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CENTRIFUGE TEST OF ROCKFILL DAM AND SEISMIC STABILITY EVALUATION BASING ON SEISMIC RESPONSE AND RESIDUAL DEFORMATION

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SUMMARY

In this study centrifuge shaking table test result under 50G is reported, and some comparison is indicated with existing shaking table tests and in-situ observations under strong earthquakes. As a result, it found that the density of the fill dam mock-up affects on the seismic response and the residual deformation. And the large residual strain by earthquake appears especially near the surface embankment and the crest. The relation between the residual crest settlement and the maximum foundation acceleration by this centrifuge test has good harmony with ones by existing shaking table tests and in-situ observations.

INTRODUCTION

Response property of rockfill dam has been studied by shaking table test and dynamic analysis [1][2][3][4][5], however the response under strong earthquake and residual deformation are not clarified. Because it is difficult to evaluate dynamic characteristics of large size particle rock and to evaluate response of actual big size dam. On the other hand, some measurement results of acceleration response and residual deformation of actual dams under strong earthquakes have been reported [6][7] and these data has been clarifying the actual behavior. And then centrifuge test is able to simulate actual size model and the test equipment has been modified and scaled up [7][8]. In this study, detailed acceleration response and residual deformation of rockfill dam by centrifuge test under strong earthquake are presented, and some comparison with already done shaking table tests and in-situ observation and then stability evaluation based on residual deformation and dam function are discussed.

CENTRIFUGE TEST

The owner of the centrifuge test equipment is Obayashi Corp. and its main property is shown in Table 1. Photo 1 shows the general view and the shaking table bucket of the centrifuge. The centrifuge acceleration of this test is 50G. Fig.1 shows the container and the dam model and one side wall of the container is made in acryl resin so as to see the residual deformation after shaking. Furthermore Teflon sheet is placed between the sidewall and dam model and grease is applied between them to decrease the friction between them.

Fig.1 also indicates the dam model. The main feature of this model is that the gradient is 1:1.4, which gradient is most steep among existing Japanese dams. There are 2 reasons to apply the gradient. One is that residual deformation of rockfill dam is expected to be small and it needs bigger deformation by the

test as possible. Another is as followed. Namely for Concrete Facing Rockfill Dam (CFRD) which is main type from 1920's to present time all over the world except seismic zone such as Japan, the gradient is applied almost 1:1.4, and in this case the slope angle is equal to 35.5°, so if the design internal angle of rock material is 40° which is adopted in many Japanese RD, statical seismic coefficient would be close to zero. But RD with 1:1.4 has a seismic stability with some degree because internal angle of rock material is bigger than 40° as described later. Therefore it needs to know the degree of seismic stability.

Table 1 Main Profile of Centrifuge

turning radius to shaking table	6.86m
Shaking table bucket	30N
Shaking table	2.2m×1.07m
Maximum gravity acceleration	50G
Maximum exciting force	1200N

Photo 1 General view and shaking table bucket of centrifuge (owner: Obayashi)

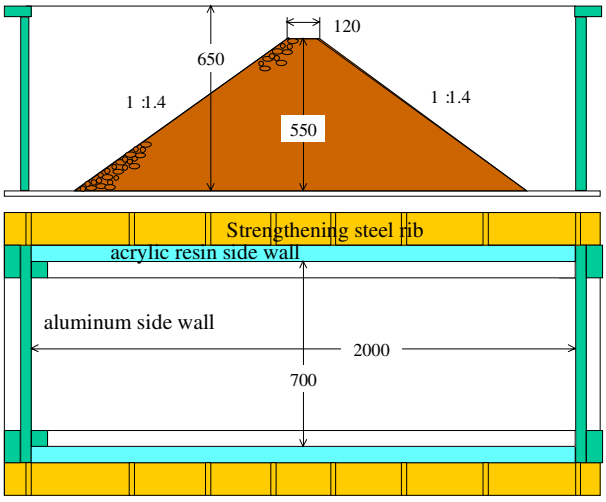

Fig.1 Container and dam model

The test sample is $C_M \sim C_L$ class rock material, which is a little bit soft and weathered comparing with main rock material ($C_M \sim C_H$). The grading curve shown in Fig.2, is a little bit coarse comparing with already embanked rock [10], and is similar to 1/50 particle size of actual grading. The compaction property is shown in Fig.3. 2 Cases with dense and loose density are conducted in centrifuge test, and they are 17.6 and 16.7 kN/m^3 that correspond to 0.1 and 1.0 E_{CJIS}. If minimum and maximum density are defined to be for placing in mould and compacting with 5E_{CJIS} each other, the relative densities are calculated to be 71 and 7 %. Dense case corresponds to in-situ density compacted by large roller which are close to the density compacted in 1.0E_{CJIS}[11]. And loose case corresponds to dumped compaction of high lift method [10] that was applied almost 30 years ago.

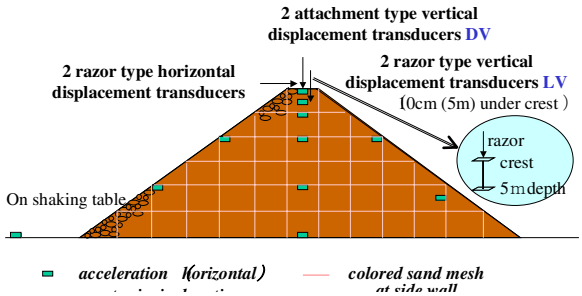

Fig.3 measuring transducer

Fig.2 indicates the drained strength of rock material with dense and loose density, which bases on internal friction defined the angle of the straight line for each failure Mohr circle [11]. The internal friction angle are less than that of already embanked rock material[11], most of which has ($C_M \sim C_H$), as shown in Fig.4, because the class is lower. Fig.5 shows the vertical deformation modulus under very small strain (almost 10-6) by cyclic triaxial test, which are smaller than that of already

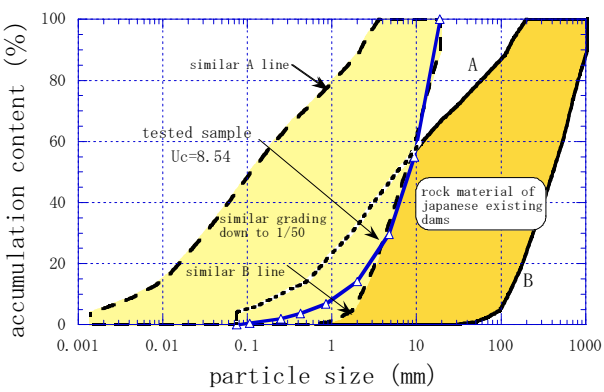

Fig.2 Physical and mechanical property of tested sample rock material (a) grading

measured values by triaxial test and in-situ elastic wave measurement [12].

Input wave are 10 cycles sin (1.8Hz) and Minowo river dam wave in Hyougo-ken Nanbu earthquake. Total test case is 4 with 2 densities and 2 input waves. Applied frequency is 50 times basing on similarity rule. Input motion is progressively increased in each case, target maximum acceleration is established 100, 200, 400, 600,800 gal in sin wave loading case and 200, 400, 800gal in Minowo river wave.

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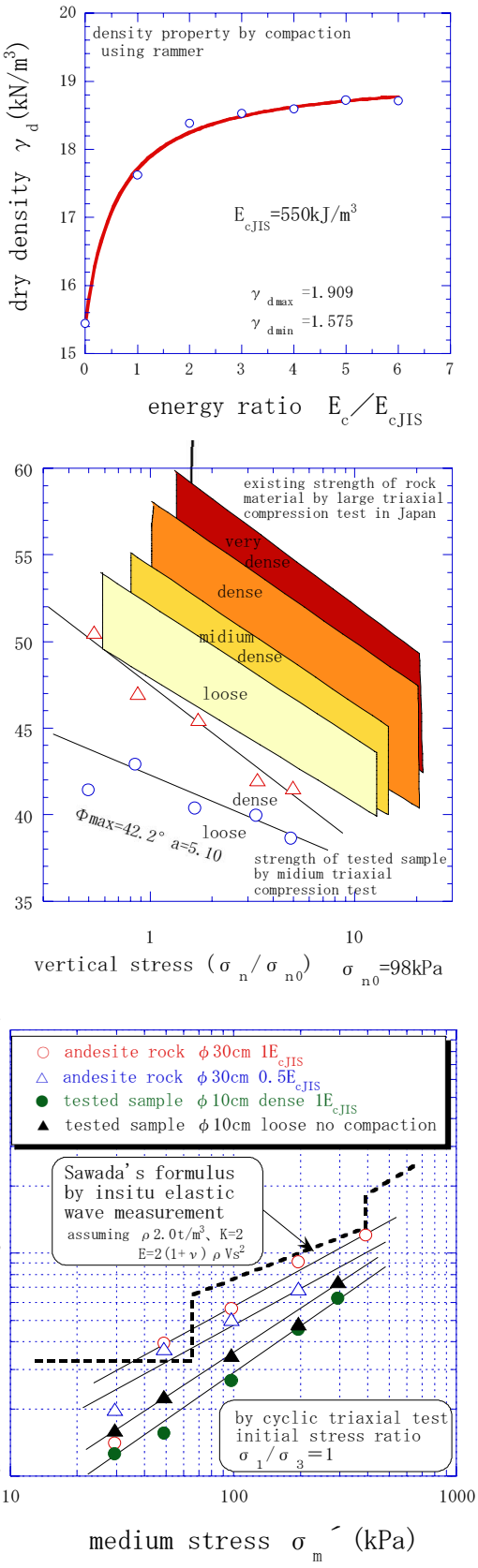

Fig.2 Physical and mechanical property of tested sample rock material
(b) compaction curve (c) triaxial compression (d) cyclic deformation

400, 600,800 gal in sin wave loading case and 200, 400, 800gal in Minowo river wave.

Both of acceleration and displacement transducer are installed as shown in Fig.6. 10 acceleration transducers are arranged 6 along the central axis, 2 on the surface and 2 in the slope. Displacement transducers are arranged 2 to the vertical direction, 2 to the horizontal direction on the crest, 1 to the vertical direction 5m deep from the crest, and also colored sand mesh is established at the side wall. It found that vertical crest residual displacement near the side wall is almost 10% less than that at the center, so it is one of the feature of this test that the residual deformation inside the embankment is clarified by colored sand mesh.

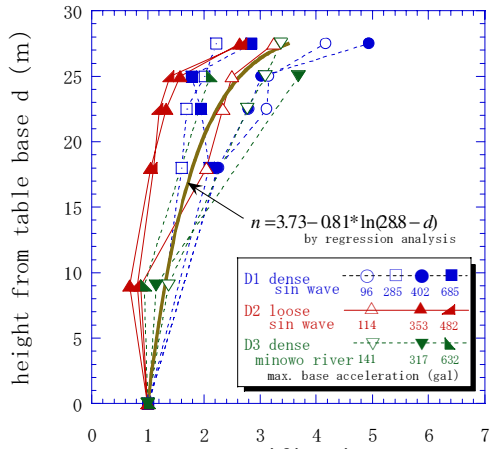

Fig.4 response magnification

TEST RESULT

Fig.4 illustrates the acceleration response result along centerline. D4 result is not shown in Fig.4 and some measurement of other case is not good. Because the shaking table cannot keep the horizontal position to shaking direction ad it sometimes rotate a little bit in centrifuge test. And the transducer installed near the surface sometimes vibrates itself. According to Fig.4 it found that bigger response acceleration magnification is restricted near the surface, it reveals at upper 2.5m of the height 27.5m, the average of maximum response magnification (max. crest acceleration / max. base acceleration) at the surface is almost equal to 3.5 and the model density clearly affects the magnification.

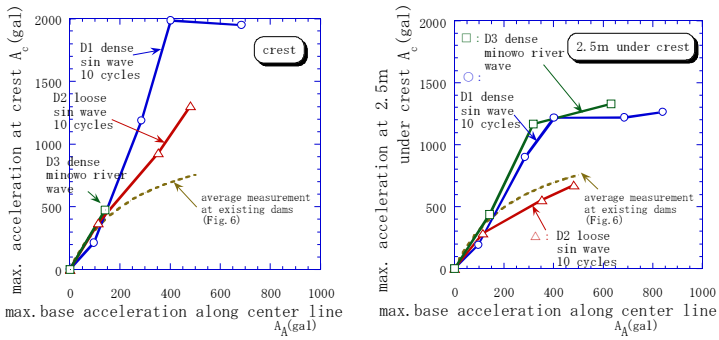

Fig.5 max. acceleration at base and crest along centerline

Fig.5 illustrates the relationship between the maximum acceleration at the base and crest or 2.5m depths for each case. When input motion is smaller i.e. max. base acceleration is almost 100 gal, the average of maximum response magnification by centrifuge test (= almost 3.5 as described before) is almost equal to the average value of existing dams by in-situ observation result due to Fig.6 [13]. The bigger the input motion is, the smaller the average of response magnification at crest by centrifuge tests is. Fig.5 reveals that the model density affects maximum response magnification, but it is not clear whether input wave affects it. Especially it found that maximum response magnification by loose case of centrifuge test is close to that of existing dams in Fig.6. The reason seems that many of the dams damaged by strong earthquake in Fig.6 were constructed before 1965 and at that time the compaction of their rock zones were not adequate [13].

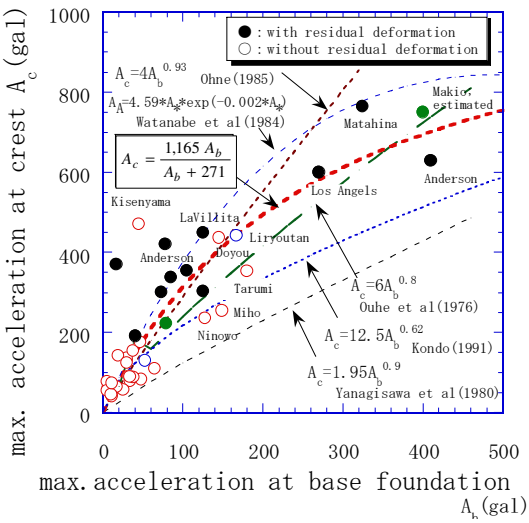

Fig.6 max. acceleration at base and crest for in-situ observation

Fig.7 shows shear strain distribution for D2 (loose Sin wave) and D3 (dense Minowo wave), biggest and least residual deformation of 4 cases. Generally shear strain is expressed as $\gamma = (\partial v/\partial x) + (\partial u/\partial y)$, in this test it can be assumed as $\gamma \cong \partial u/\partial y$ because vertical displacement u is adequately less than horizontal displacement v. So shear strain are calculated by the displacement of colored sand mesh. Fig.7 reveals that sand mesh diminished and rock material deposit beyond 30% of shear strain, big displacement of the embankment is restricted near the surface and the crest, slip circle is not observed, and shear strain depends on the density and loading wave.

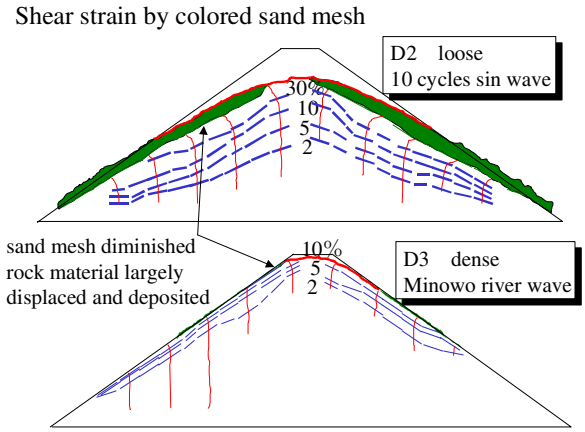

Fig.7 Final residual deformation and shear strain

Fig.8 shows detailed displacement near the crest, and Fig.9 shows the vertical strain along the centerline. The vertical displacement transducer LV installed 5m deep from the crest hardly move except D2 case. Therefore vertical displacement at the crest depends on the strain of the layer upper than 5m from the crest. Also horizontal displacement near the crest happens in the layer upper than 5m from the crest.

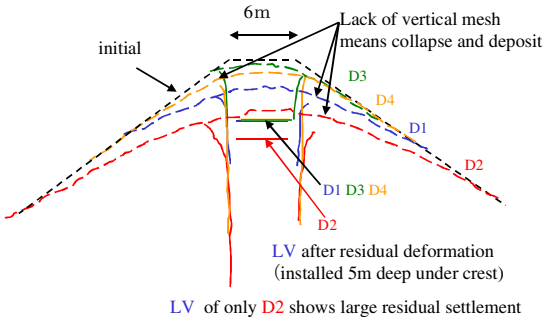

Fig.8 Detail residual deformation near crest

CONSIDERATION ON RESIDUAL DEFORMATION

(1) In-situ observation of existing dams

Type text immediately below subheadings Actual crest settlements have been reported by in-situ observation [6] [13]. For Matahina dam 80 cm settlement was observed at the upstream side top of slope, but the settlement at the downstream side top of the slope was 10.2 cm. The larger settlement is 60 cm for Cogoti, 50 cm for Makio and 32 cm for LaVillita, and many of the settlement are less than 10 cm and more for 50～130m of dam height.

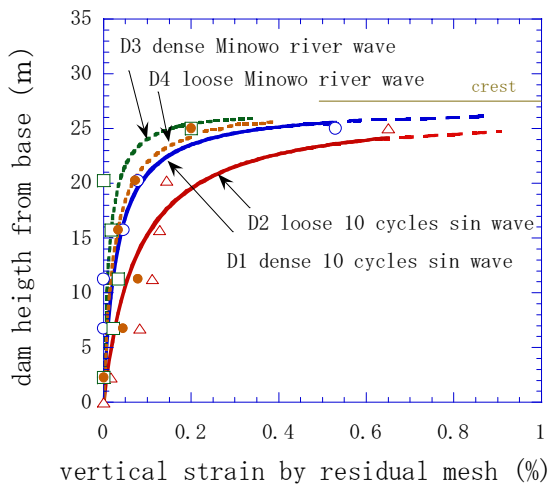

Fig.9 Vertical strain along center axis after shaking

If settlement ratio is defined to crest settlement / dam height, the relationship between settlement ratio and maximum base acceleration is shown in Fig.10. The estimated maximum acceleration value of Cogoti dam is reviewed recently [14]. According to Fig.10, settlement ratio is 0.008 in maximum and is less than 0.005 in many cases. Damaged earthfill dams showed 1.0 m and more even for 10 m height, so settlement ratio exceeded 0.1 in many cases. Because earthquakes often caused liquefaction phenomena in the embankment and / or the foundation of earthfill dams [15]. Therefore settlement ratio of rockfill dam is adequately less than that of earthfill dam.

The relationship between settlement ratio and maximum base acceleration of existing dams is widely distributed and the maximum base acceleration is 50～400 gal for the settlement ratio as described above. Beside all center soil core type RDs have adequate poundage function without leakage increment after earthquake, so the settlement ratios as described above reveals to be adequately small. On the other hand for concrete facing type RD Cogoti and Minase dam caused leakage increment even though settlement ratio was less than 0.005. So detailed dam functional consideration is needed.

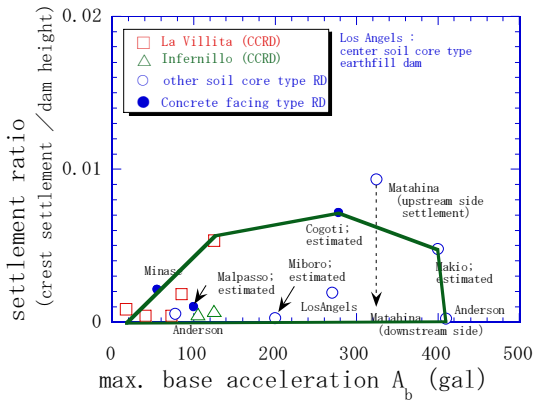

Fig.10 Residual crest settlement and max. base acceleration of in-situ observation

(2) Already done shaking table test

Most of already done shaking table test did not include the measurement of the displacement of rock fill dam, though their main object was both of the response and failure mode. In this study one of the main object is the function evaluation of rock fill dam by the measurement of the displacement. Some of already done shaking table test presented the displacement measurement results and the failure start point by visible observation as shown in Fig.11 [1] [2] [3] [4] [5] [7].

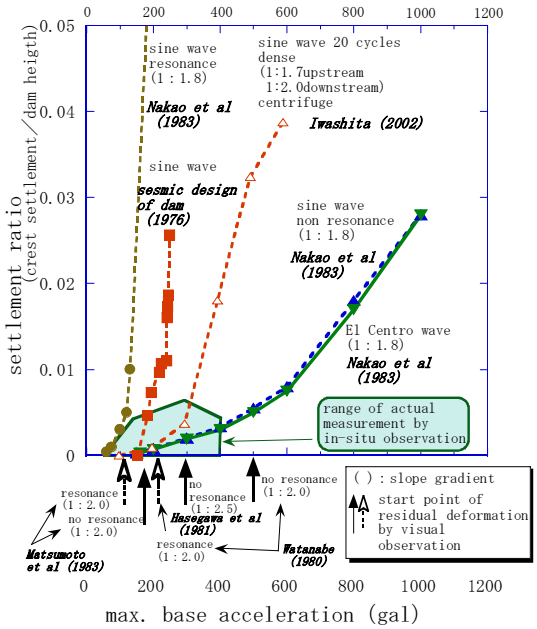

Fig.11 Residual crest settlement and max. base acceleration by existing shaking table test

Watanabe (1980) defined the failure start point as first rolling drop of rock particle by visible observation and other authors defined it as apparent large displacement. Both seem to be same, and they seem to correspond 1 mm settlement (50 mm in actual size) at crest in this test. Due to Fig.11, the failure start point by visible observation is 150～200gal in resonance condition and 200～500gal in no resonance condition by sin wave loading. These tests were conducted under 1G, densities in these tests were not clear, and only Watanabe's test satisfied similarity law

Concerning already done shaking table test with measurement of displacement, Iwashita (2002) based on centrifuge test of center soil core type RD and others tests were under 1G. All of these tests did not indicate density degree of embankment. There are big difference between resonance and non-resonance condition by sin wave loading and the settlement by earthquake wave (El Centro) is close to that by non-resonance loading.

(3) Evaluation of this test

Several items should be discussed before comparison with shaking table tests and in-situ observation results. Firstly the relationship between crest settlement and maximum base acceleration is valid and typical for residual deformation under strong earthquake, so crest settlement ratio is useful for this comparison as a non-dimensional parameter. And 1:1.4 of slope gradient is applied in this test, therefore the effect of slope gradient on the relationship shall be taken into account and this is evaluated in next chapter. Also sin and Minowo river waves are applied in this test, so the effect of input motion shall be taken into account, this is evaluated here. And it is discussed here whether accumulated or incremental settlement obtained in centrifuge test is valid as accurate one.

Fig.12 shows the accumulated crest settlement ratio and maximum base acceleration. After Minowo river wave with 591 gal of maximum base acceleration is loaded in dense condition in Fig.12, additional same loading is applied. The incremental crest settlement by 2nd loading almost agrees with that by 1st loading and the residual deformation mode does not change. Therefore it is recognized that incremental settlement is valid for the relationship. Fig.13 indicates the incremental crest settlement and it found that rigid and dash line can estimate till almost 400-600gal of maximum base acceleration.

In case of sin wave loading, start point of residual deformation is almost 50gal for both of D1 (dense) and D2 (loose) condition and it corresponds to biggest deformation case for actual in-situ observation and shaking table test. The crest settlement of D2 (loose) is almost 2 times as D1 (dense) and is close to biggest deformation case for actual in-situ observation and shaking table test, but does not show rapid increment as resonance condition. On the other hand, in case of Minowo river wave loading, it found that crest settlement ratio of D3 (dense) is close to Nakao's non-resonance sin and El Centro case and smallest case of in-situ observation results. The crest settlement of D4 (loose) is almost 2 times as D3 (dense) and is close to bigger deformation case for actual in-situ observation and to average for shaking table test.

Fig.14 indicates energy of input motion to clarify the position of sin wave comparing with actual earthquake. According to Fig.1.2Hz sin wave has biggest energy of strong earthquake and Minowo river wave has smaller energy comparing with magnitude. Therefore it can be recognized that in case of same maximum acceleration 1.8Hz sin wave loading shows almost upper biggest residual deformation and Minowo wave loading shows almost lower smallest residual deformation.

EVALUATION OF ROCKFILL DAM FUNCTION FROM VIEWPOINT OF CREST RESIDUAL SETTLEMENT

If slip circle slice method is applied in seismic design of rockfill dam, static horizontal seismic coefficient has large effect on cross section design i.e. slope gradient. However dynamic behavior of rockfill dam shall be clarified taking into account of dam damage

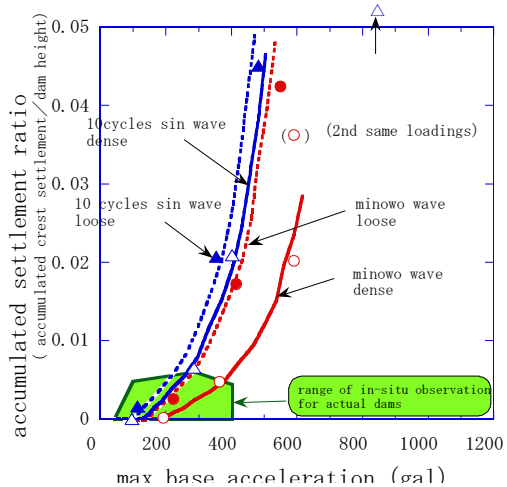

Fig.12 Accumulated crest settlement and max. base acceleration by this centrifuge test

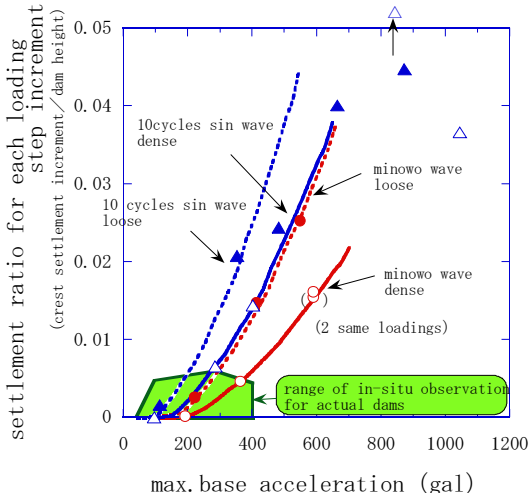

Fig.13 Incremental crest settlement and max. base acceleration by this centrifuge test

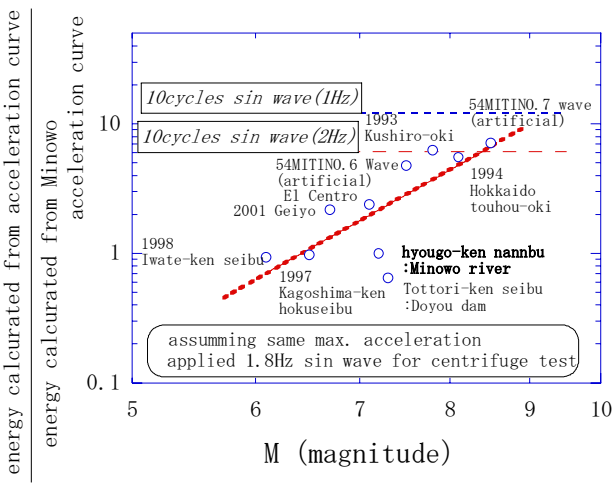

Fig.14 Energy comparison of earthquake and sin wave

by earthquake, residual deformation, dam function of impounding and so on.

(1) Effect of slope gradient on crest settlement

Fig.15 and 16 are results considering the effect of slope gradient on crest settlement using Fig.10 and 14. If 1.8Hz sin-wave loading shows almost upper biggest residual deformation and Minowo wave loading shows almost lower smallest residual deformation, the relationship between crest settlement ratio and max. base acceleration by this centrifuge test with 1.4 of slope gradient is in good harmony with that by in-situ observation.

As above –mentioned, 0.1-1.0% is critical crest settlement ratio in which impervious zone has some damage but not big enough to be able to repair the dam. Due to Fig.15 and 16, it found that crest settlement of dense dam is adequately small even if slope gradient is 1:1.4 in case of 200 gal of max. base acceleration, and crest settlement ratio sometimes exceed 0.1-1.0% even if adequate compaction in case of 400 gal of max. base acceleration.

(2) stability taking account of impounding

Freeboard has been often applied for recent evaluation of impounding function by seismic analysis of fill dam in USA [7] [8]. It means that if calculated crest settlement does not exceed freeboard i.e. embankment height upper than H.W.L., overflow will not cause and the dam will keep impounding function. Freeboard of Japanese already constructed dams is shown in Fig.17 [19]. The minimum freeboard is considered to be 3m, and it leads that critical crest settlement ratio of this test model is 0.0109 (=3/27.5) and 800 gal of maximum base acceleration is needed for this test model to cause its ratio even for sin loading and loose condition.

Leakage increment after earthquake has not been reported for all of soil core type rockfill dams as shown in Fig.10. If allowable settlement assumes to be 50 cm which is almost maximum crest settlement of already constructed dams and by observed earthquakes as shown in Fig.10, it means that impounding function is kept and severe damage is not caused for even 50 cm settlement. In this centrifuge model, 50 cm settlement corresponds to 0.018 of crest settlement ratio, the model would be able to tolerate severe earthquake if compaction is adequate.

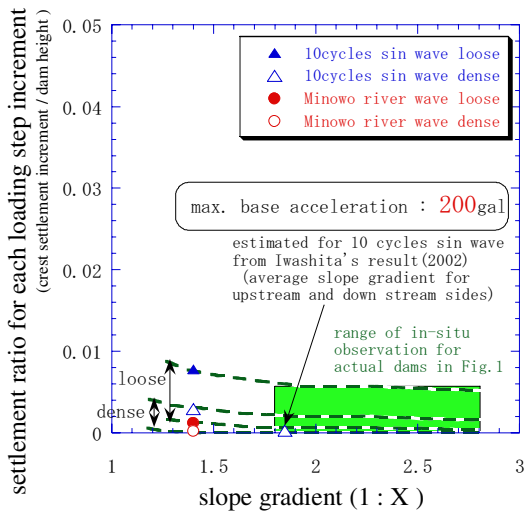

Fig.15 Effect of slope gradient on residual settlement (max. base acceleration 200gal)

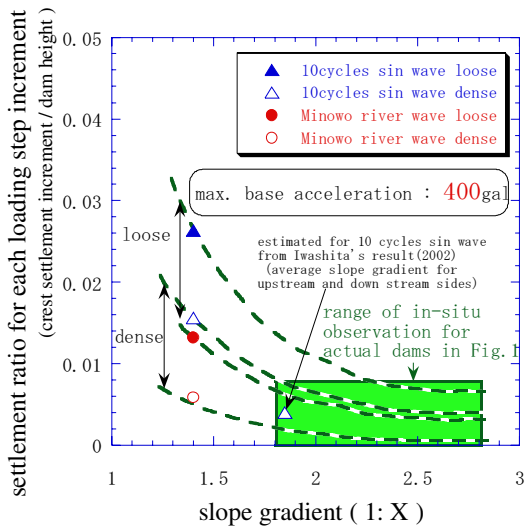

Fig.16 Effect of slope gradient on residual settlement (max. base acceleration 400gal)

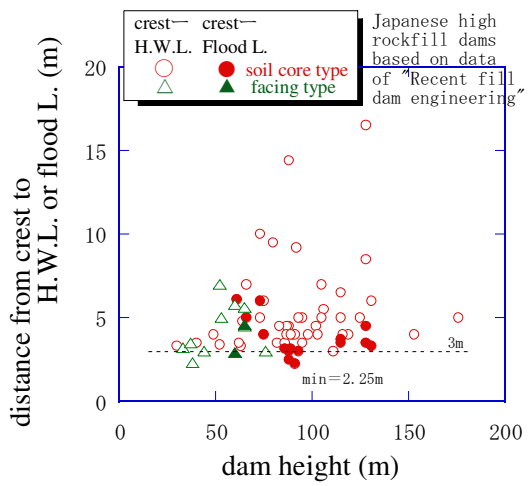

Fig.17 Freeboard of Japanese existing dams

Only examples of Malpasso, Cogoti and Minase are reported for Concrete Facing RD and Cogoti and Minase [6] were repaired for some leakage increment after earthquake. Crest settlement ratios are 0.00097, 0.0072 and 0.0021 for Malpasso, Cogoti and Minase. If critical crest settlement ratio is 0.001-0.008, it leads that it would cause leakage increment and damage of facing but it is not severe damage and it would be able to be repaired. And also it corresponds that maximum base acceleration is 200-400gal for some condition of adequate compaction, slower slope gradient and smaller magnitude.

Table 1 Criteria of Residual Settlement

Damage or stable condition	Allowable level	Allowable quantity	Allowable Settlement or ratio for 50～200m of dam height
Overflow	Freeboard	> Almost minimum is 3m due to each dam	> Almost 6～1.5%
Stable for facing typeBut leakage increment	Almost max. observed settlement or settlement ratio	Almost 0.5m	1.0%～0.25%
		Almost 0.8%	0.4～1.6m
Stable for center soil core typeAnd no leakage increment	Almost max. observed settlement or settlement ratio	Almost 0.5m	1.0%～0.25%
		Almost 0.8%	0.4～1.6m

CONCLUSION

Centrifuge test result of rockfill dam is compared with already done shaking table test and in-situ observation of existing dam. It is concluded as follows. Density of embankment affects response acceleration and response magnification is bigger within 50m deep from surface. Relationship between crest settlement and maximum base acceleration in this test almost agrees with that of in-situ observation for existing dams and already done shaking table test, and it is affected by slope gradient and density of embankment. Beside allowable and critical settlement ratio is discussed for soil core and facing type RD.

REFFERENCES

1. Watanabe H. “Dynamic property of fill dam by shaking table model test, Tsuchi-to-kiso, 1980, No.5, pp.25-32
2. Matsumoto N, Toyoda M, Shiga M. “Shaking table model test of rockfill dam” Transaction of civil engineering, 1983, vol.25-6, pp.45-50
3. Nakao S, Takezawa Y. “Study of response behavior of rockfill dam under earthquake” Report of general laboratory of Kansai Electric Power Company, 1983, vol.26
4. Ohne Y, Tatebe H, Narita K, Okumura T. “Basically study of seismic design of fill dam” proc. of Japanese society of civil engineering, 1983, vol.339, pp.127-136
5. Fill dam section of civil engineering laboratory of Japanese construction ministry. “Report of developing project examination of rivers – seismic design of dam” 1976
6. Okamoto,T. “Evaluation of measured vertical and horizontal residual deformation at crest of rockfill dam under earthquake” IS-Shikoku 99, International Symp. on Slope Stability Engineering : Geotechnical and Geoenvironmental Aspects, 1999, Nov. 8-11, pp.631-636
7. Bureau G, Volpe RL, Roth WH and Udaka T. “Seismic analysis of concrete face rockfill dams” proc. of the symposium on concrete face rockfill dams, annual convention ASCE, 1985, pp.479-508
8. Iwashita T. “Centrifuge test of rockfill dam and analysis by dynamic elasto-plastic effective stress program” dam engineering, 2002, No.184, pp.32-42
9. Uchita, Y., S.Tsuruta, Y.Hoshino, H.Yamaguchi, K.Sato and T.Matsuda “Evaluation of deformation property of rockfill dam centrifuge shaking test” 57th annual meeting of Japanese civil engineering society, T-768, 2002
10. Society of Electric Power Engineering “Recent fill dam engineering” 1970
11. Okamoto.T. “Evaluation of in-situ strength of rock material by in-situ compaction and laboratory tri-axial test” proc. of 47th symposium of geotechnical engineering, Japanese society of geotechnical engineering, pp.301-308, 2002

12. Okamoto.T. H.Tochigi and T.Nishiuti, “Dynamic deformation and damping characteristics of fresh andesite rock material by large cyclic triaxial test” – effect of stress condition at slope and evaluation of stress induced anisotropy, CRIEPI report U01027, 2001
13. Okamoto.T. “Recent trend for observed residual settlement of existing rockfill dams by earthquake and some consideration effecting factors on them” proc. of 46th symposium of geotechnical engineering, Japanese society of geotechnical engineering, pp.65-70, 2001
14. Noguera.G.L. and O. Espinoza B. “Seismic history of Cogoti dam” *Symp. Of Rehabilitation of dams*, pp.213-222, 1998
15. Tani.S. “earthquake damages of existing earth fill dam and evaluation of safety” Japanese society of large dam, No.140, pp.32-50, 1992
16. Ohne Y. “seismic behavior of Makio dam” 20th of symposium of soil engineering, special session, pp.47-54, 1985
17. Makdisi,F.I., A.T.Roodsair, M.Nmadianos and Z.L.Wang “Evaluation of seismic stability of Seven Oaks Dam” *Seismic Design and Performance of Dams*, USCOLD, pp.111-125,1996
18. Beikae, M., M.Luebbers, L.Barneich and D.Osmun “Seismic deformation analysis of Eastside Reservoir Dam” *Seismic Design and Performance of Dams*, USCOLD, pp.315-331,1996

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