Composite Beam Design to AS 2327-2017

Rev 0, Updated 10 September 2021

AS 2327-2017 Composite Structures involves the following key changes relating to Structural Toolkit:

- Capacity reduction factors (ø) revised
- Changes to effective width calculation
- γ and ψ components removed from øMb calculation simplifying the bending/shear interaction
- Revised minimum β (degree of composite action)
- Vertical shear capacity now includes concrete slab component
- Shear connector design capacities and requirements revised
- Longitudinal shear fully revised
- Creep and shrinkage serviceability changes

This document aims to discuss the relevant changes to AS 2327-2017 affecting the Composite Design V5.09 and Structural Toolkit V5.5 and has therefore omitted changes that do not affect Structural Toolkit.

The previous AS 2327-2003 Part 1: Simply supported beams, has been incorporated into the new 2017 standard as section 3, Composite Beams. The new standard includes slabs, columns, joints and floor systems which are not covered within Structural Toolkit.

The general approach to the new composite standard remains similar, however, capacities will differ due to many clause changes in the new 2017 standard. Due to the ambiguity of certain clauses in the new standard, several user options are given to allow the Professional Engineer to decide how the composite beam is designed in accordance with the new standard.

Several examples from seminars and workshops have been reviewed with high degrees of ambiguity and errors. We also instigated discussion with the code committee and Steel Institute to attempt to resolve these ambiguities, however, the new standard remains ambiguous in several areas. There is further ambiguity given the removal of Appendixes B and D containing design equations. The committee suggested a commentary is a more appropriate place for this, with no indication of when this may be published. The Steel Institute (AISC at the time) published an excellent “Composite Beam Design Handbook (SAA HB91-1997)” which was used to validate the previous releases of the Composite Design module. The Steel Institute advised that a revised publication is something that is planned in the future.

Capacity reduction factors

The new code AS 2327-2017 presents a revised capacity reduction table. This table now separates concrete and steel for each type of action. A comparison of the two tables can be seen below

CAPACITY FACTOR FOR THE STRENGTH LIMIT STATE

Type of action effect	Capacity factor (ϕ)
Bending	
(a) Propped construction: Construction Stage 5 (see Clause 4.2.3)	0.70
(b) All other cases	0.90
Vertical shear	0.90
Longitudinal shear	
(a) Concrete slab	0.70
(b) Shear connectors	0.85

AS 2327-2003 Table 3.1

CAPACITY FACTOR FOR THE ULTIMATE LIMIT STATE

Type of action effect	Capacity factor (ϕ)
Axial	
(a) Concrete in compression	0.65
(b) Steel in compression	0.90
(c) Concrete in tension	0.80
(d) Steel in tension	0.90
Bending of composite beams and composite slabs	
(a) Steel	0.90
(b) Concrete	0.80
Vertical shear	
(a) Steel	0.90
(b) Concrete	0.60
Longitudinal shear	
(a) Concrete slab	0.63
(b) Shear connectors	0.80

AS 2327-2017 Table 1.4.3

Bending capacity factors

Bending is now split into 0.8 for concrete and 0.9 for shear, as compared to a general 0.9 previously (or 0.7 if propped). The effect this has on new designs will vary greatly depending on how the new standard is interpreted. In the 2003 standard, the ø factor is applied after the Mb (bending moment capacity) was calculated – which is the general approach for applying these factors. However, in the new 2017 code this approach appears to be no longer followed as shown by Eq 3.5.4.2(1).

$$F_{cc} = \min(N_{y,d}, N_{c,d}) \quad \text{. . . 3.5.4.2(1)}$$

where

$N_{y,d}$ = design axial force resisted by the steel section at yield

$= f_{y,d}A$

$f_{y,d} = \phi f_y$

ϕ = capacity reduction factor (see Table 1.4.3)

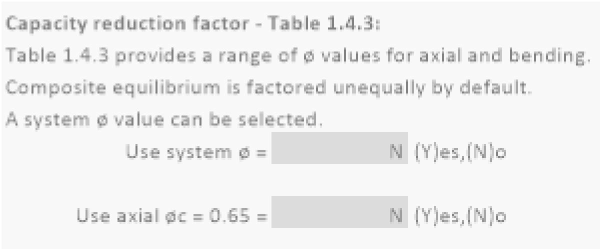
$N_{c,d} = 0.85 f_{cd} h_c b$

$f_{cd} = \phi_c f'_c$

ϕ_c = capacity reduction factor (see Table 1.4.3)

Fcc, being the axial force used to calculate the bending capacity for full composite action, is now calculated already including the capacity reduction factors for steel and concrete. This is likely a result of the removal of an overall (global) reduction factor and appears to be a similar approach to that of European codes where the factors are applied to material elements rather than the entire section.

This new approach makes it unclear which capacity factors should be used for this equation. As the forces Ny,d and Nc,d are axial, there is an argument that the capacity reduction factor for concrete should be 0.65, rather of 0.8 (for the 2 examples we had, one used 0.65 and the other 0.8). A similar argument could be made for it to be 0.8, as the forces contribute to a bending capacity (with the 0.65 used for compression members ie. columns. Further to this, one could disagree with the entire approach and prefer an overall capacity reduction factor – which would follow the 2003 code, after all, the individual components are actual forces and are equilibrium equations. To address this ambiguity in the 2017 code, the new Composite Beam V5.09 module allows the user to decide which interpretation is more appropriate. This feature can be found in the [Mb] tab to the right.



The default factors are set to ø=0.8 for concrete, and ø=0.9 for steel as per table 1.4.3y
. The options provided allow the user to set an overall system ø or set the concrete factor to ø=0.65 (being axial).

Changing these options will have significant effects on the overall bending capacity of the beam and will need to be considered carefully.

Shear capacity factors

As Cl 3.5.5 for vertical shear capacity now includes a concrete slab component, a concrete capacity reduction factor of ø=0.6 has been added.

Longitudinal shear now uses ø=0.63 for concrete rather than ø=0.70. This will result in reduced longitudinal shear capacities, but needs to be considered alongside the changed capacity formula in order to gain a full understanding of the changes (Refer to the Longitudinal shear section of this document). Shear connecters are also now ø=0.80 instead of ø=0.85.

Effective section

In Cl 3.4.2.1 the effective width of a composite slab is no longer dependent on bsf/2 + 8Dc (being half the steel flange width and 8 times the depth of the concrete). This means that in certain arrangements (typically in shallow slabs with wide loadwidths) the area of concrete used in bending moment capacity is greater than previously, possibly resulting in a greater bending moment capacity – this will depend on the critical axial force between concrete and steel for Fcc. In cases where this term of the effective width is not critical, this change will have no effect.

The λ term in Cl 3.4.2.2 has been removed, with the area of concrete below the ribs being considered based on the ribs being perpendicular (angle greater than 15°) or parallel (angle 15° or smaller) – i.e. if the ribs are parallel, all of the concrete below the ribs is considered, and none for perpendicular. This is a more simplified approach compared to the previous code, with there no longer being a proportional section between 15° and 60°. This will mean for sheeting between these angles, there will be less of a concrete area used for bending capacity than before – resulting in lower bending capacities in some cases, being dependent on the Fcc calculation.

Moment capacity

The fundamental process of deriving the bending moment capacity is similar, but with the removal of the γ shear ratio component (and subsequently also the ψ value), it is now a more simplified approach. As the 2017 standard no longer provides explicit formula for each force distribution case and β value, the new Composite module refers to the formulas within Appendix D of the 2003 standard which are based on static equilibrium. The removal of the shear ratio (γ) means that sections with a shear capacity utilization of over 0.5, the bending capacity is no longer reduced. However, Cl 3.5.6 introduces a combined moment and shear capacity requirement, meaning that the total bending capacity is still reduced by the design shear.

Cl 3.5.8.3 provides an equation for minimum degree of shear connection (β), being previously only defined as β=0.5 at the point of maximum moment in the 2003 standard. This new requirement is a function of effective span (Le) and steel yield (fy), with a lower bound of β=0.4. This may result in some designs requiring a lower minimum ratio of shear connection than before (β=0.4 at 7m, and β =0.5 at 10m). This clause also provides a secondary method for calculating minimum β, with a number of very specific requirements including 1 stud per tray. Using this simplified method (which is not particularly simple) results in the Moment capacity being calculated using the linear interaction method (Figure 3.5.4.3(B)). This method is present in Composite Beam V5.09 under the [Design] tab. If all the requirements of the method are met, the user can use the simplified β value as seen below (if the requirements are not met, the option cannot be selected).

Minimum composite (β.min = 0.40) - Cl 3.5.8.3

Beam length (Le) =	10500	mm	
Yield strength of steel (fy) =	320	MPa	
Simplified β.min method valid =	Y	(Y)es,(N)o	(Refer notes)
Use simplified β (β.min.1) and interpolate =	Y	(Y)es,(N)o	
One stud per tray β.min.1 = 1-(355/fy)*(1.0-0.04*Le) =	0.36		Eq 3.5.8.3(5)
β.min.2 = 1-(355/fy)*(0.75-0.03*Le) =	0.517		Eq 3.5.8.3(1)
β.min = β.min.1 ≥ 0.4 =	0.400		
øFcc =	1468	kN	
øFcp.min =	587	kN	
Minimum composite (øMb.min (β.min=0.40)) =	285.3	kNm	

Simplified (interpolated method):		
Height of connector ≥ 76mm =	OK	Cl 3.5.8.3(a)
Shank is 19mm dia. =	OK	Cl 3.5.8.3(a)
Rolled equal flanges =	OK	Cl 3.5.8.3(b)
Profiled sheeting =	OK	Cl 3.5.8.3(c)
Perpendicular =	OK	Cl 3.5.8.3(c)
Ribs continuous =	OK	Cl 3.5.8.3(c)
One stud per rib =	OK	Cl 3.5.8.3(d)
Bo/hp ≥ 2 =	OK	Cl 3.5.8.3(e)
hp ≤ 60 =	OK	Cl 3.5.8.3(e)
Simplified βmin. valid =	Y	(Y)es,(N)o

Vertical shear capacity

The 2017 standard now explicitly incorporates a concrete shear component into the vertical shear capacity in Cl 3.5.5. This new component is a function of the steel flange width, concrete strength and depth, and a slenderness value based on the depth of slab to composite member. This function also incorporates the new capacity reduction factor of ø=0.6 for vertical shear in concrete. This addition will result in greater vertical shear capacities than the previous code. The steel component of the shear capacity is the same as the 2003 code – being based on Vuw.

$$V_{\mathrm{comp}} = V_{\mathrm{pl,Rd}} + V_{\mathrm{slab}} \qquad \ldots 3.5.5(1)$$

$$V_{\mathrm{slab}} = \phi_s f(\lambda_{\mathrm{sd}})(b_f D_{\mathrm{slab}})^{0.7}\sqrt{f'_c} \qquad \ldots 3.5.5(2)$$

Shear connectors

Shear connecters in the 2017 standard have revised capacity formulas and changes to shear connector detailing requirements.

Shear connector capacity

The shear connector capacity formulas under Cl 3.6.2.3 are now slightly different to the previous 2003 code. Eq 3.6.2.3(1) now uses an αshear value of 0.7 (stud) or 0.5 (bolt) instead of the original 0.63. Eq 3.6.2.3(2) is factored by 0.29 instead of 0.31. These changes will result in slightly higher or lower base connector capacities than before, depending on the connector type and critical value.

$$f_{vs} = 0.63\, d_{bs}^{2}\, f_{uc}\text{; or} \qquad \text{. . . 8.3.2.1(1)}$$

$$f_{vs} = 0.31\, d_{bs}^{2} \sqrt{f'_{cj}\, E_c} \qquad \text{. . . 8.3.2.1(2)}$$

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$$P_{Rk} = \alpha_{shear} d_{bs}^{2} f_{uc}\text{; or} \qquad \text{. . . 3.6.2.3(1)}$$

$$P_{Rk} = 0.29 d_{bs}^{2} \sqrt{f'_{cj} E_c} \qquad \text{. . . 3.6.2.3(2)}$$

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Channels are now no longer an option for a shear connector, and have also been removed from the Composite design module.

High-strength structural bolts are no longer factored by 80% for lightweight concrete.

Cl 3.6.2.4 introduces kl and kt factors for dealing with parallel and perpendicular sheeting, respectively. Depending on the profile type, and geometry of the ribs, these factors will result in reduced the shear connector capacities to the previous 2003 code.

It should also be noted that with the 2003 standard, the shear connector capacity includes a multiplier factor kn which depends on the number of connectors on a composite beam. This sharing factor is not present in the new standard.

Shear connector detailing requirements

Cl 3.6.2.7.1 introduces new requirements for where an edge of a concrete flange is within 300mm to the centreline of the nearest connector row. This is present in Composite Beam V5.09 as a warning when a side of beam is defined as an edge, and its width is within the 300mm limit to the connector.

Left side of beam	Check requirements for less than 300mm edge of slab to connector (Cl 3.6.2.7.1)		
Is this an edge beam =	Y	(Y)es/(N)o	
Slab width left (width.l) =	250	mm	Distance to edge of slab
Left loadwidth (cts.l) =	250	mm	

Longitudinal Shear

Longitudinal shear has been significantly changed. The longitudinal design shear formula has been altered (Eq 3.8.2(1) and 3.8.2(2)), alongside the capacity formula (Eq 3.8.3). The nomenclature and descriptions for the shear failure planes have also been revised, with slight changes to the type of shear planes considered.

Longitudinal design shear

VL* represents the longitudinal design shear force per unit length (kN/m). The 2003 code calculated this force by dividing the calculated shear connector capacity by its spacing (and number of connectors in a group where relevant). The 2017 code instead based the force on the calculated Fcc (or Fcp if partial shear), dividing it by spacing. These formulas can be seen below.

$$V^*_{L.tot} = \frac{n_x\, f_{ds}}{s_c} \qquad \text{. . . 9.3.2}$$

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$$v_L^* = F_{cc}/s \qquad \ldots 3.8.2(1)$$

$$v_L^* = F_{cp}/s \qquad \ldots 3.8.2(2)$$

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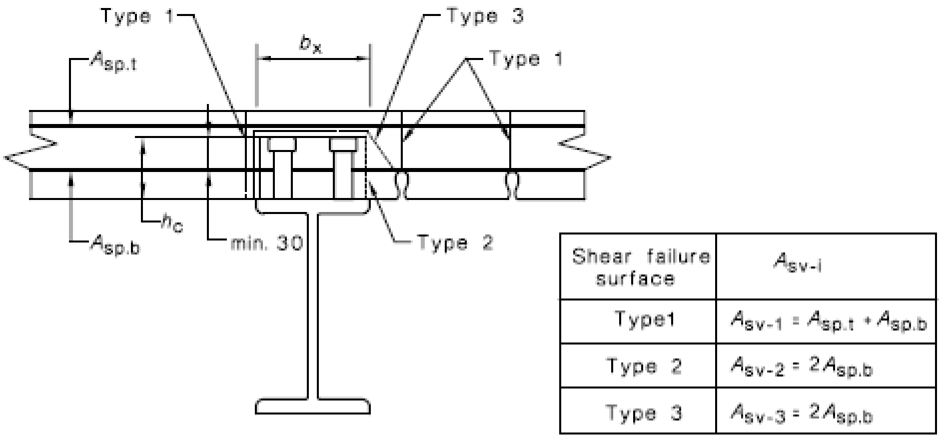
However, the 2017 formula does not result in a true VL*, as it needs to consider the number of shear connectors to convert into the kN/m needed. The Composite Beam V5.09 therefore uses the formula:

$$V_L^* = \frac{n_x * F_{cp}}{n * s_c}$$

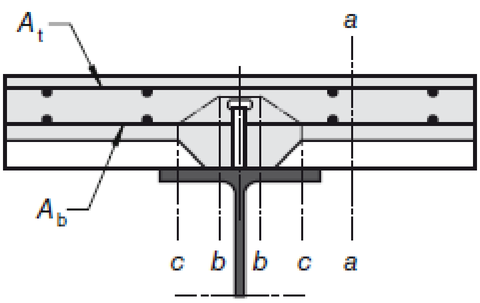
Where n is the number of shear connectors used. This formula results in a similar value to the 2003 code, except it does not include the assumption that the shear connectors will reach their full shear capacity (e.g., if 15.2 connectors are needed for shear connection requirements, then 16 will be specified – resulting in the connectors not reaching their shear capacity). This means that VL* will often be a lower value than previously.

Shear failure planes

Shear failure planes are now presented with different nomenclature and descriptions. These changes are also reflected in Composite Beam V5.09. Below is a representation of some of these changes.



AS 2327-2003



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Type 1 is now the equivalent of type a-a and type 2 is b-b. Type 3 is effectively type c-c, but no longer incorporates the vertical portion of type b-b (see Figure 9.4.2.5 in AS2327-2003 for more details). These changes are present in the new Composite Beam V5.09, but type c-c is omitted, due to the assumed shear plane passing through the rib profile, type b-b will always be critical.

Longitudinal shear capacity

Cl 3.8.3 introduces a new longitudinal shear capacity formula, and although it includes the main components of the previous 2003 formulas, it also includes an element of stud bearing capacity (Ppb,Rd).

$$V_L = u(0.36\sqrt{f'_c}) + 0.9A_{sv}f_{yr} \qquad \ldots 9.6(1)$$

$$V_L = 0.32 f'_c u \qquad \ldots 9.6(2)$$

AS 2327-2003

Conc. plane contrib.

Steel reinf. contrib.

Stud bearing contrib.

$$v^*_{Lp} \leq \phi\tau_u \qquad \ldots 3.8.3$$

$$\tau_u = \left(\mu A_{ts} f_{sy} + k_{co} f'_{ct} A_{tc}\right) + P_{pb,Rd}/s$$

$$\leq \text{lesser of } 0.2 f'_c A_{tc} \text{ and } 10A_{cv}$$

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The component µ*Ars*fsy (Ars is taken as = Ats) in Eq 3.8.3 appears to be a substitute for 0.9*Asv*fyr from the 2003 code, being the longitudinal steel reinforcement contribution. The new µ value results in a smaller contribution of this component when the surface condition of the shear plane is smooth (µ=0.6) as per Table 3.8.3. kco*f'ct*Atc is now possibly in place of u(0.36√f'c), being the concrete shear plane component. The limit of 0.2*f'c*Atc is also possibly the replacement of 0.32*f'c*u from the 2003 formula.

The final term in the equation Ppb,Rd represents a stud bearing capacity from Eq 2.7.2(5), being based on the sheeting and stud geometry. It should be noted that the entire Eq 3.8.3 has a reduction capacity factor of ø=0.63 applied to it (compared to the ø=0.7 from the 2003 code), being the ø value for concrete in longitudinal shear. However, with the introduction of the stud bearing component, it could be argued that the ø value for shear connectors in longitudinal shear (ø=0.80) should somehow be included in this formula. As there is no clear answer in the code regarding this, the new Composite Beam V5.09 uses the conservative ø=0.63.

The other limiting factor of Eq 3.8.3, 10Acv. The Acv is not defined in the code, and the units produced from Eq 3.8.3 are someshat hidden (being force per unit length – kN/m). Composite Beam V5.09 assumes that Acv = Atc, with the value 10 is comprised of a limiting stress. It appears this value may be a limiting upper bound to the equation.

These changes generally result in lower longitudinal capacities than the 2003 standard. However, due to the addition of a stud bearing capacity component, some situations may yield higher capacities.

Serviceability

Creep and shrinkage have changed in the new 2017 standard. The new methods in Cl 3.10.3.3 and Cl 3.10.3.4 now incorporate the creep coefficient from AS3600 into an effective elastic modulus (Eef.cc and Eef.cs, being for creep and shrinkage, respectively).

$$E_{ef.cc} = \frac{E_c}{1 + \phi_{cc}} \qquad \ldots 3.10.3.3$$

$$E_{ef.cs} = \frac{E_c}{1 + 0.55\phi_{cc}} \qquad \ldots 3.10.3.4(1)$$

The Eef.cc (creep) is incorporated into the Ief for calculating long term deflection and the Eefcs (shrinkage) is used in Cl 3.10.3.4 to calculate additional deflection due to shrinkage.

The new methods appear to result in significantly less deflection due to creep and shrinkage than the 2003 methods. This is partly due to the shrinkage deflection reduction as a result of creep (as seen in Eq 3.10.3.4(1)), where the suggestion is that creep will reduce the deflection from shrinkage.

Examples using these methods that are available widely differ on the values used in them. Although Composite Beam V5.09 has incorporated the new methods, the old methods can also still be used. This can be done through the [Serv] tab on the right (see below).

Use effective modulus for long term: Cl 3.10.3.3
Use AS2327-2003 shrinkage and creep method
Use = N (Y)es,(N)o

Additional features

Along with the changes to the composite code, there are several new features in the Composite Beam V5.09.

Sheeting types

New sheeting types have been added (Fielders KingFlor products) along with their relevant geometry and values (see [Input] tab). Users can now also specify a custom sheet. However, it is up to the user to check that all sheeting geometry requirements are met according to AS 2327-2017 – mostly for when unusual sheet values are used.

	Bondek II	Condeck	RF55	KF57	KF40	KF70	Solid	Custom	
Name	Bondek II	Condeck	RF55	KF57	KF40	KF70	Solid	Custom	
Profile	R	P	R	P	O	O	S	R	
Height of ribs (hr)	54	55	54	57	40	70	0	54	mm
Spacing of ribs (bs)	200	300	200	300	245	300	1000	200	mm
Width rib top ht (br)	32	0	26	0	65	112	0	26	mm
Width rib mid ht (brm)	22.5	0	21	0	93	132	0	21	mm
Width rib bottom ht (bb)	13	0	15	0	100	166	0	15	mm
Profile area	1678	1620	1650	1593	1334	1467	0	1650	mm²/m
Sheet yield	550	550	550	550	550	550	550	550	MPa
Conc. area below rib (Acb)	48400	55000	20000	57000	24400	42600	0	20000	mm²/m
Conc. centroid below rib	25	28	25	29	24	40	0	25	mm
Iwet.s (0.60BMT)			0.416	0.402	0.187			0.416	$\times10^6$ mm⁴
Iwet.s (0.75BMT)	0.511	0.405	0.520	0.503	0.233	0.584		0.520	$\times10^6$ mm⁴
Iwet.s (0.90BMT)		0.498							$\times10^6$ mm⁴
Iwet.s (1.00BMT)	0.674	0.569	0.559	0.671	0.280	0.745		0.559	$\times10^6$ mm⁴
Iwet.c (0.60BMT)			0.458	0.402	0.185	0.000		0.458	$\times10^6$ mm⁴
Iwet.c (0.75BMT)	0.425	0.340	0.559	0.503	0.185	0.584		0.559	$\times10^6$ mm⁴
Iwet.c (0.90BMT)		0.408							$\times10^6$ mm⁴
Iwet.c (1.00BMT)	0.587	0.486	0.559	0.671	0.239	0.745		0.559	$\times10^6$ mm⁴

.s = single span, .c = continuous
sheet area based on 1.00BMT
concrete centroid from bottom of sheeting

afs
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Stage 1 design capacity

The stage 1 loading requirement of a 10kN (live load) point load is now checked against the capacity of the fully unrestrained steel member under the [Loads] tab to the right.

Stage 1		
S.Wt & 10kN construction LL		
	M.1*=	46.1 kNm
	øMbx.1 =	47.5 kNm

The user will get a warning if this requirement is not met.

Ponding

The user is now given a warning to consider additional ponding due to beam deflection if sufficient precamber has not been specified. These warnings will appear on the [Loads] and [Serv] tab. A similar warning will also show if too much precamber has been specified.

Additional Ponding:
Beam deflection causes additional ponding (3mm sag) - Refer [Serv] tab

[Loads] tab

Additional Ponding:
Beam is in positive deflection (3mm sag)
after precamber under stage 3 loading.
Additional ponding is to be allowed for.

[Serv] tab

Should you have any questions regarding the changes made to the Composite Beam design module, contact our support team.