The calculation of the cross section bearing capacity of the concrete bending member under the bending bearing capacity and the compression member

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Abstract

Flexural member and compression member is the important content of concrete structure course, and through the concrete flexural members compression member of learning, mainly for learners to deepen to the principle of steel and concrete can work together and common force when the mechanical behavior.In this paper, the reinforcement calculation of bending member and the compression member are discussed in the form of two examples, and the corresponding deflection and fracture checking are discussed.

Keywords

Bending member; compression member; reinforcement; immunity; fissure.

1. Introduction

Flexural members mainly refer to various types of beams and slabs, which are the most commonly used components in civil engineering. The section perpendicular to the calculated axis of the member is called the normal section. Structures and components should meet the requirements of the ultimate state of bearing capacity and normal service limit state. The calculation of bending capacity of normal section of beams and slabs is based on the ultimate state of bearing capacity. While guaranteeing the normal section bearing capacity of flexural members, the inclined section bearing capacity should also be guaranteed, which includes the shear capacity of the inclined section and the flexural capacity of the inclined section. In engineering design, the shear capacity of oblique section is satisfied by calculation and construction, while the flexural capacity of oblique section is guaranteed by the structural requirements of longitudinal reinforcement and stirrups. Reinforced concrete compression members include axial compression members and eccentric compression members (also known as compression-bending members). Structures and components should meet the requirements of the ultimate state of bearing capacity and normal service limit state. In practical engineering, the walls and columns are all such components, so they are widely used.

2. Calculation of Section Bearing Capacity of 1Flexural Members

2.1 Section Size and Material Selection

The section sizes of cast-in-place beams and slabs should be adopted as follows:

(1) The ratio of height to width of rectangular cross-section beams is generally 2.0-3.5, and that of T-section beams is 2.5-4.0 (where B is the rib width). The width of the rectangular section or the rib width of the T-shaped section is generally taken as 100 mm, 120 mm, 150 mm, (180 mm), 200 mm, (220 mm), 250 mm and 300 mm, and the gradient over 300 mm is 50 mm; the values in brackets are only used for wooden models.

(2) Beam height h = 250 mm, 300 mm, 350 mm, 750 mm, 800 mm, 900 mm, 1000 mm, etc. The gradient below 800 mm is 50 mm and above 100 mm.

2.2 The material selection is as follows:

2.2.1 Strength grade of concrete

The common concrete grades of cast-in-situ concrete beams and slabs are C25 and C30, which generally do not exceed C40. This is to prevent excessive shrinkage of concrete, and the effect of increasing concrete strength grade on increasing flexural capacity of normal section of flexural members is not obvious.

2.2.2 Strength Grade and Common Diameter of Reinforcement Bars

(1) HRB400 and HRB500 grades are suitable for longitudinal reinforcement in beams. The commonly used diameters are 12 mm, 14 mm, 16 mm, 18 mm, 20 mm, 22 mm and 25 mm. The diameter of steel bar under longitudinal force should not be less than 10 mm when the height of beam is greater than or equal to 300 mm, and not less than 8 mm when the height of beam is not less than 300 mm.

(2) The stirrups of beams should be HPB400, HRB335 and a small amount of HPB300 steel bars. The commonly used diameters are 6mm, 8mm and 10mm.

In this paper, the longitudinal reinforcement in the beam is HRB400 grade steel bar with a diameter of 20 mm, the stirrup is HPB300 grade steel bar with a diameter of 6 m, and the strength grade of concrete is C30. The span of the beam is 3 m and the section size is B *h=250 mm *500 mm (as shown in Figure 1). Internal force diagram of beam (as shown in Fig. 2)

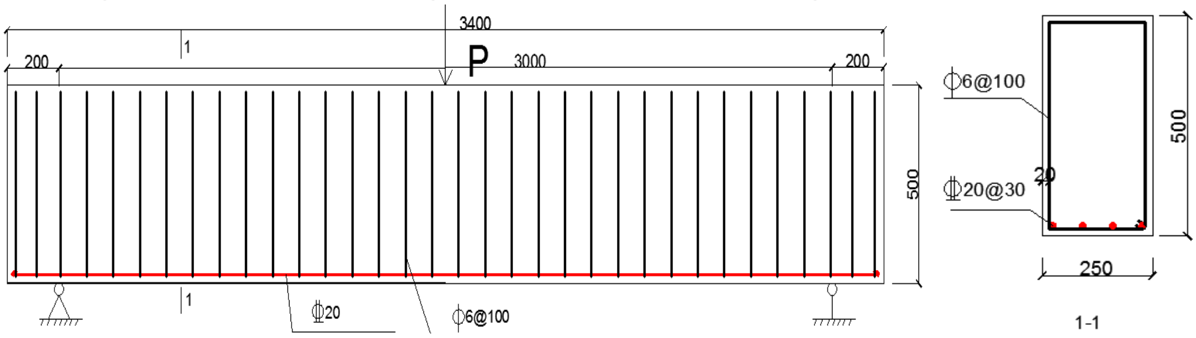
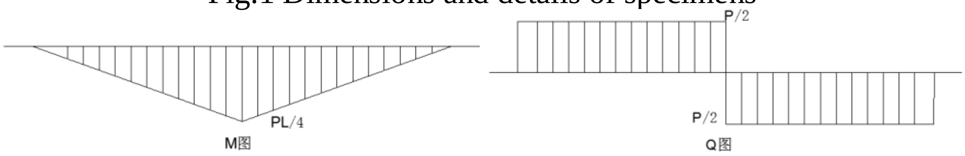


Fig.1 Dimensions and details of specimens



(a) Bending moment diagram (b) Shearing force diagram

Fig.2 Internal force of beam

2.3 load of bending members

$$\alpha f_c bx = f_y A_s \qquad x = \frac{f_y A_s}{\alpha_1 f_c b} = \frac{360\times1256}{1\times14.3\times250} = 126mm$$

$$M_{cr} = f_t W + 0.87\sigma_s h_0 = f_t W + 0.87\frac{f_t E_s}{E_c} A_s h_0 =$$

$$1.43\times\frac{250\times500^2}{6} + 0.87\times\frac{1.43\times2\times10^5}{3\times10^4}\times1256\times460 = 19.71kN$$

$$P = 26.28kN$$

$$M_u = a_1 f_y A_s (h_0 - \frac{x}{2}) = 1\times360\times1256\times(460 - \frac{126}{2}) = 178.6kN\cdot m$$

$$P_u = 238.1kN$$

2.4 Reinforcement

Checking section condition

$$h_w = h_0 = 460\,mm,\ \frac{h_w}{b} = \frac{460}{250} = 1.84 < 4$$

$$0.25\beta_c f_c bh_0 = 0.25 \times 1 \times 1.43 \times 250 \times 460 > V_A = V_B = 119.05kN$$

The section meets the requirements

Determine the area of hoop

$$V_{cs} = \alpha_{cv} f_t bh_0 + f_{yv}\frac{A_{sv}}{s}h_0,\ V_c = \frac{1.75}{\lambda+1} f_t bh_0,\ \lambda = \frac{a}{h_0} = \frac{150}{460} = 3.26 > 3, \lambda = 3$$

$$V_c = 1.75/(1+3) \times 1.43 \times 250 \times 460 = 71.95kN < V_{cs} = 119.05kN$$

Therefore, it is necessary to be equipped with hoop tendons.

$$\frac{nA_{sv}}{s} = \frac{V_{cs} - \alpha_{cv} f_t bh_0}{f_{yv}h_0} = \frac{(119.05 - 71.95)\times 10\text{^}3}{360 \times 460} = 0.3mm\text{^}2/mm$$

Select of double limb hoops

$$\frac{nA_{sv1}}{s} = \frac{2 \times 28.3}{100} = 0.57mm\text{^}2/mm > 0.3mm\text{^}2/mm$$

$$\rho_{sv} = \frac{nA_{sv1}}{bs} = \frac{2 \times 28.3}{250 \times 100} = 0.228\% > \rho_{sv,\min} = 0.113\%$$

2.5 Deflection calculation.

(1) Calculation of related parameters

$$\alpha_E \rho = \frac{E_s}{E_c} \times \frac{A_s}{bh_0} = \frac{2\times 10\text{^}5}{3 \times 10\text{^}4} \times \frac{1256}{250 \times 460} = 0.073\quad \rho_{te} = \frac{A_s}{A_{te}} = \frac{1256}{0.5 \times 250 \times 500} = 0.020$$

$$\sigma_{sq} = \frac{M_q}{\eta h_0 A_s} = \frac{178.60 \times 10\text{^}6}{0.87 \times 460 \times 1256} = 355.32N/mm\text{^}2$$

$$\psi = 1.1 - 0.65\frac{f_{tk}}{\rho_{te}\sigma_{sq}} = 1.1 - 0.65 \times \frac{2.01}{0.020 \times 355.32} = 0.916$$

$$B_s = \frac{E_s A_s h_0\text{^}2}{1.15\psi + 0.2 + \dfrac{6\alpha_E\rho}{1+3.5\gamma'_f}} = \frac{2\times 10\text{^}5 \times 1256 \times 460\text{^}2}{1.15 \times 0.916 + 0.2 + 6 \times 0.916} = 7.88 \times 10\text{^}12\ N\cdot mm\text{^}2$$

(2) Calculation deflection

$$f = \frac{1}{12} \times \frac{M_q l_0^2}{B_s} = \frac{1}{12} \times \frac{178.60 \times 10^6 \times 3000^2}{7.88 \times 10^{12}} = 17mm$$

2.6 Calculation of maximum crack

$$w_{\max} = \alpha_{cr}\psi\frac{\sigma_{sq}}{E_s}(1.9c_s + 0.08\frac{d_{eq}}{\rho_{te}}) = 1.9 \times 0.916 \times \frac{355.32}{2 \times 10\text{^}5}(1.9 \times 26 + 0.08\frac{20}{0.020}) = 0.4mm$$

3. Calculation of Bearing Capacity of Compressed Members

3.1 Section dimensions and material properties

In order to make the template, the section of the axial compression component is usually square or rectangular, sometimes circular or polygons. Eccentrically compressed members are generally rectangular.

The section size of square column should not be less than 250mm × 250mm. In order to construct the supporting mould conveniently, the section size of the column should be integer, 800mm and below, the multiple of modulus 50mm, the multiple of 800mm or above, and the multiple of password.

Material strength requirements

The strength grade of concrete has a great influence on the bearing capacity of compressed members. In order to reduce the section size and save steel, high strength concrete should be used. C30, C35 and C40 are generally used. For the bottom columns of high-rise buildings, high strength grade concrete can be used if necessary.

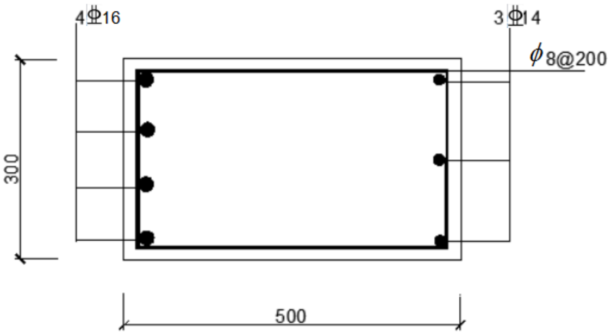


Fig.1 Dimensions and details of specimens

3.2 Calculation of reinforcement of columns

$$C_m = 0.7 + 0.3\frac{M_1}{M_2} = 1$$

$$\xi_C = \frac{0.5 f_C A}{N} = 0.5 \times \frac{14.3 \times 300 \times 500}{650 \times 10^3} = 1.43 > 1$$

$$e_a = \max\{\frac{h}{30}, 20\} mm = 20 mm$$

$$\eta_{ns} = 1 + \frac{1}{1300(\frac{M_2}{N} + e_a)/h_0}(\frac{l_c}{h})^2 \xi_c = 1 + \frac{1}{1300 \times 0.451}(\frac{4000}{500})^2 \times 1 = 1.11$$

$$M = C_m \eta_{ns} M_2 = 1 \times 1.11 \times 150 = 166.5 kN \cdot m$$

$$e_0 = \frac{M}{N} = \frac{166.5 \times 10^6}{650 \times 10^3} = 256 mm$$

$$e_i = e_0 + e_a = 256 + 20 = 276 mm$$

$$e_i = 276 mm > 0.3h_0 = 0.3 \times 460 = 138 mm$$

$$e = e_i + \frac{h}{2} - a_s = 276 + \frac{500}{2} - 40 = 486 mm$$

Calculation of reinforcement area in compression area

$$A_s^` = \frac{Ne-\alpha_1 f_c b x_b (h_0 - 0.5x_b)}{f_y^`(h_0 - a_s^`)} = \frac{Ne-\alpha_1 f_c b h_0^2 \xi_b (1-0.5\xi_b)}{f_y^`(h_0 - a_s^`)}$$

$$= 650\times 486\times 10^3 - 1.0\times 14.3\times$$

$$\frac{300\times 460^2\times 0.518\times(1-0.5\times 0.518)}{360(460-40)} < 0$$

$$A_s^` = \rho_{\min}^` bh = 0.2\%\times 300\times 500 = 300\,mm^2$$

Three steel bars with a diameter of 14 mm are used.

$$A_s^` = 461\,mm^2$$

$$N_u e = \alpha_1 f_c bx(h_0 - \frac{x}{2}) + f_y^` A_s^` (h_0 - a_s^`)$$

$$x = 150\,mm$$

$$A_s = \frac{\alpha_1 f_c bx + f_y^` A_s^` - N_u}{f_y}$$

$$= \frac{1.0\times 14.3\times 300\times 150 + 360\times 461 - 650\times 10^3}{360}$$

$$= 443\,mm^2$$

Four HRB400 steel bars with diameter of 20mm $A_s = 804\,mm^2$

Checking and calculating reinforcement ratio

$$0.6\% < \rho = \frac{A_s + A_s^`}{bh} = \frac{461+804}{300\times 500} = 0.84\% < 5\%$$

3.3 Checking calculation of cracks in columns

1) Calculation of relevant parameters

$$\frac{l_0}{h} = \frac{4000}{500} = 8 < 14, \eta_s = 1.0 \quad h_0 = 460\,mm \quad a_s = 40\,mm \quad e_0 = 256\,mm$$

$$e = \eta_s e_0 + \frac{h}{2} - a_s = 1\times 256 + 250 - 40 = 466\,mm$$

$$\eta h_0 = [0.87 - 0.12(\frac{h_0}{e})^2]h_0 = [0.87 - 0.12(\frac{460}{466})^2]\times 460 = 346\,mm$$

$$\sigma_{sq} = N_q(e - \eta h_0) / (A_s \eta h_0) = 650\times 10^3\times(466-346)/(804\times 346) = 280.3N/mm^2$$

$$\rho_{te} = \frac{A_s}{0.5bh} = \frac{1520}{0.5\times 300\times 500} = 0.02$$

$$\psi = 1.1 - 0.65 f_{tk} / (\rho_{te}\times\sigma_{sq}) = 1.1 - 0.65\times 2.01/(0.02\times 280.3) = 0.87$$

calculation of crack width

$$.\ \omega_{\max} = 1.9\psi\frac{\sigma_{sq}}{E_s}(1.9c_s + 0.08\frac{d_{eq}}{\rho_{te}}) = 1.9\times 0.87\times\frac{280.3}{2\times 10^5}(1.9\times 24 + 0.08\frac{14}{0.02}) = 0.235\,mm$$

3.4 Checking calculation of deflection of columns

1) Calculation of relevant parameters

$$\alpha_E\rho = \frac{E_s}{E_c}\cdot\frac{A_s}{bh_0} = \frac{2\times10^5}{3\times10^4}\cdot\frac{804}{300\times460} = 0.039$$

$$\sigma_{sq}=N_q(e-\eta h_0)\big/(A_s\,\eta h_0) = 650\times10^3\times(466-346)\big/(804\times346) = 280.3N\big/mm^2$$

$$\psi = 1.1-0.65 f_{tk}\big/(\rho_{te}\times\sigma_{sq}) = 1.1-0.65\times2.01\big/(0.02\times280.3) = 0.87$$

$$B_s = \frac{E_s A_s h_0^2}{1.15\psi+0.2+0.6\alpha_E\rho} = \frac{2\times10^5\times804\times460^2}{1.15\times0.87+0.2+0.6\times0.039} = 2.78\times10^{13}N\cdot mm^2$$

Calculation of deflection

$$f = \frac{1}{8}\times\frac{Ml^2}{B_s} = \frac{1}{8}\times\frac{166.5\times10^6\times4000^2}{2.78\times10^{13}} = 11.98mm$$

4. Conclusion

The reinforcement calculation of flexural and compressive members and the checking calculation of deflection and crack deflection are calculated. The results are good and can be used as a reference.

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