requires a limitation on the area of the reinforcement crossing the critical section (refer to figure C4.3/2) and the termination point of any reinforcement crossing this point.

The yield capacity limitations provided in 4.3.6.4 (c)(i) (as amended by this amendment) assume a probable lower limit on the capacity of the hollow-core floor, and overstrength of the reinforcement. The minimum development length beyond the critical section is increased by 400 mm to accommodate the possibility of diagonal cracking.

Detail (c)(ii) requires that within the core, plain round bars are placed only at the bottoms of the filled cells. The provision of multiple layers of reinforcement in the filled cores can, if over reinforced, result in the filled core effectively behaving as a short cantilever that can pry apart the top and bottom of the hollow-core units. To ensure that hinging does not occur at the ends of the filled cores under negative moment, the topping reinforcement crossing the critical section is to extend beyond the filled sections by a development length plus 400 mm.

Where the prescribed solutions are not adopted, capacity design principles are to be applied to the hollow-core unit and its supports. This process is undertaken to ensure that in the event of relative displacement between the support and units (due to rotation and/or elongation) cracking will be confined to the critical section between the end of the hollow-core unit and the supporting beam. The structural over-strength actions transmitted across the critical section into the hollow-core unit are to be calculated assuming that the reinforcement sustains a stress of αf_y where α is 1.6 for Grade 300 and 1.5 for Grade 500 reinforcement. The area of added reinforcement is calculated to ensure the nominal bending moment and axial load capacity of the hollow-core unit is greater than the maximum structural actions induced by the over-strength actions at the critical section together with gravity load and vertical seismic actions, and that the shear stresses induced in the negative moment flexurally cracked tension zone are not sufficient to cause a diagonal tension failure.”

(Amendment No.3, March 2004)

– –

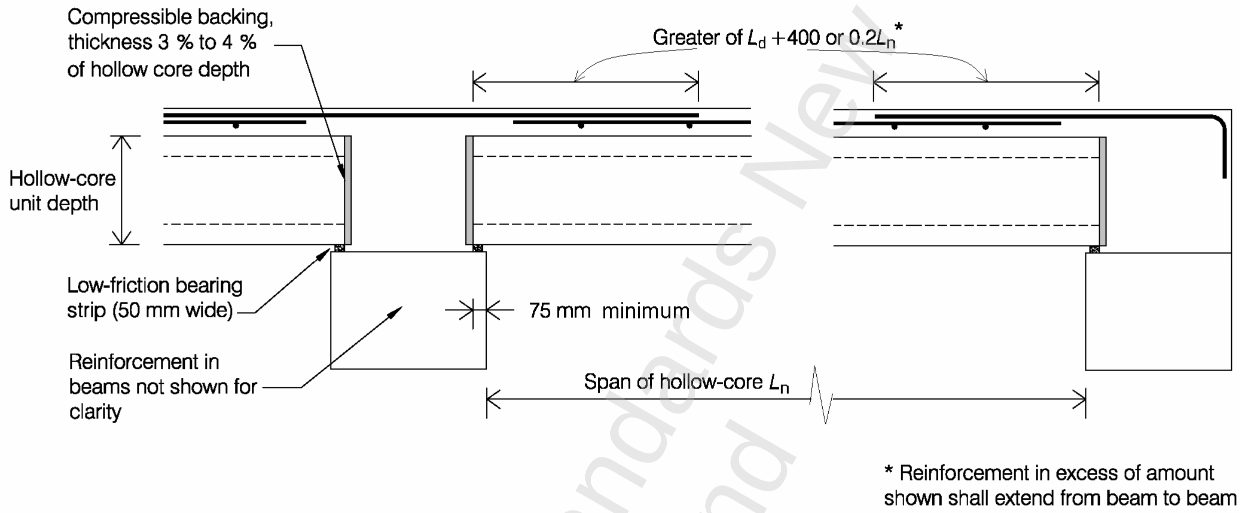
Figure C4.3 (page 41)

Delete "L/180 ≥ 50 mm (slabs)"

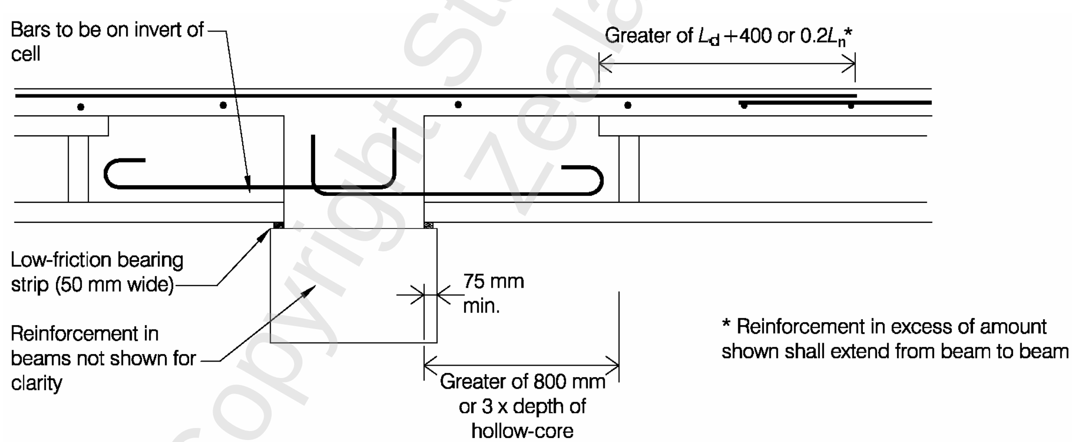
Delete "beams or ribbed floor"

(Amendment No.3, March 2004)

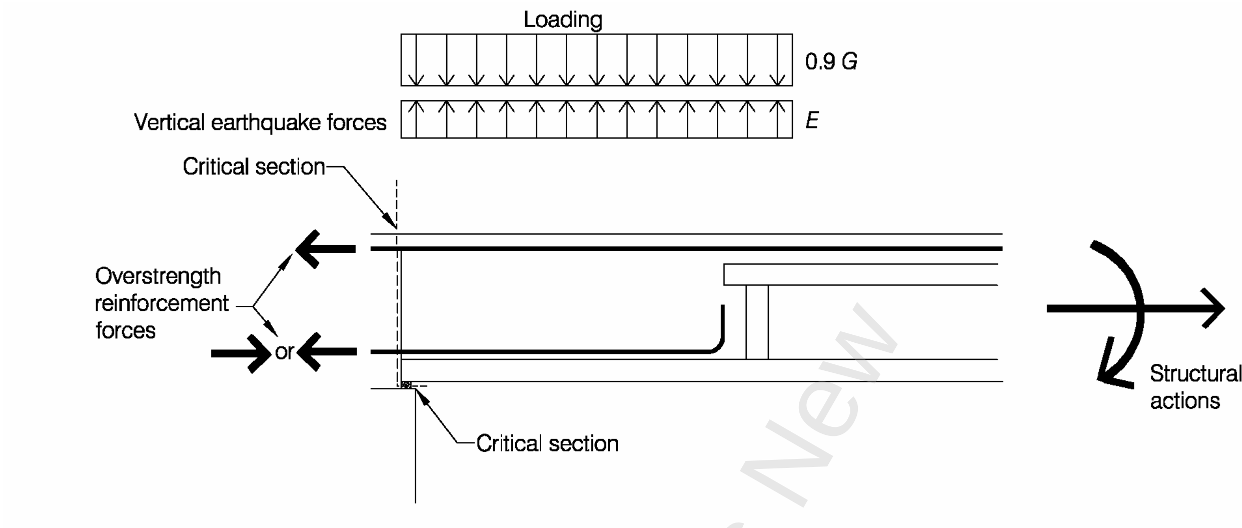
Figure C4.3/2 (new) (page 41)
Add new figure:



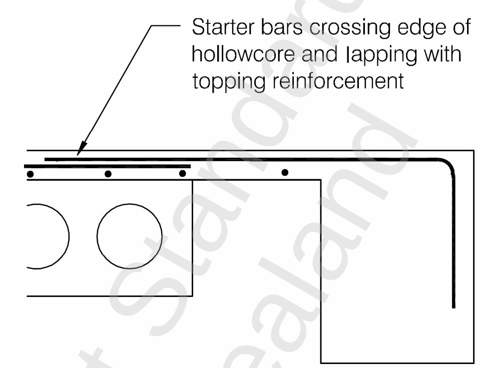
(a) Hollow-core with compressible backing on low friction bearing strips



(b) Hollow-core with 2 – 2 leg R 16 hairpins on low friction bearing strips



(c) Capacity design actions in hollow-core



(d) In situ edge slab reinforcement

Figure C4.3/2 – Support for hollow-core flooring units

(Amendment No.3, March 2004)

On page 42 **add** a new clause following C4.3.6.7:

“ **C4.3.6.8**
Where a hollow-core unit runs parallel and adjacent to a beam, the potential exists for damage to occur to the brittle hollow-core unit due to a need for deformation compatibility at the interface. The placing of the hollow-core unit a distance of 600 mm away from the beam is to provide a more flexible link between the two components.”

(Amendment No.3, March 2004)

C4.4.1.2 (page 44)

Add new paragraphs to the end of the existing clause:

"The specified overstrength factors shall only be used where the reinforcement fully complies with the manufacturing requirements of AS/NZS 4671 and in particular the compliance testing specified in appendix B1.2.

The appropriate overstrength factor for reinforcement for which documented compliance with AS/NZS 4671 is questionable will need to be determined from testing of the reinforcement. The overstrength factor for bending may then be calculated as-

(i) Beams shall be calculated assuming the maximum stress in the reinforcement is equal to 1.35 times a yield stress level, which is determined from the test results as specified in clause 8 of AS/NZS 4671, such that 95 % of all the test specimens have a yield stress of less than this value;

(ii) Columns shall be calculated assuming the maximum reinforcement stress equal to 1.35 times the yield stress level determined as for beams above, together with a concrete strength of (f'_c + 15) MPa, but modified, where required by 4.4.1.2 (b) (as amended by this amendment).

Interpretation of the tests required in (i) and (ii) above shall be-

a) In terms of AS/NZS 4671 B7, a "batch" shall be interpreted as any bundle of reinforcement to be used. Each Grade of bar, round or deformed profile and bar size shall be treated as a discrete test unit to be individually reviewed;
b) Verification that all products in the test unit are from the same cast is to be by the manufacturer´s or processor´s or supplier´s certificate;
c) From the 15 test pieces per test unit of no more than a 100 tonnes (or part of), the test results shall be used to determine compliance with NZS 4671. Up to 60 test pieces per 100 tonnes may be required by clause B7, NZS 4671, depending on the lack of compliance of the first 15 test pieces;
d) Should the test unit not conform to NZS 4671 then the material of the test unit shall not be used in the structural elements being designed to NZS 3101.

(Amendment No.3, March 2004)

C7.3.1.2 (page 78)

Add a new subclause following **C7.3.1.1**:

" C7.3.1.2

"Ductile reinforcement bars, Grade 300E or 500E, should be used in all structural elements, which may be subjected to appreciable yielding in the ultimate limit-state. For members that are not subjected to significant seismic actions Grade N reinforcement may be used provided it can be shown that the strain does not exceed $^2/_3$ of 0.05. This value (0.05) is the minimum specified strain at maximum stress as given in AS/NZS 4671.

In assessing the maximum strain in reinforcement, allowance must be made for the yield strain associated with moment redistribution and due movements occurring as a result of shrinkage, thermal strains and creep redistribution. Junctions between precast units are particularly susceptible to these actions.

When it is intended to use Grade N reinforcement in members subjected to seismic actions a more restrictive strain limit is required as detailed time history analyses demonstrate that strain levels cannot be accurately predicted in design. In addition it is now generally accepted in major international codes that a small margin of protection against structural collapse should be provided for an earthquake with a 2500 year return period.

In assessing the strains induced in plastic hinge regions it should be noted in regard to elongation that:

1) The strains in reinforcement in a reversing plastic hinge region, due to elongation, are typically twice the value calculated from rotation alone.
2) The rotation demand on a unidirectional plastic hinge is typically three times the corresponding rotation imposed on a reversing plastic hinge.
3) The deformation, which occurs in plastic hinge regions due to rotation and elongation, can impose high strains on reinforcement in diaphragms. Generally only ductile reinforcement should be used in diaphragms.

At the time of production of amendment No 3, "ductile mesh" exists in the New Zealand market, which can achieve a uniform elongation of 10 % but will not satisfy the strength requirements of AS/NZS 4671. The wording of the clause requires only the uniform elongation to satisfy AS/NZS 4671, while manufacture is still required to comply with NZS 3422 through 7.3.12.1 of NZS 3101.

Subclauses (i) and (ii) of 7.3.1.2 (as amended by this amendment) provide the conditions where lesser ductile mesh may be used. A typical example of a situation where yielding of reinforcement does not affect the structural integrity could be a slab on ground.

Designers need to be mindful of the stringent requirements of NZS 3109 and AS/NZS 1554.3 including those for rebending and appropriately account for them in their designs."

(Amendment No.3, March 2004)

C7.3.16 (page 87)

Add new paragraphs to the end of the existing clause:

"The current Standard for welding of reinforcement, AS/NZS 1554.3, allows the use of welding consumables with a minimum strength of 550 MPa (E5518) or 620 (E6218). It is considered unlikely that the use of the lower strength electrode will provide an appropriately high probability that the full strength of a Grade 500 bar will be achieved. Whether the full strength of the bar in the upper characteristic range for Grade 500 reinforcing can be achieved with the higher strength electrode requires verification.

Yielding of the weld is undesirable as the plastic deformation is limited to a short length. This can greatly reduce the ductility of the bar and lead to a brittle type of failure of a member. For this reason welded Grade 500 reinforcement should be approached with caution where plastic deformation may be required.

Welding of in-line quenched and tempered bars can have detrimental effects on the strength and ductility of the bars and associated connection. AS 3600 requires designers to assume that the strength of such reinforcement has a design strength of 250MPa when raised to the temperatures associated with welding, galvanizing or hot bending. Such a requirement is considered inappropriate in a seismic country where concentration of yielding at a weld position would be undesirable, and where inaccurate determination of overstrength could lead to the formation of a soft storey mechanism.

Designers should avoid the need to weld reinforcing steel if possible as:

(a) Where butt jointing is required there is a good range of coupling devices available. Lapping, particularly of smaller bars, may also be an option;

(b) Tack welding of stirrups or ties to main bars may result in a reduction in capacity of the main bar, either through metallurgic changes, or the generation of notches due to undercut if the procedures of AS/NZS 1554.3 are not followed;

(c) Where welds are required to provide lightning protection, care should be taken to choose a route through non-critical members.

The designer´s written approval should be obtained for any welding as what seems to be an unimportant weld to a site operative could affect a critical member."

(Amendment No.3, March 2004)

C7.3.16.5 (page 88)

In the last paragraph of C7.3.16.5:

Delete "In 7.3.16.5(c)" and **substitute** "In 7.3.16.6".

(Amendment No.3, March 2004)

C7.5.2.5 (page 99)

Add two new paragraphs after the existing second paragraph.

Tests have shown that with increased yield stress levels in reinforcement there is a decrease in the bond performance of beam bars passing through beam column joint regions when they are subjected to cyclic conditions involving yielding. The degradation arises due to cyclic yielding of the beam reinforcement in the joint regions. The higher strains associated with high grade reinforcement result in a more rapid degradation in bond and consequently the criteria developed for Grades 300 and 430 reinforcements need to be modified for use with Grade 500 reinforcement. Analysis of test results on internal beam column joints, published in the literature, shows that the current criteria for Grade 300 reinforcement works adequately for Grade 500 reinforcement provided the inter-storey drifts are limited to 1.2 % for structures greater than 30 m high and 1.6 % for structures less than 15 m. These limitations include an allowance for deflections being calculated with S_p= 0.67 in NZS 4203. For drift levels in excess of these, the bar diameter needs to be reduced.

Failure in bond of beam bars passing though an internal beam column joint generally results in a very significant loss of stiffness and it can be associated with a loss in strength.

(Amendment No.3, March 2004)

STANDARDS NEW ZEALAND
PRIVATE BAG 2439
WELLINGTON 6020