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THE SEISMIC ANALYSIS OF AN EARTH-FILL DAM ON THICK LIQUEFIABLE GROUND AND COUNTERMEASURES AGAINST A LARGE EARTHQUAKE

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ABSTRACT

This paper introduces the seismic analysis of an earth-fill dam on a thick layer of liquefiable ground and countermeasures that can be taken to counteract the effect of a large earthquake by using effective stress with dynamic analyses using Finite Element Methods (FEM). According to these analyses, the dam's crest subsidence is limited after the earthquake due to the restriction of the liquefaction potential by adopting the counterweight filling method

1. INTRODUCTION

Japan Water Agency manages dams in various parts of Japan. In recent years, many large earthquakes have occurred in Japan and some dams suffered damages from the earthquake. Since dam is one of the most essential facilities for water supply, security of dams against the expected large earthquakes are required in order to keep the facility functions, so Japan Water Agency have checked the security of dams against the expected large earthquakes by using seismic analysis.

2. TYPICAL CASE HISTORY:HATTACHI DAM

The target facility is Hattachi Dam which was constructed in Aichi Prefecture in 1967. Hattachi Dam is a central core type earth-fill dam of which height is 22.5m, crest length is 346.5m, total reservoir capacity is 1,700,000 m^3

Figure 1 shows a standard cross section of Hattachi Dam and the ground conditions. The ground consists of, from the surface, saturated sandy layers (Dg2, Ds) of 15m thickness, soft clayey layer (Dc) of 10m thickness, sand and gravel (Dg1, Dm, Dt) of 15m thickness, and chert (CH) which is an engineering foundation. Saturated sandy soil below the ground surface is associated with liquefaction during the earthquake because FL (Factor of Liquefaction) value of specifications for highway bridge (Japan Road Association. 2002) is estimated from 0.2 to 0.3.

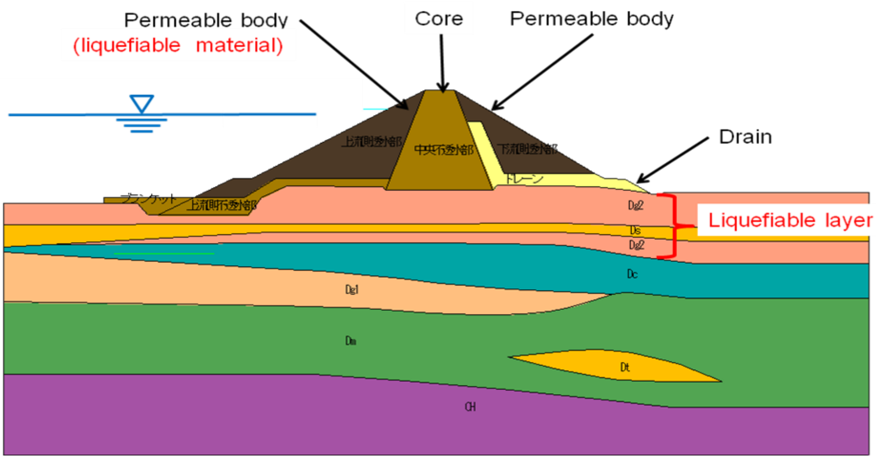


Figure 1. Cross section of Hattachi Dam

Figure 2 shows the locations of earthquakes for verifications of seismic performance. The Tokai Quake (epicenter is Suruga-trough), The Tonankai Quake (epicenter is located offshore from Kii Peninsula) and The Nankai Quake (with epicenter somewhere in the Nankai trough) is expected to occur in the near future, so early implementation of the countermeasures for Hattachi Dam is required.

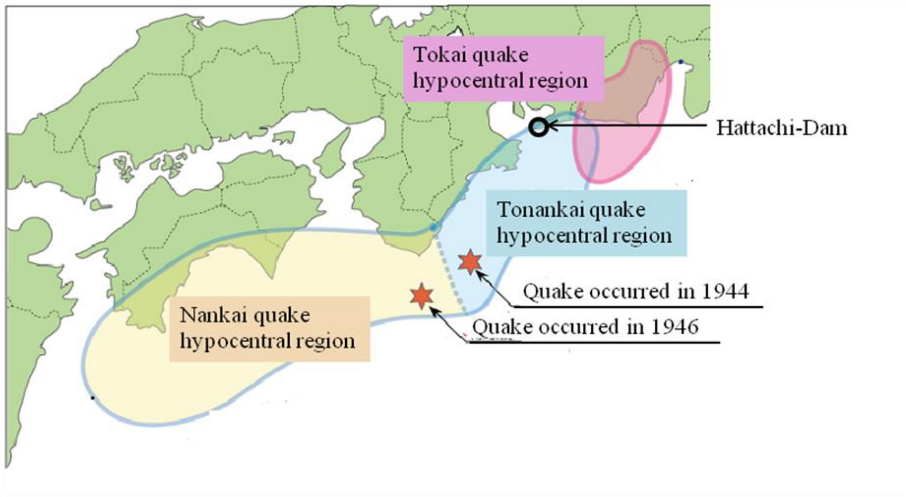


Figure 2. The target quake

3. PROCEDURES FOR THIS STUDY

Figure 3 shows the procedure of this study. At first, parameter values for each zone of dam body and each layer of the ground were assumed. The next step was soil surveying in the field and soil test in the laboratory. Thereafter, the seismic performance after a earthquake was investigated by seismic analysis using 2D effective stress dynamic FEM, LIQCA (Oka et al. 1994) and, according to the level of requirement, a minimum countermeasure that satisfies the seismic performance of the facility was proposed.

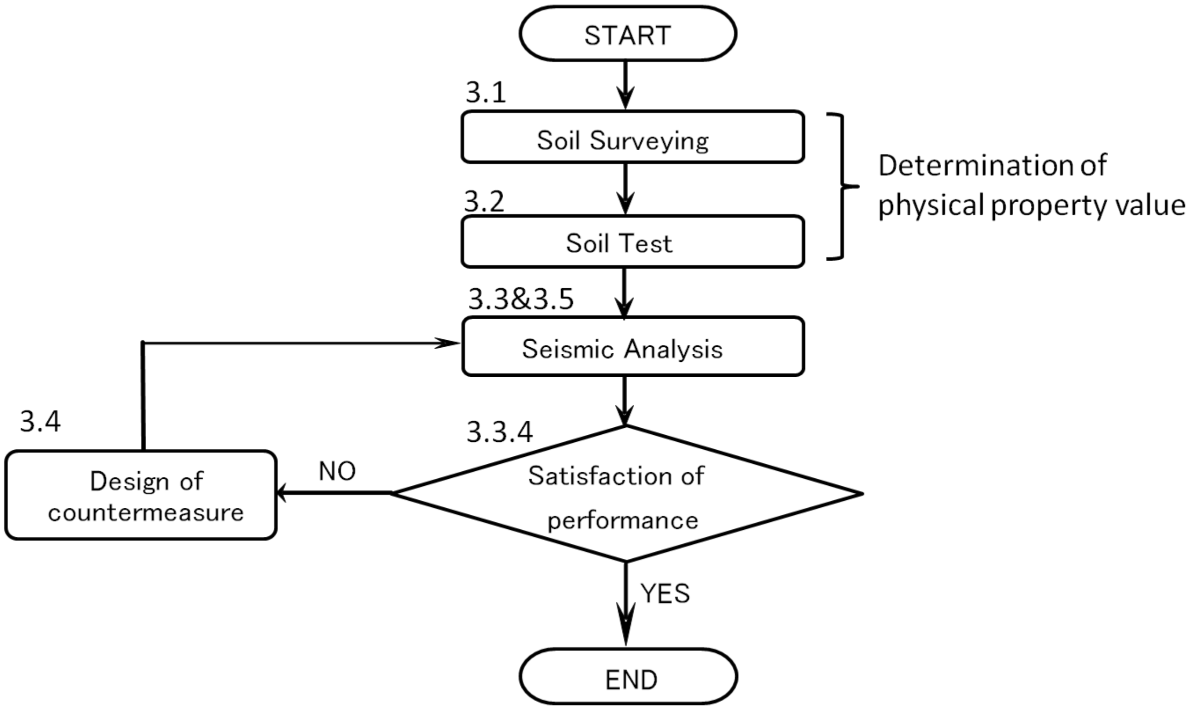


Figure 3. Procedures for this study

3.1 Soil Surveying

Figure 4 shows boring positions for soil surveying at the dam site. Boring positions were planned in line of dam's axial direction and dam's cross direction. The biggest cross section is located at STA3+20.0 and the cross section having the deepest engineering foundation is located at STA2. Figure 5 shows geological cross sections at STA2 and STA3+20.0.

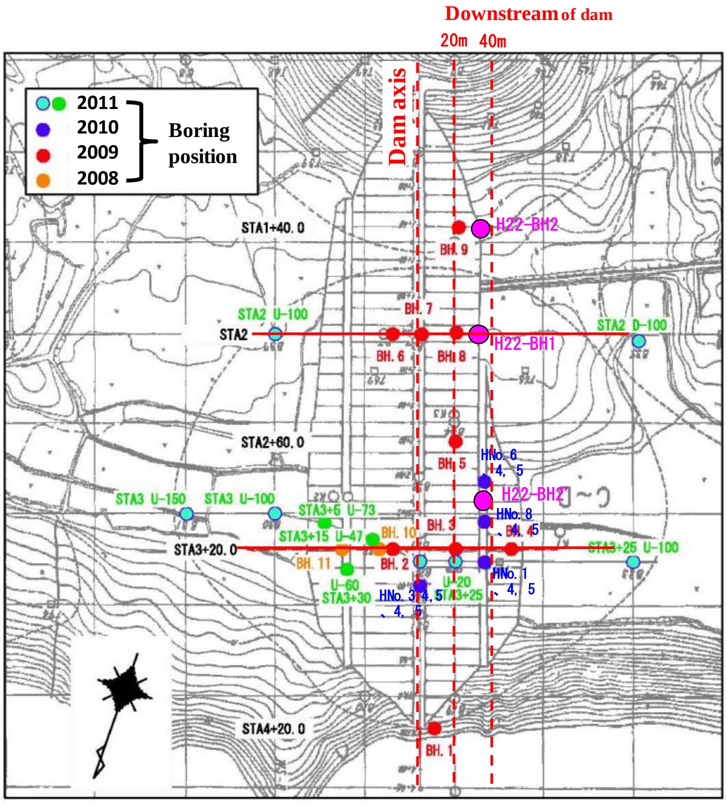


Figure 4. Boring positions

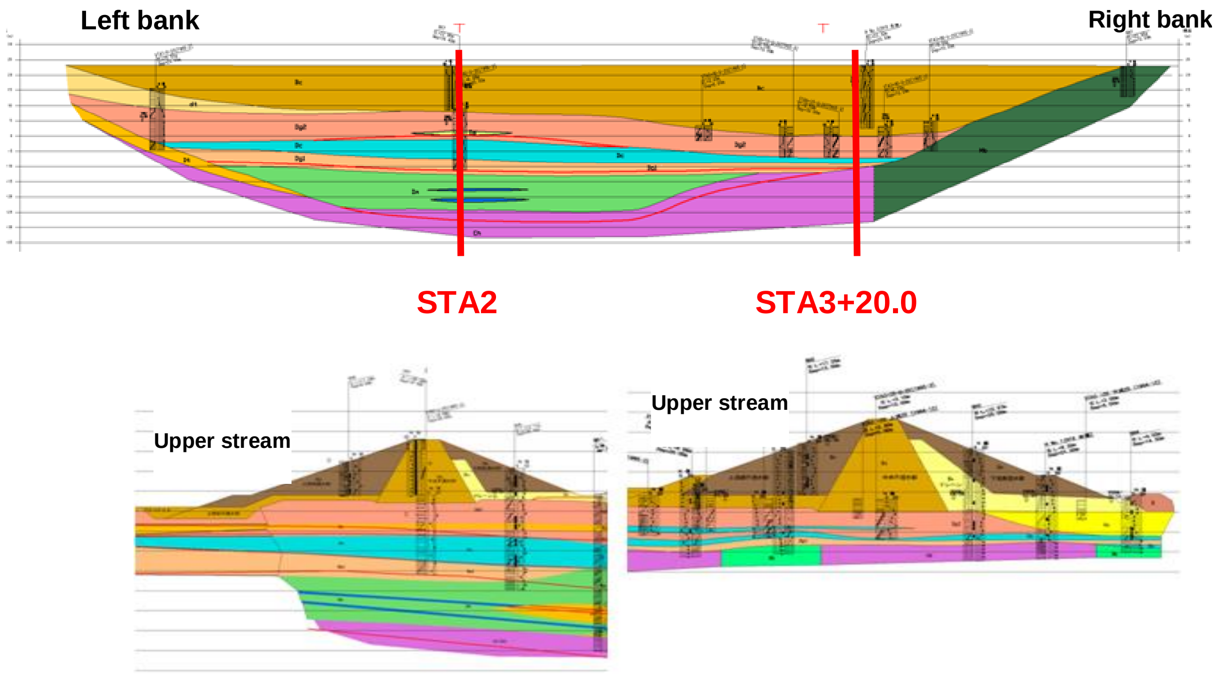


Figure 5. Geological cross sections at STA2 and STA3+20.0

3.2 Soil test

To decide physical property value used in seismic analysis, various soil tests, e.g. triaxial shear test, consolidation test, and permeability test, were carried out by using undisturbed samples obtained by boring at the dam site. Especially about liquefiable materials of the dam body and the ground, dynamic deformation test and liquefaction strength test were carried out. Analysis parameters of liquefiable materials were decided by the results of element simulation compared with laboratory tests of repeated compression triaxial test under non-drainage. As an example of element simulation, Figure 6 shows liquefaction strength curve of Dg2, and it also shows relationship diagram between loading shear stress ratio of samples (A: $\sigma_d/2\sigma'_0$) and repeated loading times at the time of samples leading to liquefaction (B: times).

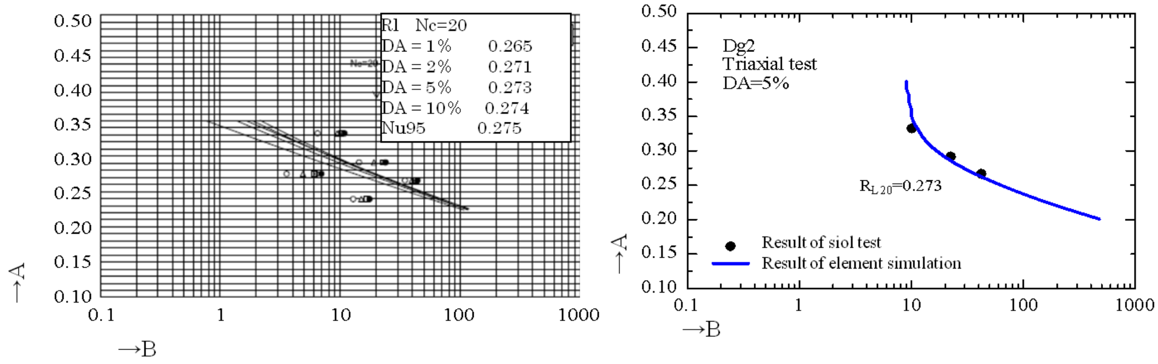


Figure 6. Liquefaction strength curve

3.3 Seismic analysis in case of without countermeasures

Seismic analysis method selected for this study was 2D effective stress dynamic FEM, LIQCA to reproduce liquefaction phenomenon. This analysis code has been often used successfully in the case of seismic analysis including consolidation process for liquefiable ground and banking structure. Seismic analysis was carried out about cross sections for STA2 and STA3+20.0. In this paper, only the result of cross section along STA2 is described, which was the more severe case.

3.3.1 Analysis model

Figure 7 shows the analysis model. This model was 2D FEM model of cross section of STA2. The width of analysis model was five times of the dam height. Lateral boundary condition was horizontal fix and vertical roller, and bottom boundary condition was viscous boundary (Oka F, et al. 1999).

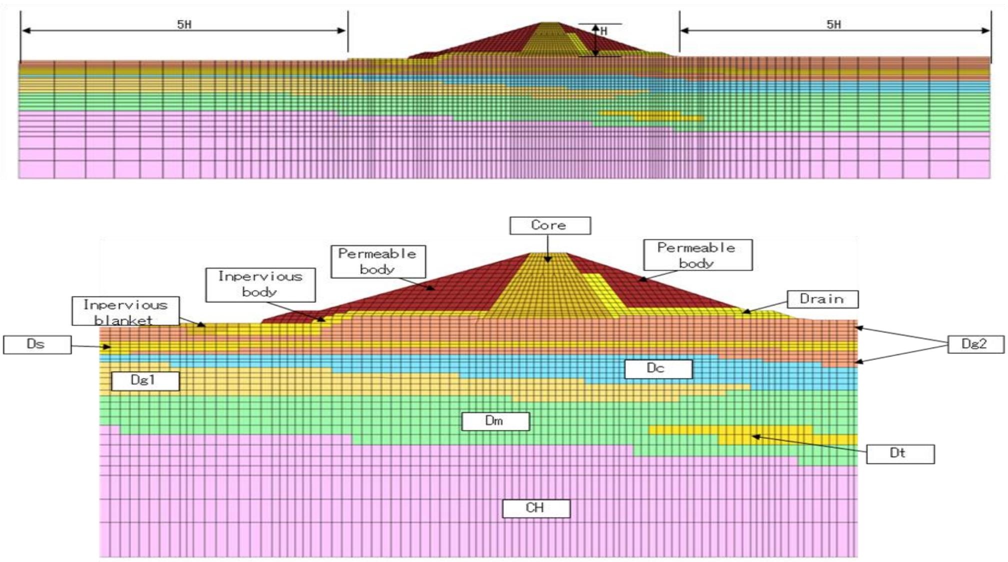


Figure 7. Analysis model

3.3.2 Input quake wave

Input quake wave was the Simultaneous Tokai, Tonankai and Nankai Earthquake declared by the Central Disaster Prevention Council, Cabinet Office, Government of Japan (Homepage 2006). Figure 8 shows the time-history waves input to the bottom of analysis model at the time of dynamic analysis. Quake duration is 327 seconds and the peak acceleration is 5.71m/s^2(571gal) with respect to horizontal upstream-downstream direction component.

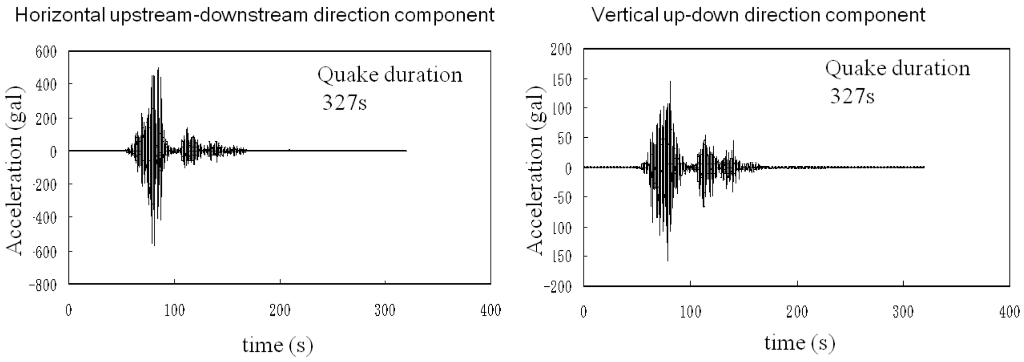


Figure 8. Time history of the earthquake

3.3.3 Limit state seismic performance

Seismic performance requirement for Hattachi Dam is stipulated to show the restrictive damage (i.e. the function of storing water can be expected to be retained after a quake and permanent repair can be made without difficulty), so it's criteria for verification to seismic performance is less than 1m(i.e. the design allowance) of subsidence for dam's crest.

3.3.4 Analysis result

Figure 9 shows the amount of subsidence at the time consolidation subsidence was over (42 hours after the end of a quake) as a result of the analysis in case of without countermeasures. Subsidence of the base was 1.030m and subsidence of the crest was 1.769m, so subsidence of the dam body itself was 0.739m. Figure 10 shows an excess pore water pressure ratio distribution at the time the quake was over. It can be confirmed that the saturated sandy layers (Dg2, Ds) of 15m thickness and upstream dam body liquefied with this figure. Consequently it was concluded that the occurrence of the liquefaction of upstream dam body and liquefiable ground layer was a main factor of subsidence.

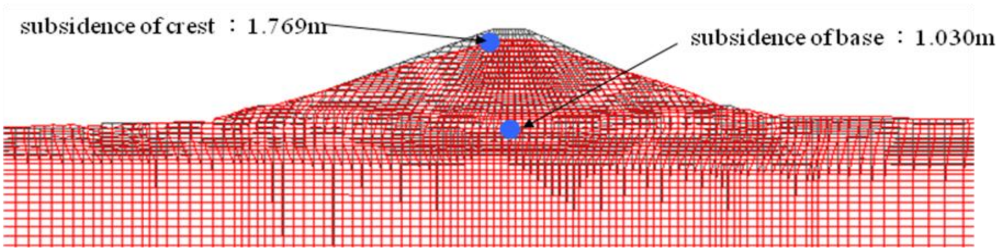


Figure 9. Deformation at the time consolidation subsidence was over(42 hours after the end of a quake)

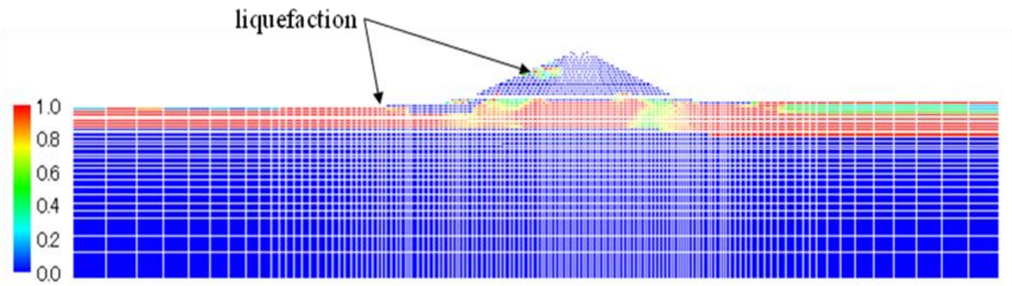


Figure 10. Excess pore water pressure ratio distribution after the earthquake

Figure 11 shows vertical displacement of section of dam center. Blue line represents vertical displacement at the time a quake was over and pink line represents vertical displacement at the time consolidation subsidence was over. It can be confirmed that most of the subsidence of liquefiable layer was occurred during quake and most of the subsidence of dam body was occurred in the process of consolidation.

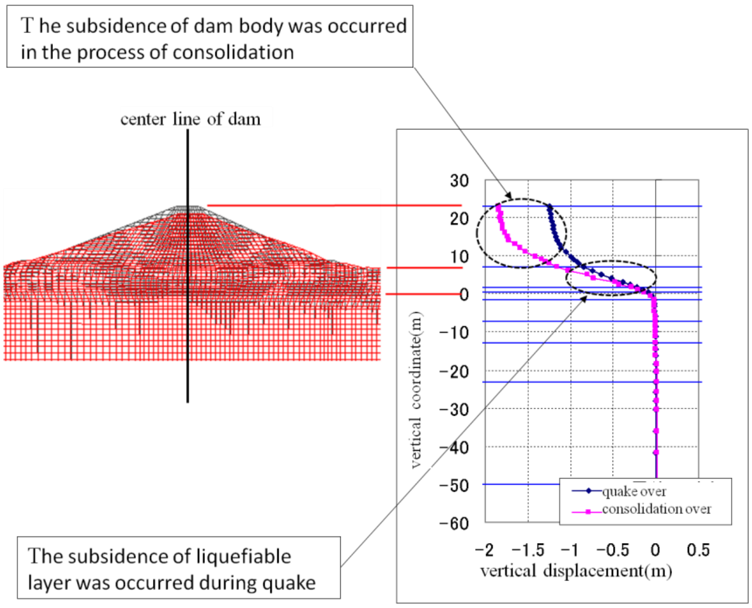


Figure 11. Vertical displacement of section of dam center at the time a quake was over and consolidation subsidence was over

3.4 Comparison of countermeasures

At first, among the generally used countermeasures against liquefaction, compaction method, consolidation method and replacement method were excluded because the target is the existing structure (Public Works Research Institute. 2000). Next, groundwater level decreasing method was excluded because the target is the dam having storage function. Rather than preventing the occurrence of liquefaction, restriction of liquefaction grounded on performance based design (PBD) concept was considered as a reasonable countermeasures, which can restrict the effect of liquefaction and can satisfy the seismic performance requirement of the facility (Kato T, et al. 2009). As a method to restrict liquefaction, steel pipe method and counterweight filling method were compared by seismic analysis. Steel pipe method couldn't satisfy the limit state seismic performance (i.e. less than 1m amount of subsidence amount of dam's crest) according to the result of seismic analysis, so that was omitted in this paper.

Figure 12 shows counterweight filling method. Crushed stones were selected for upstream materials because certain weight and strength are necessary to restrict liquefaction of the upstream dam body. In order to prevent the decrease of water storage capacity due to upstream counterweight fills, the soil excavated in reservoir was used for the material of downstream counterweight fills. The optimum gradient to satisfy the required performance was determined by seismic analyses.

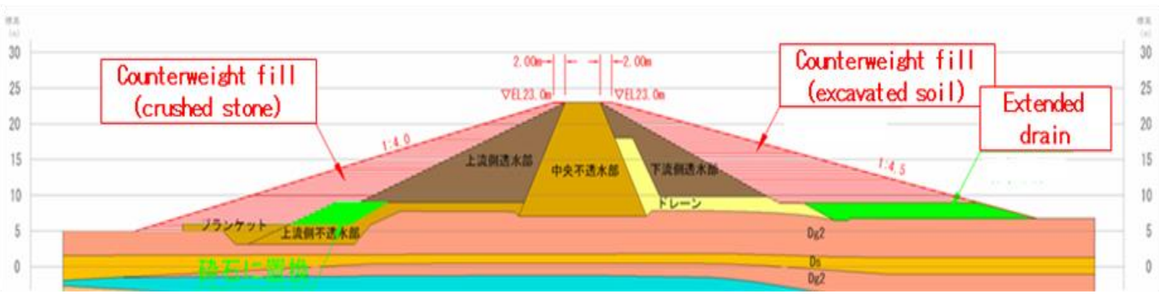


Figure 12. Counterweight filling method

3.5 Seismic analysis after countermeasures

Figure 13 shows an analysis model after countermeasures. Figure 14 shows an amount of subsidence at the time consolidation subsidence was over (21 hours after the end of a quake) as a result of the analysis of after countermeasures. Subsidence of the base was 0.60m and subsidence of the crest was 0.95m, so subsidence of the dam body itself was 0.35m. Figure 15 shows an excess pore water pressure ratio distribution at the time a quake was over. It can be confirmed that liquefaction of saturated sandy layers (Dg2, Ds) of 15m thickness and the upstream dam body was restricted in comparison with the case of no countermeasures taken. Therefore, it was concluded that the effect of counterweight fills was the decrease of subsidence due to the restriction of liquefaction and shortening of consolidation time.

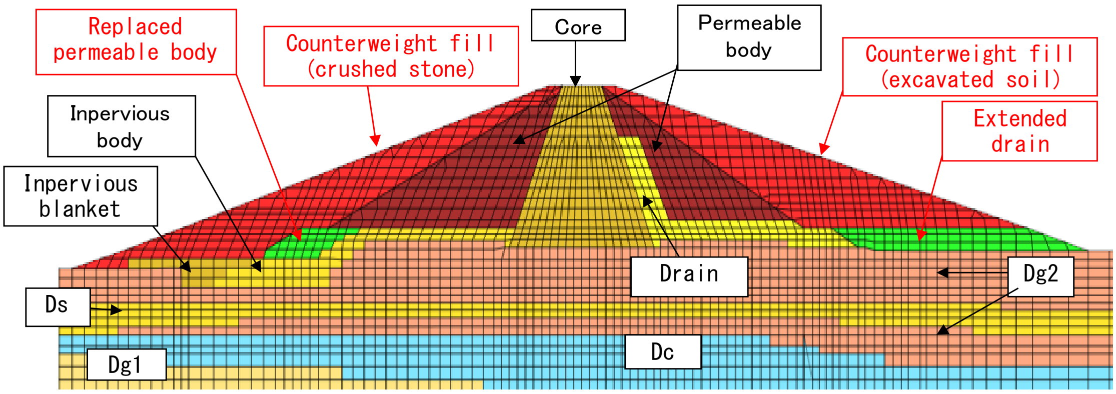


Figure 13. Analysis model after countermeasures

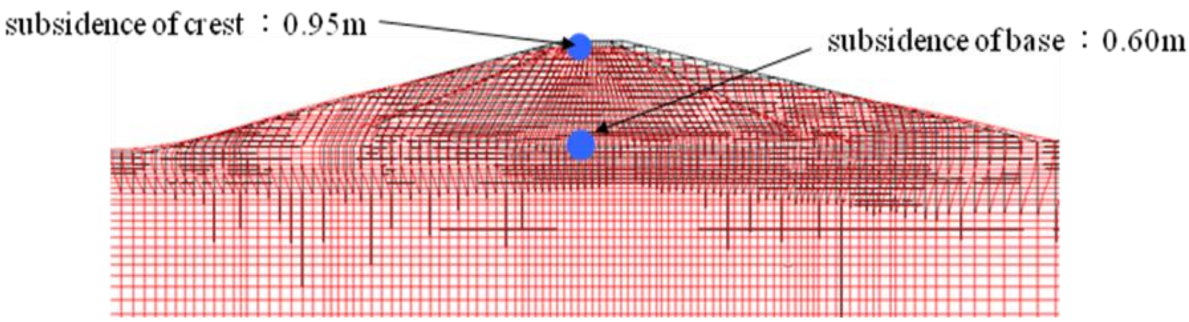


Figure 14. Deformation at the time consolidation subsidence was over (21 hours at the end of the earthquake)

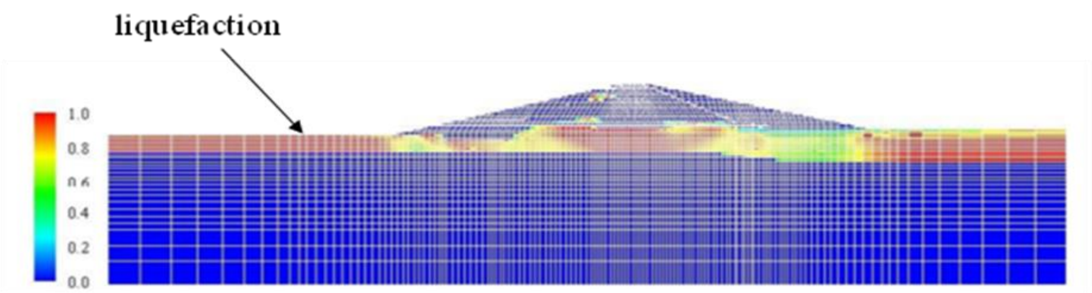
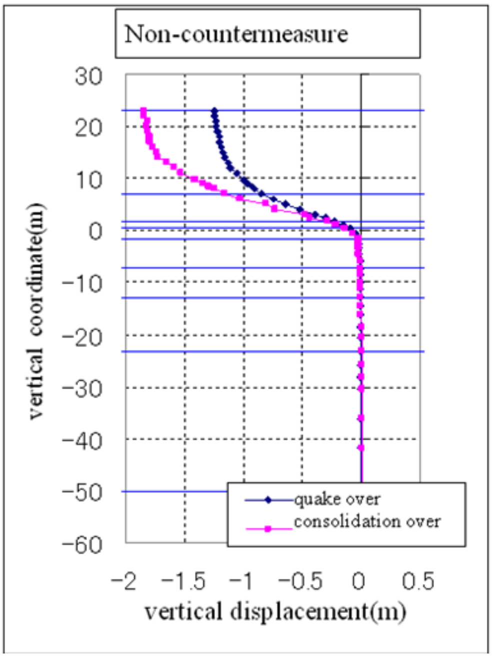


Figure 15. Excess pore water pressure ratio distribution after the earthquake

This is apparent when viewing Figure 16. After the countermeasures subsidence of liquefiable layer was greatly reduced and subsidence during consolidation process was greatly reduced due to shortening of consolidation time.



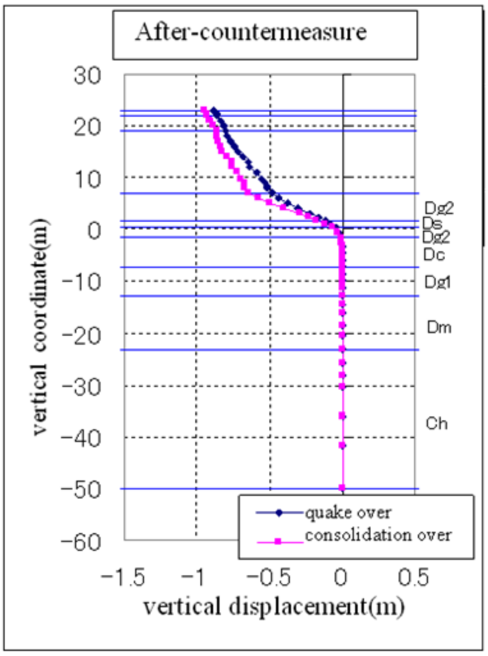


Figure 16. Comparison of vertical displacement of section of dam center at the time a quake was over and consolidation subsidence was over

4. CONSTRUCTION OF COUNTERMEASURES

Figure 17 shows the upstream site of the embankment (Left figure is distant view and right figure is near view). Crushed stones adopted for upstream counterweight fills were purchase materials, so that construction was easy because of its stable quality. On the other hand, the materials of downstream counterweight fills were the soil excavated in reservoir, so that construction was difficult because of its unstable quality.

Figure 17. The upstream side of the embankment

Figure 18 shows comparison of grading curve of the soil excavated in reservoir and the soil in stock pile. The majority of the soil excavated in reservoir was inhomogeneous viscous soil having high moisture ratio. And it was difficult to ensure the suitability by using this material directly, so it was decided to mix viscous soil and gravel in a stock pile. Figure 19 shows mixing process in the stock pile. This measure improved as shown by the red line in the Figure 18, and the design values for the material of counterweight fills were satisfied in field compaction tests and laboratory tests.

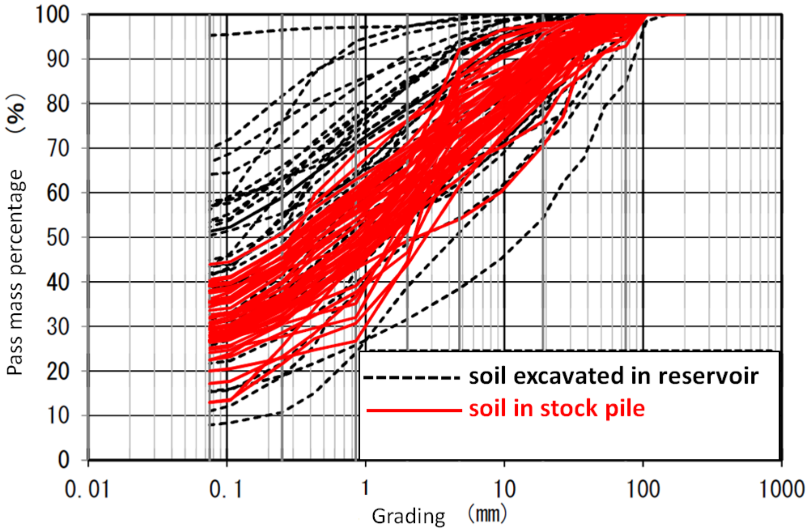


Figure 18. Comparison of grading curve of the soil in reservoir and the soil in stock pile

Figure 19. Mixing in the stock pile

5. CONCLUSION

Development of practical seismic countermeasures for existing structures for the early implementation is important issue under the situation that the occurrences of big earthquakes are predicted throughout Japan. The countermeasure against them for large scale existing facility, in particular, has been a controversial issue because of the difficult implementation due to the high cost of the measures. It is considered that the same issue is shared many countries, where such hazard of the occurrence of large earthquakes has been predicted. This paper could be helpful for those countries.

6. ACKNOWLEDGEMENTS

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7. REFERENCES

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