Geotechnical
& Material
Testing
GMT Center

SITE INVESTIGATION REPORT

For The Proposed

HEBRON COURTHOUSE

LOCATED IN

HEBRON - PALESTINE

Report No.SI 09/ 0441

SUBMITTED TO:

CIDA

SUBMITTED BY:

GEOTECHNICAL & MATERIAL TESTING CENTER
(GMT)
A SUBSIDIARY OF
ENVIRONMENTAL & CIVIL
ENGINEERING STUDIES CO.

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Date : January ,20th /2010
Our Ref.: Report SI 09/0441

Messers. CIDA

Subject: Site Investigation Report for the Proposed Hebron Courthouse

Dear Sir,

It is of our pleasure to submit you this geotechnical report for the site mentioned above. This investigation was carried out according to your request.

This report includes the results of field investigation, laboratory results, and the required conclusions recommendations needed for the design & construction of the most suitable and economical foundation.

For any further information or clarifications, please don't hesitate to contact us.

We would like to thank you for your condience, hoping to cooperate with you in the near future.

Yours Sincerely,

Eng. Mahmoud Abdallah
General Manager

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SUMMARY OF INVESTIGATION

The purpose of this report is to investigate and provide reliable, specific and detailed information about the physical and mechanical properties, Such as seismic conditions, site geology and any instalations within the investigated area. The investigation included four boreholes, one borehole with 20.0m deep, one with 15.0m and two boreholes 6.0 m deep each.

The subsurface investigation revealed that the material at the site can be described as " a layer of fill material with variable depth (5.0-12.0)m, then a layer of very pale brown fractured weak lime stone up to the end of borings" as shown in the logs of boring in **(Appendix A)**

The ground water was not encountered within the drilled depth.

In addition to conventional site investigation with boring method, two geophysical survey methods were conducted by seismography survey and electrical resisitivity. Both method showed continous layers without cavities or geological faults.

The investigated site located at a side of slopy area. The seismic zone is 2A with 0.15 g value for ground acceleration .

The table below is including the summary of the obtained values and describtion of findings and recommendations.

Table 1: Summary of results

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8.9	Soil Clasification According to UBC for earthquake design	Type S_B soil	27
8.11	Inspection of the Foundation Ground, and Verification	After the foundation excavation and before the construction of foundation, we should be notified in order to inspect the foundation ground.	27
	Electrical Resistivity	Form the above sounding points and profiles it is clear that the site lithology composed of successive layers of limestone and marly limestone. Debris depth reached about 10 m depth in Profile (I). There are absolutely no indications of any types of caves (cavities) within the rock	Appendix C

		layers. The lithologies of these three profiles are composed form limestone and marly limestone.	
	Seismic Investigation	Based on the outcropping geological cross-section in the study area and the ground velocity models deduced from the P-waves velocities of this study, the subsurface geological formations beneath the profiles P1, P2, P3, P4 and P5 are interpreted as non consolidated sediments of poorly sorted gravels, alluvial and debris which form the first two layers with a depth of 5-12 meters. And the third layer is explained as consolidated carbonates of limestone dolostone and chalk. The first two layers are highly recommended to be totally removed, and the excavation should reach the third layer.	Appendix D

1. INTRODUCTION

This report is prepared upon the request of ***CIDA.*** It includes the final results of the geotechnical site investigation, the laboratory tests and **conclusions & recommendations** for the proposed ***Hebron courthouse*** located in Plot # ????(part of 130), Basin # (34406) Sibta – Hebron City - Palestine.

2. ABOUT THIS STUDY

2.1 Purpose of the study

The purpose of this site investiagtion is to determine the existing soil profiles and engineering charateristics of the subsurface conditions at the site and to provide the designer with comments on the following:

- Suitable footing types, founding depths and geotechnical design parameters which will be required for a safe and economic design and excavation of the engineering works, such as the soil bearing capacity, expected foundation settlement, side slope stability, hydorlogical conditions at the site and other special recommendation which depends on the site nature.
- Methods of construction of foundation and footings, site seismity characters, groundwater conditions, quality controle requirements and outdoor subgrade and soil retaining parameters.

2.2 Scope Of Investigation:

The scope of investigation for this study comprices the following:-

1. Collecting information such as geological and geotechnical maps related to the project site, public services, and land use maps.
2. Making visits for site reconaisance in order to collect information about site nature, topography of the site, geological features and other properties concerning the project site.
3. Drilling of ***four*** boreholes and sampling of disturbed and undisturbed samples.
4. Conducting geophysical investigation by electrical resistivity test and seismography analysis.
5. Performing all necessary field and laboratory tests, including chemical test, to obtain physical, chemical and mechanical properties of the subsurface soil. This leads to the geological description of the obtained materials.
6. Applying engineering analysis and evaluation of field findings and laboratory results.

7. Developing conclusions and recommendations concerning design and construction of the most safe and economical foundations, site preparation, road and parking areas and retaing walls.
8. Submitting this report.

3. SITE RECONAISANCE

3.1 General:

The proposed Courthouse will be constructed on a plot of about (5012m2). The plot survey is shown in **Figure No. 1.** It is locateded at steep terrain area. In the nearby street, electrical supply is passing, and water supply is existing. The southern street adjacient to the plot are paved with asphalt layers with 16 m width. The neighboring lots are still empty except of the eastern side where where a security employed building is located.

3.2 Site location

The site lies in Plot # (part of 130), Basin # (34406) Sibta – Hebron City - Palestine.
According to surveying map prepred by our subcontractor surveyor, the coordinates of the four corners surrounding the plot are as follows:-

Corner	Y-Coordinate	X-coordinate
North – east	158985	105090
North – west	158875	105090
South – east	158875	105025
South – west	158985	105025

The plot survey is shown in Figure No. 1.

3.3 Site Topography

The plot is located at a side of steepy sloping area which sloped from south side toward north side the site is dumped with fill material up to approximately horizontal level to the adjacent front street. In general, the accessibility for the drilling work around the plot was possible, but it was impossible to conduct drilling inside the plot due to unstable fill.

3.4 Site Geology

Referring to the available geologic map of Israel (former Palestine), scale (1:50,000), printed by the Geological Survey of Israel in 1989, the following geological properties can be summarized:

1) The project site lies within *Judea Group*.

2) **Formation:** *Derorim,Shivta and Nezer formation.*
3) **Composition of the formation:** *Lime stone, marl, dolostone.*
4) **Geological Age:** *Turonian.*
5) **Surface faults:** *No faults were observed within the tested area.*

A geologic map of the site, which is extracted from the geological map of Ramallah, is shown in **Figure No. 2.**

Figure No. 1
Site Plan and Borholes Location

Figure No. 2
Site Geology

4. GEOTECHNICAL EXPLORATION

4.1 Number and Depth of Boreholes:

4.1.1 General

The number and disposition of the borings should be such as to reveal any major changes in thickness, depth or properties of the strata to be affected by the expected works and immediate surroundings.

Exploration, in general, will be carried out to a depth up to which the increase in pressure due to structural loading is likely to cause perceptible settlement or shear fialure. Such a depth, known as the ***significant depth***, depends upon the type of structure, its weight, size, shape and disposition of the loaded areas, and the soil profile and its properties. It is generally safe to assume the significant depth up to a level at which the net increase in vertical pressure becomes less than 10% of the initial overburden pressure. Alternatively, a pressure bulb bounded by an isobar of one-fifth or one-tenth of the surface loading intensity is sometiems assumed to define the minimum depth of exploration. This depth may be assumed to be equal to one-and a half to two times the width (smaller lateral dimension) of the loaded footing area.

4.1.2 Code Requirements

The Jordanian code of site investigation No. 3-92 is the applicable code in our case. In refference to **Table No. 1 page 18** in the mentioned code, the minimum required number and depth of boreholes is illustrated upon the area of ground floor and the number of floors to be expected.

The following Table shows the finished drilling program for the project site:

Borehole #	Depth (m) (1)	Date of drilling	Location (2)	Approximate MSL (3)
BH-1	15.0	13/9/2009	See Fig. No. 1	947.0
BH-2	20.0	13/9/2009	See Fig. No. 1	947.0
BH-3	6.0	14/9/2009	See Fig. No. 1	932.0
BH-4	6.0	14/9/2009	See Fig. No. 1	931.0

Notes:

(1) All depths are below the existing ground level.
(2) The borehole locations as will as their drilling depths were specified by **GMT** Center and shown in Fig. No. 1.
(3) **MSL** is the mean sea level where the elevations for the boreholes were taken with refference to the topography level.

The drilling was performed using ***Mobile Rig (B-31)*** by applying rotary drilling method.

4.2 Sampling:

Samples were taken through the whole drilled depths of boreholes. Disturbed and undisturbed samples were taken by applying down the rotary percusion tools. The samples were then brought to our laboratory, described, classified and tested. All samples were placed in waterproof plastic bags in order to conserve their moisture contents. The description and classification were carried out according to **ASTM standards**, the soil/rock color was determined using the standard soil color charts.

5. LABORATORY TESTING & RESULTS

5.1 Laboratory Testing:

After classification and carrying out the geological description on the obtained samples, a laboratory tests program was issued; this program contained the required tests on selected samples in order to determine the physical and mechanical properties of the ground materials. Where applicable, The performed tests where performed according to **American Society for Testing and Materials (ASTM)** Standard as follow:

Table No.2: Common Soil Laboratory Tests Used in Geotechnical Engineering

Type of condition (1)	Soil Properties (2)	Specification (3)
Index Tests	Classification Particle size Atterberg limits Water (or moisture) content Wet density Specific gravity Sand equivalent (SE)	ASTM D 2487-93 ASTM D 422-90 ASTM D 4318-95 ASTM D 2216-92 Block samples or sampling tubes ASTM D 854-92 ASTM D 2419-95*
Settlement	Consolidation Collapse Organic content Fill compaction: Standard Proctor Fill compaction: Modified Proctor	ASTM D 2435-96 ASTM D 5333-96+ ASTM D 2974-95 ASTM D 698-91 ASTM D 1557-91
Expansive Soil	Sell Expansion index test	ASTM D 4546-96 ASTM D 4829-95 OT UBC 18-2
Shear strength for slope movement	Unconfined compressive strength Unconsolidated undrained Consolidated undrained. Direct shear Ring shear Miniature vane (i.e., torvane)	ASTM D 2166-91 ASTM D 2850-95 ASTM D 4767-95 ASTM D 3080-90 Stark and Eid, 1994 ASTM D 4648-94
Erosion	Dispersive clay Erosion potential	ASTM D 4647-93 Day, 1990b
Pavements and	Pavements: CBR	ASTM D 1883-94

deterioration	Pavement: R-value Sulfate	ASTM D 2844-94 Chemical analysis
Permeability	Constant head Falling head	ASTM D 2434-94 ASTM D 5084-97+

* This specification is in the ASTM Standards Volume 04.03.
** These specifications are in the ASTM Standard Volume 04.03. All other ASTM standards are in Volume 04.08.
*** Other tests will be defined depending on the clients and engineers requirement as shown in attachment V if applicable.

5.2 Laboratory Tests Results:

The results of the laboratory tests are summarized in **Appendix (A).**

6. GENERAL SITE PREPERATION:

The first step in site construction work is the grading of the site.
Grading consists mainly of cutting or filling of the ground to create a leveled building pad upon which the structure can be built.
The steps of grading operation can be summarized as:

- **Easments**: the first step in grading operation is to determine the location utilies of the site, if there is any utilies it should be protected so that it will not be damaged during the grading operation.
- **Clearing**: brushing and grupping to remove any vegetation, stockpiled and then removed from the site.
- **Clean out**: the unsuitable material at the site should be remved.
  In our case the encountered material at the site is not suitable for structural use, so the cut material should be transported out side the site.
- **Scarifying and recompaction**: in areas where replacement will be used, the existing ground material shouuld be scarifying and recompact to get a good bond between the inplace material and compacted fill.

In our case, it is deserved to notice that the site contains a huge quantity of depris and loose material which should be removed totally from the site before the start of construction, because if footings is constructed on this fill it will be subjected to a large amount of settlement.

7. SURFACE AND SUBSURFACE CONDITIONS

7.1 Properties of Ground Materials:

According to our exploration, findings, and the geological description for the obtained samples, there are general similarities and continuities of the subsurface materials, however some local variations were noticed along the drilled depths as illustrated by the general section through the boreholes.

For the ***four boreholes***, a longitudinal generalized subsurface section was constructed, as presented in **Figures No. 3**. This section links the eight boreholes.

The profile was constructed by direct interpolation between the materials encountered in the boreholes. The lines connecting the various ground strata are made for illustration purposes only and are not to be considered as actual field conditions.

7.2 Ground Water and Cavities:

Water Table is defined as underground border between the ground in which all spaces are filled with water and the ground above in which the spaces contain some air. The level of the water table tends to follow the shape of the overlying ground surface, rising under hills and dipping in valleys, but with a gentler slope than the ground. The level of the water table also vaires with the climate, rising during rainy periods and falling during dry season.

At the inspected site, neither ground water nor cavities were encountered. The samples were obtained continously without any interruption in sampling.

7.3 Expansive Soil and Swell Properties

In our case the encountered natural material at the site which can be described as "very palle brown fractured weak lime stone" is not viable for swell action.

Figure No. 3 Generalized Subsurface Longitudinal Section

8. CONCLUSIONS AND RECOMMENDATIONS

According to the field exploration, laboratory testing, subsurface conditions, and engineering analysis, it can be concluded that the existing natural ground at the investigated site can support the expected building loads, provided that the following recommendations are applied:

8.1 Foundation Ground,Depth & Type:

8.1.1 Foundation Ground:

Basen on our findings and the encountered material, it is recommended that the foundations of the proposed building will be laid on **"A layer of very pale brown to pale yellow fractured weak lime stone and marl stone".** This bedding layer is located far below the existing top level of the investigated lot.

In all cases all the **friable and loose** materials should be removed before laying the foundations, this means that the foundation depth may vary according to the quantity of these undesirable materials. Differences in foundation depth for adjacent footings can be filled with lean concrete. Reinforced wall beams are also recommended where big differences in cut levels is occurred.

The description for the boreholes log are as follows:

- **BH # 1 (15.0)m**
  A layer of pale yellow fill material with stone pieces up to a depth of (5.5) m, then a layer of pale yellow fractured weak lime stone with marl filling the fractures up to the end of boring at a depth of (15.0)m.

- **BH#2 (20.0)m**
  A layer of fill material with stone pieces up to a depth of (3.0) m, then a layer of pale yellow weak marl stone up to a depth of (4.0)m, then a layer of very pale brown fractured weak lime stone with traces of chalky material up to a depth of (6.5)m, then a layer of pale yellow fractured weak lime stone with marl filling the fractures up to a depth of (15.0) m, then a layer of pale yellow weak lime stone up to a depth of (16.5) m, then a layer of pale yellow fractured weak lime stone with marl filling the fractures up to the end of boring at a depth of (20.0)m.

- **BH#3 (6.0)m**
  A layer of pale yellow fractured weak lime stone with marl filling the fractures up to a depth of (0.5)m, then a layer of pale pale brown fractured weak lime stone with marl filling the fractures up to the end of boring at a depth of (6.0)m.

- BH#4 (6.0)m

  A layer of light reddish silty clay material with stone pieces up to a depth of (1.0) m, then a layer of very pale brown fractured weak lime stone with marl filling the fractures up to a depth of (1.5)m. then a layer of pale yellow fractured weak lime stone with marl filling the fractures up to the end of boring at a depth of (6.0)m.

8.1.1.1 Design of on-site road ways and Parking area:

In our case, any organic material, topsoil, softened or disturbed soils should be removed from the road sub grade to a depth of minimum (100) cm. The exposed subgrade layer should be rolled with a loaded tandem axle truck or heavy roller. Any soft or distressed areas identified should be sub-excavated. Sub-excavated areas and raised areas should be backfilled with selected suitable materials that meet requirements of Jordanian specifications for roads construction, (simillar to AASHTO). Backfills placed within the upper (1) m of the roadway should be compacted to 95% Modified Proctor maximum dry density and the top layer of the sub-grade surface should be compacted to 98% of modified proctor.

In our case the encountered material (except debris and fill that encountered at the top 5.0 to 12.0 m), if crushed it can be used as subgrade and aggregate base course for multible construction purposes.

8.1.1.2 Design of structural fill:

The imported structural fill is to be granular medium to coarse, grained very low plastic, free drainage, compactable, and within the following gradation:

Maximum size, by sieve	6"
Passing sieve #4	40%-85%
Passing sieve #40	30%-70%
Passing sieve #200	5%-15%

During the placement of any structural fill, it is recommended that a sufficient amount of field tests and observation be performed under the direction of the Geotechnical Engineer. Any areas of fill or sub-grade instability encountered during the construction are to be immediately brought to the attention of the Geotechnical engineer so that recommendations for stabilization can be given.

8.1.1.3 Coefficient of Subgrade Reaction (Ks):

For the encountered material, the modulus of subgrade reaction could be only estimated at this stage. If Accurate ***Ks*** will be requested, it could be obtained from the plate bearing test to be implemented after the excavation on the actual foundation ground level.

Because the scale and height of the designated building are comparatively small and low height.The estimated Ks is (140000) KN/m3.

8.1.1.4 Excavation support and retaining:

The suitable type of retaining depend on the hight of the backfill, generally the cantilever retaining wall is suitable up to a hight of (8)m.
Initial sizing of the retaining wall can be as shown in figure below.
The wall should be checked for stability against sliding, over turning and bearing capacity.

Table No.3 – Unfactored Retaining Wall design Parameters

Parameter	Symbol	Granular fill
Unit weight	γ_b	18KN/m³
Angle of friction	Φ	33°
Angle of wall friction	δ	20°
Active earth pressure coefficient	K_a	0.33
At-rest earthpressure coefficient	K_o	0.5
Passive earth pressure coefficient	K_p	3.0

8.1.1.5 Conventional Slab-on-Grade Foundation:

- After removing the fill material, a layer of compacted aggregate base coarse (25) cm is added to compensate for the excavated depth.
- To isolate the SOG, at the top of compacted aggregate base coarse layer, two folds of polyethylene plastic is used to protect the slab from migreated water.
- The indoor slab is recommended to be at least (10) cm thick reinforced with a mesh of (10) mm bars at (20) cm both direction.

- If the area of the slab is large it is recommended to provide an expantion joint every (5) m, or every 25m^2 area.
- For the outdoor and indoor parking slab the thickness will be at least (12)cm and a mesh of (10)mm bars at (20)cm in both direction is used. Top and bottom mesh reinfocement is recommended for the trafic pathways and parking lot slabs.

8.1.2 Foundation Depth:

8.1.2.1 General

Foundation must be located properly so as not to be adversely affected by outside influence (adjancent structures, water, frost action, significant soil volume change, underground defects). Thus the depth and location of foundations depend on:

(a) Depth of the bearing stratum.
(b) Frost action.
(c) Ground water location.
(d) Existence of soil which exhibit volume change.
(e) Adjacent structures.
(f) Underground defects (caves, utility pipes).

8.1.2.2 Recommended Depth of Foundation

The foundation depth could vary due to different levels in the site and differences in the depth of the bearing layer. In our case the site is covered by a thick layer of fill material, thus, the footing depth will vary and should be laid on the natural bearing layer at a depth (5.0-12.0)m and excavated at least (1.0) m in the natural rock bedding.

8.1.3 Foundation Type:

8.1.3.1 General:

Foundation can be defined as that part of the structure, which transmit the building load to the foundation soil in a way so that the supporting soil is not over stressed and does not undergo excessive settlement.
Foundation generally can be divided into two major groups:

(a) **Shallow foundations:** which is located at shallow depth (usually depth of the footing is less than two times the footing width) below the ground surface.

This type of footing is used if a good bearing stratum is located at shallow depth.

Example of these footing are: ***spread footing, combined footing, wall footing & mat foundation.***

(b) **Deep foundations:** used when the upper layer is weak and there is a good bearing layer or bed rock at a great depth.

Example of these footings are: piles and piers foundation.
Any how, the foundation type should be defined by the A/E designer depending on many factores, of which the main importants are the following:

1. The foundation material type and bearing capacity.
2. Type of structure and structural system.
3. The value of imposed loads on footings.
4. The siesmic charactersitics of the location.

8.1.3.2 Recommended Type of Footing

It is up to the engineer to decide his best alternative regarding the above mentioned factors from the following types:

- Spread **footings with tie beams.**
- **Strip footing** can be used where surrounding walls bearing directly on the foundation ground, or,
- A **combination** of the above recommended types can be used properly.

8.2 Allowable Bearing Pressure

For the recommended foundations ground, and making use of the unconfined compression test results of the undisturbed core sample. The following relationship is suggested by the Jordanian Code for "Footings & Retaining Walls".

$$Q'ult = y\ q_{unc}^{(1)}$$

Where: q'ult: Reduced ultimate bearing capacity.
Qunc: Unconfined Compressive Strength for the core specimen.
y : 5 + (RQD-25) (0.3), with max RQD = 75%.

RQD = Rock Quality Designation (in percent).
q_{an} = q'_{ult}./ F.S.
q_{an} = Allowable Net Bearing Pressure, kg/cm^2
F.S. = Safety Factor depending on RQD value.
F.S. = 20 – (RQD – 25) 0.3.

It is common to use building code values for the allowable bearing capacity of rock, however, geology, rock type, and quality (as RQD) are significant parameters which should be used together with the recommended code value. It is common to use large safety factors in rock capacity. The F.S should be somewhat dependent on RQD as shown above.

Making use of the calculated unconfined compressive strength of the encountered material and comparing the value obtained with that given in **(Table No. 4)**, an

analysis was done, taking into account the fractured nature of the encountered material, the strength description for the obtained samples and the scale effect.

8.2.1 Bearing Capacity:

In our case:

The allowable bearing capacity for the natural bearing layers is (3.5) Kg/cm^2.

Important Notes:

- For the proper selection of the bearing layer, please see logs of boring.
- This value of bearing pressure is valid, provided that the foundation ground, depth and type conform to that given above. (Section 8.1).

Note: No core samples were extracted from BH#1&2 because these boreholes are within the fill area, so there is possibility of failure of the sides of the hole if we try to take core samples, while BH#3&4, it was drilled in a hurry since there was some problems and tension with the people at the site.

Table No. 4 – Indices describing rock material

B. Rock material strengths

Term	Unconfined Strength (Mpa)	Compressive (p.s.i.)	Field estimation of strength
Very strong rock	> 200	> 30000	Requires many blows with geological pick to break intact sample.
Strong rock	100 – 200	15000-30000	Hand-held sample breaks with one firm blow with hammer end of geological pick.
Moderately strong rock	50 – 100	75000-15000	Knife cannot scrape or peel surface, shallow indentation under firm blow from hammer end of geological pick.
Moderately weak rock	25 – 50	35000-7500	Shallow cuts or scraping with difficulty with knife, pick point of hammer indents deeply with firm blow.
Very weak rock	1 – 25	150-3500	Can be cut with knife, crubles under sharp blow with pick point of hammer.
Very stiff soil	0.6 – 1.0	80 – 150	Very tough, difficult to move with hand pick, pneumatic spade required

			for excavation.
Stiff soil	0.15 – 0.6	20 – 80	Cannot be molded with fingers, or cut with hand spade, required hand picking for excavation.
Firm soil	0.08 – 0.15	10 – 20	Very difficult to mould wint finger, indented with finger nail, difficult to cut with hand spade.
Stiff soil	0.04 – 0.08	5 – 10	Mould with strong pressure from fingers, shown faint marks.
Very soft soil	< 0.04	< 5	Easily moulded with fingers, shown distinct heel marks.

Notes: (a) The compressive strengths for soils are double the unconfined shear strengths.
(b) Strength values are those given by Hock and Bray (1981).

8.3 Foundation Settlement:

Settlement is of concern same as the bearing capacity and most test effor is undertaken to determine the in-situ deformation modulus E and Poisson's ratio so that some type of settlement analysis can be made.

Different sources of settlement include:

a) Settlement caused by the structural loads (foundation settlement).
b) Settelement due to the weight of recently placed fill.
c) Settelment due to falling ground water table.
d) Settlements caused by underground mining or tunneling.
e) Settlements caused by the formation of sinkholes.
f) Lateral movements resulting from nearby excavations that indeirectly cause settlement.

In this report the estimated settelement will be that one resulting from the structural loads which are a combination of:

- Immediate settlement (Elastic).
- Consolidation settelement.

8.3.1 Elastic Settlement (S_i) [2]:

$$Si = (q_{fm}\ B'\ \frac{1 - \mu^2}{E_d}\ C_d)$$

Where:

Si: Immediate , or elastic foundation settlement.
q_{fm}: specified maximum net foundation pressure.
B': Characteristic Dimension of the foundation.
μ: Poissons Ration, for the encountered material.
E_d : Deformation Modulus obtained from unconfined compression tests results.
C_d : Shape & rigidity Correction Factor given in **Table 5**.

Table 5: Shape and rigidity factores Cd for calculating settlements of points on loaded areas at the surface of an elastic half space (after Winterkorn and Fang. 1975).

Shape	Centre	Corner	Middle of short side	Middle of long side	Average
Cirlce	1.00	0.64	0.64	0.64	0.85
Circle (rigid)	0.79	0.79	0.79	0.79	0.79
Square	1.12	0.56	0.79	0.79	0.95
Square (rigid)	0.99	0.99	0.99	0.99	0.99
Rectangle					
Length/Width					
1.5	1.36	0.67	0.89	0.97	1.15
2	1.52	0.76	0.98	1.12	1.30
3	1.78	0.88	1.11	1.35	1.52
5	2.10	1.05	1.27	1.68	1.83
10	21.53	1.26	1.49	2.12	2.25
100	4.00	2.00	2.20	3.60	3.70
1000	5.47	2.75	2.94	5.03	5.15
10000	6.90	3.50	3.70	6.50	6.60

An analysis was carried out, the following assumptions are applied:-

g) The foundation material is composed of homogenous, isotropic rock (Figure No. 3 case a)
h) Load on the footing is 3000 KN.
i) If square rigid footing with B = 3.0 x3.0m is used.
j) Estimated moduls of elasticity of the encountered material is (Es = 250qu = 8.75 x 10^4 KN/m^2).

$$Si = \frac{350(3.0)(1-0.3^2)(1.15)(1000)}{8.75*10^4} = 12.6 \text{ mm.}$$

Analysis was carried out and showed that the expected elastic settlement is very small and acceptable.

8.3.2 Consolidation Settlement (S_c)

This type of settlement is time dependent settlement, and it is occur if the footing is located on a compressible layer or if this layer is located within the significant depth

below the footing. This type of settlement is calculated using the following equation assuming the compressible layer is normally consolidated clay[3]:

$$S_c : \frac{H}{1+e_o} C_c (\log \frac{P_o + \Delta P)}{P_o}$$

S_c : Consolidation Settlement.

H : Thickness of the compressible layer which will be a flected by the load imposed by the building (significant depth).

e_o : Initial void ratio.

Cc : Compression index.

P_o : Initial effective over burden pressure.

ΔP : Increase in the pressure at the middle of the layer due to footing load.

In our case footing will be located on a layer of weak lime stone where the consolidation settlement could not occur.

	Geological condition	Settlement calculation
	(a) Homogenous, isotropic, half space.	(a) Determine shape factor C_d from Table 5.4; (b) calculate settlement using equation (5.18).
	(b) Compressible layer on rigid base.	(a) determine ratios H/B, L/B; (b) determine shape factor C'_d from Table 5.5; (c) calculate settlement using equation (5.18).
	(c) Compressible bed within stiffer formation $E_1 > E_2$.	(a) Determine ratios $(H_1 + H_2)/B$, L/B; (b) calculate weighted modulus E for upper two beds $E = (E_1H_1 + E_2H_2)/(H_1 + H_2)$; (c) determine shape factor C'_d for ratio $(H_1 + H_2)/B$ from Table 5.5; (d) calculate settlement using equation (5.18).
	(d) Stiff bed overlying compressible formation $E_1 > E_2$.	(a) Determine ratios H/B, E_1/E_2; (b) determine correction factor a from Table 5.6; (c) determine shape factor C_d from Table 5.4; (d) calculate approximate settlement from equation (5.18) using elastic parameters E_2, v_2 for overall foundation; (e) calculate actual settlement using equation (5.19).
	(e) Inclined, non-uniform bed of compressible rock.	Use numerical analysis to accurately model foundation geometry.
	(f) Transversly isotropic rock.	Use equations (5.20), (5.21) and (5.22).

Figure No. 4

Settlement calculation for foundation on isotropic, layered and transversely isotropic

8.4 Excavation Methods:

It is expected that the foundation excavations will be through ***a layer of very pale brown to pale yellow weak lime stone.*** Therefore, rock breakers will be needed for the foundation excavation. In addition to the conventional excavation equipment such as loaders and dozers, for the excavation works.

8.5 Surface & Subsurface Drainage:

- **Drainage System:**

  The material encountered at the site if crushed can be used for backfilling purpose.
  To prevent biuld up of hydrostatic water pressure on the retaining wall, a drainagee system is often constructed at the heel of the wall, The drainagesystem consist of rows of weep holes which is located at different levels behind the wall, also a perforsted pipe surrounded by open graded gravel can be used a long the wall to collect the water and drain it away.
  The drainage system will be more effective if highly permeable soil, such as clean granular soil, is used as backfill.

It is recommended to protect the foundation ground and excavation from surface water both during and after construction by providing proper drainage and protection system. Surface water, if existed, should be diverted away from the edges of the excavations.

No ground water was encountered within the site, even though a proper drainage system is required and the foundation system shall be isolated using a proper isolation material. The supervisor engineer according to the required specifications shall select and specify this material. The extent of isolation should be up to the finished ground floor level.

A special drainage system for surface and subsurface water should be maintained for the service life of the building.

8.6 Excavation Side Slopes:

Excavations can be divided into two category.

8.6.1- Shallow Excavation

When the depth of excavation is less than (6m) in this case the side can be protected by provided a suitable slope sides which depend on the type of the cut material, but if a vertical cut is made, a suitable type of side support should be used.

8.6.2- Deep Excavation

When the depth of cut is more than (6m) the sides should be supported, by proper means of supports.

In our case the excavation will be mainly through a lyaer of lime stone. The temporary side excavation during construction should be sloped at a face inclination not steeper than **one horizontal to eight vertical (1H:8V)**[4].

The reocmmended slope depend on the nature of the encountered material.

8.7 Earth Pressure[5]*:*

The underground structures, if existed, shall be designed for an equivalent fluid pressure of (800 kg/m^3), however, the uniform lateral pressure that corresponds to the maximum expected surface loads, should be added to the earth pressure.

The backfill material is recommended to be granlar material, good drainage, so that no hydrostatic pressure will be developed behined the wall.
Angle of internal friction for this type of fill is about (30°).
Using Rankine's theory for active and passive pressure, so the coefficients can be callculated as:

1. Coefficient for lateral active pressure (Ka) is:

$$Ka = \frac{1 - \sin\phi}{1 + \sin\phi} = 0.33$$

2. Coefficient for lateral passive pressure (Kp) is:

$$Kp = \frac{1 + \sin\phi}{1 - \sin\phi} = 3.0$$

If the wall will be constrained (at rest) So (Ka) in this case is known as coefficient of lateral earth pressure at rest (Ko):

$$Ko = 1 - \sin\phi = 0.5$$

If the walls are designed to resist earthquake forces in this case an additional lateral force (PE) should be added.

$$P_E = \frac{3}{8} \frac{a_{max}}{g} \gamma_t H^2$$

Where:

amax : maximum ground acceleration (in our case amax = 0.15g).
γ_t : unit weight of the back fill material, can be taken as (18 KN/m3).
H : height of retaining wall.

Point of application of this force is at (0.6H) above the base of the wall.

8.8 *Site Seismisity*:

8.8.1 General

Earthquake is defined by it's intensity and it's magnitude. Intensity scales depend on human perceptib Earthquake can be defined as a sudden vibration felt on the earth surface, due to the sliding of rock slabs beneath the earth surface, this sliding occur when the energy stored in the rock overcome the frictional resistance between the rock masses causing the rock slab to slide past each other and releasing the stored energed as a waves radiate in all direction.

Earthquake is deined by it's intensity and it's magnitude. Intensity scales depend on human perceptibility and destructivity of the earthquakes (qualitative concept) several earthquake intensity scales have been proposed, the widely used one is shown in **(Table No. 5)** is the Mercalli scale.

While the magnitude of the earthquake is instrumentally measured quantity related to the total energy released during an earthquake. In1935 Richter advised a logarithmic scale for comparing the magnitudes of earthquakes.

Earthquakes of magnitude (5) or greater usually cause damage to the structures, **the amount of damage depend on**:

1- Mangitue of the earthquake.
2- Type and design of the structure.
3- Type of the foundation material
4- Duration of the earthquake.
5- Distance from the center (focus) of the earthquake.

An earthquake does not impart simple motion to a building but in general there are elastic impacts followed by forced and free vibrations of the structure, shallow foundations are sensitive to the vertical displacement component while deep foundations generally are not hazardous in an earthquake.

In order to take the effect of earthquake on the building the seismic factor concept (factor of intensity) or peack ground acceleration (PGA) had been introduced into building design . This concept means that the building should be strong enough to resist a horizontal force equal to a certain proportion of the weight of the building.

8.8.2 Seismic factor

For seismic consideration in Palestine, we refer to the ***Seismic Risk Map*** shown in **Figure in appendix D**,, which divides the region into three zones dependin on th expected horizontal ground acceleration coefficient (0.075g– 0.3g).

According to the official seismic risk map in palestine,The site of the project lies in zone **2A** with an accelration coefficient of **(0.15g)**.

8.9 Soil Classification according to UBC 1997[6]

After the analyses of results for the foundation soil, the encountered material is matching type S_B soil according to table 16-J of the Uniform Building Code. This information is useful for the design of the proposed development for earthquake.

8.10 Construction quality control & testing:(foundation construction)

8.10.1 To ensure the quality of works and performance of the foundation and footings, a verification tests should be performed on the excavated soil material including the followings:

1. Visual inspection and reconassance of material.
2. Moisture content test.
3. Seive analyses and soil classification.

If, by any reason, the encountered soil is not matching to this soil report, the geotechnical Engineer should be notified and other tests should be performed such as plate bearing, direct shear and vane shear, consolidation test and swell analyses.

8.10.2 The reinforcing steel bars should be tested by a qualified lab. For tensile strength and bending – rebending quality.

8.10.3 The concrete quality of the cast footing and ground beams should be controled by fresh concrete sampling from every concrete mixure at site.

8.10.4 For the construction of outdoor parking lot, roadway and landscape, where replacement of the top (1)m layer should be taken and new selected

material should be compacted, a field density, proctor and CBR test should be conducted for the sub-base and the basecoarse material and layers. For the basecoarse material, a seive analyses, Abrasion and sound test should be specified.

8.11 Inspection of the foundation ground, and verification:

After the foundation excavation and before the construction of pile foundation, we should be notified in order to inspect the foundation ground. This inspection is very important to confirm that the required ground is reached and all the undesirable and loose materials are removed.

References:

(1)Jordanian Code (Footing and Retaining walls)
(2),(3),(4),(5) Joseph E. Bowles "Foundation Analysis and Design" Third Edition,
(6)Uniform Building Code 1997

Table No. 6 : Modified Mercalli Intensity Scale, MMI (Abridged)	
Intensity	**Effects**
I	Not felt except by a very few under especially favorable circumstances.
II	Felt only by a few persons at rest, especially on upper floors of building. Delicately suspended objects may swing.
III	Felt quite noticeable indoors, especially on upper floors of building, but many people do not recognize it as an earthquake. Standing motor cars may rock slightly. Vibration like passing of truck. Duration estimated.
IV	During the day felt indoors by many, outdoors by few. At night some awakened. Dishes, window, doors disturbed; walls make cracking sounds. Sensation like heavy truck striking building; standing motor cars rocked noticeably.
V	Felt by nearly everyone; many awakened. Some dishes, windows, etc., broken; a few instances of cracked plaster; unstable objects overturned. Disturbance of trees, poles, and other tall objects sometimes noticed. Pendulum clocks may stop.
VI	Felt by all; many frightened and run outdoors. Some heavy furniture moved; a few instances of fallen plaster or damaged chimneys. Damage slight.
VII	Everybody runs outdoors. Damage negligible in building of good design and construction; slight to moderate in well-built ordinary structures; considerable in poorly built or badly designed structures; some chimneys broken. Noticed by persons driving motor cars.
VIII	Damaged slight in specially designed structures; considerable in ordinary substantial buildings, with partially collapse; great in poorly built structures. Panel walls thrown out of frame structures. Fall of chimneys, factory stacks, columns, monuments, walls. Heavy furniture overturned. Sand and mud ejected in small amounts. Changes in well water. Disturbs persons driving motor cars.
IX	Damage considerable in specially designed structures; well-designed frame structures thrown out of plumb; damage great in substantial buildings, with partial collapse. Buildings shifted off foundations. Ground cracked conspicuously. Underground pipes broken.
X	Some well-built wood structures destroyed; most masonry and frame structures with foundations destroyed. Ground badly cracked. Rails bent. Landslides considerable from river banks and steep slopes. Shifted sand and mud. Water splashed (slopped) over banks.
XI	Few, if any (masonry) structures remain standing. Bridges destroyed. Broad fissures in ground. Underground pipelines completely out of service. Earth slumps and land slips in soft ground. Rails bent greatly.
XII	Damage total. Waves seen on ground surfaces. Lines of sight and level distorted. Objects thrown upward into the air.

End of Report

Approved by: _____	**General Manager: _____**

APPENDIX A

LOGS OF BORING

Project No: SI-09-0441

Borehole Number: BH.01

Project: Hebron Courthouse

Proj. Consultant:

Client: CIDA

Geotech. Eng.: SAHEEM

Site Location: Hebron - Palestine

Geologist: AHMED

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GMT

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Ramallah - Beit Assia Bldg.

SUBSURFACE PROFILE				SAMPLE								Atterberg Limits			Size Distribution		
Depth	Symbol	Description	Depth/Elev.	Number	Type	Symbol	W.C%	RQD	qu (Kg/cm2)	qall (Kg/cm2)	Dry Density	LL%	PL%	PI%	Gravel %	Sand %	Silty and Clay %
		Ground Surface	0.0														
0		***FILL MATERIAL*** Pale yellow fill material with stone pieces.	0.0														
1				1	Dis.		15.4			1.0							
2																	
3																	
4																	
5			-5.5														
6		***LIME STONE*** Very pale brown fractured weak lime stone with marl filling the fractures.	5.5	2	Dis.		9.6			4.0							
7																	
8				3	Dis.		4.8			4.0							
9																	
10				4	Dis.		5.2			4.0							
11																	
12																	
13				5	Dis.		6.5			4.0							
14																	
15			-15.0														
16		**End of Borehole**	15.0														
17																	

Drilled By: SHAREEF

Hole Size: 100 mm

Drill Method: A/P

Datum:

Drill Date: October 2009

Sheet: 1 of 1

Project No: SI-09-0441

Borehole Number: BH.02

Project: Hebron Courthouse

Proj. Consultant:

Client: CIDA

Geotech. Eng.: SAHEEM

Site Location: Hebron - Palestine

Geologist: AHMED

Geotechnical & Material Testing Center
GMT

GMT CENTER
Ramallah - Beit Assia Bldg.

SUBSURFACE PROFILE				SAMPLE								Atterberg Limits			Size Distribution		
Depth	Symbol	Description	Depth/Elev.	Number	Type	Symbol	W.C%	RQD	qu (Kg/cm2)	qall (Kg/cm2)	Dry Density	LL%	PL%	PI%	Gravel %	Sand %	Silty and Clay %
		Ground Surface	0.0														
0		***CHALK*** Whitish chalky material.	0.0														
1				1	Dis.		19.2			1.5							
2																	
3			-3.0														
		MARL STONE Pale yellow weak marl stone	3.0	2	Dis.		14.7			1.0							
4																	
5		***LIME STONE WITH CHALK*** Very pale brown fractured weak lime stone with traces of chalky material.		3	Dis.		11.9			3.5							
6																	
7				4	Dis.		10.0			3.5							
8																	
9			-9.0														
		LIME STONE Very pale brown fractured weak lime stone with marl filling the fractures.	9.0														
10				5	Dis.		8.0			4.0							
11																	
12																	
13				6	Dis.		7.4			4.0							
14																	
15			-15.0														
		LIME STONE Pale yellow weak lime stone.	15.0	7	Dis.		8.8			2.5							
16			-16.5														
17		***LIME STONE*** Very pale brown fractured weak lime stone with marl filling the fractures.	16.5														
18				8	Dis.		4.7			3.0							
19																	
20			-20.0														
		End of Borehole	20.0														
21																	
22																	

Drilled By: SHAREEF

Hole Size: 100 mm

Drill Method: A/P

Datum:

Drill Date: October 2009

Sheet: 1 of 1

Project No: SI-09-0441

Borehole Number: BH.03

Project: Hebron Courthouse

Proj. Consultant:

Client: CIDA

Geotech. Eng.: SAHEEM

Site Location: Hebron - Palestine

Geologist: AHMED

Geotechnical & Material Testing Center
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Ramallah - Beit Assia Bldg.

SUBSURFACE PROFILE				SAMPLE								Atterberg Limits			Size Distribution		
Depth	Symbol	Description	Depth/Elev.	Number	Type	Symbol	W.C%	RQD	qu (Kg/cm2)	qall (Kg/cm2)	Dry Density	LL%	PL%	PI%	Gravel %	Sand %	Silty and Clay %
		Ground Surface	0.0														
0		***LIME STONE*** Pale yellow fractured weak lime stone with marl filling the fractures.	0.0 / -0.5	1	Dis.		4.0			3.5							
			0.5														
1		***LIME STONE*** Very pale brown to whitish fractured weak lime stone with marl filling the fractures.		2	Dis.		3.2			4.0							
2																	
3				3	Dis.		2.7			4.0							
4																	
5				4	Dis.		2.3			4.0							
6			-6.0														
		End of Borehole	6.0														
7																	
8																	

Drilled By: SHAREEF

Hole Size: 100 mm

Drill Method: A/P

Datum:

Drill Date: October 2009

Sheet: 1 of 1

Project No: SI-09-0441

Borehole Number: BH.04

Project: Hebron Courthouse

Proj. Consultant:

Client: CIDA

Geotech. Eng.: SAHEEM

Site Location: Hebron - Palestine

Geologist: AHMED

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Ramallah - Beit Assia Bldg.

SUBSURFACE PROFILE				SAMPLE								Atterberg Limits			Size Distribution		
Depth	Symbol	Description	Depth/Elev.	Number	Type	Symbol	W.C%	RQD	qu (Kg/cm2)	qall (Kg/cm2)	Dry Density	LL%	PL%	PI%	Gravel %	Sand %	Silty and Clay %
		Ground Surface	0.0														
0		***SILTY CLAY*** Light reddish silty clay material with stone pieces.	0.0	1	Dis.		5.7			0.5							
1			-1.0														
		LIME STONE Very pale brown fractured weak lime stone with marl filling the fractures.	1.0 -1.5	2	Dis.		4.1			3.0							
2			1.5														
		LIME STONE Pale yellow fractured weak lime stone with marl filling the fractures.															
3																	
				3	Dis.		3.2			4.0							
4																	
5																	
			-6.0														
6		**End of Borehole**	6.0														
7																	
8																	

Drilled By: SHAREEF

Hole Size: 100 mm

Drill Method: A/P

Datum:

Drill Date: October 2009

Sheet: 1 of 1

APPENDIX B

ELECTRICAL RESISTIVITY TEST REPORT

Geo – electrical profiling (VES) for Hebron site

Introduction:

The investigation site locates in Hebron city. Almost the site is completely filled with debris. A steep slope characterizes the topography of this site in the western side (Fig 2). Limestone and marly limestone are the main outcrop. In this site, the karstification phenomena are clear to see on the rock surface. Long weathering of carbonate rock could produce caves.

Objectives:

The objectives of this investigation are the followings:

1- To identify the structural zone/zones of weakness.
2- To identify caves under the foundation.
3- To delineate the dimensions and extensions of caves.

Methodology:

Wenner array (Fig 1) is the most pioneer work carried out for the geoelectrical sounding (Loke, 2001). Wenner array has almost horizontal contours beneath the center of the array. Because of this property, this method is relatively sensitive to the vertical changes in the subsurface resistivity mainly below the centre of the array. For this reason we choose it for resolving vertical changes "Horizontal structures". (Telford, et.al, 1990)

PA: apparent resistivity.
a: electrodes spacing.
V: potential voltage.
I: electrical currant.

Fig. 1 electrical sounding using Wenner array.

$$\rho_A = 2\pi a \frac{V}{I}$$

Apparent resistivity of the layered rock can be obtained by the following equation:

$$\rho = k \,.\Delta V/I,$$

$$k=2\pi a \ldots \text{ for Wenner Array.}$$

Where:
ρ: apparent resistivity, **k**: geometrical factor, **ΔV**: potential or voltage differences, **I**: electrical current.

A geo-electrical investigation was curried out using 21 vertical electrical sounding points (VES). This technique enables the construction of geo-electrical profiles for identifying the resistivity properties of the different lithology.

VES sounding points are accomplished using PASI instrument. The array spacing distances are illustrated in (Table 1). IPI2win electrical sounding interpretation software is used to generate the geo – electrical profiles.

Table 1 horizontal spacing of the VES investigation in Unipal site.

Spacing number	Spacing distance (m) **(a)**
1	1
2	3
3	5
4	8
5	10
6	15
7	20

Vertical Electrical Sounding using Wenner array was used in "Hebron". The site was divided into 2 VES different profiles. One profile is north – west and the other is west – east direction. The profiles contain 4 sounding points. The sounding profiles cover the total investigated site (Fig 2). The average distance between each tow points is 25 m.

Fig .2 locations of VES profiles. "Hebron site".

Results:

Profile (I):

This profile contains 2 sounding points (Fig 3). The total length of this profile is 19.5 m. Electrical sounding reaches about 20 m in depth. This profile locates close to the main street (Fig 2).

Fig .3 VES profile I.

Profile description:

The upper 10 m of this profile is clearly indicating the debris. At about 10 m depth low resistivity values found, it is indicating of the soil which covered by the debris. for the rest of the profile a will layered limestone continuo to the rest 20 m depth.

Profile (II):

This profile contains 2 sounding points (Fig 4), with total distance of 38 m. The VES sounding penetration reaches a depth of 20 m. The soil locates by the upper 0.5 m of the profile.

Fig .4 VES profile II.

Profile description:

Limestone and marly limestone stratifications are layered homogenous along the profile. Hard limestone layers could be met and this layer extends to 10 meter depth. High weathered limestone can be found under the sounding point (2-2). This weathering produces marly limestone, and the weathering depth

reach 10 m under this point. The maximum resistivity value is 500 Ω.m and the minimum resistivity value is 65 Ω.m. Resistivity values are belonging to the normal range of the limestone and marly limestone. There are no types of caves (cavities) indicated within the rocks layers

Conclusion:

Form the above sounding points and profiles it is clear that the site lithology composed of successive layers of limestone and marly limestone. Debris depth reached about 10 m depth in Profile (I).

There are absolutely no indications of any types of caves (cavities) within the rock layers. The lithologies of these three profiles are composed form limestone and marly limestone.

References:

1- Telford W M; Geldart L.P; Sheriff R.E 1990. Applied Geophysical, Second Edition, Cambridge University Press.

2- Loke M.H 2001. Tutorial 2-D and 3-D electrical imaging surveys.

3- IPI2win, resistivity sounding interpretation software. Moscow state university, 2003 Version 3.0.1.a.

APPENDIX C

SEISMOGRAPH TEST REPORT

بسم الله الرحمن الرحيم

An-Najah National University
Earth Sciences & Seismic
Engineering Center

جامعة النجاح الوطنية
مركز علوم الأرض
وهندسة الزلازل

Geophysical Study using Seismic Investigation

Hebron- Palestine

Report Submitted to the:
Geotechnical and Material Testing Center (GMT)

By:
Earth Sciences and Seismic Engineering Center
At An Najah National University
December 2009

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Geophysical Study using Seismic Investigation Hebron-Palestine

By:

Earth Sciences and Seismic Engineering Center (ESSEC)
An-Najah National University, P.O. Box 7, Nablus, Palestine
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December 2009

LOCAL GEOLOGY

Investigating the subsurface geology of a site is critical, in order to select the kind of structure foundation design in a given area, since sedimentary deposits are often the prime locations for development of urban areas.

The exposed sequence of rocks in the study area consists Samra Formation (alluvium with poorly sorted gravels); Jerusalem Formation (dolomite and thin bedded limestone); Bethlahem and Hebron Formations (karstic dolomite and chalky limestone); Yatta Formation (marl,, limestone & dolostone, chalk and clay) with ages ranging from Pleistocene to upper Cretaceous.

METHODOLOGY AND DATA ANALYSIS

Geophysical experiment

The subsurface geology is extremely important fro the development of highly populated, tectonically arid region such as the Middle East. The shallow upper part (ten to hundred meters) of the rock formation section is the most significant part fro civil infrastructures. The seismic refraction technique is considered one of the accurate geophysical methods to investigate shallow geological structures of an area. During the past decades, the seismic parameters obtained by the refraction survey have been widely used in case of site investigation as indicators of rock mass quality. The main objective of seismic refraction method is to estimate the first arrival velocities of P-waves, which are used to determine the depths of different layers and obtain the dynamic characteristics of rocks. These parameters are of great important in land use management of various civil engineering purposes.

DETECTION OF SEISMIC WAVES

Seismic waves are generated usually by weight dropping, sledge hammer. The seismic signals generated from the shot propagates in different direction, it is, reflected, refracted, or diffracted. The different seismic signals can be recorded using a system of receivers (geophones) distributed in a profile in the direction of the shot point. In detecting direct and refracted waves a number of detectors are placed on the ground along a straight line passing through the shot point, this system is known as (In-line spread) which is widely used in most seismic refraction techniques.

The profile type used in this study is the reversed profile consists of three shot points (sp): two of them are located at the two ends of the geophone spread, while the third one is located in the middle. Reversed profiles are employed to determine the true velocities of the subsurface structure. For this study the system used is the Smart Seis Exploration seismograph model S/N 70253, manufactured by Geometric Europe (U.K). The detectors used in the present study have a natural frequency of 28 Hz each, the signal is amplified and the undesirable frequencies can be filtered out. These signals, after suitable amplification and filtering, are fed into a recording unit. The recording system contains 24 channels.

Data Acquisition and Analysis

The seismic refraction survey was conducted on five profiles: P1, P2, P3 in the **N-S, NNE-SSW, NW-SE** directions (profiles have 52 meter length and separated laterally with 8-15 meters) and the P4 and P5 profiles crossed the three ones with lengths of 24, 36 meter (Figure 1). The distance between two receivers (geophone interval) were 1.5, 2, 3 meters, and have five shot points. Many interpretation techniques are published in seismic refraction data analysis, each of them depends on the character of the refractor. In the present study, the seismic refraction data was interpreted using the modeling and interactive ray tracing techniques. The travel time-distance curves and the corresponding ground models for P-waves were obtained. Depths of he interfaces are obtained from the travel time-distance curves for the P-waves.

Figure1: A sketch location map of the study area, also shown are the distribution of the seismic profile

Note: For More Detailes See Appendix 2 (Pic 1, 2, 3, 4, 5, 7, 9, 10)

Following are the results obtained from the seismic profiles for this study:

Profiles P1, P2, and P3 (N-S, NNE-SSW, and NW-SE):

The P-waves were picked up as first arrivals. The bed-rocks beneath these profiles form three main layers, the first two layers with thickness ranges between 5-12 meters, overlaying a layer up to a depth of more than 10 meters. Figures 2 -13; see Appendix 1, show the travel time curves and the corresponding velocity ground models beneath the profile P1, P2 P3, respectively. The travel time curves analyses showed longitudinal wave velocities (P-waves) in the range of 318 m/sec to 984 m/sec for the first two layers and P-waves velocities between 2255 m/sec to 2573 m/sec for the third layer. Both of the modeled first two layers are interpreted as non-consolidated sediments of alluvium and gravel materials in addition to derbis. On the other hand, the third layer consists of consolidated sediments of limestone types.

Profile P4 and P5 (East- West):

The underground model beneath profile P4 and P5 shows similar velocities to the models beneath the profiles P1 P2 P3 for the first two layers. The geological cross-section beneath these profiles P4, P5 consists of two layers with thickness up to eight meters which represents the first two layers beneath the profiles P1, P2, P3. Figure 14 -21; see Appendix 1 show the travel time curve and the corresponding ground velocity model. From the travel time curves for these segments, the longitudinal wave velocities (P-waves) are 303-432 m/sec for the first layer and 639-735 m/sec for the second layer.

CONCLUSIONS AND RECOMMENDATIONS

Based on the outcropping geological cross-section in the study area and the ground velocity models deduced from the P-waves velocities of this study, the subsurface geological formations beneath the profiles P1, P2, P3, P4 and P5 are interpreted as non consolidated sediments of poorly sorted gravels, alluvial and debris which form the first two layers with a depth of 5-12 meters. And the third layer is explained as consolidated carbonates of limestone dolostone and chalk. The first two layers are highly recommended to be totally removed, and the excavation should reach the third layer.

Appendix 1:

-Seismic Profiles

TITLE: LINE NUMBER 1-1 FILE10.LPK

DATE: 12-23-2009 TIME: 8:56

Spread A contains 4 Shotpoints and 24 Geophones. Distance units: meters.

SP	Elev	X-Loc	Y-Loc	Depth
A	.0	999.1	.0	.0
B	.0	1009.5	.0	.0
C	.0	1020.1	.0	.0
D	.0	1041.1	.0	.0

Arrival Times T and Layers L

Geo	Elev	X-Loc	Y-Loc	SP A		SP B		SP C		SP D	
				T	L	T	L	T	L	T	L
1	.0	1000.0	.0	5.3	1	21.5	2	42.5	2	48.0	2
2	.0	1001.8	.0	8.5	1	16.0	2	40.0	2	48.0	2
3	.0	1003.5	.0	15.0	1	11.3	2	41.3	2	46.3	2
4	.0	1005.2	.0	17.3	1	1.0	2	38.3	2	43.0	2
5	.0	1007.0	.0	20.5	1	.0	0	36.0	2	42.0	2
6	.0	1008.8	.0	25.5	1	2.3	1	33.3	2	40.5	2
7	.0	1010.5	.0	29.5	2	5.0	1	28.0	2	38.3	2
8	.0	1012.3	.0	31.5	2	9.0	1	25.3	2	37.0	2
9	.0	1014.0	.0	33.0	2	14.3	2	19.0	1	36.5	2
10	.0	1015.8	.0	37.0	2	18.0	2	15.0	1	36.5	2
11	.0	1017.5	.0	40.3	2	20.5	2	9.0	1	33.5	2
12	.0	1019.3	.0	42.5	2	24.0	2	3.3	1	30.3	2
13	.0	1021.0	.0	43.5	2	28.0	2	2.3	1	28.0	2
14	.0	1022.8	.0	45.0	2	32.5	2	7.0	1	27.0	2
15	.0	1024.5	.0	46.7	2	36.5	3	12.0	1	25.3	2
16	.0	1026.3	.0	46.3	3	36.3	3	16.5	1	24.5	2
17	.0	1028.0	.0	48.0	3	37.5	3	23.5	1	22.3	2
18	.0	1029.7	.0	48.0	3	38.0	3	27.5	2	21.3	2
19	.0	1031.5	.0	52.0	3	38.0	3	28.0	2	19.3	2
20	.0	1033.3	.0	53.0	3	39.0	3	28.3	2	18.0	2
21	.0	1035.0	.0	54.0	3	40.0	3	29.0	2	16.0	1
22	.0	1036.8	.0	53.0	3	40.5	3	31.0	3	13.3	1
23	.0	1038.5	.0	53.0	3	39.0	3	31.0	3	6.5	1
24	.0	1040.3	.0	54.3	3	40.3	3	29.0	3	2.5	1

Depths of layers directly beneath Shotpoints and Geophones for Spread A:

SP	X-Loc	Layer 2	Layer 3
A	999.1	2.0	14.3
B	1009.5	3.1	14.3
C	1020.1	3.4	7.0
D	1041.1	1.7	6.9
Geo			
1	1000.0	2.0	14.3
2	1001.8	2.9	14.3
3	1003.5	2.9	14.3
4	1005.2	3.9	14.3
5	1007.0	3.6	14.3
6	1008.8	3.3	14.3
7	1010.5	2.7	14.3
8	1012.3	2.7	13.5
9	1014.0	2.8	12.4
10	1015.8	3.1	11.2
11	1017.5	3.3	8.9
12	1019.3	3.4	8.1
13	1021.0	3.5	7.4
14	1022.8	3.0	7.2
15	1024.5	2.0	8.4
16	1026.3	.0	8.7
17	1028.0	.4	8.4
18	1029.7	.7	8.3
19	1031.5	.7	8.6
20	1033.3	.9	8.9
21	1035.0	1.2	8.7
22	1036.8	1.4	7.2
23	1038.5	1.6	7.2
24	1040.3	1.8	7.2

Velocities used:

Layer 1	Layer 2	Layer 3	
318.	806.	2255.	m/s

Fig 2: Times arrival of seismic waves beneath the seismic profile P1

Fig 3: Travel Time Curve for Seismic Profile P1

Fig 4: Refraction waves at the second layer surface beneath the seismic profile P1

Fig 5: Subsurface Geological cross- section beneath seismic profile P1

TITLE: LINE NUMBER 2-2 FILE16.LPK

DATE: 12-23-2009 TIME: 9:16

Spread A contains 4 Shotpoints and 24 Geophones. Distance units: meters.

SP	Elev	X-Loc	Y-Loc	Depth
A	.0	999.1	.0	.0
B	.0	1009.6	.0	.0
C	.0	1020.3	.0	.0
D	.0	1030.6	.0	.0

Arrival Times T and Layers L

Geo	Elev	X-Loc	Y-Loc	SP A		SP B		SP C		SP D	
				T	L	T	L	T	L	T	L
1	.0	1000.0	.0	4.0	1	25.5	1	46.7	1	45.0	3
2	.0	1001.8	.0	9.8	1	22.0	1	43.0	1	45.0	3
3	.0	1003.5	.0	12.8	1	17.8	1	39.3	1	45.3	3
4	.0	1005.2	.0	18.3	1	13.0	1	37.3	1	46.3	3
5	.0	1007.0	.0	22.7	1	9.0	1	33.0	1	43.5	3
6	.0	1008.8	.0	28.0	1	4.3	1	29.5	1	42.5	3
7	.0	1010.5	.0	30.0	1	4.5	1	26.0	1	42.0	3
8	.0	1012.3	.0	35.0	1	9.3	1	20.5	1	40.3	3
9	.0	1014.0	.0	38.3	1	17.0	1	16.5	1	38.0	3
10	.0	1015.8	.0	42.0	2	18.3	1	10.5	1	36.3	3
11	.0	1017.5	.0	47.5	2	22.7	1	6.3	1	33.3	2
12	.0	1019.3	.0	50.5	2	30.3	1	2.5	1	29.5	2
13	.0	1021.0	.0	50.5	2	33.0	1	2.3	1	26.5	2
14	.0	1022.8	.0	50.0	2	38.0	1	9.3	1	23.3	2
15	.0	1024.5	.0	50.0	2	42.5	1	9.0	1	16.0	1
16	.0	1026.3	.0	51.0	3	44.0	1	16.0	1	12.3	1
17	.0	1028.0	.0	51.0	3	44.0	1	20.0	1	9.5	1
18	.0	1029.7	.0	50.3	3	44.0	1	26.0	2	.0	1
19	.0	1031.5	.0	50.3	3	45.0	1	28.5	2	1.5	1
20	.0	1033.3	.0	50.0	3	46.0	1	33.0	2	4.3	1
21	.0	1035.0	.0	49.5	3	45.3	1	32.0	2	9.3	1
22	.0	1036.8	.0	50.0	3	45.5	1	34.3	2	11.0	1
23	.0	1038.5	.0	49.0	3	45.0	2	34.0	2	11.3	1
24	.0	1040.3	.0	49.0	3	46.3	2	35.5	2	15.3	1

Depths of layers directly beneath Shotpoints and Geophones for Spread A:

SP	X-Loc	Layer 2	Layer 3
A	999.1	6.7	12.4
B	1009.6	6.2	13.7
C	1020.3	5.6	10.7
D	1030.6	3.5	8.3
Geo			
1	1000.0	6.7	13.5
2	1001.8	6.6	13.5
3	1003.5	6.5	13.5
4	1005.2	6.4	13.3
5	1007.0	6.3	13.2
6	1008.8	6.2	13.0
7	1010.5	6.2	13.6
8	1012.3	6.1	12.8
9	1014.0	6.0	12.4
10	1015.8	5.9	12.1
11	1017.5	5.8	11.7
12	1019.3	5.7	10.8
13	1021.0	5.5	10.6
14	1022.8	4.9	11.2
15	1024.5	4.5	11.1
16	1026.3	4.1	10.5
17	1028.0	3.7	9.9
18	1029.7	3.4	8.9
19	1031.5	3.6	7.7
20	1033.3	3.7	7.1
21	1035.0	3.4	6.6
22	1036.8	3.1	6.3
23	1038.5	2.7	5.8
24	1040.3	2.5	5.0

Velocities used:

Layer 1	Layer 2	Layer 3	
403.	984.	2573.	m/s

Fig 6: Times arrival of seismic waves beneath the seismic profile P2

Fig7: Travel Time Curve for Seismic Profile P2

Fig 8: Refraction waves at the second layer surface beneath the seismic profile P2

Fig 9: Subsurface Geological cross– section beneath seismic profile P2

TITLE: LINE NUMBER 3-3 FILE22.LPK

DATE: 12-23-2009 TIME: 9:50

Spread A contains 4 Shotpoints and 24 Geophones. Distance units: meters.

SP	Elev	X-Loc	Y-Loc	Depth
A	.0	999.0	.0	.0
B	.0	1011.0	.0	.0
C	.0	1023.0	.0	.0
D	.0	1035.0	.0	.0

Arrival Times T and Layers L

Geo	Elev	X-Loc	Y-Loc	SP A		SP B		SP C		SP D	
				T	L	T	L	T	L	T	L
1	.0	1000.0	.0	3.0	2	27.0	2	46.3	3	47.0	3
2	.0	1002.0	.0	9.3	2	24.0	2	46.3	3	46.7	3
3	.0	1004.0	.0	13.0	2	20.0	2	46.3	3	47.5	3
4	.0	1006.0	.0	20.0	2	15.0	2	39.5	3	45.0	3
5	.0	1008.0	.0	25.0	2	12.3	2	36.5	2	45.3	3
6	.0	1010.0	.0	28.0	2	3.5	1	30.0	2	45.0	3
7	.0	1012.0	.0	32.0	2	3.5	1	28.0	2	43.0	2
8	.0	1014.0	.0	35.0	2	8.0	1	22.3	2	42.5	2
9	.0	1016.0	.0	36.3	2	15.0	2	14.0	2	40.0	2
10	.0	1018.0	.0	42.5	3	18.0	2	10.0	2	38.3	2
11	.0	1020.0	.0	44.0	3	20.0	2	5.0	2	35.5	2
12	.0	1022.0	.0	46.0	3	24.3	2	.0	2	35.0	2
13	.0	1024.0	.0	46.7	3	31.0	2	1.3	2	31.0	1
14	.0	1026.0	.0	47.0	3	36.0	2	6.5	2	27.0	1
15	.0	1028.0	.0	48.0	3	40.5	2	9.3	2	19.5	1
16	.0	1030.0	.0	47.0	3	42.5	2	13.0	2	11.5	1
17	.0	1032.0	.0	47.5	3	43.5	2	22.0	2	6.3	1
18	.0	1034.0	.0	49.0	3	45.0	2	28.0	2	2.0	1
19	.0	1036.0	.0	49.0	3	45.0	2	32.0	3	.0	0
20	.0	1038.0	.0	48.0	3	46.0	2	34.0	3	0.0	1
21	.0	1040.0	.0	48.3	3	46.5	2	36.5	3	14.5	1
22	.0	1042.0	.0	47.0	3	47.0	2	37.5	3	22.5	1
23	.0	1044.0	.0	50.3	3	47.3	3	36.5	3	27.5	2
24	.0	1046.0	.0	51.3	3	49.0	3	39.3	3	31.3	2

Depths of layers directly beneath Shotpoints and Geophones for Spread A:

SP	X-Loc	Layer 2	Layer 3
A	999.0	.7	11.7
B	1011.0	2.6	10.0
C	1023.0	1.5	7.6
D	1035.0	1.7	6.1
Geo			
1	1000.0	1.1	11.7
2	1002.0	1.0	11.7
3	1004.0	2.6	11.7
4	1006.0	3.0	11.0
5	1008.0	2.9	11.4
6	1010.0	2.4	11.0
7	1012.0	2.7	10.7
8	1014.0	3.1	10.3
9	1016.0	2.3	10.0
10	1018.0	1.3	8.9
11	1020.0	1.2	8.1
12	1022.0	1.3	7.6
13	1024.0	1.6	7.6
14	1026.0	1.7	7.5
15	1028.0	1.7	7.1
16	1030.0	1.7	6.6
17	1032.0	1.8	6.3
18	1034.0	1.5	6.2
19	1036.0	1.9	6.1
20	1038.0	2.5	5.8
21	1040.0	2.9	5.6
22	1042.0	3.3	5.3
23	1044.0	3.4	5.0
24	1046.0	3.6	5.1

Velocities used:

Layer 1	Layer 2	Layer 3	
364.	591.	2295.	m/s

Fig 10: Times arrival of seismic waves beneath the seismic profile P3

Fig 11: Travel Time Curve for Seismic Profile P3

Fig 12: Refraction waves at the second layer surface beneath the seismic profile P3

Fig 13: Subsurface Geological cross- section beneath seismic profile P3

TITLE: LINE NUMBER 4-4 FILE27.LPK

DATE: 12-20-2009 TIME: 2:06

Spread A contains 5 Shotpoints and 24 Geophones. Distance units: meters.

SP	Elev	X-Loc	Y-Loc	Depth
A	.0	999.2	.0	.0
B	.0	1008.3	.0	.0
C	.0	1017.2	.0	.0
D	.0	1026.3	.0	.0
E	.0	1035.3	.0	.0

Arrival Times T and Layers L

Geo	Elev	X-Loc	Y-Loc	SP A		SP B		SP C		SP D		SP E	
				T	L	T	L	T	L	T	L	T	L
1	.0	1000.0	.0	2.0	1	20.5	1	31.3	2	41.0	3	50.0	2
2	.0	1001.5	.0	6.5	1	15.0	1	30.0	2	38.0	3	49.3	2
3	.0	1003.0	.0	11.5	1	13.3	1	29.5	2	38.5	3	44.5	2
4	.0	1004.5	.0	18.3	2	9.0	1	27.0	2	37.3	3	42.3	2
5	.0	1006.0	.0	21.3	2	6.0	1	27.5	2	36.0	3	45.3	2
6	.0	1007.5	.0	26.5	2	3.0	1	27.0	2	36.0	3	45.0	2
7	.0	1009.0	.0	29.5	2	3.0	1	19.3	1	35.5	3	44.5	2
8	.0	1010.5	.0	33.0	2	1.5	1	17.0	1	34.3	3	42.0	2
9	.0	1012.0	.0	34.0	2	3.5	1	7.0	1	33.0	2	41.5	2
10	.0	1013.5	.0	38.0	2	7.5	2	5.0	1	33.3	2	41.5	2
11	.0	1015.0	.0	38.0	2	12.5	2	3.0	1	31.3	2	39.0	2
12	.0	1016.5	.0	40.3	2	18.5	2	.4	1	30.3	2	37.3	2
13	.0	1018.0	.0	41.0	2	22.7	2	3.3	1	27.0	2	35.5	2
14	.0	1019.5	.0	42.0	2	27.4	3	6.0	1	24.3	2	35.0	2
15	.0	1021.0	.0	44.0	2	28.9	3	8.5	1	21.3	2	34.3	2
16	.0	1022.5	.0	46.5	2	31.6	3	9.0	1	16.5	1	32.0	2
17	.0	1024.0	.0	47.5	2	34.3	3	12.5	2	7.0	1	39.5	1
18	.0	1025.5	.0	48.0	2	35.5	3	17.0	2	2.0	1	32.3	1
19	.0	1027.0	.0	50.5	2	36.0	3	22.0	2	6.0	1	29.3	1
20	.0	1028.5	.0	52.0	2	38.0	3	26.5	2	7.0	1	24.3	1
21	.0	1030.0	.0	55.5	2	41.3	3	32.5	2	12.0	1	20.0	1
22	.0	1031.5	.0	55.0	2	41.3	3	32.3	2	17.0	1	16.3	1
23	.0	1033.0	.0	58.0	2	43.5	3	33.5	3	32.3	2	11.0	1
24	.0	1034.5	.0	58.0	2	44.0	3	34.0	3	37.3	2	5.3	1

Depths of layers directly beneath Shotpoints and Geophones for Spread A:

SP	X-Loc	Layer 2	Layer 3
A	999.2	2.1	5.0
B	1008.3	3.8	4.7
C	1017.2	3.2	4.0
D	1026.3	3.4	5.0
E	1035.3	1.2	3.5
Geo			
1	1000.0	2.2	5.0
2	1001.5	2.3	5.0
3	1003.0	2.4	4.7
4	1004.5	2.2	4.5
5	1006.0	2.9	4.5
6	1007.5	3.6	4.6
7	1009.0	4.0	4.9
8	1010.5	4.1	5.1
9	1012.0	4.2	5.3
10	1013.5	3.5	5.2
11	1015.0	3.3	5.0
12	1016.5	3.1	4.6
13	1018.0	3.3	5.0
14	1019.5	3.5	5.5
15	1021.0	3.6	5.9
16	1022.5	3.9	6.2
17	1024.0	3.7	6.4
18	1025.5	3.5	6.7
19	1027.0	3.3	6.9
20	1028.5	3.1	6.0
21	1030.0	2.4	6.3
22	1031.5	1.9	5.6
23	1033.0	1.5	4.5
24	1034.5	1.3	3.5

Velocities used:

Layer 1	Layer 2	Layer 3	
432.	735.	1118.	m/s

Fig 14: Times arrival of seismic waves beneath the seismic profile P4

Fig 15: Travel Time Curve for Seismic Profile P4

Fig 16: Refraction waves at the second layer surface beneath the seismic profile P4

Fig 17: Subsurface Geological cross- section beneath seismic profile P4

TITLE: LINE NUMBER 5-5 FILE5-5.LPK

DATE: 12-20-2009 TIME: 2:12

Spread A contains 3 Shotpoints and 12 Geophones. Distance units: meters.

SP	Elev	X-Loc	Y-Loc	Depth
A	.0	999.0	.0	.0
B	.0	1011.0	.0	.0
C	.0	1023.0	.0	.0

Arrival Times T and Layers L

Geo	Elev	X-Loc	Y-Loc	SP A		SP B		SP C	
				T	L	T	L	T	L
1	.0	1000.0	.0	3.5	1	30.5	1	44.8	2
2	.0	1002.0	.0	11.5	1	25.5	1	42.5	2
3	.0	1004.0	.0	16.5	1	21.0	1	44.5	1
4	.0	1006.0	.0	23.3	2	16.3	1	43.0	1
5	.0	1008.0	.0	26.5	2	11.3	1	40.0	1
6	.0	1010.0	.0	30.3	2	6.3	1	36.5	1
7	.0	1012.0	.0	33.5	2	6.0	1	32.0	1
8	.0	1014.0	.0	35.3	2	11.3	1	26.5	1
9	.0	1016.0	.0	37.8	2	16.0	1	22.0	1
10	.0	1018.0	.0	40.5	2	20.5	1	17.3	1
11	.0	1020.0	.0	42.0	2	25.0	2	12.0	1
12	.0	1022.0	.0	43.0	2	31.0	2	5.0	1

Depths of layers directly beneath Shotpoints and Geophones for Spread A:

SP	X-Loc	Layer 2
A	999.0	1.3
B	1011.0	1.6
C	1023.0	3.2
Geo		
1	1000.0	1.3
2	1002.0	.8
3	1004.0	1.0
4	1006.0	1.3
5	1008.0	1.5
6	1010.0	1.6
7	1012.0	1.6
8	1014.0	1.5
9	1016.0	1.7
10	1018.0	2.3
11	1020.0	2.8
12	1022.0	3.0

Velocities used:

Layer 1	Layer 2	
303.	639.	m/s

Fig 18: Times arrival of seismic waves beneath the seismic profile P5

Fig 19: Travel Time Curve for Seismic Profile P5

Fig 20: Refraction waves at the second layer surface beneath the seismic profile P5

Fig 21: Subsurface Geological cross- section beneath seismic profile P5

Appendix 2:

- Site pictures

Picture 1

Picture 2

Picture 3

Picture 4

Picture 5

Picture 6

Picture 7

Picture 8

Picture 9

Picture 10

Picture 11

Picture 12

APPENDIX D

Seismic Risk Map

Seismic Zone Factor,Z

Zone	1	2A	2B	3
Z	0.075	0.15	0.20	0.3

Figure
Seismic Risk Map