COMPDYN 2009
ECCOMAS Thematic Conference on
Computational Methods in Structural Dynamics and Earthquake Engineering
M. Papadrakakis, N.D. Lagaros, M. Fragiadakis (eds.)
Rhodes, Greece, 22–24 June 2009

SEISMIC RESPONSE OF THE 35M HIGH MASONRY CHIMNEY OF THE ALLATINI COMPLEX

Christoudoulos Stylianidis[1], Anastasios Sextos[2], Kosmas Stylianidis[3]

[1]Civil Engineer, Postgraduate student, Imperial College London, U.K.
christodoulos.stylianidis08@imperial.ac.uk

[2]Assistant Professor, Division of Structural Engineering,
Department of Civil Engineering, Aristotle University of Thessaloniki, 54124 Greece
asextos@civil.auth.gr

[3]Professor, Laboratory of Reinforced Concrete and Masonry Structures,
Department of Civil Engineering, Aristotle University of Thessaloniki, 54124 Greece
kcstyl@civil.auth.gr

Keywords: Masonry, historical buildings, chimneys, seismic response, pre-earthquake assessment, dynamic analysis, finite elements.

Abstract. *This paper focuses on the chimney of the industrial "Allatini Mill" complex in Thessaloniki which was built in 1854 as the first flour mill of the city and was closed in early 90's, after 140 years of incessant operation, still tough attracting scientific attention from both an architectural and structural engineering point of view. More specifically, the static and dynamic response of the 35m high, masonry chimney of the complex is studied both analytically and numerically. The effect of foundation conditions and the subsequent soil-structure interaction is also discussed while a refined back analysis is presented that is based on the observed good performance of the tower during the 6.5Ms earthquake that jolted the city in 1978. Along these lines, the recorded motion of the above seismic event is used and properly deconvoluted to the bedrock level, while linear and non-linear site response analyses are used to generate several ground motion scenarios at the foundation-soil interface. A set of other parameters (i.e. masonry, soil and bedrock mechanical properties) were also sequentially modified within the framework of a sensitivity analysis scheme in order to account for various uncertainties involved. The range of variation of the parameters examined was back-estimated in order to comply with the fact that no significant damage was observed at the chimney during the 1978 earthquake. The results (inevitably) verify the performance of the chimney during the 1978 event but most importantly highlight the sections of potential strengthening required to enhance the standards of its seismic resistance. The overall back-analysis approach presented is also deemed as a comprehensive strategy applicable to other historical buildings or monumental structures as well, towards a more realistic vulnerability assessment and a means to preserve their architectural character whilst avoiding overdesign of the required rehabilitation and intervention measures.*

1 INTRODUCTION

During the last decades a growing interest has been developed worldwide towards the preservation of cultural heritage and historical buildings. This field of research which is of major practical interest as well, may be proven far more demanding in specific cases compared to the case of the design of a regular modern building as the first is related to a large number of uncertainties such as, the history of the structure, its condition at the time of assessment, its response under past environmental or accidental actions, but also to many requirements that have to be met (i.e. interpretation of its structural behavior, justification of any damage etc). Additionally and despite the above difficulties, it is necessary to obtain a realistic estimate of the available resistance of the historical structure under specific earthquake scenarios prior to any potential intervention; the latter also subjected to specific limitations related to the preservation of the architectural authenticity, the minimization of the intervention extent, the compatibility of the materials to be used and the reversibility of the actions to be taken, among others.

Beyond the above challenge though, is another critical issue of both theoretical and practical nature: the evaluation of the methods and procedures that are currently available for the assessment of the seismic performance of a historical structure. It is not uncommon that the designers really wonder, without an obvious answer, what is the level of reliability of the analysis and design methods used and what is the sensitivity of the predicted response on the decisions made especially in terms of the, more and more refined and computationally demanding, finite element models that can be currently developed. Given this complexity of the problem, the lack of evidence regarding the relative sensitivity of historical structures to the above uncertainties and especially the inherent major significance of such structures, the above question is transformed into a more practical problem which relates to the optimum balance between the foreseen structural safety and the fear of disproportionally excessive and unjustified intervention.

Along these lines, the present study emphasizes to a 35m high, masonry chimney which is located in the city of Thessaloniki, Greece, within the complex of the "Allatini" Mills. The particular structure is chosen for three main reasons:

(a) its structural configuration is the simplest possible, being essentially an isostatic vertical cantilever and as such, the uncertainties typically associated with hyperstatic structures (i.e. cracking and stiffness distribution) as well as their numerical modeling implications do not consist part of the problem.

(b) the significant height of the chimney (35m), leads to a long fundamental period, thus being a system with a very clear dynamic and seismic response, which elaborates the evaluation of the currently available seismic assessment methods and tools and the quantification of the sensitivity of results obtained to the analysis assumptions made.

(c) the chimney was subjected, without suffering any visible damage, to a strong earthquake in 1978 (M_s=6.5), which was also recorded at the center of the city. It was therefore deemed necessary to consider this very good behavior of the chimney during the 1978 earthquake as the reference level of this study, in order to quantify the related numerical and physical uncertainties and evaluate the methods and tools currently available for the seismic assessment of historical structures. It is recalled herein that according to the current philosophy for the rehabilitation of historical structures, such a verification of the observed response under a previous seismic event is a prerequisite of acceptance of the analysis method used and its relevant assumptions.

The present paper therefore, aims not only to specifically assess the seismic capacity and demand of the particular chimney under earthquake loading for various levels of earthquake loading but also to contribute towards the discussion related to the overall strategy that should be adopted nowadays for the assessment of such specific structures.

2 OVERVIEW OF THE CASE STUDY

2.1 Historical and architectural background

Similar industrial brickwork chimneys were mostly built during the industrial revolution in many European countries at the end of the 19th century. Despite their operation during the 20th century, currently, very few of them remain operative since they have been replaced by modern constructions built according to the present legislative framework; they still remain primarily as a characteristic landmark that is often protected by law as part of the cultural heritage. Most chimneys are made of masonry but it is not uncommon that they are of significant height that reaches even 70 m [1]. Despite the recent advances related to seismic assessment of masonry structures, there are only few studies (i.e. [2], [3], [4]) that focus on the particular construction, a fact that can be primarily attributed to the lack of experimental evidence, the difficulties of on-site inspection and possibly to their limited usability.

The particular Mill building was constructed in 1854 but was completely burned down in 1898 and had to be built again, bigger, in the same position with plans of the well known architect Vitaliano Pozelli. Apart from the main building, the Mill complex included warehouses, workshops and smaller secondary buildings but was incinerated by fire in 1936. The Mill was reconstructed and in a few years became operative again until it was again destroyed by fire in 1951. In 1971 the site of the Allatini Mill was designated as area of public use and in 1991 the building of the Mill, the chimney under study, a number of other buildings and their surroundings were characterized as preservable monuments. Nowadays, a master plan has been developed for the rehabilitation and reuse of the overall complex.

The chimney is one of the most remarkable buildings of the complex [5] being a leading type of construction in its time, that acted as a landmark of the site; even now it still dominates the entire neighborhood due to its height. It was constructed by French engineers, definitely before 1889, since it appears in photos of the original mill (Fig. 1), that is, before the reconstruction of V. Pozelli. The technology of construction used is deemed as an important part of the history of construction in Greece.

2.2 Description of the soil-foundation-structure system

Since no foundation drawings were available, the data required for this study were taken from specific excavations that were made around the chimney in conjunction with the construction of the underground parking station. A local deepening of the excavation was also made on the east side, revealing the basis of the structure [6], surface wells and pipelines which were removed, debris down to a level of 2.0m and clay until a depth of 3.0m where the investigation terminated. Due to the proximity with the sea, the water level was expected to be rather high and was indeed found at 2.55m from the surface, that is, slightly higher than the level of foundation of the chimney which was found at a depth of 2.72m. A linear part of the foundation of 0.90m length was also revealed by the excavation. After appropriate topographic dependence of the base edge from the body of the chimney and assuming a symmetrical foundation layout, it was deemed that the prismatic foundation base must have a square plan and dimensions 5.76x5.76m, while its minimum extension from the cylindrical body of the chimney reaches 1.20m. With the use of inclined drilled steel rods outside the edge of the prismatic base, it was also found that its thickness is of the order of about 1.00m. As a result, the foundation depth was taken equal to 3.72m. In terms of soil conditions, due to lack of on-purpose measured geotechnical data, the subsoil profile below the chimney was considered parametrically, based on nearby drillings [7] and the Microzonation study of

Thessaloniki [8]. The chimney superstructure is constructed exclusively with solid brick structured in concentric rings with diameters decreasing in height in successive layers, according to the practice of the period of construction. At the base, the outer diameter is 3.36m and the wall thickness equal to 0.78m being reduced to 1.76m and 0.23m respectively at the top of the chimney. At the base and the top end of the superstructure the chimney is decorated with rings of solid brick, structured in different configurations (Fig. 1). The height of the chimney from the foundation level to its upper end is 39.35m but the exact height of the superstructure (measured from its base) is 38.35m for the reasons already explained. Similarly, the height of the visible part of the chimney is equal to 35.63m (Fig. 2). Inside the chimney, a staircase is attached to the brick walls according to the practice of that time. Four, symmetrically arranged, openings exist at the base for operative purposes; three of them were found to be closed.

Figure 1: Overview of the Chimney and the Allatini Complex.

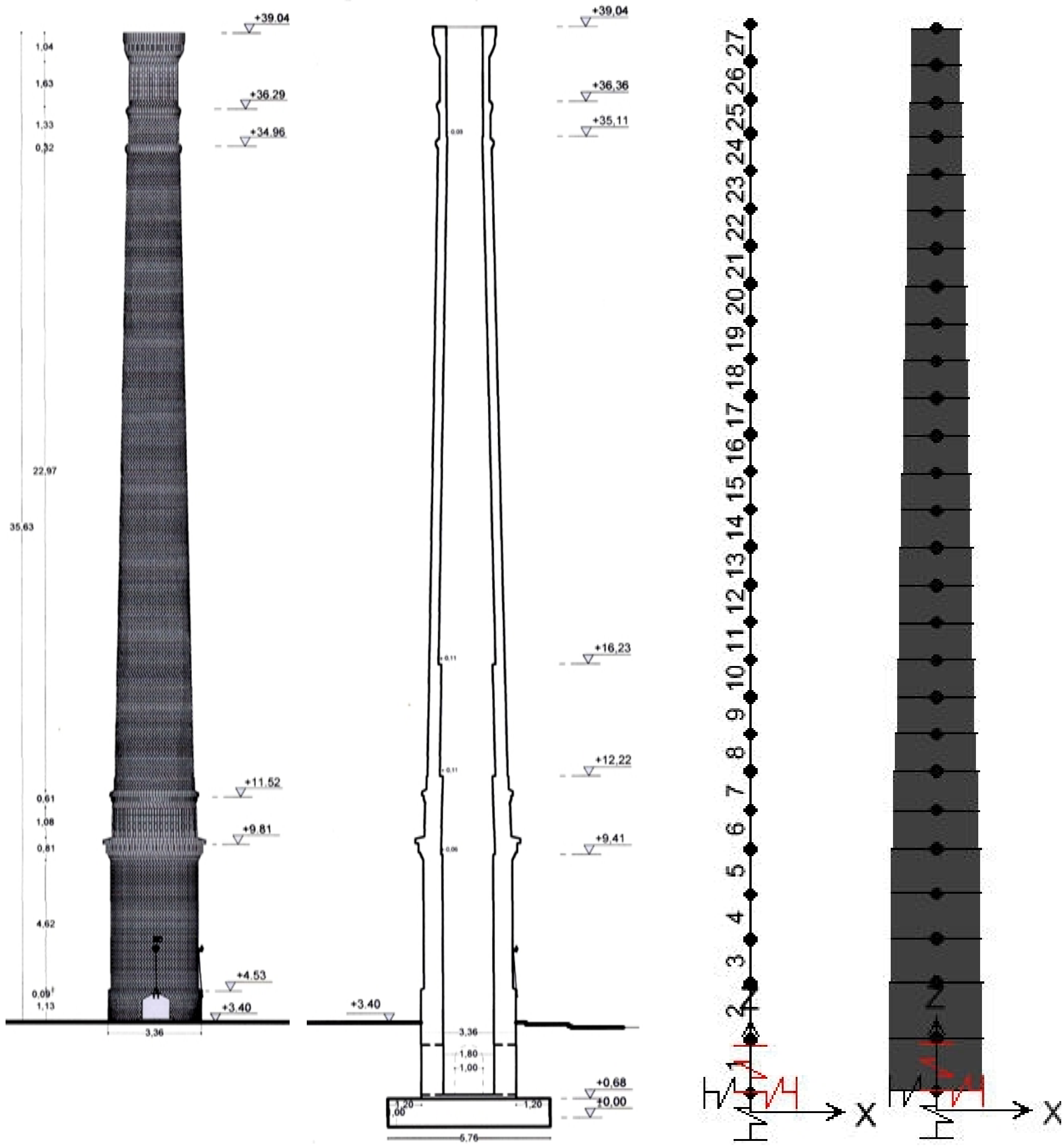

Figure 2: Geometry of the chimney and finite element model.

The chimney does not currently show any sign of significant structural damage. Local spalling and presence of salt are only observed at the outer surface of the chimney while some small parts have been detached at its top. As anticipated, particular areas close to the end of the chimney have been covered by smut. Spalling is limited inside but the smut covers larger surfaces, especially near the base. No inclination is visible and most importantly, no damage has been reported as a result of the 1978 earthquake. In general, the chimney gives the impression of a geometrically thin but stable construction.

2.3 Characterization of masonry

As it is well known, in general, masonry is a non-homogeneous anisotropic material consisting of units of bricks or stones and mortar with a strongly inelastic behavior. Due to this inhomogeneity, typically, its mechanical properties are estimated based on in-situ measurements, sampling and laboratory experiments. Along these lines, an extensive sampling program was also carefully followed for the particular case in order to provide reliable estimates of the mechanical properties of all the materials used (i.e. brick, stone, mortar) for the construction of the complex as a whole, including the chimney. The samples taken were then tested experimentally at the laboratory of Aristotle University of Thessaloniki. It was found that the masonry of the chimney consists of compact bricks with normally arranged joints of strong mortar for which their properties were defined according to Eurocode 6 [9] as follows:

- compressive strength of bricks: $f_b \cong 20.0$ MPa
- compressive strength of mortar: $f_m \cong 5.0$ MPa
- characteristic uniaxial compressive strength: $f_{ck} = K \cdot f_b^{0.65} \cdot f_m^{0.25}$ (MPa) = 5.25 MPa
- characteristic shear strength $f_{vk} = f_{vko} + \mu \cdot \sigma d = 0.20 + 0.40 \cdot \sigma d$ (MPa)
- elastic modulus: $E \cong 1000\ f_{ck} = 5250$MPa
- poisson coefficient $v = 0.2$,
- self weight: $w = 17.2$ kN/m^3

3 FLEXURAL CAPACITY ALONG THE CHIMNEY HEIGHT

All sections of the chimney are annular. The calculation of bearing capacity of such an annular section made of masonry of a given compressive strength, that is, the flexural capacity of the section under a given axial load was derived analytically [10] with the following assumptions:

(a) as the aim was to assess the flexural capacity of the sections and not to design the structure, the material safety factor was ignored.

(b) due to the significant dispersion of the masonry properties and the difficulty to evaluate the mean value of strength from the corresponding characteristic, it was the aforementioned characteristic compressive strength that was taken into account (f_{ck}=5.2MPa) and not the (higher) value of mean strength. For the sake of investigation, an alternative (lower) value of characteristic compressive strength (f_{ck} = 3.8MPa) was also considered.

(c) in order to speed up the computation process, the variation of strain along the section was taken linear but the diagram of stresses was considered rectangular instead of parabolic-rectangular.

Three cases were distinguished depending on the position of neutral axis (Fig. 3):

- **Case I (weak compression)**: corresponding to a relatively low value of axial force which results in a small area of compression. The neutral axis intersects a solid part of the section.
- **Case II (moderate compression)**: corresponding to a moderate value of axial force which results in a significant area of compression. The neutral axis intersects the circular ring.
- **Case III (strong compression)**: corresponding to a relatively large value of axial force which in turn results to an extensive compression zone. The neutral axis intersects a large part of the section, while it is only a small, solid part of the ring section that remains in tension.

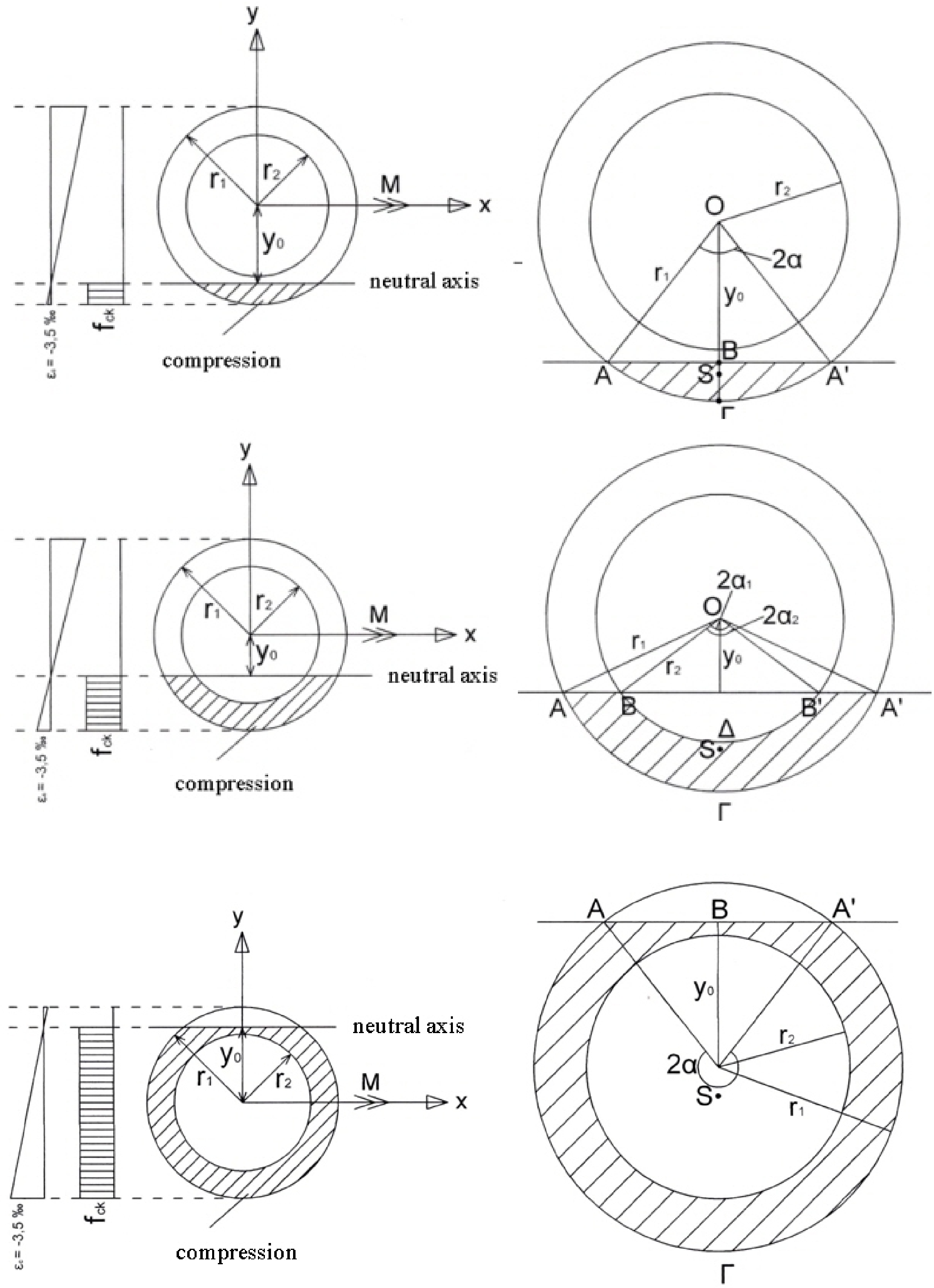

Figure 3: Alternative positions of the neutral axis for the case of an annular (ring) section.

Further information regarding the analytical approach can be found elsewhere [10]. Based on these expressions, the flexural capacity of the chimney was derived. It is recalled that as both the geometry of the chimney and the level of axial load varies with height, the capacity of the chimney is inevitably height-dependent. The resulting section strength is summarized in Table 1 for two distinct cases of masonry compression strength, i.e. 5.2 and 3.8MPa. It is obvious though, that the flexural capacity of the section is not severely affected by this variation of the masonry compression strength, since a drop of the order of 27% of the latter only leads to a minor (2%) reduction of the overall section strength.

Section level (m)	Outer radius (m)	Inner radius (m)	Axial load (kN)	Mu f_{ck}=5.2 Mpa	Mu f_{ck}=3.8 Mpa
0.68	1.68	0.90	2019.90	3033.63	2948.34
2.60	1.68	0.90	1811.11	2743.02	2672.05
4.58	1.68	0.90	1595.81	2438.61	2381.26
6.19	1.58	0.90	1449.10	2078.90	2029.04
7.80	1.58	0.90	1302.38	1881.60	1839.93
9.41	1.54	0.94	1155.67	1634.25	1599.85
10.82	1.50	0.90	1042.30	1439.95	1410.75
12.22	1.41	0.98	933.37	1207.16	1182.34
13.57	1.38	0.95	858.66	1090.84	1069.10
14.90	1.35	0.92	786.81	981.51	962.59
16.23	1.32	1.00	716.71	877.76	861.46
17.58	1.29	0.97	662.51	795.54	781.14
18.93	1.26	0.94	609.63	717.47	704.85
20.28	1.24	0.92	558.07	643.52	632.55
21.63	1.21	0.89	507.82	573.54	564.10
22.98	1.18	0.86	458.89	507.42	499.39
24.33	1.15	0.83	411.28	445.11	438.37
25.68	1.12	0.80	364.99	386.48	380.91
27.03	1.10	0.78	320.01	331.42	326.91
28.38	1.07	0.75	276.35	279.81	276.25
29.73	1.04	0.72	234.01	231.59	228.87
31.08	1.01	0.69	192.98	186.63	184.63
32.43	0.98	0.66	153.27	144.81	143.43
33.78	0.95	0.63	114.88	106.03	105.17
35.11	0.91	0.65	78.36	69.40	68.94
36.36	0.88	0.65	50.96	43.92	43.69
37.70	0.88	0.65	25.48	22.15	22.07
39.04	0.88	0.65	0	0	0

Table 1: Geometry of the chimney and variation of flexural capacity with its height.

4 EARTHQUAKE EXCITATION SCENARIOS

As already mentioned, the recorded motions of the 1978 earthquake were taken as the reference level for the assessment of the seismic performance of the chimney and the evaluation of the available computational methods. The overall area of Thessaloniki is characterized by NE-SW and NS extensional stress field driven by the Hellenic subduction zone in the Aegean Sea [11] and intense seismic activity with strong historical earthquakes having magnitudes larger than 6.0 [12]. The most recent destructive earthquake (M_s=6.5) occurred at 20/06/1978 on the Gerakarou-Stivos fault along the Mygdonian graben and caused extensive human losses and damage within the city and the surrounding villages. Given the above seismicity, the broader area of Thessaloniki belongs to Seismic Zone II (PGA=0.24g) according to the current Hellenic Seismic Code, EAK2000 [13]. It is noted that the Metropolitan city of Thessaloniki belongs to Seismic Zone I (0.16g). Three components (E-W, N-S and vertical) of the 1978 earthquake were recorded by a single accelerometric station (THE-City Hotel) at the basement of a multi-storey building at the center of Thessaloniki (Figure 5) approximately 25 km South-West of the epicenter [14].

For the development of the seismic scenario required for the assessment of the chimney, the E-W (PGA=0.143g) and the N-S (PGA=0.136g) were used herein. The soil at the location where the records were obtained is generally soft, corresponding to class D ($180<V_s\leq360$) [15], according to NEHRP [16] which is equivalent to class C of Eurocode 8 [17] and class B to C of the Hellenic Seismic Code [13]. The exact soil profile required for this study was taken herein by a recent microtremor study [7]. It is clear that the aforementioned earthquake records cannot be used directly to other locations, unless (at least) a deconvolution process is followed in order to take into consideration the differences in soil stiffness, damping and stratification between the location of the recordings and the site under study. It is noted that a similar process has been also used for the assessment of other historical structures within the city [18].

Due to the lack of detailed geotechnical data at the Allatini site, the subsoil profile below the chimney was considered parametrically based again on nearby microtremor measurements [7] and the Microzonation study of Thessaloniki [8]. As a result, multiple site response analyses had to be performed to compute a set of earthquake records at the surface of the Allatini complex and to be used in turn as the seismic input for the dynamic analysis of the chimney. The parametric study was therefore performed by modifying four analysis parameters, related to the:

- *Direction of ground motion excitation*: both the E-W and N-S components were considered (2 cases).
- *Soil stiffness of the upper soil layer*: expressed in terms of shear wave velocity V_s which was taken equal to 70m/sec or 100m/sec in conjunction to appropriate proportional modification of the stiffness of the lower soil layers (an example of the reference soil profile is shown in Figure 5). It is noted that the above profiles generally correspond to soil class B and C (2 cases).
- *Stiffness of the bedrock*: assumed either elastic (Vs=2000m/sec) or rigid (2 cases).
- *Linear or non-linear response of the soil formations*: considered through shear stiffness-shear strain-damping (G-γ-D) curves during site response analysis (2 cases).

Based on the above, from a single pair of (E-W & N-S) accelerations that deconvoluted at the bedrock level of the City Hotel, 2^3=8 pairs of records were computed at the surface of the Allatini complex site, the 5% damped response spectra of which were subsequently derived together with their mean spectrum (Fig. 6). The latter (i.e. one mean spectrum resulting from linear and one from non-linear site response analysis) was used in the global parametric scheme in order to filter out the sensitivity of the structural response of the chimney to abrupt variations of spectral acceleration. Additionally to the above time history analysis of the structure, spectrum analysis according to the Hellenic Seismic Code, EAK2000 was also performed for comparison.

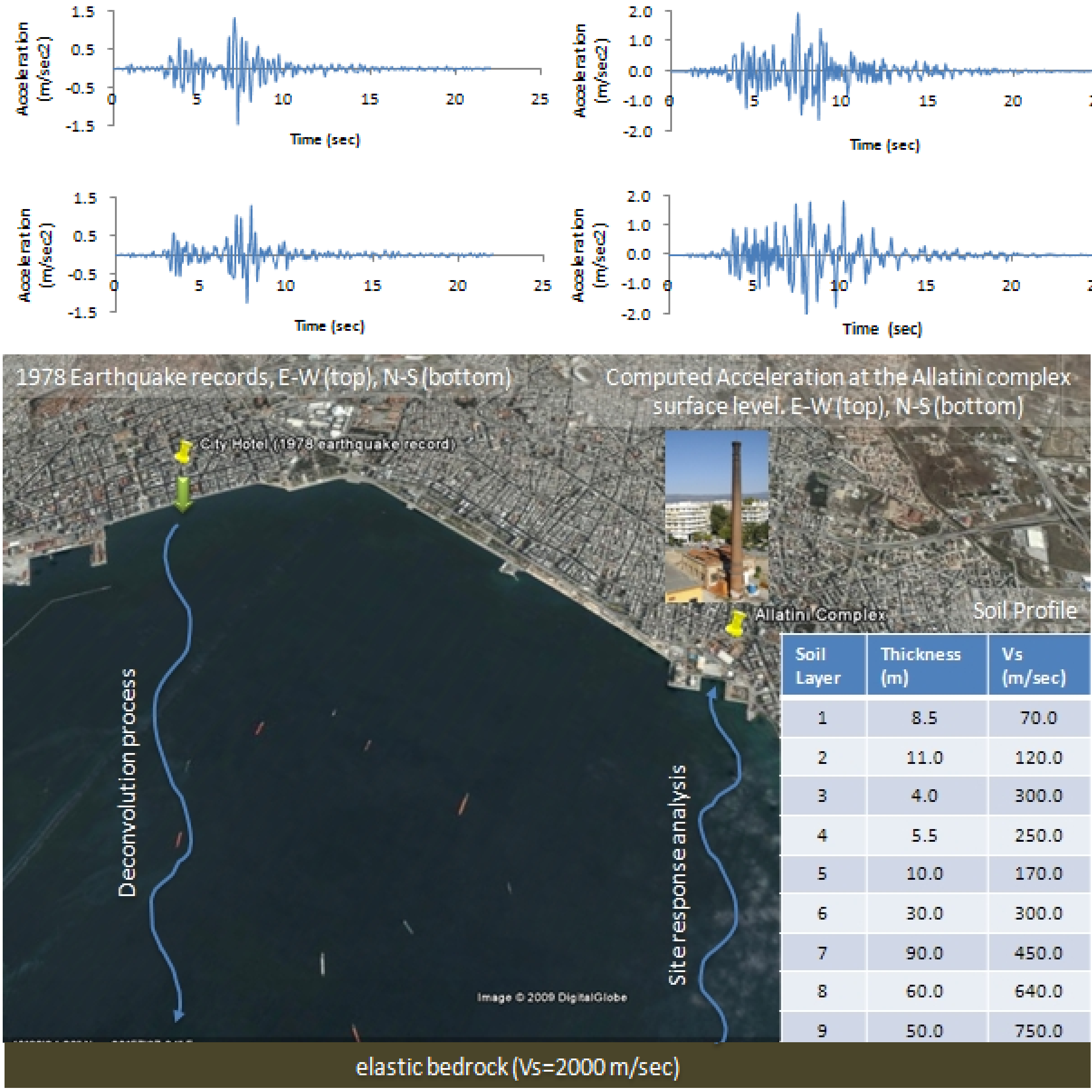

Figure 5: Deconvolution and (linear) site response analysis for the case of the reference soil profile and elastic bedrock properties (Vs=2000 m/sec).

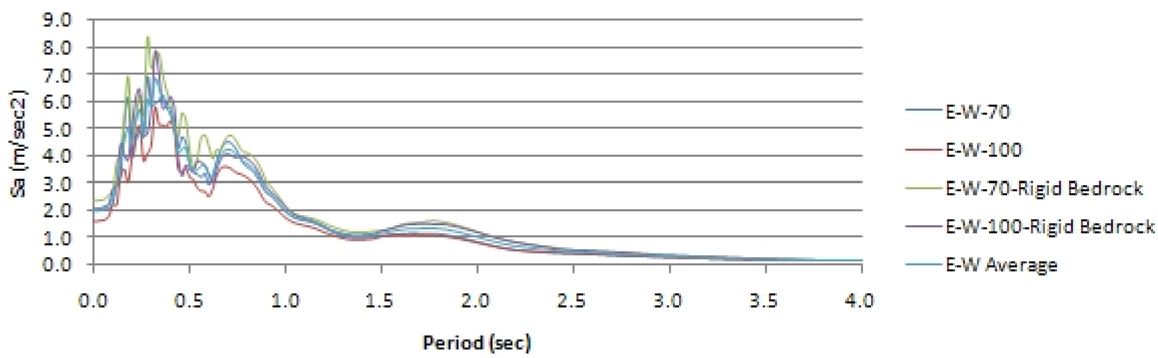

Figure 6: Response spectra of the equivalent E-W component of the 1978 earthquake event computed after linear site response analysis at the surface level of the Allatini complex for various assumptions regarding the stiffness of the upper soil layer (V_s=70 and 100m/sec) and the rigidity of the bedrock.

5 COMPUTATIONAL FRAMEWORK AND PARAMETRIC ANALYSIS SCHEME

The finite element program used for analysis was SAP2000 ver.10 [19]. Due to the structural configuration of the chimney, the model developed was intentionally chosen simple and consists of 27 linear elements (Figure 2) of identical to the ring section stiffness and mass which are deemed of adequate accuracy to predict the response of the axisymmetric structure under base excitation. The behavior of the masonry was assumed linear elastic. For the determination of the dynamic characteristics of the chimney and its dynamic response in the time and frequency domain for various earthquake scenarios, the following set of analyses was performed:

(a) static analysis for gravity loads without any safety factor in order to derive the variation of the axial load along the height of the chimney (apparently, due to the symmetric section, boundary conditions and loading, no bending moments were developed).
(b) modal analyses to compute the eigenvalues, mode shapes and participation factors of the (soil-foundation-structure) system.
(c) spectrum analyses for the 5% damped response spectra of the 1978 earthquake record scenarios as well as the EAK2000 response spectrum.
(d) time history analyses for the aforementioned 1978 earthquake record scenarios.

In order to quantify the effect of the uncertainties of the problem on the behavior of the chimney during the earthquake of 20/06/1978, the following parameters were examined:

- *Compression strength of masonry*: an upper (f_{ck} = 5.2MPa) and a lower (f_{ck} = 3.8MPa) bound was considered (2 cases).
- *Support conditions of the chimney*: taken as either fixed or flexible. For the latter case the properties of a 6-DOF spring attached at the base of the structure were computed based on the foundation dimensions, the stiffness of the subsoil and the analytical formulations proposed by Mylonakis et al. [20] (2 cases).
- *Earthquake scenarios*: based on the 4 mean spectra (grouped for the E-W & N-S component and the linear & nonlinear response of the soil) and after considering various soil profiles and two bedrock rigidity conditions as described in section 4 (4 cases).
- *Type of analysis*: time history and spectrum analysis using the accelerograms and the corresponding spectra of the above scenarios (2 cases).

Apart from these analysis combinations corresponding to 2x2x4x2 = 32cases, 2^3=8 additional spectrum analyses were performed using the *Hellenic Seismic Code EAK2000* response spectrum and another 2^3=8 analyses were conducted assuming *10% of material damping* (again for different support conditions, direction of excitation and linear/non-linear soil behavior, while assuming the compression strength of masonry fixed to 5.2MPa). As a result, a total of 48 analyses were performed, the structural response of which is presented and commented in the following section.

6 NUMERICAL ANALYSIS RESULTS AND STRUCTURAL ASSESSMENT

6.1 Dynamic characteristics of the fixed and flexibly supported chimney

As the modulus of elasticity of masonry was taken as a function of its compression strength, the dynamic characteristics of the chimney for the assumption of f_{ck}=5.2MPa and f_{ck}=3.8MPa were apparently different. Soil compliance was found to be an additional source of modification of the fundamental period of the structure. In particular, considering the flexibility of the soil-foundation system the fundamental period was increased from 1.05sec to 1.73sec (for the case of f_{ck}=5.2MPa). It has to be noted herein that, as seen in Table 2, soil-foundation flexibility, expressed in terms of the spring properties, is also a function of the earthquake scenario since the shear stiffness G of the soil is reduced with the peak ground acceleration a (where a<0.3g) according to an EC8-based expression [21]:

$$G/G_{max} = 41.6a^3-17.5a^2-0.66a+1 \quad (1)$$

The relation of the dynamic characteristics to the frequency content of the exciting ground motion for each one of the 8 earthquake scenarios presented in Table 2 is also affecting the corresponding base shear. The reduction in masonry strength leads to an increase of the fundamental period that is proportional to the square root of the ratio of the two elastic moduli. Another interesting finding is that for the case of the fully-fixed support conditions, a large number of modes of vibration is required in order to activate a significant percentage of the mass of the structure, in contrast to the case of flexible supports. For instance, for the case of masonry strength equal to 5.2 MPa, the first three modes activate 35% of the mass, while, 90% is activated when soil compliance is considered.

	Analysis case	T_1 (sec)	p. f. (%)	T_2 (sec)	p.f. (%)	T_3 (sec)	p.f. (%)	Base Shear (kN)
SSI	E-W linear	1.721	54.8%	0.374	23.5%	0.148	12.3%	340
	E-W non linear	1.721	54.8%	0.374	23.5%	0.148	12.3%	325
	N-S linear	1.742	55.0%	0.376	23.2%	0.149	12.4%	235
	N-S non linear	1.731	54.9%	0.375	23.5%	0.148	12.3%	215
FIXED	E-W linear	1.054	24.8%	0.262	9.5%	0.114	0.6%	278
	E-W non linear	1.054	24.8%	0.262	9.5%	0.114	0.6%	280
	N-S linear	1.054	24.8%	0.262	9.5%	0.114	0.6%	401
	N-S non linear	1.054	24.8%	0.262	9.5%	0.114	0.6%	323

Table 2: Periods of vibration, participation factors of the first three modes as well as the corresponding base shear for eight soil-foundation-structure systems (f_{ck}=5.2MPa, ζ = 5%).

6.2 Seismic response of the chimney in the frequency and time domain

Although there are many ways to present the results of this analysis, it was deemed preferable herein to assess the response of the chimney in terms of the variation with height of the maximum in time bending moments for the different earthquake scenarios studied. These results are presented in detail in the form of diagrams in Stylianidis (2008) [10]. It is noted that it is the *effective* bending moment values that are considered, since the maximum in time moments are reduced to their 2/3 to filter out the higher acceleration peaks [22].

It is observed that, in terms of the direction of excitation, there is no clear indication whether the E-W or the N-S component yields the most unfavourable demand, a fact that can be attributed to the aforementioned interplay between the modes of vibration and the frequency content of each acceleration record used for the excitation of the structure. The most decisive factor though, in terms of structural response is the support conditions, which in the case where soil flexibility is accounted for, lead to significantly lower demand, especially close to the base of the chimney. This fact is again related to the (modified) dynamic characteristics of the system, which this time lead to clearly lower spectral acceleration. For the same reason the bending moments developed are smaller for the case of reduced flexural capacity (and subsequent lower value of modulus of elasticity). It is also observed that spectrum analysis according to the Hellenic Seismic Code EAK2000 leads to higher demand independently of the parameter studied, a fact that is generally anticipated since the design earthquake and as such, the response spectrum provided by the code is more demanding compared to the scenarios based on the 1978 event.

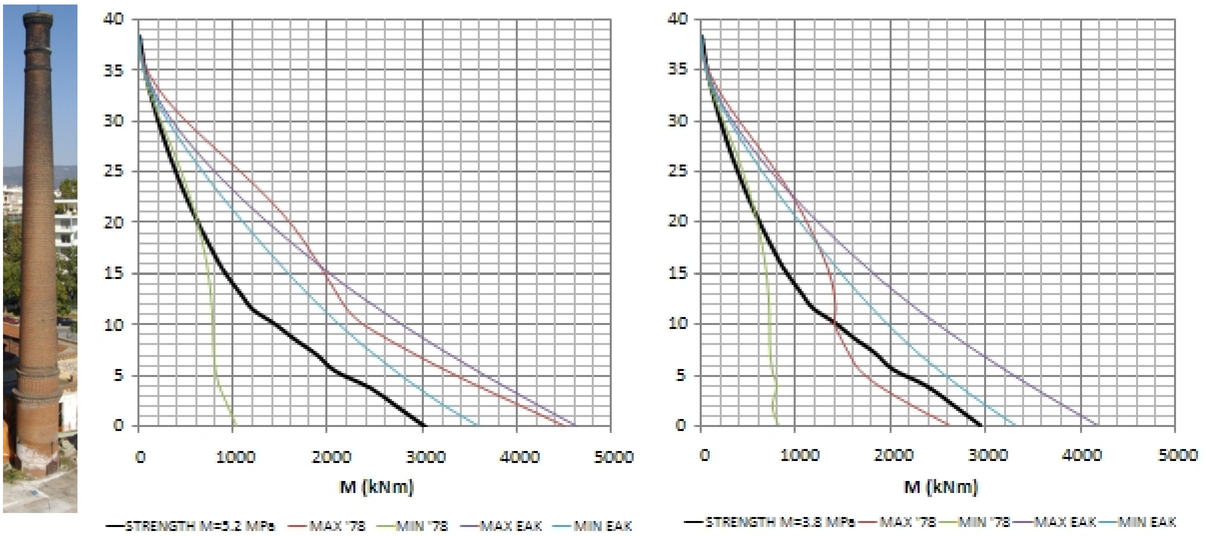

Figure 7: Envelopes of maximum and minimum bending moments for dynamic time history (1978 record) and spectrum analysis (EAK). Strength equal to 5.2 MPa (left) and 3.8 MPa (right).

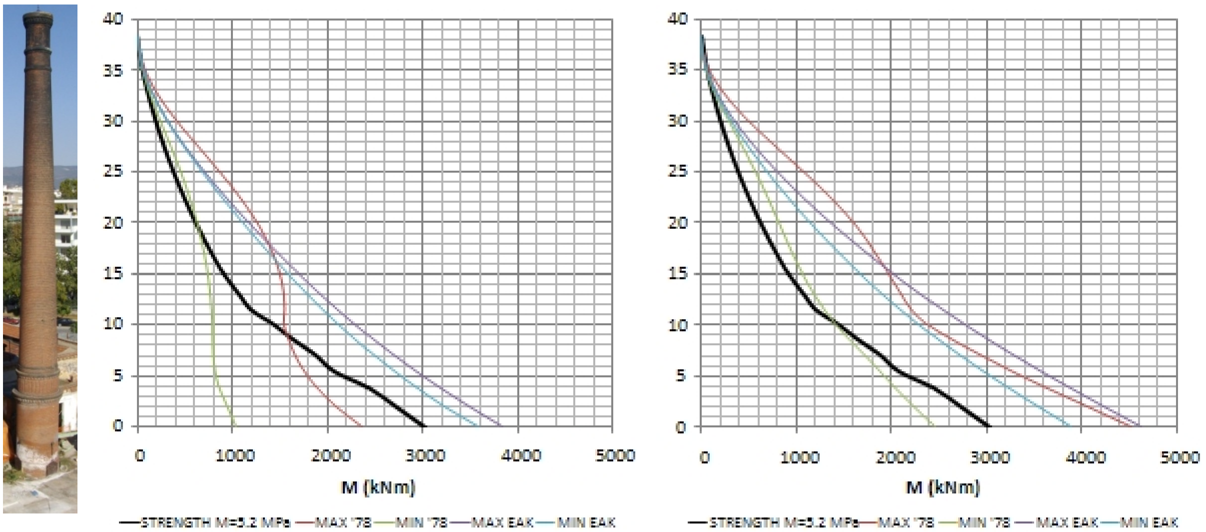

Figure 8: Envelopes of maximum and minimum bending moments for dynamic time history (1978 record) and spectrum analysis (EAK). Strength equal to 5.2 MPa. Flexible (left) and fixed (right) support conditions.

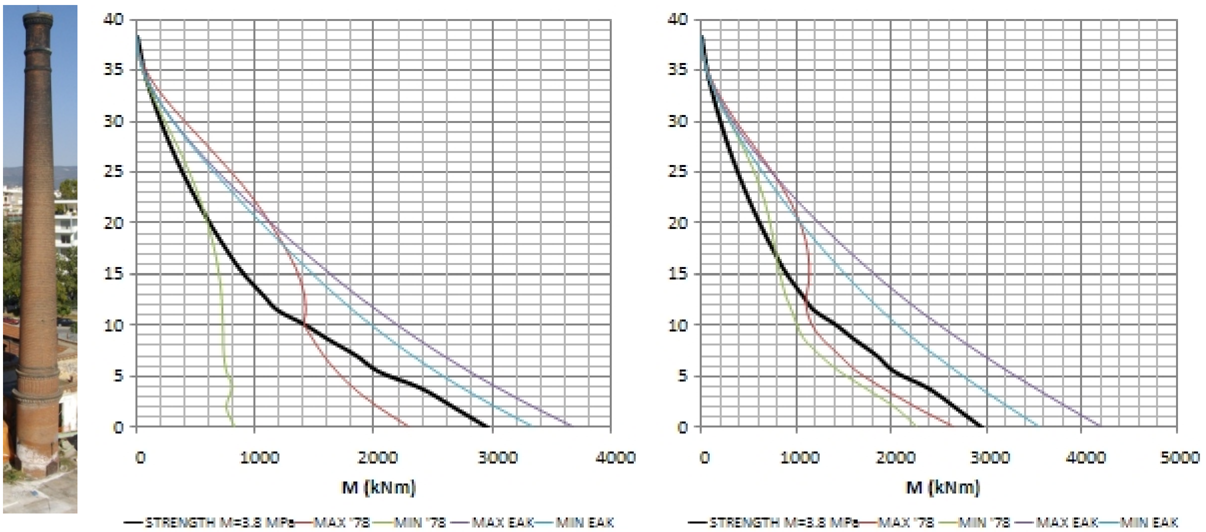

Figure 9: Envelopes of maximum and minimum bending moments for dynamic time history (1978 record) and spectrum analysis (EAK). Strength equal to 3.8 MPa. Flexible (left) and fixed (right) support conditions.

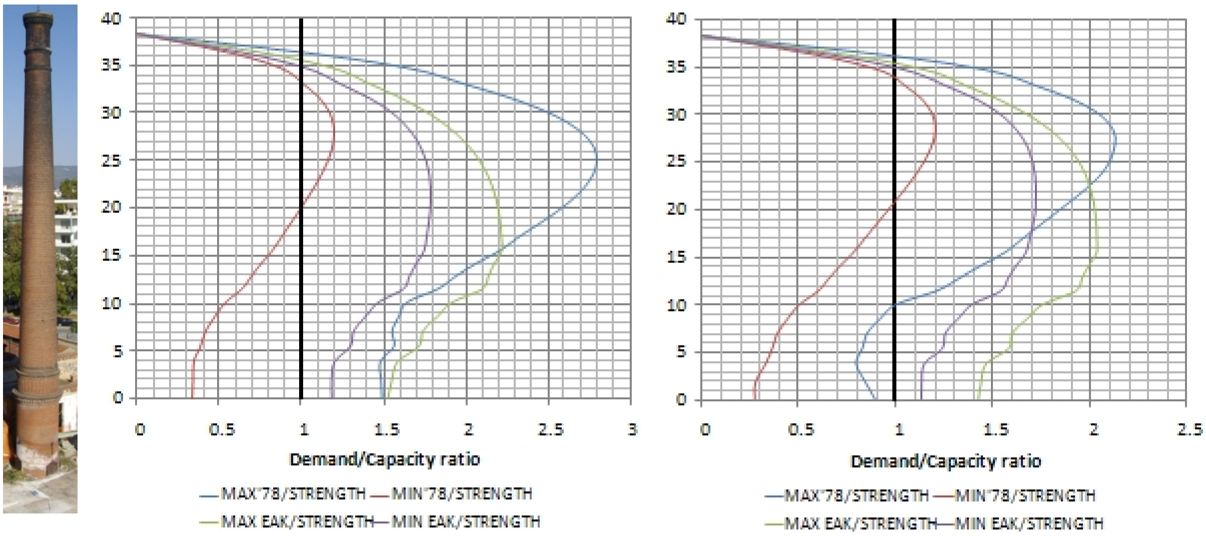

Figure 10: Demand-to-Capacity ratios along the height for dynamic time history (1978 record) and spectrum analysis (EAK). Strength equal to 5.2 MPa (left) and 3.8 MPa (right).

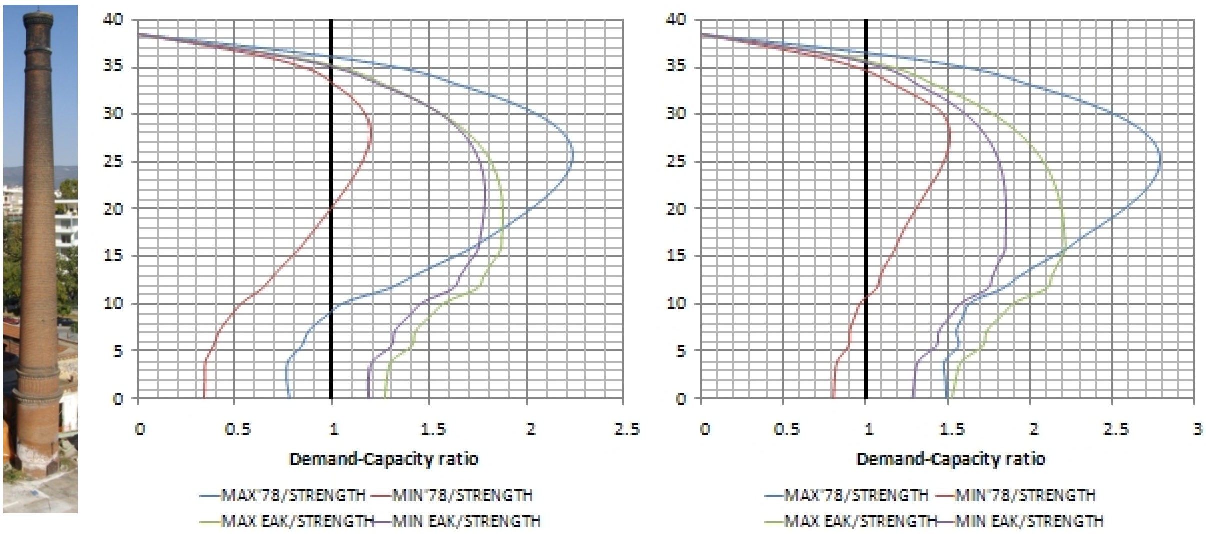

Figure 11: Demand-to-Capacity ratios along the height for dynamic time history (1978 record) and spectrum analysis (EAK). Strength equal to 5.2 MPa. Flexible (left) and fixed (right) support conditions.

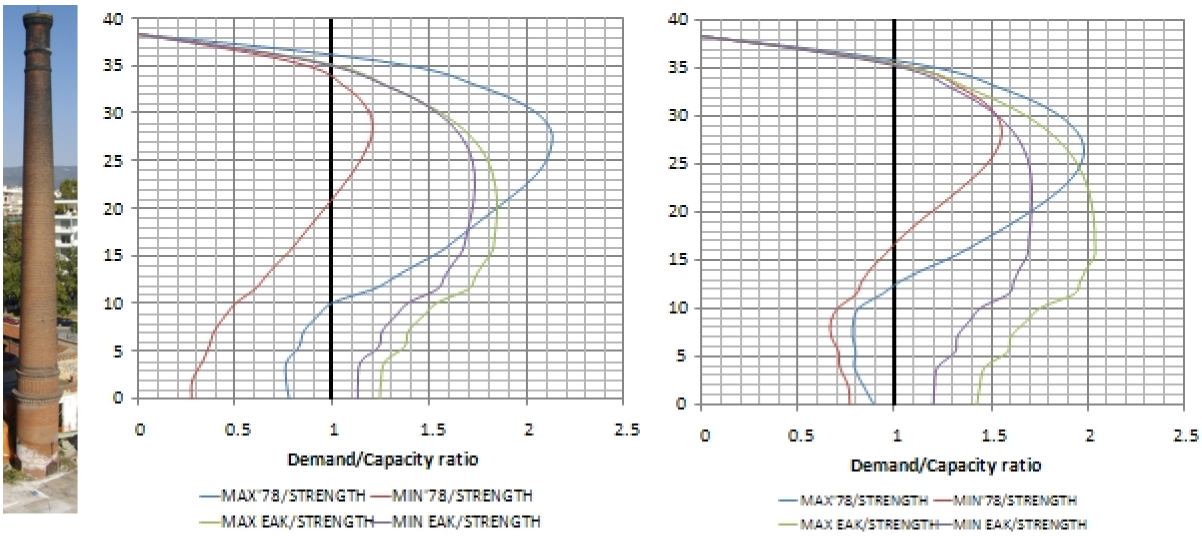

Figure 12: Demand-to-Capacity ratios along the height for dynamic time history (1978 record) and spectrum analysis (EAK). Strength equal to 3.8 MPa. Flexible (left) and fixed (right) support conditions.

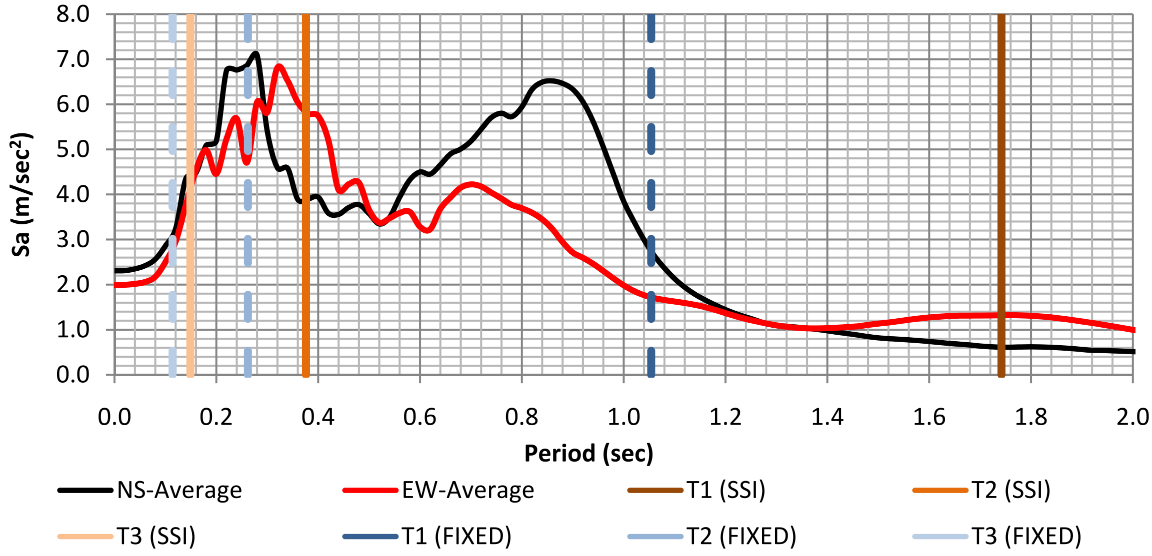

Figure 13: Relation of the mean spectra in the E-W and N-S direction with the first three eigenfrequencies of the fixed base and flexibly supported systems.

6.3 Overall assessment of the structural performance

In order to able to assess the performance of the chimney, a series diagrams (Fig. 7-12) were prepared to illustrate the variation of the envelope of the bending moments (Fig. 7-9) and the corresponding envelopes of the flexural demand-to-capacity ratios (Fig. 10-12) along the height of the chimney for the various earthquake scenarios studied. At first, as can be seen from these figures (i.e. Fig. 7) the range of variation is wide and becomes even wider for lower values of strenth (i.e. for f_{ck} = 3.8MPa), an observation that applies for all analyses, independently of whether they were preformed in the frequency or in the time domain (i.e. 1978 and EAK2000 cases). Moreover, it is also clearly seen from Figures 8 and 9 which refer to structures with common strength equal to f_{ck}=5.2MPa and 3.8MPa respectively, that the parameter which dominates the extent of variation is the flexibility of the soil-foundation system. In fact, in both cases soil compliance leads to wider dispersion of the results and as such, it is deemed to be a crucial issue that has to be thoroughly investigated before being considered in the finite element modelling process. All other parameters that were examined did not present such a decisive influence.

In terms of demand-to-capacity ratio, it is seen from Figure 10 that in the extreme case where the structure is excited at its base with the most critical 1978-compatible earthquake scenario (case: max1978/strength), the ratio at the most critical section which is located at about 2/3 of the overall height, reaches the value of 2.8. This result indicates that indeed, there is (at least one) earthquake scenario and a critical section for which the maximum bending moment developed exceeds the available flexural strength significantlty. It is noticeable that this exceedance gets lower at the base (i.e. is equal to 1.5); however is still greater than the threshold value of 1.0 which corresponds to damage initiation. It is also very interesting to notice that even in the envelope of the minimum in time demand-to-capacity ratio, a value of 1.2 is also observed (although the response as a whole is clearly beneficial, the minimun ratio value being of the order of 0.3). Such high levels of strength exceedence are also observed for the analyses using the Hellenic Seismic Code EAK2000 spectrum, where the maximum ratio was found approximately equal to 2.2) while the same observations can also be made for the case of lower strength of masonry (f_{ck} = 3.8MPa). On the other hand, Figures 11 and 12 reveal that independently of the capacity of the section, consideration of the (more realistic) soil compliance releaves the structure significantly.

7 CONCLUSSIONS

From the parametric study presented for the assessment of the seismic performance of the 35m high chimney of the Allatini complex, the following conclussions were drawn:

- In structures such as the one studied herein, even small changes in the parameters considered (i.e. material properties of soil and structure, geometry of the foundation), can lead to serious discrepancies in their dynamic characteristics. This fact, in conjunction to the equally significant effect of other assumptions related to earthquake input and site response can lead to seismic behaviour of the structure that cannot be easily assessed in advanced, since it depends on the interplay between the structural characteristics of the soil-structure system and the frequency content of incoming earthquake ground motion. As seen from Figure 13, where the mean spectra in the E-W and N-S direction and the first three eigenfrequencies of the fixed base and flexibly supported system respectively are comparatively illustrated, inclusion of soil flexibility shifts all periods to the right and in turn, to generally lower values of spectral acceleration. This phenomenon is further stressed by the fact that as shown in Table 2, in the case of flexible supports, the first three modes activate a significantly larger percentage of the mass hence the structure is less sensitive to higher modes (of shorter periods and possibly related to higher spectral acceleration). On the other hand, it is clear that this conclusion cannot be generalized since it strongly depends on the frequency content of the earthquake. It is just the specific case of this (already) very flexible chimney where it can be claimed that the softer the soil, the lower the seismic response of the soil-structure system would have been.

- Another interesting point is that even the most favourable scenario (see left curves of Fig. 10), although very close, were unable to fully explain the fact that the chimney did not suffer any damage during the 1978 earthquake. It appears that well constructed structrures are able to behave slightly better compared to prediction that can be made even with the most modern computational tools and assessment methods.

- The spectrum analysis according to the Hellenic Seismic Code was found to lead locally to unfavourable results compared to the dynamic excitation with the 1978-earthquake compatible scenarios, a fact which is anticipated given the finding of previous studies and the shape of the corresponding spectra. It is remarkable though that, at higher locations of the chimney, characterized by the highest demand-to-capacity ratios, the results of the time history analysis using the records of the 1978 earthquake were more critical. This can be primarily attributed to the fact that in the long period range the spectral accelerations of the 1978 earthquake scenario are higher. It is also confirmed that, as also stated in many modern seismic codes, the current legislative framework only covers ordinary structures, while for certain specific types of projects additional provisions are required.

- It is deemed that the particular research does not, by any means, exhaust the investigation of such a complex and multi-parametric problem. On the contrary, it is necessary to extend the parametric scheme to other case studies and address additional issues (foundation uplift and section cracking under earthquake loading, effect of the vertical component of ground motion on the axial load and in turn on the capacity of the sections among others) that may also affect the structural response of a masonry chimney.

REFERENCES

[1] G. Riva, Zorgno, A.M., Old brickwork chimneys: structural features and restoration problems, *Structural Studies, Repairs and Maintenance of Historical Buildings IV, Vol 2 Dynamics, Repairs and Restoration,* WIT Press, 1995.

[2] Pallarés, F.J., Agüero, A. & Martin, M., Seismic behaviour of industrial masonry chimneys. *International Journal of Solids and Structures*, **43**(7-8), 2076-2090, 2006.

[3] Pallarés, F.J., Agüero, A. & Ivorra, S., A comparison of different failure criteria in a numerical seismic assessment of an industrial brickwork chimney. *Materials and Structures*, **42**(2), 213-226, 2009.

[4] Vermeltfoort, A.T., Preservation and stability of industrial masonry chimneys. In: *IV International Seminar of Historical on Structural Analysis of Historical Constructions* (SAHC'04). A.A. Balkema Publishers. 2004.

[5] P. Makridis and Associates, *Architectural outline of the Allatini Complex, Technical Report*, Thessaloniki, 2006 (in Greek).

[6] C. Ignatakis, K. Stylianidis, *Final design technical report, evaluation of the current condition of the chimney*, Thessaloniki, 2006 (in Greek).

[7] P. Apostolidis, *Determination of the soil structure using microtremors. Application to the estimation of the dynamic properties and the geometry of the soil formations at Thessaloniki city*, PhD Thesis, Aristotle University of Thessaloniki, Greece, 2002.

[8] A. Anastasiadis., D. Raptakis, K. Pitilakis, Thessaloniki's Detailed Microzoning: Subsurface Structure as basis for Site Response Analysis, *Pure and Applied Geophysics – Pageoph*, **158**(12), 2597-2633, 2001.

[9] CEN, Comité Européen de Normalisation, *Eurocode 6. Design of masonry structures. ENV 1996-1 General Rules*, Brussels, 2004.

[10] C. Stylianidis, *Seismic response of the Allatini complex chimney*, Diploma Thesis, Aristotle University Thessaloniki, 2008 (in Greek).

[11] M. Goldsworthy, J. James, J. Haines, The continuity of active fault systems in Greece, *Geophysics Journal International,* **148**, 596–618.

[12] B. Papazachos, C. Papazachou, *The earthquakes of Greece*, Ziti Publications, Thessaloniki, 1997 (in Greek).

[13] Ministry of Public Works of Greece. Greek Seismic Code – EAK 2000 (amended June 2003), Athens, 2000 (in Greek).

[14] N. Theodulidis, I. Kalogeras, C, Papazachos, V. Karastathis, B. Margaris, C. Papaioannou, A. Skarlatoudis, HEAD 1.0: A Unified HEllenic Accelerogram Database, *Seismological Research Letters*, 75, 36-45, 2004.

[15] B. Margaris, and P. Hatzidimitriou, Source spectral scaling and stress release estimates using strong-motion records in Greece. *Bulletin of the Seismological Society of America,* **92**, 1040-1059.

[16] National Earthquake Hazards Reduction Program (NEHRP), Recommended provisions for seismic regulations for new buildings, *FEMA 222A/223A*, **1,** (*Provisions*) and **2** (*Commentary*), 1994.

[17] CEN, Comité Européen de Normalisation TC250/SC8, *Eurocode 8: Design provisions of structures for earthquake resistance, Part 1: General rules, seismic actions and rules for buildings*, prEN1998-1, Brussels, 2004.

[18] K. Stylianidis, A. Sextos, Back analysis of Thessaloniki Byzantine Land Walls as a means to assess its Seismic History, *International Journal of Architectural Heritage*, 3(4), 1-23.

[19] Computers and Structures Inc., SAP 2000 Nonlinear Version 10. User's Reference Manual, Berkeley, California, 2005.

[20] G. Mylonakis, S. Nikolaou, & G. Gazetas, Footings under seismic loading: analysis and design issues with emphasis on bridge foundations. *Soil Dynamics and Earthquake Engineering*, **26**(9), 824-853, 2006.

[21] A. Sextos, K. Pitilakis, A. Kappos, Inelastic dynamic analysis of RC bridges accounting for spatial variability of ground motion, site effects and soil-structure interaction phenomena. Part 1: Methodology and Analytical tools, *Earthquake Engineering and Structural Dynamics*, **32**(4), 607-627.

[22] S. Kramer, *Geotechnical Earthquake Engineering"*, Prentice Hall, Washington, 1996.