HONG KONG CODE IMPLEMENTATION IN[1] ADAPT SOFTWARE

This Technical Note details the implementation of the Hong Kong Code (CoP-HK:2004 and amendment of June 2007) in the Builder Platform programs.

The implementation follows the Hong Kong Code's procedure of calculating a "Demand," referred to as "design value" for each design section, and a "Resistance," for the same section, referred to as "design capacity." "Design value" and "design capacity" are generic terms that apply to displacements as well as actions. For each loading condition, or instance defined in Hong Kong Code, the design is achieved by making the "resistance" exceed the associated demand "Design Value". Where necessary, reinforcement is added to meet this condition.

The implementation is broken down into the following steps:

- Serviceability limit state
- Strength limit state
- Initial condition (transfer of prestressing)
- Reinforcement requirement and detailing

In each instance, the design consists of one or more of the following checks:
- Bending of section
    - o With or without prestressing
- Punching shear (two-way shear)
- Beam shear (one-way shear)
- Minimum reinforcement

In the following, the values in square brackets "[ ]" are defaults of the program. They can be changed by the user.

REFERENCES

1. Hong Kong Code of Practice: 2004, and amendment of June 2007

MATERIAL AND MATERIAL FACTORS

Concrete[2]
- Cube strength at 28 days, as specified by the user
  f_{cu} = characteristic compressive cube strength at 28 days;

- Parabolic stress/strain curve with the horizontal branch at $0.67f_{cu}/\gamma_m$; maximum strain at 0.0035; Strain at limit of proportionality is based on the following relationship.

[2] CoP-HK :2007, Section 3.1.10

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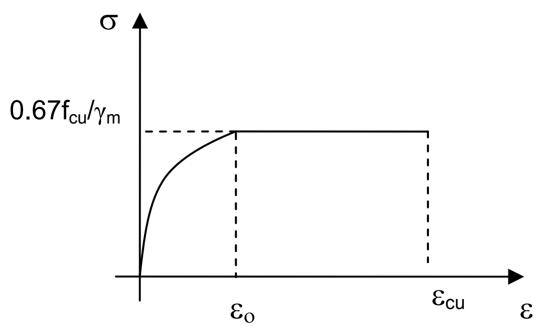


$$\varepsilon_0 = \frac{1.34\, f_{cu}/\gamma_m}{E_c}$$

$\varepsilon_{cu} = 0.0035$ for $f_{cu} \leq 60$ MPa

$\varepsilon_{cu} = 0.0035\text{-}0.00006 \times \sqrt{(f_{cu}\text{-}60)}$ for $f_{cu} > 60$ MPa

- Modulus of elasticity of concrete is automatically calculated and displayed by the program using f_{cu}, γ_m, and the following relationship of the code. User has the option to override the code value and specify a user defined substitute.

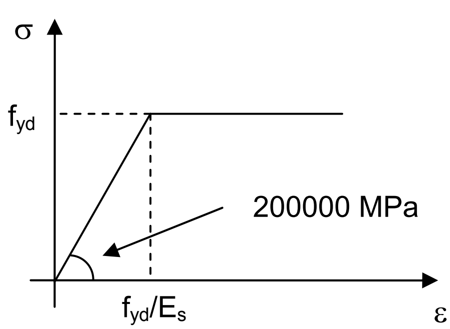
$$E_c = \left(3.46\sqrt{\frac{f_{cu}}{\gamma_m}} + 3.21\right) \times 10^3$$

where,

E_c = modulus of elasticity at 28 days [MPa];
f_{cu} = characteristic cube strength at 28 days; and
γ_m = material factor for concrete.

Nonprestressed Steel[3]

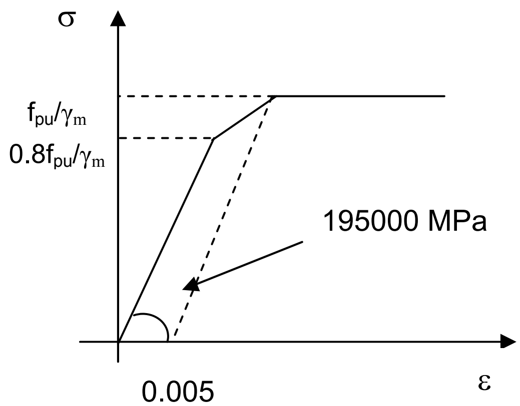
- Bilinear stress/strain diagram with the horizontal branch at $f_{yd} = f_y/\gamma_m$
- Modulus of elasticity(E_s) is user defined [200000 MPa]
- No limit has been set for the ultimate strain of the mild steel in the code.



Prestressing Steel[4]

[3] CoP-HK :2007, Section 3.2.6
[4] CoP-HK :2007, Section 3.3.5

- A trilinear stress-strain curve is assumed.
- Modulus of elasticity is user defined [195000 MPa]
- No limit has been set for the ultimate strain.



Material Factors[5]

- Concrete γ_m = 1.50
- Nonprestressed steel γ_m = 1.15
- Prestressing steel γ_m = 1.15

LOADING

Self-weight determined based on geometry and unit weight of concrete. Other loads are user defined.

SERVICEABILITY

- **Load combinations[6]**

  Permanent:

  Residential and office building

  1.0 DL+1.0 SW+ 0.25 LL+1.0 PT

  Strorage building

  1.0 DL+1.0 SW+ 0.75 LL+1.0 PT

  Transitory:

  1.0 DL+1.0 SW+ 1.0 LL+1.0 PT

- **Stress checks**

  o Concrete

  Stress limitations for compression[7] are as follows:

[5] CoP-HK :2007, Table 2.2
[6] CoP-HK :2007, Section 2.3.3
[7] CoP-HK :2007, Section 12.3.4.2

i. Stress in flexure: $0.33f_{cu}$
ii. Stress in average precompression: $0.25f_{cu}$

If stress at any location exceeds, the program displays that location with a change in color (or broken lines for black and white display), along with a note on program's text report.

Stress limitations for hypothetical tensile stress[8] for the three design options are as follows:

i. Class 1: No tensile stress
ii. Class 2:
Pre-tensioned members: $0.45\sqrt{f_{cu}}$ MPa
Post-tensioned members: $0.36\sqrt{f_{cu}}$ MPa

For temporary loads, the above value may be increased by up to 1.7MPa.

iii. Class 3: Design based on cracked section. The design values are taken from Table 12.2 based on the concrete grade, modified by coefficients given in Table 12.3.

By defining the limits of the tensile stresses, the user specifies the design Class. Should stresses exceed the threshold of the design Class specified by user, the program automatically applies the restrictions applicable to the next design Class. More reinforcement is added, where needed. Computed crack widths are limited to those specified in the code.

- Nonprestressed Reinforcement
  - No stress limits for service condition are specified – no check made
- Prestressing steel
  - No stress limits for service condition are specified - no check made

- **Crack control**
The program calculates the design crack width (w_{cr}) based on the following relationship[9] for non-prestressed members for each design section. In the following relationship, strain in the tension reinforcement is limited to $0.8f_y/E_s$.

$$w_{cr} = \frac{3a_{cr}\varepsilon_m}{1 + 2\left(\frac{a_{cr} - c_{min}}{h - x}\right)}$$

$$\left[\varepsilon_m = \frac{f_s}{E_s}\right]$$

Alternatively ε_m may be calculated as,

$$\varepsilon_m = \varepsilon_1 - \frac{b_t(h - x)(a' - x)}{3E_sA_s(d - x)}$$

where,

a' = distance from the compression face to the point at which the crack width is being calculated;

a_{cr} = distance from the point considered to the surface of the nearest longitudinal bar [user specified cover to rebar];

[8] CoP-HK :2007, Section 12.3.4.3
[9] CoP-HK :2007, Section 7.2.3

b_t = width of the section at the centroid of the tension steel [stem width for beams; tributary width for flange];
c_{min} = minimum cover to the tension steel [a_{cr}];
E_s = modulus of elasticity of the reinforcement;
f_s = tensile stress in the reinforcement;
x = depth of the neutral axis;
ε_1 = strain at the level considered, calculated ignoring the stiffening effect of the concrete in the tension zone; and
ε_m = average strain at the level where the cracking is being considered.
h = depth of the section

A negative value for ε_m indicates that the section is uncracked.

If the calculated value of a section exceeds the allowable, reinforcement is added to that section, in order to reduce the crack width to within the allowable limit. The allowable crack width depends on the exposure condition.

- o Crack width limitation for nonprestressed[10]concrete: 0.3 mm

- o Crack width limitation for prestressed[11]concrete:

Type1 and Type2 members	- no cracking
Type 3 members	
For aggressive environment	- 0.1 mm
For all other	- 0.2 mm

- o For Type 3 members, if the tensile stress exceeds the threshold, program adds rebar to limit the cracking based on the prestressing system as follows[12]:
  - For grouted post-tensioned and pre-tensioned members, $0.0025A_t$ rebar in tension zone is added for every 1MPa of stress above the allowable up to the stress of $0.25f_{cu}$.
  - For other members, $0.0033A_t$ rebar in tension zone is added for every 1MPa of stress above the allowable up to the stress of $0.25f_{cu}$.

The addition of the above rebar for excess stress satisfies the limitations on crack width.

STRENGTH

- **Load combinations[13]**
  - o 1.4 DL+ 1.4 SW+ 1.6 LL+ 1.0 Hyp
  - o 1.4 DL+ 1.4 SW+ 1.4 WL+ 1.0 Hyp
  - o 1.2 DL+ 1.4 SW+ 1.2 LL+1.2 WL+ 1.0 Hyp

- **Check for bending[14]**

  - o Plane sections remain plane. Strain compatibility is used to determine the forces on a section.
  - o Maximum concrete strain in compression is limited to 0.0035.
  - o Tensile capacity of the concrete is neglected.

[10]CoP-HK :2007, Section 7.2.1
[11]CoP-HK :2007, Section 12.1.3
[12] CoP-HK :2007, Section 12.3.4.3
[13]CoP-HK :2007, Section 2.3.2.1
[14]CoP-HK :2007, Section 6.1.2.4 & 12.3.7.1

- o Maximum allowable value for the neutral axis "x" is limited to the following based on the strength of the concrete:
  - $x \le 0.5d$ for $f_{cu} \le 45$ MPa;
  - $x \le 0.4d$ for $45 < f_{cu} \le 70$ MPa;
  - $x \le 0.33d$ for $70 < f_{cu} \le 100$ MPa and no moment redistribution.

  Where necessary, compression reinforcement is added to enforce the above requirement.
- o If a section is made up of more than one concrete material, the entire section is designed using the concrete properties of lowest strength in that section.
- o Stress in nonprestressed steel is derived from representative stress-strain curve for the type of steel used.
- o Stress in prestressing steel is calculated as:
  - For bonded tendons, stress is calculated from stress-strain compatibility of the section.
  - For unbonded tendons[15]:

$$f_{pb} = f_{pe} + \frac{7000\lambda_1}{l/d_p}\left(1 - 0.7\lambda_2 \frac{f_{pu}A_{ps}}{f_{cu}bd_p}\right) \le 0.7f_{pu}$$

where,

λ_1 = 1 for $f_{cu} \le 60$ MPa,
= $1 - 0.017\sqrt{f_{cu} - 60}$ for $f_{cu} > 60$ MPa
λ_2 = 2.58 for $f_{cu} \le 45$ MPa,
= 2.78 for $45 < f_{cu} \le 70$ MPa,
= 3.09 for $70 < f_{cu} \le 100$ MPa

f_{pe} = the effective stress in prestressing (after allowance for all prestress losses);
b = the width or effective width of the section or flange in the compression zone;
d_p = the distance between the compression face of the section to the centroid of the tendons;
A_{ps} = the area of the tendons in tensile area;
f_{pb} = design tensile strength in the tendons.

- o Rectangular concrete block is used with maximum stress equal to $0.45f_{cu}$ and the depth equal to the following:
  - 0.9x for $f_{cu} \le 45$ MPa;
  - 0.8x for $45 < f_{cu} \le 70$ MPa;
  - 0.72x for $70 < f_{cu} \le 100$ MPa. Where x is the depth of the neutral axis.
- o For flanged sections, the following procedure is adopted:
  - If x is within the flange, the section is treated as a rectangle
  - If x exceeds the flange thickness, uniform compression is assumed over the flange. The stem is treated as a rectangular section.

- **One-way shear**

  - o **Non-prestressed members[16]:**

    Nominal shear stress:

[15] CoP-HK :2007, Section 12.3.7.3
[16] CoP-HK :2007, Section 6.1.2.5

$$v = \frac{V_u}{b_v d}$$

where, V_u = shear force due to design loads;
b_v = width of the section. For flanged section, average width of the rib below the flange;
d = effective depth.

Design shear strength of concrete:

- For beams, and slabs supported by beams or walls:

$$v_c = 0.79(100\rho)^{1/3}\left(\frac{400}{d}\right)^{0.25}\frac{1}{\gamma_m}\times\left(\frac{f_{cu}}{25}\right)^{1/3}$$

where,

$$100\rho = \frac{100A_s}{b_v d} < 3 \quad \& > 0.15$$

$\left(\frac{400}{d}\right)^{1/4} > 0.67$ for members without shear reinforcement;

$\left(\frac{400}{d}\right)^{1/4} > 1$ for members with shear reinforcement;

$f_{cu} \leq 80$ MPa.

- For members under axial compression:

$$v_c' = v_c + 0.6\frac{NV_u h}{A_c M_u}$$

where,
v_c' = design concrete shear stress corrected to allow for axial forces;
N = applied axial load;
M_u = moment due to design loads;
A_c = gross area of the concrete section;
$\frac{N}{A_c}$ = average stress in the concrete at the centroid of the section;
$\frac{V_u h}{M_u} \leq 1$

Shear reinforcement:
For beams[17]:

- $v < v_c + v_r$, $\qquad A_{sv} = \frac{v_r b_v s_v}{0.87 f_{yv}}$
- $v_c + v_r < v < 0.8\sqrt{f_{cu}}$ or 7MPa $\qquad A_{sv} = \frac{b_v s_v (v - v_c)}{0.87 f_{yv}}$

For slabs[18]:
- $v < v_c$, No shear reinforcement is required.

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17 CoP-HK :2007, Table 6.2
18 CoP-HK :2007, Table 6.3

- $v_c < v < v_c + v_r,$ $\qquad A_{sv} = \dfrac{v_r b_v s_v}{0.87 f_{yv}}$
- $v_c + v_r < v < 0.8\sqrt{f_{cu}}$ or 7MPa $\qquad A_{sv} = \dfrac{b_v s_v (v - v_c)}{0.87 f_{yv}}$

where,

v_r = 0.4 for $f_{cu} \leq 40$ MPa

$v_r = 0.4\left(f_{cu}/40\right)^{2/3}$ for $40 < f_{cu} \leq 80$ MPa

Maximum spacing of the links, $s_{vmax} = 0.75d$

- **Prestressed members[19]:**
  - $V < 0.5V_c$ No shear reinforcement is required.
  - $0.5V_c < V < V_c + V_r,$ $\qquad A_{sv} = \dfrac{V_r s_v}{0.87 f_{yv} d}$
  - $V_c + V_r < V < 0.8 b_v d\sqrt{f_{cu}}$ or $7b_vd$ $\qquad A_{sv} = \dfrac{s_v (V - V_c)}{0.87 f_{yv} d_t}$

where,

$V_r = 0.4 b_v d$ for $f_{cu} \leq 40$ MPa

$V_r = 0.4\left(f_{cu}/40\right)^{2/3} b_v d$ for $40 < f_{cu} \leq 80$ MPa

Concrete Shear Resistance, V_c:

- For uncracked sections ($M<M_0$) : $V_c=V_{co}$
- For cracked sections ($M \geq M_0$) : V_c= lesser of V_{co} and V_{cr}

$$V_{co} = 0.67 b_v h\sqrt{\left(f_t^2 + 0.8 f_{cp} f_t\right)} \qquad \text{where} \qquad f_t = 0.24\sqrt{f_{cu}}$$

$$V_{cr} = \left(1 - 0.55\frac{f_{pea}}{f_{pu}}\right) v_c b_v d + M_0\frac{V}{M} \geq 0.1 b_v d\sqrt{f_{cu}}$$

where,

$$f_{pea} = \frac{PTforce}{A_{ps} + \left(\dfrac{f_y}{f_{pu}}\right) A_s}$$

$M_0 = 0.8 f_{pbot} \times S_b$ - If applied moment due to DL & LL is positive;

$M_0 = 0.8 f_{ptop} \times S_t$ - If applied moment due to DL & LL is negative;

f_{ptop} and f_{pbot} - Stresses due to prestressing only;

S_t and S_b - Top and bottom section modulii;

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[19] CoP-HK :2007, Section 12.3.8

v_c - design concrete shear stress for non-prestressed beams and slabs obtained by replacing A_s with (A_s+A_{ps}).

Maximum spacing of the links,s_{vmax} :

- For $V < 1.8V_c$, $s_{vmax} = \min\{ 0.75d_t, 4b_w \}$
- For $V > 1.8V_c$, $s_{vmax} = 0.5d_t$

Maximum lateral spacing of individual legs of the stirrups = d_t

- **Two-way shear**

Categorization of columns:

No criterion is mentioned in British code regarding categorizations of columns for punching shear check. The program uses ACI-318 criteria as detailed below.

Based on the geometry of the floor slab at the vicinity of a column, each column is categorized into to one of the following options:

1. Interior column
   Each face of the column is at least four times the slab thickness away from a slab edge

2. Edge column
   One side of the column normal to the axis of the moment is less than four times the slab thickness away from the slab edge

3. Corner column
   Two adjacent sides of the column are less than four times the slab thickness from slab edges parallel to each

4. End column
   One side of the column parallel to the axis of the moment is less than four times the slab thickness from a slab edge

In cases 2, 3 and 4, column is assumed to be at the edge of the slab. The overhang of the slab beyond the face of the column is not included in the calculations. Hence, the analysis performed is somewhat conservative.

Stress calculation[20]:

Stress is calculated for several critical perimeters around the columns based on the combination of the direct shear and moment.

For interior column and, edge column where bending about an axis perpendicular to the free edge,

$$v_u = \frac{1}{A} V_u \left(\beta + \frac{1.5M_u}{V_u x_{sp}}\right)$$

[20] CoP-HK :2007, Section 6.1.5.6 & 6.1.5.7

where V_u is the absolute value of direct shear, M_u is the absolute value of direct moment, x_{sp} is the length of the critical section parallel to the axis of bending, A is the area of the critical section, and β is 1.0 for interior column and 1.25 for edge column.

For corner column and, edge column where bending about an axis parallel to the free edge,

$$v_u = \frac{1.25V_u}{A}$$

For a critical section with dimension of b_1 and b_2 and average depth of d, A is:

1. Interior column:
$$A = 2(b_1 + b_2)d$$
2. Edge column: (b_1 is parallel to the axis of moment)
$$A = (2b_1 + b_2)d$$
3. Corner Column:
$$A = (b_1 + b_2)d$$
4. End column: (b_1 is parallel to the axis of moment)
$$A = (b_1 + 2b_2)d$$

Allowable stress[21]:

For non-prestressed and prestressed members:

$$v_c = 0.79(100\rho)^{1/3}\left(\frac{400}{d}\right)^{0.25}\frac{1}{\gamma_m}\times\left(\frac{f_{cu}}{25}\right)^{1/3}$$

where,

$$\rho = \frac{100A_s}{b_v d} < 3 \quad \& > 0.15$$

$\left(\frac{400}{d}\right)^{1/4} > 0.67$ for members without shear reinforcement;

$\left(\frac{400}{d}\right)^{1/4} > 1$ for members with shear reinforcement;

$f_{cu} \leq 80$ MPa;

γ_m= design strength factor =1.25

For prestressed members, ρ will be calculated based on sum of A_{ps} and A_s. Currently program calculates the allowable stress using minimum reinforcement ratio ρ [0.15].

Critical sections[22]:

The critical sections for stress check are:

(1) at the face of column;
(2) at 1.5d from the face of the column, where d is the effective depth of the slab/drop cap; and

[21] CoP-HK :2007, Table 6.3
[22] CoP-HK :2007, Section 6.1.5.7

(3) additional sections at 0.75d intervals, where required.

If drop cap exists, stresses are also checked at 0.75d from the face of the drop cap in which d is the effective depth of the slab. Subsequent sections are 0.75d away from the previous critical section.

Stress check[23]:

Stresses are calculated at the critical sections in two directions separately and compared against the allowable values:

If	$v_u < v_c$	no punching shear reinforcement is required
if	$v_u > v_{max}$	punching stress is excessive; revise the section
If	$2v_c > v_u > v_c$	provide punching shear reinforcement

where,

$$v_{max} = \min\begin{cases} 7 \\ 0.8\sqrt{f_{cu}} \end{cases}$$ at the face of the support

$v_{max} = 2v_c$ at critical sections other than face of support

If stress is below the permissible value in both directions, then no shear reinforcement is needed otherwise if at least in one direction, stress exceeds the permissible value, shear reinforcement should be provided.

Stress check is performed until no shear reinforcement is needed. Where drop caps exist, stresses are checked within the drop cap until the design stress is less than the permissible, then in a similar manner the stresses are checked outside the drop cap.

Shear reinforcement[24]:

Where needed, shear reinforcement is provided according to the following:

If $v_u \leq 1.6v_c$ $$A_{sv} = \frac{(v_u - v_c)ud}{0.87f_{yv}\sin\alpha} > A_{smin}$$

If $1.6v_c < v_u \leq 2.0v_c$ $$A_s = \frac{5(0.7v_u - v_c)ud}{0.87f_{yv}\sin(\alpha)} > A_{smin}$$

$$A_{smin} = \frac{v_r ud}{0.87f_{yv}\sin(\alpha)}$$

where,

$v_r = 0.4$ for $f_{cu} \leq 40$ MPa

$v_r = 0.4\left(f_{cu}/40\right)^{2/3}$ for $40 < f_{cu} \leq 80$ MPa

[23] CoP-HK :2007, Section 6.1.5.7(d & e)
[24] CoP-HK :2007, Section 6.1.5.7 (e)

Where v_u is the maximum shear stress calculated as the maximum of shear stresses of the two directions calculated in previous sections. α is the angle of shear reinforcement with the plane of slab and u is the periphery of the critical sections.

Arrangement of shear reinforcements:

Shear reinforcement can be in the form of shear studs or shear stirrups (links). In case of shear links, the number of shear links (N_{shear_links}) in a critical section and distance between the links ($Dist_{shear\_links}$) are given by:

$$N_{shear\_links} = \frac{A_s}{A_{shear\_link}}$$

$$Dist_{shear\_links} = \frac{u}{N_{shear\_links}}$$

Where, $A_{shear\text{-}link}$ is the area of the single shear link.

The calculated distance will be compared with the maximum allowable by the code and will be adjusted accordingly.

If shear studs are used, the number of studs per rail (N_{shear_studs}) and the distance between the studs ($Dist_{shear\_studs}$) are given by:

$$N_{shear\_studs} = \frac{A_s}{A_{shear\_stud} \times N_{rails}}$$

$$Dist_{shear\_studs} = \frac{s}{N_{shear\_studs}}$$

Where, s is the distance between the critical sections.

Shear reinforcement is provided in three layers (perimeters) from the face of support to the first critical section, i.e., within the distance 1.5d from the face of support.

INITIAL CONDITION

- **Load combinations**

Hong Kong code does not specify a load combination for the initial condition. ADAPT uses the following default values. User can modify these values.

1.0 SW +1.15 PT

- **Allowable stresses**[25]

Limitations for hypothetical tensile stress for the three design options are as follows:

i. Class 1: 1.0 MPa

ii. Class 2 and Class 3:

Pre-tensioned members: $0.45\sqrt{f_{ci}}$ MPa

[25] CoP-HK :2007, Section 12.3.5

Post-tensioned members: $0.36\sqrt{f_{ci}}$ MPa

Where this stress exceeded for class 3 members, the section should be considered as cracked.

Limitations for compressive stress are as follows:

i. Stress in flexure: $0.50f_{ci}$

ii. Stress in average precompression: $0.40f_{ci}$

where,

f_{ci} = Cube strength of concrete at transfer

DETAILING

- **Reinforcement requirement and placing**

Non-prestressed member[26]

- o Minimum rebar

Beams:

Minimum rebar for tension and compression is provided based on Table 9.1.

Slabs:

As $= 0.0024A_c$ -For fy = 250 MPa

$= 0.0013A_c$ -For fy = 460 MPa

Where A_c is the concrete cross-sectional area.

- o Maximum rebar

Beam - $A_{smax} = 0.04bh$

For prestressed structure, minimum rebar for crack control (serviceability check) is provided for all the systems.

APPENDIX

This appendix includes additional information directly relevant to the design of concrete structures, but not of a type to be included in the program.

- **Effective width of the flange[27]**

Effective flange width is not included in ADAPT_Floor Pro, because it is implicit in the finite element analysis of Floor Pro. But this is included in ADAPT_PT and will be calculated as follows:

- o For T-Beams

$$b_{eff} = \Sigma b_{eff,i} + b_w$$

with i = 1 or 2, and

$$b_{eff,i} = 0.2b_i + 0.1\, l_{pi} \leq 0.2l_{pi} \text{ and } \leq b_i$$

where,

b_{eff} = effective width of flange;

$b_{eff,i}$ = effective width of flange on one side of the web;

l_{pi} = distance between points of zero moments in the beam;

[26] CoP-HK :2007, Section 9.2.1 and 9.3.1

[27] CoP-HK :2007, Section 5.2.1.2

For continuous beams, l_2 = 0.7*effective span;

b_w = width of the web;

b_i = actual width of the flange on one side of the web.

- **ANALYSIS**
  - o Loading arrangement[28]:

    Vertical loads are arranged as in the following combination:
    - all spans loaded with the maximum design load (γ_f DL+ γ_f LL) ;
    - alternate spans loaded with the maximum design load (γ_f DL+ γ_f LL) and all other spans loaded with the minimum design load (γ_f DL).
    - any two adjacent spans loaded with the maximum design load (γ_f DL+ γ_f LL), and all other spans loaded with the minimum design load, (γ_f DL).

    Where, γ_f is the load factor for strength and serviceability conditions.

  - o Redistribution of moment[29]
    - No redistribution is allowed if the ultimate moment of resistance at any section of a member is less than 70% of the factored moment at that section for non-prestressed and 80% of the factored moment at that section for prestressed structure;
    - The percentage of moment redistribution should not be more than 30 for non-prestressed and 20 for prestressed structure;
    - At sections where the design moment is reduced, the following relationship will be satisfied:

$$x \le (\beta_b - 0.4)d, \qquad \text{for } f_{cu} \le 45 \text{ MPa;}$$

$$x \le (\beta_b - 0.5)d, \qquad \text{for } 45 < f_{cu} \le 70 \text{ MPa.}$$

where,

x = depth of neutral axis,

d = effective depth, and

β_b = the ratio: $\dfrac{\text{moment at the section after redistribution}}{\text{moment at the section before redistribution}} \le 1$

- **Deflection[30]**

  Total deflection - L/250

  L- span length of the member

NOTATION

A_s = area of tension reinforcement;

A_t = area of concrete in tension zone;

d = effective depth;

[28] CoP-HK :2007, Section 5.1.3. 2

[29] CoP-HK :2007, Section 5.2.9 & 12.2.3

[30] CoP-HK :2007, Section 7.3.1

d_t = depth from the extreme compression fiber either to the longitudinal bars or to the centroid of the tendons, whichever is greater;

DL = dead Load;

f_{cu} = characteristic compressive cube strength at 28 days;

f_{pu} = characteristic strength of the prestressing steel [1860 MPa];

f_y = characteristic yield strength of steel, [460 MPa];

f_{yv} = characteristic yield strength of shear reinforcement;

Hyp = hyperstatic (secondary);

h = overall depth of the beam/ slab;

LL = live load;

s_v = spacing of the stirrups;

SW = self weight of the structure;

v = design shear stress;

v_c = concrete shear strength;

x = depth of neutral axis;

w_{cr} = design (computed) crack width; and

WL = wind load.