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A case study on piping during excavation for bridge abutments

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SYNOPSIS: This paper, first, reports a case history of piping occurred in a subsoil of a bridge abutment within a cofferdam during dewatering. The countermeasures against the piping are also presented. Second, seepage flow analyses are carried out and stability with respect to seepage failure (i.e., boiling) is investigated. Finally, the causes of the piping are considered, and the following results were then obtained:

At this location, a gravel layer is overlaid with a sandy silt layer which has less permeability. In the case of layered soil of this type, head losses take place dominantly in the upper layer, as a result the large seepage force acts upward on soil particles. Seepage flow concentrated into the toe of the abutment footing through its construction on the surface of excavation, the seepage velocity became larger and the piping occurred.

1 INTRODUCTION

Piping occurred in a subsoil of a bridge abutment of the Isasa river within a cofferdam during dewatering. This paper reports the case history and the analytical consideration of the problem.

At this location, a gravel layer is overlaid with a sandy silt layer which has less permeability. It is thought that seepage flow concentrated partially through the abutment construction, and forced soil particles in the upper layer to move, i.e., the piping occurred. The piping caused settlement of the abutment about 10cm in maximum. A grouting method was adopted to prevent the piping in the ground under the bridge abutment.

FEM seepage flow analyses are carried out for the conditions before and after the construction of the bridge abutment. The stability against boiling for the soil located within the cofferdam was investigated precisely.

2 CONSTRUCTION OF BRIDGE ABUTMENTS

2.1 Site conditions and bridge abutment

The Isasa river road bridge is located on the alluvial plain in the southern part of Lake Biwa in Siga Prefecture, Japan. The road is two-lane on both sides and 20m wide. The Isasa river is 10m wide around the site. The bridge abutments are cantilever reversed-T-shaped reinforced concrete structures, are located on both sides and are separated into two parts from the center of the road, i.e. upstream and downstream parts 10.40 $\sim$ 13.46m long. The excavation of the ground and the construction of abutments were performed from the left hand side. Fig.1 shows the plane of the site. Fig.1 indicates the both sides at the same time, but the construction on the left side preceded the right side as stated above.

Fig.2 shows the soil profile at the point Bo.1 (Fig.1). It was expected from the boring log (Fig.2) that a gravel layer comes out near the base of the bridge abutment, and the subsoil of the abutment consists of an approximately uniform gravel. A cantilever sheet pile wall 9.8m long (called III-type in Japan) was adopted as an earth retaining structure. The safety of the expected bottom of excavation was checked with respect to boiling beforehand for the above conditions and the factor of safety against boiling F_s was estimated as F_s=4.48(>1.5).

2.2 Left bank bridge abutments

The left side ground was first excavated using the sumping method. The well compacted gravel layer came out at the bottom level of the abutment in the upstream part, but a sandy silt layer came out in the downstream part unexpectedly. The sandy silt is thought to be the same soil as the upper one above the gravel layer judging from the succession of soil layers (Fig.2). The N-value of this soil is about 14, which is the allowable lowest value for spread foundation. The depth of the soil is about 1.505m under the bottom level of the abutment (T.P. 87.235m), where T.P. represents the Tokyo Bay mean sea level called Tokyo Peil. The left bank abutments were constructed and the revetment work was continued within the cofferdam during dewatering.

The 4cm settlement of the left bank downstream abutment was observed at a week after the placement of the stem concrete of the abutment. Fig.3 shows the progress of the settlement of the abutment. It is found from Fig.3 that the abutment is subsiding about 1.0cm per day. A site investigation revealed that fine soil particles were being washed out by seepage water from the foundation ground under the left bank downstream abutment. The settlement of the abutment was, therefore, judged

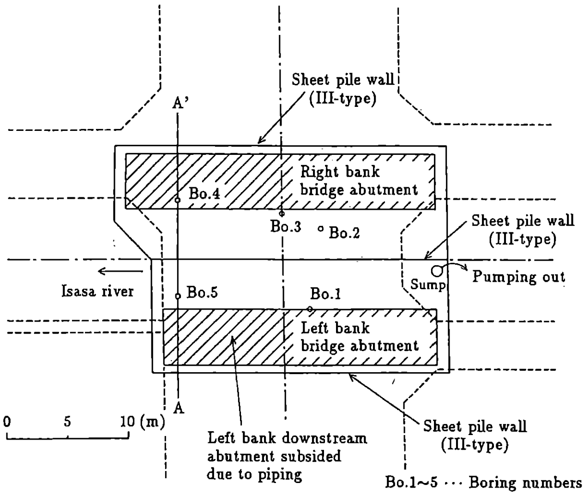


Fig.1 Plane of the site.

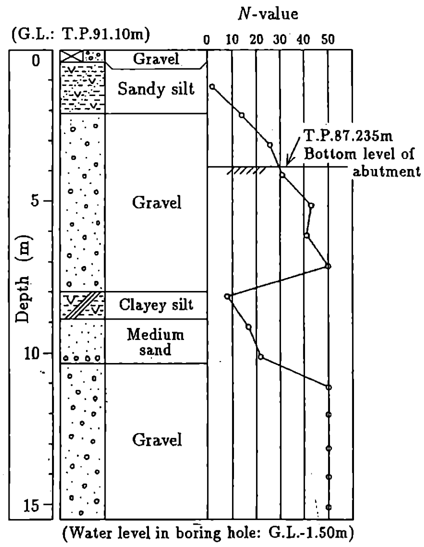


Fig.2 Boring log (Bo.1).

to be due to piping. Although the revetment work had not finished yet, the dewatering was stopped. The water level within the cofferdam came up naturally and reached the level of T.P. 88.900m after one day. As a result, the settlement of the abutment ceased (see Fig.3).

A grouting method was adopted to prevent the piping in the foundation ground under the bridge abutment, to harden and reinforce the subsoil. The grout is made up of high-early-strength-portland cement and water glass. The piping did not occur and the left bank downstream abutment did not subside during the successive excavation with dewatering for the resumption of the revetment work.

2.3 Right bank bridge abutments

In the case of the construction of the right bank bridge abutments, the well compacted gravel came out at the bottom level of the abutments as expected from boring results, and there were no settlements of the abutments.

Thus the geological features at the site are thought to be changed unexpectedly. This example indicates the difficulties of the geological survey.

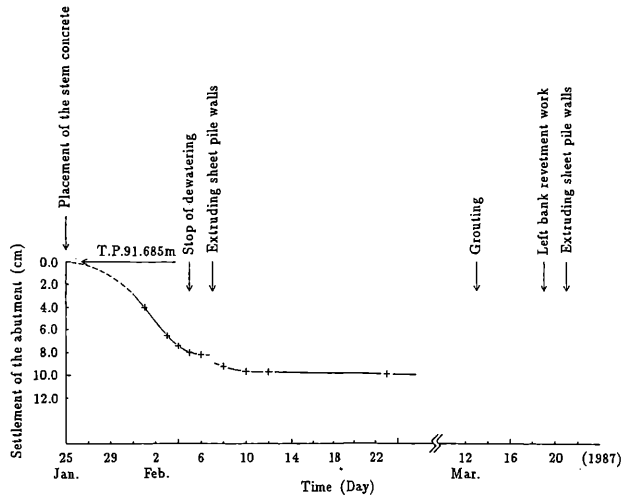


Fig.3 Observed settlement of the left bank downstream abutment.

2.4 Set up of Bridge Girder

The bridge girder was set up on the abutments. The settlement of the left bank downstream abutment was about 1.8cm two months after grouting, extruding sheet pile walls and setting up the bridge girder. No settlement is observed thereafter and the grouting method is, therefore, proved to be effected successfully.

3 MODELING OF THE FOUNDATION GROUND SUBJECTED TO PIPING

The substratum geology of the foundation ground subjected to piping is modeled as shown in Fig.4. The soil layers are thought to be inclined, but the details are not clear in the absence of enough boring data. Hence we consider the horizontal layers of sandy silt and gravel downward (overlaid with fillbank and thin clay layer).

The ground water level is relatively high from the boring logs, i.e., 0.14 $\sim$ 1.52m under the river water level. Here we assume that the ground water level is the same as the river water level (T.P. 89.460m), and is fixed.

Table 1 shows the physical properties of soils.

Table 1 Physical properties of soils.

	1. Gravel (Lower layer)	2. Sandy silt (Upper layer)
Specific gravity of soil particles G_s	2.657	2.659
Uniformity coefficient U_c	21.84	7.02
50 percent grain size D_{50} (mm)	3.87	0.219
Maximum void ratio e_{max}	–	1.093
Minimum void ratio e_{min}	–	0.561
Coefficient of permeability at 15°C k_{15} (cm/s)	2.82×10^{-2}	0.463×10^{-2}
Buoyant unit weight γ' (gf/cm^3)	1.171	0.935*

* Value at D_r=60%.

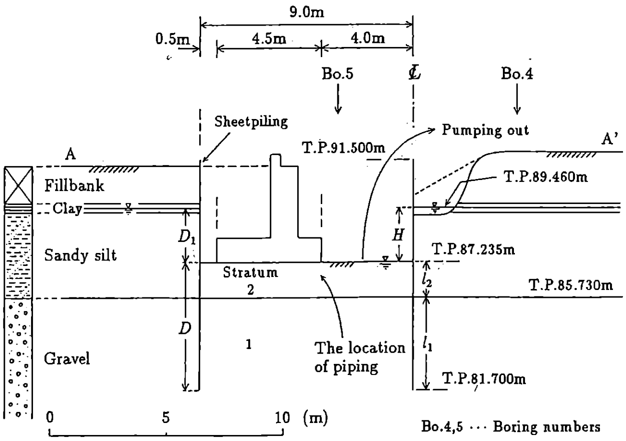


Fig.4 Modeling of the foundation ground subjected to piping (Left bank downstream abutment).

4 ANALYSIS AND STABILITY OF THE FOUNDATION GROUND

4.1 Seepage analysis for the condition before the abutment construction

FEM seepage flow analysis was carried out for the condition before the abutment construction. Figs.5(a) and (b) show the computed velocity distribution and flow net. The pattern of the computed velocity (Fig.5(a)) and the shape of the flow net (Fig.5(b)) show that the soil located within the cofferdam is approximately in the state of one-dimensionally upward seepage flow.

4.2 Stability against boiling for the bottom of excavation before the abutment construction

Homogeneity assumption

Let us now assume the soil surrounded by the cofferdam, which is actually in the two-layer case, to be one-layered homogeneous soil. The assumed one-layered soil has the safety factor F_s against boiling (Highway Earthwork Series, 1987):

$$F_s = \frac{2D\gamma'}{H\gamma_w}, \tag{1}$$

where D is the penetration depth of sheet pile wall from the bottom of excavation, D_1 is the length from the upstream water level to the excavation bottom, H is the difference between the upstream and downstream water levels, γ' is the buoyant unit weight of soil, and γ_w is the unit weight of water.

The buoyant unit weight of soil γ' is here taken as the average value of the two layers as follows:

$$\gamma' = \gamma'_{av} = \frac{\gamma'_1 l_1 + \gamma'_2 l_2}{l_1 + l_2}. \tag{2}$$

Since we have $\gamma'=\gamma'_{av}=1.107\text{gf/cm}^3$ from Eq.2, Eq.1 becomes

$$F_s = 5.51, \tag{3}$$

where we use the values: D_1=2.225m, D=5.535m, H=2.225m, l_1=4.030m and l_2=1.505m. Thus the bottom of excavation is judged to be quite stable against boiling. The soil within the cofferdam was actually thought to be near in the state of quicksand, the homogeneity assumption is not correct in this case.

Kälin (1977) concluded that in the seepage failure problem of layered soil there will be a case where the homogeneity assumption may be unreasonable from the standpoint of the critical hydraulic head. Speaking of the one-dimensional problem, the homogeneity assumption may become unreasonable and on an unsafe side when the upper stratum is more permeable than the lower one (Tanaka et al., 1981). The soil within the cofferdam treated here is just the case and should be analysed precisely taking account of the constituent layers.

Two-layer case

We can assume that the soil located within the cofferdam is in the state of one-dimensionally upward seepage flow as stated above. Hence we consider that the soil is subjected to one-dimensionally upward seepage flow and is in the state of one-dimensional stress.

Let us now consider a piece-wise homogeneous soil under a vertically upward seepage flow and in one-dimensional state of stress. The layers of the n-layered soil are named Stratum 1, Stratum 2, $\cdots$, Stratum n upward from the bottom layer, and k_j, l_j and γ'_j are the coefficient of permeability, thickness and buoyant unit weight of the Stratum j. The factor of safety F_s against seepage failure for the multi-layered soil is defined as follows (Tanaka, 1988):

$$F_s = \min\left\{\frac{h_c{}^{(m)}}{h};\quad m=1,2,\cdots,n\right\}, \tag{4}$$

where $\min\{h_c{}^{(m)}/h;\ \ m=1,2,\cdots,n\}$ means the minimum value of $h_c{}^{(m)}/h$ for m=1,2, $\cdots$, n, and h is the excess pore water pressure (expressed in water head). $h_c{}^{(m)}$ is the critical hydraulic head at a time when a multi-layered soil becomes critical imaginarily at the bottom of the Stratum m, and is then expressed as:

$$h_c{}^{(m)} = \left(p + \sum_{j=m}^{n} \gamma'_j l_j\right) \frac{\sum_{j=1}^{n} \frac{l_j}{k_j}}{\sum_{j=m}^{n} \frac{l_j}{k_j}\gamma_w}, \tag{5}$$

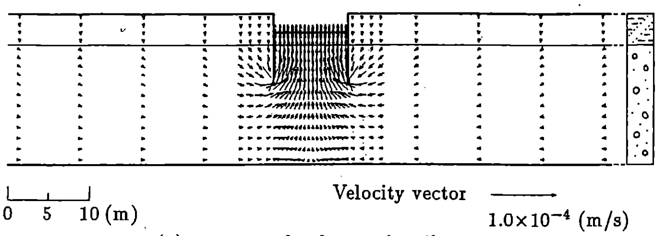
where p is the effective overburden pressure of a loaded filter. In our case of p=0 and m=1, 2, the values of $h_c{}^{(m)}$ are calculated from Eq.5 as follows: $h_c{}^{(1)}$=6.126m and $h_c{}^{(2)}$=2.026m. We can then obtain, from Eq.4,

$$F_s = 1.50, \tag{6}$$

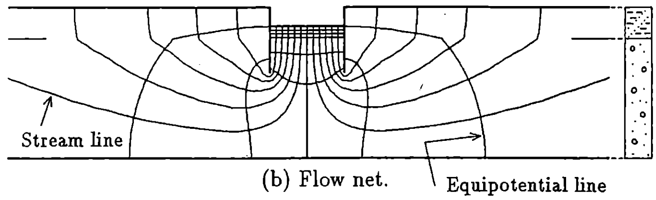
where we use the value h=1.355m obtained from FEM seepage flow analysis. This indicates that the bottom of excavation was near in the critical state for quicksand.

Hydraulic gradient in the upper layer

According to the FEM computation, the soil surrounded by the cofferdam is approximately in the state of one-dimensionally upward seepage flow. The hydraulic gradients calculated in the upper layer are directed almost

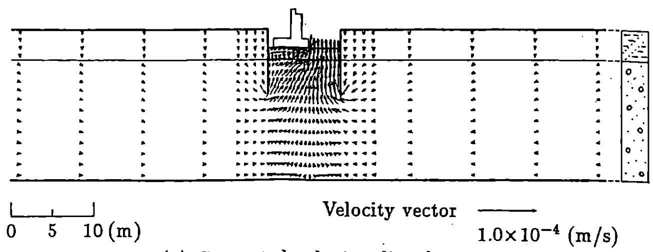


(a) Computed velocity distribution.

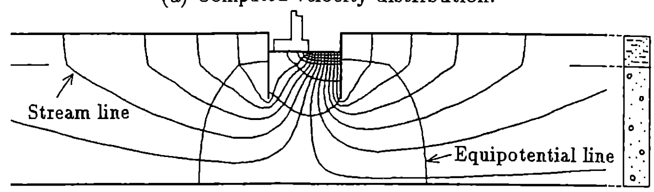


(b) Flow net.

Fig.5 Seepage flow before the abutment construction.



(a) Computed velocity distribution.



(b) Flow net.

Fig.6 Seepage flow after the abutment construction.

vertically upward, and the vertical components i_z are $0.618 \sim 0.626$. The factor of safety against quicksand F_s is defined as the ratio of the critical hydraulic gradient i_c to i_z (Terzaghi, 1922) and becomes, for our case,

$$F_s = \frac{i_c}{i_z} = 1.51 \sim 1.49\,, \tag{7}$$

which indicates that the upper layer of soil within the cofferdam was near in the quick state. This is in fact the same conclusion as stated in the two-layer case above.

4.3 Seepage flow in the foundation ground after the abutment construction

FEM seepage flow anlysis was carried out for the condition after the abutment construction. Figs.6(a) and (b) show the computed velocity distribution and flow net. The pattern of the seepage velocity (Fig.8) indicates that the velocity vectors are directed toward the toe of the abutment footing and their magnitudes become larger in the vicinity of the toe. It is then found from Fig.6(b) that the stream lines concentrate into the toe of the abutment footing and the flow net becomes complicated near the toe. Thus it is thought that the seepage flow concentrated into the toe of the abutment footing and the magnitude of the velocity became larger.

The vertical component of hydraulic gradient i_z near the abutment toe is 1.128 in maximum, which is beyond the theoretical critical one i_c (=0.935) of this soil. It is concluded that the foundation ground near the toe boiled first and piping was progressing into the subsoil.

The hydraulic head difference H is estimated larger than it really is, and the actual hydraulic gradient i_z near the toe is thought to be somewhat smaller than the computed one.

5 CONCLUSIONS

The piping occurred in a foundation ground of a bridge abutment within a cofferdam during dewatering. At this location, a gravel layer is overlaid with a sandy silt layer with less permeability. FEM seepage flow analyses were carried out for the conditions before and after the abutment construction. The stability against boiling for the foundation ground was investigated precisely, and the following results were then obtained:

(1) The bottom of excavation was near in the critical state for quicksand when the ground was excavated at the bottom level of the bridge abutment. An upward seepage flow concentrated near the toe of the abutment footing as a result of the abutment construction, the seepage velocity became larger and the piping occurred.

(2) The stability against boiling for the soil within a cofferdam should be checked for the conditions not only before but after the construction of a structure on the surface of excavation.

(3) In the seepage failure problem of a layered soil of this type, it is thought that the homogeneity assumption may be unreasonable and on an unsafe side from the standpoint of the critical hydraulic head. The problem presented here is just the case and should be analysed precisely taking account of the constituent layers.

(4) In the case where the soil behind sheet pile walls is in the state of one-dimensional seepage flow and one-dimensional state of stress, the theory of seepage failure of a multi-layered soil is available for calculating exactly the stability of the soil with respect to seepage failure.

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