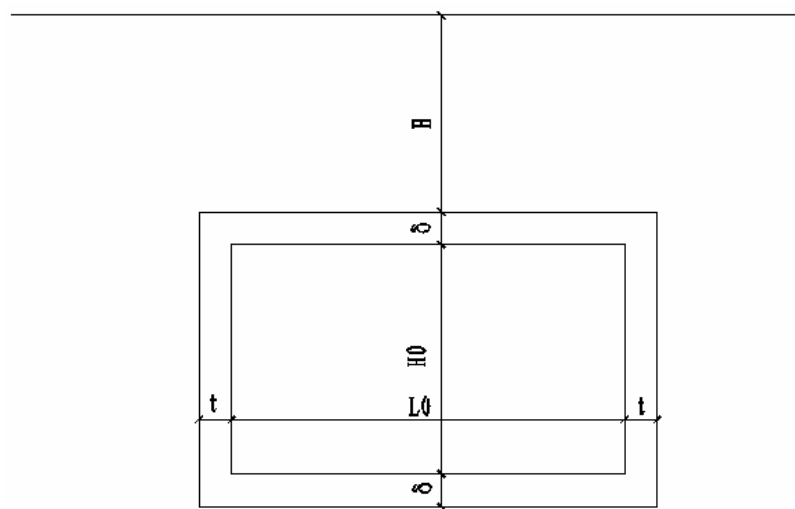


一. 设计资料

地下通道净跨径 $L_0=6\text{m}$ ，净高 $h_0=3.5\text{m}$ ，箱顶填土厚为 3m ，土的内摩擦角 ϕ 为 30° ，填土的密度 $\gamma_1=20\text{KN/m}^3$ 。箱涵主体结构混凝土强度等级为 C30，箱涵基础垫层混凝土强度等级为 C15，纵向受力钢筋采用 HRB335 钢筋。地基为强风化砂岩。汽车荷载等级为城-A 级。



二. 设计计算

(一) 尺寸拟定

顶板、底板厚度 $\delta=50\text{cm}$

侧墙厚度 $t=50\text{cm}$

故计算长度 $l = L_0 + t = 6 + 0.5 = 6.5\text{m}$

$$h = H_0 + \delta = 3.5 + 0.5 = 4.0\text{m}$$

(二) 荷载计算

1. 恒载

竖向恒载标准值 $q_v = \gamma_1 \cdot H + \gamma_2 \cdot \delta = 20 \times 3 + 25 \times 0.5 = 72.5\text{KN/m}^2$

水平恒载标准值

$$\text{顶板处 } q_{h1} = \text{tg}^2(45^\circ - \frac{\phi}{2}) \cdot \gamma_1 \cdot H = \text{tg}^2(45^\circ - \frac{30^\circ}{2}) \times 20 \times 3 = 20 \text{KN/m}^2$$

底板处

$$q_{h2} = \text{tg}^2(45^\circ - \frac{\phi}{2}) \cdot \gamma_1 \cdot (H + h + \delta) = \text{tg}^2(45^\circ - \frac{30^\circ}{2}) \times 20 \times (3 + 4.5) = 50 \text{KN/m}^2$$

2. 活载

一个汽车后轮荷载横向扩散长度 $\frac{0.6}{2} + 3 \times \text{tg} 30^\circ = 2.03 > \frac{1.8}{2}$ ，故两辆车相邻车轴由荷载重叠；一个汽车后轮荷载纵向扩散长度

$$\frac{0.25}{2} + 3 \times \text{tg} 30^\circ = 1.86 \begin{matrix} > \frac{3.6}{2} \\ < \frac{6.0}{2} \end{matrix}。按两辆车相邻计算车轴荷载扩散面积横向$$

分布长 $a = (\frac{0.6}{2} + 3 \times \text{tg} 30^\circ + 1.8) \times 2 + 1.3 = 8.96 \text{m}$ 。纵向分布长分两种情况，

第一种情况考虑 1、2、3 轴荷载重叠，此时纵向分布长

$$b = (\frac{0.25}{2} + 3 \times \text{tg} 30^\circ) \times 2 + 3.6 + 1.2 = 8.52 \text{m}；第二种情况只考虑 4 轴荷载，$$

此时纵向分布长 $b = (\frac{0.25}{2} + 3 \times \text{tg} 30^\circ) \times 2 = 3.72 \text{m}$ 。车辆荷载垂直压力，按

纵向分布第一种情况计算， $q_{v\text{车}} = \frac{2 \times (60 + 140 + 140)}{8.96 \times 8.52} = 8.91 \text{KN/m}^2$ ；按纵

向分布第二种情况计算， $q_{v\text{车}} = \frac{2 \times 200}{8.96 \times 3.72} = 12.0 \text{KN/m}^2$ 。

取车辆荷载垂直压力标准值 $q_{v\text{车}} = 12.0 \text{KN/m}^2$ 。

车辆荷载水平压力标准值

$$q_{h\text{车}} = q_{v\text{车}} \cdot \text{tg}^2(45^\circ - \frac{\phi}{2}) = 12.0 \times \text{tg}^2(45^\circ - \frac{30^\circ}{2}) = 4.0 \text{KN/m}^2$$

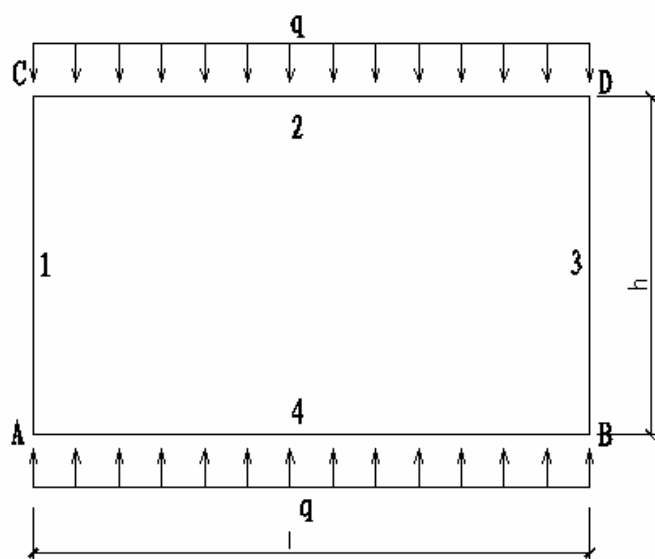
(三) 内力计算

1. 结点内力计算

$$K = \frac{h}{l} \cdot \frac{I_2}{I_1} = \frac{4}{6.5} = 0.6154$$

(1) 竖向恒载作用下

计算简图如下：



$$M_A = M_B = M_C = M_D = -\frac{q_v \cdot l^2}{12(K+1)} = -\frac{72.5 \times 6.5^2}{12 \times (0.6154 + 1)} = -158.02 \text{ KN} \cdot \text{m}$$

$$N_1 = N_3 = \frac{q \cdot l}{2} = \frac{72.5 \times 6.5}{2} = 235.63 \text{ KN} \quad N_2 = N_4 = 0$$

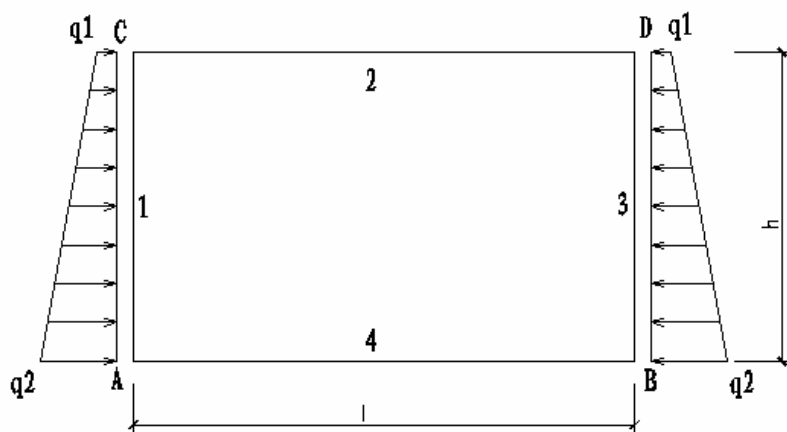
(2) 竖向活载作用下

$$M_A = M_B = M_C = M_D = -\frac{q_{v\text{车}} \cdot l^2}{12(K+1)} = -\frac{12.0 \times 6.5^2}{12 \times (0.6154 + 1)} = -26.15 \text{ KN} \cdot \text{m}$$

$$N_1 = N_3 = \frac{q \cdot l}{2} = \frac{12.0 \times 6.5}{2} = 39.0 \text{ KN} \quad N_2 = N_4 = 0$$

(3) 水平恒载作用下

计算简图如下：



$$M_A = M_B = -\frac{Kq_{h1}h^2}{12(K+1)} - \frac{(q_{h2}-q_{h1})h^2K(3K+8)}{60(K^2+4K+3)}$$

$$= -\frac{0.6154 \times 20 \times 4^2}{12 \times (0.6154+1)} - \frac{(50-20) \times 4^2 \times 0.6154 \times (3 \times 0.6154+8)}{60(0.6154^2+4 \times 0.6154+3)} = -18.46 \text{ KN} \cdot \text{m}$$

$$M_C = M_D = -\frac{Kq_{h1}h^2}{12(K+1)} - \frac{(q_{h2}-q_{h1})h^2K(2K+7)}{60(K^2+4K+3)}$$

$$= -\frac{0.6154 \times 20 \times 4^2}{12 \times (0.6154+1)} - \frac{(50-20) \times 4^2 \times 0.6154 \times (2 \times 0.6154+7)}{60(0.6154^2+4 \times 0.6154+3)} = -17.10 \text{ KN} \cdot \text{m}$$

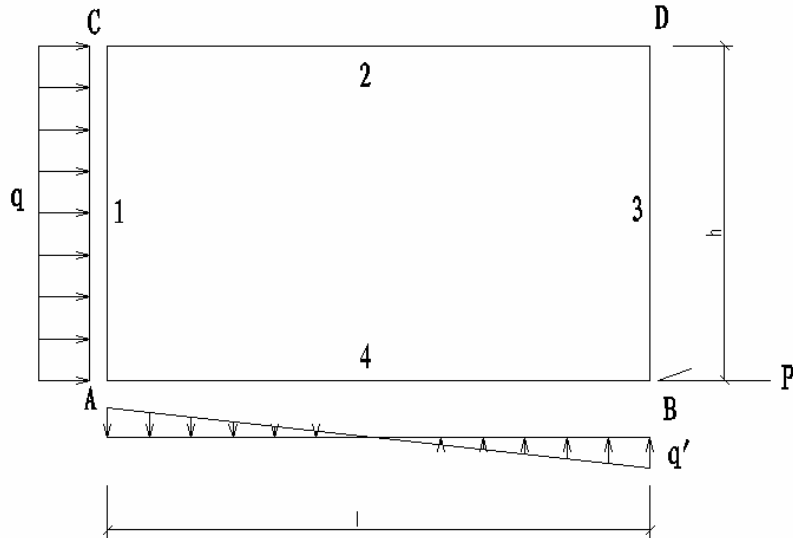
$$N_2 = \frac{q_{h1} \bullet h}{2} + \frac{(q_{h2}-q_{h1})h}{6} + \frac{M_A - M_C}{h} = \frac{20 \times 4}{2} + \frac{(50-20) \times 4}{6} + \frac{-18.46+17.10}{4} = 59.66 \text{ KN}$$

$$N_4 = \frac{q_{h1} \bullet h}{2} + \frac{(q_{h2}-q_{h1})h}{3} - \frac{M_A - M_C}{h} = \frac{20 \times 4}{2} + \frac{(50-20) \times 4}{3} - \frac{-18.46+17.10}{4} = 80.34 \text{ KN}$$

$$N_1 = N_3 = 0$$

(4)水平活载作用下

计算简图如下：



$$M_A = M_B = -\left[\frac{K}{24(K+1)} \pm \frac{5K+1}{10(3K+1)}\right]q_{h\mp}h^2 = -\left[\frac{0.6154}{24(0.6154+1)} \pm \frac{5 \times 0.6154+1}{10(3 \times 0.6154+1)}\right] \times 4 \times 4^2 = \begin{matrix} -10.18 \\ 8.15 \end{matrix} \text{ KN} \cdot \text{m}$$

$$M_C = M_D = -\left[\frac{K}{24(K+1)} \pm \frac{5K+1}{20(3K+1)}\right]q_{h\mp}h^2 = -\left[\frac{0.6154}{24(0.6154+1)} \pm \frac{5 \times 0.6154+3}{20(3 \times 0.6154+1)}\right] \times 4 \times 4^2 = \begin{matrix} 5.82 \\ -7.85 \end{matrix} \text{ KN} \cdot \text{m}$$

$$N_1 = N_3 = -\frac{M_C - M_D}{l} = -\frac{5.82+7.85}{6.5} = 2.10 \text{ KN}$$

$$N_2 = \frac{M_B - M_D}{h} = \frac{8.15+7.85}{4} = 4 \text{ KN}$$

$$N_4 = q_{h\text{车}}h - \frac{M_B - M_D}{h} = 4 \times 4 - \frac{8.15 + 7.85}{4} = 12 \text{KN}$$

(5)节点弯矩和轴力计算汇总表

荷载种类	弯矩(KN•m)				轴力(KN)			
	A	B	C	D	1	2	3	4
恒载	-158.02	-158.02	-158.02	-158.02	235.63		235.63	
	-18.46	-18.46	-17.10	-17.10		59.66		80.34
小计	-176.48	-176.48	-175.12	-175.12	235.63	59.66	235.63	80.34
活载	-26.15	-26.15	-26.15	-26.15	39.00		39.00	
	-10.18	8.15	5.82	-7.85	2.10	4.00	2.10	12.00
小计	-36.33	-18.00	-20.33	-34.00	41.10	4.00	41.10	12.00
基本组合	-262.64	-236.97	-238.61	-257.74	340.30	77.19	340.30	113.21
短期效应组合	-201.91	-189.08	-189.35	-198.92	264.40	62.46	264.40	88.74

注：基本组合 $S_{ud} = 1.2S_{GK} + 1.4S_{QK}$ ，短期效应组合 $S_{sd} = S_{GK} + 0.7S_{QK}$

2. 跨中截面内力计算

(1)顶板

顶板均布荷载设计值 $q = 1.2q_v + 1.4q_{v\text{车}} = 1.2 \times 72.5 + 1.4 \times 12 = 103.8 \text{KN/m}^2$

跨中弯矩

$$M_{\text{中}} = M_C + N_1 \cdot \frac{l}{2} - \frac{ql^2}{8} = -236.97 + 340.3 \times \frac{6.5}{2} - \frac{103.8 \times 6.5^2}{8} = 320.81 \text{KN} \cdot \text{m}$$

跨中剪力

$$V_{\text{中}} = \frac{ql}{2} - N_1 = \frac{103.8 \times 6.5}{2} - 340.3 = -2.95 \text{KN}$$

(2)底板

$$q' = \frac{3q_{h\text{车}}h^2}{l^2} = \frac{3 \times 4 \times 4^2}{6.5^2} = 4.54 \text{KN/m}^2$$

底板非均布荷载设计值

$$w_1 = 1.2q_v + 1.4(q_{v\text{车}} - q') = 1.2 \times 72.5 + 1.4 \times (12 - 4.54) = 97.44 \text{KN/m}^2$$

$$w_2 = 1.2q_v + 1.4(q_{v\text{车}} + q') = 1.2 \times 72.5 + 1.4 \times (12 + 4.54) = 110.16 \text{KN/m}^2$$

$$M_{\text{中}} = M_A + N_1 \cdot \frac{l}{2} - \frac{w_1 l^2}{8} - \frac{(w_2 - w_1) l^2}{48}$$

$$= -262.64 + 340.3 \times \frac{6.5}{2} - \frac{97.44 \times 6.5^2}{8} - \frac{(110.16 - 97.44) \times 6.5^2}{48} = 317.53 \text{ KN} \cdot \text{m}$$

$$V_{\text{中}} = \frac{w_1 l}{2} + \frac{(w_2 - w_1) l}{8} - N_1 = \frac{97.44 \times 6.5}{2} + \frac{(110.16 - 97.44) \times 6.5}{8} - 340.3 = -13.29 \text{ KN}$$

(3)左侧墙

$$w_1 = 1.2q_{h1} + 1.4q_{h\text{车}} = 1.2 \times 20 + 1.4 \times 4 = 29.6 \text{ KN/m}^2$$

$$w_1 = 1.2q_{h2} + 1.4q_{h\text{车}} = 1.2 \times 50 + 1.4 \times 4 = 65.6 \text{ KN/m}^2$$

$$M_{\text{中}} = M_C + N_2 \cdot \frac{h}{2} - \frac{w_1 h^2}{8} - \frac{(w_2 - w_1) h^2}{48}$$

$$= -238.61 + 77.19 \times \frac{4}{2} - \frac{29.6 \times 4^2}{8} - \frac{(65.6 - 29.6) \times 4^2}{48} = -155.43 \text{ KN} \cdot \text{m}$$

$$V_{\text{中}} = \frac{w_1 h}{2} + \frac{(w_2 - w_1) h}{8} - N_2 = \frac{29.6 \times 4}{2} + \frac{(65.6 - 29.6) \times 4}{8} - 77.19 = -0.01 \text{ KN}$$

(4)右侧墙

$$w_1 = 1.2q_{h1} = 1.2 \times 20 = 24 \text{ KN/m}^2 \quad w_1 = 1.2q_{h2} = 1.2 \times 50 = 60 \text{ KN/m}^2$$

$$M_{\text{中}} = M_D + N_2 \cdot \frac{h}{2} - \frac{w_1 h^2}{8} - \frac{(w_2 - w_1) h^2}{48}$$

$$= -257.74 + 77.19 \times \frac{4}{2} - \frac{24 \times 4^2}{8} - \frac{(60 - 24) \times 4^2}{48} = -163.36 \text{ KN} \cdot \text{m}$$

$$V_{\text{中}} = \frac{w_1 h}{2} + \frac{(w_2 - w_1) h}{8} - N_2 = \frac{20 \times 4}{2} + \frac{(60 - 20) \times 4}{8} - 77.19 = -17.19 \text{ KN}$$

3.作用基本组合构件内力汇总表

构件	M	N	V	M	N	V	M	N	V
C-D	C			跨中			D		
	-238.61	77.19	340.3	320.81	77.19	-2.95	-257.74	77.19	-340.3
A-B	A			跨中			B		
	-262.64	113.21	340.3	317.53	113.21	-13.29	-236.97	113.21	-340.3
A-C	A			跨中			C		
	-262.64	340.3	113.21	-155.43	340.3	0.01	-238.61	340.3	-77.19
B-D	B			跨中			D		
	-236.97	340.3	77.19	-163.36	340.3	-17.19	-257.74	340.3	-113.21

4.作用短期组合构件内力汇总表

作用短期组合构件内力计算方法及步骤同基本作用，汇总表如下：

构件	Ms	Ns	Ms	Ns	Ms	Ns
C-D	C		跨中		D	
	-189.35	62.46	242.70	62.46	-198.92	62.46
A-B	A		跨中		B	
	-201.91	88.74	241.33	88.74	-189.08	88.74
A-C	A		跨中		C	
	-201.91	264.4	-120.03	264.4	-189.35	264.4
B-D	B		跨中		D	
	-189.08	264.4	-124.00	264.4	-198.92	264.4

(四) 截面配筋计算

结构左右和上下对称配筋，采用最不利荷载计算

(1)顶板跨中

①配筋计算

计算跨度 $l_0 = 6.5m$ ，截面高度 $h = 500mm$ ，截面宽度 $b = 1000mm$ ，钢

筋保护层厚度 $a = 40mm$

$$e_0 = \frac{M}{N} = \frac{320.81}{77.19} = 4.156m$$

$$i = \sqrt{\frac{bh^2}{12}} = \sqrt{\frac{0.5^2}{12}} = 0.1443$$

$$\frac{l_0}{i} = \frac{6.5}{0.1443} = 45.03 > 17.5$$

根据《公路钢筋混凝土和预应力钢筋混凝土桥涵设计规范》（以下简称桥涵规范）JTG D62-2004 第 5.3.10 条

$$\zeta_1 = 0.2 + 2.7 \frac{e_0}{h_0} = 0.2 + 2.7 \times \frac{4156}{445} = 25.42 > 1.0, \text{ 取 } \zeta_1 = 1.0$$

$$\zeta_2 = 1.15 - \frac{0.01l}{h} = 1.15 - \frac{0.01 \times 6500}{500} = 1.02, \text{ 取 } \zeta_2 = 1.0$$

$$\eta = 1 + \frac{1}{1400e_0/h_0} \left(\frac{l_0}{h}\right)^2 \zeta_1 \zeta_2 = 1 + \frac{1}{1400 \times 4156/445} \times \left(\frac{6500}{500}\right)^2 \times 1.0 \times 1.0 = 1.013$$

按桥涵规范 (5.3.5-3) 公式, 轴向压力作用点至纵向受拉钢筋的合力点的距离:

$$e = \eta e_0 + \frac{h}{2} - a = 1.013 \times 4156 + \frac{500}{2} - 55 = 4405 \text{ mm}$$

$$Ne = f_{cd} b x \left(h_0 - \frac{x}{2}\right)$$

$$77.19 \times 1000 \times 4405 = 13.8 \times 1000 \times x \times \left(445 - \frac{x}{2}\right)$$

解得 $x = 59.3 \text{ mm} < \xi_b h_0 = 0.56 \times 445 = 249.2$, 为大偏心受压构件

$$A_s = \frac{f_{cd} b x - N}{f_y} = \frac{13.8 \times 1000 \times 59.3 - 77.19 \times 1000}{280} = 2647 \text{ mm}^2$$

实配 25@125, $A_s = 3927 \text{ mm}^2$

跨中剪力值较小, 未做斜截面抗剪承载力验算

②正常使用极限状态计算

根据桥涵规范 6.4.4 条

$$\gamma_f' = \frac{(b_f' - b) h_f'}{b h_0} = 0$$

$$l_0/h = 6500/500 = 13 < 14 \quad \text{取 } \eta_s = 1.0$$

$$y_s = \frac{h}{2} - a_s = \frac{500}{2} - 55 = 195 \text{ mm}$$

$$e_s = \eta_s e_0 + y_s = 1.0 \times 4156 + 195 = 4351 \text{ mm}$$

$$z = \left[0.87 - 0.12(1 - \gamma_s) \left(\frac{h_0}{e_s}\right)^2\right] h_0 = \left[0.87 - 0.12 \times (1 - 0) \times \left(\frac{445}{4351}\right)^2\right] \times 445 = 386.6 \text{ mm}$$

$$\sigma_{ss} = \frac{N_s (e_s - z)}{A_s z} = \frac{62.46 \times (4351 - 386.6)}{3927 \times 386.6} = 163.10 \text{ N/mm}^2$$

根据桥涵规范 6.4.3 条, 验算裂缝宽度

对带肋钢筋 $C_1 = 1.0$

$$\text{作用长期影响系数 } C_2 = 1 + 0.5 \frac{N_l}{N_s} = 1 + 0.5 \times \frac{61.26}{62.46} = 1.4904$$

$$C_3 = 0.9$$

裂缝宽度

$$\begin{aligned} W_{tk} &= C_1 C_2 C_3 \frac{\sigma_{ss}}{E_s} \left(\frac{30+d}{0.28+10\rho} \right) \\ &= 1 \times 1.4904 \times 0.9 \times \frac{163.10}{2.0 \times 10^5} \left(\frac{30+25}{0.28+10 \times 0.007854} \right) = 0.168 \text{ mm} < 0.2 \text{ mm} \end{aligned}$$

满足要求

(2)顶板节点

计算步骤同上。

$$e_0 = \frac{M}{N} = \frac{257.74}{77.19} = 3.339 \text{ m}$$

$$\eta = 1 + \frac{1}{1400 e_0 / h_0} \left(\frac{l_0}{h} \right)^2 \zeta_1 \zeta_2 = 1 + \frac{1}{1400 \times 3339 / 445} \times \left(\frac{6500}{500} \right)^2 \times 1.0 \times 1.0 = 1.016$$

$$e = \eta e_0 + \frac{h}{2} - a = 1.016 \times 3339 + \frac{500}{2} - 55 = 3588 \text{ mm}$$

$$Ne = f_{cd} b x \left(h_0 - \frac{x}{2} \right)$$

$$77.19 \times 1000 \times 3588 = 13.8 \times 1000 \times x \times \left(445 - \frac{x}{2} \right)$$

解得 $x = 47.7 < \xi_b h_0 = 0.56 \times 445 = 249.2$ ，为大偏心受压构件

$$A_s = \frac{f_{cd} b x - N}{f_{sd}} = \frac{13.8 \times 1000 \times 47.7 - 77.19 \times 1000}{280} = 2075 \text{ mm}^2$$

实配 22@125, $A_s = 3041 \text{ mm}^2$

根据《公路钢筋混凝土和预应力钢筋混凝土桥涵设计规范》(以下简称桥涵规范) JTG D62-2004 中 5.2.9 条

$$\begin{aligned} 0.51 \times 10^{-3} \sqrt{f_{cu,k}} b h_0 &= 0.51 \times 10^{-3} \times \sqrt{30} \times 1000 \times 445 = 1243.06 \text{ KN} \\ &> \gamma_0 V = 1.0 \times 340.3 = 340.3 \text{ KN} \end{aligned}$$

抗剪截面尺寸符合要求

根据桥涵规范 5.2.10 条

$$0.5 \times 10^{-3} \times 1.25 \times \alpha_2 f_{td} b h_0 = 0.5 \times 10^{-3} \times 1.25 \times 1.0 \times 1.39 \times 1000 \times 445 = 386.59 \\ > \gamma_0 V_d = 1.0 \times 340.3 = 340.3 \text{KN}$$

式中 1.25 为板式受弯构件提高系数，抗剪配筋按构造设置

(3)底板跨中

$$e_0 = \frac{M}{N} = \frac{317.53}{113.21} = 2.805 \text{m}$$

$$\eta = 1 + \frac{1}{1400 e_0 / h_0} \left(\frac{l_0}{h} \right)^2 \zeta_1 \zeta_2 = 1 + \frac{1}{1400 \times 2805 / 445} \times \left(\frac{6500}{500} \right)^2 \times 1.0 \times 1.0 = 1.019$$

$$e = \eta e_0 + \frac{h}{2} - a = 1.019 \times 2805 + \frac{500}{2} - 55 = 3054 \text{mm}$$

$$Ne = f_{cd} b x \left(h_0 - \frac{x}{2} \right)$$

$$113.21 \times 1000 \times 3054 = 13.8 \times 1000 \times x \times \left(445 - \frac{x}{2} \right)$$

解得 $x = 60.4 < \xi_b h_0 = 0.55 \times 445 = 244.75$ ，为大偏心受压构件

$$A_s = \frac{f_{cd} b x - N}{f_{sd}} = \frac{13.8 \times 1000 \times 60.4 - 113.21 \times 1000}{280} = 2573 \text{mm}^2$$

实配 25@125， $A_s = 3927 \text{mm}^2$

(4)底板节点

$$e_0 = \frac{M}{N} = \frac{262.64}{113.21} = 2.32 \text{m}$$

$$\eta = 1 + \frac{1}{1400 e_0 / h_0} \left(\frac{l_0}{h} \right)^2 \zeta_1 \zeta_2 = 1 + \frac{1}{1400 \times 2320 / 445} \times \left(\frac{6500}{500} \right)^2 \times 1.0 \times 1.0 = 1.023$$

$$e = \eta e_0 + \frac{h}{2} - a = 1.023 \times 2320 + \frac{500}{2} - 55 = 2569 \text{mm}$$

$$Ne = f_{cd} b x \left(h_0 - \frac{x}{2} \right)$$

$$113.21 \times 1000 \times 2569 = 13.8 \times 1000 \times x \times \left(445 - \frac{x}{2} \right)$$

解得 $x = 50.2 < \xi_b h_0 = 0.56 \times 445 = 249.2$ ，为大偏心受压构件

$$A_s = \frac{f_{cd} b x - N}{f_{sd}} = \frac{13.8 \times 1000 \times 50.2 - 113.21 \times 1000}{280} = 2070 \text{mm}^2$$

实配 22@125, $A_s = 3041\text{mm}^2$

(5)左右侧墙跨中

计算跨度 $l = 4.0\text{m}$, 截面高度 $h = 500\text{mm}$, 截面宽度 $b = 1000\text{mm}$, 钢筋保护层厚度 $a = 40\text{mm}$

计算步骤同上。

$$e_0 = \frac{M}{N} = \frac{163.36}{340.3} = 0.48\text{m}$$

$$\zeta_1 = 0.2 + 2.7 \frac{e_0}{h_0} = 0.2 + 2.7 \times \frac{480}{445} = 3.11 > 1.0, \text{ 取 } \zeta_1 = 1.0$$

$$\zeta_2 = 1.15 - \frac{0.01l}{h} = 1.15 - \frac{0.01 \times 4000}{500} = 1.07, \text{ 取 } \zeta_2 = 1.0$$

$$\eta = 1 + \frac{1}{1400e_0/h_0} \left(\frac{l_0}{h}\right)^2 \zeta_1 \zeta_2 = 1 + \frac{1}{1400 \times 500/445} \times \left(\frac{4000}{500}\right)^2 \times 1.0 \times 1.0 = 1.042$$

$$e = \eta e_0 + \frac{h}{2} - a = 1.042 \times 500 + \frac{500}{2} - 55 = 695.4\text{mm}$$

$$Ne = f_{cd}bx(h_0 - \frac{x}{2})$$

$$340.3 \times 1000 \times 695.4 = 13.8 \times 1000 \times x \times (445 - \frac{x}{2})$$

解得 $x = 40.4 < \xi_b h_0 = 0.56 \times 445 = 249.2$, 为大偏心受压构件

$$A_s = \frac{f_{cd}bx - N}{f_{sd}} = \frac{13.8 \times 1000 \times 40.4 - 340.3 \times 1000}{280} = 776\text{mm}^2$$

实配 22@125, $A_s = 3041\text{mm}^2$

(5)左右侧墙节点

计算步骤同上。

$$e_0 = \frac{M}{N} = \frac{262.64}{340.3} = 0.772\text{m}$$

$$\eta = 1 + \frac{1}{1400e_0/h_0} \left(\frac{l_0}{h}\right)^2 \zeta_1 \zeta_2 = 1 + \frac{1}{1400 \times 772/445} \times \left(\frac{4000}{500}\right)^2 \times 1.0 \times 1.0 = 1.026$$

$$e = \eta e_0 + \frac{h}{2} - a = 1.026 \times 772 + \frac{500}{2} - 55 = 987\text{mm}$$

$$Ne = f_{cd}bx(h_0 - \frac{x}{2})$$

$$340.3 \times 1000 \times 987 = 13.8 \times 1000 \times x \times (445 - \frac{x}{2})$$

解得 $x = 58.5 < \xi_b h_0 = 0.56 \times 445 = 249.2$ ，为大偏心受压构件

$$A_s = \frac{f_{cd}bx - N}{f_{sd}} = \frac{13.8 \times 1000 \times 58.5 - 340.3 \times 1000}{280} = 1668 \text{mm}^2$$

实配 22@125, $A_s = 3041 \text{mm}^2$