STRUCTURAL MODELLING, ANALYSIS, EVALUATION AND STRENGTHENING OF PERGE SOUTHERN GATE HELLENISTIC TOWERS

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STRUCTURAL MODELLING, ANALYSIS, EVALUATION AND STRENGTHENING OF PERGE SOUTHERN GATE HELLENISTIC TOWERS

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ABSTRACT

STRUCTURAL MODELLING, ANALYSIS, EVALUATION AND STRENGTHENING OF PERGE SOUTHERN GATE HELLENISTIC TOWERS

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The successive struggle of *Perge Antique City* to resist against aging is clearly signified by *Hellenistic Towers Ruins*, parts of which still reaches up to 20 m high. Being a most reflecting example located at Anatolia, it clearly signifies its construction period and function compared to other examples that constitutes the same features.

However, There exist a certain requirement of detailed and wide ranging conservation study for finding remedy to cope with risk of further collapse, which is originated from the slender geometry of Towers Remains. Therefore, the need of a survey on the structural behaviour of towers with non-linear analytical modelling techniques is fulfilled in this study.

Preliminary analytical modelling (linear-elastic, macro models) was performed by using SAP2000 while, following detailed discrete stone element modelling examinations were performed with ANSYS-Ls DYNA, ABAQUS Software. Verification for simulations were made with results related with ambient vibration dynamic testing performed at Eastern Tower and Closed-form, simple calculations.

In the light of results bound to structural behaviour investigation on reconstitution, stability performance of today's ruins was examined against seismic activities. Four different strengthening methods were considered and their contributions to stability were compared in order to reach at the most appropriate intervention scheme obeying contemporary restoration criteria. The study formed a significant sub branch work of a restoration project of which charge was undertaken by SAYKA Restoration, Architecture Ltd. Co. Being a part of multi-disciplinary teamwork, structural investigation research was concluded to an optimum solution, which foreseen “minimum intervention to the building” assuring performance under seismic loading of large earthquakes.

Keywords: Perge Hellenistic Towers, dry masonry, discrete element modelling, seismic vulnerability, strengthening intervention

ÖZ

PERGE GÜNEY KAPISI HELENİSTİK KULELERİ YAPISAL MODELLEME, ANALİZ, DEĞERLENDİRME VE GÜÇLENDİRME ÇALIŞMASI

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Perge Antik Şehrinin yaşlanmaya karşı verdiği mücadele,en belirgin olarak, halen bazı kısımları 20 m yüksekliğe kadar ulaşan *Helenistik Kuleler Kalıntıları'*ndan anlaşılmaktadır. Yapıldığı dönem ve mimari özellikleri açısından benzerleri ile karşılaştırıldığında Anadolu'da yer alan en belirgin örnektir.

Ancak Kule Kalıntılarının yapısal narinliklerine bağlı olarak yok olma riski altında bulunmaları, geniş kapsamlı ve ayrıntılı bir çalışma yapılmasını gerektirmektedir. Buna bağlı olarak, kulelerin yapısal davranışının lineer olmayan modelleme teknikleri ile araştırılması bu çalışma kapsamında gerçekleştirilmiştir.

Birincil analitik modellemeler (lineer-elastik makro-modeller) SAP2000 yazılımı ile gerçekleştirilirken; ayrık eleman modeleri için ANSYS-Ls DYNA, ABAQUS yazılımları kullanılmıştır. Modellemelerin doğrulanması arazide, Doğu Kule üzerinde yapılan ortam titreşim dinamik testleri sonuçları ve basit mühendislik hesaplarınca gerçekleştirilmiştir.

Restitüsyon Geometrileri üzerinde yapılan yapısal davranış araştırmaları sonucu elde edilen verilere de bağlı olarak, günümüz kule kalıntılarının, sismik hareketliliklere karşı stabilite performansları incelenmiştir. Çağdaş koruma prensipleri sınırlarında kalarak en uygun

müdahalenin çıkarılması için 4 farklı güçlendirme yönteminin stabiliteye katkıları karşılaştırılarak araştırılmıştır. Çalışma SAYKA Restorasyon, Mimarlık Ltd. Şti. tarafından üstlenilmiş Perge Helenistik Kuleleri Restorasyon Projesinin önemli bir kolunu oluşturmaktadır. Çok disiplinli bir çalışma grubuyla yürütülen, yapısal davranış araştırmalarında "yapıya minimum müdahale" öngörülmüş; büyük sismik hareketlilikler sırasında yapısal dayanım performansı gösterebilecek, optimum çözüm kararı ile sonuçlandırılmıştır.

Anahtar Kelimler: Perge Helenistik Kuleleri, kuru duvar yığma yapı, ayrık eleman modelleme, sismik hassasiyet, güçlendirme müdahalesi

To my dearest mother, Gülen Bedük

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A_0	Effective Ground Acceleration Coefficient
AFAD	Afet ve Acil Durum Yönetimi Başkanlığı
d_c	Friction Function Decay Coefficient
DEM	Discrete Element Model
e	Eccentricity
E_m	Modulus of Elasticity of Masonry in Compression
EQ	Earthquake
F_a	Allowable Compressive Stress due to Axial Load Only
f_a	Calculated Compressive Stress In Masonry due to Axial Load Only
F_b	Allowable Tensile or Compressive Stress due to Flexure Only
f_b	Calculated Compressive Stress in Masonry due to Flexure Only
FEM	Finite Element Model
FFT	Fast Fourier Transformation
F_i	Equivalent Earthquake Load at i^{th} Stone Course of Tower Ruin-Arched Wall
$f_{m'}$	Specified Compressive Strength of Masonry
h	Height of the Masonry Wall
H_i	Height of the i^{th} Stone Course from ground Level
I	Building Importance Factor
ICOMOS	International Council on Monuments and Sites
I_n	Moment of Inertia of Net Cross-Sectional Area of a Wall
ISCARSAH	International Scientific Committee on the Analysis and Restoration of Structures of Architectural Heritage
p	Surface Normal Pressure

P_e	Euler Buckling Load
PSHCGTRRP	Perge Southern Hellenistic City Gate Towers Restoration Revision Project
r	Radius of Gyration
r_i	Radius of i^{th} Level of Tower Ruin-Arched Wall
R(T)	Earthquake Load Reduction Factor
S(T)	Spectrum Coefficient
V_t	Base Shear
W	Total Weight of the Wall
w_i	Weight of i^{th} Stone Course of Tower Ruin Arched Wall
θ_i	Degree of i^{th} Stone Course of Tower Ruin Arched Wall
μ_k	Dynamic Friction Coefficient
μ_s	Static Friction Coefficient
τ_{eq}	Equivalent Shear Value
$\tau_{1,2}$	Orthogonal Directional Surface Shear Values
σ	Normal Stress
γ_{eq}	Equivalent Slip Rate
γ_1	Slip Rate at First Orthogonal Direction of Friction Surface
γ_2	Slip Rate at Second Orthogonal Direction of Friction Surface
ν	Poisson's Ratio

CHAPTER 1

INTRODUCTION

1.1 Analytical Modelling as a Tool for the Structural Analysis of Historical Masonry Buildings

The 10,000 years of building activity carried out by the human race has included a need to repair and maintain structures, not only to keep them functional, but also to protect them as social, national and religious symbols of their age. It was during the Renaissance that historical structures began to be considered as bearing witness to former civilizations, and thus a heritage to be preserved for future generations. Consequently, in modern societies, a comprehensive conservation consciousness was developed for the preservation of built heritage not only to satisfy a social need but also as a cultural responsibility. Modern (20th century) conservation considerations are: *dynamic*, in that the adopted methods should be adaptable to alterations in the conditions of both the heritage itself and the environment; *sustainable*, in that they should guarantee effectiveness in the long term; and *value centered*, in that they should promote the participation of different groups from the social spectrum to encourage autonomous preservation (Mason & Avrami, 2002).

This idealistic modern approach necessitates a thorough understanding of the architectural heritage and the preparation of a conservation project that takes into account both its tangible and intangible characteristics. This can only be achieved through multi-disciplinary scientific conservation studies. In the mid-20th century, the “Venice Charter: International Charter for the Conservation and Restoration of Monuments” was published as a guide to this multi-disciplinary approach. Specifically, Item 2 of charter cited the importance of various scientific branches and related techniques to be included within conservation activities for the protection of cultural heritage.

On the other hand, the structural consolidation and strengthening of historical heritage against the elements is vital if these historical structures are to be prevented from completely vanishing. In the case of the 18th century Baroque Cathedral of Noto in Sicily (Figure 1.1-a), a regional earthquake caused serious damage to the cathedral six years after unsuccessful attempts at consolidation in 1996 (Binda *et al.* 1999). Even masterpieces of engineering genius, such as Brunelleschi's Dome at Santa Maria Del Fiore, face a similar fate, as a series of cracks that have appeared near the ribs of the vaulted dome have decreased its bearing capacity (Ottoni *et al.* 2010). Despite the fact that the main structure of Hagia Sophia has been repaired and strengthened throughout its life following seismic activity, the shape of the dome of the structure (Figure 1.1-b) has altered significantly, thus making its collapse all the more possible (Croci *et al.* 1997). In some examples, the lack of a holistic consideration of local conditions during the design process has raised the likelihood of collapse, as in the case of the Tower of Pisa. Furthermore, the sudden collapse of the Civic Tower of Pavia shows that it is not only natural hazards or constructional deficiencies that cause collapses, but also the aging of structures (Binda *et al.* 1992). In summary, it is clear that while much building heritage is today in danger of collapse, or will become so at some point in the future, the problems are complicated and may well yet need to be determined, necessitating a detailed research-based structural behaviour investigation using contemporary tools and techniques.

To this end, over the last two decades computerized systems have been used in the creation of analytical models,[1] making it possible to view a real structure in a virtual environment and solve multi-parametric cause and effect problems within the structures using complex matrixes of mathematical calculations in a considerably short period of time.

[1] Different types of structural analysis through analytical modelling is covered in detail in Section 3.1.

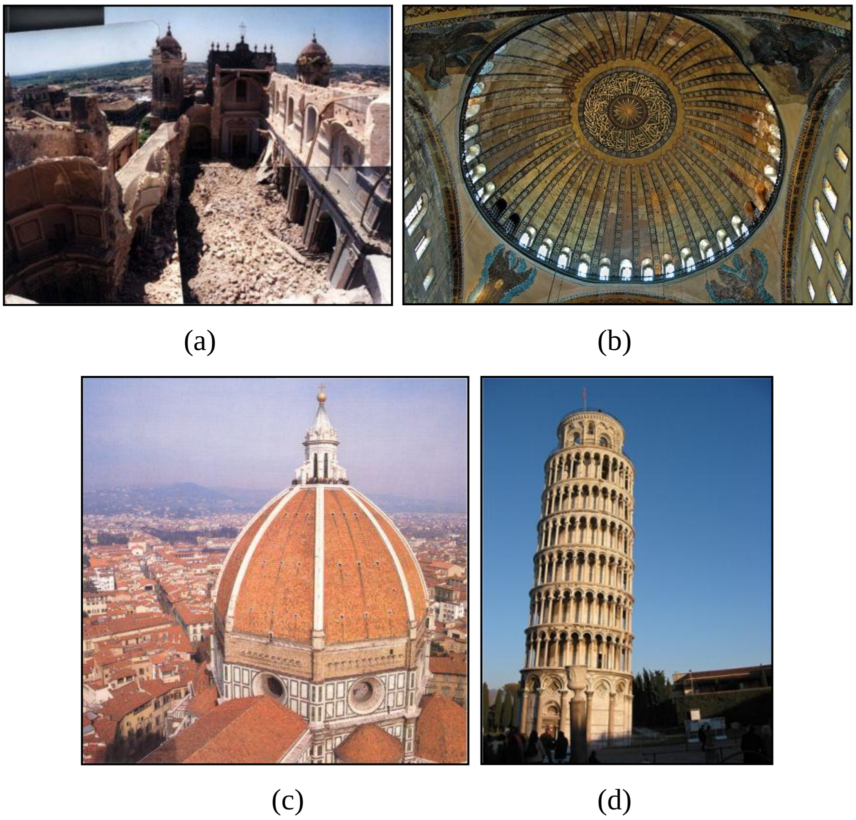

Figure 1.1 (a) Collapsed Cathedral of Noto-Sicily (Binda & Saisi, 2005) (b) Dome of Hagia Sophia (c) Brunelleschi's Dome at Santa Maria Del Fiore (d) Tower of Pisa

At first, a *linear-elastic* analytical modelling technique was used for the structural inspection of historical buildings. Although this kind of analysis did not permit the visualization of the characteristics of buildings beyond the limit of recoverable deformation, it was very effective in defining the structural behaviours of huge structures with complex geometries that could not be established by a visual inspection or basic hand calculations. The technique allowed the global structural behaviour of the Colosseum (Croci, 1995) to be understood, and the effect of the continuing settlement of the Tower of Pisa's outer pillars (Macchi *et al.* 1993).

It is understood that linear-elastic analyses are incapable of measuring the realistic behaviour of historical masonry structures that have minimal tension carrying capacity and have already been damaged beyond their elastic limit (Roca *et al.* 2010). Especially under lateral loadings resulting from such forces as earthquakes, wind and explosions, masonry can easily become exposed to tensional forces, meaning that many historic masonry structures that have already been affected by superimposed forces cannot be investigated.

At the end of 20^{th} century, the previously defined non-linear mechanical behaviours of masonry buildings were possible to be assessed through the laboratory testing of historical masonry construction techniques, providing a basis for analytical models. As a result, it became possible to simulate the heterogeneous mechanical characters of masonry construction in a virtual medium. As a result, in recent years many historical masonry structures have been studied to assess their vulnerability, stability and durability using specially developed complex research activities that draw upon data from visual inspections, laboratory tests and non-linear analytical models.

Accordingly, not only the structural characters of the historical masonry structures, under their own loads or under the effect of external forces, such as wind, earthquakes and explosions, were studied, but also structural problems related to material degradation. In the case of Pavia Tower (Ferretti & Zdenek, 2006) the newly developed technique was easy to apply, even after the structure had been completely demolished. Moreover, research activities of *reverse engineering* for objects of cultural heritage other than buildings could be made with a non-linear structural analysis; for example, studies to identify the degradation sources of the Mount Nemrut Statues (Turer, Aktas, & Ş. Guchan, 2009) and research into the structural decay of the Ebe Schooner Brig (Invernizzi *et al.* 2010).

1.2 Definition of the Problem and Selection of the Case

In trying to define the *philosophy of preservation engineering,* Kelley (2005) mentions the *medical analogy* of the patient-doctor relationship, being a compulsory cyclic scheme that comprises Anamnesis, Diagnosis, Therapy and Control. If a purposive collection of information, including previous problematic occurrences, interventions and constructional alterations, is able to be gathered from heritage building in the *Anamnesis Stage*, a successful *Diagnosis* can be reached. Only if the supported data is well processed, and associated with visual inspections, structural analyses and laboratory tests, can the exact sources of damage and safety levels to be understood, which can then form a basis for deciding upon the level and means of intervention. On the other hand, a minimal intervention scheme cannot be provided by a course of *Therapy* comprising intervention methods that target precisely the problem in both a physical and ideal manner unless the two previous links in the chain have been established successfully. Finally, the activity is *Controlled* to evaluate and verify the diagnosis and the chosen therapy to decide whether the whole work scheme is to be reactivated or not.

In recent years, the number of scientific methods facilitating the structural identification of historical masonry structures, have seen a rapid increase. However a large percentage of these approaches do not follow the routine defined above. For this reason, studies in which the researcher has been guided to imprecise results (for example, due to the use of a linear elastic analytical model that results in a level of detail that cannot possibly provide a realistic assessment) are unlikely to lead to the creation of case-specific diagnostic procedures, despite the wealth of collected data on the structural problem and the material features of the case in point. On the other hand, in some study cases, while the historical background may have been fully identified and the case-related problems identified to the required level, the results may not be used to develop an appropriate solution. Moreover, in some cases, when structural vulnerability estimation studies are made using non-linear analytical modelling techniques, which rely on insufficient base data, such as wrong material and mechanical feature characterizations or uncertain problem definitions, similar unsuccessful results are obtained . Hence, the results obtained from such studies may be inaccurate and thus may affect the accuracy of the results of the structural investigation in an imprecise manner.

In this study, a unique historical structure, that has been flagged as in danger of collapse, is to be investigated structurally and properly treated in line with the modern conservation approaches mentioned in Section 1.1. More important, a full adaptation of Kelley's medical analogy approach (2005) to modern preservation engineering is to be applied, using contemporary tools and techniques.

Figure 1.2 Perge Hellenistic Gate Towers, Southern View (Photo was taken at 2010).

Dated to the 3rd century B.C., the Fortification Towers of Hellenistic City Gate at the archaeological site of Perge in Southern Anatolia (Antalya) are considered unique, in terms of their architectural and archaeological features (Akarca, 1987). They were initially constructed for defensive purposes, in the Early Roman Empire Period. The structures, which enclose a courtyard, were turned into a monumental symbol for the city and have maintained this new identity as the *Symbol of Perge* into modern times (Özgür, 1989).

Especially for dry masonry construction, the prevention of destruction resulting from lateral loads is much more important than from vertical forces, such as dead and surcharge loads (Heyman, 1995). Considering this argument, the Hellenistic Towers, which are nearly 20 meters tall, were built to withstand lateral loads from such occurrences as seismic activity. In general, it is the upper levels of structures that are at greater risk of collapse, being subjected to greater movement; but it is the unexpected deformed shape of the Perge Hellenistic Towers, along their elevation that makes the towers interesting for investigation.

Additionally, as the towers do not feature a complex geometry or include architectural elements that are generally constructed with similar construction technique, the formation of non-linear numerical models is easier, in that highly complicated laboratory experiments or numerical extractions can be avoided.

For the above reasons, the towers were considered as an appropriate case for structural modelling, analysis, evaluation and strengthening. The study was carried out as a consultancy research project. The aim was to come up with structural intervention suggestions for the Perge Southern Hellenistic Gate Towers Restoration Revision Project (PSHGTRRP[2]), under contract to the SAYKA Arch. Rest. Cons. Company. SAYKA had previously commissioned three earlier reports and proposals from three different organizations, all of which were declined by SAYKA for different reasons.

1.3 Methodology of the Study

The study was carried out in three main stages: A research of written and visual documents; a field survey and ambient vibration tests; and a structural investigation, involving laboratory testing, computer analysis and numerical calculations.

Research of written and visual documents: A brief literature survey was carried out to ascertain contemporary conservation approaches; and the need and purpose of preservation engineering as a part of conservation efforts was mentioned in brief. As a contemporary tool used in structural investigations of heritage buildings, analytical modelling was evaluated in Chapter 1.1.; however, there is also a need for a fully comprehensive preservation engineering study, which requires a framework that follows the medical analogy mentioned in Chapter 1.2. The methodology of this thesis aims to follow this analogy.

Before beginning the modelling studies, the architectural, archaeological and structural features of the Perge Hellenistic Gate Towers were required to be fully understood. The

[2] Upon a request from SAYKA Co. a consultancy team was put together to carry out revision work for the 2002 Restoration Project of the Perge Hellenistic Gate Towers, which was approved by Antalya Regional Council for Conservation of Cultural and Natural Assets No: 5377. In the revision work, archaeological consultancy was carried out by a team from the Istanbul University Classical Archaeology Department led by Prof. Dr. Haluk Abbasoğlu. Architectural consultancy was provided by the METU Department of Architecture, Restoration Graduate Program, led by Assoc. Prof. Dr. Emre Madran. The Technical Research phase was supported by METU. The geological and geophysical investigations were carried out by a team from the METU Geological Engineering Department, led by Prof. Dr. Tamer Topal. The experimental research on material decay and the repair mortar proposal were made by the METU Department of Architecture, Material Conservation Laboratory team, led by Prof. Dr. Emine Caner Saltık. The structural condition investigation and strengthening proposal development phase, included within the scope of this thesis work, was carried out by a METU Civil Engineering Department team, led by Assoc. Prof. Dr. Ahmet Türer, all bound by an agreement between SAYKA & METU Project No: DSİM-10-03-03-1-00-75.

majority of data relating to the architectural, material and geological features of the towers was obtained from the unpublished documents in the SAYKA Archives, the company's 2002 Restoration Project and the site notes of the manager of the project, all of which were used in the determination of the geometrical features of the ruins. The dimensions of the parts and elements of the tower structures were taken from the photogrammetric documentation within the 2002 Restoration Project, while some necessary measures about the wall sections were gathered from the notes prepared during the 2009 reconstruction process of the Eastern Tower. Information, on the original condition (reconstitution) and archaeological characteristics of the towers, was gathered from the records of 19th century travellers, archaeological excavation reports, journals and thesis works (Chapter 2.2.). In the scope of PSHGTRRP, the METU teams responsible for the material investigation and the geological survey provided the data mentioned in Chapter 2.3., which formed the basis of the analytical model in terms of material properties (Chapter 3.2.2.1).

Based on reports from the SAYKA archive, the strengthening proposals, put forward as part of earlier consultancy works on behalf of SAYKA, are given in detail in Chapter 2.5. These studies provided additional data on the structural behaviour of the towers. In Chapter 4 the earlier analytical models are recreated taking into account each of the strengthening proposals, and their performance is investigated.

In order to select the most appropriate structural assessment method and tools necessary for the analytical studies in this thesis, a brief literature survey was made on the types of analytical models that are deemed appropriate for masonry structures. Since the Perge Hellenistic Gate Towers were constructed using the dry masonry technique with stone blocks of uniform shape, the discrete element modelling technique (DEM) was found to be the most appropriate tool for the structural analysis, and accordingly, a detailed survey of DEM studies related to masonry building heritage using DEM has been made (Chapter 3.1).

Field studies: Concentrating on the structural identification of the towers, two field studies were made at Perge in the preparation of this thesis on the 16th and 17th of June, 2010. On the first day of the field survey, investigations concentrated on identifying the structural form of the towers, their construction techniques and any evident structural problems. The surrounding area of the towers was also investigated to allow a comparison of the structural conditions of the towers, with buildings from the same period and constructed using similar techniques. Traces of sources of the structural degradation were sought in the field; and a photographic record was made of both the structural condition of the towers and the traces of degradation in the field. The results of this visual inspection are presented in Chapter 2.3. while defining the current condition of the towers.

On the second day of the survey, detailed measurements were taken and free-hand drawings were made on specific parts of the towers. Although in the modelling process the dimensions of the structural elements of the towers were taken from the SAYKA Archive (2010) and from archaeological and reconstitution studies found in literature, the dimensions and orientation of the stone block elements in the wall sections, the surrounding areas of the openings and at the junctions of the tower walls were measured and documented by the author with photographs and simple sketches.

An additional visual inspection was made of the points and traces defined in previous reconstitution and restitution studies. In this way, all of the necessary data relating to the architectural features of the towers, in both their current state and in their reconstitution condition, was collected in the first field study. Data on the architectural features of the tower gathered during the field survey is given in Chapter 2.2, while the usage of this data in the formation of the analytical models is explained in Chapter 3.2.1.

The second field survey, carried out on the 21st of September, 2010, aimed to determine the dynamic characteristics of the towers through ambient vibration testing. The natural modal vibration periods of the Eastern tower were captured under wind and artificial excitation, through the use of highly sensitive (resolution of 0.000004g) accelerometers located on the 32nd stone course. Six different measurements were taken at different locations under different kinds of excitation. The raw data, gathered in the field, was processed using a Fast Fourier Transformation and Wavelet Analysis to ascertain the natural frequencies of the towers. Details of the measurement and data processing stages of the dynamic testing phase are explained in Chapter 3.5.

The requisite raw data, including the geometrical features, material physic-mechanical properties and dynamic characteristics of the structures, was fully gathered, and the structural investigation continued with the creation of analytical models, laboratory tests and numerical calculations are presented in Chapter 3.

Both linear elastic continuum models and non-linear discrete element models were formed considering the reconstitution geometries of the Perge Hellenistic Gate Towers and the current condition of the Eastern Tower. A linear elastic analysis was made using SAP2000 software, while discrete model analyses, in which every stone block element was represented individually, were made using Ls-DYNA & ABAQUS software.

Preliminary analytical modelling studies were made on the reconstitution geometries of the towers using SAP2000 and Ls-DYNA software to deduce the lateral loading on the towers in terms of stress distribution, and the effect of stone orientation on resistance. Then, the collapse sources were investigated in detail using ABAQUS discrete element models. In the analysis, architectural elements assumed to have existed prior to collapse by archeologists, was also questioned. The criteria regarding the formation of the model based on the data gathered from the literature survey, archive research and field studies are presented in Chapter 3.2.1.

The seismic vulnerability of the reconstituted and original structures were made using sample earthquake data (PGA 0.5 g) corresponding to a first-degree EQ zone in Turkey. Furthermore, three additional synthetic earthquake measurements with low intensities (PGA 0.3 g on average) were taken from local seismic records (Chapter 3.2.3), with the same physic-mechanical material properties used in both the linear and non-linear models. In addition to the detailed experimental studies conducted by Topal (2010), a compression capacity test was made on 1/20-scaled tower wallettes (Chapter 3.2.2.1), while for the discrete models, the Columb friction behaviour was assigned for the interfaces between the stone blocks. The frictional behaviour was defined using the ABAQUS Library, while the required parameters were obtained from geological studies found in the SAYKA Archive (Chapter 3.2.2.2).

The results obtained from the reverse engineering studies done through an analysis of the reconstitution models, evaluated in Chapter 3.3., formed the basis of both the investigation of the existing structures and the development of a proposal for structural strengthening.

A verification of the models through numerical calculations and dynamic testing was made, and is presented in Chapters 3.4 & 3.5. The results of the analysis of the current Eastern Tower were verified with numerical calculations that estimated the risk of topple when subjected to seismic activity, after which dynamic character verification was carried out for the models by comparing the results of the dynamic testing made on the real structure and on the analytical models.

A general evaluation of the structural vulnerability estimation study is made in Chapter 3.6.

Taking into account the results of the analytical modelling investigation of the tower in both its original and existing form, a strengthening proposal was developed by the author for SAYKA. This would be the forth such attempt to come up with a solution for the strengthening of the towers company, with details of the three earlier proposals presented in Chapter 2.5. The performance of all strengthening proposals was investigated with the help of ABAQUS discrete element models. The Eastern Tower model defined in Chapter 3 was recreated to include the details of the four proposals, and a further analysis was made using the same parameters and under similar conditions (Chapter 4.1 & 4.2). Moreover, the effectiveness of the strengthening method developed by the author was also tested under the previously defined synthetic EQ records, and a detailing of the system was made taking into account the results of the analysis.

The four strengthening proposals were assessed considering not only the structural performance for compliance with the fundamental preservation principals defined within the Venice Charter, but also the case-specific conservation requirements in the towers (Chapter 4.3).

CHAPTER 2

SOUTHERN GATE HELLENISTIC TOWERS OF PERGE

This chapter focuses on the Hellenistic Towers of Perge in terms of their historic, archaeological and architectural features, drawing upon the researches, archaeological excavations and conservation studies carried out to date. In addition, as briefly mentioned in the first chapter, the structural conditions of the towers and factors that may constitute a failure risk are defined in detail based on the results of the site survey carried out as a preliminary study prior to the structural interventions.

For this purpose, first a review will be made of the previous archaeological and conservation activities carried out at the perimeter of Southern Hellenistic Gate. This will be followed by a detailed examination of some of the earlier strengthening proposals and projects that were considered appropriate for the conservation of the Perge towers

2.1 History of the City of Perge and the Southern Hellenistic Gate

Perge, a key city in the Ancient *Pamphlyia*[3] Region from the Hellenistic and Roman Age, is located 12 km north-east of the city center of Antalya (Ancient *Attalia*) (Figure 2.1). The ancient site neighbors Aksu Village, which takes its name from the river that passes 2 km away from the northern and western perimeters of Perge. A major part of the city is spread over a valley that is surrounded by three hills: *İyilik Belen* (south-east of the city), *Koca Belen* (south-west of the city) and *Asar Hill* (to the north) (Figure 2.2).

[3] *Pamphlyia* is an ancient geographic region in Asia Minor, located in South-western Anatolia. It encompasses the western part of the Taurus Mountains and the Southern Anatolian Lakes Region, and forms part of the southern seaboard. It contains numerous ancient cities, including Perge, Side, Sagalassos, Selge, etc. (From: http://en.wikipedia.org/wiki/Pamphylia)

Figure 2.1 Location of Perge at Ancient Pamphylia Region

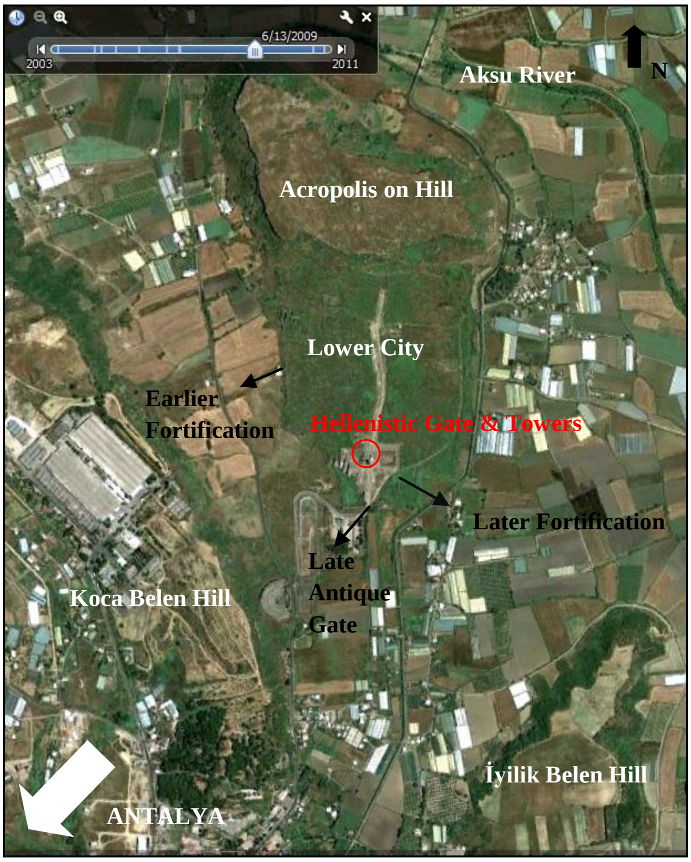

Figure 2.2 Google Earth – Aerial View of Perge and Its Surrounding Geography at 2009

The city is composed of three parts: The Acropolis to the north; the Lower City, which was surrounded by early Hellenistic Fortifications; and the Southern Extension, a part of which was also fortified with a wall in a later period. The Acropolis was established on the northern hill, and was served by an ancient road leading from the *Attalia* direction. Spreading over the valley beneath the hill, the Lower City is laid out in a grid-iron pattern, in which the Attalia Road, the N–S Colonnaded Street, forms the main axis (Akarca, 1987). The lower city perimeters are marked with the remains of a Hellenistic Fortification and the city itself features a residential area and a number of social constructions, including a bath, palaestra, agora and a well-developed water system having elements of Nymphaeum and channels. The third part of city is a later-period southern extension to the lower city, and is composed of two parts, the first of which was fortified with a later period addition to the old fortifications, while the second part was not. The additional fortifications enclose such structures as the Basilica, the Southern Bath and the Agora inside the city, while important structures like the stadium and the theatre lie outside (Figure 2.3).

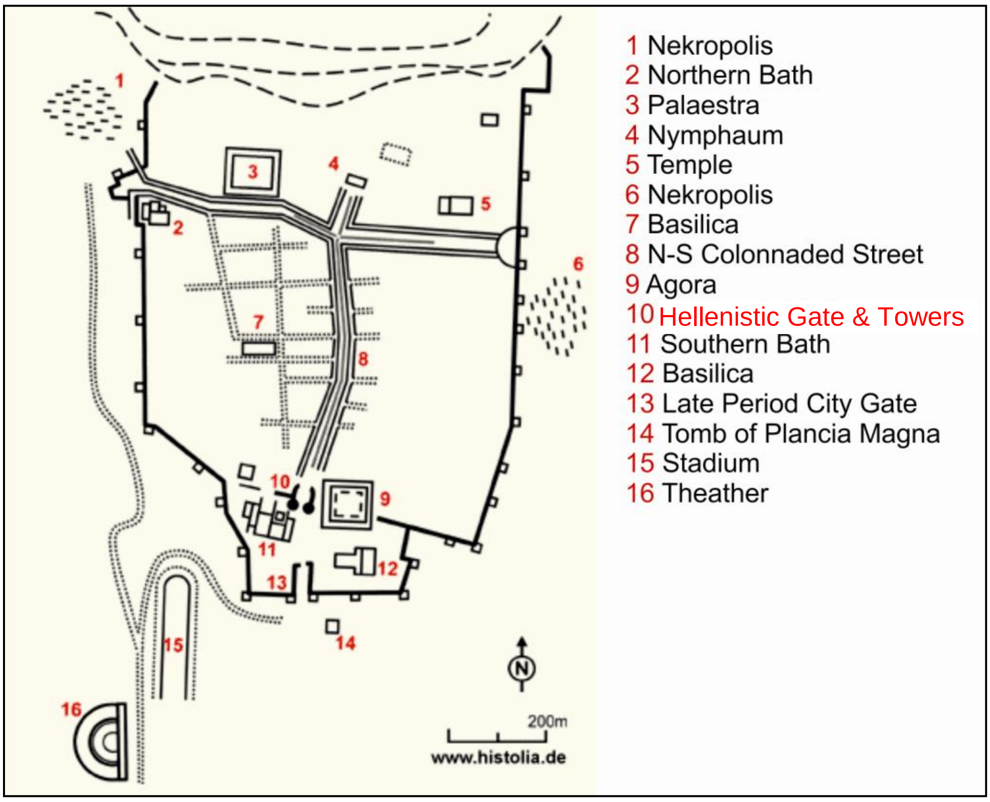

Figure 2.3 Perge City Plan (Rochow, 2011)

Philological evidence found on the northern hill offers evidence that the first settlement of the area was in the 13th century BC (Pekman, 1989). Up until the Hellenistic Period only the hill was inhabited; after which the settlement expanded into the lower parts, and thus a preliminary organized city plan was established. Although precise dating has not been possible, construction of the first fortification is thought to have been in the Seleucid[4] Period as defence against the unrest following the death of Alexander the Great, which steered development policy in the region at the time (Abbasoglu, 2001).

During the Roman Period, especially during peace period of II and III centuries, the city was extended. Secondary partial fortification was constructed. However, city was also evolved outside the fortifications. The major monumental structures in the city were also made during this period (Pekman, 1989), late periods of that the Byzantium and Christianity was taken over the control of Pamphlyia, and Perge. Being a major metropolis of the Christian Reign, a widespread church and basilica structure network can be found at Perge.. In IV and V centuries, the settlement was deceased to inner sides of the fortifications. Unrest in the Isauria[5] Region and bandit attacks specified the political state of the period during that time (Pekman, 1989). This unrest was the clear evidence for that power of Byzantium Empire over Southern Asia Minor was deceased.

Starting from first century Perge become one of two struggling, major cities for Christianity metropolis of the region. The Christianity had its climax in IV century at the Perge. The two big church of the city were constructed in this century. Up until the 7th century (the period that Byzantium Reign was started to weakened) the city was an important location of Christian world, however it had lost its importance gradually after this period (Bulgurlu, 1999). Especially, the Arabic invasion has caused big unrest at the southern Anatolia. Because of the unrest settlement ar Perge narrowed and the city left its place as a metropolis to *Attelia* (Pekman, 1989).

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[4] The Seleucid Empire was established by the successor of Alexander the Great, Seleucus, who was one of his generals. The domain of the Seleucid Empire in its golden age included South-eastern Anatolia, Mesopotamia and the region that is now Iran, Syria and Iraq (from: http://en.wikipedia.org/wiki/Seleucid_Empire).

[5] *Isauria* is an ancient geographic region in Asia Minor, located North of the *Pamphlyia*.

A detailed visual presentation of city's historical development up to 7^{th} century has been made by Bakacak (2007) (see Figure 2.4).

Beginning from XI century the invasion of Anatolia by Turks has started. Turkish tribes, up to the reign of the Seljukids in the XII century, inhabited the Pamphlyia region (Pekman, 1989). The Mongolian invasion of Anatolia at XIV century brought the chaos in the region so that Cyprus Frank were kept the control of the region a while. However, at the second quarter of the XV century the interchanging political situation of the region turned into five-century long stable control of the Ottomans, which was also finished with declaration of the Turkish Republic (Pekman, 1989).

Nor archaeological studies or historical studies determined a clear evidence of clear inhabitancy at Perge after XI century, but it was known that Perge was also included at the authority area of Theophykatos, the religious leader of the Christianity Union in the Phamphylia at XV century (Pekman, 1989).

In XX, many travellers visited the Perge ruins, state of which were defined in their publications (journals and books) with few pages. R. Walpole, W. M. Leak, F. V. J. Arundell, Ch. Fellows, E. Forbes, Ch. Texier, P. Tremaux, H. Rott, Pariberi-Romannelli, B. Pace, Viale were some of these travellers. The most detailed observations were made by another traveller Lankoronski who reserved a major part of his study for Perge; archaeology of the site, buildings, and city's history (Pekman, 1989).

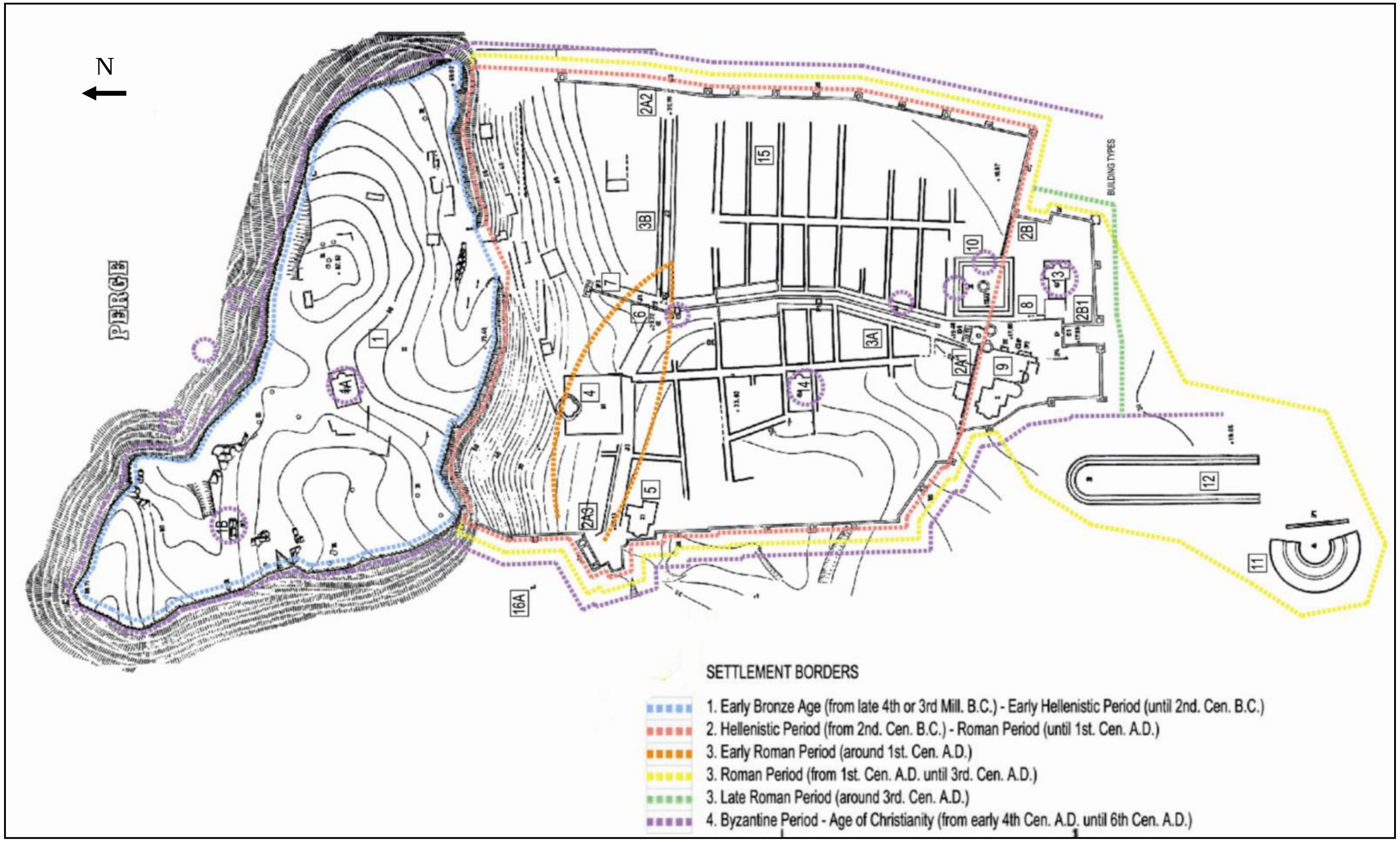


Figure 2.4 City Development History Representation (Bakacak, 2007)

The Southern City Gate Complex is the largest of the three gates in the first city fortifications, the other two are located at the edges of E-W Colonnaded Street. Today, the City Gate complex is formed by two partially collapsed towers, an oval courtyard located beneath the towers, and a triumphal arch, of which only the lower levels are still intact (Figure 2.5). A detailed description of the architectural features of the towers is given in Chapter 2.2.

Neither the architectural style, nor archaeological evidence offer clues to the actual construction date of the towers. Structures built in the “Doric Style” have been both common and widespread among architects for centuries, and thus make precise dating difficult (Bulgurlu, 1999). Despite these uncertainties, there is some historical evidence that can be used to define the construction period. Perge’s use as a garrison-town for Seleucid ruler Anthiochus III has been considered as clear evidence of the existence of a Hellenistic Fortification during this period, meaning that an upper limit of late third-century BC can be defined for the construction period (McNicoll & Milner, 1997).

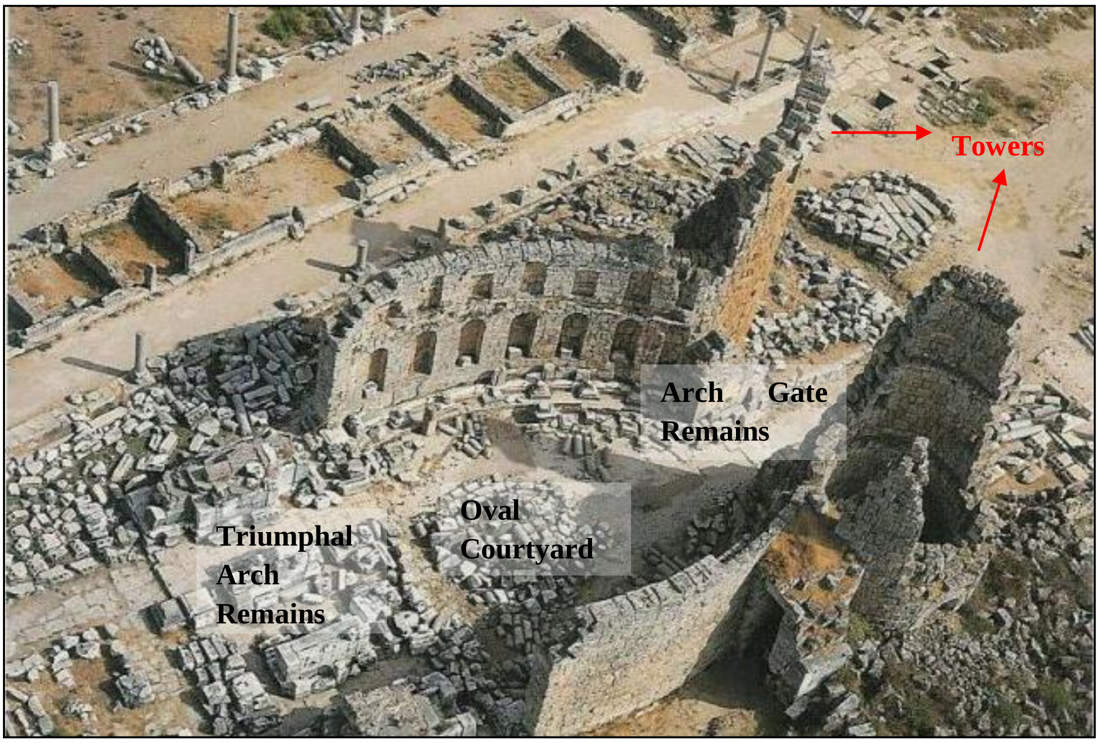


Figure 2.5 Perge Hellenistic City Gate Complex

The gate complex and all of its elements were constructed as an important part of a well-organized city fortification, representing common architectural features of its era (Lawrence, 1979). For centuries, even during the Early Roman Empire Period, the gate and towers maintained their function. During *Pax Romana,*[6] when the need for such solid defences had subsided, the city extended far beyond the fortification walls, and consequently the gate took on the role of a monumental object instead of a functioning building. During the reign of Hadrian, it gained its final form as a monument at the centre of the city. Later disturbances around the city meant that a defensive need against invasions re-emerged, and the city was provided with an additional (later period) fortification including a gate built to the south of the existing one. From that point in time until the present day the gate has maintained its importance as a monumental symbol of the city, with minor alterations made during the Byzantine period.

Initially, the City Gate was formed by two towers and an oval-shaped courtyard with an arched gateway at the far end of the curve (Abbasoglu, 2001). During the Pax Romana period, the oval shape of the courtyard was altered into a "U" shape and an additional arched gate was constructed between the two towers. This was a clear indication that the towers were no longer needed for the controlling of passage, and the gate was thus losing its function as a part of the defensive system (Bulgurlu, 1999). In the Imperial age, a major part of old southern fortification was demolished to enable the expansion of the city southward. During the reign of Hadrian the courtyard was reorganized Plancia Magna by as a monumental structure, dedicated to Perge, with such artistic elaborations as niches and marble wall coverings. In addition, a triumphal arch was built between the courtyard and the city (Pekman, 1973). In following periods, while some alterations were made to the surrounding structure, no major structural changes altered the form of the Hellenistic City Gate (Bulgurlu, 1999)

The different periods of the structure are presented in Figure 2.6

[6] Pax Romana was the golden age of the Roman Empire, and can be translated from its Latin as "Roman Peace" (From: http://tr.wikipedia.org/wiki/Pax_Romana.)

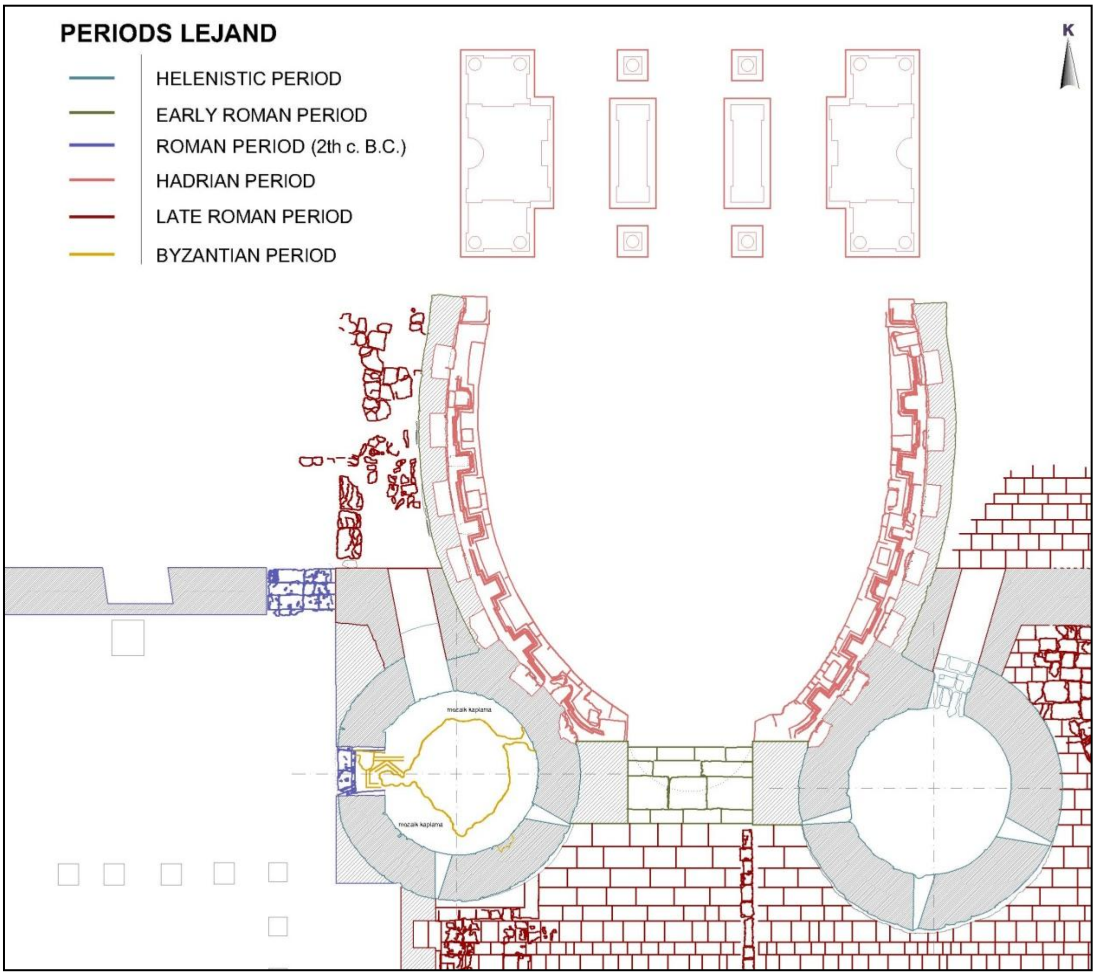
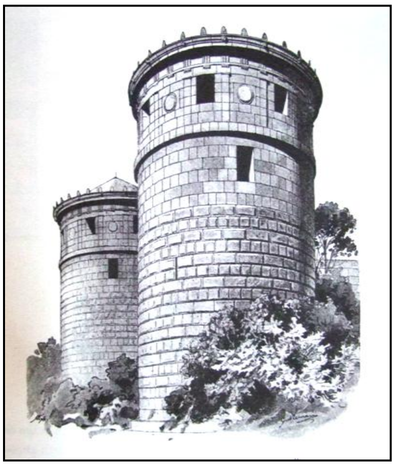


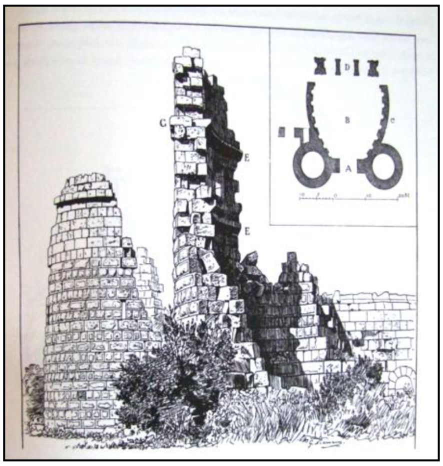
Figure 2.6 Periods of Hellenistic Gate (SAYKA, 2011)

The city of Perge attracted the attention of several scholars, including Texier and Trémaux, before the 20th century (Pekman, 1973). Although the towers had been analyzed previously, it was not until the end of the 19th century that a detailed architectural identification and documentation of their conditions was made by Lanckoronski. The City Gate was defined by Lanckoronski *et al.* (1890) as having surely been constructed for defensive purposes, based on its huge, strong towers and its particular courtyard plan. Moreover, the gate was identified as being rich in architectural diversity, bearing similarities with many examples that could be found all over the Hellenic Regions of Asia Minor and Achaia[7] (Lanckoronski *et al*., 1890).

[7] Achaea was the name given to the southern part of modern Greece in Hellenic and Roman times.



(a)



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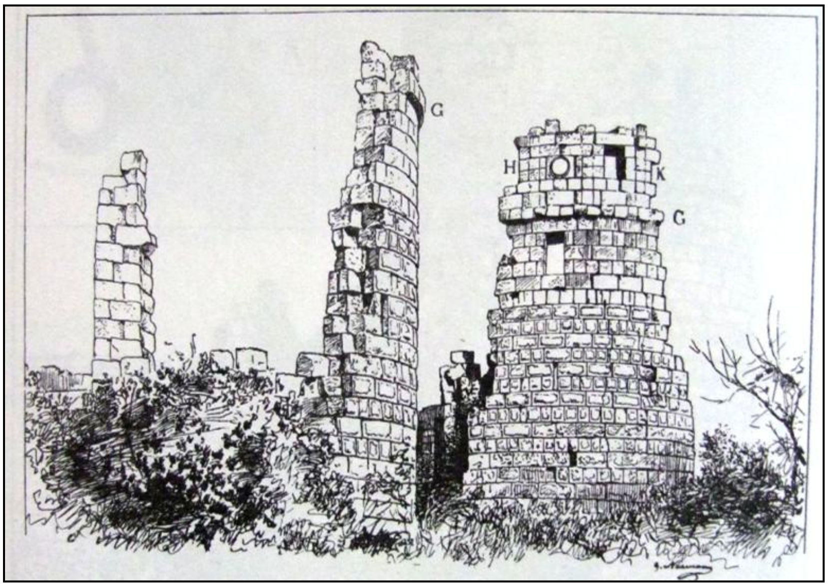


(c)

Figure 2.7 (a) Reconstitution of Hellenistic Gate by Lankoronski *et. al*. (1890) (b) Southeastern View Engraving of the Towers by Lankoronski *et. al.* (1890) (c) Southwestern View Engraving of the Towers by Lankoronski *et. al*. (1890)

Architectural excavations in Perge have been taking place, with some breaks, since 1946. Legal excavations have being performed by the Classical Archaeology Department of Istanbul University from 1946 until today. In the early stages of this timeline (1953–1955), excavations were mainly focused around the Hellenistic Gate; while in the 1995–1996 period, additional small-scale surveys were also carried out. In 2001 restoration studies were launched that included both project preparation and application, and the resulting conservation applications are still continuing today.

In Section 2.4, both previous archaeological and conservation studies are given in detail.

2.2 Archaeological and Architectural Characteristics of the Towers

When carrying out structural investigation studies of ancient buildings with analytical simulations it is important to document in detail the structure in terms of its geometry and its construction techniques so as to come up with realistic results. Visual inspection details, a detailed level of measurement and literature survey grades are collected, with attention paid to the complexity of both the structure and the precision target of the structural estimation study with non-linear analytical modelling.

In this manner, the measured drawings of the towers prepared in 2001 and 2008 by SAYKA Arch. Co. were used, with additional information gathered from reports of excavations made during the restoration applications of the company in 2010–2011. Moreover, measurements and information that can only be gathered through a visual inspection were obtained by the author during a field study in the summer of 2010. In this way it was possible to collect the required data about the geometry and structural form of the towers in their current state. It is also essential to identify the structural behaviour of a heritage building from records of its original form and its current state (ISCARSAH, 2008). However, one of the most significant problems faced in data acquisition directly from archaeological heritage buildings when attempting to gather holistic information is “lacuna”. As can be seen from Figure 2.5, a large proportion of the Perge Hellenistic Gate Towers has been lost to decay, and architectural traces have been lost or buried under the collapsed parts. In this sense, gaps in the available information were filled by drawing upon publications from previous archaeological, conservation and architectural history studies (travel reports, restoration project drawings & reports, archaeological annual study reports, books etc.).

The measurements of architectural details of the current structures were mainly gathered from SAYKA Archive (2002 drawings, revised in 2010). These drawings are given in Figure 2.8, Figure 2.9, and Figure 2.10. Thesis work of Bulgurlu (1999) and measurements made during field study provided additional data about measurements and architectural features of the towers.

Architectural Features

Being the highest of the structural remains in Perge, an astonishing view of the Hellenistic City Gate Towers greets visitors beyond the Late Antique Gate. This spectacular view of the towers has become a symbol of the city (Özgür, 1989), and while they cover a large area, they still reach up to great elevations as if in challenge to their aging.

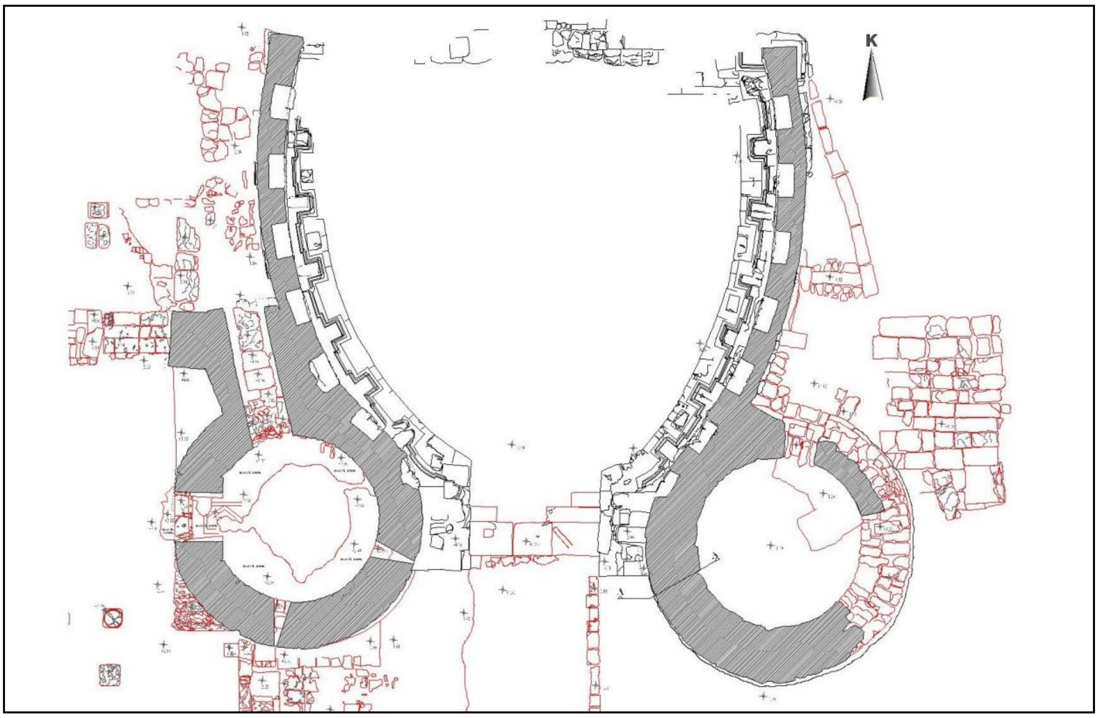


Figure 2.8 Ground Floor Plan of the Hellenistic Gate Towers

Hellenistic Gate is formed with two tubular masonry towers and courtyard walls that projecting outward from towers and forming an oval-shape at northern sides of the towers.
Two towers were constructed ~12 m away from each other. The outside radii of the eastern and western towers are 11.7 m and 11.25 m respectively (Bulgurlu, 1999). Inner radii of the towers decreases to 9.30 m and 9.15 m at the ground level.

The remains of the Eastern Tower measure 20 m in height from ground level, and 18.5 m for the western tower. As can be deduced from changes in wall thicknesses and the existence of stone courses projecting both outward and inward, the structures had four floors, excluding the foundation level.

The superstructure sits on four stone courses, which form the foundation.[8] The thickness of the wall at this level is ~240 cm for the Eastern Tower and ~215 cm for the western one. The height of each course is 60 cm at this level. The number of remaining stone courses is 39 for the eastern tower and 34 for the western.

The ground floor[9] comprises 12 stone courses, where wall thicknesses of 215 cm and 200 cm were measured for the eastern and western towers respectively. The difference in wall thickness between the towers vanishes at the transition line separating the ground and first floors. The wall recesses outward above that level to a thickness of ~175 cm, meaning that the ground floor level seems to be projected inward from the upper floors.

[8] The foundation characteristics of the wall thickness, stone orientation and number of courses were investigated on the Eastern Tower during excavation and stone cataloging activities in 2010–2011 by SAYKA Co. During the summer field visit for this thesis work, a similar investigation was also made by the author, using a one square meter hole excavated by SAYKA on the inner side of the Eastern Tower.

[9] Prior to the excavations made by SAYKA inside and around the perimeter of the towers during restoration works in 2010–2011, almost the entire first six courses were buried under rubble.

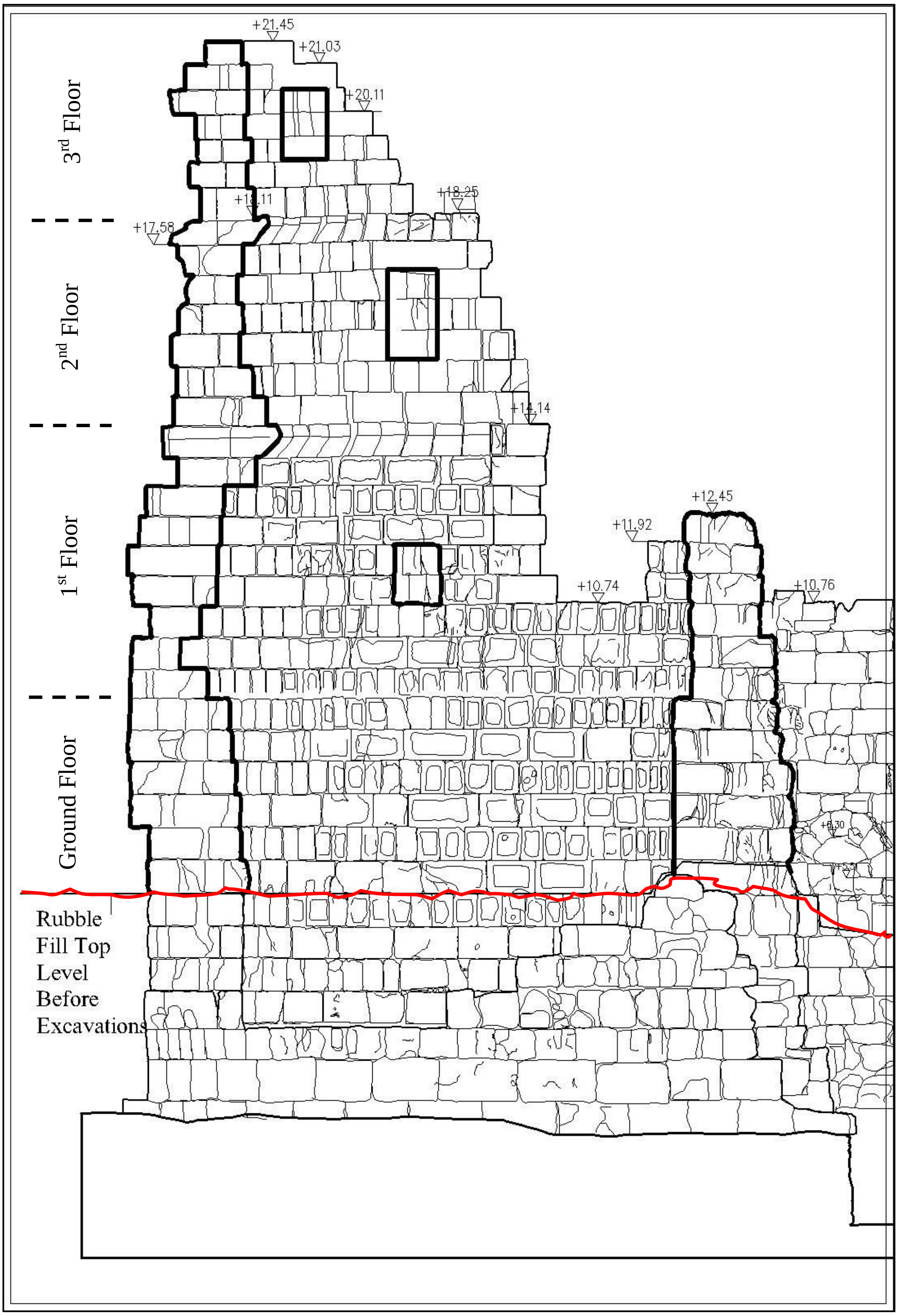
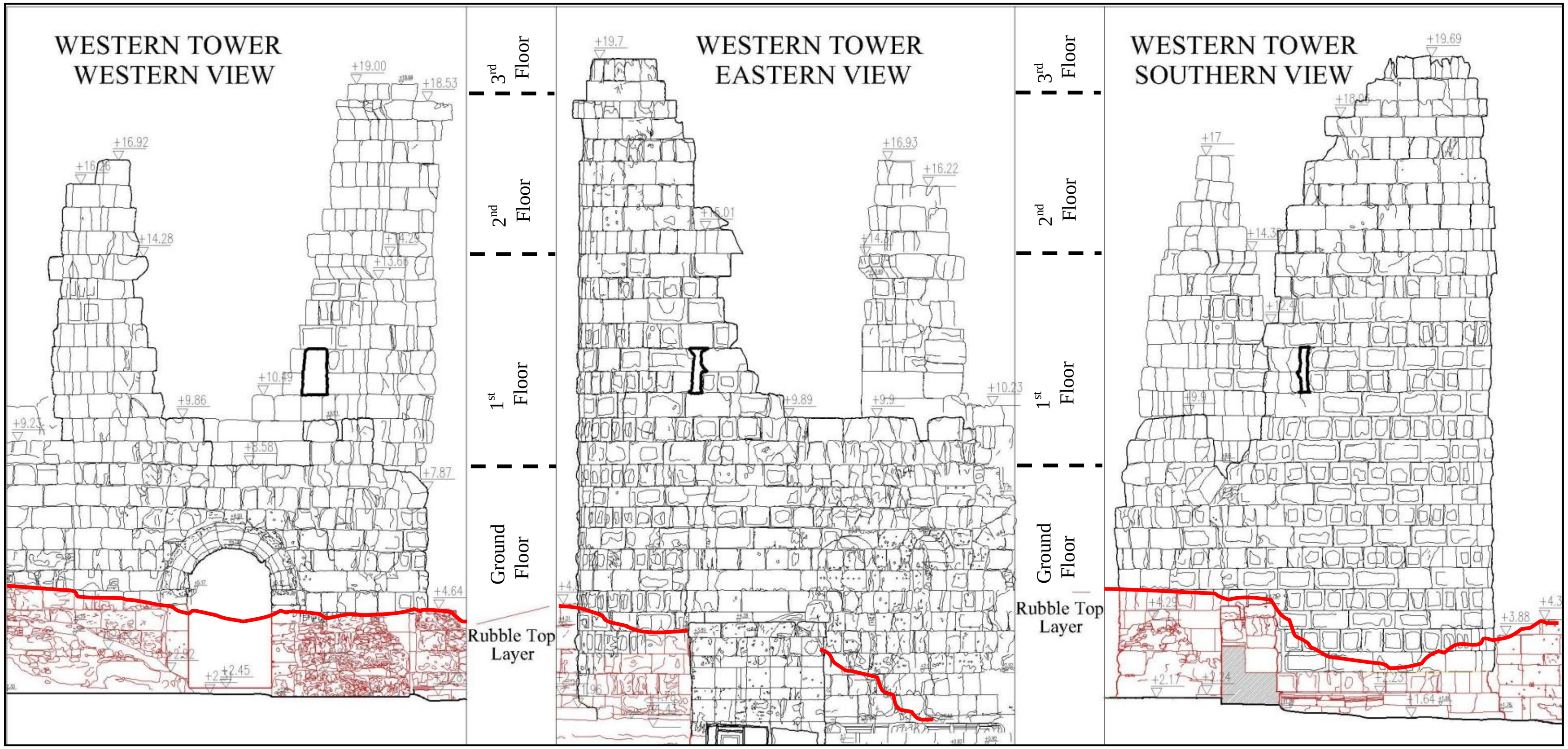


Figure 2.9 Eastern View Drawing of the Eastern Tower – SAYKA (2002, Revised in 2010)



(a) (b) (c)

Figure 2.10 Drawings of the Western Tower a) Western View b) Eastern View c) Southern View - SAYKA (2002, Revised in 2010)

The first floor comprises nine stone courses (courses 17–25). The final course (25th) of this floor projects inward 50 cm, making the wall thickness ~225 cm at this point. Above the first floor, the second floor is formed by seven stone courses (courses 26–32). The 32nd course acts also as a boundary, projecting both outward and inward by ~40 cm. The third floor begins at course 33. A group of stones belonging to the first two courses of the third floor are still in place on the Western Tower; however the Eastern Tower offers greater information about the third floor, as parts of seven of the third floor courses (courses 33–39) are still intact and are in their original positions. From ground level to the 33rd course (which is still in place) the stone blocks measure ~60 cm in height. In the courses above course the heights of each course vary, with an average height of ~45 cm.

Windows, loopholes or traces of them can be detected at every level of the Western Tower above the ground, while the Eastern Tower lacks such features at ground level. Having an inner opening width of ~70 cm and an outer width of ~25 cm, the Western Tower has a series of loopholes through the first two superstructure floors. There are two such loopholes constructed in the 7th and 8th courses of the Western Tower, one of which is directed in a southern direction and the other in an eastern direction. In the Eastern Tower, the mirrored location of western ones were filled with stone elements, having different masonry orientation. At the 20th and 21st courses of the first floor, two loopholes can be found in the Western Tower; one directed southward, and the other towards the south-east. In the eastern twin, only one such loophole has survived, located on the south-western side of tower. At the third floor, the loopholes are replaced by windows (covering courses 28–30) on the south-western side of the Eastern Tower. In contrast, in the Western Tower only an inner side cut stone pattern of a window can be detected, located on the south-eastern edge of what is left of the wall. Because of lacuna, on the last floor the only existing example of the window series can be found on the Eastern Tower, constructed on the south-western side in courses 35–37. The window is located on a 9°-wide segment of the circular wall.

The arched doors located on the northern elevations of the towers provided access, and were constructed between the 5th and 9th courses. A gallery leading to this door was constructed on the northern side of the Western Tower, and covers the full height of the ground floor. The same structural element cannot be viewed on the other tower due to collapse, meaning that its existence cannot be verified by visual inspection. On the western side of the Western

Tower there exists another arched gate, which is located one course higher than on the northern gate. It can be understood from the gate's construction technique and location on the tower plan that it was added in a later period than the northern one. The existence of a similar gate on the Eastern Tower cannot be determined, as the tower has collapsed down to the foundation level on its eastern side.

In both towers ~5 cm deep square holes were found in the 11th course of the Eastern Tower and the 16th course of the Western Tower. The holes follow a systematic order, being located ~50 cm away from each other. The same pattern can be found in the lower courses of both towers, although featuring smaller square holes, laid in a denser pattern, and are thought to have been part of an internal timber bearing system for the towers.

(a) (b)

Figure 2.11 a) Square Holes on the 11th & 16th Stone Courses of the Eastern View b) Square Sockets carved at the 16th Stone Courses of the Eastern Tower

Material and Constructional Techniques;

The towers were constructed using the dry stone masonry technique. Voussoir-like travertine cut stones, weighing on average 0.5 m^3 were laid in a circle on top of each other, to form circular pipe-like masonry towers. The Acropolis of the city lies on the northern Asar Hill, the top of which is formed by a layer of travertine, 150 m in depth. The travertine formation is typical for the Antalya Region, and so accessibility to rock deposits was easy and the establishment of quarries was very high for the area. On the other hand, during the formation of the travertine deposits, the petrifaction and rotting of organic substances like tree roots caused a high level of porosity in the stone body, giving the material a low level of durability (Topal, 2011). However this formation increased its workability, which led to its extensive use in historical masonry structures of the Pamphlyia Region. Being constructed from travertine and built close to a huge source of travertine deposits, the Perge Hellenistic Gate Towers' main material should belong to a quarry, which should be sided to the Asar Hill.

Constructed with Isodomic ashlar using an alternating course technique, the dry masonry walls have four different bond patterns depending on the wall thickness. Two different bonds were used for the lower structure wall segments (foundation and ground floor). The cross section of the first bond reveals a construction of two headers and one stretcher stones that are ~110 cm length, ~55 cm wide and ~60 cm high. The second bond includes one header stone located at the centre of the cross section, while the two faces are formed by stretcher stones. These alternating stone courses were laid staggered in contrast to the adjacent courses, by which a regular spread of head bed jointing was obtained. The same construction technique can be found at the higher parts of the towers; however where the wall thickness decreases, a single stretcher and header bond was used.

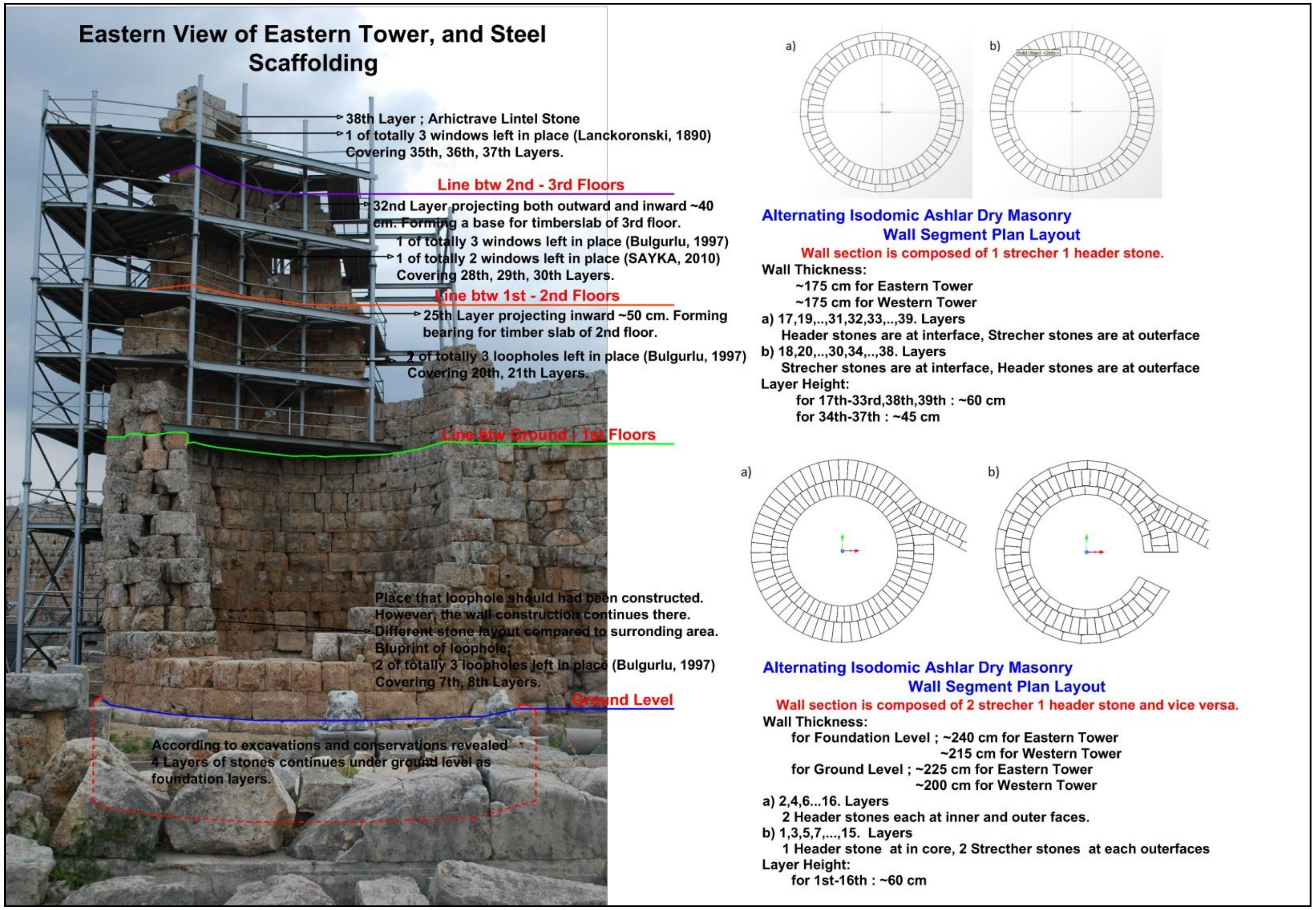


Figure 2.12 Stone Layout Presentation on Eastern View of the Eastern Tower and the Steel Scaffolding

Travertine is much more fragile and solidifies easier than such hard rock types as marble and andesite, and so it was not possible to obtain very smooth surfaces on the blocks, making the anathyrosis technique impossible. All surfaces abutting neighbouring stones were left roughly flattened; however the outer and inner faces of the stones were specially treated. As previously defined, the structures were built using a dry masonry technique, with no traces found of either metal joining elements (clamps, dowels, etc.) or mortise geometry. The only evidence of metal elements (clamps) can be found on the voussoirs in the arches of the gates of the towers (Bulgurlu, 1999). Stone faces at the lower levels of the towers (ground and first floors) were left in bossage technique,while those on the upper parts were smoothed as much as possible. A rope-sling system, handling bosses, side grooves, top grooves, lewises and lifting pins have been defined as techniques which were frequently used in lifting-placing activities in Hellenistic Masonry Construction (Adam, 1994); however in the case of the Perge Hellenistic Towers, the blocks lack traces (small sockets, channels) of the use of metal elements for both handling and jointing (metal clamps, dowels). For this reason, it can be assumed that handling bosses may have been used for lifting, and removed by carving after placement.

The constructional form and architectural element orientation of towers is summarized in Figure 2.12.

Defining Architectural Features at Lacuna according to Previous Reconstruction Studies;

Although is not possible to make a detailed classification of the Hellenistic City Gates due to the fact that there are many variations among existing examples in terms of their construction techniques and functions, a simple classification was made by Akarca (1987) based on their plan type and elements, defining three main types of Hellenistic city gate plan (1987). In the first type, the gates were constructed as simple openings through fortifications, formed by overlapping two segments of curtain wall and leaving a gap between them. Simple examples of gates located at the edge of a fortification segment that are V-shaped in plan are also included in this type defined by Akarca. In the second type, being a transition to a more complicated gate form, towers were added to each side of the overlapping gaps in the wall. The gates at Mantineia and Stimfalos belong to this group. This type of gate was also constructed by deploying a city gate between two towers that projected outwards, however they lacked a courtyard behind. Starting from the 4th century BC, the construction of a courtyard behind the gate became widespread. Akarca defined these gates with courtyards as the third type (1987). Courtyards that are rectangular, trapezoidal, semi-circular, full circle, ellipse and U-shaped in plan can all be found in Hellenistic Architecture (Lawrence, 1979). The courtyards were built to give a monumental visage to the city gate, while also allowing enemy forces to be defended against in a narrow area should they manage to breach the gate (McNicoll & Milner, 1997).

The main objective in building towers next to the gate was to provide a higher ground from which to defend. The plan schemes of gate towers were square, rectangular, polygon and circular (Akarca, 1987). In the 3rd century BC, although rectangular construction was a common architectural trend for the Hellenistic period, the construction of towers that were circular in plan saw an increase at the time. The City Gate at Miletos is a good example of the transition from rectangular to circular plan, as one of the towers of the city gate is square in plan, while the other is trapezoidal. As it was necessary to produce stones in curved forms, the effort and resources required for the construction of a circular tower were greater than for polygonal ones; however circular towers were much more durable against impact. For this reason, many examples of Hellenistic City Gate Towers that are circular in plan can be found, with examples in Athens (Peiraeios Gate), Peiraeios (Aitoneis Gate), Mantineia, Korinth (İsthmos Gate), Megara Hiblaia and Perge. Although the city gates of Silyon, Side and Perge are the most distinct examples of the Hellenistic Period, Perge is the only example of a gate featuring a courtyard and circular towers in Asia Minor. In spite of the fact that the characteristics of Hellenistic City Gate of Perge have been altered over time,[10] it possesses a more unique character in its function, architectural features and history when compared to other examples.

As previously defined, the percent of lacuna is very large in these structures. In particular, information about the number of openings and their locations had to be gathered from previous reconstitution and historical studies (Abbasoglu, 1996; Nosov, 2009; Mansel, 1974; McNicoll & Milner, 1997).

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[10] The architectural and functional alterations that have occurred throughout history are presented in Chapter 2.1.

The southern gate towers stand out as a clear symbol of the fortification at Perge. The first initial architectural form of the Hellenistic Gate comprised circular towers in front of a closed-form oval courtyard (Abbasoglu, 1996). The only entrance to this part of the city fortification was provided by an arched door located at the far end of the courtyard (Abbasoglu, 1996). According to the reconstitution presentation made by McNicoll & Milner (1997), the towers were divided into four floors (Figure 2.13). Timber floors were assumed to have been positioned at the base of each floor, able to carry heavy artillery for defence. The existence of carved holes at the ground floors and projected levels at the transitions zones between floors were likely to have been used to carry bearings for the timber floor elements. The non-systematic order of the square-shaped carved holes on the inner surface of the Eastern Tower indicates that the timber floors were probably semi-circular and/or smaller (Bulgurlu, 1999). Originally, the existence of a roof structure was argued by Lanckoronski (1890) as protection for the war machines against the elements (Nosov, 2009).

During Pax Romana, an arched city gate was constructed between the Perge Gate Towers, the form of the courtyard was altered into an open-ended oval shape, and a Triumphal Arch dedicated to Perge by Plancia Magna was constructed between the city and the courtyard. An arched gate, located on the western side of the Western Tower, was also added in that period. As a result, the city gate lost its intended function as a defensive structure as the need for accessibility from the surrounding structures arose. The gate facilitated access to the Roman Bath beside the towers. Although doubted by Abbasoğlu (2001), Mansel (1974) stated that a similar gate was also constructed in the Eastern Tower.

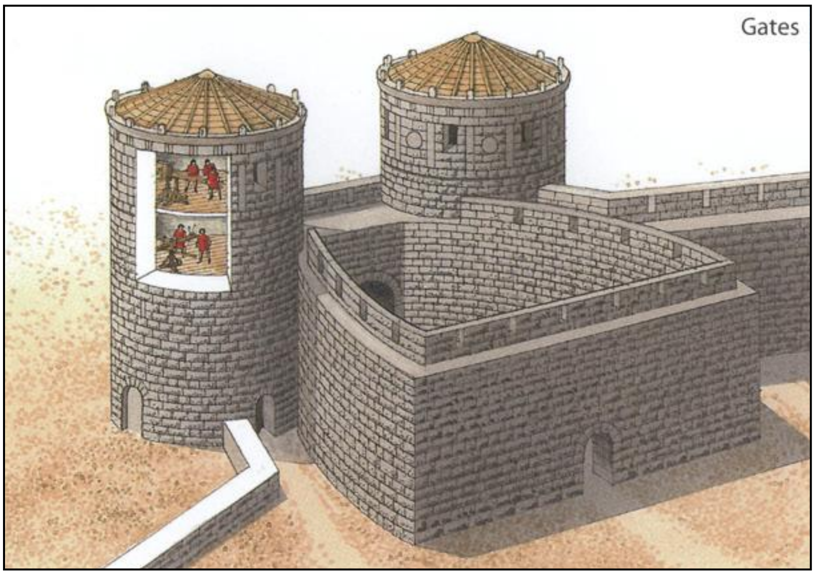


Figure 2.13 A Reconstruction for Initial State of Southern Hellenistic Gate of Perge by Nosov, K. S. & Delf, B. (2009)

It has been suggested that three loopholes had been deployed at intervals smaller than a quarter at 1st and ground floors in the original structures (Bulgurlu, 1999). At the 2nd floor, Lanckoronski et al. suggested that there had been four windows positioned equidistant around the circumference of the tower (1890); however, the existence of a window on the northern elevation has been disputed by Bulgurlu & Abbasoğlu (1997), and in the reconstitution study carried out by SAYKA (2010) put the number at two. For the top floor, the eight-window assumption made by Lankoronski et al. (1890) has been accepted during all reconstitution studies performed to date. A Hellenistic period reconstitution prepared by SAYKA (2011) is given in Figure 2.14.

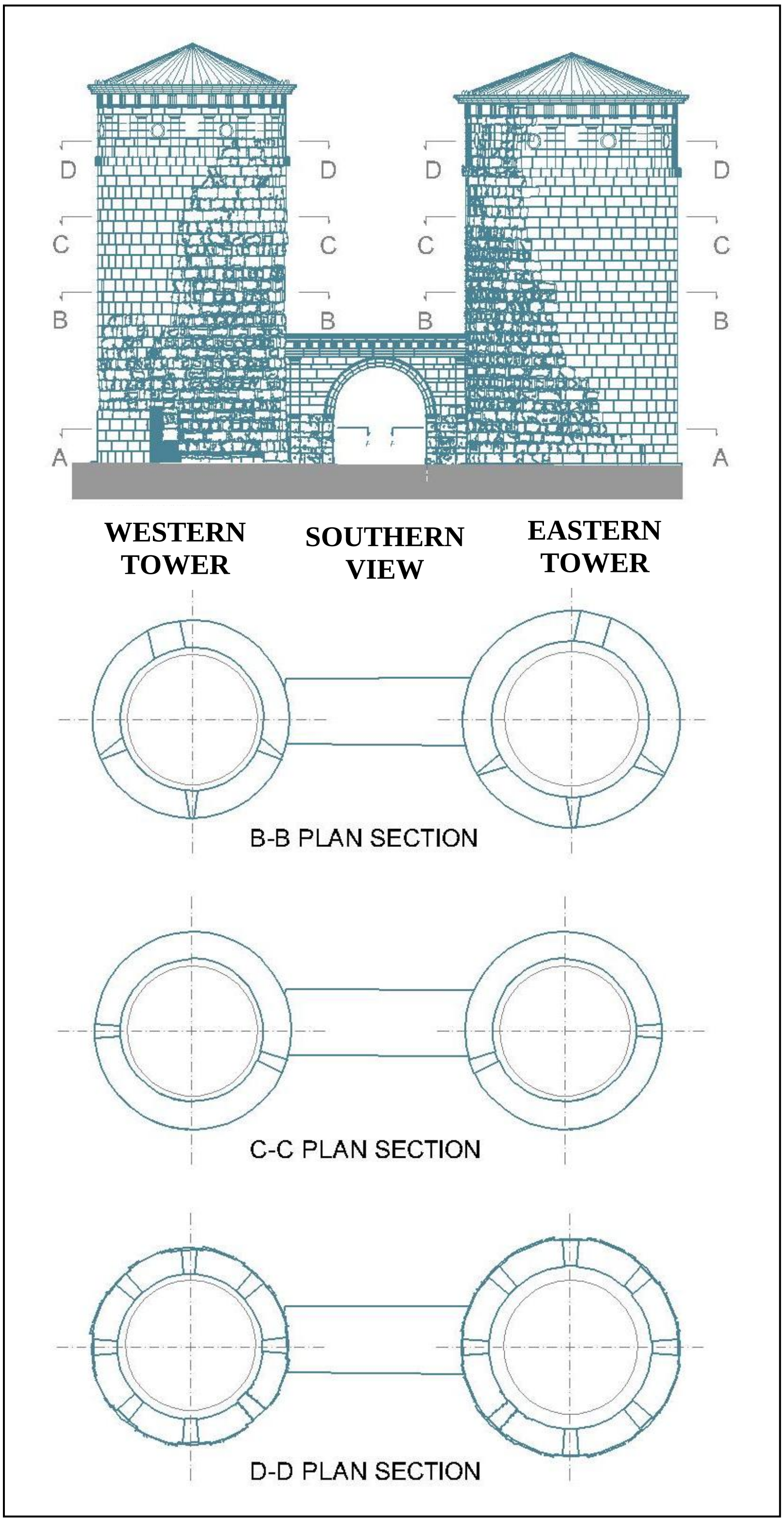


Figure 2.14 Reconstitution Drawings of the Towers by SAYKA Co. (2011).

The reconstitution study of the 3rd floor was first made by Lankoronski (1980), and then developed by Bulgurlu & Abbasoğlu (1999) according to archaeological findings (Figure 2.15). The 3rd floor and the Doric entablature on top of it are the most spectacular features of Hellenistic Architecture in the structure.

A series of large stone blocks carved with shield-like reliefs were located between the windows (eight in total) on this floor (Lanckoronski *et al.*, 1890). Piers were constructed to fill the spaces between the windows and stones with shield reliefs. Between 33rd and 37th block courses, piers made of five stone blocks were constructed between the windows and stones bearing shield reliefs. Today, only one pier remains in its original condition on the Eastern Tower. Column capital reliefs were carved on the topmost (37th course) stones of the piers. On top of the windows and piers, architrave and lintel-architrave stones were used following the order, while a frieze layer (39th course – Metop, Triglyph Series) was laid as the final course under the Doric cornice (Bulgurlu, 1999). The only remaining stone corresponding to the roof cornice is a geison (horizontal) stone bearing a mutulus. There is not enough architectural evidence to allow a clear understanding of the roof shape and character; however, assumptions have been made based on comparative studies of examples from the same era (Lanckoronski *et al.*, 1890).

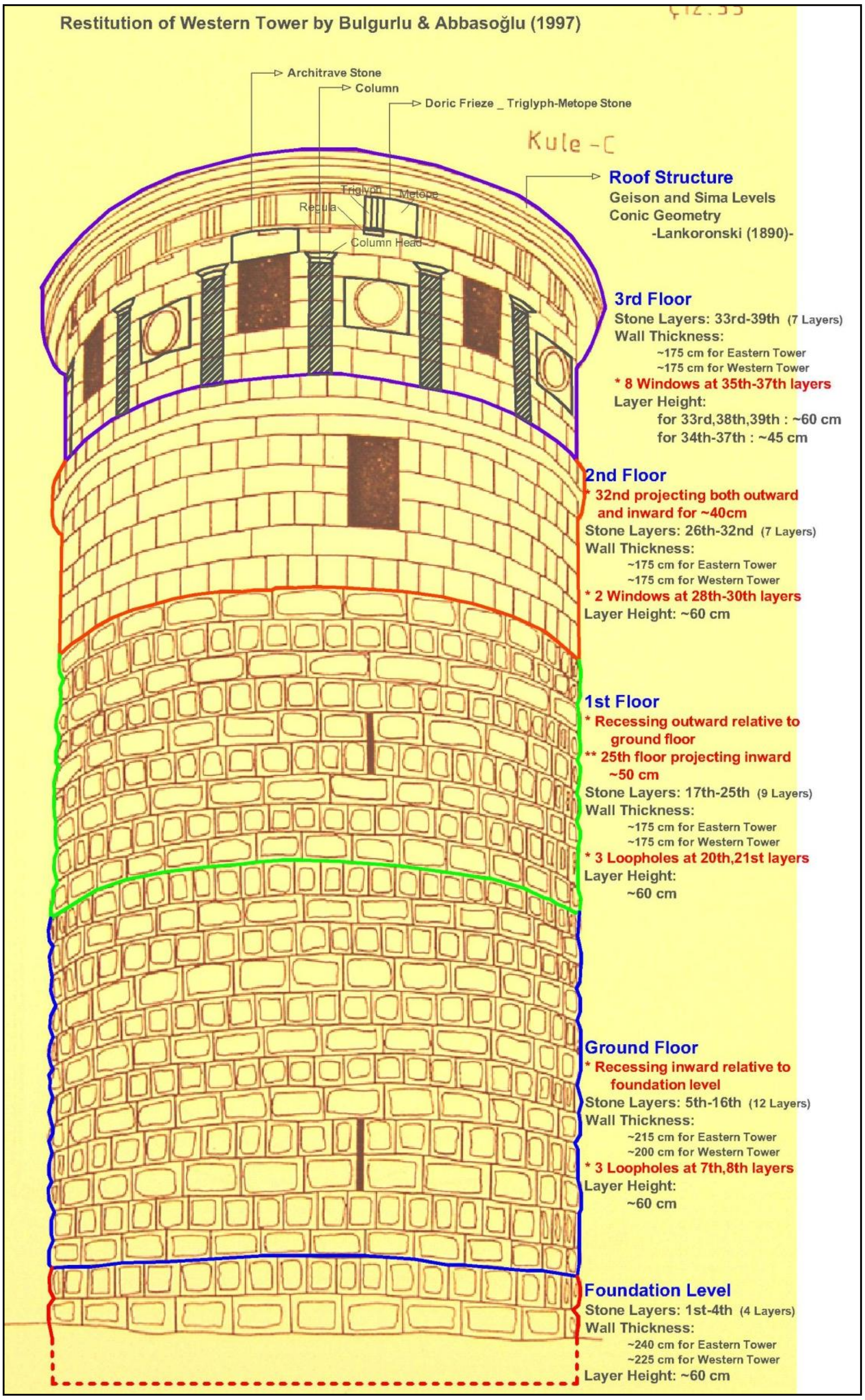


Figure 2.15 Southwestern View Restitution of the Western Tower (Bulgurlu, 1999)

2.3 Structural Features and Current Structural & Material Conditions of the Towers

A holistic estimation of structural behaviour and material features is required in the scope of any successful restoration work (ISCARSAH, 2008), and identifying both structural and material conditions is one of the primary tasks to be carried out in a qualitative investigation. In this way, preliminary links between these aspects can be understood, and a preliminary diagnosis made of any problems. To this end, an examination of the structural system and structural decay of the site was made during a field study in the summer of 2010. The investigations not only concentrated on the towers, as possible sources of structural decay were also sought in the surrounding area.

In masonry structures, in particular for dry stone masonry, the strength of the masonry units is not the main parameter when specifying structural durability as maintaining stability is the primary goal. Moreover, in dry stone masonry structures the general form may be retained in spite of slippages of unit blocks (Heyman, 1995). The most challenging problem in historic dry masonry is the propagation of this kind of degradation over the course of the history of the structures.

In the case of Perge, it is known that the site was abandoned in the 7th century; and as a result the towers suffered the progressive effect of aging in the absence of any preventative measures. This phenomenon becomes apparent primarily at a material level (stress concentrations, local collapse), and presents itself at a global level in the form of instability and vulnerability to occurrences. In this sense, the structure's physico-mechanical characteristics, level of degradation and sources of degradation at a material level should be well established in order to verify the involvement of material character in structural performance. For this purpose, a full geological investigation involving a field survey and laboratory research was carried out by Topal (2010),[11] including foundation characterization (soil layers beneath towers) and a seismic fracture direction investigation.

[11] In a sub-branch activity of the "Perge Hellenistic City Gate Towers Restoration Revision Project", a consultation agreement was made between METU-SAYKA (No: 10-03-09-1-00-14), and the ensuing works were coordinated by Prof.Dr. Tamer Topal in the name of the METU Geological Engineering Department.

The site is located on top of a travertine deposit at the northern “Asar” Hill top most rock formation of travertine is 150 m deep. Underlying this formation is a “Kurşunlu” formation of sandstone, followed by a “Yenimahalle” formation of marl, clay, siltstone and sandstone (Topal, 2011). Therefore, the structural materials are mentioned to be carved from near quarries (Topal, 2011). Some material investigations were made on the specimens gathered from near travertine formations to the Perge City by Topal (2011) in the framework of the PHCGTRRP.

The wetting-drying cycle tests carried out by Topal (2011) revealed that the travertine stones used in the towers were very weak against continuous water exposure. The tested travertine samples (5x5x5 cm cubic specimens) showed ~66% decrease in their uniaxial compressive strength and ~100% increase in their porosity level, and secondary cracks were detected in the body of the sample after 20 cycles. Furthermore, a change of colour was witnessed, from cream to brown, which could be detected easily with the naked eye. This color change was also viewed on the Hellenistic Towers during site visits, leading to the belief that the material decay at the lower levels of the structures was a result of the continuous exposure of the lower courses to water during winter floods (Figure 2.16).

Schmit Hammer testing carried out at site by Topal (2011) revealed that the ratio of strength values between the lower and higher levels of towers was nearly 2:3. This decay, related to water exposure, is seen mostly on the surface of the wall stone elements. Therefore, this cannot be deduced as the sole cause of structural decay. However, it is known that the lower surfaces of the wall may be subjected to a high concentration of stress during lateral loading; and consequently it can be assumed that the structural vulnerability is based on a combination of both material decay and stress distribution due to the load-bearing system.

Figure 2.16 Surface Decay at Lower Levels of the Inner Surface of Tower Walls due to Continuous Wetting Drying Cycles (Topal, 2011).

Perge has a well-spread infrastructure, however after being left to ruin for thousands years the system has become blocked with fill. The city is often exposed to flooding due to the non-functioning drainage system. In order to understand the effect of this poor drainage system and its effect on the structural decay of the towers (possible swelling-shrinkage cycles in the ground carrying the foundations must be the reason for the tilting of the towers), geophysical soil layer investigations were carried out by the Geology Study Team of PHCGTRRP. The tests revealed that the soil is wholly composed of impermeable layers of clay material (1–6 m depth; clay silt, 6–35 m; clay and 35 m < main rock). No clues were detected as to the extent the permanent deformations and foundation movements were due to alterations in the level of the water table.

It is known that no significant structural alterations were made to the towers due to weaknesses or local collapse before the area was abandoned. It can be considered that the confining effect of the roof structure and the floor slabs, in addition to the thickness of the walls and the “arched plan geometry” allowed them to remain intact. After the structures were abandoned and left to ruin, both the material and structural decay rates should have been accelerated because the towers were much more drastically exposed to climatic actions.

As previously defined, the towers are circular in plan, with the thickness of the walls accounting for 33% of the diameter. This significant wall thickness was common in Hellenistic fortifications, and was based on the need to withstand high impacts during sieges. In addition, the circular geometry assists the structures in resisting lateral loads from all directions, while also preventing edge zone stress concentrations. The towers were built to a height of 20 m from ground level, so as to allow control over larger distances, however this would have resulted in a decrease in the flexural capacities of the circular walls when compared to similar structures of that age.

In Chapter 2.2 it is defined that the structures were built using ~0.5 m^3 travertine blocks, laid in a staggered pattern in both horizontal and vertical directions. The 15 cm (on average) bond between each stone result in a contact and stitch between not only the neighbouring ones, but also those laid oppositely. However, due to a lack of uniformity in the dimensions of the stone blocks due to low-quality workmanship, the distance between two adjacent bed joints and the staggers in the wall section joints decreases to 5 cm in some areas, resulting in a weakness in the stitching in a vertical direction. Cracks and deformations can be viewed following the paths of closely staggered jointing, and in these areas the flexural resistance is lower than in other parts (Figure 2.17).

A clear difference in wall elevations around the whole circle can be seen on the ruins of both tower ruins. Flexural cracks that follow weak staggering zones and openings such as windows can be distinguished with the naked eye. It is obvious that much of the structural integrity was lost due to initial partial failures, and the gradual collapse of wall segments has occurred due to the distinct decrease of structural inertia. Being proof of a hundred years long - staged deformation, the huge mass that was documented at the end of 19th century by Lanckoronski *et al.* does not exist today.[12]

[12] The defined mass of blocks was recorded as being on the north-western side of the third floor window of the Eastern Tower. This block mass comprised a huge block with shield carvings and 4–5 blocks surrounding it, as documented by Lankoronski *et al.* in 1890 (Figure 2.7), however these blocks are no longer in place.

Figure 2.17 A Vertical Crack at the Eastern Tower Following a Pattern in which Non-Successful Vertical Staggering at Stones exists. Photo by SAYKA (2002)

Traces of the source of collapse were sought all over the site; and the collapse mechanism of the towers was compared with other structures to find similarities that would indicate a source of the fractures.

Figure 2.18 Overturned Columns in North-South Direction at the East-West Colonnaded Street, 16.07.2010

During the field survey, it was observed that some parts of the structure, especially in the city walls, had remained intact at high elevations. As a general observation it can be stated that some walls had remained intact, while oppositely placed walls related to the same building had suffered collapse, even to foundation level. Flexural cracks, occurring due to stress concentrations at wall intersections related with the independent flexural movement of the walls under lateral loading, were also detected frequently. The column remains along Colonnade Street, which forms the east-west axis, were also investigated. Looking at the distribution of previous archaeological excavations in Perge (Bakacak, 2007), it can be stated that this street has not been studied in as much detail as the main axis. Hence, it can be considered that there has been no intervention that might have caused alterations to the layout of the collapsed column parts that line the street. It was viewed that the columns had generally overturned in same north-south direction, leading to the belief that seismic activity directed along the north-south axis may have been responsible (Figure 2.18).

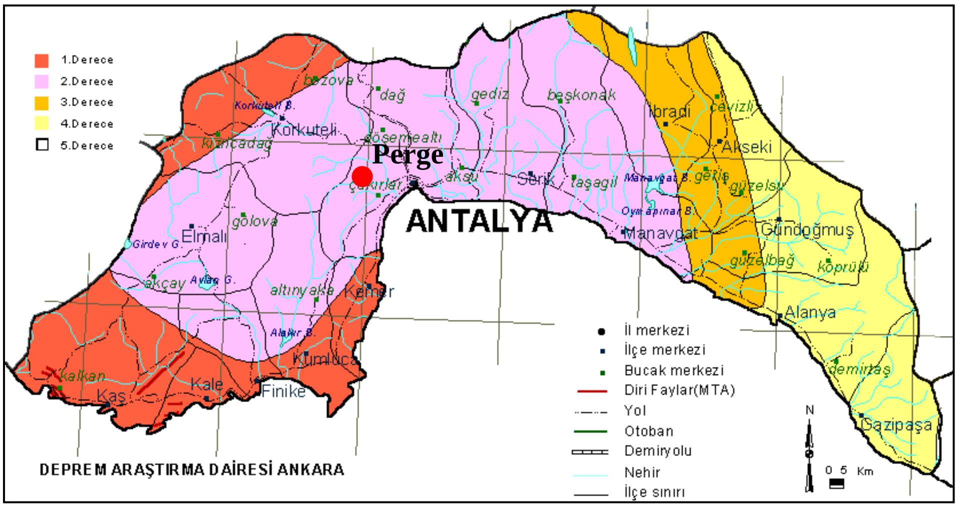


Figure 2.19 Earthquake Map of Antalya Province (retrieved from:http://www.deprem.gov.tr/sarbis/shared/DepremHaritalari.aspx , 2011)

On a map of Turkey’s Earthquake Zones (Figure 2.19), Perge is defined as being located in a second EQ Risk Region of 5 levels. Although a catastrophic earthquake has not been recorded in the region in the last century, it is obvious that the area is at risk of a destructive EQ occurrence. The Mediterranean has always been prone to geological hazards, such as volcanic eruptions, tsunamis and earthquakes; and in the Hellenistic era earthquakes resulting from volcanic eruptions were common all along the Mediterranean coast that were impressive enough to have been spoken about by philosophers (Kozak, 2010). Catastrophic occurrences and losses due to earthquakes were also recorded by historians over the centuries up until the Middle Ages. Seismic activities struck such important urban areas as Lycia, Antioch, Crete and Cyprus in the Mediterranean (Ambraseys, 2009). Although not at the centre of a seismically active region, shockwaves related to catastrophes in the wider vicinity certainly affected the Antalya Region (Ambraseys, 2009). In the Middle Ages it is known that earthquakes struck with epicenters in the Pamphlyia Region. One particularly destructive earthquake hit Antalya on the 8th March, 1743, causing severe damage to local housing and provoking landslides and rock falls (Ambraseys & Finkel, 1995). From this data it can be assumed that Perge also would have been affected by this series of earthquakes, which would have had a major destructive impact on the tall Hellenistic Gate Towers of the city. The results of the geological survey and observations reveal that three local active seismic fault lines may affect the area, running from north to south; from north-east to south-west; and from east to west (Topal, 2011). The relationship between the directions of these fault lines and the destruction of the towers is given in Figure 2.20.

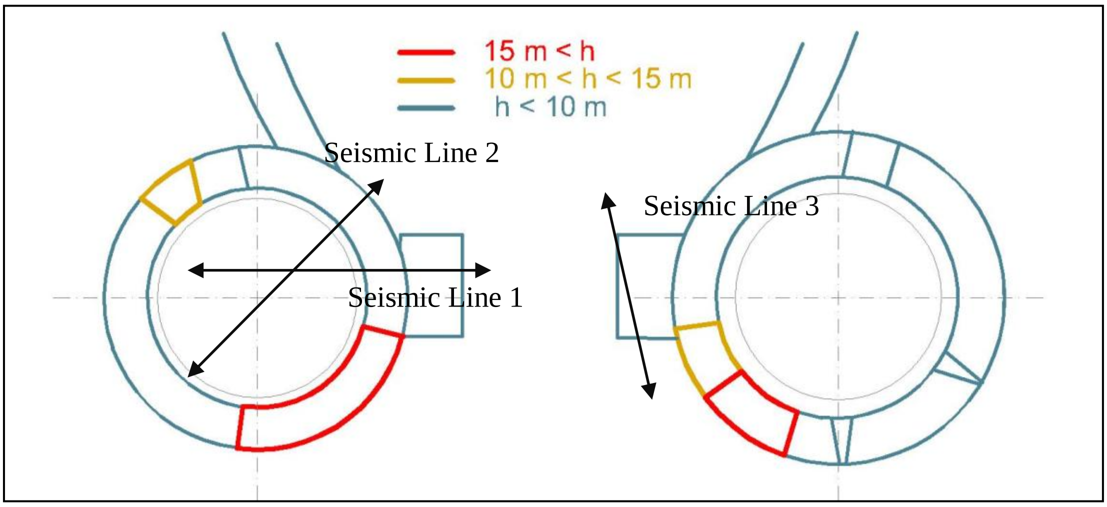


Figure 2.20 Seismic Fracture Directions defined by Topal (2011) On the Non-Scaled Plan Drawing of the Towers

2.4 Previous Archeological and Conservation Studies on the Towers

The first systematic and organized archeological study at Perge did not take place until 1946; and studies of the site have been carried out by Istanbul University, Department of Classical

Archaeology up until today. Excavations started with a season-long study by an archaeology team led by Prof.Dr. Arif Mufit Mansel in 1946. After a six-year break, the studies continued between 1953–1957. The last excavation organized by Dr. Mansel took place between 1967–1974; after which excavations coordinated by Prof.Dr. Jale Inan continued between 1975–1987. Since 1988, continuous archaeological studies have been directed by Prof.Dr. Haluk Abbasoğlu.

During the 1953–1954 excavations, the first archaeological studies were made of the Hellenistic Gate and the courtyard. Activities were concentrated on clearing the inner side of the courtyard, and cataloguing and storing the statues and valuable architectural elements found during the excavations around the Gate Complex (Mansel, 1954), and in the following year the Triumphal Arch was fully exposed. During the 1970s, 1/50-scaled drawings of the towers and courtyard walls were prepared by the METU Architectural Photogrammetry Center, however at the time the lower 4–6 stone courses above ground level could not be documented due to the presence of debris, covering both the perimeter and inner sides of the towers. The studies of Prof.Dr. Jale İnan focused not on the city gate, but rather on cataloguing the fallen stone blocks in the courtyard, and uncovering the statue bases bearing inscriptions in the niches. In the 1990s Prof.Dr. Haluk Abbasoğlu and his team continued their studies of the Oval Courtyard and the Triumphal Arch to determine their construction dates and forms (Abbasoglu, 1996b).

In 2001, a private restoration firm, SAYKA, started work in Perge in order to update previous documentations and to prepare a restoration/conservation project. The studies were conducted with the co-operation of an Archeological Team led by Dr. Abbasoğlu, with additional consultancy provided by the METU Restoration Program and an engineering firm. Drawings made by METU were used as the basis of the project and were updated for the later stages of the conservation project (SAYKA, 2001). To perform a comprehensive structural investigation of the towers, the documentation and interventions carried out on the towers in earlier periods were studied in detail by SAYKA. The results of these studies are summarized below.

The following problems were defined:

- Material loss, which is a common problem at archaeological sites
- Wall segments formed by large groups of stone blocks had lost their bearing capacity, in particular on the 3^{rd} floor remains of the Eastern Tower and the thin wall segment remaining on the northern part of the Western Tower
- Separation of joints in walls (both <10 cm and >10 cm)
- Cracks in the bodies of travertine blocks
- Surface loss due to decay of the stone blocks

Considering the above-listed problems, a consolidation oriented restoration project was prepared by SAYKA for the towers and courtyard. The restoration project prioritized the interventions listed below:

- Partial reconstruction using newly cut stones to provide additional stability to the structure

  - A staged reconstruction of the two edges of the southern wall segment (consisting 2–3 stone blocks at each level) of the Eastern Tower. The eastern addition reaches up to the top level of the tower, while the western addition was proposed to reach the start of the 3^{rd} floor

  - A similar reconstruction was proposed for the Western Tower, including the reconstruction of four entire courses, starting from the 15^{th} course.

- Dismantling and reconstruction of weak spots where bearing has been lost with metal clamps and mortise connections
- Injection of a repair mortar into the cracks and at joints that have suffered separation of less than 10 cm
- The filling of joints that have suffered separation of larger than 10 cm with an imitation stone and injection with a repair mortar to fill the gaps

- This activity was proposed not only for the towers, but also for the courtyard walls. The objective is to fill the large cracks starting from the lower segment of the Eastern Tower up to the top floor window span

- Plastic surface repair of the stones that have suffered >10 cm deep surface decay/loss
- The implementation of a steel lintel system developed by a structural engineering consultancy firm was proposed for both towers[13]
- Anchorage of a steel box-frame at the 2nd floor window of the Eastern Tower to prevent the progression of the largest crack through the window span.

The project was accepted by the Antalya Regional Council for the Conservation of Cultural and Natural Assets in May 2002, and after a gap of 6–7 years, applications started in 2008. Excavations, and the systematic removal and cataloguing of stones that covered both the surrounding area and the inside of the towers were made at the beginning of the application process. The stone blocks were documented and were systematically laid out in an empty area located between the Late Antique Gate and the towers. The applications proposed for the Eastern Tower called for the erection of a scaffolding system, and consequently a steel two-sided scaffold was constructed at the start of the implementation period. The southern part of the scaffolding is designed to be carried by four legs based at ground level, while the northern side is supported by the 1st floor projection of the tower wall. The adjustment and reconstruction works defined in the project were carried out for:

- 3rd floor part of the Eastern Tower.
- 2nd and 3rd floors edge stones.

  - The newly laid stones were clamped to the existing stones.

- Repair mortar injection to the adjusted areas and filling of large gaps with imitation stones on some locations of the Eastern Tower.
- Filling of gaps, finishing of decayed stones and capping of the top level of the courtyard walls.

[13] Details of the system are given in Section 2.5.

Excavations to clear the piled rubble were completed both on the outside perimeter and the inside of the towers. The stone courses (lowest 9–10 courses) that had not previously been documented were exposed and documented. For the first time, the foundation system was investigated through the excavation of a ~1 m^2 large hole at the ground level of the Eastern Tower. Some archaeological remains, both moveable and unmovable, were found during these excavations, the most interesting being a Byzantine Mosaic, found at the ground level of the Western Tower during the excavations.

(a) (b)

Figure 2.21 (a) Reconstructed 3rd Floor Part of the Eastern Tower. (b) Readjustment and Attachment with Metal Clamps of 25th Course Western Edge Stones at the Eastern Tower, 16.07.2010

Consequently, by the end of 2010 certain parts of the Ministry of Culture & Tourism-funded project, had been applied. At this point the Ministry decided to improve the existing Restoration Projects with the participation of a multidisciplinary group, including architects, archaeologists, engineers and conservation scientists, who cooperated in both problem diagnosis and the development of solutions.

The revision of the Restoration Project continued to be carried out by SAYKA throughout 2010. A multidisciplinary group formed by SAYKA, including an Istanbul University

Classical Archaeology Team lead by Dr. Abbasoğlu and a number of METU Research Groups, performed the revision work of the 2002 projects, and the work was completed after one year of investigation and research. In May 2010, the Antalya Regional Council for the Conservation of Cultural and Natural Assets approved the revision to the 2002 Project.

(a) (b)

Figure 2.22 (a) Steel Support Frame Anchored to the Inner Side of 2nd Floor Window of the Eastern Tower. (b) Consolidated Part of Courtyard Wall with Filling and Injection (SAYKA, 2009).

2.5 Previous Structural Investigation Studies and Strengthening Proposals for the Towers

To date, only three study groups have investigated the structural performance of the towers through a visual inspection or using computer-based techniques. Following the structural performance estimation studies of these researchers, some strengthening suggestions were also developed.

To date, only three study groups have investigated the structural performance of the towers through either a visual inspection or using computer-based techniques. Following the structural performance estimation studies of these researchers, some strengthening suggestions were also developed.

1^{st} Proposal: Lintel-Girder System: As a part of the 2001 Restoration Project for the towers, an engineering firm, working as a consultant, developed a strengthening method that included the strengthening of the towers with the confinement of the wall segments through the application of a lintel system at the transition levels between the 1^{st}–2^{nd} and 2^{nd}–3^{rd} floors. A steel arch segment plate which was proposed to be bedded on the projecting stone layers (at the 16^{th} & 25^{th} courses), and anchored to every stone element beneath it. Another, thinner plate was proposed to be inserted at the interface between the two adjacent stone courses above the projecting stone course. The two parallel plates were to be bonded using anchor bolts, and in this way bonding of inner side stones of 17^{th} and 26^{th} courses were supposed to be fulfilled to form lintel course. Consequently, the bonded stone rows would act like tie beams, increasing the flexural capacity of the entire wall. Details of the steel system can be seen in Figure 2.23–Figure 2.26.

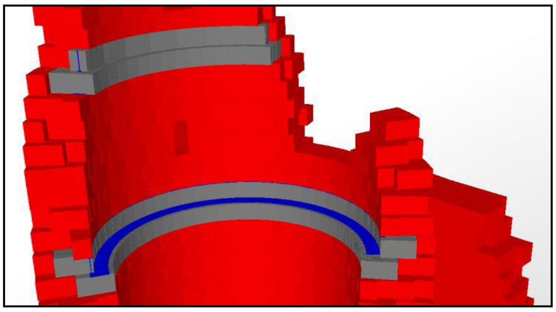


Figure 2.23 3D View of Lintel System and Eastern Tower (Lintels on 16^{th} and 25^{th} layers)

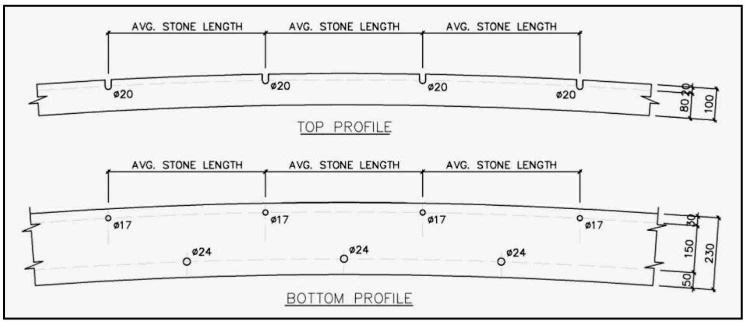


Figure 2.24 Steel Support Frame Anchored to Inner Side of 2^{nd} floor Window of the Eastern Tower

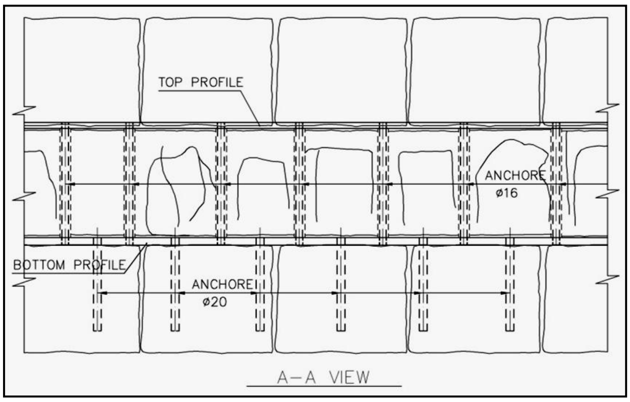
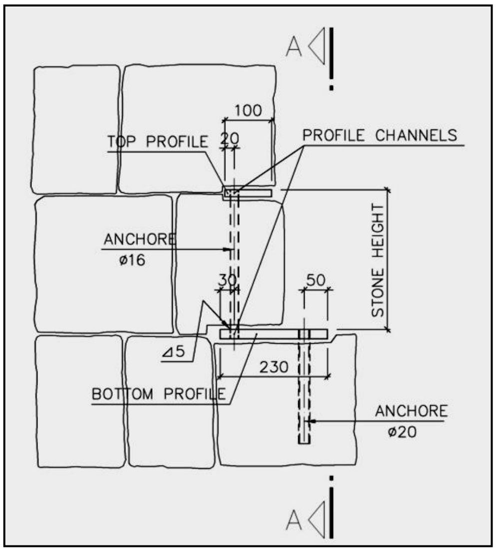
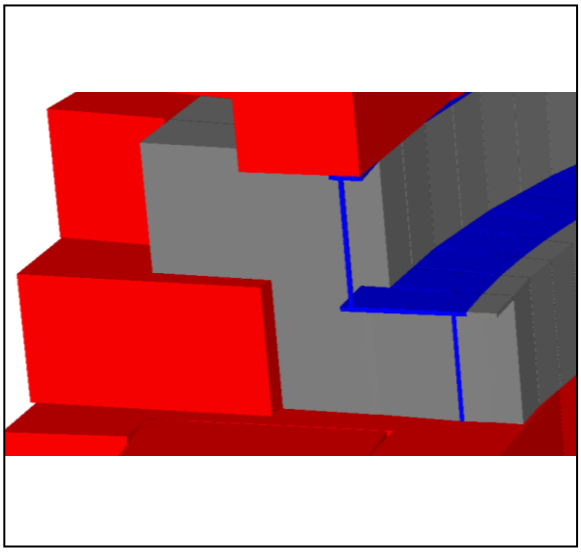


Figure 2.25 Frontal View, Drawing of the Lintel System SAYKA (2002)



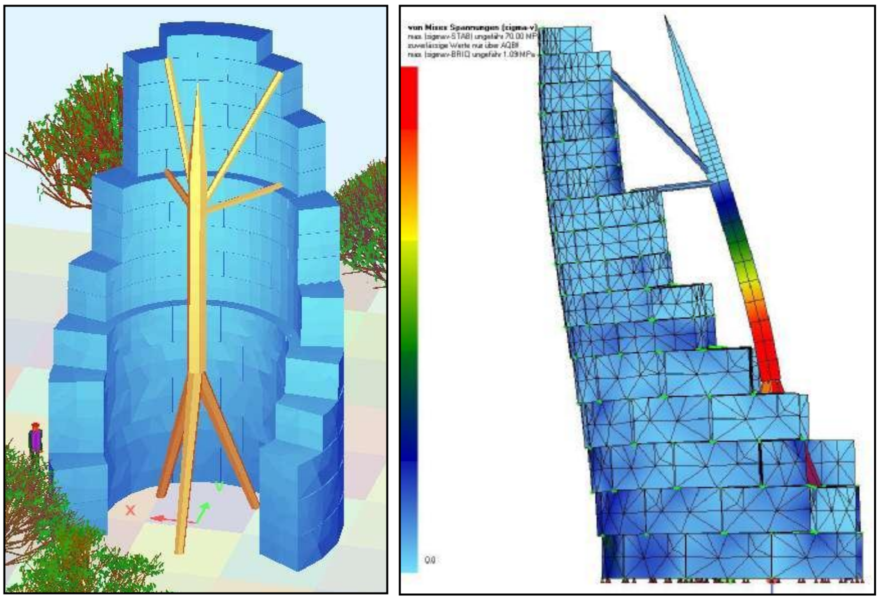


(a) (b)

Figure 2.26 (a) Side Section of Wall and Anchored-Lintel System Drawings SAYKA (2002). (b) 3D Side View of the Lintel System

2nd Proposal: A Tripod system proposal was made by a group of Construction and Design engineers from the Kaiserslauten University of Applied Science (2010). Different from the previous study, the performance of the suggested system was investigated through non-linear analytical modelling. The stability of the towers was proposed to be improved by increasing the inertia of the towers against lateral effects. It was suggested that a steel 3-legged tree-like support system to be constructed inside the tower remains would result in the requisite increased inertia. The “trunk” of the tree was to be a Φ800 and 14 m high main support, with four Φ600 pipe section sub-branches that were to be projected towards the top floor corners and stitched into the masonry. Hence, the overturning risk of the upper wall segment was aimed to be prevented from the inside the tower. In order to ensure the stability of the support system, a rigid foundation system was also planned. The main branch was to sit on three legs made of Φ600 pipe section. Each leg would transfer its loading to a ~1m deep square pad foundation, under which four 7 m long φ300 pipe section micro piles would provide a connection with the soil layers (Figure 2.27-a).

The performance of the system was investigated by researchers using a discrete modelling technique with non-linear material properties. A push-over analysis performed by the team revealed that the system would be able to prevent collapse under lateral loading so long as a perfect bonding between the steel sub-branches and the wall could be provided (Figure 2.27b).



(a) (b)

Figure 2.27 (a) Tripod System 3D View. (b) Discrete Analytical Modelling, Pushover Analysis - Limit State Result Representation.

3rd Proposal: Inner steel frame tube. This was a conceptual idea developed by an Italian consultancy team led by Prof.Arch. Gianni Perbellini. The intention was to support the towers from the inside with another tower constructed using a steel frame system. The system was planned to be constructed to at least the height of the masonry towers to maximize performance. The aesthetic problem resulting from the bare steel frame system was proposed to be overcome by applying a sleeve over the modern tower, like a pipe (Figure 2.28).

(a) (b)

Figure 2.28 (a) View of Suggested Support System and the Towers from the South. (b) Southern View Drawing of the Towers and the Support System (Inner Steel Frame Tube) Perbellini (2009)

CHAPTER 3

MODELLING OF PERGE TWIN TOWERS AND STRUCTURAL ANALYSES

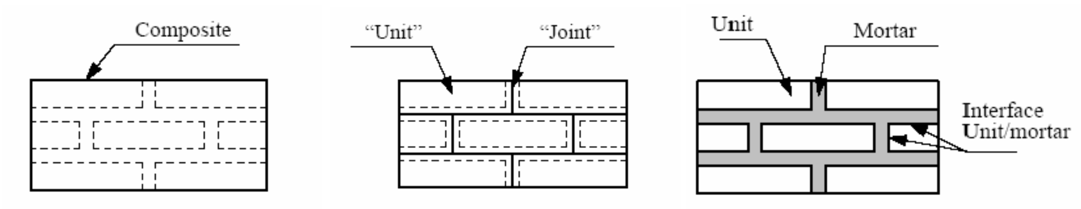
In this chapter, the study is concentrated on estimation of structural characteristics and problems of towers which is then followed by intervention decisions in the next chapter. The methods used for the generation of analytical models were defined for Perge Hellenistic Gate Towers' original form and current geometries.

In order to understand damage sources and collapse mechanisms of towers; linear dynamic analysis in SAP2000 software and nonlinear dynamic analysis in Ls-DYNA and ABAQUS software were made. The remaining parts of the towers were assessed to see if they are collapse critical using hand calculations and analytical models. Verification of analytical models s was made based on analysis of dynamic data gathered with field ambient vibration testing. Comparison of natural vibration frequencies obtained from both real structures and analytical models allowed verification of analytical models. The determination of uniaxial compressive strength of towers' masonry walls were investigated with laboratory experimentations.

3.1 Review on Structural Investigation by Analytical Modelling of Historic Masonry Structures: Discrete Element Modelling

Different from modern structures, historic buildings inherit many uncertainties about their construction materials and structural details. Lack of being built according to any standardized modern engineering calculations and design, estimation of structural features of this type of buildings is a difficult task. Combined with this issue, masonry construction possesses a complex medium that includes more than one material having different physico-mechanical properties. Then it becomes a struggling task to rationalize structural behaviour of historical masonry structure with numerical and mathematical methods. In this sense, analytical models ease the task by solving complex mathematical relations that may be valid for historic masonry structures.

However, in order to obtain realistic results, proper representation of masonry elements is a must. Several analytical member types and solution methods to represent masonry have been developed in last decades. Representation of masonry piers, spandrels, and shear walls with simplified (equivalent) finite frame elements was an initial attempt to reach global behaviour of historic masonry structures in a virtual environment; however, determination of realistic results about a shear deformation dominant structure with these methods appeared as another problem. Therefore, masonry structure elements were started to be modelled with planar elements of shells or solids. Today macro, discontinuous, micro finite element models and discrete element models are used for this purpose.



(a) (b) (c)

Figure 3.1 a) Macro model b) Simplified Micro Model c) Detailed Micro Model

In macro modelling to minimize the mesh intensity and the number of degrees-of-freedom, the brick unit, mortar, and their interfaces are smeared in a homogenous continuum, which is meshed regardless of the masonry units' locations (Figure 3.1-a). However, homogenization of masonry requires realistic definition of anisotropic yield and plasticity concerns of masonry walls. Therefore, macro-models are formed based on case-related laboratory investigations. On the other hand, smearing existing discontinuities such as joints and cracks, causes inappropriate definition of the damage simulation consequently may result in insufficient definition of the targeted structure

Macro-FEMs that are separated by discontinuities at the location of large existing cracks are also being used to define structural damage in analytical form to overcome the phenomenon of insufficiently defined target structure by using smeared modelling. Discontinuities in the form of constructional joints, severe cracks, which had been detected to be indicating structural failure, were added to macro models as discontinues interfaces to visualize possible failure mechanisms of mass separations, shear sliding, flexural rotations.

The most appropriate modelling of masonry behaviour in analytical environment can be provided by using micro models in which separate anisotropic mechanical properties of mortar and unit is implemented; however, the analytical models can become too large to be solved using currently available computer technology. To overcome this difficulty, homogenization for half part of bed and head mortar joints and brick units are made in simplified micro-models while the interface between each simplified micro-model is still defined and formed separately (Figure 3.1-b). Using simplified micro-models enables the reduction of degree-of-freedom (and node) numbers and introduction of flexibility in definition of new units, which may not strictly fit into original masonry unit-mortar pattern. However, the homogenization of materials should rely on exact mathematical definitions related with experimental material investigation. Furthermore, the pattern of the masonry should be organized to allow simplified micro-model definition; irregular masonry pattern may not be suitable for simplified micro-modelling.

On the other hand, detailed micro-model consists of separate definition for brick unit, mortar, and unit-mortar interface is used to take influence of every constructional detail in the structural response of the analytical model (Figure 3.1-c). The most accurate benefit of micro-models is that "all the different failure mechanisms can be considered." Roca (2010), whereas the increased number of joints and therefore degrees-of-freedom makes it impractical to solve and obtain analysis results with the currently available computer technology. Furthermore, the large number of equations to be solved in a single analysis causes numerical errors to be accumulated during simultaneous linear equation solution and the accuracy of the results may be adversely affected by accumulated numerical errors. Another problem associated with detailed micro-modelling is the increased number of structural parameters which may not be accurately known. The 'uncertainties' associated with the various material characteristics, interaction between different material layers, etc brings additional sources of error in structural analysis.

Detailed discrete element models (DEM) may be considered to be an improved version of micro models and are formed by representing every masonry unit as part of a discrete assembly and definition of unit-to-unit interfaces separately. The DEMs are usually nonlinear in solution; therefore, there might be conversion problems as well as extended amounts of time to complete analysis since iterative solutions are necessary. This technique has been implemented on the cases of heritage structures, which have simple geometry or uniformly shaped masonry elements like cut stone blocks.

As a distinct example of DEM, 2D model of a barrel vault from Mattera were investigated under vertical loading to their limit states (Baggio & Trovalusci, 1998). In the study, analysis with discrete dry block stones and friction based interface enabled the modelling of sliding, rotation, and separation at joints; therefore, a realistic failure mechanism was able to be determined.

Another study targeting structural behaviour of a severely damaged building bound to the earthquakes is S. Vincente Monastery's inner cloister façade modelling (Pegon *et al.* 2001). The study consists of both laboratory push-over testing on real cloister model and nonlinear analytical modelling to make comparison between each investigation to form a base for further global models of structure. In analytical model the masonry system including dry cut stone masonry pillars, arches, and brick masonry upper walls was formed by using discrete elements for stones and wall segments. But also meshing of each discrete element enabled the visualization of stress and strain distribution on structure in detail at the limit state. The study resulted in a good correlation for both real and analytical investigations which are concentrated on cyclic behaviour of cloister.

A part of the Parthenon Pronaos was studied with analytical discrete element modelling in which column drums and architrave stone blocks were represented as separate assemblies (Psycharis *et al.* 2003). All stone blocks were meshed, thus imperfection of structure was able to be fully modelled. The integration of metal clamps were added into the interface behaviour. The effect of architrave stones on global structural stability against seismic activity was investigated in this study. Effectiveness of strengthening ideas was also estimated as a result of modelling studies. Seismic vulnerability study was performed with DEM for same type of structure of Concordia Temple Ruins (Bisswurm *et al.* 2010). Possible failure patterns and conditions related to the intensity and type of earthquake is mentioned as a result of the study.

All these studies show the suitability of discrete element models to be used as a tool for structural analysis of block type historic masonry structures under the effect of seismic actions. In this sense, DEM tool was selected to tackle structural modelling problems of the Hellenistic Gate Towers of Perge.

3.2 Analysis Technique and Tools

As the amnesia activity of Perge Hellenistic Twin towers, in-field study of preliminary diagnosis based on visual inspection and adequately gathered structural/material/geometrical data led to detailed structural investigations using DEM based simulations and analyses. Two types of analysis were performed with a) linear-elastic macro model and b) nonlinear discrete element models (DEM).

Preliminary ideas about structural behaviour of towers were tried to be captured by linear models while more detailed examinations were followed by using DEM. Study also enabled a comparison between these two types of tools regarding their capability to provide realistic results about dry masonry structures. In order to establish a correct diagnosis, it is important to evaluate target buildings' original (that belong to earlier periods) and current structural characteristics. Therefore, study was initiated with formation of models referring to original form before any structural collapse of the towers. In this sense, sources of structural decay, possible accruing patterns of the collapse, relation between construction techniques and the decay were tried to be determined before recent structural characters and problems are investigated.

Linear elastic analyses were made using SAP2000 Software. Modal analysis and dynamic time history analysis were made for the Eastern Tower. Results of modal analysis were used to verify linear models by comparing them with natural vibration periods belong to real structure[14]. In time-history analysis, effect of earthquake loading on the original form of the Eastern Tower and stress distribution on overall structure were evaluated with SAP2000 analysis to determine weakness zones on structure due to earthquake loads.

[14] The methodology of work done to acquire dynamic features of the Todays Eastern Tower and related results are provided in Chapter 3.5.

In the case of dry stone masonry structures, the interaction between each building block has contact and friction issues which is expected to be more complex than a standard linear FEMs can simulate; therefore, geometry of Perge twin towers were decided to be also investigated using discrete element modelling (DEM).

DEMs were formed at relatively complicated analysis software of Ls-DYNA and ABAQUS. Preliminary nonlinear inspection was made using Ls-DYNA models. Effect of masonry construction technique on global structural behaviour, in the case of towers to be constructed with stone blocks that had been staggered in horizontal plane was estimated with the Ls-DYNA time history analysis on the DEM of Eastern Tower. Further investigation of collapse sources was performed with ABAQUS model, which had revisions and improvements of geometrical stone staggering with respect to the initial Ls-DYNA model. Architectural elements of neighbouring structures, such as courtyard wall and entrance tunnel, were added and time history analysis accompanied with self-weight analysis (in which material decay effect was also included) were made with ABAQUS. Relation between possibly existed architectural elements in lacuna and global structural behaviour were included in investigations to support archaeological studies. Furthermore, recent ruined geometry of the Eastern Tower was analysed under lateral loading to understand further possible collapse risks.

In addition to all these modelling studies, further modified ABAQUS models, details of which are given in Chapter 4, were formed for previous and newly developed strengthening methods. Effectiveness of the strengthening methods was studied using modified ABAQUS analytical models.

3.2.1 Formation of Model Geometry of Superstructure

As it was previously stated, the analytical modelling studies were concentrated on the Eastern Tower geometry. Although towers had deformed in different geometrical alterations, the source and response relations, that was formed with visual inspection, exhibits same way of structural behaviour. Therefore, it was though that concentrating on the Eastern Tower, which had deformed in a relatively simpler shape, could be studied with smaller time of investigation.

Required geometries were sketched in programs having CAD properties (SAP2000, Rhinoceros and Gambit). SAP2000 model was formed in a mesh of 3D 8-noded solid elements. For DEM models 3D 8-node brick elements were also used; however, they were assembled as separate individual stone blocks. Therefore, in every joint moment and tensional force transfer was removed and nonlinear analyses were performed (different from the case at SAP2000 model).

The architectural form of the structure in SAP2000 and Ls-DYNA were formed in similar shapes. The reconstruction geometry of Lancoronski's study (ref?) was taken as the basis for analytical modelling, where all openings' numbers and shapes were defined according to this study. Especially loophole and window directing toward city were also added. In both of the models existence of Roman Gate (later period gate of eastern side) was ignored. In addition, structural elements like gallery wall and courtyard wall were not considered as they were not required to be added to these 'preliminary' models formed at SAP2000 and Ls-DYNA. The preliminary models were made to reveal initial results related with load bearing mechanisms and effect of constructional technique on load bearing character. The participation of those structural elements was assumed to be minimal which makes it possible to be ignored in the preliminary models.

In Ls-DYNA DEM (different from linear elastic models), all structural stone block elements, which are ordinary wall stone block units, lintel stones, shield carved ornamental stones, gate arch voussoirs, were represented with an individual assembly as closely as possible to their original dimensions[15].

More than one ABAQUS discrete element model was formed. Eastern and Western towers' original shapes (forms that were assumed as the shapes right before severe global destruction) were established with the similar rate of detail at Ls-DYNA model, however courtyard and gallery walls are also added. Furthermore, models (including suspicious architectural elements like Eastern Roman Gate at the Eastern Tower) were modelled to question their existence and their possible structural effect on the collapse.

[15] In analytical discrete element models similar stone block elements were defined in uniform dimensions which had been assumed as a result of dimensional optimization related to drawings of SAYKA (2010).

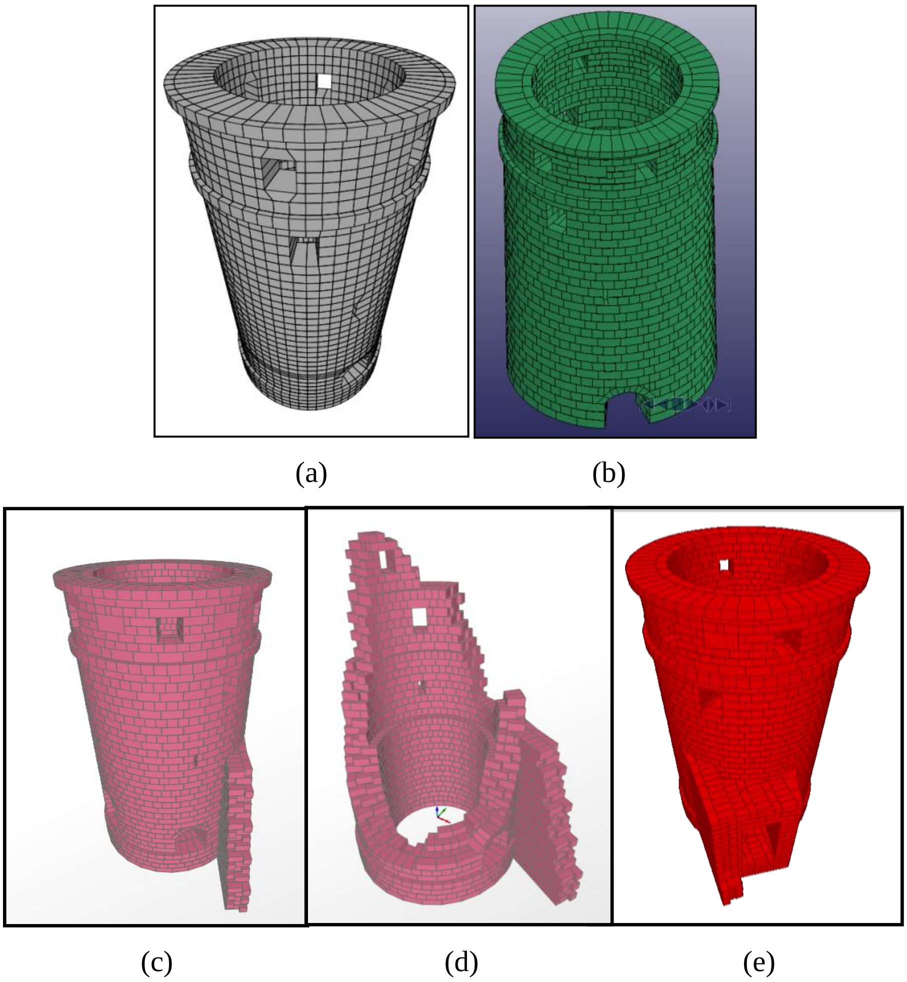


Figure 3.2 a) SAP2000 Model of the Original Eastern Tower b) Ls-DYNA DEM of the Original Eastern Tower c) ABAQUS DEM of the Original Eastern Tower d) ABAQUS DEM of the Ruined Eastern Tower e) ABAQUS DEM of the Original Western Tower

In all of the models speculative building members of timber slabs and roof was avoided in the modelling process. Because, it was considered that the structural decay should had started long after this confining part had been lost. Therefore, the roof structure that starts at 40th stone layer was represented with an additional stone layer projecting outward in the models.

3.2.2 Masonry Unit Physico-Mechanical Parameters and Unit Interface Models used in FEM

Input data to define material properties and dry masonry wall construction in the manner of physico-mechanical parametric definition was supported by laboratory experiments made by Topal (2010) on travertine stone material. Dry masonry wallet uniaxial compressive strength testing was also made in the scope of this thesis. In both linear elastic and nonlinear DEM analyses, stone blocks were assumed as exhibiting linear elastic behaviour. On the other hand, the major concern about structural behaviour of interfacial friction behaviour between stone blocks was defined with simple Columb's isotropic stick/slip (static/dynamic) frictional behaviour.

3.2.2.1 Physico-Mechanical Properties of Travertine Units and Masonry Wall due Laboratory Experimentation and Specifications

Perge Hellenistic Gate Tower's masonry unit, *travertine* physico-mechanical properties were examined by Topal (2010) in the scope of consultancy work of PHCGTRRP. Density porosity, ultrasonic pulse velocity, uniaxial compressive strength test were made on 7x7x7 cm cubic samples that were cut from stone specimen which are belong to Asar Hill. Moreover, a group of similar samples was exposed to wetting drying cycles to investigate aging effect of travertine. Results of experiments, which were used as input parameters for models, are given at Table 3.1.

Although uniaxial compressive strength of travertine masonry blocks were revealed, it is important to form relation between unit features and masonry wall characteristics by also considering constructional techniques. Because towers were being constructed in heterogeneous blocky form instead of homogenous form like a solid pipe; it is not feasible to assume masonry walls would also comprise a compression strength same as a unit block itself. Therefore, laboratory experimentation was made in order to investigate constructional features on strength capacity of walls.

Table 3.1 Physico-mechanical features of travertine stones of the towers

Parameter Name	Value
Dry Unit Weight (kN/m^3)	17.27
Wet Unit Weight (kN/m^3)	19.61
Porosity (%)	23.9
Dry Uniaxial Compressive Strength (MPa)	15.79
Dry Uniaxial Compressive Strength (MPa)	14.45
Average Uniaxial Compressive Strength (MPa)	15.0
Tensile Strength (MPa)	1.5 (Assumption ~1/10$f_{m'}$)
Poisson's Ratio	0.25 – 0.28
Static Frictional Coefficient	0.7
Dynamic Frictional Coefficient	0.3
Young' Modulus (GPa)	2.5

Andesite blocks of 1/20 scale were prepared in similar geometrical shapes of original structure units. Uniaxial compressive strength testing was made on scaled units, prisms and wallets that were constructed in similar orientation of stones at original tower walls. As a result of testing, it can be said that for andesite blocks, which are uniaxially loaded, first cracking (can be named as yield strength) occurs between ~1/3 to ~1/4 ratio of the ultimate strength and same ratio is followed in the wallet specimens. Furthermore, the ultimate strength of wallet stays at about 1/4 ratio of one single block strength. Therefore, if same relationship is assumed for travertine, the yield strength of wall should be 1/9 to 1/16 of the material ultimate strength while ultimate strength of wallet stays at about 1/4 of a single blocks' compressive strength.

Maximum allowable uniaxial compressive load on masonry wall according to ACI530-02/ASCE 5-02/TMS 402-02 "Building Code Requirements for Masonry Structures" was limited with 1/4 of the buckling load (P_e). From Equation 3.2, buckling load is found as 131.77 MN for eccentricity (e) = 0.3 due to the thickness difference in wall segments along the height. The permitted axial load capacity of masonry wall is obtained by 1/4 of P_e which gives 32.94 MN (3358.13 ton) for the Eastern Tower wall, which reaches up to highest altitude, defined as rectangular section, has average thickness of 1.8 m, height of 21 m, and width of 6 m. The limiting value of allowable compressive strength due to buckling becomes as 3.05 MPa.

Radius of gyration belong to section can be calculated as r = t/(3.464) = 0.5196. Then h/r ratio takes the value 40.41. Therefore, allowable compressive strength defined by the code due to axial loading for walls having h/r < 99 can be determined with Equation 3.3 from where F_a takes the value **3.667 MPa**. (Therefore, the limiting value is 3.05 MPa)

Table 3.2 Uniaxial Compressive Strength Testing Results

Specimen Photo	**Specimen Type**	**Initial Cracking Strength (MPa)**	**Ultimate Strength (MPa)**
-	Single *Red* Andesite 1	~40.00	116.05
-	Single *Red* Andesite 2	Could not be detected	131.44
-	Single *Red* Andesite 3	Could not be detected	103.02
-	Single *Red* Andesite 4	~35.00	116.05
-	*White* Single Andesite	~60.00	136.82
	*Red* Andesite Wall Segment - Representing 2^{nd} & 3^{rd} floor walls of the towers	~12.00	33.21
	*White* Andesite Wall Segment - Representing 1^{st} floor walls of the towers	~12.00	50.19
	*White* Stone Prism	~42.00	139.71
	*White* Stone Pyramite	~11.00	63.49
	*White* Stone Cube	~43.00	104.82

$$P \leq (1/4)\, P_e \tag{3.1}$$

$$P_e = \frac{\pi^2 E_m I_n}{h^2}\left[1 - 0.577\left(\frac{e}{r}\right)^2\right] \tag{3.2}$$

For wall section h/r ratio < 99 :

$$F_a = \left(\frac{1}{4}\right) f'_m \left[1 - \left(\frac{h}{140r}\right)^2\right] \tag{3.3}$$

3.2.2.2 Interface Element Models used in FEM

A Simple isotropic frictional behaviour defined by Columb’s, was considered as valid for interface behaviour between stone units of the towers. According to frictional behaviour relative motion can only occur if equivalent shear stress, which is resultant of orthogonal shear stress on the interface, overpasses critical stress, which is the limiting value bound to μ; coefficient of friction and ρ; surface pressure

$$\tau_{eq}^2 = \sqrt{\tau_1^2 + \tau_2^2} \tag{3.4}$$

$$\tau_{crit} = \mu \rho \tag{3.5}$$

The resistance during sliding is governed by kinetic friction coefficient μ_k that has lower value than static one μ_s. The transition between static to kinetic friction is defined by exponential decay function, which depends on d_c; decay coefficient, $\dot{\gamma}_{eq}$; equivalent slip velocity and friction coefficients.

$$\dot{\gamma}_{eq}^2 = \sqrt{\dot{\gamma}_1^2 + \dot{\gamma}_2^2} \tag{3.6}$$

Friction transition behaviour:

$$\mu = \mu_k + (\mu_s - \mu_k)e^{-d_c \dot{\gamma}_{eq}} \tag{3.7}$$

The values of frictional coefficients were provided by laboratory testing conducted by Topal (2010). The value of decay coefficient was selected as 1000 accordingly.

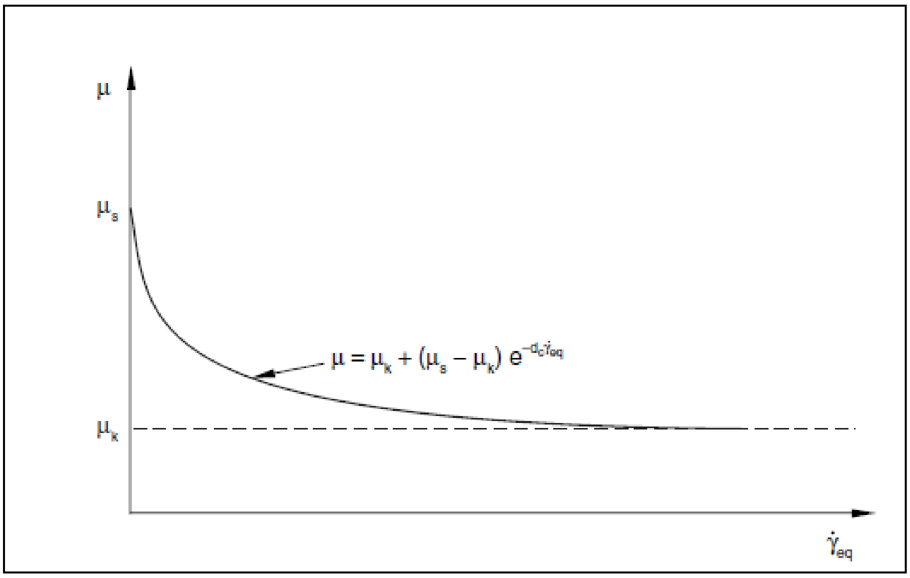


Figure 3.3 Transition Behaviour Between Static and Kinetic Frictional Coefficients (ABAQUS 6.10 Analysis User's Manual)

3.2.3 Earthquake Loading Data

In investigation of collapse mechanism of original towers, it is important to use a seismic data with high intensities in the analytical simulation, thus a global deformation is assured by the effect of an earthquake (EQ). Research on earthquake simulation and therefore digital earthquake records can only be found from early 20th century, which does not provide large historic earthquakes in the past to correspond life span of the Perge Towers. Moreover, it is not possible to define type and form for an antique earthquake that might have caused severe damage at the towers considering many parameters related with geological features of the region where two thousand years old structure exists. Therefore, 1992 Erzincan Earthquake was used for seismic vulnerability investigation of Perge towers in analytical models by omitting participation of geological features of terrain that towers are located in the selection of sample EQ. The earthquake has peak ground acceleration of 0.5153 g in the north direction at 2.885 sec and also 0.4909 g at 2.995 sec in the east direction (Figure 3.4). The earthquake record was used as-is in SAP2000 and Ls-DYNA models, while new records were derived by

- Elongating record adding more than one Erzincan ‘92 EQ back of each other
- Interchanging directional components of Erzincan ‘92 EQ
- Altering the scale of Erzincan ‘92 EQ

in ABAQUS analysis in addition to original one.

On the other hand, in addition to Erzincan ‘92 EQ 3 synthetic EQ data that were derived from local earthquakes[16] (epicentered at most 50 km away from Perge);

- EQ-1) Coordinates: 37.24820 N-30.64970 E, PGA: 2.05 gal, date: 2011/03/16
- EQ-2) Coordinates: 37.15780 N-30.67680 E, PGA: 1.03 gal, date: 2010/10/29

were used in Chapter 4.

Synthetic earthquake generation was made with RSCA software, which was established by M. Thiele (2002) and was modified by Domaniç (2008). The RSCA program uses a locally recorded small earthquake data, which has low PGA in the order of 0.01g, and amplifies it with white-noise function to acquire a new earthquake data, which gives a spectrum compatible with the graph[17] given in the Turkish Earthquake Code (TEC2007). Especially, for first two modal frequencies[18] of the Eastern Tower, synthetic earthquake spectrums show an acceptable correlation with spectrum given in TEC2007 (Figure 3.6).

Earthquake time history data was also applied to linear elastic model as a lateral force in ratio with joint masses, whereas EQ vibrations were applied at base plate as acceleration data in ABAQUS and Ls-DYNA models.

[16] Local earthquake records were taken from database of AFAD Deprem Dairesi Başkanlığı, web site: http://www.deprem.gov.tr/sarbis

[17] According to soil classification of TEC2007, soil under the towers can be defined as Z2 class. Therefore, the general response spectrum defined for this class in TEC2007 has plateau limits of T_a:0.15, T_b:0.40 sec.

[18] Modal frequencies of the Eastern Tower was found in dynamic testing which is described in Chapter 3.5.

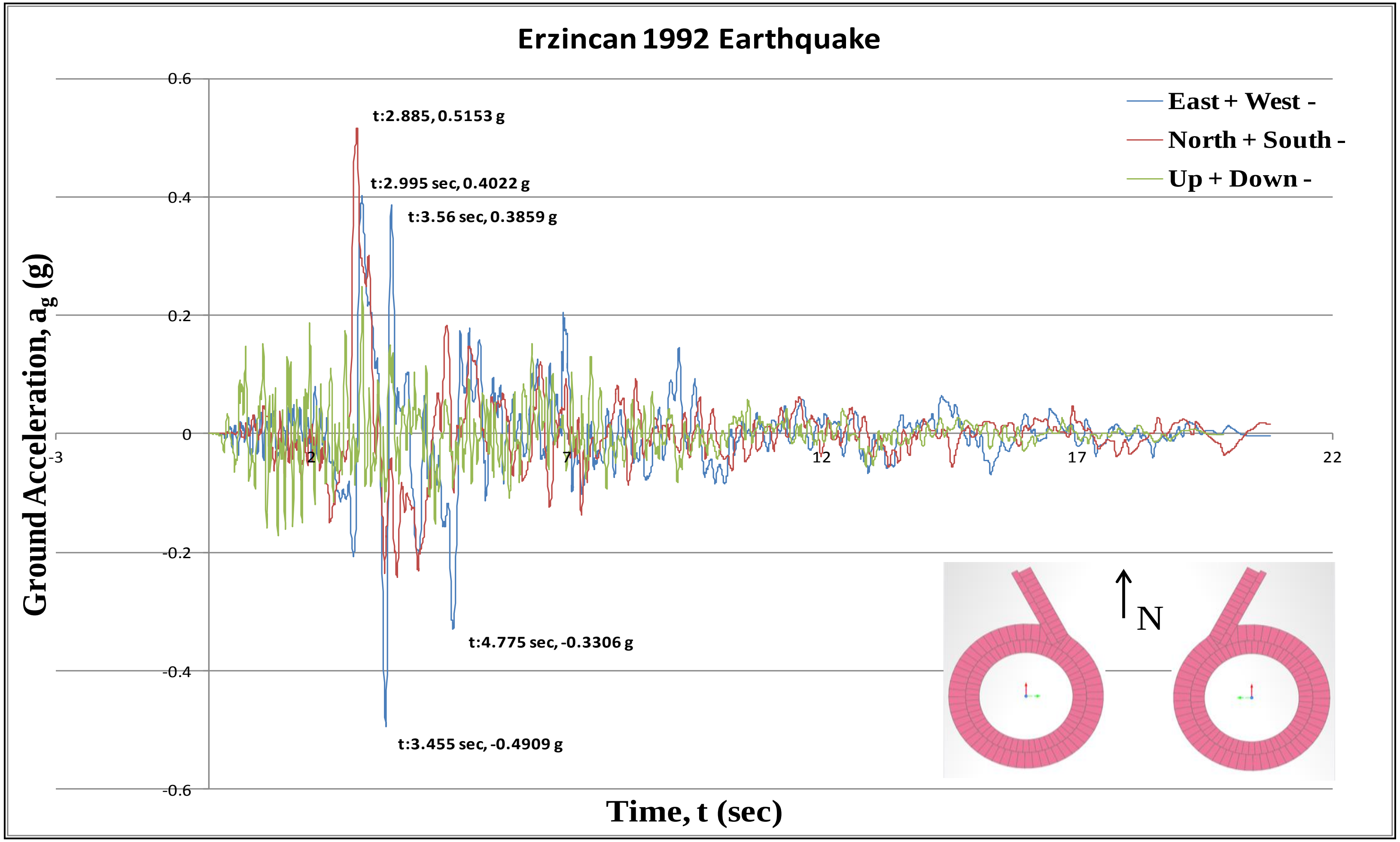


Figure 3.4 1992 Erzincan Earthquake Record

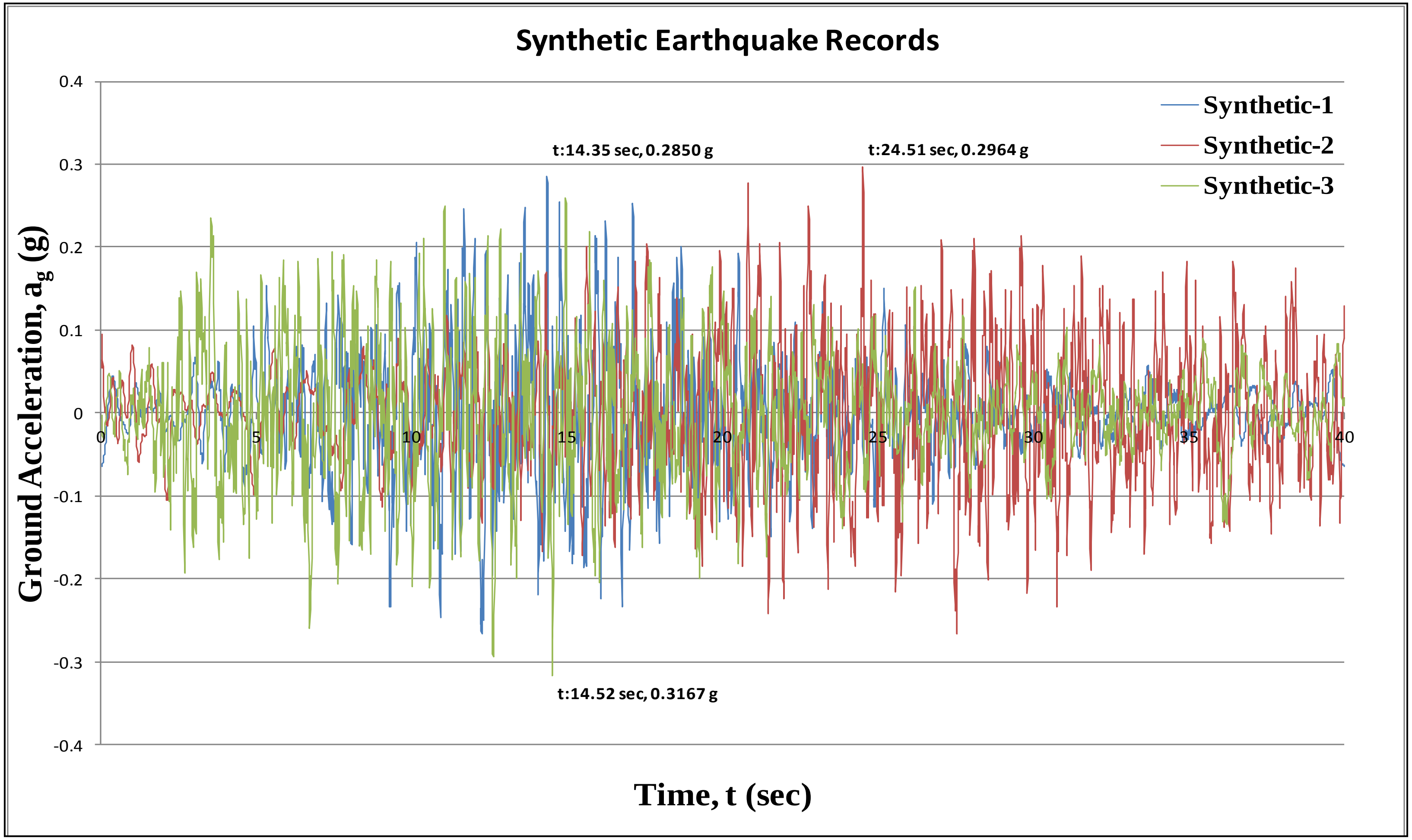


Figure 3.5 N-S Components of Synthetic EQs derived with RSCA Software

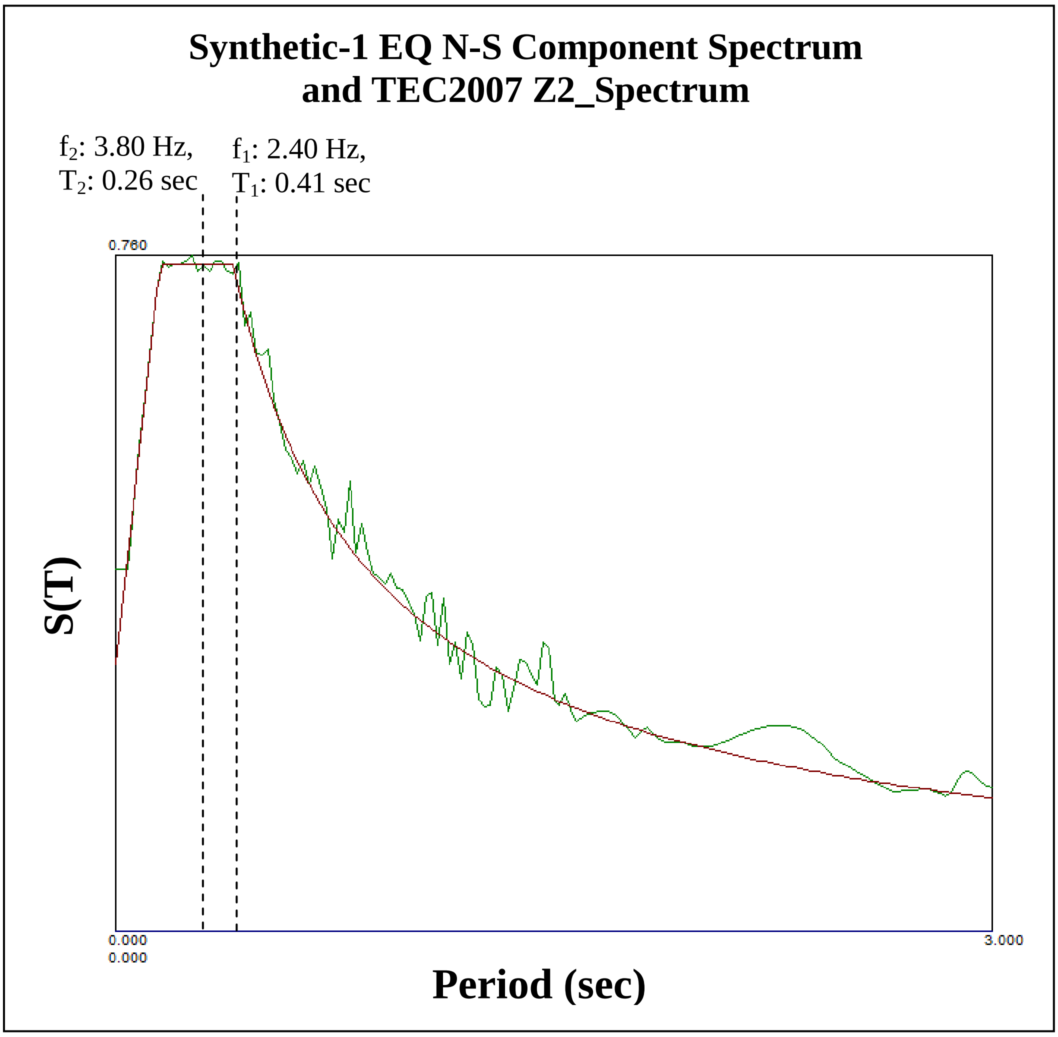


Figure 3.6 Response Spectrum of Synthetic-1 EQ N-S Direction. Correlation between TEC2007 Z2 Spectrum and Synthetic one

3.3 Analytical Modelling to Simulate Actual Observed Damage Condition

In this section of thesis, the detailed definition of modelling studies, which were the main tool that have been used for reversed engineering studies, are given. Studies concentrated on estimation of collapse sources and mechanisms, failure pattern related with original structures, and collapse risk of current structures.

Analytical modelling study includes two types of investigation on towers;

- Linear-elastic dynamic testing on continuum models of SAP2000 software;
- Nonlinear dynamic discrete element models of Ls-DYNA and ABAQUS software.

It was previously defined that the structural investigation with analytical models would better be made with nonlinear techniques, which can provide more realistic and detailed data about historic structures. However, linear elastic modelling was made to get preliminary results about global behaviour of the towers. Furthermore, linear and nonlinear models' results were compared against observed existing structural condition of the towers to evaluate the success of different models to be used for dry masonry historic structures.

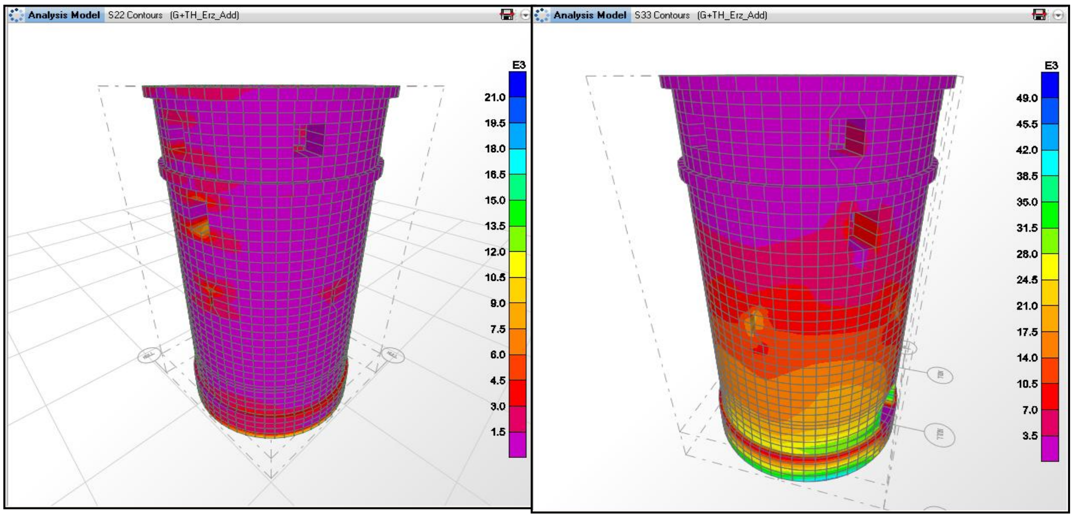
Preliminary investigation was made by using SAP2000. Natural vibration frequency values and mode shapes were found with modal analysis made on continuum model of SAP2000 (Table 3.3). Predominant lateral deformation effect of the flexural modes on global behaviour was seen from the modal analysis since the maximum mass participation ratios were obtained in the flexural modes.

Table 3.3 Frequency Values and Modal Shape Definitions obtained from SAP2000 Modal Analysis

Modal Frequency	Frequency (Hz)	Modal Mass Participation (%)	Mode Shape Definition
1st	8.919	56	Flexural Global Movement in N-S Direction
2nd	8.919	57	Flexural Global Movement in E-W Direction
3rd	18.556	22	Second Degree Flexural Movement in E-W Direction

The stress distribution results of the original Eastern Tower was obtained using linear time-history analysis. Erzincan 19'92 Earthquake was used in the analytical simulations as ground shaking. As a result of linear modelling, 40 MPa vertical (S33) and 20 MPa horizontal (S22) stress values were reached (Figure 3.7). By considering compressive strength of travertine block (15 MPa) provided in laboratory experiments, it is not possible that such high values of stresses can occur. Furthermore, the capacity of the masonry was was obtained to be in the range of 3.05 MPa. Blocky wall sections are expected to crush or buckle under such high stresses in the range of 40 MPa and therefore, the SAP2000 analyses are not valid. Nonlinear modelling techniques should be used which can incorporate crack opening, dissipation of energy, etc.

High vertical stresses are concentrated at lower parts of the structure according to linear elastic analysis results; whereas, the observed damage pattern in Perge Towers, which is the failure starting from upper parts and progressing down in the form of flexure, could not be estimated by SAP2000 analysis. On the contrary, horizontal stress accumulations at the lower parts of the towers and around window and door openings of the structures were obtained as weakness zones in SAP2000 analysis.

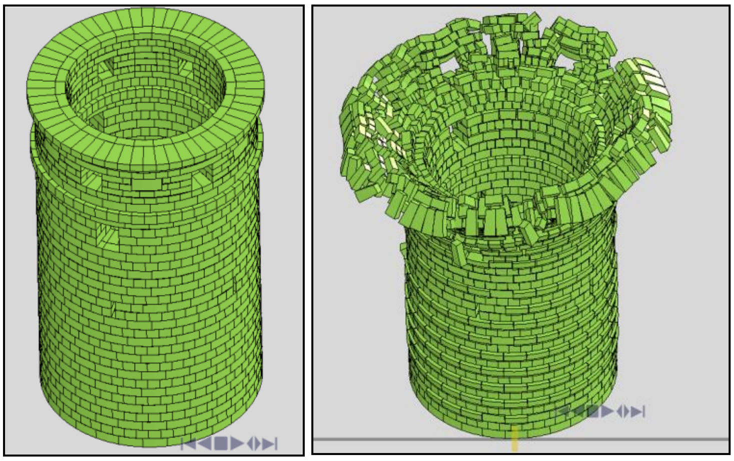


(a) (b)

Figure 3.7 Horizontal (S22) and Vertical (S23) tension stress distribution on linear modelling with effect of Erzincan Earthquake on the Eastern Tower; a) South-western View (S22 Distribution) b) Northeastern View (S33 Distribution)

The need for realistic structural behaviour modelling, which could not be fulfilled with linear-elastic continuum models to show deformation like cracking and local collapses, was tired to be solved with further studies using DEM.

Before going into further investigation on multi-parametric DEM analysis, the effect of horizontal staggering of same level stones uniformity of which was not certainly be proven by both visual inspections and drawings, was tried to be dictated using Ls-DYNA DEM time – history analysis.



(a) (b)

Figure 3.8 DEM of the Eastern Tower formed by Non-Staggered Stone Blocks in Horizontal Plane, Representation of Tower during Seismic Excitation a) at the start of Erzincan EQ b) 5^{th} sec of Erzincan '92 EQ

Result of DEM analysis shows that circular shaped walls had higher tendency to overturn towards outside while the lack of interlocking mechanism between stone blocks caused a weakness in flexural behaviour. Different from the current shape of ruins, a uniform deformation instead of differential one among elevation was obtained in conclusion. Moreover, it was seen that circular shape of walls prevents the flexural deformation inward due to the thrust effect of wall section. However, a certain part of wall was found lying inside the towers at the end of the earthquake simulation, which means a mass block collapse directing inside have also occurred. To summarize, the missing horizontal staggering in the walls found to be irrational, whereas at some locations because of constructional mistakes coinciding joints might have formed non staggered weak zones.

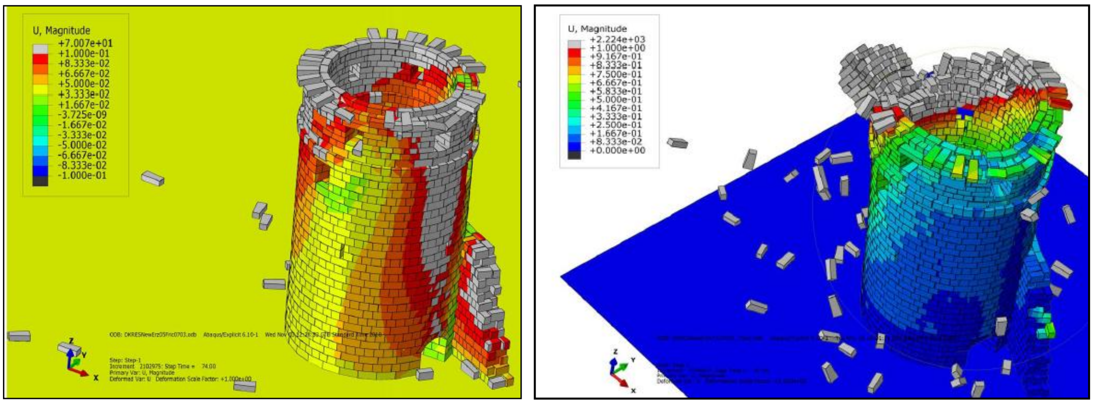
In following studies, cause-effect relationship was investigated regarding the collapse of towers in ABAQUS Software. Several analyses were performed by altering parameters like earthquake intensity, earthquake component direction, uncertainties regarding existence of structural elements due to the former reconstruction studies in lacuna. The following geometries were all formed with uniform staggering of stone in both vertical and horizontal planes.

Table 3.4 ABAQUS DEM Models: Definition of Different Parameters used

Model No	Tower Model	Earthquake Number[19]	Earthquake Intensity Coefficient	Earthquake Component Direction	Geometrical Alterations
1	Original Eastern	1	0.5	Original with Erz.'92	Courtyard Wall added
2	Original Eastern	1	1.0	Original with Erz.'92	Courtyard Wall added
3	Original Eastern	1	1.0	Substitution between. N-S & E-W components	Courtyard Wall added
4	Original Eastern	2	1.0 & 0.5 respectively	"	Courtyard Wall added
5	Original Eastern	3	1.0 & 0.5 & 0.5 respectively	"	Courtyard Wall added
6	Original Eastern	-	-	-	Courtyard Wall added, Lintels were deleted.
7	Original Eastern	1	1.0	Substitution between. N-S & E-W components	Courtyard Wall added, Lintels were deleted.
8	Original Eastern	3	1.0 & 0.5 & 0.5 respectively	"	Courtyard Wall added Eastern Late Period Door were (?) added
9	Original Eastern	1	1.0	Original with Erz.'92	Courtyard Wall added City Side Windows and Loopholes were deleted
10	Original Western	3	1.0 & 0.5 & 0.5 respectively	Substitution between. N-S & E-W components	Courtyard Wall and Northern Gallery were added
11	Current Eastern	1	1.0	Original with Erz.'92	Structural Cracks were added

[19] New earthquake records were acquired, by adding sample of Erzincan '92 EQ more than one to the end of each other.

Reversed engineering studies were concentrated on the original form of the Eastern Tower. All visual investigation and geological survey results were pointing out to the lead role of seismic activity in structural damage. Initially, the effect of seismic activity intensity was estimated. Result of comparison between the Eastern Tower models excited with Erzincan 1992 EQ with PGA: 0.25 g and 0.5 g revealed that intensity has a great role on the collapse of towers. The weak earthquake caused only sliding of uppermost layer stones, whereas the stronger one caused an overturning of a big mass block in the outward direction at the southern upper part of the tower. The analysis showed the vulnerability of towers against an earthquake with PGA of 0.5 g even in their original confined state. However, a single strong EQ was not enough to cause a catastrophic collapse similar to the observed level of damage in the towers.

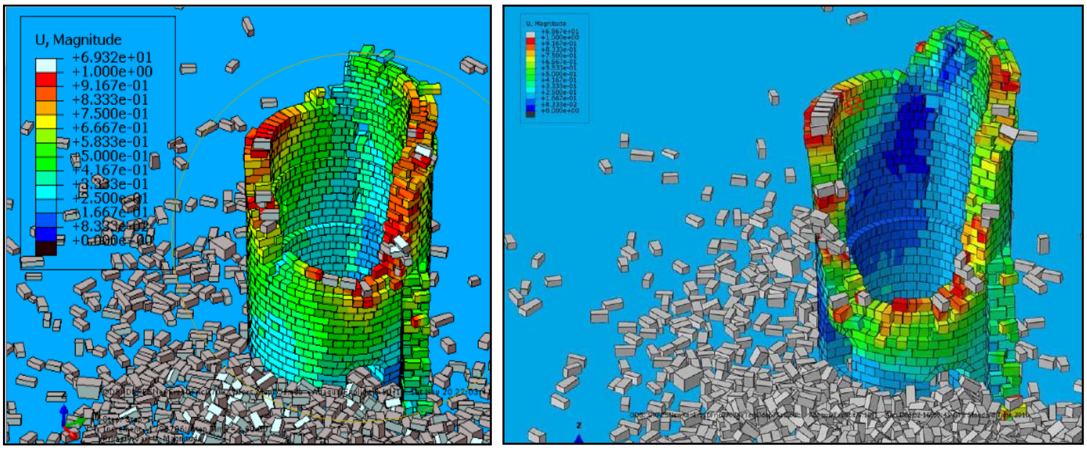


(a) (b)

Figure 3.9 a) Deformation at the Eastern Tower after being affect by the Erzincan ‘92 EQ with 0.5 Intensity Rate b) Deformation at the Eastern Tower during Excitation of the Erzincan ‘92 EQ with 1.0 Intensity Rate

The destruction related with Erzincan ‘92 EQ was mostly at the upper parts of the tower, while these parts are still intact. Therefore, the direction components of sample earthquake were altered in further analyses. On the other hand, new earthquake data was formed by adding copies (with 0.5 intensity) of sample earthquake one after another, because towers have been withstanding two thousand years in which the occurrences of seismic activity must have been more than a single earthquake. As a result of analysis (given with #5 at Table 3.4), a similar deformed shape was found with current tower ruins (Figure 3.10-a).

Then it was understood that the collapse of towers and finally taking the current shape might have caused by progressive set of earthquakes that occurred during two thousand years of period.

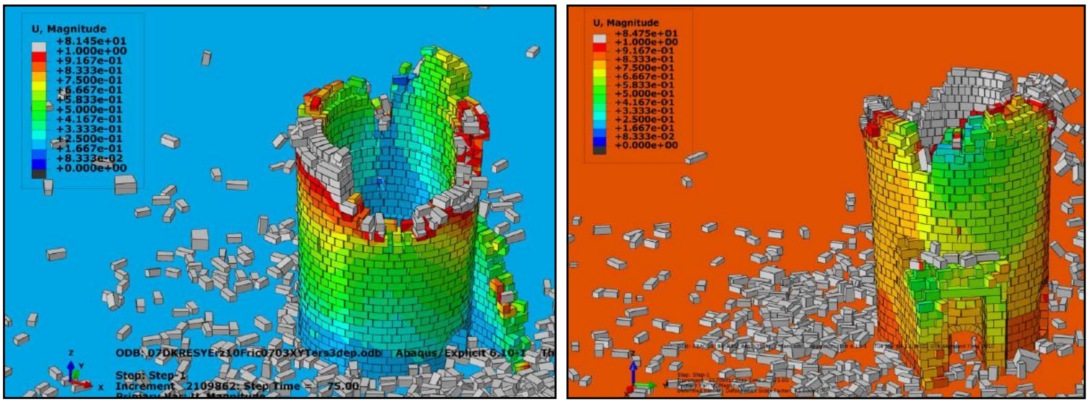


(a) (b)

Figure 3.10 a) Deformed Shape similar with Current One as a result of #5 analysis of Table 3.4 b) Result of the Analytical Model (#8 of Table 3.4) having same Analysis Parameters with #5 but addition of Later Period Eastern Gate (Existence of which is claimed by Mansel)

The effect of material decay on structural behaviour was also investigated. In analysis #7, some stone elements having key features such as lintel stone blocks were deleted from the model and investigation was made under (self weight)dead loading effect. It was viewed that, as expected from a masonry structure, newly formed vertical arching effect caused collapse to stay in minimal rate like falling of a few stones. However, when the same model was affected by only the single record of Erzincan '92 EQ, a global collapse pattern similar with #5 analyses was obtained. This showed that material decay might also cause an increase in the progressive failure due to seismic activity.

The uncertainty about architectural features in lacuna was also investigated with ABAQUS DEM. The later period eastern door existence of which is uncertain but was claimed by Mansel (1954) was added to the model #5. Under same circumstances, analysis results revealed a more accurate differential failure pattern, which is much more similar with current state of the Eastern Tower (Figure 3.10). On the other hand, possibility for existence of loopholes and windows at the city side of the Eastern Tower was considered positively due to the comparison between results of the analysis #9 and #5 of Table 3.4.



(a) (b)

Figure 3.11 a) Result of Analyses #9 having same Parametric and Geometric Condition with #5 except, Lack of Loopholes and Windows at City Side of the Tower b) Results of Analyses #10 made on the Western Tower. Analysis having the same Parametric Conditions with #5

According to result of analysis #10, which was made with same conditions of #5, a much more uniform deformation was obtained (Figure 3.11-b) when compared to the one in the Eastern Tower(Figure 3.11-a). This difference in collapse level or uniformity was considered to be due to the effect of gallery projected to the northern side of the tower. The upper level of deformed shape was fitting into the current level of tower damage, while large cracks that were formed as a result of analysis was pointing the separation zones of large mass blocks like the one existing on the northern side of the Western Tower today. Therefore, it was concluded that the seismic activity that generates analytical damage similar to the observed damage in Eastern tower did not cause a damage similar to the actual damage for the Western tower. It is not possible to know the exact earthquake record combined with exact material decay that the twin towers have experienced; the modified Erzincan ’92 EQ had been successful in replicating the observed damage on the Eastern tower only.

The results of ABAQUS DEMs were also evaluated in the manner of stress concentrations. All of the seismic load simulation analyses have shown stress concentrations along the vertical cracks and near locations of architectural openings. The highest vertical S33 and S22 values observed during seismic activity did not pass the ~2 MPa compressive stress range. Therefore, the main damage mechanism in the EQ simulations of towers is not material overloading but the separation of building blocks by forming vertical cracks leading to collapse.

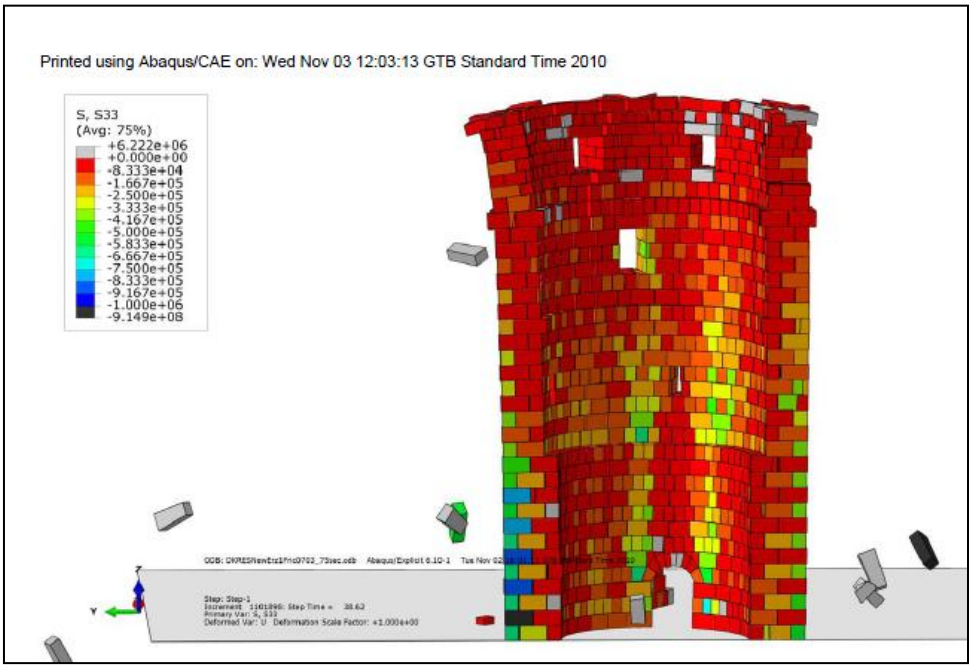


Figure 3.12 Vertical Compressive Stress Distribution during 3rd sec of #5 Analysis.

Even the original tower models that were maintaining their structural integrity show a clear vulnerability under seismic action of Erzincan ’92 EQ. Therefore, defining same risk for current ruins is meaningful to mention. A DEM analysis considering current structural form of the Eastern Tower, also containing the vertical cracks, was made to visualize behaviour under a similar earthquake. Overturning of the upper parts of tower in the outward direction has showed itself as the main damage pattern. The level of turning was found about 10 m high from ground level (Figure 3.13). The remaining portion of the collapsed Eastern tower resembles some narrow standing part of the Western tower indicating that consequent earthquakes might have caused a similar damage.

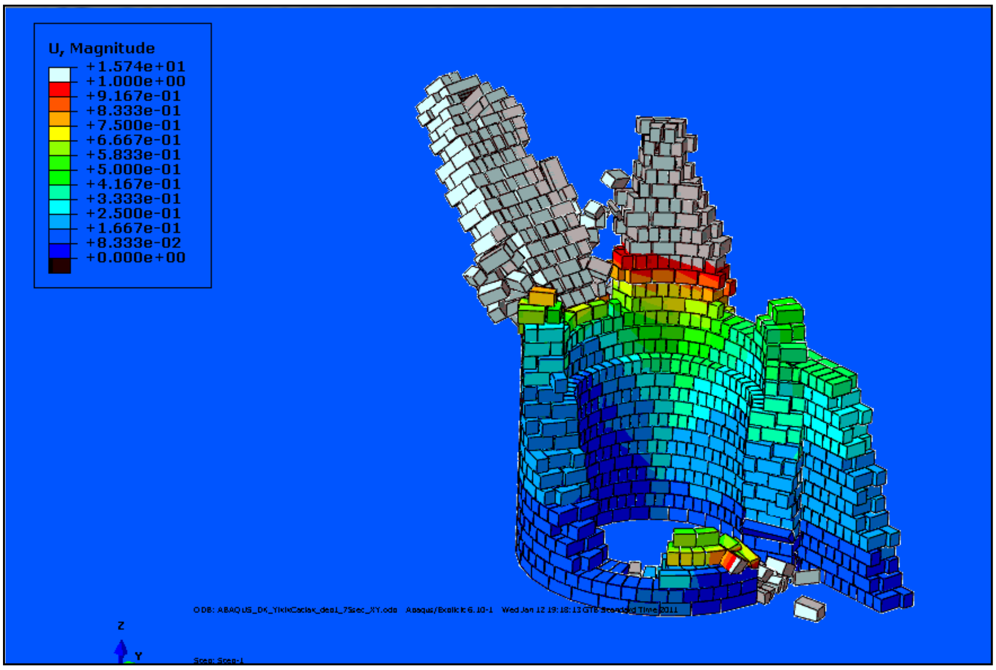


Figure 3.13 Overturning of Upper Part of the Current Eastern Tower under Seismic Action.

3.4 Structural Capacity Evaluation for Current Condition with Simple Engineering Calculations and Simulation Results

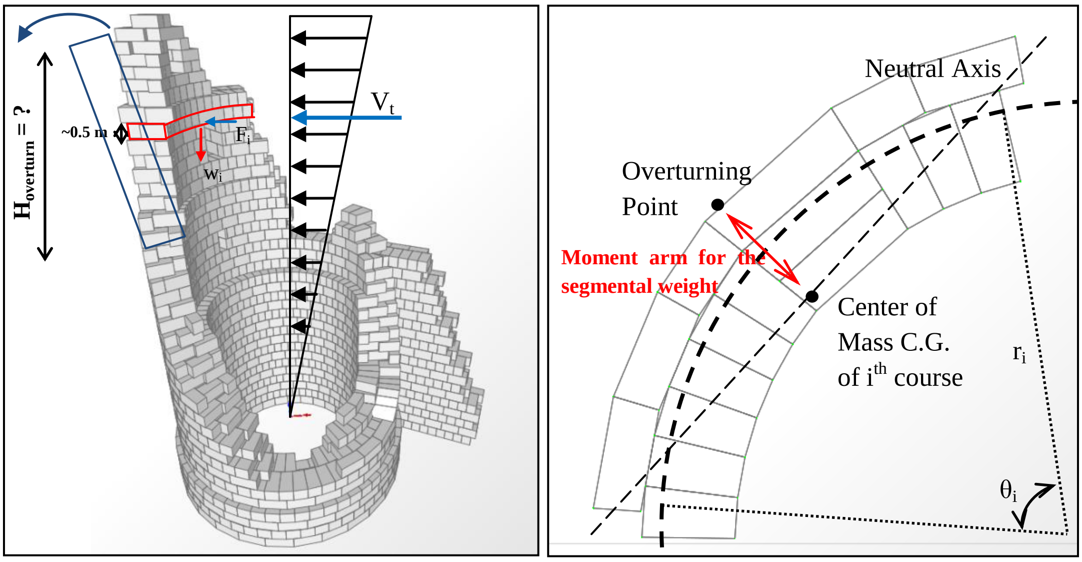
Analytical modelling studies showed that towers have very low flexural resistance against especially lateral forces directed towards outside of the wall. This weakness had also acted as dominating aspect in the way of global collapse of the structure. Today, even not possessing a fully integrated structural body, the arched wall segments of the towers exhibit major risk of overturning toward outside in any occurrence of seismic activity. DEM analysis that was performed for the recent Eastern Tower Structure, also validated the risk under seismic activity. However, the intensity of earthquake and the level of the wall segment that possesses the highest risk in the way of overturning is also investigated by means of hand calculations to verify the results of analytical models.

Total equivalent earthquake load was calculated using *Equivalent Earthquake Load Method*, which is defined in TEC2007 Section 2.7. Total lateral load was found from Equation 3.5., in which $S(T_1)$ was taken as 2.5 because the first natural vibration period was found to be on the flat part of response spectrum in TEC2007 Spectrum (See Chapter 3.5). Importance factor (I) was taken as 1.0 for non-functional building, $R(T_1)$ was taken 1.0 for non-plastic masonry structure. The lateral force that should affect every level is found from the Equation 3.9 in which H_i; height of i^{th} stone level from foundation level, w_i; weight of i^{th} stone level.

$$V_t = \frac{W\, A_0\, I\, S(T_1)}{R(T_1)} \tag{3.8}$$

$$F_i = V_t \frac{w_i H_i}{\sum_{j=1}^{N} w_j H_j} \tag{3.9}$$

Every segmented arch part having 0.5 m height was considered as a layer of stone. By locating centre of mass of every segment, a relation between resisting moment and overturning moment was found due to various intensities of PGA (Figure 3.14). Consequently, it was found that even ground acceleration having 0.2 g poses danger to overturn upper parts of the tower. On the other hand, it was clearly seen that the most risky part susceptible to overturning is about ~8 m from ground level. When compared with the result of analytical studies, which yields overturning location of about 9 m height, the hand based numerical calculations verifies the result of the DEM (Figure 3.15).



(a) (b)

Figure 3.14 a) Equivalent Earthquake Load affecting the Tower b) Calculation Assumptions for Each Stone Layer.

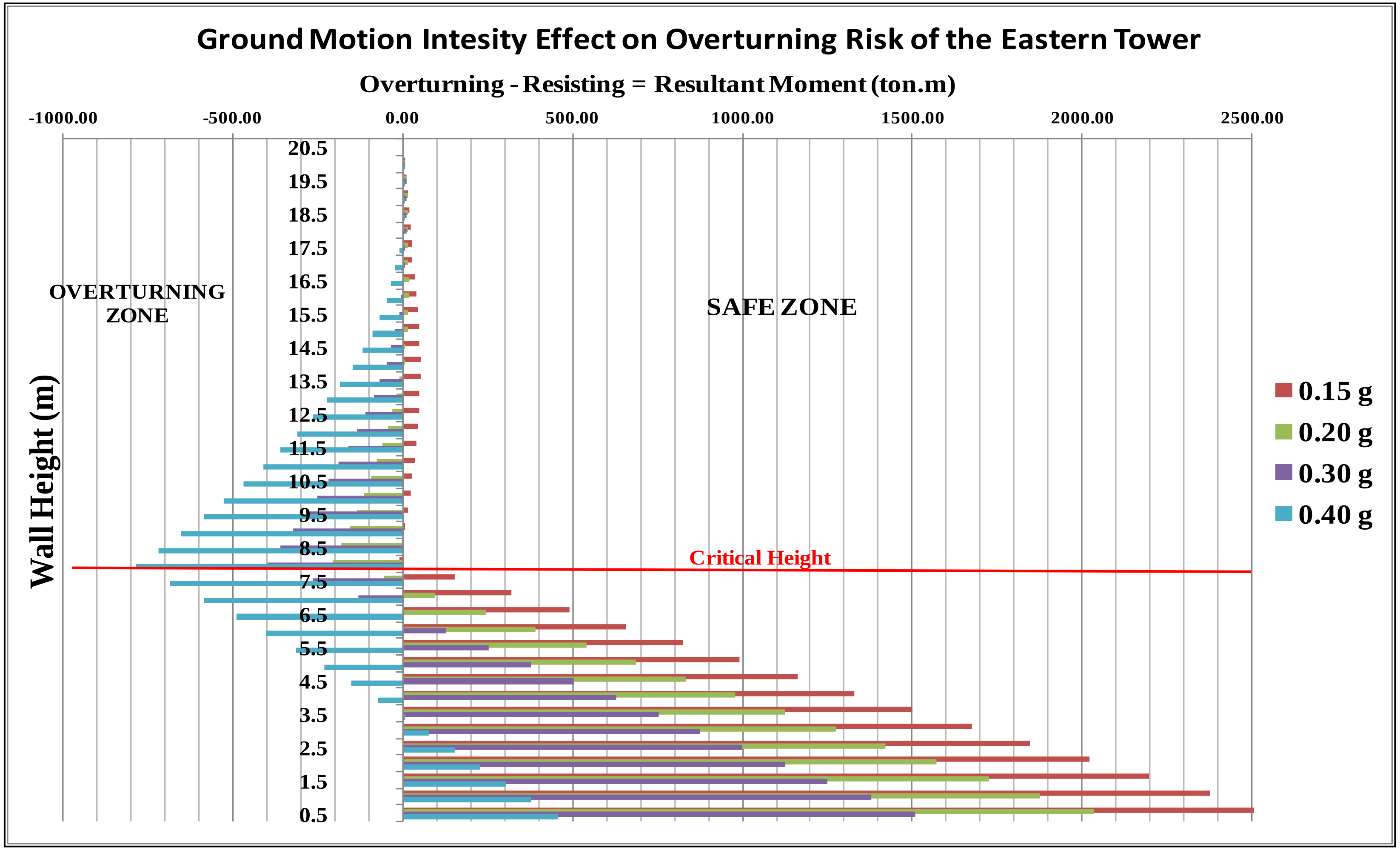


Figure 3.15 Overturning Risk Analysis on the Eastern Tower

3.5 Dynamic Testing on Field and Verification of Simulation Results

At summer 2010, ambient vibration testing was made at the Eastern Tower. Because it was easy to reach upper levels of the Eastern Tower, by the help of existing steel scaffolding, dynamic measurements were taken from 2nd stone level above 6th scaffolding level (29th stone layer, being about 15m height from the ground level) using sensitive accelerometers. In order to sensitively and consistently obtain the natural vibration frequencies that are related with global vibration modes, more than one set of measurements (6 set of data) were taken at the selected areas. The location of data collection points, where the 6 sets of data were taken, are shown in the Figure 3.16, on both elevation and plan. Testing was made with using equipment of PCB-B05 piezoelectric accelerometers, which has resolution of 0.000004 g and gathers data between - 0.5g and +0.5 g and National Instruments 24 bit ADC ,4 channel mobile Data Accusation System. Therefore, with this level of sensitivity minimal effects of both wind and artificial impulse excitations could be observed on collected acceleration data. Among six data groups, the longest measurement was gathered for about 8 minutes (nearly 980000 data points) with 2 kHz in measurement #6 (Figure 3.17). In this measurement, modal frequencies that can be excited with both natural (wind) and artificial (impact) effects were tried to be obtained simultaneously.

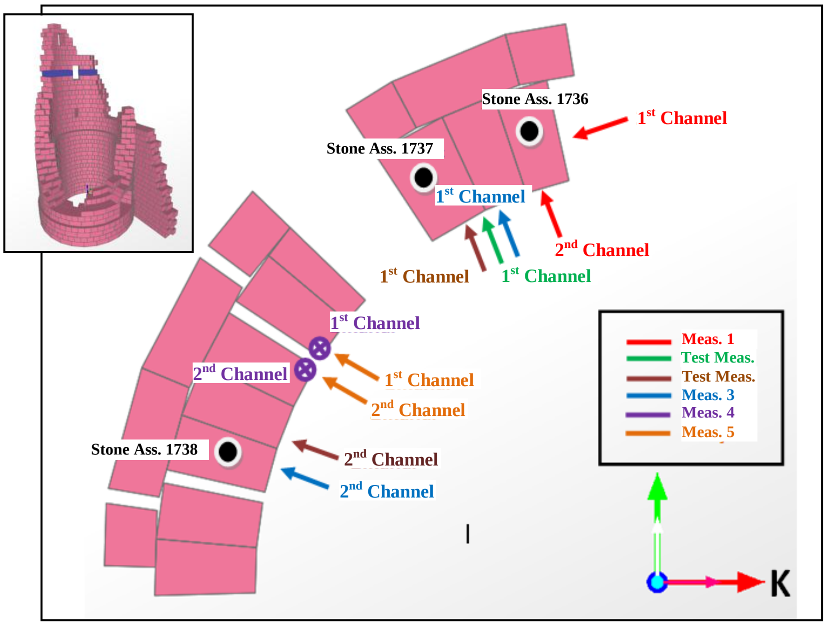


Figure 3.16 Field-Dynamic Testing, Measurement Scheme

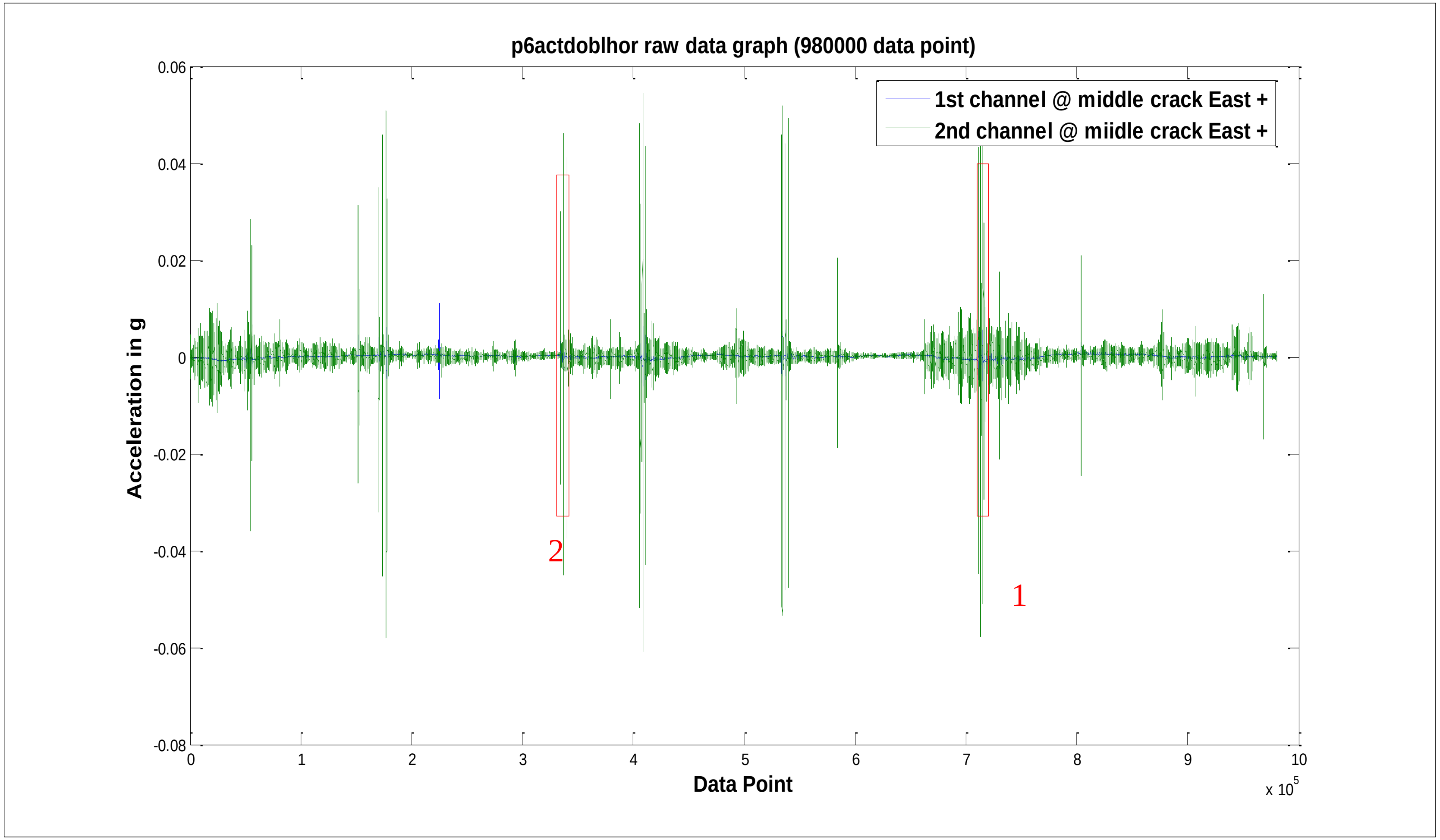


Figure 3.17 Acceleration Data Graphic of the 6th Measurement.

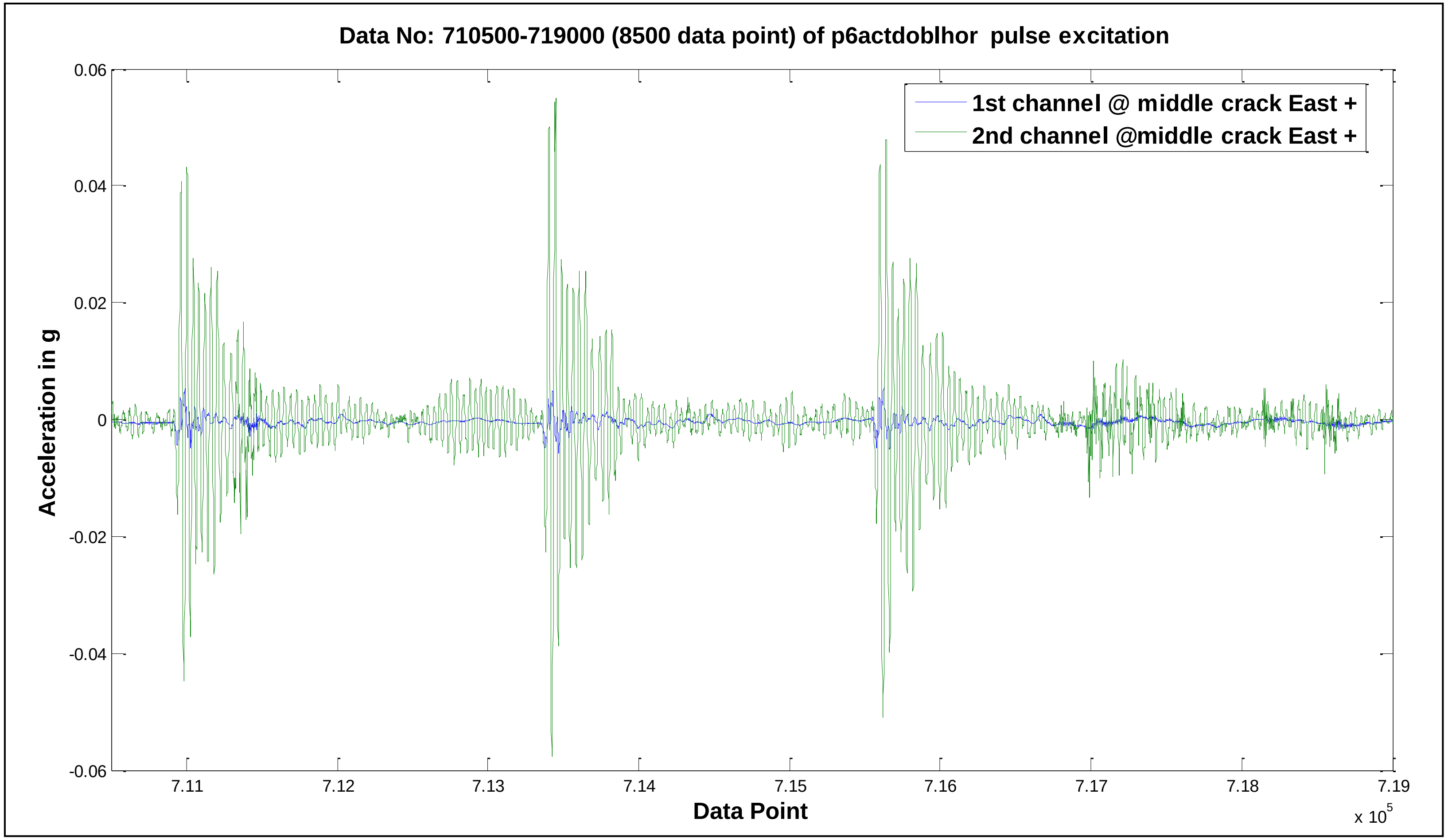


Figure 3.18 Acceleration Data Graph of Measurement #6 (shown with 1 in Figure 3.17)

(Artificial Excitation by Three Sequential Impulse due to Punching Tower's Inner Face)

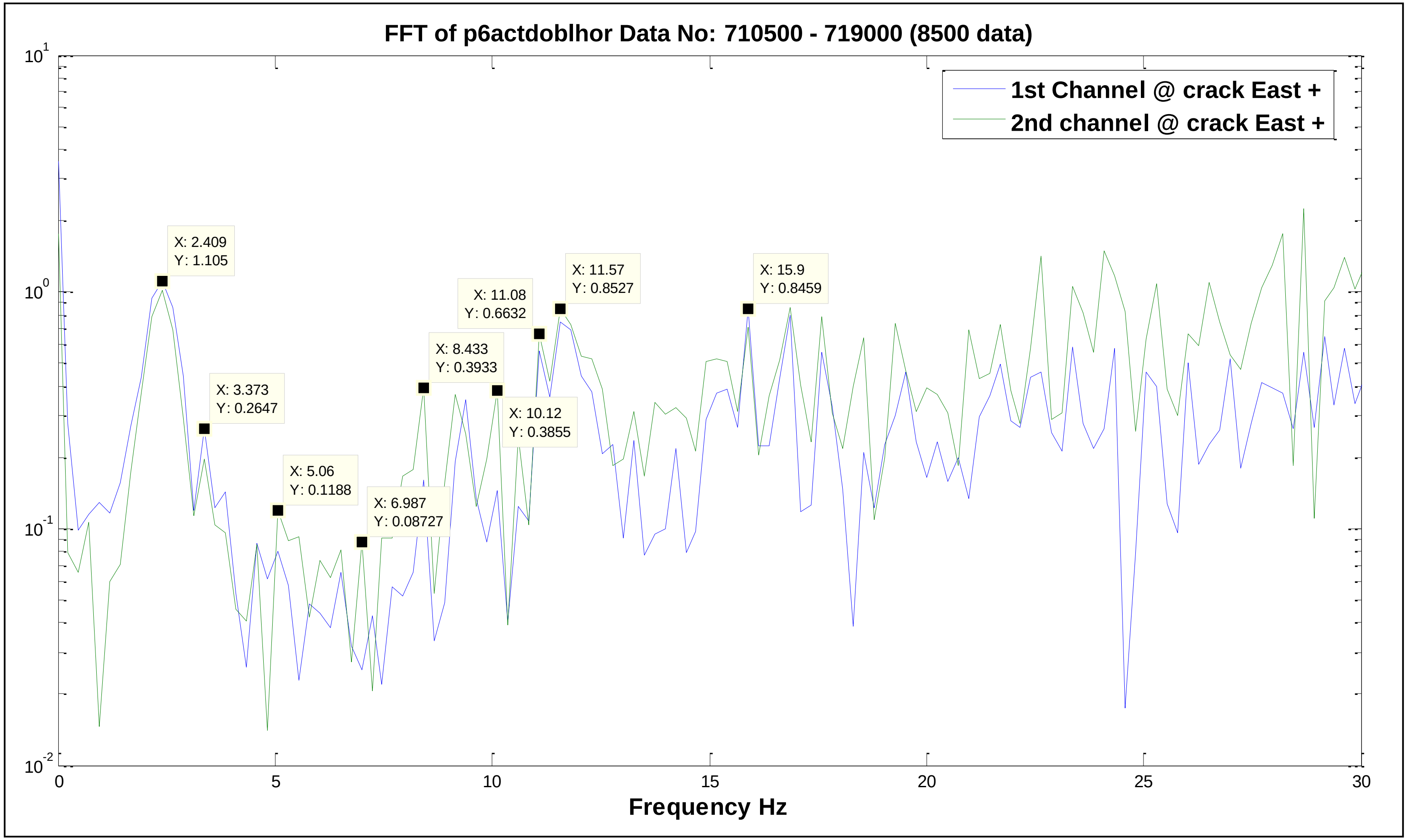


Figure 3.19 Data Group Number 6. FFT Analysis Results Graphic of Part Number 2.

The collected raw data converted from time defined region to frequency defined region by Fast Fourier Transformation (FFT). However, data that was collected during wind excitation used in Wavelet Analyses to reach natural frequencies. As examples; modal frequencies that were found with FFT by process of raw data of 6th measurement can be found at Figure 3.19 while the first flexural modal frequency that was found with the wavelet analysis from the data which is taken at measurement #4 ,channel #2 is given at Figure 3.20.

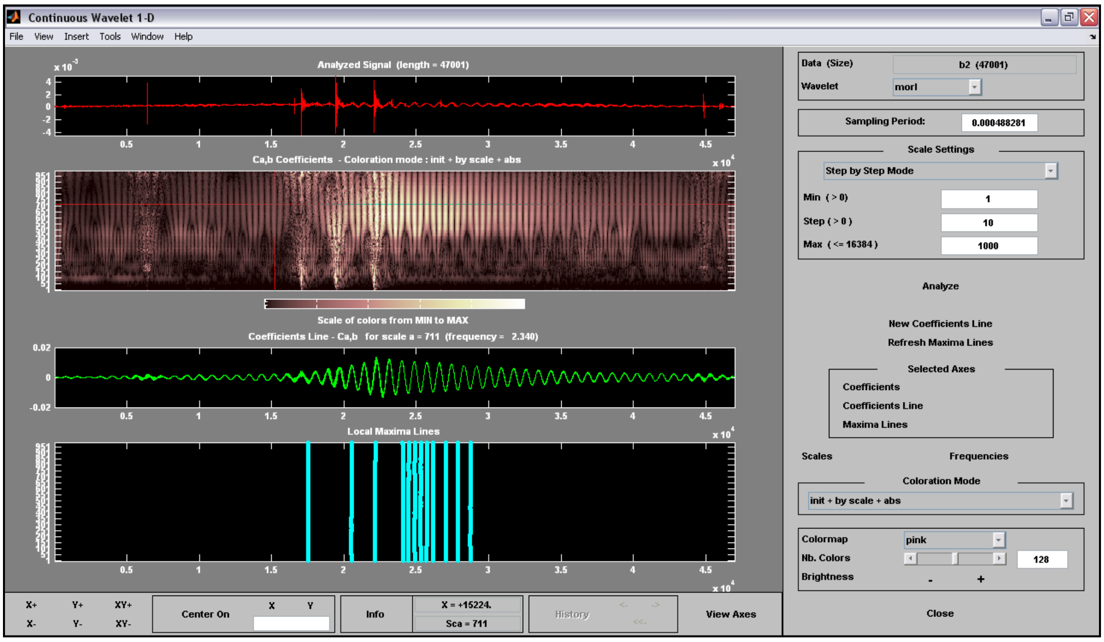


Figure 3.20 Wavelet Analysis on Data of 4th Measurement 2nd Channel which was excited by wind (First Modal Frequency Value corresponding to Flexural Behaviour was found as 2.340 Hz)

In order to make a comparison betweenbetween analytical models and real cases, same dynamic studies were applied for response of models to dynamic excitation. A step impulse function was used to excite ABAQUS Model of the Eastern Tower (Figure 3.21). Therefore, a raw data of acceleration responses of tower was obtained and corresponding modal vibration frequencies were obtained again with FFT procedure. Nodal acceleration data of three stone assemblies were used for this process. Assemblies were especially selected from the stone layer at which field dynamic testing was performed (Figure 3.16). On the other hand, the existence of large scaffolding at eastern tower and its contribution to structural dynamic response might have caused some uncertainty in measurements, which was deemed to be negligibly small since the mass and stiffness of the towers was too large compared to the scaffolding.

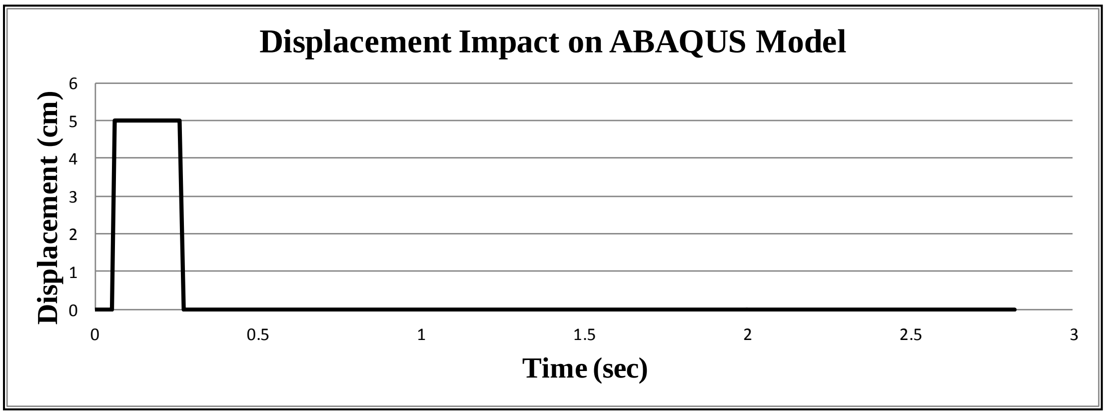


Figure 3.21 Displacement Based Impact that was used to excite Analytical Model.

In Table 3.5, results of both field dynamic tests and analysis on the analytical model can be found. A good correlation between values of artificial and real modal frequencies was able to be achieved especially in lower modes that should correspond to the flexural behaviour. However, an exact connection cannot be defined between some of the higher modes. The dominating structural behaviour of the towers is flexural movement which can be defined with modal shapes having vibration frequency values around 2.40 Hz and 3.80 Hz. These values were detected in nearly every measurement and analysis has clear correlation. Therefore, the correlation results verify the reliability of analytical models' dynamic behavior in the small vibration range.

Table 3.5 Results of Dynamic Testing on the Real Structure and Analytical Model: Natural Frequency Values of the Eastern Tower

Data Code	Accelerometer Channel Code	Natural Frequency													
		Impact Excitation													
1) Backup.lvm	Channel 1		3.13	3.86		5.06	5.54	6.51		9.70					
	Channel 2	1.93	3.13	3.86		5.06	5.54	6.51	8.192						
2) Test_001.lvm (test2)	Channel 2	2.46			4.10			6.14		9.42		11.50		17.20	19.25
3) Act_doblsym_001.lvm	Channel 1	2.43	3.20	3.84		5.20	5.85	6.53	8.43	9.86			14.59		
	Channel 2	2.43		3.84	4.35			6.53							
4) Act_doblevert_001.lvm	Channel 1	2.39	3.07	3.76				6.23							
	Channel 2	2.39	3.07	3.76				6.23		9.05	10.20		14.93		
5) Act_doble_hor_001.lvm	Channel 1	2.41	3.37							9.60		11.60	15.90		
	Channel 2	2.41	3.37			5.06		6.99	8.43	9.60	10.10	11.60	15.90		
		EQ Excitation													
Stone Assembly 1736	S-N Direction	2.40		3.80			5.60	6.20				11.40			
	E-W Direction			3.80			5.60	6.20	8.40			11.40			
Stone Assembly 1737	S-N Direction	2.20	3.40	3.80			5.80		8.40						
	E-W Direction		3.40	3.80			5.80								
Stone Assembly 1738	S-N Direction	2.40			4.60		5.40	6.40			10.40				
	E-W Direction	2.40			4.60			6.40			10.40				
		Wind Excitation													
1) Backup.lvm	Channel 1			3.78			5.94								
	Channel 2			3.39			5.92					11.80			
3) Act_doblsym_001.lvm	Channel 1	2.31		3.86					8.32					18.29	
	Channel 2	2.34		3.86											

3.6 Structural Identification Study Results

Structural behaviour identification studies (diagnosis) can be summarized in the terms given below;

1. Analytical model parameters and obtained failure mechanisms depend on some assumptions such as EQ record or removing some lintel blocks to simulate material decays. Stone blocks were defined in orientation and dimensions, which are close to their original ones. However, there are some differences between existing stone block volumes and dimensions in the real case. Certain approximations were made for minimizing the number of different stone block types of the towers in order to optimize model generation time. Results showed that approximations on some parameters like material characteristic, joint locations, etc. was successful to represent structural behaviour in general sense.

2. Dynamic (EQ) loads on analytical models revealed the trend of tower walls to turnover and collapse outwards direction, whereas inward loads were better resisted with an arching effect in the diagonal direction. In other words arching effect occurs from loading point through sides and diagonally downward direction of the tubular form of the towers. Vertical cracks were observed leading to overturning from the base for the outward collapse trend, instead of diagonal ones that might form in a wall loaded in its in-plane shear direction. Once the width and number of these cracks increase, stone wall segments located between these cracks possessed lower stability against overturning.

3. Joint pattern and staggering found to have great influence on structural rigidity. Moreover, different joining techniques like usage of mortise and clamps between stone blocks were not present at the Perge Towers and the stability was solely dependent on the joining pattern. Sliding type of failure was generally not an issue in the simulations since the out-of-plane failure was more dominant as well as relatively large shear stresses was formed between stone blocks due to self weight of the structure. Thus, failure of upper levels, at which smaller axial loads that could not generate enough frictional resistance, seemed to be much easier. In addition, amplifications of EQ accelerations at upper levels played a role in the collapse of upper layer. The wall thicknesses being relatively thinner at the upper levels caused further weakness against overturning.

4. Several earthquake loading combinations, which are identified in the models, could not exactly create the observed real collapse pattern by itself, whereas combination of EQ loading and material decay in analytical models as an input showed a better collapse match with the real observed damage.

5. Uncertainties of architectural elements regarding once existed structural parts in today's lacuna were also questioned with analytical models. Although it cannot be directly bounded to the structural projection of their existence, reconstructed analytical models including these architectural elements were possible to obtain a damaged geometry which is similar to the current condition of the towers.

6. The recent towers possess much lower structural stability compared to the original towers. Since the existing condition was reached from a more stable original condition, the current state of towers being less stable necessitates proper strengthening applications as a precaution against future possible earthquakes.

7. The analyses of linear SAP2000 were shown to be inadequate in simulating the structural response of towers and did not realistically reflect the damage mechanisms. DEM was proved to be much more successful in representing structural response of dry masonry structures against damaging effects of earthquakes.

CHAPTER 4

STRENGTHENING PROPOSALS AND THEIR EVALUATION WITH THE HELP OF STRUCTURAL ANALYSIS

After the structures being fully investigated with field studies of visual inspection and dynamic testing; laboratory experimentation and analytical modelling studies were finalized with *Theraphy* Process. In addition to three former strengthening proposal that were mentioned in Chapter 2.5, a new one was developed by considering the structural behaviour of ruins that was found in the scope of this thesis work.

Discrete element models that were used in previous stages were modified by addition of details of new strengthening proposals. Detailed performance inspection was made for each proposal except the one that was previously made by researchers[20]. As a performance indication, lateral movement resistance of the Strengthened Eastern Tower was estimated under seismic action of the sample earthquake.

After performance of proposal were estimated the implementations were compared in the manner of contemporary conservation principals in addition to the Perge Hellenistic Gate Towers Case related preservation plan.

4.1 Analytical Modelling Studies on Former Strengthening Methods and Results

Three different strengthening proposal some which is based on visual inspection and simple hand calculations and some that have been made according to the results of detailed analytical modelling, were made by group of engineers and construction firms.

The first suggestion that was made by German group of engineers was tripod like steel structures supporting the tower from inside (Bisswurm & et, 2010). In the study of

[20] The researches were briefly mentioned in Chapter 2.5.

researchers, the contribution of steel tripod against lateral movement was proven with push over analysis on simplified discrete element model . In the model due to the simplification, instead of each stone block being discretely defined, larger group of blocks were represented with single cubic discrete element. Therefore, the number of nodes was decreased to minimize the analyses time without vanishing the precision of analyses. Furthermore, the bonding between steel system and masonry wall was defined with full constrained which is not be possible to provided in real application. But both macro based discritization and full bonding between branches of steel support and masonry wall might showed the performance of steel system greater than as it is possible in real circumstances.

The other two proposals had been defined as result of simple calculations and visual observations, thus detailed performance estimation was made in the scope of this study.

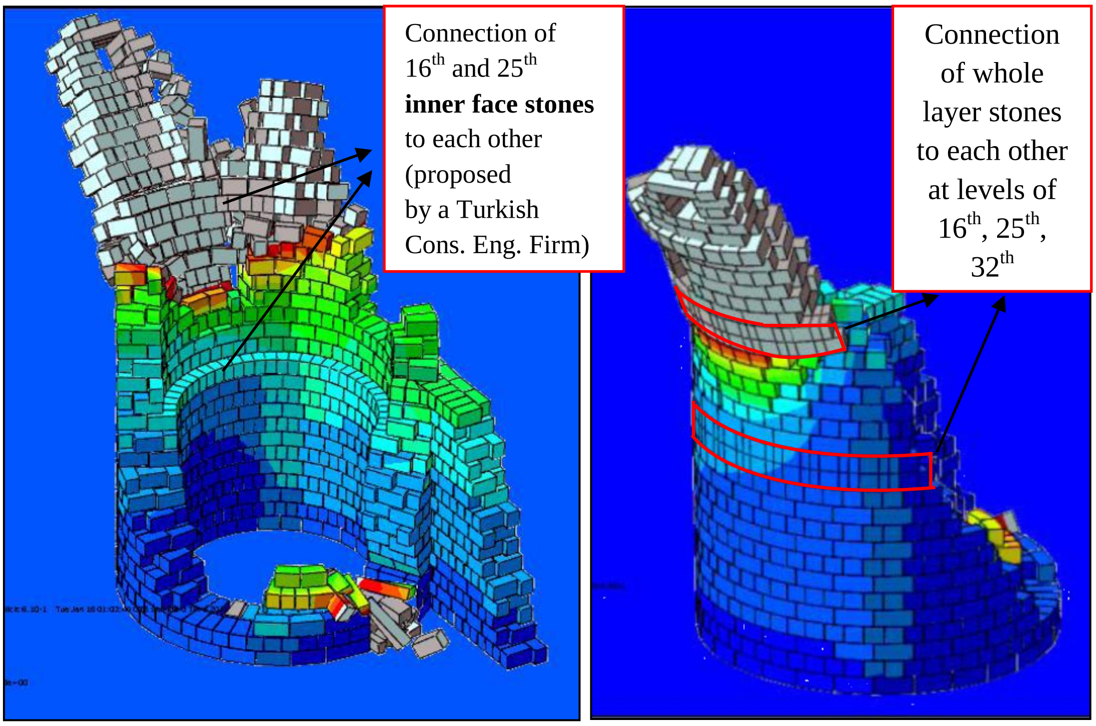
The first one of two remaining former suggestion is *Girder System*. The aim of the system is formation of girders from stone layers at specific locations. The inner stone blocks of 16^{th} and 25^{th} layers were supposed to be bound each other with steel arched beams to form local integration of wall segments against flexural movement. By following same principle two other alternatives of system were also considered in analytical modelling investigation.

Three different girder system strengthening performance estimation were made by updating DEM of Current Eastern Tower as given below;

a) Connection of 16^{th} and 25^{th} inner face stones to each other.
b) Connection of 16^{th} - 17^{th} and 25^{th} - 26^{th} outer face stones with each other
c) Connection of whole layer stones to each other at levels of 16^{th}, 25^{th}, 32^{th}.

In reality it is not possible to provide a full bonding between target stones, however in models in order to investigate the maximum contribution that can be provided to structural rigidity, the full body constrained was assigned for girder stones. Updated models were analyzed under dynamic action of original Erzincan ‘92 EQ.

It was observed that none of three able to prevent collapse at the Eastern Tower under seismic action (Figure 4.1 & Figure 4.2).



(a) (b)

Figure 4.1 a) Girder System -a- Analysis b) Girder System -b- Analysis

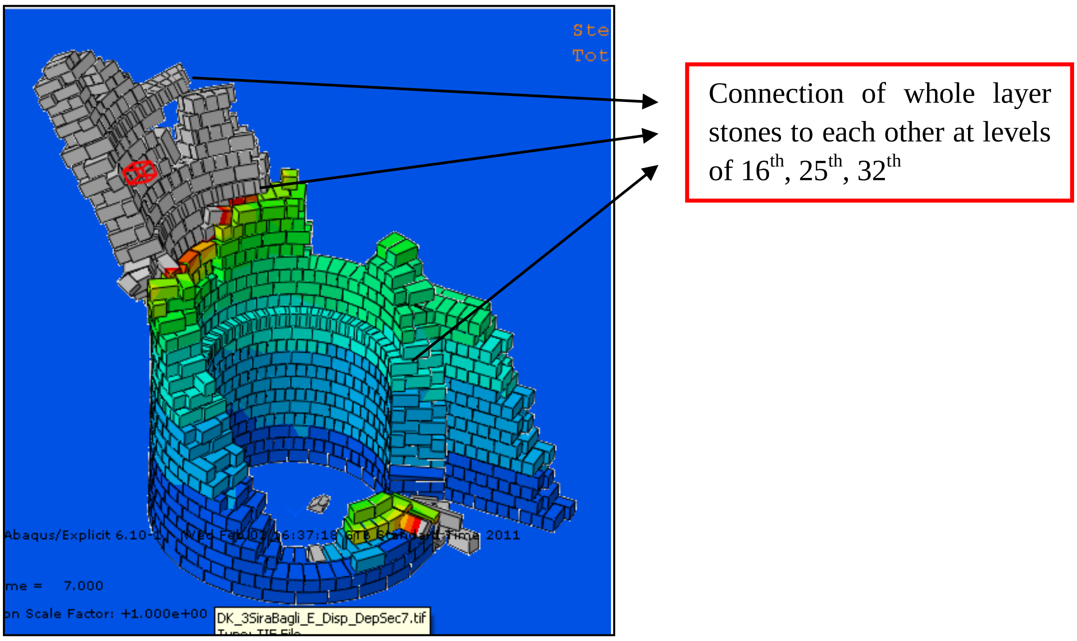


Figure 4.2 a) Girder System -c- Analysis

The second proposal the performance of which was investigated with DEM is pipe like steel frame system that was suggested to be constructed inside of the towers. The system was proposed by Italian Consultancy Team of SAYKA as a conceptual suggestion. The proposal was lack of detailed structural system and drawings. Therefore, the pipe like steel system was defined in models in relevant dimensions.

The frame system (7 storey structure) was formed by 25 cm square solid columns (laid out in arch intervals of 3.5m) and 25 cm square solid sections connecting them to each other. Arms that projecting outward from main structure and bending sides of the masonry structure to prevent flexural failure of tower was also added to steel model. However, in real conditions it is not possible to use solid sections having such geometries, thus towers would had been modelled much stiffer than it is. Related with this issue in order to visualize realistic deformations another model in which a steel material having 10 times lower elastic modulus was formed. The foundation constraint of steel system was provided with a stone layer high inner circle for what material assignment was made same with travertine stone. Linear elastic steel material unit weight; 7850 kN/m^3, elastic modulus; 199.9E+08 Pa, Poisson's ratio: 0.3 were used.

Even at the model, which was strengthened with steel system composed of lower inertia members (2^{nd} model), global deformation was able to be prevented (Figure 4.3). Only slippage of some blocks at uppermost part of the tower occurred.

At the steel system, made with solid steel members, maximum stress levels reaches up to 15 MPa (tensional_S33-vertical) in the analysis. In a real case assuming box sections having nearly 1 cm thickness was used for members, the realistic stress levels would had reach up to 6-7 times greater values of ~90-150 MPa.

At the result of second model support system was still able to prevent global collapse mechanism. According to stress distribution, at 2^{nd} 30 MPa Mises stress concentration was detected at joints (6-7 amplification 180-210 MPa in real). Therefore, it was concluded that, even assumed dimensions of steel members and number of columns were not enough to resist stress values under seismic action. A system having diagonal elements and column-beam frame with larger members should be designed which signs a need of huge steel mass to be built inside of the towers.

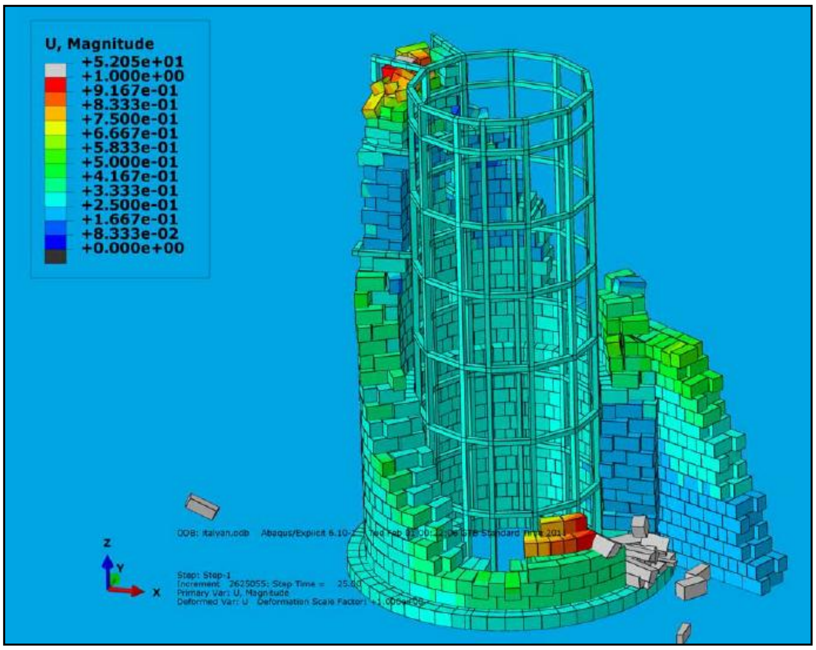


Figure 4.3 Deformation of the Eastern Tower with Pipe Steel System during PGA of Erzincan ‘92 EQ

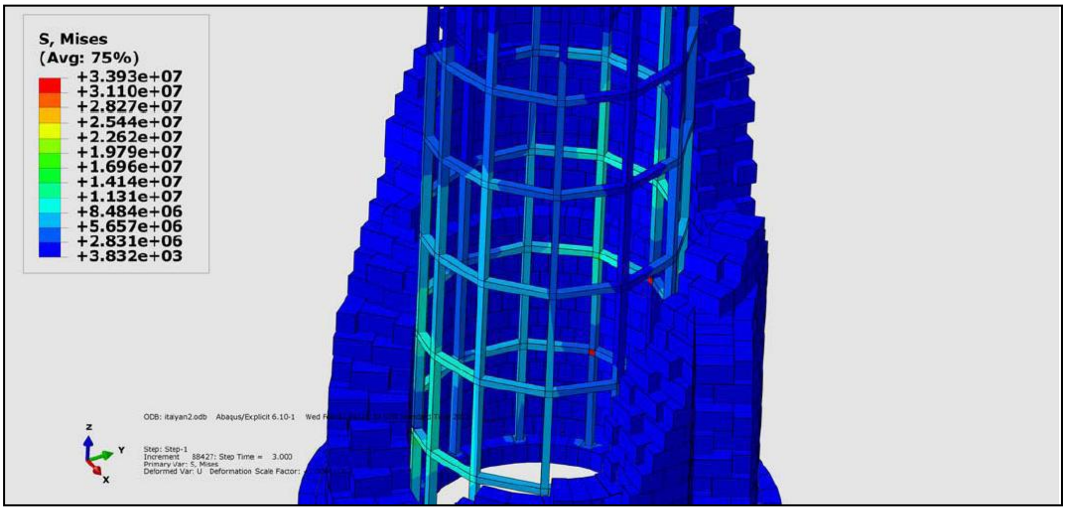


Figure 4.4 Result of 2nd Model in which the Eastern Tower being strengthened: Von Mises Stress Distribution indicates High Stress Concentrations

4.2 Results of Post-Tensioning System

An additional alternative that respects load bearing principle of masonry towers and provide additional rigidity against flexural movement was developed in the scope of this study. The main principle was sitting on increasing the axial load in wall section. In this way, both increase in frictional resistance at interfaces would be assured and increase in the resisting moment against overturning movement would be provided.

Additional concentric loads were suggested to be applied with post-tensioning, cramp like, systems. According to the risk assessments on both towers, it can be stated that wall segments of each tower;

- Southern Part of the Eastern Tower reaching up to ~20 m height
- Southern Part of the Western Tower reaching up to ~16 m height
- Northern Part of the Western Tower reaching up to ~13 m height

involve a risk of demolish. Therefore, all of them were considered as targets of post-tensioning system. However, again evaluation of new system was made on models of the Eastern Tower.

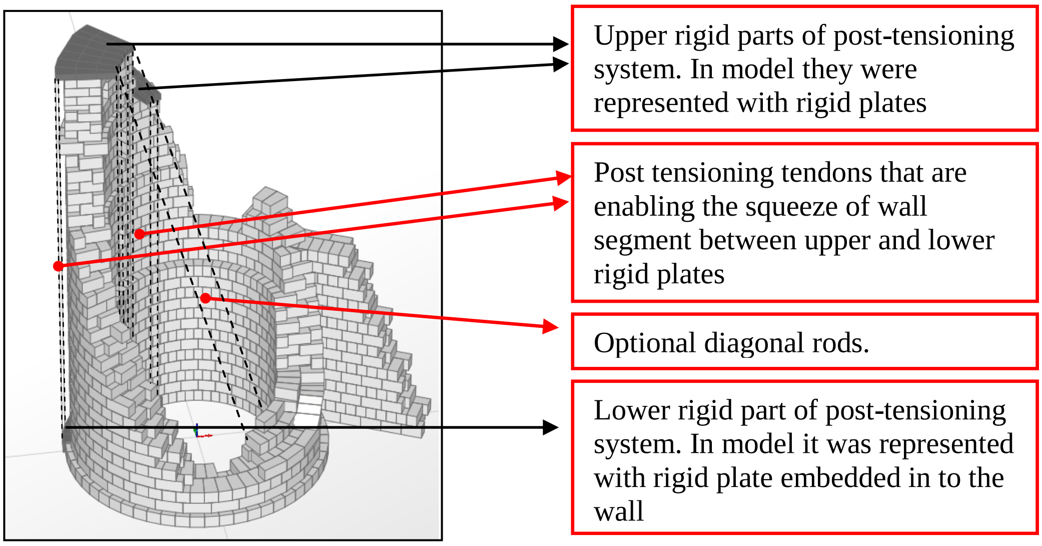
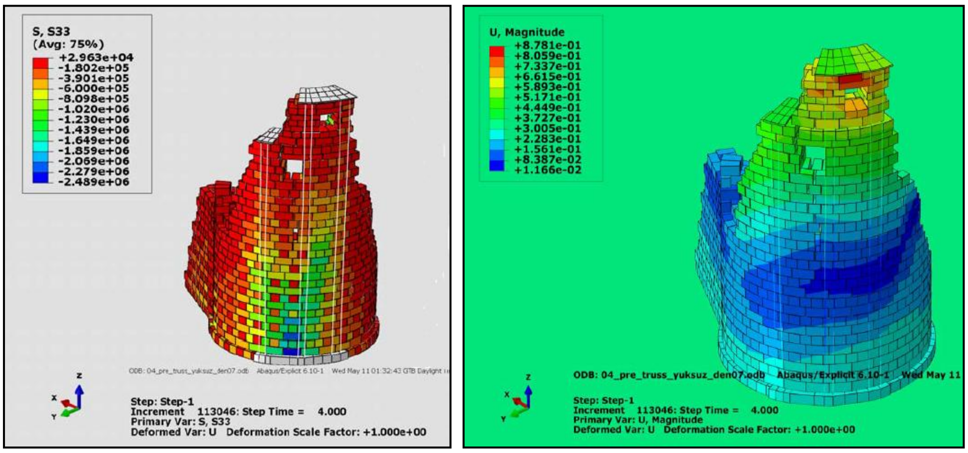


Figure 4.5 Eastern Tower Strengthening Model: Post-Tensioning System

In models, lower and upper rigid parts were defined as rigid plates having the specific gravity of steel. In addition, the post-tensioning rods were defined as steel members with 43 mm diameter. The ends of rods were tied to nodes of plates. Therefore, the load transfer between these objects was ensured. In average ~5 tons of prestressing load was given to tendons in order to prevent slippage of rigid bodies during dynamic loading of analytical model with Erzincan '92 EQ.

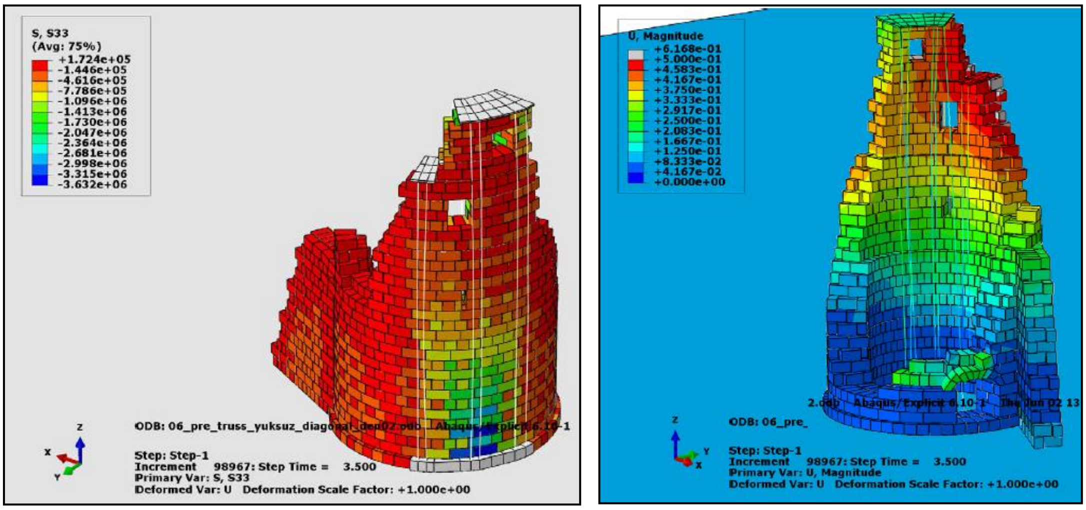


(a) (b)

Figure 4.6 Results of the Eastern Tower Model strengthened with Post-Tensioning System: The Moment of PGA of Erzincan '92EQ a) S33 (Vertical) Stress Distribution b) Displacement

Initial results showed that system was able to prevent the collapse. During Erzincan '92 EQ, high relative displacements at top of tower was occurred (~80 cm). Some of this displacement (~50 cm) was recovered after the earthquake by the flexibility that support system was provided for tower. However, a temporary large crack occurred following the top window span. On the other hand, maximum stress level at the bottom stone blocks of the structures reach up to ~2.5 MPa compressive stress during pick acceleration of the earthquake. The largest tensional load that occurred in rods was equal to 52.2 tons.

To solve the displacement problem 2 additional rods, which thought to be bound to the opposite side of the circular tower wall (near northern gate) at their lower end and to the upper larger rigid plate at their upper end, were added. Therefore, participation of other parts of the tower arched wall segments about the flexural resistance of northern part was provided a little.



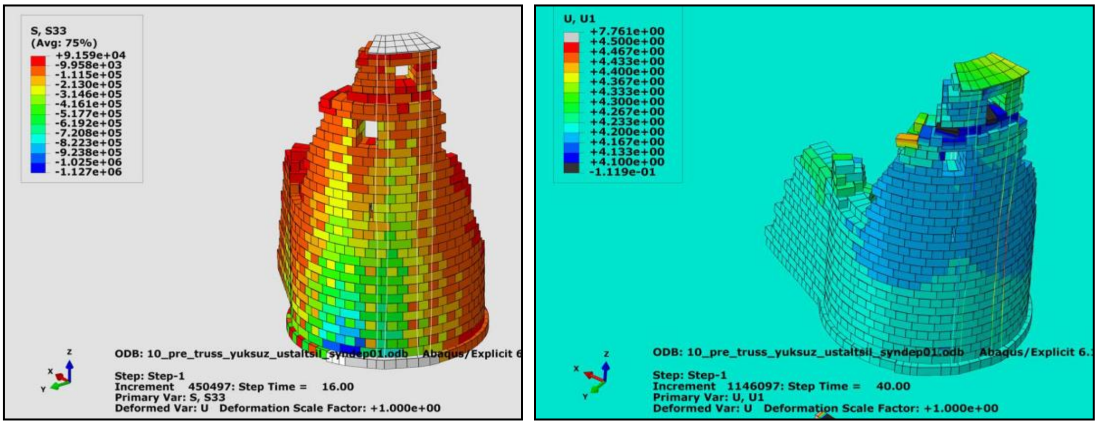
(a) (b)

Figure 4.7 Results of the Eastern Tower Model strengthened with Post-Tensioning System including Additional Diagonal Rods : The Moment of PGA of Erzincan '92EQ a) S33 (Vertical) Stress Distribution b) Displacement Distribution

Existence of diagonals enabled the relative displacement of top layer decreasing to ~45 cm ~60 % of which was recovered after EQ. The crack that was occurred at previous analysis was barely formed in this one. However, the maximum compressive stress was increased to ~3.5 MPa, which is known that it cannot be affordable by masonry wall from the calculations made at Chapter 3.2.2.1. Moreover, the maximum vertical rod tensional load found as 55.8 tons while diagonal rod loads reached up to 88.4 tons.

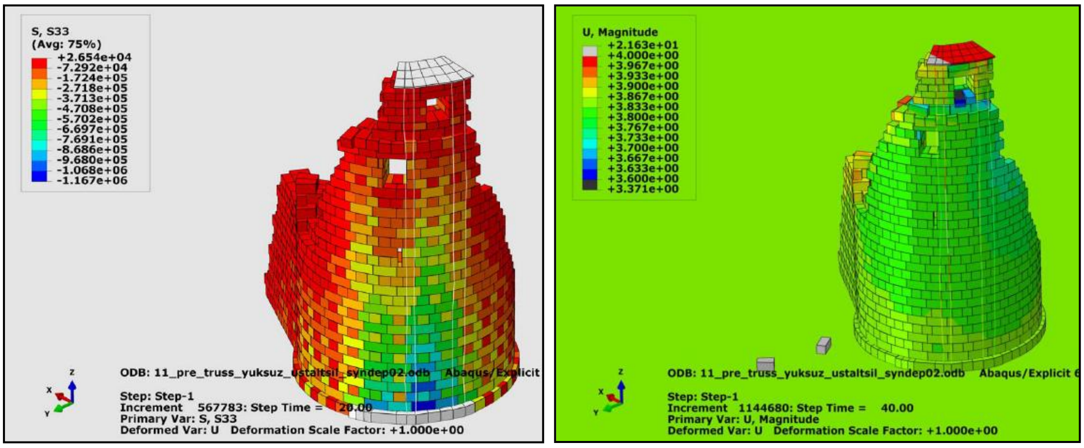
In addition to performance of strengthened system against Erzincan '92 EQ with 0.5 g PGA investigation was also performed with synthetic records. Establishment of these synthetic records is mentioned in Chapter 3.2.3. They were formed due to the local geological features of Antalya, thus it was considered to obtain more realistic results with this records. Analysis were made with three different records.

On the other hand, in the analysis importance of the smaller part of upper steel structure, which was located at lower segment of southern wall, to be built was also questioned. In these models, diagonal rods were mot used.



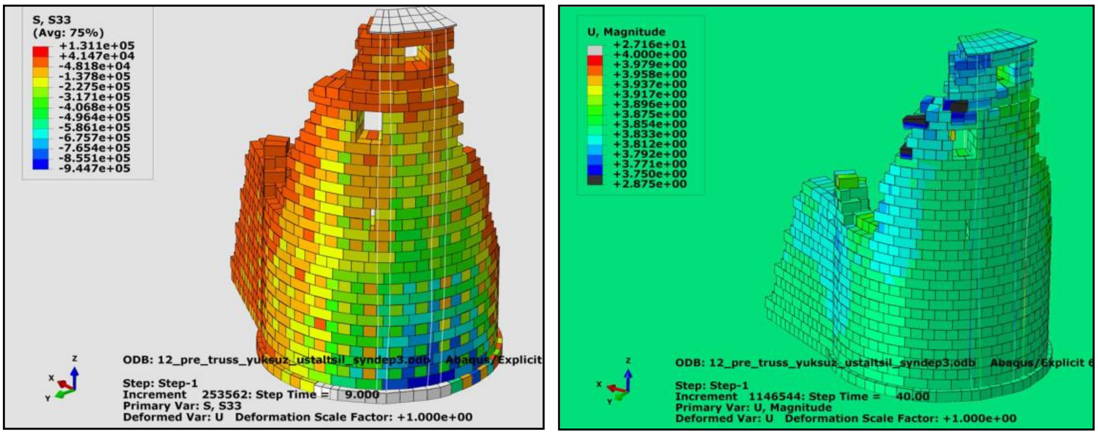
(a) (b)

Figure 4.8 Eastern Tower Model strengthened with Post-Tensioning System and loaded with the Synthetic EQ-01 a) S33 during PGA (t = 16) of EQ b) Deformed Shape at the End of EQ



(a) (b)

Figure 4.9 Eastern Tower Model strengthened with Post-Tensioning System and loaded with the Synthetic EQ-02 a) S33 during PGA (t = 20) of EQ b) Deformed Shape at the end of EQ



(a) (b)

Figure 4.10 Eastern Tower Model strengthened with Post-Tensioning System and loaded with the Synthetic EQ-03 a) S33 during PGA (t = 20) of EQ b) Deformed Shape at the end of EQ.

These synthetic earthquakes having PGA around 0.30 g did not possessed higher risks then Erzincan '92. It was seen that the maximum relative displacement reached up to ~20 cm, ~50 % of which was recovered after EQ. Moreover, the maximum compressive stress at lower parts of the tower ~1.0 MPa while tension load on rods was found as 8 tons. Lack of being supported by smaller upper plate some blocks located at south-western part of the wall fell because of slippage. But no serious damage was occurred due to fact that secondary plate had been removed.

Consequently, by considering the lower intensity local earthquakes it was possible to say that strengthening system is capable of to prevent collapse, furthermore this success can also be achieved for strong motions like Erzincan '92 EQ However, the in such cases the deformations reaches up to risky values in which the loss of stability is possible.

According to results of analysis the loads that has accrued on system was used to the design strengthening steel system. Therefore, the conceptual proposal was able to be represented in a realistic level.

Strengthening support structure consists of steel sub systems located at lowest and highest levels of the wall and post-tensioning rods to connect them. The aim of these two systems is to form rigid constrains which do not deform in considerable level to transform axial load originated from tension rods to tower walls. In lower system, it was possible to propose construction of a rigid circular ring that sits on foundation level in concrete or steel. However, the key idea is formation of diaphragm among the connection points of rods to the system therefore the individual movement of outer and inner supports of rods. On the other hand, it was known that system should not be in contact with ground level because of archaeological founding like Byzantium mosaics that was found in the Western Tower Ground Level at 2010. Therefore, lower system should be flying system anchored to walls.

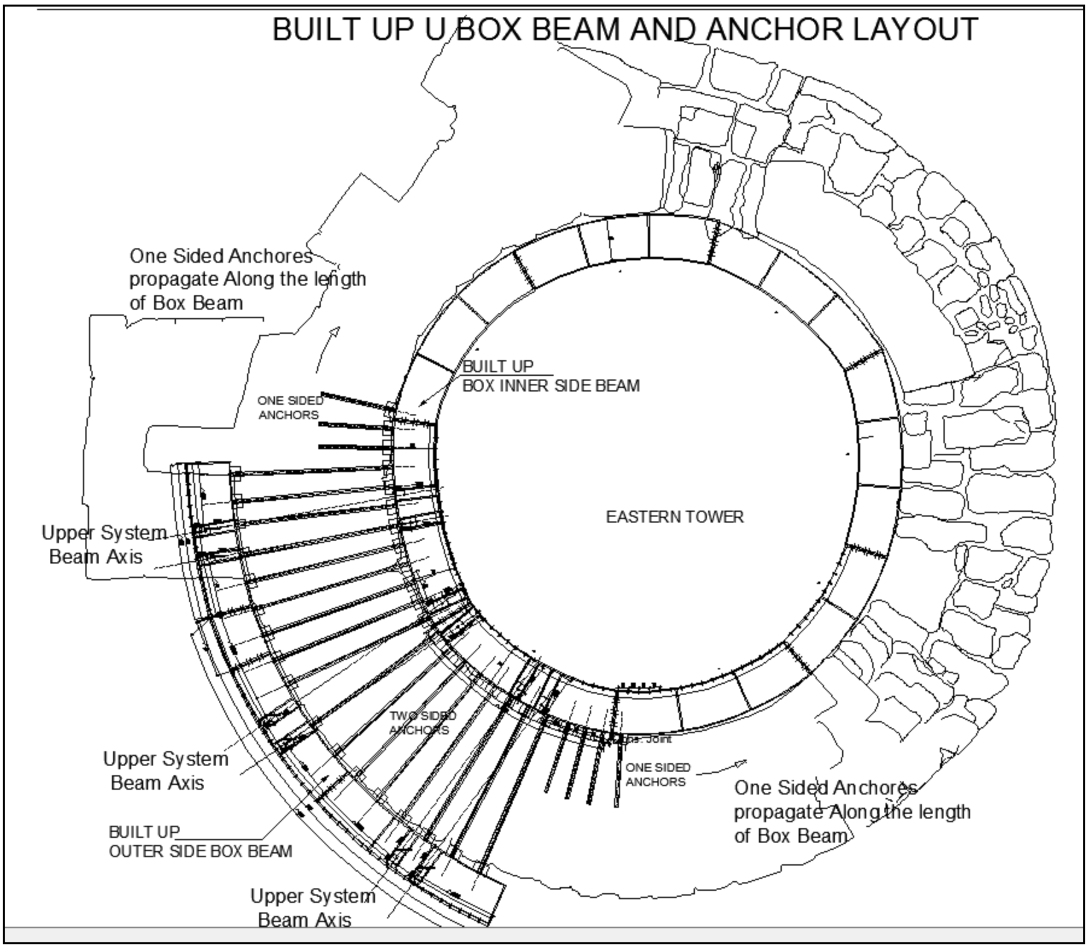


Figure 4.11 Lower System Anchorage Pattern

For the inner face of the 5th stone layer a built up beam, circular in plan, and for the outer face a segmental built up beam were thought to be anchored to the wall (Figure 4.11). Outer one was only covering the part of the circular face corresponding to southern high wall of the tower. ~3 m long arched segments, having C built up section with ~80 cm web and ~60 cm flanges were suggested to be bound together to form inner and outer beam systems. These segments were recommended to connect each other with 12 x 10.9/M50 bolts from the head plates that had already been welded to C sections in the factory (Figure 4.12).

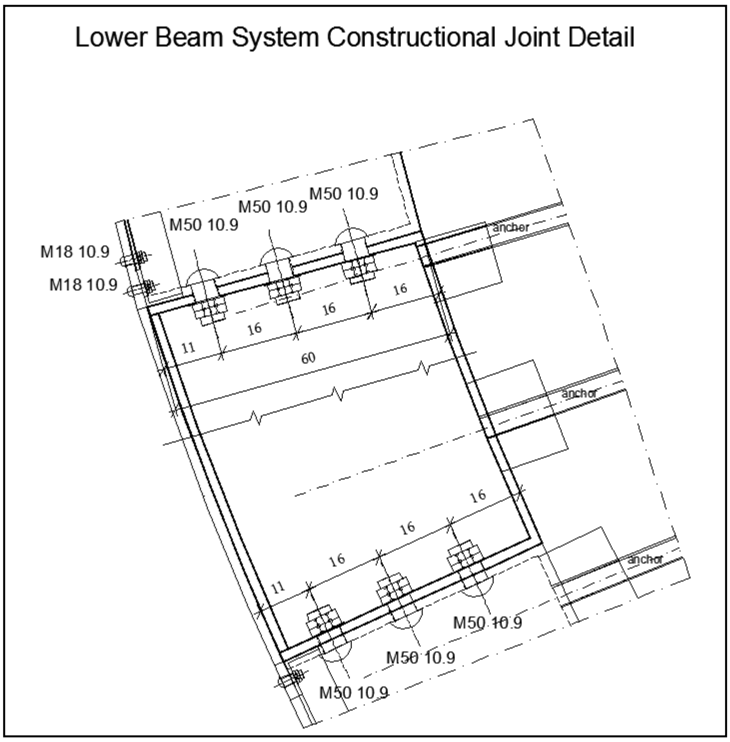


Figure 4.12 Lower Beam System Constructional Joint Connection Detail (Plan Section)

Anchorage of beams to the wall follows the joints between the stone blocks in order to minimize irreversible implementation to the tower. Beams were supposed to be anchored to the wall from their web. Anchors follows a staggered orientation, the first one was located at the joint between 6^{th} and 7^{th} layers while second one stuck into the joint between 5^{th} and 6^{th}, and the others was organized as following this routine. At the parts which outer and inner beams were coinciding, the anchors were designed to be two sided in order to bind two beams to each other. By giving a little tensional force at these anchors with application of torsion to the end bolts of two sided anchors, rigid body with wall and beams was able to be ensured. The sticking of tower with remaining parts of inner beam was proposed to be made with one-sided anchors projecting half of the wall section. At the locations, that anchors just penetrating the wall in order to prevent high vertical stresses caused by anchors a reliving element was designed. Element was including a steel pipe hole of which is 2 mm larger than anchor diameter and a steel plate (150 x 200 mm large) welded on top of tube. This element was suggested to be embedded to the edge of holes that was planned to be drilled for anchors in to the wall (Figure 4.13).

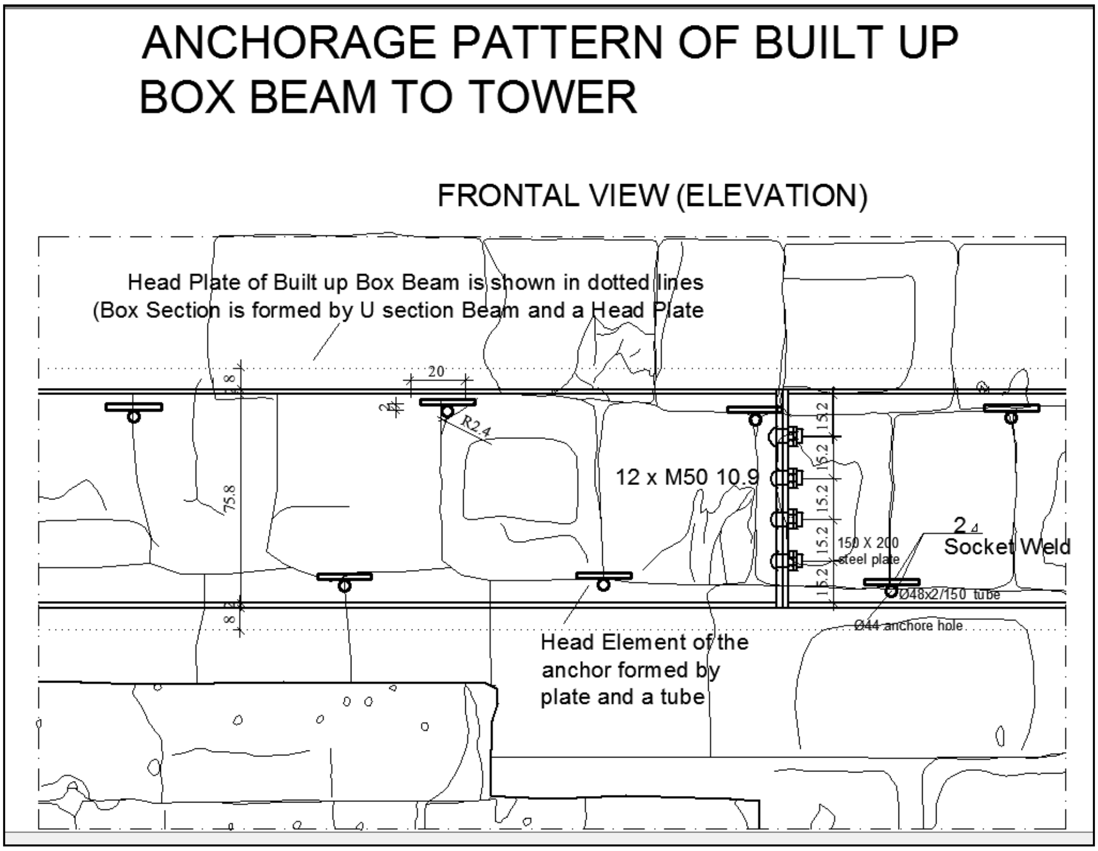


Figure 4.13 Anchorage Pattern View of Lower Box Beam System to the Tower

Tension rods were considered to be fit into the holes, which were opened at both upper and lower flanges of C Section. The ends of the rods were designed to be stabilized at lower face of lower flange with bearing plates and bolts (Figure 4.14). Then, the open faces of C section thought to be closed with curved plates in order to form a box section which is more resistant to the local buckling effect originated from tension rods on flanges of C section.

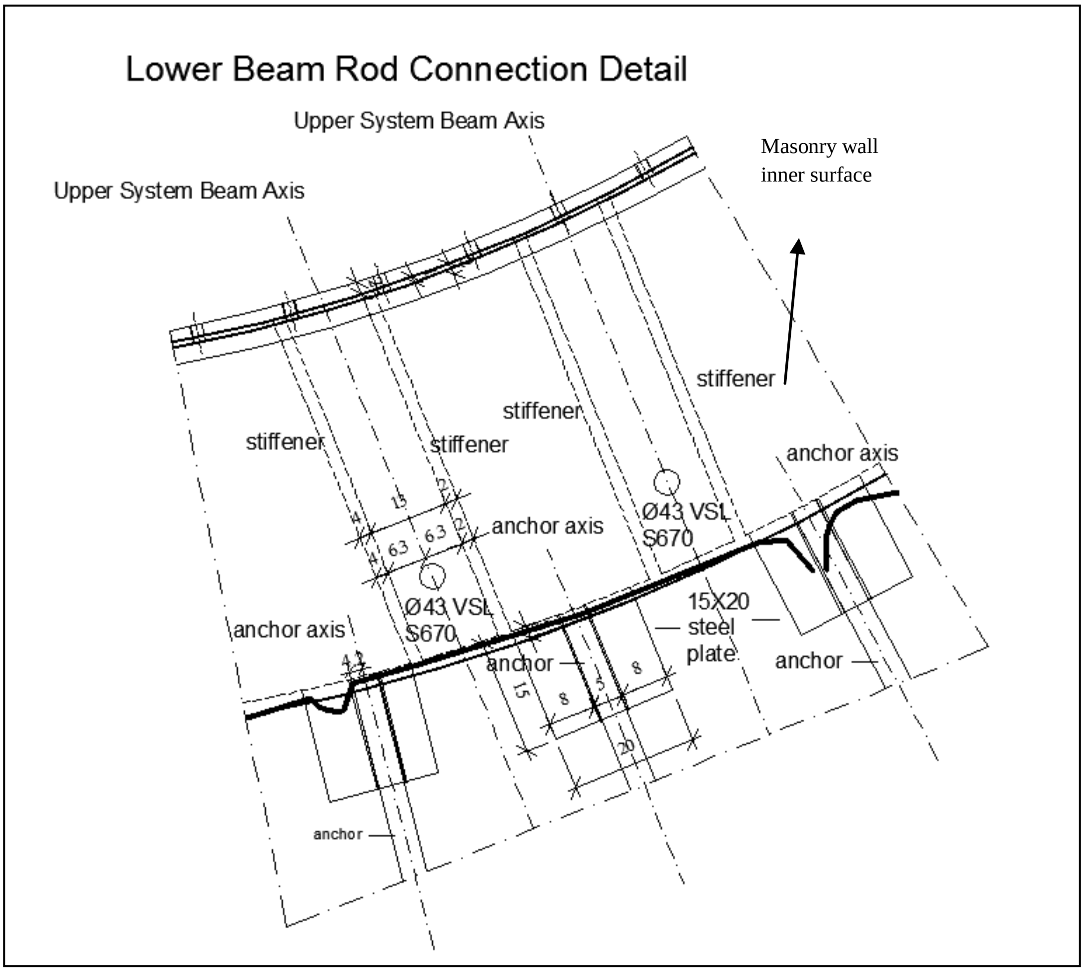


Figure 4.14 Lower Beam System – Post Tension Rod Connection Detail (Plan Section)

IPE 600 beams that lay on a steel base plate form the upper system. Before base plate is placed on the walls, the smooth surface was considered to be achieved by partial reconstruction on top of walls and capping with repair mortar on top of upmost wall layer. I beams were thought to be fastened to base plate with stiffeners. Rods were designed to be connected to edges of I beams. The rod connections was suggested to be made to flanges of the I beams because of the lateral buckling or flange buckling risk for them. Two plates forming triangular plan shape was designed to be welded to the head plates of I beams to form a space that rods can be fit into (Figure 4.16). Therefore, the rods would have apply a force that cause only shear on I Beam sections. Upper system assemblies were formed with pairs of I beams and this assemblies were considered to be fit on risky parts of towers (Figure 4.17).

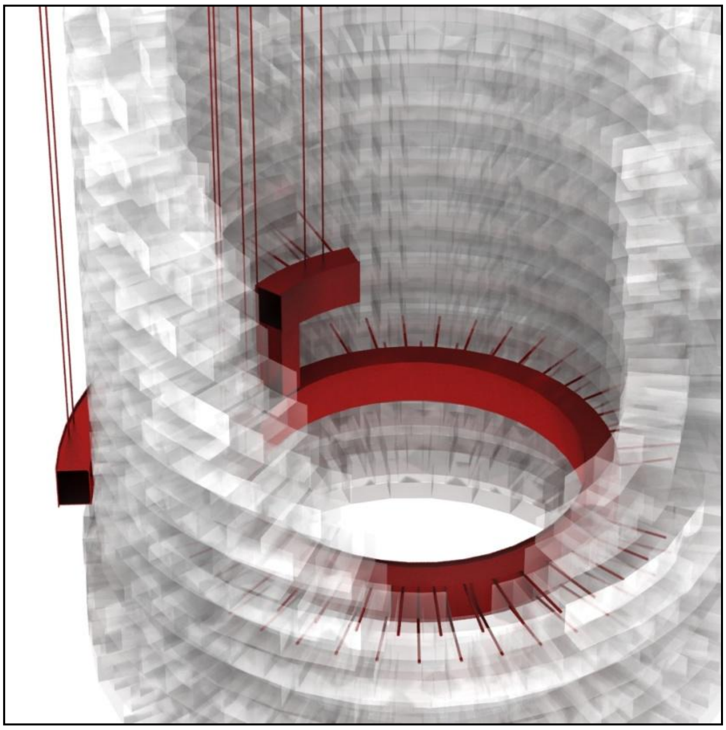


Figure 4.15 3D Max representation of the Eastern Tower post-tensioning system lower part view made by Seray Türkay.

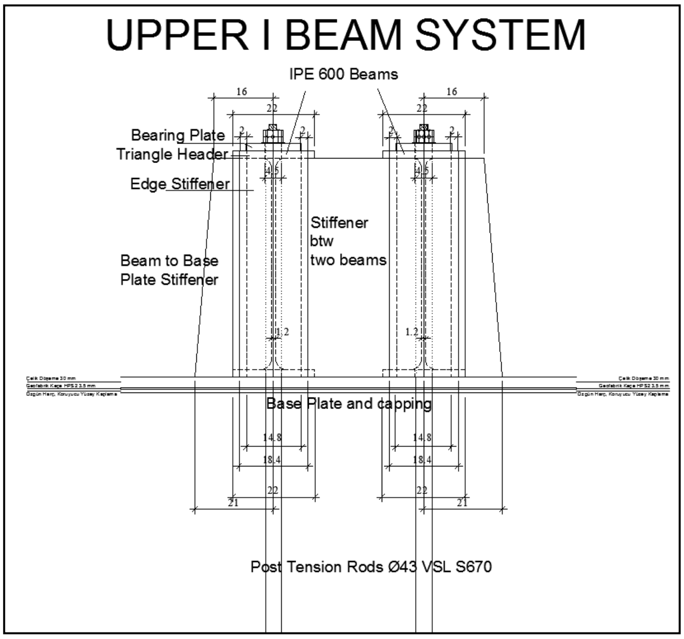


Figure 4.16 Upper Beam System Assembly Front View

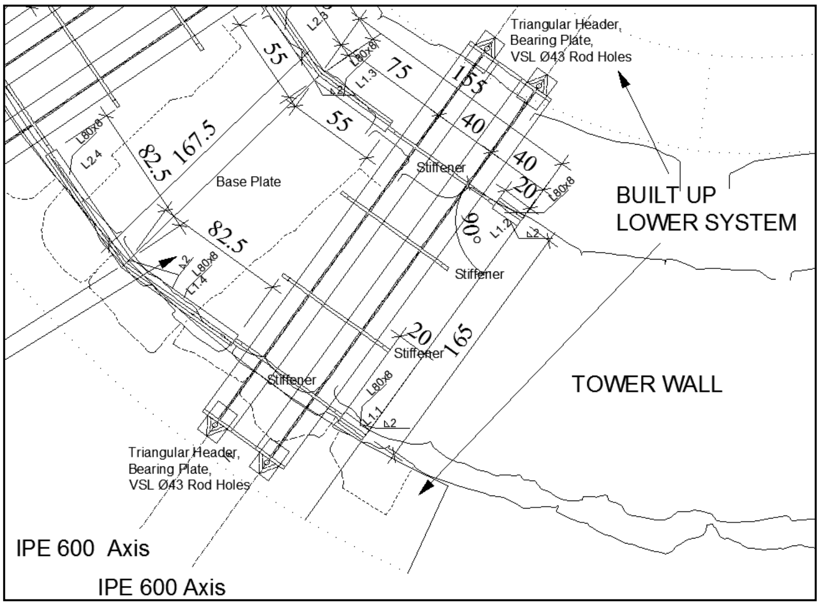


Figure 4.17 View of Upper System: An Assembly composed of Two Beams and Base Plate

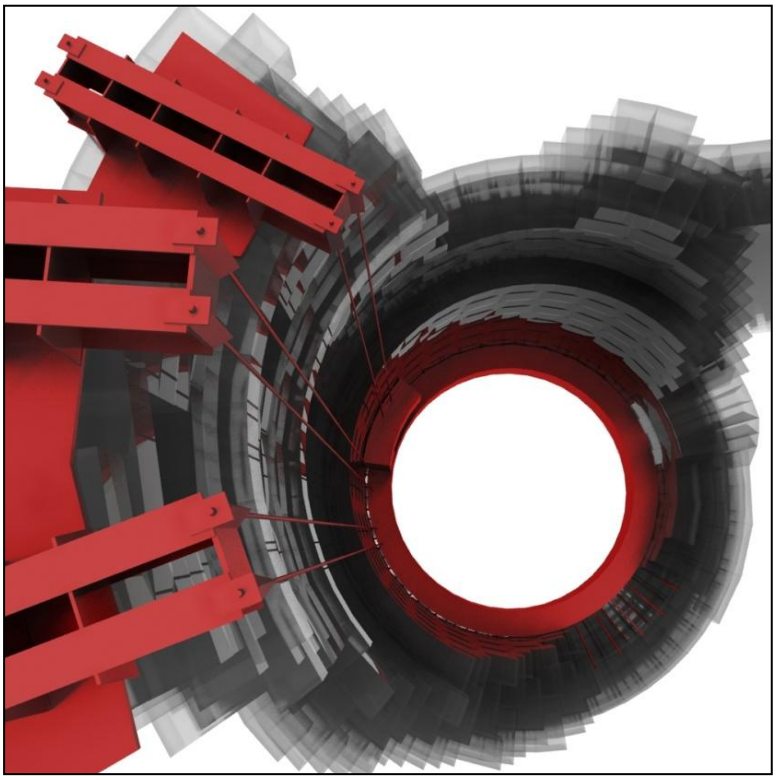


Figure 4.18 3D Max representation of the Eastern Tower post-tensioning system upper part view made by Seray Türkay.

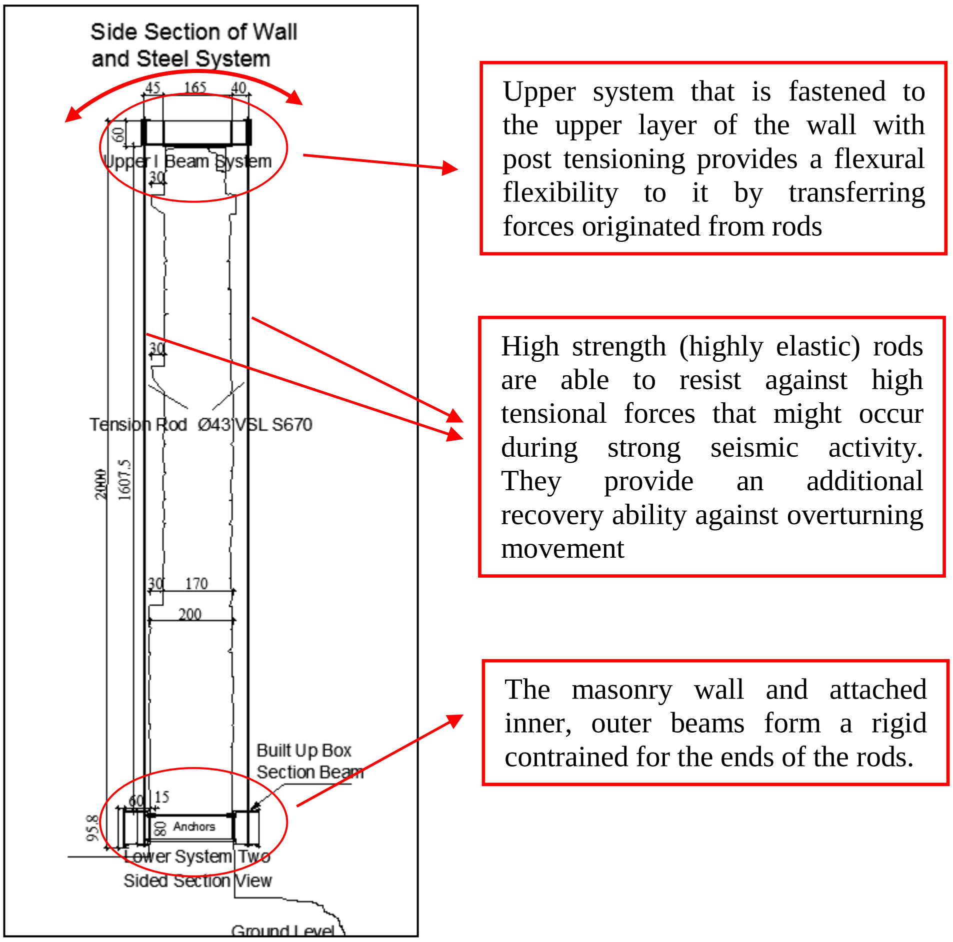


Figure 4.19 Side Section of the Tower and the Strengthening System

4.3 Comparison of Intervention Methods

Four different methods of intervention were compared by considering not only their structural contribution to global behaviour of the towers, but also they were evaluated in the manner of contemporary preservation principles for building heritage.

Girder System; All along the proposed strengthening methods, the lintel system was the least interventionist idea if the partial reconstruction at the east side of the Eastern Tower is ignored. Nor it disturbs aesthetical meaning of the towers or alters the mass and color relation of the architectural features of the structure because it includes small amount of volumetric addition that can barely be seen. On the contrary, according to analysis made, the proposed system and its derivatives are not adequate to prevent collapse of the towers during a strong seismic activity. Moreover, although it was not suggested to be a compulsory work to be done, the reconstruction of eastern side of the Eastern Tower up to the 16th stone layer is not a acceptable intervention while the archaeological and architectural features of the *lacuna* is still have not been clearly defined.

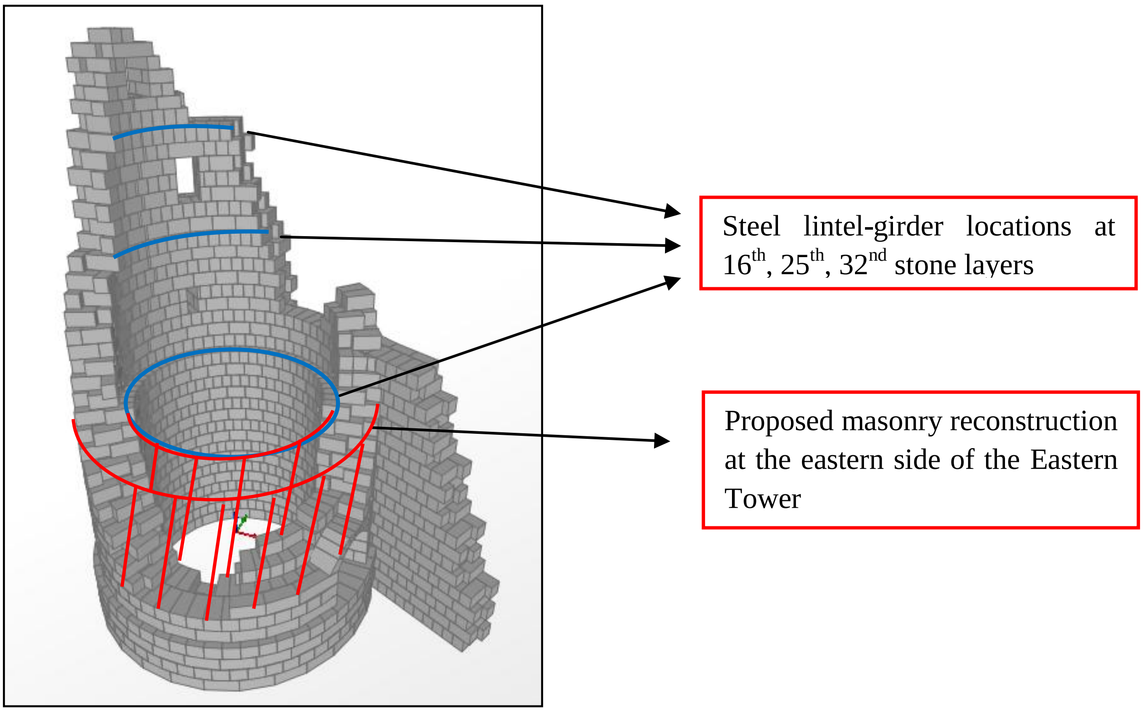


Figure 4.20 The Reconstruction that was proposed in the Project of Strengthening with Lintel-Girder System.

Tripod System;

The steel structure of the tree-like tripod system was analyzed by designer and proven that they could adequately support towers. Especially contribution to the overturning resistance was planned to be validate by supporting projecting stone layers from their top therefore, eccentric vertical forces are applied to form resisting moments. However, in order to reach 25th 32nd levels of towers a tall system having long arms was designed to be fit into the middle location of towers' inner hole. A slender main branch of the support system was supposed to be compensating large deformations related with both eccentric forces due to branches and lateral forces due to the seismic affect on masonry structures. On the other hand, this stability problem requires a stiff foundation to be built under tower. In this sense a concrete foundation which is supported with ~7.5 m long micro piles had been included in the proposal by German team (Figure 4.21). In addition to the fact that structures are being located at an archaeological site, the need of reversibility to be assured with preservation attitude are wiping out the reality of this system is being accurately fit in to the conditions of Perge Hellenistic Gate Towers.

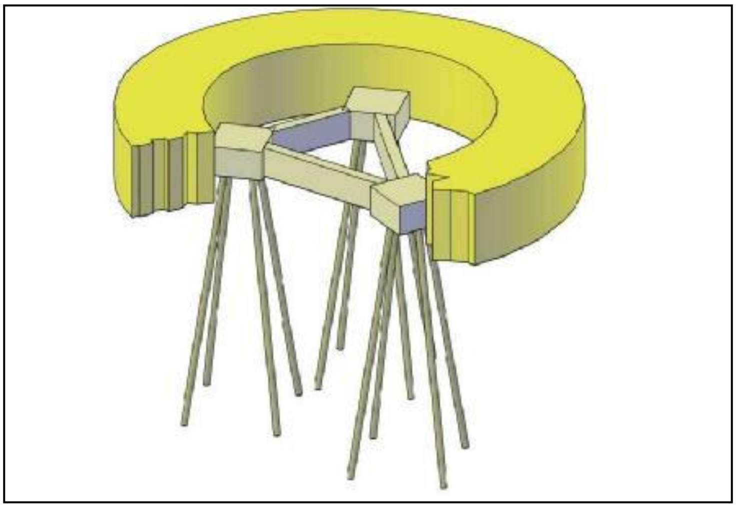


Figure 4.21 Foundation System that was suggested by German Team to carry Tripod Structure

The implementation of this support system requires the Byzantium mosaics (Figure 4.22) that were found at the ground level of the Western Tower at 2010 whereas this kind of decision was strictly forbidden by the Article 8 of the Venice Charter.

Not only being constructed in modern materials and styles, huge mass of supporting system disturbs the authentic perception of the towers. More importantly, the interesting architecture of the tripod causes it to be considered as an object having an architectural meaning in contrast with the masonry towers' instead of being an ordinary strengthening system

Tubular Frame System;

The structural analysis made for the suggestion made by Italian team was revealed that it is possible to prevent collapse with this tubular form steel tower. However, the detailed structural investigation and calculations for design of steel system was not made by proposer. From the analytical models it was found that a huge structure which has densely located columns and beam having large sections is required to obtained required performance from the structure.

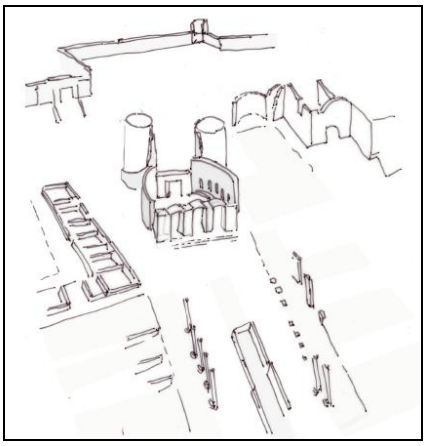
(a) (b)

Figure 4.22 Ground Level of Western Tower after 2010 Excavations by SAYKA; a) Byzantium Mosaics covering the Ground Level b) Closer View of Doorstep Mosaics

Since it is not possible to anchor this huge slender system to the walls due to the additional lateral seismic forces originating from steel tower to masonry ones, a thick concrete foundation is required to be built inside of the towers. The need of deep excavations and vibrations that might be affecting the towers during a dense constructional work, make intervention as a very aggressive one compared to other three choices. On the other hand, the irreversibility is not possible for foundation although the steel superstructure can be built in bolted modular form.

The aim of preservation is given by Venice Charter Article 3 defines the preservation of built heritage because they are being unmovable historical documents. Therefore, especially, archaeological preservation should target the idea of conserving structures without disturbing ability of structure to be examined with visual aspects. Since the steel tubular form is covering the whole inner space of the towers, it hinders the visual perception from inside.

(a)



(b)

Figure 4.23 a) The Northern View of Proposed System and Masonry Towers b) Hand Sketch of Hellenistic Propylea and Steel Support Towers (Perbellini, 2010).

On the other hand, because of the huge mass of new tubular structure which is even reaching higher altitude than original towers a scale dilemma occurs about the both relation between original and new towers. More importantly, the contrast between new and old becomes an accurate problem about not only the Hellenistic Towers but also the Perge Site itself.

Post-tensioning System;

The last recommendation of post-tensioning structure can be considered as most appropriate one among four alternatives.

Structural analysis results showed that it was successful to consolidate structure against destructive actions of self-weight and seismic activity. It was dictated by Heyman (1995) that the frictional resistance of masonry against lateral forces is the main concern of dry masonry resistance instead of vertical bearing action. This method takes in to the consideration this resistance behaviour and increases the frictional endurance of it, which means not only conserves the visual (architectural features) and historical aspects of the towers but also flashes load bearing style of masonry towers out. On the other hand, the segmental arched tower walls can be confined against lateral action.

Because it is not possible to intervene the ground level of structures (excavations) the strengthening system was proposed to fastened to the towers by anchoring. Hence, irreversibility about the post-tensioning system arouse from the permanent anchor holes designed to be located at lower site of the masonry walls. No other irreversible part was included in the general design of system and being fully designed in steel material makes it easily to be dismantled.

Simple steel sections and rods were used to form whole system which do not comprise a architectural aesthetic, thus no alteration disturbing affect was supposed to be made on the mass, scale and colour relation about the authentic towers. On the contrary, system was being planned to be constructed in modern materials makes intervention details to be easily distinguished from original masonry elements while no masking of visual aspects of the tower was made by the additions of strengthening system.

A comparison summary of four alternatives is given at the Table 4.1.

In this study, the first three stages of medical analogy was tried to be defined and fulfilled. However, in the cyclic work scheme of analogy the last stage *Control* forms a significant stage of the whole conservation process. In the case of post-tensioning strengthening of the Perge towers, control is also needed to be done. It was previously defined that the material decay is very serious aspect which also affects structural performance. Therefore, mapping of decay condition, which was provided with quantitive investigation for stone elements, should be made in a period of time. Moreover, the performance of the strengthening system by monitoring of steel cable tensional forces and critical wall inclinations should be made. Meanwhile, other sources of collapse and material decay like wind loads or climatic action can also be monitored.

Figure 4.24 3D Max model representing possible form of the Eastern Tower after strengthening (model was made by Seray Türkay)

Table 4.1 Comparison Table for Intervention Methods in the Frame of Contemporary Preservation Concept and Structural Efficiency

		Preservation Articles given by Venice Charter						
		Considering the main aim of conservation by Article 3 Should not mask the feature of heritage building being a historical document	**Article 4, 10 Performance to provide sustainability; Structural performance is proven with scientific study**	**Article 6 No alteration in mass and colour relation is permitted**	**Article 8 Paintings, decorations should not be removed if possible to be preserved in place**	**Article 9 Reveal and preserve aesthetic value of the monument**	**Reconstruction must be avoided as possible; especially in an archaeological site.**	**Reversibility & Rate of Intervention**
Intervention Definition	**Steel Lintel - Girder System**	Have no such a negative effect	**It is not structurally efficient. Do not prevent collapse of the towers.**	Have no major effect on alteration in mass and colour balance	The decorative structural elements do not need to be moved	Do not have any effect on aesthetical value of the structure	Applications suggests the partial reconstruction at eastern side lacuna of the Eastern Tower architectural features of which is still not exactly be defined by researchers	Can be detected from stones but the anchor holes will be remain existed after removal (minimal Intervention)
	Steel Tree-like Tripod Support	Although designed as a big mass, do not prevent accessibility to any section of structure and do not prevent visual inspection on structure	Affectivity was proven by scientific study by designer of the system	Being constructed in the heart of structure with a modern techniques and material with a huge mass might be considered as a intervention getting out of the scale	Especially the requirement of stiff foundation including 10 m micro piles to be built both causes the removal of Byzantium Mosaics in The Western Tower and archaeological objects that might be existing under unexcavated rubble.	The system itself holds a new aesthetical value, which may be in contrast with the one of the towers.	Requires no reconstruction	Micro piles and the required foundation cannot be removed whereas the upper structure can be removed without leaving any traces on tower walls
	Inner Steel Tubular Frame System	Even the accessibility inside the steel tube is still possible; the visual perception about monument is partially blocked. The possibility to investigate structure from inside is reduced.	Affectivity was proven by scientific study in this thesis work by analytical modelling	Being constructed as an another tower inside the monuments totally alters the traditional setting scale	The requirement of huge foundation causes Byzantium Mosaics at the Western Tower to be removed and revelas the need for further excavations.	Being constructed as a new tower with contemporary material and technique the effect of the new tower on old ones aesthetic is debatable	Requires no reconstruction	The foundation was supposed to be built under tower is assumed to be huge mass which can not taken back from the site whereas the dismantling of the steel tower is possible
	Post-tensioning System	Have no such a negative effect	Affectivity was proven by scientific study in the scope of this study with a holistic approach including contemporary scientific tools and techniques	Compared to mass of target structure it do not alter the balance in the perception of mass and colour relation	The decorative structural elements do not need to be moved	Although the dimensions of intervention system is small compared to 2nd and 3rd methods, being in contrast with authenticity it decreases the aesthetical value of the towers	Considerable very small amount of reconstructions in order to achieve smooth surfaces	Anchors in the lower side of the structure should be left in place if the recovery of system is required. Therefore, it is partially reversible

CHAPTER 5

CONCLUSION

In this thesis, the adaptation of the of the medical analogy to “preservation engineering” defined by Kelley (2005) and mentioned in the ICOMOS Recommendations (2008), which includes stages of anamnesis, diagnosis and therapy, was adapted and applied for the structural modelling, analysis, evaluation and strengthening of the Perge Southern Gate Hellenistic Towers. In the first step, an anamnesis, including a comprehensive literature survey and field works focusing on architectural and archaeological features, material decay, and a structural condition assessment was conducted on the towers. This was followed by a diagnosis which was based on ambient vibration tests, axial compressive strength experiments in the laboratory, hand calculations, analytical models. The analytical models were verified with data obtained from dynamic tests and numerical calculations. Finally, a course of therapy was suggested involving a series of strengthening proposals for the towers and a verification of their efficiency through the use of analytical models for earthquake simulations.

In the earlier states of the study, it was realized that linear elastic analytical models failed to represent the realistic behaviour of the towers. Models that do not take into account existing discontinuities (contact surfaces between stone blocks, existing cracks etc.), current deformations, and effect of material decay on the input parameters of material physic-mechanical properties, cannot reach to a realistic structural analysis for dynamic loads. Moreover, the lack of discrete movement of mesh elements and non-splitting joints cannot model and simulate energy dissipation mechanism when compared to the existing structure. For this reason, an unrealistic stress distribution mapping of the towers was obtained from linear elastic analytical models tested under seismic excitation, from which it was concluded that linear elastic continuum models were unsuitable for structural investigations of historical dry masonry structures.

The use of nonlinear models for structural investigations of heritage buildings is necessary for the realistic monitoring of structural behaviour from the linear elastic region through to the plastic region of degradation, and further to collapse (Lourenço, 1996). In this sense, the nonlinearity issue in the collapse investigation and structural performance inspection of the Perge Hellenistic Gate Towers was resolved with an analysis of discrete element models.

Previous studies using DEM as a tool for seismic vulnerability estimations of masonry heritage building (Baggio, C et al. 1998; Pegon, P. et al. 2001; Psycharis, I. et al. 2003; Invernizzi, S. et al. 2010) have focused on representative parts of the structures or analytical models with relatively small degrees of freedom for dry masonry heritage buildings; but in contrast, this study has been made for a larger scale object (dry masonry structure having ~5000 elements) in order to capture the overall behaviour.

The global nonlinear behaviour of the towers, majorly depending on the contact surface separation and nonlinear friction surface character between stone blocks, was successfully represented in DEMs. The accuracy of the analyses may be verified by simplistic limit state hand calculations and through a comparison of analytical models and dynamic tests on real structures for micro level vibrations.

On the other hand, used discrete stone block based analytical modelling has linear elastic models for the stone blocks. Therefore, crushing and cracking of stone blocks during the nonlinear earthquake simulation is neglected. Although the nonlinear model used in this study is far better and linear elastic models, the results can be further improved if more sophisticated models (including meshed discrete blocks) that can simulate block cracking and crushing were used. Uncertainty of the blocks' material properties due to variable nature of the material as well as small differences of the stone building blocks' dimensions makes the use of more sophisticated methods inappropriate.

It is clear that the performance of discrete element models on the Perge Towers is a case-dependent issue. The selection of the modelling technique depends on the architectural features, and especially the construction technique of the towers. This means that the same accuracy may not necessarily be achieved for structures built using different construction techniques, such as brick-mortar masonry.

The earthquake simulations conducted on the original (undamaged) Perge Towers resulted in understanding possible collapse mechanisms. The same analytical model (modified to represent currently existing damaged form of the Perge Towers) was used to simulate possible future earthquake damage scenarios in a more realistic way. The prediction of current damaged condition starting from the original (reconstituted) form can be defined as “reverse engineering” and allowed to make predictions with more confidence.

The analyses showed that even the original circular plan forms of the Perge Towers, which still maintain their structural integrity at every stone level, were vulnerable to strong seismic activity (with PGA: 0.4-0.5 g). In contrast, it was determined that the vulnerability level has intensified for the towers in their present state since they have lost their cylindrical shape and remaining parts stand close to planer walls, with the analytical simulation evidence indicating that they would suffer total collapse in the event of relatively smaller earthquakes (with PGA: $\cong$ 0.2 g).

The outward flexural movements of the tower walls were found to be the most likely cause of collapse for both towers. It was determined that the inward collapse is prevented by arch formation of the cylindrical form, further improved the construction pattern of the stone blocks tapered for and staggered bond increased the resistance of the wall sections against inward flexural movement, while the stone orientation would not contribute to an outward overturning as much.

The strengthening performances of the three previously proposed strengthening methods were evaluated and a new system was determined based on a DEM analysis. Considering the estimated structural behaviour and vulnerability of the towers against lateral loading, a strengthening proposal that aims to increase axial compressive stress, and thus contribute to both the sliding and overturning resistance of walls, was developed. According to the results, all of the proposed strengthening systems, aside from the lintel-girder system, were found to increase structural stability. A comparison of each proposal showed that the most appropriate system according to contemporary conservation approaches was the post-tensioning system proposed in this study. Detailed comparison of methods is provided in Chapter 4. The advantages of the proposed post-tensioning proposal system are i) it is a sustainable and ii) reversible system, iii) does not generate a distinct visual effect on the architectural perception of the towers. Based on a minimal intervention approach, it provides structural resistance without the addition of a large mass preventing increase of earthquake inertial forces, increases friction force between building blocks, provides tension rebars against overturning, prevents comprehensive reconstruction or the removal of any parts of the structure, and protects it in its current form. The disadvantage of tripod system was the necessity of micro pile foundation, which was not permitted in archaeological sites.

Both immediately before and after strengthening activity key aspects of material decay, stress rate on tension bars, inclination of risky tower walls, wind and acceleration excitations, and climatic action should started to be monitored.

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