

Geotechnical Evaluation and Deep Foundation Design Strategy in Gypseous Soils: A Case Study of the Airport Intersection Bridge Project in Najaf, Iraq

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التقييم الجيوتقني واستراتيجية تصميم الأساسات العميقة في التربة الجبسية: دراسة حالة لمشروع
جسر تقاطع المطار في النجف، العراق

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Abstract:

The paper discusses the integrated geotechnical evaluation of the sub-surface conditions at the proposed site of Najaf Airport bridge, Iraq, based on the field investigation data, standard penetration tests (SPT), laboratory soil characterisation, shear-strength testing and chemical analysis of soil and ground water. The data obtained indicate heterogeneous alluvial soil profile with prevalent sandy deposits interbedded with isolated silty and clayey horizons, with considerable accumulation of gypsum in the near-surface zone. The laboratory tests on soil revealed that the tested materials were mainly poorly graded sand (SP) and silty sand (SM), while the fine-grained deposits at depth were classified as MH and MI. The corrected SPT resistance generally increased with depth, presenting maximum values of 64, 59 and 58 for BH1, BH2 and BH3 respectively, indicating denser and stronger deposits with depth. The friction angle ranged from 30° to 41.2° as shown by the direct shear tests indicating that sandy soils behave essentially in a frictional manner. Groundwater was encountered at a depth of about 4.2 m below ground surface and chemical analyses indicated significant levels of sulphate, chloride, total dissolved solids and gypsum indicative of potentially aggressive subsurface conditions. The average calculated allowable bearing capacity of the investigated raft foundation condition was 14.5 ton/m² with a range of allowable values of 14.2-14.8 ton/m². The estimated allowable axial capacity of a 1.2-m diameter bored pile increased from approximately 145 tons at a pile length of 15 m to 310 tons at 20 m. The results show that generally the engineering properties of the site vary significantly in the spatial and vertical directions. The results provide a preliminary basis for foundation selection and indicate that foundation design should consider the groundwater conditions, the presence of gypsum in soils, the chemical aggressiveness, and the variation of soil resistance with depth. Full-scale pile load testing is recommended to confirm analytical pile capacity estimates prior to the final foundation design.

Keywords: Geotechnical investigation; Najaf Airport; soil classification; SPT; shear strength; gypsum-bearing soil; groundwater; bearing capacity; bored piles; foundation engineering.

المخلص

تناقش هذه الورقة البحثية التقييم الجيوتقني المتكامل للظروف تحت السطحية في الموقع المقترح لجسر مطار النجف في العراق، وذلك استناداً إلى بيانات التحريات الميدانية، واختبارات الاختراق القياسية (SPT)، وتوصيف التربة مخبرياً، واختبارات مقاومة القص، والتحليلات الكيميائية للتربة والمياه الجوفية. وتشير البيانات المستمدة إلى وجود قطاع تربة رسوبية غير متجانسة، تغلب عليه الرواسب الرملية المتداخلة مع طبقات معزولة من التربة الغرينية والطينية، مع وجود تراكم ملحوظ للجبس في النطاق القريب من السطح. وقد أظهرت الاختبارات المخبرية أن المواد المختبرة كانت في الغالب رمالاً ضعيفة التدرج (SP) ورمالاً غرينية (SM)، في حين صنفت الرواسب دقيقة الحبيبات الموجودة على أعماق أكبر ضمن الفئتين (MH) و (MI). وبشكل عام، لوحظ تزايد في قيم مقاومة الاختراق القياسي (SPT) المصححة مع زيادة العمق، حيث بلغت القيم القصوى 64 و 59 و 58 في الآبار الاستكشافية (BH1) و (BH2) و (BH3) على التوالي، مما يشير إلى وجود رواسب أكثر كثافة وقوة مع العمق. وتراوحت زاوية الاحتكاك بين 30 درجة و 41.2 درجة وفقاً لاختبارات القص المباشر، مما يدل على أن التربة الرملية تسلك سلوكاً يعتمد أساساً على الاحتكاك. وقد تم رصد المياه الجوفية على عمق يبلغ حوالي 4.2 متر تحت سطح الأرض، وأظهرت التحاليل الكيميائية مستويات مرتفعة من الكبريتات والكلوريدات والمواد الصلبة الذائبة الكلية والجبس، مما يشير إلى ظروف تحت سطحية قد تكون مسببة للتآكل. وبلغ متوسط قدرة التحمل المسموح بها المحسوبة لنظام الأساسات الحصىرية (Raft Foundation) المدروس 14.5 طن/متر مربع، ضمن نطاق يتراوح بين 14.2 و 14.8 طن/متر مربع. كما تبين أن قدرة التحمل المحورية المسموح بها المقدر لوتد محفور (Bored Pile) بقطر 1.2 متر تزداد من حوالي 145 طناً عند طول 15 متراً إلى 310 أطنان عند طول 20 متراً. وتُظهر النتائج أن الخصائص الهندسية للموقع تتفاوت بشكل ملحوظ في الاتجاهين الأفقي والرأسي. وتوفر هذه النتائج أساساً أولياً لاختيار نوع الأساسات، وتشير إلى ضرورة أن يأخذ تصميم الأساسات في الاعتبار ظروف المياه الجوفية، ووجود الجبس في التربة، والطبيعة الكيميائية للتربة، وتغير مقاومة التربة مع العمق. ويوصى بإجراء اختبارات تحميل فعلية (على نطاق كامل) للاوتاد للتحقق من التقديرات التحليلية لقدرة تحمل الاوتاد قبل إعداد التصميم النهائي للأساسات.

الكلمات المفتاحية: التحريات الجيوتقنية، مطار النجف، تصنيف التربة، اختبار الاختراق القياسي (SPT)، مقاومة القص، تربة محتوية على الجبس، المياه الجوفية، قدرة التحمل، الركائز المحفورة، هندسة الأساسات.

1. Introduction

The engineering characteristics of the underlying soil deposits strongly control the performance and long-term serviceability of transportation infrastructure. This is particularly the case in alluvial environments where there may be large spatial variations in soil composition, density, groundwater conditions, and geotechnical properties over relatively short distances (Craig, 2004). Therefore, a detailed geotechnical investigation is necessary to determine subsurface conditions that may affect foundation selection, bearing capacity, and settlement control.

Al-Najaf Al-Ashraf governorate is located in an area of Quaternary and younger sedimentary deposits related to the Mesopotamian basin and the Najaf plateau. The geological setting of the area is the result of interaction between the regional depositional processes and the structural framework of central and south Iraq. The project site is a relatively level area underlain by

Quaternary sediments, and the regional geology is also influenced by the transition between the Stable Shelf and the Mesopotamian Zone, as shown in Figure 2.

From an engineering point of view, the presence of heterogeneous sandy, silty, clayey and locally gypseous deposits can affect strongly the foundation performance. For example, soils with gypsum may experience significant changes in their engineering behaviour when in contact with water. Changes in the density of the soil and the groundwater conditions may affect the bearing capacity and deformation properties. Thus, the characterisation of the subsurface profile should be based on field investigation and laboratory testing, not only on the general regional geological information (Bowles, 1996).

The geotechnical assessment of large infrastructure must also consider the seismic setting of Iraq. Iraq is located in a tectonically active zone associated with the convergence of the Arabian and Eurasian plates. In general, northern and north-eastern Iraq is more seismically active. The central and southern parts of the country are less seismically active than the Zagros belt, but seismic loading is still an important consideration for the design of critical infrastructure.

This paper seeks to evaluate the geotechnical conditions of the site of an airport bridge junction in Al-Najaf Al-Ashraf Governorate, Iraq. The evaluation was done by field investigation and laboratory tests at the Quality Control Laboratory at the University of Kufa. The investigation involved borehole drilling, sampling, Standard Penetration Testing (SPT), groundwater observations, physical and chemical characterisation of the soils, and engineering assessment of shallow and deep foundation options. Of particular interest are the subsurface stratigraphy, soil classification, groundwater conditions, bearing capacity, settlement considerations, and suitability of bored pile foundations.

The primary goal of this study is to convert the geotechnical investigation for the project to a comprehensive scientific assessment of the engineering behaviour of the investigated soils. The results are discussed within the framework of established principles of soil mechanics and foundation engineering. Particular attention is paid to the implications of the observed soil conditions for foundation selection and construction practice (Peck et al., 1974; Terzaghi & Peck, 1967; Das, 2004).

2. Materials and Methods

2.1 Study Area and Geological Setting

The studied site is located in Al-Najaf Al-Ashraf Governorate, about 150 km away from Baghdad to the south Figure 1. The site is characterised by relatively flat terrain and is located in an area dominated by Quaternary sediments. The geotechnical investigation report shows that the regional geological succession is made up of Pliocene-Pleistocene deposits related to the Dibdibba Formation.

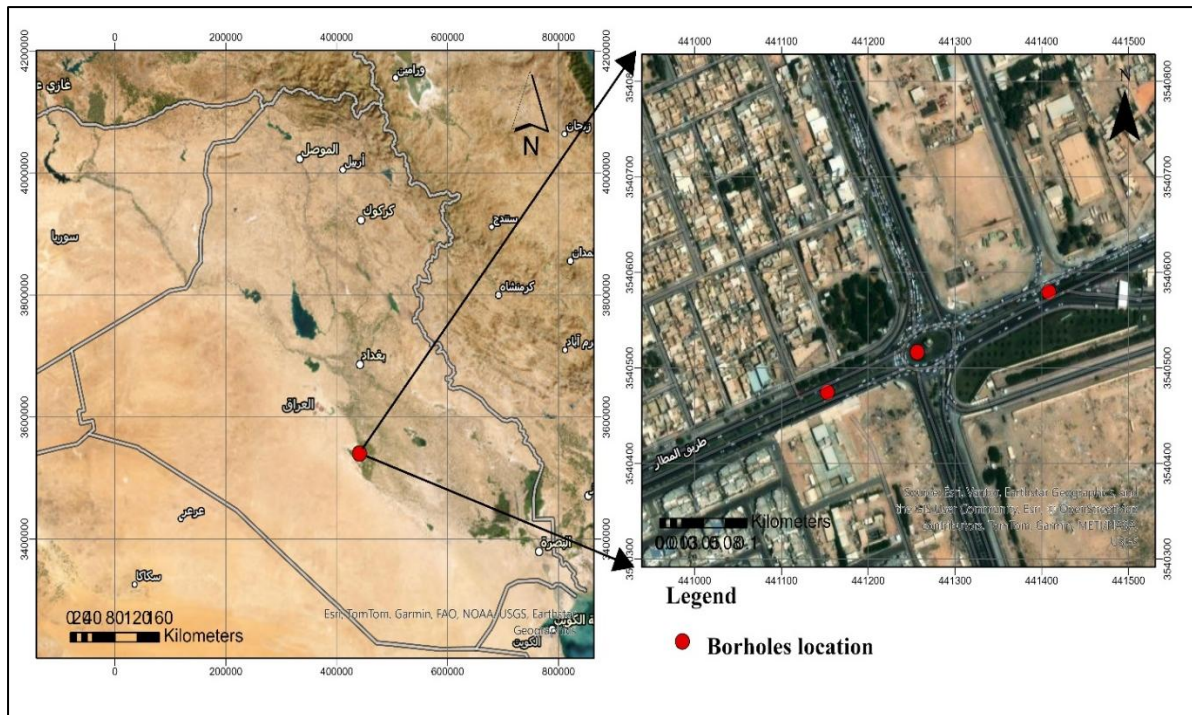


Figure 1. The location of the Airport Intersection Bridge and Borholes

The Dibdibba Formation is described as being widely exposed in the upper part of Al-Najaf Governorate and consists mainly of sandstone and pebbly sandstone. The lithological characteristics of the sandstone vary spatially in colour, grain size, sorting, roundness and sphericity. The main components are quartz, chert, rock fragments and feldspar. The cementing material can be clayey, calcareous or gypsiferous according to the local lithology. The project area is described as being located near the boundary between the Stable Shelf and the Mesopotamian Zone Figure 2.

The regional geology provides a useful background to the interpretation of the engineering behaviour of the soils encountered during the site investigation.

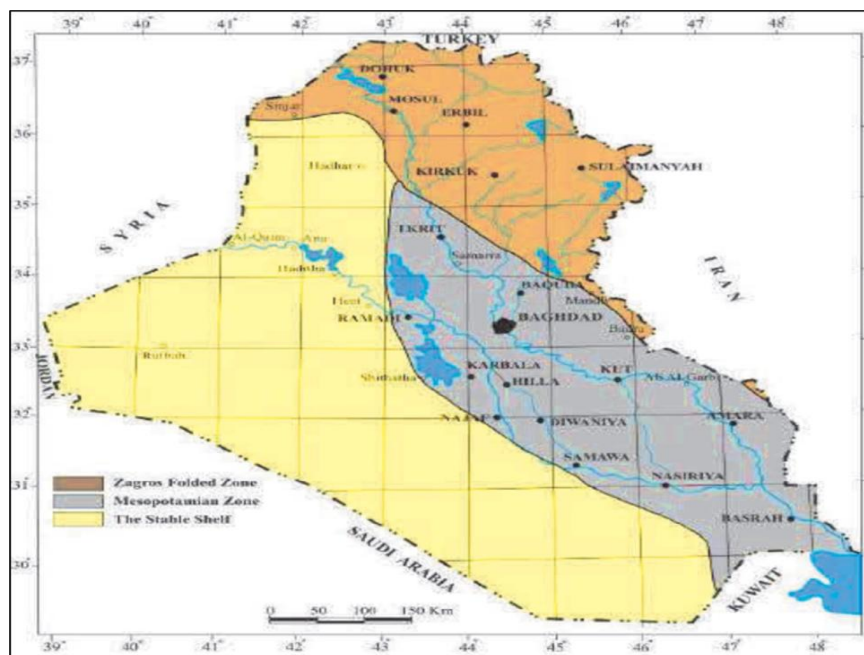


Figure 2: General Geological Setting of Iraq

2.2 Field Investigation

The field investigation was conducted from 6 to 11 September 2025. Borehole drilling was performed using a mechanical flight-auger drilling system according to ASTM D1452 and ASTM D5783. The borehole diameter reported was approx. 100 mm.

The investigation report states that three boreholes were investigated and are named BH1, BH2, and BH3. The reported depths were 17 m for BH1 and 21 m for BH2 and BH3, respectively, resulting in a total drilled depth of 59 m.

Three main sample types were obtained during the investigation:

1. Disturbed samples (DS) taken at intervals of about 0.5 to 1.5 m for soil identification and classification.
2. SPT samples (SS) taken by the standard split-spoon sampler in Standard Penetration Testing.
3. Undisturbed samples (US) taken using Shelby tubes at approximately 1–2 m intervals or where significant changes in soil strata were encountered.

The sample types and corresponding laboratory testing protocols were selected based on the nature and condition of the recovered materials.

2.3 Standard Penetration Testing

The Standard Penetration Tests (SPT) were performed at selected depths in accordance with ASTM D1586. The test used a standard split-spoon sampler with an internal diameter of approximately 35 mm and external diameter of approximately 50.8 mm.

The standard hammer mass was 63.5 kg and the nominal drop height was 760 mm. The penetration resistance was determined from the number of hammer blows needed to drive the sampler through the specified penetration intervals. The field SPT N-value was taken as the sum of the blow counts for the last two 150-mm intervals.

Corrections were considered in the investigation because measured SPT resistance may be affected by hammer efficiency, drill-rod length, sampler configuration, borehole diameter, and effective overburden pressure. The report gives the corrected SPT resistance in the following form:

$$N_{60/70} = C_N \cdot N \cdot \eta_1 \cdot \eta_2 \cdot \eta_3 \cdot \eta_4$$

Where C_N is the overburden pressure correction factor ($C_N = (95.76/\sigma'_o)^{0.5} \leq 2.0$) per Peck et al. (1974), and $\eta_1, \eta_2, \eta_3, \eta_4$ represent correction factors for hammer efficiency, rod length, sampler type, and borehole diameter, respectively. Groundwater corrections were applied to fine sand and silty layers below the water table following Terzaghi & Peck (1967) for $N_c > 15$ ($N' = 15 + 0.5(N_c - 15)$).

2.4 Groundwater Investigation

Groundwater conditions during drilling were studied. Upon completion of boring, the groundwater level was allowed to stabilise for approx. 24 h and thereafter the depth of groundwater was recorded in relation to the natural ground surface.

The reported stabilised water level in the ground was approximately 4.2 m below the ground surface. This groundwater condition is of particular relevance to the engineering assessment, as appreciable gypsum and soluble salts are present at the site and groundwater may also affect the effective stress state and durability of buried concrete elements.

2.5 Laboratory Testing

Laboratory tests were carried out on representative samples taken from the boreholes examined. The testing program involved physical, chemical, strength, consolidation, and collapse-related investigations.

2.5.1 Physical Properties

The physical characterization included:

- Natural moisture content.
- Grain-size distribution.
- Atterberg limits.

- Specific gravity.
- Natural and dry unit weight.

These tests were used to identify and classify the soil materials and to evaluate their basic index and engineering characteristics.

2.5.2 Chemical Properties

Chemical testing included determination of:

- Total soluble salts.
- pH.
- Chloride content.
- Gypsum content.
- Carbonate content.
- Organic matter.
- Sulfate content.

The physical and chemical tests were conducted according to the standards cited in the report, including BS 1377:1990 and relevant ASTM procedures.

2.5.3 Force et consolidation

The strength tests were conducted on the basis of soil type and included direct shear and triaxial tests for the sandy and clayey materials. The compressibility and settlement behaviour was evaluated by means of consolidation tests on clayey samples.

2.5.4 Potential for Collapse

Particular interest was paid to collapse behavior as the soils studied contain significant amounts of gypsum. The investigation report states that the infiltration of water into the vulnerable soil strata may cause structural damage due to the dissolution and leaching of the gypsum. The potential for collapse was evaluated using a one-dimensional wetting test as per ASTM D5333-2003. The collapse indices for a stress level of 200 kPa are ranked as follows:

Collapse Index I_c (%) Degree of Collapse (0) = None, (0.1-2.0) = Slight, (2.1-6.0) = Moderate, (6.1–10.0) = Moderate severe, and (10.0) = Serious.

This test is particularly relevant to the studied site as gypsum dissolution can lead to an irreversible volume reduction when susceptible soils are wetted under load.

2.6 Seismic Description of the Project Site

The seismic characterisation of the site was performed by merging the information from the geotechnical investigation report on regional seismic zoning, the subsurface conditions obtained from the boreholes and the Standard Penetration Tests (SPT). The seismic evaluation in the investigation report mentions that the project site falls in Seismic Zone II and the seismic hazard factor is about 0.20g. This value was used as an initial regional seismic design parameter for the site considered. The seismic classification given should be considered as a preliminary characterisation and not as a substitute for a detailed seismic hazard assessment, as there were no direct shear-wave velocity measurements or site-specific dynamic response analyses part of the available investigation.

The subsurface conditions are of special interest for the seismic response, since the investigated profile is dominantly formed of sandy materials, e.g. poorly graded sand (SP) and silty sand (SM) with intercalated fine-grained deposits at larger depths. The corrected SPT resistance generally increases with depth, indicating a transition from relatively less resistant near-surface deposits to denser and stronger deposits at depth. Such vertical variation in stiffness and density may affect the propagation and amplification of seismic waves through the soil column. Therefore, the seismic response of the site should be estimated based on its geotechnical properties rather than on its regional seismic-zone classification.

3. Results and Discussion

3.1 Subsurface Stratigraphy and Soil Classification

The field investigation showed a vertical variation of the subsurface sequence. It is mainly composed of sandy materials with limited silty and clayey horizons and significant accumulation of gypsum in the shallow zone. The soil profiles derived from the three boreholes examined suggest that the site can be broadly divided into four engineering zones: a superficial fill layer; a shallow sandy zone containing gypsum blocks; a middle zone of pale-gray to yellowish fine-grained sandy soil with pebbles; and a deeper zone comprising sandy clayey silt.

BH1 The uppermost 0.0–0.5 m consists of fill material, followed by pale-brown sandy soil containing gypsum blocks. Between about 2 and 5 m the soil is a fine-grained sand of pale brown to yellowish colour. The sandy layer with pebbles is 5 to 8 m thick, and the deep part of the borehole is represented by greyish fine-grained sandy soil.

The BH2 profile is similarly arranged in general. The upper 0.5 m consists of fill and is underlain by pale-brown to yellowish sandy soil containing gypsum blocks. Shallow to intermediate depths is a yellowish sandy soil with pebbles followed by a thick grayish-to-whitish fine-grained sandy layer. The material changes to pale-green to whitish sandy clayey silt at depth.

The results of the classification confirm that the deposits investigated are mainly sandy. Most of the intervals tested in BH1–BH3 were classified as poorly graded sand (SP) with a few deeper or finer-grained intervals classified as silty sand (SM) or SP–SM. The most deeply studied samples in BH2 and BH3 were classified as high plasticity silt (MH) and medium plasticity silt (MI), respectively. The results are summarized in Table 1.

Table 1. Summary of soil classification for the investigated boreholes

Depth (m)	USCS classification (BH1)	USCS classification (BH2)	USCS classification (BH3)
0.0–0.5	Fill soil	Fill soil	Fill soil
1.5–2.0	Poorly graded sand (SP)	Poorly graded sand (SP)	Poorly graded sand (SP)
4.5–5.0	Poorly graded sand (SP)	Poorly graded sand (SP)	Poorly graded sand (SP)
6.0–6.5	Poorly graded sand (SP)	Poorly graded sand (SP)	Poorly graded sand (SP)
7.5–8.0	Poorly graded sand (SP)	Poorly graded sand (SP)	Poorly graded sand (SP)
9.0–9.5	Poorly graded sand (SP)	Poorly graded sand (SP)	Poorly graded sand (SP)
12.0–12.5	Silty sand (SM)	Silty sand (SM)	Silty sand (SM)
15.0–15.5	SP–SM	SP–SM	SP–SM
16.5–17.0	SP–SM	SP–SM	SP–SM
20.5–21.0	-----	High-plasticity silt (MH)	Medium-plasticity silt (MI)

The table summarises ranges where multiple adjacent sample intervals were reported in the original laboratory tables. Individual values should be retained in the supplementary dataset if the manuscript is submitted with a full data appendix.

3.3 Shear Strength Characteristics

The direct shear tests imply that the sandy deposits are mainly responding in a frictional manner. In BH1, the cohesion is normally zero, and the friction angle is between approximately 30.0° and 41.2°. The maximum measured value of 41.2° occurs at 16.5–17.0 m.

The friction angles of BH2 are from about 32.1° to 40.1°. The cohesion value of 58 kN/m² is reported at 20.5–21.0 m. BH3 shows friction angles in the range of 30° to 39°. Cohesion is 55 kN/m² at 19.0–19.5 m and 40 kN/m² at 20.5–21.0 m (Table 2).

Table 2. Direct shear strength parameters

Depth (m)	BH1		BH2		BH3	
	Cohesion, c (kN/m ²)	Friction angle, ϕ (°)	Cohesion, c (kN/m ²)	Friction angle, ϕ (°)	Cohesion, c (kN/m ²)	Friction angle, ϕ (°)
1.5–2.0	0	32.4	0	33	0	32
3.0–3.5	0	30	0	31.5	0	32.1
4.5–5.0	0	31.5	0	33.8	0	31.5
6.0–6.5	0	31.2	0	34.5	0	30
7.5–8.0	0	31.3	----	----	----	----
9.0–9.5	0	35.3	0	35.5	0	31
11.5–12.0	0	32.4	11	35.7	0	35.5
14.5–15.0	0	32.1	0	32.3	0	35
16.5–17.0	10	41.2	0	32.1	0	34.5
19.0–19.5	----	----	----	----	55	39
20.5–21.0	----	----	58	40.1	40	34

The results show that the shallow sandy deposits have quite homogeneous friction angles, usually 30–36°. In the deepest fine-grained intervals, the increase in apparent cohesion is associated with a transition from predominantly sandy material to silty or clayey soil.

3.4 SPT Resistance and Depth-Dependent Soil Behavior

The corrected SPT values are one of the best indicators of the vertical variation of soil resistance at the site.

For BH1, the corrected N_{60} values are between 10 and 64, with a strong increase below about 15 m. The highest reported value of 64 occurs at 16.5–17.0 m.

Corrected values for BH2 range from 15 to 59 and generally increase with depth up to 59 at 20.5–21.0 m.

Corrected values for BH3 are reported between ~10.5 and 58, with a steep increase between ~18.5 and 19.0 m, where a value of 58 is reported Table 3.

Table 3. Field and corrected SPT values

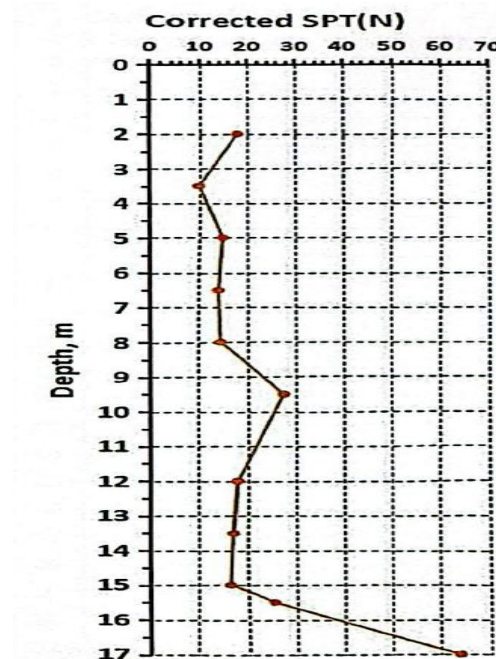
Depth (m)	BH1		BH2		BH3	
	Field N	Corrected N'_{60}	Field N	Corrected N'_{60}	Field N	Corrected N'_{60}
1.5–2.0	17	18	21	20	15	17
3.0–3.5	10	10	16	15	20	17
4.5–5.0	16	15	31	22.5	17	15
6.0–6.5	16	14	40	25	12	10.5
7.5–8.0	16	14.5		--	--	--
9.0–9.5	48	27.6	50	29	17	15
10.5–11.0	25	18	50	29	48	28
13.0–13.5	25	17.5	50	27.5	25	17.5
13.5–15.0	25	17	25	17	50	26.5
15.0–15.5	47	25.5	--	--	--	--
16.5–17.0	>50	64	--	--	--	--
18.5–19.0	--	--			>50	58
20.5–21.0	--	--	>50	59	50	24

The SPT profiles show an overall increase of penetration resistance with depth but not monotonically. This variation is important because it indicates changes in soil density and/or composition, not a homogeneous soil profile.

3.5 Graphical Representation of SPT Results

The original report has three SPT-depth plots. These should be kept in the scientific manuscript, but redrawn digitally to publication quality.

In the BH1 profile, the corrected SPT resistance is relatively moderate in the upper 15 m. The corrected SPT resistance then sharply increases to $N'_{60}=25.5$ at 15.0–15.5 m and around 64 at 16.5–17.0 m. This increase is consistent with the denser/deeper sandy material found in the borehole logs Figure 3.

**Figure 3.** Corrected SPT resistance versus depth, BH1

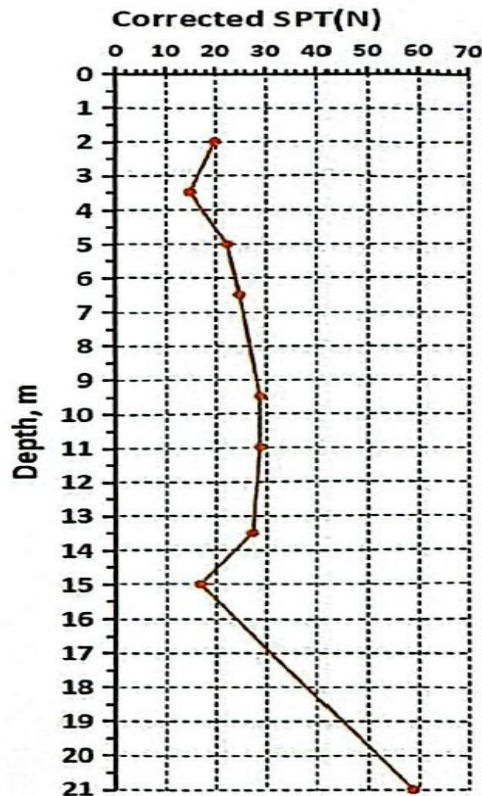


Figure 4. Corrected SPT resistance vs depth for BH2

In several shallow and intermediate intervals, BH2 shows better penetration resistance than BH1. The corrected value is about 59 at 20.5–21.0 m, which implies a significant increase in resistance at the bottom of the borehole Figure 4.

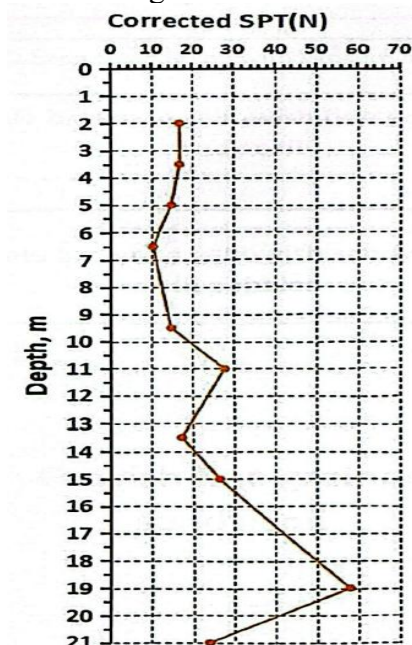


Figure 5. SPT resistance versus depth corrected for BH3

BH3 shows a significant increase in the corrected SPT resistance at approximately 18.5–19.0 m, where the N_{60}^c is 58. However, the value falls to around 24 at 20.5–21.0 m, suggesting significant local variability within the deeper deposits Figure 5.

3.6 Groundwater Conditions

Groundwater was observed at approximately 4.2 m below ground surface. This is consistent with the groundwater level in the soil-profile diagrams of BH1 and BH2.

The groundwater level is important because it intersects the shallow-intermediate sandy deposits with gypsum. The interaction of soluble gypsum, groundwater and foundation loading is an important geotechnical consideration because uncontrolled water movement can change the fabric and engineering behaviour of susceptible soils.

The relatively shallow groundwater table also affects the effective stress state used in bearing-capacity calculations and should therefore be included in the final foundation design.

3.7 Chemical Characteristics of the Soil

Chemical testing results indicate that soil chemistry varies greatly with depth and between boreholes. The most important parameters are sulphate, electrical conductivity, total dissolved solids, carbonate content, gypsum and organic matter Tables 4 & 5.

Table 4. Chemical properties of soil samples from BH1

Depth (m)	Sulfate (%)	EC ($\mu\text{S/cm}$)	pH	TDS (ppm)	CaCO ₃ (%)	Gypsum (%)	O.M. (%)
1.5–2.0	2.50	10546	6.0	6750	73.1	5.5	2.90
3.0–3.5	1.30	6875	6.5	4400	73.5	2.7	1.15
7.5–8.0	1.20	10156	7.0	6500	72.3	2.6	1.62
15.0–15.5	0.27	4062	7.5	2600	91.3	0.6	1.77
16.5–17.0	0.19	3900	7.5	2100	93.0	0.4	0.83

Table 5. Chemical properties of soil samples from BH2

Depth (m)	Sulfate (%)	EC ($\mu\text{S/cm}$)	pH	TDS (ppm)	CaCO ₃ (%)	Gypsum (%)	O.M. (%)
1.5–2.0	5.80	14533	7.8	8720	73.0	12.6	4.0
4.5–5.0	3.90	11300	8.0	6800	71.0	8.3	4.5
9.0–9.5	0.60	5300	8.15	3180	68.3	1.3	5.3
10.5–11.0	0.80	1933	8.3	1150	68.3	1.8	2.5
20.5–21.0	2.00	6800	8.3	1200	39.0	4.5	3.7

Table 6. Chemical properties of soil samples from BH3

Depth (m)	Sulfate (%)	EC ($\mu\text{S/cm}$)	pH	TDS (ppm)	CaCO ₃ (%)	Gypsum (%)	O.M. (%)
1.5–2.0	7.20	17300	8.6	10625	81.0	15.6	4.7
4.0–4.5	3.60	18350	7.9	9875	85.4	7.7	3.2
7.5–8.0	0.34	12850	8.4	7450	79.8	0.7	2.7
10.5–11.0	3.70	3825	8.17	1612	79.6	8.1	7.8
20.0–20.5	0.40	3450	8.4	1260	94.5	0.9	3.4

One feature of particular importance is the concentration of gypsum in the shallow deposits. The highest reported value is 15.6% (at 1.5–2.0 m in BH3), Table 6, 7.2% sulphate. BH2 also has a high gypsum content of 12.6% at the same depth interval.

The decrease in gypsum content with depth in several samples suggests that the effect of evaporitic accumulation is stronger in the shallow soil zone. This feature has direct consequences for the durability of foundations and the manner in which the soil can react to water infiltration.

3.8 Groundwater Chemical Characteristics

The groundwater sample associated with BH1, BH2, and BH3 exhibited the following characteristics:

Table 7. Chemical properties of groundwater

Parameter	Reported value
Sulfate	1350 ppm
Electrical conductivity	7800 $\mu\text{S}/\text{cm}$
pH	7.3
TDS	5000 ppm
Alkalinity as HCO_3	915 ppm
Chloride	2100 ppm

The sulphate and chloride contents suggest that the groundwater should be classified as chemically aggressive to buried concrete and reinforcement. The investigation therefore recommends the use of sulphate-resistant cement for pile construction and protective treatment for shallow foundation elements.

3.9 Allowable Bearing Capacity of Shallow Foundations

The report uses SPT-based correlations to evaluate shallow foundations, taking into account the failure of bearing capacity and settlement criteria (Meyerhof, 1976). The minimum width of the foundation in the raft-foundation case was taken as 10 m, Table 8.

Table 8. Allowable bearing capacity for raft foundations

Borehole	Allowable bearing capacity (ton/m^2)
BH1	14.2
BH2	14.8
BH3	14.5
Average	14.5

The three calculated values are fairly close, from 14.2 to 14.8 ton/m^2 , and indicate broadly similar bearing conditions for the assumed raft geometry. But this result must be considered with settlement and groundwater conditions.

The original report also indicates an allowable bearing capacity of about 7.7 ton/m^2 for the strip-footing condition considered. Thus, the larger raft foundation has a higher allowable pressure under the assumptions adopted by the report, in part because the larger foundation distributes the applied load over a larger area.

3.10 Bored-Pile Capacity

From the results of the site investigation, the report recommends a bored-pile foundation. The diameter of the pile was taken as 1.2 m and the pile capacities were estimated using the SPT results by the Meyerhof approach.

The report considers for compression loading:

$$Q_s = 0.1NA_s$$

$$Q_b = 1.4N \frac{L}{B} A_b$$

$$Q_u = Q_s + Q_b$$

and:

$$Q_a = \frac{Q_u}{F.S.}$$

where Q_s is the shaft resistance, Q_b is the base resistance, Q_u is the ultimate pile capacity, Q_a is the allowable pile capacity, N is the corrected SPT resistance, L is pile length, B is pile diameter, and A_s and A_b are the shaft and base areas, respectively.

Table 9. Reported bored-pile capacities for a 1.2-m-diameter pile

Pile length (m)	Average ultimate capacity Q_u (ton)	Factor of safety	Average allowable capacity Q_a (ton)
15	362.5	2.5	145
18	630	2.5	252
20	775	2.5	310

The calculated capacity in Table 9 shows a large increase with pile length. The reported allowable capacity is thus increased from about 145 tonnes at 15 m to 310 tonnes at 20 m. However, these values are analytical estimates from empirical SPT correlations. The original report clearly recommends a field pile-loading test in accordance with ASTM D1143 to determine the actual ultimate load and settlement response of the pile. It is suggested to keep this distinction in the scientific manuscript to separate calculated capacity from experimentally verified performance.

3.11 Integrated Geotechnical Interpretation

The combined results suggest there is a large amount of vertical and horizontal variation in soil properties at the site. The shallow deposits are mainly sandy, locally with high content of gypsum, whereas the deeper materials possess higher SPT resistance and locally contain silty or sandy clayey horizons.

The SPT profiles indicate that generally the soil resistance increases with depth, but locally sudden changes occur. This is in agreement with the borehole logs of stratigraphic variability. The most important engineering problem is not the calculated bearing capacity. The most important problem in engineering is the interaction between the type of soil, the gypsum content, the groundwater, and the load of the foundation. The high content of gypsum, especially in the shallow zone, indicates that uncontrolled water infiltration may be a source of ground deformation. The groundwater level of about 4.2 m suggests the need for appropriate water management.

The allowable pressures reported for shallow foundations are preliminary design values, but the relatively high gypsum content and chemical aggressiveness suggest that additional precautions are needed. For heavily loaded bridge foundations, the calculated bored-pile capacities indicate that increasing the pile length gives a substantial increase in calculated axial capacity. However, final pile dimensions and lengths should be verified by field load testing.

3.12 Original Soil-Profile Figures

The following figures from the geotechnical report should be included in the manuscript by digital redrawing and not by scanning images.

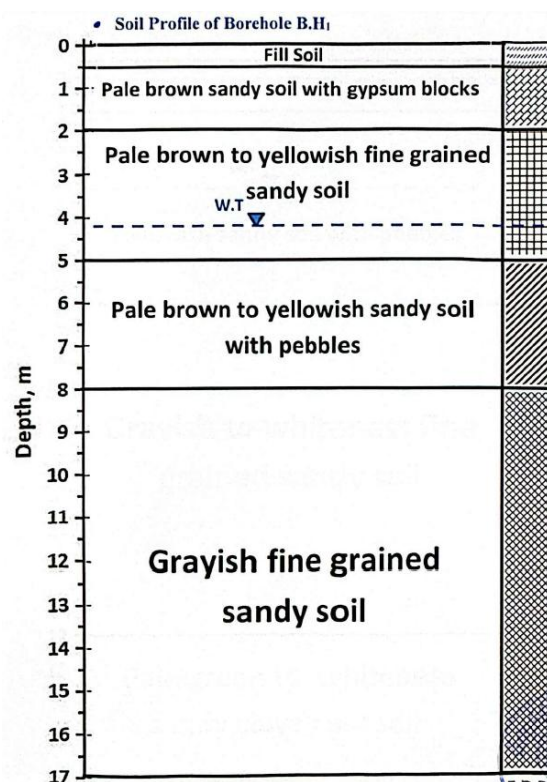


Figure 6. Soil profile for borehole BH1.

Figure 6 for BH1 profile is fill soil near the surface, pale-brown sandy soil with gypsum blocks, pale-brown to yellowish fine-grained sandy soil, sandy soil with pebbles and deeper greyish fine-grained sandy soil. The groundwater table lies at about 4.2 m below ground level.

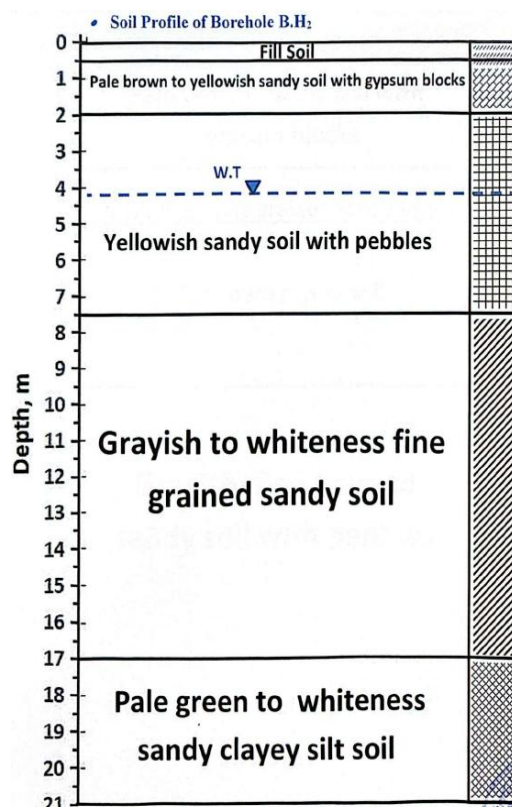


Figure 7. Soil profile Borehole BH2.

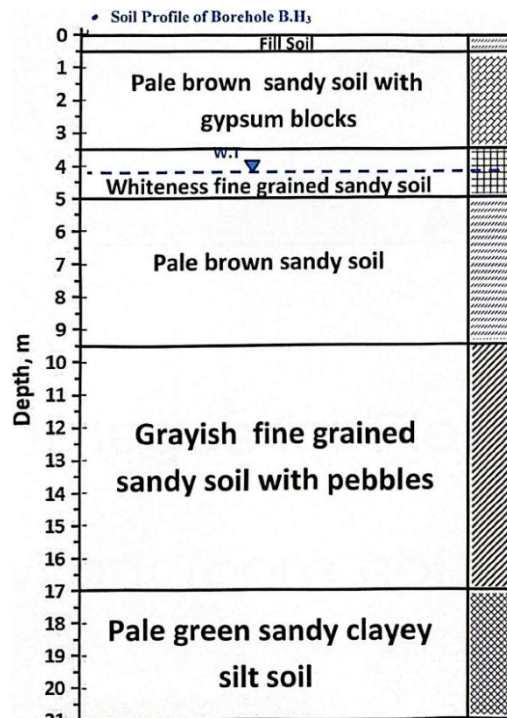


Figure 8. Soil profile of Borehole BH3.

The profile of BH2 consists of fill, pale-brown to yellowish sandy soil with gypsum blocks, yellowish sandy soil with pebbles, grayish-to-whitish fine-grained sandy soil, and pale-green to whitish sandy clayey silt at greater depths, as shown in Figure 7. The groundwater level is estimated to be about 4.2 m. The profile of BH3 shown in Figure 7 consists of the same sequence as the profile of BH2.

4.13 Seismotectonic setting and seismic site conditions

The seismicity of the investigated bridge site should be considered in the broader tectonic setting of Iraq and the northeastern Arabian Plate. Iraq is located on the northeastern margin of the Arabian Plate, where the Arabian Plate meets the Eurasian Plate. The regional tectonic regime is mainly controlled by the Arabian–Eurasian continental collision and the associated Zagros Fold–Thrust Belt, which is the main active deformation and earthquake-generating zone in the region. Recent studies of the Iraqi earthquake catalogue reveal that the earthquake activity is mainly concentrated in the Zagros Fold–Thrust Belt, decreasing toward the Mesopotamian Foredeep and the Inner Arabian Platform (Abdulnaby et al., 2026). (OUP Academic) The studied site is situated in the central part of Iraq, where the regional geological setting is related to the Mesopotamian basin and its thick Quaternary sedimentary succession. The seismicity in this part of Iraq is much lower than in the Zagros Fold-Thrust Belt. But the relatively lower level of seismic activity does not mean that there is no seismic hazard. Probabilistic seismic hazard studies in Iraq have identified the Mesopotamian Foredeep as a distinct seismic source region and have shown that earthquake sources in and around this region should be taken into account in seismic hazard assessments (Onur et al., 2017). According to the Iraqi Seismic Code (2017), the site is classified as a Seismic Zone IV with a preliminary seismic coefficient of about 0.20 g. This value was taken as the regional seismic design parameter of the investigated site in the report. The report also assesses the soil profile by using SPT-based criteria, and the site is classified as Site Class D, which corresponds to stiff-soil conditions. The Iraqi Seismic Code classifies the sites based on the shear-wave velocity, SPT resistance, and undrained shear strength. Site Class D is defined as stiff soil conditions (Iraqi Seismic Code, 2017), Figure 9.

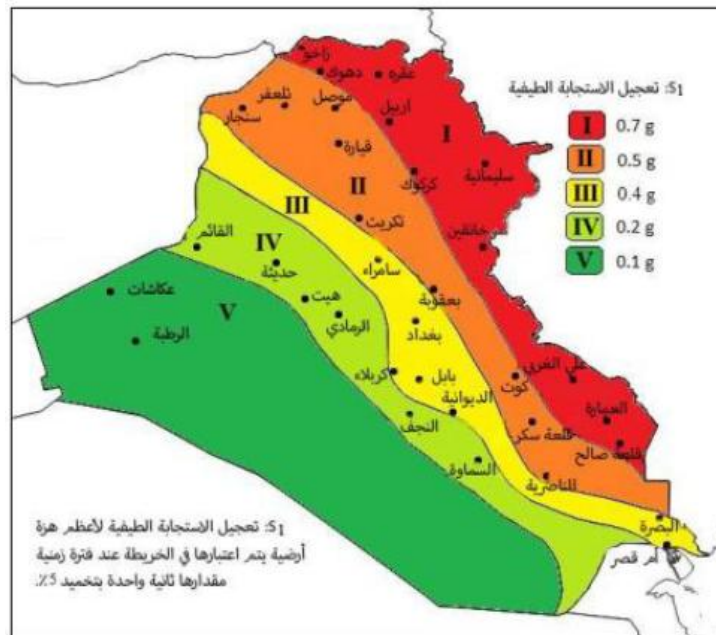


Figure 9: The Iraqi seismic codes map (ISC 2017).

4.14 SPT-Based Seismic Site Classification

The seismic classification of the site is directly related to the measured SPT resistance. The boreholes examined indicate a general increase in penetration resistance with depth, though local variations exist between the boreholes. Corrected SPT values vary from about 10 to 18 blows in some shallow intervals to higher than 50 blows in the deeper and denser deposits. This trend indicates that the subsurface profile becomes denser and mechanically stronger with depth.

The site corresponds to Site Class D (stiff soil) according to the seismic site-classification criteria adopted in the investigation. The classification is consistent with the general range of corrected SPT resistance in the profile investigated. Likewise, the Iraqi Seismic Code defines Site Class D as a stiff-soil class with intermediate shear-wave velocities and SPT resistance typically in the range of about 15–50 blows, depending on the classification criterion used (Iraqi Seismic Code, 2017).

However, note that the present classification is based on geotechnical investigation data mainly and does not incorporate direct measurement of shear-wave velocity. Therefore, for the final seismic design of the bridge, a direct geophysical determination of the average shear-wave velocity in the upper 30 m (V_{s30}) would provide a more robust characterisation of the dynamic response of the site.

4. Conclusions

The combined interpretation of the field and laboratory investigation results leads to the following conclusions:

1. The studied site is covered by a heterogeneous sequence of Quaternary deposits dominated by sandy soils with localised silty and clayey materials.
2. Appreciable gypsum is present in shallow soil profile with maximum value of 15.6% in BH3 at 1.5–2.0 m depth.
3. Soil is mostly SP and SM, deeper fine-grained materials are MH in BH2 and MI in BH3.
4. The corrected SPT resistance generally increases with depth. The values of 64 in BH1, 59 in BH2 and 58 in BH3 have been reported at selected deeper depths.
5. The water table is about 4.2 m below the ground surface, resulting in a significant interaction between groundwater, soluble minerals and foundation materials.

6. Direct shear testing shows mainly frictional behaviour of the sandy deposits with friction angles in the range of about 30° to 41.2°.
7. The permissible bearing capacity of raft foundation is calculated to be in the range of 14.2 to 14.8 ton/m² with an average of 14.5 ton/m².
8. The reported allowable capacities for the investigated 1.2-m-diameter bored piles increase from 145 tonnes for 15 m pile length to 310 tonnes for 20 m.
9. The sulphate concentrations in both soil and groundwater are high and concrete durability measures are necessary including the use of sulphate resistant cement as advised in the original investigation.
10. Before final pile design, full-scale pile load testing shall be performed in accordance with ASTM D1143 since pile capacities are based on empirical SPT correlations.
11. The studied site is located in the central part of Iraq where the seismic activity is relatively less than the Zagros Fold-Thrust Belt, but the seismic effects are still important for the design of the proposed bridge. From the investigation report, the site falls under Seismic Zone II with a preliminary seismic coefficient of about 0.20g. The soil profile is classified as Site Class D based on the available geotechnical information. The sandy deposits, the variable SPT resistance and the presence of groundwater at about 4.2 m depth indicate the need of considering local soil amplification and the possibility of pore pressure generation due to earthquakes. A site-specific seismic investigation including direct Vs measurements and quantitative liquefaction assessment is recommended to support the final structural and foundation design.

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