

INTERNATIONAL SOCIETY FOR SOIL MECHANICS AND GEOTECHNICAL ENGINEERING



This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

<https://www.issmge.org/publications/online-library>

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Compression of alluvial deposits due to wetting

Compression des alluvions après immersion

I. ORDEMIR, Professor of Civil Engineering, Middle East Technical University, Ankara, Turkey
 Y. ÖZKAN, Associate Professor of Civil Engineering, Middle East Technical University, Ankara, Turkey

SYNOPSIS Settlement of soils due to wetting with or without additional loading is a widespread phenomenon and it is not confined to a particular soil type. Settlement due to wetting which is generally termed "collapse" usually occurs in loose granular materials such as rockfills, compacted sand-gravel fills, or natural loess, silt, sand or gravel deposits at low water content. Various types of clays may also experience compression under load when wetted provided that certain initial conditions are satisfied. The case study presented in this paper aims to illustrate the behaviour of dry alluvial deposits upon reservoir filling.

INTRODUCTION

An actual case of compression of thick natural dry alluvial deposits due to wetting was investigated by the authors. The aim of the investigation and the experimental study was to understand the behavior of dry alluvial deposits on the reservoir area of Karakaya Dam being constructed on Euphrates River which flows through southeast Turkey. It was decided to construct a new railroad bridge at the reservoir area of the dam to meet the new requirements of the maximum water level of the reservoir lake. Studies conducted made it necessary to design and construct a viaduct 2030 m long, with two abutments and twenty eight towers spaced 70 m center to center. Some towers as tall as 65 m above the base are to be constructed at the locations where the existing alluvial deposit is very thick and the groundwater at great depth. Considering the potential settlement of the dry alluvion upon reservoir filling the following alternative design solutions were studied:

- a) Construction of bridge towers directly on dry alluvion by shallow foundations,
- b) Construction of towers on foundations supported by piles constructed in dry alluvion,
- c) Construction of towers on foundations supported by piles whose tips extend below original groundwater,
- d) Replacement of viaduct at the deep dry alluvion zone with a high rockfill embankment.

In order to decide on one of the alternatives mentioned above, the compression behavior of the dry alluvion due to wetting upon reservoir filling when the dam under construction is in service will have to be investigated. For this purpose, field and laboratory tests were carried out in order to predict the percent compression of the alluvion. After studying the behavior of many earth and rockfill dams, and conducting one dimensional compression tests

it was concluded that various factors may be the cause of the compression induced by wetting (Nobari and Duncan, 1972). Initial conditions such as initial water content and initial dry density, amount of fines, uniformity of gradation, maximum particle size, type of aggregate and type of loading may be listed among such factors. At Karakaya Viaduct construction site, the initial water content and dry density, and gradation of the dry deposit are believed to be the main factors affecting compression due to wetting.

SOILS OF THE CONSTRUCTION SITE

The study was made at two different but typical locations at the site, namely the tower No.12 and No.22 locations. As it can be seen in Figures 1 and 2 that very thick medium to very dense alluvial deposits cover the construction area. The thickness of the alluvion varies between 10 m below the riverbed and more than 70 m at the left bank of the river. The groundwater table ranges from low to very low, consequently the upper part of the alluvion is dry. The thickness of the dry alluvion increases at the tower locations near the left bank of the valley where the groundwater is as deep as 25 m from the present ground surface. The Standard Penetration Tests have been made where soil conditions permitted and the pressuremeter tests have been carried out in alluvion using Menard GA type pressuremeter and 77 mm and 58 mm diameter probes. SPT and pressuremeter tests revealed that the alluvion was in a medium to very dense condition. The relationship between the Pressuremeter Modulus E_M and the Limit Pressure p_1 of the alluvions as shown in Figure 3 is an indication of dense to very dense granular material. Figure 3 has been prepared using the results of the pressuremeter tests made in borehole at maximum 100 m

Depth (m)	Soil	Symbol	SPT	P _l (kpa)	E _M (kpa)
5	Sand With Gravel	SP-SM	21		
			20	1150	11900
			47	1700	18900
			32	2250	29600
10	Silt and Sand	ML-CL		1000	29200
			33		
15	Gravel With Sand	GP-SP	28	550	8300
			>50	4200	82300
			30	1800	10200
20	Gravel With Sand	GP-SP		2200	21500
				1600	31800
				540	10900
25	Bedrock			500	10800

Fig.1. Soil Profile and in-situ Test Results at No.12 Tower Location

Depth (m)	Soil	Symbol	SPT	P _l (kPa)	E _M (kPa)
5	Very Dense Sand and Gravel	SP-GP	>50	3800	53200
			>50	2200	29100
			>50	4600	75800
			>50	5000	83200
			>50	4800	78200
10	Sand and Gravel	SP-GP	>50	5500	94300
			>50	5600	100400
			>50	7400	148300
20	Sand and Gravel with silt and clay bands	SM-CL-GC	>50	8600	176600
			>50	11000	220400
			>50		
			>50		

Fig.2. Soil Profile and in-situ Test Results at No.22 Tower Location

distance from the center of No.12 and No.22 towers.

The typical viaduct tower which is shown schematically in Figure 4 was planned as a reinforced concrete box shaped vertical shaft resting on a reinforced concrete footing pad which in turns will be supported by soil if predicted future settlements following reservoir filling are acceptably small or by large diameter piles if settlements are excessive.

FIELD TESTS

In addition to SPT and pressuremeter tests made in boreholes, three types of plate loading tests were carried out at 3,5 m depth at No.12 and No.22 tower locations. The first type of test was carried out on natural or dry soil. The second type of test was made loading in dry condition up to 300 kN/m² and continuing after flooding at 300 kN/m². The third type of test was run by flooding the soil at the beginning of the test. Foundation soil in all tests was loaded by using a circular plate of 750 mm diameter. Loads were applied by a heavy duty hydraulic jack and settlement readings were taken using 0.001 mm accurate dial gauges.

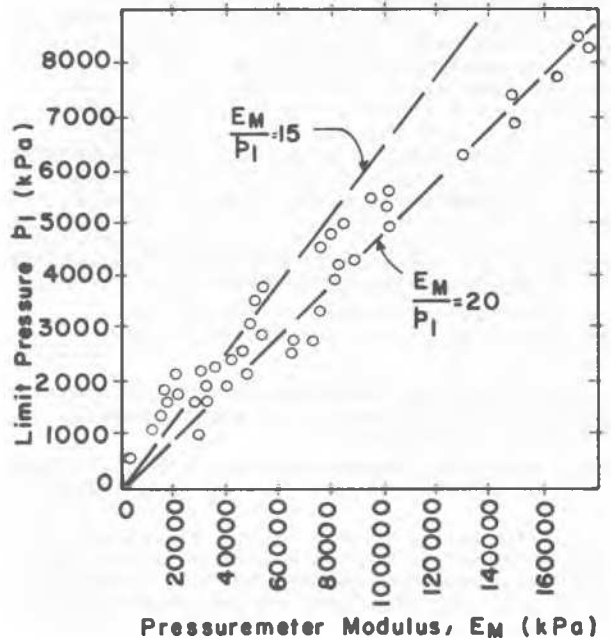


Fig.3. Relation Between Limit Pressure and Pressuremeter Modulus of Alluvial Deposits at Karakaya Railroad Viaduct

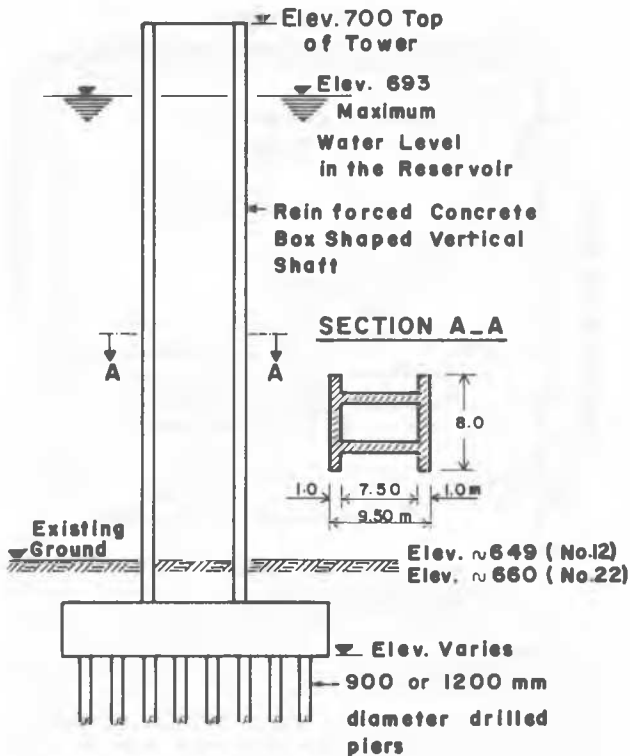


Fig. 4. Side View of A Typical Tower of Karakaya Viaduct (Yüksel Proje A.Ş.)

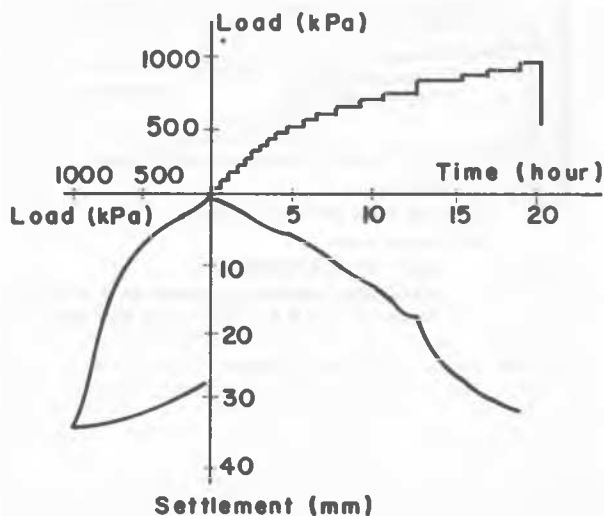


Fig. 5. Plate Load Test on Natural Soil at Tower 12 (Yüksel Proje A.Ş.)

Loading increments of 50 kN/m^2 were used during the tests, and at each load step readings were taken every 5 minutes for a minimum of 30 minutes. To saturate the soil tested, the pit was flooded for twelve hours by keeping a 0.3 m constant water height in the pit throughout the

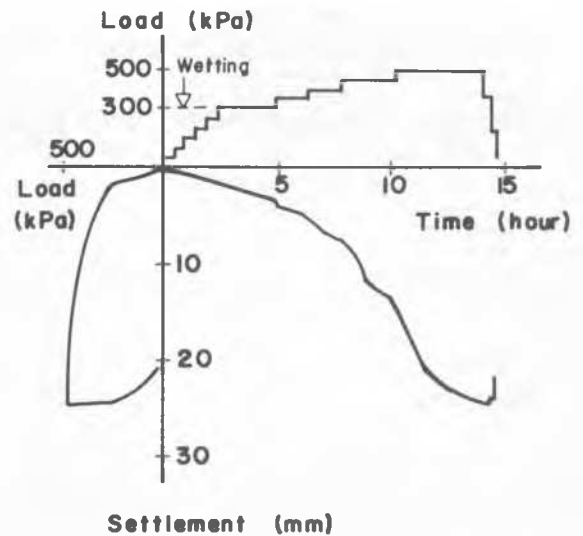


Fig. 6. Plate Load Test on Soil Flooded at 300 kN/m^2 (Yüksel Proje A.Ş.)

test. All tests were carried out up to 1000 kN/m^2 or failure load. The subsequent unloading was made in three steps.

Load test results obtained through the first and second type test series at No.12 tower location are given in Figures 5 and 6 respectively.

Comparison of the test results indicated that the rate of settlement increased upon wetting. It may also be observed that the settlements increased suddenly after 600 kN/m^2 load in the dry test, and after 250 kN/m^2 in the second type of test. At No.22 location alluvion was found to be very dense and no failure was observed at 1000 kN/m^2 load in either of the test series. In the third type of test at No.22 location, a settlement of 0.33 mm was recorded after wetting at 300 kN/m^2 load.

LABORATORY TESTS

One dimensional compression tests were run on the undisturbed and compacted soil samples taken from the same location and depth where the field load tests were made. Some physical properties of the soils taken by large molds are given in Table I.

Samples taken from the field were tested in the laboratory in a large diameter consolidation unit (permeability apparatus) consisting of mainly of a steel mold with an inside diameter of 204 mm, a height of 204 mm and a wall thickness of 5 mm. The specimen which is completely confined by the metal mold was placed on a lower porous stone and covered by an upper porous stone. The compression was measured by two dial gauges fastened to the top porous stone. The specimen was loaded by means of a heavy duty hydraulic jack, and after each load increment the nuts on the loading plate were tightened in order to keep a constant load on soil sample. Load on soil was measured by a gauge attached to the jack. The load observed was fairly constant despite small

TABLE I

Properties of Soils Taken from No.12 and No.22 tower locations

Soil Property	No.12 Location	No.22 Location
Silt and Clay(%)	10	18
Sand (%)	66	69
Gravel and Cobble (%)	24	13
Natural Water Content(%)	5.8	5.4
Bulk Unit Weight (kN/m ³)	19.5	19.8
Void Ratio (%)	40	40

deformations of the spring due to compression of the soil sample. The soil sample was saturated from the lower porous stone when wetting was required. The loading steps used during the test were 25,50,100,200,400 and 800 kN/m². The minimum waiting period at each load increment was 30 minutes. The samples were flooded at 400 kN/m² in three tests and 800 kN/m² in one test. Results of the large diameter consolidation tests are given in Figures 7,8,9 and 10.

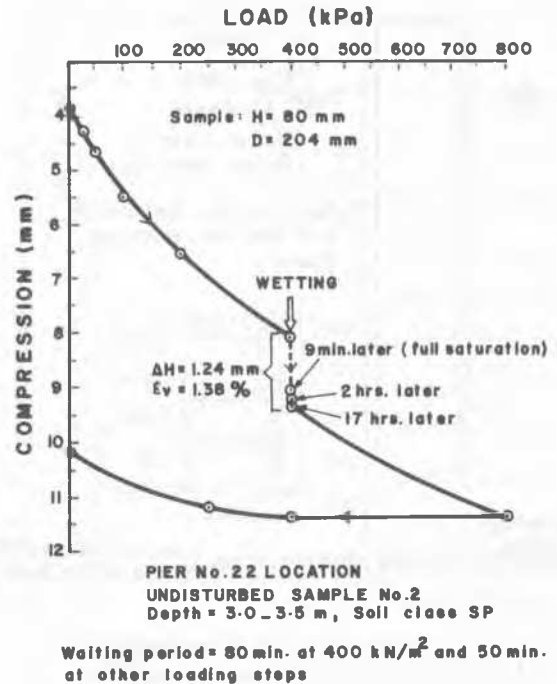


Fig.8. Consolidation Test on Undisturbed Soil Sample (Ordemir and Özkan)

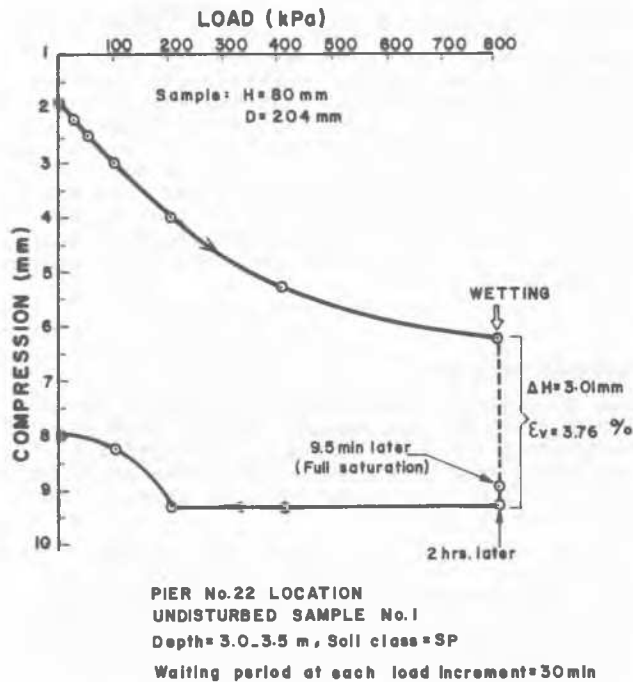


Fig.7. Consolidation Test on Undisturbed Soil Sample (Ordemir and Özkan)

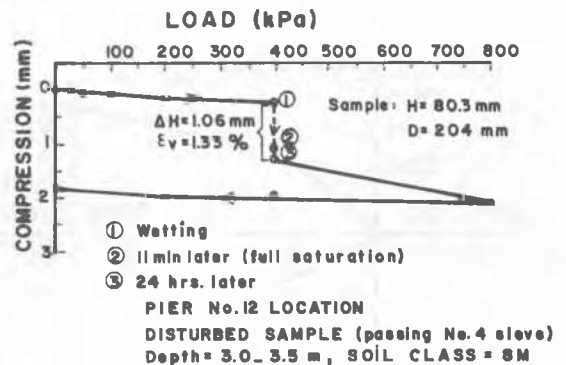


Fig.9. Consolidation Test on Compacted Soil Sample (Ordemir and Özkan)

Compacted samples were tried to be prepared at about in-situ unit weight. It may be seen from the figures that a considerable compression takes place after saturating the soil. Amount of the percent compression due to wetting varies

between 1.33% and 2.75% for the specimens flooded at 400 kN/m². The sample saturated at 800 kN/m² experienced a considerably large percent compression of 3.76%.

CONCLUSIONS

Both field load tests and laboratory consolidation tests indicate that dry alluvion above the present groundwater will experience considerable amount of compression when saturated during reservoir filling. It may be concluded that:

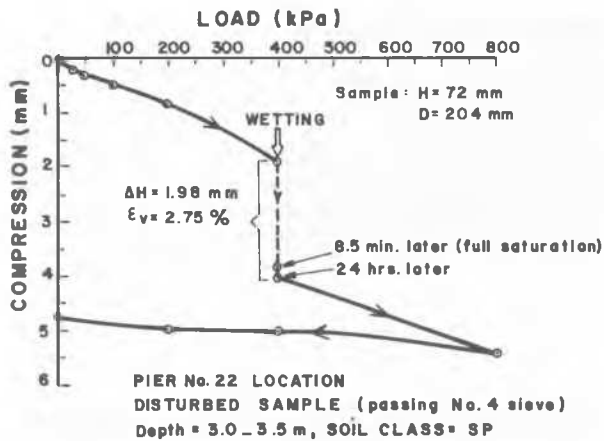


Fig.10. Consolidation Test on Compacted Soil Sample (Ordemir and Özkan)

- the viaduct towers will settle excessively if they are supported by shallow foundations constructed on dry alluvion,
- the towers will settle excessively if they are supported by piles not extending below present groundwater,
- the railroad if constructed on a rockfill embankment which may partly replace the viaduct will settle excessively partly due to compression of the dry alluvion and partly due to compression of the rockfill itself.

The amount of the settlement in any of the three alternative solutions cited above depends largely on the thickness of the dry alluvion and the height of the rockfill, and their relative density. In any case, however, percent compression as determined by tests is rather large and the resulting settlement cannot be tolerated. Predicted percent compression by tests is compatible with the values obtained by other investigators in similar soil conditions.

In order to overcome the settlement problem it was decided to design and construct a viaduct to cross throughout the valley and support its towers in alluvion zone by thick mat foundations resting on 900 mm and 1200 mm diameter drilled piers extending below present groundwater. Drilled piers were selected because they have many advantages in medium to very dense alluvions in which other types of piles can be hardly driven or constructed. Some theoretical analysis were made to evaluate the load bearing capacity of drilled piers considering potential negative skin friction and group action reduction. Then series of load tests on actual drill piers constructed were made to determine experimentally the vertical and horizontal load bearing capacity of 900 mm and 1200 mm piers. After having considered the theoretical approach and the results of compressive, uplift and lateral pier tests, and various methods of interpretation of pier load test results, the design loads as listed in Table II were recommended :

TABLE II
Design Loads of Drilled Piers

Diameter of Drilled Pier (mm)	Allowable Axial Compressive Load (kN)	Allowable Horizontal Load (kN)
900	1300 to 1500	80 to 120
1200	2500 to 3000	120 to 170

All drilled piers are to be extended a minimum of 5 mm below present groundwater.

REFERENCES

- Nobari, E.S. and Duncan, J.M. (1972). Effects of Reservoir Filling on Stresses and Movements in Earth and Rockfill Dams. Special Report No. TE-72-1, College of Engineering, Office of Research Services, University of California, Berkeley, California.
- Ordemir, İ. and Özkan, Y. (1982). Foundations of Karakaya Viaduct. Middle East Technical University, Project No. 81-04-03-123, Ankara.
- Sipahi, E.C. (1984). Compression of Granular Materials due to Wetting. Master Theses. Middle East Technical University, Ankara.
- Soydemir, Ç. and Kjaernsli, B. (1975). A Treatise on the Performance of Rockfill Dams with Unyielding Foundations in Relation to the Design of Storvass Dam. Norwegian Geotechnical Institute, Report No. 53203, Oslo.
- Tezcan, S. and Durgunoğlu, T. (1981). Geotechnical Report on Karakaya Viaduct. Boğaziçi University, Earthquake Engineering Research Institute, Report No. 81-36T, Istanbul.