SRIA's Class L Mesh Elevated Slab Tests: Part 2A – Strength Design to Concrete Structures Standard AS 3600–2009

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ABSTRACT
SRIA commissioned Curtin University to test SSOW-series, DSOW-series and TW-series of reinforced-concrete slabs incorporating Ductility Class L mesh. Test details and ultimate strength results are presented in Part 1. In this part (Part 2A), design in accordance with AS 3600–2009 is described including for robustness. All methods of analysis permitted for low-ductility steel are examined. In Part 2B, design and test strengths are compared to examine the structural safety of typical elevated-concrete slabs incorporating Class L mesh. The analysis methods examined are: static analysis of determinate members; linear elastic analysis ignoring moment redistribution; finite-element analysis; non-linear frame analysis with non-linear geometric effects, including compressive membrane action; and simplified flexural analysis of continuous one-way slabs, or two-way slabs supported on four sides. No safety issues arose. For example, for eight statically determinate SSOW-series slabs with unrestrained ends, the ratio of the ultimate test load divided by ultimate design live load (with a load factor of 1.5) varied from 2.11 to 2.54 for the same slabs. Much higher values (max. 6.52) were obtained for doubly-reinforced slabs with fully restrained ends or edges. It is apparent that Class L main bars near the compressive face actually yielded in tension and added up to 20% to the moment capacity of critical cross-sections.

KEYWORDS
Elevated Slab; Low-Ductility Mesh; Structural Design; Structural Safety.

STRENGTH DESIGN OF SLAB TEST SPECIMENS IN ACCORDANCE WITH LATEST CONCRETE STRUCTURES STANDARD AS 3600–2009

Introduction

The one-way slabs (series SSOW and DSOW) are referred to herein as "unrestrained" or "restrained". By "unrestrained" it is meant that during a test their ends were free to move longitudinally or axially in or out, and they were also free to rotate about the orthogonal axis through the centre of each set of end roller bearings with a fixed axle. In contrast, "restrained" one-way slabs had their ends effectively fully built-in, with all translational and rotational movements severely restricted. Similarly, the two-way slab (series TW) had all four of its edges "restrained", or effectively fully built-in. Therefore, it was designed with four continuous edges, while the potentially more conservative design of the slab with four discontinuous edges was also considered with the more conservative designer in mind. The slabs were principally designed for ultimate strength (in accordance with Clause 2.2 of AS 3600–2009 (SA, 2009)) rather than for serviceability limit states, e.g. vertical deflection or cracking. However, each slab was practically proportioned with a span-to-depth ratio representative of normal construction, and also reinforced in all flexural tensile regions. Also, on account of the realistic span-to-depth ratios and the test loading patterns or configurations, failure due to flexure rather than shear was anticipated.

Each slab test specimen was designed for strength in accordance with Clause 2.2.2 *Simplified check procedure for use with linear elastic methods of analysis, with simplified analysis methods and for statically determinate structures* of AS 3600–2009. Specifically, the strength calculations were performed in accordance with all of the following items as appropriate.

(i) Design capacity, R_d (= ϕR_u) equalled or exceeded design action effect, E_d, for all potentially critical cross-sections, such that: for bending, $\phi M_{uo} \geq M^*$; and for vertical shear, $\phi V_{uc} \geq V^*$.

(ii) Design capacity, R_d, was obtained using the appropriate value of ϕ given in Table 2.2.2 of AS 3600–2009 for members with Class L reinforcement, viz.: (a) ϕ as per Eq. 2 in Part 1 of this paper, for bending without axial force; (b) $\phi = 0.6 + (\phi$ as per Eq. 2 in Part 1 – 0.6)(1 – N_u/N_{ub}) for bending with axial compression, and $N_u < N_{ub}$ (applicable to the restrained slabs exhibiting compressive membrane action) within the limits $0.6 \leq \phi \leq 0.64$; and (c) $\phi = 0.7$ for vertical shear.

(iii) Ultimate strength, R_u, was determined in accordance with the relevant sections of AS 3600–2009 using characteristic material strengths, viz.: (a) for bending M_{uo} was calculated as per Part 1; and (b) for vertical shear along a continuous support perpendicular to the span V_{uc} was calculated using Clause 8.2.7 *Shear strength of a beam excluding shear reinforcement* with all reinforcing steel included, i.e. no strength penalty was applied to Class L.

(iv) The design action effect, E_d, was determined for the normal combination of factored actions $1.2G + 1.5Q$ (dead load G and live load Q are both load per unit length) or $1.2g + 1.5q$ (dead load g and live load q are both pressures), using at least one of the methods of analysis described in Part 1 of this paper, as applicable for each slab tested, viz.:

(a) static analysis: design action effects for all of the unrestrained SSOW slabs were determined using statics;
(b) linear elastic analysis: this method of analysis was applied to restrained slab SSOW-ST1, all four redundant, continuous two-span DSOW slabs, and also redundant one-way strips in the continuous two-way slab TW-ST1;
(c) finite-element analysis: two-way slab TW-ST1 was analysed when it was loaded by four patches to failure, assuming the concrete was uncracked and behaved linear elastically;
(d) non-linear frame analysis: the restrained SSOW and DSOW slabs (SSOW-ST1, DSOW-ST1 and DSOW-ST2) were also designed taking into account compressive membrane action assuming the negative and positive plastic hinges formed simultaneously (Park and Gamble, 2000); and
(e) simplified flexural analysis in accordance Clause 6.10.3 *Simplified method for reinforced two-way slabs supported on four sides*: it was applied to slab TW-ST1 for the initial water proof-loading stage, applicable for slabs with Class L mesh – see Patrick, et al. (2005) for derivation of rules. (Linear elastic analysis was used in preference to Clause 6.10.2 for the one-way slabs with four line-loads applied to simulate uniformly-distributed design loads.)

Design Moment Capacity (ϕM_{uo}) & Shear Capacity (ϕV_{uc}) of Critical Sections.
Assuming singly-reinforced cross-sections (conservatively ignoring any main steel near the compressive face), nominal dimensions and design material properties, the values of design moment capacity in pure bending, ϕM_{uo}, calculated ignoring compressive membrane action, and design shear capacity, ϕV_{uc}, of the potentially critical cross-sections in the positive (sagging) and negative (hogging) moment regions of all the test specimens are summarised in Table 1 per metre width of slab.

STRENGTH DESIGN OF SSOW SLABS

Unrestrained SSOW Slabs (SSOW-ST2 to SSOW-ST8 & SSOW-TRIAL)

Simply-supported, statically determinate slabs SSOW-ST2, SSOW-ST4 to ST8 and SSOW-TRIAL were designed according to the test set-up geometry to support dead load G=0.11x24=2.64 kN/m and a service design line-load, P_s^*, at 4 points along the span of 2290 mm with 205 mm cantilevers. With the maximum positive bending moment at mid-span assigned equal to $\phi M_{uo.x}^+$, it can be shown that $4P_s^*=2.279(\phi M_{uo.x}^+ - 2.01)$ kN/m. Values of the total applied live load, $4P_s^*$, are given in Table 2 using the values of $\phi M_{uo.x}^+$ in Table 1. The corresponding value of ultimate design applied load $4P^* = 1.5 \times 4P_s^* = 6P_s^*$ is also given in Table 2 for each slab. Similar values are given in Table 2 for slab SSOW-ST3, which had a single line-load applied at mid-span. Assuming the design of each slab was governed by bending rather than shear, the corresponding values of maximum design vertical shear force, V_{max}^*, are given in Table 2. They are all less than the corresponding value of $\phi V_{uc.x}^+$ in Table 1, thus confirming the validity of this latter assumption.

Table 1. Design moment capacity, ϕM_{uo} in kNm/m & design shear capacity, ϕV_{uc} in kN/m.

Test Series	Test Specimen No.	$\phi M_{uo.x}^+$ ($\phi V_{uc.x}^+$)	$\phi M_{uo.x}^-$ ($\phi V_{uc.x}^-$)	$\phi M_{uo.y}^+$ ($\phi V_{uc.y}^+$)	$\phi M_{uo.y}^-$ ($\phi V_{uc.y}^-$)
SSOW	SSOW–ST1	9.29 (50.74)	7.71 (47.63)	-	-
	SSOW–ST2	7.71 (47.63)	-	-	-
	SSOW–ST3	7.71(47.63)	-	-	-
	SSOW–ST4	9.29 (50.74)	-	-	-
	SSOW–ST5	14.93 (60.36)	-	-	-
	SSOW–ST6	15.30 (63.48)	-	-	-
	SSOW–ST7	13.50 (58.42)	-	-	-
	SSOW–ST8	14.04 (61.66)	-	-	-
	SSOW–TRIAL	9.29 (50.74)	-	-	-
DSOW	DSOW–ST1	7.71 (47.63)	9.29 (50.74)	-	-
	DSOW–ST2	7.71 (47.63)	9.29 (50.74)	-	-
	DSOW–ST3	7.71 (47.63)	9.29 (50.74)	-	-
	DSOW–ST4	7.71 (47.63)	9.29 (50.74)	-	-
TW	TW–ST1	9.29 (50.74)	7.71 (47.63)	8.21 (48.99)	6.91 (46.15)

Table 2. Strength design of unrestrained SSOW slabs (SSOW-ST2 to ST8 & SSOW-TRIAL).

Test Series	Test Specimen No.	SSOW-ST2, SSOW-ST4 to SSOW-ST8 and SSOW-TRIAL			SSOW-ST3		
		$4P_s^*$ (kN/m)	$4P^* = 6P_s^*$ (kN/m)	V_{max}^* (kN/m)	P_s^* (kN/m)	$P^* = 1.5P_s^*$ (kN/m)	V_{max}^* (kN/m)
SSOW	SSOW–ST2	12.99	19.49	13.38	-	-	-
	SSOW–ST3	-	-	-	6.64	9.96	8.61
	SSOW–ST4	16.59	24.89	16.08	-	-	-
	SSOW–ST5	29.45	44.18	25.72	-	-	-
	SSOW–ST6	30.29	45.44	26.35	-	-	-
	SSOW–ST7	26.19	39.29	23.28	-	-	-
	SSOW–ST8	27.42	41.13	24.20	-	-	-
	SSOW–TRIAL	16.59	24.89	16.08	-	-	-

Restrained SSOW Slab (SSOW-ST1)

Statically indeterminate slab SSOW-ST1 was designed three ways, with all the results shown in Table 3, viz.: (i) linear elastic analysis in accordance with Clause 6.2 of AS 3600–2009; (ii) plastic analysis ignoring compressive membrane action; and finally (iii) plastic analysis including compressive membrane action (Park and Gamble, 2000).

In accordance with Clause 6.7 *Plastic Methods of Analysis* of AS 3600–2009, Ductility Class N bars shall be used throughout for flexural reinforcement when plastic collapse analysis is used to design one-way slabs. Nevertheless, the design values in Table 3 for this case have been computed assuming fully built-in ends, 4-point loading, simplistically ignoring the small slab self-weight, an effective span of 2290 mm, and fully-developed negative plastic hinges at both slab ends with $\phi M^{-}_{uo.x} = 7.71\,\text{kNm}$ and positive plastic hinges at both inner loading points with $\phi M^{+}_{uo.x} = 9.29\,\text{kNm}$. It follows that the design loads are about twice those based on linear elastic analysis. Even so, it can be seen that vertical shear strength still does not govern.

Table 3. Strength design of restrained SSOW slab (SSOW-ST1).

Strength Design Method	$4P^{*}_{s}$ **(kN/m)**	$4P^{*} = 6P^{*}_{s}$ **(kN/m)**	V^{*}_{max} **(kN/m)**
Linear elastic	21.00	31.50	19.38
Plastic / compressive membrane action ignored	40.10	60.15	30.08
Plastic / compressive membrane action	63.5	95.25	47.63 (shear controls)

Slab SSOW-ST1 was designed taking into account compressive membrane action as a consequence of non-linear geometric effects, in accordance with Clause 6.5 *Non-Linear Frame Analysis*. It is stated in Clause 6.5.3 *Non-linear geometric effects* that "*Equilibrium of the structure in the deformed condition shall be considered whenever joint displacements or lateral deflections within the length of the members significantly affect the action effects or overall structural behaviour*". Park and Gamble (2000) use plastic theory to derive the load-deflection curve of a restrained one-way slab strip (like test specimen SSOW-ST1) while approaching and after initial ultimate (first peak) load, when compressive membrane forces are present. Their equations for calculating the increased ultimate moment capacity of the negative or positive plastic hinges under the action of axial compressive force were modified for design purposes to conform to AS 3600–2009. A single positive plastic hinge was assumed to form at the mid-span of the test slab (as observed during the test on slab SSOW-ST1). Their formulation includes the lateral stiffness of the end supports, estimated to be 250 kN/mm with a steel tubular tension tie fixed in between the long sides of the tubular ringbeam. Assuming the negative and positive plastic hinges to simultaneously develop their full bending strength, and ultimate strength under compressive membrane action to be reached at a mid-span deflection of *D*/2 or 55 mm, resulted in $4P^{*}_{s} = 80\,\text{kN/m}$. Ignoring dead load, the ultimate design vertical shear force acting at each end support equals $V^{*}_{max} = 1.5\left(2P^{*}_{s}\right) = 3P^{*}_{s} = 60\,\text{kN/m}$. Ignoring the beneficial effects of resultant axial compressive force, from Table 1, $\phi V^{-}_{uc.x} = 47.63\,\text{kN/m}$, so $V^{*}_{max} > \phi V^{+}_{uc.x}$, whereby vertical shear governs the strength design. It follows that $3P^{*}_{s} = 47.63\,\text{kN/m}$, whereby $4P^{*}_{s} = 63.5\,\text{kN/m}$ instead of 80 kN/m. Finally, it follows that $4P^{*} = 1.5 \times 63.5 = 95.25\,\text{kN/m}$, as given in the last row of Table 3.

STRENGTH DESIGN OF DSOW SLABS

Support Settlement

Two of the DSOW slabs (DSOW-ST2 and DSOW-ST4) were lifted up by 5 mm at their centre support prior to being tested to failure, i.e. both end supports effectively settled by this amount relative to the centre support. If a designer elected to account for the effects of this relative support settlement using elastic analysis, the design load-carrying capacity of the slab could be very significantly reduced. For example, for restrained slab DSOW-ST2, linear-elastic analysis assuming fully built-in ends and ignoring flexural cracking gives rise to maximum service positive and negative bending moments, $M^{*+}_{s.max}$ = 17.47 kNm/m and $M^{*-}_{s.max}$ = -19.80 kNm/m. They both well exceed the design ultimate moment capacity of their respective section, viz. ϕM^{+}_{uo} (= 7.71 kNm/m) and ϕM^{-}_{uo} (= 9.29 kNm/m) as given in Table 1. Similarly, for unrestrained slab DSOW-ST4, $M^{*-}_{s.max}$ = -11. 03 kNm/m > ϕM^{-}_{uo} too.

It follows that due to support settlement alone, both slabs would have been deemed to have failed in bending, unable to support any design superimposed dead or live loads. As a result, neither of these slabs was designed for relative support settlement, which is in accordance with AS 3600–2009. Instead, it was assumed that the slabs, despite incorporating Class L mesh, could accommodate the potentially large amount of moment redistribution. Therefore, both restrained slabs (DSOW-ST1 and DSOW-ST2) were designed exactly the same way as each other, as were both unrestrained slabs (DSOW-ST3 and DSOW-ST4), by completely ignoring support settlement as per AS 3600.

Restrained DSOW Slabs (DSOW-ST1 & DSOW-ST2)

Slabs DSOW-ST1 and DSOW-ST2 were designed as being nominally identical to each other, using linear elastic analysis in accordance with Clause 6.2 *Linear Elastic Analysis* of AS 3600–2009. Moment redistribution was not applied when determining the design ultimate bending moments. Also, both spans were assumed to be equally loaded, thus ignoring pattern-loading effects, as was the case in all the DSOW slab tests, i.e. both spans were loaded equally at all times. It can be shown that $4P_s^* = \min.\left[7.300\left(\phi M_{uo}^+ - 0.70\right), 3.333\left(\phi M_{uo}^- - 1.39\right)\right]$, which results in the "Linear elastic" solution shown in Table 4, noting that vertical shear obviously does not govern the design.

Table 4. Strength design of restrained DSOW slabs (DSOW-ST1 & DSOW-ST2).

Strength Design Method	$4P_s^*$ **(kN/m)**	$4P^* = 6P_s^*$ **(kN/m)**	V_{max}^* **(kN/m)**
Linear elastic	26.33	39.50	23.38
Plastic / compressive membrane action ignored	38.91	58.37	29.18
Plastic / compressive membrane action	67.7	101.50	50.74 (shear controls)

The plastic design solution calculated ignoring compressive membrane action is shown next in Table 4. Negative plastic hinges (ϕM_{uo}^- = 9.29 kNm/m) were assumed to be present at the centre support and at both end supports, simultaneously with positive plastic hinges (ϕM_{uo}^+ = 7.71 kNm/m) at the second loading points in from each fixed end. Again, vertical shear did not govern. The last solution in the table was determined in a similar fashion to that described above for restrained slab SSOW-ST1, and like for this slab, much greater strength is displayed leading to shear governing.

Unrestrained DSOW Slabs (DSOW-ST3 & DSOW-ST4)

In a similar way to the restrained DSOW slabs, unrestrained slabs DSOW-ST3 and DSOW-ST4 were designed elastically ignoring moment redistribution. In this case it can be shown that $4P_s^* = \min.\left[3.738\left(\phi M_{uo}^+ - 1.15\right), 2.222\left(\phi M_{uo}^- - 2.06\right)\right]$ with design moment capacities as per Table 1, resulting in the "Linear elastic" solution shown in Table 5, with vertical shear obviously not governing the design. The plastic solution in Table 5 was derived ignoring compressive membrane action, with a negative plastic hinge (ϕM_{uo}^- = 9.29 kNm/m) at the centre support and simultaneously positive plastic hinges (ϕM_{uo}^+ = 7.71 kNm/m) at the second loading points in from each pinned end.

Table 5. Strength design of unrestrained DSOW slabs (DSOW-ST3 & DSOW-ST4).

Strength Design Method	$4P_s^*$ **(kN/m)**	$4P^* = 6P_s^*$ **(kN/m)**	V_{max}^* **(kN/m)**
Linear elastic	16.07	24.11	19.74
Plastic / compressive membrane action ignored	25.68	38.52	23.30

STRENGTH DESIGN OF TW SLAB (TW-ST1)

Simplified Flexural Analysis based on Elastic Finite-Element Analysis

Patrick et al. (2005) used elastic finite-element analysis without moment redistribution to derive the values of the two-way bending moment coefficients α_x, α_y, β_x and β_y given in Table 6.10.3.2(B) of Clause 6.10.3 *Simplified method for reinforced two-way slabs supported on four sides* in AS 3600–2009. This analysis was specifically performed assuming uniformly-loaded, prismatic solid reinforced-concrete slabs incorporating Class L (mesh) reinforcement, with slab corners tied down to rigid supports to resist uplift. These coefficients are used in the following formulae to calculate the maximum design positive and negative bending moments in the (short) primary (x) and (long) secondary (y) orthogonal directions: $M_x^{*+} = \beta_x F_d L_{eff.x}^2$; $M_y^{*+} = \beta_y F_d L_{eff.x}^2$; $M_x^{*-} = \alpha_x M_x^{*+}$; and $M_y^{*-} = \alpha_y M_y^{*+}$; where F_d is the uniformly-distributed design load per unit area factored for strength, and for the design of slab test specimen TW-ST1, F_d =1.2g+1.5q; and $L_{eff.x}$=shorter effective span of the rectangular slab supported on all four sides, i.e. in primary spanning direction, x, while in accordance with Clause 1.7 *Notation* of AS 3600–2009, $L_{eff.x}$=min. ($L_{n.x}$+D, L_x) = min. (2140+110, 2140+150)=2250 mm, and $L_{eff.y}$=min. ($L_{n.y}$+D, L_y) = min. (4440+110, 4440+150)=4550 mm. Design edge shear forces were determined using Clause 6.10.3.4 *Load allocation*.

Design assuming Four Edges Continuous under Uniform Water Pressure

From Table 6.10.3.2(B), for Case 1 (four edges continuous), i.e. all edges effectively fully built-in to prevent any translation or rotation from occurring: $L_{eff.y}/L_{eff.x}$=4550/2250=2.02=2.0 $\Rightarrow$ α_x=2.00, α_y=2.69, β_x=0.042 and β_y=0.020, whereby it can be shown that the controlling design moment capacity is along the long support, i.e. $\phi M_{uo.x}^{-} = 7.71 = 2.00 \times 0.042 \times 2.25^2 F_d \quad \Rightarrow F_d = 18.13\ \text{kPa}$. Therefore, design live load q =(F_d -1.2×g)/1.5=(18.13-1.2×2.64)/1.5=9.98 kPa, or 10.0 kPa. According to Clause 6.10.3.4 of AS 3600–2009, the corresponding maximum design end shear force, V_{max}^{*}, can simply be computed as $V_{max}^{*} = F_d L_{n.x} / 2 = 18.13 \times 2.14 / 2 = 19.4\ \text{kN/m}$, whereby $V_{max}^{*} < \phi V_{uc.x}^{-}$, with $\phi V_{uc.x}^{-} = 47.63\ \text{kN/m}$ from Table 1, confirming that bending rather than shear governed the design.

Design assuming Four Edges Discontinuous under Uniform Water Pressure

If a designer doubted that the connection of the test slab to the tubular steel ringbeam could provide a level of rotational restraint equivalent to full continuity of the slab over an interior support, then they could assume that the edges were discontinuous. The slab design was also examined for this extreme case, and not shown to be affected, i.e. from Table 6.10.3.2(B), for Case 9 (four edges discontinuous): $L_{eff.y}/L_{eff.x}$=4550/2250=2.02=2.0 $\Rightarrow$ α_x=0, α_y=0, β_x=0.100 and β_y=0.049, whereby it can be shown that positive bending in the primary short-spanning direction governs, i.e. $\phi M_{uo.x}^{+} = 9.29 = 0.100 \times 2.25^2 F_d \quad \Rightarrow F_d = 18.35\ \text{kPa}$, effectively the same as determined above assuming continuous edges.

Design for Bending and Shear (including Punching Shear) under 4 Patch Loads

The final loading configuration used to test slab TW-ST1 to failure comprised four equal patch loads, each applied to the top surface of the slab over a 200 mm wide square area. Various ways of designing the slab for this loading configuration are presented under the following subheadings, with results summarised in Table 6. It will be shown first that local punching shear did not govern the design in any case, i.e. no value of 4P* in Table 6 exceeds 4×182.4=729.6 kN. For brevity, the calculations for diagonal shear design will not be presented, as this mode was not critical (see V_{max}^{*}).

Table 6. Strength design of restrained TW slab under 4 patch loads (TW-ST1).

Strength Design Method	$4P_s^*$ (kN)	$4P^* = 6P_s^*$ (kN)	V_{max}^* (kN/m)
Linear elastic / one-way strip	45.52	68.28	15.57
Plastic / one-way strip ignoring compressive membrane action	72.53	108.80	19.43
Plastic / one-way strip including compressive membrane action	not calculated	not calculated	not calculated
Finite-element linear elastic / two-way flexural action	51.56	77.34	15.94
Yield-line analysis / two-way flexural action	153.47	230.20	not calculated

Design for Punching Shear. In accordance with Clause 9.2.3 *Ultimate shear strength where M_v^* is zero*, in the absence of a shear head and any prestressing (and also ignoring any longitudinal restraint effects): $V_{uo} = ud_{om}f_{cv}$ where V_{uo}=ultimate (nominal) shear strength with bending moment transferred to a support, $M_v^* = 0$; d_{om}= mean value of d_o, around the critical shear perimeter = mean value of d_L^+= (85.3+75.8)/2=80.5 mm; u=4×(200+d_{om})=1122 mm; and concrete shear strength $f_{cv} = 0.17(1+2/\beta_h)\sqrt{f_c'}$ with β_h=ratio of longest overall dimension of the (patch) effective loaded area to the overall dimension measured perpendicularly. Therefore, for a square patch, β_h=1, while f_c'=32 MPa for test specimen TW-ST1, and therefore f_{cv}=2.885 MPa. Noting that ϕ equals 0.7, the design shear strength of the slab around each loading patch equals: $\phi V_{uo} = \phi u d_{om} f_{cv} = 0.7 \times 1122 \times 80.5 \times 2.885 / 1000 = 182.4$ kN.

Linear Elastic, One-Way Strip Design. A simple approach is to design the slab in accordance with Clause 9.6 *Moment Resisting Width for One-way Slabs supporting Concentrated Loads* of AS 3600–2009, whereby the effective width of a one-way strip, b_{ef}, is given by $b_{ef} = \text{load width} + 2.4a(1 - a/L_n)$ where load (patch) width = 200 mm; a=perpendicular distance from the (inside edge of the) nearer support to the section under consideration, which is the critical section in bending, i.e. (between) the centre of each pair of patch loads on a strip, whereby a=(L_n-600)/2=(2140-600)/2=770 mm. It follows that b_{ef}=200+2.4×770×(1-770/2140)=1383 mm, or rounding up b_{ef}=1400 mm. Similar to the elastic design of restrained one-way slabs SSOW-ST1, DSOW-ST1 and DSOW–ST2, assuming a prismatic section with built-in ends and 2-point loading ($2P_s^*$) per 1.4 metre wide strip of slab, elastic analysis gives the result of $2P_s^* = \min.\left[4.405\left(1.4\phi M_{uo}^+ - 0.937\right), 2.551\left(1.4\phi M_{uo}^- - 1.873\right)\right]$, and therefore $4P_s^* = 45.52$ kN.

Plastic, One-Way Strip Design ignoring Compressive Membrane Action. Assuming: a prismatic section throughout the length of each 1.4 m wide strip; fully built-in ends; 2-point loading per strip, i.e. $2P^*$ per span; slab self-weight can be ignored; each span has an effective span of 2250 mm; negative plastic hinges develop at both end supports with $\phi M_{uo.x}^- = 7.71$ kNm/m; and positive plastic hinges at both patch loads with $\phi M_{uo.x}^+ = 9.29$ kNm/m; then simple plastic analysis ignoring axial forces developed due to membrane action gives $4P^* = 1.5 \times 4P_s^* = 6P_s^* = 108.80$ kN. The plastic design method accounting for compressive membrane action described previously in the paper could have similarly been applied to the design of the strip to obtain a less conservative design value, but for brevity this was omitted from the study – see yield-line analysis below, instead.

Finite-Element Linear Elastic Analysis of Two-Way Flexural Action. Linear elastic stress analysis was undertaken in accordance with Clause 6.4 of AS 3600–2009 using finite-element analysis to account for two-way flexural action, modelling the slab as a flat plate assuming all four edges fully built-in, with effective spans $L_{eff.x}$=2250 mm and $L_{eff.y}$=4550 mm. In-plane restraint effects were conservatively eliminated by setting Poisson's ratio for the concrete to zero, and also suppressing membrane action by assigning pure bending action. The concrete was modelled as uncracked in accordance with Clause 2.2.3(a) of AS 3600–2009. The design values calculated using this method are shown in the second last row of Table 6.

Yield-Line Analysis of Two-Way Flexural Action. Plastic analysis using yield-line theory was undertaken in accordance with Clause 6.7.3.2 *Yield line method for slabs* of AS 3600–2009 to model two-way flexural action, conservatively ignoring compressive membrane action. Assuming: four built-in edges; effective spans $L_{eff.x}$=2250 mm and $L_{eff.y}$=4550 mm; negative yield lines ($\phi M_{uo}^{-} = 6.91\,\text{kNm/m}$ –Table 1, conservatively the lesser value in both orthogonal directions) formed around the full perimeter of the slab (noting that in the actual test to failure, these yield lines were rounded in all four slab corners – Fig. 7(c) of Part 1); positive yield lines ($\phi M_{uo}^{+} = 8.21\,\text{kNm/m}$ – the lesser value in both orthogonal directions) formed parallel to the slab edges between adjacent patch loads, radiating from each patch load to the nearest corner; it can be shown that $4P^{*} = 2\left(\phi M_{uo}^{-} + \phi M_{uo}^{+}\right)(L_{eff.x}/1.32 + L_{eff.y}/0.77) = 2\left(6.91 + 8.21\right)(2.25/1.32 + 4.55/0.77) = 230.2\ \text{kN}$.

CONTINUED IN PART 2B.