Interdependence between Structural Engineering and Construction Management
Edited by Ozevin, D., Ataei, H., Modares, M., Gurgun, A., Yazdani, S., and Singh, A.

ISBN: 978-0-9960437-6-2

INVESTIGATION ON THE EFFECT OF COUNTERMEASURES FOR SUBSIDENCE AT THE APPROACHING AREAS OF ABUTMENTS

YUKIHIDE KAJITA[1], KAZUKI ONODA[1], TAIJI MATSUDA[1], KUNIHIKO UNO[2], and TAKESHI KITAHARA[3]

[1]*Dept of Civil and Structural Engineering, Kyushu University, Fukuoka, Japan*
[2]*Institute of Technology, Penta-Ocean Construction Co. Ltd., Tochigi, Japan*
[3]*Dept of Science and Engineering, Kanto Gakuin University, Yokohama, Japan*

The purpose of this study is to confirm the effect of soil improvement methods on preventing ground subsidence at the back of abutments. Earthquake seismic analysis is performed for three models. One is a model with no ground improvement. Next is a model with deep mixing method. The third is the model with lightweight banking method. As a result, from the perspective of the passage possibility of the emergency vehicles, both the deep mixing method and lightweight banking method are effective in preventing ground subsidence at the approaching area of abutments. However, in the case of the deep mixing method, it is found that the maximum bending moment of the pile under the liquefaction layer increases because a lump of improved rigid soil that falls down toward the piles.

Keywords: Ground subsidence, Liquefaction, Structure-soil interaction, Deep mixing method, Lightweight banking method, Effective stress analysis.

1 INTRODUCTION

The subsidence of the soil at the approaching area of the abutment and the horizontal movement of the abutment were observed as shown in Figure 1 in 2011 off the Pacific Coast of Tohoku earthquake and the 2016 Kumamoto earthquake. This is because the lateral flow was generated in the soft ground due to the severe ground excitation (Japan Society of Civil Engineers 2017). If the amount of the subsidence was large, the emergency cars would not be able to pass through the bridge just after a severe earthquake. Some types of ground improvement methods have been introduced to improve the ground stiffness hardness or reduce the surcharge load. However, previous studies have not mentioned how the abutment and the back embankment of the abutment behave when an earthquake occurs. The purpose of this study is to confirm the effect of soil improvement methods for prevention of ground subsidence at the back of the abutment. The deep mixing method which improves the liquefaction layer and the lightweight banking method which reduces surcharge load and soil pressure is adopted in this study. In this research, the effect of the deep mixing method and the lightweight banking method for subsidence of soil at the approaching area of the abutment is investigated by performing seismic response analysis.

Figure 1. Ground subsidence at the back of abutment.

2 THE OUTLINE OF TWO-DIMENSIONAL EFFECTIVE STRESS ANALYSIS

2.1 The Modelling of Structures and Ground

In this analysis, FLIP (Finite element analysis program of liquification process/response of soil-structure systems during an earthquake), which is a dynamic effective stress analysis program is used (Morita *et al.* 1997, Iai *et al.* 1992). Figure 2 shows the numerical model in this analysis. The girder (orange line) is connected to the pier rigidly. The clearance between the girder and the abutment is set to be 10 cm. The ground consists of 5 layer types. The physical properties of each layer are shown in Table 1.

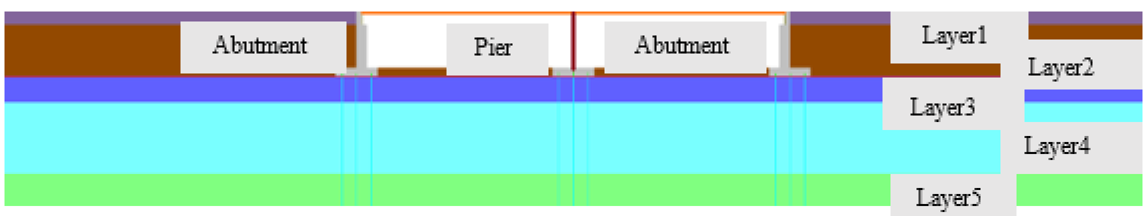


Figure 2. Numerical model in this analysis.

In Table 1, Layer 1 and Layer 2 are the ground at the back of the abutment. Layer 3 is the ground just under the foundation of the abutments and the pier. In this analysis, it is assumed that only Layer 3 is a liquefaction layer. So, the excess pore water pressure ratio at Layer 3 is calculated. The excess pore water pressure ratio has defined the ratio of the excess pore water pressure to the initial effective mean stress. This analysis is conducted under the undrained condition. The moment-curvature relationship of the piles is defined as the bi-linear model as shown in Figure 3. The foundation piles are steel tube piles whose diameter is 800mm, and thickness is 12mm. The foundation piles are arranged three in the bridge-axial direction, four in the orthogonal direction. The foundation piles of the abutment and the ones of the pier is the same material and size. The bottom of the numerical model in Figure 2 is the surface of the engineering base. In this model, the node of the end member for the steel pile is located at the bottom of the numerical model. That means the movement of the end member for steel pile is the same as the movement of the engineering base surface.

Table 1. The physical properties of each layer.

	Thickness of a layer	Average effective constraing pressure	Density	N-value	Internal friction angle
	m	kPa	ton/m^3		degree
layer1	2.1	13.90	1.8	5	38
layer2	7.9	80.12	1.8	10	37
layer3	5	147.15	2	2	30
layer4	10	215.82	2	10	33
layer5	5	353.16	2	20	36

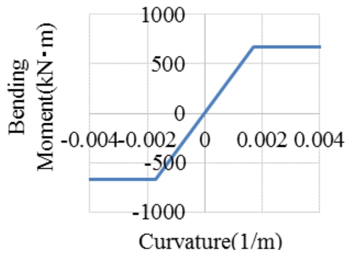


Figure 3. Moment-curvature relationship of piles.

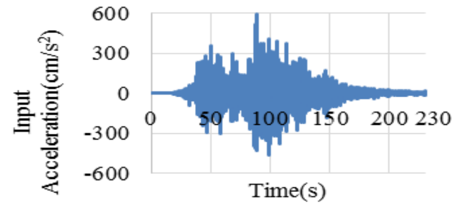


Figure 4. Acceleration at the engineering base.

2.2 Input Earthquake Motion at the Engineering Base

Firstly, the ground surface motion, which is the observation record in 2011 off the Pacific coast of Tohoku Earthquake is prepared. This record is the ground surface acceleration of the east-west direction and this is recorded in Soma city, Fukushima Prefecture. The Observation point is FKS001 by Strong-motion seismograph networks in Japan. The characteristic of this ground surface motion is that the maximum acceleration is over $300cm/s^2$ and both the low and high-frequency components are considerably included. In this analysis, the acceleration at the engineering base is needed. So, the acceleration at the engineering base is calculated by using one-dimensional wave propagation analysis. Figure 4 shows the input wave at the engineering base.

2.3 The Case of Analysis

Two types of ground improvement methods are investigated in this study. One is the deep mixing method and the other is the lightweight banking method. So, in this paper, 3 cases of analytical models are prepared. Case1 is the model with no ground improvement. Case2 is the model with the deep mixing method. Case3 is the model with the lightweight banking method. The Deep mixing method is a ground improving method for solidifying ground by mixing the improved material and the soft soil. By conducting a lightweight banking method, reduction of surcharge load and soil pressure to the back of abutment prospects. Therefore, the reduction of subsidence at the approaching area of the abutment is also prospected by reduction of lateral movement of the abutment.

2.4 The Model of Deep Mixing Method

In this research, the deep mixing method is conducted to liquefaction layer for reducing the subsidence of soil at the approaching area of the abutment. Improving area is determined by the past technical report (Public Works Research Institute 2010). The improving area at the back of the left side abutment is shown in Figure 5. In Figure 5, the green area surrounded by red line is the improving area by deep mixing method. In this study, the improving area is modeled as a solid material like cement mortar. This is because the improving area is solidified. So, the improving area of the deep mixing method is modeled by the liner plane element. The physical property is decided from the result of the past research (Yamamoto *et al.* 2006).

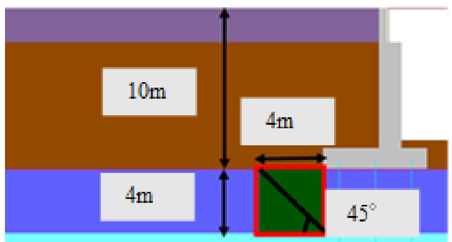


Figure 5. The improving area of deep mixing method.

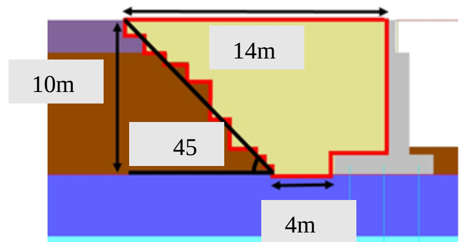


Figure 6. The improving area of lightweight banking method.

2.5 The Model of Lightweight Banking Method

The physical properties of the lightweight banking method are decided from the test data of the private company (Taiheiyo Cement Corporation 2018). The density of the lightweight banking is about 60% of the original ground. The place replaced by lightweight banking is shown in Figure 6. In the case that the lightweight banking method is conducted, the density and the stiffness are changed from the original ground. In this analysis, we assumed that the lightweight soil ground exists from the beginning. In real site situation, when the lightweight banking method is adopted, the original soil ground has already existed, and then we replace the original soil with the lightweight soil. Comparing these two cases, that means the case that the lightweight soil is existed from the beginning and the case that the lightweight soil is replaced the original soil with, the surcharge load at the liquefaction layer might be different. This difference is ignored in this research.

3 ANALYTICAL RESULTS AND DISCUSSIONS

3.1 The Amount of Vertical Gap

The contour figure of the excess pore water pressure ratio of Case 1(no ground improvement) is shown in Figure 7. When the excess pore water pressure ratio is close to 1, the color turns to be from blue to red. Figure 8 shows the time history of the excess pore water pressure ratio in the middle of Layer 3. As shown in Figure 8, the excess pore water pressure ratio is rising sharply from 40 seconds in all cases which is the time that the acceleration of the input wave shown in Figure 4 starts to be large. The excess pore water pressure ratio is closer to 1 at the end of analysis in all case. In other words, the liquefaction occurs in Layer 3. The yellow points in Figure 7 are the node of the top of the abutment and the node of ground which is located at the 4.5m back of the abutment. In this study, the vertical gap is defined as the vertical relative displacement between two points. Figure 9 presents the time history of the vertical gap at the right-side abutment. If the node of back fill soil ground is located below against the node of the abutment, the vertical gap is a positive value. From Figure 8 and Figure 9, when the excess pore pressure ratio becomes 0.8, the amount of vertical gap increases sharply. And it is confirmed that the horizontal displacement of the right abutment also becomes the maximum value. This is because the stiffness of the liquefaction layer is starting to be reduced. Figure 10 shows the vertical gap at the end of the analysis. As shown in Figure 10, the node of backfill soil is located above against the top of the abutment in Case 3 (The lightweight banking method). This is because the surcharge load of liquefaction layer under the replaced area is reduced. The soil just under the abutment foundation might be easy to move to the area just under the replaced area. So, the abutment might sink in contrast with the backfill soil of the abutment. It is reported that the traffic speed of the emergency vehicle is limited when the vertical gap is 10cm and more (Tokida and Oda 2009). Therefore, in this research, 10 cm is defined as the limiting vertical gap. The vertical gap exceeds the limiting vertical gap in Case1, which is the model with no ground improvement. In the other two cases, the vertical gap is smaller than the limiting vertical gap. The absolute value of the vertical gap of the model with deep mixing method (Case 2) is smaller than the absolute value of gap of the model with lightweight banking method (Case 3). In this analysis, the liquefaction is the main factor of the occurrence of the vertical gap. The deep mixing method improves the liquefaction layer directly. Therefore, the deep mixing method has a great effect on preventing ground subsidence at the approaching area of abutments.

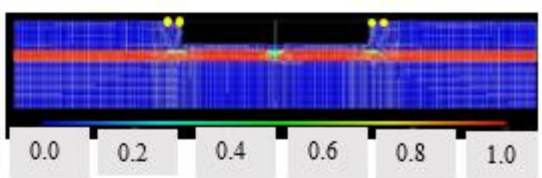


Figure 7. The excess pore water pressure ratio in Case 1 (no ground improvement).

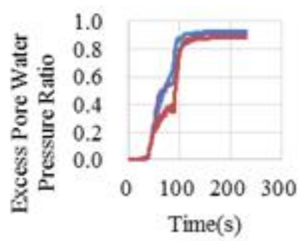


Figure 8. The time history of excess pore water pressure ratio.

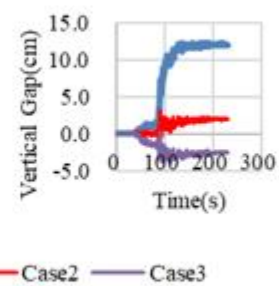
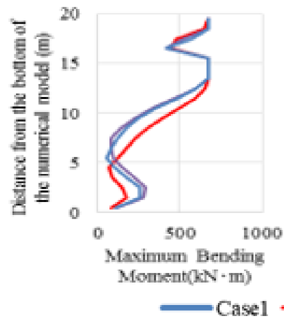


Figure 9. The time history of vertical gap.

3.2 The Response of Piers

Figure 11 and Figure 12 present the maximum bending moment and the maximum horizontal movement of the foundation pile which is located at the center of the right-side abutment. The value of the bending moment and horizontal movement is the absolute value. The yield bending moment is set to be 670 kNm in this model. The bottom surface of the abutment foundation is located at 20m upward from the bottom of this numerical model. And the thickness of the liquefaction layer (Layer 3) is 4m. So, 16m from the bottom of the numerical model indicates the boundary between Layer 3 (Liquefaction layer) and Layer 4 (Non-liquefaction layer). As shown in Figure 11, the bending moment reaches the yield value from 12.5m to 15.5m and from 18.5m to 19.5m from the bottom. However, the bending moment doesn't reach the yield value from 15.5m to 18.5m from the bottom. This decrease in bending moment is caused by liquefaction. From Figure 12, the horizontal displacement is almost the same from the bottom to 16m. The displacement increases sharply from 16m due to the liquefaction layer (Layer 3). The displacement at the top of the pile in Case 2 (the deep mixing method) is smaller than the other two cases. Therefore, it is thought that the deep layer mixing which improves liquefaction layer directly has an effect on the response of the piles. As shown in Figures 11 and 12, the bending moment in Case 2 is larger than the other two cases and the displacement is a little larger under the liquefaction layer. It is confirmed that the improving area moves to diagonally forward downward due to the surcharge load. It is thought that this displacement causes the increasing of the bending moment of the pile.



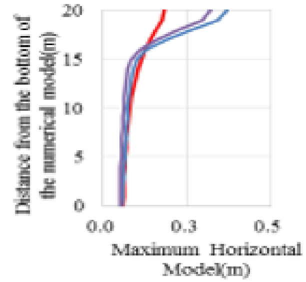


Figure 10. Final vertical gap.

Figure 11. Maximum bending moment.

Figure 12. Maximum horizontal displacement.

4 CONCLUSIONS

In this research, the seismic response analysis which is considered the interaction between ground and bridge is performed for investigating the behavior of abutment and background and confirming the effect of ground improvement.

(i) The vertical gap in no ground improvement model is larger than 10 cm. However, the vertical gap in other cases is smaller than 10 cm. From the perspective of the passage possibility of the emergency vehicles, these two improvement method has the effect for prevention of ground subsidence at the back of the abutment.

(ii) In the deep mixing method model, the bending moment of the pile and the maximum horizontal displacement increases under the liquefaction layer. This is because the improvement area (the rigid solid area) moves to diagonally forward downward and then the improvement area pushes the soil and pile under the liquefaction layer.

(iii) In this research, input earthquake acceleration data is one case. Therefore, the increase of input earthquake data and three-dimensional analysis are future topics of discussion.

Acknowledgments

This work was supported by JSPS KAKENHI Grant Number JP25KO6182.

References

Iai, S., Matsunaga, Y., and Kameoka, T., Analysis of Undrained Cyclic Behavior of Sand under Anisotropic Consolidation, *Soils and Foundations*, 32(2), 16-20, June 1992.

Japan Society of Civil Engineers, *Report on Damage Surveys and Investigations Following the 2016 Kumamoto Earthquake* (in Japanese), Japan, 2017.

Morita, T., Iai, S., Liu, H., Ichii, K., and Sato, Y., Simplified Method to Determine Parameter of FLIP (in Japanese), *Technical Notes of The Port and Harbor Research Institute*, 869, 1-36, 1997.

Public Works Research Institute, Investigation on The Guideline of Countermeasures for Lateral Flow of Abutments (in Japanese), 2, *Technical notes of PWRI 4174*, Japan, 2010.

Taiheiyo Cement Corporation, Artificial Light Weight Filling Material (in Japanese), 2018. Retrieved from http://www.taiheiyo-cement.co.jp/service_product/aggregate/pdf/keiryo-morido2018.pdf.

Tokida, K., and Oda, K., Future Views on Seismic Performance-Based Design Reinforcement and Management of Road Embankment (in Japanese), *Journal of Japan Society of Civil Engineers*, 65, 857-873, November 2009.

Yamamoto, Y., Takahashi, N., and Kurokawa, Y., Analytical Study on Liquefaction Countermeasure Effect of the Pile Type Deep Mixing Method (in Japanese), *Technical Report of The Research Institute of Mitsui-Sumitomo Construction*, 45-53, Japan, 2006.