THE TORSION RULES IN THE AUSTRALIAN BRIDGE DESIGN CODE AS5100

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SYNOPSIS

The author is a member of the Standards Association BD002 Committee revising the AS3600 Concrete Structures Standard, and has submitted a proposal for revising the Shear and Torsion rules for that Standard. There are a number of errors and shortcomings in the present set of rules, which, whatever problems they may create in buildings, will be significantly worse for bridges. The Bridge Design Standard, AS5100 and AS3600 both contain errors and shortcomings at present, and the author believes both should be corrected, improved and made as compatible as possible. For bridges, in some instances the errors produce very conservative results, and in other instances unconservative results. This paper outlines the author's proposed rule changes and discusses how they might affect bridge designs ranging from Super Ts through to large box girder bridges, using specific examples. It is a subject on which anyone designing any type of concrete box superstructure, including Super Ts should be informed.

1. INTRODUCTION

While the focus of this paper is on the Torsion clauses of the Australian Bridge Design Standard (Ref 1), also referred to as the Bridge Code, some mention will also be made of the Shear clauses. It will be argued that essentially the Shear clauses as they stand are reasonably sensible and technically correct, although there is certainly still room for debate which is beyond the scope of this paper. The Torsion clauses by comparison contain a number of errors, which must be corrected, and the whole layout of the Torsion clauses should be reviewed to make them much simpler – in fact it will be argued that an implied shear should be extracted from torsion actions, and simply then added to flexural shear, with a few minor qualifications. The effects of the existing errors and proposed changes will be shown in a Super T example and a large box girder example.

2. WHAT IS TORSION?

Torsion is the action effect of an applied torque, or twisting moment on a beam. What that action effect actually causes is a shear flow which twists around the beam, producing a spiralling compression stress field along the beam, and for a concrete beam, longitudinal and transverse tensile forces. As can be observed in Figure 1, each face then becomes a shear plane which is exactly the same as a web in flexural shear, and these longitudinal and transverse tensile forces are then carried by reinforcement or prestressing steel, longitudinally and as ties, just as in a flexural member. Figure 1 also shows how a "line beam" torsion flexure model represents a single cell box girder for eccentric loads, and that the design issue is the face

(generally a web) where flexural and torsion shears are additive. This line beam with a torsional stiffness could also be incorporated into grillage models.
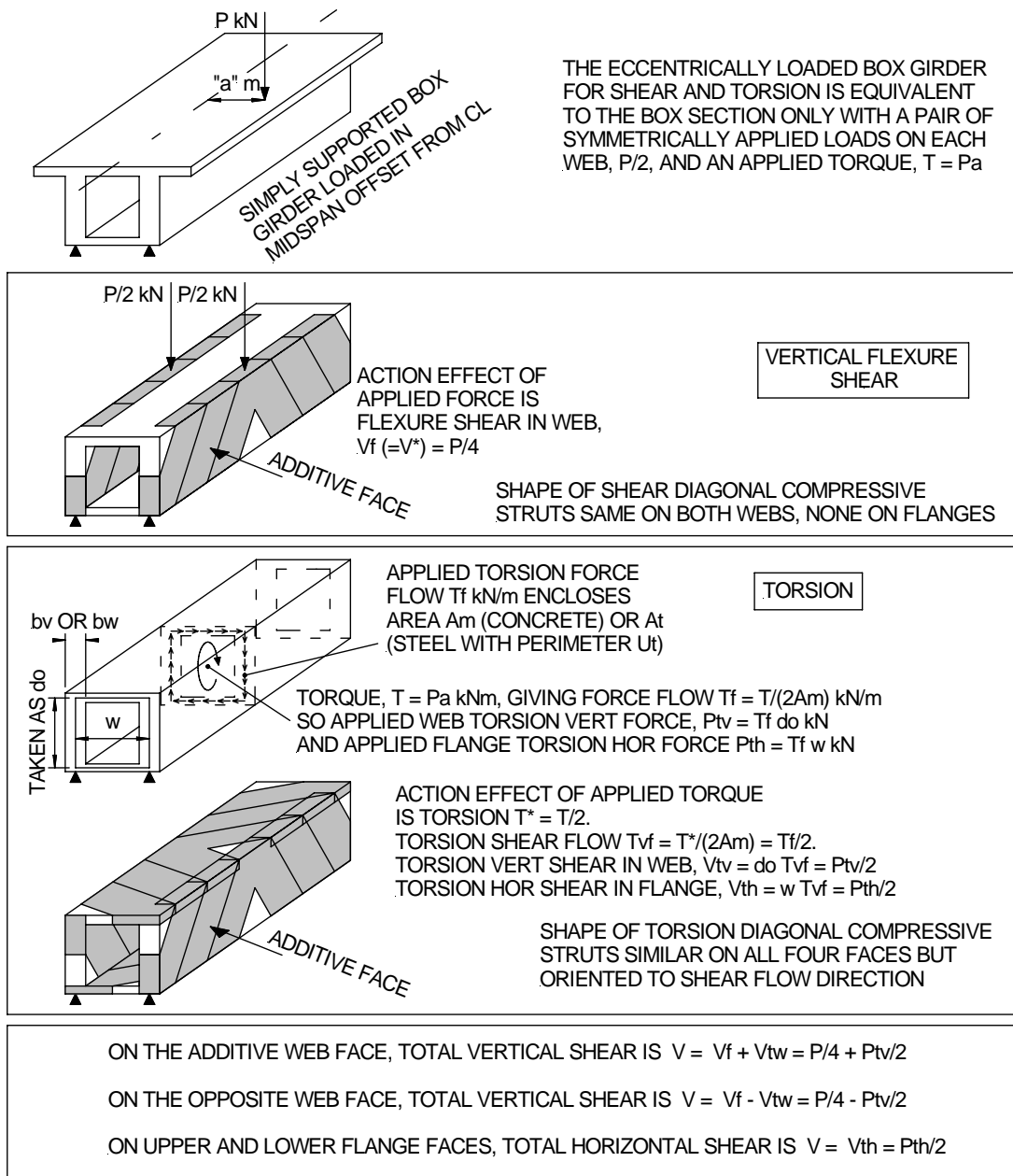

THE ECCENTRICALLY LOADED BOX GIRDER FOR SHEAR AND TORSION IS EQUIVALENT TO THE BOX SECTION ONLY WITH A PAIR OF SYMMETRICALLY APPLIED LOADS ON EACH WEB, P/2, AND AN APPLIED TORQUE, T = Pa

VERTICAL FLEXURE SHEAR

ACTION EFFECT OF APPLIED FORCE IS FLEXURE SHEAR IN WEB, Vf (=V*) = P/4

SHAPE OF SHEAR DIAGONAL COMPRESSIVE STRUTS SAME ON BOTH WEBS, NONE ON FLANGES

TORSION

APPLIED TORSION FORCE FLOW Tf kN/m ENCLOSES AREA Am (CONCRETE) OR At (STEEL WITH PERIMETER Ut)

TORQUE, T = Pa kNm, GIVING FORCE FLOW Tf = T/(2Am) kN/m
SO APPLIED WEB TORSION VERT FORCE, Ptv = Tf do kN
AND APPLIED FLANGE TORSION HOR FORCE Pth = Tf w kN

ACTION EFFECT OF APPLIED TORQUE IS TORSION T* = T/2.
TORSION SHEAR FLOW Tvf = T*/(2Am) = Tf/2.
TORSION VERT SHEAR IN WEB, Vtv = do Tvf = Ptv/2
TORSION HOR SHEAR IN FLANGE, Vth = w Tvf = Pth/2

SHAPE OF TORSION DIAGONAL COMPRESSIVE STRUTS SIMILAR ON ALL FOUR FACES BUT ORIENTED TO SHEAR FLOW DIRECTION

ON THE ADDITIVE WEB FACE, TOTAL VERTICAL SHEAR IS V = Vf + Vtw = P/4 + Ptv/2

ON THE OPPOSITE WEB FACE, TOTAL VERTICAL SHEAR IS V = Vf - Vtw = P/4 - Ptv/2

ON UPPER AND LOWER FLANGE FACES, TOTAL HORIZONTAL SHEAR IS V = Vth = Pth/2

Figure 1 Line Beam Torsion Flexure Model for Shear

It can also be readily observed from Figure 1 that the torsional shear, Vtw, on the additive, or any other face can easily be derived from the very simple cellular structure formula, Vtw = T*do/(2Am), or Vth = T*w/(Am), where the torsional shear flow round all faces is Tvf = T*/(2Am).

2.1 Three Dimensional Analysis Versus Line Beam Torsion Flexure Models

Considering the information presented in Figure 1, the question might well be asked whether there is any difference between the action effects from a Line Beam Torsion Flexure model and a three dimensional analysis such as a finite element model, or WW space frame (Ref 2), which gives more directly usable results. The answer is that there is essentially no difference, especially for a single cell girder where the interaction of the torsional and flexural stiffnesses is not an issue. Where there are multiple girders, or multi-cell girders, caution should be exercised, as the flexural out of plane distortions of the boxes can significantly affect the pure torsional stiffness:

$J = 4\ Am^2/\Sigma(d_s/t)$, where d_s = web or flange length, and t = web or flange thickness

The J value may also be significantly reduced by torsional cracking relatively more than the flexural stiffness. Hambly (Ref 3) discusses this and shows examples where the effective elastic J is several times smaller than the pure J, but notes that the effective J will also be stiffened back up towards the pure J by diaphragms. For the single cell example given later in the paper, a finite element model was used to compare the stress action effects from loading with (A) a single eccentric point load, and (B) a symmetric pair of point loads plus a direct torsion force flow as in Figure 1. The effects were similar, though somewhat affected by local distortions close to the applied load, and very similar further away from the applied load. In this case, unlike Hambly (Ref 3), the torsional rotations were remarkably similar.

In general the Line Beam Torsion Flexure model is a reasonably good representation, especially for a single cell box, but in larger structures a three dimensional model should also be used, at least to "calibrate" the line beam model.

3. ERRORS AND SHORTCOMINGS WITH TORSION IN THE BRIDGE CODE

3.1 The Steel Truss Model and the "Concrete Contribution"

Australian Bridge Codes over the years since the 1980s have not fundamentally varied the approach on torsion. A review of several of the references cited in the Commentary to the Austroads Bridge Code (Ref 4) was made, and these are presumed to apply to the current Bridge Code (Ref 1) clauses. The references reviewed include the CEB/FIP Model Code for Concrete Structures (Ref 5) plus a source paper for Ref 5 by Thurlimann (Ref 6), Collins and Mitchell (Ref 7), and Walsh (Ref 8). In addition the Eurocode for Concrete Structures (Ref 9), and the ACI Concrete Code, ACI 318-02 (Ref 10) were perused. ACI 318-05 was recently published, but the author understands that the relevant sections are unchanged.

There are two fundamental aspects of torsion, one of which is agreed upon by all of these references and by the Australian Bridge Code (Ref 1), and the other by all except the Bridge Code (Ref 1), and Walsh (Ref 8) who was writing about Australian Concrete Standards, and therefore takes the same position as they do. These two aspects are:

- All sources agree on the truss models for shear and torsion, for assessing the contribution of steel strength to shear capacity in a web or flange, for shear

and torsion. There is no controversy about this and for convenient reference the truss equations are derived in the format of the Bridge Code (Ref 1) in an Appendix to this paper.

- All sources, except Walsh (Ref 8) incorporate the same allowances for the "concrete contribution" for both shear and torsion, within each source. Note that between the various sources there are widely differing opinions as to the nature and magnitude of the "concrete contribution", but only the Australian Standards, AS3600 (Ref 11) and the Bridge Code (Ref 1) and, by extension, Walsh (Ref 8) incorporate a concrete contribution for pure shear, and then remove it both for shear and torsion when torsion is present. This is the most important basic problem with torsion as treated in Australian Standards. By disallowing the "concrete contribution" (Vuc in the Bridge Code, Ref 1) the torsion clauses, which are mandatory, inevitably dominate a much more conservative design than would have been done with say a finite element or WW Space Frame model, incorporating Vuc. The only explanation the author has found for this is in Walsh (Ref 8), who says "*The draft code rules have to deal with the inconsistency that the torsion is resisted entirely by the truss action, whereas part of the shear resistance is provided by Vuc, i.e. the "concrete" component. It assumes very conservatively that for significant torsion Vuc should be taken as zero.*" To the author this does not seem to recognise that, as shown in Figure 1, shear and torsion are one and the same, and that therefore as all other sources tend to do, they should be treated the same in allowing the Vuc to be counted. Some extra observations about the various versions of "concrete contribution" will be made later in the paper.

3.2 List of Shortcomings in the Australian Bridge Code

Apart from the issue of the inconsistency of approach to Vuc, the following list of items is also suggested as requiring improvement, taking them in the order they appear in the Bridge Code (Ref 1). Some items also apply to clauses on Shear:

3.2.1 **"D-shift" rule.** The "D-shift" should really be $d_o\cot\theta v$ in Clause 8.1.8.2, in line with the truss analogy.

3.2.2 **End anchorage.** End anchorage of longitudinal reinforcement and prestress for shear and torsion is presently covered by Clause 8.1.8.3, which requires 1.5V* to be anchored beyond the face of the support. To be correctly aligned with the truss analogy this should require that V*cotθv be anchored beyond the compression node over the support. A simple deemed to comply rule for the compression node location would be halfway across the bearing, and the anchorage also needs a Strength Reduction Factor of 0.7. This clause needs altering because the current version of the Code (Ref 1) tends to lead to flat truss angles approaching 30° which need much more anchorage for shear at a support than the 1.5V* provides. The latter implies a truss angle of about 45°. It would also be useful, especially for members such as Super Ts, to allow truss angles higher than 45° (by incorporating a few more ties) which would then require less anchorage in what is often a very limited zone. This clause should also make it clear that longitudinal torsion forces must be anchored (in the upper chord as well) in addition to the 1.5V*. The anchorage of tendons, clause 13.3.2, should be revised to 0.1 Lpt, instead of 0.1 Lp (which seems to be an unintended error), and it should also be made clear that even at the

Strength Limit, development at the end of the strand initially follows the transmission length path i.e. is non-linear between transmission through to full development. This is also very important for Super Ts.

3.2.3 Vuc zero with torsion present. While the author takes the position that Vuc should be allowed with torsion present, if it is not to be allowed, a forward reference should be given in Clause 8.2.2 to Clause 8.3.4, which will determine whether Vuc must be taken as zero. As presently written the Code (Ref 1) is very confusing.

3.2.4 Vu.max. The formula, 8.2.6 (1) in Clause 8.2.6 should include a $\sin 2\theta v$ term (see truss formulations in Appendix). See also comments in 3.2.7 below.

3.2.5 Vo or Vt with torsion present. If Vuc is to be allowed with torsion present, in Clause 8.2.7.2, for calculating the cracking moment Mo, the axial tensile force caused by the torsion should be included, but the torsion shear would not be included in the V* associated with M*. The axial force also affects Vt.

3.2.6 Range of θv values, with inclusion of torsion shear. The present formulation of Clause 8.2.10 effectively produces only one value of θv, although it does add that it may be "conservatively" taken as 45°. The value which it produces between 30° and 45° is effectively a minimum value which approaches 45° as the concrete stresses increase towards a crushing failure, i.e. thinner webs. Other Codes allow values from as low as 22°, but as explained in Collins and Mitchell (Ref 7) the mechanism to ensure yielding in the ties for flat angles, or yielding longitudinal reinforcement for steep angles will limit the range of possible truss angles as the concrete stress in the struts increases. Clause 8.2.10 reflects this by effectively setting a lower limit to θv. However it seems reasonable (and in line with mechanisms described by Collins and Mitchell, Ref 7) to allow a free choice of truss angles, bounded on either side of 45° by the same formula as is presently incorporated into Clause 8.2.10. Although steep truss angles are usually uneconomic, for special situations such as end anchorage of shear in Super Ts, they can be useful. When torsion is also present, torsion shear (see Section 2 above) should be added to flexure shear for the effective total V* in determining the θv range.

3.2.7 Torsional web crushing. For Clause 8.3.3 it would be much more transparent and direct to calculate torsion shear (see Section 2 above) and add it directly to flexure shear in the formula at Clause 8.2.6 (see 3.2.4 above), which also will bring in the duct diameter reductions for torsion shear. At present this does not happen which is wrong. As with Clause 8.2.6, there should be a $\sin 2\theta v$ term.

3.2.8 Threshold torsion capacity and error calculating T*. Clause 8.3.4 (a) provides threshold values of torsion capacity, as a proportion of torsional concrete cracking capacity below which torsion can be ignored. This is similar in principle to ACI (Ref 10), but it would seem much more preferable to simply calculate the torsion shear present (see Section 2 above) and add it to the flexural shear, which is the Eurocode (Ref 9) approach. The examples given later in the paper show how use of this threshold value can lead to very unconservative designs. If this Clause is to be retained, there is also what seems to be an unintended but quite serious error in determining the T* to be used in checking whether the section has sufficient torsion cracking strength. The clause says "T* shall be calculated taking into account the effect of cracking on the torsional stiffness" (Ref 1), when it should actually say T* shall be calculated using the uncracked section stiffness. Even if the threshold

allowance is dispensed with, these criteria really need to be retained for determining whether using torsion redistribution in accordance with Clause 8.3.2 requires minimum torsion reinforcement in accordance with Clause 8.3.7. (See also 3.2.12 below).

3.2.9 **Torsion and shear steel strength interaction error, and Pv error.** At Clause 8.3.4 (b), the interaction formula is simply wrong. It confuses the strength associated only with closed ties with the total strength and gives wrong and conservative results. The effects of this are shown in a subsequent example. This is also the only place in the Code (Ref 1) which indirectly requires that Vuc be taken as zero for shear when torsion is present. This is very confusing (see also 3.2.3 above). This interaction formula should be deleted, and steel for torsion strength included as described in 3.2.10 below. Also in the transition from the previous to the current Bridge Code (Ref 1) an unintended error occurred where ϕ(Vus+Pv) was changed to (ϕVus+Pv).

3.2.10 **Error in torsion truss angle and steel contribution to torsion capacity.** Clause 8.3.5 (b) uses a different symbol θt, though it must be the same angle θv as in Clause 8.2.10. It would be much simpler to calculate the torsion shear (see Section 2 above) and add it to the flexural shear in Clause 8.2.10 (see also 3.2.6 above). The torsion shear has to be calculated to correctly determine θv anyway.

3.2.11 **Longitudinal torsion force.** Clause 8.3.6 should be altered to require a single tensile force be added to the section halfway between the upper and lower chords. The force should also be required to be anchored at the ends of the beam (see 3.2.2 above) and incorporated in the computation of Vo or Vt, if Vuc is to be allowed (see 3.2.5 above). Detailing rules in Clause 8.3.8 should also allow prestress for this force.

3.2.12 **Minimum torsional reinforcement.** Clause 8.3.7 gives Asw/s > 0.2y1/fsy.f as a new minimum torsion reinforcement, which is a significant improvement because the old one, Tuc was much too high. For some reason the old one is retained and it is not made clear that the lesser of the two may be used. AS3600 (Ref 11) uses the same minimum for torsion as for shear, being 0.35bw/fsy.f, taken as closed ties. This seems reasonable when it is considered that torsion is really only shear anyway, but the author believes that if significant torsional redistribution is permitted as in Clause 8.3.2, and especially if the threshold cracking torsion capacity of Clause 8.3.4 is retained, then it is prudent to have a higher value. Compare the following shear flow values (see Section 2 above) implied by the three options mentioned:

Min torsion option	Shear Flow, kN/m	
		Assumptions: f'c = 50 MPa
0.35bw/fsy.f	90 to 420	$\theta v = 30^o$
0.2y1/fsy.f	350 to 2080	bw = 150 to 700 mm
Tuc, rf conc	550 to 2570	y1 = 1000 to 6000 mm
Tuc, ps conc	820 to 3820	avge concrete ps = 6 MPa

The Bridge Code (Ref 1) value of 0.2y1/fsy.f is about half the old version, Tuc (with prestress) which is still quite a "jump" above the threshold ϕ0.25Tuc. Even if the "ignore torsion" threshold is abolished it is suggested that for

uncracked analysed torsion below $\phi 0.25 T_{uc}$, $0.35 b_w/f_{sy}.f$ could then be used as the minimum in lieu of $0.2y_1/f_{sy}.f$, with the associated axial force.

3.3 Further Comment on Concrete Contribution

Walsh (Ref 8) gives the rationale for the concrete contribution, Vuc in the Australian Standards, explaining that the truss analogy could not account for all the shear strength experimentally present, especially at low levels of shear reinforcement. At that time there was a lively debate about what the "concrete contribution" actually was, and there were two schools of thought - (A) that there is a concrete contribution similar in principle to the shear friction concept in ACI (Ref 10), and also reflected in the longitudinal shear Clause 8.4 in the bridge Code (Ref 1); and (B) that the truss theory could explain all test results, and that previous researchers had simply underestimated how flat truss angles could get. The latter view is expressed in conclusion 3 of Collins and Mitchell (Ref 7) who say "*Because θ can vary within such wide limits, it is not necessary to introduce an empirical correction term (a "concrete contribution") to account for the strength of lightly reinforced members.*" However in the same set of conclusions they speak of "*mechanisms of interface shear transfer*" being affected by excessive cracking, and one wonders whether they can really explain away the apparent increase in non-truss strength reflected by increased percentages of flexural tensile steel, and further enhanced by prestressing. Both phenomena would be predicted by a shear friction concept, and both are reflected in the Vuc (including Vo) equation 8.2.7.2(1) of the Bridge Code (Ref 1). For a concept comparison, but not quantified, the following Table is presented comparing the Eurocode (Ref 9), which uses flat truss angles, ACI (Ref 10) which uses a shear friction type concrete contribution, and the Bridge Code (Ref 1) (and AS3600, Ref 11) which is similar to ACI (Ref 10).

	Eurocode (Ref 9)	ACI (Ref 10)	Bridge Code (Ref 1)
θv range	22-45°	45° shear, 30-60° torsion	30-45°
θv ductility restriction	Yes	shear 45°, torsion 'by analysis'	Tends to 45° at max strut compression.
Web loses thickness at ultimate?	No	No	Yes, by restricting comp. to 0.4f'c. (See Appendix.)
Vuc (inc Vo)	None	Yes	Yes
Vert p/s, Pv?	Include as load	Include as resistance	Include as resistance
Threshold torsion?	No. Convert and add to other shear.	Yes	Yes
Redistribute compatibility torsion?	Yes, with minimum reinforcement.	Yes, with detailing rules.	Yes, with minimum reinforcement.
Same shear strength with torsion present?	Yes	Yes	No. All concrete contribution disallowed.

In the author's opinion, the Australian Standards have the most reasonable concepts for concrete contribution to shear, but have omitted it completely when torsion is contributing to shear for no discernible reason.

Given that the Bridge Code (Ref 1) does not allow any concrete contribution (Vuc) once torsion is present, one may ask how much shear strength is lost. The following table gives the proportion of Vuc to total strength, Vu (including steel, Vus). It was derived for 50 MPa concrete, over an extensive range of reasonable combinations of web and flange thicknesses, heights and widths, and various proportions of reinforcing and prestress. It considers the lesser of web tensile and flexure induced Vuc values, using the ratio of the moment and shear capacity of the section as the M*/ V* ratio for calculating Vo. Minimum practical shear reinforcement was used, which tends to be higher than minimum reinforcement to Clause 8.2.8.

	Reinforced Concrete, Vuc/Vu		Prestressed Concrete, Vuc/Vu	
	With minimum practical ligs	With concrete crushing	With minimum practical ligs	With concrete crushing
Mean	0.20	0.08	0.39	0.24
Std Deviation	0.02	0.01	0.04	0.04

This shows that a very large proportion of strength is lost, especially for prestressed concrete, right through the range from very low levels of reinforcement up to when crushing failure occurs below tie strength.

4.   WORKED EXAMPLES

4.1 Super T

Figure 2 shows a real case, analysed first, as required by Bridge Code (Ref 1) Clause 8.3.4 (ignoring the error described at 3.2.8) using the uncracked stiffness, and then using Clause 8.3.2 for compatibility torsion redistribution, with zero torsional stiffness.
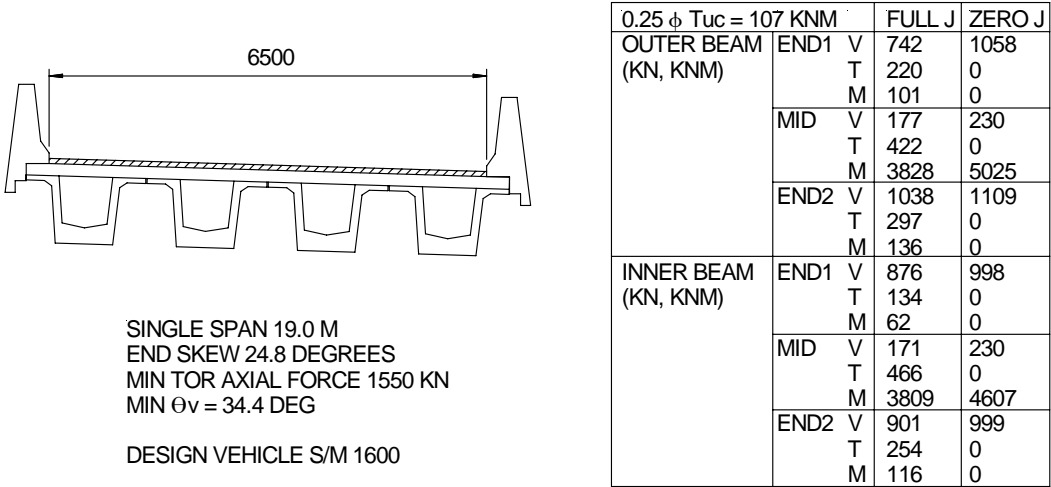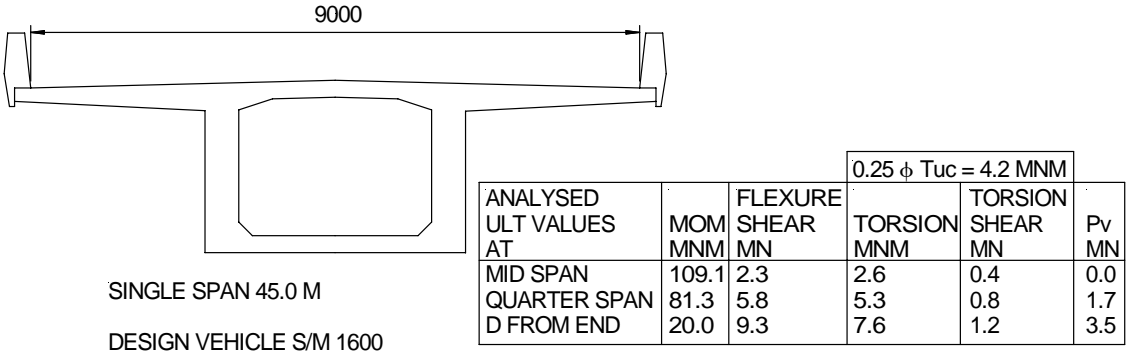

0.25 φ Tuc = 107 KNM			FULL J	ZERO J
OUTER BEAM (KN, KNM)	END1	V	742	1058
		T	220	0
		M	101	0
	MID	V	177	230
		T	422	0
		M	3828	5025
	END2	V	1038	1109
		T	297	0
		M	136	0
INNER BEAM (KN, KNM)	END1	V	876	998
		T	134	0
		M	62	0
	MID	V	171	230
		T	466	0
		M	3809	4607
	END2	V	901	999
		T	254	0
		M	116	0

6500

SINGLE SPAN 19.0 M
END SKEW 24.8 DEGREES
MIN TOR AXIAL FORCE 1550 KN
MIN Θv = 34.4 DEG

DESIGN VEHICLE S/M 1600

Figure 2 Super T Example

Firstly it can be seen that the threshold uncracked torsion of 0.25 ϕTuc is exceeded all over the bridge, and secondly the torsional stiffness has a significant influence on the action effects. Note that even with uncracked stiffness for the girders, the torsion constant was made zero for the deck slab and end diaphragms, and the author believes a set of guidelines about these assumptions should be issued by the Road Authorities. In this case then Vuc must be made zero, and the minimum torsion reinforcement of Clause 8.3.7 is required, including the longitudinal extra tensile force. One might then suggest that if this reinforcement has to be included anyway, the torsion constant could be included at say 25% of its value so that torsion is present at least up to this minimum level, with a beneficial reduction in moment.

What is clear from this example is that if the torsion stiffness is made zero, and then the threshold torsion is taken as not exceeded, a considerable illegal advantage would be gained by including Vuc. Anecdotally, this seems to occur quite commonly, and the author is even aware of inexperienced designers including the torsion stiffness at full value and then proceeding to ignore torsion completely.

4.2 Box Girder

Figure 3 shows an example of a reasonably large single cell box girder. The data presented are a realistic set of values for the various bases, particularly including instances where torsion is below the threshold 0.25ϕTuc, and where the minimum reinforcement Tmin = 0.2y1/fsy.f, or Vmin = 0.35bw/fsy.f can govern for the proposals of this paper (see 3.2.12). Minimum practical shear reinforcement is 1100 mm^2/m, but out of plane web or flange bending has not yet been considered.

SINGLE SPAN 45.0 M

DESIGN VEHICLE S/M 1600

0.25 ϕ Tuc = 4.2 MNM					
ANALYSED ULT VALUES AT	MOM MNM	FLEXURE SHEAR MN	TORSION MNM	TORSION SHEAR MN	Pv MN
MID SPAN	109.1	2.3	2.6	0.4	0.0
QUARTER SPAN	81.3	5.8	5.3	0.8	1.7
D FROM END	20.0	9.3	7.6	1.2	3.5

BASIS	SPAN LOC	TOTAL PER WEB MM^2/M	CLOSED TIES MM^2/M	AXIAL FORCE TOR MN	AXIAL AS EXTRA RF MM^2	MIN Θv DEG
RF REQD	MID	1080	280	4.7	4200	30.0
FOR PURE	1/4 PT	2436	644	8.6	0	32.9
TRUSS	END	5086	1309	8.7	0	42.5
CODE AS IS	MID	1100	0 T'hld	0	0	30.0
WITH ALL	1/4 PT	3407	1701	20.9	0	34.0
ERRORS ETC	END	7031	3511	22.6	0	43.0
WIDEN TO	MID	1100	0 T'hld	0	0	30.0
INCREASE	1/4 PT	1100	0 T'hld	0	0	30.0
THRESHOLD	END	1100	0 T'hld	0	0	34.6
EXPRESS T*	MID	1100	1100	5.9 Vmin	5800	30.0
AS SHEAR	1/4 PT	2540	1340 Tmin	22.5 Tmin	0	34.2
NO VUC	END	5190	1340 Tmin	22.5 Tmin	0	43.0
EXPRESS T*	MID	1100	1100	5.9 Vmin	5800	30.0
AS SHEAR	1/4 PT	1340 Tmin	1340 Tmin	22.5 Tmin	0	34.2
WITH VUC	END	3300	1340 Tmin	22.5 Tmin	0	43.0

Figure 3 Large Box Girder Example

The following points are worth noting from this example:

- Rigidly following the Code (Ref 1) produces a very conservative design at the end and quarter points, but possibly an unconservative design at midspan.
- The most economic design would be to simply widen the lower flange about 900 mm, which means all torsion would be below the threshold, torsion could be completely ignored, and full Vuc may be counted.
- The minimum torsion reinforcement $0.2y1/fsy.f$ looks a bit high and demands a substantial extra axial force capacity, although it did not affect this example particularly adversely. The various code committees should review this.
- If the Code (Ref 1) was changed as suggested in this paper, the quantities with T^* expressed as shear, and including Vuc would be used, which are much more economical and rational than the present Code (Ref 1).
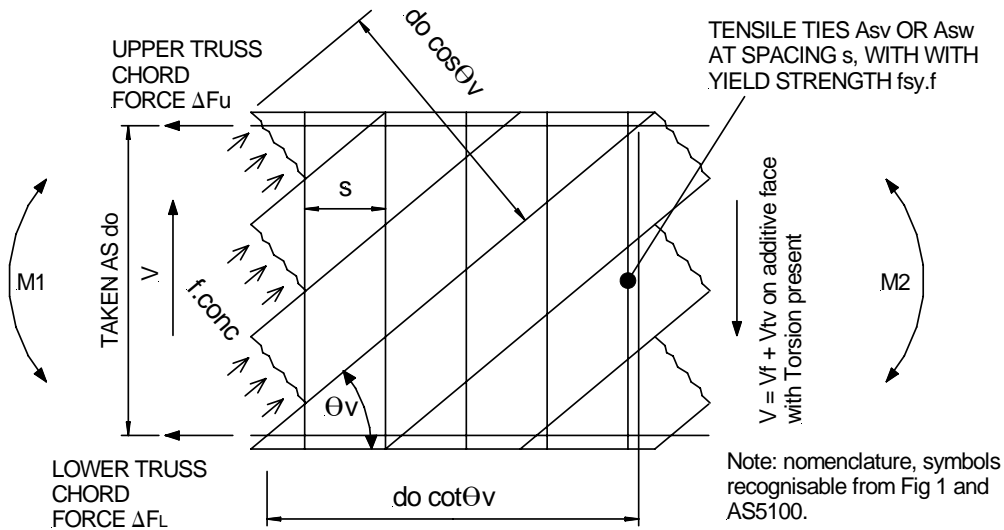
5. CONCLUSIONS

The following conclusions may be drawn from what has been presented in this paper:

- The Bridge Code (Ref 1) contains a number of errors and shortcomings in the torsion clauses, which can lead to either unconservative or conservative design, but virtually never to a correct design.
- The following Table of changes could or should be made to AS5100. Fuller explanation is given in the main text:

Clause No	Proposed Change
8.2 and 8.3	In general, obtain shear from T^* as $T^*do/(2Am)$ and add to flexural shear. Then design for flexure and torsion shear simultaneously.
8.1.8.2	Base D-shift on $d_o\cot\theta v$.
8.1.8.3	End anchorage of horizontal shear beyond half width of bearing, based on $d_o\cot\theta v$.
8.2.2	Disagree that Vuc = 0 with torsion but if AS5100 to retain this concept, need forward warning in Clause 8.2.2
8.2.6	Include $\sin 2\theta v$ term in calculation of Vu.max
8.2.7.2	Vuc should be allowed with torsion and this will affect calculation of Vo or Vt, by inclusion of axial torsion tensile force.
8.2.10	A range of θv values should be allowed, with a minimum based on the Vu.min to Vu.max range. Minimum θv value must reflect flexure and torsion shear simultaneously i.e. add them together.
8.3.3	Allow for Tu.max by adding torsion shear to flexure shear in Vu.max formula at Clause 8.2.6.
8.3.4	Disagree with "threshold T^* ignore" approach but if retained, base on T^* calculated with uncracked stiffness.
8.3.4 (b)	Interaction formula is wrong. Delete it. This is handled by adding flexural and torsion shear together in Clause 8.2 generally.
8.3.5 (b)	"θt" should be "θv"
8.3.6	AS5100 should incorporate longitudinal torsion tensile force directly instead of as calculated reinforcement areas.
8.3.7	Review/ clarify minimum torsion reinforcement rule.

- All Codes and sources reviewed agree on the truss model for steel strength contribution (see Appendix).
- There are widely varying opinions as to the nature of "concrete contribution" reflected in the Eurocode (Ref 9) – the "flat truss" approach, and the ACI (Ref 10) – the "shear friction" approach, similar to Australian models. Neither of these Codes distinguish between shear and torsion in accepting the "concrete contribution", as the Bridge Code (Ref 1) does. This is ironic because, in the author's opinion, the Bridge Code (Ref 1) and AS3600 (Ref 11) have a better, more rational model for "concrete contribution" than either of the other two Codes.

6. APPENDIX – DERIVATION OF CODE TRUSS FORMULAE

UPPER TRUSS CHORD FORCE ΔFu

$do \cos\Theta v$

TENSILE TIES Asv OR Asw AT SPACING s, WITH WITH YIELD STRENGTH fsy.f

TAKEN AS do

M1

V

f.conc

Θv

$V = Vf + Vtv$ on additive face with Torsion present

M2

LOWER TRUSS CHORD FORCE ΔFL

$do \cot\Theta v$

Note: nomenclature, symbols recognisable from Fig 1 and AS5100.

FOR CONCRETE CRUSHING

DIAGONAL COMP FORCE = V / SINΘv, so f.conc = (V/SINΘv) / (bv doCOSΘv) = V / (bv do SINΘvCOSΘv), from which V = f.conc bv do SINΘvCOSΘv = 0.5 f.conc bv do SIN2Θv. SETTING f.conc TO A MAXIMUM = 0.4f'c gives Vu.max = 0.2 f'c bv do SIN2Θv   (Note that SIN2Θv taken as 1.0 in Australian Standards)

CONSIDER TORSION SHEAR SEPARATELY, USING SAME CRITERION
Vu.max.tor = 0.2 f'c bw do SIN2Θv = Tu.max do / (2Am), so Tu.max = 0.2 f'c (2Am bw) SIN2Θv, i.e.
Tu.max = 0.2 f'c Jt SIN2Θv

FOR TIES

NUMBER OF TIES CROSSED BY COMP STRUTS = do COTΘv/s, so Vus = Asv (or Asw)fsy.f (do COTΘv/s), i.e.
Vus = (Asv fsy.f do/s) COTΘv

CONSIDERING TORSION SHEAR SEPARATELY, USING SAME CRITERION
Vus.tor = (Asw fsy.f do/s) COTΘv = Tus do/ (2At), from which
Tus = fsy.f (Asw/s) 2At COTΘv

UPPER AND LOWER CHORD FORCES

FOR FLEXURAL SHEAR, M1 - M2 = ΔM = Vf (do COTΘv) = ΔFL (or ΔFu) do i.e.
ΔFL and ΔFu = Vf COTΘv
Note that M1 going to M2 is moving relatively from hogging to sagging moment for the slope of the struts as shown. ΔFL and ΔFu are both tensile. ΔFL increases tension (reduces compression) from M1 to M2, while ΔFu increases tension from M2 to M1.

FOR TORSION SHEARS, At each corner there will be a tensile component from a flange or web torsion shear, interacting with a compressive component from the adjoining web or flange. The compressive strut angle means that the tensile component from say the web "leads" the compressive component from the adjoining flange, resulting in a net tensile force at each corner. For any section the interaction ensures that the total tensile force is the sum of the the tensile components of the shear forces from the torsion shears from all four shear faces. It also ensures that this force is distributed equally to the four corners. With "closed" ties, Θv is made the same on all four faces.

So Vth = w Tus/(2At), and Vtv = do (Tus/2At) i.e. flange width or web height times shear flow. Total tensile force thus = 2 Vth COTΘv + 2 Vtv COTΘv = 2 (w Tus/(2At) COTΘv + do Tus/(2At) COTΘv) = 2 (w + do) Tus/(2At) COTΘv = Ut Tus/(2At) COTΘv = Ut (fsy.f (Asw/s) 2At COTΘv)/(2At) COTΘv = fsy.f (Asw/s) Ut COT^2 Θv, half each to flexural tensile zone, 0.5 fsy.f (Asw/s) Ut COT^2 Θv and
flexural compressive zone, 0.5 fsy.f (Asw/s) Ut COT^2 Θv

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