# GEOTECHNICAL PROPERTIES OF MARINE SEDIMENTS FROM KOPER BAY

# PROPRIETES GEOTECHNIQUES DES SEDIMENTS RECENTS DE LA BAIE KOPER

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SYNOPSIS: The first part of this paper briefly describes the method of building the Port of Koper and the geotechnical properties of recent marine sediments in Koper Bay. The second part explains the method of improving subsoil at the bulk terminal by preloading the subsoil only.

## INTRODUCTION

Koper is situated where the Adriatic Sea most deeply indents into the European Continent, and where there exist convenient natural passages from Central Europe to the Adriatic Sea. The Port of Koper was and still is being built in a unique way: the water ways (canals) to the coastal constructions, which were erected later and founded mostly on steel pipe piles (Sovinc et al, 1985), on circular cells (Sovinc, 1972) and/or caissons are being built by the use of suction dredgers. The material excavated hydraulically is, for the most part, transported to the sites of the projected piers and to the background where fine mineral grains of sand, silt and clay mixed with organic material settle. At the end of pipeline (the eruption mouth) the diluted suspension flows out in cloddish granular particles. The cloddish fractions remain near the mouth of the outflow eruption pipe, while the unbound grains in the diluted suspension settle further on from the outflow point. The dredged material is therefore heterogeneous and of various densities. It was laid down at various thicknesses, in some places soon, in others treadable only after several years, etc. In the Soil Mechanics Laboratory IMFM of the University of Ljubljana parallel to the indicated way of building the harbour many original laboratory tests on samples of cloddish granular material as well as on settled samples taken at various distances from the end of the pipes (2, 5, 12, 15 & 100 m), were investigated. Further more, one of the main geotechnical properties of hydraulically transported material is grain size distribution and the time of sedimentation of dredged material. The second part of the present work deals with settlement calculations and "in situ" measurements of dredged material as well as of the natural ground under the dredged fill. Therefore it should be pointed out that by dredging, several million m$^3$ of original marine sediments were excavated and hydraulically transported. They were covered by ca. 2 million m$^3$ of weathered flysh and about 1 million m$^3$ of stone plug. This represents a great redistribution of earth mass and enormous supplemented loads on the primary surface of the sea bed.
Fig. 1 shows an arial photograph of the town of Koper before the construction of the harbour had commenced. Fig. 2 was taken in 1971, whereas Fig. 3 shows the harbour in the year 1990.

## A SHORT GEOLOGICAL REVIEW AND DESCRIPTION OF THE SEDIMENTATION PROCESS

The basic rock in Koper Bay represents Eocene flysh, which is composed of alternate layers of marl and sandstones. It can be seen throughout the outlying areas of Koper and forms the basis of the island where the town of Koper was built. Until the end of Second World War the town was surrounded only by the sea. The geologic-geotechnical profile of the harbour region proves that the deepest point in the flysh lies at an elevation of -55 to -60 m. The indent in the flysh is filled from an elevation of -1.5 m at the beginning to -5 m at the end of the piers by coherent and non-coherent sediments brought by torrents from the surrounding heights. The alluvia were smoothed and ground by sea waves. Clay grains brought by fresh water in suspensions in the heavier sea water settled in a light flake structure. Under the pressure of new alluvia layers of these less permeable clays started to consolidate, and at that time their porosity was reduced. However, since the natural growth of loading in consecutive and slow, generally only the loose structure remained during consolidation. Thus thick layers of very porous coherent soils several metres thick have appeared and are still being formed. According to factors such as the changing situation of the brood and torrent beds, their deltas, the water load flowing into the sea under various weather conditions, and the effects of sea waves layers of sand or lenses of sand appeared in some places, especially near to the outflow into the sea among porous less permeable cohesive soils. These clay-sand alluvia are mainly lightly settled and their porosity is large, whereas the structure is labile.

The dredged material is distributed in the harbour at the pier construction usually at an elevation of around +1.5 m in the way described at the beginning of this report. Some years ago a geotechnical map was elaborated with the isolines of hard flysh stratum, the surface of weathered flysh and the upper surface of hard moulded sand and gravel clays wich cover the flysh. The cards do not show a more detailed packet of clay sediments since individual layers are fairly unbound and lens-like. With classic sounding, vane shear tests, CPT tests and similar "in situ" tests the heterogeneity of

**Fig. 1.** Aerial shot of the town of Koper before the construction of Luka Koper (Koper Harbour)

**Fig. 2.** View of the town and Koper Harbour in 1971

**Fig. 3.** View of the town and Koper Harbour in 1990

marine sediments is proven, rather than by the results which could in fact be used in geotechnical calculations. Thus, for each larger building additional geotechnical research is necessary. The first general overview on geotechnical properties of marine sediments was published 30 years ago (Sovinc, 1963). The first description of the problem of the foundations of round steel tanks with special attention on distortion deformations was also published in the same year (Šuklje & Sovinc, 1963). Further interesting solutions for foundations of storage buildings in the Koper harbour area can also be seen in the reports by authors of this paper (Sovinc & Vogrinčič, 1974, 1989, 1991; Sovinc, 1972; Sovinc & Vidmar, 1973).

The above mentioned articles deal with methods for the improvement of natural soil, calculation methods for the estimation of settlement, comparaisons of evaluations of settlement with "in situ" measurements, etc.

Before describing a further example of the building of the bulk terminal we would like to enumerate and show only superficially diverse "unconventional" laboratory tests as performed over the last 30 years. In the laboratory several strain shear tests performed in shear boxes as well as in the triaxial apparatuses were compared. The differences in strain characteristics according to the momentarily more or less continual loading stages (loading by drops) were shown and it was further proven that the strain characteristics are almost equal, if samples are loaded in a vertical or horizontal directions. The consolidation curves from oedometer tests were performed in accordance with Terzaghi's consolidation theory. At the oedometers and in triaxial deformation tests the strains were monitored so long that the slope angle or secondary settlement could clearly be determined, etc.

## GEOTECHNICAL TESTING AT THE LOCATION OF BULK CARGO TERMINAL

For the construction requirements of the bulk cargo terminal, which comprises large surfaces along the coast (see x on Fig. 3), numerous boreholes were made and undisturbed soil samples for laboratory testing were taken. In Fig. 4 the ground plan of preloading embankments is shown; they were erected with the intention of improving soil properties and preventing failures under the fills. The primary depression surface (points 40 to 70 in Fig. 4) was at an elevation of -1.5 to -2 m. This was raised by dredging to an elevation of ca. +2 m. In the area of points 1 to 37 the elevation after dredging was ca. +1.5 m. On this dredged fill, stone and gravel material was brought to a height of 2.2 m and in this surface steel meshes $\phi$ 6 mm at a raster of 10 x 10 cm were laid. Over the meshes earth material (marl and flysh) was laid on the elevations as shown in Fig. 4. Fig. 5 shows the geotechnical profile as well as the cross-section of the embankment B-B. The individual typical soil layers are marked by letters **b** to **g** and dredged material by letter **r**, which have the following meanings:

**b)** uniform medium and fine sand of grey colour with organic additions, interrupted by thin layers of clay and silty clay,
**c)** silty clay of grey colour, low plasticity, interrupted by thin lenses of fine sand, with several organic additions,
**d)** silty clay with organic additives, mussels and thin layers of peat,
**e)** poorly granulated gravel and sandy clay,
**f)** decayed flysh,
**g)** solid flysh,
**r)** dredged material.

The main soil caracteristics are given here only in brief for layers **b**, **c**, **d** and **r** which are of most interest according to the anticipated settlement of natural soil under the weight of the preloading embankment and subsequently under the weight of the bulk terminal and other structures.

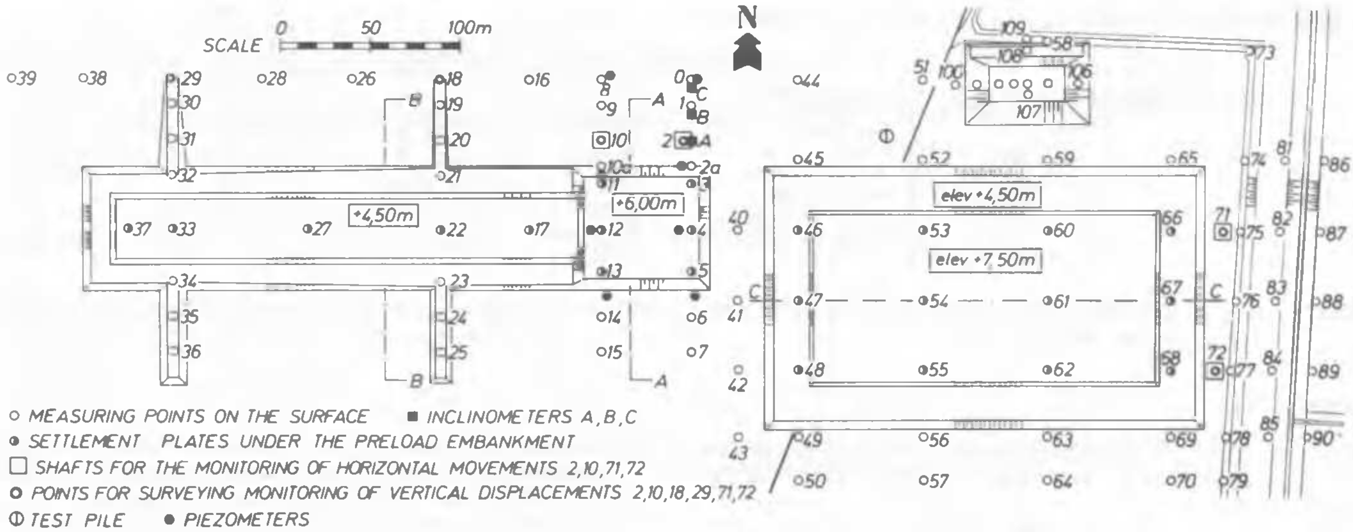


Fig. 4. Ground plan of preload embankments and measurment points

**Layer b** is uniform medium and fine sand SU of a grey colour with organic additives interrupted by thin layers of silty clay. The water content is about 40 %. Grain size tests have shown that the sandy fractions range from 50 to 75 %; the rest is silt. The shear strength angle lies between 35° and 38°. The oedometer modulus in the domain of primary pressures lies between 1500 and 2000 kPa and the coefficient of permeability is probably $10^{-8}$ to $10^{-9}$ m/s.

**Layer c** is organic silty clay MI to MH of a grey colour and low consistancy, interrupted in some places by fine sand which is very organic. The water content is around 45 %, plasticity index $I_P = 25\%$, and index of consistency $I_C = 0.30$. Silty fractions are between 40 to 80 %, the rest being sandy with no more than 7 % of clay fractions. The shear angle is about 21° and the apparent cohesion 5 kPa. The shear strength as tested by simple uniaxial tests is an average 20 kPa and the strength determined "in situ" by using the pocket penetrometer 19.5 kPa; the average shear strength obtained by vane shear tests after the first shearing is 25 kPa. The modulus of compressibility in the range of primary stresses spans 1300 to 2000 kPa; the coefficient of permeability is estimated to be $5 \cdot 10^{-10}$ m/s.

**Layer d** is silty clay CH-MI-MH, and comprises a large number of mussels and peat. Average water content is around 45 %, index of plasticity $I_P = 35\%$ to 40 %, and index of consistency lies in a range from 0.40 and 0.55. There are about 5 % to 35 % of sandy fractions, clay fractions comprising not more than 15 %; the rest is silt. The shear angle is about 15° and the cohesion between 10 and 16 kPa. Unconfined strength at failure is an average 30 kPa, the average strength achieved by vane tests after remoulding 37 kPa, and pocket penetrometer strength ca. 0.4 kPa. The moduli of compressibility lie in the range of primary pressures between 1500 and 2000 kPa; the coefficient of permeability is estimated to be $10^{-11}$ m/s.

**Layer r** represents dredged material. For the reasons given at the begining of this report it is rather difficult to provide some average characteristics. According to our experience it is necessary to make some shallow soundings and remove as much undisturbed soil samples as possible for the laboratory tests.

For monitoring the movements on site of bulk terminal a system of measurements has been established and consist of monitoring vertical displacements on the surface as well as under the embankments, measuring relative distance between several points, horizontal displacements by use of inclinometers as well as permanent surveying points from the wider surroundings of the terminal.

The time-settlement diagram for measurment points 26 to 36 is shown in Fig. 6 (at the time of the first observation the height of the embankment was 0.6 m; at the time of second observation it was already 4.5 m). The time-settlement curve for measurment point 4 is shown in Fig. 7. Compared to the calculations, it was made by using the classic one-dimensional theory of consolidation. However, we would like to mention that point 4 was laid on the place which was already slightly consolidated at the beginning of preloading.

After preloading the surface was lowered to an elevation of +2.2 m, and soon there after reloaded by high bulk cargoes. As shown in Fig. 3 the bulk cargo terminal is more or less stable despite settlements occuring rather intensively. Control at present is performed by measurements of pore water pressure (piezometers) and by estimation of effective pressure in the soil.

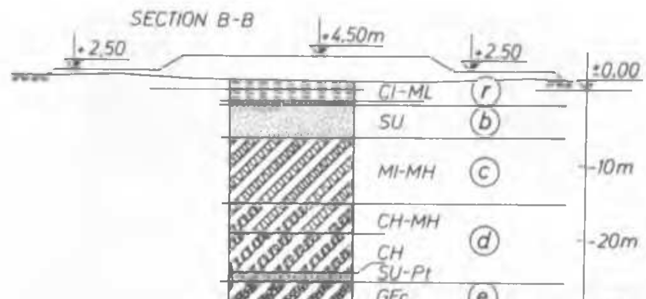


Fig. 5. Cross-section B-B of the preload embankment and geotechnical soil profile

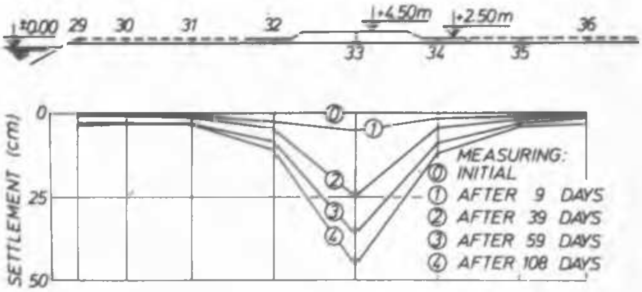


**Fig. 6.** Progress of settlement of preload embankment at measurement points Nos. 29 - 36

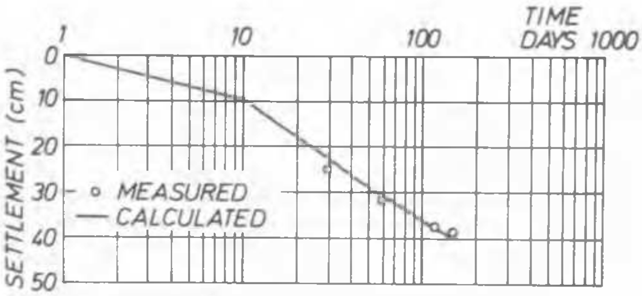


**Fig. 7.** Coparison of measured and evaluated settlement values for measurement point No. 4 (logarithmic time scale)

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