Concrete Design to AS 3600-2018

Rev 1, Updated 22 August

AS 3600-2018 introduces several new features including:

- Increases to the capacity reduction factors (ø factors) in Table 2.2.2
- Modified alpha2 (α2) for the compression block maximum force, Eq 8.1.3(1) for beams and Eq 10.6.2.5(1) for columns
- Modified alpha1 (α1) for column squash load, Eq 10.6.2.2
- Modified gamma (γ) with γkud being the compression block depth, Eq 8.1.3(2) for beams and Eq 10.6.2.5(2) for columns
- Changes to crack control stress limits and characteristic crack widths as well as the introduction of crack width calculations, Cl 8.6.2.3
- An entirely new approach for shear and torsion design using Modified Compression Field Theory (MCFT)
- Refinement to the effective stiffness (Ief) formula being a modified Eurocode 2 approach, Eq 8.5.3.1(1)
- Changes to wall slenderness limits before a double layer of reinforcement is required

Beam capacity

The bending capacity reduction factor has increased from 0.8 to 0.85, Table 2.2.2(b), representing an increase of 6.25%, however, because of the changes to the alpha2 (α2) and gamma (γ) equations the actual capacity increase varies depending on the beam properties.

Deflection by Simplified Calculation

The effective stiffness (Ief) equation $Ief=Icr+(Ig–Icr)x(Mcr/Ms)^3$ (Branson 1963) has been part of AS 3600 since the initial release of AS 3600. In fact, this equation was a “suggested” formula in the preceding AS 1480 where the following note was stated “The Standards Association of Australia is not prepared to recommend a value for effective moment of inertia, but suggest that the following formula may be used”.

Kilpatrick and Gilbert 2009, suggested a modified Eurocode 2 approach better predicted deflections according to Reinforced Concrete Basics 2E by Foster, Kilpatrick and Warner, published in 2010. The Branson equation has been shown to significantly underestimate measured deflections according to the same publication.

This new equation for the effective inertia of a cracked section is $Ief = Icr/[1-(1-Icr/Ig)x(Mcr/Ms^*)^2] < Ief.max$ Eq 8.5.3.1(1)

The Ief should only be calculated when M* > Mcr, otherwise the effective stiffness equals the gross stiffness (Ief=Ig)

Crack Control

The maximum steel stresses specified in Tables 8.6.1(A) and 8.6.1(B) of AS 3600-2009 are intended to ensure that maximum crack widths will not exceed 0.4mm in width according to the 2009 commentary.

The new code introduces characteristic crack widths citing 0.2, 0.3 and 0.4mm choices. Clause 8.6.2.3 provides general equations for calculating actual crack width under the service conditions. The Reinforced Concrete Building Series Design Booklet RCB-1-1(1) Crack Control of Beams, Part 1:AS 3600 Design, by OneSteel Reinforcing, August 2000 provides comprehensive background and equations into calculating the bar stresses.

Structural Toolkit's Concrete Member design provides an input option for specifying maximum characteristic crack width (w'max) which calculates the limiting steel stress within the reinforcement based on Tables 8.6.2.2(A) & (B) for beams and 9.5.2.1(A) & (B) for slabs. The actual crack width is also calculated based on the equations of Cl 8.6.2.3 and the OneSteel publication.

There appears to be a very close correlation between the limiting stress of the characteristic crack width and the actual crack width calculated.

ACI 224R-01 Control of Cracking of Concrete Structures, 2001 (reapproved 2008) Table 4.1 shows a "Guide to reasonable crack widths, reinforced concrete under service loads". This table is reproduced in Reinforced Concrete Basic 2E, Table 3.3. (Note that these crack widths are a conversion from US imperial units to metric)

Exposure Condition	Maximum allowable Crack Width (mm)
Dry air or protective membrane	0.41
Humidity, moist air, soil	0.30
Deicing chemicals	0.18
Seawater and seawater spray, wetting and drying	0.15
Water retaining structures	0.10

Shear Capacity

AS 3600-2018 introduces an entirely new approach to shear design based on Modified Compression Field Theory (MCFT). This approach is derived from the Canadian code approach CSA A23.3. The Bridge Code AS 5100.5-2017 had already introduced this approach, but it must be noted that differences currently exist between AS 3600 and AS 5100.5.

Two methods are provided for shear design; a more rigorous general approach and a simplified approach which is a derivation of the general approach. The simplified approach derives the concrete and shear reinforcement contribution capacity based on fixed kv and ø constants and is independent of the applied loads. The general approach calculates the kv and ø values, and thus concrete and shear reinforcement contribution, based on the bending, torsion, shear and axial applied loads at the point being considered.

A notable feature of this new approach is the inclusion of coincidental torsion which can increase the equivalent shear at the section being considered; and the introduction of additional flexural steel requirements as a result of shear and torsion induced tension.

Shear design capacity reduction factor (ø)

When the area of shear reinforcement is less than the minimum requirements; for the determination of shear strength by web crushing; and when low ductility fitments are used, the capacity reduction factor (ø) is 0.7.

For sections with shear reinforcement more than minimum and with normal ductility fitments, the capacity reduction factor (ø) is 0.75.

Minimum shear reinforcement area

Minimum shear reinforcement requirement has been revised, increasing the coefficient at the front of Eq 8.2.8 of 2009 code of 0.06, to 0.08 in Eq 8.2.1.7 of 2018 code. This change represents a increase of 33% to minimum area of shear reinforcement.

The bridge code AS 5100.5 currently specifies the AS 3600-2009 approach in Cl 8.2.1.7 with a lesser minimum requirement.

Structural Toolkit provides an option to use the Bridge Code value where applicable.

Minimum shear reinforcement - Cl 8.2.1.7, Cl 8.2.1.7 - AS 5100.5:
The minimum has changed in AS3600-2018 which does not match
the bridge code, and results in higher area requirements

Use AS5100 =	N	(Y)es, (N)o

Torsion

The importance in considering torsion in design is highlighted by clause 8.2 beginning with "Combined flexure, torsion and shear".

When torsion (T*) is applied it must be considered in design of the member. When T* < 0.25 øTcr, the effects of torsion can be neglected providing the minimum torsional reinforcement is provided, according to Cl 8.2.1.2. Obviously when T* is 0 kNm, then the minimal torsional requirements are not applicable.

When T* > 0.25 øTcr, the torsion effect increases the applied shear using an equivalent shear force (Veq*) calculated in Eq 8.2.1.2(3). Both the increased shear (Veq*) and Torsion (T*) must then be designed for with appropriate closed ligs covering both these applied loads.

Unreinforced section

The 2009 code required shear reinforcement when the design shear V* > 0.5 øVuc in Cl 8.2.5, (and V* > øVuc for band beam type situations).

AS 3600 -2018 requires shear reinforcement when V* > øVuc with no reduction coefficient present like the 2009 standard. This has caused some confusion and although the methodology for deriving Vuc is completely different, there is some discussion that a reduction coefficient may be reinstated in a future amendment.

It should be noted that when minimum shear reinforcement is not provided, the new simplified approach code has a capacity reduction factor (ø) of 0.7 and kv of 0.10 (or less). When greater than minimum is provide, the capacity reduction factor (ø) is 0.75 and kv is 0.15. The direct comparison is not that simple as the Vuc in the 2009 code uses a $f'c^{1/3}$ and the 2018 uses a $f'c^{1/2}$.

Simplified approach

The basic equation for concrete shear capacity contribution is Vuc = kv*bv*dv*√f'c, Eq 8.2.4.1

The first thing to note is that dv is not the depth to outermost layer of tensile reinforcement as per the 2009 standard, but is now defined in Cl 8.2.1.9 being the greater of 0.72D and 0.9d where d is the distance to the centroid of tensile reinforcement in the half-depth of the section.

Using the simplified approach, the capacity of the section in shear uses a capacity reduction factor (ø) of 0.7, and a reduced kv which is no greater than 0.10 when the area of shear reinforcement is less than the minimum specified in Cl 8.2.1.7. The angle of inclination of the compression strut (θ) is also set to 36° for calculation of the reinforcement contribution Vus, Cl 8.2.4.3.

When the shear reinforcement provided is greater than minimum, the capacity reduction factor (ø) is 0.75, kv is 0.15 and θ is 36°. These constants are unaffected by the applied loads at the section being considered.

This simplified kv is derived by setting the value of εx of the general method in Cl 8.2.4.2.2 to 0.85 fsy /(2 Es) according to Stephen Foster of UNSW.

General Approach

The general approach provides a rigorous evaluation of εx in Eq 8.2.4.2.2(1) & (2) and 8.2.4.2.3(1) & (2) where torsion is present. εx is then used to derive the kv value, for calculation of Vuc, from Eq 8.2.4.2(1)-(4) and the angle of inclination of the compression strut (θ) from Eq 8.2.4.2(5) for calculation of the reinforcement contribution Vus.

The εx value requires the applied design values for M*, V*, N* and T* as well as the flexural reinforcement at that section.

One significant feature of the general approach is that changing the design actions (M*, V*, T*, N*) changes the kv and θ thus affecting the Vuc (which is dependent on kv) and Vus (which is dependent on θ) components. So, taking a random or nominal combination of shear and moment and determining the maximum capacity øVu is erroneous.

Additional longitudinal tension forces caused by shear (and torsion)

Shear and torsion cause additional longitudinal tension forces in the flexural reinforcement and these are dealt with in Cl 8.2.7 and Cl 8.2.8.

Previously in the 2009 standard Cl 8.3.6 dealt with the additional flexural reinforcement as a result of torsion, but shear was not considered.

The applied shear and torsion and shear reinforcement contribution result in an additional shear/torsion induced tensile force in flexural reinforcement, Eq 8.2.7.1(1) & (2).

The coincidental moment and axial then combine with this additional force, Eq 8.2.8.2(1), which results in total longitudinal steel area requirement in Eq 8.2.8.2(2).

The calculation for the effects in the compression zone is also considered in clause 8.2.8.3 which may result in additional compression steel requirements.

The statement below equation 8.2.8.2(1) states that the total steel demand (from moment, axial and shear/torsion forces) is "not more than that required at the section with the maximum tension force demand for flexure".

This limits the additional demand to be no greater than the maximum flexural demand of the beam, and this is shown in figure 8.2.8.

We have observed that both the simplified approach and general approach often result in additional longitudinal reinforcement requirement above the maximum flexural demand. Following discussions with Stephen Foster, UNSW, we have concluded that if moment and shear forces from a section near the support is used, (within the non-flexural or strut-and-tie zone) the results of both methods may not be reliable and will likely indicate a requirement for additional longitudinal tensile reinforcement. It may also erroneously indicate greater shear capacity using the simplified approach compared to the general method, which should not be the case.

Structural Toolkit provides an option called Limit Ttd to remove the limiting requirement. This results in flagging additional flexural reinforcement requirements beyond the maximum flexural demand. Note that when additional flexural reinforcement is required, shear ratios less than one are also flagged as "No Good".

Changing the Limit Ttd option will limit the additional reinforcement requirement and a note may appear stating the additional reinforcement has been limited.

Limit Ttd to max. flexural requirement - Cl 8.2.8.2:

Limit Ttd =	N	(Y)es, (N)o

Limiting Ttd to maximum flexural requirements results in gross overestimation of the simple approach shear capacity and possible overestimation of the general shear capacity

It is also important to ensure that the maximum flexural demand moment is correctly nominated.

Maximum tension force demand for flexure - Cl 8.2.8.2

Section with the maximum demand for flexure (Mmax*)

Considering the above, it is suggested that in situations when the total longitudinal reinforcement demand is greater than the maximum flexural demand, the design action for the section are within the support zone. Thus, it is inappropriate to use the moment and shear at the support or centerline of support, or even 0.7 asup for determining shear capacity

Comparison of approaches

In simple comparisons, we have observed varying differences between the 2009 and 2018 simple and general approaches.

For deep beams, the capacities of the simple approach are comparable to the 2009 approach with a ±3% variation. The capacity for the general approach however, yields a capacity range between 39% less (1.62 ratio) and 30% greater (0.77 ratio).

For typical beams, the capacities of the simple approach are comparable to the 2009 approach with a ±4% variation. The capacity for the general approach however, yields a capacity range between 18% less (1.21 ratio) and 59% greater (0.63 ratio).

For band beams, the capacities of the simple approach are consistently 10%-12% greater than the 2009 approach. The capacity for the general approach however, compared to the simple approach, yields a capacity range varying ±25%.

For slab/footings the capacity of the simple approach for a thin slab (with no ligs of course) is 47% less (1.89 ratio) than the 2009 approach. Increasing the slab thickness results in better performance for the simple approach compared to the 2009 approach. Capacity from the general approach were consistently more than the simple approach (between 28% to 170% greater).

According to Stephen Foster, UNSW, it is expected that the general approach will provide a better result for shear capacity. Where we've observed a reduced capacity from the general approach to the simple approach, it is generally within close proximity to the supports (Within the non-flexural or strut-and-tie zone).

Shear strength limited by web crushing Cl 8.2.3.3

There are significant differences currently between the Bridge Standard AS 5100 and AS 3600 relating to the kc factor. The kc factor is set at 0.55 in AS 3600 while varying according to concrete strength in AS 5100.

In Structural Toolkit, an option is provided to use the Bridge Code kc where applicable.

Shear strength limited by web crushing (kc) - Cl 8.2.3.3, Cl 8.2.3.3 - AS 5100.5:
Use AS5100 = N (Y)es, (N)o

Combined shear and torsion strength limited by web crushing Cl 8.2.3.4

There are also significant differences between the Bridge Standard and AS 3600 relating to the limit for web crushing due to combined shear and torsion effect. AS 5100 sets the limit to 0.2*ø*f'c while AS 3600 limits the strength to øVu.max/(bv*dv). The difference results in an around 30% higher capacities in AS 3600 due to the inclusion of the cot(θv) terms.

In Structural Toolkit, an option is provided to use the limit from the Bridge Code where applicable.

Web crushing limited by shear and torsion - Cl 8.2.3.4, Cl 8.2.4.5 - AS 5100.5:
Use AS5100 = N (Y)es, (N)o

Determination of εx for combined shear and torsion Cl 8.2.4.2.3

In determining εx for a section with torsion, Eq 8.2.4.2.3(1) & (2), a variable Ao is specified without a commentary to increase the shear requirements.

Although it has been stated to us that the code is based on MCFT, the term is almost identical to the ACI formula for combined shear and torsion design which defines Ao as being 0.85*Aoh in Cl 11.5.3.6.

Structural Toolkit, by default sets the value of Ao using the ACI code resulting in a more conservative answer, however, provides an option to set the value of Ao to that of Aoh.

Acp & Ao - ACI Cl 11.5.3.6		
ACI 318 permits Ao to be 0.85*Aoh		
Use ACI =	Y	(Y)es, (N)o

Columns

The strength of columns increases as a result of the capacity reduction factor (ø) as well as a new capacity factor for short columns.

The other notable change is the alpha 1 (α1) for squash load and alpha 2 (α2) for the compression block maximum force are different and are different with Cl 10.6.2.4 stating a linear relationship occurs between these points. Alpha 2 (α2) is applicable when ku is less than the decompression point (where strain at the extreme tensile fibre is 0)

We've noticed some peculiar "kinks" in the capacity curve in this region as a result and have provided an option to tweak this zone by setting α1 = α2. This results in a slight reduction of capacity between the decompression point and the squash load, noting that Cl 10.1.2 still requires a minimum design eccentricity reducing the benefits of the higher α1 capacity. The kinks occur due to the complex behavior of the capacity calculation when ku is becoming increasingly large, close to the squash load.

Uniform compression stress block α1 and α2 factors:		
Interpolating compression α between α2 (where less than decompression) and α1 (where pure axial) can result in irregularities in the diagram between those points.		
A slightly conservative approach which will reduce the axial capacity at min. ecc is to take α1 = α2		
Use α1 = α2 =	N	(Y)es,(N)o

If you wish to take advantage of the improved capacity reduction factor ø for a short column then you must manually change the below setting and this will be flagged in the capacity tables.

Bending with axial compression - Table 2.2.2 (d):		
Short and Q/G ≥ 0.25 =	n	(Y)es,(N)o
øo =	0.60	

Walls

Using the 2018 code, the capacity of walls increase as a result of an increased capacity reduction factor (ø) from 0.6 to 0.65.

The slenderness limit has been changed and now requires a doubly reinforced walls when slenderness ratio exceeds 20 in Cl 11.5.2(c) to improve the ductility in earthquake conditions.

Providing the axial load does not exceed 0.03*f'c*Ag as per Cl 11.1(b)(i), Section 9 (slabs) may be used for the design the walls, providing the ratio of height to thickness does not exceed 50 thus working around the double layer reinforcement requirements (depending on the slab capacity design).

A notable aspect of this section is that there may be situations that the ratio is less than 50, giving a capacity of 0.03*f'c*Ag which exceeds the capacity obtained from Eq 11.5.3 for the axial design strength of a wall.

For example, for a 40MPa wall, 4500mm high, 2 layers of reinforcement and load eccentricity of 30mm, the capacity is 93.6kN/m, while the 0.03*f'c*Ag results in 180kN/m providing the flexural reinforcement requirements are catered for in the slab section.

Another area to be wary of is Cl 11.2.1 stating that the capacity can be derived from Cl 11.5 if the section is in compression over the entire section and within the limits of Cl 11.2.1(a)(i); or as a column which is based on Cl 11.2.1(a)(ii).

This can be dangerous as a slender wall, such as a 150 thick, 4400 high, with 2 layers gives a capacity of 261kN/m using the Eq 11.5.3 and this column has a buckling load Ncx of 200kN/m in accordance with Cl 10.4.4. This wall buckles and does not have the stated capacity when assessed as a column.