SRIA's Class L Mesh Elevated Slab Tests: Part 2B – Comparison of Design Strengths to AS 3600–2009 with Test Results

Scott Munter* and Mark Patrick**

** Steel Reinforcement Institute of Australia, PO Box 418, Roseville, New South Wales, 2069, Australia (E-mail: scott.munter@sria.com.au)*

*** MP Engineers Pty Limited, 6 Doowi Court, Greensborough, Victoria, 3088, Australia (E-mail: mp-engs@bigpond.net.au)*

ULTIMATE DESIGN LOAD-CARRYING CAPACITY VS APPLIED TEST LOAD

The values of ultimate design load-carrying capacity determined in Part 2A of this paper are summarised in Table 7. For each method of analysis used (see 3rd column), these values are compared with the ultimate applied test loads given by Chandler and Lloyd (2012), which are reproduced in the second last column, by calculating load ratio, $P_u/(1.5Q)$, values of which are given in the last column. The load ratio equals the ultimate applied test load, P_u, (i.e. the total load applied by the hydraulic jacks, including the dead weight of any ancillary items of the loading system supported on the top surface of a test slab) divided by the factored (ultimate) design live load (equal to 1.5 times design live load, Q, for the slab designed in accordance with AS 3600–2009).

Table 7. Comparison of ultimate design load-carrying capacity and ultimate applied test load.

Test Series	Test Specimen No.	AS 3600–2009 Method of Analysis	Ultimate Design Load *(Dead Load Factor=1.2; Live Load Factor=1.5)*		Ultimate Applied Test Load, P_u	*Load Ratio: $P_u/(1.5Q)$*
			Type	Factored Live Load, $1.5Q$		
SSOW	SSOW-ST1	Linear elastic	$4P^*$	31.50 kN/m	167.5 kN/m	5.32
		Plastic / membrane		95.25 kN/m		1.76
		Plastic		60.15 kN/m		2.78
	SSOW-ST2	Statically determinate	$4P^*$	19.49 kN/m	46.6 kN/m	2.39
	SSOW-ST3	Statically determinate	P^*	9.96 kN/m	25.3 kN/m	2.54
	SSOW-ST4	Statically determinate	$4P^*$	24.89 kN/m	58.5 kN/m	2.35
	SSOW-ST5	Statically determinate	$4P^*$	44.18 kN/m	93.1 kN/m	2.11
	SSOW-ST6	Statically determinate	$4P^*$	45.44 kN/m	101.0 kN/m	2.22
	SSOW-ST7	Statically determinate	$4P^*$	39.29 kN/m	84.0 kN/m	2.14
	SSOW-ST8	Statically determinate	$4P^*$	41.13 kN/m	87.2 kN/m	2.12
	SSOW-TRIAL	Statically determinate	$4P^*$	24.89 kN/m	56.8 kN/m	2.28
DSOW	DSOW-ST1	Linear elastic	$4P^*$	39.50 kN/m	168.0 kN/m	4.25
		Plastic / membrane	$4P^*$	101.50 kN/m		1.66
		Plastic	$4P^*$	58.37 kN/m		2.88
	DSOW-ST2	Linear elastic ignoring support settlement	$4P^*$	39.50 kN/m	163.0 kN/m	4.13
		Plastic / membrane	$4P^*$	101.50 kN/m		1.61
		Plastic	$4P^*$	58.37 kN/m		2.79
	DSOW-ST3	Linear elastic	$4P^*$	24.11 kN/m	91.0 kN/m	3.77
		Plastic	$4P^*$	38.52 kN/m		2.36
	DSOW-ST4	Linear elastic ignoring support settlement	$4P^*$	24.11 kN/m	90.0 kN/m	3.73
		Plastic	$4P^*$	38.52 kN/m		2.34
TW	TW-ST1	Simplified flexural – 4 continuous edges	udl (water)	15.0 kPa	15.5 kPa proof load (1.58 m of water)	Not tested to failure, and undamaged
		Simplified flexural – 4 discontinuous edges	udl (water)	15.2 kPa		
		Linear elastic – 2 one-way strips	$2\times2P^*$	68.28 kN	445.0 kN	6.52
		Plastic – 2 one-way strips	$2\times2P^*$	108.80 kN		4.09
		Linear elastic stress FEM two-way action	$4P^*$	77.34 kN		5.75
		Plastic / yield-line	$4P^*$	230.2 kN		1.93

Load ratio values shown in the shaded boxes in the last column of Table 7 correspond to conventional design practice, being based on either linear elastic analysis (without moment redistribution) in the case of the redundant slabs, or simple statics for the simply-supported slabs (when moment redistribution cannot occur). For the SSOW slabs incorporating a mix of Class L and N main bars, they are based on the conservative assumption that all of the main bars are low ductility, i.e. Class L.

ULTIMATE DESIGN CAPACITIES VS ULTIMATE TEST ACTION EFFECTS

In all the tests with unrestrained edges, which excludes slabs SSOW-ST1, DSOW-ST1, DSOW-ST2 and TW-ST1 with restrained edges, it was possible to accurately calculate the ultimate test action effects (viz. maximum test bending moment, M^+_{max} or M^-_{max}, and maximum test vertical shear force, V^+_{max} or V^-_{max}) at critical or potentially critical cross-sections in bending or shear, respectively. In Tables 8 and 9 these values are compared directly with the corresponding calculated ultimate design section capacities given in Table 1 of Part 2A (values of design moment capacity $\phi M^+_{uo.x}$ or $\phi M^-_{uo.x}$ and values of design transverse shear capacity $\phi V^+_{uc.x}$ or $\phi V^-_{uc.x}$) by calculating their respective ratios.

Table 8. Ratio of maximum test bending moment to design moment capacity.

Test Series	Test Specimen No.	$\phi M^+_{uo.x}$ (kNm/m)	$\phi M^-_{uo.x}$ (kNm/m)	M^+_{max} (kNm/m)	M^-_{max} (kNm/m)	$\frac{M^+_{max}}{\phi M^+_{uo.x}}$	$\frac{M^-_{max}}{\phi M^-_{uo.x}}$
SSOW	SSOW–ST2	7.71	-	15.22	-	1.97	-
	SSOW–ST3	7.71	-	16.07	-	2.08	-
	SSOW–ST4	9.29	-	18.74	-	2.02	-
	SSOW–ST5	14.93	-	28.82	-	1.93	-
	SSOW–ST6	15.30	-	31.18	-	2.04	-
	SSOW–ST7	13.50	-	26.18	-	1.94	-
	SSOW–ST8	14.04	-	27.14	-	1.93	-
	SSOW–TRIAL	9.29	-	18.24	-	1.96	-
DSOW	DSOW–ST3	7.71	9.29	19.5	*18.7* – *not a hinge*	2.53	*2.01* – *not a hinge*
	DSOW–ST4	7.71	9.29	19.4	20.6	2.52	2.22

All the SSOW slabs in Table 8 failed in flexure (by necking/fracture of Class L main bars). Therefore, all the values of maximum test positive bending moment, M^+_{max}, that apply to the simply-supported SSOW test slabs, equal the actual moment capacity of the critical cross-sections in bending.

The values of M^+_{max} and M^-_{max} for each DSOW slab test were not concurrent. They were calculated based on the equilibrium state at maximum applied load using the measured applied loads and central support reactions. The values of M^-_{max} were calculated 75 mm either side of the centreline of the centre support, corresponding to the outer edges of the central steel support plate. The values of M^+_{max} and M^-_{max} all correspond to hinge formation (and subsequent mesh bar fracture) except for M^-_{max} for slab DSOW-ST3, i.e. hinges were not fully developed at centre support before bottom bars broke.

None of the slabs listed in the Table 9 failed by vertical shear. Instead, the maximum load applied to each slab was governed by the bending strength of one or more critical cross-sections in flexure. It follows that for all the simply-supported SSOW test slabs (which excludes SSOW-ST1), the low values of $V^+_{max} / \phi V^+_{uc.x}$ in the second last column of the table do not imply low shear strengths.

Table 9. Ratio of maximum test vertical shear force to design shear capacity.

Test Series	Test Specimen No.	$\phi V^{+}_{uc.x}$ (kN/m)	$\phi V^{-}_{uc.x}$ (kN/m)	V^{+}_{max} (kN/m)	V^{-}_{max} (kN/m)	$\dfrac{V^{+}_{max}}{\phi V^{+}_{uc.x}}$	$\dfrac{V^{-}_{max}}{\phi V^{-}_{uc.x}}$
SSOW	SSOW–ST1	50.74	47.63	-	86.67	-	1.82
	SSOW–ST2	47.63	-	26.17	-	0.55	-
	SSOW–ST3	47.63	-	15.52	-	0.33	-
	SSOW–ST4	50.74	-	32.18	-	0.63	-
	SSOW–ST5	60.36	-	49.42	-	0.82	-
	SSOW–ST6	63.48	-	53.45	-	0.84	-
	SSOW–ST7	58.42	-	44.91	-	0.77	-
	SSOW–ST8	61.66	-	46.55	-	0.75	-
	SSOW–TRIAL	50.74	-	31.33	-	0.62	-
DSOW	DSOW–ST1	47.63	50.74	-	86.60	-	1.71
	DSOW–ST2	47.63	50.74	-	84.70	-	1.67
	DSOW–ST3	47.63	50.74	-	61.35	-	1.21
	DSOW–ST4	47.63	50.74	-	60.41	-	1.19

DISCUSSION

Eight Statically Determinate SSOW Slabs with Unrestrained Ends

For slabs SSOW-ST2 to SSOW-ST8 & SSOW-TRIAL supported on rollers, the values of load ratio, $P_u / 1.5Q$, in Table 7 varied from 2.11 to 2.54, with a mean value of 2.27 which is 46% above a value of (1/ϕ) = (1/0.64)=1.56 corresponding to collapse occurring for design in accordance with AS 3600– 2009. The values of moment ratio, $M^{+}_{max} / \phi M^{+}_{uo.x}$, in Table 8 varied from 1.93 to 2.08, with a mean of 1.98 which is 27% above a value of 1/ϕ =1/0.64=1.56 corresponding to flexural failure. With flexural strength controlling, the corresponding low shear ratios in Table 9 do not imply low shear strength.

Four Statically Indeterminate Slabs (from SSOW, DSOW & TW Test Series) with Restrained Ends or Edges – Designed using Linear-Elastic Analysis

For restrained slabs SSOW-ST1, DSOW-ST1, DSOW-ST2 (this latter slab being initially subjected to significant relative vertical support movement) and TW-ST1, linear-elastic analysis without moment redistribution was used in accordance with AS 3600–2009 to determine the design action effects of ultimate bending moment and vertical shear force at potentially critical cross-sections. The design live load Q was then determined depending on the design moment or shear capacity of the critical cross-sections. In accordance with AS 3600–2009, the effects of relative vertical support settlement were ignored when designing test slab DSOW-ST2. Compressive membrane action, which involves the development of large resultant axial compressive forces that significantly increase the moment capacity of critical cross-sections (just like for normal beam-columns) and therefore the load-carrying capacity of all of these axially restrained test slabs, was also ignored.

For slab SSOW-ST1, load ratio $P_u / 1.5Q$ was a very large value of 5.32 (see Table 7). For slabs DSOW-ST1 and DSOW-ST2, it was 4.25 and 4.13, respectively. For TW-ST1 it was 6.52 or 5.75 with the slab modelled as two one-way strips or as two-way, respectively. The mean value was 5.19, which is 233% above a value of 1/ϕ =1/0.64=1.56 corresponding to collapse by flexural failure. Despite maximum vertical shear force being limited by the moment capacity of critical cross-sections, for slabs SSOW-ST1, DSOW-ST1 and DSOW-ST2 values of the ratio $V^{-}_{max} / \phi V^{-}_{uc.x}$ in Table 9 are between 1.67 and 1.82. Design vertical shear capacity, $\phi V^{-}_{uc.x}$, was calculated in accordance with AS 3600–2009 based on the entire cross-sectional area of the Class L mesh, using a strength reduction factor, ϕ, of 0.7. All of these values exceed 1/ϕ =1.43, which corresponds to theoretical shear failure in design.

Two Statically Indeterminate DSOW Slabs with Unrestrained Ends – Designed using Linear-Elastic Analysis

For slabs DSOW-ST3 and DSOW-ST4 (the latter initially subjected to relative vertical support movement) linear-elastic analysis without moment redistribution was also used. Similarly high values of load ratio, $P_u / 1.5Q$, of 3.77 and 3.73 are given in Table 7, respectively, irrespective of settlement. The mean value of 3.75 is 140% above $1/\phi$ =1/0.64=1.56 corresponding to collapse by flexural failure. Like for the statically determinate SSOW slabs, with flexural strength controlling maximum load, the corresponding low shear ratios in Table 9 do not imply low shear strengths for these DSOW tests.

The values of moment ratio, $M^{+}_{max} / \phi M^{+}_{uo.x}$, in Table 8 were 2.52 and 2.53, with a mean of 2.52 (Curtin University and SRIA, 2011) which is significantly higher (about 27%) than the mean of 1.98 for the tests described at the start of this Discussion. A detailed investigation was made as to the reason for this apparent disparity. Using the best available estimates for parameters that define the geometry and material properties of the test slabs; moment-curvature analysis to accurately predict cross-section positive and negative moment capacities; and confirming that these predictions agreed with the observed sequence of positive and negative hinge formation; it was concluded that the additional bending strength (+ve=20%; -ve=13%) of these doubly-reinforced DSOW slabs, when compared with the SSOW slabs with unrestrained ends, was due to the continuous main mesh bars nearer the compressive face of the critical section actually nearly yielding in tension.

The following strain-compatibility and force equilibrium assumptions can be used to calculate nominal moment capacity, M_{uo}, of a doubly-reinforced Class L cross-section: plane sections normal to the longitudinal axis remain plane after bending; the concrete has no tensile strength; the resultant compressive and tensile forces in the steel and concrete balance; the maximum strain in the extreme compressive fibre, ε_c, equals 0.003 (as shown in Fig. 2 of Part 1A, it can be much less); a uniform compressive stress of $\alpha_2 f'_c$, where α_2=1.0 – 0.003f'_c within the limits 0.67≤α_2≤0.85, acts on an area bounded by the vertical edges of the cross-section and the extreme compressive face; and the tensile strain in the layer of steel nearest the tensile face equals the lower characteristic uniform strain, ε_{su}=1.5%=0.015=6ε_{sy} with a corresponding resultant tensile stress f_u (=1.03f_{sy}). The strain in the layer of steel nearest the compressive face, ε_{sc}, is directly and simply calculated as $\varepsilon_{sc} = -\varepsilon_c + d_{sc}/d_{st}(\varepsilon_{su} + \varepsilon_c)$ where d_{sc}=the effective depth of the layer of steel nearest the compressive face; and d_{st}=the effective depth of the layer of steel nearest the tensile face. Using this approach, 0.6 ≤ϕ≤ 0.64 for Class L steel.

All Statically Indeterminate Slabs mentioned above (SSOW-ST1, DSOW-ST1 to DSOW-ST4 & TW-ST1) – Designed using Plastic Analysis

Although not permitted by AS 3600–2009 when designing slabs incorporating Class L main reinforcement, plastic analysis was used to design all of the statically indeterminate slabs, possibly accounting for compressive membrane action, which gave significantly reduced but still acceptable load ratio values (see unshaded cells in last column of Table 7). Therefore, plastic analysis more accurately represented the behaviour of the indeterminate slabs than elastic analysis, particularly when large amounts of moment redistribution occurred during a test, e.g. due to support settlement.

CONCLUSIONS

For each of the nine single-span one-way (SSOW) slabs, four double-span one-way (DSOW) slabs and the two-way (TW) slab tested, the paper describes: the design of the slabs in accordance with AS 3600–2009; the calculation of load ratio, $P_u / 1.5Q$, equal to the ultimate test load (i.e. the total load applied by the hydraulic jacks, including the dead weight of any ancillary items of the loading system supported on the top surface of a test slab) divided by the ultimate design load-carrying capacity; and a comparison of ultimate design section capacities with ultimate test action effects, at least for the slabs with unrestrained edges for which the action effects could be accurately calculated.

A major finding was that elastic methods of analysis applied to the design of typical redundant slabs with restrained ends or edges can be much too conservative, and that plastic analysis can much more accurately model real behaviour and predict ultimate strength, particularly if compressive membrane action can develop. Also of major significance is that tests on doubly-reinforced slabs (typical of actual construction) infer that the continuous main Class L mesh bars near the compressive face of a critical section could yield in tension, which contributed as much as 20% to the moment capacity of the sections. Therefore, there is a strong case that the 20% penalty applied to $\phi=0.8$ when calculating the design moment capacity of doubly-reinforced sections like those tested should be waived, if, as is normally the case, they are treated in design as singly-reinforced sections. This has been shown to be a direct consequence of the ductility of the Class L bars. Alternatively, a design method was given to more accurately calculate the nominal moment capacity of doubly-reinforced sections with Class L bars using lower characteristic uniform strain, ε_{su}=1.5%, design tensile strength $f_u = 1.03 f_{sy}$, and $0.6 \leq \phi \leq 0.64$.

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