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Review of the Seismic Retrofitting of Sardis Dam

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ABSTRACT

Most of the dams in Central United States, built in earlier part of 20th century are hydraulic fill dams. At the time of construction of these dams, earthquake engineering was still in its infancy and hence the dams lacked proper seismic design. Failure or near failure of earth dams as a consequence of earthquakes forced the United States Army Corps of Engineers (USACE) to re-evaluate the seismic performance of major earth dams in the Central United States, one of these dams was Sardis Dam.

Investigations of Sardis Dam revealed a potentially liquefiable silty clay layer under the upstream shell. It was determined that an earthquake of intensity 7.5 on the Richter scale can cause liquefaction of this weak layer leading to excessive deformations of the dam. Pre-stressed concrete piles were driven to give additional shear resistance to the weak layer. This paper presents a revision of the rehabilitation work at Sardis Dam and a prediction of the performance of rehabilitated dam using limit equilibrium methods.

The study verifies the results of deformation analysis carried out using Finite element procedures (TARA-3FL). It is shown that deformation analysis is a useful tool that can be employed reliably to predict the deformations of an embankment dam. The study also shows that the used reinforced zone was very conservative with a factor of safety greater than 2 with negligible seismic induced displacement under the most unfavorable conditions. Therefore it was possible to make the dam safe using less piles and smaller reinforced width which will highly reduce the cost of the remediation work.

INTRODUCTION

Damage to Dry Canyon Dam (45 miles from epicenter) and South Haiwee Dam (95 miles from epicenter) during 1952 Kern County earthquake raised concerns about the seismic stability of hydraulic fill dams (Babbitt, 1993). The near failure of Lower San Fernando dam and displacement of Upper San Fernando dam during 1971 earthquake further confirmed the potential failure of hydraulic fill dams due to earthquake induced vibrations. The lower San Fernando dam nearly failed as a result of an earthquake of magnitude 6.6 on the Richter scale and forced evacuation of 80,000 people from the area (Fin et.al, 1998). The lack of proper seismic design of hydraulic fill dams built in the earlier part of 20th century; and consequent large deformations associated with earthquakes which were observed in many hydraulic fill earth dams forced United States Army Corps of Engineers (USACE) to re-evaluate the seismic stability of all of its embankments dams. Sardis Dam was one of the dams which were investigated for its seismic stability. The investigations revealed presence of a weak layer, susceptible to liquefaction, beneath the upstream shell. The seismic rehabilitation using driven piles was carried out to safeguard against the potential liquefaction hazard in event of an earthquake of magnitude 7.5 on the Richter scale. The remediation measures taken at Sardis Dam to safeguard against the potential failure due to liquefaction are reviewed and the results of deformation analysis are compared with those of limit equilibrium method in this paper.

DAM LOCATION, DESCRIPTION AND FOUNDATION CONDITIONS

Sardis dam is located in Northwestern Mississippi approximately nine miles from the town of Sardis on the Little Tallahatchie River as shown in Fig. 1. The dam is located in the New Madrid Seismic Zone which has a history of some significantly large earthquakes. Sardis is one of the four major dams (Arkabutla, Enid and Grenada being three others) which were built in Yazoo Basin mainly to control flood and to provide opportunities for recreation and improvement of the local navigation in the area.

Sardis Dam is a hydraulic fill embankment dam constructed by U.S. Army Corps of Engineers (USACE) in the late 1930s and was placed in service in 1940. The foundation soil consists of a zone of natural clayey silt, 3 m to 6 m thick, called the top stratum clay. The top stratum clay is underlain by pervious dense alluvial sands (substratum sands) approximately 12 m thick, which in turn are underlain by Tertiary silts and clays (Finn et al. 1998). Vertical cross section of foundation soils and longitudinal distribution of liquefiable zones are respectively shown in Figs. 2 and 3.

Fig. 1: Location of Sardis Dam (Hall, 2001)

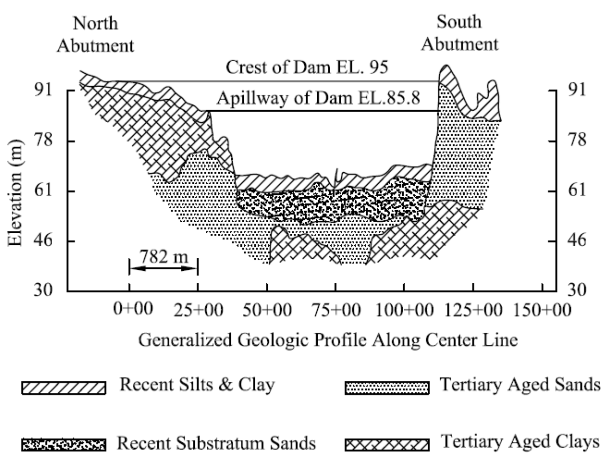

Fig. 2: Vertical profile of foundation soils (reproduced from Finn et al. 1998)

The dam was constructed in three sections with clay dikes in between the sections (Finn et.al, 1998). During dam construction the top stratum clay was removed from the downstream portion to help in seepage control. The overall length of dam is approximately 4600 meters. The central portion of dam, which was constructed by hydraulic filling, has a length of about 2620 meters with a height varying between 28 to 36 meters. The dam has a clayey silt core flanked by sand shells on both sides. 12 meter wide Crest of the dam consists of a rolled clay cap. The upstream slope varies between 1:15 at bottom to 1:2.75 at the top. The gradient of downstream slope is 1:7 at bottom and 1:3.25 at the top (Fig. 4). The top stratum clay was considered sufficient to control the seepage; however, this layer was removed beneath the downstream shell to reduce the uplift pressure.

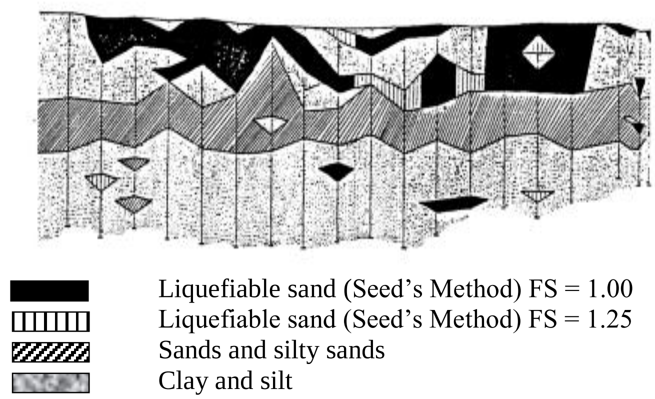

Fig. 3: Longitudinal distribution of liquefiable zones (Wu, 1992)

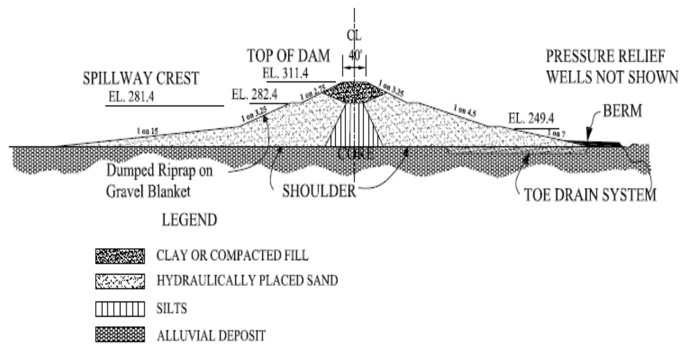

Fig. 4: Typical section of Sardis Dam (reproduced from Llopis, 1995)

INVESTIGATIONS

Several studies were undertaken by The U.S. Army Engineer District, Vicksburg, to evaluate the behavior of the dam during and after the earthquake. The first step in the evaluation was to determine the suitable seismic hazard criterion for the dam. Two records of the 1952 Kern county earthquake in California were modified and taken as the design earthquake for the Sardis dam (Finn et.al, 1998; Sharp & Matheu, 2005). The main consideration for selecting these earthquakes was distances from the epicenter. It was determined that an earthquake of magnitude 7.5, with a duration of 15 seconds, can produce an acceleration of 0.2g in the dam. This acceleration will produce a peak velocity of 35-40 cm\sec and can cause liquefaction in the weak layer that in turn can cause the failure.

The investigations were performed using Standard penetration test (SPT) and Field vane shear tests (FVST). Laboratory tests including classification tests, static and cyclic triaxial tests were also conducted on undisturbed samples from the dam itself and the foundation soils (Finn et.al, 1998). Field and laboratory testing and seismic stability analyses indicated that the downstream slope would remain stable during shaking; however, a significant strength loss or liquefaction could occur to threaten the upstream stability. The Liquefaction may occur in (*a*) the hydraulically placed silty core, (*b*) the upper 3 m to 9 m of sand shell along the lower portion of the upstream slope, and (*c*) a discontinuous layer (1.5 m to 4.5 m thick) of clayey silt located in the top stratum clay as shown by red color in Fig. 5 (Finn et al. 1998, Hall, 2001). The deformation analyses showed that the deformations in the silt core as well as in the upstream shell will remain within the tolerable limits, however, the silty clay layer was identified as the critical layer that could result in excessive deformations in the dam as a result of liquefaction. Initially, the extent of the weak layer was considered to be 310 meters along the upstream of dam, however, additional explorations revealed other pockets of weak material which increased the overall length of remediation to 823m (Sharp & Matheu, 2005).

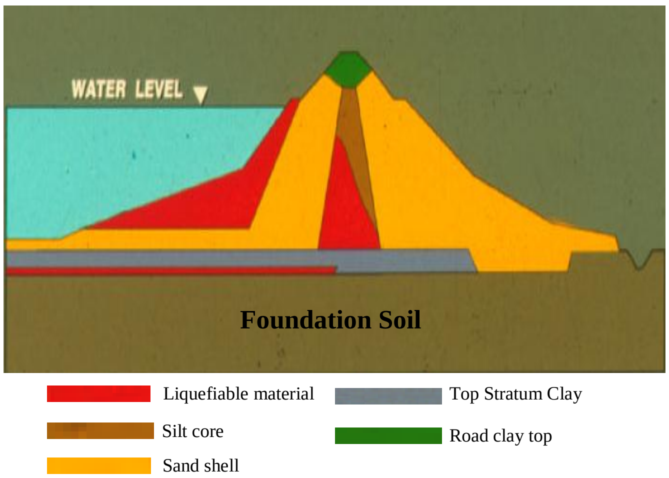

Fig.5: Section of Sardis dam showing location for potential liquefaction (Hall, 2001)

The SPT profile along the upstream of the dam is shown in Fig. 6, from which it can be seen that the 'N' values from most of the borings are less than 5 at an elevation of 200ft. These low N values indicate the presence of a weak layer at an elevation of 200ft. The N value for SES-59-85 between elevations of 220 and 230ft is also less than 5; however the other borings at the same depth indicate much higher N values and thus explains why deformation of this zone was estimated to be within the tolerance limits of the dam.

The peak and residual in situ strengths of weak silty clay were estimated using field vane test, the location of these tests are shown in Fig. 7. These tests and the limit equilibrium analyses for a factor of safety of 1.5 to 2 suggest an average peak S_u/p' ratio of about 0.2 to 0.3 for the weak layer and a residual S_u/p' of 0.075 (Finn et.al, 1998).

In order to ascertain the liquefaction potential of the dam material Chinese criteria developed by Wang (1979) was used (Finn, et.al, 1998). According to this criterion; a fine grained soils meeting following criterion are susceptible to liquefaction:

- Percent finer than 0.005mm $\leq$ 20%
- Liquid limit, LL $\leq$ 35%
- Natural water content $\geq$ 0.9 LL

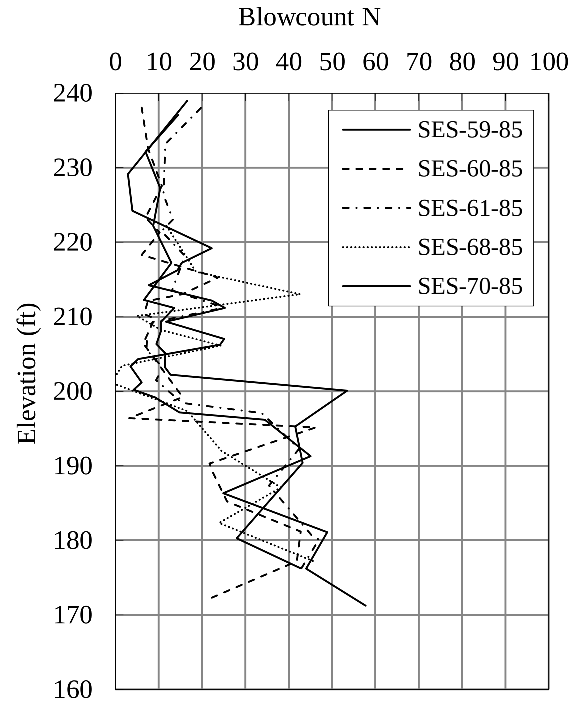

Fig. 6: SPT profile along the upstream shell (reproduced from Salah et.al, 1993).

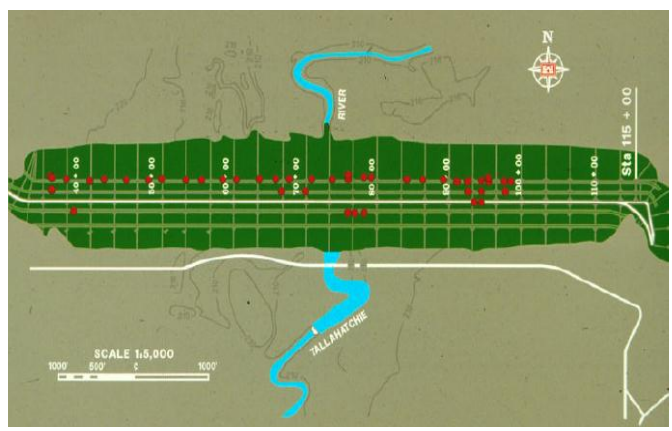

Fig. 7: Locations for Field vane shear test (Hall, 2001)

The criterion was initially applied as such; however, on recommendations on consultants, owing to differences between the testing procedures and equipment between Chinese and US practices, following modifications were applied to Chinese criterion and applied strictly (Finn et al. (1998, Wu 1992).

- Percent finer than 0.005mm $\leq$ 15% (decreased by 5%)
- Liquid limit, LL $\leq$ 36% (increased by 1%)
- Increase the natural moisture content by 2%

In addition to above, soils with SPT N value $\leq$ 4 was also assumed to liquefy if sufficient data were not available to apply the Chinese criteria.

Computer program, TARA-3FL was used to analyze the post liquefaction behavior of Sardis dam. The program resulted in a factor of safety that varies from 1.15 to 0.8 as the residual strength is reduced from 30 to 10 kPa. The minimum strength of weak layer was estimated to be 17.5kPa which corresponds to a factor of safety of 0.75 (Finn et al, 1998). The deformation analysis predicted a complete collapse of core of the dam and a loss of free board in excess of 1.5m as a result of liquefaction of silty clay layer. The estimated deformed shape of dam after shaking (as predicted by TARA-3FL program) is shown in Fig. 8.

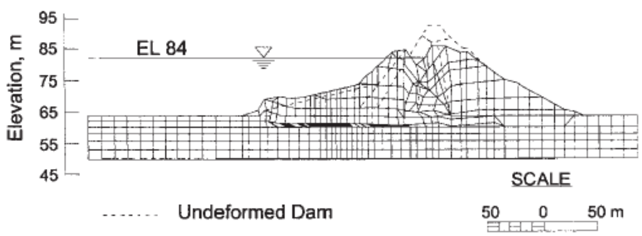

Fig. 8: Estimated deformed shape of Sardis Dam (Finn 1998)

REMEDIATION

Babbitt (1993) reported 132 cases where the dam embankments were improved using different types of techniques. In most cases, either a permanent component was added or replaced, or the performance of dam was improved by imposing storage restrictions. Only nine out of a hundred and thirty two cases dealt with improving the foundation conditions. For Sardis dam also, a number of options were considered which included jet grouting, vibro-grouted stone columns, vibro-grouted concrete columns, driven pre-stressed concrete piles and deep soil mixing (Finn et al. 1998; Sharp and Matheu, 2005). The methods were compared considering following criterion (Finn et al. 1998):

- Operation of the reservoir
- Reliability and verification of improvement method
- Experience of available contractors
- Impact on environment
- Cost of improvement techniques

Improvement options involving grouting were found expensive as well as having an adverse affect on the reservoir operation. Analysis showed that pre-stressed concrete piles asible solution for the remediation. To verify the results, piles were driven in a test section and ground improvement after driving of piles was evaluated by using SPT, CPT, and cross-hole shear wave velocity tests. The drivability of piles and improvement effects were verified by post-driving investigations. A total of 2594 pre-stressed heavily reinforced concrete piles, with a cross sectional area of $0.6m^2$, were driven into a 90m wide and 823m long area (Sharp 2005). The function of the pile group was to form a shear plug that will transfer the load resulting from the movement of upstream embankment to the substratum sand. The lengths of piles varied between 16 to 20m with an embedment depth of 4.6m into the dense substratum sands. The piles were driven from a floating platform without lowering the reservoir. Fig. 9 presents the sectional view of piles used for remediation.

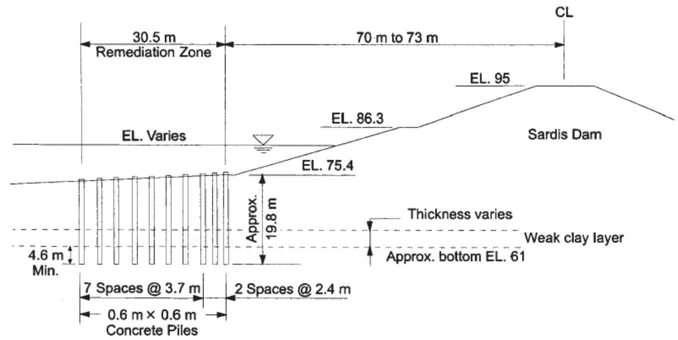

Fig. 9: Section of Sardis Dam with reinforced zone (Finn 1998)

STABILITY ANALYSIS

Before Remediation

A parametric study for the stability analysis of Sardis dam was performed using Geoslope (2004). The soil properties used for the study are as shown in Table 1 below and Morgenstern and Price method was used for calculating the factor of safety.

In the first stage of analysis, FS was calculated before any remediation. The analysis was performed by varying the liquefied strength ratio (Su/p') of the weak layer from 0.3 (average) to 0.075 (residual). The properties of all other materials were kept constant. A critical slip surface passing through the weak layer was determined by performing a number of trials.

Table 1: Material properties of Sardis Dam (after Finn et.al, 1998)

Material Type	γ (KN/m^3)	Φ (deg.)	c (kPa)
Rolled clay cap	18.0	0	-
Silt core			
• Residual	18.9	0	5
• Unsaturated	18.9	20	15
Sand shell			
• Saturated	19.7	35	0
• Unsaturated	19.7	35	0
• Residual	19.7	0	0
Top clay stratum	18.9	0	100
Weak layer	18.9	0	0.075p'

Figure 10 shows the dam cross section that was used in the analysis.

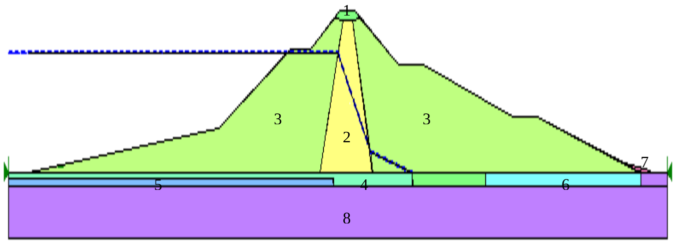

Fig. 10: Dam cross section used for the analysis

The variation in FS with change in Su/p' is shown in Fig. 11. Assuming an average effective overburden pressure of 150kPa over the improved zone, residual strength reported by Finn et.al (1998) was expressed in terms of liquefied strength ratio and incorporated in Fig. 11.

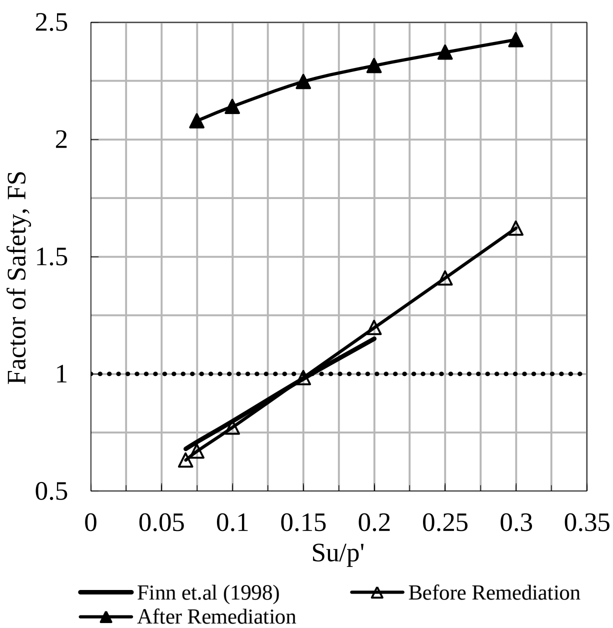

Fig. 11: Variation in FS with liquefied strength ratio

<u>After Remediation</u>

For post earthquake analysis, a 90m wide reinforced zone extending from upstream slope to well below the weak layer was added to the dam. The reinforced zone was modeled as a dense material having an angle of internal friction of at least 35°. This is a conservative value as in addition to improving the shear resistance in the weak layer, the pile driving also results in densification and thus helps in improving the properties of the weak layer. The critical failure surface was moved to maximize its length in the weak layer. Location of reinforced zone is shown in Fig. 12 All the materials were assigned their residual strengths as mentioned in Table 1.

The analysis resulted in a minimum factor of safety of 2.08 which increase with the increase in angle of internal friction and the liquefied strength ratio which is also shown in Fig. 13.

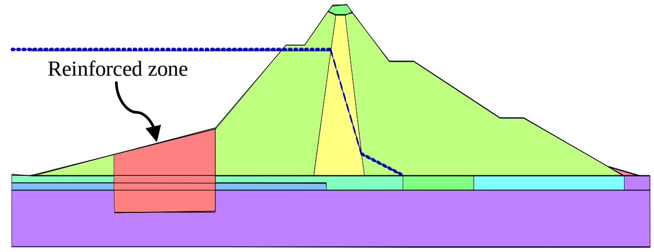

Fig. 12: Location of Reinforced zone

That high factor of safety after remediation led to another question which needs to be investigated; what the factor of safety would be if smaller width of that reinforced zone was used? To answer this question another analyses were performed using the same previous assumptions but with smaller length of that reinforced zone. Fig. 13 shows the vibration in FS with the change in reinforcement length. Assuming average shear strength of about 200kPa in a reinforced zone of about 36m long resulted in a factor of safety of about 1.15 reported by Finn et.al (1998) and also shown in the Fig. 13.

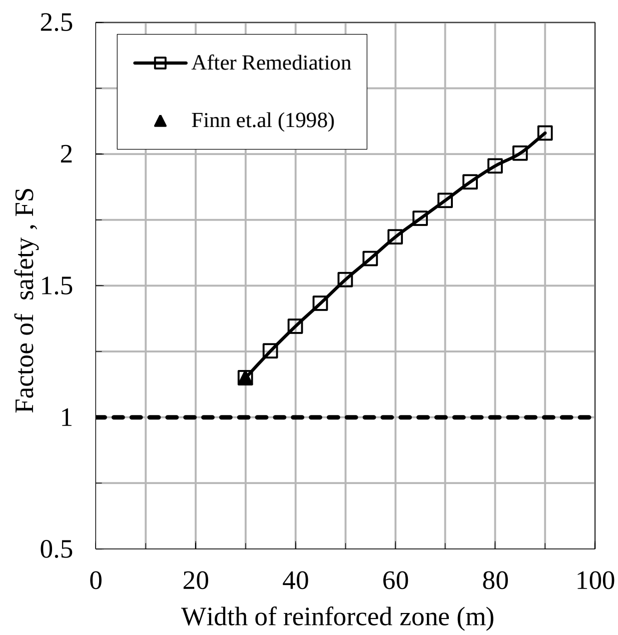

Fig. 13: Variation in FS with reinforced zone width

The analysis showed that it was possible to achieve FS of 1.5 using 40m reinforced zone width instead of 90m, and to achieve FS of 1.15 using 30m width which is the FS that can be used in the case of liquefaction.

Crosshole shear wave velocity test were performed before and after the remediation to ascertain the improvement achieved as a result of rehabilitation. The results are shown in Fig. 14. It can be seen from Fig. 14 that there is a noticeable increase in the shear wave velocity in almost all the layers except in the dense substratum sands. The average increase in the shear wave velocity was around 20% whereas the increase in weak layer was about 25% (Llopis & Ballard, 1995).

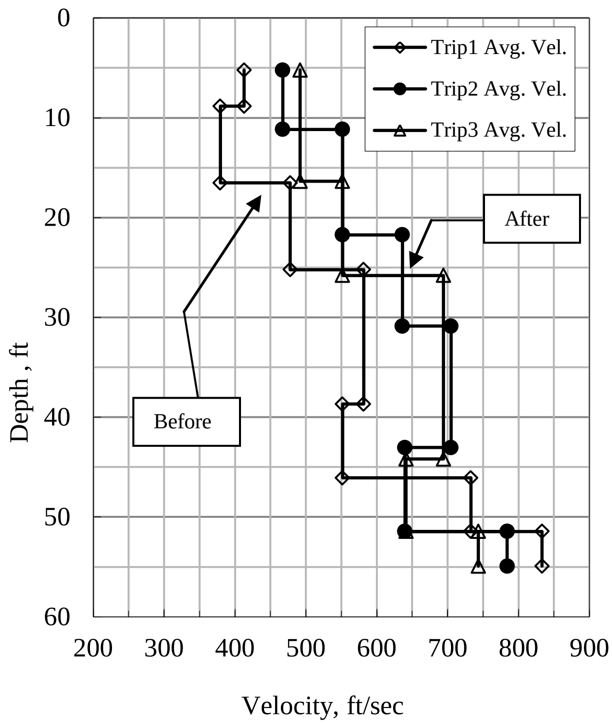

Fig. 14: Shear wave velocity profile before and after rehabilitation (reproduced from Llopis & Ballard, 1995)

Permanent Deformations

Since the serviceability of slope after an earthquake is controlled by its deformations, it was important to study the permanent deformations before and after the remediation work sing Makdisi and Seed (1978) analytical method.

The following procedure was followed to study the permanent deformations:

1. Pseudostatic analysis was performed using 80% of the silt core strength and the residual strength for the weak layer (Makdisi and Seed, 1978). This analysis was carried out without including the reinforced zone. From this analysis it was found that the yield acceleration (*ky*) for the dam before remediation is about 0.03g.
2. The same analysis was performed after remediation using 90m and 50 m width for the reinforced zone and that resulted in yield accelerations of about 0.2g and 0.1g respectively
3. Using Fig. 15, the maximum acceleration ratio was determined, and then the maximum acceleration was found to be 0.16g.
4. The seismically induced displacement (*U*), were determined using Makdisi and Seed (1978) displacement chart (see Fig. 16). From the chart it was found that an earthquake of intensity 7.5 on the Richter scale can cause 160 cm permanent deformations before having the remediation work. While using 90m width reinforced zone resulted in yield acceleration greater than the maximum acceleration which means that there will be negligible deformations after remediation. And even using 50m width of reinforced zone also resulted in a noticeable drop in the magnitude of permanent deformations which is found to be less than 7cm and that reflects the efficiency of the used remediation and in the same time how much conservative was the width of that reinforced zone.

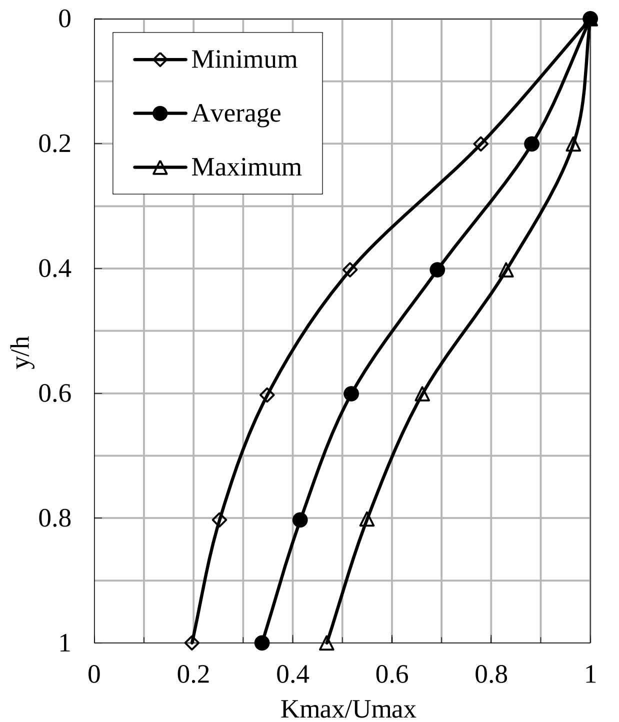

Fig. 15: Variation of maximum acceleration ratio wity depth of sliding mass (reproduced from Makdisi and Seed, 1978)

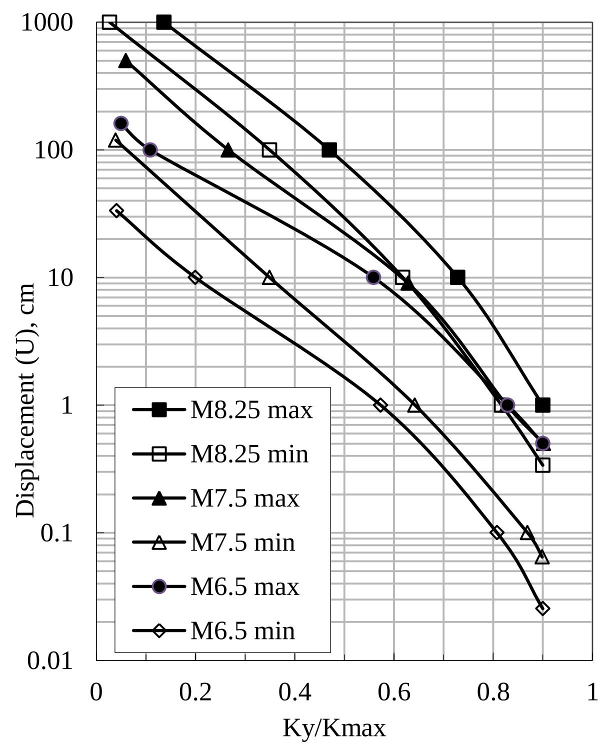

Fig. 16: Variation of permanent displacement with yield acceleration (reproduced from Makdisi and Seed, 1978)

CONCLUSIONS

The use of driven piles for liquefaction remediation is a practically viable and economically feasible solution. This method helped in applying the required remediation without lowering the reservoir. Pile driving potentially increases the density of both dam and foundation materials as indicated by crosshole test results, this also justifies the modeling of reinforced zone as a dense material having a high angle of internal friction. The study verifies the results of deformation analysis carried out using Finite element procedures (TARA-3FL). It is shown that deformation analysis is a useful tool that can be employed reliably to predict the deformations of an embankment dam.

The study also shows that the used reinforced zone was very conservative; this was proven from both the post liquefaction factor of safety and the permanent deformations. Therefore, it was possible to make the dam safe using smaller width which will highly reduced the cost of remediation and keep the dam safe.

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