IN-SITU TESTS ON THE INTERNAL WESTERN COLUMNS OF THE PARTHENON AND MODELING OF THE JOINTS’ MECHANICAL PROPERTIES

Eleni-Eva Toumbakari¹, Dimitrios Egglezos²

ABSTRACT

The paper reports a) the in-situ trial load tests, which were carried out on the four, out of six, internal columns of the Parthenon western side, the Opisthodomos, during the recent restoration project (2001-2004) and b) the analytical assessment of their structural response, including foundation, based on the obtained experimental data. The dismantling of the entablature members during the anastylosis project gave the opportunity for the performance of in situ trial load tests in order to study the response of the columns, interacting to their foundation. The testing project consisted in the application of a horizontal force of 10-11kN at the capital of each column. At the same time, the horizontal displacement of the capital and the vertical displacement of the base marble block, were measured. The test was carried out twice in each direction, to account for differences in the foundation conditions. For the evaluation and the interpretation of the test results a back-analysis was performed. For the (3-d nonlinear elastic) analysis Rock-Mechanics principles were applied. The idea is that dry masonry structure, from natural rock stone blocks connected with frictional “joint” forces are analogous to a jointed rockmass system. The comparison of measured values and analytical predictions are generally in good agreement, offering increased accuracy. According to the results, discrete modelling with properly determined parameters for monument’s geomaterials can satisfactorily be applied to restoration analyses of this type of structure and could form the base for more complex calculations involving dynamic effects e.g. earthquake.

Keywords: Heritage, Acropolis, Parthenon, Dry masonry, Joints, In-situ tests, Discrete structure modeling

1. INTRODUCTION

The project described in this paper is part of the actions undertaken during the recent (2001-2004) restoration project for the anastylosis of the Opisthodomos (i.e. the internal colonnade of the western side) of the Parthenon.

1.1. The Parthenon

The Parthenon, the masterpiece of Classical art, was built on the Acropolis of Athens in a short period of time, between the years 447 and 438 B.C by the architects Iktinos and Kallikrates and the sculptor Pheidias who spent 5 years more, to place the sculptures in position on the pediments [1]. The temple, dedicated to Athena, Goddess of Wisdom, still exists, till our days dominating the summit of the Acropolis Hill (Fig. 1).

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1.2. The Opisthodomos
The Opisthodomos, the western part of the temple (Fig. 2), was directly exposed to the disastrous fire in 267 A.D. and experienced the explosion of 1687 – which resulted in the collapse of the eastern and middle part of the building – as well as the various earthquakes that have occurred in the Athens area. Therefore, it bears the signs of a complicated mechanical history. The structural restoration of the Opisthodomos was the main target of the anastylosis between 2001 and 2004 [2]. In the years 2001-2002 the totality of the architrave blocks were removed in order to be repaired, leaving the four northern Opisthodomos columns standing free (the southern two columns are in contact to the medieval staircase masonry). One of the actions undertaken in 2002-2003 was the planning and execution of in-situ trial load tests on those four columns, in order to assess their structural and foundation conditions.

1.3. The Foundation
Parthenon is based partly on the natural limestone of the hill and- due to the slopy ground of the hill- on an artificial stereo-prism consisting of discrete large stone blocks from soft rock material (poros stone) from the Piraeus coast (Fig. 3). The height of the foundation prism is 11 m on the SE edge, 6 m on the SW and 2 m on the NW.
2. TESTING PROJECT

In order to carry out the trial load tests, first a reinforced scaffolding was built and tested, to serve as the reaction frame. A force up to 10 kN was gradually applied at the middle of each side of the capitals by means of a simple scaffolding screw abutted on the reaction frame. The applied force was measured with a load cell and the horizontal displacement of the capital as well as the vertical displacement of the isolated marble block, on which each column stands, were measured with mechanical extensometer and LVDTs respectively [3]. As a total, 30 trial load tests were carried out and the corresponding load-displacement curves of the Opisthodomos columns OK1, OK2, OK3 and OK4 and their foundation were obtained in the 4 directions (to the north, south, east, west).

3. ANALYTICAL EVALUATION AND INTERPRETATION OF TEST RESULTS

For the evaluation and the interpretation of the test results in respect to the foundation, an analytical approach was performed. This analysis was based mainly on Rock-Mechanics principles. The basic idea was that the nature of structural materials, that is, natural rock stone blocks (marble or such) connected with frictional “joint” forces in a dry masonry structure, are fully analogous to a jointed rockmass system. Accordingly, the loaded columns are simulated as a discrete element structure consisting of the column drums and the column capital, with frictional compression-only springs between a) successive drums and b) the column base and the underlying monument base (crepis and foundation blocks) lying upon limestone bedrock.

The sub-base crepis and the underlying discrete foundation (consisting mainly of marly blocks with potential existence of soil intercalations, and locally of dolomite blocks), is simulated as a two layered half space on rigid limestone bedrock. In this case the geomaterials are modeled as an elastic – perfectly plastic material. The typical formation of the opisthodomos column sub-base is presented in the section of Fig. 4 [4]. It is obvious that the column is based on a two layered half space, consisting of the superficial marble crepis and the (much thicker) foundation from marly (or locally dolomite) blocks. The geometry of this layering is differentiated in respect to the direction (west vs east) of the underlying geomaterials, due to the thickness of the marble crepis (0.62m thick to westside – 0.30 m thick to the eastside).
3.1. Geomaterials
The geomaterials involved for analytical interpretation of the results are the following:

- Marble drums and Crepis
- Poros Footing Blocks (marly limestone)
- Dolomite Footing Blocks
- Limestone Bedrock

The footing blocks came from the rocky geological formation of the Piraeus coast, known as Aktite, which appears typically either as a soft rock (marl to marly limestone) or as a dolomite.

<table>
<thead>
<tr>
<th>Table 1 Mechanical properties of Geomaterials</th>
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<td>E (MPa)</td>
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<tr>
<td>Marble</td>
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<tr>
<td>Marl</td>
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<tr>
<td>Dolomite</td>
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<td>Limestone</td>
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</table>

The assignment of mechanical and geostuctural properties to the above materials (Young Modulus E, Poisson ratio ν) is a prerequisite for the conduction of the analytical calculations. Since these materials are classified as rock, the application of GSI method [5-7] applies for the estimation of the rockmass with application of Roclab software code [8]. Indeed, as it has been shown in previous research ([9], [10]), large scale dry masonry structures from discrete blocks (as the case in hand of the large foundation of Parthenon) consisting of rocky blocks, may successfully be simulated as a continuum, applying rock-mechanics principles to fractured rockmass. In addition, this approach may also apply in small scale, for evaluation of isolated structural block properties (accounting for ageing effects and presence of microfracture net). The initial mechanical parameters for application of the GSI method (uniaxial compressive strength qo and E of intact rock), are obtained from relevant literature [9-12], while the value of GSI is based on expertise macroscopic estimation. The initial parameters and the results for rock mass properties are summarized in Table 1.

3.2. Analytical procedure
The comparison of experimental measurements to analytical predictions allows for a) calibration of geomaterial parameters and, b) interpretation of experimental results on the basis of geotechnical conditions prevailing below the base of the columns. Since the horizontal loading of Opisthodomos columns was kept in relatively low levels, the unloading of the column was fully reversible. Thus, a nonlinear elastic approach is justified for prediction, according to the following steps:

- Application of elastic solutions, for calculation of (elastic) settlements/subsidence at the measurement points
- Assignment of geostuctural properties to proper complex springs, for modeling the contact interfaces of the column drums discrete structure (interface springs) and the behavior of the foundation underneath (base springs).
- 3-D nonlinear elastic analysis with use of Poros Footing Blocks (either marly limestone or dolomite)

The presentation of the above analyses is made in the following paragraphs.

3.3. Elastic solutions for calculations of settlements /subsidence
For calculation of settlements/subsidence of a layered halfspace, uniformly loaded on circular or rectangular superficial area, the following well established analytical solutions were applied:

1. According to Winterkorn and Fang [12,13]

\[ s = \frac{c_d qB(1-\nu^2)}{E} \]  

2. According to Steinbrenner [13, 14]

\[ s = qB \frac{1-\nu^2}{E} f(I_1, I_2, I_F) \]  

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In the above formulae, $c_d$ accounts for the geometry of foundation, $B$ = diameter of circular areas or, width of rectangular ones and $l_i$ = factors depending on geometry.

As far as the geometry of loaded surface is concerned, two cases are examined:

- uniform press on circular superficial footing of diameter equal to that of the Opisthodemos column base ($D \approx 1.71$ m)
- uniform press on a square surface with area equal to half the circular area of the Opisthodemos column base, in order to account for the eccentric vertical loading $P_v$ of the column base, arising from the horizontal thrust of the column capital (couple of forces $\pm P_n$, $P_v \approx 100$ kN).

Finally, for each column (OK1 – OK4) four layered profiles were considered (tot. 16 profiles)

- Two plate thick marble (0.62 m) on marly halfspace (foundation) – referring to the west side of the column base
- One plate thick marble (0.30 m) on marly halfspace (foundation) – referring to the east side of the column base
- Two plate thick marble (0.62 m) on dolomite halfspace (foundation) – referring to the west side of the column base
- One plate thick marble (0.30 m) on dolomite halfspace (foundation) – referring to the west side of the column base

The limestone bedrock of the rockhill is considered rigid, since it appears in a practically non affected from the loaded surface depth.

**Table 2 Base Springs (MN/m^3)**

<table>
<thead>
<tr>
<th></th>
<th>Porous Blocks</th>
<th>Dolomite Blocks</th>
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</thead>
<tbody>
<tr>
<td></td>
<td>East</td>
<td>West</td>
</tr>
<tr>
<td>OK-1</td>
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<tr>
<td>OK-4</td>
<td>708</td>
<td>1223</td>
</tr>
</tbody>
</table>

Totally, 64 parametric analyses were performed (2 methods × 2 loaded areas × 16 geotechnical profiles). Indicative results of the analyses (values of base springs for sq. surface, according to Winterkorn & Fang) are summarized in Table 2.

**3.4. Geostuctural parameters (spring constants)**

The mechanical properties of geomaterials and the analytical results from subsidence calculations are used for the estimation of proper spring constants, accounting for the simulation of contact surface between the drums and subbase foundation, respectively.

**3.4.1. Interface (contact) Springs**

The interface springs (12 peripheral and one central) per unit area, are calculated according to

$$k_n = \frac{E_i}{h_o}, k_s = k_n / 10$$

where $k_n$ represents compression-only properties and $k_s$ frictional behavior.

In the above formulae, the Young Modulus $E_i$ refers to a notional loose geomaterial, filling the joints between drums, while $h_o = 1$ mm, is the nominal joint opening. These springs are considered to have constant unit area value for all the peripheral and the central spring respectively. Two sets of contact springs are examined for $E_i = 15000$ kN/m² and $E_i = 7500$ kN/m² respectively.

**3.4.2. (sub)Base Springs**

The sub-base springs are evaluated according to the definition formula:

$$k_v = \frac{q}{s_v}, k_s = k_v / 10$$

Where, $q$ = contact press on the loaded area of the column base and $s_v$ = the relevant subsidence, on the assumption of rigid base.
3.4.3. 3-D Analyses

The results of the previous paragraph (spring values) are utilized for nonlinear elastic 3-D analyses. The modeling comprises the exact (idealized geometry) of the columns, with contact springs between the drums and base springs for modeling of the sub-base foundation (Fig. 5). The 3-D analyses results are more or less consistent to the measured values. Totally, 96 analyses were performed, corresponding to a) different base springs, arising from the elastic calculations of the 3.1 paragraph, b) difference in geotechnical subbase conditions (marly or dolomite blocks) and c) difference in contact spring constants. The results of calculated horizontal drift and subsidence from the 3-D analyses are summarized in Table 3.

**Table 3 Measurements vs Predictions**

<table>
<thead>
<tr>
<th>MEASUREMENTS</th>
<th>( \delta_{0} (\text{mm}) )</th>
<th>( s_{0}(\mu\text{m}) )</th>
<th>( E-W^2 )</th>
<th>( W-E^2 )</th>
<th>( N-S^2 )</th>
<th>( S-N^2 )</th>
<th>( E-W^2 )</th>
<th>( W-E^2 )</th>
<th>( N-S^2 )</th>
<th>( S-N^2 )</th>
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<td>(13,17,22)</td>
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<td>PREDICTIONS (elastic solutions)</td>
<td>( \delta_{0} (\text{mm}) )</td>
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<td>PREDICTIONS (3-D analyses)</td>
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(1) Winterkorn & Fang (equiv. square to half circular base), (2) loading direction, (3) extrapolation for \( P_s=10\text{kN} \), (4) dolomite foundation, (5) for \( P_s=3,4,6\text{ kN} \) respectively.
From, the results it is obvious that the best simulation corresponds a) to subbase springs arising from a uniform loaded square with area equal to that of the half circular column base, for the elastic solution according to Winterkorn and Fang [11] and b) to contact springs for notional infill material of $E_i = 7500 \text{ kN/m}^3$. Indicative comparison between measurements and analytical predictions for column OK-4 are shown in Fig. 6.

![Fig. 6 Comparison between measurements and analytical predictions for column OK-4 (loading direction: S to N)](image)

4. CONCLUSIONS

The main general conclusions arising from this research can be summarized as follows:

- The large scale non-destructive testing is invaluable for the interpretation of the structural behavior of monuments.
- The performance of loaded dry masonry structures is simulated satisfactorily with the application of rock-mechanics principles.
- The proper analytical procedures supported by experimental results offer a stiff frame for successful estimation of the essential mechanical parameters of monuments’ geomaterials.

In addition, the main findings from the interpretation of the experimental results comprise the following:

- The columns horizontal drift is governed mainly by the mechanical properties of contact surfaces.
- The subsidence depends mainly on the sub-base foundation material properties. The assumption that the foundation is made from soft rock marly limestone blocks interprets the relatively high subsidence measurements. On the other hand the rather limited number of low value subsidence measurements may be attributed to the local presence of dolomite foundation blocks.

The practical “equal” drift and the settlements that have been measured on columns OK-3 and OK-4 verify that the effect of foundation thickness is limited to ca. 5-6 m below the Opisthodomos column base.

Finally, according to the results, the modeling of discrete structures with properly determined parameters for monument’s geomaterials, could satisfactorily be applied to restoration analyses and could form the base for more complex calculations involving dynamic effects e.g. earthquake. The comparison of measured values and analytical predictions are generally in good agreement, offering increased accuracy. The proposed modeling approach, supported by the carefully planned and executed trial load procedure, offers a well-founded basis for the analysis of the highly complex and demanding issues of classical monuments’ restoration.

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REFERENCES


