Suggested Changes to NZS3101:2006 with Amendments 1 and 2

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Introduction

NZS 3101: 2006 Concrete Structures Standard is a design code published by Standards New Zealand. Since the publication of the second amendment to NZS 3101: 2006 a number of questions about specific clauses in the standard have been sent to Standards New Zealand and members of the Standards committee. The authors have sighted these questions and believe that a number of changes should be considered to correct errors, simplify interpretation of clauses and prevent unintended consequences or clashes with other clauses.

The suggested changes, which are detailed below, are made by the authors and they have **not** been considered by Standards New Zealand or the NZS3101 Standards Committee. The authors hope that in time the suggested changes will be put forward by SESOC, as a NZS3101 committee nominating organisation, and subsequently considered for the next amendment to the Standard. We hope this paper will be of assistance to designers using the Structural Concrete Standard in the interim. Readers are referred to the disclaimer on page 2 of this journal for the conditions of use.

In addition to questions raised by practicing structural engineers the Department of Building and Housing (DBH) has imposed several modifications and limitations on the Standard in order for it to be used under the New Zealand Building Code in their B1 Structure Compliance Document, B1/VM1, effective from September 2010. These changes are noted and in some cases the rational for these changes is questioned by the authors.

The suggested changes are grouped according to the main sections in the Standard. Explanations and comments on the suggested changes are in italics.

Section 1

Clause 1.5

Add to the end of definition of Pier the sentence, “For design purposes piers shall be considered as columns.”

Explanation

A number of other changes are suggested to the section on columns to make the provisions more appropriate for the design of bridge piers and columns where the ratio of length to width of a section is greater than 2.

Section 2

Clause 2.6.1.3.2(b) (ii)

This sub clause relates to the calculation of rotation in unidirectional plastic hinges.
Replace the equations in 2.6.1.3.2 (b) (ii) by;

$$\text{“}1+0.63(\mu-1) \quad for\ 1.0 \le \mu \le 2.0$$

$$1.63\sqrt{(u-1)} \quad for\ 2.0 \le \mu \le 6.0\text{”}$$

Explanation

This change reduces the discrepancy in the calculation of the plastic hinge rotations for unidirectional plastic hinges calculated using NZS1170.5 and NZS3101: 2006. The equations *are based on a limited number of analyses, which are reported in reference 2.20 in the commentary to NZS 3101: 2006.*

Section 3

Clause 3.10 and C3.10

Questions have arisen about the commentary to 3.10. It is accepted that as the number of cycles the concrete is subjected to below zero temperature increases so should the strength of the concrete. However, the commentary to 3.10 provides a table which details the number of frost cycles expected in various locations. What is not explained is the relationship between these cycles and the number of cycles the concrete experiences. The thermal mass of the ground and the concrete itself should have some impact on reducing the number of cycles. The question has arisen as many driveways constructed of 20MPa concrete are performing quite adequately in areas prone to many frost cycles. The issue needs further debate amongst the durability group.

Section 5

Clause 5.2.1

This clause gives the minimum specified design concrete strength, f_c', as 25MPa. This is in conflict with section 3, which deals with durability, where the permitted range of design concrete strengths is given as 20 to 100MPa.

The tentative suggestion is that the minimum design concrete strength specified in 5.2.1 be reduced to 20MPa.

Section 6

Clause 6.9.1.1;

Change title to *"Section properties for seismic analyses"*

Explanation
This is an editorial change.

Clause 6.9.1.2: *ULS deflections to allow for post-elastic effects*

Change the text to:
"Assessment of structural deflections and inter-storey drifts for the ultimate limit state involving seismic actions shall make due allowance for the elastic and inelastic deformation as specified in NZS 1170.5 Section 7, or other appropriate referenced loading standard, and for cracking in concrete sections and the reinforcement grade that is used as specified in NZS3101: 2006, clauses 6.8 and 6.9.

Explanation
This is mainly an editorial change but with the addition of the reference to NZS1170.5 as the source criteria defining the inelastic deformation.

C6.9.1: *Linear elastic analysis*

Immediately below the title insert-
"C6.9.1 and C6.9.2: *Section properties and deformation of structures"*

Replace the 4th paragraph by;
"As an alternative to the values in Table C6.6 for T and L beams the effective second moment of area of the beam may be taken as the value calculated from Equation 6-2, assuming the flexural tension reinforcement in the potential plastic hinges at each end of the beam just reaches its design yield stress and that the beam is supporting the gravity load associated with the seismic load cases. Where there is a major difference in the effective second moments of area for zones subjected to negative and positive flexure the effective stiffness properties of a

beam may be calculated using first principles from section properties calculated at regular intervals along the member. The effective section properties for individual sections may be calculated from Equation 6-2 with the 2 terms $(M_{cr}/M_a)^3$ being replaced by $(M_{cr}/M_a)^4$, see reference 6.16 in the commentary."

Explanation
This change brings the assessment of section properties into line with the values for non seismic cases as described in 6.8.3.

Clause C6.9.1 on page C6-19
Replace equation C6-11(b) by-

$$\text{“} A_{shear} = \frac{V_y L}{G \delta_y} \qquad \text{(Eq. C6-11(b))}$$

Where G is the shear modulus ($0.4E_c$)."

Explanation
This change corrects an error in Equation C6-11(b).

Section 7

Clause 7.1

In the definition of t_c, replace *"0.75 A_{co}/p_o"* by "*0.75A_{co}/p_c*"
In the definition of t_o replace *"0.75 A_o/p_c"* by *"0.75 A_o/p_o"*

Explanation
This change corrects errors in the definition of the above two variables.

Clause C7.6.1.2

Add two new paragraphs after the first three paragraphs in this commentary clause.

"Where torsional moments are induced in members due to twist associated with compatibility requirements the torsional moments may be neglected provided the requirements of 7.6.2 are satisfied. This step may be made as the torsional stiffness of a member decreases to a small proportion of its initial stiffness when torsional cracks form, which allows torsional moments to be redistributed.

A method of assessing the torsional stiffness for both equilibrium and compatibility induced torsion after torsional cracks have formed is given in C7.6.1.3."

Clause 7.6.2.1: *Minimum reinforcement for compatibility torsion*

Replace Equation 7-10 and the portion of the clause above this equation by;

"Where required by 7.6.1.3(b), closed stirrup and longitudinal reinforcement meeting the requirements of 7.6.3 shall be provided for a minimum torsional moment, T_n, equal to or greater than the smaller of:

(a) $T_n = T^*/\phi$, where T^* is calculated neglecting reduction in stiffness due to torsional cracking, or

(b) $$T_n = 0.10 A_{co} t_c \sqrt{f'_c} \left[1 + \frac{N^*}{0.33 A_g \sqrt{f'_c}} \right] \qquad \text{(Eq. 7-10)}$$

Where N^* is taken as positive for axial compression.

Explanation
This change reduces the torsional moment required by Equation 7-10 in (b).

Clause C7.6.2

Replace the second paragraph in this clause by-

"The minimum stirrup and longitudinal reinforcement required to satisfy Equation 7-10 corresponds to a torsional moment of approximately 22% of the average torsional cracking moment. This reinforcement ensures that the member has adequate ductility to enable redistribution of the torsional actions, which occurs due to the loss of torsional stiffness associated with torsional cracking, but does not significantly decrease the flexural or shear strengths of the member."

Explanation
With the suggested changes to 7.6.2 the minimum amount of torsional reinforcement for compatibility induced torsion is significantly reduced from that previously required in clause 7.6.2. This change brings the requirements of the clause into line with ACI-318: 2005. The previous edition of NZS3101 (1995) stated that the torsional reinforcement should be designed to sustain the torsional cracking moment, though the equation for calculating this moment was only given in the commentary.

Clause 7.6.4.1 *Design moment for torsion*
Replace existing clause by;
"Where the torsional action, T^*, exceeds the limit given by 7.6.1.2, closed stirrup and longitudinal reinforcement shall be designed to meet the requirements of 7.6.4.2 for a nominal torsional moment, T_n, which is equal to or greater than the larger of*:*

(a) $T_n = 0.10 A_{co}\, t_c \sqrt{f'_c}\left[1 + \dfrac{N^*}{0.33 A_g \sqrt{f'_c}}\right]$ or,

(b) $T_n = T^*/\phi$,

where T^* is required for equilibrium and N^* is taken as positive for axial compression."

C7.6.4.1 and C 7.6.4.2: *Torsional design moment and torsional reinforcement*
Replace the first paragraph by-
"Where a torsional moment required for equilibrium exceeds the limit given in 7.6.4.1, the required nominal torsional strength is taken as the larger of T^*/ϕ or the torsional moment corresponding to the value given by the equation in (a). The equation given in (a) corresponds approximately a quarter of the average torsional cracking moment. In practice there is a wide variation in the torsional shear stress that initiates diagonal cracking. The reinforcement requirements corresponding to equations in (a) and (b) ensure that the member has adequately ductility to prevent a brittle failure in the event of an over-load or in a situation where torsional cracking occurs at a low torsional shear stress.

Section 9

Clause 9.3.9.4.13*; Minimum area of shear reinforcement*
Replace (c) by;
"(c) In insitu slabs with a depth equal to or less than 400mm, and in composite floor slabs containing precast prestressed units where the maximum clear spacing between the webs is equal to or less than 750mm and the overall depth is equal to or less than 400mm;"

Explanation
The change (c) should remove possible confusion in the interpretation of this sub-clause. The DBH requires a change to the sub-clause (c) and this is addressed together with other DBH requirements later in this paper.

Section 10

Clause 10.2: *Scope*
To the end of the paragraph add-

"The requirements for piers shall be the same as those for columns."

The references to "piers" in 10.4.3, 10.4.5, 10.4.6 and 10.4.7 are redundant and could be deleted.

Clause 10.3.2.3
Replace the line below (Eq. 10-1)" by;

(c) The slenderness ratio, which is given by $\frac{k\,L_u}{r}$, is equal to or less than 100;

Where L_u is defined in 10.3.2.3.1, k is defined in 10.3.2.3.2 and r is defined in 10.3.2.3.3.

Explanation
The slenderness limit, which was in earlier versions of the standard, had been omitted in the current version of the Standard.

Clause 10.4.7.2.6
Replace the existing (b) by;

"(b) Within the ductile detailing length for ductile plastic regions the nominal shear stress resistance of concrete, v_c, is given by-

$$v_c = 3v_b \left[\frac{N_o^*}{A_g\,f_c'} - 0.1 \right] \geq 0.0 \qquad \text{(Eq. 10-34)"}$$

Explanation
This change corrects an error in the existing (b) in this clause.

Clause 10.4.7.5.1
Add a new sub clause after sub clause (a) and re-label the current sub clause (b) to (c).
The new sub clause is-

"(b) For rectangular columns where h/b >2.0 and $N_o^* / A_g\,f_c' \leq 0.25$ and the seismic forces induce bending moments about the strong axis, the reinforcement normal to the longer side, h, shall be equal to or greater than the appropriate value given below.

(i) In the end regions of the column, which are outside the mid region defined in 10.4.6.3, the minimum area of a single tie, A_{t1}, is given by $A_{t1} \geq \frac{A_{sh}}{h''} s_t$;

(ii) In the mid region defined in 10.4.6.3 the minimum area of a tie may be reduced to $A_{t1} \geq 0.5\frac{A_{sh}}{h''} s_t$;

(iii) In all cases the total area of transverse reinforcement shall equal or exceed the area specified in 10.3.10.6.1;

Where A_{sh} is given by Equation 10-40, and s_t is the spacing of the ties normal to the longer side of the column.

Explanation
This is a suggested addition to the clause, which brings the confinement requirements of concrete into line with the existing relaxation allowed for the spacing of longitudinal bars in columns with high length to width ratios, see clause 10.3.8.3.

C10.4.7.4.2 to C10.4.7.5.4
Add to the end of the existing commentary-
"The sub clause (c) in 10.4.7.5.1 is intended to cover the case which frequently arises in bridge piers. In Section 10 bridge piers are treated as columns. In these structural elements

the predominant seismic forces often act in the plane of the pier inducing high compression stresses in the end zones of the pier section. Consequently in the ductile detailing length these zones need to be confined to ensure that ductile behaviour can be sustained. However, the mid regions of the pier are only required to sustain relatively low compression stresses and consequently the level of confinement to the concrete in this area may be reduced. It should be noted that all longitudinal bars in the pier should be restrained against buckling as required in sub clause (c). In addition, the minimum requirements of 10.3.5.2 apply over the whole section and the full length of the column."

Section 11

General comment on chapter 11
This chapter covers detailing requirements for walls, which range from slender tilt up walls to major structural walls in multi-storey buildings. There has been limited research on the stability of tilt up walls and on the shear performance of major structural walls. Consequently this chapter was difficult to write and not all the situations that may arise in practice have been adequately covered by the Standard.

Clause 11.3.5.2.2

Move the label for Equation to the RHS of the page;

$$\frac{k_{ft}\,L_n}{t} \leq 65 \qquad \text{(Eq. 11-10)}$$

Explanation
This is an editorial change.

Clause 11.4.7.3

Replace title and clause by-
11.4.7.3 *Shear strength provided by concrete and maximum shear strength*
"(a) Shear strength provided by concrete"
In wall subjected to an axial load,. N^*, the concrete shear strength provided by concrete, V_c, in the end region defined by 11.4.3 shall not exceed

$$V_c = \left(0.27\lambda\sqrt{f_c'} + \frac{N^*}{4A_g}\right)A_{cv} \geq 0.0 \qquad \text{(Eq. 11-28)}$$

Where
λ = 0.25 for ductile plastic regions
λ = 0.5 for limited ductile plastic regions
N^* the design action axial force is taken as negative for tension.

(b) Maximum nominal shear strength
The nominal shear strength, V_n, shall be equal to or smaller than-

$$V_n = \left(\frac{\phi_{ow}}{\alpha} + 0.15\right)\sqrt{f_c'}A_{cv} \leq v_{\max}\,A_{cv} \qquad \text{(Eq. 11-29)}$$

Where
v_{max} is given in 7.5.2
α = 3.0 for limited ductile plastic regions
α = 6.0 for ductile plastic regions as defined in Table 2.4.
Linear interpretation for α may be used between the values given above when the calculated curvature ductility lies between the limits provided in Table 2.4 for limited ductile and ductile plastic regions.

Explanation
The suggested changes to the text are editorial in nature and they should improve the clarity of this clause.

Section 12

Clause 12.7.3.2

Two lines below Equation 12-8, replace "$1.0 \leq k_{ds} \leq 0.5$" by "$0.5 \leq k_{ds} \leq 1.0$".

Explanation

This corrects an error in the Standard.

Section 17

Clause 17.1 Notation

In the definition of c_2, replace "c" by "c_1".
Replace the existing definition of c_1 by-
"c_1 the distance from the centre of an anchor to the edge of the concrete in the direction in which the load is applied, mm

Clause 17.5.8.1, page 17-8

The current wording for the c_1 is a copy of ACI-318: 2002. However ACI has amended the definition so the intention is to adopt the latest definition from ACI-318.

In the last sentence of the definition of c_1 delete "shall be limited to *h*/1.5" and replace with "shall not exceed the greater of c_2/1.5 in either direction, *h*/1.5, and one third the maximum spacing between anchors within the group."

Clause 18.6.7.2, page 18-6

Replace the first two lines of text by-
"Where hollow-core flooring is adjacent to a beam, a wall or other structural element, which may deform in a direction normal to the plane of the floor, either;"

Explanation

As written in the Standard a flexible linking slab is required where a hollow-core unit is adjacent to a beam which is parallel to the span of the hollow-core units. When a plastic hinge forms in such a beam it deflects in a vertical direction relative to the floor. This differential movement has been observed to cause extensive cracking in the webs of adjacent hollow-core units separating the top and bottom flanges of the hollow-core units and contributing to the premature collapse of a floor. However, other structural elements such as walls or beams in eccentrically braced frames may also generate differential displacement relative to the plane of the floor. The suggested modification generalises the requirement for linking slabs to be used in all situations where vertical displacement can occur between a structural element and an adjacent hollow-core unit.

Clause C18.6.7 on page C18-9

In (a) replace "or 1.0 where lower ductility reinforcement is used" by "or 1.25 where lower ductility reinforcement is used"

Explanation

The smallest value of structural ductility factor in NZS3101 is 1.25.

APPENDICES

Appendix A

***Clause A5.2**: Effective compressive strength of concrete strut*

Replace "α_1 is given by 7.4.2.1 (c)" by "α_1 is given by 7.4.2.7 (c)"

Appendix D

Clause D3 2 3: *Dynamic magnification and modification factors*

In part (c) replace the second paragraph by-

"In the top storey the minimum value of $\omega\beta$ shall be equal to or greater than 1.2."

Explanation
This value had been omitted in error from the Standard.

Restraint of topping concrete above precast units

Questions have been asked about the need to restraint of topping concrete above precast units where this topping concrete is subjected to a shear stress of more than $0.3\sqrt{f'_c}$ *due the transfer of lateral forces to the lateral force resisting elements (walls or frames). NZS310:1995 clause 13.4.3 required concrete topping to be restrained by a nominal amount of ties when the shear stress sustained by the topping concrete exceeded the limit of* $0.3\sqrt{f'_c}$ *. In the current edition this requirement has been removed except for the case for ductile diaphragms, which are required to dissipate energy by inelastic deformation, see Clause 14.4.3.1. For these unusual elements, nominal ties are required regardless of shear stress that the ductile diaphragm is required to sustain. This requirement will seldom be critical as it is only in exceptional situations that a ductile diaphragm is used. Where a ductile diaphragm is used nominal ties are required as cyclic inelastic deformation in the reinforced concrete topping can lead to extensive delamination between the topping and precast floor units.*

The general requirement for tie reinforcement between topping and precast units when the shear stress exceeded the limit of $0.3\sqrt{f'_c}$ *was not maintained in NZS 3101: 2006 for the following reasons;*

- *Any tie between precast and topping concrete is unlikely to be fully effective due to the difficulty of anchoring the ties in the thin layer of topping concrete;*
- *Design for a diaphragm is in general based on a strut and tie analysis, which makes it difficult to assess the equivalent shear stress for a diaphragm, or of a region in a diaphragm.*
- *No theoretical or experiment evidence was found to justify the requirement for these ties.*

Amendments by DBH for use with B1/VM1

(refer to http://www.dbh.govt.nz/building-code-compliance-documents-downloads)

Clause 4.8

When AS/NZ1170 was cited by the DBH, some modifications were made to the load cases required to be considered during and after fire. These necessitate some small modification to clause 4.8 of NZS3101. The amendment to B1 of the compliance documents included the citation of NZS3101 and included a small modification to clause 4.8 of NZS3101: 2006. It is our intention to include these modifications in the amendment of NZS3101.

Clause 9.3.9.4.13: *Minimum area of shear reinforcement*

In many situations for beams and slabs nominal shear reinforcement is required if the design shear force exceeds 50 percent of the design shear strength provided by the concrete $\left(V^* \geq 0.5\phi V_c\right)$. This clause details the situations where this requirement need not be applied. The DBH amendment changes part (c) of this clause, which relates to the case of a floor slab that may be constructed either, entirely with insitu concrete or, from precast pretensioned units, such as such as ribs, double tees and hollow-core units, joined together with insitu concrete that generally takes the form of a reinforced concrete topping. The current sub-clause (c) indicates that nominal shear reinforcement is not required where the total slab thickness is equal to or less than 400mm. This limit applies to both the insitu floor and the composite precast floor provided in the latter case the web spacing does not exceed 750mm. For floors which satisfy this condition when the shear force, *V**, is less than ϕV_c^*, nominal shear reinforcement is not required.

The DBH modification changes this to; (c) “in slabs, including floor slabs containing pretensioned units, where the maximum clear spacing between webs is equal to or less than 750mm; and the depth of the precast unit is equal to or less than 300mm”.

Comment

The DBH modification has 2 significant effects, which are noted below.

1. *The previous requirement to have nominal shear reinforcement in one way insitu slabs for beam type shear where the depth is greater than 400mm and the design shear force, V*, lies between* $0.5\phi V^*$ *and* ϕV^* *has been removed.*

   This change is very significant as some deep slabs that would have required shear reinforcement according to the existing clause 9.3.9.4.13 will no longer require such reinforcement. This reduces the factor of safety of these structural elements. The provision in NZS3101: 2006 was added due to analytical and experimental work carried out by Collins and others, which showed that deep slabs could have significantly lower shear strengths than had previously been assumed, see reference [1] and other references in the commentary to NZS3101: 2006.

2 *With composite slabs there is also a significant change in that the depth of insitu concrete added above the precast units could be greatly increased without invoking the* $V^* \geq 0.5\phi V_c$ *rule, which requires nominal shear reinforcement to be used. A consequence of this is that greater depths of insitu concrete can now be used. For the case where shear is critical near a support in negative moment zones this is a rational change, which will safely reduce the level of conservatism inherent in the existing provisions in the Standard for the case where the insitu concrete depth is greater than 100mm. This reduction in conservatism with the DBH change arises as nominal shear reinforcement is no longer required if* $0.5\phi V_c \leq V^* \leq \phi V_c$. *However, where shear is critical in positive moment zones and the flexural shear condition is critical the increase in depth reduces the shear stress that can be sustained at shear failure. In this situation the shear stress that results in failure decreases with an increase in the total depth (precast and insitu concrete topping). Hence excluding the overall depth from the* $0.5\phi V^*$ *rule reduces the factor of safety where the overall depth of the hollow-core unit and insitu concrete topping exceeds 400mm and high concentrated loads act in the mid-span positive moment region of the floor. This situation may only occur in unusual situations.*

Explanation

The shear stress that can be transferred across a crack reduces as the crack width increases. With increasing slab depths the spacing of primary cracks increases and this results in wider cracks in the mid depth region of the flexural tension zone and a reduction in shear transfer by aggregate interlock action in these regions. This results in a reduction in the shear stress level that can be sustained prior to diagonal cracking (shear failure) as the depth increases in insitu concrete slabs, see reference [1]. Hence with the DBH modification the factor of safety is reduced.

The flexural shear strength of prestessed concrete beams and slabs depends to an appreciable extent on shear transfer across cracks by aggregate interlock action. Close to a support negative moments can act and flexural cracks can extend through the insitu concrete into the top of the precast units and into their webs. However these web cracks are located close to the neutral axis, and consequently the crack widths are small and high shear stresses can be *sustained across this region of these cracks. This enables the composite floor to sustain a relative high shear stress in the zone where the web width is a minimum. In the region where the cracks are wide the width of concrete available to sustain the shear flow is large and consequently the shear stress is low. Hence the shear strength is high compared with that of typical a rectangular or Tee beams from which the shear design equations were developed.*

The situation is different for flexural cracking in positive moment zones. In this case the flexural cracks develop from the bottom fibre and extend into the web. Consequently, the critical cracks in the flexural tension zone are in a region of high tensile strain, which results in wide cracks and a reduction in the shear stress that can be sustained by aggregate interlock action. For the positive moment region the greater the construction depth (precast plus insitu concrete) the lower the shear stress that can be sustained by the concrete in the critical region of the web.

In NZS3101: 2006, the limit of 400mm was used to cover both cases. This was a conservative assumption for negative moment shear strength near a support but a reasonable limit for shear strength provided by concrete in the mid region positive moment zone. The DBH change reduces the conservatism for floors with thick insitu concrete toppings in the negative moment zones, which is not an issue, but does not account for the anticipated reduction in flexural shear strength in positive moment zones where a thick concrete topping is used. Hence the DBH change could be un-conservative in some unusual situations for composite floor slabs.

Clause 18.7.4: *Floor or roof members supported by bearing on a seating*
The DBH require an amendment to this clause, which is to add to the end of clause 18.7.4 (e) to following sentence.

"The details given by C19.6.7(e) may be applied to hollow-core units where the depth of the precast unit is equal to or less than 300mm".

Comment
There is no clause or commentary clause 19.6.7 (e). However, there is a commentary clause C18.6.7(e), which is probably what was intended by the DBH. The intent of this modification is to prevent the detail described in this sub clause from being used where the depth of the hollow-core units exceeds 300mm. ***No*** *rational explanation was given for this modification.*

The clause C18.6.7(e) detailed a method of support for hollow-core units. It was tested in a large scale floor test using 300mm deep hollow-core units [2]. Experimental results showed that this detail was superior to all other support details that were examined, which is not surprising as this can also be shown using standard structural theory. The recommended detail, which was illustrated in Figure C18.4 is also illustrated below in Figure 1.

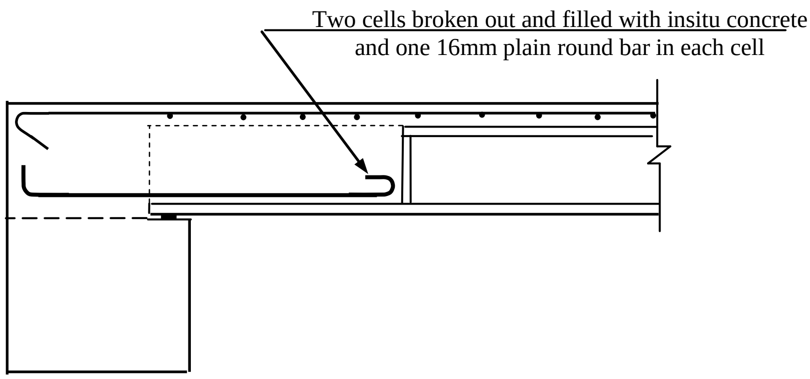

Figure 1: Recommended support detail for hollow-core floor units

Concept behind recommended detail
Two cells at the end of each unit are broken out and filled with insitu concrete. Each of these cells is reinforced with a single a 16mm plain round bar, which is placed in the bottom of the

cell. The addition of the insitu concrete in the cells increases the flexural cracking strength of the hollow-core unit and hence it increases the likelihood that when relative movement occurs between the unit and supporting beam, due to relative rotation or elongation, a crack will form at the back face of the hollow-core unit rather than in the unit close to the face of the supporting ledge. The 16mm bar in effect laps the pretension strands, which gives the support zone to the unit some positive moment flexural strength. Without this reinforcement the eccentric gravity load from the unit induces torsion in the support beam, which can be a problem as the development of plastic hinges in a beam destroys torsional resistance. The positive moment flexural strength provided by the 16mm bars allows redistribution to occur with the torsional moment in the support beam reducing and the reaction from the hollow-core unit effectively moving to the centre-line of the beam section.

The 16mm bar needs to be placed at the bottom of the cell. In this position it laps the pretension strands and it is in the optimum position for increasing the positive moment flexural strength. Placing this reinforcement higher up in the cell has the adverse effects of;

- *Increasing the negative moment that can be transferred to the hollow-core floor, which could require additional reinforcement to be placed in the insitu concrete in the span of the floor to resist the induced negative moments;*
- *Increasing the length of the negative moment zone that can result in a reduction of the shear strength of the hollow-core floor.*

Plain round 16mm bars are used to allow yielding to extend along the bar in the event that elongation of beams parallel to the span of the hollow-core units opens up wide cracks between the floor and supporting beams. In addition in the unlikely event that the hollow-core unit is pulled off the supporting ledge the 16mm bars can prevent complete collapse by developing a kink in the bars, which enables each unit to safely resist a gravity load of 120kN without collapsing.

Scale effects occur in reinforced concrete members. These arise:

- *When in modelling a member the courser aggregate sizes are reduced. This change can lead to the ratio of tensile strength to compressive strength increasing, which in turn results in the model sustaining a higher shear stress at failure than the full sized member;*
- *The flexural tensile stress, which results in the formation of a flexural crack, decreases with an increase the depth of the member due to non-linear behaviour of concrete in tension [3]. Consequently, where a strength depends on the flexural tensile strength of the concrete increasing the depth of a member reduces the stress which results in failure;*
- *Increasing the depth of a member increases the spacing of primary flexural cracks and this increases the crack width, which reduces shear transfer by aggregate interlock action. As a result the shear stress sustained at diagonal cracking in members without shear reinforcement reduces with increasing depth of the member [1].*

None of the above effects influence the flexural strength of reinforced concrete and consequently there appears to be no rational explanation of why the recommended support detail in C18.6.7(e) should be limited to hollow-core units with a depth equal to or less than 300mm. This modification by the DBH is likely to reduce the robustness of hollow-core floors built with hollow-core units with a depth greater than 300mm.

References

1 Collins, M P and Kuchmas, D., "How safe are our large lightly reinforced concrete beams, slabs and footings?", ACI Structural journal, Vol. 96, No. 4, July-Aug. 1999, pp482-490

2 MacPherson, C J., “Seismic analysis of precast concrete hollow-core floor super-sub-assembly”, ME Thesis, Civil Engineering, University of Canterbury, 2005

3 CEB-FIP, Comite Euro-Internationale du Beton and Federation Internationale de la Preconstrainte, “CEB-FIP Model Code, 1990”, Thomas Telford, London, 1993

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UNREFERREED