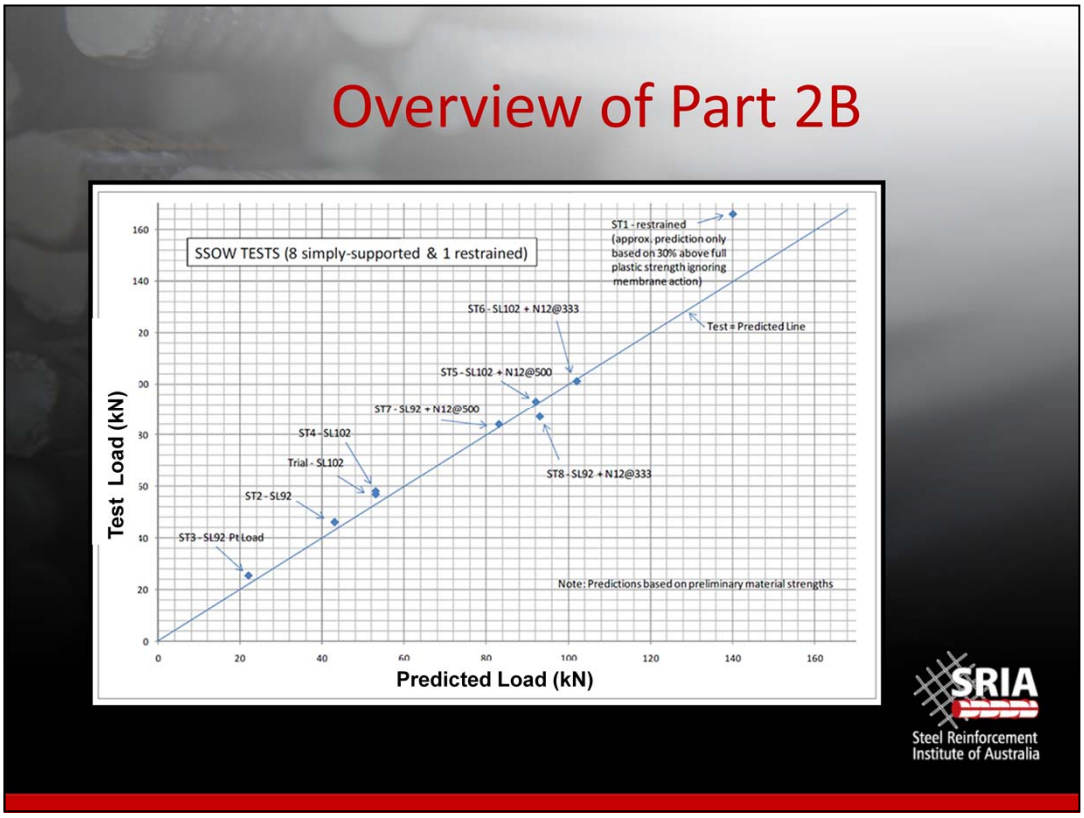
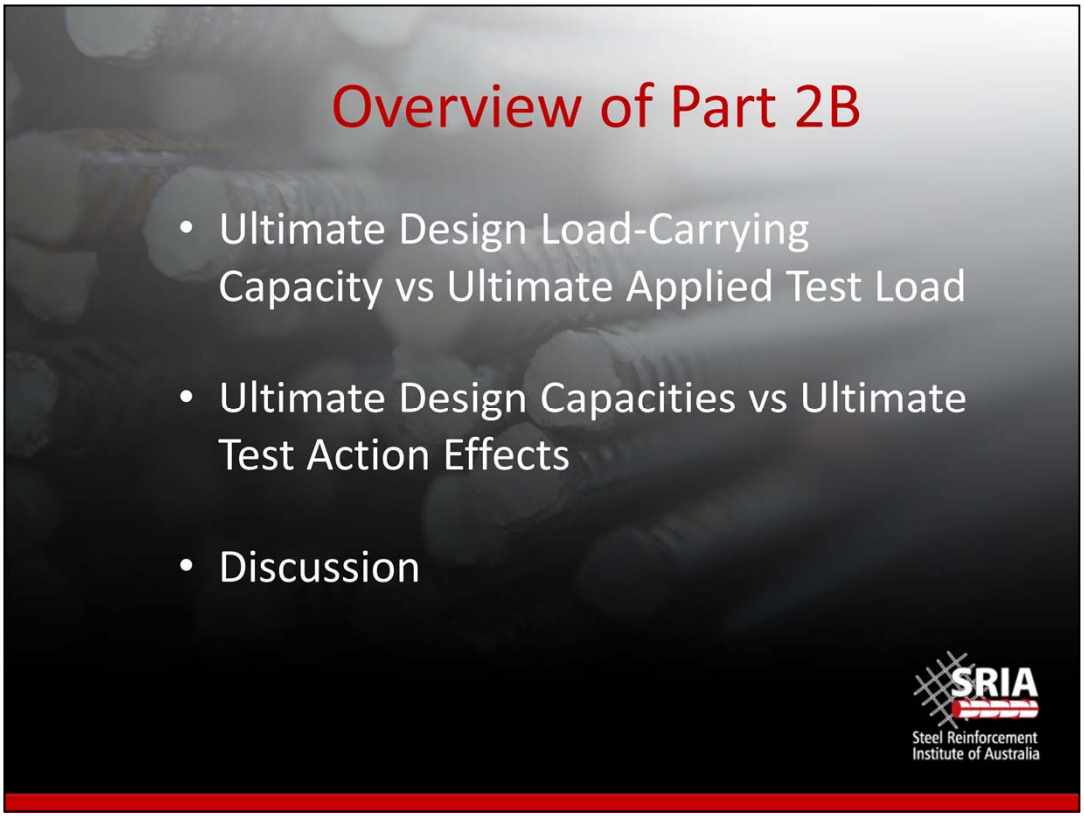
In this Part 2B presentation, the design strengths of the slabs, determined in accordance with AS 3600–2009, will be compared directly with the test results.



It should be noted that the Part 2 paper is principally concerned with comparing the test results with design strengths based on nominal slab geometries and material properties, rather than predicted strengths based on the measured slab geometries and material properties described at the end of the Part 1 presentation.

This graph was prepared for the Peer Review Panel at the time the two-way slab was being tested, and shows that the moment capacities of the 8 simply-supported SSOW slabs could be predicted reasonably accurately using preliminary tensile data for the reinforcing steels. A study of all the test results along these lines has yet to be fully completed, although extensive checks have been made to confirm the accuracy of the Curtin Test Report results.



For all of the 14 tests being studied here, it is possible to compare the Ultimate Design Load-Carrying Capacity calculated for each of the methods of analysis examined, with the Ultimate Applied Test Load recorded in the Curtin Test Report. This comparison will be considered first.

For all the tests with unrestrained ends, which excludes one of the SSOW slabs, two DSOW slabs and of course the TW slab, it is possible to calculate the ultimate test action effects, i.e. maximum bending moments and vertical shear forces at critical and potentially critical cross-sections. Therefore, in 10 of the tests the design cross-section strengths can be compared with the test strengths.

The ratios calculated by these two approaches will then be discussed in more detail by grouping the results according to whether the slabs were determinate or indeterminate, unrestrained or restrained, and the method of analysis employed.

Ultimate Design Load-Carrying Capacity vs Applied Test Load

- Summary for 14 slabs tested:

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) Type	Factored Live Load, 1.5Q	Ultimate Applied Test Load, P_u	*Load Ratio: $P_u/(1.5Q)$*
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	2×2P*	68.28 kN	445.0 kN	6.52
		Plastic – 2 one-way strips	2×2P*	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

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All of the results for this comparison are summarised in this table from the paper.

The ultimate design load is defined as the factored live load, 1.5*Q*. The ultimate applied test load, P_u, is the maximum load applied per span. The values in the last column in the table are P_u divided by 1.5*Q* and are called the Load Ratio.

Values shown shaded correspond to conventional design practice, being based on either linear elastic analysis without moment distribution in the case of the redundant slabs, or simple statics for the simply-supported slabs.

For the slabs with mixed Class L and N bars, all the steel has been treated as Class L in design. The load ratio values will be discussed shortly.

Ultimate Design Capacities vs Ultimate Test Action Effects

- Summary for 10 slabs tested: BENDING

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

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In all of the 10 tests with unrestrained ends, it was possible to accurately calculate the maximum test bending moments, in the positive, and if applicable, negative moment regions.

Summarised in the last two columns of this table are the values of the ratio of the maximum test bending moment to the corresponding design moment capacity. The values of these ratios will be discussed shortly.

Of course, it should be remembered that all of these slabs failed in flexure with bar fracture.

Ultimate Design Capacities vs Ultimate Test Action Effects

- Summary for 12 slabs tested: SHEAR

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)	$\frac{V^{+}_{max}}{\phi V^{+}_{uc,x}}$	$\frac{V^{-}_{max}}{\phi V^{-}_{uc,x}}$
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

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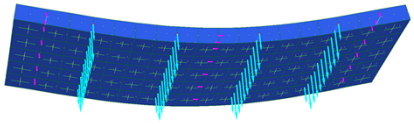
Adding two of the restrained DSOW slabs to this group when assessing shear, the last two columns of this table contain values of the ratio of the maximum test vertical shear force to the corresponding design shear capacity.

Bearing in mind that shear failure did not occur in any of these 12 tests, the low values of this ratio for the 8 simply-supported SSOW slabs, as seen in the second last column, do not imply low shear strengths.

Discussion

- Eight Statically Determinate SSOW Slabs with Unrestrained Ends



SSOW-ST2 to SSOW-ST8
& SSOW-TRIAL

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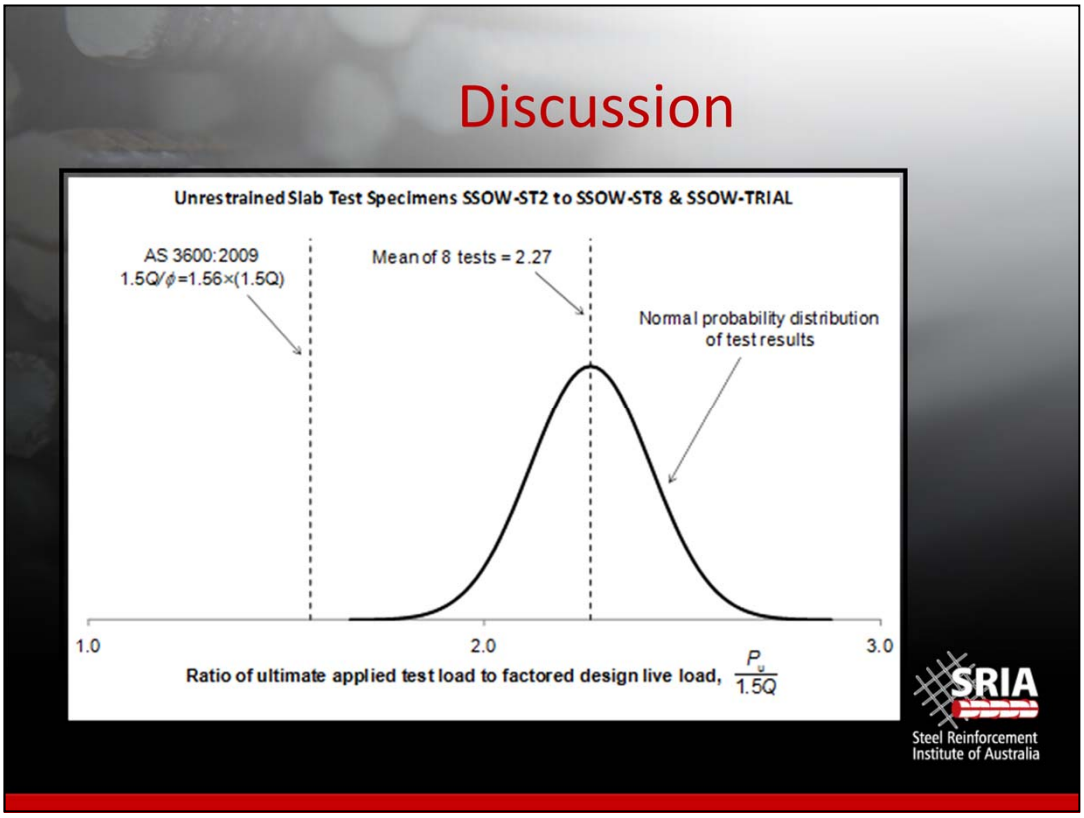
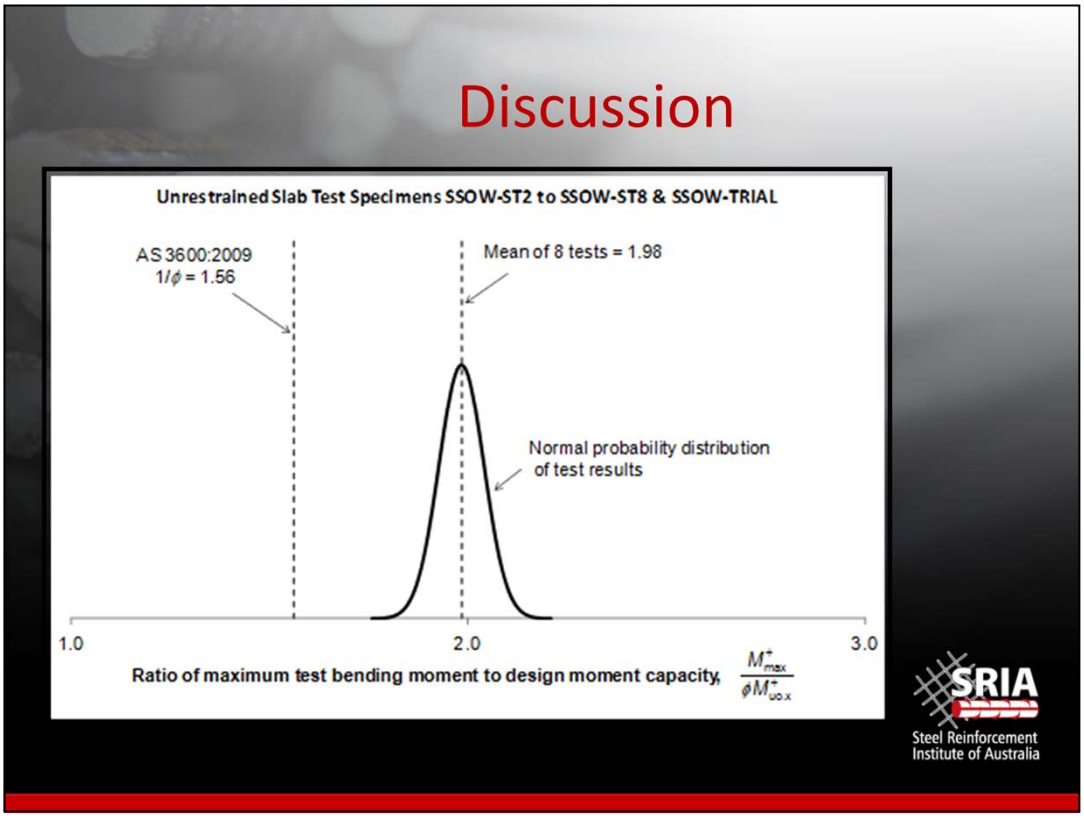
We will consider the ratio values in more detail for the eight statically determinate SSOW slabs with unrestrained ends.



For slabs SSOW-ST2 to ST8 & SSOW-TRIAL supported on rollers, the values of load ratio $P_u/1.5Q$ varied from 2.11 to 2.54, with a mean value of 2.27 which is 46% above a value of $(1/\phi)$ = (1/0.64)=1.56 corresponding to collapse occurring for design in accordance with AS 3600– 2009.

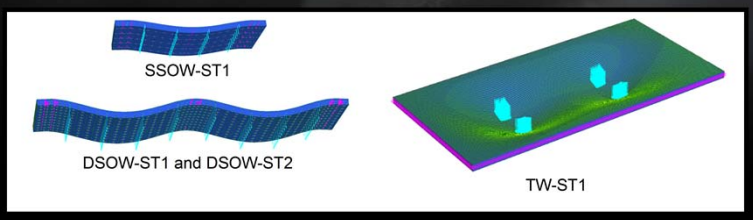
Moreover, note that the bell representing the normal probability distribution of the test results falls well above the 1.56 line.



For the same SSOW slabs, the values of moment ratio varied from 1.93 to 2.08, with a mean of 1.98 which is 27% above a value of 1.56 corresponding to flexural failure.

Discussion

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



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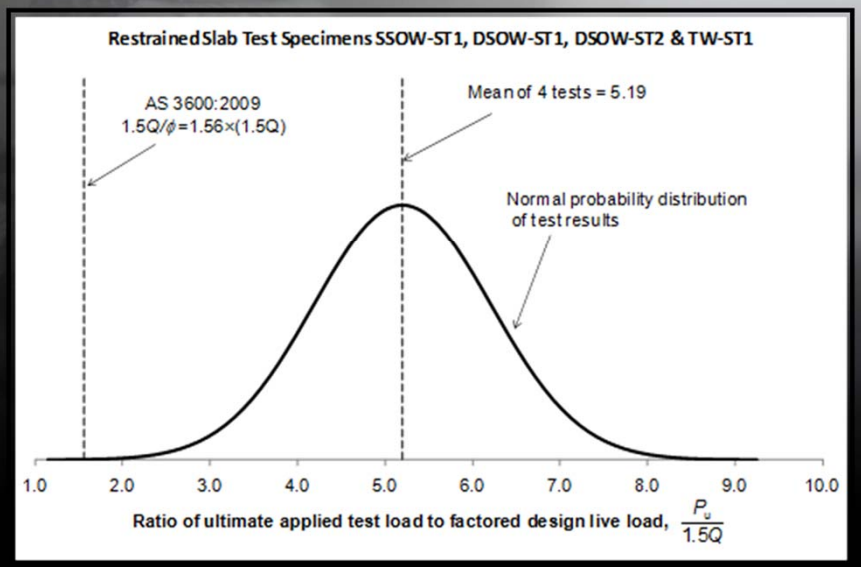
Next we'll consider the load ratio values for the four statically indeterminate slabs with restrained ends or edges when designed using linear-elastic analysis. The internal bending moments and shear forces have not been estimated for these tests, yet.

Discussion



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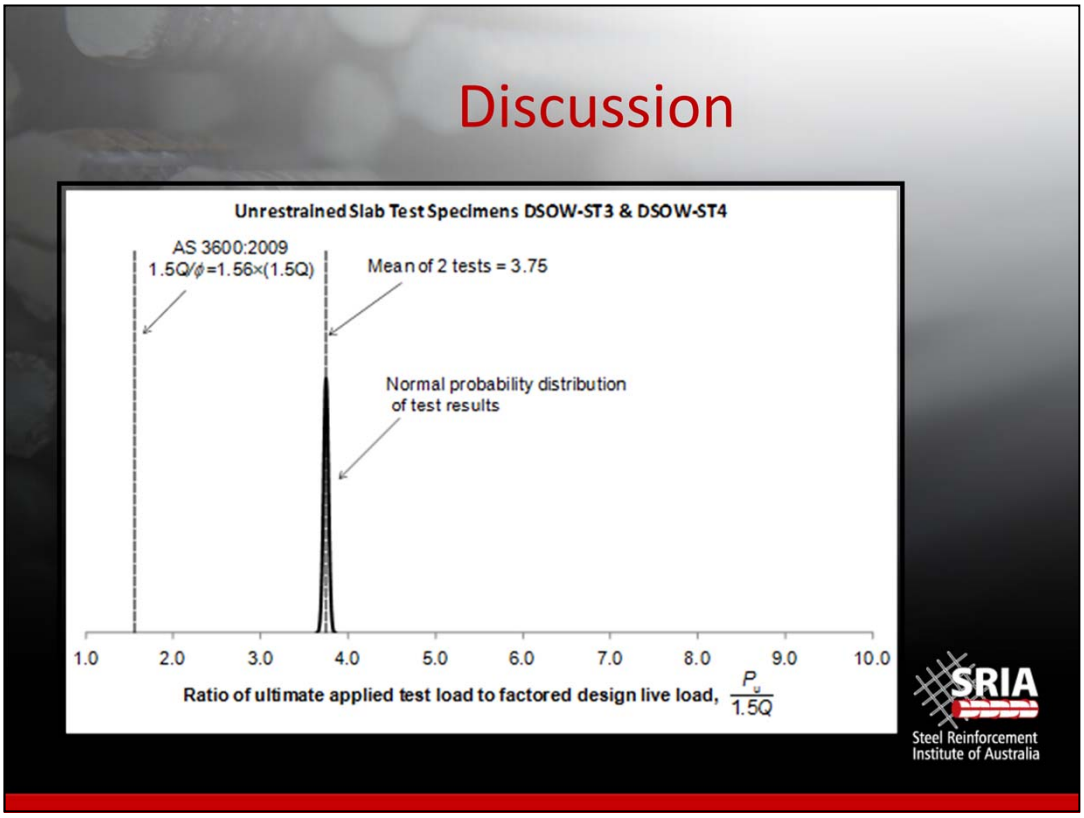
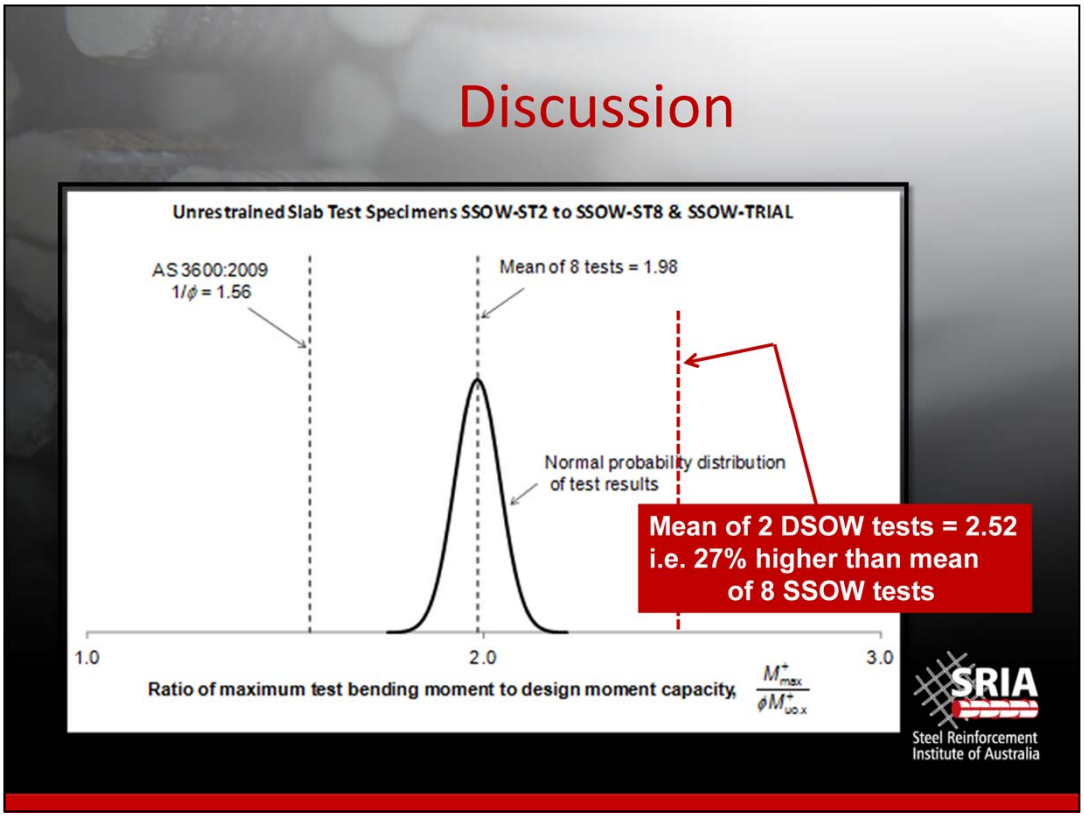
For slab SSOW-ST1, load ratio $P_u/1.5Q$ was a very large value of 5.32. For slabs DSOW-ST1 and DSOW-ST2, it was 4.25 and 4.13, respectively, even with support settlement. For TW-ST1 it was 6.52 or 5.75 with the slab modelled as two one-way strips or as two-way, respectively. The mean value was 5.19, which is 233% above a value of 1.56 corresponding to collapse by flexural failure in design.

Now we'll consider the ratio values for the two statically indeterminate DSOW slabs with unrestrained ends when designed using linear-elastic analysis ignoring moment redistribution.



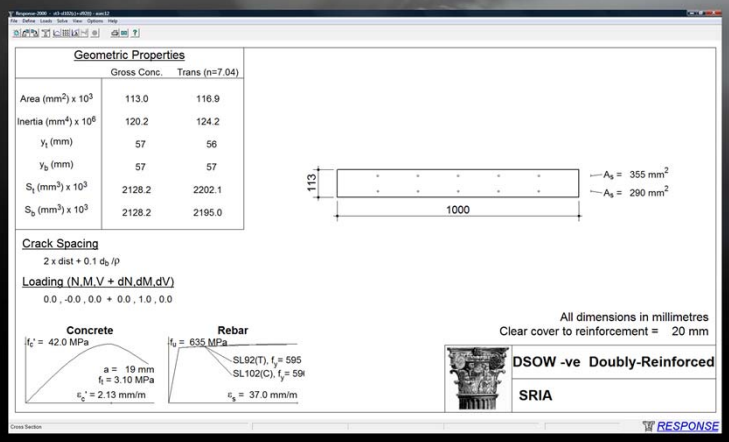
For slabs DSOW-ST3 and DSOW-ST4 (the latter initially subjected to support settlement) similarly high values of the load ratio of 3.77 and 3.73 occurred. The mean value of 3.75 is 140% above 1.56 corresponding to collapse by flexural failure in design.



The values of moment ratio for DSOW-ST3 and ST4 were both about 2.52, which is significantly higher (about 27%) than the mean of 1.98 for the eight unrestrained SSOW slabs shown again here. A detailed investigation was made to explain this apparent anomaly.

Discussion

- Moment-curvature analysis of doubly-reinforced Class L cross-sections



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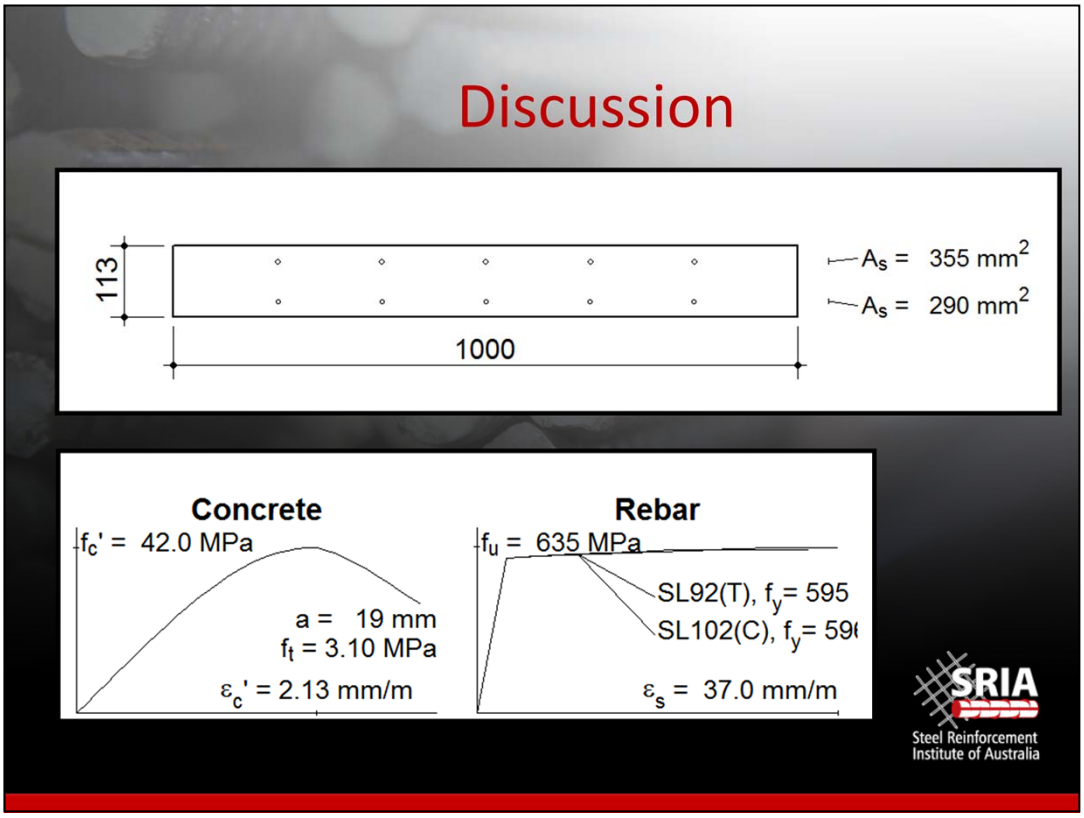
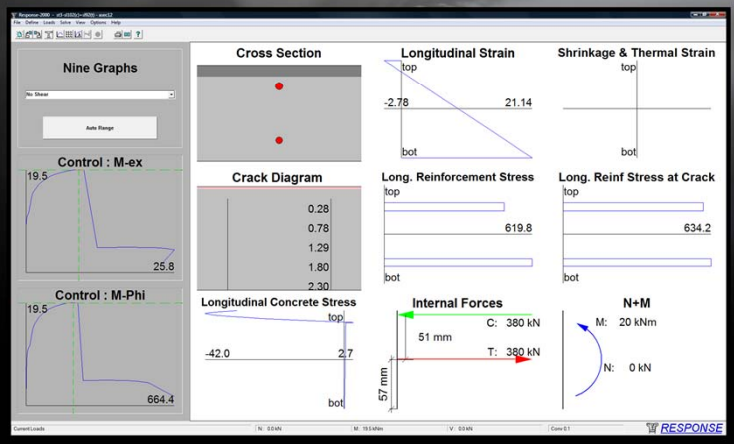
Using the best available estimates for the geometric and material properties of the test slabs, moment-curvature analysis was used to study the behaviour of the doubly-reinforced Class L cross-sections in negative or positive bending. This slide shows a summary-input screen of the software used.



This figure shows that the average overall depth of the slabs was 113 mm, and also the stress-strain curves assumed for the concrete and bottom SL92 and top SL102 meshes.

Discussion

- Moment-curvature analysis of doubly-reinforced Class L cross-sections



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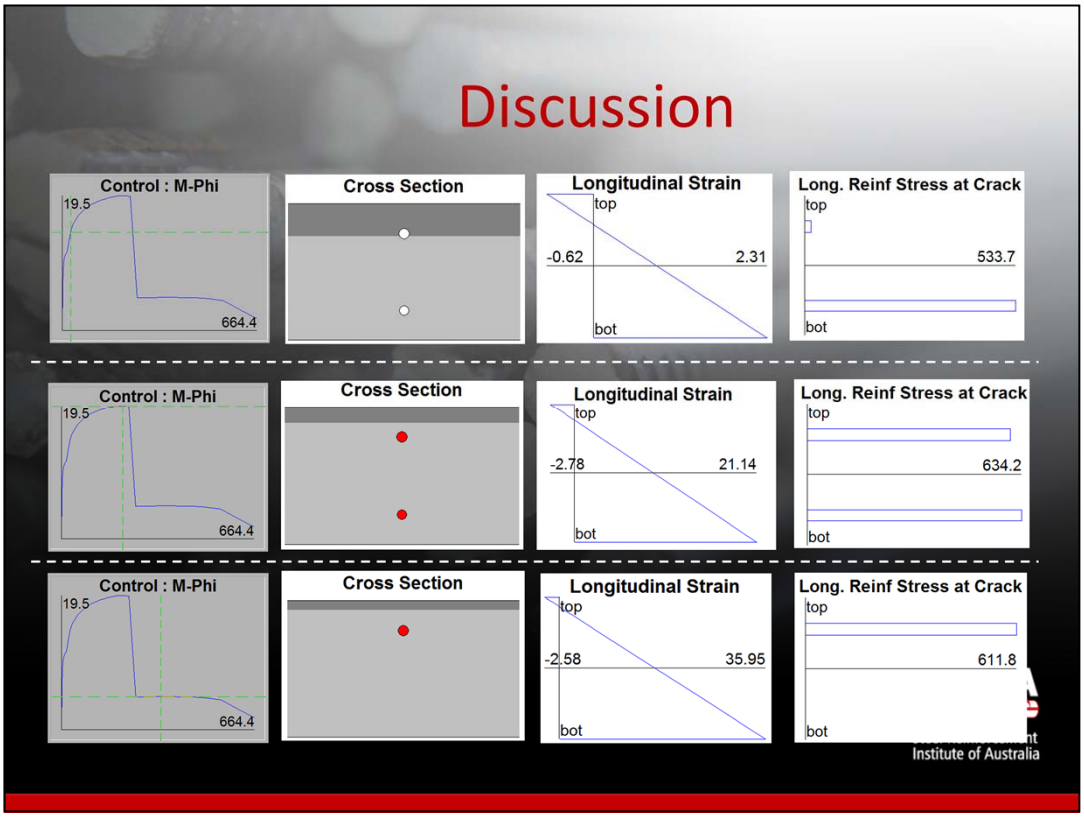
This slide shows a summary output screen of the software used.

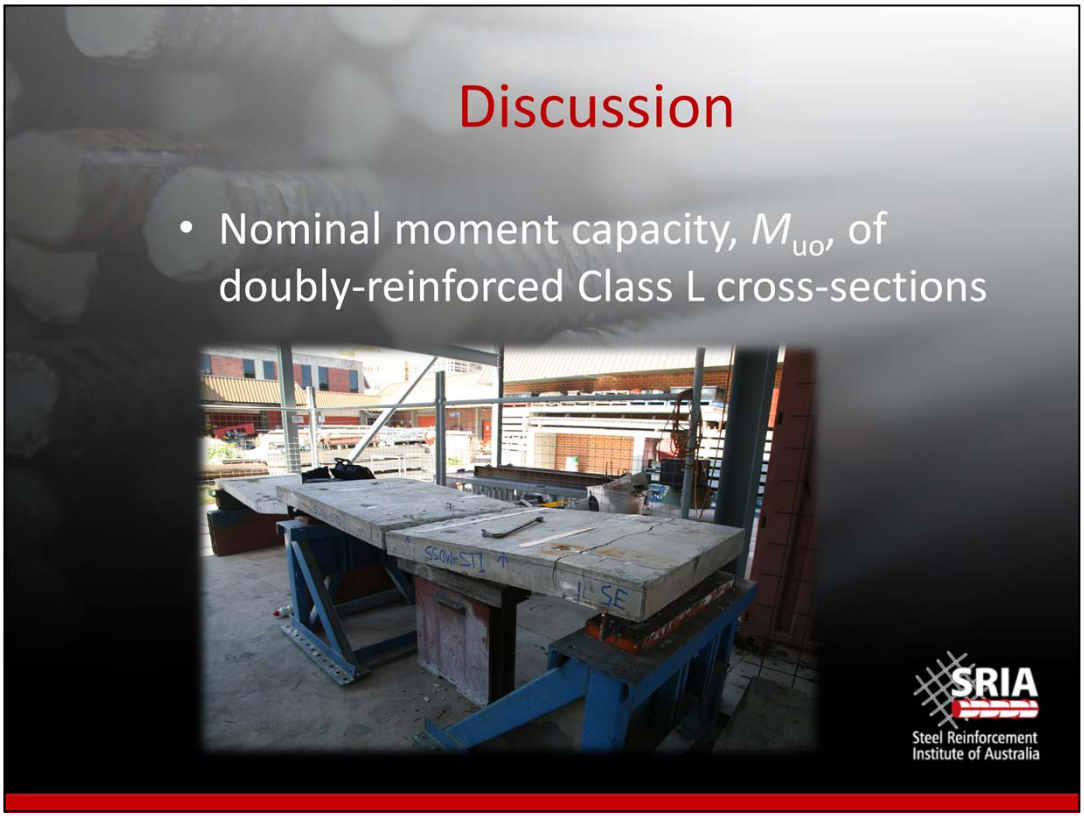


Here the results of the moment-curvature analysis are shown at three different curvatures.

The top line of four figures corresponds to when about 3 quarters of the peak moment was reached, at which stage the top figure on the right shows that the bottom bars were highly stressed but the top bars were just outside the compressive zone, and therefore carry little if any tensile force.

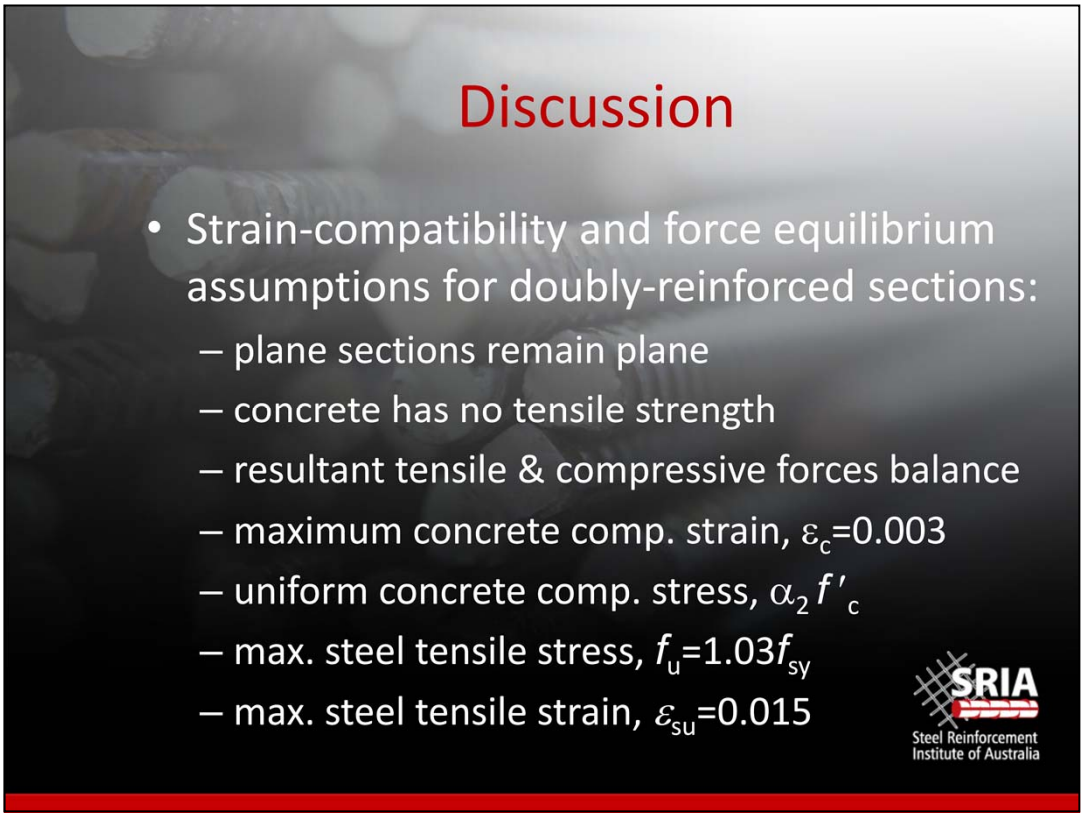
The middle line of figures shows that at peak moment even the top layer of steel is highly stressed.

The bottom line of figures shows what happens after the bottom tensile bars break, and some moment capacity still exists.



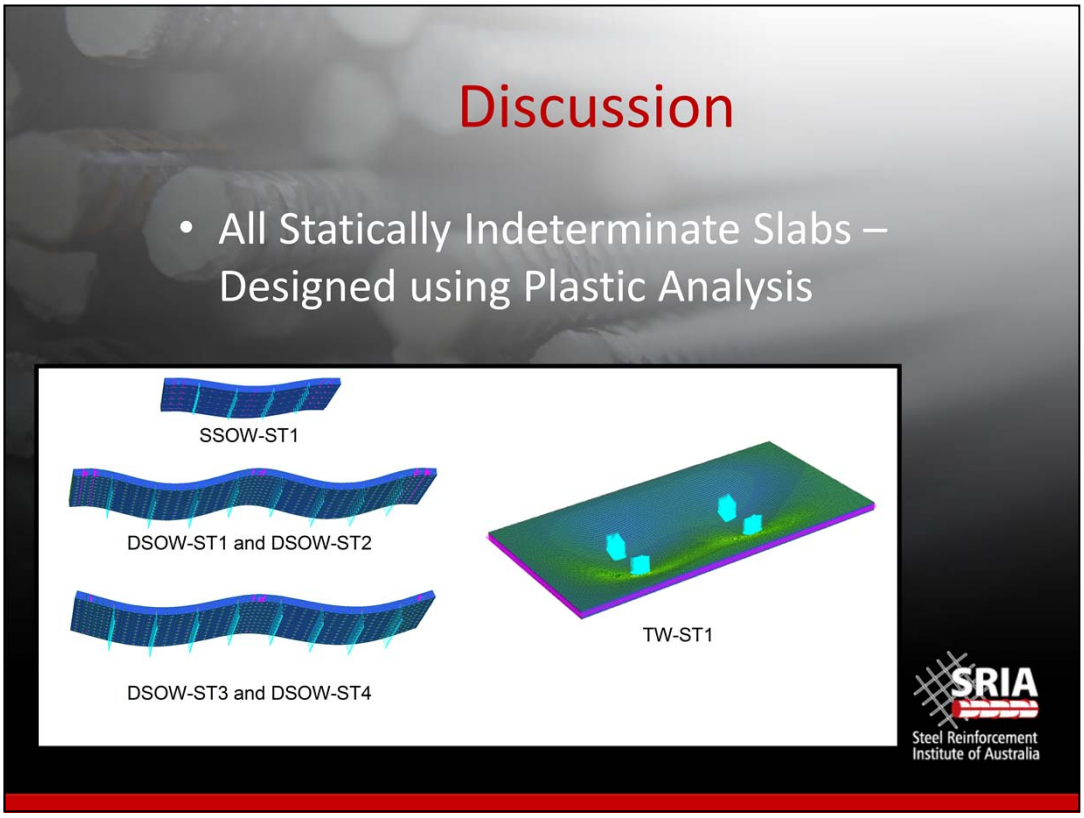
It can be concluded that the doubly-reinforced Class L cross-sections had additional moment capacity due to significant additional tensile force developing in the layer of steel nearest the compressive face, despite there only being 20 mm of cover. This reflects the significant ductility of the Ductility Class L mesh used in the tensile face.

This can be accounted for in design when calculating the nominal moment capacity, M_{uo}.

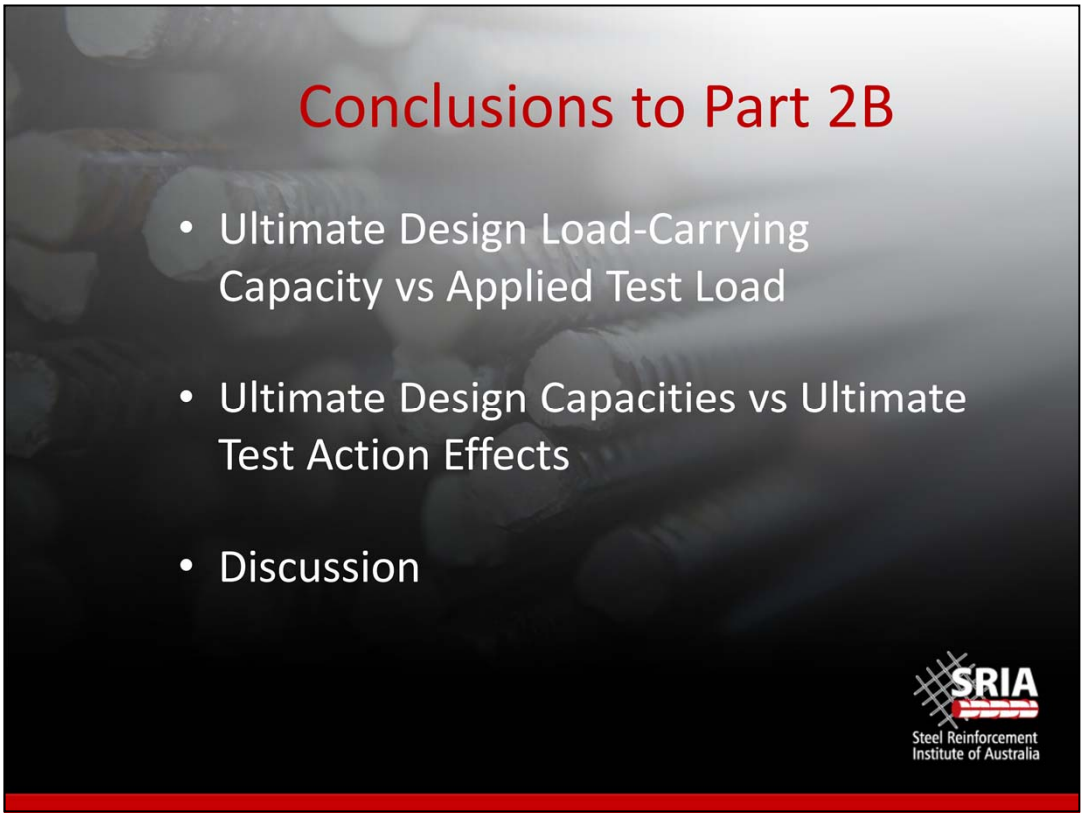


For this purpose, the following strain-compatibility and force equilibrium assumptions may be made. The paper shows how as a result the tensile stress of the layer of steel nearer the compressive face may be calculated.

Even assuming the minimum permitted uniform strain of 1.5%, the design moment capacities of the test slabs with doubly-reinforced sections would be increased by about 10 to 20%.



Although not permitted by AS 3600–2009 when designing slabs incorporating Class L mesh, plastic analysis more accurately represented the behaviour of the indeterminate slabs, particularly when compressive membrane action could develop.



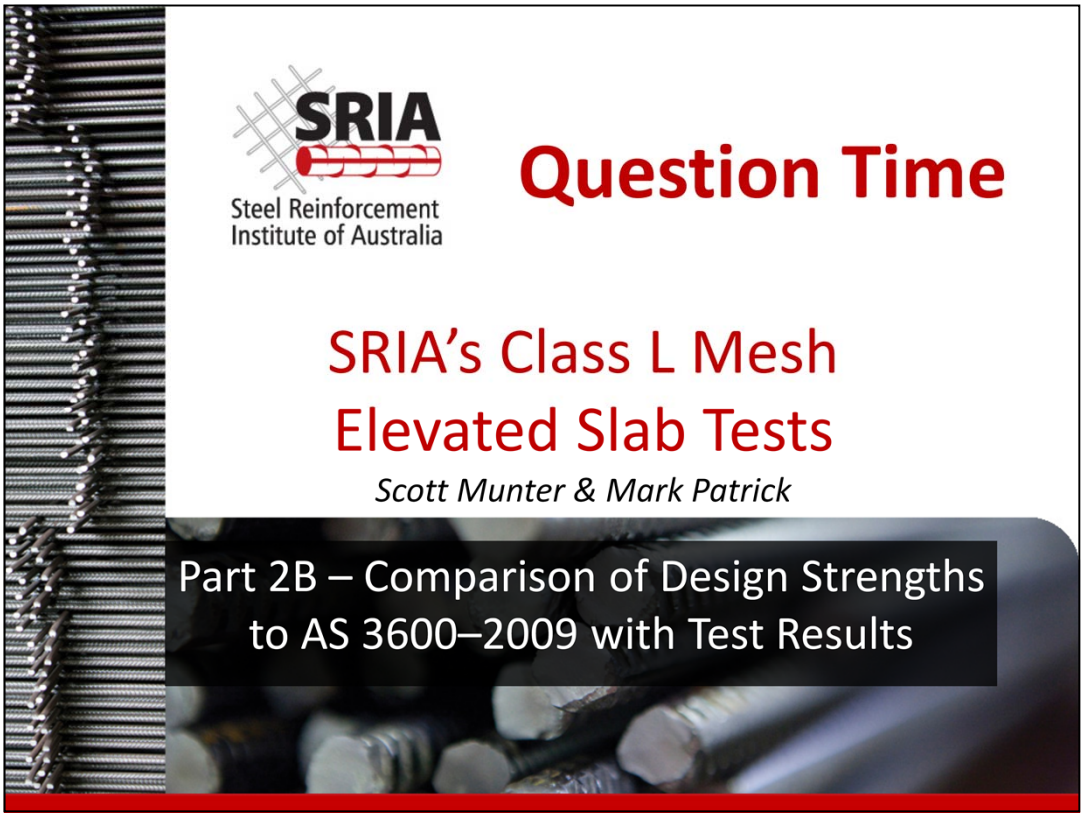
In conclusion:

The elastic methods of analysis in AS 3600–2009, when applied to the design of the redundant slabs with restrained ends or edges, have been shown to be very conservative, particularly with the 20% penalty applied to ϕ.

Interestingly, plastic analysis could more accurately model real behaviour and predict ultimate strength, particularly when compressive membrane action developed. As proven by the support settlement tests, significant amounts of moment redistribution can actually occur without affecting the ultimate strength of the slabs designed elastically ignoring moment redistribution.

Vertical shear strength was also considered, and no problems are apparent with the design rules in AS 3600–2009 applicable to slabs without shear reinforcement.

It's been shown why the typical doubly-reinforced cross-sections were stronger than expected. Designers could consider either directly accounting for the two layers of mesh using the method described, or else not applying the 20% penalty and treating the slab as if it were only singly-reinforced.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Question Time
SRIA's Class L Mesh
Elevated Slab Tests
Scott Munter & Mark Patrick
Part 2B – Comparison of Design Strengths
to AS 3600–2009 with Test Results