New Developments in the Testing, Design and Construction of Concrete Structures Incorporating Class L Reinforcing Mesh

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1. Introduction

The following significant new developments in the testing, design and construction of concrete structures incorporating Class L reinforcing mesh (see Fig. 1) are briefly described in this paper.

- The details of a *new major experimental test program* being undertaken by the SRIA in collaboration with leading Australian academics and design practitioners are briefly described. The tests are designed to provide new research findings regarding the effects of: mixing Class L and N reinforcing steels; end or edge restraints representative of real construction, as opposed to free restraints normally used in laboratory tests, including membrane action; two-way action; and relative support movement or settlement.

- In light of the latest amendments to AS 3600–2001 for designing slabs and beams incorporating Class L mesh, explaining the significant economic advantage to be gained by adopting *a new design and construction approach* of sizing the reinforcing mesh to control cracking due to temperature and shrinkage effects (when the mesh is not penalised for its low ductility), and then using short Class N bars lapped with the mesh to provide the necessary additional bending strength in peak moment regions, thereby reducing the impact on the total amount of reinforcing steel for a construction project, often to a negligible level.

- Recognising the conservatism of the traditional practice of using the minimum cross-sectional areas of standard meshes based on a typical internal bar at its standard centre-to-centre spacing, to recommend another *new design and construction approach* of taking fully into account, typically an average 5-10% extra steel area due to lapping.


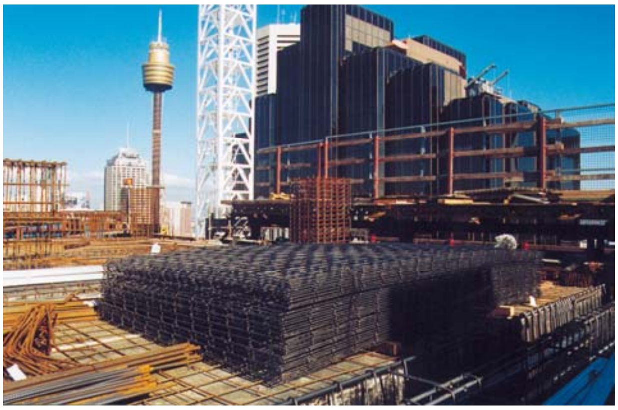

Figure 1 Examples showing low-ductility mesh being used in high-rise building construction

Low-ductility (Class L) mesh produced in accordance with AS/NZS 4671 (Standards Australia & Standards New Zealand 2001) is used extensively as main reinforcement in suspended concrete floors. The Australian Concrete Structures Standard AS 3600 (Standards Australia 2001) allows engineers and builders to design and build many types of concrete structures incorporating Class L mesh, the quality of which has improved to satisfy these complementary material and design Standards, both referenced in the Building Code of Australia (BCA 2008).

Recognition of the importance of reinforcing steel ductility in Australia coincided with the move to 500 MPa as the primary standard strength grade for main reinforcing steels in the form of bars or mesh. Low ductility mesh had been used in Australia as a higher yield strength (450 MPa) alternative form of reinforcement to 400 MPa hot-rolled bars. Hot-rolled Class N bars and Class L mesh, both produced with a yield strength of 500 MPa, may now be used interchangeably or together in suitable types of concrete structures designed in accordance with AS 3600. However, a lower value of capacity reduction factor, ϕ (0.64 instead of 0.80) now applies to calculate the design moment capacity of under-reinforced critical sections incorporating mesh (neutral axis parameter, $k_u \leq 0.4$). The members must be designed elastically assuming no moment redistribution. Several other restrictions are that: (i) the Simplified and Idealized Frame Methods for two-way slab systems in Clauses 7.4 and 7.5 of AS 3600, respectively, may not be used (until they are investigated and possibly modified); (ii) plastic methods of design in Clause 7.9 are deemed unsuitable, as excessively large amounts of moment redistribution could be required to form a full plastic hinge mechanism; and (iii) the types of unbraced moment-resisting frames designed for moderate to severe earthquakes in accordance with Appendix A should not use Class L mesh as main reinforcement. However, these latter three restrictions normally have little significance in everyday design, e.g. plastic design can cause serviceability issues, so is seldom used in practice. *The new rules specifically included in AS 3600–2001 and referenced in the BCA to cater for designing suspended concrete floors incorporating Class L mesh are discussed in Section 2.*

The results of twenty-six (26) full-scale structural tests reported in the literature, performed in Australia on concrete beams and one-way slabs incorporating low-ductility reinforcing steels, the earliest dating back more than a decade ago, are *collectively reviewed in Section 3 for the first time in relation to the design rules in AS 3600*. The tests consistently show that: (i) tensile reinforcing steel can fracture in peak moment regions; (ii) a full plastic-hinge mechanism may not form in a continuous member if a large amount of moment redistribution is assumed in design; (iii) steel fracture can occur when the flexural members have deflected only relatively small amounts; and (iv) sudden collapse of a span can occur if steel fracture at a single hinge causes a mechanism to form. The consequences of these simple phenomena need to be well understood, as the misconceptions amongst some engineers are that (i) tensile reinforcing steel should yield but never fracture; (ii) full plastic-hinge mechanisms should form in continuous reinforced-concrete beams and slabs, irrespective of the effects of steel ductility and redistribution; and (iii) deflections should always be large enough to provide imminent warning of collapse, thus not appearing sudden. As a result of these misconceptions, some of the test results have become controversial, and caused design restrictions and a penalty on the strength reduction factor, ϕ, to be applied when Class L mesh is used as main reinforcement in suspended floors.

Results from the Australian tests are examined in relation to some fundamental aspects of the design rules in AS 3600 developed for Class L reinforcement, viz.: (i) use of ordinary rectangular stress block theory to calculate nominal moment capacity, M_{uo}, ignoring possible steel fracture (as almost universally used by designers in calculations); (ii) formation of full plastic hinge mechanisms in continuous members designed ignoring moment redistribution; (iii) use of elastically-based methods of analysis to calculate design action effects at ultimate load; (iv) ignoring deflections at ultimate load when designing for strength; and (v) penalising designs that could fail suddenly by effectively treating mesh as Grade 400 MPa steel at ultimate load.

None of the Australian tests performed to date on a continuous member has strictly complied with the design requirements of AS 3600–2001. Nor has any of the tests included a mix of reinforcing steels of different ductility classes (Classes L and N), which can frequently occur in practice, partly due to standard Australian meshes having limited cross-sectional areas. Important axial and rotational restraint effects that occur in real structures have also conservatively been ignored. The SRIA-funded research program is being undertaken to address these issues, and to examine what is believed to be a high level of conservatism of the design rules in AS 3600 for flexural members incorporating Class L mesh. Aspects of the tests are very briefly described in Section 4. The test results will be openly reported and will be the subject of future technical papers.

The new, more efficient design and construction approaches involving mixing Class L and N steels (a situation not directly addressed in AS 3600) or using average instead of minimum mesh cross-sectional areas are briefly described in Sections 5 and 6, respectively.

2. AS 3600 design rules for low-ductility main reinforcement

2.1 Calculation of design action effects – elastic analysis with no moment redistribution

Scott and Whittle (2005) confirm that normal practice, as permitted by AS 3600, is to calculate design bending moment and shear force distributions using linear elastic analysis, and that this is endorsed by all the major international design codes for both serviceability and ultimate load conditions, despite non-linear effects due to cracking, creep, shrinkage, temperature, etc. In accordance with Clause 7.6.5 of AS 3600 where the general principles of linear elastic analysis are stated, an estimate of the flexural stiffness of each member may be based on either (i) the dimensions of the uncracked (gross) cross-sections; or (ii) other reasonable assumptions, which better represent conditions at the limit state being considered. Scott and Whittle investigate using the *uncracked concrete section* (ignoring the reinforcement), the *uncracked gross section* (including the reinforcement using a modular ratio) or the *cracked transformed section* (ignoring concrete in tension). They explain that because the reinforcement details are not known at the start of the design process, the uncracked concrete section is normally used, while the other approaches can involve significant iteration depending on how accurately the designer attempts to model the situation. They further explain that moment redistribution will arise at the serviceability and strength limit states due to these and other inaccuracies in the modelling. They recommend for normal design that the simplest *uncracked concrete section* approach be used, as per AS 3600.

In accordance with Clause 7.6.8.3 of AS 3600, for beams, one-way slabs and two-way slab systems with Class L reinforcement, they must be designed to carry the elastic distribution of bending moments at all locations. Readjustment of the elastically-calculated bending moments, i.e. moment redistribution, is not permitted, which in any case is normally best avoided also for serviceability design. These principles of elastic analysis were applied in a detailed study by Patrick et al. (2005) to improve the simplified design methods in Clause 7.2 of AS 3600 for continuous beams and one-way slabs, and Clause 7.3 for two-way slabs supported on four sides.

2.2 Calculation of cross-section moment capacity – rectangular stress block theory

The nominal moment capacity, M_{uo}, of reinforced concrete cross-sections incorporating Class L mesh may be calculated using the basic principles set down in Clause 8.1.2 of AS 3600, which includes the rectangular stress block rules in Clause 8.1.2.2. In normal singly-reinforced slabs (i.e. slabs with one layer of tensile reinforcement) with under-reinforced sections, it is *unnecessary* to consider the possibility of premature steel fracture, using strain compatibility, as Class L steel with a uniform strain, ε_u, of at least 1.5% (as required by AS/NZS 4671) is sufficiently ductile to attain its full tensile strength in a destructive test to failure (and yield strength in design) - see below.

2.3 Mixing reinforcement types (Class L mesh, Class N bars & prestressing tendons)

Class L mesh is often used in conjunction with Class N bars or bonded prestressing tendons, but no specific rules exist in AS 3600 about how they can be used together, which is permitted. It is particularly useful to supplement reinforcing mesh with short additional Class N bars in critical regions of peak bending moment. This way, it may be possible to avert having to use a heavier mesh than is required for secondary effects (shrinkage and temperature), and thus maintain a lighter design. This important design and construction issue is discussed in Section 5 with reference to a fully worked example prepared by the SRIA (Steel Reinforcement Institute of Australia 2008).

3. Review of Australian test data – beams and one-way slabs

3.1 General

Some details of the twenty-six Australian tests on beams and one-way slabs with low-ductility reinforcing steel reported in the following technical papers are summarised in Appendix A: (1) Patrick, Akbarshahi and Warner (1997); (2) Adams et al. (1997); (3) Smith and Gilbert (2003); (4) Gilbert and Smith (2006); (5) Gilbert (2005); (6) Gilbert, Sakka and Curry (2006); and (7) Siddique (2005) – see Column 3 in the table in Appendix A for these reference numbers.

3.2 Cross-section nominal moment capacity, M_{uo} – rectangular stress block theory

Rectangular stress block (simple plastic) theory has been used to predict the moment capacity, $M_{u.RSBT}$, of every peak moment region that failed in flexure in each of the tests. This is the most commonly used design method. (An alternative, more complex approach would be to account for strain compatibility and use a representative stress-strain curve for the concrete according to Clause 6.1.4 of AS 3600, which would account for the low concrete strain in lightly-reinforced sections.) Representative mechanical testing of the steel and concrete in the test specimens was performed for all the tests. Because every specimen was under-reinforced, every failure involved necking followed by tensile fracture of the low-ductility reinforcing steel, and therefore the maximum bending moment reached was controlled in every case by the tensile strength of the main steel. In contrast, the compressive strength of the concrete only had a small influence, as this only slightly affects the calculated depth of the compressive stress block and therefore the lever arm between the idealised resultant tensile and compressive forces of the singly-reinforced sections.

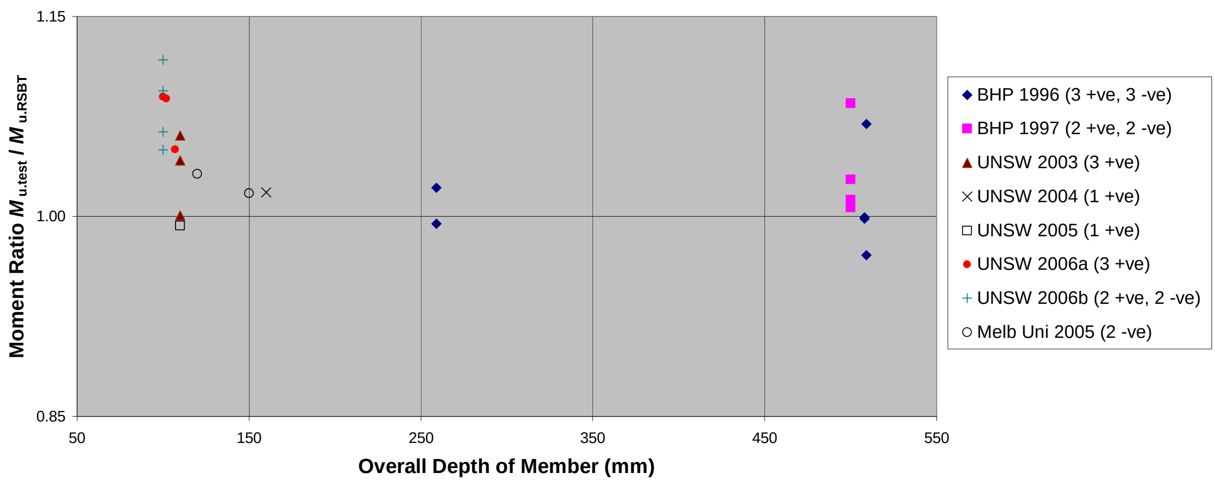

Figure 2 Moment ratio $M_{u.test}/M_{u.RSBT}$ for all Australian tests when the tensile steel fractured

All the two-span beams of Patrick, Akbarshahi and Warner (1996) were tested in two stages in order to obtain the moment capacity, $M_{u.test}$, of the critical sections in both positive and negative bending. Therefore, six points in Fig. 2 were obtained from the three tests labelled *BHP 1996*. However, in the two-span slab tests by Smith and Gilbert (2003) the bending moments could not be reliably determined, so no results are shown. It follows from Fig. 2, in which the average value of the moment ratio $M_{u.test}$ / $M_{u.RSBT}$ equals 1.04, that rectangular stress block theory is a reasonably accurate, conservative predictor of moment capacity in slabs and beams incorporating low ductility steel for a wide range of overall depths. Therefore it can be used with confidence in design to calculate M_{uo} for singly-reinforced sections incorporating Class L mesh, in either negative or positive bending, without having to be concerned about possible steel fracture.

A reason for the consistently higher test results is the effect of the tensile strength of concrete, as borne out by more accurate moment-curvature analysis in which full stress-strain relationships of the steel and concrete are modelled. Variation of tensile strength between wires of a mesh (which come off different coils during production) would also explain some of the scatter in Fig. 2. There was a degree of uncertainty about the accuracy of the dead loading in Siddique's tests.

3.3 Formation of full plastic hinge mechanism in continuous members designed ignoring moment redistribution

It was stated in the Introduction that none of the continuous Australian tests performed to date strictly complies with the design requirements in AS 3600–2001. The original numerically-based parametric studies undertaken by the University of Adelaide, supported by testing and partly reported by Patrick, et al. (1996), showed that typical continuous members incorporating Class L steel should form full plastic hinge mechanisms if designed elastically ignoring moment redistribution. However, the proportions of reinforcement in most of the continuous specimens tested differed to that required by the theoretical elastic distribution of bending moments.

For example, consider two-span slab OWC5 reported by Gilbert (2005). Initially the slab exhibited very large amounts of moment redistribution, i.e. -38% and +25% in the peak negative and positive moment regions, respectively (Keith et al. 2007), consistent with the findings of Scott and Whittle (2005). By the end of the test when the first hinge formed in the positive moment region, the redistribution was only about +6% and -4%, so the bending moments were close to the elastic values. However, only 93% of the plastic collapse load corresponding to a full plastic hinge mechanism was reached. Elastically, the nominal negative moment capacity of the interior support should have been 20% larger than the nominal positive moment capacity of the mid-span regions. Instead, the nominal ratio was 1.59, while the ratio of actual moment capacities was shown to be 1.43. Accordingly, a significantly larger peak negative bending moment would have had to be developed compared with the elastic value, for a full plastic hinge mechanism to be achieved. A close examination of the test results shows that a full mechanism would have formed had the actual ratio of negative and positive moment capacities equalled 1.2 while leaving the positive moment capacity unchanged, thus conforming to AS 3600–2001. Moreover, a similar finding applies to all the Australian continuous slab tests that have adequate test data, confirming the findings of the original Adelaide University study and the correctness of the design principle in AS 3600–2001 that this will occur if moment redistribution is ignored in design.

3.4 Elastically-based methods of analysis to calculate ultimate design action effects

All of the continuous slabs for which adequate test data were obtained have been analysed elastically, like just explained above for slab OWC5. With large amounts of moment redistribution typically occurring during the course of loading, it is clear that full plastic hinge mechanisms would have effectively formed had the reinforcing steel been distributed according to the approved elastically-based methods of analysis in AS 3600. In the case of the two slabs tested by Siddique (2005), which were both subjected to significant support movements prior to loading to failure, this was still the case even though the slabs were designed using the moment coefficients appropriate to Class N steel in Clause 7.2, as the rules in AS 3600 had not been modified at the time. Other examples of full plastic mechanisms effectively forming in slabs with non-conforming reinforcement layouts are slabs CS1 and CS3 of Gilbert, Sakka and Curry (2006), while it is entirely expected that this did not occur in slab CS2, which had about twice the amount of top steel required elastically, and therefore only failed in the mid-span region.

3.5 Ignoring deflections at ultimate load when designing for strength

Warning of impending failure or imminent collapse is not a design requirement in Australian structural design Standards or the BCA. Many forms of construction would potentially be ruled out if it were, e.g. cold-formed steel. The incident with the mobile crane in Fig. 3 illustrates sudden failure, but the crane weighed 19.2 times the rated vehicle load, and the simply-supported concrete panels would have collapsed suddenly irrespective of what they were made of, i.e. in this case, post-peak-strength response would not have been improved at all by greater ductility.

Many factors can significantly affect deflection at ultimate load. However, as shown by the Australian test data in Fig. 3 with maximum deflection at peak load plotted against overall member depth, there is no definite trend regarding articulation (i.e. 1- or 2-span). The data includes a brittle slab with insufficient steel to form multiple cracks, some quite stocky slabs, many slabs with single line loads, and none with any significant restraint effects, all factors that can reduce the deflection significantly. Without any stated basis, Gilbert and Smith (2006) imply that span/68 provides "*ample warning of failure*", achieved in some single-span tests. However, it would be impractical to design many types of structures for such an arbitrary ultimate deflection limit.

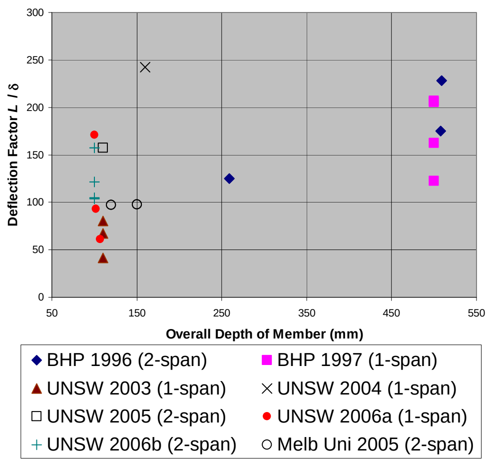

Figure 3 Sudden failure of severely overloaded slab, & deflection at failure in Australian tests

3.6 Penalty for sudden failure, and resulting design conservatism using AS 3600

Despite the fact that Class N bars can fracture too if the curvature in critical regions is sufficiently large, flexural members incorporating Class L mesh have been arbitrarily penalised by 20%, effectively reducing the yield strength to 400 MPa for strength design. As a result, it is seen from Fig. 4 that design in accordance with AS 3600 is very conservative indeed; viz. the real moment capacity of a plastic hinge can be expected to be at least twice the design moment capacity, while the method of analysis and load factors can add significant extra conservatism again. It follows that by using a method of analysis based on elastic theory and ignoring moment redistribution when determining the design bending moments at critical sections in accordance with AS 3600, that an average factor of safety against collapse of about 2.0×1.35=2.7 is expected. The reliable, high tensile strength of Class L mesh contributes significantly to this high level of safety, and so too does its improved, reliable level of ductility, as the test results show.

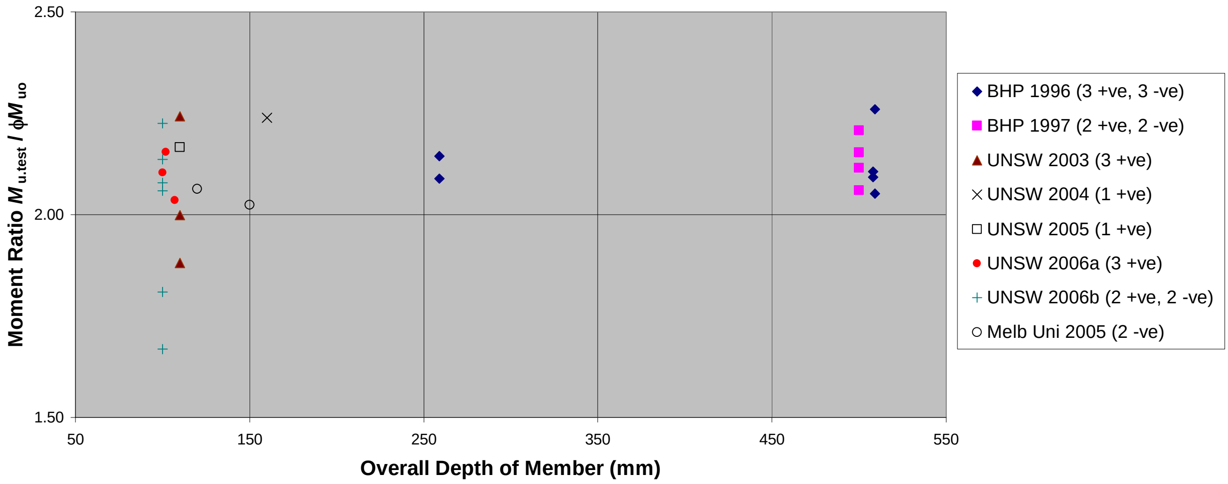

Figure 4 Moment ratio $M_{u.test}/\phi M_{uo}$ for Australian test data in Fig. 2

4. New SRIA Class L Experimental Test Program

A special tubular steel ringbeam shown in Fig. 5 is being used as a universal test rig to study three different types of slabs (single-span one-way, two-span one-way or two-way) with either restrained or pinned edge support conditions. In the two-span tests, the middle support, viz. the middle beam in Fig. 5, can be moved up or down to simulate differential settlement. The overall depth of the slabs is 110 mm, which contain SL92 and/or SL102 meshes, and possibly N12 supplementary bars. All of the slabs have been designed in accordance with the latest design provisions of AS 3600–2001. The main issues to be investigated were briefly described in Section 1, and include some important factors not previously tested in Australia. One of these factors involves membrane action in the two-way slab with restrained edges, while Bailey et al. (2008) recently reported on the significance of membrane action in two-way slabs incorporating low ductility mesh with unrestrained edges, which they accurately modelled using finite elements. It is planned to take a similar approach, and to develop an accurate numerical model to explain the observed behaviour. For example, in some of the single span tests the loading configuration will be altered, and it should be possible to explain the effect this has on the load-deflection response using numerical modelling.

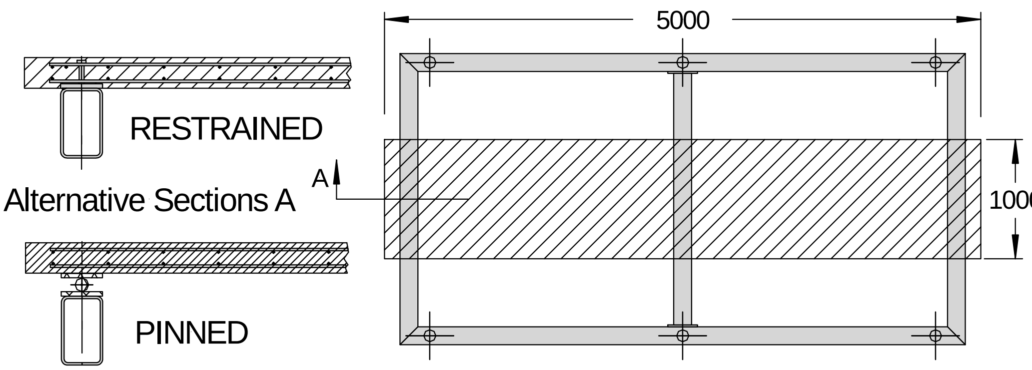

Figure 5 Tubular steel ringbeam universal test rig (two span one-way slab shown)

5. New Design & Construction Approach – Mixing Class L & N Steels

As already mentioned, the issue of mixing Class L and N reinforcing steels together is not directly addressed in AS 3600. It only addresses the simpler cases of reinforced-concrete beam or slab cross-sections containing main steel of one ductility class. The issue is to what degree the Class N steel is stressed before the Class L mesh reaches its tensile strength corresponding to the onset of necking and then fractures. It was mentioned in the last section that mixing steels together is being investigated experimentally. However, theoretical studies conducted by the authors using moment-curvature analysis modelling typical steel stress-strain curves have confirmed that when the two types of steel are effectively in the same plane, i.e. close to the same effective depth, that for design purposes they may be considered to achieve their full strengths. Therefore, the equivalent area of tensile reinforcement, A_{stN}, when the two types of steel are mixed this way, simply equals A_{Nb} + $0.8\,\overline{A}_{b}$ for the calculation of design strength in bending, ϕM_{uo}, using f_{sy}=500 MPa, where A_{Nb} is the cross-sectional area of the Class N bars, and $\overline{A}_{b}$ is the cross-sectional area of the Class L bars, for the same slab width. Using this approach, ϕ=0.8. A worked example is given in SRIA (2008).

6. New Design & Construction Approach – Using Average Mesh Areas

Table 1 contains cross-sectional areas of commonly available Class L meshes used to construct suspended concrete floors like those in Fig. 1. The variables A_{bl} and A_{bt} are the cross-sectional areas of the longitudinal and transverse bars, respectively, *based on the minimum intensity of bars ignoring edge effects and lapping*. These values have historically been used in design.

Table 1 Cross-sectional areas of standard Australian Class L meshes

Ref. No.	Longitudinal		Transverse		Ref. No.	Longitudinal		Transverse	
	A_{bl}	$\overline{A}_{bl}$	A_{bt}	$\overline{A}_{bt}$		A_{bl}	$\overline{A}_{bl}$	A_{bt}	$\overline{A}_{bt}$
RL1218	1112	**1215**	227	**243**	SL81	454	**495**	454	**470**
RL1118	899	**982**	227	**243**	SL102	354	**372**	354	**380**
RL1018	709	**774**	227	**243**	SL92	290	**303**	290	**311**
RL918	581	**634**	227	**243**	SL82	227	**247**	227	**243**
RL818	454	**495**	227	**243**	SL72	179	**190**	179	**192**
RL718	358	**390**	227	**243**	SL62	141	**157**	141	**152**

However, *in uniformly stressed areas* it may be more appropriate to use the *larger average areas* $\overline{A}_{bl}$ and $\overline{A}_{bt}$. For example, the design bending moments, M^*, determined using either of the simplified methods in Clauses 7.2 and 7.3, or general linear elastic analysis in accordance with Clause 7.6 of AS 3600, are normally averaged over a significant width of slab. Therefore, it is normally acceptable to use the appropriate average mesh area $\overline{A}_{bl}$ or $\overline{A}_{bt}$, taking into account the orientation of the mesh bars, as illustrated in a worked design example in SRIA (2008). It can be determined from the values in Table 1 that the average areas are approximately between 5 and 10 percent larger than the minimum areas, as already mentioned, which can be useful in design.

7. Conclusion

Significant new developments in the testing, design and construction of concrete structures incorporating Class L mesh have been described. They include a major experimental test program currently being undertaken by the SRIA, and two important new design and construction approaches that lead to significantly more economical design solutions in the light of recent changes to the design rules in AS 3600, as well as utilising the extra steel at laps in meshes. Important design advice has also been given about accounting for mixing of steel with N and L ductility classes. Detailed analysis of the results of the Australian tests on simply-supported and continuous one-way concrete members incorporating low ductility steel shows that suspended floors incorporating Class L mesh can be safely designed in accordance with AS 3600, which limits the amount of moment redistribution peak moment regions are likely to experience in practice.

8. References

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APPENDIX A – AUSTRALIAN LOW DUCTILITY REINFORCEMENT IN BEAM AND SLAB TESTS: ONE-WAY ACTION

Research Organisation	Year Reported	Ref. No. in §3.1	Type of Member	Test Code No.	Member Dimensions (width × depth × span)	Span & Loading Configuration	Main Reinforcement
BHP	1997	1	Beam	ADF.B01	300 x 250 x 6000	2-span, 2-pt loading per span	Ribbed wire
"	"	"	"	ADF.B02	300 x 500 x 6000	"	"
"	"	"	"	ADF.B03	300 x 500 x 6000	"	"
"	1999	2	T-Beam	Sagging prototype	(600 x 100 top flange 400 x 300 web) x 3900	1-span, 1-pt loading	"
"	"	"	"	Test 1	(600 x 100 top flange 400 x 300 web) x 3900	"	"
"	"	"	"	Test 4	(600 x 100 top flange 400 x 300 web) x 3900	"	"
"	"	"	"	Hogging prototype	(600 x 100 top flange 400 x 300 web) x 6000	1-span, 2-pt loading (beam inverted)	"
"	"	"	"	Test 7	(600 x 100 top flange 400 x 300 web) x 6000	"	"
"	"	"	"	Test 8	(600 x 100 top flange 400 x 300 web) x 6000	"	"
UNSW	2003	3	Slab	S1	850 x 110 x 3500	1-span, 1-pt loading	Ribbed mesh
"	"	"	"	S2	850 x 110 x 3500	1-span, 1-pt loading	"
"	"	"	"	S3	850 x 110 x 3500	1-span, 1-pt loading	"
"	"	"	"	S4	850 x 110 x 1750	2-span, 1-pt loading per span	"
"	"	"	"	S5	850 x 110 x 1750	2-span, 1-pt loading per span	"
"	"	"	"	S6	850 x 110 x 1750	2-span, 1-pt loading per span	"
"	2004	4	"	S7	850 x 160 x 3500	1-span, 1-pt loading	"
"	2005	5	"	OWC5	850 x 110 x 2500	2-span, 1-pt loading per span	"
"	2006	6	"	SS1	850 x 100 x 2000	1-span, 1-pt loading	"
"	"	"	"	SS2	850 x 100 x 2000	1-span, 1-pt loading	"
"	"	"	"	SS3	850 x 100 x 2000	1-span, 1-pt loading	"
"	"	"	"	CS1	850 x 100 x 2000	2-span, 1-pt loading per span	"
"	"	"	"	CS2	850 x 100 x 2000	2-span, 1-pt loading per span	"
"	"	"	"	CS3	850 x 100 x 2000	2-span, 1-pt loading per span	"
"	"	"	"	CS4	850 x 100 x 2000	2-span, 1-pt loading per span	"
Melbourne Uni.	2005	7	Slab	Test Slab 1	600 x 150 x 5000	2-span, supposedly udl, mid support lifted 17 mm	Ribbed mesh
"	"	"	"	Test Slab 2	600 x 120 x 4000	2-span, supposedly udl, mid support dropped 17 mm	"