

A REVIEW OF RECENT AUSTRALIAN BOND TEST RESULTS AND THE NEW STRESS DEVELOPMENT DESIGN RULES OF AS 3600–2009

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ABSTRACT

While new design rules for stress development of straight D500N reinforcing bars by end anchorage or lap splicing were being written for inclusion in Section 13 of the Australian Concrete Structures Standard AS 3600-2009, bond test series were being undertaken independently at three Australian universities. One series focussed on lap splices in slabs characterised by small-diameter, widely-spaced bars without side edge effects. Transverse reinforcement was absent, and lap lengths were short so bond failure always occurred. Another test series used wide concrete blocks in unconventional pull-out tests, again with large side cover to laps between small-diameter bars, while transverse bars were included in some of the specimens but not between the main bars and the nearest concrete surface. The Steel Reinforcement Institute of Australia (SRIA) commissioned a third test series involving widely-spaced large-diameter bars lap-spliced in large-scale flexural specimens which were similar to a wide specimen tested decades ago in America, referenced in an ACI database of flexural bond tests with or without transverse steel present, on which the stress development design rules in ACI 318-08 are based. The SRIA tests are briefly described. Ultimate bond stresses reached in the Australian tests and tests from the ACI database are compared with design values calculated using AS 3600–2009 and ACI 318-08. Conclusions are drawn regarding the ductility requirements for D500N bars, and the effect of bar diameter. Advice about using the AS 3600–2009 design rules in light of the tests is provided.

NEW DESIGN RULES IN AS 3600–2009 FOR CALCULATING TENSILE DEVELOPMENT AND LAP LENGTHS

The new design rules in Section 13 of AS 3600–2009 for stress development of straight D500N reinforcing bars with design yield stress $f_{sy}=500$ MPa by anchorage or lap splicing are described by Munter et al. (2010). In normal-density concrete and for uncoated (bare steel) bars, basic development length ($L_{sy.tb}$) is calculated as:

$$L_{sy.tb} = \frac{0.5k_1k_3f_{sy}d_b}{k_2\sqrt{f'_c}} \geq 29k_1d_b \dots\dots\dots (1)$$

where: $k_1 = 1.3$ when more than 300 mm of concrete is cast below the bar (otherwise $k_1=1.0$); $k_2=(132-d_b)/100$ and $k_3=\{1.0 - 0.15(c_d - d_b)/d_b\}$ such that $0.7 \leq k_3 \leq 1.0$, with c_d being either the cover to the bar or half the clear distance to the next bar being developed ($a/2$), whichever is the smaller. In wide members like slabs and walls the cover to the tensile face only has to be considered, i.e. side cover may be ignored.

Factor k_2 was copied from Eurocode 2 (BSI 2004) to account for the greater bond stress of small compared to large diameter bars, e.g. comparing N12 and N24 bars, the difference equals $(132-12)/(132-24)=1.11$ or 11%, compared to a significantly greater allowance of 20% in ACI 318-08 (ACI 2008) for bars with 19 mm or smaller diameter.

Refined development length ($L_{sy.t}$) may be determined according to Eq. 2, where coefficients k_4 and k_5 account for transverse reinforcement and pressure, respectively:

$$L_{sy.t} = k_4k_5L_{sy.tb} \dots\dots\dots (2)$$

In particular, $k_4 = 1.0 - K\lambda$ ($0.7 \leq k_4 \leq 1.0$), and equals 1.0 when there is no transverse steel between the anchored or lapped bars and the concrete tensile face, i.e. $K=0$, as to be effective transverse steel must cross a potential splitting crack passing through a main bar.

In wide members (such as slabs and walls), lap length ($L_{sy.t.lap}$) is calculated using Eq. 3:

$$L_{sy.t.lap} = k_7L_{sy.t} \geq 29k_1d_b \dots\dots\dots (3)$$

Factor k_7 shall be taken as 1.25 unless the stress in the lapped bar at the ultimate limit state is less than or equal to $0.5f_{sy}$ and no more than half the reinforcement at the section is spliced, in which case k_7 may be taken as 1.0. For bars lapped in the same plane, clear distance, a , shall be determined assuming contact lapped splices, i.e. lapped bars shall be assumed to be touching each other, even if they do not.

Factor k_7 was copied from ACI 318-08. Thompson et al. (2002) explain that this factor (equal to 1.3 in ACI 318-08, but 1.25 in Eurocode 2) is “*a penalty (by increasing the lap length) to dissuade designers from needlessly or unwisely using lap splices in a structural design and particularly to prevent the Class B splice situation in which all tensile bearing bars in a section are spliced at a single location*”. Therefore, test data should support using Eqs 1 and 2 to calculate lap length, with clear distance, a , calculated appropriately.

RECENT AUSTRALIAN UNIVERSITY BOND TEST SERIES

Three independent bond test series were recently conducted at three Australian Universities, all involving contact lap splices of D500N bars in normal density concrete.

University of New South Wales (UNSW)

Gilbert (2008) described ten flexural-bond tests undertaken by Yeow (2008). Each simply-supported specimen had a span of 1800 mm with line loads applied at third-span positions giving a uniform-moment region of 600 mm to accommodate short tensile lap splices of between 150 and 280 mm long for N12 or N16 bars. They were all 850 mm wide by 150 mm deep, with no transverse reinforcement. Side cover was 125 mm,

bottom cover varied between 25 and 40 mm, and with 3 or 4 laps present, minimum half clear distance ($a/2$) between adjacent bars developing stress was 78 mm, and therefore theoretically bottom cover controlled their ultimate (average) bond stress, calculated for each test using elastic cracked-section theory and the maximum test bending moment outside the laps. The UNSW results are plotted in Fig. 1, where the vertical axis is the ratio of the ultimate bond stress reached in a test and the design ultimate bond stress ($=f_{sy} d_b / 4/L_{sy,t.lap}$) determined using Eqs 1 to 3 with $f_{sy} = 500$ MPa, refining coefficients product $k_4k_5 = 1.0$, and $k_7 = 1.25$. No account was taken that ultimate bond stress can reduce with increasing lap length (Esfahani and Rangan, 1996). The top cluster of six results above a ratio of 2.0 corresponds to the N12 bars, and the others to the N16 bars. Values on the horizontal axis were computed similarly but to ACI 318-08 with a lap-splice length penalty of 1.3.

University of Queensland (UQ)

Twenty-one unconventional pull-out tests were performed using 700 mm wide by 200 mm deep concrete blocks incorporating pairs of N16 lapped bars with only 15 mm minimum cover to allow for construction tolerance, much less than the large side cover of 250 mm, chosen to simulate slab or wall conditions. Some specimens had transverse bars, but unlike required by AS 3600–2009 in order to be assumed effective in design, they were not placed in the controlling cover to the main bars.

Bars fractured in tension in all but seven of the tests, with the mean tensile strength of the N16 bars equal to 637 MPa. Only the results of the seven UQ tests that exhibited bond failure are plotted in Fig. 1, three of which had the transverse steel. The top two points in Fig. 1 correspond to these latter three tests, but noticeably the other result is clustered with the tests without transverse bars. Lap length in the specimens that failed in bond was either 432 or 540 mm, which equalled the length of the test specimens.

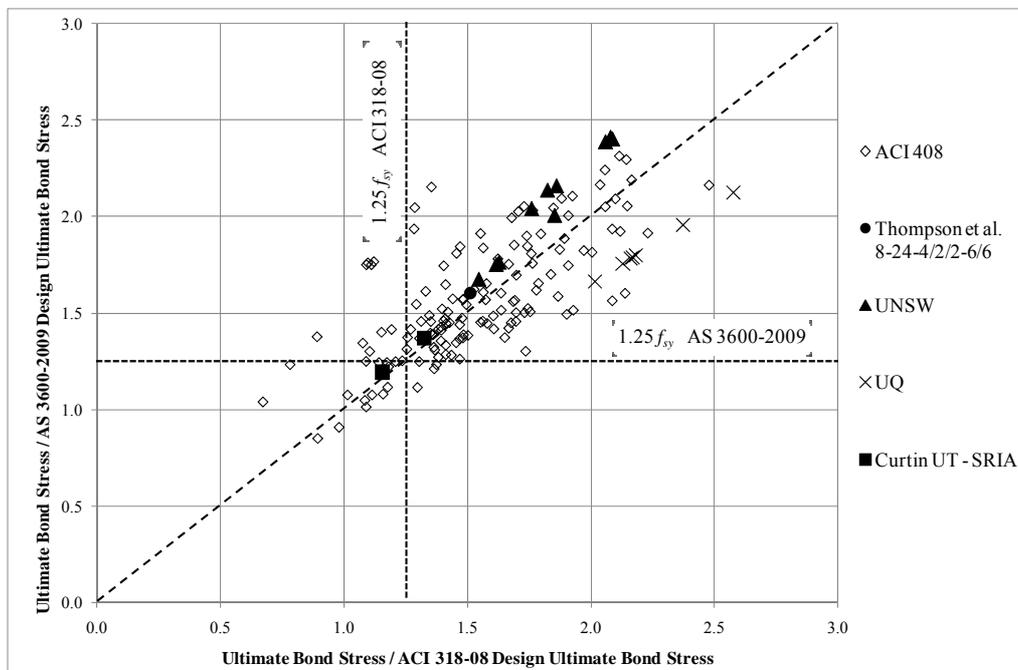


Fig.1: ACI 408 database (ACI 2003) & all three Australian university test series results.

Curtin University of Technology (CUT) – SRIA Tests

Patrick (2009) designed two similar specimens that were tested by Chandler and Lloyd (2010). One only had 6 lapped N24 bars, and was similar to a wide specimen tested by Thompson et al. (1979), viz. specimen no. 8-24-4/2/2-6/6, which is included in a database of important bond tests available from the ACI (2003). Results from this database for specimens incorporating bottom bars and without transverse bars are plotted in Fig. 1, where the test by Thompson et al. (1979) is also specifically identified.

The 900 mm wide by 300 mm deep test specimen was designed assuming $f'_c = 25$ MPa, and $c_d = \min.(a/2, c) = \min.((150-2 \times 24)/2, 50) = 50$ mm, while using Eq. 1, $k_1 = 1.0$ (as the bars were poured on the bottom) and $L_{sy.tb} = 930$ mm. A nominal 900 mm lap length ($L_{sy.t.lap}$) was used, corresponding to $k_7 = 1.0$ in Eq. 3, compared with 610 mm (24") used by Thompson et al. (1979). The companion specimen had 4 lapped bars with side cover increased from 50 to 126 mm, and half clear distance ($a/2$) from 51 to 76 mm, but with the same minimum cover, so lap length was 900 mm too. Both specimens were poured together and tested at a similar age with concrete compressive strength about 32 MPa.

The SRIA tests were conducted more rigorously than normal by including cyclic loading at high bar stress levels while approaching design ultimate load. The average yield stress of the bars was 549 MPa and the tensile strength was 653 MPa, reaching a tensile strain of about 5% at 625 MPa. Both specimens failed in bond, and using moment curvature analysis the maximum bar stresses were estimated to have reached 522 and 599 MPa in the 6-bar and 4-bar specimens, respectively – see 2 points in Fig. 1.

Also, the 4.5 metre long specimens were tested up-side-down to observe concrete crack development on the exposed top face in tension. Both specimens are shown after testing in Fig. 2, where the failure modes are evident: with 6 bars side splitting occurred with a horizontal crack finally forming between all the bars; while with 4 bars splitting failure was confined to the edge with face splitting occurring along each pair of lapped bars. These observations were predicted using a physical model proposed by Canbay and Frosch (2005), and the maximum stresses reached were also predicted to within $\pm 4\%$. It is also clear from Fig. 2 that bar prying would have had an effect, with the straight ends lifted up. This phenomenon is known to adversely affect bond, particularly if the lapped bars are relatively stiff and no fitments are present to tie the bars to the body of concrete in regions of curvature due to bending, e.g. see Thompson et al. (2002).



6-bar: side splitting with bar prying



4-bar: face splitting and edge side splitting

Fig.2: SRIA test specimen failure modes with N24 bars and 50 mm top cover

DISCUSSION & CONCLUSIONS

Clause 13.2.6 of AS 3600–2009 states that “*Welded or mechanical splices formed between Ductility Class N bars shall not fail prematurely in tension or compression before the reinforcing bars, unless it can be shown that the strength and ductility of the concrete member meets the design requirements*”. In accordance with ACI 318-08 and ACI 439.3-R07 (ACI 2007), under low-seismic design conditions this requirement would be deemed to be met if the splice (referred to as Type 1) allows a tensile stress of at least $1.25f_{sy}=625$ MPa to develop in D500N bars. This same stress limit has historically been used when formulating design rules for lap splices, but more rigorous reliability analysis can be undertaken instead (ACI 2003). Under seismic conditions, Type 2 mechanical or welded splices are required to develop at least 1.25 times the actual yield stress, while using lap splices under these conditions is not recommended.

The dashed diagonal line in Fig. 1 corresponds to when the design methods for calculating lap length ($L_{sy,t.lap}$) in AS 3600–2009 (with $k_7=1.25$) and ACI 318-08 (with a penalty factor of 1.3) give the same value of lap length for $f_{sy}=500$ MPa. The horizontal and vertical lines correspond to a test tensile stress of $1.25f_{sy}=625$ MPa.

A general observation of Fig. 1 is that there are a number of test results from the ACI 408 database either to the left of the dashed vertical line (indicating unconservative designs to ACI 318-08), or below the dashed horizontal line (indicating unconservative designs to AS 3600–2009). Accordingly, it is explained in ACI 408R-03 that “*the factor 1.3 for Class B splices provides strength that helps make up for some unconservative aspects of the ACI 318 bond provisions*”. Given that the test data in Fig. 1 tends to be reasonably evenly spread about the diagonal line, it can be concluded that AS 3600–2009 and ACI 318-08 generally give similar design values of $L_{sy,t.lap}$. It follows that k_7 too is not just a penalty factor, but also a correction factor, which explains why the additional stress criterion of $0.5f_{sy}$ should also apply in the new Australian rules.

Concerning the Australian tests, the very high AS 3600–2009 test/design values of 2.0 or more corresponding to the N12 bar tests by the UNSW tend to imply that factor $k_2=(132-d_b)/100$ could be increased, subject to a more detailed investigation however. High test/design values were also obtained in the UQ tests, but these test results need to be interpreted cautiously as the test specimens were not subjected to curvature, and therefore could be misleadingly high. Regarding the SRIA 6-bar test performed at CUT, for which the test result in Fig. 1 falls slightly below the dashed horizontal line indicating that AS 3600–2009 would provide a slightly unconservative design even with $k_7=1.25$, i.e. if $L_{sy,t.lap} = 1.25 \times 930 = 1160$ mm for $f'_c = 25$ MPa. However, the specimen was tested more rigorously than normal (which could in part explain why the result is significantly less than for historical specimen 8-24-4/2/2-6/6), but more particularly it would be normal for such closely-spaced large diameter (N24) bars to be enclosed by fitments for shear resistance, in which case it appears that satisfactory performance would be achieved designing to AS 3600–2009. As illustrated in Fig. 2 and confirmed by newly-developed physical models by Canbay and Frosch (2005) and Wang (2009), the failure mode of the SRIA 4-bar specimen tested at CUT was less severe. Despite having the same minimum cover that controlled the design value for lap length ($L_{sy,t.lap}$), the result from the 4-bar test indicates that such an arrangement of bars could be designed satisfactorily using AS 3600–2009, even without fitments present.

Following conducting the two tests at CUT, the SRIA is continuing to research the new stress development design rules in AS 3600–2009 and their application in practice.

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Scott Munter is a structural engineer and Executive Director for the Steel Reinforcement Institute of Australia (SRIA). Previously Scott worked for BlueScope Steel for almost 3 years as the Lysaght National Structural Decking Manager then High-Rise Business & Engineering Manager for BlueScope Buildings.

Scott served for 7 years with Australian Steel Institute as the State Manager-NSW then National Engineering Construction Manager working on a variety of key projects such as the Steel Connection Design Series. Scott also has a broad 15 year commercial, industrial and residential track record as a Civil & Structural Consulting Engineer with SCP Consulting in the Engineering Design and Construction field.

He graduated with a Bachelor of Structural Engineering (under the part-time attendance program, 6 year degree) from the University of Technology, Sydney in 1991 with 1st Class Honours, University Medal and the Engineers Australia Medal. As a Member of Engineers Australia he holds Charter Professional Engineer & NPER (Structural) status. He is a member of a number of Standards Australia committees including BD-002 Concrete Structures.

Mark Patrick holds BE and MEngSc degrees from Melbourne University and a PhD from Sydney University. For 6 years he was a consulting structural engineer; researched composite and concrete construction at BHP Melbourne Research Laboratories for 15 years; held a professorial position at the University of Western Sydney for 5 years; before starting a specialist structural engineering consultancy practice six years ago. He is a member of several Standards Australia committees including BD-002 Concrete Structures.