TS 500-2000 Approximate Method

The Moment Magnifier Method

- In this method, the sway and non-sway frames are treated separately.
- According to TS500-2000, a story in a frame is said to be non-sway, if the increase in end moments resulting from slenderness effect does not exceed 5 percent of the first-order moment.
- The method for determining this increase can be based on the stability index, ϕ

$$\phi = 1.5\Delta_i \frac{\Sigma\left(N_{di} / \ell_i\right)}{V_{fi}}$$

Where:

- V_{fi} = factored horizontal shear in i^{th} story,
- Δ_i = relative displacement between the top and bottom of the i^{th} story due to V_{fi}, (first order analysis)
- N_{di} = factored axial load at a given eccentricity, for each column in the ith story,
- ℓ_i = length of an individual column in ith story, center-to-center distance between top and bottom joints.

If the ϕ value, calculated by using Eq. (1), comes out to be less than 0.05, the frame can be identified as a non-sway frame.

If the length of all columns in the ith story is the same, the previous can be rewritten as;

$$\phi = 1.5\frac{(\Sigma N_{di})\Delta_i}{V_{fi}\ell_i}$$

To simplify the design, codes permit the designer to neglect the second order moments when the slenderness ratio of the compression member, l_k / i, is smaller than a certain value.

- *For columns of non-sway frames*

$$\left(\ell_k / \mathrm{i}\right) \leq 34 - 12 \frac{M_{d1}}{M_{d2}}$$

- *For columns of sway frames*

$$\left(\ell_k / \mathrm{i}\right) \leq 22$$

The radius of gyration of the cross-section may approximately be taken as 0.3h, where “h” is the dimension of the column in the direction of bending for rectangular sections and 0.25h for circular sections.

The compression members, with slenderness ratios (l_k/i) values grater than the limits defined by Eq.(3) and smaller than 100, shall be designed for factored axial load N_d and moment amplified for the second order effects.

The terms M_{dl} and M_{d2} are the column end moments obtained from first order elastic analysis. Here M_{d2} is the larger end moment ($M_{d2}>M_{d1}$) and M_{d2}/M_{d1} ratio should be taken as positive for columns in single curvature. When the column is in double curvature, the M_{d2}/M_{dl} ratio should be taken as negative, indicating a less critical situation when compared with the former case.

The effective length of a compression member can be found by multiplying the unsupported length of the column by the effective length factor.

The unsupported length of the column is the clear distance between the top of the floor and the bottom of the shallowest beam or other members capable of providing lateral support in the direction being considered.

In TS-500-2000, the effective length factors, k, for *braced* and *unbraced* compression members are defined differently. The effective length factor of braced columns, which are restrained at both ends, is given as:

$$
\begin{aligned}
k &= 0.7+0.05(\alpha_1 + \alpha_2) \\
&\leq 0.85+0.05\alpha_1 \\
&\leq 1.0
\end{aligned}
$$

The effective length factor given above is a function of the relative stiffness at each end of the column, i.e. α_1 and α_2.

- The α values can be expressed as:

$$\alpha_1 = \frac{\Sigma\left((I/\ell)\right)_{columns}}{\Sigma(I/\ell)_{beams}} \quad , \qquad \alpha_2 = \frac{\Sigma\left((I/\ell)\right)_{columns}}{\Sigma(I/\ell)_{beams}}$$

- The effective length factor, k, for columns of unbraced frames can be taken as:
- $\alpha_m = 0.5(\alpha_1 + \alpha_2)$

If α_m<2.0, $k = \dfrac{20-\alpha_m}{20}\sqrt{1+\alpha_m}$

If α≥2.0, $k = 0.9\sqrt{1+\alpha_m}$

In ACI318-95 and TS500-2000 codes, the slenderness is accounted for by magnifying the column end moments.

In the moment magnifier method, the design moment, is obtained from:

$$M'_d = \beta M_{d2} \quad \text{or} \quad M'_d = \beta_s M_{d2}$$

In this equation, $M_{d2} \geq N_d \times e_{min}$ is the factored larger first order column end moment and, β and β_s are moment magnifiers defined separately for non-sway and sway columns.

Moment magnifier for columns in non-sway frames:

$$\beta = \frac{C_m}{1 - \frac{N_d}{N_{cr}}}$$

$$C_m = (0.6 + 0.4\frac{M_{d1}}{M_{d2}}) \geq 0.4$$

Here, M_{d2} > M_{d1} and N_d and N_{cr} are the design axial load and the buckling load of the column, respectively.

- The coefficient C_m is known as the 'equivalent moment correction factor' and it may take values as low as 0.4 especially when the column is in double curvature and the maximum moment occurs at one end of the column.
- For members with transverse loads between the supports, it is possible that the maximum first order moment will develop at a section away from the end of the member. If this occurs, the value of the largest calculated moment occurring anywhere along the member should be used instead of M_{d2} and C_m must be taken as 1.0 for this case.

Moment magnifier for columns in sway frames:

β_s should be calculated as an average value for the entire story by considering all columns in that story

$$\beta_s = \frac{1.0}{1 - \dfrac{\Sigma N_d}{\Sigma N_{cr}}}$$

β_s: Moment magnification factor for the unbraced frame, to reflect lateral drift resulting from lateral and gravity loads.

ΣN_d: Summation of the design axial loads of all columns in the story.

ΣN_{cr}: Summation of the critical loads of all columns in the story.

Individual β value for each column must also be computed by:

$$\beta = \frac{1.0}{1 - \frac{\Sigma N_d}{\Sigma N_{cr}}}$$

The larger of the two moment magnifiers β_s and β will be used in the design of the individual column in consideration.

The critical load to be used in β or β_s calculations, i.e. N_{cr}, can be calculated by using Euler's buckling formula. This equation involves both flexural rigidity EI and effective length l_k.

$$N_{cr} = \frac{\pi^2 EI}{\ell_k^2}$$

The creep of concrete, the cracking of the member and the non-linearity of concrete stress-strain curve significantly influence the flexural rigidity, EI, of the column.

This has to be taken into account

If $0.01 < \rho_t < 0.02$ then $$EI = \frac{E_c I_c}{2.5} \times \frac{1}{1 + R_m}$$

If ρ_t is high then $$EI = \frac{0.2 E_c I_c + E_s I_s}{1 + R_m}$$

where;

EI effective flexural rigidity

E_c modulus of elasticity of concrete

I_c moment of inertia of gross concrete area of column

E_s modulus of elasticity of steel

Is moment of inertia of the column longitudinal reinforcement about the centroid of gross concrete area

R_m creep factor

- For non-sway frames, R_m is the ratio of the maximum factored axial dead load to the total factored axial load, $R_m = N_{gd}/N_d$
- For sway frames, R_m is the maximum factored sustained shear within a story to the total factored shear in that story.

$$R_m = \frac{\Sigma V_{gd}}{\Sigma V_d}$$

V_{gd} factored design shear due to gravity loads, $V_{gd}=1.4V_g$

V_d total design shear force in the story

MINIMUM REQUIREMENTS FOR COLUMNS

Definition	TS500-2000 + TEC-1998	Remarks
Minimum cross-sectional dimension	250 mm for rectangular sections 300 mm for spiral columns	The minimum diameter for spiral columns refers to diameter of the gross section.
Minimum cross-sectional area	75,000 mm^2	
Min ℓ_c	h, ℓ_n /6 , 500 mm	
	min 0.01, max 0.04	In regions of lapped splices, max r_t=0.06. TS-500-2000 minimum r_t may be taken as low as 0.005.
$\rho_t = A_{st} / A_c$	min 4-ϕ16 or 6-ϕ14	
	min 6-ϕ14 (spiral columns)	
Minimum diameter of the tie or spiral steel	8 mm	
Maximum tie or spiral spacing along the column height	b/2 , 200 mm , 12ϕ_ℓ (TS500-2000)	Here ϕ_ℓ is the diameter of the longitudinal steel in mm.
Length of confined regions at column ends	≥ d, ≥ l_n/6 ≥ 500 mm	
Maximum tie or spiral spacing in the confined rgions, s_c	Tied columns: b/3, 100 mm. Spiral columns: D/5, 80 mm where 'D' is the core diameter.	
Min s_c	50 mm	
Max N_d	$0.6 f_{ck} A_c$ (TS500-2000) $0.5 f_{ck} A_c$ (TEC-1998)	
Minimum diameter for the longitudinal bars, ϕ_ℓ	14 mm	
e_{min}	15 mm + 0.03h	