Analysis of the Semi-Empirical methods used for the estimation of the load capacity of the Soil-Pile, using the stratificated Soil.

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Abstract: This scientific work, is the result of the study of some Semi-Empirical methods used for estimation of the load capacity of the soil-pile assembly, using data from the Standard Penetration Test (SPT). Like case of study, the construction of the Central Hospital of Jing Zhou (1st phase), located in China, Hubei Province, City of Jing Zhou is used. For this paper, three Semi-Empirical methods, developed in countries outside China, and already used in other parts of the world, were chosen. These methods are: the methods of Aoki and Velloso, the method of Decourt and Quaresma, and the method of Meyerhof. The results of the Semi-Empirical methods under study are compared with the test results of static load tests performed on site, as recommended by the national Standards (Chines), and local regulations. In addition to the Semi-Empirical methods mentioned above, the relevant provisions of the technical specifications for building foundations (DB42/242-2003) and technical specifications for construction piles (JGJ94-2008) are used. At the end, comparisons are made between the results of the methods under study, the results of static load tests (slow loading) performed on site, the results of the GEO5 Software, and the results of characteristics value of the single pile (Ra), according (DB42/242-2003). In a general vision, with this work the following conclusion was reached: Using the Meyerhof method, the value of Qu was greater than the value of the load test, but it is also the method that presents the values closest to the values of the static load test performed on the spot, with a deviation of 2% in relation to the value of the load test; The Aoki and Velloso method, in addition to providing a value of Qu lower than the value of the static load test, thus ensuring greater safety, also provides acceptable values in comparison to the values of the load test, with a deviation of 6%; The Decourt and Quaresma method, although giving a value of less than the value of the static load test, gave values with a very large deviation from the value of the static load test, being above 10% ; In relation to the software Geo 5, the results are very satisfactory, with a deviation of 0%, in relation to the static load test values.

Key Words: Foundations on piles, Semi-empirical methods in piles, Piles Load Capacity, Soil Load Capacity.

1 Introduction

The fundamental purpose of deep foundations is to transfer the load from the superstructure to the ground. It is important to understand that with the evolution of the technical-scientific knowledge in the area of Civil Engineering in general, and Geotechnical Engineering in particular, together with the current trend in relation to the globalized market, competition has been increasing in the area of civil construction.

This requires more and more companies to adopt increasingly competitive prices, which raises the need to study various methods of analyzing structures, which may be inexpensive (without some soil testing), but without causes the structure to function unproperly.

It was with this intention, that emerged over the years, several Empirical and Semi-Empirical methods, for the estimation of the load capacity of the piles, worldwide.

Because these methods have been developed in certain geographic areas, sometimes with unique characteristics, it is necessary to make some comparative studies of the results of these methods, before they are applied in these new areas of geographic location.

For these reasons, in this work, we will study some Semi-Empirical methods developed in some countries of America and Europe.

According to Citra (2010), in some cases, the theoretical formulas of prediction of the load capacity in stakes, may not offer adequate results; therefore, there is need to study and use the semi-empirical formulas that are usually based on empirical correlations, and in-situ test results in the end adjusted with load tests.

The objective of this research is to study some semi-empirical methods used for the estimation of the load capacity of soil and piles based on the SPT tests, Load capacity tests and local standard of Hubei Province, for the estimation of the load capacity of piles, “Technical Specifications for Building Foundations” (DB42/242-2003). As case study will be use the project of the Jing Zhou Cetral Hospital (phase I), located in Jing Zhou city, Hubei province, China.

1.1 Jing Zhou Project Overview

The project is located on the north side of Chuyuan Road in Jing Zhou city, approximately 1 km from the city Government. During the research period, the elevation of the site varied between 30.15 to 38.24 m. Figure 1.1. shows some details of the side.

Figure 1.1. Location Map.

In figure 1.2, is showed the architectural design of Jing Zhou Central Hospital, and Table. A1. (in annex) shows the specific conditions of the proposed buildings. According to the "Geotechnical Research Code" (GB50011-2001, 2009 edition), as to its importance, the proposed project is level 1; in relation to the positioning, the project corresponds to level 2 (Medium complex); and the Geotechnical Research Degree, is Grade 2.

Figure.1.2. Architectural Renderings.

2 Profiles and Geotechnical Data of the Site

Because it is the tallest building of all (First Stage of JingZhou Central Hospital) and the building that carries the most ground demands, our study will be fixed in the main building. The building has a height of approximately 55.7 m, consisting in 20 floors of the visible part, and 2 underground floors, with a building area of approximately 69842 m2. Following the contours of the building, in the area of implantation, 20 drilling holes with the following references were opened: ZK 20, ZK 21, ZK 22, ZK 23, ZK 24, ZK 25, ZK 27, ZK 28, ZK 29, ZK 30, ZK 31, ZK 32, ZK 33, ZK 34, ZK 36, ZK 37, ZK 38, ZK 39 e ZK 40, ZK 41, as shown in Figure 2.1. The spacing between exploration points was defined according to the Geotechnical Code (GB50021-2001) in use in China, which defines the spacing between 15 and 30 meters for foundations of level 2, which is the case of this building.

Figure 2.1. Exploration holes in the main building.

According to the design of the building, and the Geotechnical exploration plan (Fig. 1.2 and 2.1. respectively), Points ZK 30 and ZK 31, are the ones that transmit greater efforts of the building structure, to the foundation.

In figure 2.2, the profiles of the holes ZK 30 and ZK 31 are presented, with detail of the stratification of the constituent layers. For this building, bored piles, molded in-situ, with a diameter of 800 mm and 1000 mm, and 15 m in length were used. The piles is seated in <5-2>, layer of Pebble.

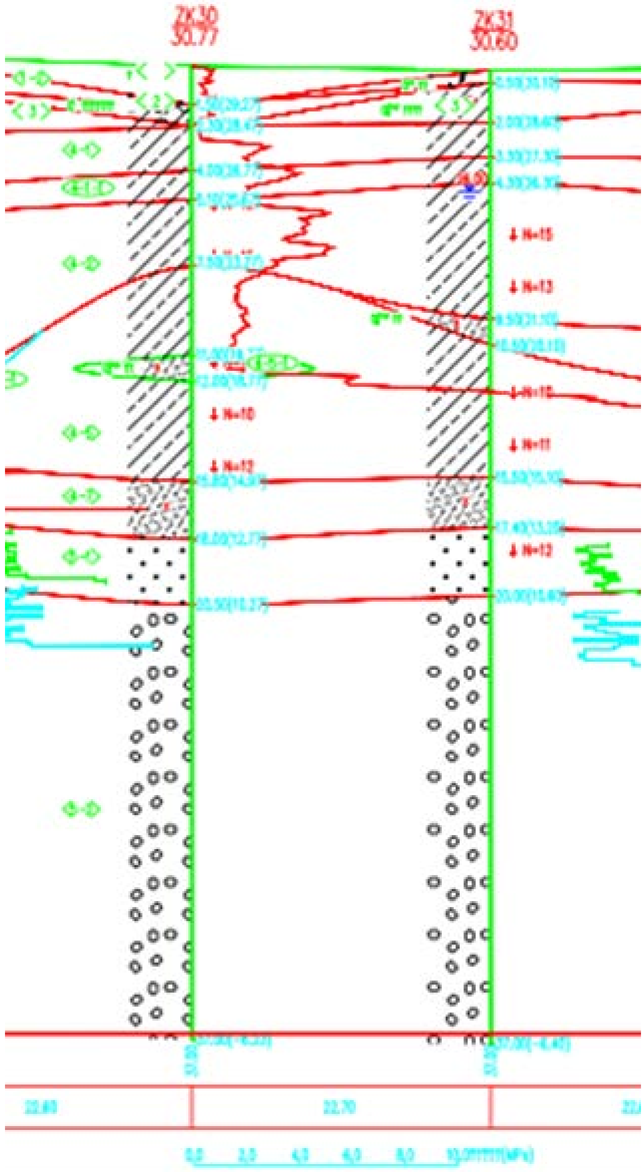

Figure 2.2. Geotechnical Profiles of the exploration points ZK 30 and ZK 31.

After the data collection on the site, the work of data processing follows. Tables A1, are the result of the statistical processing of the SPT test data, performed on site.

2.1 Static Load Test Results carried out on Site

According to the loading capacity of the design piles, equal to 6400 kN, the loading is divided into 6 stages, and the unloading is divided into 4 stages, as indicated in chart 3.4 (chart resulting from the static load tests carried out on site). The geotechnical report of the static load tests carried out in situ, defines -12.10 m as the top level of the pile, and the stratum <5-2> as where the base of the pile stay. According to the Chinese regulation, the carracteristic value (Ra), from the maximum load (Qu), can be determined according to equation 2.1.

$$R_a = \frac{Q_u}{2} \qquad \text{Eq. 2.1.}$$

3 Analysis of Dadas

3.1 Results of ZK 31 Profile Load Capacity and Pile Material

The ZK31 profile is composed of the layers <5-2>, <5-1>, <4-7>, <4-5>, <4-5-1> and <2-1>. The pile crosses the strata, <5-2> (9.20 m): <5-1> (2.60 m); <4-7> (1.90 m); <4-5> (3.40 m).

The pile in study is Drilling pile (reinforced concrete) , with a diameter (d) of 800 mm, and 17.10 m in length (l).

3.1.1 Estimation of the carracteristic value of the vertical support capacity of single pile (Ra).

According to national (Chinese) and local standards, "Technical Specifications for Foundations Construction" (DB42 / 242-2003) in Hubei Province, the vertical support capacity of the single pile, can be determined by equation 3.1.

$$R_a = q_{pa} A_P + u_P \sum q_{sia} l_i \,(D \le 800mm) \qquad \text{Eq. 3.1.}$$

Where: **R$_a$** - Characteristic value of the vertical load capacity of the single pile; ***q$_s$*** - Characteristic value of lateral resistance of the soil around the pile; ***q$_p$*** - Characteristic value of the final resistance of the pile; ***A$_P$*** - The cross-sectional area of the pile; ***l*** - Pile length; ***u$_p$*** – Base Perimeter of the pile;

Table 3.1 shows the results of the characteristic value of the vertical capacity of a single pile, determined according to local standards, calculated according to the data found in table A2.

Building	Hole Number	Pile Type	Pile Top Elevation (m)	Pile Length (m)	Bearing layer	Pile Diameter (mm)	Vertical bearing Capacity of Single pile Characteristic value Ra (kN)	
							Before Grouting	After Grouting (1.3)
Inpatient Building	ZK31	Drilling (punching) hole pouring pile	19.9	17.10	<5-2>	800	3238.51	4210.06

Table 3.1. Results of the estimation of Ra, according to DB42 / 242-2003.

3.1.2 Aoki and Velloso

According to these authors, the ultimate bearing capacity of a pile, can be evaluated by the equation 3.2.

$$Q_u = A_b \frac{N_{SPT}^{L}}{F_1} + P \sum \frac{\alpha N_{SPT}^{m}}{F_2} \Delta L \qquad \text{Eq. 3.2.}$$

Where:

P **-** is the perimeter of the pile;

ΔL **-** is the thickness of the soil layer (m);

N_{SPT}^{L} is the N_{SPT} near the point of the pile;

N_{SPT}^{m} is the N_{SPT} average for each ΔL;

F_1 and F_2 are coefficients of correction of the end and side resistances.

By analyzing the results for each layer, for this method (Aoki and Velloso), the highest value of Qu is reached in the last layer of the pile (<5-2>). These results can be seen clearly in chart 3.1.

Chart 3.1. Layers x Qu by the method of Aoki and Velloso.

3.1.3 Decourt and Quaresma

In 1996, Decout introduced factors α and β, in the part of tip resistance, and lateral resistance respectively, to adapt the method to excavated cuttings with bentonitic mud, generally excavated cuttings, continuous flight auger, root cuttings, and stakes injected under high pressure; obtaining equation 3.3.

$$Q_u = \alpha C N_P A_P + \beta 10\left(\frac{N_L}{3}+1\right)UL \quad \text{Eq. 3.3.}$$

Analyzing the results for each layer, in Decourt and

Quaresma method, the highest value of Qu is reached in the last layer of the pile (<5-2>). These results can be seen clearly in chart 3.2.

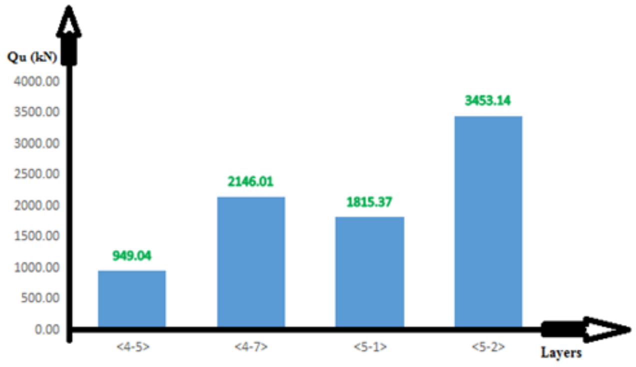

Chart 3.2. Layers x Qu by the method of Decourt and Quaresma.

3.1.4 Method of Meyerof

According to the Meyerof method, the maximum load capacity of the pile is 6541.86 kN. In this method it is proposed that the resistant capacity of a spiked pile is obtained by Eq.3.4.

$$Q_u = 400NA_b + 2\bar{N}A_S \quad \text{Eq. 3.4.}$$

Where:

R is the bearing capacity of the pile (kN);

N is the number of strokes (SPT);

A_b is the area of the pile tip (m^2);

$\bar{N}$ is the average value of N along the length of the pile;

A_s is the lateral area of the pile (m^2).

The maximum resistance capacity of the layer <5-2> (the last layer) is approximately 6540 kN, and be seen clearly in chart 3.3.

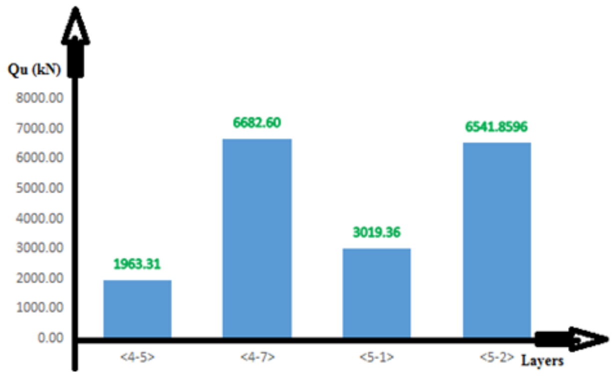

Chart 3.3. Layers x Qu by the method of Meyerof.

3.1.5 Static Load Test D-Z59

The static load test D-Z59 obtained a maximum load of 6400 kN, as shown in chart 3.4, and maximum settlement of 14.26 mm. The characteristic value is 3200 kN, corresponding to 4.03 mm of settlements. The residual settlement is 11.47 mm.

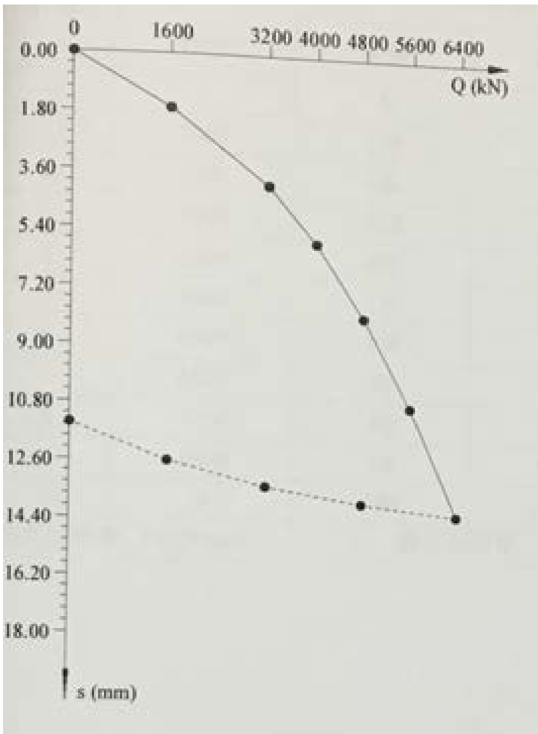

Chart 3.4. Load (Q) x Displacement (S).

3.2 Simulation in GEO 5 Software.

3.2.1 The results of GEO 5 Software

In the GEO 5 software, the conditions of the Geotechnical profile of the site, and parameters of the load test pile D-Z59 were produced. The result of the profile and the pile is shown in figure 3.1.

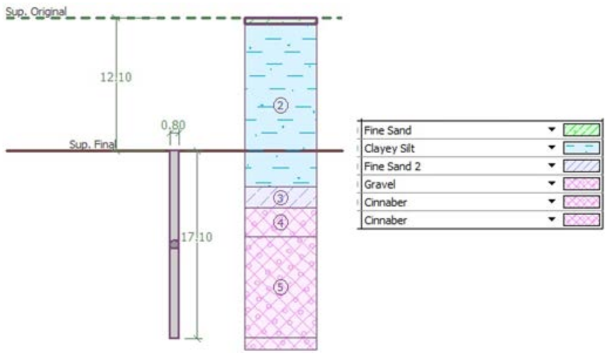

Figure 3.1. Profile of the pile and soil, in GEO5.

According to the result founded using GEO 5 software, the maximum load capacity of the pile (Qu), is 6401.22 kN. In relation of Ra, considering a safety factor (Fc = 2), as recommended by the Chinese Geotechnical Engineer, the value of Ra is equal to 3200.61 kN, as showed in table 3.2.

Layer	Qp[kN]	Ql[kN]	Qu[kN]	Ra[kN]
<5-2>	396.01	6005.21	6401.22	3200.61

Table 3.2. Results of the GEO5 software.

4 Comparison of Results

According to the results of the semi-empirical methods under study, the Meyerof method is one that has values of Qu that are closer to the value of the static load test; with a deviation of 2%, in relation to the static load test. Then, we have the method of Aoki and Velloso, with a deviation of 6%, in relation to the static load test value. Finally we have the method of Decourt and Quaresma, with a deviation of 46%, in relation to the static load test value. In chart 4.1, the comparative values between the Qu, and the static load test of the slow loading are presented. The red horizontal line (serie-2) represents the value of static load test performed at the site.

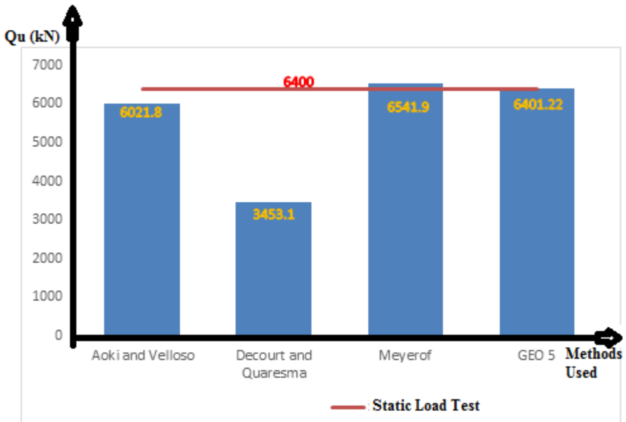

Chart 4.1. Results of Qu [kN] in all method used, and comparing with Static Load Test.

The chart 4.2 shows the comparative values between the Ra values of all the methods used, the Ra value of the static loading test performed in the work field, and the characteristic value of the pile, according to (DB42 / 242-2003) . The horizontal brown line (value 4210.1 kN) represents the service value of the pile, after grouting, and the horizontal blue line (value 3238.5 kN) represents the service value of the pile, befor grouting. According to chart 4.4, all the methods offer a safety admissible, in relation to the vertical load capacity of the single pile.

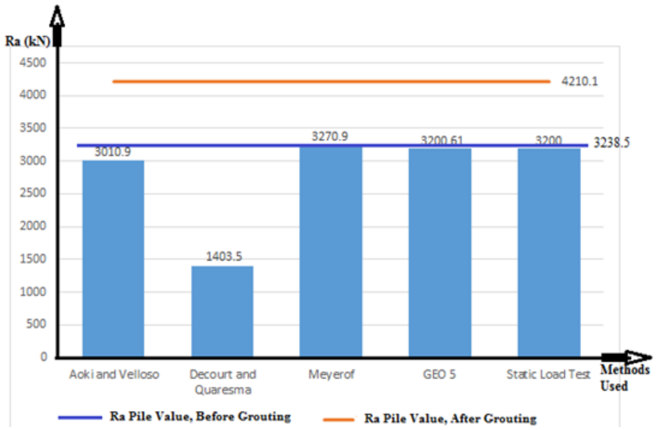

Chart 4.2. Results of Ra in all method used, and comparing with Static Load Test.

5 Conclusions

According to the results of the semi-empirical methods studied, can be concluded that the Meyerhof method, although it has given a value of Qu greater than the value of the load test, is the method that presents the closest values in relation to the values of the static load test carried out in the place, with a deviation of 2% in relation to the value of the load test. The Aoki and Velloso method, in addition to providing a value of the Qu lower than the value of the Static load test, thus ensuring greater safety, also provides acceptable values in comparison to the values of the load test, with a deviation of 6%, and the method of Decourt and Quaresma, although it gave a value of Qu lower than the value of the test of static load, is the one that gave values with a very great deviation, in relation to the value of the test of static load. In relation to the software Geo 5, the results are very satisfactory, with a deviation of 0%, in relation to the values of the static load test. According to the results of Geo 5, the load of the base of the pile contributes with smaller percentage in the total resistance of the stake.

Analyzing the results obtained in a safety-oriented view, it can be stated that the method of Decourt and Quaresma is the one that offers greater security, following the method of Aoki and Velloso, while the method of Meyerhof, for providing values of the Qu higher than the limit determined by the static load test, can be consideret low safety. If we look at the economic point of view, since the Meyerhof method gives a higher value of Qu, it can reduce the number of stakesof the project, which also reduces the amount of material to be used, so it can be the favorable method. In relation to the characteristic value of the single pile, all the methods under analysis, work in safety, mainly, if considering the pile, after grouting.

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Table A1. SPT Strokes count statistics.

Number of strata	Type of soil	Situation	Number of tests n	Basic value			Standard deviation σ	Coefficient of variation δ	Statistical correction factor Ψ	Standard valueN
				Max	Min	μ				
<3>	Clayey Silt	Plastic	37	11	7	**9.1**	1.026	0.11	0.968	8.8
<4-1>	Clayey Silt	Hard plasticity	37	15	11	**13**	0.902	0.07	0.980	12.5
<4-1-1>	Clayey Silt	Plastic	19	10	7	**9**	1.079	0.12	0.951	8.5
<4-2>	Clayey Silt	Hard plasticity	266	18	11	**13**	1.208	0.09	0.990	13.0
<4-3>	Fine Sand	Slightly Dense	29	17	10	**12**	1.885	0.16	0.950	11.5
<4-4>	Fine Sand	Mean Density	22	27	13	**18**	3.741	0.21	0.923	16.8
<4-5>	Clayey Silt	Plastic	103	11	8	**9**	0.863	0.09	0.984	9.1
<4-5-1>	Fine Sand	Mean Density	6	24	16	**19**	3.098	0.16	0.865	16.4
<4-5-2>	Fine Sand	Mean Density	10	35	25	**29**	3.502	0.12	0.930	27.3
<4-6>	Clayey Silt	Compact	51	12	7	**10**	1.012	0.10	0.975	9.5
<4-7>	Fine Sand	Compact	69	36	26	**31**	2.485	0.08	0.983	30.5
<5-1>	Gravel	Mean Density	61	11.	9.1	**12**	0.572	0.06	0.955	10.1
<5-2>	Cinnabar	Compact	61	13.	11.0	**24**	0.453	0.04	0.992	11.8

Table A2. Suggested Values for Stake Foundation Project Parameters

Stratum Code	**Name of Soil**	Compactness	Prefabricated piles			Drilling Pile			Coefficient of increase of post-bitumen strengthβs / βp
			q_{sia} (kP_a)	q_{pa} (kP_a)		q_{sia} (kP_a)	q_{pa} (kP_a)		
				h≤10m	10m<h ≤15m		10m<h ≤15m	h≥15m	
<3>	Clayey Silt	Malleable	32	/	/	35	/	/	1.4/2.2
<4-1>	Clayey Silt	Rigid	36	/	/	40	/	/	
<4-1-1>	Clayey Silt	Malleable	31	/	/	36	/	/	
<4-2>	Clayey Silt	Rigid	36	/	/	42	/	/	
<4-3>	Fine Sand	**Slightly dense**	20	/	/	15	/	/	1.6/2.4
<4-4>	Fine Sand	**Mean density**	28	/	/	24	/	/	
<4-5>	Clayey Silt	Malleable	31	/	/	30	/	/	1.4/2.2
<4-5-1>	Fine Sand	**Mean density**	30	/	/	26	/	/	1.6/2.4
<4-5-2>	Fine Sand	**Mean density**	33	/	/	25	/	/	
<4-6>	Clayey Silt	**Compact**	28	/	/	32	/	/	1.4/2.2
<4-7>	Fine Sand	**Compact**	40	/	/	34	/	/	1.6/2.4
<5-1>	Gravel	**Mean density**	65	3900		70	1000		2.4/3.2
<5-2>	Pebble	**Compact**	70	4500		75	1250		

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