Recent Developments in the Design and Construction of Concrete Structures incorporating Low-Ductility Steel Reinforcement

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Abstract: Low-ductility steel reinforcement is used as main and/or secondary reinforcement and as fitments in many different types of concrete structures around the world. Produced in various forms, the practical and economic advantages of which can outweigh any non-essential benefits possibly gained from using more ductile reinforcement, the performance requirements of the structural design can be met. A number of major Australian experimental research programs have been undertaken over the past two decades to investigate the behaviour of suspended reinforced-concrete floor beams, one-way slabs, and two-way slabs incorporating low-ductility deformed steel reinforcement. Theoretical studies have also been undertaken to support the development of design rules. Some major findings of these research programs and studies are described and discussed with respect to the latest design rules in AS 3600–2009 for using low-ductility reinforcement (500 MPa Ductility Class L mesh) as longitudinal tensile reinforcement in suspended floors. Ductility of the reinforcing steel, reinforced cross-sections and critical bending regions are all discussed. Alternative methods for calculating cross-section design bending strength and internal design action effects are described. Design issues concerning mixing steels of different ductility classes, and the greater bending strength of doubly-reinforced compared with singly-reinforced sections, are addressed. Use of mesh in construction is illustrated with a practical case study.

Keywords: steel reinforcement; steel ductility; Ductility Class L; reinforced concrete; AS 3600.

1. Introduction

Low-ductility steel reinforcement (5 to 12 mm) is used as main and/or secondary reinforcement and as fitments in many different types of concrete structures around the world. Produced in various forms, the practical and economic advantages of which can outweigh any non-essential benefits possibly gained from using more ductile reinforcement, the performance requirements of the structural design can be met.

For example, in North America, plain or deformed low-carbon cold-worked wire or bar is used extensively in construction of buildings and bridges (1).

Deformed wire defined by ASTM A496/A496M (2) has a minimum yield stress of 515 MPa (determined at a strain of 0.50%) and minimum tensile strength of 585 MPa, and is bend tested for ductility. However, there is no specific minimum ratio of tensile strength to yield stress, and neither is there for uniform strain (strain at peak stress) or elongation after fracture, so the ductility requirements are less stringent than those in AS/NZS 4671 (3) for Ductility Class L reinforcement which have applied in Australia since 2001.

Nevertheless, deformed steel wire is permitted in ACI 318-11 (4) for use in buildings as general concrete reinforcement with a design yield stress of up to 550 MPa for flexure, shear and confinement determined at a strain of 0.35% requiring stress-strain data to be supplied, otherwise it equals 420 MPa. It is most commonly used to manufacture deformed welded wire reinforcement (WWR) defined by ASTM A497/A497M (5), with wire diameters normally from 5.75 to 12.8 mm. Uniform strains (ε_u) varying from about 1.5 to 4.5% can be expected (6,7), with a sample mean in percent of ε_u=0.37d_b with nominal wire diameter (d_b) in millimetres (6). However, unlike in Australia, significantly lower values occur, e.g. 0.78% in Ref. 8. Applications are extensive and include walls and suspended floors. WWR is manufactured in rolls, or flat sheets that can be cold bent into various shapes like stirrup baskets for beams, and closed shear and confinement cages for beams or columns. Ref. 9 summarises the relevant design rules in ACI 318.

Similarly, for bridges designed in accordance with the AASHTO LRFD Bridge Design Specifications (10), its commentary states that the same WWR material (referred to therein as welded wire fabric) "*has been increasingly used in bridge applications in recent years, especially as auxiliary reinforcement in bridge I- and box beams and as primary reinforcement in slabs*". Recent experimental and analytical studies into optimising the topping slab reinforcement of composite bridge decks by Klingner et al. (11) and Foster (12) have shown that WWR is more economical than bars when compressive membrane action is present and temperature and shrinkage control requirements govern the amount of longitudinal and transverse steel. Hon et al. (13) also tested and analysed decks of beam-and-slab bridges containing low-ductility reinforcement and confirmed the importance of compressive membrane action at increasing strength.

Design rules in the Australian Concrete Structures Standard AS 3600–2009 (14) relating to using low-ductility steel (Ductility Class L mesh) as longitudinal tensile reinforcement in suspended concrete floors are described in Section 2. The experimental research programs conducted over the past 20 years, and several theoretical studies conducted concerning the rules are briefly described and discussed in Sections 3 and 4, respectively. With this background information, the design rules are discussed in Section 5. Insight into using Ductility Class L mesh in construction is given by a case study in Section 6.

2. AS 3600–2009 Design Rules for Ductility Class L Reinforcement

2.1 Introduction

While Grade 500 MPa, Ductility Class L bar is commonly used as fitments, e.g. spirals, ties or stirrups, the focus of this discussion is its use in slabs and beams as longitudinal tensile reinforcement in the form of square or rectangular deformed ribbed reinforcing meshes produced in accordance with AS/NZS 4671 (3).

2.2 Clause 1.1.2 Application (c)(ii)

Owing to the low ductility of Class L mesh it has some design restrictions placed on its use when acting as longitudinal tensile steel compared with Ductility Class N steel. These design restrictions are given in later clauses of the Standard, and automatically prevent the Class L reinforcing steel from being used in any situation where it would be required to undergo large plastic deformation under strength limit state conditions. Therefore, designers do not have to be concerned about this clause when they comply with the usual design restrictions, such as assuming zero moment redistribution while employing linear elastic analysis of floors with Ductility Class L mesh in accordance with Clause 6.2 of AS 3600–2009.

2.3 Clause 2.1.1 Design for Strength and Serviceability

When using Ductility Class L mesh as tensile steel the value of the capacity reduction factor ϕ given in Table 2.2.2 is between 0.6 and 0.7 depending on the type of action effect. For bending with or without axial force, the maximum value of ϕ equals 0.64. A new note to the table states that when Class L mesh and Class N bars are mixed together as tensile steel, all the steel should be treated as if Ductility Class L.

The Standard also permits testing of a structure in lieu of calculation to check that the strength and serviceability requirements can be achieved. Proof testing in accordance with Appendix B of AS 3600–2009 defines strength and deflection criteria for acceptance without incurring any significant damage.

2.4 Clause 2.1.3 Design for Robustness

Robust concrete structures can withstand local damage caused by accidents or unforeseen events without progressive collapse, and must satisfy Section 6 of AS/NZS 1170.0 (15). Minimum lateral design forces are specified, and floors are treated as diaphragms to distribute the required wall anchorage forces.

2.5 Clause 2.1.4 Design for Durability and Fire Resistance

The fire resistance period (FRP) for structural adequacy or insulation for a concrete beam or slab incorporating Class L mesh tensile reinforcement may be established using standard solutions (Clause 5.3.1(a) – see Clause 5.4 for beams and Clause 5.5 for slabs) or by calculation ((Clause 5.3.1(b)).

2.6 Clause 2.1.5 Material Properties

Deformed ribbed Grade 500 MPa low-ductility (D500L) mesh has a lower characteristic yield stress, f_{sy}, of 500 MPa, uniform strain, ε_u, of 1.5%, as designated in Table 3.2.1, and a modulus of elasticity of 200 GPa.

2.7 Section 6 Methods of Structural Analysis

The following methods of analysis may be used with Ductility Class L mesh present: (i) static analysis of determinate members or structures permitted by Clause 6.1.3 *Methods of Analysis*; (ii) linear elastic analysis in accordance with Clause 6.2 *Linear Elastic Analysis*, with no moment redistribution assumed to occur at the strength limit state; (iii) finite element analysis, in accordance with Clause 6.4 *Linear Elastic Stress Analysis*; (iv) non-linear frame analysis accounting for non-linear geometric effects, including the effects of compressive membrane action, in accordance with Clause 6.5 *Non-Linear Frame Analysis*; and (v) simplified flexural analysis in accordance with Clause 6.10.2 *Simplified methods for reinforced* *continuous beams and one-way slabs* or Clause 6.10.3 *Simplified method for reinforced two-way slabs supported on four sides*.

2.8 Sections 8 & 9 Design of Beams and Slabs for Strength and Serviceability

Design strength of beam or slab cross-sections in bending, ϕM_{uo}, shall be calculated in accordance with Clause 8.1, normally using rectangular stress block theory in accordance with Clause 8.1.3, while for critical sections it is necessary that $M_{uo} \geq 1.2 M_{cr}$, where M_{cr} is the cracking moment.

2.9 Section 13 Stress Development of Reinforcement and Tendons

Revised rules concerning the development length of deformed mesh in tension are given in Clause 13.1.8, and in Clause 13.2.3 for lapped splices in tension.

2.10 Section 17 Materials and Construction Requirements

Clause 17.2.1.1 requires that Ductility Class L mesh "*shall not be substituted for Ductility Class N reinforcement unless the structure is redesigned*".

3. Australian Experimental Research Programs

3.1 Introduction

The main Australian experimental research programs conducted since about 1995 into the behaviour of reinforced-concrete beams and slabs incorporating low-ductility reinforcement are briefly described. Some of the results and conclusions are discussed in Section 5 in relation to the design rules in AS 3600–2009.

3.2 Continuous Rectangular Beams and T-Beams

Patrick et al. (16) describe the first Australian tests performed on concrete beams incorporating low-ductility reinforcing bar with known tensile properties, e.g. ductility parameters uniform strain ε_u=1.6% and tensile-strength-to-proof-stress ratio=1.057 (see Fig. 1(a)). The tests were overseen by the Concrete Structures Committee BD-002 when the minimum ductility requirements for Class L steel were being set by Reinforcing and Prestressing Committee BD-084, and AS 3600–1994 was amended in August 1996 to not allow moment redistribution when using elastic analysis to design flexural members incorporating mesh produced then which could be significantly less ductile than Ductility Class L mesh produced today.

(a) Two-span beam that achieved 99% of full plastic mechanism while exhibiting significant redistribution

(b) Short slabs incorporating different ductility steels, tested with a central line load until fracture of steel reinforcement occurred

(c) Continuous slab with Ductility Class L mesh subjected to a standard fire – no steel fractured and gross deflections occurred

Figure 1 Several early Australian experimental research programs conducted into the behaviour of beams and slabs incorporating low-ductility reinforcement (from 1996 to 1997)

In 1998 Adams et al. (17) tested six continuous under-reinforced T-beams as elements of stiffened rafts used in slab-on-ground construction. Reinforcing bar ductility varied significantly between otherwise identical beams. A common phenomenon observed was bar fracture, even for Ductility Class N bars.

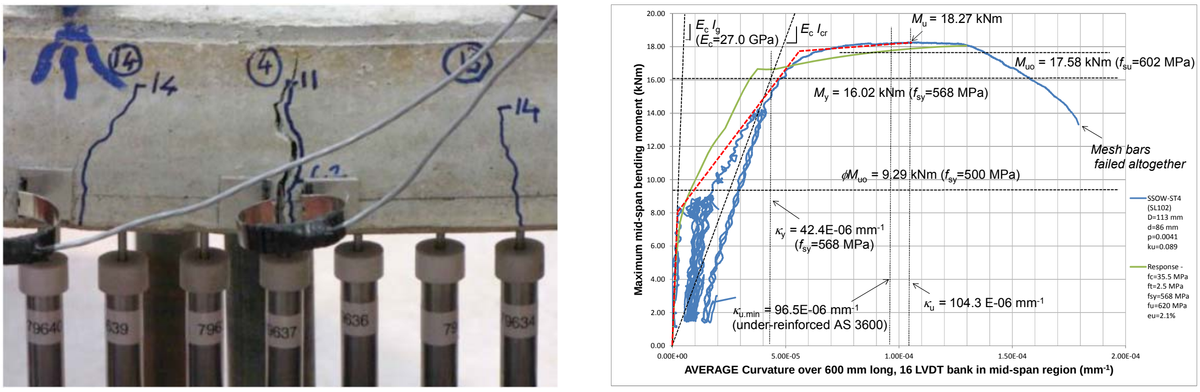
3.3 Unrestrained Continuous One-Way Slab under Standard Fire Conditions

Two continuous slabs with 4.7 m spans were fire tested at BHP Melbourne Research Laboratories in 1997. The 120 mm deep slab shown in Fig. 1(c) was longitudinally unrestrained and incorporated top and bottom continuous layers of low-ductility mesh with uniform strain ε_u =2.75% and tensile-strength-to-proof-stress ratio=1.05 (29). The support regions were conservatively designed for -30% redistribution under ambient temperature conditions, and were under-reinforced ($p^{-}=A^{-}_{st}/(bd^{-})$=0.0033). Plastic hinges quickly formed in these critical regions, extended in length during the initial heating period and remained intact until the test was ultimately terminated due to excessive slab deflection. No reinforcing bars fractured during the fire test. Companion, nominally identical short slabs were tested to failure under ambient temperature conditions to gain information about the support regions during the initial heating period (see Fig. 1(b)) with steel ductility a variable. A slab incorporating Ductility Class N reinforcement was also tested. A full report on these important tests will be published by the SRIA as background information to the fire design rules in AS 3600–2009 for solid reinforced-concrete slabs.

3.4 Simply-Supported One-Way Slabs

Patrick and Keith (18) reviewed seven tests performed by the University of New South Wales on simply-supported one-way under-reinforced slabs incorporating Ductility Class L mesh. There was only one line load per span, which would have reduced the ultimate deflection compared to uniform loading. Consistent with previous studies it was observed that at ultimate load: (i) bars fractured in the peak moment region; (ii) maximum deflection could be relatively small; and (iii) failure appeared sudden under sustained loading and without any end restraint with the slab ends supported on rollers. More importantly, however, Patrick and Keith used the peak moments from these statically determinate tests to show that rectangular stress block theory (see Section 2.8 above) could be used with confidence in design to calculate M_{uo} for singly-reinforced sections incorporating Class L mesh without being concerned about possible steel fracture.

Munter and Patrick (19) report on eight simply-supported slabs tested by Chandler and Lloyd (20). For the first time in Australia, some of the slabs had a mix of Class L mesh and Class N bars. For these 8 tests the ratio of maximum test moment to design moment capacity (ϕM_{uo} with ϕ=0.64) varied from 1.93 to 2.08. During seven of these tests, sixteen LVDT's were positioned at 40 mm centres along the uniform moment region between the inner two line loads (see Fig. 2(a)). By assuming the deflected shape of the slab over this 600 mm long region was circular, the average curvature was computed and plotted as a function of maximum bending moment (e.g. see Fig. 2(b)). This is the first time a moment-curvature relationship has been derived experimentally for concrete slabs incorporating Class L mesh in Australia. Its relevance for the whole span has been verified by using it to accurately calculate the load-deflection curve. The values of M_y and κ_y shown in the graph to predict when tension stiffening should be minimal were calculated using conventional Eqs 5.30 and 5.31 of Warner et al. (21) using f_{sy}=568 MPa for the SL102 bottom mesh.



(a) Test set-up for vertical deflection measurement (b) Moment-curvature relationship (Slab SSOW-ST4)

Figure 2 Example average moment-curvature relationship for a uniform moment region

For the Curtin University tests by Chandler and Lloyd (20), the mechanical properties of the reinforcing bars were studied in considerable detail (22). Using the American equation above, ε_u=0.37d_b, for SL92 and SL102 meshes ε_u=3.2% and 3.5%, respectively, while mean test values were 3.07% and 3.5%.

3.5 Two-Span Continuous One-Way Slabs

Patrick and Keith (18) reviewed eight tests performed by the University of New South Wales on two-span continuous one-way under-reinforced slabs incorporating Ductility Class L mesh. The ends were on roller supports, and again there was only one line load per span. The slabs were not designed elastically with zero moment redistribution when the reinforcement was detailed, and therefore their details did not comply with AS 3600–2009. Patrick and Keith concluded that despite large amounts of moment redistribution occurring, the full plastic hinge mechanisms did not form. This was also the case for a beam designed for 30% redistribution in the study by Patrick et al. (16). It was also concluded that there was no definite trend regarding the effect of articulation (i.e. 1- or 2-span) on maximum deflection at peak load.

3.6 Two-Span Continuous One-Way Slabs, with or without Relative Support Settlement and/or Restrained Ends

Munter et al. (23) have reviewed three independent test series undertaken at Australian universities to examine the potentially detrimental effect that relative support movement could have on the load-carrying capacity of continuous one-way slabs incorporating Class L mesh. Despite inducing a large amount of moment redistribution by imposing significant differential support settlement before loading a slab to failure, this had little effect on load-carrying capacity. This capacity was estimated either analytically or preferably from a test on a companion slab tested in its original position without support settlement. The ductility of the mesh bars in all of the tests exceeded the minimum ductility requirements of AS/NZS 4671, but was representative of the ductility commonly achieved in practice.

For two companion Curtin University two-span slabs with unrestrained ends tested with or without relative support settlement, the maximum positive bending moments were found to be about 27% higher than expected. Investigation showed that this was due to tension developed in the mesh nearer the compressive face, noting these were the first Australian tests to have doubly-reinforced critical sections.

3.7 Single-Span One-Way Slabs with Restrained Ends

Munter and Patrick (19) explain that for the Curtin University tests performed by Chandler and Lloyd (20) it was considered very important at least in some tests to comply with the robustness design requirements of AS 3600 (see Section 2.4 above), provide continuity of bottom tensile steel at supports, and to also allow compressive membrane action to develop. In a single-span one-way slab both ends were effectively fully built-in, and the load-carrying capacity was increased enormously with the ratio of ultimate test load to factored live load $P_u/1.5Q$=5.32, with Q based on elastic analysis without moment redistribution and noting that dead load was only small. This load ratio was over twice that reached in unrestrained tests. Plastic analysis was able to much more accurately model behaviour and predict ultimate strength.

3.8 Single-Panel Two-Way Slabs

Munter and Patrick (19) mention the tests that have been performed in Australia on single-panel two-way slabs incorporating Class L mesh. Even in tests with rotationally unrestrained edges membrane action was significant and increased strength. At Curtin University a rectangular single-panel two-way slab with all four edges effectively fully built-in was tested. Initially it was proof-tested according to Appendix B of AS 3600–2009 and calculating design action effects using Clause 6.10.3, but remained uncracked. When tested to failure with four concentrated loads, load ratio $P_u/1.5Q$=6.52 assuming one-way elastic strips, and it required plastic analysis to much more accurately model behaviour and predict ultimate strength.

4. Theoretical Studies to Develop AS 3600–2009 Design Rules

4.1 Introduction

It is apparent from Section 3 that in Australia there has now been extensive experimental testing of slabs incorporating Class L mesh. In comparison, the theoretical studies performed to date have been relatively limited, and the opportunity exists to study and use all of the test data in much more detail in the future.

4.2 Simplified Methods of Analysis

Patrick et al. (24) reviewed the simplified methods of analysis for reinforced continuous beams and one-way slabs, and reinforced two-way slabs supported on four sides with regard to the amount of moment redistribution incurred in critical moment regions under both serviceability and ultimate load conditions. The theoretical action effects were conservatively determined assuming prismatic, uncracked members ignoring any moment redistribution. Finite element analysis accounting for torsion was used for the parametric study to derive moment coefficients and investigate current reinforcement detailing rules. The Australian tests on beams and slabs incorporating low-ductility reinforcement (see Section 3) have demonstrated that significant amounts of moment redistribution can occur without any detrimental effects, and accordingly it was assumed that up to 10% moment redistribution can be reliably accommodated. Therefore, while conducting the analysis for beams and one-way slabs, maximum moments calculated to be within 10% of the negative or positive design moments in AS 3600–2001 were deemed to be satisfactory and no change was made in such cases. In the case of slabs supported on four sides, moments derived from the finite element analysis were averaged over middle strips such that the peak values could be up to 10% greater than the design value.

Similar studies could be undertaken in the future to investigate how some of the other simplified methods of analysis in AS 3600–2009, e.g. the Idealized Frame Method of Analysis in Clause 6.9, could possibly be modified to accommodate using Class L mesh as longitudinal tensile steel.

4.3 Strength Design Penalty based on Strain Localisation Theory

Foster and Kilpatrick (25,26) subsequently undertook a theoretical study in order to review the arbitrary 20% penalty (18) when using Class L mesh as longitudinal tensile reinforcement by reducing the maximum value of ϕ in Table 2.2.2 from 0.8 to 0.64 (see Section 2.3). No statistically-based reliability analysis has yet been undertaken to confirm this penalty, and on the contrary, a preliminary strength assessment of the Curtin University test data shows that elastic analysis ignoring moment redistribution can be excessively conservative (27). Options for improving the design efficiency are given in Section 5.3.

Foster and Kilpatrick (25) used a theoretical tension-chord model based on an assumed bond-shear-stress-slip relationship and crack spacing to predict the mean strain in the tensile reinforcement (ε_{sm}) over a uniform-moment region and thus calculate their strain localisation factor (SLF) given by SLF=$\varepsilon_u/\varepsilon_{sm}$, where ε_u is the maximum tensile strain in the bars at a cracked section and was taken as the lower characteristic uniform strain for the ductility class of reinforcement, e.g. 0.015 for Class L mesh. Cross-section equilibrium of forces at ultimate moment based on the mean steel strain between cracks gave the maximum concrete compressive strain, ε_{cu}, and therefore the ultimate (average) curvature, κ_u.

Stating that there is experimental evidence for slabs incorporating Class L mesh that flexural cracks coincide with the location of mesh transverse bars, they assume a flexural crack spacing of 200 mm in their examples. Such an observation implies poor bond, and yet they are critical of the bond developed by deformed ribbed bars as being too high. Moreover, a typical pattern of flexural cracks in a uniform moment region of a 110 mm deep slab in the Curtin University tests (which had mesh with transverse bars at 200 mm centres), as seen in Fig. 2(a), clearly shows that this assumption is wrong and much too conservative. Their theory predicts much smaller values of average ultimate curvature, κ_u, than have now been determined experimentally, such as shown in Fig. 2(b) where κ_u (=104.3x10^{-6} mm^{-1}) actually exceeds the minimum ductility requirement of 0.0083/d_o (=96.5x10^{-6} mm^{-1}) of AS 3600–2009. Correspondingly, plastic hinge lengths are also much longer than they suggest which can be proven knowing the moment-curvature relationship, slab articulation and loading configuration. The large amount of post-peak curvature exhibited in Fig. 2(b), which occurred after the onset of necking of the reinforcing bars, can also significantly increase the amount of moment redistribution approaching ultimate load.

5. Discussion of AS 3600–2009 Design Rules for Ductility Class L Reinforcement

5.1 Introduction

The authors provide the following commentary on some of the main design rules described in Section 2.

5.2 Clause 1.1.2 Application

By large plastic deformation it is meant reaching longitudinal tensile strains at any point along the bar or wire in excess of uniform strain, ε_u, which corresponds to the onset of local necking when the maximum stress is reached in the bars. Moreover, at strains in excess of the uniform strain a reinforcing bar would be in a state of incipient failure due to bar fracture. Premature fracture of the reinforcing bars at a critical section of an indeterminate member could prevent the formation of a complete collapse mechanism and thereby unduly weaken the member which should be avoided. Munter and Patrick (19) explain this using a bi-linear stress-strain curve for reinforcement, which conforms to Clause 3.2.3 of AS 3600–2009, which for example can be employed using non-linear stress analysis in accordance with Clause 6.6 of AS 3600.

5.3 Clause 2.1.1 Design for Strength and Serviceability

Concerning the 20% penalty on ϕ in Table 2.2.2 discussed in Section 4.3, test results show that it can lead to very conservative designs when elastic analysis without moment redistribution, or simplified methods of analysis which are based on this approach, are employed (19,27). As mentioned in Sections 3.7 and 3.8, alternative methods of analysis including plastic analysis can give more realistic designs, particularly if significant compressive membrane action can develop.

In practice, critical regions of solid reinforced-concrete slabs incorporating Class L mesh are doubly reinforced. Further to the discussion in Section 3.6, Munter and Patrick (19) have proposed a simplified calculation procedure to determine how much the layer of mesh nearer the compressive face may actually contribute in tension. This contribution can potentially fully negate the 20% penalty.

Concerning the mixing of Ductility Class L mesh and Class N bars in a single layer of tensile reinforcement, as can occur in slabs where the mesh is supplemented over supports with bars, or mesh in a slab forming a beam flange acts with bars in a beam, the new note to Table 2.2.2 mentioned in Section 2.3 suggesting that all of the steel should be penalised merits further consideration. A more reasonable yet simple, less conservative design approach is proposed in Ref. 28. Further studies are still required to address more general cases with steel bars of different ductility classes at different depths.

Further to the discussion in Section 3.6, support settlement no longer normally needs to be considered in design using AS 3600–2009, noting that it is too conservative to try and estimate its effects using elastic analysis. Further design advice on this topic is given in Ref. 29.

5.4 Clause 2.1.3 Design for Robustness

Slabs on roller supports, as used in many of the Australian laboratory tests, obviously do not satisfy the design requirements of Section 6 of AS/NZS 1170.0. Therefore, the only test members described in Section 3 that complied with AS 3600–2009 in this regard were the restrained slabs tested at Monash and Curtin Universities. Compressive membrane action developed, greatly enhancing their load-carrying capacity and post-peak behaviour. Straightforward strength calculations normally determine the adequacy of the mesh to connect and tie the structure together to satisfy the structural robustness requirements.

5.5 Clause 2.1.4 Design for Durability and Fire Resistance

Beams and slabs incorporating Ductility Class L mesh acting as longitudinal tensile reinforcement may be designed for fire resistance in accordance with Section 5 of AS 3600–2009 provided the designs satisfy all of the relevant design and detailing criteria for ambient temperature conditions in the Standard.

5.6 Clause 2.1.5 Material Properties

When undertaking non-linear frame analysis in accordance with Clause 6.5 of AS 3600–2009, calculations shall be undertaken using the mean values of all relevant material properties, with additional analysis to allow for variability. Random sampling of Ductility Class L mesh for all the Australian slab tests mentioned in Section 3 indicates that mean values for the ductility parameters such as uniform strain, ε_u, are significantly higher than the minimum values specified in AS/NZS 4671. It should also be borne in mind that a sheet of mesh is normally made from steel off multiple coils, further increasing the probability that the average properties will be well above the minimum requirements.

5.7 Section 6 Methods of Structural Analysis

Reference 27 clearly illustrates how the various methods of analysis described in Section 2.7 can be applied in practical design situations when Ductility Class L mesh is used as main tensile reinforcement.

5.8 Section 8 & 9 Design of Beams and Slabs for Strength and Serviceability

The Curtin University tests on mixed steel described in Section 3.4 and theoretical moment-curvature analysis of cracked sections confirm that strain compatibility does not have to be considered in design when the tensile reinforcement is in one layer. This confirms using rectangular stress block theory for singly-reinforced sections as described in Section 2.8. For more complex situations potentially involving low-ductility tensile reinforcement in multiple layers (see discussion in Section 5.3 for doubly-reinforced sections) or mixed steel (Classes L and N) in multiple layers, non-linear moment-curvature analysis taking into account the stress-strain curves of the reinforcement may be undertaken to satisfy Clause 1.1.2 of AS 3600 – 2009, as described in Sections 2.2 and 5.2 above.

5.9 Section 13 Stress Development of Reinforcement and Tendons

The new rules for anchoring or lapping mesh were taken directly from ACI 318-08.

5.10 Section 17 Materials and Construction Requirements

Redesign of affected members would be necessary if *significant* amounts of moment redistribution at the strength limit state were assumed in design. In accordance with the description in Section 4.2, by *significant* it would be reasonable to assume in excess of 10% redistribution, and only where the moment redistribution assumed caused a change in the amount of Class L mesh required.

6. Case Study illustrating Use of Ductility Class L Mesh in Construction

The practical and economic advantages of using Ductility Class L mesh in construction can outweigh any non-essential benefits gained from using more ductile reinforcement such as Ductility Class N bars. Several buildings are described to illustrate this point, all of which were under construction at the time this paper was written. By using the relevant design rules in AS 3600–2009, as described in Section 2 and discussed in Section 5 above, the performance requirements of the structural design can be readily met and the Building Code of Australia satisfied.

The fire-rated concrete building in Fig. 3(a) comprises 19 above-ground floors, each a conventional cast-in-situ Grade 32 MPa, 200 mm thick one-way or two-way slab or flat plate, doubly-reinforced with standard SL102 D500L mesh sheets, and only relatively few additional N bars (typically N16 in the slabs) where required in isolated locations such as over columns, resulting in a very simple, repetitive reinforcement layout. The slabs span from about 4.1 up to about 7.3 metres and are supported by precast panels and blade columns. Accounting for the effects of minimum mesh lapping (18,28), average steel cross-sectional areas in the longitudinal and transverse directions are increased an average of about 6% for SL102 mesh. The minimum bending strength requirements of Clauses 9.1.1 and 8.1.6 of AS 3600–2009 are therefore met for both the two-way and one-way areas, respectively.

(a) External precast concrete panel façade

(b) Typical floor – manual placement of SL102 mesh sheets

(c) Typical floor – chairing mesh sheets

(d) Typical floor – mixed steel at a column

(e) Typical floor – mixed steel at pour joint

Figure 3 Case study of a medium-rise residential apartment building with Ductility Class L mesh

By using mesh as the primary reinforcement rather than individual reinforcing bars, an 8-day floor cycle (1200 m^2 per floor) was achieved, estimated by the builder to be a saving of two days per floor using the same labour force. A major cost saving resulted by reducing the time a tower crane was on site.

Using a standard SL102 mesh with a 200×200 mm bar grid allows workers to stand directly on the formwork and fit chairs through the mesh wherever required, as shown in Fig. 3(c), which is important for constructability. Weight is another important factor, and as seen in Fig. 3(b) two men could lift and place a standard 2.4×6.0 m sheet of SL102 mesh. The sheets of mesh could be efficiently placed directly over column starter bars, and as shown in Fig. 3(d) the simple addition of N16 top bars for additional shear strength completed reinforcement detailing in this area. Another example of simple detailing, and also mixed construction with both Ductility Class L and N reinforcement, was at pour joints (see Fig. 3(e)).

The Steel Reinforcement Institute of Australia is currently gathering detailed information on numerous other Ductility Class L reinforcement projects involving both mesh and bar. Case studies are being prepared featuring these new projects that range from: the common one- to four-storey walk-ups; to multi-level residential apartments over commercial offices and carparks; to multi-level feature-buildings. These studies will be showcased on SRIA's website (www.sria.com.au) to capture the key benefits for builders and designers that this essential construction solution delivers for the Australian building sector.

7. Conclusions

This paper has primarily been written to bring design and construction engineers up with the very latest developments in Australia concerning the use of conventional D500L reinforcing mesh in buildings designed in accordance with Concrete Structures Standard AS 3600–2009. In the introduction it was explained that low-ductility mesh products are also manufactured in Northern America, where they are used extensively in bridges as well. Australian and American experience extends over many decades with countless successful projects, both technically and economically. Mesh was introduced into Australia in 1918, and produced in all States by 1963. Awareness about design issues concerning low-ductility mesh has widely increased in Australia since the move to high-strength, Grade 500 MPa reinforcing steels and the later adoption of Steel Reinforcing Materials Standard AS/NZS 4671–2001. While research on this topic is still on going, a major milestone reached – to give engineers confidence to use economic D500L mesh and other forms of low-ductility, multi-purpose reinforcement in building construction, although in cases very conservatively – is publication of AS 3600–2009 (and its commentary, anticipated late 2013).

8. Acknowledgements

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