Seismic assessment of Nagoya Port Island against Nankai Trough earthquake

Takayuki Sakai i) and Kentaro Nakai ii)

i) Researcher, Disaster Mitigation Research Center, Nagoya University, Furo, Chikusa, Nagoya, 464-8603, Japan.
ii) Associate professor, Department of Civil Engineering, Nagoya University, Furo, Chikusa, Nagoya, 464-8603, Japan.

ABSTRACT

In order to maintain the function of Nagoya Port, 1.3 million m$^3$ soil must be dredged every year which is currently filled at Nagoya Port Island (hereinafter, PI) as a temporary disposal area. However, the initial envisioned capacity of the PIs to accept dredged soil has already been exceeded, even though the temporary banks built on top of the reclaimed ground have increased their capacities. Therefore, should a megathrust earthquake occur, it is likely that, in addition to damaging the revetment and temporary banks, the earthquake would cause large volumes of high-piled dredged soil to flow into the harbor, where it could block ship channels. This would not only reduce the functionality of Nagoya Port, it would also lead directly to delays in disaster relief and recovery/reconstruction support. For this reason, seismic assessment of the PIs and improvements to their earthquake resistance are urgent issues. In this study, we used a computer simulation to reproduce a PI in its current state, while taking into consideration its actual construction history, in order to gain an understanding of the damage that would occur during and after the largest envisioned Nankai Trough earthquake. Furthermore, the model used predicted how earthquake damage would progress if dredged soil were to continue to be added onto the PI without restriction. The main conclusions are as follows. 1) If the current situation remains unchanged, a certain amount of deformation would occur on the PI as a result of the earthquake, but the damage extent would not be such that dredged soil would flow into the harbor. 2) On the other hand, if the existing height were raised by 4 m, it would be impossible to prevent the lateral displacement of dredged soil, and there would be a high risk of dredged soil flowing into the harbor. These results indicate that continuing to raise the height of the reclaimed land is dangerous in terms of PI earthquake resistance. The analysis code used in this paper is the soil–water coupled finite deformation analysis code GEOASIA.

Keywords: reclamation, soft ground, seismic response analysis

1 INTRODUCTION

Nagoya Port is a shallow-water harbor prone to sediment accumulation, which makes it necessary to dredge approximately 1.3 million m$^3$ of soil annually in order to keep the port in operation. Construction of the first Nagoya Port Island (hereinafter, PI) began in 1975 as the sole disposal site for the dredged soil. To date, three PI have been completed, and the total area of reclaimed land is approximately 220 ha, which is equivalent to 50 Nagoya Domes (see Figure 1). However, the initial envisioned capacity of the PIs to accept dredged soil has already been exceeded, even though the temporary banks built on top of the reclaimed ground have increased their capacities. For example, the reclaimed land crown in the case of the first PI was 6 m above sea level during the planning stage, but because temporary banks have been added three times to date, the crown height has reached approximately 16 m above sea level, and the addition of further landfill is under discussion.
designed and constructed with the minimum required structural strength. Therefore, should a megathrust earthquake occur, it is likely that, in addition to damaging the revetment and temporary banks, the earthquake would cause large volumes of high-piled dredged soil to flow into the harbor, where it could block ship channels. This would not only reduce the functionality of Nagoya Port, it would also lead directly to delays in disaster relief and recovery/reconstruction support. For this reason, seismic assessment of the PI’s and improvements to their earthquake resistance are urgent issues.

In this study, we used a computer simulation to reproduce a PI in its current state, while taking into consideration its actual construction history, in order to gain an understanding of the damage that would occur during and after the largest envisioned Nankai Trough earthquake. Furthermore, the model used predicted how earthquake damage would progress if dredged soil were to continue to be added onto the PI without restriction. The analysis results showed that: (1) If the current situation remains unchanged, a certain amount of deformation would occur on the PI as a result of the earthquake, but the damage extent would not be such that dredged soil would flow into the harbor. (2) If the existing height were raised by 4 m, the foundation soil would be subjected to unbalanced loading, which would cause the ground directly beneath the revetment to be significantly deformed by the earthquake, thus reducing the revetment functionality. (3) At such time, it would be impossible to prevent the lateral displacement of dredged soil, and there would be a high risk of dredged soil flowing into the harbor. These results indicate that continuing to raise the height of the reclaimed land is dangerous in terms of PI earthquake resistance. The analysis code used in this paper is the soil–water coupled finite deformation analysis code GEOASIA (Noda et al., 2008) mounted with an elasto-plastic constitutive equation (SYS Cam-clay model (Asaoka et al., 2002)), which describes sand to intermediate soil and clay using the same theoretical framework.

2 ELASTO-PLASTIC MODELING NAGOYA PORT ISLAND

This chapter explains the elasto-plastic PI modeling process used. In order to prepare for this stage, various mechanical tests were performed on undisturbed soil samples collected from the site. Then, after identifying the elasto-plastic parameters, mechanical behaviors were reproduced using the SYS Cam-clay model. Because the PI has been built to a height of 16 m above sea level, the state of the foundation soil was expected to have undergone complex changes during the construction process. This research identified and reproduced the current PI soil conditions by faithfully reproducing its construction history using the method shown in Figure 2, which involved adding a finite element mesh onto the foundation soil. Soil–water two-phase elasto-plastic finite elements were then added for the dredged sediment, one layer at a time, and single-phase elasto-plastic finite elements were added for the concrete revetment.

Figure 3 shows the ground cross-section of the first PI, which was used as the analysis object of this study. Construction of this PI began in 1974, and it took approximately 30 years for the crown height to reach 16 m. While clayey soil is predominant in the PI foundation, there is an inclusion of dense sandy soil with an N-value of around 30. After construction of a rubble revetment on top of the foundation, temporary banks were built and filled with 26 m of dredged soil. Figure 4 shows grain size distribution curves for the Ac and Bc layers. Despite variations depending on the site where the sample was collected, the dredged soil used in the landfill and the cohesive soil of the foundation were taken as the same material and classified into Bc, AcU1–U3, and AcL according to differences in
Material constants (elasto-plastic parameters and evolution rule parameters) were determined along with the initial condition of the skeleton structure by trial and error using the SYS Cam-clay model in order to ensure that multiple test results of the same material could be reproduced from one set of material constants. Additionally, by simulating the (ideal) sampling process from in situ conditions, not only were laboratory test results reproduced, we could also estimate state quantities during natural sedimentation prior to PI construction. Figure 5 shows an example of the reproduction results, and Table 1 shows a summary of elasto-plastic behavior. As can be seen in the figure and table, the As layer is dense and strong, but all of the cohesive soil layers are weak, with a liquidity index of almost 1. Thus, the ground was modeled as being in a highly structured state. The most highly structured layer is the surface layer Bc, and the structure degree decreases with depth. Specific volume and structure degree were assumed to be uniform within each layer, and the over-consolidation ratio was distributed according to the overburden pressure (Noda et al., 2005). The concrete was taken to have Young's modulus $E = 2.35 \times 10^7$ kPa, Poisson's ratio 0.2, and density 2.7 g/cm$^3$, based on the actual concrete used. Additionally, although the AcU layer directly beneath the revetment has undergone improvement work to promote consolidation using the sand drain method, in this analysis, the only sand drain improvement effect considered was increased ground permeability, and hydraulic conductivity was only increased 100-fold in the model.

![Fig. 4. Grain size distribution curves.](image1)

![Fig. 5. Reproduction result of CU tests.](image2)

**Table 1. Material constants and initial conditions for this analysis.**

| Material name          | AcU1 | AcU2 | AcU3 | AcL | As | Bc | Rubble Bank |
|------------------------|------|------|------|-----|----|----|-------------|
| **Elasto-plastic parameters** |      |      |      |     |    |    |             |
| Compression index      | 0.18 | 0.18 | 0.18 | 0.18| 0.05| 0.18| 0.18        |
| Swelling index         | 0.019| 0.019| 0.019| 0.019| 0.0002| 0.019| 0.0005 | 0.019   |
| Limit state index      | 1.60 | 1.60 | 1.60 | 1.60| 1.10 | 1.60 | 1.70 | 1.60    |
| NCL intercept (98.1 kPa) | 2.22 | 2.22 | 2.22 | 2.22| 1.95 | 2.22 | 1.895 | 2.22   |
| Poisson’s ratio        | 0.30 | 0.30 | 0.30 | 0.30| 0.30 | 0.30 | 0.30 | 0.30    |
| **Evolution rule parameters** |      |      |      |     |    |    |             |
| Normal consolidation index | 3.00 | 3.00 | 3.00 | 3.00| 0.12 | 3.00 | 0.12 | 3.00   |
| $a$                     | 0.30 | 0.30 | 0.30 | 0.30| 0.30 | 0.30 | 2.00 | 0.30   |
| $b$                     | 1.00 | 1.00 | 1.00 | 1.00| 1.00 | 1.00 | 1.00 | 1.00   |
| $c$                     | 1.00 | 1.00 | 1.00 | 1.00| 1.00 | 1.00 | 1.00 | 1.00   |
| $c_s$                   | 0.40 | 0.40 | 0.40 | 0.40| 1.00 | 0.40 | 1.00 | 0.40   |
| Rotational hardening index | 0.001| 0.001| 0.001| 0.001| 0.001| 0.001| 0.001| 0.001 |
| Rotational hardening limit constant | 1.00 | 1.00 | 1.00 | 1.00| 0.90 | 1.00 | 0.001| 1.00   |
| **Other parameters**    |      |      |      |     |    |    |             |
| Soil particle density (g/cm$^3$) | 2.71 | 2.71 | 2.71 | 2.71| 2.69 | 2.67 | 2.593| 2.67   |
| Hydraulic conductivity (cm/s) | $2.0 \times 10^7$ | $2.0 \times 10^7$ | $2.0 \times 10^7$ | $1.0 \times 10^7$ | $2.0 \times 10^7$ | $1.0 \times 10^7$ | $1.0 \times 10^7$ | $1.0 \times 10^7$ |
| **Initial conditions**  |      |      |      |     |    |    |             |
| Specific volume        | 2.50 | 2.40 | 2.30 | 2.20| 1.84 | 2.80 | 1.593| 2.64   |
| Structure              | 9.00 | 7.00 | 5.00 | 4.00| 2.00 | 2.00 | 1.00 | 200    |
| Stress Ratio           | 0.00 | 0.00 | 0.00 | 0.00| 0.23 | 0.00 | 0.00 | 0.00   |
| Anisotropy             | 0.23 | 0.23 | 0.23 | 0.23| 0.23 | 0.00 | 0.00 | 0.00   |

774
Figure 6 shows the entire analysis cross-section for post-construction PI in its current state (before seismic input). With regard to the hydraulic boundaries, the ground surface above the water level was assumed to have a constant hydraulic pressure of zero (atmospheric pressure conditions), the ground surface below the water level was assumed to be a permeable boundary subjected to a pressure equal to the hydrostatic pressure, and both edges and the bottom surface of the ground were assumed to be impermeable boundaries. Additionally, all ground was assumed to be saturated and concrete was taken as an impermeable boundary. Figure 7 shows the input seismic motion. Since PI envisions the results of the largest anticipated Nankai Trough earthquake, which will involve a long-duration seismic motion with a large maximum acceleration of approximately 500 gal, the bottom face viscous boundary (William et al., 1975) (Vs = 300 m/s) was set at the ground’s bottom edge nodes, and lateral boundary element simple shear deformation boundaries (Yoshimi and Fukutake, 2005) were established at the elements on both side edges of the foundation soil.

Figure 8 shows shear strain distribution after 100 years from the start of landfill construction. Shear strain of approximately 10% occurs in the Bc layer as a result of the added dredged soil, but the layer is stable during and after soil addition. Figure 9 shows the structure degree distribution before and after PI construction. As can be seen in the figure, even though the AcU1 layer is directly beneath the revetment, and the As layer became slightly less structured as a result of PI construction, almost all areas are in a highly structured state, including the newly added dredged soil.

Figure 10 shows shear strain distribution after the earthquake. Figure 11 shows the structure degree distribution after the earthquake.
3 SEISMIC ASSESSMENT OF EXISTING NAGOYA PORT ISLAND IN THE EVENT OF NANKAI TROUGH MEGATHRUST EARTHQUAKE

This chapter investigates the seismic behavior of the PI in the event of a Nankai Trough earthquake in 2015, which is 41 years after PI construction began. Figure 10 shows shear strain distribution directly after the earthquake and 4 years later. The seismic motion has generated large shear strain at reclaimed ground. However, as is evident from Figure 10, the dredged soil does not cross the revetment and sediment does not flow into the harbor. And most of the deformation occurs during the earthquake, and the post-earthquake deformation is not significant. Figure 11 shows the structure degree distribution. As can be seen in the figure, there is considerable landfill structural degradation in areas where the seismic motion has generated large shear strain. Figure 12 shows the behavior of those elements in the relevant locations. It also shows that the rigidity of the dredged soil decreases significantly, along with effective stress, with cyclic loading. This generates large shear strain during the earthquake. A swelling is apparent at the toe of the rubble mound and, as is evident from Figure 10, this is because the rubble mound as a whole forms a circular slip surface within the ground as it deforms. This does not lead to a large deformation under the existing conditions; however, it suggests that reinforcing the reclaimed ground and the area around the toe of the mound would be effective as an earthquake countermeasure aimed at reducing damage.

4 SEISMIC ASSESSMENT OF RAISED NAGOYA PORT ISLAND IN THE EVENT OF NANKAI TROUGH MEGATHRUST EARTHQUAKE

A valid method of utilizing the large volumes of soil generated from Nagoya Port dredging has not yet been identified, so the use of PIs as temporary storage sites can be expected to continue into the future. This chapter investigates the seismic behavior of a PI if landfill continues and the current height is raised by a further 4 m.

Figure 13 shows shear strain distribution after 100 years from the start of landfill construction. It is evident that even if the height of the landfill is raised by an additional 4 m, there will be no problems with stability at ordinary times during or after construction.

Figure 14 shows shear strain distribution immediately after, and 4 days after the earthquake. A circular slip surface occurs underneath the rubble mound, and that damage is caused to the revetment. With the function deterioration of the revetment, there is increased danger that the raised dredged soil will cross over the revetment and flow into the harbor. Figure 15 shows the structure degree distribution. As is evident when compared with Figure 11, there is considerable local structure degradation in the AcU layer directly beneath the revetment. Figure 16 shows the behavior of the elements in the relevant location. In places subject to unbalanced loading, such as directly beneath the revetment, effective stress and rigidity decrease significantly, even in cohesive soil, partly because the vertical load has increased as a result of raising the landfill height, which results in large shear deformation during the earthquake.
Dredged soil is generated continually in Nagoya Port and continues to be temporarily stored on PIs so that the function of the port can be maintained. The height of reclaimed ground on the analyzed PI has reached 16 m above sea level, and it is feared that, in the event of a Nankai Trough megathrust earthquake, a large volume of dredged soil will flow into the harbor and block shipping routes.

In this chapter, we reported on a seismic assessment of a PI that attempts to clarify dredged soil behavior in the event of the largest envisioned Nankai Trough earthquake that is expected to occur within the foreseeable future. The numerical analysis results showed the following: (1) PIs are built on foundations deposited clayey soil under embankment loading, Soils and Foundations, Vol.45, No.5, 39-51.

However, should a Nankai Trough earthquake strike after heights of existing PI were raised by an additional 4 m, the increase in vertical load would cause a disruption of the weak clay foundation, and when combined with increased revetment deformation, would create a high risk of dredged soil flowing into the harbor.

These results indicate that continuing to raise the height of the reclaimed ground is dangerous in terms of the PI earthquake resistance. Therefore, from the standpoint of reducing earthquake damage, it is essential to identify valid uses for dredged soil and refrain from increasing the height of the PIs any further. In fact, the height of the PI crown analyzed in this study should be reduced somewhat. In light of the results of this analysis, the Chubu Regional Development Bureau has announced a plan to impose a limitation to the dredged soil height, and intends to strengthen earthquake resistance by implementing earthquake countermeasures, such as placing a counterweight in front of the revetment (Nagoya port review meeting).

**ACKNOWLEDGEMENTS**

The research in this report was supported by Grants-in-Aid for Scientific Research (S) (No. 21226012) and (A) (No. 25249064). We would like to express our thanks to the Nagoya Port Authority, Ministry of Land, Infrastructure, Transport and Tourism, for their assistance and cooperation with this research.

**REFERENCES**

1) Asaoka, A., Noda, T., Yamada, E., Kaneda, K. and Nakano, M. (2002): An elasto-plastic description of two distinct volume change mechanisms of soils, Soils and Foundations, Vol.42, No.6, pp.47-57.

2) Nagoya port review meeting countermeasure for tsunami and earthquake (the sixth) http://www.pa.cbr.mlit.go.jp/NAGOYA/topics/140311/index_files/data04_3.pdf (in Japanese)

3) Noda, T., Asaoka, A., Nakano, M., Yamada, E. and Tashiro, M. (2005): Progressive consolidation settlement of naturally deposited clayey soil under embankment loading, Soils and Foundations, Vol.45, No.5, 59-51.

4) Noda, T., Asaoka, A. and Nakano, M. (2008): Soil-water coupled finite deformation analysis based on a rate-type equation of motion incorporating the SYS Cam-clay model, Soils and Foundations, Vol.48, No. 6, pp. 771-790.

5) William, B., Joyner and Albert T. and F. CHEN. (1975): Calculation of nonlinear ground response in earthquakes, Bulletin of the Seismological Society of America, Vol.65, No.5, pp. 1315-1336.

6) Yoshimi, Y. and Fukutake, K. (2005): Physics and evaluation, countermeasure technology of the ground liquefaction ISBN 978-4-7655-1693-8, Gihoudou Print Co. Ltd. (in Japanese)