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Geotechnical, geophysical and geostructural analysis for conservation of the Stylite Tower of Umm er-Rasas

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1. INTRODUCTION – SCOPE OF THE WORK

The present work reports the results of an engineering geotechnical, geophysical, seismic and structural analysis conducted by a joint team of expert from ISPRA, Geological Survey of Italy and ENEA, Department for Natural Risk Prevention and Effect Mitigation.

The work is finalized for the conservation of the Stylite Tower of Umm er-Rasas for providing a technical support to the Department of Antiquities of the Hashemite Kingdom of Jordan, for long-term restoration and structural conservation of the structure.

The Tower, even if with some damage, is still in intact condition and represents the unique surviving example of this kind of monuments, given that a similar one in Syria has been severely damaged by an earthquake. The height of the tower is about 13.50 m. The structure is constructed with trimmed stone blocks.

Investigation has been mainly focused on the following aspects:

- ✓ Analysis of potential causes and effects that lead to present damage of the tower;
- ✓ Analysis of available technical documentation and data from investigation already undertaken in recent years;
- ✓ Implementation of further investigation based on engineering geological techniques, geophysical analysis and structural modelling vs. seismicity to complete the scenario of damage cause-effect;
- ✓ Proposal of a set of potential works for the reinforcement of the tower in the light of a long-term conservation strategy.

The field activities have been undertaken in the site on July 2014 whereas analysis and interpretation of data were conducted from September to December 2014.

Results of the study have been presented on 21st December 2014 in Amman, Department of Antiquities, for discussion and agreement of preliminary proposals of reinforcement works.

2. DESCRIPTION OF THE SITE AND INVESTIGATION PLAN

The Byzantine Stylite Tower of the UNESCO site of Umm er-Rasas (Kastron Mefa'a) is sited ca. 1.2km NW from the core archaeological site, (31.30'43.60"N; 35.55'18.92"E) at an elevation of approx. 735m a.s.l (Figure 1).

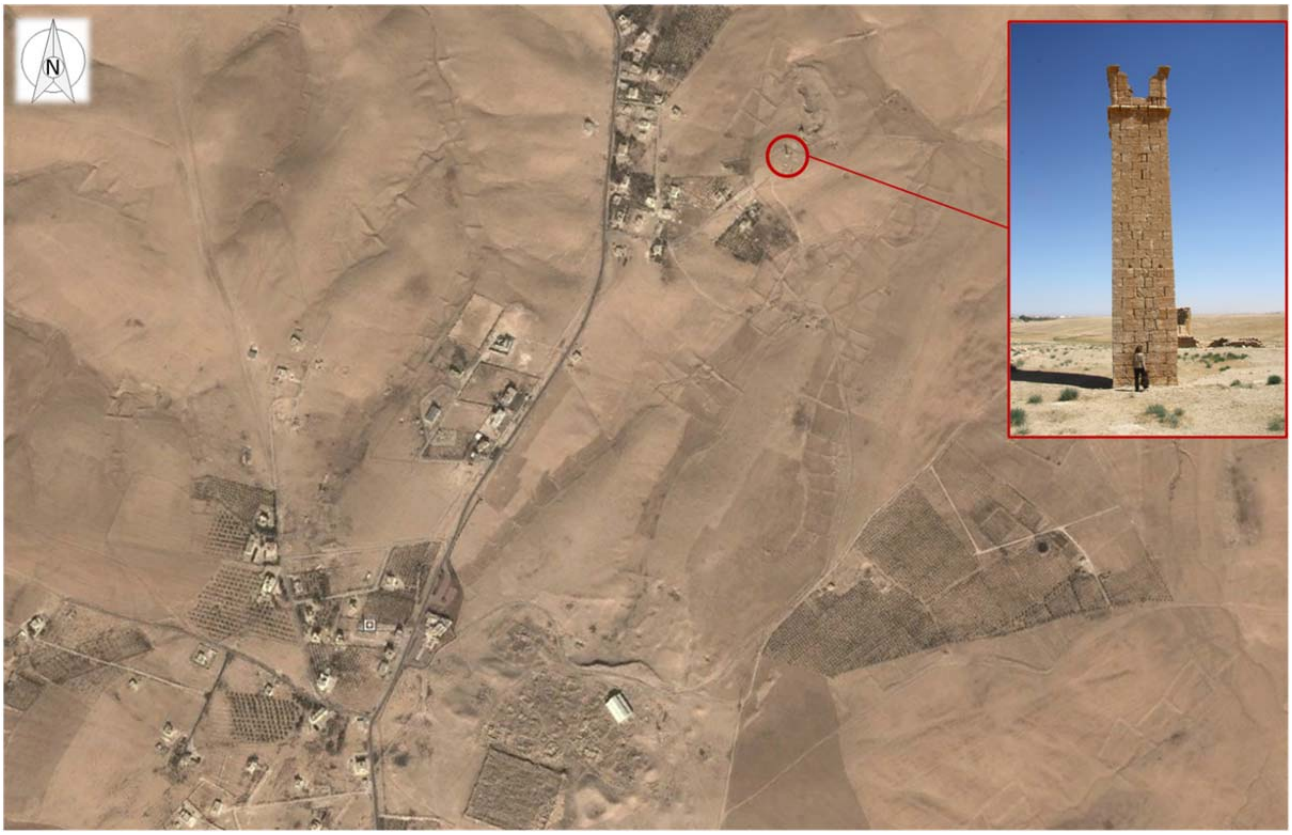


Figure 1. Aerial view of the main archeological area of Umm ar-Rasas; the red circle shows the location of the Stylite Tower (top-right image)

According to available sources of information[1], the tower was constructed in the first half of the 6th century and was originally surrounded by a wall. The tower, with a square base area of 2,52x2,52m and a height of 13,50m, is made of 35 rows of trimmed local limestone stone blocks. The structure is directly founded on the bedrock at a approx. 1m depth. The original chamber on the top along with the dome/vault collapsed, probably as a result of a seismic event occurred in 779. The collapse produced an accumulation of debris and roof stones inside the structure; further filling of stones and gravel until 1/3 of the height was probably done to provide some stability of the structure.

[1] Koellisch W. (2006). *Report on the investigation of the Stylite Tower of Umm er Rasas.* DoA/CIM, p. 56.

The tower presents cracked stone blocks, especially on the base layers (Figure 2), and displacement of blocks located at different heights, temporarily filled with stone slabs (Figure 3).

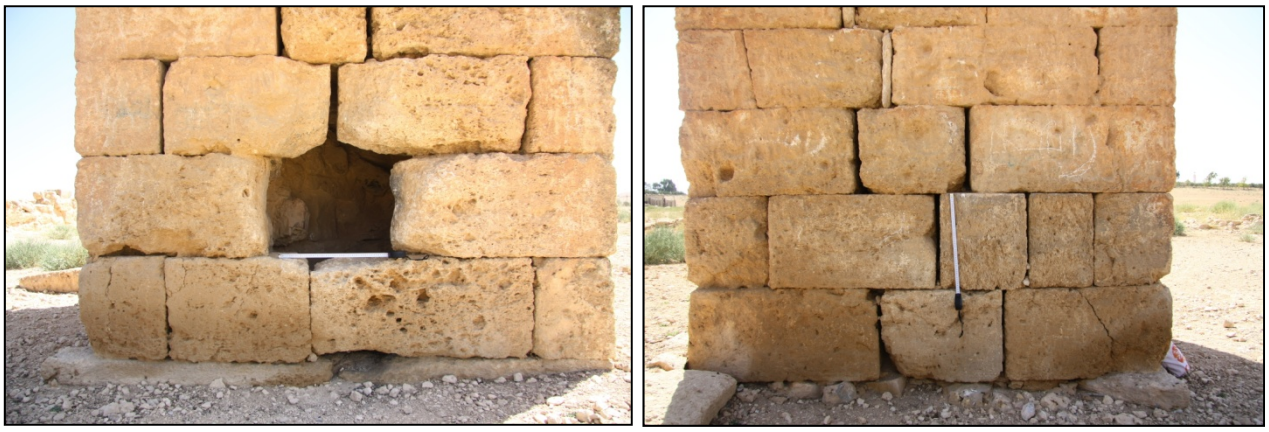


Figure 2. Cracked and displaced blocks located on the N (left) and E (right) sides of the tower

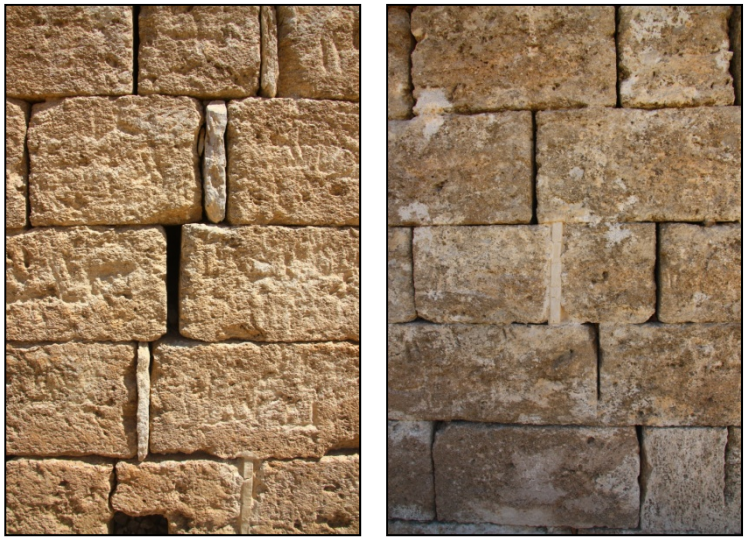


Figure 3. Displacement of blocks partially filled with stone slabs

The limestone blocks are also affected by different nature and degree of weathering.

From a geological point of view[2], the area of Umm er-Rasas characterized by the presence of 55-65m thick Al-Hisa Phosporite formation (Campanian-Maastrichtian) composed by three different members. The Bahiya coquina member (from few meters to 30m thick), mostly composed by fossiliferous limestone - rich in e.g. oyster shell, foraminifera, gasteropods, bivalves - is outcropping in the area. The blocks forming the tower are composed by such a material.

[2] Azzam M. and Doukh F. (2009). *Site investigation for Saint Stylite Tower at Um er Rasas.* Natural Resources Authority, p. 20.

According to the map of seismic zones in Jordan, the area is characterized by high seismicity (2B) with the following reference parameters:

- seismic zone factor: *z=0.2*
- Soil section type: *Sb*
- Shear waves velocity: *(Vs)=760-1500m/s*
- Earthquake responding coefficient for acceleration: *Ca=0.2*
- Earthquake responding coefficient for velocity: *Cv=0.2*

Boreholes and laboratory testing on rock materials (Unconfined Compressive Strength, Point Load tests, rock specific gravity & absorption, chemical analysis) were performed in November-December 2009 and results reported in Azzam and Doukh (2009).

The following methods and techniques (field and data analysis) have been applied for reconstruction of geotechnical, geophysical and structural characteristics of the Stylite Tower area and structure:

a. Visual analysis of the lithological component of the bedrock and stone blocks that constitute the structure of the tower;

b. Execution of no. 310 Schmidt-hammer tests on 31 base blocks and data analysis for *in-situ* assessment of the Uniaxial Compressive Strength (UCS) of the limestone material that form the structure;

c. Execution of no. 4 seismic passive tests using a digital tromograph (Tromino® instrument) and data analysis with HVSR technique to investigate the fundamental frequencies of the soil/bedrock and of tower structure;

d. Preliminary structural modeling of the tower under static and dynamic conditions for analysis of potential damage.

The main scope of the above studies is the elaboration of a set of potential typologies of reinforcement works for a long-term sustainable structural conservation of the Stylite Tower that have been elaborated in the present study.

3. Analysis of Uniaxial Compressive Strength (UCS)

Assessment of geomechanical strength of blocks forming the tower has been undertaken through in-situ Schmidt-Hammer test. Analysis of UCS has been carried out on 31 stone blocks that compose the first 3 base rows of the Stylite Tower, divided following geographical orientation of the walls (Figure 4).

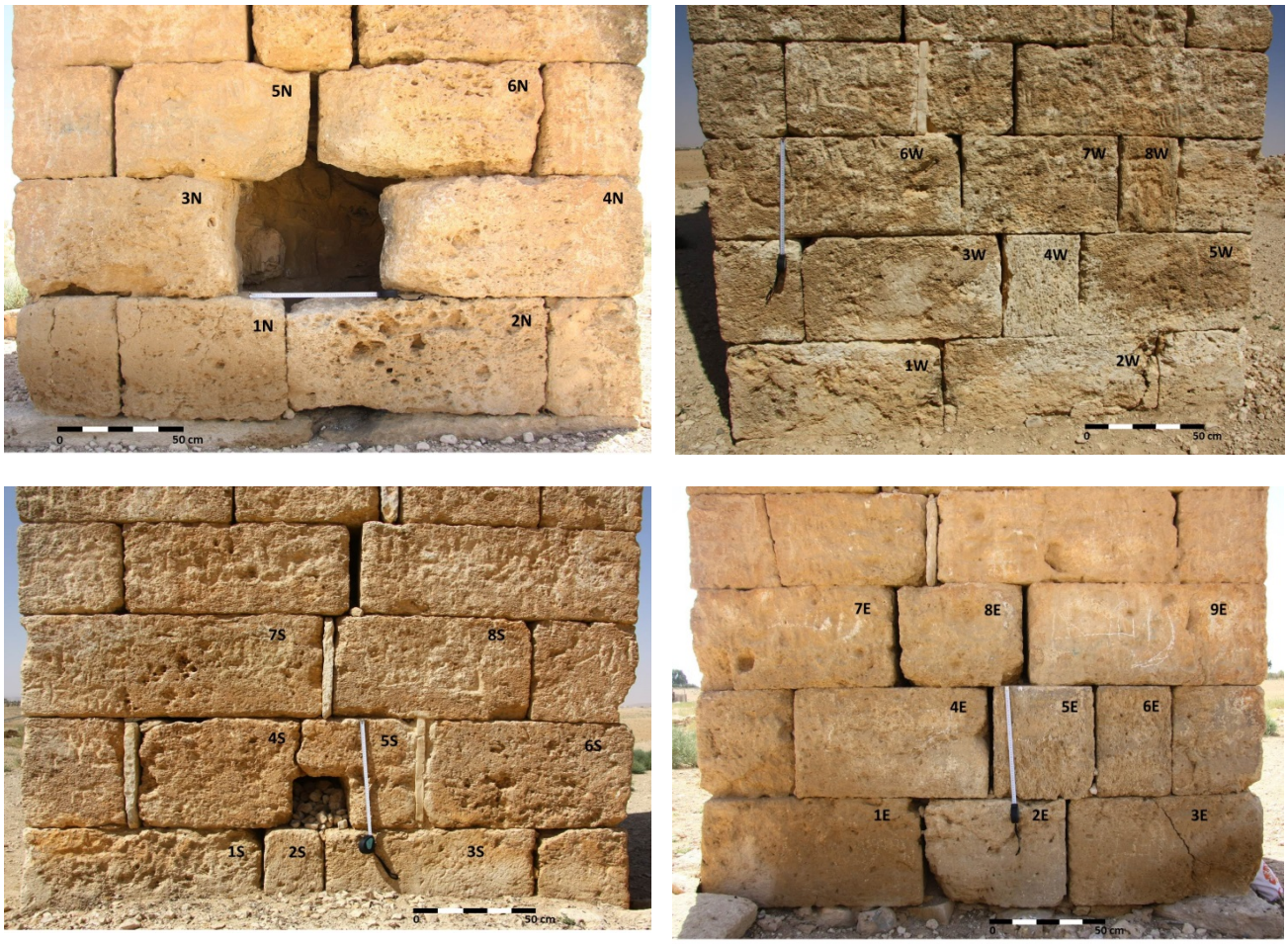


Figure 4. Identification and numbering of the stone blocks vs. wall orientation: N side (upper left); W side (upper right); S side (lower left); E side (lower right).

On each block, therefore, a mechanical test with the Schmidt-hammer instrument has been executed (Figure 5).

The Schmidt-hammer test is an empirical, non-destructive site analysis for the reconstruction of the uniaxial compressive strength (UCS) of concrete or natural rock.

The international reference standards are those proposed by ISRM[3] (1978), and ASTM[4] (2001).

Figure 5. Execution of Schmidt-hammer test on the western wall of the Stylite Tower

The Schmidt hammer consists of a spring-loaded piston that, when pressed orthogonally against a surface, is automatically released onto the plunger. Part of the impact energy of the piston is consumed largely by absorption, by plastic deformation of rock material under the plunger tip, and transformation into heat and sound. The remaining energy represents the impact penetration resistance, or hardness, of the surface and enables the piston rebound. The harder the surface, the shorter the penetration time or depth and hence the greater the rebound. The distance travelled by the piston after rebound, expressed as a percentage of the initial extension of the key-spring, is called the rebound value (R), which is considered to be an index of surface hardness.

The standard L-type Schmidt hammer, used for rocks, provides a 0.735Nm impact energy. Provided that the hammer impact results in uniform compaction:

[3] ISRM (1978). *Suggested methods for determining hardness and abrasiveness of rocks*. Int. J. Rock Mech. Min. Sci., Geomech. Abstr. 15, 89– 97.
[4] ASTM (2001). *Standard test method for determination of rock hardness by rebound hammer method*. ASTM Stand. 04.09 (D 5873-00).

- Ratios of rebound values measured on different homogeneous surfaces at two different energy levels should be constant; and
- Higher impact energy, corresponding to probing a larger volume of material by a deeper and wider penetration, should reduce scatter in rebound values obtained at different points on a heterogeneous surface.

Schmidt-hammer rebound values obtained along non-horizontal impact directions are influenced by gravitational forces to varying degrees. To annul these effects, non-horizontal rebound values must be normalized with reference to the horizontal direction.

Rules have yet to be established for determining the number of impact points necessary to capture hardness variations (at the micro structural scale) over a specimen surface and for reducing the corresponding rebound value readings to a representative value (Shorey et al.[5], 1984).

This reduction process is carried out by removing the lowest readings attributed to the presence of hidden cracks and averaging the others. ISRM (1978), recommended averaging the upper 50% of at least 20 single impact readings, after eliminating any reading from points that show signs of cracking.

ASTM (2001) suggested taking at least 10 single impact readings, discarding those differing from the average by more than 7 units, and averaging those left.

Both standards require that the impact points be separated by at least one plunger diameter. Shorey et al. (1984), on the other hand, preferred lower mean values because they correlated better with UCS on the other hand, preferred lower mean values because they correlated better with UCS.

Amaral et al[6]. (1999) indicated that it is important to understand the variations in the readings because they are strictly related to the material heterogeneity and therefore, all values should be taken into account. In our survey, all rebound values have been taken into account since lowest

[5] Shorey P.R., Barat D., Das M.N., Mukherjee K.P., Singh B. (1984). *Schmidt hammer rebound data for estimation of large scale in situ coal strength.* Int. J. Rock Mech. Min. Sci., Geomech. Abstr., 21, 39-42.

[6] Amaral P.M., Rosa L.G., Fernandes J.C. (1999). *Determination of Schmidt rebound hardness consistency in granite*. Int. J. Rock Mech. Min. Sci., 36, 833-837.

values can reflect better local conditions of investigated materials (i.e. weathering state, superficial water content, textural irregularity).

The instrument utilized in the field is a Geohammer® with the following characteristics:

Range of measurements: 10–250N/mm^2

Impact energy: 0.705 N/mm^2

International standard of reference: ASTM D5873 UNI EN 12504-2.

The correlation curves IRbm (Rebound Index)/UCS are:

$$R_m = aI^b$$

where:

R_m = average compressive strength of the rock (MPa);

I = rebound index;

a,b = calibration coefficient derived by destructive tests on rock samples.

For any analyzed stone block, a series of 12 readings (N), with a minimum distance of 50mm, have been executed. The readings have been distributed and executed according to the exterior quality of the material (Figure 5). Weathered and/or cracked portions of the stone blocks, when present, have been selected for the test in order to reconstruct the minimum value of UCS.

Uniaxial compressive strength values (σc) are calculated through the average of N values and the corresponding value of strength in a diagram that takes into account the rebound value (R) and the angle between the horizontal and the axis of the instrument (Figure 6).

The A total of 310 registrations of rebound values (N total) on 31 single limestone blocks have been done (Table 1 and Annex 1).

The same values have been classified according to rock strength classes proposed by ISRM (1978) (Table 2) in order to provide a general view of the rock quality for any column block analyzed.

The same data have been reported in Figure 7 where all single results (in MPa) are displayed according to walls orientation and grouped in rock strength classes, following the above ISRM classification (Figure 8).

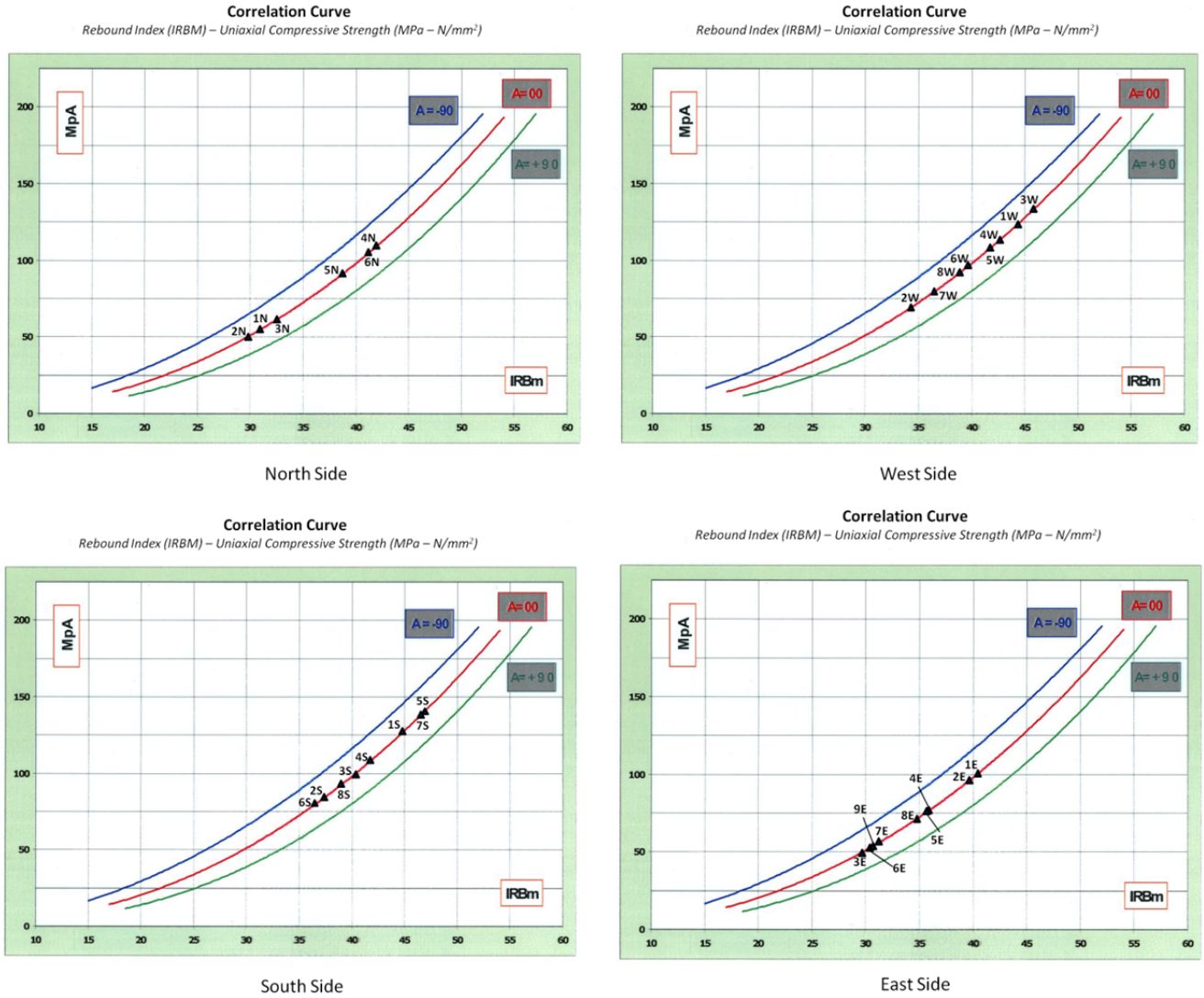


Figure 6. Correlation between Rebound Index (IRBm) – UCS (MPa) following direction of test execution vs. stone blocks.

As already mentioned, the quality of the rock is strongly depending on lithological aspects (i.e. texture, type and quality of cement, mineralogical content, presence and orientation of bedding, macro and micro-structural discontinuities); presence and type of degradation (i.e. chemical, physical, mechanical weathering).

UCS analysis reports that ca. 6% of samples are in R2 class, whereas 58% and 34% are respectively in R3 and R4 classes. This means that 29 out of 31 limestone blocks analyzed present a medium-high value of strength.

Lower values of strength are mainly due to presence of high weathering conditions and presence of cracks that affect the limestone material mostly concentrated in the NE corner of the structure.

	Block no.	UCS (MPa)		Block no.	UCS (MPa)		Block no.	UCS (MPa)		Block no.	UCS (MPa)
NORTH WALL	1N	54.8	EAST WALL	1E	100.1	SOUTH WALL	1S	126.4	WEST WALL	1W	123.8
	2N	48.9		2E	94.5		2S	83.8		2W	67.3
	3N	59.4		3E	48.2		3S	99.0		3W	133.1
	4N	108.9		4E	75.8		4S	110.1		4W	117.5
	5N	90.7		5E	75.3		5S	139.9		5W	108.9
	6N	106.5		6E	50.8		6S	79.3		6W	95.6
				7E	54.4		7S	137.8		7W	79.2
				8E	69.1		8S	89.1		8W	89.6
				9E	51.6						

Table 1. UCS values calculated for single stone blocks of the Stylite Tower

	Block no.	UCS (MPa)		Block no.	UCS (MPa)		Block no.	UCS (MPa)		Block no.	UCS (MPa)
NORTH WALL	1N	R3	EAST WALL	1E	R4	SOUTH WALL	1S	R4	WEST WALL	1W	R4
	2N	R3		2E	R3		2S	R3		2W	R3
	3N	R3		3E	R2		3S	R3		3W	R4
	4N	R4		4E	R3		4S	R4		4W	R4
	5N	R3		5E	R3		5S	R4		5W	R4
	6N	R4		6E	R3		6S	R3		6W	R3
				7E	R3		7S	R4		7W	R3
				8E	R3		8S	R3		8W	R3
				9E	R3						

Table 2. Classifications of the stone blocks of the Stylite Tower with respect to mechanical strength classes for rocks (ISRM, 1978). R1: very weak rock; R2: weak rock; R3: moderately weak rock; R4: strong rock; R5: very strong rock

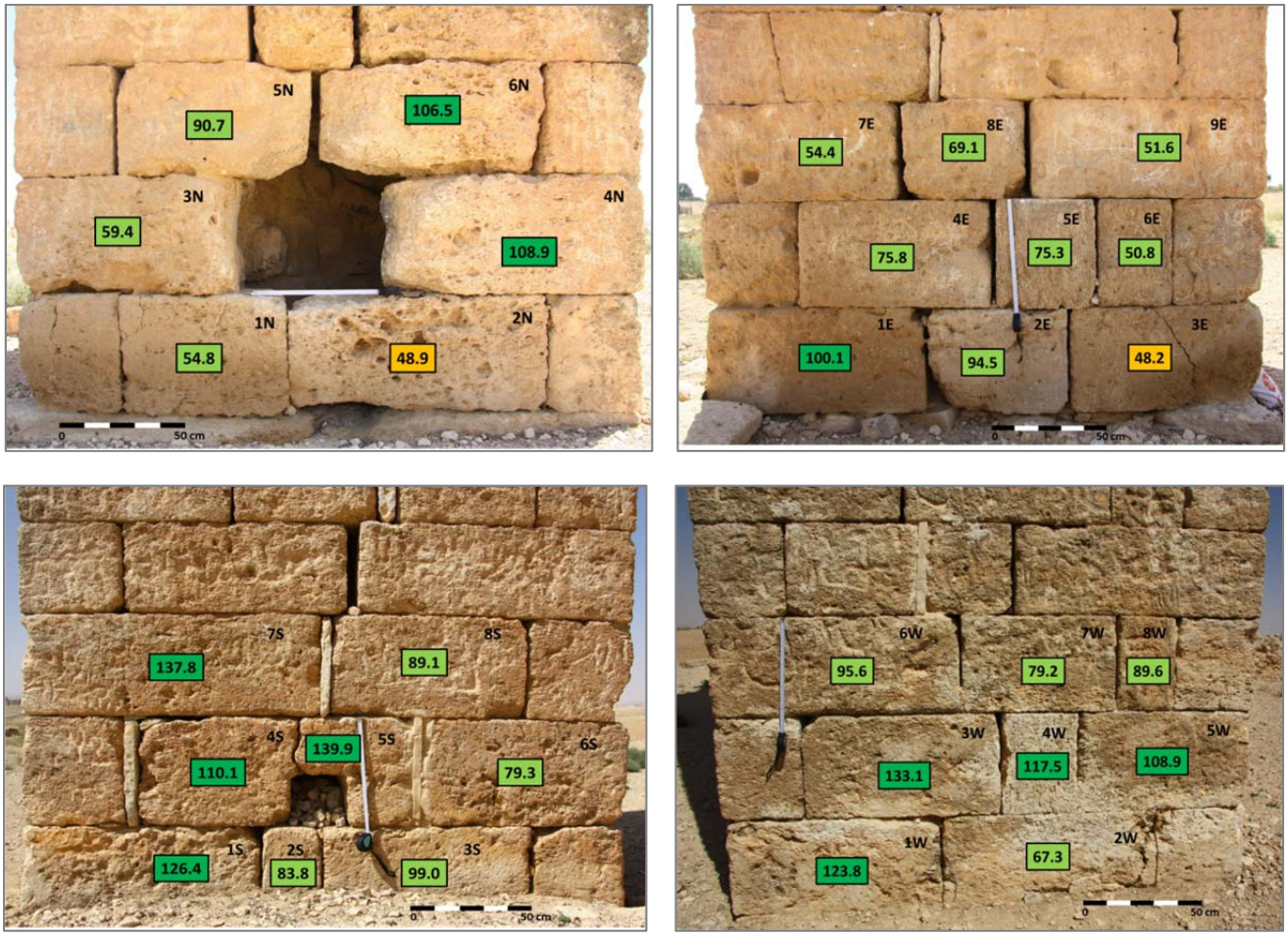


Figure 7. UCS values of the limestone blocks represented as classes of strength (see Table 2) according to ISRM (1978)

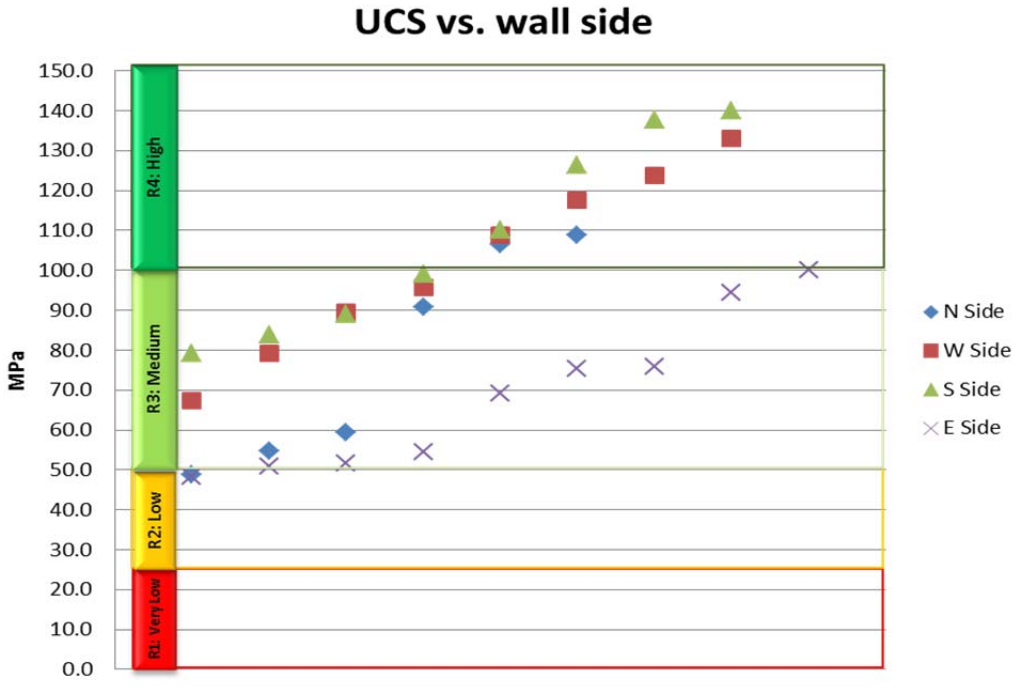


Figure 8. Diagram of UCS values and strength classes (ISRM, 1978) of the limestone blocks vs. wall orientation

4. PASSIVE TROMOGRAPHY

4.1 Theoretical Background

Investigation of the TWL subsoil has been undertaken using a passive seismic technique (HVSR: Horizontal to Vertical Spectral Ratio) through a portable and non-invasive high resolution instrument (Tromino®).

The HSRV technique is usually employed as a very cost-effective geophysical method to investigate a site about the presence of a large impedance contrast at some depth, very likely able to amplify the ground motion. The same technique can be employed also for stratigraphic purposes such as: i) to locate the most representative sites on which further investigation(s) should be repeated/refined with other active and passive seismic techniques, ii) to verify, when other data already exist, the presence of significant heterogeneities around the investigated sites, iii) to supplement other investigations when these do not lead to clear interpretations.

From a theoretical point of view, the shape of a seismic wave recorded by an instrument at a site *x* is a function of:

1. The wave form produced from the source *s*;
2. The wave path from the source *s* to the site *x* (considering attenuations, reflections, refractions, wave guide phenomena);
3. The instrumental response.

The source is constituted by seismic tremor or seismic ambient noise, that is present everywhere on the Earth's surface, generated by atmospheric phenomena (ocean waves, wind), human activity as well as, obviously, the Earth's dynamics. It is also called microtremor because it involves very small oscillations (10–15 $[m/s^2]^2$ in acceleration), much smaller than those induced by earthquakes of any size in the near field.

The geophysical methods based on tremor are called passive because they do not require any signal generated ad hoc, as for example the explosions used in the seismic prospection.

In the frequency spectrum of tremor on flat rock, peaks at 0.14 and 0.07Hz are commonly interpreted as originated by ocean waves. These spectral components are only slightly attenuated

even after thousands kilometers because of wave guide effects. Local anthropogenic sources (i.e. traffic, industrial activity as well as the simple walk of a man) and natural sources overlap this general pattern, that is always present, but attenuate strongly at frequencies above 20Hz due to the anelastic absorption produced by rock internal friction.

Along the path between the source *s* and the site *x*, elastic waves (be them earthquake or tremor waves) undergo reflections, refractions and trapping due to wave guide effects, as well as attenuations, which depend on the nature of the subsoil along the path. On the one hand this means that the information relative to the source is quickly lost, so that the classical "ray tracing" techniques of seismology cannot be applied.

On the other hand, this also means that the only (weakly) coherent part of the signal will contain information relative to the local structure of subsoil in the vicinity of the sensor. Therefore seismic noise, which traditionally constitutes the part of the signal discarded in the classical seismologic/seismic investigation, contains valuable information about the subsoil structure. This information is deeply buried into random noise, and can only be extracted through appropriate techniques: HVSR is one of them.

The HVSR was originally developed to assess seismic risk in Japan (Nogoshi and Igarashi[7], 1970; Nakamura, 1989[8]; 2000[9]). Superficial sediment is widely known to produce enhanced shaking during an earthquake, partially as a consequence of horizontally oscillating shear waves arriving from below at the fundamental resonant frequency of the sediment. At this frequency a significant part of the shear wave energy is trapped within the sediment in a state of constructive interference. As a result, the horizontal oscillations are amplified relative to the vertical oscillations. Thus, on a 3-component seismic record, the averaged spectra of the horizontal components divided by the vertical spectrum should produce a prominent peak at the fundamental resonant frequency of the sediment. Although data should be collected during earthquakes, the method proposed by Nakamura works effectively with ambient vibration, increasing its value as a preemptive tool for earthquake risk assessment. The sources of the ambient vibrations in the wide range between 0.1Hz to 60Hz, are known (Bonnefoy-Claudet et

[7] Nogoshi M. and T. Igarashi (1971). *On the amplitude characteristics of microtremor* . J. of Seismol. Soc. of Japan, 24, 26-40.
[8] Nakamura Y. (1989). *A method for dynamic characteristics estimation of subsurface using microtremor on the ground surface*. Quarterly Report Railway Tech. Res. Inst., 30(1), 25-30.
[9] Nakamura Y. (2000). *Clear identification of fundamental idea of Nakamura's technique and its applications*.12th World Conference on Earthquake Engineering. CD-ROM.

al.[10], 2006): signals below 0.3Hz are primarily caused by oceanic waves, winds and other meteorological sources whereas signals from 1Hz and above are largely due to cultural activities such as industrial, agricultural, and traffic noise.

Beyond scientific theoretical debate regarding the type of seismic waves that actually produce peaks on the HVSR curve, this method has proven to be useful to estimate the fundamental resonant frequency in soil deposits.

The theoretical bases of HVSR are relatively simple in a 1-D layered configuration. Considering a system (Figure 9) where layers 1 and 2 differ for their density value (ρ_1 and ρ_2) and seismic wave propagation velocities (V_1 and V_2).

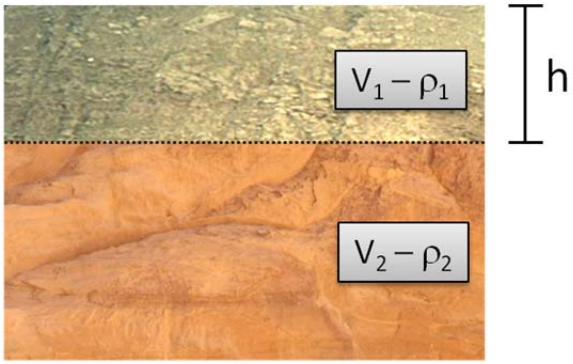


Figure 9. Schematic model of a 2-layers subsoil with different densities and seismic wave velocities

A wave travelling across layer 1 is (partially) reflected at the transition with layer 2, and the reflected wave will interfere with incident waves and will give a maximum amplitude (fundamental resonance frequency), when the wavelength of the incident wave (λ) is 4 times (or odd multiples) the

thickness h of the first layer. In other words, the fundamental resonance frequency (f_r) of layer 1 for the P waves is:

$$f_r = V_{P1} / (4\ h) \quad [1]$$

while the fundamental resonance frequency for the S waves is:

$$f_r = V_{S1} / (4\ h) \quad [2]$$

[10] Bonnefoy-Claudet S., Cotton F., Bard P.Y., Cornou C., Ohrnberger M., Wathelet M. (2006), *Robustness of the H/V ratio peak frequency to estimate 1D resonance frequency*. Third International Symposium on the Effects of Surface Geology on Seismic Motion, Grenoble, France, 30 August-1 September 2006, Paper Number 85, 10p.

Theoretically, this effect is summable so that the HVSR should show (as relative maxima) the resonance frequency of layers. This, together with an estimate of seismic velocities which is generally available at least as a first approximation, allows one to predict the thickness h of the first (or more that one) layer. This information is mostly contained in the vertical component of motion, but the idea of using the ratio of horizontal to vertical spectra rather than the vertical spectrum alone derives from the fact that the ratio provides for an important normalization of the signal for a) frequency content, b) instrumental response and c) overall signal amplitude if the recordings are taken at times of higher or lower noise levels. This normalization, which makes signal interpretation simpler, is at the basis of HVSR popularity.

An example of HVSR curve derived from a model built as in Figure 9, of passive soil motion recorded at a site is given in Figure 10.

An apparent problem with this view is that microtremors are only partially constituted of body waves, P or S, and are also composed of surface waves (and in particular of Rayleigh waves). However, since surface waves are produced by the constructive interference of the body waves, the above equations still hold approximately.

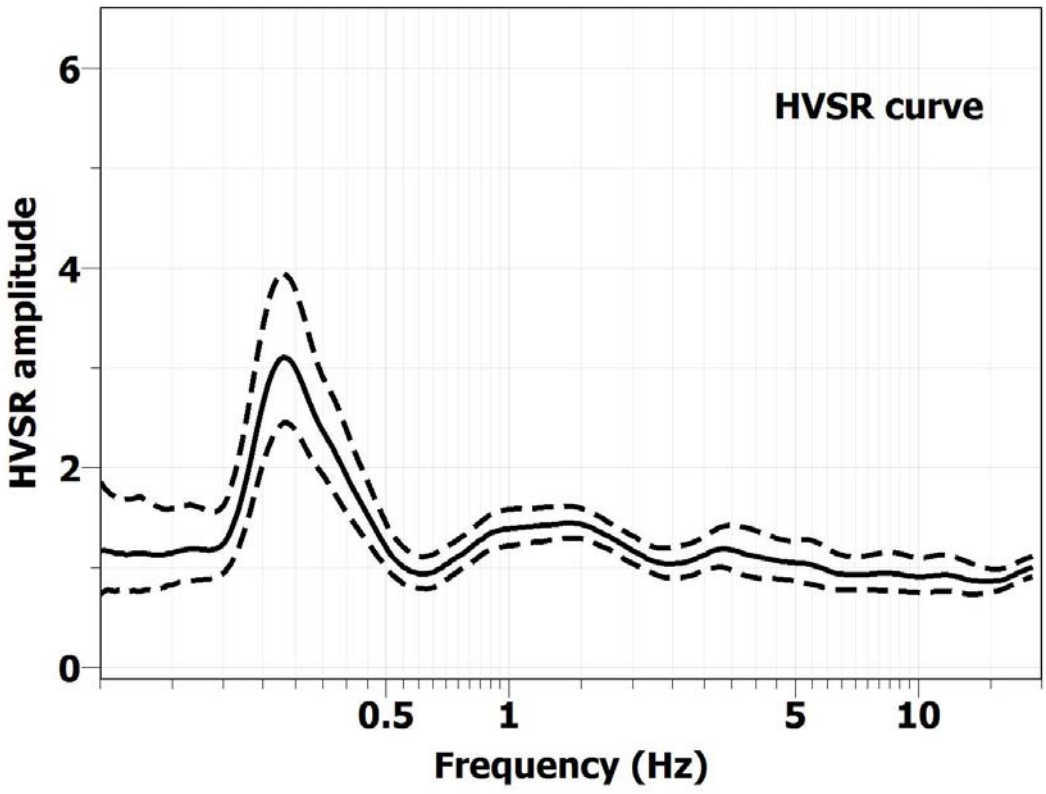


Figure 10. Example of HV ratio for different impedance contrasts z between two layers

The practical applicability of equation [2] has already been discussed in many studies both in the fields of geophysical prospection and engineering. Since the situation depicted in Figure 9 is typical of the sedimentary covers overlying the bedrock, the HVSR method was first of all applied to the problem of determining the thickness of the sedimentary covers (Ibs-Von Seht and Wohlenberg[11], 1999).

4.2 Field Investigation in the Stylite Tower area

HVSR analyses in Umm er-Rasas have been conducted using Micromed tromograph "Tromino"® TEP (Tromino Engy Plus) model, specifically designed for HVSR technique applied to a variety of engineering- and geology-related applications. This self-contained, broadband instrument is compact (1dm^3) and light weight (1kg), equipped with three orthogonal electrodynamic sensors (velocimeters) - powered by two 1.5 V AA batteries - and an internal GPS antenna (Figure 11).

Analysis of data collected is accomplished using preliminarily the Micromed software "Grilla" ver.6.4 and then after with open-source software Geopsy (vers.2.5.0 available at http://www.geopsy.org/download.php last access January 2015).

At a field station, the Tromino was pressed into a flat area of the ground (usually inside the the sandy/gravel level of the superficial soil) such that the three 5-cm-long spikes mounted on the underside of the unit penetrate the soil and form a solid anchor. A bubble level on top of the unit provides for leveling, but, priority is given to a solid ground connection over exact leveling.

Figure 11. Model of Tromino used for passive tromography at the TWL

[11] Ibs-von Seht M., and Wohlenberg,J. (1999). *Microtremors measurements used to map thickness of soft soil sediments*. Bulletin of the Seismological Society of America, v. 89, p. 250-259.

As a convention, the instrument was oriented towards geographic north in order to assess any directional preference in the ambient noise.

A total of 4 measurements were taken in the vicinity of the Stylite Tower. Location of the tests is reported in Figures 12 and 13.

Figure 12. Position of passive tromography tests (UAR01-UAR04) in the Stylite Tower area

Seismic noise registrations undertaken in the vicinity of the tower have been analyzed and further processed using the open source Geopsy[12] software (ver. 2.5.0).

Each test was performed using a sampling frequency of 128Hz and an acquisition time of 20 minutes, following the SESAME[13] guidelines to process data and for the implementation of the H/V spectral ratio technique, providing, for each motion component, a valid and reliable Fourier spectrum (not affected by *aliasing*) in the range of frequencies between 0.5-20 Hz. The quality of records acquisition has been checked directly in the field and validated for further data processing.

[12] *https://download.geopsy.org/index.php? Download* , last access December 2014
[13] SESAME (2004). *Guidelines for the implementation of the H/V spectral ratio technique on ambient vibrations measurements, processing and interpretation.* SESAME European research project, WP12, Deliverable D23.12. EC-Research General Directorate, Project No. EVG1-CT-2000-00026 SESAME

The sampling rate and recording interval depends on the expected range of frequencies in the HVSR spectra, which in turn depends on the expected range of thickness and shear wave velocity of the sediment.

Procedures of data analysis are based on the following operations:

1. Acquired signal is divided into windows of length L (in this work usually 30s);
2. Each window is detrended;
3. Tapered with a cosine window (2%) and padded with zeros;
4. The Fast Fourier Transform (FFT) is then computed for each window as well as the amplitude spectrum;
5. The spectra of each window are smoothed according to a function *f*, and the horizontal to vertical spectral ratio is computed at each frequency, for each window.

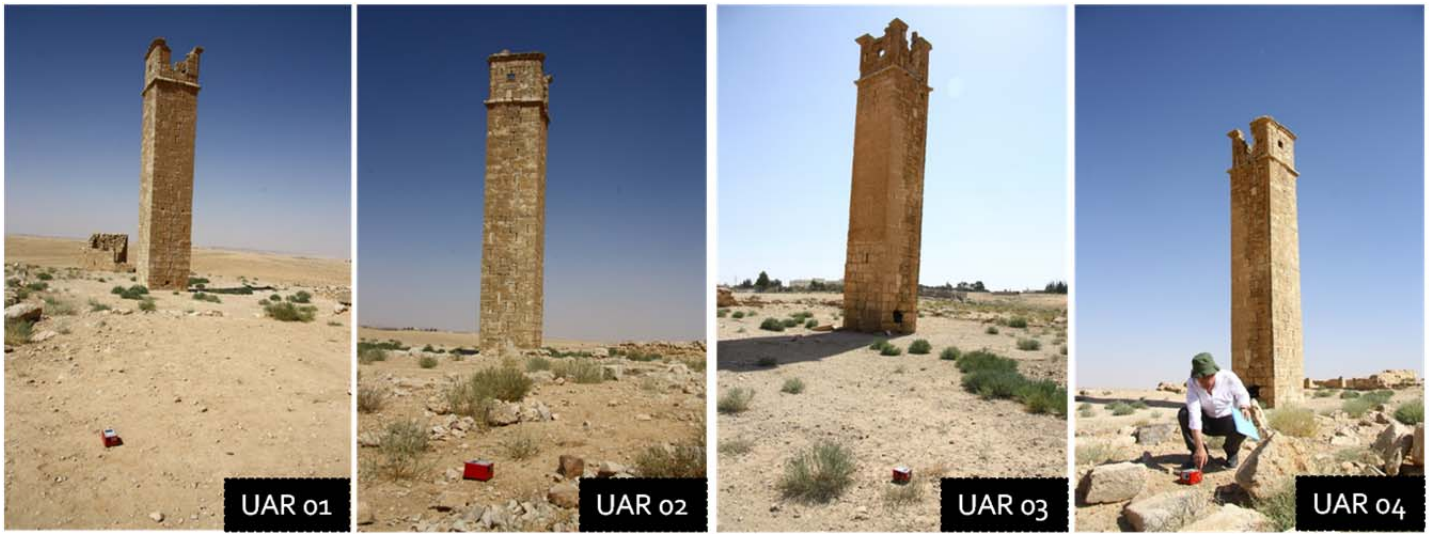


Figure 13. Passive tromography tests undertaken in the ground level of the Stylite Tower area

The final HVSR function at each frequency is given by the average of the HVSR of each window.

From application of the spectral ratio technique, an average H/V curve with its corresponding peak of frequency F_0 and peak of amplitude A_0 were retrieved for each site. In this work, the HVSR have been computed by averaging the horizontal spectra with the quadratic average and dividing it by the vertical spectrum.

4.3 Results from Geophysical Analysis

All the HVSR presented in this work have been obtained by smoothing the spectra with a Konno-Ohmaci smoothing function (*f*) choosing "*b*" parameter values between 35 to 45.

N-S and E-W spectra horizontal components have been combined (geometric average) to obtain the horizontal spectrum H before calculating H/V spectral ratio for each frequency (curve H/V). For any site, besides the H/V function, spectral ratios between the single horizontal components (N: channel direction N-S; E: channel direction E-W) and the vertical component of the motion (N/V function; E/V function) have been also calculated.

Figure 14 reports the curves of E/V, N/V and H/V spectral ratios for any recording site.

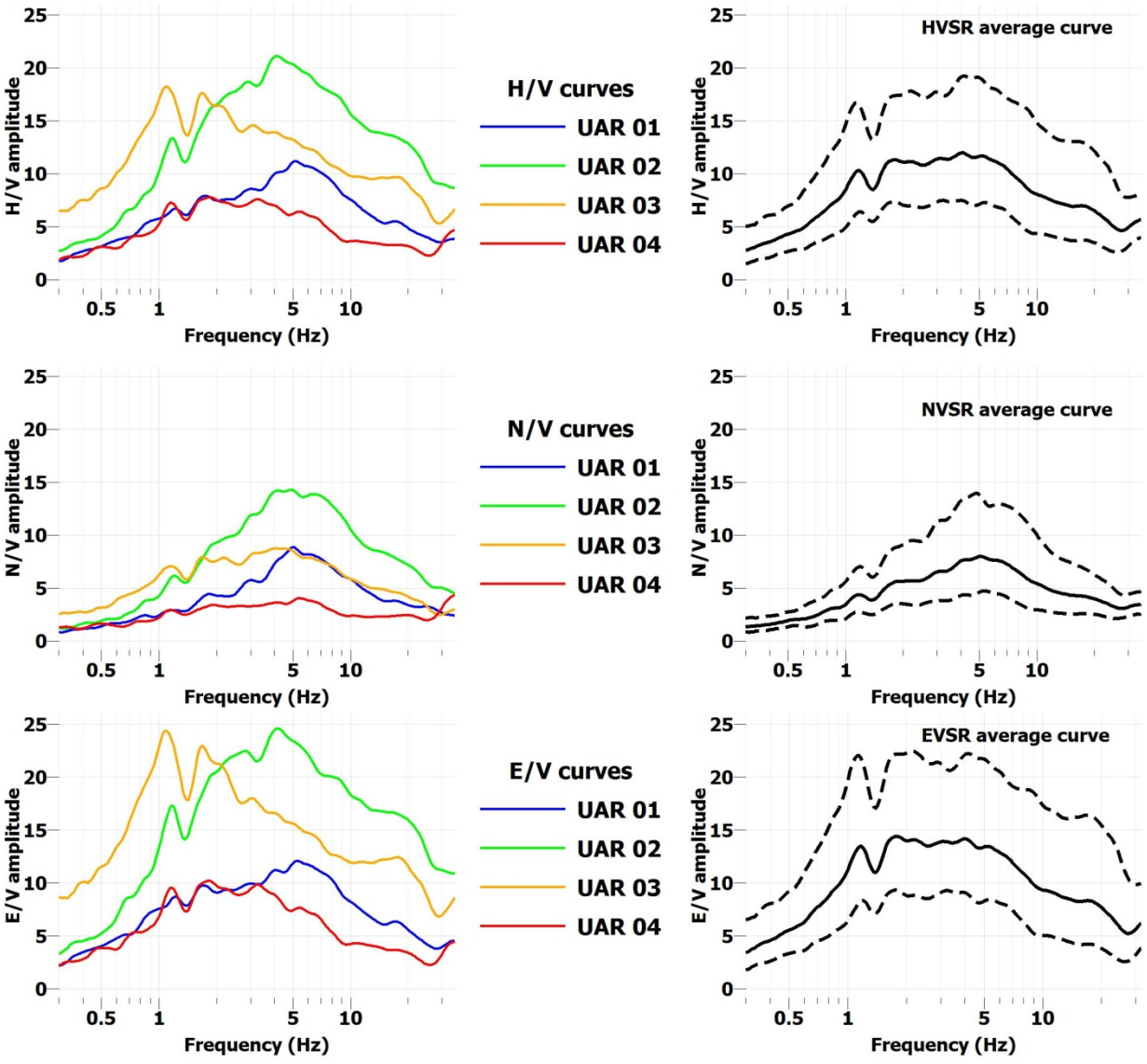


Figure 14. E/V, N/V and H/V curves for tests collected on the ground of the Stylite Tower

As a general rule, records are characterized by very small amplitude of the signal. This is coherently related with the location of the site in low densely populated area, far from usual sources of anthropogenic noise (e.g. traffic, industrial activities). This situation has promoted the

registration of spectral ratio curves very useful to the detection of peak frequency values that can be reasonably considered equal to the fundamental frequency resonance of the terrains. Such frequencies are characterized at the same time by very high values of amplitude that are not correspondent to the actual conditions of the ground motion amplification.

Data elaboration has evidenced the presence of two distinct ranges of frequency where potential amplification effects of ground motion can occur: the first range is comprised between 1.0 and 2.5 Hz and can be quite clearly detected in all observed sites and in all curves except N/V for the sites UAR01 and UAR04; the second range is comprised between 4.0 and 6.0 Hz with lesser amplitudes on N/V curves. Diagrams of H/V peaks directionality are shown in Figure 15: all peaks, except those recorded in the site UAR01, show a pronounced directionality in the angular range between 50° and 120°.

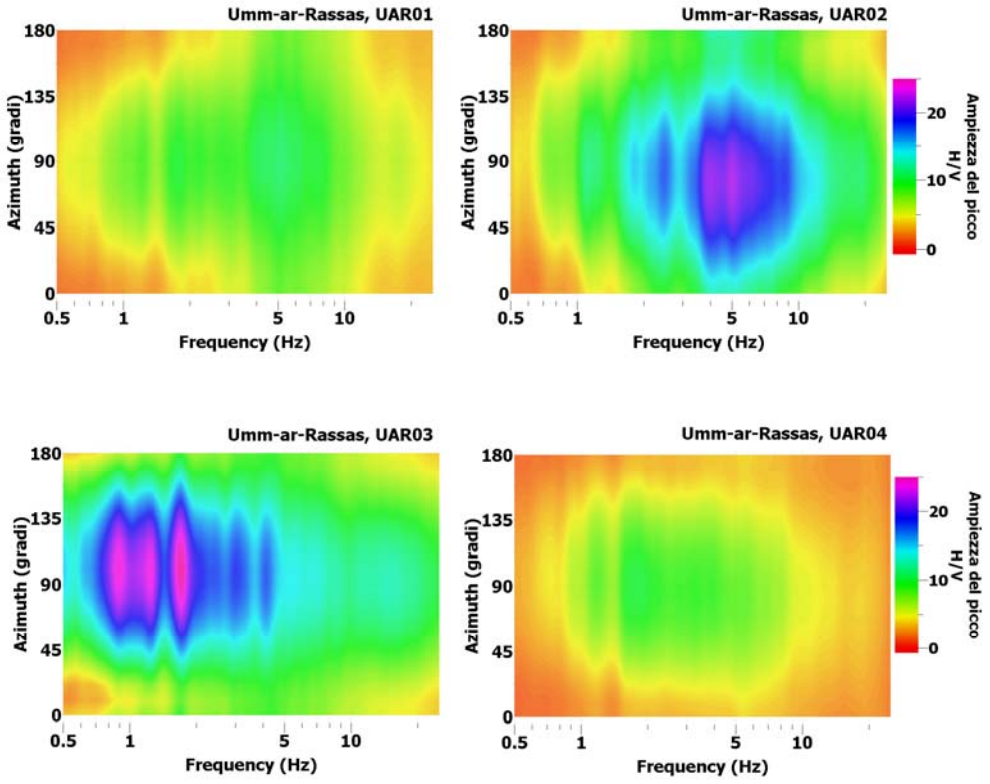


Figure 15. H/V diagrams vs. direction for tests collected on the ground of the Stylite Tower

Detailed information on passive tromography tests is reported in Annex II.

5. STRUCTURAL MODELING

5.1 Introduction

This section is focused on the safety conditions of the Stylite Tower considering the implementation of a structural modeling under static and dynamic conditions.

The Tower, even if with some damage, is still in intact condition and represents the unique surviving example of this kind of monuments, given that a similar one in Syria has been severely damaged by an earthquake.

5.2 Static Analysis

A static analysis of the Stylite Tower has been initially implemented in order to have a first evaluation of the effect of the debris/stone filling, that has been recently removed from the structure.

According to some experimental results, the weight per unit volume of the stone has been assumed equal to 23 KN/m^3; the elastic modulus has been assumed equal to 1.5x10^{10} N/m^2.

The tower was filled in the past by means of irregular stones. Therefore, a static analysis is firstly performed in order to evaluate the effect of the filling. In particular, the pressure given by the filling on the tower walls is evaluated by analogy making use of the Eurocode 1, Section 4 - Actions in silos and tanks. The friction on the wall gives a horizontal (ph) and a vertical (pw) components of pressure:

$$p_h(z) = \frac{\gamma A}{\mu U} C_z(z) \qquad p_w(z) = \frac{\gamma A}{U} C_z(z)$$

where γ is the filling weight per unit volume, μ the friction coefficient, z the depth, A and U the area and the perimeter of the tower cross-section, respectively, and

$$C_z(z) = 1 - e^{-\frac{z}{z_0}}$$

where $z_0 = A/(K_s \mu U)$ and K_s is the ratio between horizontal and vertical pressure.

The following parameters have been assumed in this preliminary analysis:

$$\gamma = 16\,KN/m^3, \qquad K_{s,m} = 0.45, \qquad \mu_m = 0.6$$

where the subscript “m” indicates the mean value. According to Eurocode, the design values of the horizontal pressure are derived as follows:

$$K_s = 1.15 K_{s,m} \qquad \mu = 0.9 \mu_m$$

whereas for the vertical pressure:

$$K_s = 1.15 K_{s,m} \qquad \mu = 1.15 \mu_m$$

The pressure profiles reported in Figure 16 are derived.

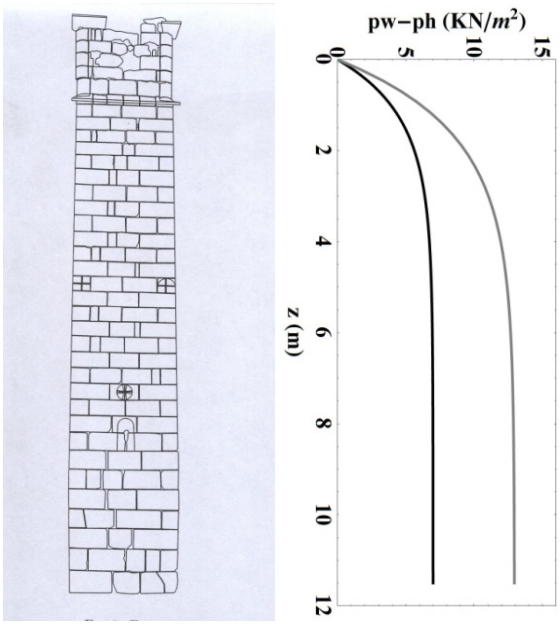


Figure 16. Pressure of the filling along the tower height: vertical component (black) and horizontal component (gray)

In this preliminary analysis, in order to analyze the tower behavior, a finite element model in SAP2000 has been considered (Figure 17), using shell elements. A more refined models, based on nonlinear material behavior, will be defined later.

Figure 18 shows the normal force along the height of the tower whereas the normal forces per unitary length in horizontal (F11) and vertical (F22) directions are represented in Figures 19. The value at the base is about 2.7 MN, with a stress in the walls of 1.45 N/mm^2 (without fill the stress reduces to 0.56 N/mm^2). The stress on the foundation, supposed to be with the same area of the tower base, is equal to 0.42 N/mm^2. These values are anyway bearable, taking into account that the experimental tests show strength values between 5.3 and 8.5 N/ mm^2.

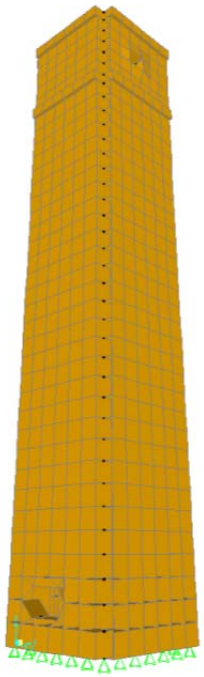


Figure 17. Finite element model of the tower

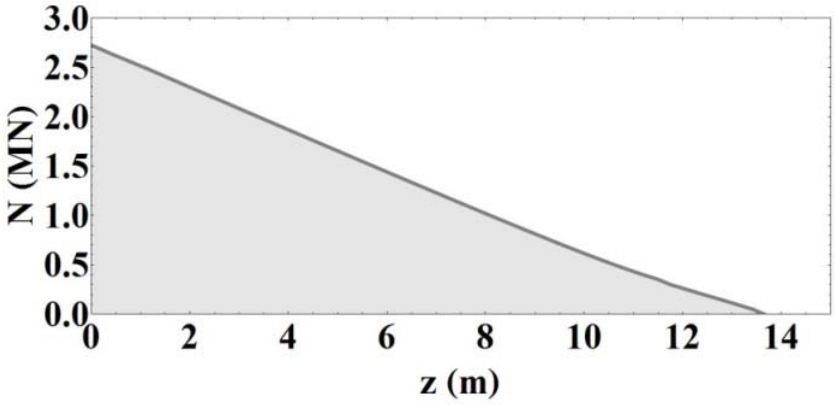


Figure 18. Normal force vs. tower height

Even if the normal stresses at the base are low, the pressure on the walls given by the filling leads to bending stresses at the corners and at the middle points of the walls, as depicted in Figure 19. As has been said previously, in this first model, shell elements capable to bear bending, shear and normal stresses have been used. However, the actual structure, made of discontinuous blocks, bases its resistance on friction between blocks. Therefore, the bending stresses can cause relative displacements between blocks, with crack openings along the discontinuities.

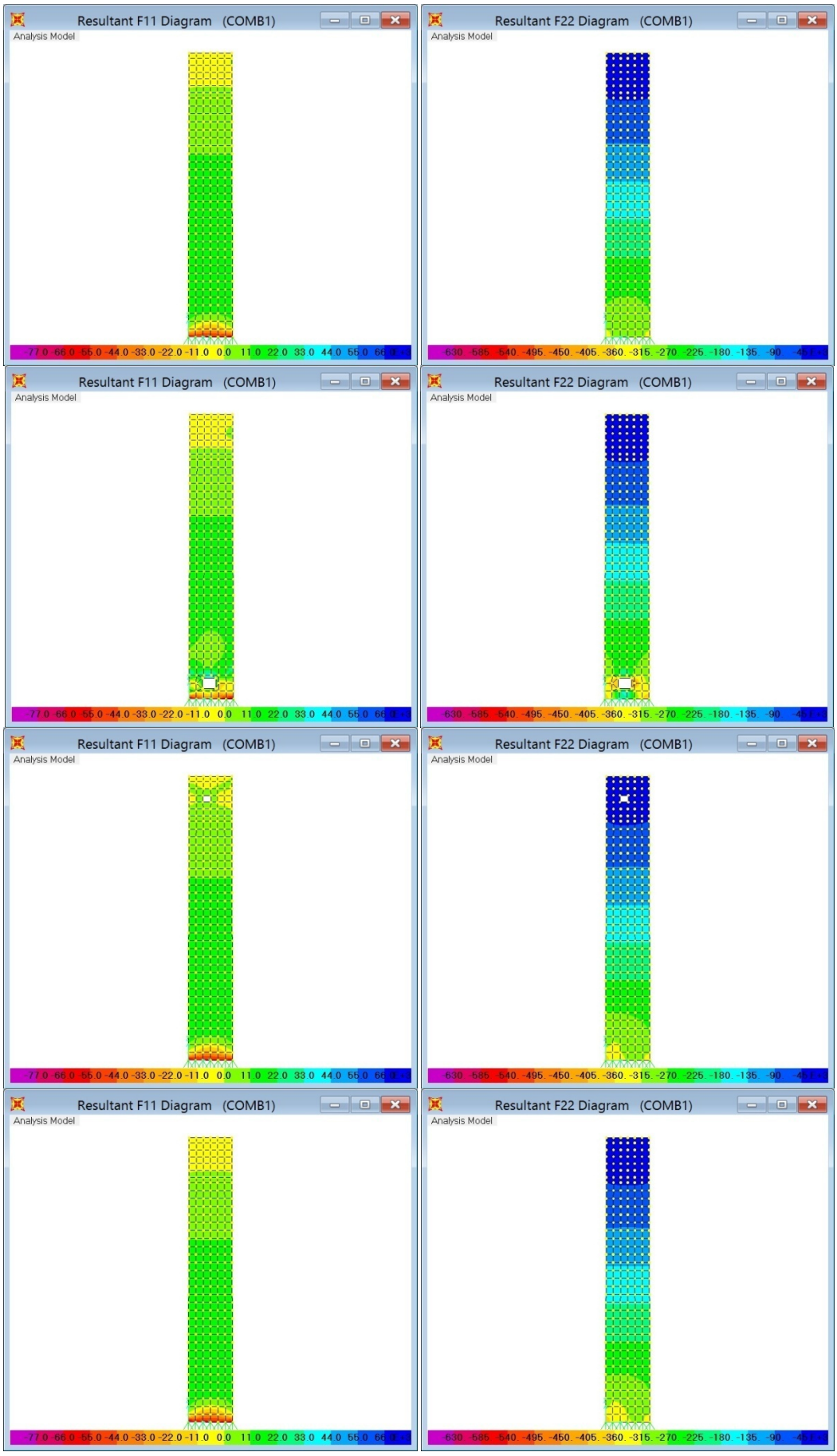


Figure 19. Horizontal (left side) and vertical (right side) element normal force vs. unitary length

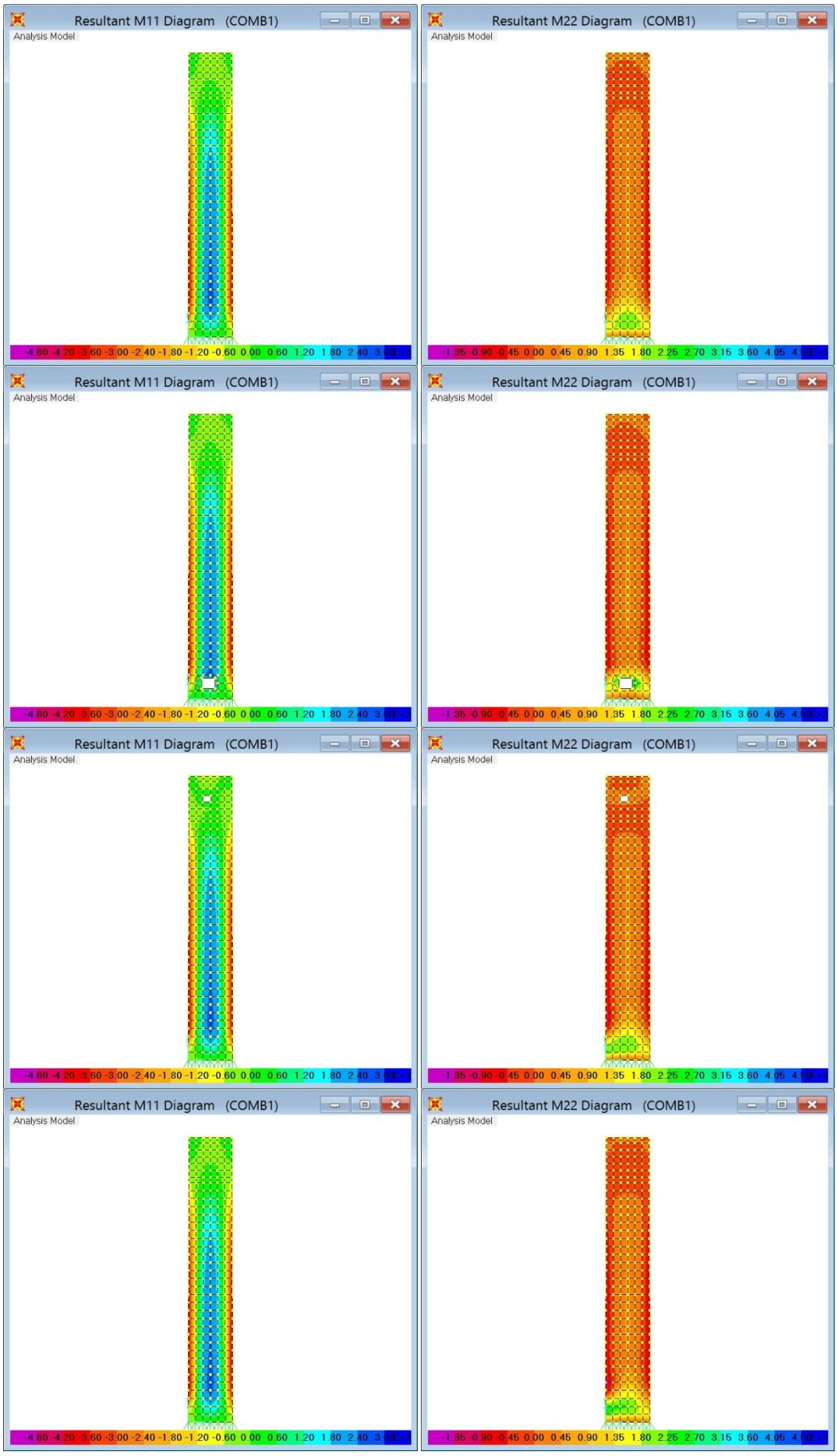


Figure 20. Bending stresses on the walls along horizontal (left side) and vertical (right side) elements

5.3 Modal and Seismic Analysis

According to available sources, the Stylite Tower has been heavily damaged by a strong earthquake occurred in 759AD, that caused the fall of the upper part of the structure (e.g. dome, vaulted roof).

The so-called 'Jericho earthquake' occurred on 11 July 1927 (M_L=6.25) was the strongest and most recent destructive event that affected the area of Umm ar-Rasas[14] (Figure 21) that heavily damaged the cities of Salt and Amman in present Jordan and might have caused further damage to the Stylite Tower due to its vicinity to the epicentral area[15].

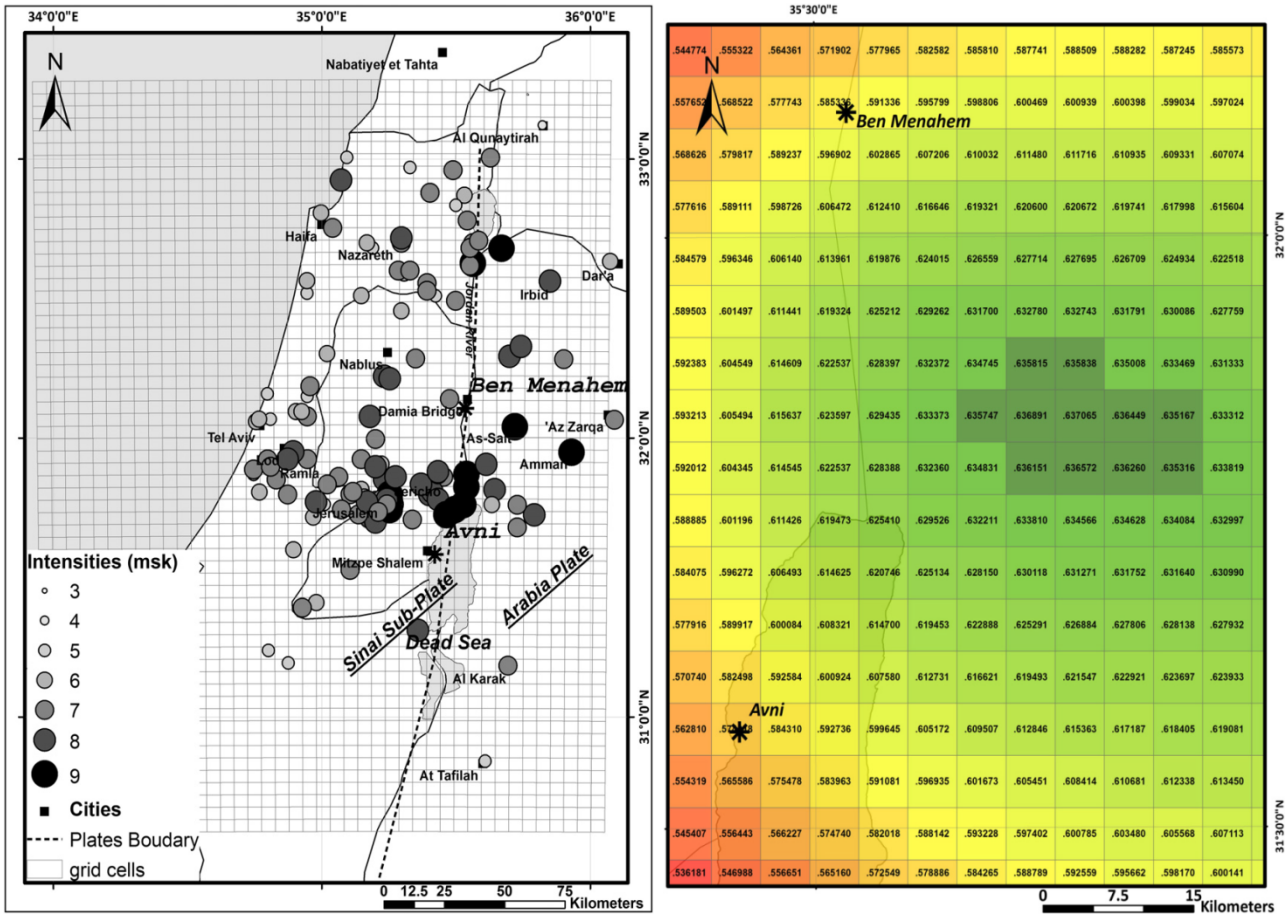


Figure 21.Map of spatial distribution of intensities (left) due to historical earthquakes(Avni, 1999) and new epicentral area re-calculated for the 1927 Jericho earthquake (Zohar and Marco, 2012)

[14] Avni R. (1999). *The 1927 Jericho Earthquake. Comprehensive macroseismic analysis based on contemporary sources.* Ben Gurion University of the Negev, Beer Sheva.

[15] Zohar M. and Marco S. (2012). *Re-estimating the epicenter of the 1927 Jericho earthquake using spatial distribution of intensity data.* Journal of Applied Geophysics, 82, 19-29.

The site of Umm er-Rasas falls in the 2B zone of the Jordan seismic map (orange in Figure 22).

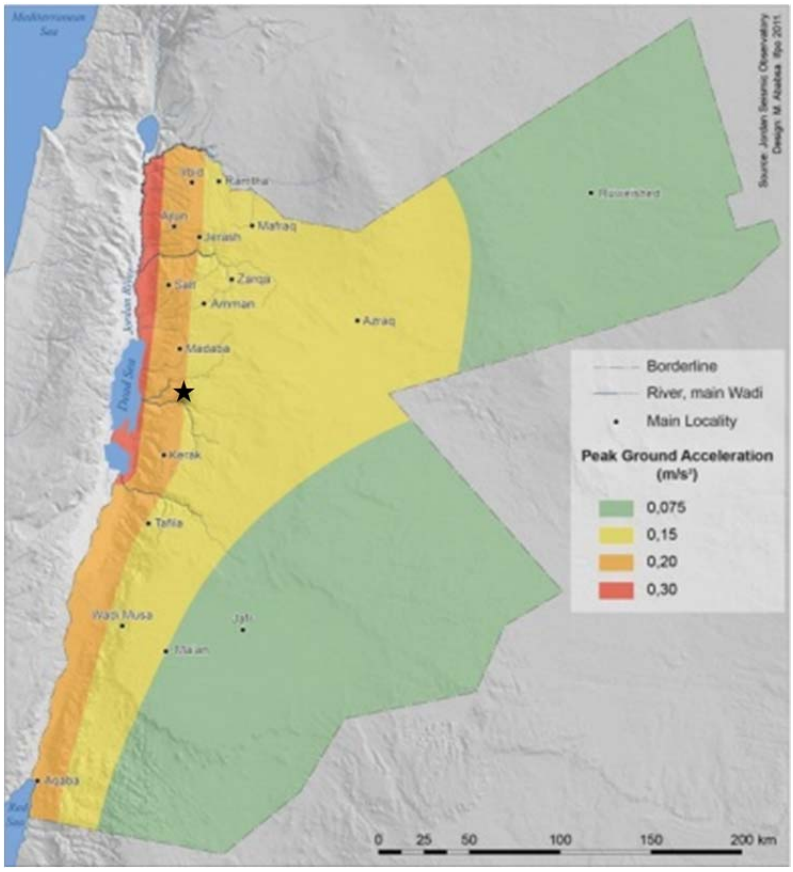


Figure 22. Jordan seismic zones (the star indicates the site of Umm er-Rasas

The earthquake responding coefficient for acceleration is C_a=0.2, equal to the coefficient for velocity C_v=0.2. The Jordan seismic code is defined according to the UBC97. Therefore, the elastic response spectrum is defined as in the following Figure 23.

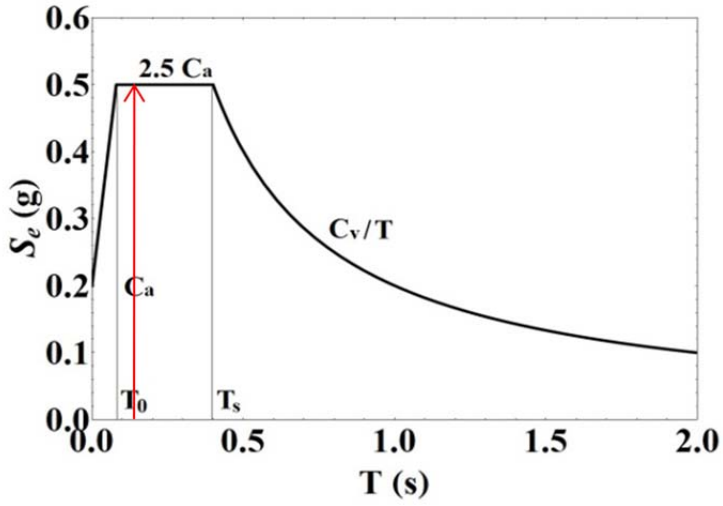


Figure 23. Elastic response spectrum: the red line shows the period equal to 0.16s (2 out of 5 possible seismic modes occurring at Umm er-Rasas

where $T_0 = 0.2T_s$ and $T_s = C_v / (2.5C_a)$.

In the case of the seismic analysis, the tower was considered as empty. The modal analysis results are the frequencies and modal shapes of the structure. Figure 24 shows the first five modal shapes: the first two and the last two are related with the translational motion of the tower in the two orthogonal directions, whereas the third mode is torsional.

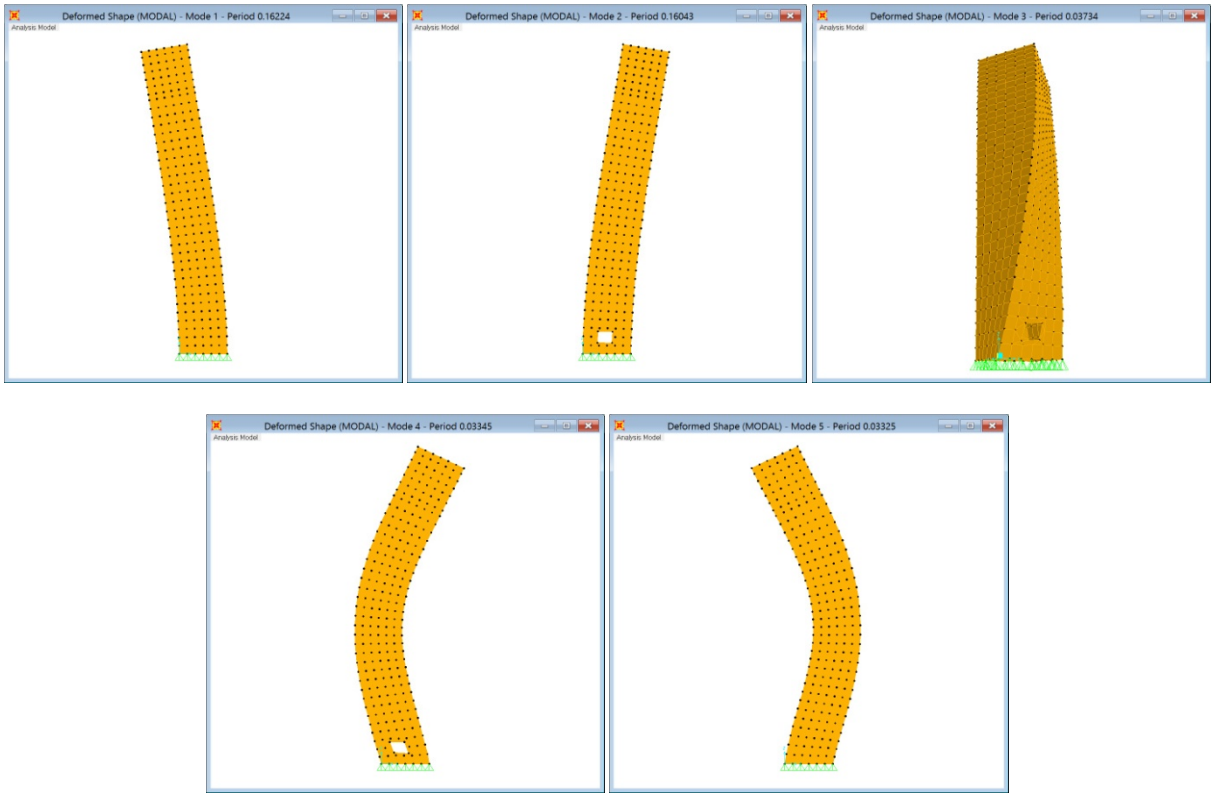


Figure 24.Modal shapes of the Stylite Tower

StepType	Period	Frequency	UX	UY	UZ	SumUX	SumUY	SumUZ
Text	Sec	Hz	Unitless	Unitless	Unitless	Unitless	Unitless	Unitless
Mode	0.162235	6.16	0.007916	0.62	0.000005652	0.007916	0.62	0.000005652
Mode	0.160433	6.23	0.62	0.007873	2.51E-08	0.63	0.63	0.000005677
Mode	0.037341	26.78	0.001302	0.000001959	0.000000016	0.63	0.63	0.000005693
Mode	0.033447	29.90	0.22	0.0001385	4.641E-07	0.85	0.63	0.000006157
Mode	0.033255	30.07	0.0001595	0.22	0.00008291	0.85	0.85	0.00008906

Table 3. Modal participating mass ratios

Table 3 shows periods and participating mass ratios for the first five modes.

With the chosen elastic modulus, the frequencies of the first modes fall in the zone with constant values in the spectrum, leading to large seismic forces on the structure.

The response spectrum analysis with seismic load along x direction allows to evaluate normal forces at the base, represented in Figure 25 for seismic load and dead loads. The estimated base moment is equal 3.3 MNm whereas the total vertical force is equal to 1.06 MN. This means an eccentricity value of about 3.1 m. Figure 26 shows the vertical forces per unitary length.

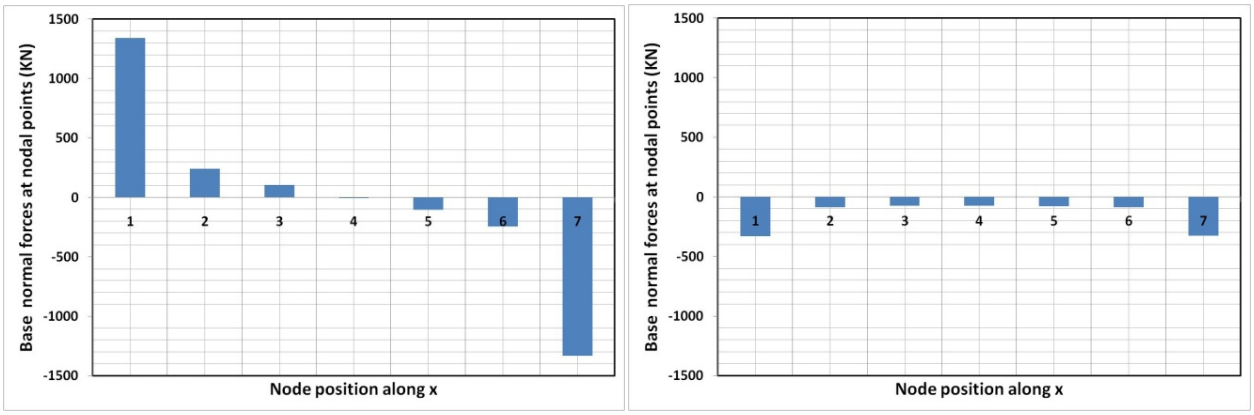


Figure 25. Normal forces at nodal positions along x (summed along y): Seismic loads (left diagram) b) Dead load (right diagram)

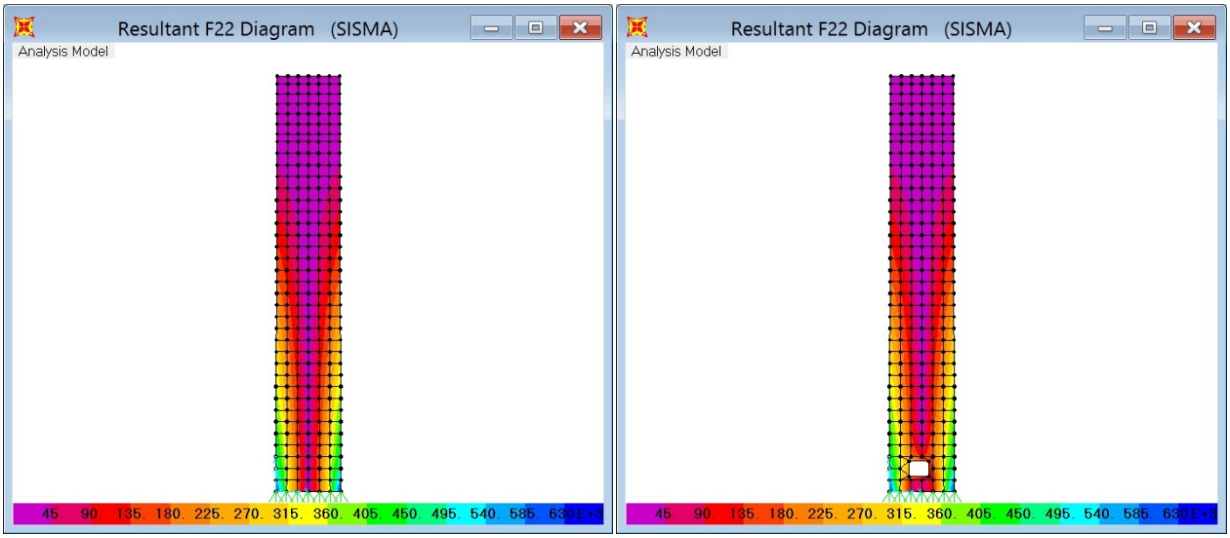


Figure 26. Normal vertical forces per unitary length, under seismic loads

6. RESULTS FROM INVESTIGATIONS AND PROPOSED TYPOLOGIES OF INTERVENTION

6.1 Summary of Results

According to results derived by geotechnical and geophysical analyses, and structural modeling of the tower, the following conclusions and recommendations can be done vs. the main issues and aspects to which the present work has been addressed.

- ✓ **Geomechanical characteristics of the stone blocks of the tower**

Investigation with Schmidt-hammer test on basal rows of the blocks provides evidence of a differentiated behavior of the material according to weathering and mechanical degradation of the stone. Although majority of blocks present a medium-high values of UCS, low values of UCS have been measured in a couple of fractured blocks sited in the NE corner of the structure. In this part of the tower the lowest values of UCS (<60 MPa) have been generally assessed, probably for effect of maximum stress due to the leaning of the tower in such a direction.

It is strongly advised to provide mechanical consolidation or substitution of fractured blocks and stones that exhibit the lower values of UCS in the basal rows (e.g. 1N, 2N, 3N, 3E, 6E, 7E, 2W).

Extension of mechanical analysis (or at least visual analysis) for detection of damaged/degraded limestone blocks to consolidate or substitute is also recommended.

- ✓ **Geophysical analysis of the Stylite Tower area**

Passive tromography executed on the site has stressed the presence of two main ranges of frequencies for which soil amplification can occur: 1-2.5 Hz and 4-6 Hz. The second range of frequencies can be reasonably associated to the presence of the tower and might result in a negative dynamic behavior in case of strong motion due to a seismic event.

- ✓ **Static-dynamic analysis of the Stylite Tower**

The values of normal forces acting on the tower resulted from static analysis are actually bearable, taking into account also experimental tests that show values of 53-85 MPa (laboratory test) and 48-140 MPa (in-situ Schmidt-hammer test).

The results evidence also presence of bending moments with maximum values at about 1/3 of the height. These moments can lead to generation and/or enlargement of discontinuities, given that the out-of-plane stability of the stone wall depends essentially by friction between the blocks.

Dynamic modeling of the tower provided an estimated base moment equal to 3.3 MNm whereas the total vertical force is equal to to 1.06 MN. This means an eccentricity value of about 3.1 m. Such a large eccentricity is unacceptable for a material without tensile strength. Therefore, the analysis, even if not conclusive, evidences the high vulnerability of the tower under a credible earthquake.

6.2 Proposed interventions

As a general guideline, conservation strategies of Cultural Heritage have to be addressed to respect and preservation of their historical and architectural values. At the same time, restoration works, and above all structural works, have to safeguard effectiveness in the long-term of implemented actions with regard to events and factors that has caused degradation, damage or disruption of the monuments.

Some conservation strategies have already proposed in past studies that consider the following works:

a) Restoration of limestone blocks and consolidation of voids with mortar;
b) Insertion of carbon fibers or stainless steel bars;
c) Anastylosis.

Such typologies of works, namely those addressed to preservation of the architectural aspect of the Tower, as long as replace the original characteristics of the monument, on the other hand bring back the structure to the same unsafe conditions in case of earthquakes.

Under the above assumptions, different interventions for the structural reinforcement of the Stylite Tower have been proposed that take into account:

- Results from studies and analysis carried out in the last decade;
- Results derived by the present research as completion of data needed for proper design of consolidation works;
- Analysis of causes-effects that have promoted damage of the tower;

- Selection of reinforcement works capable to provide long-term conservation of the monument vs. occurrence of future seismic events that are deemed to be the main cause of actual damage conditions;
- Minimization of impacts by proposed works;
- Integration of present structural proposals with some restoration/conservation strategies suggested in previous studies;
- Sustainability of works by local expertise;
- Knowledge transfer of specific innovative techniques to local expertise.

Typology 1

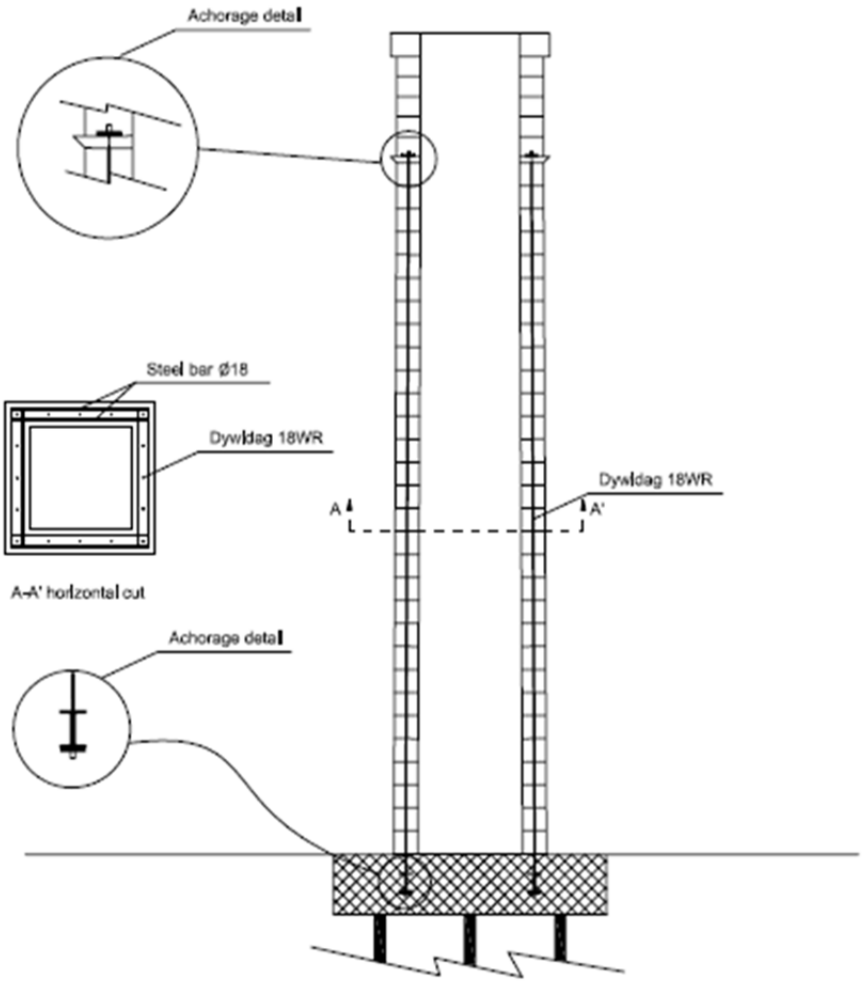


Figure 27. Proposal of reinforcement works – Typology 1

Structural reinforcement of the tower is provided by steel wire cables and bars, concrete foundation, micropiles (Figure 27).

The main operations are the following:

- Dismounting of the tower;
- Excavation of the foundation (also for archaeological survey) until bedrock;
- Execution of micropiles;
- Construction of reinforced concrete foundation of around 4.5mx4.5m
- Drilling of stone blocks for insertion of wire cables;
- Replacement of blocks and installation of stainless steel wire cables (Dywidag 18WR) and horizontal steel bars (ϕ18mm) located every 2-3m between the stone rows;
- Anchoring of cables on the top of the structure with the horizontal bar, whereas the base of the wires are anchored to the foundation of the tower.

Typology 2

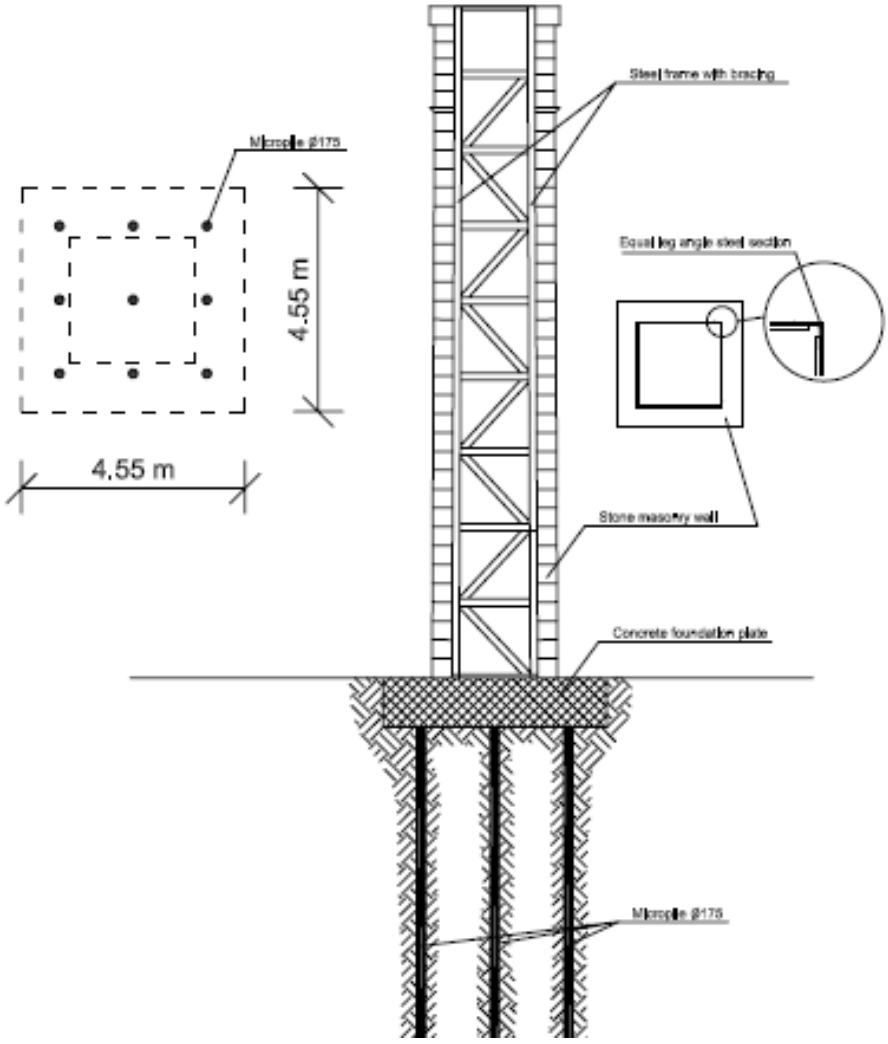


Figure 28. Proposal of reinforcement works – Typology 2

Structural reinforcement of the tower is provided by internal braced steel structure, concrete foundation, micropiles (Figure 28) through:

- Excavation of the foundation (also for archaeological survey) until bedrock without dismantling the structure;
- Execution of micropiles from outside;
- Construction of reinforced concrete foundation of around 4.5mx4.5m;
- Installation of stainless steel braced structure connected with horizontal steel bars located every 2-3m between the stone rows;
- Restoration/replacement of damaged stone block (*cuci-scuci* technique).

Typology 3

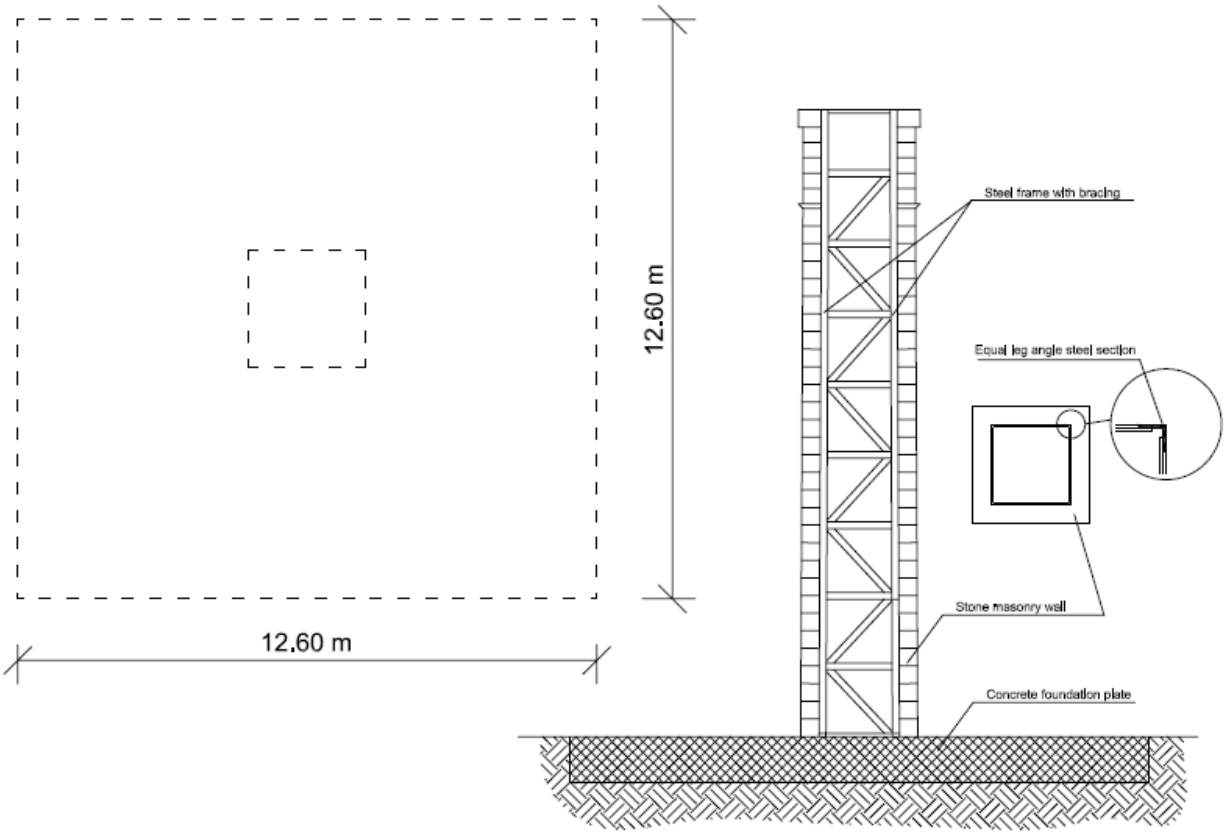


Figure 29. Proposal of reinforcement works – Typology 3

This intervention is very similar the previous (Typology 2), where a larger foundation substitutes micropiles (Figure 29). The work considers:

- Excavation of the foundation (also for archaeological survey) until bedrock without dismantling the structure;
- Construction of reinforced concrete foundation of around 12.6mx12.6m;
- Installation of stainless steel braced structure connected with horizontal steel bars located every 2-3m between the stone rows;
- Restoration/replacement of damaged stone block (*cuci-scuci* technique).

Typology 4

This typology is very similar to the first proposed intervention (Typology 1) where internal cables are substituted with special devices (SMAD – Shape Memory Alloy Devices) that provide higher resistance to the tower increasing tensional strength of materials and decreasing acceleration motion to the structure due to seismic events.

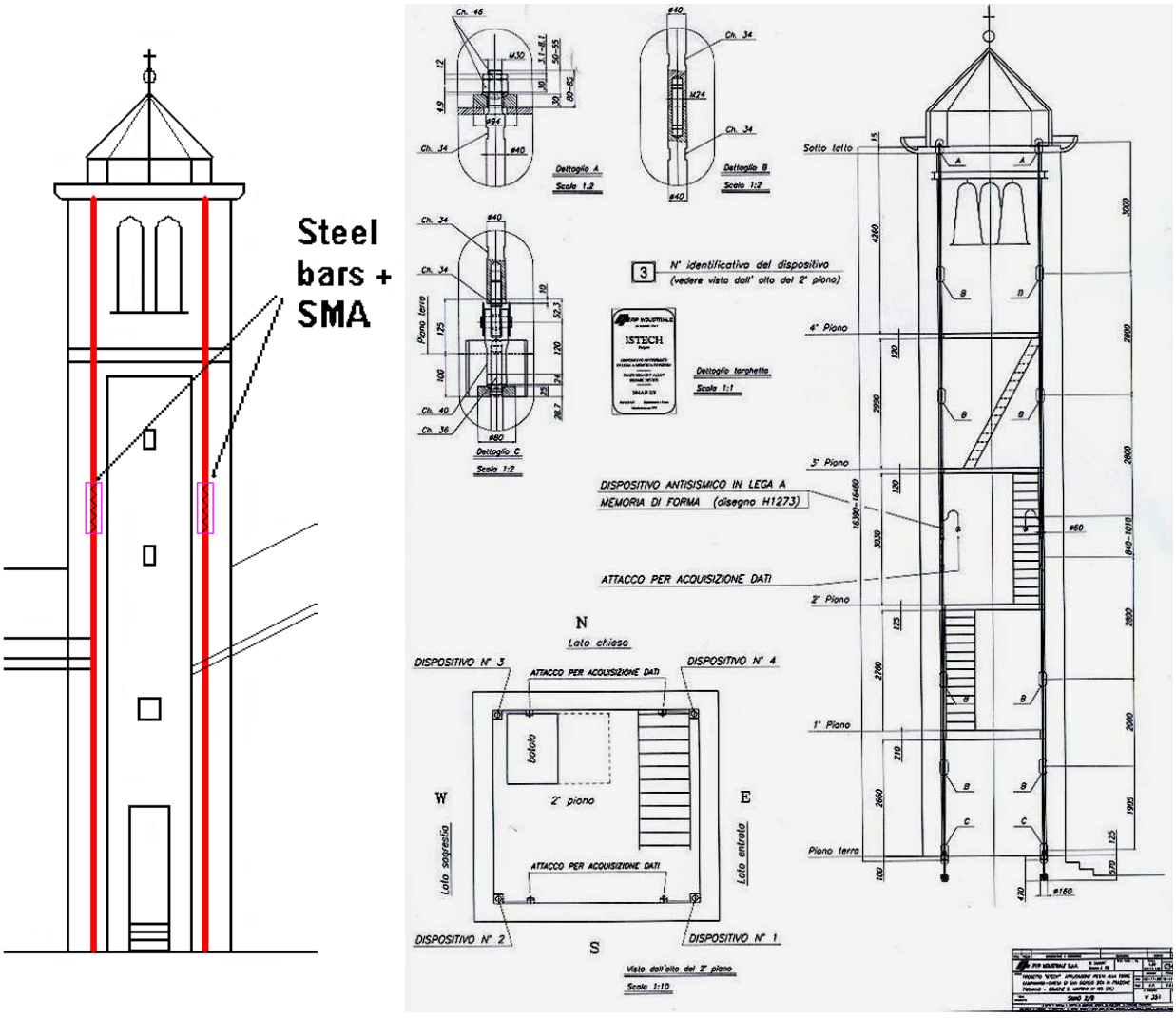


Figure 30. Proposal of reinforcement works – Typology 4, example of the Church of S: Giorgio a Trignano (Italy)

Application of SMADs has been implemented in some case studies in Italy as for the medieval Bell Tower of San Giorgio a Trignano (Emilia Romagna Region, Italy)[16] and such methodology can be successfully applied to the Stylite Tower of Umm er-Rasas (Figure 30 and 31).

The main operations are the following:

- Excavation of the foundation (also for archaeological survey) until bedrock without dismantling the structure;
- Execution of micropiles from outside;
- Construction of reinforced concrete foundation of around 4.5mx4.5m;
- Installation of stainless steel bars connected with SMADs anchored to the top of the tower and foundation
- Restoration/replacement of damaged stone block (*cuci-scuci* technique).

Figure 31. Application of Shape Memory Alloy (upper left) and anchoring of the bars to the top and foundation (bottom left) of the Church of S: Giorgio a Trignano (Italy).

[16] Clemente P., Bongiovanni G. and Buffarini G (2002). *Experimental seismic behaviour of a cracked masonry structure.* 12th European Conference on earthquake Engineering, 9-13 Sept. 2002, London, UK. Elsevier, Paper Ref. 104.

6.3 Feasibility analysis of proposed interventions

The following Table 4 reports a general analysis of the main elements in terms of technicality and impacts of the proposed interventions in order to support the Department of Antiquities of Jordan in the choice of the best solution to be implemented.

	Typology 1	*Typology 2*	*Typology 3*	*Typology 4*
Technical feasibility	Difficult in positioning internal wire cables	Some difficulty in construction of the steel braced bar	Some difficulty in construction of the steel braced bar	SMADs installation has to be done by skilled workers
Potential Impacts	Low	Low	High for possible presence of archaeological elements in the foundation area	Low
Technological content	Low	Low	Low	High
Capability from local expertise	High	High	High	Low for installation of SMADs High for the rest of operations
Knowledge transfer to local system	Low	Low	Low	High

Table 4. Synoptical feasibility analysis vs. typologies of proposed works

It is highly advised that any selected typology of works has to be properly redesigned according to further modification and integration that might occur during the finalization stage of the proposal.

The final design has to take into account a structural modeling in order to validate reinforcement works in compliance with Jordan building codes with regard to the expected seismicity in Umm er-Rasas.

Further archaelogical investigation is strongly suggested in the surrounding of the foundation area of the Stylite Tower before the beginning of works.

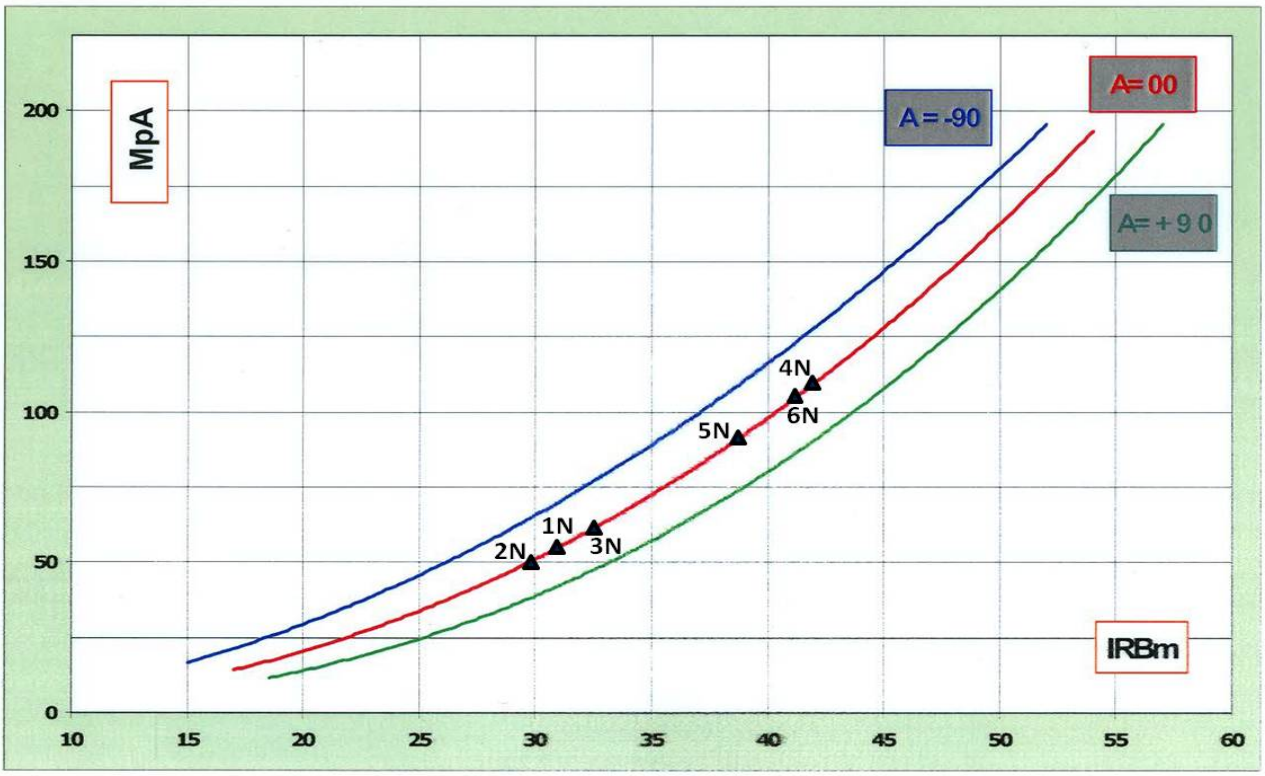
ANNEX I

Schmidt-Hammer Test

NORTH SIDE

Correlation Curve

Rebound Index (IRBM) – Uniaxial Compressive Strength (MPa – N/mm²)



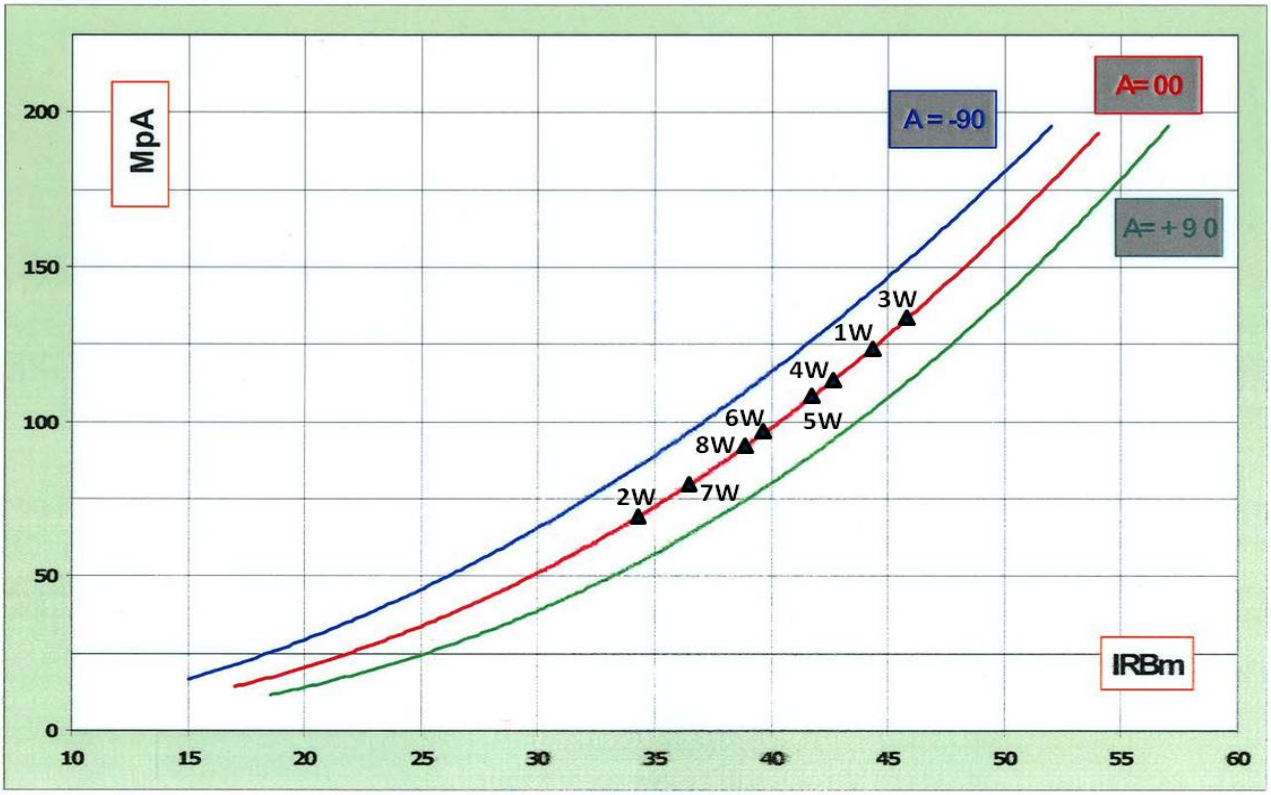
North Side

	N (1N)	N (2N)	N (3N)	N (4N)	N (5N)	N (6N)
	26	24	28	38	33	32
	27	26	30	40	33	39
	28	26	30	40	35	40
	29	27	30	41	37	40
	31	27	32	42	37	41
	31	29	32	42	40	41
	33	30	34	43	41	42
	34	33	34	44	43	45
	36	36	36	45	45	45
	38	40	38	46	45	52
Average	***31.3***	***29.8***	***32.4***	***42.1***	***38.9***	***41.7***
UCS	**54.8**	**48.9**	**59.4**	**108.9**	**90.7**	**106.5**

WEST SIDE

Correlation Curve

Rebound Index (IRBM) – Uniaxial Compressive Strength (MPa – N/mm²)



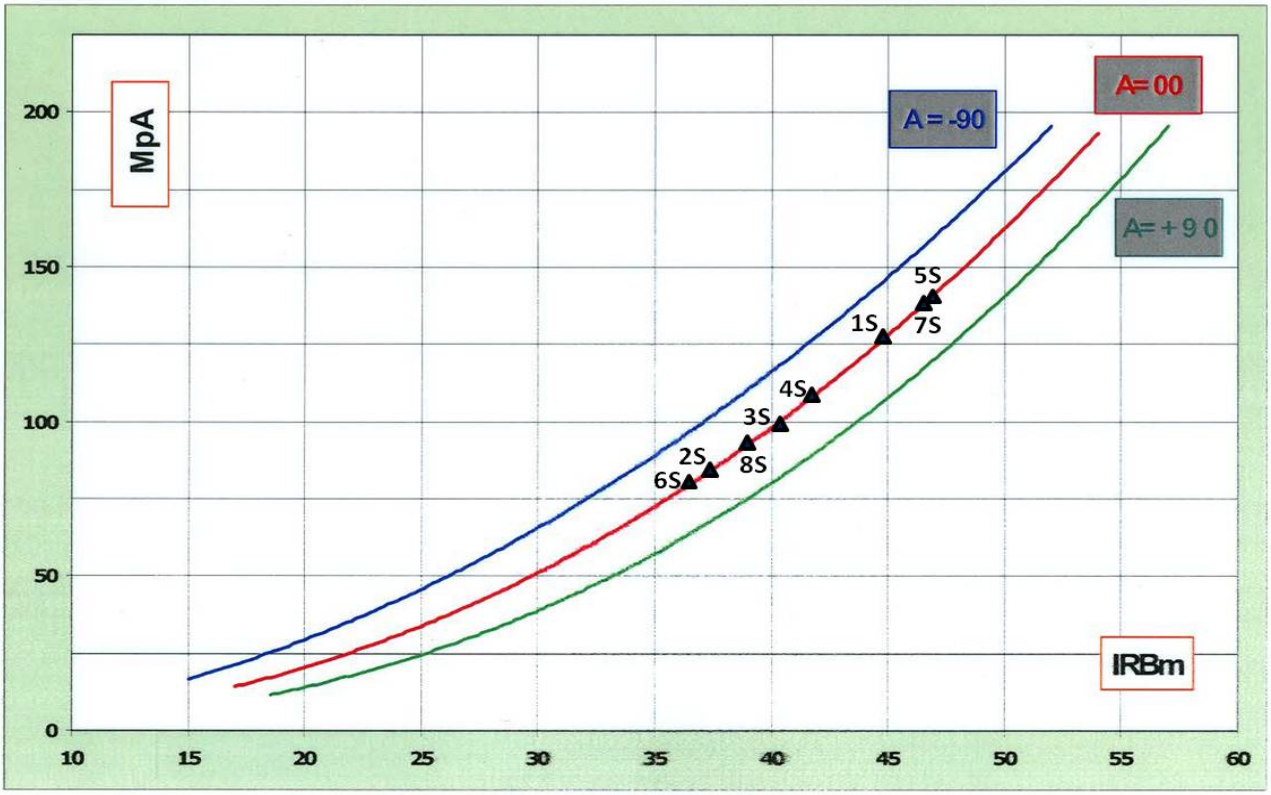
West Side

	N (1W)	N (2W)	N (3W)	N (4W)	N (5W)	N (6W)	N (7W)	N (8W)
	34	25	32	38	35	33	30	30
	38	28	38	39	37	33	30	33
	38	28	42	43	40	35	35	33
	40	30	42	43	40	38	35	36
	43	35	44	44	40	40	36	38
	44	37	45	44	40	40	39	40
	49	37	50	45	44	42	39	40
	52	40	54	45	47	44	40	44
	53	40	54	46	48	45	40	45
	54	42	58	48	50	48	43	48
Average	***44.5***	***34.2***	***45.9***	***43.5***	***42.1***	***39.8***	***36.7***	***38.7***
UCS	**123.8**	**67.3**	**133.1**	**117.5**	**108.9**	**95.6**	**79.2**	**89.6**

SOUTH SIDE

Correlation Curve

Rebound Index (IRBM) – Uniaxial Compressive Strength (MPa – N/mm²)



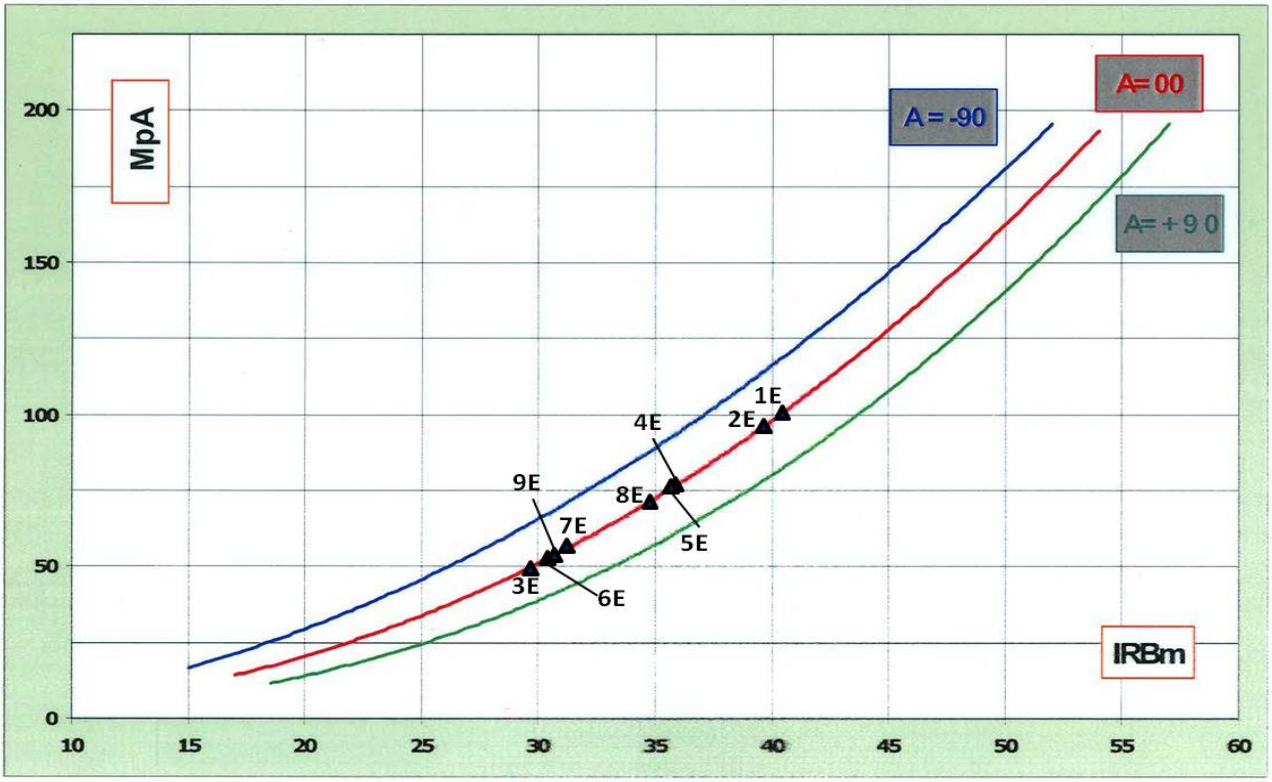
South Side

	N (1S)	N (2S)	N (3S)	N (4S)	N (5S)	N (6S)	N (7S)	N (8S)
	37	31	31	32	35	28	35	33
	39	35	32	33	37	30	36	35
	40	35	34	35	44	33	41	35
	41	36	35	36	47	35	45	38
	43	37	37	41	48	35	46	39
	45	38	40	45	49	35	48	39
	49	38	43	46	50	38	50	40
	50	39	45	47	53	43	55	40
	51	40	53	53	53	45	55	42
	54	47	54	55	53	45	55	45
Average	***44.9***	***37.6***	***40.4***	***42.3***	***46.9***	***36.7***	***46.6***	***38.6***
UCS	**126.4**	**83.8**	**99.0**	**110.1**	**139.9**	**79.3**	**137.8**	**89.1**

EAST SIDE

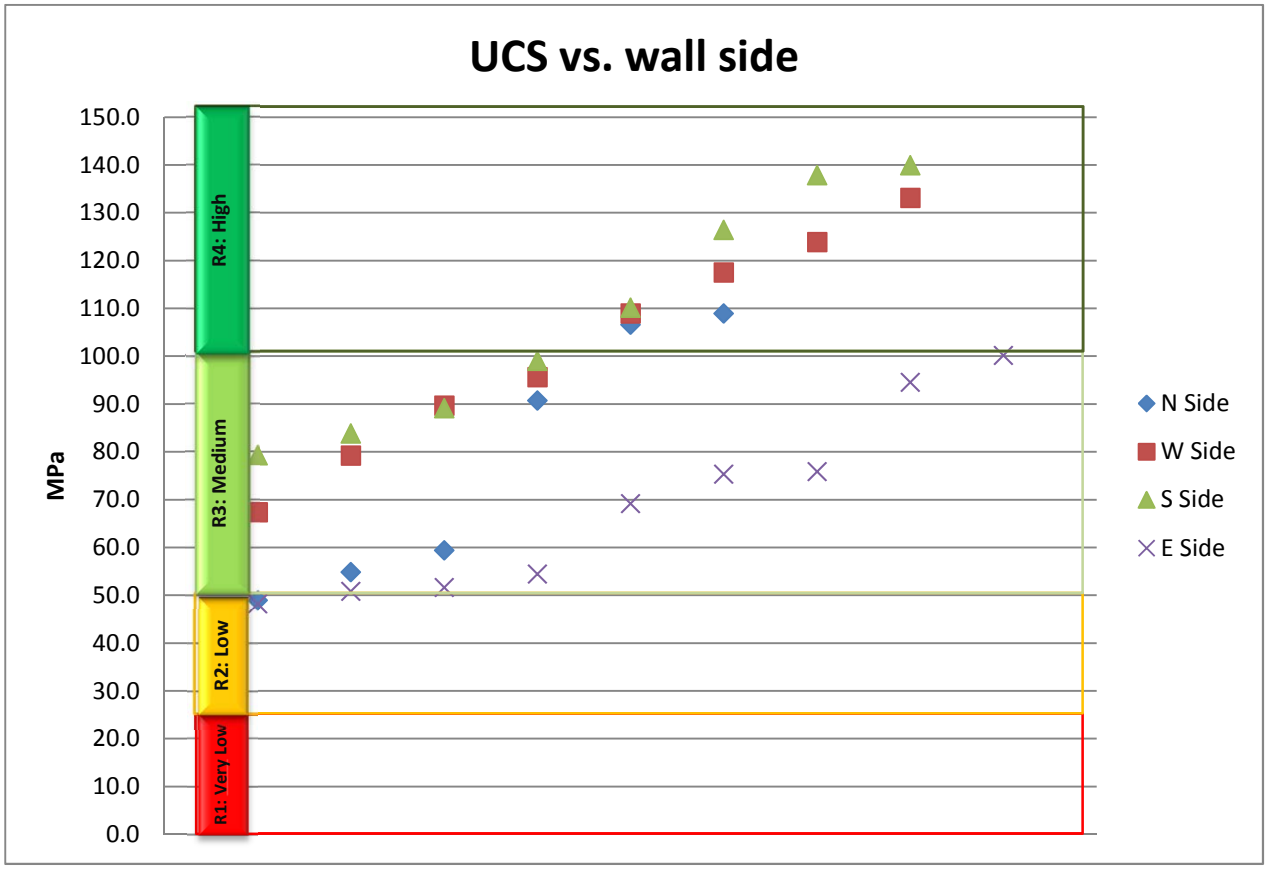
Correlation Curve

Rebound Index (IRBM) – Uniaxial Compressive Strength (MPa – N/mm²)



East Side

	N (1E)	N (2E)	N (3E)	N (4E)	N (5E)	N (6E)	N (7E)	N (8E)	N (9E)
	33	33	26	29	30	25	28	30	28
	37	33	28	33	32	27	28	33	29
	38	36	28	35	33	27	29	33	29
	38	40	29	35	34	28	31	33	30
	40	40	30	35	35	30	31	35	30
	40	40	30	35	35	30	32	35	30
	44	40	30	35	36	31	32	36	31
	45	41	31	36	38	31	33	36	32
	45	46	31	42	41	32	33	37	33
	46	47	33	45	45	42	35	38	33
Average	***40.6***	***39.6***	***29.6***	***36.0***	***35.9***	***30.3***	***31.2***	***34.6***	***30.5***
UCS	**100.1**	**94.5**	**48.2**	**75.8**	**75.3**	**50.8**	**54.4**	**69.1**	**51.6**



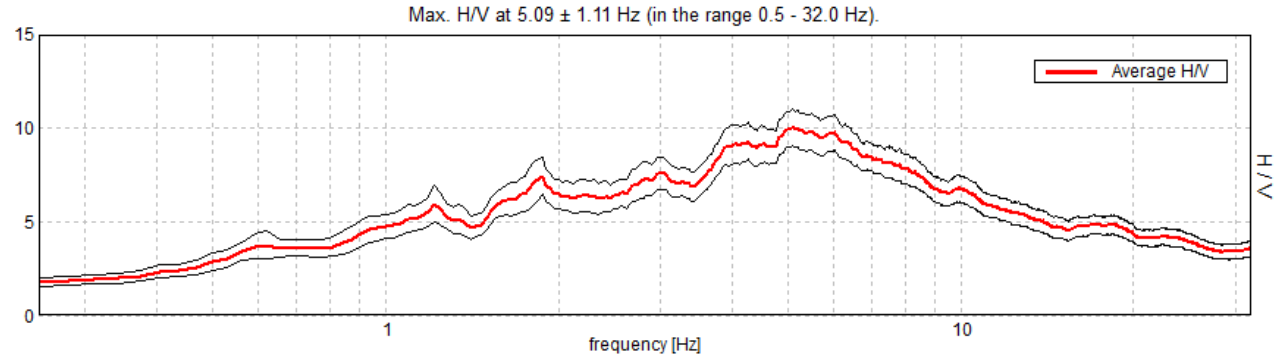
	N Side	W Side	S Side	E Side
	48.9	67.3	79.3	48.2
	54.8	79.2	83.8	50.8
	59.4	89.6	89.1	51.6
	90.7	95.6	99.0	54.4
	106.5	108.9	110.1	69.1
	108.9	117.5	126.4	75.3
		123.8	137.8	75.8
		133.1	139.9	94.5
				100.1
Average	***78.2***	***101.9***	***108.2***	***68.9***

ANNEX II Passive Tromography

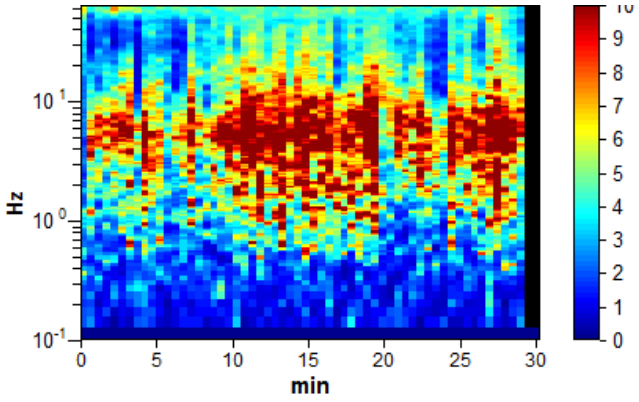
UMM-AR-RASSAS, UAR01

Instrument: TEP-0036/01-09
Data format: 16 byte
Full scale [mV]: n.a.
Start recording: 02/08/14 11:12:31 End recording: 02/08/14 11:42:41
Channel labels: NORTH SOUTH; EAST WEST ; UP DOWN ; GPS + GPS -
GPS location: 035°55.3090 E, 31°30.7154 N (741.4 m)
(UTC time synchronized to the first recording sample): 11:12:31 + 0 + 0 samples
Satellite no.: 04
Trace length: 0h30'19''. Analysis performed on the entire trace.
Sampling rate: 128 Hz
Window size: 30 s
Smoothing type: Rectangular window
Smoothing: 5%

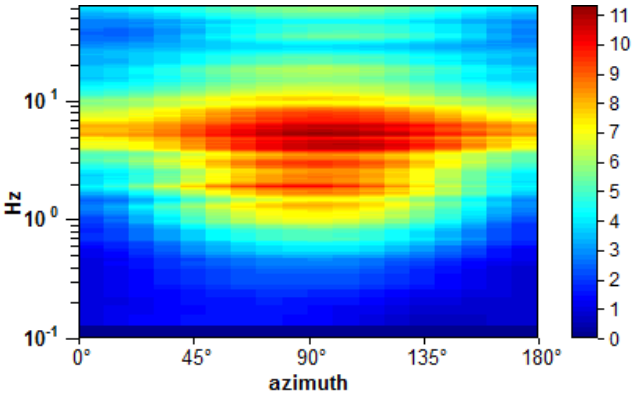
HORIZONTAL TO VERTICAL SPECTRAL RATIO



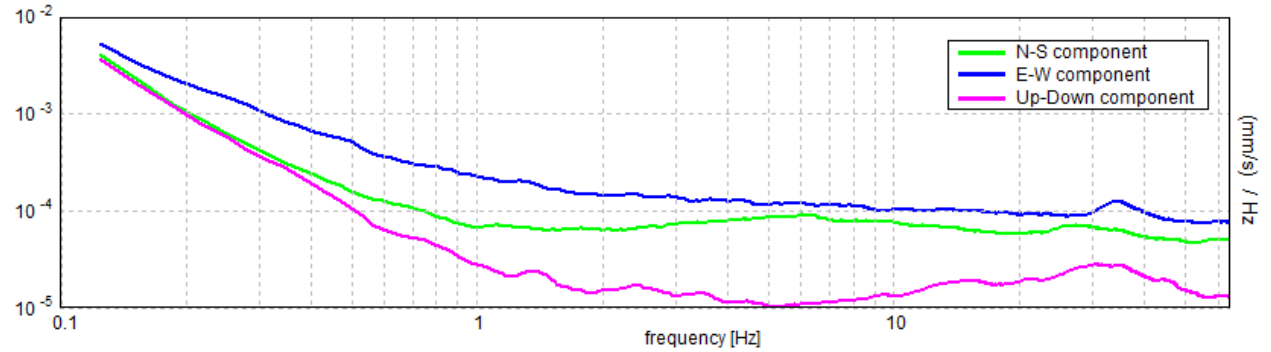
H/V TIME HISTORY



DIRECTIONAL H/V



SINGLE COMPONENT SPECTRA



[According to the SESAME, 2005 guidelines. **Please read carefully the *Grilla* manual before interpreting the following tables**.]

Max. H/V at 5.09 ± 1.11 Hz (in the range 0.5 - 32.0 Hz).

Criteria for a reliable H/V curve [All 3 should be fulfilled]			
$f_0 > 10 / L_w$	5.09 > 0.33	OK	
$n_c(f_0) > 200$	9168.8 > 200	OK	
$\sigma_A(f) < 2$ for $0.5f_0 < f < 2f_0$ if $f_0 > 0.5$Hz **$\sigma_A(f) < 3$ for $0.5f_0 < f < 2f_0$ if $f_0 < 0.5$Hz**	Exceeded 0 out of 246 times	OK	
Criteria for a clear H/V peak [At least 5 out of 6 should be fulfilled]			
Exists f^- in $[f_0/4, f_0]$ | $A_{H/V}(f^-) < A_0 / 2$	1.469 Hz	OK	
Exists f^+ in $[f_0, 4f_0]$ | $A_{H/V}(f^+) < A_0 / 2$	13.938 Hz	OK	
$A_0 > 2$	10.05 > 2	OK	
$f_{peak}[A_{H/V}(f) \pm \sigma_A(f)] = f_0 \pm 5\%$	|0.21828| < 0.05		NO
$\sigma_f < \varepsilon(f_0)$	1.11186 < 0.25469		NO
$\sigma_A(f_0) < \theta(f_0)$	0.9935 < 1.58	OK	

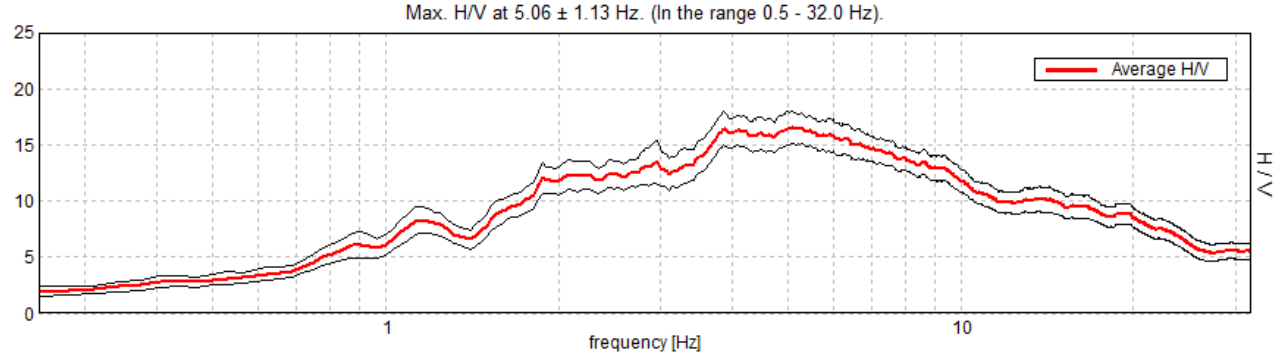
L_w	window length
n_w	number of windows used in the analysis
$n_c = L_w n_w f_0$	number of significant cycles
f	current frequency
f_0	H/V peak frequency
σ_f	standard deviation of H/V peak frequency
$\varepsilon(f_0)$	threshold value for the stability condition $\sigma_f < \varepsilon(f_0)$
A_0	H/V peak amplitude at frequency f_0
$A_{H/V}(f)$	H/V curve amplitude at frequency f
f^-	frequency between $f_0/4$ and f_0 for which $A_{H/V}(f^-) < A_0/2$
f^+	frequency between f_0 and $4f_0$ for which $A_{H/V}(f^+) < A_0/2$
$\sigma_A(f)$	standard deviation of $A_{H/V}(f)$, $\sigma_A(f)$ is the factor by which the mean $A_{H/V}(f)$ curve should be multiplied or divided
$\sigma_{logH/V}(f)$	standard deviation of log $A_{H/V}(f)$ curve
$\theta(f_0)$	threshold value for the stability condition $\sigma_A(f) < \theta(f_0)$

Threshold values for σ_f and $\sigma_A(f_0)$					
Freq. range [Hz]	< 0.2	0.2 – 0.5	0.5 – 1.0	1.0 – 2.0	> 2.0
$\varepsilon(f_0)$ [Hz]	0.25 f_0	0.2 f_0	0.15 f_0	0.10 f_0	0.05 f_0
$\theta(f_0)$ for $\sigma_A(f_0)$	3.0	2.5	2.0	1.78	1.58
log $\theta(f_0)$ for $\sigma_{logH/V}(f_0)$	0.48	0.40	0.30	0.25	0.20

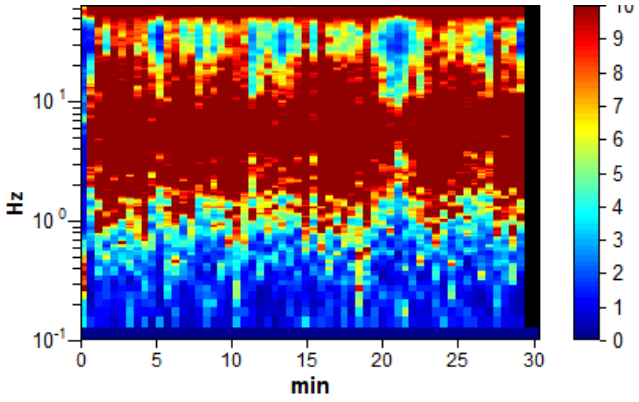
UMM-AR-RASSAS, UAR02

Instrument: TEP-0036/01-09
Data format: 16 byte
Full scale [mV]: n.a.
Start recording: 02/08/14 11:46:33 End recording: 02/08/14 12:16:51
Channel labels: NORTH SOUTH; EAST WEST ; UP DOWN ; GPS + GPS -
GPS location: 035°55.2978 E, 31°30.7276 N (730.8 m)
(UTC time synchronized to the first recording sample): 11:46:33 + 0 + 0 samples
Satellite no.: 04
Trace length: 0h30'28''. Analysis performed on the entire trace.
Sampling rate: 128 Hz
Window size: 30 s
Smoothing type: Rectangular window
Smoothing: 5%

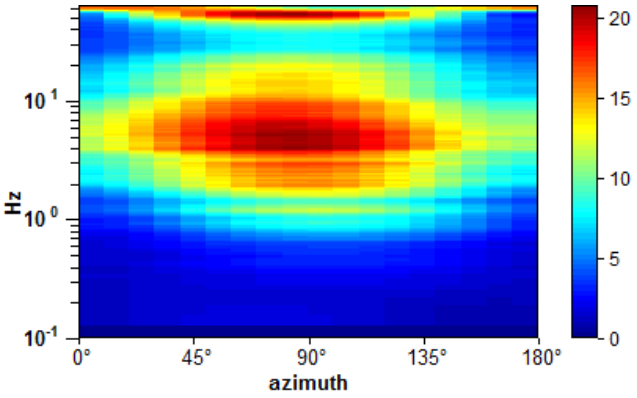
HORIZONTAL TO VERTICAL SPECTRAL RATIO



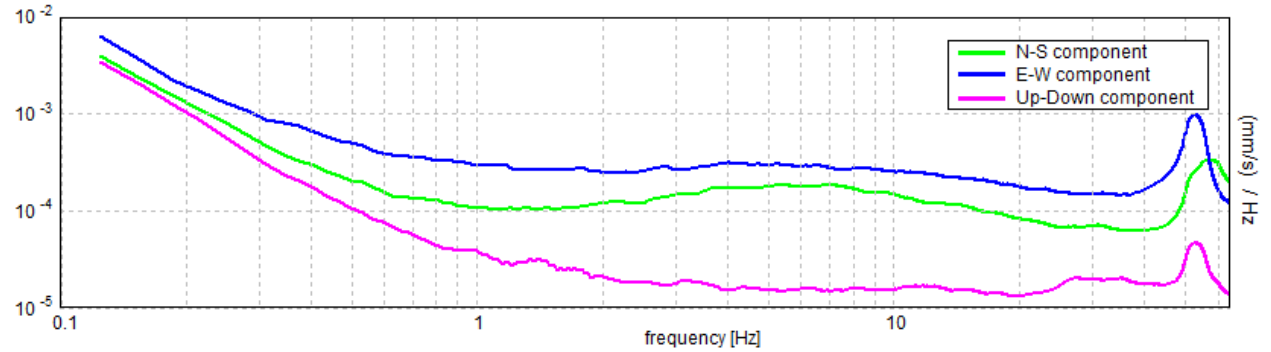
H/V TIME HISTORY



DIRECTIONAL H/V



SINGLE COMPONENT SPECTRA



[According to the SESAME, 2005 guidelines. **Please read carefully the *Grilla* manual before interpreting the following tables**.]

Max. H/V at 5.06 ± 1.13 Hz (in the range 0.5 - 32.0 Hz).

Criteria for a reliable H/V curve [All 3 should be fulfilled]			
$f_0 > 10 / L_w$	5.06 > 0.33	OK	
$n_c(f_0) > 200$	9112.5 > 200	OK	
$\sigma_A(f) < 2$ for $0.5f_0 < f < 2f_0$ if $f_0 > 0.5$Hz **$\sigma_A(f) < 3$ for $0.5f_0 < f < 2f_0$ if $f_0 < 0.5$Hz**	Exceeded 0 out of 244 times	OK	
Criteria for a clear H/V peak [At least 5 out of 6 should be fulfilled]			
Exists f^- in $[f_0/4, f_0]$ | $A_{H/V}(f^-) < A_0 / 2$	1.5 Hz	OK	
Exists f^+ in $[f_0, 4f_0]$ | $A_{H/V}(f^+) < A_0 / 2$			NO
$A_0 > 2$	16.53 > 2	OK	
$f_{peak}[A_{H/V}(f) \pm \sigma_A(f)] = f_0 \pm 5\%$	|0.22376| < 0.05		NO
$\sigma_f < \varepsilon(f_0)$	1.13277 < 0.25313		NO
$\sigma_A(f_0) < \theta(f_0)$	1.4616 < 1.58	OK	

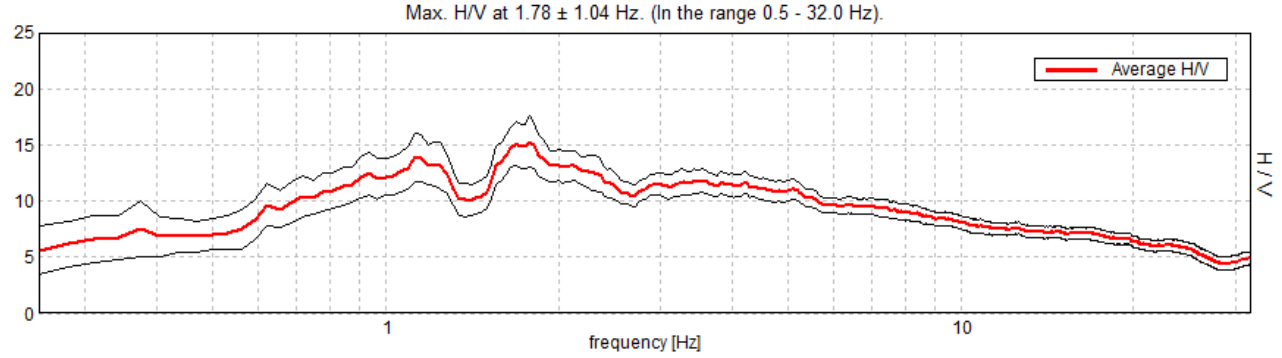
L_w	window length
n_w	number of windows used in the analysis
$n_c = L_w n_w f_0$	number of significant cycles
f	current frequency
f_0	H/V peak frequency
σ_f	standard deviation of H/V peak frequency
$\varepsilon(f_0)$	threshold value for the stability condition $\sigma_f < \varepsilon(f_0)$
A_0	H/V peak amplitude at frequency f_0
$A_{H/V}(f)$	H/V curve amplitude at frequency f
f^-	frequency between $f_0/4$ and f_0 for which $A_{H/V}(f^-) < A_0/2$
f^+	frequency between f_0 and $4f_0$ for which $A_{H/V}(f^+) < A_0/2$
$\sigma_A(f)$	standard deviation of $A_{H/V}(f)$, $\sigma_A(f)$ is the factor by which the mean $A_{H/V}(f)$ curve should be multiplied or divided
$\sigma_{logH/V}(f)$	standard deviation of log $A_{H/V}(f)$ curve
$\theta(f_0)$	threshold value for the stability condition $\sigma_A(f) < \theta(f_0)$

Threshold values for σ_f and $\sigma_A(f_0)$					
Freq. range [Hz]	< 0.2	0.2 – 0.5	0.5 – 1.0	1.0 – 2.0	> 2.0
$\varepsilon(f_0)$ [Hz]	0.25 f_0	0.2 f_0	0.15 f_0	0.10 f_0	0.05 f_0
$\theta(f_0)$ for $\sigma_A(f_0)$	3.0	2.5	2.0	1.78	1.58
log $\theta(f_0)$ for $\sigma_{logH/V}(f_0)$	0.48	0.40	0.30	0.25	0.20

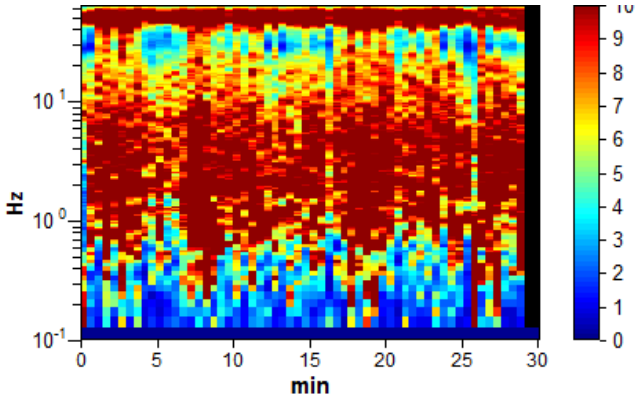
UMM-AR-RASSAS, UAR03

Instrument: TEP-0036/01-09
Data format: 16 byte
Full scale [mV]: n.a.
Start recording: 02/08/14 12:23:11 End recording: 02/08/14 12:53:15
Channel labels: NORTH SOUTH; EAST WEST ; UP DOWN ; GPS + GPS -
GPS location: 035°55.3247 E, 31°30.7327 N (747.5 m)
(UTC time synchronized to the first recording sample): 12:23:11 + 0 + 0 samples
Satellite no.: 05
Trace length: 0h30'12''. Analysis performed on the entire trace.
Sampling rate: 128 Hz
Window size: 30 s
Smoothing type: Rectangular window
Smoothing: 5%

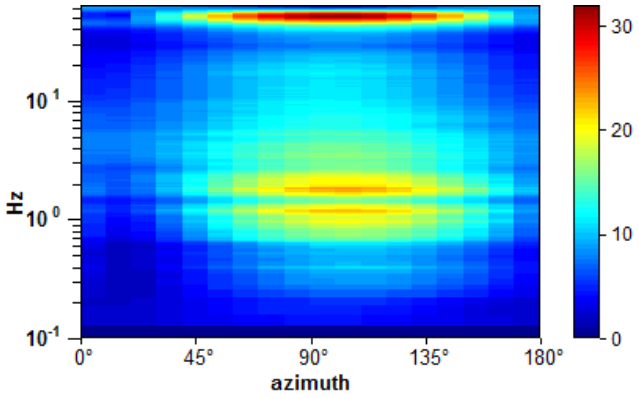
HORIZONTAL TO VERTICAL SPECTRAL RATIO



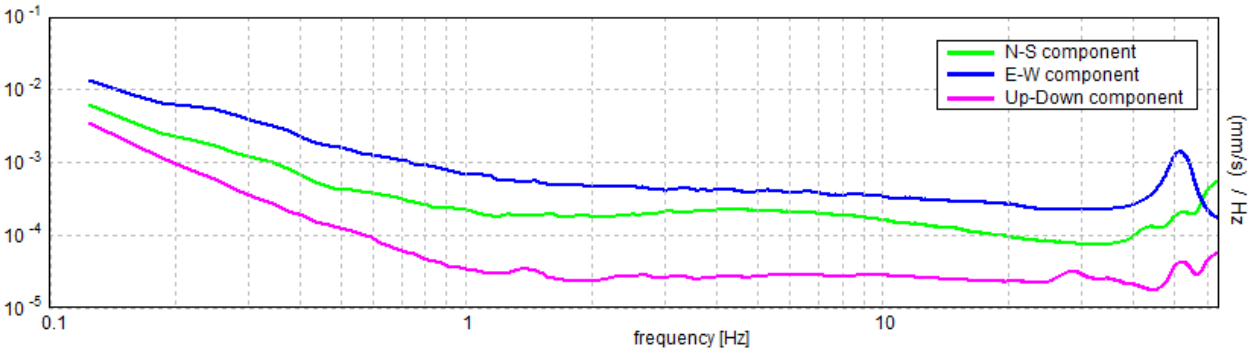
H/V TIME HISTORY



DIRECTIONAL H/V



SINGLE COMPONENT SPECTRA



[According to the SESAME, 2005 guidelines. **Please read carefully the *Grilla* manual before interpreting the following tables**.]

Max. H/V at 1.78 ± 1.04 Hz (in the range 0.5 - 32.0 Hz).

Criteria for a reliable H/V curve [All 3 should be fulfilled]			
$f_0 > 10 / L_w$	1.78 > 0.33	OK	
$n_c(f_0) > 200$	3206.3 > 200	OK	
$\sigma_A(f) < 2$ for $0.5f_0 < f < 2f_0$ if $f_0 > 0.5$Hz **$\sigma_A(f) < 3$ for $0.5f_0 < f < 2f_0$ if $f_0 < 0.5$Hz**	Exceeded 6 out of 86 times		NO
Criteria for a clear H/V peak [At least 5 out of 6 should be fulfilled]			
Exists f^- in $[f_0/4, f_0]$ | $A_{H/V}(f^-) < A_0 / 2$	0.563 Hz	OK	
Exists f^+ in $[f_0, 4f_0]$ | $A_{H/V}(f^+) < A_0 / 2$			NO
$A_0 > 2$	15.26 > 2	OK	
$f_{peak}[A_{H/V}(f) \pm \sigma_A(f)] = f_0 \pm 5\%$	|0.58349| < 0.05		NO
$\sigma_f < \varepsilon(f_0)$	1.03934 < 0.17813		NO
$\sigma_A(f_0) < \theta(f_0)$	2.2967 < 1.78		NO

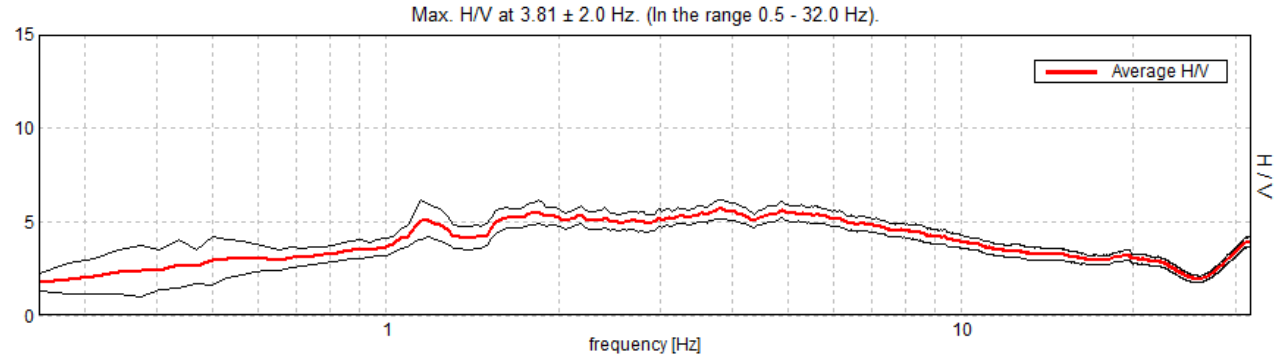
L_w	window length
n_w	number of windows used in the analysis
$n_c = L_w n_w f_0$	number of significant cycles
f	current frequency
f_0	H/V peak frequency
σ_f	standard deviation of H/V peak frequency
$\varepsilon(f_0)$	threshold value for the stability condition $\sigma_f < \varepsilon(f_0)$
A_0	H/V peak amplitude at frequency f_0
$A_{H/V}(f)$	H/V curve amplitude at frequency f
f^-	frequency between $f_0/4$ and f_0 for which $A_{H/V}(f^-) < A_0/2$
f^+	frequency between f_0 and $4f_0$ for which $A_{H/V}(f^+) < A_0/2$
$\sigma_A(f)$	standard deviation of $A_{H/V}(f)$, $\sigma_A(f)$ is the factor by which the mean $A_{H/V}(f)$ curve should be multiplied or divided
$\sigma_{logH/V}(f)$	standard deviation of log $A_{H/V}(f)$ curve
$\theta(f_0)$	threshold value for the stability condition $\sigma_A(f) < \theta(f_0)$

Threshold values for σ_f and $\sigma_A(f_0)$					
Freq. range [Hz]	< 0.2	0.2 – 0.5	0.5 – 1.0	1.0 – 2.0	> 2.0
$\varepsilon(f_0)$ [Hz]	0.25 f_0	0.2 f_0	0.15 f_0	0.10 f_0	0.05 f_0
$\theta(f_0)$ for $\sigma_A(f_0)$	3.0	2.5	2.0	1.78	1.58
log $\theta(f_0)$ for $\sigma_{logH/V}(f_0)$	0.48	0.40	0.30	0.25	0.20

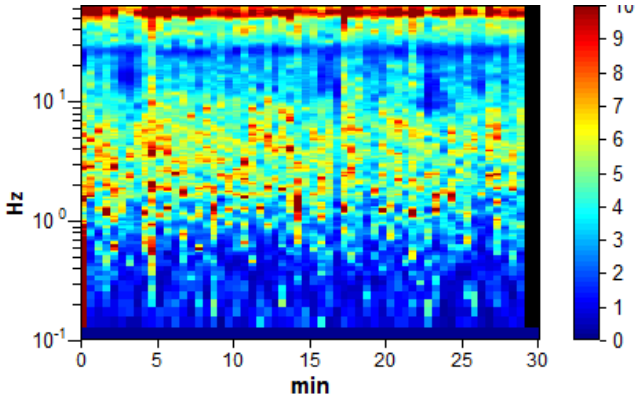
UMM-AR-RASSAS, UAR04

Instrument: TEP-0036/01-09
Data format: 16 byte
Full scale [mV]: n.a.
Start recording: 02/08/14 13:00:41 End recording: 02/08/14 13:30:49
Channel labels: NORTH SOUTH; EAST WEST ; UP DOWN ; GPS + GPS -
GPS location: 035°55.3129 E, 31°30.7384 N (749.3 m)
(UTC time synchronized to the first recording sample): 13:00:41 + 0 + 0 samples
Satellite no.: 04
Trace length: 0h30'12''. Analysis performed on the entire trace.
Sampling rate: 128 Hz
Window size: 30 s
Smoothing type: Rectangular window
Smoothing: 5%

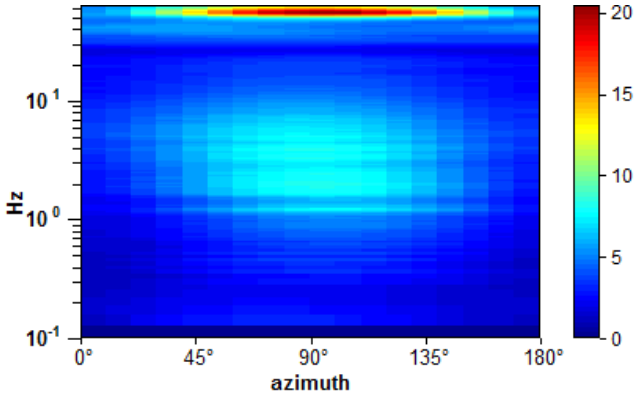
HORIZONTAL TO VERTICAL SPECTRAL RATIO



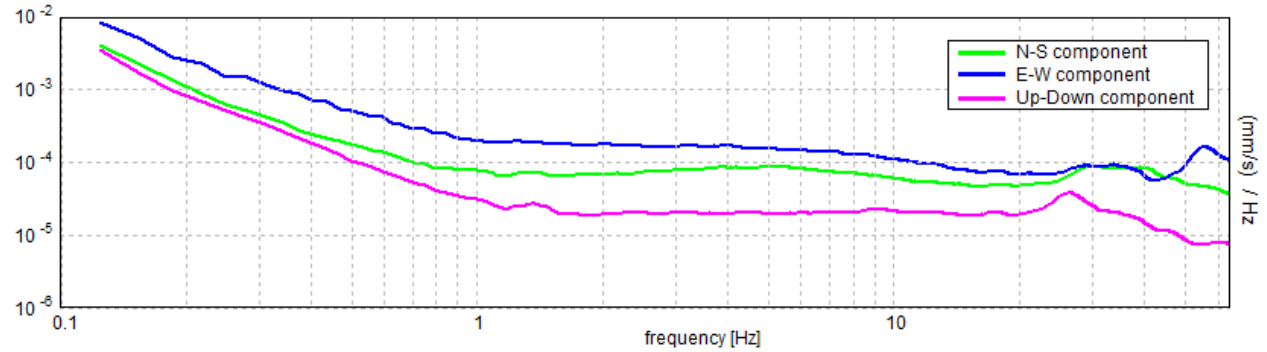
H/V TIME HISTORY



DIRECTIONAL H/V



SINGLE COMPONENT SPECTRA



[According to the SESAME, 2005 guidelines. **Please read carefully the *Grilla* manual before interpreting the following tables**.]

Max. H/V at 3.81 ± 2.0 Hz (in the range 0.5 - 32.0 Hz).

Criteria for a reliable H/V curve [All 3 should be fulfilled]			
$f_0 > 10 / L_w$	3.81 > 0.33	OK	
$n_c(f_0) > 200$	6862.5 > 200	OK	
$\sigma_A(f) < 2$ for $0.5f_0 < f < 2f_0$ if $f_0 > 0.5$Hz **$\sigma_A(f) < 3$ for $0.5f_0 < f < 2f_0$ if $f_0 < 0.5$Hz**	Exceeded 0 out of 184 times	OK	
Criteria for a clear H/V peak [At least 5 out of 6 should be fulfilled]			
Exists f^- in $[f_0/4, f_0]$ | $A_{H/V}(f^-) < A_0 / 2$			NO
Exists f^+ in $[f_0, 4f_0]$ | $A_{H/V}(f^+) < A_0 / 2$			NO
$A_0 > 2$	5.68 > 2	OK	
$f_{peak}[A_{H/V}(f) \pm \sigma_A(f)] = f_0 \pm 5\%$	|0.52563| < 0.05		NO
$\sigma_f < \varepsilon(f_0)$	2.00395 < 0.19063		NO
$\sigma_A(f_0) < \theta(f_0)$	0.504 < 1.58	OK	

L_w	window length
n_w	number of windows used in the analysis
$n_c = L_w n_w f_0$	number of significant cycles
f	current frequency
f_0	H/V peak frequency
σ_f	standard deviation of H/V peak frequency
$\varepsilon(f_0)$	threshold value for the stability condition $\sigma_f < \varepsilon(f_0)$
A_0	H/V peak amplitude at frequency f_0
$A_{H/V}(f)$	H/V curve amplitude at frequency f
f^-	frequency between $f_0/4$ and f_0 for which $A_{H/V}(f^-) < A_0/2$
f^+	frequency between f_0 and $4f_0$ for which $A_{H/V}(f^+) < A_0/2$
$\sigma_A(f)$	standard deviation of $A_{H/V}(f)$, $\sigma_A(f)$ is the factor by which the mean $A_{H/V}(f)$ curve should be multiplied or divided
$\sigma_{logH/V}(f)$	standard deviation of log $A_{H/V}(f)$ curve
$\theta(f_0)$	threshold value for the stability condition $\sigma_A(f) < \theta(f_0)$

Threshold values for σ_f and $\sigma_A(f_0)$					
Freq. range [Hz]	< 0.2	0.2 – 0.5	0.5 – 1.0	1.0 – 2.0	> 2.0
$\varepsilon(f_0)$ [Hz]	0.25 f_0	0.2 f_0	0.15 f_0	0.10 f_0	0.05 f_0
$\theta(f_0)$ for $\sigma_A(f_0)$	3.0	2.5	2.0	1.78	1.58
log $\theta(f_0)$ for $\sigma_{logH/V}(f_0)$	0.48	0.40	0.30	0.25	0.20