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NUMERICAL INVESTIGATION OF THE SEISMIC BEHAVIOUR OF CONNECTIONS OF ANCIENT COLONNADES

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Abstract. *As a common practice, restoration projects of ancient colonnades have to deal with joining together fragments of architectural members using threaded titanium bars (reinforcement) fixed into place with cement mortar. The basic criterion for the design of such connections is that, in case of a seismic event, the reinforcement should absorb the seismic energy and fail before the marble suffers any damage. For the dimensioning of the titanium bars, a methodology based on the capacity design philosophy is usually implemented. The accurate calculation of the forces that will be induced to the reinforcement during an earthquake is not an easy task, since the response of the structure is governed by the rocking and the sliding of the individual stone blocks.*

In this paper, the efficiency of the reinforcement used for the connection of complements, which has been calculated with the above-mentioned capacity design methodology, is investigated for selected severe seismic excitations. The analyses were performed for the case study of the restoration of the colonnades of the Southern part of the Ancient Agora of Kos in Greece. The induced forces were calculated by dynamic analyses, using an accurate numerical description of the restoration's structural scheme and earthquake motions of various characteristics, selected to be compatible with the seismological history and the soil conditions at the site. All the analyses were performed using the code 3DEC, which is based on the distinct element method and has been verified and calibrated by comparison of the numerical results with experimental data. The results show that the simplified design that is applied in practice is adequate, as the stresses induced to the reinforcement bars were always less than the ultimate strength and, in many cases, significantly less than the yield resistance as well.

1 INTRODUCTION

Ancient colonnades consist of stone blocks of different sizes and shapes made of marble, stiff limestone or porous stone, depending on the available material in the nearby region. Typically, the blocks are not connected to each other and the structure behaves as a system of discrete blocks, except of connectors (clamps and dowels) that are provided in certain places only. In current restoration practice, ancient mortises that are preserved in such places are used to connect the stone blocks with new clamps and dowels made of titanium. The basic principle that is followed for the design of the new connectors is that, in case of a seismic event, the connectors should absorb the seismic energy and fail before the surrounding marble suffers any damage.

Apart from the connectors, the use of titanium bars is also common for joining together fragmented ancient blocks or fragments of blocks with new complements so as to restore the unity of each discrete element of the ancient structure. The principle in designing the bars that are used as reinforcement is that those should bear the induced forces in a seismic event and maintain the discrete block as a whole, while the marble does not suffer any damage.

In general, the design of the restoration anticipates the following sequence of response: The joints between independent blocks are the first to be activated. This does not necessarily imply engagement of the connectors (clamps and dowels), because, typically, there is a gap between them and the mortises of the stones. However, forces are induced to the connections between blocks and new complements. When the movements of the blocks exceed certain values, the clamps and dowels are activated, reducing, in general, the forces applied to the restored interfaces of fracture. After the failure of the connectors (clamps and dowels), it is possible that rehabilitated members of the structure lose their integrity; in that event, the titanium bars of the reinforcement should yield prior to any other damage to the marble.

In order to follow the above-mentioned procedure, one should know the forces that will be induced to the connectors and the reinforcement during an earthquake. However, the calculation of these forces is not an easy task, since the response of the structure is governed by rocking and sliding of the individual stone blocks. Previous investigations [1-6] on the dynamic behaviour of single freestanding columns and sub-assemblages of ancient temples have pointed out that the response of these discrete structures is highly nonlinear and very sensitive to even small changes in the parameters. Thus, the imposed excitation and the frequency content of the ground motion, the degree of the accuracy of the numerical model concerning the geometry of the structure and the assumptions adopted in the analysis (joint properties, friction coefficient, etc.) may affect significantly the results of even rigorous nonlinear analyses. For this reason, the dynamic analyses of such structures contain an inherent uncertainty and their outcomes should be used with caution.

In practice, simplified analyses are usually applied for the design of the connections that are implemented during interventions. These analyses are based on the capacity design philosophy, thus they end up with the maximum forces that can be developed theoretically, independently of the earthquake excitation. In this paper, the response of connections of complements, which have been designed by such methodologies, under strong earthquake excitations, is investigated. For this purpose, nonlinear numerical analyses are performed using the discrete element method and the forces induced to the reinforcements are compared with their strength. The analyses are performed for the case study of the restoration of the colonnades of the Southern part of the Ancient Agora of Kos in Greece.

2 PRINCIPLES OF INTERVENTION

2.1 General philosophy

Restoration projects nowadays follow very specific guidelines in order to ascertain the required quality of the intervention. The main scope is the minimum, yet necessary and sufficient, intervention in the monument's inherent characteristics. Of main importance is the respect for the original building techniques, the original structural system and the original materials. Authenticity is a beyond debate concern and goal of the project, in order to maintain the monument at the best possible status and to minimize the alterations. Additional requirements might be reversibility, meaning the ability to revert the monument to its previous - before the restoration- state, maintenance of the structural function and consistency of the individual architectural members.

The use of new material for the complement of missing parts of structural elements is generally restricted to the absolutely necessary and must be kept in a low proportion compared to the original material. Such decisions must not be based only on stability issues, but also take under consideration the forms and volumes, the visitor's perception of the monument and aesthetic issues. It should be kept in mind that the main "recipients" of the monuments are their visitors and that the cultural heritage that they carry is not addressed to scholars and connoisseurs only, but mainly to the public.

2.2 Structural restoration

The term ‘structural restoration’ signifies the series of interventions that are necessary to ensure the bearing capacity of the structure and of its individual parts. To this end, restoration of the connectors between the structural elements and re-composition of the original geometry of the stone pieces that were retrieved during the excavation by connecting fragments and/or complement of stone elements is deemed necessary in many cases.

In antiquity, the connection elements were made of bronze or iron and were covered with lead, cast in the mortises, which after its congelation offered high insulation to the metallic connectors, protecting them from the oxidization and the corrosion. At the same time, the connectors, being ductile materials, contributed to the overall behaviour of the structure in case of an earthquake. Two types of connectors were used: dowels, which connected elements between consequent layers along the height and resisted the shear forces; and clamps, which connected stones belonging to the same layer and prevented their relative dislocation through their tensile resistance. In the ultimate limit state, the dowels and the clamps were meant to fail before the failure of the stone.

In many restorations realized in the 19th ant the 20th century, the structural steel that was used in typical constructions was applied also to monuments for the enhancement of their bearing capacity and for the connection of fragments of architectural members. The steel elements were usually cast in lead, as a follow up of the ancient practice. Cement mortars were widely used for covering mass lacks. This technique caused significant damage to the monuments, because the cast lead failed to reassure the same impermeability as the ancient one and environmental actions, due to their intense corrosive character, led to iron's oxidization and subsequently to fracture of the architectural members.

In modern restoration projects, mostly inorganic materials are adopted, in order not to provoke any additional problems in the long term. Thus, for the connections between restored members of the monument (clamps and dowels), specially formed titanium sheets are commonly used, fixed in place with the use of inorganic mortars. Similarly, for joining together fragments and/or complements, threaded titanium bars are applied.

2.3 Calculation of the reinforcement of the connections

The ancient and the new pieces are typically connected with titanium threaded bars that are inserted in properly drilled holes and fixed into place by mortar. Mortar is also used as the bonding material at the interface of the fraction. As mentioned above, a proper dimensioning of the reinforcement would require difficult nonlinear analyses, which are seldom performed in practice. Usually, analyses based on capacity design effects are performed [7, 8], which lead to the required reinforcement for resisting the maximum forces that can be induced, without restoring completely the strength of the original material. In example, the design of the connection of fractured architraves is based on the assumption that the architrave is subjected mainly to bending under increased gravity loads by a factor around 1.50; for the connection of fragments at column drums and stylobate blocks, the required reinforcement is calculated from equilibrium conditions under capacity actions that include the friction forces, assuming that sliding occurs at the joint, and the ultimate resistance of any existing dowels.

3 CASE STUDY – DESCRIPTION OF THE MONUMENT

The analyses presented herein are based on the part of the southern Arcade (Stoa) of the Ancient Agora in the island of Kos in Greece that has been proposed to be restored. The monument is situated in the center of the modern city of Kos.

An unexpectedly large number of structural members were found in situ. The location where the members were found and the study of the mortises of the connectors confirmed that they derived from specific parts of the building. Thus, the 'erection' of a small part of the Stoa was proposed, using a significant portion of the found ancient members.

The restoration project concerns three columns of the Stoa with the respective parts of the crepis and the entablature (Figure 1). In this restoration, 37 from the 62 ancient members are to be used. In addition, seven new blocks are to be used to ensure the stability of the structure and, also, for aesthetic reasons.

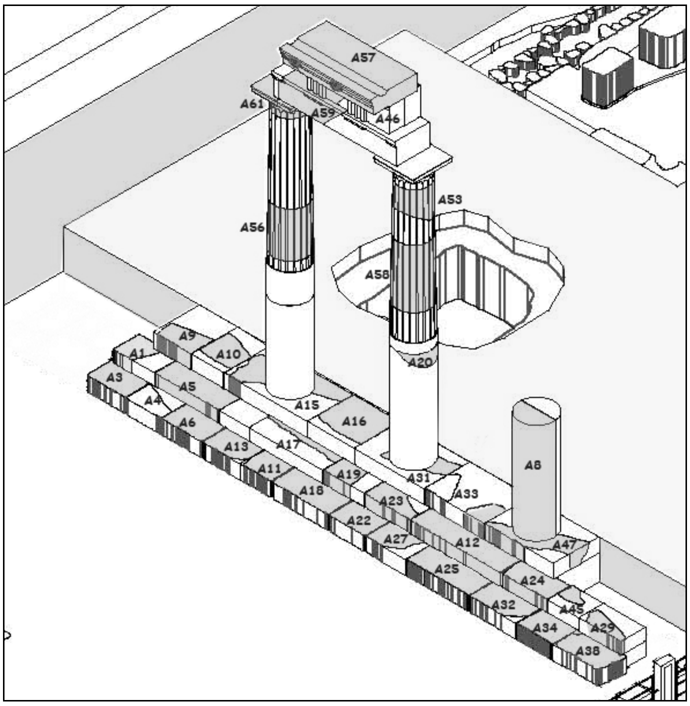

Figure 1: Drawing of the restoration proposal. The ancient members and fragments are shown in gray and the members and complements that will be made of new marble are shown in white.

The original structure rested on a two layered base of height 0.55 m, made of porous blocks that lied under the crepidoma (crepis). The crepis consisted of two steps and the stylobate. The first step had height 0.33 m and was made of gray limestone blocks of varying plan dimensions (their length was varying from 0.60 to 1.40 m and their width from 0.50 to 0.70 m). The second step had height 0.29 m and was made of marble blocks of the same overall dimensions. The stylobate was made of marble blocks of height 0.30 m, width about 1.00 m and varying length.

The marble columns of the Stoa were of doric style without fluting at their lower part, up to a height of 2.07 m. The columns consisted of four drums of uneven height with base diameter 0.78 m. The axial distance between the columns was 2.66 m and their overall height was 5.61 m. The diameter of the capital was 0.635 m and its height was 0.38 m, approximately, while the abacus had plan dimensions 0.85 m × 0.85 m and height equal to 0.11 m.

The architraves consisted of single blocks, 2.66 m in length, 0.71m in width and 0.47 m in height. The frieze was made of blocks 1.73 m in length and 0.59 m in height that included two triglyphs and two metopes. Those blocks were either single of full width (~0.47 m) or were supplemented by other blocks of approximately the same dimensions that completed the width of the layer; the latter is the case of the part of the structure that is considered in this analysis. The cornice had height 0.42 m and was projecting 0.325 m. The block that will be used in the restoration is 1.95 m long.

4 NUMERICAL ANALYSIS

4.1 General

The structural and the dynamic analysis of ancient temples or sub-assemblages of ancient temples differ significantly from the analysis carried out for modern structures, mainly because of their articulate construction. During a seismic event, rocking and/or sliding of the stones, independently or in groups, may occur, which results in highly nonlinear behaviour [1-6]. Additionally, the response is very sensitive to the details of the geometry, the characteristics of the ground motion and the joint parameters.

The complexity and the special character of the response of the structure (rocking and sliding) create computational requirements hard to meet with the incorporation of conventional software. For the numerical analyses presented herein, the code 3DEC by Itasca Consulting Group, Inc. [9] was employed, which is based on the discrete element method. The code is designed to allow significant displacements and rotations of the blocks, even total detachment. During the calculation process, the code locates the contacts between the blocks and computes the motion of each block from the forces (normal and shear) that are developed at the joints. The contacts are divided in sub-surfaces, while various types of contact are considered (apex to apex, apex to edge etc.). In this way, rocking and sliding are accurately addressed.

The code 3DEC has been verified and calibrated for the response of ancient colonnades through comparisons of the numerical results with experimental data obtained from shaking table tests performed at the Laboratory for Earthquake Engineering of the National Technical University of Athens [2, 4-6].

4.2 Numerical model

The numerical model used in the analyses was based on the actual restoration proposal. The connections between the ancient fragments and the new complements were assumed from titanium bars and were designed according to the above-mentioned methodology. The exact geometry and dimensions of the reinforcement were implemented in the numerical model (Figure 2).

The mortar typically used at the interfaces of the connected blocks was not considered, because its actual mechanical properties cannot be precisely defined. In addition, the dowels and the clamps were not included in the numerical model, because sliding can occur before these connectors are activated, since a gap of 1 to 2 cm, not filled with mortar, is commonly left between them and the edge of the mortises. Both assumptions are to the safety side and lead to the upper limit of the forces that can be induced to the reinforcement.

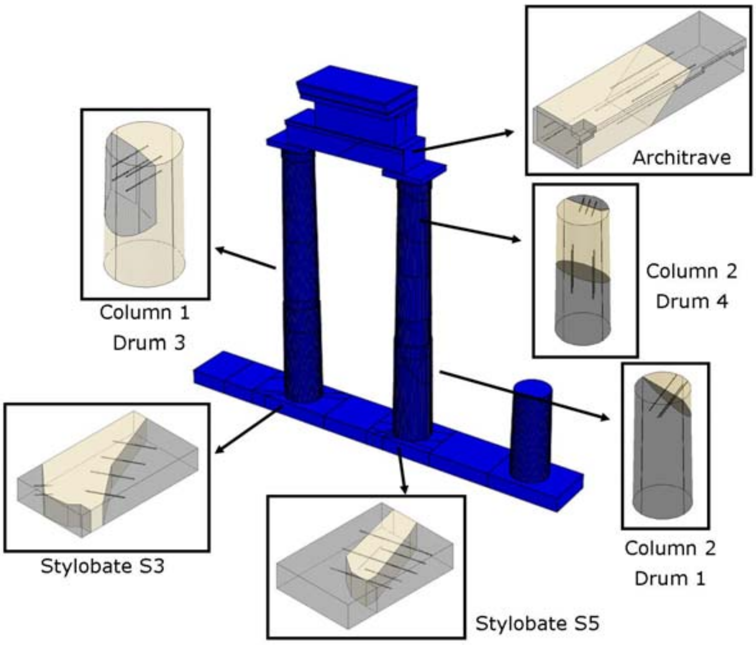

Figure 2: Numerical model used in the analyses with 3DEC. Details of the connections considered in the model are also shown.

The analyses were performed assuming that all the structural elements are rigid blocks. For rigid blocks problems, the 3DEC code gives an accurate contact formulation in which the interaction takes place at a number of contact points [9]. Each contact is assigned a contact area, which is used to calculate the local point stiffness, in terms of the user-defined stiffness of the discontinuity surface. The joint stiffness used in the model was based on former studies [2, 4-6] and was equal to 5×10^9 Pa/m in the normal direction and 1×10^9 Pa/m in the tangential direction. A 10% mass-proportional damping at ω = 0.3 Hz was also considered. The friction coefficient was taken equal to 0.75.

The reinforcement (titanium bars) were simulated as nonlinear springs for which the elastic stiffness, the yield force and the ultimate strain were assigned in both the axial and the shear directions. Since pullout test results were not available, the following theoretical expressions, given in [10] and proposed by 3DEC [9], were used to estimate the axial stiffness, K_a and the shear stiffness, K_s:

$$K_\mathrm{a} = \pi \cdot k \cdot d_1 \tag{1}$$

$$K_\mathrm{s} = E_\mathrm{b} \cdot I \cdot \beta^3 \tag{2}$$

where d_1 is the diameter of the reinforcement; $k = \left[\frac{1}{2} \cdot G_g \cdot E_b / (d_2 / d_1 - 1) \right]^{1/2}$; G_g is the shear modulus of the grout; E_b is the Young's modulus of the reinforcement material; d_2 is the diameter of the hole; I is the second moment of area of the reinforcement element; and $\beta = \left[K / (4 \cdot E_b \cdot I) \right]^{1/4}$, with $K = 2 \cdot E_g / (d_2 / d_1 - 1)$.

The ultimate axial strength, P_{ult}, and the shear strength, $F_{s,b}^{max}$, of the titanium bars were calculated using the formulas proposed in [11] and [12], respectively, and adopted by 3DEC [9]:

$$P_{ult} = 0.1 \cdot \sigma_c \cdot \pi \cdot d_2 \cdot L \tag{3}$$

$$F_{s,b}^{max} = 0.67 \cdot d_1^2 \cdot \left(\sigma_b \cdot \sigma_c \right)^{1/2} \tag{4}$$

where σ_c is the uniaxial compression strength of the marble (up to a maximum value of 42 MPa); L is the bond length; and σ_b is the yield strength of the reinforcement.

4.3 Selection of base motions

For the selection of the base motions, the geological and seismological data in the area of the monument were taken under consideration. The basic ground formation in the area, where the ancient Agora is situated, is alluvial coastal depositions with significantly low mechanical properties and poor geotechnical behaviour [13-15].

There are many references of large historic earthquakes in the wider area of Kos, with magnitudes up to M = 6.5 [13-16]. During these earthquakes, ground raptures, liquefaction and collapse of many structures were reported. It should be pointed out that part of the ancient Agora collapsed in the historic earthquake of 556 A.D. [13]. In Figure 3, the map of the geological hazard of the island of Kos, according to [13], is presented. Note that a normal fault is located in a distance less than 6 kilometers from the area under consideration.

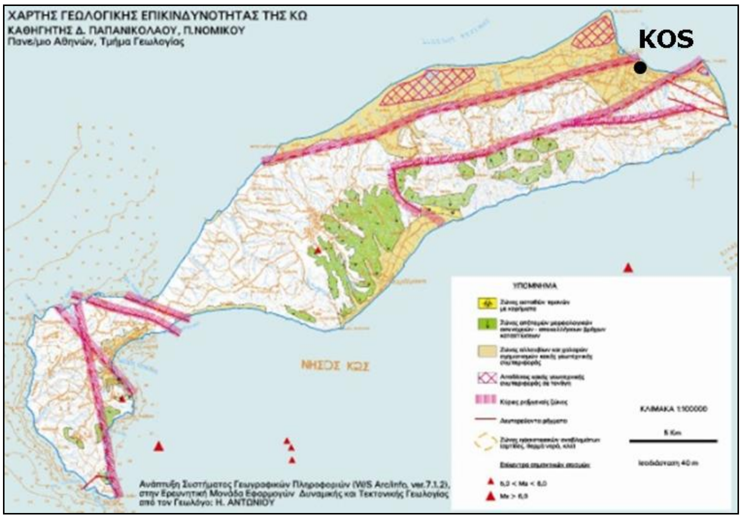

Figure 3: Seismological map of the island of Kos, showing the main faults (reproduced from [13]).

For the selection of the earthquake records to be used in the analyses, the above-mentioned seismotectonic data and the ground profile of the Ancient Agora of Kos were considered. More specifically, the following criteria were posed: adjacency to the seismic fault (near field earthquakes); magnitude around 6.5; and ground motions recorded on soft soil conditions.

Based on these criteria, six earthquake records were selected from the strong motion data bases: Cosmos Virtual Center; Pacific Earthquake Engineering Research Center (PEER); European Strong-Motion Database (ESD); National Observatory of Athens; Institute of Technical Seismology and Earthquake Resistant Structures (ITSAK), which had different characteristics and covered a wide range of spectral accelerations. Both horizontal components of each earthquake were applied as the base excitation. The selected earthquakes and their characteristics are shown in Table 1.

From the analyses performed, it was found that collapse of the structure occurs for the Sylmar, Northridge, USA earthquake. The collapse mechanism is shown in Figure 4. Since the main scope of this research was to determine the behaviour of the reinforcement assuming that collapse does not occur, this record was not further examined. The response spectra of the component that was applied in the transverse direction of the structure of the five remaining records are presented in Figure 5.

Location	Date	Fault Mechanism	Magnitude M_w	Soil Type	Distance from fault (km)
Northridge, USA/ Sylmar County Hospital	17/01/94	SS	6.7	SL	5.6
Imperial Valley, CA/ Aeropuerto Mexicali St.	15/10/79	SS	6.5	SL	0.34
Aigion, Greece	15/06/95	NM	6.3	SL or SR	6.0
Kalamata, Greece	13/09/86	NM	6.0	SL or SR	10.0
Lefkada, Greece	14/08/03	SS	5.9	SL	4.8
Athens, Greece/ Sepolia	7/09/99	NM	5.9	SL	17.0

Table 1: Selected earthquake excitations. The record in red leads the structure to collapse.

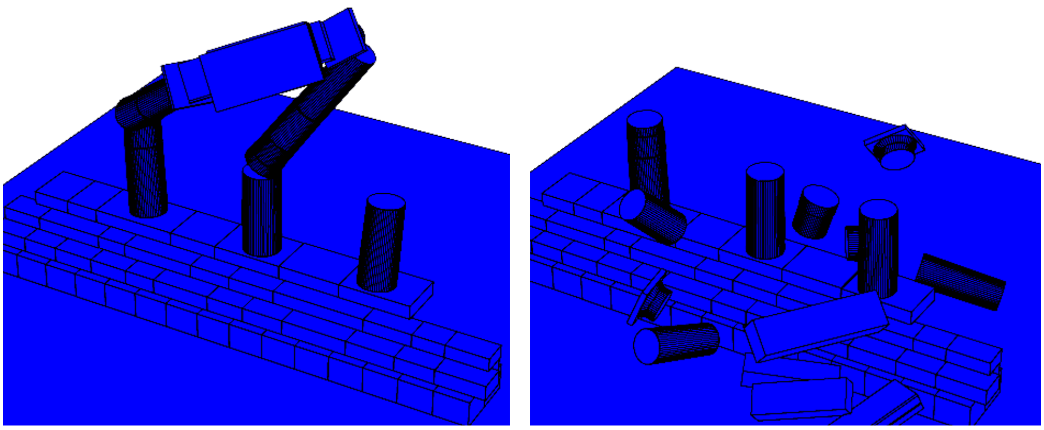

Figure 4. Collapse mechanism of the structure for the Sylmar, Northridge, USA record.

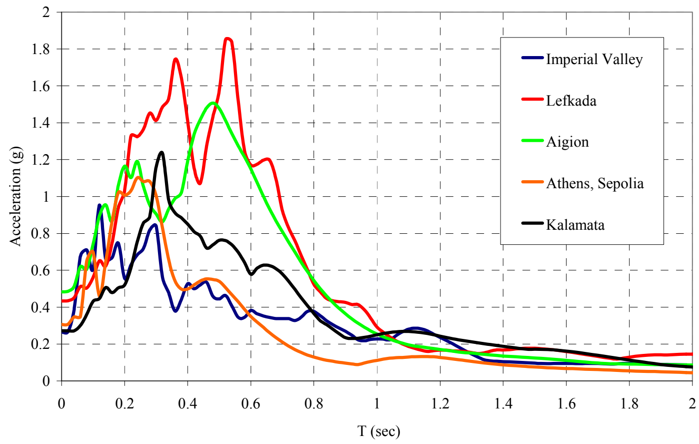

Figure 5: Acceleration response spectra for 5% damping of the component of the base motion that was applied in the transverse direction of the structure.

5 RESULTS

For all the titanium bars that were used in the connections, the time histories of the axial and the shear forces were obtained from the analyses. Indicative results are shown in Figure 6, where the time histories of the axial force of the four reinforcement bars that were used to connect the two fragmented pieces of the architrave beam are depicted.

From these results, the efficiency factor for each bar was derived as the ratio of the maximum uniaxial stress, $\sigma_{M,max}$, that was developed during each earthquake motion over the yield stress, σ_y. The uniaxial stress was calculated using the Von Mises yield criterion:

$$\sigma_M = \sqrt{\sigma^2 + 3 \cdot \tau^2} \tag{5}$$

where σ is the axial stress of the bar and τ is the shear stress. The yield stress σ_y and ultimate stress σ_u of the threaded titanium bars were taken equal to 330 MPa and 420 MPa, respectively.

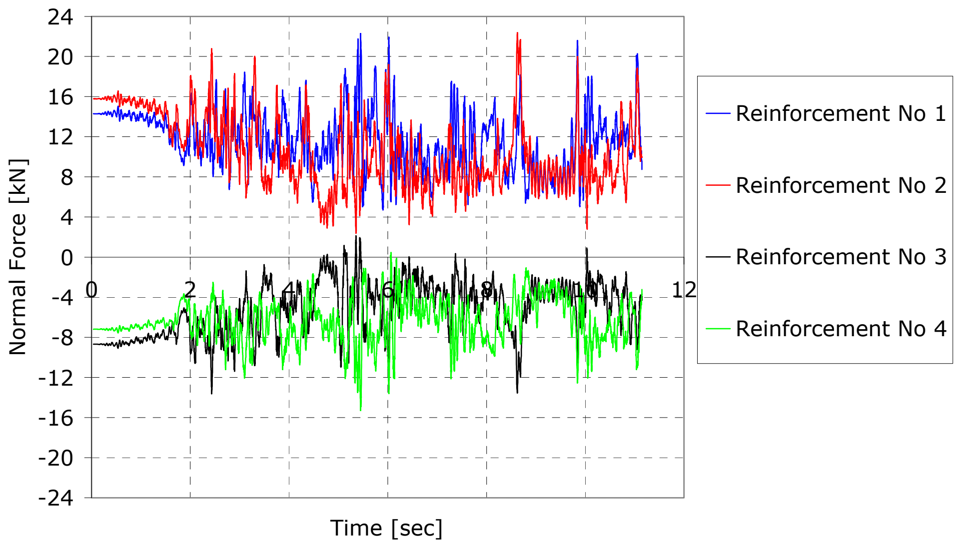

Figure 6: Time histories of the axial force of the 4 titanium bars of the architrave connection (Figure 5) for the Imperial Valley earthquake.

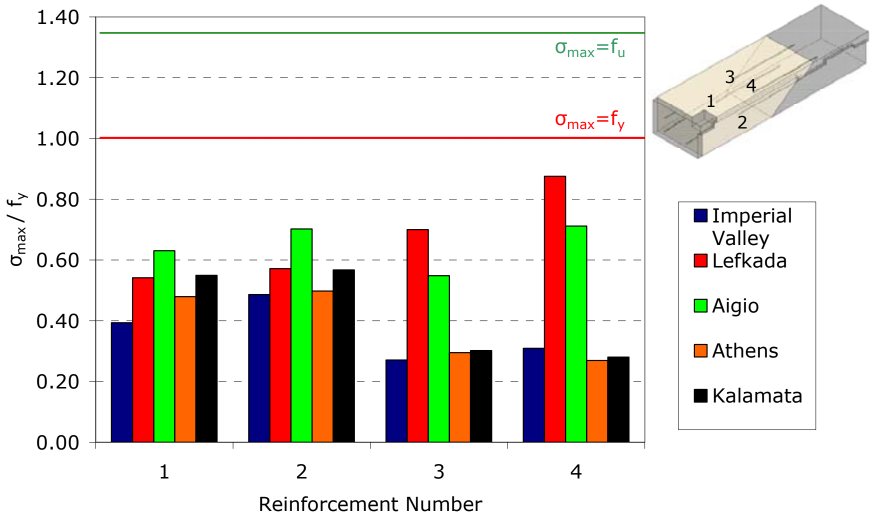

Figure 7: Accomplished efficiency factors of the four titanium bars of the architrave connection for the five earthquake excitations used in the analyses.

Figure 7 shows the accomplished efficiency factor of the four reinforcement bars applied at the connection of the architrave. None of the bars reached its yield strength for the earthquakes examined. The two lower reinforcement bars (No 1 and 2), which were in tension under gravity loads, showed an efficiency factor around 0.60, not much affected by the base motion. This value is close to $1/\gamma_G$, where $\gamma_G = 1.5$ is the load factor for the gravity loads that was used for the design of these reinforcements according to the methodology described above. The upper reinforcement bars (No 3 and 4) were stressed significantly less for three of the earthquake excitations (Imperial Valley, Athens and Kalamata), as expected; however, large stresses were induced to these bars for the Lefkada and the Aigion earthquakes, showing that large displacements occurred at the architrave during these ground motions.

Figures 8 to 10 show the efficiency factors of the reinforcement bars used for the connection of the fragments at the drums of the columns. It can be observed that significantly different stresses were induced to the titanium bars for each type of fragmentation. The worst case was observed for the fragment at the first drum of column 2 (Figure 9), in which two out of three bars yielded in most earthquakes. The smallest forces were induced to the connection of the third drum of column 1 (Figure 8), where efficiency factors less than 0.70 were achieved in most cases. Similar was the behaviour of the titanium bars of the connections at the fourth drum of column 2 (Figure 10), which were stressed significantly lower than their yield limit in most cases. It should be noted that the maximum stress was less than the ultimate limit strength of the reinforcements in all cases.

It is evident from these results that both the inclination of the connection and its position along the height of the column affect significantly the forces that are induced to the reinforcement bars. The worst case concerns fragments that form a wedge and are located at the lower part of the column, where the gravity loads are larger. During rocking, large forces are developed at such connections; this was also observed in [17].

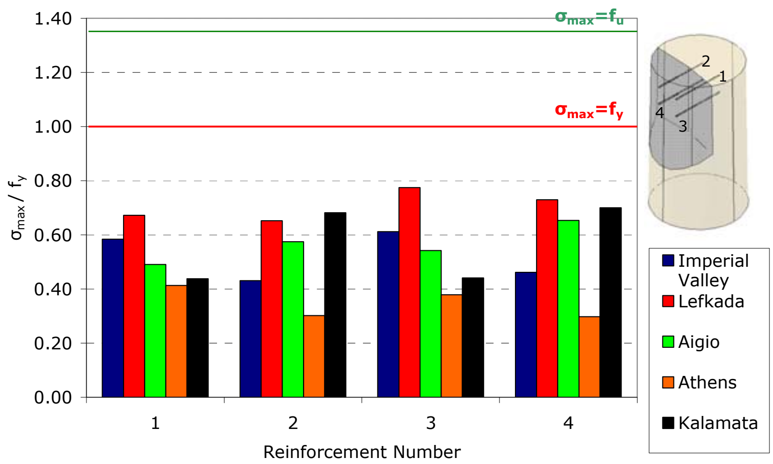

Figure 8: Efficiency factors of the four titanium bars used for the complement of the third drum of Column 1, for the five earthquake excitations used in the analyses.

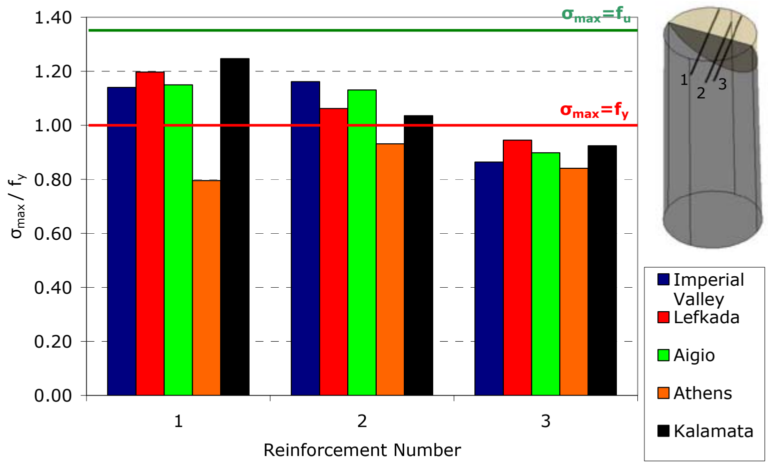

Figure 9: Efficiency factors of the three titanium bars used for the complement of the first drum of Column 2, for the five earthquake excitations used in the analyses.

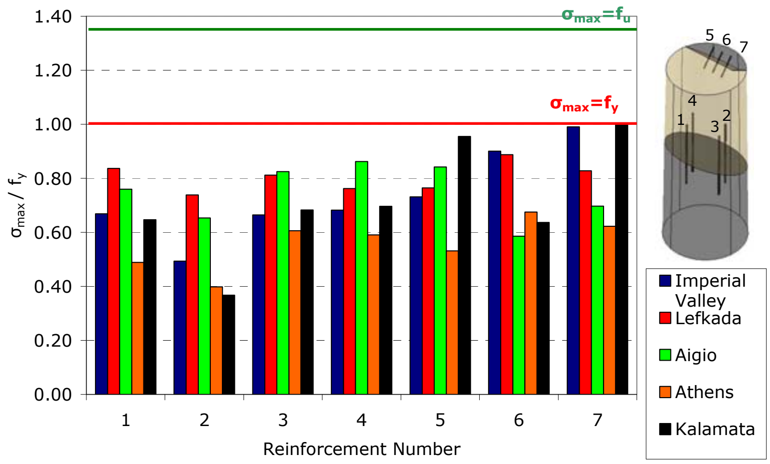

Figure 10: Efficiency factors of the seven titanium bars used for the complements of the fourth drum of Column 2, for the five earthquake excitations used in the analyses.

The results for the connections at the stylobate are shown in Figures 11 and 12. In case of S3 (Figure 11), in which extended connections were made, the stresses induced to the titanium bars were significant, more than 80% of the yield resistance in most of them (No. 1, 2, 3, 4) for all earthquake motions. In this case, the response of the reinforcement was not affected significantly by the ground motion characteristics. On the contrary, the reinforcement bars of the connection at stylobate S5 showed different behaviour for each earthquake. It is interesting to note that, although the efficiency factors attained at S5 were generally smaller than the ones at S3, probably due to the fact that the new complement was “encased” by the original block, large stresses were induced to some reinforcement bars of S3, which, in one case, surpassed the yield resistance. It should be mentioned that, for the stylobate blocks, reinforcement larger than the required according to the current design methodology was applied, in general, for constructional reasons.

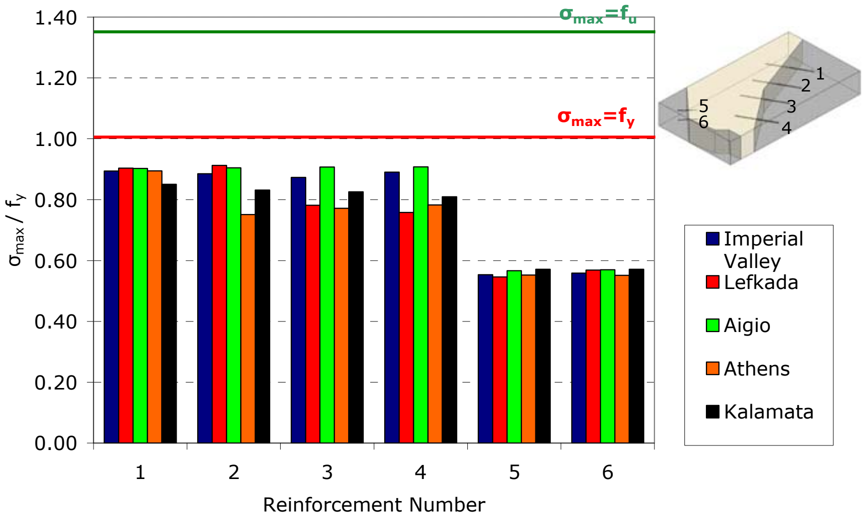

Figure 11: Efficiency factors of the six titanium bars used for the complements of the stylobate S3, for the five earthquake excitations used in the analyses.

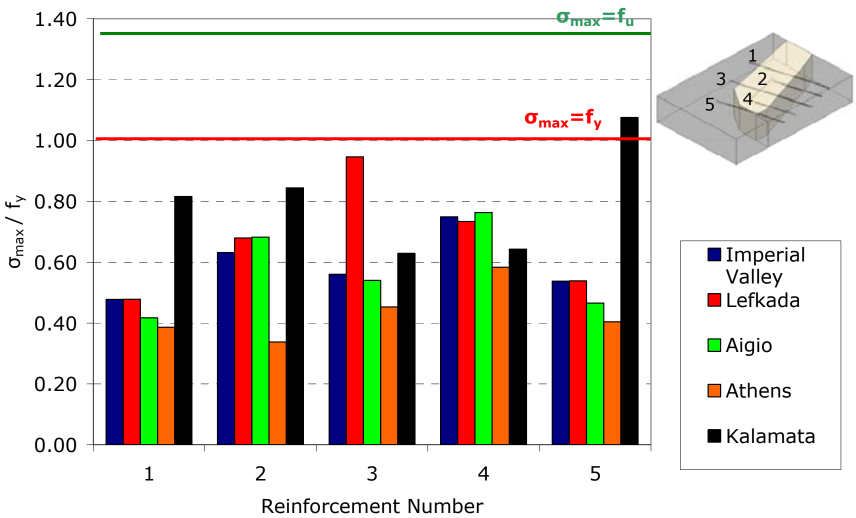

Figure 12: Efficiency factors of the five titanium bars used for the complement of the stylobate S5, for the five earthquake excitations used in the analyses.

6 CONCLUSIONS

- The simplified methodology that is used in practice for the dimensioning of the reinforcement of connections between new and old complements, which is based on the capacity design philosophy, was proved to be adequate, since the ultimate strength of the titanium bars was never reached. In most cases, the stresses induced to the reinforcement were significantly less than the corresponding yield stresses.
- The most severe situation was observed for fragments of column drums that form a wedge and are located at the lower part of the column.
- Significant stresses were observed at the connections of the stylobate, which were not expected intuitively, since only sliding can occur at these places. It should be mentioned, however, that failure of the reinforcement at such places does not, in general, puts the structure in danger of collapse.

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