1. INTRODUCTION

1.1 Background

Rawabi is a new Palestinian planned city to be located north of Ramallah as shown in Figure 1.1. Rawabi aims to be a response to the severe shortage of affordable housing in Palestine, to reverse the substantial decline in the construction sector activity and to stimulate the Palestinian economy. Upon completion, Rawabi will have a population of 40,000 with an extent of 6,300 dunums (630 hectare). The ultimate goal is to create a sustainable development framework and a prototype for development in Palestine.

The Earth Sciences and Seismic Engineering Center (ESSEC) at An-Najah National University (NNU) was approached by Bayti Real Estate Investment Company seeking to conduct an assessment of seismic site effect. This ASSE investigation provides engineering data and recommendations to mitigate the seismic site effect.

Figure 1.1: Geographic setting of Rawabi in the West Bank (Source:www.rawabi.ps)

1.2 Problem Statement

Seismic information including historic and prehistoric data indicates that major destructive earthquakes have occurred along the Dead Sea Transform (DST). The DST is a left-lateral fault between the Arabia and the Sinai tectonic plates that stretches from the opening at the Red Sea to the Taurus-Zagrous collision zone. The estimated MMS intensities of historical earthquakes in the Dead Sea region reach up to X, where the determinable magnitudes of the recorded earthquakes range between 1.0 and 6.5, on the local magnitude scale, ML. These damaging earthquakes caused, in several cases, severe destruction and many hundreds and sometimes thousands of fatal casualties.

In addition to the seismic risk, landslide hazards are a frequent threat in the Palestinian regions. Generally, local site effects (landslides, liquefaction, amplification and faulting systems) play an important role in the intensity of earthquakes. Thus, Earthquake-resistant design of new structures and evaluating the seismic vulnerability of existing buildings take into account their response to site ground motions. Geophysical studies of seismic activity in Palestine, deep seismic sounding, paleoseismic excavation, and instrumental earthquake studies of half a century [1-14] demonstrate that damaging earthquakes occurred along the Dead Sea Rift/Transform fault (Fig. 1.2). The topography, geomorphology and geology of the West Bank have been the main reasons behind several sizeable landslides that occurred around ten years ago in different parts of the West Bank. Also, it has been shown that Palestine suffered from several landslides during historical earthquakes.

Based on the seismic peak ground acceleration map (PGA Map) for the region, Rawabi area is located in zone 2A and it is very close to zone 2B. Therefore, it is recommended that the Rawabi area be considered as zone 2B (see figure 1.3 or appendix no. 1). The seismic zone factor (Z) on the rock for the zone 2A is equal to 0.15 and 0.2 for zone 2B. According to the Uniform Building Code (UBC97), International Building Code (IBC), Jordanian Building Code 2005 and Arab Uniform Code 2006, it can be considered as moderate seismic area.

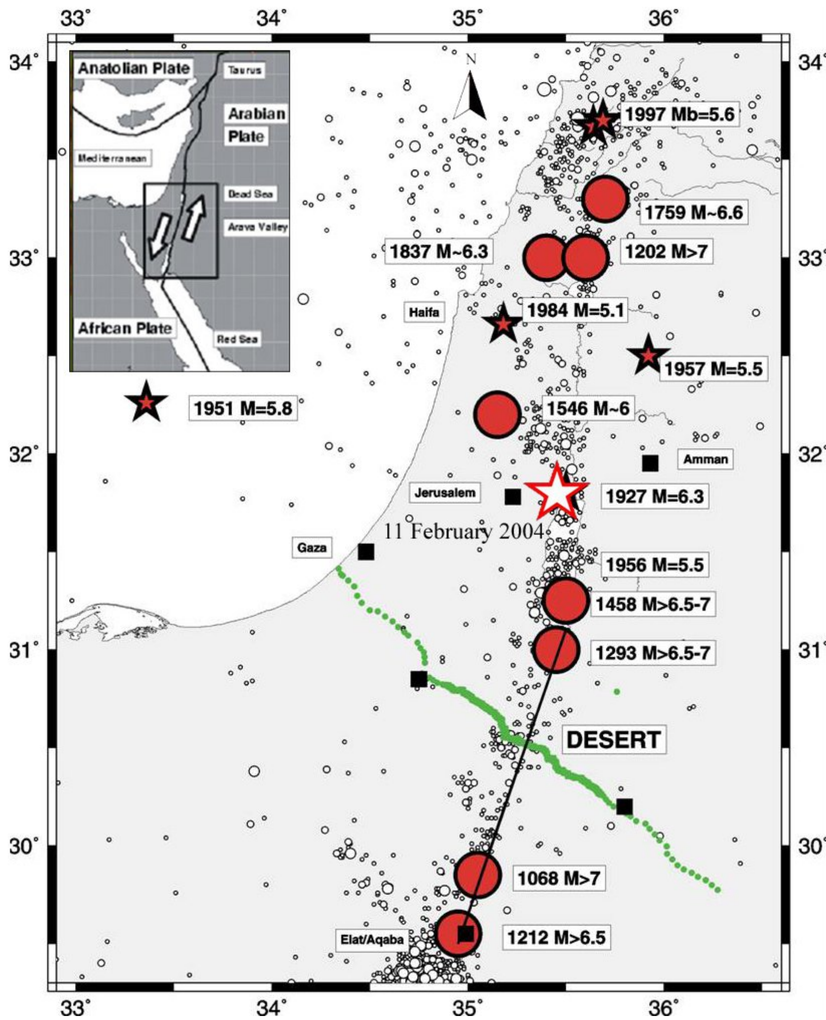

Figure 1.2: Seismic activity in the Dead Sea Transform region; the map shows locations of historical earthquakes [11-14]. Also shown is the most recent earthquake of 11 February 2004, ML 5.2.

1.3 The scope of Assessment of Seismic site effect (ASSE)

The Earth Sciences and Seismic Engineering Center (ESSEC) at An-Najah National University (NNU) was approached by Bayti Real Estate Investment Company, to conduct an assessment of seismic site effect. This ASSE investigation provides engineering data and recommendations to mitigate the seismic site effect.

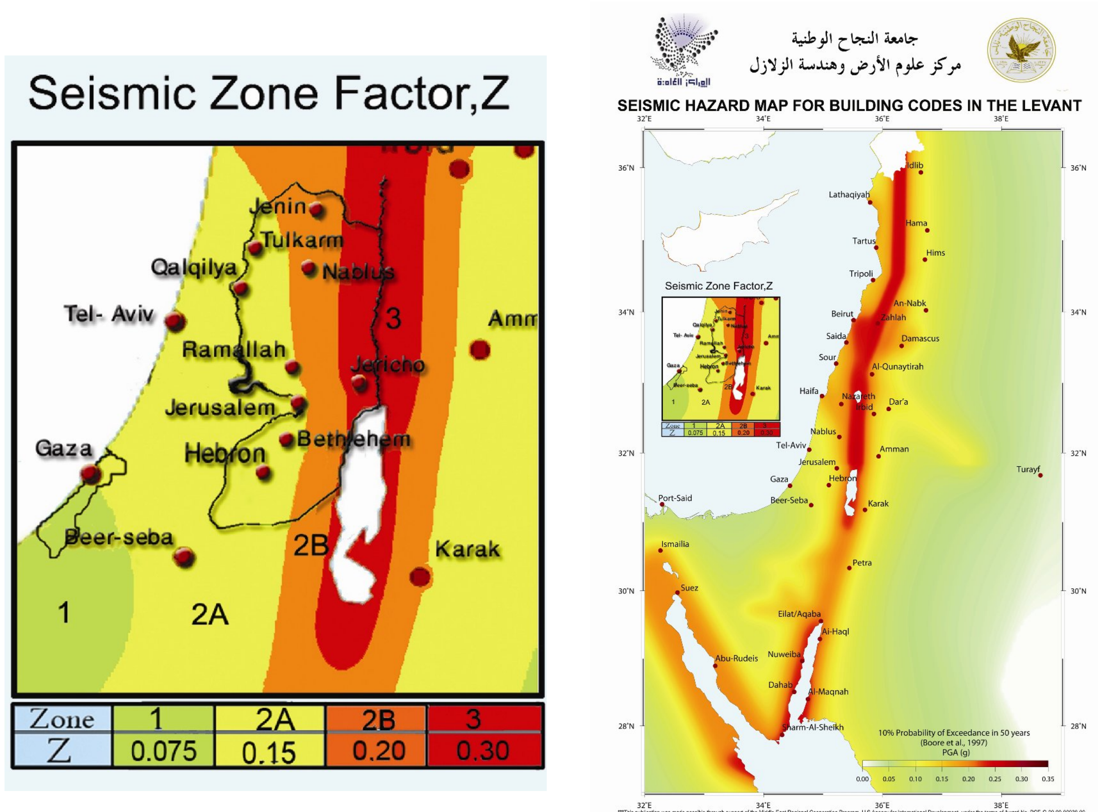

Figure 1.3: Seismic Hazard Map and Seismic Zone Factor (Source ESSEC)

Bayti has requested that the study take place over a ‘two stage’ process, identifying the 300 dunum land mass allocated for the Rawabi construction as ‘stage 1’ and the lower 550 dunum land mass as ‘stage 2’ (see figure 1.4).

Based on the scope of services, the seismic investigation under the contract (the contract signed between Bayti and the ESSEC) should deliver the following tasks:

- Micro-Zonation maps:
  1) Fundamental nature frequency map ($\mathbf{T_s}$) for part (1)
  2) Soil profiles for part (1) and most of part (2).
  3) Shear wave velocity maps for part (1)
  4) Landslides maps for part (1) and part (2).
- Data collection, field visits, field surveys, data acquisition, data analysis
- Seismic report no. (1) as described in Annex (2) in the contract.

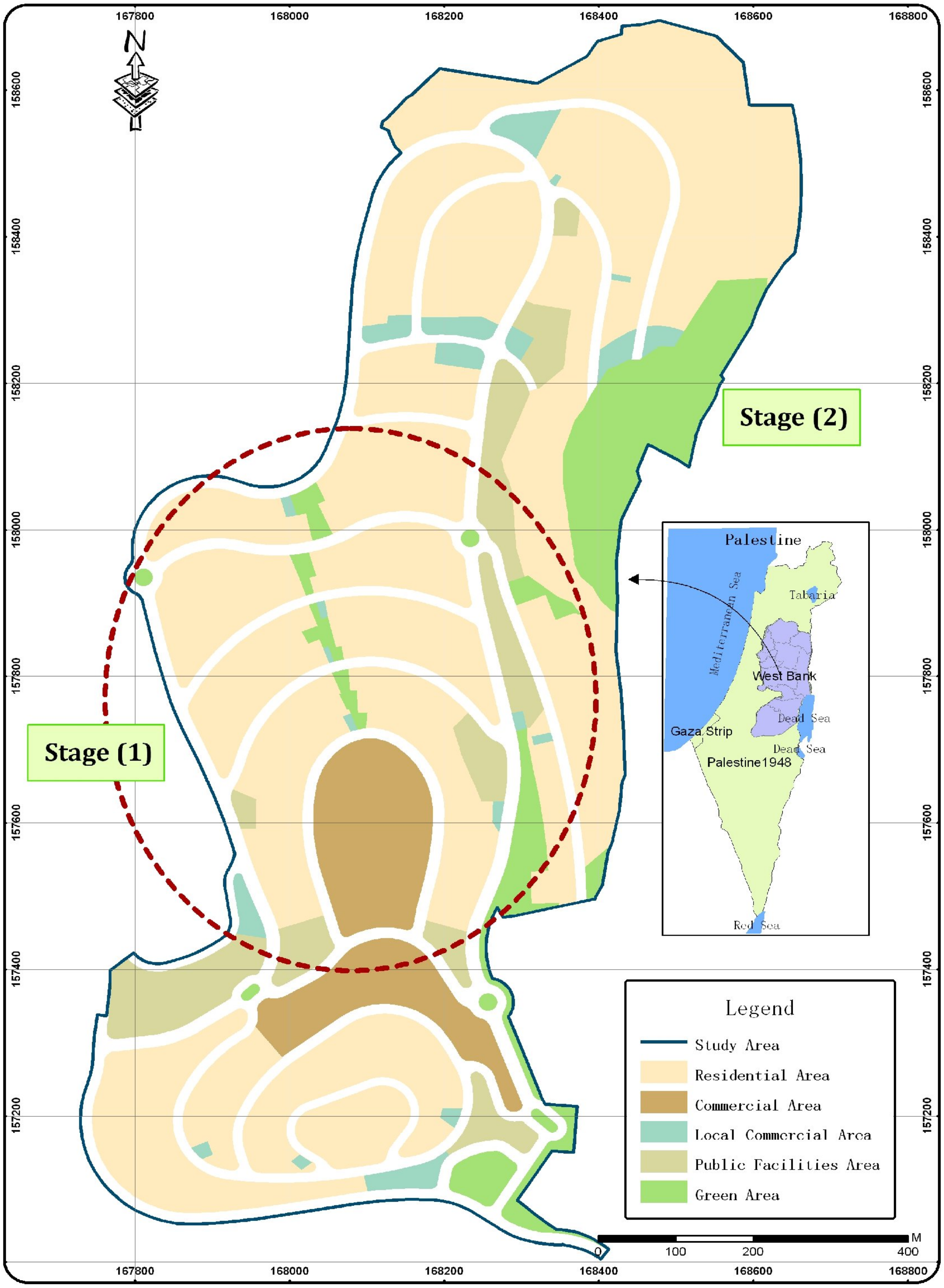

Figure 1.4: Rawabi Site Plan and the study area (Stage 1 and Stage 2).

2. GEOPHYSICAL SEISMIC STUDY: Site Investigations

2.1 Local Geology

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

The exposed sequence of rocks in Rawabi area (Ramallah Group) mainly consists of carbonates; limestone, dolomite, marl and chalk and it includes other sediments such as chert, clay, with ages ranging from lower Cretaceous to upper Cretaceous. The formations, outcrops, and reported lithology from groundwater boreholes indicate that the limestone thickness ranges from about 50-210 m with dolomite, some chalk, chalky marl and marl appearing at different locations of the formations.

2.2 Cavities in Rock

A topic of concern in many projects involving rock excavation is whether or not there are undetected cavities below an apparently solid bedrock surface or whether cavities could develop after construction. These cavities may occur naturally in karst or pseudokarst terrains, may be induced by human interference in natural processes, or they may be totally due to man's activities. The term "cavities" is used since it covers all sizes and origins of underground openings of interest in rock excavations.

The presence of cavities has a number of rock engineering implications, including:

(I) Irregular or potentially irregular bedrock topography due to collapse or subsidence and associated unpredictable bearing surface elevations.

(2) Excavation difficulties, with extensive hand-cleaning, grouting, and dental treatment requirements.

(3) Questionable support capacity with a potential for collapse or subsidence over cavities, or settlement of debris piles from prior collapses, all of which may be concealed by an apparently sound bedrock surface.

(4) Ground water flow problems, with requirements for tracing flow paths, or sealing off or diverting flows around or through the project area. Surface water flows may be affected by underground cavities, sometimes by complete diversion to the subsurface.

(5) Contaminants may flow rapidly into open channels, with minimal natural filtration and purification, possibly contaminating local water supplies.

Most natural and induced cavities develop in soluble rocks, most notably limestone, dolomite, gypsum, and rock salt. Typical karst conditions develop in limestone and dolomites by the solution-widening of joints and bedding planes caused by flowing ground water. Eventually, this process develops into a heterogeneous arrangement of cavities with irregular sinkholes occurring where cavity roofs have collapsed. However, the amount of solution-widening that occurs in limestone and dolomite is negligible in the lifetime of a typical project. Already existing cavities are, therefore, the major concern.

Gypsum and anhydrite are less common than limestones, but they have the additional concern of solution and collapse or settlement during the useful life of a typical structure. Flow of ground water, particularly to water supply wells, has been known to dissolve gypsum and cause the collapse of structures. Rock salt is probably one of the most soluble of common geologic materials, and may be of concern in several areas in the world. While natural occurrences of cavities in rock salt are rare, cavities may have been formed by solution mining methods and collapse and creep have occurred in some mined areas. As cavities are difficult to detect, a combination of detailed preconstruction investigations and construction investigations should be expected in potential cavity areas. The occurrence of cavities on a local scale is more difficult to determine, and many significant cavities can be missed by a typical exploration program.

Geophysics may be of some use in initial site investigations in locating larger cavities but may also miss smaller ones. Remote sensing using air photos, infrared imagery, and side-looking radar are useful in determining trends of cavities and jointing in an area, as well as determining the structural geology features associated with rock salt exposures.

Since cavity occurrence is difficult to determine on a local scale, the only practical solution, after initial site studies, is to place a test boring at the location of each significant load-bearing member. Such an undertaking is costly, but represents the only reasonable approach in areas of high concern.

A number of techniques/methods are available for addressing design and construction problems associated with project sites where cavities are present. The following provides a brief listing of alternative techniques.

a. Avoid the area for load-bearing use if possible.
b. Bridge the cavity by transferring the loads to the sides of the cavity.

c. Allow for subsidence and potentially severe differential settlements in the design of the foundation and structure.

d. Fill in the cavities to minimize subsidence, prevent catastrophic collapse, and prevent progressive enlargement. Support piers or walls may be used for point supports in larger cavities, or cavities may be filled with sand, gravel, and grout. Cement grout can be used to fill large cavities to prevent roof slabs from falling, eliminating a potential progression to sinkholes. Grout can also fill cavities too small for convenient access, thereby reducing permeability and strengthening the rock foundation.

e. Avoid placing structures over gypsum, salt, or anhydrite beds where seeping or flowing water can rapidly remove the supporting rock.

2.3 Methodology and Data Analysis

2.3.1 Geophysical experiment

The subsurface geology is extremely important for the development of highly populated, tectonically arid regions such as the Middle East. The shallow upper part (ten to hundred meters) of the rock formation section is the most significant part for civil infrastructures. The seismic refraction technique is considered an accurate geophysical method to investigate the shallow geological structures of an area. During the past decades, the seismic parameters obtained by a refraction survey have been widely used in cases of site investigation as indicators of rock mass quality. The main objective of the 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 importance in land use management of various civil engineering purposes.

2.3.2 Detection of Seismic Waves

Seismic waves are generated usually by weight dropping, i.e. a sledge hammer. The seismic signals generated from the shot propagate 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) and is widely used in most seismic refraction techniques.

The profile type used in this study is the reversed profile and 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 was 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.

2.3.3 Data Acquisition and Analysis

The seismic refraction survey was conducted on more than 30 seismic profiles (see Fig. 2.1 and the photos in appendix no. 2). The distance between the two receivers (geophone interval) was between 3-5 meters, with three shot points. Many interpretation techniques are published in seismic refraction data analysis and 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 the interfaces were obtained from the travel time-distance curves for the P-waves. Table 2.1 summarizes the results obtained from the seismic profiles for this study:

The P-waves were picked up as first arrivals. The underground model beneath the profiles indicates different velocities for the materials beneath the seismic lines; the first layer represents the soil cover in the studied area, such as:

- Weathered surface material with maximum thickness of about 5 m and P-waves velocities range between 300 m/sec and 600 m/sec,
- Marl, marly limestone and limestone materials with maximum thickness of about 17 m and P-waves velocities range between 600 m/sec and 1465 m/sec,

The travel time curves analyses of layer two showed longitudinal wave velocities (P-waves) in the range of 1193 m/sec to 2867 m/sec, and P-waves velocities between 1868

Figure 2.1: Sketch location map of the study area, also shown are the distribution of The seismic profiles.

Table 2.1: Summary of results obtained from the seismic profiles

Line 1	Layer 1		Layer 2		Layer 3	
	V m/sec	Thickness (m)	V m/sec	Thickness (m)	V m/sec	Thickness (m)
Line 1-1	614	0.3 - 1.5	1505	7 - 9.5	4106	∞
Line 1-2	615	0.0 – 2.5	1377	1 - 8	2401	∞
Line 1-3	693	1 - 4	2042	7.5 - 10	3247	∞
Line 2-1	803	0.5 - 5	1819	∞	-------	∞
Line 22-33	919	3.5 - 5	2867	∞	-------	∞
Line 4-1	305	0 – 0.5	1306	9 - 15	2262	∞
Line 4-2	864	0 - 6	1207	5 - 9	2531	∞
Line 4-3	582	3.5 - 5	2027	9 - 10	3365	∞
Line 4-4	305	0 – 1.5	1863	17.5 - 21	3333	∞
Line 5-5	772	0.5 - 2	1422	∞	------	∞
Line 6-1	930	0.6 - 1	2555	0 - 1	2674	∞
Line 6-2	1689	5 - 7	2009	12.5 - 20	4241	∞
Line 6-3	305	0 - 1	1660	11 -12.5	3125	∞
Line 7-7	455	0.1 – 2.5	2144	20 – 22.5	3333	∞
Line 8-8	929	2.5 - 7	1429	12.5 - 20	3151	∞
Line 9-9	385	0.3 - 1	1193	5.5 – 7.5	3413	∞
Line 10-10	597	2.5 – 2.65	1953	10 – 10.5	3859	∞
Line 11-11	522	1.5 – 1.6	1564	7.5 - 10	3208	∞
Line 12-12	1465	15 – 17.5	2275	∞	-----	∞
Line 13-13	1200	0.25 – 2.5	1952	15 – 16.5	4000	∞
Line 14-14	587	1 - 2	1414	2 - 5	1868	∞
Line 15-15	1003	0 – 3.5	1229	2.5 - 15	2852	∞
Line 16-16	851	1.5 – 2.5	2050	15 – 17.5	4268	∞
Line 17-17	879	2.5 - 3	1485	12.5 - 15	4308	∞
Line 18-18	1001	2 – 2.7	1579	3.5 – 6.5	2461	∞

m/sec 4308 m/sec for the third layer. Both of these modeled layers (layer two and layer three) are interpreted as carbonate sediments of different types: the second layer of clayey marly sediment material; and consolidated carbonate materials of limestone, chalky limestone, and dolomite limestone are the typical lithology of layer three. Appendix 2.2: shows the travel time curves and the corresponding velocity ground models (geological cross sections for the three layers) beneath for selected profiles from 30 Seismic Profiles, and for more details, see all the seismic profiles in appendix no. 6.

2.4 Safe Cover Thickness over Caves

All the available information suggests that the "rule-of-thumb" that cover thickness should exceed cave width is excessively over-conservative in most of the strong limestones that form cavernous karst. Evidence from the various available sources suggests that a roof thickness of about half the cave width is stable and safe under most conditions of loading. In view of the extreme variability of karstic ground conditions, a guideline that roof thickness should exceed 70% of cave wide (i.e., thickness/width = t/w = 0.7) is probably more appropriate in most karst terrains in strong limestone. This value is still conservative under normal structural loading, and is very conservative under highway loading.

An alternative approach to the safe cover thickness is based on the decline of imposed stress at increasing depths beneath a loaded foundation structure. It has been suggested that induced collapse of a cave roof is unlikely where the loading stress is less than 5-10% of the existing overburden stress. References to the undistorted bulbs of pressure perceived by foundation engineers suggest that this stress ratio is reached at a depth of about 4m beneath a foundation pad 1 m^2 carrying a load of 1 MN, where overburden stress increases by 25 KPa per meter depth. This takes no account of cave width and assumes there is no cave roof at a critical state of imminent collapse. It is however slightly controversial because it does not account for stress redistribution around an open cave, where wall failure is unlikely. A safe thickness of 4m is commensurate with guideline figures derived from other considerations. Where a foundation pad 2 m^2 carries a load of 4 MN, even with an applied stress of 1 MPa, the imposed stress exceeds 10% of overburden stress at a depth of about 6m. This is implies that greater thicknesses of sound rock should be proven where heavy structural loads are placed on karstic rock that may contain large caves. There are multiple benefits in using larger footings that impose lower stresses on cavernous ground.

2.5 Conclusions and Recommendations

Based on the outcropping geological cross-section in the study area and the ground velocity models deduced from the P-wave velocities of this study, the subsurface geological formations beneath the seismic profiles are interpreted as soil cover of soft weathered material (clay and marly-clay materials) which forms the first layer in several sites in the studied area, with a maximum depth of 5 meters. The second layer is explained as non-consolidated carbonates of marly sediment materials in the southern part of the studied area (upper part and city center), as well as consolidated carbonates in the northern part. Whereas the third layer is interpreted as consolidated carbonate materials of limestone, chalky limestone, and dolomite limestone.

The corresponding velocity ground models (geological cross sections for the three layers) beneath all the seismic profiles show clearly an overlapping between layer one and layer two as well as between layer two and layer three at different locations of the study area which means that there are lateral and vertical variations in the mineralogy and the geomorphology of the layer boundaries.

The investigated subsurface geology beneath the profiles does not show clear cavities at shallow depth but slight morphological differences at the interface of layer two with layer three could indicate small scale change voids.

It is recommended that the soft weathered material and most of clay-marly sediment materials be totally removed and that the excavation should reach the consolidated carbonate materials (limestone, chalky limestone, and dolomite limestone). And, consequently, this leads to harmony and more or less to homogeny in the physical properties of the engineering soil.

Based on the values of P-wave velocities in the two or three layers and using the approximate values of the Poisson's Ratio for each layer (ν = 0.25, 0.30 and 0.35), the value of shear wave velocity (V_s) will be as follows:

- $\mathbf{V_s}$ = 250 – 695 m/sec for the first layer
- $\mathbf{V_s}$ = 495 – 1590 m/sec for the second layer
- $\mathbf{V_s}$ = 934 – 2390 m/sec for the third layer

The values of shear wave velocities (V_s) at the proposed foundation levels will be around between 500 m/sec and 1500 m/sec. Based on international and regional seismic design codes, such as: Uniform Building Code 97, International Building Code IBC, Jordanian Building Code 2005 and Arab Uniform Building Code 2006 the type of soil profile for the shear wave velocities mentioned above (500 m/sec and 1500 m/sec) will be S_C and S_B. In design it is recommended to use:

- S_B for most of the buildings in studied area in Stage 1 and Stage 2.
- S_C for the buildings founded on marly-limestone soil foundations.

For more details about the type of soil profiles and the shear wave velocities (V_s) in the studied area see the microzonation map presented in appendix no. 5.