Oasys

Oasys GSA

Slab Design – RCSlab

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Introduction

RCSlab is a design postprocessor within GSA for reinforced concrete two-dimensional elements of uniform thickness subject to any combination of in-plane axial or shear force and out-of-plane bending moment and torsion. The calculations can be made following the principles of the most co mmonly used concrete codes. *RCSlab* is unable to allow for out-of-plane shear and through-thickness forces.

The input to the postprocessor comprises applied forces and moments, section depth, reinforcement positions, and material properties. The reinforcement orientations can be in general directions, referred to as θ_1 and θ_2, which need not be orthogonal. The results comprise either areas of reinforcement for each face of the section in the two specified directions, or else an indicator to the effect that *RCSlab* is unable to find a solution for the current data. Early versions of the program were known as RC2D.

Data requirements

Each run of *RCSlab* obtains the following data in any consistent set of units from the GSA analysis or *RCSlab* design data as appropriate:

N_{xx} ultimate applied axial force per unit width in the x-direction

N_{yy} ultimate applied axial force per unit width in the y-direction

M_{xx} ultimate applied bending moment per unit width about the x-axis

M_{yy} ultimate applied bending moment per unit width about the y-axis

N_{xy} ultimate applied in-plane shear force per unit width

M_{xy} ultimate applied torsion moment per unit width

e_{add} additional eccentricity ($e_{add} > 0$) – considered as acting in both senses

e_{min} minimum eccentricity ($e_{min} > 0$) – considered as acting in both senses

h section thickness ($h > 0$)

z_{t1} position of top reinforcement centroid in direction 1 ($0 < z_{t1} < h/2$)

z_{t2} position of top reinforcement centroid in direction 2 ($0 < z_{t2} < h/2$)

z_{b1} position of bottom reinforcement centroid in direction 1($-h/2 < z_{b1} < 0$)

z_{b2} position of bottom reinforcement centroid in direction 2 ($-h/2 < z_{b2} < 0$)

θ_1 angle of reinforcement in direction 1, anticlockwise with respect to x-axis

θ_2 angle of reinforcement in direction 2, anticlockwise with respect to x-axis

$A_{st1,min}$ minimum top reinforcement to be provided in direction 1($0 < A_{st1,min}$)

$A_{st2,min}$ minimum top reinforcement to be provided in direction 2 ($0 < A_{st2,min}$)

$A_{sb1,min}$ minimum bottom reinforcement to be provided in direction 1 ($0 < A_{sb1,min}$)

$A_{sb2,min}$ minimum bottom reinforcement to be provided in direction 2 ($0 < A_{sb2,min}$)

f_{cd} compressive design strength of concrete ($f_{cd} > 0$)

$f_{cd,t}$ compressive design strength of top layer of concrete ($f_{cd} > 0$)

$f_{cd,b}$ compressive design strength of bottom layer of concrete ($f_{cd} > 0$)

f_{cdc} cracked compressive design strength of concrete ($f_{cdc} > 0$)

f_{cdu} uncracked compressive design strength of concrete ($f_{cdu} > 0$)

f_{cdt} tensile design strength of concrete ($f_{cdt} > 0$)

ε_{ctrans} compressive plateau concrete strain $(\varepsilon_{ctrans} \geq 0)$

ε_{cax} maximum axial compressive concrete strain $(\varepsilon_{cax} \geq \varepsilon_{ctrans})$

ε_{cu} maximum flexural compressive concrete strain ($\varepsilon_{cu} \geq \varepsilon_{cax}$)

β proportion of depth to neutral axis over which rectangular stress block acts ($\beta \leq 1$)

$(x/d)_{max}$ maximum value of x/d, the ratio of neutral axis to effective depth, for flexure: $(x/d)_{min} < (x/d)_{max} \leq 0.5/[\beta(0.5 + \min\{zt1, zt2, -zb1, -zb2\}/h)]$

E_s elastic modulus of reinforcement

f_{yd} design strength of reinforcement in tension ($f_{yd} > 0$)

f_{ydc} design strength of reinforcement in compression, ($f_{ydc} > 0$)

f_{lim} maximum linear steel stress of reinforcement ($f_{lim} > 0$)

ε_{plas} yield strain of reinforcement in tension ($\varepsilon_{plas} > 0$)

ε_{plasc} yield strain of reinforcement in compression ($\varepsilon_{plasc} > 0$)

ε_{su} design value of maximum strain in reinforcement

φ_Δ maximum permitted angle between applied and resulting principal stress

In addition, the program needs to know whether to use, where appropriate, the faster approach and, if so, what the maximum area of reinforcement so calculated should be before the rigorous approach is used.

Within *RCSlab* the reinforcement positions are measured with respect to the mid-height of the section, the positions being measured positively upwards. The reinforcement angles are specified with respect to the x-axis and measured positively in an anticlockwise direction looking from above. It should be noted that the concrete is assumed to have zero tensile strength in the analysis; the tensile strength, f_{cdt}, is only used to calculate the compressive strength when tensile strains are present.

The results of each run consist of the required area of reinforcement, negative if tensile, in each direction in the top and bottom faces or an error flag indicating that a solution could not be found.

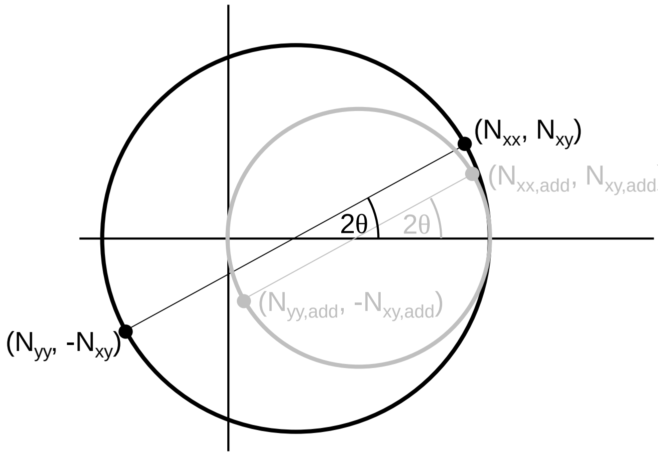
RCSlab analysis procedure

The following su mmarizes the procedure followed by *RCSlab*:

1. Adjust, where necessary, the applied moments for minimum eccentricities.
2. Split the section into three layers with the central layer unstressed and the outer layers taking in-plane stresses, the thicknesses corresponding to an acceptable neutral axis depth; calculate the stresses applied to each layer.
3. Calculate the stress to be taken by the concrete in each layer and the stress from each layer to be taken by reinforcement.
4. Calculate the force to be taken by each of the four sets of reinforcement (two faces, two directions) taking into account their positions relative to the layers.
5. Determine section strains co MPatible with the neutral axis depths implied by the layer thicknesses in 5.0.2 and the concrete strains in the outer layers from 5.0.3 for top and bottom layers.
6. Determine reinforcement strains co MPatible with the section strains.
7. From the strains, calculate the stress in each of the four sets of reinforcement.
8. Knowing the force to be taken by each of the four sets of reinforcement and the stress in each set of reinforcement, calculate the reinforcement areas required; these should not be less than the specified minimum values.
9. Repeat as necessary from 5.0.2, adjusting the layer thicknesses to achieve the minimum total area of reinforcement.
10. Where in-plane effects dominate, repeat from 5.0.2 adopting a model with the central layer stressed.
11. The design reinforcement areas correspond to the layer arrangement giving the minimum total area of reinforcement.
12. To speed up the calculation, an option is available to adopt a non-iterative technique where the loading is primarily either in-plane or out-of-plane. This approach is likely to lead to slightly more conservative results. The user can choose to use this approach in appropriate situations and can specify a total area of reinforcement, as a percentage of the cross-sectional area, above which a rigorous, iterative solution is used.

Inclusion of moments resulting from additional and minimum eccentricities

The applied moments are adjusted to take into account the additional and minimum eccentricities of applied axial forces. The additional eccentricity, which can be used to model tolerances and second-order effects, is determined by the user; applied bending moments are increased by compressive principal axial forces but are not adjusted for tensile principal axial forces. The components of in-plane force in the orthogonal directions for use with the additional eccentricity, $N_{xx,add}$, $N_{yy,add}$ and $N_{xy,add}$, are calculated assuming the angle between the principal direction and the x-axis is unchanged.



The default value of the minimum eccentricity, which can be overwritten, is taken from the chosen design code; this value, and all other code-dependent values, are given in Appendix 3. If the absolute value of the applied moment exceeds the sum of the additional and minimum eccentricity moments for Mxx, Myy and Mxy, then the applied moments are increased in magnitude by their respective additional moments. Otherwise two sets of applied moments are calculated, corresponding to eccentricities applied in the two senses.

Where $|M + j.N_{add}.e_{add}| > |N.e_{min}|$, the design moment $M_d = M + j.N_{add}.e_{add}$;

otherwise, $M_d = M + j.(N_{add}.e_{add} + N.e_{min})$ but restricted to the range $-|N.e_{min}|$ to $|N.e_{min}|$

For example, if the applied, additional and minimum eccentricity moments were 75kNm, 50kNm and 60kNm respectively, the design moments for the two sets would be 75-50-60 = -35kNm and 75+50 = 125kNm respectively. It should be noted that no specific allowance is made for slenderness.

Distribution of reinforcement

RCSlab calculates the area of reinforcement required at each node. Since the reinforcement distribution corresponds to the force and moment distributions with their concentrations and peaks, there may be locations where no satisfactory reinforcement arrangement can be determined because the concrete is overstressed in shear. If these points, which are left black when contouring, are isolated, they can probably be ignored but larger areas will require changes to the geometry or material properties.

It is also usually appropriate to average values of reinforcement in areas of great change. For example, reinforcement requirements in flat slabs can be averaged over the central half of the column strips, the outer portions of the column strips and the middle strips, as when following code methods. It is hoped that future developments within GSA will help automate this averaging process.

Concrete code related data

Codes with strength reduction factors

	ACI318-08	ACI318-11	ACI318-14	AS3600
Concrete strength	f_c'	f_c'	f_c'	f'_c
Steel strength	f_y	f_y	f_y	f_{sy}
Strength reduction factor for axial compression* - ϕ_c	$\phi = 0.65$ [9.3.2.2]	$\phi = 0.65$ [9.3.2.2]	$\phi = 0.65$ [21.2.2]	$\phi = 0.6$ [Table 2.2.2]
Strength reduction factor for axial tension* - ϕ_t	$\phi = 0.9$ [9.3.2.1]	$\phi = 0.9$ [9.3.2.1]	$\phi = 0.9$ [21.2.2]	$\phi = 0.8$ (N bars) $\phi = 0.64$ (L bars) [Table 2.2.2]
Uncracked concrete design strength for rectangular stress block f_{cdu}	$0.85\ f_c'$ [10.2.7.1]	$0.85\ f_c'$ [10.2.7.1]	$0.85\ f_c'$ [22.2.2.4.1]	$\alpha_2 f'_c$ Where $\alpha_2 = 1.00-0.003\ f'_c$ but within limits 0.67 to 0.85 [10.6.2.5(b)]
Cracked concrete design strength (equal to twice the upper limit on shear strength) f_{cdc}	$(5/3)\sqrt{f_c'}$ (f_c' in MPa) *$20\sqrt{f_c'}$ (f_c' in psi)* [11.2.1.1 & 11.4.7.9]	$1.66\sqrt{f_c'}$ (f_c' in MPa) *$20\sqrt{f_c'}$ (f_c' in psi_* [11.2.1.1 & 11.4.7.9 11.9.3]	$1.66\sqrt{f_c'}$ (f_c' in MPa) *$20\sqrt{f_c'}$ (f_c' in psi)* [11.5.4.3]	$0.4\ f'_c$ [11.6.2]

	ACI318-08	ACI318-11	ACI318-14	AS3600
Concrete tensile design strength (used only to determine whether section cracked) f_{cdt}	$(1/3)\sqrt{f_c'}$ (f_c' in MPa) *$4\sqrt{f_c'}$ (f_c' in psi)* [11.3.3.2]	$0.33\sqrt{f_c'}$ (f_c' in MPa) *$4\sqrt{f_c'}$ (f_c' in psi)* [11.3.3.2]	$0.33\sqrt{f_c'}$ (f_c' in MPa) *$4\sqrt{f_c'}$ (f_c' in psi)* [22.5.8.3.3]	$0.36\sqrt{f_c'}$ [3.1.1.3]
Compressive plateau concrete strain ε_{ctrans}	0.002 [assumed]	0.002 [assumed]	0.002 [assumed]	0.002 [assumed]
Maximum axial compressive concrete strain ε_{cax}	0.003 [10.2.3]	0.003 [10.2.3]	0.003 [22.2.2.1]	0.0025 [10.6.2.2(b)]
Maximum flexural compressive concrete strain ε_{cu}	0.003 [10.2.3]	0.003 [10.2.3]	0.003 [22.2.2.1]	0.003 [8.1.2.(d)]
Proportion of depth to neutral axis over which constant stress acts β	$0.85-0.05(f_c'-30)/7$ (f_c' in MPa) *$0.85-0.05(f_c'/1000-4)$* *(f_c' in psi)* but within limits 0.65 to 0.85 [10.2.7.3] β_1	$0.85-0.05(f_c'-28)/7$ (f_c' in MPa) *$0.85-0.05(f_c'/1000-4)$* *(f_c' in psi)* but within limits 0.65 to 0.85 [10.2.7.3] β_1	$0.85-0.05(f_c'-28)/7$ (f_c' in MPa) *$0.85-0.05(f_c'/1000-4)$* *(f_c' in psi)* but within limits 0.65 to 0.85 [22.2.2.4.3] β_1	$1.05-0.007 f_c'$ but within limits 0.67 to 0.85 [10.6.2.5(b)] γ

	ACI318-08	ACI318-11	ACI318-14	AS3600
Maximum value of ratio of depth to neutral axis to effective depth in flexural situations $(x/d)_{max}$	$\frac{1}{(1+0.004/\varepsilon_{cu})}$ [10.3.5] $(c/d)_{max}$	$\frac{1}{(1+0.004/\varepsilon_{cu})}$ [10.3.5] $(c/d)_{max}$	$\frac{1}{(1+0.004/\varepsilon_{cu})}$ [7.3.3.1 & 8.3.3.1] $(c/d)_{max}$	0.36 [8.1.5] $k_{u.max}$
Elastic modulus of steel E_s	200 GPa [8.5.2]	200 GPa [8.5.2]	200 GPa [20.2.2.2]	200 GPa [3.2.2(a)]
Design strength of reinforcement in tension f_{yd}	f_y [10.2.4]	f_y [10.2.4]	f_y [20.2.2.1]	f_{sy} [3.2.1]
Design strength of reinforcement in compression f_{ydc}	f_y [10.2.4]	f_y [10.2.4]	f_y [20.2.2.1]	f_{sy} [3.2.1]
Maximum linear steel stress f_{lim}	f_y [10.2.4]	f_y [10.2.4]	f_y [20.2.2.1]	f_{sy} [3.2.1]
Yield strain in tension ε_{plas}	f_y/E_s [10.2.4]	f_y/E_s [10.2.4]	f_y/E_s [20.2.2.1]	f_{sy}/E_s [3.2.1]
Yield strain in compression ε_{plasc}	f_y/E_s [10.2.4]	f_y/E_s [10.2.4]	f_y/E_s [20.2.2.1]	f_{sy}/E_s [3.2.1]
Design strain limit ε_{su}	[0.01] assumed	[0.01] assumed	[0.01] assumed	Class N 0.05 Class L 0.015 [3.2.1]

	ACI318-08	ACI318-11	ACI318-14	AS3600
Maximum concrete strength	-	-	-	-
Maximum steel strength	-	-	-	$f_{sy} \leq 500$ MPa [3.2.1]
Minimum eccentricity	0.10 h [R10.3.6 & R10.3.7]	0.10 h [R10.3.6 & R10.3.7]	0.10 h [R22.4.2.1]	0.05 h [10.1.2]
Minimum area compression reinforcement	-	-	-	1% (0.5% each face) [10.7.1 (a)]
maximum permitted angle between applied and resulting principal stress φ_{Δ}	-	-	-	-

*Applied forces and moments are divided by the strength reduction factor to obtain design values for use within RCSlab. The appropriate vales are determined as follows:

$M = abs(M_{xx} + M_{yy})/2 + \sqrt{[(M_{xx} - M_{yy})^2/4 + M_{xy}^2]}$

$N = (N_{xx} + N_{yy})/2 + \sqrt{[(N_{xx} - N_{yy})^2/4 + N_{xy}^2]}$

$z_{min} = \min\{z_{t1}, z_{t2}, -z_{b1}, -z_{b2}\}$

ACI318

$k_{uc} = \varepsilon_{cu}/(\varepsilon_{cu} + f_{yd}/E_s)$
$k_{ut} = \varepsilon_{cu}/(\varepsilon_{cu} + 0.005)$
$M_c = \phi_c k_{uc} \beta f_{cdc} \times (1 - k_{uc}\beta/2) \times (h/2 + z_{min})^2 - N \times z_{min}$
$M_t = \phi_t k_{ut} \beta f_{cdc} \times (1 - k_{ut}\beta/2) \times (h/2 + z_{min})^2 - N \times z_{min}$

If $M \le M_t : \phi = \phi_t$

If $M \ge M_c : \phi = \phi_c$

Otherwise:

$$\phi = \frac{\left[(M_c - M)\phi_t + (M - M_t)\phi_c\right]}{(M_c - M_t)}$$

AS3600

$k_{uc} = (1.19 - \phi_c) \times 12/13$
$k_{ut} = (1.19 - \phi_t) \times 12/13$
$k_{ub} = \varepsilon_{cu}/(\varepsilon_{cu} + f_{yd}/E_s)$
$M_c = \phi_c k_{uc} \beta f_{cdc} \times (1 - k_{uc}\beta/2) \times (h/2 + z_{min})^2 - \min(0, N) \times z_{min}$
$M_t = \phi_t k_{ut} \beta f_{cdc} \times (1 - k_{ut}\beta/2) \times (h/2 + z_{min})^2 - \min(0, N) \times z_{min}$
$N_b = [\phi_c k_{ub} \beta f_{cdc} \times (1 - k_{ub}\beta/2) \times (h/2 + z_{min})^2 - M] / z_{min}$

If $M \le M_t : \phi_b = \phi_t$

If $M \ge M_c : \phi_b = \phi_c$

Otherwise:

$$\phi_b = \frac{\left[(M_c - M)\phi_t + (M - M_t)\phi_c\right]}{(M_c - M_t)}$$

If $N \le 0 : \phi = \phi_t$

If $N \ge N_b : \phi = \phi_c$

Otherwise:

$$\phi = \phi_b \frac{\left(1 + \sqrt{\left[1 - 4(\phi_b - \phi_c) \times (N/N_b)/\phi_b^2\right]}\right)}{2}$$

Current codes with partial safety factors on materials

	EN1992-1-1 2004 +A1:2014	EN1992-2 2005	Hong Kong Buildings 2013	Hong Kong Structural Design Manual for Highways and Railways 2013	Indian concrete road bridge IRC:112 2011	Indian concrete rail bridge IRS 1997	Indian building IS456
Concrete strength	f_{ck}	f_{ck}	f_{cu}	$f_{ck,cube}$	f_{ck}	f_{ck}	f_{ck}
Steel strength	f_{yk}	f_{yk}	f_y	f_{yk}	f_{yk}	f_y	f_y
Partial safety factor on concrete	$\gamma_C = 1.5$ [2.4.2.4(1)]	$\gamma_C = 1.5$ [2.4.2.4(1)]	$\gamma_{mc} = 1.5$ [Table 2.2]	$\gamma_c = 1.5$ [5.1]	$\gamma_C = 1.5$ [A2.10]	$\gamma_C = 1.5$ [15.4.2.1(b)]	$\gamma_{mc} = 1.5$ [36.4.2.1]
Partial safety factor on steel	$\gamma_S = 1.15$ [2.4.2.4(1)]	$\gamma_S = 1.15$ [2.4.2.4(1)]	$\gamma_{ms} = 1.15$ [Table 2.2]	$\gamma_s = 1.15$ [5.1]	$\gamma_S = 1.15$ [Fig 6.2]	$\gamma_m = 1.15$ [15.4.2.1(d)]	$\gamma_{ms} = 1.15$ [36.4.2.1]
Uncracked concrete design strength for rectangular stress block f_{cdu}	$f_{ck} \leq 50$ MPa $\alpha_{cc}f_{ck}/\gamma_C$ $f_{ck} > 50$ MPa $(1 - (f_{ck}-50)/200) \times \alpha_{cc}f_{ck}/\gamma_C$ α_{cc} is an NDP* [3.1.7(3)] ηf_{cd}	$f_{ck} \leq 50$ MPa $\alpha_{cc}f_{ck}/\gamma_C$ $f_{ck} > 50$ MPa $(1 - (f_{ck}-50)/200) \times \alpha_{cc}f_{ck}/\gamma_C$ α_{cc} is an NDP* [3.1.7(3)] ηf_{cd}	$0.67f_{cu}/\gamma_{mc}$ [Figure 6.1]	$0.67f_{ck,cube}/\gamma_c$ [Figure 5.3]	$f_{ck} \leq 60$ MPa $0.67f_{ck}/\gamma_C$ $f_{ck} > 60$ MPa $(1.24-f_{ck}/250) \times 0.67f_{ck}/\gamma_C$ [6.4.2.8 A2.9(2)] ηf_{cd}	$0.60f_{ck}/\gamma_{mc}$ [15.4.2.1(b)]	$0.67f_{ck}/\gamma_{mc}$ [Figure 21]

	EN1992-1-1 2004 +A1:2014	EN1992-2 2005	Hong Kong Buildings 2013	Hong Kong Structural Design Manual for Highways and Railways 2013	Indian concrete road bridge IRC:112 2011	Indian concrete rail bridge IRS 1997	Indian building IS456
Cracked concrete design strength (equal to twice the upper limit on shear strength) f_{cdc}	$0.6\times(1-f_{ck}/250)\times f_{ck}/\gamma_C$ [6.2.2(6)] νf_{cd}	$0.312\times(1-f_{ck}/250)\times f_{ck}/\gamma_C$ [6.109 (103)iii] (see also φ_Δ) νf_{cd}	$\min\{17.5, 2\sqrt{[f_{cu}]}\} /\gamma_{mc}^{0.55}$ [6.1.2.5(a)]	$0.6\times(1-0.8f_{ck,cube}/250)\times 0.8f_{ck,cube}/\gamma_C$ [5.1]	$f_{ck} \leq 80$ MPa $0.6\times0.67f_{ck}/\gamma_C$ 80 MPa< $f_{ck} \leq$ 100 MPa $(0.9-f_{ck}/250)\times 0.67f_{ck}/\gamma_C$ $f_{ck} > 100$ MPa $0.5\times0.67f_{ck}/\gamma_C$ [10.3.3.2] $\nu_1 f_{cd}$	$\min\{11.875, 1.875\sqrt{[f_{ck}]}\}/ \gamma_{mc}^{0.55}$ [15.4.3.1]	$1.6\sqrt{[f_{ck}]} / \gamma_{mc}^{0.55}$ [Table 20]
Concrete tensile design strength (used only to determine whether section cracked) f_{cdt}	$f_{ck} \leq 50$ MPa $\alpha_{ct}\times0.21 f_{ck}^{2/3}/\gamma_C$ $f_{ck} > 50$ MPa $\alpha_{ct}\times1.48\times \ln[1.8+f_{ck}/10] /\gamma_C$ α_{ct} is an NDP* [Table 3.1] f_{ctd}	$f_{ck} \leq 50$ MPa $\alpha_{ct}\times0.21 f_{ck}^{2/3}/\gamma_C$ $f_{ck} > 50$ MPa $\alpha_{ct}\times1.48\times \ln[1.8+f_{ck}/10] /\gamma_C$ α_{ct} is an NDP* [Table 3.1] f_{ctd}	$0.36\sqrt{[f_{cu}]}/ \gamma_{mc}$ [12.3.8.4]	$f_{ck} \leq 60$ MPa $[0.025f_{ck,cube} + 0.6] /\gamma_C$ $f_{ck} > 60$ MPa $2.1 /\gamma_C$ [Table 5.1]	$f_{ck} \leq 60$ MPa $0.1813f_{ck}^{2/3}/\gamma_C$ $f_{ck} > 60$ MPa $1.589\times \ln[1.8+f_{ck}/12.5]/\gamma_C$ [A2.2] f_{ctd}	$0.36\sqrt{[f_{ck}]}/ \gamma_{mc}$ [16.4.4.2]	$0.5\sqrt{[f_{ck}]}/ \gamma_{mc}$ [From 6.2.2 (70% of SLS value / γ_{mc})]

	EN1992-1-1 2004 +A1:2014	EN1992-2 2005	Hong Kong Buildings 2013	Hong Kong Structural Design Manual for Highways and Railways 2013	Indian concrete road bridge IRC:112 2011	Indian concrete rail bridge IRS 1997	Indian building IS456
Compressive plateau concrete strain ε_{ctrans} ε_{ctrans}	$f_{ck} \le 50$ MPa 0.00175 $f_{ck} > 50$ MPa $0.00175 + 0.00055 \times [(f_{ck}-50)/40]$ [Table 3.1] ε_{c3}	$f_{ck} \le 50$ MPa 0.00175 $f_{ck} > 50$ MPa $0.00175 + 0.00055 \times [(f_{ck}-50)/40]$ [Table 3.1] ε_{c3}	0.002 [assumed]	$[0.026 f_{ck,cube} + 1.1] / \gamma_C$ [5.2.6(1) & Table 5.1] ε_{c2}	$f_{ck} \le 60$ MPa 0.0018 $f_{ck} > 60$ MPa $0.00175 + 0.00055 \times [(0.8 f_{ck}-50)/ 40]$ [Table 6.5 & A2.2] ε_{c3}	0.002 [assumed]	0.002 [Figure 21]
Maximum axial compressive concrete strain ε_{cax}	$f_{ck} \le 50$ MPa 0.00175 $f_{ck} > 50$ MPa $0.00175 + 0.00055 \times [(f_{ck}-50)/40]$ [Table 3.1] ε_{c3}	$f_{ck} \le 50$ MPa 0.00175 $f_{ck} > 50$ MPa $0.00175 + 0.00055 \times [(f_{ck}-50)/40]$ [Table 3.1] ε_{c3}	$f_{cu} \le 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035 - 0.00006 \times \sqrt{[f_{cu}-60]}$ [Figure 6.1]	$[0.026 f_{ck,cube} + 1.1] / \gamma_C$ [5.2.6(1) & Table 5.1] ε_{c2}	$f_{ck} \le 60$ MPa 0.0018 $f_{ck} > 60$ MPa $0.00175 + 0.00055 \times [(0.8 f_{ck}-50)/ 40]$ [Table 6.5 & A2.2] ε_{c3}	0.0035 [15.4.2.1(b)]	0.002 [39.1a]
Maximum flexural compressive concrete strain ε_{cu}	$f_{ck} \le 50$ MPa 0.0035 $f_{ck} > 50$ MPa $0.0026 + 0.035 \times [(90-f_{ck})/ 100]^4$ [Table 3.1] ε_{cu3}	$f_{ck} \le 50$ MPa 0.0035 $f_{ck} > 50$ MPa $0.0026 + 0.035 \times [(90-f_{ck})/ 100]^4$ [Table 3.1] ε_{cu3}	$f_{cu} \le 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035 - 0.00006 \times \sqrt{[f_{cu}-60]}$ [Figure 6.1]	$f_{ck,cube} \le 60$ MPa 0.0035 $f_{ck,cube} > 60$ MPa $0.0035 - 0.00006 \times \sqrt{[f_{ck,cube}-60]}$ [5.2.6(1)]	$f_{ck} \le 60$ MPa 0.0035 $f_{ck} > 60$ MPa $0.0026 + 0.035 \times [(90-0.8 f_{ck})/ 100]^4$ [Table 6.5 & A2.2] ε_{cu3}	0.0035 [15.4.2.1(b)]	0.0035 [38.1b]

	EN1992-1-1 2004 +A1:2014	EN1992-2 2005	Hong Kong Buildings 2013	Hong Kong Structural Design Manual for Highways and Railways 2013	Indian concrete road bridge IRC:112 2011	Indian concrete rail bridge IRS 1997	Indian building IS456
Proportion of depth to neutral axis over which constant stress acts β	$f_{ck} \leq 50$ MPa 0.8 $f_{ck} > 50$ MPa $0.8-(f_{ck}-50)/400$ [3.1.7(3)] λ	$f_{ck} \leq 50$ MPa 0.8 $f_{ck} > 50$ MPa $0.8-(f_{ck}-50)/400$ [3.1.7(3)] λ	$f_{cu} \leq 45$ MPa 0.9 $45 < f_{cu} \leq 70$ 0.8 $f_{cu} > 70$ MPa 0.72 [Figure 6.1]	$f_{ck,cube} \leq 45$ MPa 0.9 $45 < f_{ck,cube} \leq 70$ 0.8 $70 < f_{ck,cube} \leq 85$ 0.72 [Figure 5.3]	$f_{ck} \leq 60$ MPa 0.8 $f_{ck} > 60$ MPa $0.8-(f_{ck}-60)/500$ [A2.9(2)] λ	1.0 [15.4.2.1(b)]	0.84 [38.1c]
Maximum value of ratio of depth to neutral axis to effective depth in flexural situations $(x/d)_{max}$	$f_{ck} \leq 50$ MPa $(1-k_1)/k_2$ $f_{ck} > 50$ MPa $(1-k_3)/k_4$ k_1, k_2, k_3 and k_4 are NDPs* [5.5(4)]	$f_{ck} \leq 50$ MPa $(1-k_1)/k_2$ $f_{ck} > 50$ MPa $(1-k_3)/k_4$ k_1, k_2, k_3 and k_4 are NDPs* [5.5(104)]	$f_{cu} \leq 45$ MPa 0.50 $45 < f_{cu} \leq 70$ 0.40 $f_{cu} > 70$ MPa 0.33 [6.1.2.4(b)]	$f_{ck} \leq 50$ MPa 0.344 $f_{ck} > 50$ MPa $0.6/\{0.6 + 0.4/(2.6 + 35[(90-f_{ck})/100]^4)\}$ [5.1]	[upper limit]	$\dfrac{1}{(1+\varepsilon_s/\varepsilon_{cu})}$ where $\varepsilon_s = 0.002 + f_y/(E_s\gamma_m)$ [15.4.2.1(d)]	$f_y = 250$ 0.53 $f_y = 415$ 0.48 $f_y = 500$ 0.46 [38.1f] $x_{u.max}/d$
Elastic modulus of steel E_s	200 GPa [3.2.7(4)] E_s	200 GPa [3.2.7(4)] E_s	200 GPa [Figure 3.9]	200 GPa [5.1] E_s	200 GPa [6.2.2] E_s	200 GPa [Figure 4B] E_s	200 GPa [Figure 23B]
Design strength of reinforcement in tension f_{yd}	f_{yk}/γ_s [3.2.7(2)] f_{yd}	f_{yk}/γ_s [3.2.7(2)] f_{yd}	f_y/γ_{ms} [Figure 3.9]	f_{yk}/γ_s [5.1]	f_{yk}/γ_s [6.2.2] f_{yd}	f_y/γ_m [Figure 4B]	f_y/γ_{ms} [Figure 23B]

	EN1992-1-1 2004 +A1:2014	**EN1992-2 2005**	**Hong Kong Buildings 2013**	**Hong Kong Structural Design Manual for Highways and Railways 2013**	**Indian concrete road bridge IRC:112 2011**	**Indian concrete rail bridge IRS 1997**	**Indian building IS456**
Design strength of reinforcement in compression f_{ydc}	f_{yk}/γ_s [3.2.7(2)] f_{yd}	f_{yk}/γ_s [3.2.7(2)] f_{yd}	f_y/γ_{ms} [Figure 3.9]	f_{yk}/γ_s [5.1]	f_{yk}/γ_s [6.2.2] f_{yd}	$(f_y/\gamma_m)/[1+ (f_y/\gamma_m)/2000]$ [15.6.3.3] f_{yc}/γ_m	f_y/γ_{ms} [Figure 23B]
Maximum linear steel stress f_{lim}	f_{yk}/γ_s [3.2.7(2)]	f_{yk}/γ_s [3.2.7(2)]	f_y/γ_{ms} [Figure 3.9]	f_{yk}/γ_s [5.1]	f_{yk}/γ_s [6.2.2]	$0.8f_y/\gamma_m$ [Figure 4B]	f_y/γ_{ms} [Figure 23B]
Yield strain in tension ε_{plas}	$f_{yk}/(\gamma_s E_s)$ [3.2.7(2)]	$f_{yk}/(\gamma_s E_s)$ [3.2.7(2)]	$f_y/(\gamma_{ms} E_s)$ [Figure 3.9]	$f_{yk}/(\gamma_s E_s)$ [5.1]	$f_{yk}/(\gamma_s E_s)$ [6.2.2]	$f_y/(\gamma_m E_s) + 0.002$ [Figure 4B]	$f_y/(\gamma_{ms} E_s)$ [Figure 23B]
Yield strain in compression ε_{plasc}	$f_{yk}/(\gamma_s E_s)$ [3.2.7(2)]	$f_{yk}/(\gamma_s E_s)$ [3.2.7(2)]	$f_y/(\gamma_{ms} E_s)$ [Figure 3.9]	$f_{yk}/(\gamma_s E_s)$ [5.1]	$f_{yk}/(\gamma_s E_s)$ [6.2.2]	0.002 [assumed]	$f_y/(\gamma_{ms} E_s)$ [Figure 23B]
Design strain limit ε_{su}	NDP* [ε_{ud}]	NDP* [ε_{ud}]	$(10\beta-1)\times\varepsilon_{cu}$ [6.1.2.4(a) (v)]	Grade 250 0.45 Grade 500B 0.045 Grade 500C 0.0675 [5.1(1) & 5.3(1) CS2:2012 Table 5 UKNA EN1992-1-1]	[0.01] assumed	[0.01] assumed	[0.01] assumed

	EN1992-1-1 2004 +A1:2014	EN1992-2 2005	Hong Kong Buildings 2013	Hong Kong Structural Design Manual for Highways and Railways 2013	Indian concrete road bridge IRC:112 2011	Indian concrete rail bridge IRS 1997	Indian building IS456
Maximum concrete strength	$f_{ck} \leq 90$ MPa [3.1.2(2)]	$f_{ck} \leq 90$ MPa [3.1.2(2)]	$f_{cu} \leq 100$ MPa [TR 1]	$f_{ck,cube} \leq 85$ MPa [5.2.1(2)] C_{max}	$f_{ck} \leq 110$ MPa [A2.9(2)]	$f_{ck} \leq 60$ MPa [Table 2]	$f_{ck} \leq 80$ MPa [Table 2]
Maximum steel strength	$f_{yk} \leq 600$ MPa [3.2.2(3)]	$f_{yk} \leq 600$ MPa [3.2.2(3)]	$f_y = 500$ MPa [Table 3.1]	$f_{yk} \leq 600$ MPa [5.1]	$f_{yk} \leq 600$ MPa [Table 6.1]	-	$f_y \leq 500$ MPa [5.6]
Minimum eccentricity	max{h/30, 20 mm} [6.1(4)]	max{h/30, 20 mm} [6.1(4)]	min{h/20, 20 mm} [6.2.1.1(d)]	max{h/30, 20 mm} [5.1]	0.05 h [7.6.4.2]	min{0.05 h, 20 mm} [15.6.3.1]	max{h/30, 20 mm} [25.4]
Minimum area compression reinforcement	-	-	-	-	-	-	-
Maximum permitted angle between applied and resulting principal stress φ_Δ	-	$\lvert\theta-\theta_{el}\rvert = 15°$ [6.109 (103)iii] (see also f_{cdc})	-	-	-	-	-

* NDPs are nationally determined parameters.

Superseded codes with partial safety factors on materials

	BS8110 1997* & Concrete Society TR49	BS8110 1997 (Rev 2005) & Concrete Society TR49	BS5400 Part 4 & Concrete Society TR49	Hong Kong Buildings 2004#	Hong Kong Buildings 2004 AMD1 2007	Hong Kong Highways 2006
Concrete strength	f_{cu}	f_{cu}	f_{cu}	f_{cu}	f_{cu}	f_{cu}
Steel strength	f_y	f_y	f_y	f_y	f_y	f_y
Partial safety factor on concrete	$\gamma_{mc} = 1.5$ [2.4.4.1]	$\gamma_{mc} = 1.5$ [2.4.4.1]	$\gamma_{mc} = 1.5$ [4.3.3.3]	$\gamma_{mc} = 1.5$ [Table 2.2]	$\gamma_{mc} = 1.5$ [Table 2.2]	$\gamma_{mc} = 1.5$ [4.3.3.3]
Partial safety factor on steel	$\gamma_{ms} = 1.05$ [2.4.4.1]	$\gamma_{ms} = 1.15$ [2.4.4.1]	$\gamma_{ms} = 1.15$ [4.3.3.3]	$\gamma_{ms} = 1.15$ [Table 2.2]	$\gamma_{ms} = 1.15$ [Table 2.2]	$\gamma_{ms} = 1.15$ [4.3.3.3]
Uncracked concrete design strength for rectangular stress block f_{cdu}	$0.67\ f_{cu}/\gamma_{mc}$ [Figure 3.3]	$0.67\ f_{cu}/\gamma_{mc}$ [Figure 3.3]	$0.60\ f_{cu}/\gamma_{mc}$ [5.3.2.1(b)]	$0.67\ f_{cu}/\gamma_{mc}$ [Figure 6.1]	$0.67\ f_{cu}/\gamma_{mc}$ [Figure 6.1]	$0.60\ f_{cu}/\gamma_{mc}$ [5.3.2.1(b)]

	BS8110 1997* & Concrete Society TR49	**BS8110 1997 (Rev 2005) & Concrete Society TR49**	**BS5400 Part 4 & Concrete Society TR49**	**Hong Kong Buildings 2004#**	**Hong Kong Buildings 2004 AMD1 2007**	**Hong Kong Highways 2006**
Cracked concrete design strength (equal to twice the upper limit on shear strength) f_{cdc}	$2\sqrt{[f_{cu}]}/\gamma_{mc}^{0.55}$ [3.4.5.2 & TR 3.1.4]	$2\sqrt{[f_{cu}]}/\gamma_{mc}^{0.55}$ [3.4.5.2 & TR 3.1.4]	min {11.875, $1.875\sqrt{[f_{cu}]}\}/\gamma_{mc}^{0.55}$ [5.3.3.3]	min{17.5, $2\sqrt{[f_{cu}]}\}/\gamma_{mc}^{0.55}$ [6.1.2.5(a)]	min{17.5, $2\sqrt{[f_{cu}]}\}/\gamma_{mc}^{0.55}$ [6.1.2.5(a)]	min {11.875, $1.875\sqrt{[f_{cu}]}\}/\gamma_{mc}^{0.55}$ [5.3.3.3]
Concrete tensile design strength (used only to determine whether section cracked) f_{cdt}	$0.36\sqrt{[f_{cu}]}/\gamma_{mc}$ [4.3.8.4]	$0.36\sqrt{[f_{cu}]}/\gamma_{mc}$ [4.3.8.4]	$0.36\sqrt{[f_{cu}]}/\gamma_{mc}$ [6.3.4.2]	$0.36\sqrt{[f_{cu}]}/\gamma_{mc}$ [12.3.8.4]	$0.36\sqrt{[f_{cu}]}/\gamma_{mc}$ [12.3.8.4]	$0.36\sqrt{[f_{cu}]}/\gamma_{mc}$ [6.3.4.2]
Compressive plateau concrete strain ε_{ctrans}	0.002 [assumed]	0.002 [assumed]	0.002 [assumed]	0.002 [assumed]	0.002 [assumed]	0.002 [assumed]

	BS8110 1997* & Concrete Society TR49	**BS8110 1997 (Rev 2005) & Concrete Society TR49**	**BS5400 Part 4 & Concrete Society TR49**	**Hong Kong Buildings 2004#**	**Hong Kong Buildings 2004 AMD1 2007**	**Hong Kong Highways 2006**
Maximum axial compressive concrete strain ε_{cax}	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.001\times[(f_{cu}-60)/50]$ [TR49 3.1.3]	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.001\times[(f_{cu}-60)/50]$ [TR49 3.1.3]	$f_{cu} \leq 60$ MPa 0.0035 [5.3.2.1(b)] $f_{cu} > 60$ MPa $0.0035-0.001\times[(f_{cu}-60)/50]$ [TR49 3.1.3]	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.00006\times\sqrt{[f_{cu}-60]}$ [Figure 6.1]	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.00006\times\sqrt{[f_{cu}-60]}$ [Figure 6.1]	0.0035 [5.3.2.1(b)]
Maximum flexural compressive concrete strain ε_{cu}	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.001\times[(f_{cu}-60)/50]$ [TR49 3.1.3]	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.001\times[(f_{cu}-60)/50]$ [TR49 3.1.3]	0.0035 [5.3.2.1(b)]	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.00006\times\sqrt{[f_{cu}-60]}$ [Figure 6.1]	$f_{cu} \leq 60$ MPa 0.0035 $f_{cu} > 60$ MPa $0.0035-0.00006\times\sqrt{[f_{cu}-60]}$ [Figure 6.1]	0.0035 [5.3.2.1(b)]
Proportion of depth to neutral axis over which constant stress acts β	0.9 [Figure 3.3]	0.9 [Figure 3.3]	1.0 [5.3.2.1(b)]	0.9 [Figure 6.1]	$f_{cu} \leq 45$ MPa 0.9 $45 < f_{cu} \leq 70$ 0.8 $f_{cu} > 70$ MPa 0.72 [Figure 6.1]	1.0 [5.3.2.1(b)]

	BS8110 1997[*] & Concrete Society TR49	**BS8110 1997 (Rev 2005) & Concrete Society TR49**	**BS5400 Part 4 & Concrete Society TR49**	**Hong Kong Buildings 2004[#]**	**Hong Kong Buildings 2004 AMD1 2007**	**Hong Kong Highways 2006**
Maximum value of ratio of depth to neutral axis to effective depth in flexural situations $(x/d)_{max}$	[upper limit] $(x/d)_{max}$	[upper limit] $(x/d)_{max}$	$\frac{1}{(1+\varepsilon_s/\varepsilon_{cu})}$ where $\varepsilon_s = 0.002 + \frac{f_y}{(E_s\gamma_{ms})}$ [5.3.2.1(d)] $(x/d)_{max}$	$f_{cu} \le 45$ MPa 0.50 $45 < f_{cu} \le 70$ 0.40 $f_{cu} > 70$ MPa 0.33 [6.1.2.4(b)] $(x/d)_{max}$	$f_{cu} \le 45$ MPa 0.50 $45 < f_{cu} \le 70$ 0.40 $f_{cu} > 70$ MPa 0.33 [6.1.2.4(b)] $(x/d)_{max}$	$\frac{1}{(1+\varepsilon_s/\varepsilon_{cu})}$ where $\varepsilon_s = 0.002 + \frac{f_y}{(E_s\gamma_{ms})}$ [5.3.2.1(d)] $(x/d)_{max}$
Elastic modulus of steel E_s	200 GPa [Figure 2.2]	200 GPa [Figure 2.2]	200 GPa [Figure 2] E_s	200 GPa [Figure 3.9]	200 GPa [Figure 3.9]	200 GPa [Figure 2] E_s
Design strength of reinforcement in tension f_{yd}	f_y/γ_{ms} [Figure 2.2]	f_y/γ_{ms} [Figure 2.2]	f_y/γ_{ms} [Figure 2]	f_y/γ_{ms} [Figure 3.9]	f_y/γ_{ms} [Figure 3.9]	f_y/γ_{ms} [Figure 2]

	BS8110 1997[*] & Concrete Society TR49	**BS8110 1997 (Rev 2005) & Concrete Society TR49**	**BS5400 Part 4 & Concrete Society TR49**	**Hong Kong Buildings 2004[#]**	**Hong Kong Buildings 2004 AMD1 2007**	**Hong Kong Highways 2006**
Design strength of reinforcement in compression f_{ydc}	f_y/γ_{ms} [Figure 2.2]	f_y/γ_{ms} [Figure 2.2]	$(f_y/\gamma_{ms})/[1+(f_y/\gamma_{ms})/2000]$ [Figure 2]	f_y/γ_{ms} [Figure 3.9]	f_y/γ_{ms} [Figure 3.9]	$(f_y/\gamma_{ms})/[1+(f_y/\gamma_{ms})/2000]$ [Figure 2]
Maximum linear steel stress f_{lim}	f_y/γ_{ms} [Figure 2.2]	f_y/γ_{ms} [Figure 2.2]	$0.8f_y/\gamma_{ms}$ [Figure 2]	f_y/γ_{ms} [Figure 3.9]	f_y/γ_{ms} [Figure 3.9]	$0.8f_y/\gamma_{ms}$ [Figure 2]
Yield strain in tension ε_{plas}	$f_y/(\gamma_{ms}E_s)$ [Figure 2.2]	$f_y/(\gamma_{ms}E_s)$ [Figure 2.2]	$f_y/(\gamma_{ms}E_s) + 0.002$ [Figure 2]	$f_y/(\gamma_{ms}E_s)$ [Figure 3.9]	$f_y/(\gamma_{ms}E_s)$ [Figure 3.9]	$f_y/(\gamma_{ms}E_s) + 0.002$ [Figure 2]
Yield strain in compression ε_{plasc}	$f_y/(\gamma_{ms}E_s)$ [Figure 2.2]	$f_y/(\gamma_{ms}E_s)$ [Figure 2.2]	0.002 [Figure 2]	$f_y/(\gamma_{ms}E_s)$ [Figure 3.9]	$f_y/(\gamma_{ms}E_s)$ [Figure 3.9]	0.002 [Figure 2]
Design strain limit ε_{su}	$(10\beta-1)\times\varepsilon_{cu}$ [3.4.4.1(e)]	$(10\beta-1)\times\varepsilon_{cu}$ [3.4.4.1(e)]	([0.01] assumed	$(10\beta-1)\times\varepsilon_{cu}$ [6.1.2.4(a)]	$(10\beta-1)\times\varepsilon_{cu}$ [6.1.2.4(a)]	[0.01] assumed
Maximum concrete strength	$f_{cu} \le 100$ MPa [TR 1]	$f_{cu} \le 100$ MPa [TR 1]	-	$f_{cu} \le 100$ MPa [TR 1]	$f_{ck} \le 80$ MPa [Table 2]	-

	BS8110 1997* & Concrete Society TR49	BS8110 1997 (Rev 2005) & Concrete Society TR49	BS5400 Part 4 & Concrete Society TR49	Hong Kong Buildings 2004#	Hong Kong Buildings 2004 AMD1 2007	Hong Kong Highways 2006
Maximum steel strength	f_y = 460 MPa [Table 3.1]	f_y = 500 MPa [Table 3.1]	-	f_y = 500 MPa [Table 3.1]	f_y = 500 MPa [Table 3.1]	-
Minimum eccentricity	min{h/20, 20 mm} [3.9.3.3]	min{h/20, 20 mm} [3.9.3.3]	0.05h [5.6.2]	min{h/20, 20 mm} [3.9.3.3]	min{h/20, 20 mm} [6.2.1.1(d)]	0.05h [5.6.2]
Minimum area compression reinforcement	-	-	-	-	-	-
maximum permitted angle between applied and resulting principal stress φ_Δ	-	-	-	-	-	-

* BS8110: 1985 is similar to BS8110: 1997 but with a value of 1.15 for γ_{ms}.

Hong Kong 1987 code is similar to BS8110: 1985.

Current tabular codes

	PR China GB 50010 2002
Characteristic concrete cube strength	$f_{cu,k}$ (value after ‘C’ in grade description)
Characteristic steel strength	f_{yk} – related to bar type in Table 4.2.2-1
Design concrete strength	f_c - related to $f_{cu,k}$ in Table 4.1.4
Uncracked concrete design strength for rectangular stress block f_{cdu}	$f_{cu,k} \le 50$ MPa f_c $f_{cu,k} > 50$ MPa $[1 - 0.002(f_{cu,k}\text{-}50)]\times f_c$ [7.1.3] $\alpha_1 f_c$
Cracked concrete design strength (equal to twice the upper limit on shear strength) f_{cdc}	$f_{cu,k} \le 50$ MPa $0.4f_c$ $f_{ck} > 50$ MPa $0.4\times[1 - 0.00667(f_{cu,k}\text{-}50)]\times f_c$ [7.5.1] $0.4\beta_c f_c$
Concrete tensile design strength (used only to determine whether section cracked) f_{cdt}	f_t - related to $f_{cu,k}$ in Table 4.1.4
Compressive plateau concrete strain ε_{ctrans}	$f_{cu,k} \le 50$ MPa 0.002 $f_{cu,k} > 50$ MPa $0.02 + 0.5(f_{cu,k}\text{-}50)\times10^{-5}$ [7.1.2] ε_0
Maximum axial compressive concrete strain ε_{cax}	$f_{cu,k} \le 50$ MPa 0.002 $f_{cu,k} > 50$ MPa $0.02 + 0.5(f_{cu,k}\text{-}50)\times10^{-5}$ [7.1.2] ε_0
Maximum flexural compressive concrete strain ε_{cu}	$f_{cu,k} \le 50$ MPa 0.0033 $f_{cu,k} > 50$ MPa $0.0033 - (f_{cu,k}\text{-}50)\times10^{-5}$ [7.1.2] ε_{cu}

	PR China GB 50010 2002
Proportion of depth to neutral axis over which constant stress acts β	$f_{cu,k} \leq 50$ MPa 0.8 $f_{cu,k} > 50$ MPa 0.8-0.002($f_{cu,k}$-50) β_1
Maximum value of ratio of depth to neutral axis to effective depth in flexural situations $(x/d)_{max}$	$\beta_1/[1+f_y/(E_s\varepsilon_{cu})]$ [7.1.4 & 7.2.1] ξ_b
Elastic modulus of steel E_s	$f_y < 300$ MPa 210 GPa $f_y \geq 300$ MPa 200 GPa [4.2.4] E_s
Design strength of reinforcement in tension f_{yd}	f_y – related to f_{yk} in Table 4.2.3
Design strength of reinforcement in compression f_{ydc}	f'_y – related to f_{yk} in Table 4.2.3
Maximum linear steel stress f_{lim}	f_y – related to f_{yk} in Table 4.2.3
Yield strain in tension ε_{plas}	f_y/E_s
Yield strain in compression ε_{plasc}	f'_y/E_s
Design strain limit ε_{su}	0.01 [7.1.2(4)]
Maximum concrete strength	$f_{cu,k} \leq 80$ MPa [Table 4.1.3]
Maximum steel strength	$f_{yk} \leq 400$ MPa [Table 4.2.2-1

	PR China GB 50010 2002
Minimum eccentricity	max{h/30, 20 mm} [7.3.3]
Minimum area compression reinforcement	0.2% each face [Table 9.5.1]
maximum permitted angle between applied and resulting principal stress φ_{Δ}	-

Codes with resistance factors on materials

	CSA A23.3-04	CSA A23.3-14	CSA S6-14
	Compulsory input parameters	**Compulsory input parameters**	**Compulsory input parameters**
Concrete strength	f_c'	f_c'	f_c'
Steel strength	f_y	f_y	f_y
	Code parameters that can be overwritten	**Code parameters that can be overwritten**	**Code parameters that can be overwritten**
Resistance factor on concrete	$\phi_c = 0.65$ [8.4.2]	$\phi_c = 0.65$ [8.4.2]	$\phi_c = 0.75$ [8.4.6]
Resistance factor on steel	$\phi_s = 0.85$ [8.4.3(a)]	$\phi_s = 0.85$ [8.4.3(a)]	$\phi_s = 0.9$ [8.4.6]
	Derived parameters that can be overwritten	**Derived parameters that can be overwritten**	**Derived parameters that can be overwritten**
Uncracked concrete design strength for rectangular stress block f_{cdu}	Max{0.67, 0.85-0.0015×f_c'}×$\phi_c f_c'$ [10.1.7]	Max{0.67, 0.85-0.0015×f_c'}×$\phi_c f_c'$ [10.1.7]	Max{0.67, 0.85-0.0015×f_c'}×$\phi_c f_c'$ [8.8.3(f)]
Cracked concrete design strength (equal to twice the upper limit on shear strength) f_{cdc}	$0.5\phi_c f_c'$ [11.3.3]	$0.4\phi_c f_c'$ [21.6.3.5]	$0.5\phi_c f_c'$ [8.9.3.3]
Concrete tensile design strength (used only to determine whether section cracked) f_{cdt}	$0.37\phi_c\sqrt{[f_c']}$ [22.4.1.2]	$0.37\phi_c\sqrt{[f_c']}$ [22.4.1.2]	$0.4\phi_c\sqrt{[f_c']}$ [8.4.1.8.1]

	CSA A23.3-04	**CSA A23.3-14**	**CSA S6-14**
Compressive plateau concrete strain ε_{ctrans}	0.002 [assumed]	0.002 [assumed]	0.002 [assumed]
Maximum axial compressive concrete strain ε_{cax}	0.0035 [10.1.3]	0.0035 [10.1.3]	0.0035 [8.8.3(c)]
Maximum flexural compressive concrete strain ε_{cu}	0.0035 [10.1.3]	0.0035 [10.1.3]	0.0035 [8.8.3(c)]
Proportion of depth to neutral axis over which constant stress acts β	Max{0.67, 0.97-0.0025×f_c'} [10.1.7(c)] β_1	Max{0.67, 0.97-0.0025×f_c'} [10.1.7(c)] β_1	Max{0.67, 0.97-0.0025×f_c'} [8.8.3(f)] β_1
Maximum value of ratio of depth to neutral axis to effective depth in flexural situations $(x/d)_{max}$	[upper limit] $(c/d)_{max}$	[upper limit] $(c/d)_{max}$	[upper limit] $(c/d)_{max}$
Elastic modulus of steel E_s	ϕ_s × 200 GPa [8.5.3.2 & 8.5.4.1]	ϕ_s × 200 GPa [8.5.3.2 & 8.5.4.1]	ϕ_s × 200 GPa [8.4.2.1.4 & 8.8.3(d)]
Design strength of reinforcement in tension f_{yd}	$\phi_s f_y$ [8.5.3.2]	$\phi_s f_y$ [8.5.3.2]	$\phi_s f_y$ [8.4.2.1.4 & 8.8.3(d)]
Design strength of reinforcement in compression f_{ydc}	$\phi_s f_y$ [8.5.3.2]	$\phi_s f_y$ [8.5.3.2]	$\phi_s f_y$ [8.4.2.1.4 & 8.8.3(d)]
Maximum linear steel stress f_{lim}	$\phi_s f_y$ [8.5.3.2]	$\phi_s f_y$ [8.5.3.2]	$\phi_s f_y$ [8.4.2.1.4 & 8.8.3(d)]

	CSA A23.3-04	CSA A23.3-14	CSA S6-14
Yield strain in tension ε_{plas}	f_y/E_s [8.5.3.2]	f_y/E_s [8.5.3.2]	f_y/E_s [8.4.2.1.4]
Yield strain in compression ε_{plasc}	f_y/E_s [8.5.3.2]	f_y/E_s [8.5.3.2]	f_y/E_s [8.4.2.1.4]
Design strain limit ε_{su}	[0.01] assumed	[0.01] assumed	[0.01] assumed
	Other parameters	**Other parameters**	**Other parameters**
Maximum concrete strength	$f_c' \le 80$ MPa [8.6.1.1]	$f_c' \le 80$ MPa [8.6.1.1]	$f_c' \le 85$ MPa [8.4.12]
Maximum steel strength	$f_y = 500$ MPa [8.5.1]	$f_y = 500$ MPa [8.5.1]	$f_y = 500$ MPa [8.4.2.1.3]
Minimum eccentricity	0.03h + 15 mm [10.15.3.1]	0.03h + 15 mm [10.15.3.1]	0.03h + 15 mm [8.8.5.3(g)]
Minimum area compression reinforcement	-	-	-