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Calculation of Bearing Capability for Pier Base While Highway Crossing under Existing Railway Bridge

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ABSTRACT: With the rapid development of transportation, more and more expressways cross railway bridges. However, the design and construction of the railway constructed first did not consider the impact of the subsequent expressway construction and its load, so the expressway undercrossing the railway has brought hidden dangers to the railway safety. Taking an expressway crossing a railway as an example, this paper analyzes the overturning stability under uneven soil pressure, and effectively anticipates the possible adverse conditions in advance, considering that the bridge pier bears the traffic load, wind load and other loads from the upper railway bridge, as well as the comprehensive action of the backfill load on the pier foundation and the traffic load on the expressway brought by the new expressway subgrade, relevant treatment measures shall be formulated.

Key words:express; cross under; existing railway; bearing capability; calculation

I. INTRODUCTION

With the rapid development of highways and railways, there are increasingly more engineering cases of highways passing under railway bridges. However, as railways were built before highways and their loads were not considered, the construction of highways passing under railways has brought potential safety hazards to the railway. In order to ensure the safety of both the railway and the highway, it is necessary to calculate the bearing capacity of the foundation of existing railway bridge piers under which the highway will pass, and based on the calculation results, take corresponding safety control measures.

Regarding the current research on intersectional engineering, more emphasis has been placed on the impact of tunnels and excavations on existing routes. Lo et al. (1991) studied excavations above the Toronto subway as the research object and found that excavation construction had an impact on the longitudinal and transverse deformation of existing tunnels. They also evaluated the project, analyzed the construction control standards and operational requirements of existing tunnels. Kulesza et al. (1998) and Wilson et al. (2000) studied the new MUNI tunnel project above the existing BART tunnel and conducted a displacement allowance value study based on the BART tunnel during the design stage. They proposed a measurement and monitoring plan and process for existing tunnel projects. In terms of research on highway and railway intersectional engineering, Xu (2013) and Lu (2013) simulated the deformation of railway subgrade under multiple factors during intersectional engineering construction by establishing a finite element analysis model. They analyzed the main sources of danger and safety risk factors for highway and railway intersectional engineering and proposed a corresponding safety risk assessment indicator system. Qian et al. (2008) summarized the numerical modeling research methods for predicting ground movement and surface settlement in crossing projects both domestically and internationally. They compiled a set of new theoretical numerical models for underground engineering, opening up a new direction for large-scale geotechnical engineering research.

Currently, there has been some research on the bearing capacity of the foundation of existing railway bridge piers under which highways pass, but each project has its own characteristics and lacks universality. Therefore, this paper takes a specific case of a highway passing under a railway and proposes corresponding research for the specific project, providing reference for similar projects.

1 Engineering background

The Tongtianlong Viaduct at K181+270 of a certain highway is located in Tongtianlong, Huaihua City. The roadbed segment behind the Huaihua side abutment of the bridge passes under the

Chongqing-Huaihua Railway. The crossing angle between the highway and the railway is 86°and the station number at the intersection is highway K181+308.06=railway K610+816.36. The railway underpass is located on a gentle curve with a radius of 1,200 meters.

The highway passes under the second and third spans of the existing railway bridge, and pier 2# of the railway bridge is located in the central median of the highway. Considering that pier 2# not only bears the loads from the upper railway bridge, such as the train and wind loads, but also the loads from the fill soil and the highway traffic loads on the pier foundation brought by the newly constructed highway embankment, and the stability of pier 3# facing the uneven soil pressure, the bearing capacity of pier 2# foundation is calculated, and the overturning stability of the pier foundation of pier 3# is also checked to effectively predict possible adverse situations in advance.

2 Calculation of pier stability

2.1 Calculation basis

According to the calculation loads stipulated by the Chinese Railway Bridge and Culvert Design Code. Using a static calculation method, evaluate the impact of pier pressure on the base under various adverse load combinations based on the design-related data such as dimensions, elevations, hydrology, geology.

2.2 Load and load combination calculation

(1) Main load

①Constant load

(a) Weight of superstructure on bridge (sidewalks on both sides)

$$N_1^{'} = 2212.22 + 32.73 \times 36.6 = 3410.14\text{kN} \tag{1}$$

$$N_2^{'} = 1672.18 + 24.74 \times 36.6 = 2577.66\text{kN} \tag{2}$$

$$N_1 = \frac{N_1^{'} + N_2^{'}}{2} = \frac{3410.14 + 2577.66}{2} = 2993.90\text{kN} \tag{3}$$

Where, 32.73m is the sum of the left beam length and the joint, and 24.74m is the sum of the right beam length and the joint.

(b) Weight of top cap and tray

$$N_2 = \frac{1}{2} \times 2.7 \times 6 \times 0.5 \times 25 + 2.7 \times 6 \times 0.5 \times 25 = 303.75\text{kN} \tag{4}$$

$$N_3 = \frac{1}{2} \times (5.6 + 3.2) \times 1.5 \times 23 = 151.80\text{kN} \tag{5}$$

(c) Weight of pier body

When the height of pier is h, the corresponding volume is obtained as Eq. (6).

$$V = 2.3 \times 3.6h + (\frac{3.6}{59} + \frac{2.3}{42})h^2 + \frac{4}{3 \times 59 \times 42}h^3 = 8.28h + 0.116h^2 + \frac{h^3}{1858.5} \tag{6}$$

Therefore, the weight of pier body is determined according to the value of h, as displayed in Table 1.

Table 1 Weight of the pier body for different condition of h

h/m	Volume (V/m³)	Weight (kN)
0	0	0
2	17.0283	391.65
3.68	32.0678	737.56
6	53.9722	1241.36
9	84.3123	1939.18
12	116.9898	2690.77
14	140.1365	3223.14

②Live load

According to the definition of centrifugal force rate C, it can be calculated by Eq. (7).

$$C = \frac{V^2}{127R} = \frac{140^2}{127 \times 1200} = 0.1286 < 0.15 \tag{7}$$

(a) Light load for the condition of single hole (Fig. 1)

The reaction force of static live load is calculated as Eq. (8).

$$R_1 = \frac{1}{32.03} \times \left[5 \times 220 \times (3 - 0.35) + 92 \times 25.23 \times (32.73 - 0.35 - \frac{25.23}{2})\right] = 1523.34\text{kN} \tag{8}$$

The eccentric bending moment of reaction force of static live load on pier center is calculated as Eq. (9).

$$M_{R1} = R_1 \times 0.35 = 1523.34 \times 0.35 = 533.17\text{kN} \cdot \text{m} \quad (9)$$

The centrifugal force is calculated as Eq. (10).

$$P_y = R_1 \times 0.1286 = 1523.34 \times 0.1286 = 195.90\text{kN} \quad (10)$$

The force arm from action point to pier shaft top is calculated as Eq. (11).

$$C_y = 2 + 0.15 + 3.1 + 2 = 7.25\text{m} \quad (11)$$

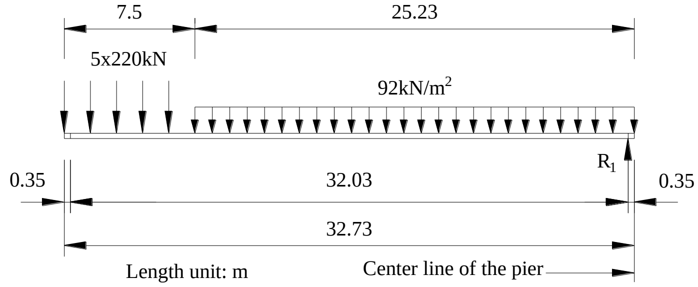

Fig. 1 Diagram of light load for the condition of single hole

(b) Heavy load for the condition of single hole (Fig. 2)

The reaction force of static live load is calculated as Eq. (12).

$$R_1 = \frac{1}{32.03} \times \left[5 \times 220 \times (32.73 - 0.35 - 3) + 92 \times 25.23 \times \left(\frac{25.23}{2} - 0.35\right)\right] = 1897.82\text{kN} \quad (12)$$

The eccentric bending moment of reaction force of static live load on pier center is calculated as Eq. (13).

$$M_{R2} = 1897.82 \times 0.35 = 664.24\text{kN} \quad (13)$$

The centrifugal force is calculated as Eq. (14).

$$P_y = 1897.82 \times 0.1286 = 244.06\text{kN} \quad (14)$$

The force arm from action point to pier shaft top is calculated as Eq. (15).

$$C_y = 2 + 0.15 + 3.1 + 2 = 7.25\text{m} \quad (15)$$

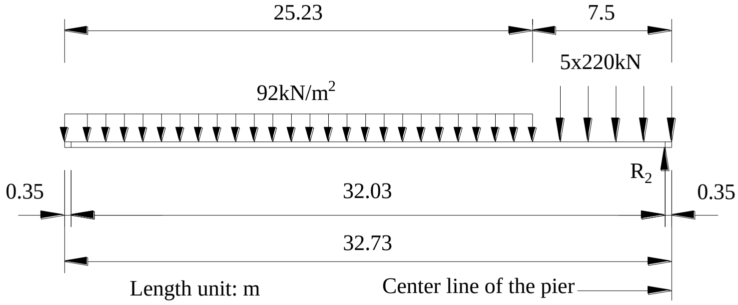

Fig. 2 Diagram of heavy load for the condition of single hole

(c) Heavy load for the condition of double holes (Fig. 3)

The lengths of beams on both sides of the pier are unequal, thus the value of x in

$$\begin{cases} G_1 = 5 \times 220 + 92 \times (24.88 - x) \\ G_2 = 92 \times (5.12 + x) + 80 \times (19.27 - x) \end{cases} \quad (17)$$

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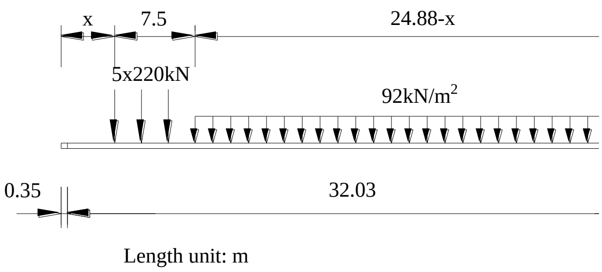

Fig. 3 is deduced according the torque balance (Eq. (16)). The G1 and G2 is expressed as Eq. (17), as a result, the x is equal to 6.64m.

$$\frac{G_1}{32.73} = \frac{G_2}{24.74} \qquad (16)$$

$$\begin{cases} G_1 = 5\times 220 + 92\times(24.88 - x) \\ G_2 = 92\times(5.12 + x) + 80\times(19.27 - x) \end{cases} \qquad (17)$$

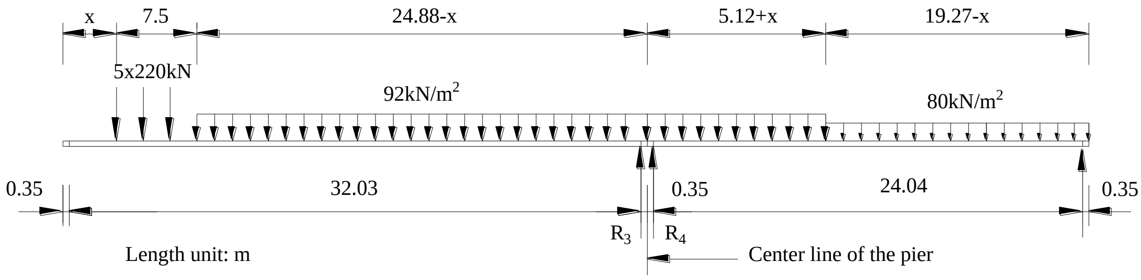

Fig. 3 Diagram of heavy load for the condition of double holes

The reaction force of static live load is calculated as Eq. (18), Eq. (19), Eq. (20).

$$R_3 = \frac{1}{32.03}\times[5\times 220\times(6.64+3) + 92\times 18.24\times(32.38 - \frac{18.24}{2})] = 1549.68\text{kN} \qquad (18)$$

$$R_4 = \frac{1}{24.04}\times[80\times 12.63\times\frac{12.63}{2} + 92\times 11.76\times(24.39 - \frac{11.76}{2})] = 1098.46\text{kN} \qquad (19)$$

$$R_3 + R_4 = 1549.68 + 1098.46 = 2648.14\text{kN} \qquad (20)$$

The eccentric bending moment of reaction force of static live load on pier center is calculated as Eq. (21).

$$M_R = (R_3 - R_4)\times 0.35 = (1549.68 - 1098.46)\times 0.35 = 157.93\text{kN}\cdot\text{m} \qquad (21)$$

The centrifugal force is calculated as Eq. (22).

$$P_y = (R_3 + R_4)\times 0.1286 = (1549.68 + 1098.46)\times 0.1286 = 340.55\text{kN} \qquad (22)$$

The force arm from action point to pier shaft top is calculated as Eq. (23).

$$C_y = 2 + 0.15 + 3.1 + 2 = 7.25\text{m} \qquad (23)$$

(d) Empty load for the condition of double holes (Fig. 4)

The reaction force of static live load is calculated as Eq. (24), Eq. (25), Eq. (26).

$$R_5 = \frac{1}{32.03}\times[10\times 32.73\times(\frac{32.73}{2} - 0.35)] = 163.65\text{kN} \qquad (24)$$

$$R_6 = \frac{1}{24.04}\times[10\times 24.74\times(\frac{24.74}{2} - 0.35)] = 123.7\text{kN} \qquad (25)$$

$$R_5 + R_6 = 163.65 + 123.7 = 287.35\text{kN} \qquad (26)$$

The eccentric bending moment of reaction force of static live load on pier center is calculated as Eq. (27).

$$M_R = (R_5 - R_6)\times 0.35 = (163.65 - 123.7)\times 0.35 = 13.98\text{kN}\cdot\text{m} \qquad (27)$$

The centrifugal force is calculated as Eq. (28).

$$P_y = (R_5 + R_6)\times 0.1286 = (163.65 + 123.7)\times 0.1286 = 36.95\text{kN} \qquad (28)$$

The force arm from action point to pier shaft top is calculated as Eq. (29).

$$C_y = 2 + 0.15 + 3.1 + 2 = 7.25\text{m} \qquad (29)$$

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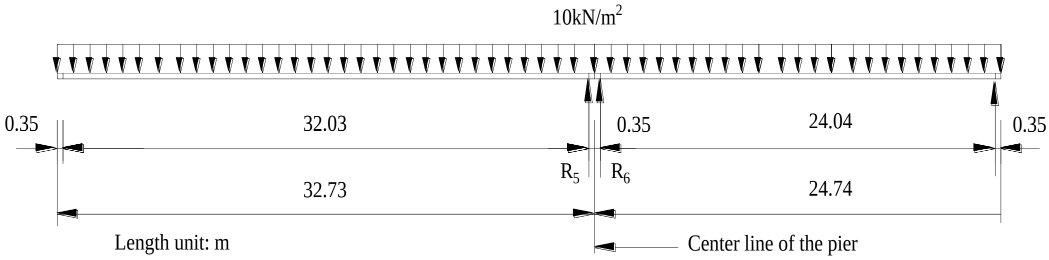

Fig. 4 Diagram of heavy load for the condition of double holes

(2) Additional load

①Braking force

(a) Train braking force P_x under light load for the condition of single hole is calculated as Eq. (30), and the height c from the point of application of braking force to the top of pier shaft is calculated as Eq. (31).

$$P_x = 0.1 \times (5 \times 220 + 92 \times 25.23) = 342.12\text{kN} \quad (30)$$

$$c = 0.32 + 2 = 2.32\text{m} \quad (31)$$

(b) Train braking force P_x under heavy load for the condition of single hole is calculated as Eq. (32), and the height c from the point of application of braking force to the top of pier shaft is the same as Eq. (31).

$$P_x = 0.1 \times (5 \times 220 + 92 \times 25.23) = 342.12\text{kN} \quad (32)$$

(c) Train braking force P_x under heavy load for the condition of double holes is composed of two parts, one is the braking force transmitted through fixed supports (Eq. (33)), another one is the braking force transmitted through sliding bearings (Eq. (34)), thus the total braking force is calculated as Eq. (35).

$$0.1 \times (5 \times 220 + 82 \times 18.24) \times 100\% = 259.57\text{kN} \quad (33)$$

$$0.1 \times (5 \times 220 + 82 \times 11.76) \times 50\% = 103.22\text{kN} \quad (34)$$

$$P_x = 259.57 + 103.22 = 362.79\text{kN} > 342.12\text{kN} \quad (35)$$

(d) Train braking force P_x under empty load for the condition of double holes is composed of two parts, one is the braking force transmitted through fixed supports (Eq. (36)), another one is the braking force transmitted through sliding bearings (Eq. (37)), thus the total braking force is calculated as Eq. (38).

$$0.1 \times 10 \times 32.73 = 32.73\text{kN} \quad (36)$$

$$0.1 \times 10 \times 24.74 \times 50\% = 12.37\text{kN} \quad (37)$$

$$P_x = 32.73 + 12.37 = 45.10\text{kN} \quad (38)$$

②Wind load

The strength of wind load W is set to 0.8kN/m^3 with train condition and 1kN/m^3 without train condition.

(a) The train wind force P_{y1} is calculated as Eq. (39), in which the wind force is slightly higher based on the calculation of trains with two full holes due to the small value of x, so it is calculated based on the left hole, and the train height is known to be 3m. In addition, the force arm from the point of action of the train wind force to the pier top is calculated as Eq. (40).

$$P_{y1} = 3 \times 32.73 \times W = 3 \times 32.73 \times 0.8 = 78.55\text{kN} \quad (39)$$

$$C_y = 2 + 0.15 + 3.1 + 2 = 7.25\text{m} \quad (40)$$

(b) The wind force on the beam is calculated as Eq. (41) with train condition, and Eq. (42) without train condition. In addition, the force arm from the point of action to the pier top is calculated as Eq. (43).

$$P_{y2} = (2.7 + 0.15) \times 32.73 \times 0.8 = 74.62\text{kN} \quad (41)$$

$$P_{y2}^1 = (2.7 + 0.15) \times 32.73 \times 1.0 = 93.28\text{kN} \quad (42)$$

$$C_y = \frac{1}{2} \times 2.85 + 0.4 + 2 = 3.83\text{m} \quad (43)$$

(c) The longitudinal wind force on the top cap is calculated as Eq. (44), while the transverse wind force on the top cap is calculated as Eq. (45) with train condition and Eq. (46) without train condition. In addition, the force arm from the point of action to the bottom section of the tray is calculated as Eq. (47).

$$P_{x1} = 6 \times 0.5 \times 0.8 = 2.40\text{kN} \quad (44)$$

$$P_{y3} = 2.7 \times 0.5 \times 0.8 = 1.1\text{kN} \quad (45)$$

$$P_y^1 = 2.7 \times 0.5 \times 1.0 = 1.4\text{kN} \quad (46)$$

$$C_{x1} = \frac{1}{2} \times 0.5 + 1.5 = 1.75\text{m} \quad (47)$$

(d) The longitudinal wind force on the tray is calculated as Eq. (48), while the transverse wind force on the tray is calculated as Eq. (49) with train condition and Eq. (50) without train condition. In addition, the force arm from the point of action to the

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bottom section of the tray is calculated as Eq. (51).

$$P_{x2}=\frac{1}{2}\times(5.6+3.2)\times1.5\times0.8=5.28\text{kN}$$
(48)

$$P_{y4}=2.3\times1.5\times0.8=2.8\text{kN} \quad (49)$$

$$P_{y4}^{1}=2.3\times1.5\times1.0=3.5\text{kN} \quad (50)$$

$$C_{y4}=\frac{1}{2}\times1.5=0.75\text{m} \quad (51)$$

(e) The longitudinal wind force on pier shaft is calculated as Eq. (52), in which dis expressed as Eq. (53), and the distance from wind action point to check section is calculated as Eq. (54), thus the wind bending moment of the pier shaft at the check section is calculated as Eq. (55). By contrast, the transverse wind force on pier shaft is calculated as Eq. (56), in which bis expressed as Eq. (57), and the distance from wind action point to check section is calculated as Eq. (58), thus the wind bending moment of the pier shaft at the check section is calculated as Eq. (59). The values of abovementioned valuables are determined according to the value of h, as displayed in

Table 2 and

Table 3.

$$P_{x5}=\frac{1}{2}\times(3.6+d)hW \quad (52)$$

$$d=3.6+0.0476h \quad (53)$$

$$C=\frac{h}{3}\times\frac{7.2+d}{3.6+d} \quad (54)$$

$$M_y=P_{x5}\times C=\frac{1}{6}\times(7.2+d)h^2W \quad (55)$$

$$P_{y5}=\frac{1}{2}\times(2.3+b)bW \quad (56)$$

$$b=2.3+2\times\frac{h}{59}=2.3+0.0339h \quad (57)$$

$$C=\frac{h}{3}\times\frac{4.6+b}{2.3+b} \quad (58)$$

$$M_x=P_{y5}\times C=\frac{1}{6}\times(4.6+b)h^2W \quad (59)$$

Table 2 Calculation of longitudinal wind force on pier shaft

Height of pier h (m)	d (m)	P_{x5} (kN)	M_y (kN·m)
2	3.6952	5.84	5.81
3.68	3.7752	10.86	19.82
6	3.8856	17.97	53.21
9	4.0284	27.46	121.27
12	4.1712	37.3	218.33
14	4.2664	44.05	299.66

Table 3 Calculation of transverse wind force on pier shaft

Height of pier h (m)	b (m)	P_{x5} (kN)		M_y (kN·m)	
		Train	Without train	Train	Without train
2	2.3678	4.42	5.53	3.72	4.65
3.68	2.4248	4.58	5.73	12.68	15.86
6	2.5034	4.81	6.01	34.1	42.62
9	2.6051	5.11	6.39	77.82	97.27
12	2.7068	5.42	6.78	140.29	175.36
14	2.7746	5.63	7.04	192.72	240.9

③Transverse sway load of train

The transverse sway loads of light train regarding single hole, heavy train regarding single hole and heavy train regarding double holes are calculated as Eq. (60), Eq. (61) and Eq. (62) respectively, and the transverse sway load is not considered for the condition of empty load regarding double holes.

$$P_y=32.73\times5.5\times\frac{1}{2}=90<195.9+78.55=274.45\text{kN}$$
(60)

$$P_y = 32.73 \times 5.5 \times \frac{1}{2} = 90 < 244.06 + 78.55 = 322.61\text{kN}$$
(61)

$$P_y = \frac{1}{2} \times (32.73 \times 5.5 + 24.74 \times 5.5) = 158.04 < 78.55 + 3 \times 24.74 \times 0.8 + 340.55 = 478.48\text{kN}$$
(62)

It should be noted that the sway force is smaller than the sum of centrifugal force and wind force, therefore, the transverse sway load of train is not considered in this paper.

④ Water load
The hydrogeological conditions in the bridge site area are simple, and both surface water and groundwater are not well developed, so the water load is not considered in this paper.

⑤ Fill load of highway subgrade above the foundation

The average height of the fill is 7.227m, and the relevant physical and mechanical parameters are determined by indoor test, as shown in

Table 4.

Table 4 Physical and mechanical parameters of fill

Cohesive force c (kPa)	Friction angelφ	Weightγ (kN/m^3)
10	10	18.5

Take the soil within 45°+φ/2 above both ends of the fill foundation as the scope of influence, as shown in Fig. 5. Therefore, the volume of soil on the left side of the pier centerline (including the pier part) is calculated as Eq. (63), and the volume of soil on the right side of the pier centerline (including the pier part) is calculated as Eq. (64), as a result, the total volume is calculated as Eq. (65).

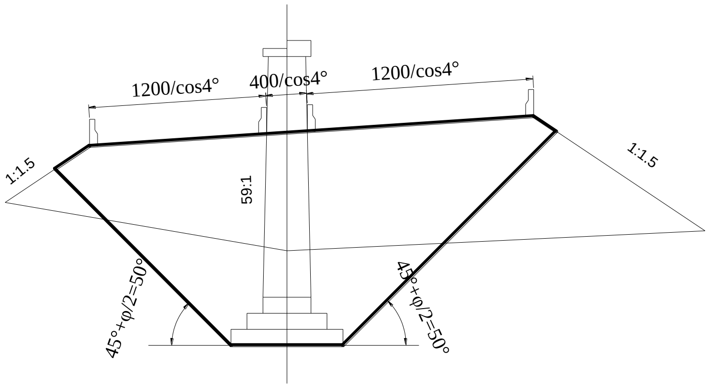

Fig. 5 Diagram of calculation range of overlying fill

$$V_L = [14.79 \times 14.34 - \frac{1}{2} \times 11.57 \times 13.79 - \frac{1}{2} \times 0.82 \times 0.55 - \frac{1}{2} \times 14 \times \tan 4^\circ \times 14] \times 8.67 = 1085.79\text{m}^3 \quad (63)$$

$$V_{右} = \left[14 \tan 4^\circ \times 14 + \left(14 \tan 4^\circ + 16.14\right) \times 1.45 + \left(3.22 + 16.14\right) \times 10.94\right] \times \frac{1}{2} \times 8.67 = 1085.14\text{m}^3 \quad (64)$$

$$V_t = V_L + V_R = 1085.74 + 1085.14 = 2170.88\text{m}^3 \quad (65)$$

Taking the fill and foundation into consideration, the volume V_f of fill is the remaining part of soil after eliminating the volume of pier and foundation, expressed as Eq. (66), in which V_P is the volume of pier within the soil, V_{fd} is the volume of foundation.

$$V_f = V_t - V_p - V_{fd} = 2170.88 - 108.07 - 100.44 = 1962.37\text{m}^3$$

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(66)
According to the design data of the railway bridge, the soil situation of the overburden fill on the foundation is shown in Table 5. The terms numbered 1, 2, and 3 are the different soil layers of the overburden fill from the surface layer to the base layer of the foundation.

Table 5Physical and mechanical parameters of overburden fill on the foundation

No	Soil layer	Thickness of soil layer (m)	Weightγ (kN/m^3)	Friction angelφ	Basic bearing capacity (kPa)	σ_0	Friction coefficient f (kN/m^2)	Foundation coefficientC_0
1	sand clay	1.5m	19.2	17º	150		3.0	8000
2	Sandstone intercalated with mudstone	3.89m	17.8	26º	350		3.5	10000
3	Sandstone intercalated with mudstone	1m	20	33º	500		4.0	15000

According to the different stratum conditions of the fill, the weight of the overlying fill on the foundation is calculated, and its unit weight is taken as the weighted unit weight of each different soil layer, which is calculated as Eq. (67). Furthermore, the weight of the fill can be determined by Eq. (68).

$$\gamma = \frac{18.5\times 7.227+(19.2-10)\times 1.5+(17.8-10)\times 3.89+(20-10)\times 1.2}{7.227+1.5+3.89+1.0} = 13.96\text{kN/m}^3 \quad (67)$$

$$N_{填土} = V_{填土}\times\gamma_{加权} = 1962.37\times 13.8 = 27080.71\text{kN} \quad (68)$$

⑥Calculation of Highway Traffic Load

According to the standard axle load BZZ-100 specified in the current pavement design specifications in China, the wheel load P is set to 25kN, the corresponding pressure p is set to 700kPa. For the double wheel set axles, the equivalent diameter d of double circular load is derived by Eq. (69). Furthermore, the corresponding equivalent lane uniform load is determined according to load equivalent conversion principle (Fig. 6), expressed as Eq. (70).

$$d = \sqrt{\frac{4P}{\pi p}} = 0.213\text{m} \quad (69)$$

$$q = \frac{\frac{1}{4}\times\pi\times 0.213^2\times 700\times 2\times 2\times 2}{\left(12/\cos 4^{\circ}\right)\times 2\times 1} = 8.3\text{kN/m}^2$$

(70)

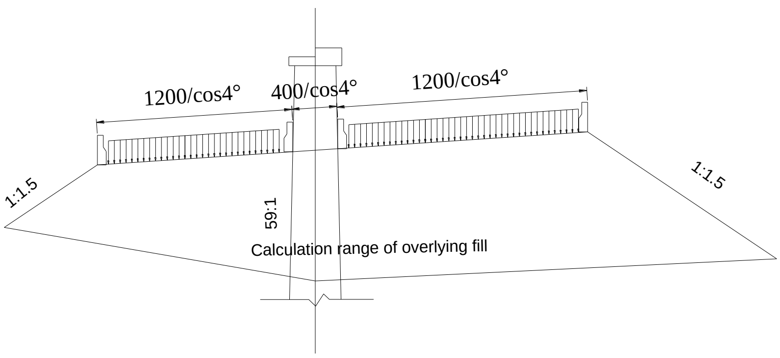

Fig. 6 Diagram of equivalent uniform load of expressway traffic load

The vertical pressure σ_z of the linearly distributed uniformly distributed load on the foundation is calculated as Eq. (71), in which θ_1, θ_2 are the included angle between the vertical line passing through the N point and the connecting line from the N point to the points on both sides of the load.

$$\sigma_z = \frac{2q}{\pi}\int_{\theta_1}^{\theta_2} \cos^2\theta d\theta = \frac{q}{\pi}\left[\sin(\theta_2-\theta_1)\cdot\cos(\theta_2+\theta_1)+(\theta_2-\theta_1)\right] \quad (71)$$

By Analyzing the Eq. (71), the distribution of vertical pressure is approximately parabolic, as shown in Fig. 7. In addition, it is further simplified to the form of trapezoidal linear distribution, as shown in Fig. 8. As a result, the equivalent concentrated force transmitted by highway traffic loads to the foundation can be obtained by Eq. (72).

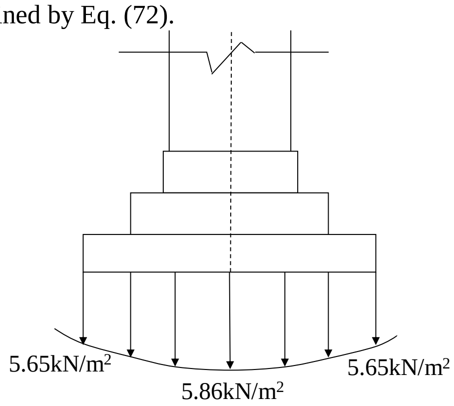

Fig. 7 Foundation pressure caused by highway traffic load

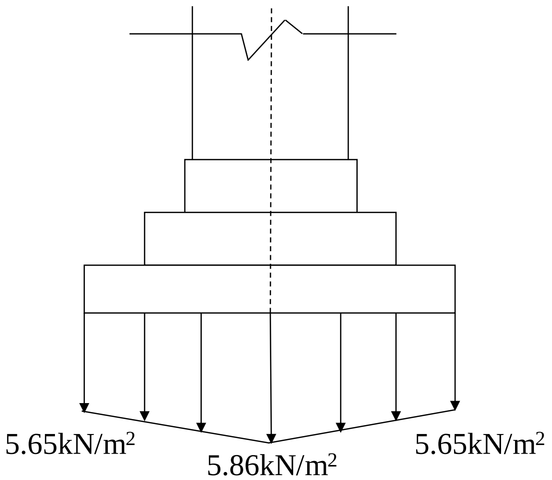

Fig. 8 Equivalent trapezoidal distributed load of highway traffic load

$$P_q = \frac{\frac{1}{2}(5.65+5.86)\times 3.22\times 2}{6.44}\times 6.44\times 8.67 = 321.61\text{kN} \quad (72)$$

2.3 Safety verification

2.3.1 Checking calculation of foundation overturning stability

The earth pressure on the bridge pier is shown in Fig. 9. It can be seen that the left side of the bridge pier bears a uniformly distributed load of 90.27kN/m^2, which consists of two parts, one is the uniformly distributed pressure (1.15kN/m^2) transmitted by highway traffic load, another one is the uniformly distributed pressure (89.12kN/m^2) transmitted by the volume and weight of the fill within the 45°+φ/2 extension line above the left end of the foundation.

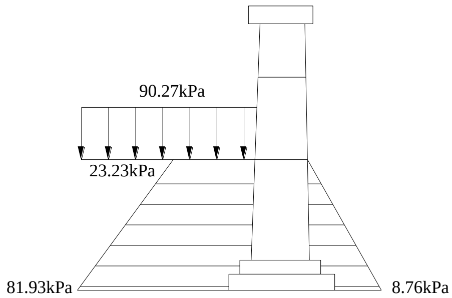

Fig. 9 Diagram of earth pressure action on bridge piers

The active earth pressure at the left fill surface is calculated by using Eq. (73), in which γ=13.8kN/m^3, φ=20° and c=15kPa. The active earth pressures at the left and right base are calculated by using Eq. (74) and Eq. (75) respectively.

$$P_{a0} = (\gamma z+q)\times\tan^2\left(45^\circ-\frac{\phi}{2}\right)-2c\times\tan\left(45^\circ-\frac{\phi}{2}\right)$$
$$=(90.27+13.8\times 0)\times\tan^2 35^\circ-2\times 15\times\tan 35^\circ = 23.25\text{kN/m}^3 \quad (73)$$

$$P_{a1} = (\gamma_1 z+q)\times\tan^2\left(45^\circ-\frac{\phi}{2}\right)-2c\times\tan\left(45^\circ-\frac{\phi}{2}\right)$$
$$=(90.27+13.8\times 8.68)\times\tan^2 35^\circ-2\times 15\times\tan 35^\circ = 81.93\text{kN/m}^3 \quad (74)$$

$$P_{a0} = (\gamma_1 z+q)\times\tan^2\left(45^\circ-\frac{\phi}{2}\right)-2c\times\tan\left(45^\circ-\frac{\phi}{2}\right)$$
$$=(0+8.47\times 7.17)\times\tan^2 35^\circ-2\times 15\times\tan 35^\circ = 8.76\text{kN/m}^3 \quad (75)$$

By combining the Eq. (73), Eq. (74) and Eq. (75), the overturning stability coefficient can be calculated by Eq. (76).

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$$K_q = \frac{\text{Stabilizing moment}}{\text{Overturning moment}}$$

$$= \frac{2468.14 \times 2.94}{2468.14 \times 0.35 + \frac{1}{3} \times 8.68 \times \frac{1}{2} \times (58.7 - 7.17) \times 8.68 + 23.23 \times 8.68 \times \frac{8.68}{2}} = 2.96 > 1.5 \quad (76)$$

According to Eq. (76), the calculated overturning stability coefficient is larger the safety coefficient, which indicates that the foundation of the bridge pier meets the requirements for foundation overturning stability.

2.3.2 Checking calculation of foundation bearing capacity

According to the design data, the pier is a three-layer expanded foundation located in sandstone intercalated with mudstone. The initial bearing

Table 5, the maximum and minimum values of foundation reaction under unfavorable loads (combination of main load and longitudinal additional load) for the condition of heavy load regarding double holes can be obtained (σ_{max}=766.75kPa, σ_{min}=638.63kPa). In addition, the maximum and minimum values of foundation reaction under unfavorable loads (combination of main load and transverse additional load) for the condition of heavy load regarding double holes can be obtained (σ_{max}=754.53kPa, σ_{min}=650.69kPa). It can be seen that the calculated results are less than the allowable foundation bearing capacity [σ].

II. CONCLUSION

In this paper, a set of methods for checking the bearing capacity and anti-overturning stability of bridge foundations is proposed. Taking a highway crossing an existing railway as an example, the problems of foundation bearing capacity and overturning stability under uneven earth pressure are analyzed. After theoretical calculation, it is found that they meet the requirements for foundation bearing capacity and foundation overturning stability, and the feasibility of highway crossing an existing railway bridge is verified. The results of this study can provide technical support for the safety of train and highway operations.

capacity σ_0 of the foundation at the base is 500 kPa, and groundwater is abundant.

Due to the fact that when the foundation width exceeds 2m, the buried depth h of the foundation bottom surface exceeds 3m, whenh/b≤4, the allowable bearing capacity of the foundation needs to be considered for width and height correction, which can be calculated by Eq. (77), in which σ_0=500kPa, k_1=4, k_2=10, γ_1=10kN/m^3,γ_2=8.47kN/m^3.

$$[\sigma] = \sigma_0 + k_1\gamma_1(b-2) + k_2\gamma_2(h-3)$$
$$= 500 + 4 \times 10 \times (6.44 - 2) + 10 \times 8.47 \times (6.39 - 3) = 964.73\text{kN/m}^2 \quad (77)$$

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According to the geological data presented in

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