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Grouted Dowels using AS5216:2021 - Their Capacities and Suitability for Temporary Movement Joints

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R2	27-04-2026	Service, tension and conclusion chapters significantly amended and other general wording changes	DM

Abstract

The use of proprietary grouted dowel sleeve systems across Temporary Movement Joints (TMJs) in concrete floors has increased in Australia, often supported by capacity data that does not explicitly demonstrate compliance with AS5216:2021 – Design of post-installed and cast-in fastenings in concrete. This paper assesses the ultimate and serviceability capacities of grouted dowels designed in accordance with AS5216, with particular emphasis on combined shear, bending and axial tension.

Dowels spanning a TMJ may develop plastic hinges under conditions of shear, with capacity governed by interaction between bending and axial tension. Both N-grade reinforcing steel and stainless-steel dowels are considered. The analysis demonstrates that maximum shear capacity coincides with negligible reserve tension capacity, and vice versa, significantly reducing available resistance compared with shear-only assumptions.

Using an example, the paper further examines axial tension that develops in dowels after TMJs are locked by grouting, arising from restrained concrete shrinkage, post-tensioning precompression loss and cracking behaviour. A staged analytical approach indicates that, for slabs restrained between large stiff structural elements, tension forces may dominate dowel design and frequently exceed the capacity of practical dowel arrangements. The study concludes that grouted dowels working in

combined shear and tension may be a potentially unsafe TMJ solution, and that alternative load-path strategies or permanent movement joints should be considered.

Introduction

The market in Australia is becoming saturated with grouted dowel sleeve products that offer short term shrinkage relief in concrete floors with some suppliers reporting capacities that seem unrealistic using simple theory. This report calculates the ultimate capacities of two different grouted dowel solutions based on the Australian standard AS5216:2021 Design of Post-Installed and cast-in fastenings in concrete [1] and is aimed at those trying to determine dowel capacities for compliance with AS5216. Using an example of an application in a Post Tensioned concrete slab, this report also intends to address their suitability for use in Temporary Movement Joint (TMJ's). AS5216 is referenced to in clause 19.3.2 in AS3600 [2], as the basis for the design of Post-Installed and Cast-in fastenings. Australian dowel sleeve suppliers or manufacturers that are providing capacity tables in their brochures, do not appear to claim compliance with AS5216, which is concerning.

This report concentrates only on the steel strength of the dowel and not the bearing or breakout force of the concrete which may be more onerous particularly where the shear load is high and the concrete element is thin. To calculate these capacities, the reader is referred to the AS5216 for which the grade of concrete and thickness of slab will be relevant to determine the ultimate capacity of the complete system. AS5216 typically directs guidance to AS3600 for the design of supplementary reinforcing to enhance the breakout characteristics of the edge in accordance with clause 3.4.2.1. Reinforcement should only be considered to enhance breakout loads when it acts across the concrete breakout plane parallel to the direction of the force as per AS5216 figures 3.4.2.1 (g) & (h) and figures 3.4.3.1 (e) & (f).

The two dowel materials compared are an 'N' grade 500 reinforcement bar and a Stainless-Steel Bar Grade 316, both with a diameter of 24mm. Although the characteristic ultimate and yield strengths are well documented for N reinforcement grade, the grade of 316 stainless-steel is less controlled and varies based on the manufacturing method and the supply chain. We have assumed a high characteristic yield and characteristic tensile strength for Stainless Steel 316 in the calculations of 415MPa for yield and 620MPa for tensile strength. We believe these values represent the uppermost limit of a likely available Stainless-Steel 316 bar supply. For those manufactures providing stainless-steel dowel solutions, full disclosure of the characteristic material strengths allows for checking of their capacity claims. Where disclosure is opaque, design engineers and product manufacturers/suppliers should understand the risk.

Theory and Code

The capacity of the dowels will be determined by the interaction of all stresses on the system at the critical point. That is the shear, tension and bending stresses at the plastic hinge. The following two sections address these interactions.

Bending and Shear

The design of dowels that span across a gap is well understood and the value of the maximum bending moment can be quantified using the method of virtual work, Ie the energy absorbed by the system equals the work done on the system. All structural engineers should be aware of the approach through their engineering studies as the subject is elemental and is usually taught in the initial stages of their degree. See figure 1 below.

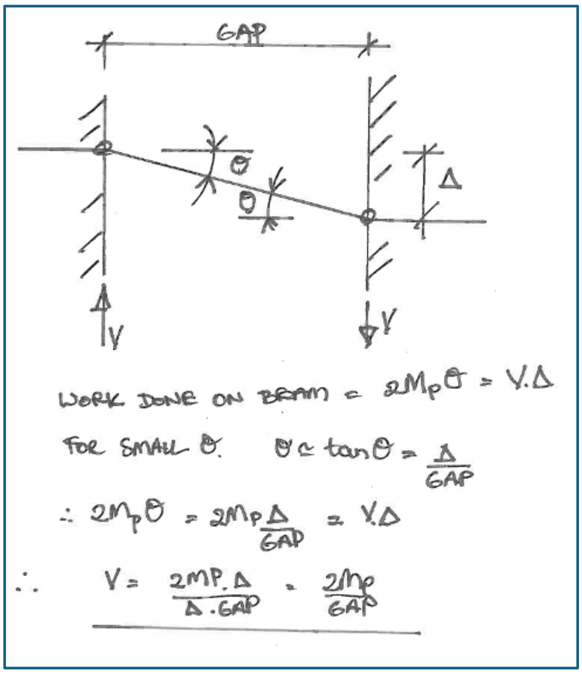

Figure 1 - Beam spanning a gap - Method of Virtual Work

From above, the shear to cause plastic hinges in the beam is equal to 2 times the Plastic Moment capacity divided by the gap. In the case of a dowel, it should be recognised that concrete at the face providing the bearing seat under the dowel, will receive significant stresses and be unreliable in fully supporting the dowel at the face of the joint. As such the effective gap forming the bending moment is wider than the gap between the slab edges and the plastic hinge will occur just inside of the face of the concrete edge. The effective gap becomes the joint width plus the ineffective support distances inside the face of the concrete.

Clause 4.2.2.4 of AS5216, addresses capacity for dowels in shear that are projecting from one face of the concrete and develop only one plastic hinge. The formula in the above figure 1 needs to be adjusted for the method provided in this clause and demonstrated by Figure 4.2.2.4(a) of AS5216. This is achieved by dividing the concrete gap by 2 so that e1 in figure 4.2.2.4(a), is equal to Slab Gap (TMJ Gap) divided by 2. This clause also introduces a small additional lever length a_3 to account for the concrete crush or lack of reliable support at the concrete face and what should be the softening of the negative bending moment over the bearing length beyond both the face of the support and the distance a_3. This value a_3 in AS5216 is equal to the bar diameter divided by two. The value α_M in this clause equals one for grouted dowels since there is no fixed restraint applied to the bar outside the face of the concrete midway between the two faces of the joint.

The equation for the maximum design ultimate shear force in the dowel (formula 4.2.2.4 in AS5216) can be rearranged with la = (Joint Gap+bar diam)/2 and α_M=1.

$$V^* = \frac{Mp^*}{l_a} = \frac{Mp^*}{\frac{Dia}{2}+\frac{Gap}{2}} = \frac{2Mp^*}{(Dia+Gap)} \tag{a}$$

The only item in the above formula that is introduced by AS5216 is the Diameter/2 allowance of a_3. Otherwise, the formula represents statics derived by the principle of virtual work. Without the a_3=Dia/2 consideration in the above formula, the denominator would become “Gap”, which is the same formula in Figure 1.

Some may recognise that there is an opportunity to increase dowel capacities by supporting the edge of the concrete, at least on the dowel sleeve side to reduce the effects of a bearing crush or local spall distance a_3. The author suggests such an approach will need extensive testing, and we must recognise that results will come from laboratory tests. In the laboratory, compaction will be optimal and can be expected to be better than that performed at site. Such an approach to achieve a small influence on the shear capacity is not recommended as bearing loads closer to the slab face still need to be resolved back through the cover to the vertical U-bar legs adjacent to the dowel.

The capacity reduction factors to apply to V* in equation (a) above, come from table 3.2.4 of AS5216. With permission of Standards Australia, the table is reproduced in part below (Table 1)[1].

Table 3.2.4 — Capacity reduction factors for modes of failure of post-installed and cast-in fasteners

Mode of failure	Capacity reduction factor		
Steel failure — Fasteners			
Tension	ϕ_{Ms}	=	$5f_{yf}/(6f_{uf}) \leq 1/1.4$
Shear — with and without lever arm	ϕ_{Ms}	=	$f_{yf}/f_{uf} \leq 0.8$ when $f_{uf} \leq 800$ MPa and $f_{yf}/f_{uf} \leq 0.8$
		=	$2/3$ when $f_{uf} > 800$ MPa or $f_{yf}/f_{uf} > 0.8$

Table 1: Extract from AS5216:2021 Capacity factors for steel failure

Tension Shear and Bending Interaction

The tension capacity of the dowel is addressed in clause 7.2.2.3 of the AS5216, "Shear Force with Lever arm". This clause makes the above equation (a) redundant when tension occurs. In un-grouted and more onerously grouted dowel bars in TMJ's, tension should always be understood to occur.

For the case of the dowel being un-grouted, tension can be estimated by the coefficient of friction between the dowel and the sleeve. This tension arises due to shear load and the friction generated through opening of the temporary movement joint. AS3600 suggests a coefficient of friction of 0.2 for tendon friction in pt ducts and a minimum coefficient of 0.2 in clause 12.3(b) for corbels. We would caution that no dowel sleeve manufacturer has included friction coefficients in their literature so checking of any provided capacity tables is made harder. It is likely that an 'N' deformed bar will have a higher friction coefficient than a smooth round stainless-steel bar. In this report and without a coefficient of friction being published by dowel sleeve manufacturers, a coefficient of 0.2 is adopted under ultimate load conditions for both dowel types in un-grouted conditions.

The relevant clause 7.2.2.3 is extracted from AS5216 as follows[2]:

7.2.2.3 Shear force with lever arm

Where the fastener is considered to include a lever arm (see Clause 4.2.2.3) the characteristic steel shear strength to steel failure ($V_{Rk,s,M}$) shall be calculated as follows:

$$V_{Rk,s,M} = \frac{\alpha_M M_{Rk,s}}{l_a} \quad 7.2.2.3(1)$$

where

α_M = parameter accounting for the degree of restraint, of the lever arm, given in Clause 4.2.2.4

l_a = length of lever arm as illustrated in Figure 4.2.2.4

$$M_{Rk,s} = M^0_{Rk,s}\left(1 - \frac{N^*}{\phi_{Ms} N_{Rk,s}}\right) \quad 7.2.2.3(2)$$

N^* = resultant design tensile load applied to a fastener or group of fasteners

$N_{Rk,s}$ = characteristic tensile strength of a fastener to steel failure

The basic characteristic flexural strength of the fastener ($M^0_{Rk,s}$), the characteristic tensile strength of the fastener to steel failure ($N_{Rk,s}$) and the material capacity reduction factor for steel failure (ϕ_{Ms}) shall be determined in accordance with Appendix A.

$N_{RK,S}$ comes from clause 6.2.2 which cross references AS4100 [3] and specifies the capacity reduction factor to use is from Appendix A. For materials not covered by AS4100 reference is made back to table 3.2.4 above. One of the references in Appendix A is the "EAD 330232-02-0601 (Sept 2024) Mechanical Fasteners for Use in Concrete" [4]. The capacity reduction factors for both shear and tension in the EAD document (section 2.2.9.2) match the capacity reduction factors for AS5216.

$M^0_{RK,S}$ is defined as the "reference characteristic flexural strength of a fastener". This is formula is defined in the same EAD document under clause 2.2.7.1 or the alternative document "EAD 330499-01-0601 Bonded Fasteners for Use in Concrete (July 2017)" [5] under a similar clause 2.2.7.1. Either formula from these EAD documents, limit the reference characteristic flexural moment to 1.2 times the elastic section modulus times the ultimate tensile stress.

$$M^0_{Rks} = 1.2 \times Z \times f_{uf} \quad \text{(b)}$$

The author would caution that if the bar material does not demonstrate adequate ductility (strain after yield), using the ultimate tensile strength may be unconservative in

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bending. The capacity reduction factor for shear does rectify this in replacing f_{uf} with f_{yf} once the factor is applied.

On combining the two formulas of clause 7.2.2.3 and applying the capacity reduction capacity factor $\emptyset_{Ms}$, we can arrange the following expression for the maximum ultimate design shear, V^*.

$$V^* = \emptyset_{Ms}\left(\frac{\alpha_M}{l_a}\right)M^0_{Rks}\left(1-\frac{N^*}{\emptyset_{Ms}N_{Rks}}\right) \quad \text{(c)}$$

As per clause 8.1.1 in AS5216:2021, the interaction of shear and tension is undertaken in the above equation of 7.2.2.3(2) for dowels subject to shear loads with a lever arm. With respect to axial load, gap width or material strength, the above expression (c) is linear and the dowel capacity is inversely proportional to la, i.e. double the la distance and the shear capacity is halved. The first capacity reduction factor $\emptyset_{Ms}$ is for dowel in shear and the second in the denominator is the capacity reduction for dowel in tension. Unfortunately, AS5216 does not distinguish between the two factors with a unique subscript so care must be exercised with the application of the respective capacity reduction factors. The value of α_M, equals 1 as there is no moment restraint to the dowel at the centre of the joint.

The capacity reduction factor is dependent on the yield and tensile strengths of the different steels, thus varies across the two different dowel materials. This capacity factor is calculated in the spreadsheeted values of Table 2 below based on the formula in table 3.2.4 of AS5216.

In table 2 for ULS capacities, the green row is the capacity with 20% friction on the dowel load. This is the expected capacity before grouting. The pink row is without friction and should not be used as there is no situation that occurs without tension. Shear is calculated through a range of tension loads. The values in table 2 reflect the interaction between tension and shear on the dowels. Figures 2 & 3 illustrate the values from table 2 and the linear relationship of the interaction between tension and shear for any joint width. Based on the results of the calculations in the spreadsheet, a friction of 20% has only a very minor effect on the shear capacity. When the maximum shear occurs, there is no reserve capacity for the dowel to take tension. This should be a concern for those that choose to use dowels rated for their maximum shear and ignore the significant tension loads that may develop after the dowels are grouted.

The interaction should come as no surprise to engineers that are familiar with bolt design in AS4100. An interaction between shear and tension is the basis for bolt design, however in bolt design there is no consideration for bending, as bolts are not expected to bridge gaps over the shear interfaces. In AS4100 cl 9.4.3 Pin in Bending, there is a design method for bending on a pin, but unfortunately the interaction with tension is not included. Introducing tension into this clause would be necessary should the pin have tension. Note the recommendations of Gorenc et Al [6, p. 258] on determining bending moments for pins are conservatively calculated taking the full width of the all the plies rather than the distance between the centre line of the outside plies. This conservative recommendation should be considered for those that wish to still try to reduce the effective gap width by reducing a_3 arguing a stiff dowel sleeve as mentioned above.

Note although reference to shear is made in the clause title and throughout this document, the check on the ultimate capacity of the dowel is not a dowel shear check but a bending check in combination with tension. As bending stress and tension stress act in the same direction longitudinal to the dowel axis, the interaction formula is additive rather than the shear versus tension approach for bolts in AS4100 where the interaction under ultimate conditions is parabolic due to shear planes being in contact.

On using the ULS capacity table 2 particularly for the N grade bar, care should be exercised in determining the gap width as we are aware some dowel sleeves have a seal or unique geometry which cannot effectively provide support for the dowel at the edge. Such seal widths or geometry need to be added to the estimated joint width.

Dowels to AS5216:2021 - Ultimate Capacity

	SS 316 Grade	N Bar G500	
Dowel Bar f_{yf}	415	500	Mpa
Dowel Bar f_{uf}	620	540	Mpa
AS5216:2018 Cl 3.2.4			
Steel Capacity Reduction Factor Tension Φ_{ms}	0.56	0.71	
Steel Capacity Reduction Factor Shear Φ_{ms}	0.67	0.67	
Allowance distance for bearing typ 0.5xDiameter (a3/dia)	0.50	0.50	

SS 316 Grade									
Diameter	24	mm							
M^{o}_{RKS}	1.01	KN.m							
N^{o}_{RKS}	280.5	KN							
Max Shear (kN)	187.7	kN	cl 7.2.2.2 k7=1						
Max Tension (kN)	156.5	kN	cl 6.2.2						
Joint Gap (mm)	0	5	10	15	20	25	30	35	40
Effective Span of dowel (mm)	24	29	34	39	44	49	54	59	64
V*max (N*=0) (kN)	56.3	46.6	39.8	34.7	30.7	27.6	25.0	22.9	21.1
V*with (N*=.2V*)(kN)	52.5	44.0	37.8	33.2	29.6	26.6	24.3	22.3	20.6
N* @ .2V*	10.5	8.8	7.6	6.6	5.9	5.3	4.9	4.5	4.1
V* at N* = 10kN	52.7	43.6	37.2	32.4	28.8	25.8	23.4	21.4	19.8
V* at N* = 20kN	49.1	40.7	34.7	30.2	26.8	24.1	21.8	20.0	18.4
V* at N* = 30kN	45.5	37.7	32.1	28.0	24.8	22.3	20.2	18.5	17.1
V* at N* = 40kN	41.9	34.7	29.6	25.8	22.9	20.5	18.6	17.1	15.7
V* at N* = 50kN	38.3	31.7	27.1	23.6	20.9	18.8	17.0	15.6	14.4
V* at N* = 60kN	34.7	28.7	24.5	21.4	18.9	17.0	15.4	14.1	13.0
V* at N* = 70kN	31.1	25.8	22.0	19.2	17.0	15.2	13.8	12.7	11.7
V* at N* = 80kN	27.5	22.8	19.4	16.9	15.0	13.5	12.2	11.2	10.3
V* at N* = 90kN	23.9	19.8	16.9	14.7	13.0	11.7	10.6	9.7	9.0
V* at N* = 100kN	20.3	16.8	14.3	12.5	11.1	10.0	9.0	8.3	7.6
V* at N* = 110kN	16.7	13.8	11.8	10.3	9.1	8.2	7.4	6.8	6.3
V* at N* = 120kN	13.1	10.9	9.3	8.1	7.2	6.4	5.8	5.3	4.9
V* at N* = 130kN	9.5	7.9	6.7	5.9	5.2	4.7	4.2	3.9	3.6
V* at N* = 140kN	5.9	4.9	4.2	3.6	3.2	2.9	2.6	2.4	2.2
V* at N* = 150kN	2.3	1.9	1.6	1.4	1.3	1.1	1.0	0.9	0.9
V* at N* = 155kN	0.5	0.4	0.4	0.3	0.3	0.3	0.2	0.2	0.2

N Bar G500									
Diameter	24	mm							
M^{o}_{RKS}	0.88	KN.m							
N^{o}_{RKS}	244.3	KN							
Max Shear (kN)	162.9	kN	cl 7.2.2.2 k7=1						
Max Tension (kN)	174.5	kN	cl 6.2.2						
Joint Gap (mm)	0	5	10	15	20	25	30	35	40
Effective Span of dowel (mm)	24	29	34	39	44	49	54	59	64
V*max (N*=0) (kN)	48.9	40.4	34.5	30.1	26.6	23.9	21.7	19.9	18.3
V*with (N*=.2V*)(kN)	46.3	38.6	33.2	29.1	25.9	23.3	21.2	19.4	17.9
N* @ .2V*	9.3	7.7	6.6	5.8	5.2	4.7	4.2	3.9	3.6
V* at N* = 10kN	46.1	38.1	32.5	28.3	25.1	22.6	20.5	18.7	17.3
V* at N* = 20kN	43.3	35.8	30.5	26.6	23.6	21.2	19.2	17.6	16.2
V* at N* = 30kN	40.5	33.5	28.6	24.9	22.1	19.8	18.0	16.5	15.2
V* at N* = 40kN	37.7	31.2	26.6	23.2	20.5	18.4	16.7	15.3	14.1
V* at N* = 50kN	34.9	28.8	24.6	21.5	19.0	17.1	15.5	14.2	13.1
V* at N* = 60kN	32.1	26.5	22.6	19.7	17.5	15.7	14.2	13.0	12.0
V* at N* = 70kN	29.3	24.2	20.7	18.0	16.0	14.3	13.0	11.9	11.0
V* at N* = 80kN	26.5	21.9	18.7	16.3	14.4	13.0	11.8	10.8	9.9
V* at N* = 90kN	23.7	19.6	16.7	14.6	12.9	11.6	10.5	9.6	8.9
V* at N* = 100kN	20.9	17.3	14.7	12.8	11.4	10.2	9.3	8.5	7.8
V* at N* = 110kN	18.1	14.9	12.7	11.1	9.8	8.8	8.0	7.3	6.8
V* at N* = 120kN	15.3	12.6	10.8	9.4	8.3	7.5	6.8	6.2	5.7
V* at N* = 130kN	12.5	10.3	8.8	7.7	6.8	6.1	5.5	5.1	4.7
V* at N* = 140kN	9.7	8.0	6.8	5.9	5.3	4.7	4.3	3.9	3.6
V* at N* = 150kN	6.9	5.7	4.8	4.2	3.7	3.4	3.0	2.8	2.6
V* at N* = 160kN	4.1	3.4	2.9	2.5	2.2	2.0	1.8	1.7	1.5
V* at N* = 170kN	1.3	1.0	0.9	0.8	0.7	0.6	0.6	0.5	0.5

Table 2 Dowel Ultimate Capacity tables.

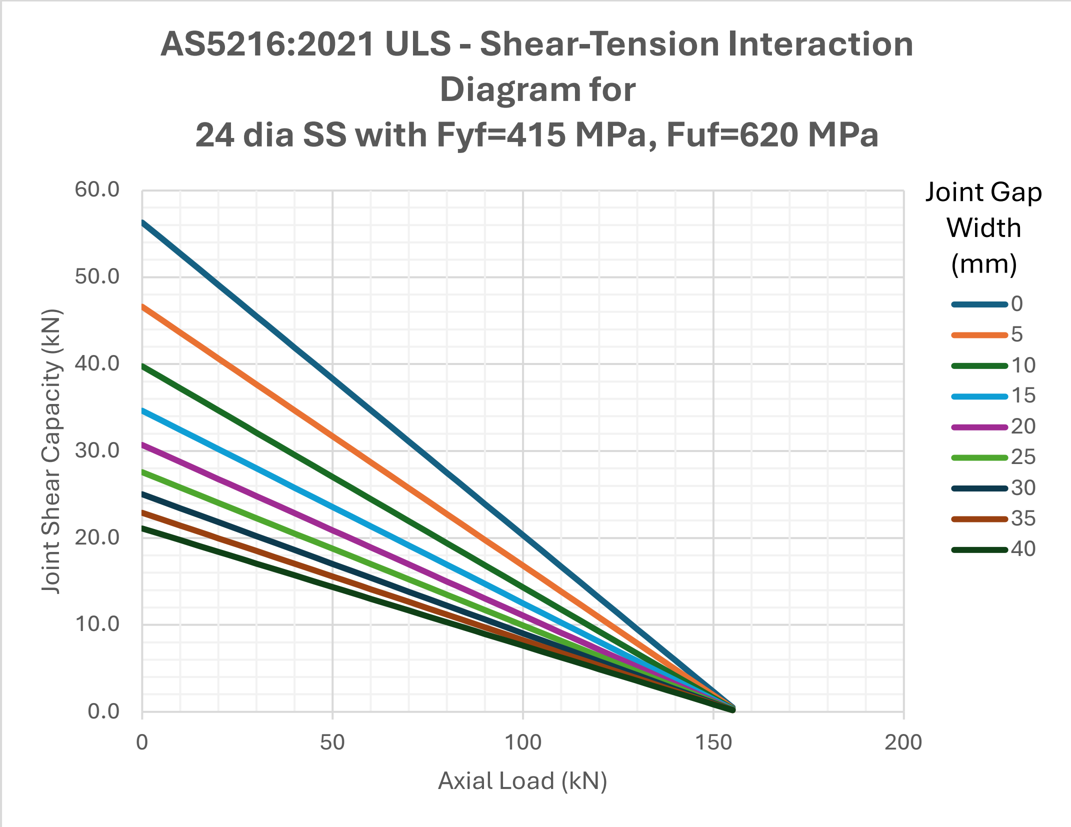

Figure 2 - ULS Interaction Diagram for 24mm SS Bar with assumed material properties.

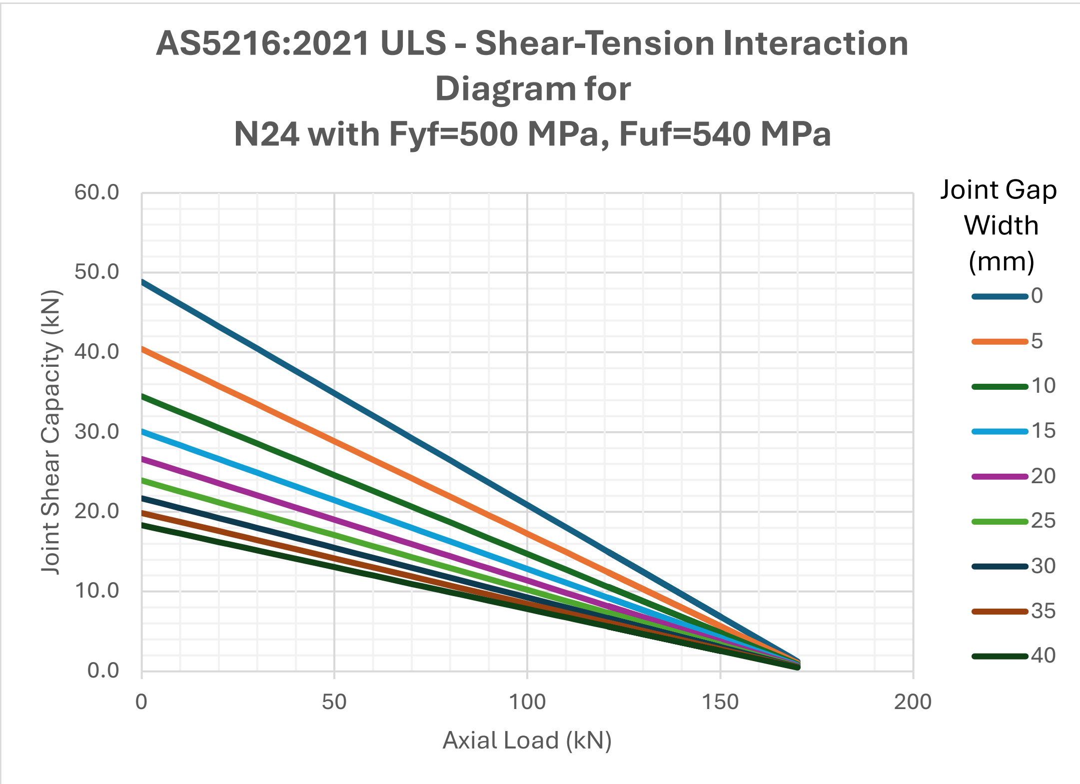

Figure 3 - ULS Interaction Diagram for N24mm Bar

After Grouting and Floor Tension

Floors that may be perceived to benefit from a delayed grouted joint or pour strip are those floors that have significant separated stiff restraints such cores, buttressed walls, long blade walls, very stiff columns or a combination of any of these close to a floor or foundation providing lateral restraint against shrinkage. The delay provided by the TMJ in floors bridging restraints, provides temporary shrinkage relief thereby avoiding short term tension loads. The TMJ joint is usually specified to be grouted between 30 and 56 days, with most engineers preferring the later age but builders preferring earlier ages particularly as the building nears completion and finishes trades need unrestricted access to the floors. Often there is a compromise with the result not necessarily changing the performance of the floor particularly in areas between large stiff restraints.

For those considering TMJ's, they should be aware that there is still a significant shrinkage strain that needs to be accounted for after the grouting of the dowels and substantial tension loads will develop after grouting of such joints between heavy unyielding restraints. For a Post Tensioned 250 thick floor plate, it is reasonable to expect magnitudes of long term restrained shrinkage to be more than 200 micro-strains after 56 days in an internal environment.

After grouting of the TMJ, a modest elastic precompression of 30 micro-strains under a PT load of 1MPa will likely to be lost quickly between very stiff restraints. For such stiff unyielding restraints, after the dowels are grouted, the dowel forces will build up to carry the PT load plus the force that is needed to form the first crack. After the first crack, the load will relax to the PT load plus the load of any reinforcement that bridges the crack. If enough reinforcement is provided throughout the remaining of the uncracked slab, further cracks will develop and a general relaxation of the tension loads will keep occurring, but the load on the dowels will not reduce less than the precompression load plus the yield strength of the reinforcement crossing the crack. Estimating shrinkage loads is not straightforward but can be undertaken by the following strategy. The author expects the strategy will be considered both conservative and unconservative across the range of peers that review this document, but tension should not be ignored or underestimated.

For our example below, we will assume a PT slab of thickness of D=250mm, stiff unyielding restraints, an effective long term precompression load of 1MPa and a characteristic strength at both 28 days and 56 days of 40MPa (56-day strength assumption is unconservative as it is likely to be significantly higher). After grouting we can expect loads to develop on the dowels, with magnitudes estimated in the following steps.

Step 1). PT Load

The PT load is the effective stress in the slab times the cross section. Once the shrinkage strain exceeds the precompression strain, the full precompression load will be transferred to the dowel. If the effective long-term stress is 1MPa then the Load on the joint when the precompression stains are lost becomes:

$$PT\ Load = \frac{P}{A} \times bD \times 10^{-3} = \mathbf{250kN/m}$$

Step 2). The force to cause the 1st crack in the slab.

After the precompression load is lost, a tension load in the concrete will begin to develop until a crack forms. The force to produce the first crack will be determined by the weakest concrete. The cracking strength of the concrete can possibly be assumed to be represented by the characteristic tensile strength of the concrete ($f'_{ct}=0.36\sqrt{f'_c}$) and perhaps reduced for flexure by 25% according to the implied factor in AS3600 cl9.5.3.2(b). All slabs will have inherent weaknesses and are unique. If no bridging reinforcement is across the crack, then no other cracks are likely. The crack will widen as the bond between the pt duct, and the concrete is poor and tendons are widely spaced. We will assume the pt load at the crack does not increase with a widening of the crack which is conservative in determining reinforcement estimates to control crack widths, however if no reinforcement is added across the 1st crack, the pt load must increase as the crack widens. If enough reinforcement bridges the crack and does not yield, then the crack width should stay controlled.

Ignoring the minor contribution of un-tensioned steel to reducing the cracking strength and ignoring it in steps 1 to 4, the maximum force we can expect to add to item 1 just prior to the first crack and using the characteristic tensile strength is:

$$\begin{aligned} 1st\ Crack\ Load &= 0.75 \times .36\sqrt{f'c} \times bD \times 10^{-3} \\ &= 1.71bD \times 10^{-3} \\ &= 426\ kN/m \end{aligned}$$

Therefore, our maximum Load that we realised prior to the crack is now:

$$PT\ Load + 1st\ Crack\ Load = \ 250 + 426 = \mathbf{676\ kN/m}$$

If no reinforcement bridges the crack, the slab restraint force will relax back to the minimum PT load of 250kN/m plus a small additional PT load due to uncontrolled widening of the crack. As we don't want wide cracks in our structure, hopefully there is reinforcement in the slabs to control crack widths and achieve more controlled cracks. One wide uncontrolled crack is not desirable and may cause alarm and serviceability issues.

Step 3). Increase in force due to reinforcement bridging the 1st crack.

If reinforcement bridges the crack, then the relaxed force reduces to the value of $PT\ Load + Asfy$. If we have 0.1% of N grade reinforcement through the region of the first crack, then we can expect the load will relax down to:

$$PT\ Load + Asfy = 250 + \frac{0.1}{100} \times fsy \times bD = 250 + 125 = \mathbf{375 kN/m}$$

Cracking progresses with further shrinkage and the next crack will not find the weakest concrete. It will find the next slightly stronger concrete and if we have provided enough reinforcement at each crack to prevent wide cracks, this procedure will continue until we have exhausted our shrinkage. Each crack being stronger than the preceding crack. Gilbert [7] provides Figure 4, that illustrates this strengthening as the numbers of cracks increases.

We should consider using at least the mean uniaxial tensile strength in our maximum cracking force and not the lower characteristic value. One could argue that the maximum characteristic tensile strength should be conservatively used, however the author does not have that opinion since we have removed approximately 50% of the shrinkage through the 56-day TMJ and are expecting a lower quantity of cracks to occur. In AS3600 the mean uniaxial tensile strength is estimated at 1.4 times the characteristic uniaxial tensile strength. The problem is accurately determining the cracking strength for concrete at 56 days (lockup age) and beyond, particularly with post tensioned concrete mixes. Post-tensioned concrete strengths at 56-days, usually far exceed the design 28-day strength of Normal grade mixes due to the concrete being a special class mix. For winter concrete mixes, it is not uncommon to expect higher cement ratios in the mix to achieve a necessary 4- or 5-day stressing strength in cold conditions with the concrete supplier expected to be making dynamic adjustments to the mix design if needed with feedback from the crushing tests. Accurate in-situ cracking strength is almost impossible to accurately predict, and the estimate of uniaxial strength should not be understood to be precise.

Step 4). Increase in force due to the mean tensile strength.

Applying the mean to our formula for the much later and possibly last (nth) crack when the mean tensile strength is dominant, results in:

$$\begin{aligned} nth\ Crack\ Load &= 0.75 \times 1.4 \times .36\sqrt{f'c} \times bD \times 10^{-3} \\ &= 2.39bD \times 10^{-3} \\ &= 598\ kN/m \end{aligned}$$

Adding to the PT load we have a maximum load of:

$$PT\ Load + nth\ Crack\ Load = 250 + 598$$
$$= \mathbf{848\ kN/m}$$

This load builds up gradually and significant relaxation after the crack will be instantaneous, however the nth crack cannot occur if low reinforcement rates have been provided. With 1% reinforcement bridging the 1st crack, the relaxed load will be 375kN/m with the maximum load of 676kN/m as per step 2. The load prior to the 1St crack will govern and the 1st crack will widen uncontrolled.

If there was 2% of reinforcement crossing the 1st crack and generally throughout the floor then the load restraining the 1st crack would become 500kN/m, still below nth crack load. The width of the first crack will still be uncontrolled.

Step 5). Provide a sufficient and justified reinforcement quantity in the slab to control cracking without being too conservative.

It is normally desired to have many fine cracks form instead of a couple of wide cracks after the precompression load is lost. For this to occur then significant reinforcement to control cracking needs to be added.

The amount of reinforcement to control the nth crack (last crack)at the concrete's mean assumed tensile strength can be calculated as follows where '*p*' represents the proportion of reinforcement (As/Ac). Caution again as we are using 28-day concrete strength estimates instead of higher 56-day estimates.

$$PT\ Load + nth\ Crack\ Load = Pt\ Load + Asfsy$$

$$nth\ Crack\ Load = As \times fsy = pbD \times fsy$$

$$0.75 \times 1.4 \times .36\sqrt{f'c} \times bD = pbD \times fsy$$

At 40MPa, and fsy = 500 MPa,

$$2.39bD = pbD \times 500$$
$$2.39 = p \times 500$$
$$p = \frac{2.39}{500} = 0.48\% \ \{\text{at } 50\text{MPa}\ p = 0.53\%\}$$

We could reason that this is a general approximate lower bound reinforcement percentage for the slabs in the heavy restraint zones between the joint and the restraint. The tension load on the dowels is now:

$$Dowel\ Load\ per\ meter = \left(\frac{P}{A} + 0.0048fsy\right) D \times 10^{-3}\ \ kN$$

For this example, with D at 250mm thick and P/A=1MPa, the designed dowel load in tension after cracking is

$$Dowel\ Load\ per\ meter = (1 + 0.0048 \times 500) \times D \times 10^{-3}$$

$$= 3.4D \times 10^{-3}$$
$$= \mathbf{850\ kN/m}$$

In addition to the dowels required for vertical shear across the gap, we could expect approximately another 5.4 (850/156.5) numbers of Stainless-Steel Dowels per meter and 4.9 (850/174.5) numbers of N Grade bars per meter (based on maximum allowable Service load of Ns without shear, see table 5 below).

Given the width of these components, the extra numbers required for shear and the requirements for U bars to strengthen the edge, a grouted dowel solution working in shear would exhaust all available room and be practical.

There is conservatism in the solution above because there is no allowance for the quantity of reinforcement that is provided (0.48%) to reduce the tensile cracking strength of the concrete. Reinforcement has a restraining effect and should be allowed to reduce the cracking strength. Now adopting a less conservative approach to our estimate and accounting for a reinforcement quantity of x% and an approximate Ec(long-term)/Ec(short-term) of 0.26. With 0.26 derived from $\left(1/(1 + .85 \times \emptyset cc_{(10000-5)}\right) = .26,\ \emptyset cc_{(10000-5)} \sim 3.3)$. The maximum long-term restraint from the un-tensioned steel reinforcement can be estimated to be:

With $Ec_{ST(28day)}$ of 32000 MPa, reinforcement approximately 0.5% added to the floor and fsy=500 MPa

$$Ft = \frac{Ec_{lt}}{Es} \times fsy \times \frac{X}{100}$$
$$Ft = \frac{32000 \times 0.26}{200000} \times 500 \times \frac{.5}{100}$$
$$= 0.1\ MPa$$
$$= 0.1D\ kN/m$$

Providing there is 0.5% reinforcement added in at least in the direction of restraint, this reduces the 250 thick slab force in this example by 25 kN/m to (850-25) **825 kN/m** (3.3MPa). However, the 825 kN/m estimate is still subjective to an accurate tensile strength prediction and thus has significant uncertainty. Given there is such a small influence in the restraining effects of the reinforcement, the restraining effects will be ignored for the dowel demand calculation.

Step 6). Sensibility Check (based on shrinkage estimates)

To assess our load for sensibility based on shrinkage estimates, an assessment needs to determine an approximate free opening of the TMJ joint both long term (if un-grouted for full 30 design life) and prior to lockup (grouting of the dowels at 56 days.). Assuming the TMJ lockup happened at 56 days, and our heavy restraints were 40m apart, slab depth D=250mm, an internal environment, basic shrinkage strain of 800 micro-strains and a PT load of 1MPa. From a separate analysis, and without conservatism in the shrinkage results and the restraining effects of reinforcement being ignored, we would expect the joint to open approximately 15mm at 56 days and 25mm if left unrestrained after 30 years, i.e. a difference of 10mm.

To estimate the restraining force to make a 10mm difference between 56 days and 30 years, without cracking and with the assumptions of Ec, age adjustment factor χ =0.85, $\emptyset cc = 1.8$ for creep after load applied at 56days and assuming the creep factor for tension is the same as would be that for compression, we can use the following formula. We have chosen to ignore the effects of Steel Restraint, since our shrinkage estimate of 10mm was based on no reinforcement in the cross section.

$$\frac{Force}{Area} = \frac{Ec}{(1 + \chi \emptyset cc)} \times Strain$$

$$Force = \frac{32000}{(1 + .85 \times 1.8)} \times \frac{10}{40000} * bD\ kN$$

$$= 3.2DkN/m$$

For our 250 thick slab example, this equates to 800kN/m of force. This is below but close to our tension force of 850kN/m in step 5 for concrete with 0.48% reinforcement, however we must also recognise that the joint gap widths were rounded up to the nearest 5mm. If we were a little more conservative and used a higher modulus of elasticity to represent concrete older than 28 days and a more generous shrinkage strain when determining our joint width estimate (AS3600 suggests the basic drying shrinkage strain could vary ± 30%) the estimated loads for this approach, would be significantly higher.

Using another method to check our estimates, Gilbert [7] determines a restraining force in a 150mm thick reinforced concrete slab of 1.61 MPa in example (a) of the article. Gilbert's example is not necessarily comparable to the Post Tensioned example above and Gilbert is using a slightly different approach that includes crack width and estimated crack spacing with the calculation performed relative to higher shrinkage strains that occur due to restraint at the time of pour. Adding an additional 1 MPa of PT load to Gilbert's example would suggest a restraining force of 2.61MPa Long Term on the joint is possible (2.61x250=653 kN/m). Obviously only a ballpark number as shrinkage strains, reinforcement quantities, creep, reinforcement yield, elastic modulus and cracking strength differ across Gilbert's example and what is being considered in this report. However using Gilbert's procedure and adjusting for the necessary time delay and material properties, we can derive the tension load like Gilbert in table 3 below. Note the tension load calculated in our example needs the PT precompression load added.

Comparison of Shrinkage Tension Load Determination with Gilbert's example (a)						
Parameter		**Unit**	**Gilbert's Example a (Column A)**	**This report (56day lockup, 28 day characteristic) (Column B)**	**This report (56day lockup) More Conservative Concrete Properties (Column C)**	**This report (56day lockup), More Conservative Concrete Properties, Crack Limited to 0.3mm (Column D)**
Constants						
F'c		Mpa	Unknown	40	50	50
Creep Coeff at day of lockup (Age adjusted Effective Modulus for B, C & D, χ=0.85, Ø*=1.8)			2.5	1.53	1.53	1.53
Ec		MPa	25000	32000	34800	34800
P/A		MPa	0	1	1	1
Thickness		mm	150	250	250	250
Length between restraints		m	5	40	40	40
Drying Shrinkage Strain after lockup			600.E-06	250.E-06	325.E-06	325.E-06
Locked In Shrinkage after Precompression loss			600.E-06	220.E-06	296.E-06	296.E-06
Axial Tensile Load before 1st crack (characteristic A & B, Mean C & D)		MPa	2	2.3	3.6	3.6
Reinforcement %			0.5	0.48	0.59	0.85
Es		Mpa	2.00E+05	2.00E+05	2.00E+05	2.00E+05
Fy		Mpa	400	500	500	500
Calculations	Gilbert Eq					
So, (for 40MPa mature concrete we have reduced So slightly as bond on bars is improved with higher concrete strengths)	(1)	mm	240	192	192	192
E*e	(22)	MPa	7143	12648	13755	13755
n*=Es/E*e			28.0	15.8	14.5	14.5
n.st			8.0	6.3	5.7	5.7
C1	(6)		0.033	0.003	0.003	0.003
Ncr	(12)	kN/m	161	513	812	834
σ_1	(9)	MPa	1.11	2.06	3.26	3.34
σ_{ave}	(19)	MPa	1.56	2.17	3.41	3.45
ξ	(28)		0.236	0.021	0.016	0.022
Crack Spacing s	(27)	mm	837	6238	7997	5945
m (nos of cracks)		no.	6	7	6	7
Revised crack spacing s		mm	833	5714	6667	5714
C2	(16)		0.238	0.023	0.020	0.023
Restraining Load N(∞)	**(25)**	**kN/m**	**241**	**510**	**728**	**837**
σ_{S2} (Maximum Steel Stress at Crack)	(17)	MPa	322	425	494	394
σ_{S1} (Steel Stress Away from Crack)	(15)	MPa	-76	-10	-10	-9
σ_{c1} (Concrete Stress Away from Crack)	(23)	MPa	1.99	2.09	2.97	3.43
Estimated crack width w	(30)	mm	0.31	0.34	0.56	0.30

Table 3 - Comparison with Gilbert** [7] **but adjusted for our lockup age and concrete parameters.

Our result (Column B) with the Gilbert method is 760 kN/m (250kN+510kN) of restrained shrinkage load to be applied to the dowels. If we took more conservative values (Column C) of shrinkage strain being 30% higher ($286x10^{-6}$), f'cmi=53MPa (f'c=50MPa AS3600 Table 3.1.2), E=34,800MPa and f'ct=$1.4x.36x50^{1/2}$, then the tension load would become 978kN/m (728+250) but crack width would be considered too wide at 0.56mm. Adding reinforcement (column D) to control the crack width to around 0.3mm requires 0.85% reinforcement. With 0.85% reinforcement, the load increases to 1087kN/m (837+250).

Step 7). Result (Ultimate Limit State).

If our slab example had 1kPa SDL, 3kPa LL and a 2.5m load width on the joint, our ultimate shear would be 32kN/m. Finding a solution between the two dowel types from Table 2 above (Joint Gap=15mm at lockup) and using an estimate design tension load on the joint as 850kN/m instead of the higher 1087kN/m:

Stainless Steel Dowel – 7 Nos per meter. 121 kN/dowel in tension, 55 kN/m shear capacity across the dowels.

"N" Bar Dowel – 6 Dowels per meter. 141 kN/dowel in tension, approximately 35 kN/m total shear capacity across the dowels.

Again, the number of dowels would not be a practical solution, and this assessment is heavily reliant on the predicted accuracy of the maximum tensile forces in the slab.

Step 8). Conclusion.

We have provided a simple approach to the determination of tension forces between unyielding restraints after the TMJ is locked by the grouting operation. Shrinkage loads are complex, and we do not purport to have offered an authoritative or conservative approach. Readers wishing to know more are referred to reference [8] particularly chapters 5, 6 & 7 and references [9], [10], [11] and [12]. It should be understood this report provides only ballpark tension loads for consideration and an appropriate factor of safety against the tension load should be applied. The magnitude of the FOS should reflect the certainty of prediction.

No amount of Post Tensioning in the slabs adjacent to the joint can adjust equation 9.5.3.4(a)(iii) in AS3600 across the joint and higher pt loads in the slabs adjacent to the joint, will result in larger tension loads on the dowels If the floors aren't post tensioned either side of the joint, then the tension forces will be reduced across the TMJ's.

Our steps above are summarised in below Table 4. The percentage indicates the percentage reinforcement provided in the slabs and not the joint.

Step	Tension Load on Dowels (kN/m)	Comment (All analysis has been based on 250 thick concrete, 56-day lockup, final drying basic shrinkage strain ($\varepsilon_{csd.b}$) of 800 micro strain, Interior Environment, Fsy=500MPa, F'c=40MPa, Ec_{st}=Ec_{lt}=32000MPa and P/A=1MPa).
1	250	PT Load
2	676	PT + 1st Crack
3	375	With 1% Reinforcement & relaxation. Crack(s) uncontrolled
4	848	PT + Nth Crack (crack uncontrolled)
5	850, 825	PT+.5% Reinforcement. Crack width deemed controlled, restraining effects of reinforcement both ignored and considered
6	800, 760, 978,1087	Sensibility Checks
7	Dowel Result using Step 5 loads of 850kN/m and based on ultimate shear of 32kN/m is 7no 24mm diameter SS dowels and 6N24 dowels per meter.	
8	Conclusion – To provide the required number of Dowels in the TMJ is not possible given the restriction of faceplate width and need to provide adequate reinforcement between each dowel to cater for shear and tension. Tension loads dominate the dowel solution	

Table 4 – Summary of Steps to determine Tension.

We must recognise there are also superimposed thermal effects occurring seasonally and daily and temperature changes in the floor plates may cause either tension or compression. A one-degree temperature decrease in a floor plate spanning between unyielding restraints, correlates to 0.35 MPa tension. Unyielding restraint will dictate the maximum tension loads across a TMJ. As the stiffness of the restraints reduce, so do the joint tensile forces. Tension is a significant force that may drive the quantity of dowels across a joint and should not be ignored. Tension loads need to be conservatively calculated, and the above adoption of 850kN/m may still not be considered conservative at ages greater than 56 days.

If the design tension loads are unconservative and the slab is stronger than the tension capacity of the dowels, then the fuse for failure is the dowels and not the next crack in the slab. This is an unsafe design strategy as a failure of the dowels could cause loss of amenity and result in significant remedial cost and joint uncertainty as opposed to the failure of the slab which would only be another crack.

Why haven't we seen dowels fail?

Failure has occurred, it just doesn't become widely reported and dowels have been seen to form mechanisms. The author is aware of a project where 16mm centrally placed dowels were substituted for 24mm centrally placed dowels as a cost cutting exercise. Not because of the cost of the dowel, but because of the cost of the dowel sleeve. Even without a superimposed load on the concrete floor, as soon as the formwork underneath was removed, the dowel formed a mechanism and the supported slab edge dropped significantly. Joint width at failure less than 5mm, straight after support removal. It is believed the original designed N24 dowels would not have failed under working loads but could not demonstrate an adequate reserve of strength predicted by AS5216 under the ultimate conditions and were specified without an assessment for tension. The author is thus concerned that the use of grouted dowels is becoming misunderstood, and design approaches are not being performed with tension as a significant consideration.

Serviceability Considerations

Clause 9.1 in AS5216 stipulates that the dowel remain in the elastic state under service loads which is the similar basis for the design of all steel members and connections in steel design. Yield should not occur under service loads. For dowels, the author believes a reduction of the live load to represent short term durations like deflection estimates, should not be undertaken in calculating the service load for the shear estimation. The maximum service load for shear should be the full design service load. It is also worth considering small contingencies on dead load estimates as the floor plate thickness can be underestimated due to the allowed tolerances on slab thickness and the likelihood of formwork deflection during the pour. Superimposed dead and live loads supported by the floor plate can also be hard to accurately predict.

The aim of a service load check is to check that plasticity does not occur in the dowel under working loads, and the deflection can rebound after the load is removed and the magnitude of the deflection can be tolerated. The author recommends that on conducting a check, if the value for a_3 for serviceability loads is unavailable by testing, a_3 for service loads should be slightly reduced below the ultimate condition, possibly to a value of 40% of the bar diameter. We should recognise the load should still be significant enough to cause crushing or spalling of the concrete local to the bearing edge of the dowel.

In Table 5 below, for the nominated gap widths and varying tension loads, the maximum shear loads are calculated prior to the onset of yield. The vertical deflections and bar extensions are also presented in the table for reference and suggests overall vertical displacement of joints under service loads are negligible.

In Table 6 below, Bar extensions under tension are listed for each dowel and are based on the nominated forces, applied over the design joint width and 75% of one full anchorage length as determined by AS3600 and calculated based on bar load. The length of the extension assessment in the table is not scientifically based but interpreted from the very low 'So' lengths that Gilbert uses in his analysis and report. For the smooth SS bar, we have applied an extra 50% based on clause 13.1.3 of AS3600. Without testing the dowel systems, the extension calculations suggest there is likely inadequate crack control at the joints particularly as the tension loads increase. We would need to reduce loads down to approximately 230MPa in the deformed bar and 180MPa in the round bar to achieve possibly a tolerable 0.3mm crack after grouting of the joint. Thus, maximum service loads could be limited to 100KN in the deformed bar and 80kN in the SS smooth bar should it be desirable to limit the crack width.

Manufacturers of grouted dowel systems should be testing and providing the results of tension tests and joint opening so a better prediction of the opening of the joint can be made, and in service monitoring of the joint is possible. Table 6 should be understood to be a guestimate only and needs verification by testing, as dowel sleeves would also impact the distribution and development of the load into the adjacent concrete. Only dowel testing can establish a reliable joint opening prediction.

Deflection of Dowels and Limit of Yield under maximum service stress

	SS 316 Grade	N Bar G500
Dowel Bar E (Mpa)	195000	200000
Bar Fy (Mpa)	415	500
Allowance distance for bearing (a3/dia)	0.4	0.4

SS 316 Grade									
Diameter	24	mm							
Ns.max without shear	188	kN							
Joint Gap (mm)	0	5	10	15	20	25	30	35	40
Design Gap (Gap+2a$_3$) (mm)	19	24	29	34	39	44	49	54	59
Vs.max to limit Yield (Ns=0) (kN)	58.7	46.6	38.6	33.0	28.8	25.5	22.9	20.8	19.0
Vs(kN) at Ns = 10kN	55.6	44.1	36.5	31.2	27.2	24.1	21.7	19.7	18.0
Vs(kN) at Ns = 20kN	52.4	41.6	34.5	29.4	25.7	22.8	20.5	18.6	17.0
Vs(kN) at Ns = 30kN	49.3	39.1	32.4	27.7	24.2	21.4	19.2	17.5	16.0
Vs(kN) at Ns = 40kN	46.2	36.7	30.4	25.9	22.6	20.1	18.0	16.4	15.0
Vs(kN) at Ns = 50kN	43.1	34.2	28.3	24.2	21.1	18.7	16.8	15.3	14.0
Vs(kN) at Ns = 60kN	39.9	31.7	26.3	22.4	19.6	17.4	15.6	14.1	13.0
Vs(kN) at Ns = 70kN	36.8	29.2	24.2	20.7	18.0	16.0	14.4	13.0	11.9
Vs(kN) at Ns = 80kN	33.7	26.7	22.2	18.9	16.5	14.6	13.1	11.9	10.9
Vs(kN) at Ns = 90kN	30.6	24.2	20.1	17.2	15.0	13.3	11.9	10.8	9.9
Vs(kN) at Ns = 100kN	27.4	21.8	18.0	15.4	13.4	11.9	10.7	9.7	8.9
Vs(kN) at Ns = 110kN	24.3	19.3	16.0	13.6	11.9	10.6	9.5	8.6	7.9
Vs(kN) at Ns = 120kN	21.2	16.8	13.9	11.9	10.4	9.2	8.3	7.5	6.9
Vs(kN) at Ns = 130kN	18.1	14.3	11.9	10.1	8.8	7.8	7.0	6.4	5.9
Vs(kN) at Ns = 140kN	14.9	11.8	9.8	8.4	7.3	6.5	5.8	5.3	4.8
Vs(kN) at Ns = 150kN	11.8	9.4	7.8	6.6	5.8	5.1	4.6	4.2	3.8
Vs(kN) at Ns = 160kN	8.7	6.9	5.7	4.9	4.2	3.8	3.4	3.1	2.8
Vs(kN) at Ns = 170kN	5.5	4.4	3.6	3.1	2.7	2.4	2.2	2.0	1.8
Vs(kN) at Ns = 180kN	2.4	1.9	1.6	1.4	1.2	1.1	0.9	0.9	0.8
Vs(kN) at Ns = 185kN	0.9	0.7	0.6	0.5	0.4	0.4	0.3	0.3	0.3
Deflection due to Joint Shear per 10kN (mm)	0.002	0.004	0.007	0.010	0.016	0.023	0.031	0.042	0.054
Deflection at Vs.max Joint Shear (mm)	0.011	0.017	0.025	0.035	0.045	0.058	0.072	0.087	0.104

N Bar G500									
Diameter	24	mm							
Ns.max without shear	226.2	kN							
Joint Gap (mm)	0	5	10	15	20	25	30	35	40
Design Gap (Gap+2a$_3$) (mm)	19	24	29	34	39	44	49	54	59
Vs.max to limit Yield (Ns=0) (kN)	70.7	56.1	46.5	39.7	34.6	30.7	27.6	25.1	22.9
Vs(kN) at Ns = 10kN	67.6	53.6	44.4	38.0	33.1	29.4	26.4	23.9	21.9
Vs(kN) at Ns = 20kN	64.5	51.2	42.4	36.2	31.6	28.0	25.2	22.8	20.9
Vs(kN) at Ns = 30kN	61.3	48.7	40.3	34.4	30.0	26.6	23.9	21.7	19.9
Vs(kN) at Ns = 40kN	58.2	46.2	38.3	32.7	28.5	25.3	22.7	20.6	18.9
Vs(kN) at Ns = 50kN	55.1	43.7	36.2	30.9	27.0	23.9	21.5	19.5	17.9
Vs(kN) at Ns = 60kN	52.0	41.2	34.2	29.2	25.5	22.6	20.3	18.4	16.9
Vs(kN) at Ns = 70kN	48.8	38.7	32.1	27.4	23.9	21.2	19.1	17.3	15.8
Vs(kN) at Ns = 80kN	45.7	36.3	30.1	25.7	22.4	19.9	17.8	16.2	14.8
Vs(kN) at Ns = 90kN	42.6	33.8	28.0	23.9	20.9	18.5	16.6	15.1	13.8
Vs(kN) at Ns = 100kN	39.5	31.3	25.9	22.2	19.3	17.1	15.4	14.0	12.8
Vs(kN) at Ns = 110kN	36.3	28.8	23.9	20.4	17.8	15.8	14.2	12.9	11.8
Vs(kN) at Ns = 120kN	33.2	26.3	21.8	18.6	16.3	14.4	13.0	11.8	10.8
Vs(kN) at Ns = 130kN	30.1	23.9	19.8	16.9	14.7	13.1	11.7	10.7	9.8
Vs(kN) at Ns = 140kN	27.0	21.4	17.7	15.1	13.2	11.7	10.5	9.5	8.7
Vs(kN) at Ns = 150kN	23.8	18.9	15.7	13.4	11.7	10.3	9.3	8.4	7.7
Vs(kN) at Ns = 160kN	20.7	16.4	13.6	11.6	10.1	9.0	8.1	7.3	6.7
Vs(kN) at Ns = 170kN	17.6	13.9	11.6	9.9	8.6	7.6	6.9	6.2	5.7
Vs(kN) at Ns = 180kN	14.4	11.5	9.5	8.1	7.1	6.3	5.6	5.1	4.7
Vs(kN) at Ns = 190kN	11.3	9.0	7.4	6.4	5.5	4.9	4.4	4.0	3.7
Vs(kN) at Ns = 200kN	8.2	6.5	5.4	4.6	4.0	3.6	3.2	2.9	2.7
Vs(kN) at Ns = 210kN	5.1	4.0	3.3	2.8	2.5	2.2	2.0	1.8	1.6
Vs(kN) at Ns = 220kN	1.9	1.5	1.3	1.1	0.9	0.8	0.8	0.7	0.6
Vs(kN) at Ns = 225kN	0.4	0.3	0.2	0.2	0.2	0.2	0.1	0.1	0.1
Deflection due to Joint Shear per 10kN (mm)	0.002	0.004	0.006	0.010	0.015	0.022	0.030	0.041	0.053
Deflection at Vs.max Joint Shear (mm)	0.013	0.020	0.030	0.041	0.053	0.068	0.084	0.102	0.122

Table 5 - Service Deflections and combined actions of tension and shear

Possible joint opening at Tension load.

	SS 316 Grade	N Bar G500
Dowel Bar E (Mpa)	195000	200000
Bar Fy (Mpa)	415	500
Allowance distance for bearing (a3/dia)	0.4	0.4

SS 316 Grade											
Diameter			24								
Clause 13.1.3 factor			1.5								
Ns.max without shear			188								
Joint Gap (mm)			0	5	10	15	20	25	30	35	40
Design Gap (Gap+2a$_3$) (mm)			19	24	29	34	39	44	49	54	59
Bar Load (kN)	Bar Service Stress (Mpa)	Anchorage Length (mm)	Joint Opening (mm)								
Ns = 0kN	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ns = 10kN	22	41	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ns = 20kN	44	82	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ns = 30kN	66	122	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.1
Ns = 40kN	88	163	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Ns = 50kN	111	204	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Ns = 60kN	133	245	0.1	0.1	0.1	0.1	0.2	0.2	0.2	0.2	0.2
Ns = 70kN	155	285	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Ns = 80kN	177	326	0.2	0.2	0.2	0.3	0.3	0.3	0.3	0.3	0.3
Ns = 90kN	199	367	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Ns = 100kN	221	408	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Ns = 110kN	243	449	0.4	0.4	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Ns = 120kN	265	489	0.5	0.5	0.5	0.5	0.6	0.6	0.6	0.6	0.6
Ns = 130kN	287	530	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7
Ns = 140kN	309	571	0.7	0.7	0.7	0.7	0.7	0.7	0.8	0.8	0.8
Ns = 150kN	332	612	0.8	0.8	0.8	0.8	0.8	0.9	0.9	0.9	0.9
Ns = 160kN	354	652	0.9	0.9	0.9	0.9	1.0	1.0	1.0	1.0	1.0
Ns = 170kN	376	693	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Ns = 180kN	398	734	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Ns = 185kN	409	754	1.2	1.2	1.2	1.3	1.3	1.3	1.3	1.3	1.3

N Bar G500											
Diameter			24								
Ns.max without shear			226								
Joint Gap (mm)			0	5	10	15	20	25	30	35	40
Design Gap (Gap+2a$_3$) (mm)			19	24	29	34	39	44	49	54	59
Bar Load (kN)	Bar Service Stress (Mpa)	Anchorage Length (mm)	Joint Opening (mm)								
Ns = 0kN	0	0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ns = 10kN	22	27	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ns = 20kN	44	54	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ns = 30kN	66	82	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
Ns = 40kN	88	109	0.0	0.0	0.0	0.1	0.1	0.1	0.1	0.1	0.1
Ns = 50kN	111	136	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Ns = 60kN	133	163	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.1
Ns = 70kN	155	190	0.1	0.1	0.1	0.1	0.1	0.1	0.1	0.2	0.2
Ns = 80kN	177	217	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Ns = 90kN	199	245	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2	0.2
Ns = 100kN	221	272	0.2	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Ns = 110kN	243	299	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3	0.3
Ns = 120kN	265	326	0.3	0.4	0.4	0.4	0.4	0.4	0.4	0.4	0.4
Ns = 130kN	287	353	0.4	0.4	0.4	0.4	0.4	0.4	0.5	0.5	0.5
Ns = 140kN	309	381	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5	0.5
Ns = 150kN	332	408	0.5	0.5	0.6	0.6	0.6	0.6	0.6	0.6	0.6
Ns = 160kN	354	435	0.6	0.6	0.6	0.6	0.6	0.7	0.7	0.7	0.7
Ns = 170kN	376	462	0.7	0.7	0.7	0.7	0.7	0.7	0.7	0.8	0.8
Ns = 180kN	398	489	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8	0.8
Ns = 190kN	420	516	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9	0.9
Ns = 200kN	442	544	0.9	1.0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
Ns = 210kN	464	571	1.0	1.0	1.1	1.1	1.1	1.1	1.1	1.1	1.1
Ns = 220kN	486	598	1.1	1.1	1.2	1.2	1.2	1.2	1.2	1.2	1.2
Ns = 225kN	497	612	1.2	1.2	1.2	1.2	1.2	1.3	1.3	1.3	1.3

Table 6 – Dowel Service Stress and Possible Joint Opening

If we reconsider our previous example above with a working maximum shear of 25kN/m and tension load of 850kN/m and using table 5 (with table 6 for predicted joint opening), the dowel demand becomes:

Stainless Steel – 6 Nos (1.08% dowel area), Tension = 850/6=142 kN/dowel/m.

Maximum shear capacity of 6 dowels with joint at 15mm is 41 kN/m. (working load per dowel = 25/6 = 4.2 kN/dowel)

$$Deflection = \frac{4.2}{10} \times .010 = .004\,mm$$
$$Possible\ LT\ Joint\ opening = 0.7\,mm$$
$$Dowel\ Bar\ Max\ Service\ Stress = 315\,MPa$$

N24 – 5 Nos (0.9% dowel area), Tension = 850/5=170 kN/dowel/m.

Maximum shear capacity of 5 dowels with joint at 15mm is 49 kN/m. (working load per dowel = 25/5 = 5 kN/dowel)

$$Deflection = \frac{5}{10} \times .010 = .005\,mm$$
$$Possible\ LT\ Joint\ opening = .7\,mm$$
$$Dowel\ Bar\ Max\ Service\ Stress = 377\,MPa$$

This is one less dowel per meter than the Ultimate Limit State requirement. In this example, the ULS demands are driving the dowel count which is anticipated. Unfortunately, the dowel service stress in both solutions is very high and thus we do not believe we have adequately controlled the long-term joint opening at the TMJ.

As the calculated deflections under maximum working shear are insignificant, we have a tool for building owners or asset managers to check the performance of existing dowelled joints. Should a measurable vertical offset exist, we should be concerned and there is a possibility the dowelled joints are under designed. Caution should be exercised here, as some dowels may have a small vertical tolerance in the sleeve before taking up load. This vertical tolerance will be exhausted after formwork and back-propping is removed from the supported pour. If the dowel sleeve is carefully designed, the dowel in the sleeve should be a vertical snug fit. Similarly, by measuring the gap opening (induced crack after grouting of the joint) of any dowelled TMJ, we may be able to estimate the axial load in the dowel, however without manufacturers providing test data, such an estimate may be unreliable.

Temporary movement joints imply that the effect of the joint is temporary. Tension across at temporary movement joint once grouted, will slowly develop and direct stick brittle finishes such as tiles should be expressed with a relief at the joint. Grouting of TMJ's after grouting the dowel sleeves, needs to be robust enough so that grout stays in the joint under any expected forces of tension and shear. The TMJ has no precompression to keep grout in place and the faces of the edges of the pours will be smooth and may have residual debonding agent on them if a debonding agent was used. In non-corbelled joints, should joint grout crack and fall from the joint, the debris will become a hazard for occupants below and the joint deterioration may become a potential bridge for fire.

To the authors knowledge, there has not been any survey with long term monitoring of TMJ doweled joints after construction. A comprehensive review of the performance of these joints may help to justify their use and address the concerns of grout being robust in the joint.

Basis of Table 5 calculations

The below formulas (d) and (e) are used to determine the maximum shear load at service stress conditions. They are dependent of the yield strength of the materials and not the ultimate tensile strength. Equation (d) represents the stress at the extreme edge of the bar influenced by both bending and axial stress. Equation (e) represents the Principal Stress (averaged across the cross section) at the neutral axis of the dowel where the position is not subject to bending stress, only axial and shear stress. For simplicity and without checking stresses at any other points in the cross section that may be slightly more critical, our limiting stress formulas are:

Top edge of bar (No shear stress)

$$fsy \geq \frac{M}{Z} + \frac{N}{A}$$

For a bar of diameter d, Moment M=V(a_3+Gap/2), Axial Load= N thus rearranging the above equation:

$$V \leq \frac{\left(fsy - \frac{N}{A}\right)Z}{\left(a_3 + \frac{Gap}{2}\right)}$$

Substitution of the tension stress ($\frac{N}{A} = \frac{4N}{\pi d^2}$), the bar area ($A = \frac{\pi d^2}{4}$) the elastic section modulus (Z= $\frac{\pi d^3}{32}$) and choosing an a_3 value of 40% of the bar diameter, our equation becomes:

$$V \leq \frac{\pi d^3\left(fsy - \frac{4N}{\pi d^2}\right)}{32(0.8d + Gap)} \qquad (d)$$

Centroid of bar (Max shear stress and no bending stress)

$$fsy \geq \frac{\sigma}{2} + \sqrt{\frac{\sigma^2}{4} + \tau^2}$$

Rearranging the formula to solve for the maximum principal stress at the centroid based on shear stress $\left(\tau = \frac{VQ}{It} = \frac{16V}{3\pi d^2}, where\ Q = \frac{d^3}{12},\ I = \frac{\pi d^4}{64},\ t = d\right)$, the axial stress due to tension $\left(\sigma = \frac{4N}{\pi d^2}\right)$, our check on the neutral axis unaffected by the bending moment becomes:

$$V \leq \frac{3\pi d^2}{16}\sqrt{\left[\left(f_{sy} - \frac{2N}{\pi d^2}\right)^2 - \frac{4N^2}{\pi^2 d^4}\right]} \tag{e}$$

Deflection of bar (Assuming shear deflection negligible)

The deflection is calculated from a cantilever formula projecting to half the gap width multiplied by 2. With Youngs Modulus E, the formula for deflection across the joint is as follows:

$$Deflection = \frac{2V\left(a_3 + \frac{Gap}{2}\right)^3}{3EI}$$

In terms of a_3=40% of dowel diameter (a_3=0.4d) and the incorporating the second moment of area for a bar I$\left(I = \frac{\pi d^4}{64}\right)$, the equation becomes,

$$Deflection = \frac{128V\left(0.4d + \frac{Gap}{2}\right)^3}{3E\pi d^4} \tag{f}$$

We have ignored shear deflections in the deflection calculations as they are expected to be many orders of magnitude less than the deflection formulated above for bending.

Extension of bar (Assuming bars fully developed beyond distance a_3 from face of joint).

The extension is calculated from formula (g) below. Without testing, the values may be unreliable particularly as tension loads increase, so does the demand for bar anchorage beyond a_3.

$$Extension = \frac{4\text{N}(0.8d + Gap)}{E\pi d^2} \tag{g}$$

Concluding Remarks

The aim of the above report is to provide a guide for engineers to assess round bridging dowels on joints to comply with the design check requirement in AS5216:2021. The predicted magnitudes of tension loads for those dowels to be grouted in a TMJ, assumed the restraints were infinitely stiff. Such a situation could be expected to occur between large cores or otherwise buttressed walls near the foundation or deep transfer floors.

It is the author's opinion that there is a restricted market for grouted dowels working in shear combined with tension. Should these dowels fail, there is a possibility of significant yielding and a plastic mechanism forming in the dowels to cause alarm and loss of amenity. Should one or more dowels fail, there is a further concern that there may be no reserve in strength of the adjacent dowels to avoid more dowels yielding and forming plastic hinges. When the plastic hinge forms in the dowels, tension from will remain in the floor plate and the tension load may increase in the dowels as the mechanism forms and the supported slab edge deflects.

A better option would be to resolve shear in TMJ's with an un-grouted dowel or corbel and supplement the joint with grouted dowels working only in tension. In this way if the tension loads are underestimated, the grouted tension dowels may yield but the floor will remain propped by the un-grouted shear dowels or corbel and the joint will open slightly. There will be no concern for collapse or the development of a hinged mechanism.

Temporary movement joints are being frequently specified when permanent movement joints are usually a better option and work long term to relieve restraint loads from shrinkage and thermal effects. There is much authoritative literature on permanent movement joint spacings in buildings, and the author recommends we keep our floor plate sizes between movement joints within the recommendations.

Significant crack control reinforcement is still needed to control shrinkage cracks that may occur much later in the building's life if the precompression is lost after a temporary movement joint is grouted and restrained. The higher the concrete strength is at the age that the concrete cracks occur, the higher is both the demand for reinforcement to control cracking and the demand for tension on dowels grouted into a TMJ.

TMJ's are most beneficial to avoid or reduce restraint loads on supports that may not have the strength or tolerance to otherwise resist the slab shrinkage loads or displacements. For instance, blade columns and walls that are restrained by and near to thick transfers or foundations. Providing a TMJ will reduce the lateral displacement of any supporting weak and sensitive vertical elements which could reduce the chances of shear cracks and reduce load eccentricity and moments for load bearing elements with incompatible displacements between floors. TMJ's are thus beneficial when the floor element is stronger than the restraining elements and generally not the other way around. In other words, TMJ's would be beneficial in situations where the forces to crack the slab exceed the forces that can be tolerated on the restraining element in the short term.

There may be other circumstances where temporary relief of early shrinkage forces and displacements are required, and a permanent solution is not practical. This may be the case where the connection strength at points of high restraint (e.g. cores) may be insufficient until the floor plate/pours are complete, however we must be aware that providing a TMJ between stiff restraints incurs a significant tension penalty on dowels.

The author suggests that we should be questioning the need for TMJ's between heavy restraints as it may be simpler to add the necessary reinforcement for crack control in post tensioned slabs without a TMJ given the tension-shear interaction issues with grouted dowels.

For those continuing to use grouted dowelled joint solutions, joint width estimates and tension loads should be conservative. The author is not aware of any authoritative software or literature that can be relied on for an accurate assessment given concrete properties are highly variable. Assumed design shrinkage values can easily be exceeded through mix design, extra water added to increase concrete workability at site, lower humidities and many other factors. Concrete strengths can also be significantly higher than specified characteristic values used in the slab design particularly in post tensioned slabs where early strength requirements often dictate the cement content and strength design. Be conservative when calculating joint widths and tension loads.

Every floor needs unique consideration. Popular Finite Element Analysis programs such as Ram Concept, do not calculate tension loads that develop from shrinkage or thermal restraint. Where manufacturers are providing grout sleeve capacities that do not allow for reductions in capacities under tension, be wary and undertake your own simple assessment as per AS5216. As maximum shear is reached, the dowel should not be expected to have a reserve tension capacity, and similarly as the maximum tension is reached, the dowel should not be expected to have a reserve shear capacity.

Shrinkage loads build up over time so we cannot be complacent and believe there are no latent issues developing in projects that have dowels that are working in both shear and tension. **Restraining shrinkage in concrete comes with a significant penalty. A TMJ should not be understood to be a feature that eliminates tension.**

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