STRUCTURAL ANALYSIS OF HISTORIC ASPENDOS THEATRE

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ABSTRACT

STRUCTURAL ANALYSIS OF HISTORIC ASPENDOS THEATRE

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Aspendos Theatre still stands in fairly good condition although it has been constructed about 2200 years ago in Serik village of Antalya, Turkey. Aspendos Theatre is one of the most valuable historical buildings in Turkey. The fact that the structure had overcome numerous possible earthquakes during its lifespan in Antalya and located in second degree earthquake zone, makes the subject an interesting research topic. The earthquake analysis of Aspendos Theatre was conducted using “Specification for Structures to be Built in Disaster Areas” code and stress levels are investigated using 3D FE modeling. Also, the resonance state of the theatre under sound induced forces due to concerts and exhibitions performed in the theatre has been examined. Structural identification is performed to obtain certain structural characteristics by comparing experimentally measured and analytically obtained natural frequencies. The analytical model is constructed using solid members and the analysis is performed by using SAP2000 software. The elastic modulus of conglomerate used as building blocks in the Theatre is taken as 2350 MPa based on the experimental and analytical studies. The compressive and tensile strength of the theatre wall material is taken as 12

MPa and 1.2 MPa, respectively based on the previous studies conducted on conglomerate. When the maximum stress levels under combined effect of response spectrum and dead load analyses are examined, the level of compressive stress is found to be about 60% of the compressive strength. On the other hand, the tensile stresses developing at upper corners and bottom middle parts of the stage wall and mid-height central location of the exterior wall (on the vicinity of the front door) are calculated to be about 6.6 MPa, which are more than the assumed tensile strength. It has also been calculated that the level of sound that generates tensile failure is about 125 dB as the theatre gets into resonance state.

Keywords: Aspendos, Resonance, Earthquake

ÖZ

TARİHSEL ASPENDOS TİYATROSUNUN YAPİSAL ANALİZİ

Boz, Berk
Yüksek Lisans, İnşaat Mühendisliği Bölümü
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Antalya ili sınırları içinde Serik ilçesinde bulunmakta olan Aspendos Tiyatrosu, 2200 yılı aşkın süredir ayakta kalmış ve hala bazı sanatsal etkinliklerde kullanılmaktadır; ülkemizde bulunan tarihi eserlerin en önemli ve değerlilerinden birisidir. İkinci derece deprem bölgesinde bulunan tarihi yapının ömrü boyunca bir çok büyük depremi başarıyla atlatmış olması araştırmaya değer bir konudur. Aspendos tarihi tiyatrosunun afet yönetmeliğinde belirlenen deprem yükleri altında dinamik analizi gerçekleştirilmiş ve üç boyulu sonlu elemanlar modellemesi kullanılarak yapıda oluşan gerilmelerin seviyeleri araştırılmıştır. Ayrıca Aspendos Tiyatrosunda sık sık gerçekleştirilen konser ve gösteriler sırasında oluşan ses dalgalarının yapının resonans durumuna girmesine sebep olup olmadığı da araştırılmıştır. Analitik modelden elde edilen doğal salınım periyotları tiyatrodan alınan dinamik ölçümler ile karşılaştırılmış ve gerçekçi bir model için gerekli olan yapı karakteristikleri elde edilmiştir. Model dolu (solid) elemanlar kullanılarak oluşturulmuş, analizler SAP2000 programı kullanılarak yapılmıştır. Yapıda kullanılan konglomera tipi doğal taşın elastik modül değeri deneysel ve analitik çalışmalar sonucu 2350 MPa olarak alınmış, basma ve çekme gerilim kapasiteleri sırasıyla 12 MPa ve 1,2 MPa olarak kabul edilmiştir. Tepki spektrumu analizi ve kendi ağırlığı altında oluşan gerilmelerin toplam etkisi incelendiğinde malzemenin basma kapasitesinin en çok %60 mertebesinde zorlandığı; çekme yönünde ise iç duvarın alt kotu, üst köşeler ve dış duvarın orta seviyesinde (kapının bulunduğu bölgede) Kabul edilen çekme kapasitesinin (1.2MPa) aşıldığı (6.6 MPa) görülmüştür. Ayrıca, tiyatronun resonans içerisinde iken uygulanabilecek maksimum ses seviyesinin belirtilen varsayımlar altında 125 dB olabileceği hesaplanmıştır.

Anahtar Kelimeler: Aspendos, Resonans, Deprem

To My Parents

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The author wishes to present his eternal gratitude to his supervisor Assist. Prof. Dr. Ahmet Türer for his assistance, guidance, advices, encouragements, patience and friendly cooperation throughout the research. He is highly appreciated for advising such an interesting and important subject that is really worthy to study extensively with curiosity and great concern.

The author would also like to thank to the director, Sabri Ünal of Antalya Museum, fire department of Antalya and the staff of Aspendos theatre for their great support during the tests performed in Antalya, which were required for the successful completion and execution of this study.

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CHAPTER 1

INTRODUCTION

1.1 GENERAL INFORMATION

The historical Aspendos Theatre is located at a distance of 38 km from Side. The theatre was built in B.C. 200 by Xenon. Xenon was an architect in Aspendos city which was built on a mountain at the edge of Perge plain. The secret of the amazing acoustics of the theatre created by the architect Xenon is still unknown.

The theatre is known as one of the most preserved amphitheatre in the world. The city of Aspendos and the theatre is well kept by Pergamums until 129 B.C. Like its neighbor Perge, Aspendos, which was attached to Rome in 129 A.D., was looted by the Roman politician Verres. He removed all the sculptures and statues from the structure. However, there was no damage on the main body of the theatre. Starting from early 13th century, Aspendos began to exhibit signs of settlement by the Seljuk Turks. During the rule of Alaeddin Keykubad I in particular, the theatre was restored and used as a caravanserai. This is one of the reasons it has survived in such good condition. Between 13th century and the present time, the theatre was kept undamaged and well preserved. Although at the present time, many music concerts and exhibitions in the theatre are being performed, the general condition of the theatre is worse than the condition it should be (e.g. plants growing on cracks, poor structural condition, unstable stones, etc.).

The future preservation of the historical Aspendos theatre is very important. The theatre has seen many civilizations and witnessed many phenomenon, wars and battles in the last 22 centuries but kept as almost in the same condition built. Like the other great historical heritages left by the previous civilizations in Anatolia, it should be taken into consideration that the conservation of this special theatre is an obligation for us, the present owners of these lands.

Several studies on the famous acoustics of the theatre have been performed in the last few decays. But, this study is the first one which examines the effects of the sound induced forces on the structure.

1.2 OBJECTIVES OF THE STUDY

The main purpose of this study was to find out the effects of the sound induced forces on Aspendos Theatre. There are many music concerts and exhibitions in the theatre. Every music and sound has a frequency. The frequency ranges and the magnitudes of the sound pressures have been determined by the measurements. The natural frequencies of all the parts of the structure have been investigated by modeling of the Aspendos Theatre. Also, the natural frequency measurements have been taken in Aspendos theatre for the parts which has been selected as critical after performing the model analysis. Comparisons for the natural frequencies of the structure between the model analysis and tests preformed in Aspendos theatre have been made. After getting the correct frequencies of the parts of the structure, comparisons between the frequencies of the sound source and the natural frequencies of the structures have been made. When the frequencies are matching, the structure has been analyzed in its resonance state. Structures may suffer extraordinary damages in their resonance states when they are applied even comparatively small forces due to the large deformations.

The earthquake analysis of the Aspendos Theatre has been also performed. The results have been compared with the results of sound induced forces analysis to see the differences and to determine the limit of sound pressure level.

Another objective of the study was investigation of non-destructive testing method for condition evaluation of the theatre columns. To fulfill this objective, first bending natural frequencies of the backside peripheral columns were measured and compared against visual inspection results.

CHAPTER 2

LITERATURE REVIEW

2.1 INTRODUCTION TO FAILURES DUE TO RESONANCE STATE OF THE STRUCTURES

The breaking of glass by a sound with a specific frequency is a well known example to explain the effect of the resonance on the structures.

Glass has a natural resonant frequency at which it will vibrate easily. If the sinusoidal force making the glass vibrate is large enough, the deformation due to vibration will become so large that the glass will break.

The procedure listed below can be followed for this test;

1. The sound, which is used for this test, must contain a large amount of energy at the resonant frequency of the glass.
2. A sine wave generator which creates sound at the resonant frequency only, is used,
3. The frequency of the signal should match the resonant frequency of the glass.
4. The resonant frequency using an FFT analyzer is measured.
5. The glass is excited by rubbing a damp finger on the top of the glass.
6. The signal's frequency is fine tuned until a maximum displacement was reached.
7. At the maximum displacement of glass, the breaking occurs (http://www.acoustics.salford.ac.uk acoustics_world/glass/glass.htm and http://www.physics.ucla.edu/demoweb/ demomanual/acoustics).

2.2 HISTORICAL FAILURES DUE TO RESONANCE

Resonance may cause violent swaying motions in improperly constructed structures such as bridges. Both the Tacoma Narrows Bridge (nicknamed Galloping Gertie) and the London Millennium Footbridge (nicknamed the Wobbly Bridge) exhibited this problem. A bridge can even be destroyed by its resonance; that is why soldiers are trained not to march in lockstep across a bridge, but rather in breakstep.

2.2.1 THE TACOMA NARROWS BRIDGE FAILURE

Location: Washington State, USA

Design and Construction:

- Seattle to Tacoma with nearby Puget Sound Navy Yard was connected by this bridge.
- Two 128-m. towers were connected by 853-m. main span.
- This bridge was the third longest bridge in the world.
- Tacoma Narrows was much narrower, lighter, and more flexible than any other bridge of its time. With a 12-m wide and 2.4-m deep concrete deck, it accommodated two lanes of traffic quite comfortably.
- The appearance of the bridge was sleek. It was not an illusion that Tacoma Narrows had a light appearance. Its dead load was 1/10 of that of any other major suspension bridges.
- Because of these unique characteristics combined with its low dampening ability, large vertical oscillations happened in even the most moderate of winds.

Details of the Failure:

- *At 7:30 A.M. on November 7, 1940:* Although the wind was not extraordinary, the bridge was undulating noticeably and the stays on the west side of the bridge which had broken loose were flapping in the wind.

- *Just before 10:00 A.M.:* After the wind speed is measured as 42 mph, the bridge was closed to all traffic due to its dangerous movement (38 oscillations/minute with amplitude of 0.9m). After a couple of minutes, the stiffening girders in the middle of the bridge buckled and the collapse has been initiated. The Photos 2.1 & 2.2 shows the twisting motion of the center span just prior to failure. Then the suspender cables broke and large sections of the main span dropped. The weight of the sagging side spans pulled the towers 3.7 m towards them and the ruined bridge finally came to a rest (see Photo 2.3)

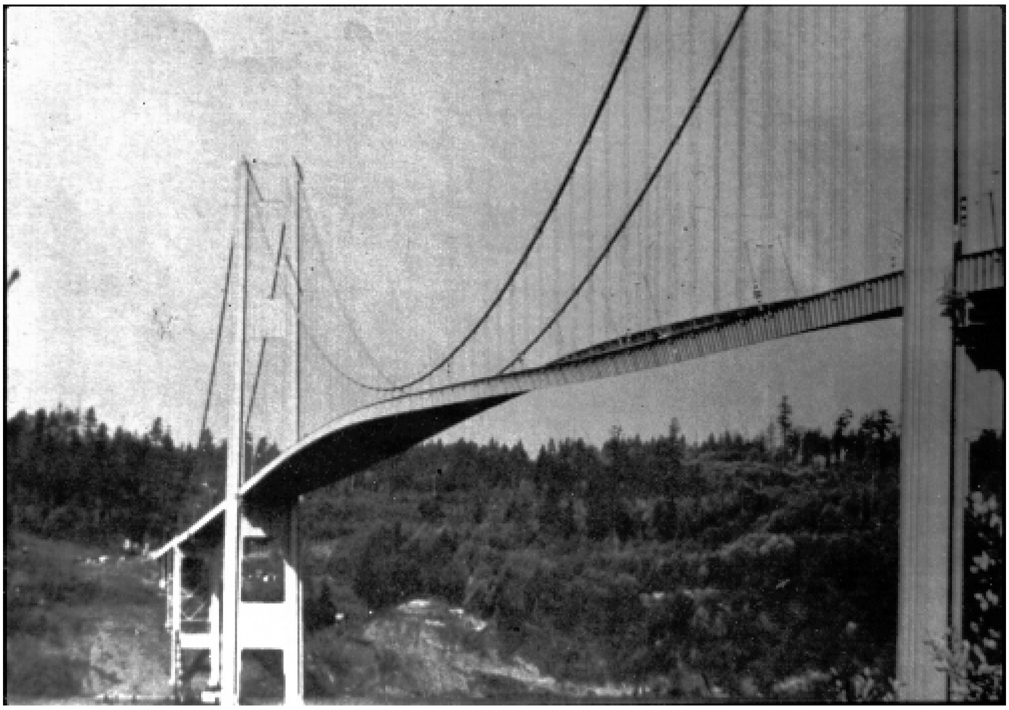


Photo 2.1 Twisting motion of the center span just prior to failure

Photo 2.2 Twisting motion of the center span just prior to failure

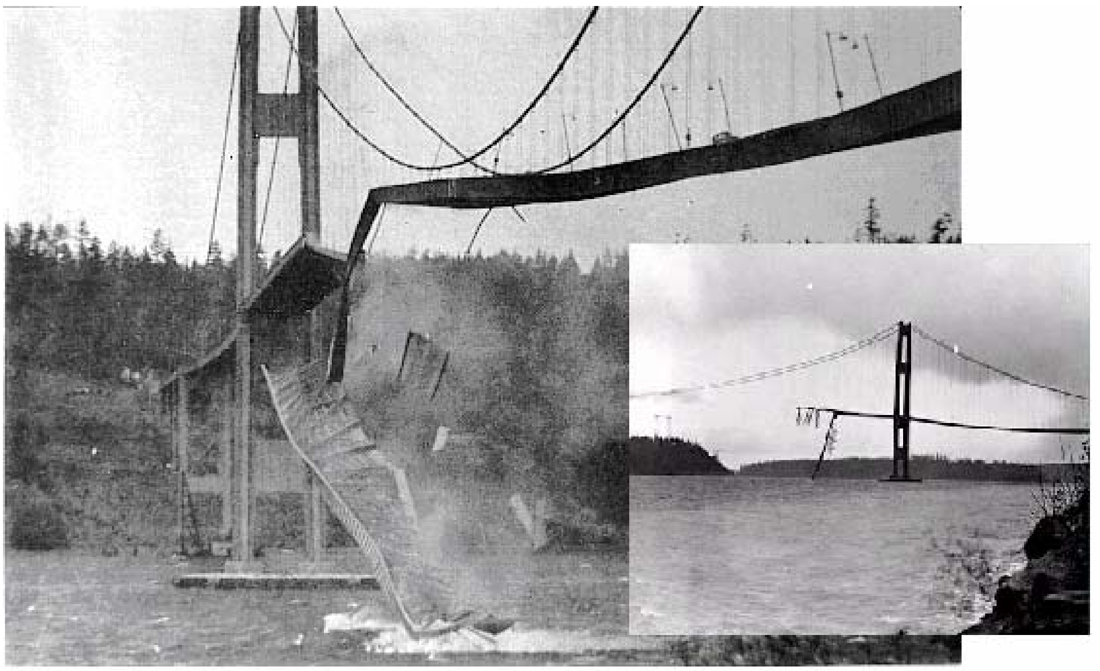


Photo 2.3 The failure

Causes of the Failure:

- There was a steady wind of about 68 at the morning of the collapse although the bridge had been designed to withstand winds greater than 193 kmph.
- The collapse has been concluded by the engineers who were examining the events around, materials used in the construction of the bridge were sound. Also, the design of the bridge was according to the standards.
- There are several theories propesed for the collapse of the bridge. All of the theories agree that the wind blowing over the bridge created pulsating aerodynamic forces. The wind produced a fluctuating resultant force in resonance with the natural frequency of the structure. This caused a steady increase in amplitude until the bridge was destroyed. The period of these force pulses very nearly matched the natural period of the bridge. The driving force for vortex formation feeds off the motion of the structure (see Picture 2.1). A resonance between the forces and the structure was matched and the disaster has been soon followed (James Koughan, 1996 and http://www.vibrationdata.com/Tacoma.htm, 1999).

Picture 2.1 Vortex shedding

2.2.2 THE VIBRATION OF THE LONDON MILLENNIUM FOOTBRIDGE

Location: London, UK

Design and Construction:

- A new footbridge has been built across the river Thames in London.
- The bridge has been called as Millennium Bridge to mark the millennium.
- It is a shallow suspension bridge linking St. Paul's Cathedral on the north side of the river with the Tate Modern art gallery on the south side.
- The bridge is over 300 metres long with three spans, the longest being the centre span of 144 metres.

Photo 2.4 London Millennium Bridge

Details of the Failure:

- The bridge was opened on 10 June 2000.
- For the opening day, there were a crowd of over 1000 people on the south half of the bridge including a band in front of them
- When the people started to walk with the band in front of them, a unexpected excitation occurred on the bridge. The lateral movement was so large that the people had to stop after a while.
- The bridge has been closed for temporarily and not opened until 22 February 2002.

Causes of the Failure:

- There was a significant wind blowing on the opening days. However, it has been realized that the wind was not the main reason for vibration of the bridge.
- It has been realized that the problem was due to the lateral excitation (Photo 2.5).

Photo 2.5 Lateral Excitation by foot steps of humans

- The footbridge had a lowest lateral mode of 1.1Hz.
- The frequency of excitation was very close to the mean pacing rate of walking of about 2 Hz.

- There were average 300-400 people walking on the bridge at the same time. Since the footbridge had a lowest lateral mode of about 1.1 Hz, an almost resonating vibration occurred.
- Moreover it could be supposed that in this case the pedestrians synchronized their step with the bridge vibration, thereby enhancing the vibration considerably" (Bachmann, 1992, p. 636). The problem is said to have been solved by the installation of horizontal tuned vibration absorbers.(Deyan Sudjic, 2001, Tony Fitzpatrick, 2001, H Bachmann, 1992, http://www2.eng.cam.ac.uk/ den/ICSV9_04.htm).

2.3 STUDIES ON HISTORICAL BUILDINGS

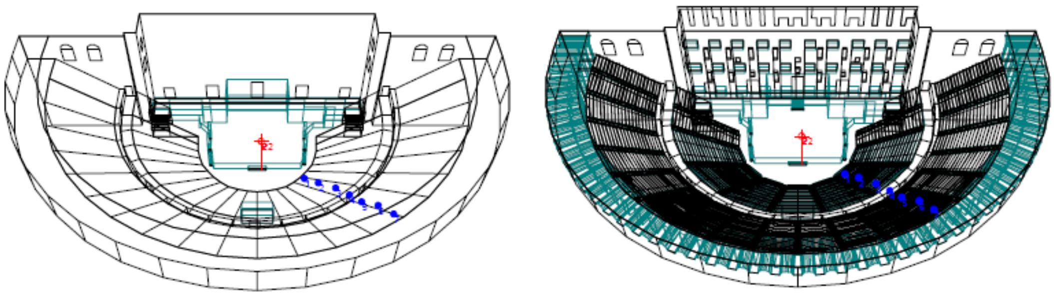
This section explain previous studies conducted by other researchers on historical buildings. Each subtitle includes the reference next to it and the study is briefly explined under each subtitle.

2.3.1 ROMAN THEATRE ACOUSTICS (Gade, 2003)

Aspendos Theatre has been examined in this study by Gade. The analytical model of the theatre has been prepared and examined for its acoustics.

Room acoustics measurements have been carried out in this best preserved Roman theatre. The results are compared with the simulated values from the analytical model of the theatre.

For modeling purposes, “Odeon” program has been used by the researcher (Gade, 2003). During the measurement that had been made in the Aspendos theatre, there was a music concert and the stage for this concert has been also put in the analytical model.



Picture 2.2 Computer model of Aspendos Theatre (Gade, 2003)

Two models of the theatre have been prepared for various levels of analysis (Picture 2.2). The shell elements have been used in the model. The first model includes 362 surfaces, however, the second one includes 6049 surfaces.

The measurements have been performed from different locations. These locations were on two radial lines. The “Dirac” software which is installed on a portable computer has been used for acoustic measurements.

2.3.2 VULNERABILITY INVESTIGATION OF ROMAN COLOSSEUM

(Nakamura, 2000)

Among other wonders of the world, Roman Colesseum has a special place with its big shape and important role in the history (Photo 2.6).

The construction of the Colesseum has been completed from AD 75 to AD 80. It was built on an area that was previously occupied by an artificial lake. The structure has been also destroyed because of the earthquakes during its lifetime.

The purpose of the study was to investigate the dynamic characteristics of Colesseum. Also, identifying main damage mechanism to evaluate the risk of damage or collapse of various parts of the structure was another aim of the study.

Photo 2.6 Colosseum (Nakamura, 2000)

Detailed measurements have been performed at each floor of the structure. The weak points that may collapse during the earthquakes have been found after these measurements.

The analysis has resulted that the dynamic characteristics of Colosseum consists of two parts; inner and outer walls. From the analysis results, it has been found that the predominant frequency and the amplification factor for the outer wall are 1.5Hz. and 3, respectively. They were 3Hz. and 7-10 for the inner wall.

Different mode shapes of the structure have been also simulated in this study.

2.3.3 THE STRUCTURAL BEHAVIOUR OF COLOSSEUM OVER THE CENTURIES (Cerone, 1997)

The aim of this study was to understand this structure’s present structural situation.

Many different mathematical Finite Element models were carried out to simulate the various types of structural behaviors.

The geometrical and mechanical characteristics of the analytical model are the followings:
* Models of the original situation of the Flavian time that consider the interaction with the soil low stiffness and the effect of the first earthquakes
* Models relative to the situation before the 19th century interventions
* Models according to the present situation (Picture 2.3).

The results of the analyses relative to the original configuration (the Flavian time structure) clearly showed that the damages with the distribution and succession of collapses historically documented, are due both to

- The non-homogeneous stiffness of the soil and
- Seismic actions producing adverse combined effects in the area of the east entrance and in the southern sector.

The settlements and the soil movements have certainly contributed to make more critical the effects of the earthquakes. Also, Along the centuries, successive earthquakes together with the progressive lack of maintenance and the subsequent violations, have caused the development of asymmetrical damages and collapses of the structure.

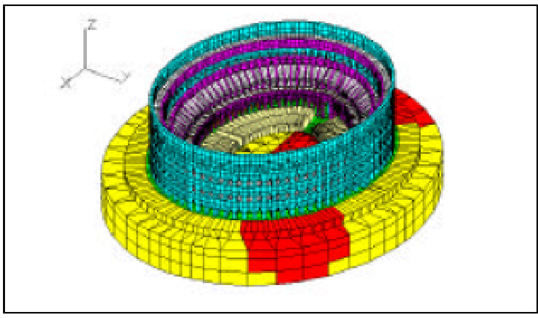


Figure 1 : The Flavious Amphitheatre model with the foundational soil

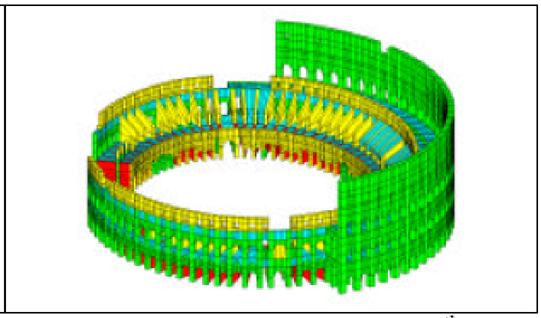


Figure 2 : The structures before the 19th century restorations

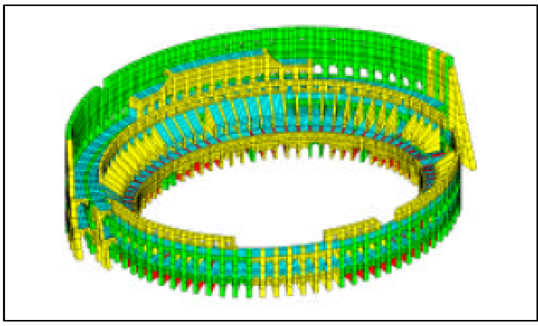


Figure 3 : The present situation

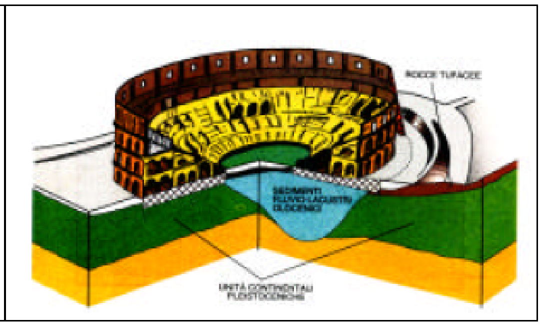


Figure 4 : Schematic geomorphological situation of the site

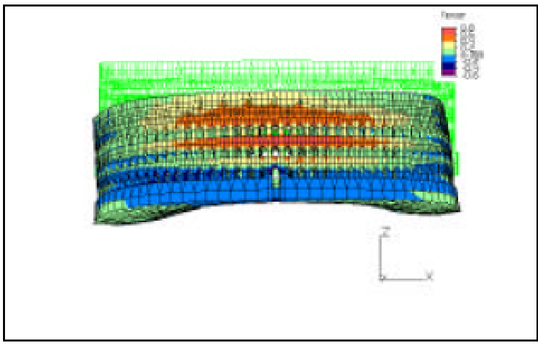


Figure 5 : Annular tensile stresses and deformation, north side view

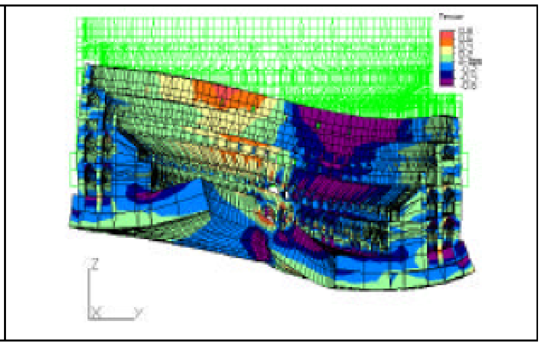


Figure 6 : Section with annular tensile stress and deformation, east side view

Picture 2.3 Analytical model of Colosseum (Cerone, 1997)

2.3.4 LIMIT ANALYSIS OF THE EXTERNAL WALL OF COLOSSEUM (Como, 1996)

The aim of this study was to evaluate the safety condition of the Colosseum's structures under the vertical loading condition.

This study is based on the limit analysis which is developed for masonry structures. The model adopts a rigid no-tension and no-sliding materials.

The structure's external wall has been analyzed. Under the vertical loading of the structure, the possible failure shape has been found and it is shown on Figure 2.1.

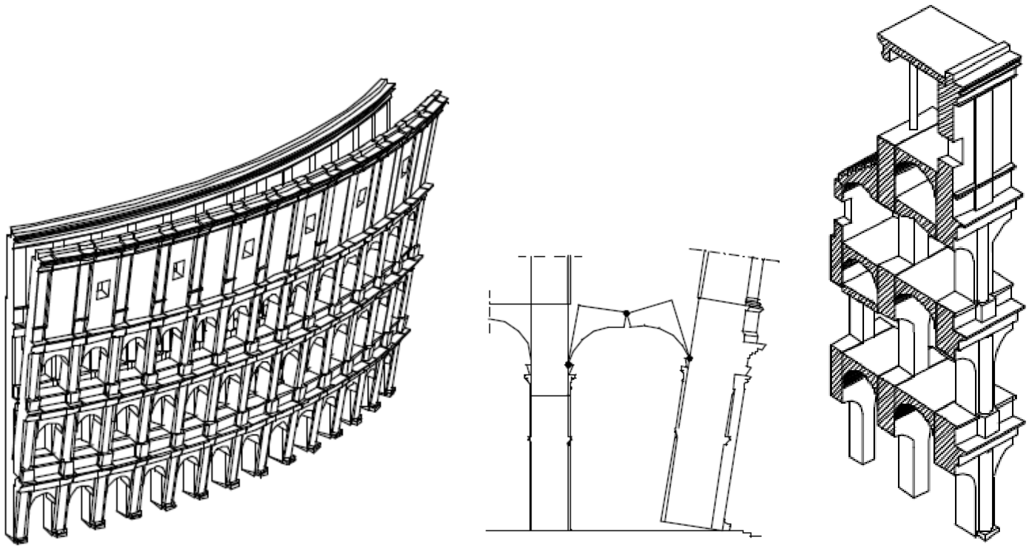


Figure 2.1 The failure mechanism of External wall (Como, 1996)

2.3.5 NUMERICAL MODELING OF STONE-BLOCK MONUMENTAL (Pegon, 2001)

The aim of the study is to describe how 2D and 3D numerical modeling of a built cultural heritage structure for testing at the laboratory and to characterize its behavior.

The S. Vicente monastery (Photo 2.7) has been selected for this study. A realistic model of the column has been built in the laboratory. The structure is made of large blocks with mortar, and masonry.

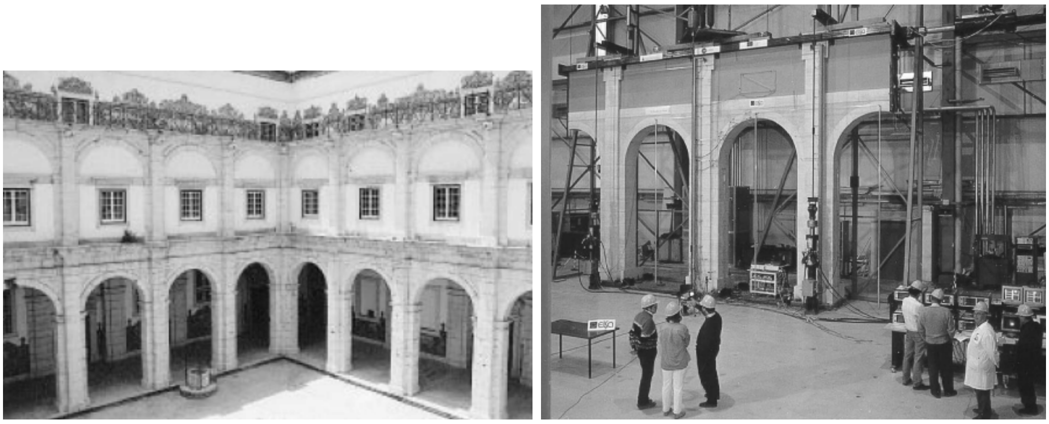


Photo 2.7 Full scale model of the column structures (Pegon, 2001)

Also, 2D and 3D numerical models have been prepared. The experiments have been performed on the columns and the deflection shapes have been obtained (Figure 2.2).

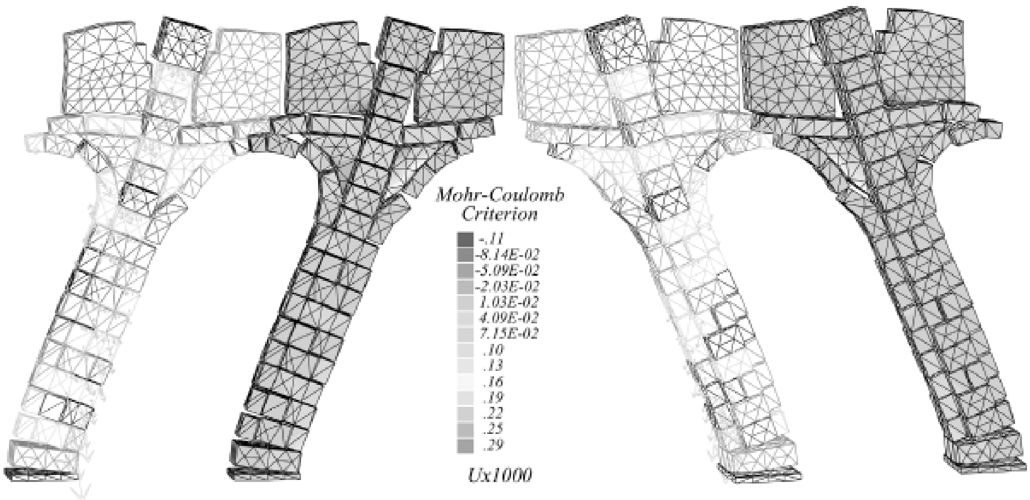


Figure 2.2 The failure mechanism of columns (Pegon, 2001)

After the tests, it has been obtained that 3D analyses allowed to characterize better behavior of the column structure. The calibration of the material parameters was also more efficient. However, because 3D modeling was too time consuming, the prime importance has been give to derive realistic 2D models from 3D analyses.

CHAPTER 3

GENERAL CONCEPTS ON SOUND AND VIBRATION

3.1 GENERAL INFORMATION ON SOUND FREQUENCY, VIBRATIONS AND RESONANCE

3.1.1 FREE VIBRATIONS

Free vibration is the periodic motion occurring when an elastic system is displaced from its equilibrium position.
If an object vibrates without interference at a rate of determined by its physical characteristics, including its mass, tension, and stiffness, these vibrations are called as free vibrations.

3.1.2 NATURAL FREQUENCY

Natural frequency is defined as the number of cycles of oscillation that occurs in a time period when moved from its normal position and allowed to vibrate freely.

3.1.3 RESONANCE

Resonance occurs when the frequency of excitation is equal to the natural frequency of the system. When resonance happens, the amplitude of vibration increases and it is only limited by the amount of damping. Also, resonant frequency is a frequency at which resonance exists.

An example would be pushing a child in a swing. If you time your pushes to match the natural frequency of the swing, the small pushes can be added up to a rather large effect. This rapid increase off of energy and amplitude in the vibrating system is called as resonance.

3.1.4 BEAT FREQUENCY

The difference between two frequencies of a sound gives the beat frequency of this sound.

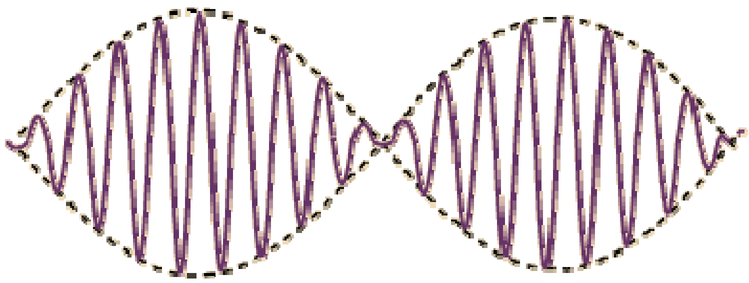


Figure 3.1 Beat frequency as the sum of two waves

A beat frequency is the combination of two frequencies that are very close to each other. For example, if there are two waves at 440 Hz and 445 Hz, the beat frequency will be 5 Hz.

The beats in a song such as regular powerful bass effects during a section of the song can be also considered as beat frequency.

3.1.5 TEST FOR THE RELATION BETWEEN BEAT FREQUENCY AND RESONANCE

A test has been performed to show the relationship between beat frequency and resonance. Sound with various beat frequencies has been given to a structure. The aim of the test was to demonstrate the effect of the beat frequency on the resonance state of the structure. In order to get the resonance state of the structure, a continuous sound with a beat frequency, which is the same as the natural frequency of the structure, has been applied to the structure.

The instruments and materials used during the test are;

1. A sound source with a powerful subwoofer,
2. A metal ruler with a length of 1m.,
3. A square metal plate with a 10cm edge dimension and 3 mm thickness,
4. A pincer to fasten the metal sheet to the ruler and two clamp fasten the ruler to the test floor,
5. A computer with a program called “Goldwave” in order to produce sound with various beat frequencies.
6. A stopwatch

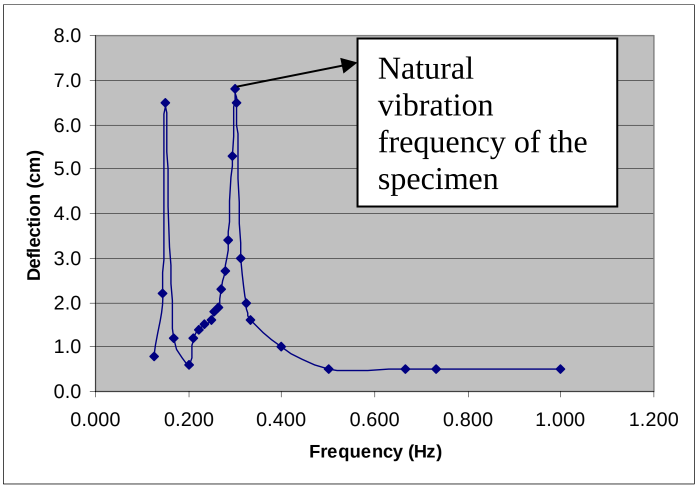
First the ruler has been attached to the test floor by the clamps. The plate has been attached at the top of the ruler by a pincer. Then a single force has been applied to the plate in order to make the ruler swing. By means of a stopwatch, the cycle times have been counted for both 30 seconds. The ruler has swung 9 times in 30 seconds. So, the natural frequency has been calculated as 9 cycles / 30 seconds = 0.3 Hz. The same test repeated for a reading time of 60 seconds instead of 30 seconds. The cycle quantity was 18 in 60 seconds and the natural frequency was 18 cycles / 60 seconds = 0.3 Hz (Brater, 1959). The subwoofer has been placed in front of the ruler and at the elevation level of the plate (see Photo 3.1).

Photo 3.1 Subwoofer in front of the plate

With the computer program “Goldwave”, totally 25 sound with various beat frequencies has been obtained. The range of the beat frequencies was from 0.125 Hz to 1 Hz. The maximum deflection of the plate has been measured for each sound trial and noted (see Table 3.3).

The curves showing the deflection versus the applied beat frequencies are given in Graph 3.3. It is seen that the deflection of the plate is much larger at a beat frequency of 0.3 Hz which is equal to the natural frequency of the test structure. It was also observed that at a beat frequency of 0.15 Hz, the deflection of the plate is also large but not more than the deflection under the beat frequency of 0.3 Hz.

As a result, it is seen that the deflection of the top part of the test structure is very large when the beat frequency is equal to the natural frequency of the structure. Also when the beat frequency is equal to the frequency obtained by dividing the natural frequency of the structure by an integer number (e.g. 2, 3, 4 etc), the structure had large deflections.



Graph 3.1 Frequency vs. Deflection graph

Table 3.1 Frequency vs. Deflection table

Trial #	T (sec)	Freq. (1/sec)	Deflection (cm)
1	1.000	1.000	0.5
2	1.364	0.733	0.5
3	1.500	0.667	0.5
4	2.000	0.500	0.5
5	2.500	0.400	1.0
6	3.000	0.333	1.6
7	3.100	0.323	2.0
8	3.200	0.313	3.0
9	3.300	0.303	6.5
10	3.333	0.300	6.8
11	3.400	0.294	5.3
12	3.500	0.286	3.4
13	3.600	0.278	2.7
14	3.700	0.270	2.3
15	3.800	0.263	1.9
16	3.900	0.256	1.8
17	4.000	0.250	1.6
18	4.250	0.235	1.5
19	4.500	0.222	1.4
20	4.750	0.211	1.2
21	5.000	0.200	0.6
22	6.000	0.167	1.2
23	6.666	0.150	6.5
24	7.000	0.143	2.2
25	8.000	0.125	0.8

3.1.6 FOURIER ANALYSIS

Fourier analysis of a periodic function refers to the extraction of the series of sinus and cosines which when superimposed will reproduce the function. This analysis can be expressed as a Fourier series. The fast Fourier transform (FFT) is a mathematical method for transforming a function of time into a function of frequency. It can be described as transforming from the time domain to the frequency domain. A sample periodic waveform and its FFT is given in Figures 3.2 and 3.3, respectively.

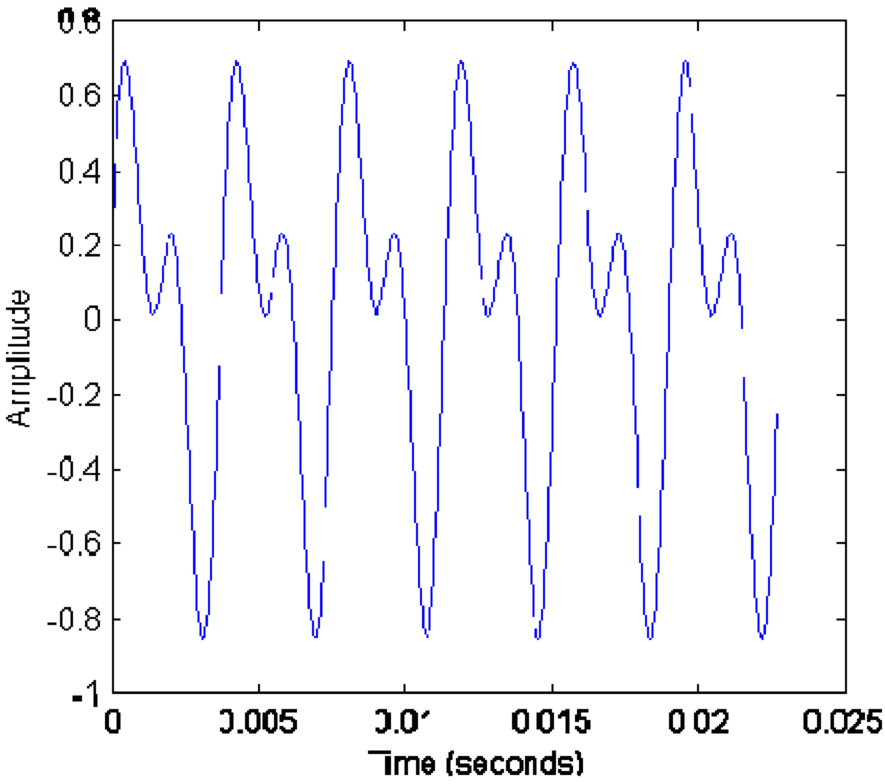


Figure 3.2 A periodic waveform

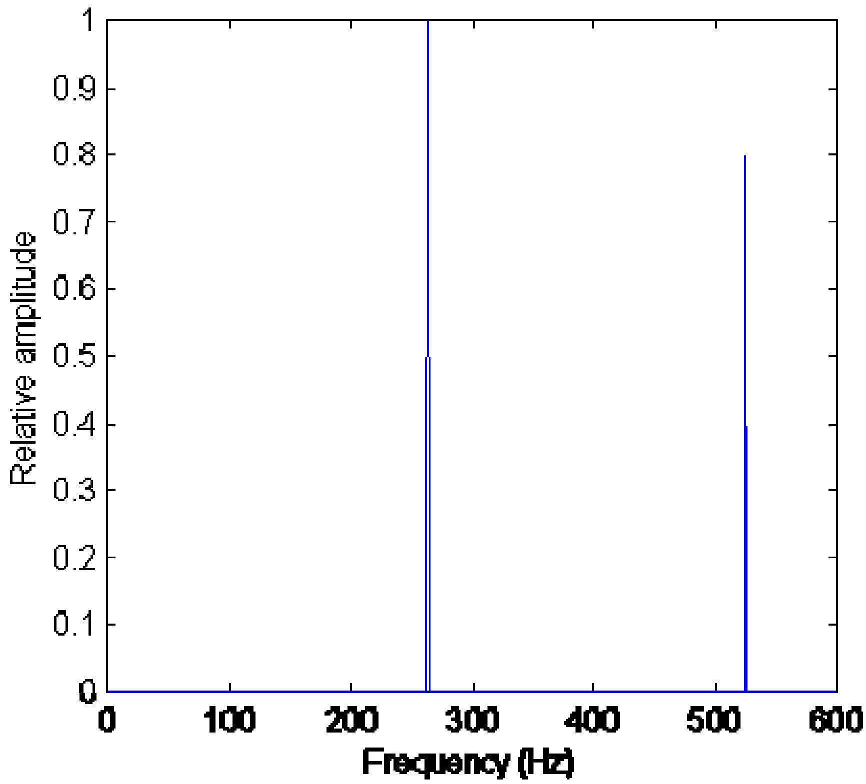


Figure 3.3 FFT example

As seen on Figure 3.6, there are two sharp and extremely narrow peaks at around 260 Hz and 520 Hz. The lower frequency peak rises to a height of 1 whereas the lower frequency rises to 0.8 on the vertical axis. These numbers as the relative amplitudes of the two sine waves that have been added together to create the original complex tone (DeLong, 1989).

3.1.7 SOUND POWER LEVEL AND DECIBEL

The decibel (dB) is a logarithmic unit used to describe a ratio of power, voltage, intensity. The most physically understandable quantity used in determining the size of a sound signal is the Pressure Amplitude. Pressure amplitude is a measure of the size of the variation in air pressure caused by a sound wave. The decibel scale for pressure amplitude is called Sound

Pressure Level, typically abbreviated SPL. The decibel SPL value for a sound with pressure amplitude P is given by the relation

$$\text{Sound Pressure Level (SPL)} = 20 \log (P/Po) \qquad \text{(Eq. 3.1)}$$

where "P" is the root mean square pressure (N/m2). The usual reference level "Po" is defined as 2e-5 Newton per square meter, the threshold of hearing. The pressure amplitude for the weakest sound that is audible to the average person is referred to as the threshold of hearing.

To get a feel for decibels, a table below gives values for the sound pressure levels of common sounds in our environment. Also, the corresponding sound pressures and sound intensities are shown on the Table 3.2 (http://www.sengpielaudio.com/TableOfSoundPressureLevels.htm).

Table 3.2 Sound levels and corresponding sound pressure

Examples	Sound Pressure Level dBSPL	Sound Pressure p N/m^2 = Pa
Jet aircraft, 50 m away	140	200
Threshold of pain	130	63.2
Threshold of discomfort	120	20
Chainsaw 1m distance	110	6.3
Disco, 1 m from speaker	100	2
Diesel truck, 10 m away	90	0.63
Kerbside of busy road, 5 m	80	0.2
Vacuum cleaner, distance 1 m	70	0.063
Conversational speec, 1m	60	0.02
Average home	50	0.0063
Quiet library	40	0.002
Quiet bedroom at night	30	0.00063
Background in TV studio	20	0.0002
Rustling leaf	10	0.000063
Threshold of hearing	0	0.00002

As an example, the pressure amplitude corresponding to a decibel reading of 35 dB is calculated as follows;

From eq. 3.1;
35dB = 20 log (P/2e-5) which yields P = 1.12e-3 N/m2

The decibel quantities can also be combined. The combination of the decibel levels are calculated seperately for incoherent and coherent sources.

Incoherent Sources

Some sources have different frequencies and random phase relation. These are called incoherent sources. Total energy from two incoherent sources equals the sum of the energy from each. Since the total intensity is the sum of the intensity from each individual source, we can calculate the total pressure:

$$P_T^{\ 2} = \sum_{i=1}^{n} P_1^{\ 2} + \sum P_1^{\ 2} + P_2^{\ 2} + ...P_n^{\ 2} \qquad \text{(Eq. 3.2)}$$

and in dB;

$$10\log_{10}(\frac{P_T}{P_{ref}})^2 = 10\log_{10}\sum_{i=1}^{n}(\frac{P_i}{P_{ref}})^2 = 10\log_{10}\sum_{i=1}^{n}10^{L_{pi}/10} \qquad \text{(Eq. 3.3)}$$

For example, the combined sound pressure level due to two incoherent sources of 90 and 88 dB is calculated as 92.1 dB.

Coherent Sources

If sources are coherent (exactly the same frequency), phase must be considered. The total combined pressure is:

$$P_T^{\ 2} = P_1^{\ 2} + P_2^{\ 2} + 2P_1P_2\cos(\beta_1 - \beta_2) \qquad \text{(Eq. 3.4)}$$

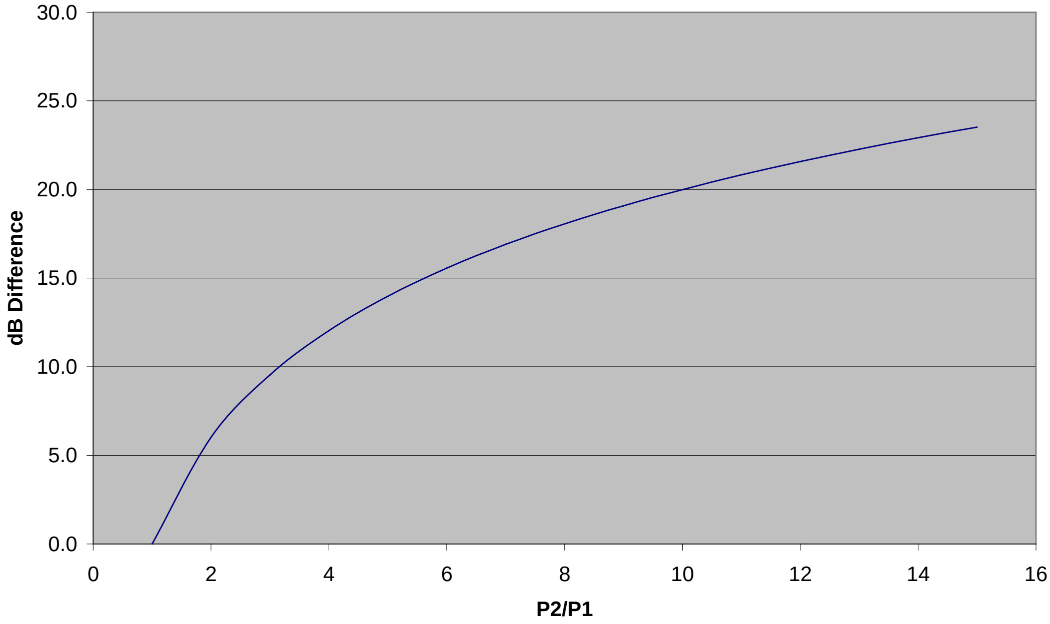
$$10\log_{10}(\frac{P_T}{P_{ref}})^2 = 10\log_{10}\sum_{i=1}^{n}(\frac{P_i}{P_{ref}})^2 \qquad \text{(Eq. 3.5)}$$

Addition of two coherent sources (totally in phase) adds 6 dB to the level of either alone.

A table and a corresponding graph showing the relation between P2/P1 ratio and dB is shown below (see Table 3.3 and Graph 3.2).

Table 3.3 Combining Decibel Quantities

P2/P1	dB
1	0.0
2	6.0
3	9.5
4	12.0
5	14.0
6	15.6
7	16.9
8	18.1
9	19.1
10	20.0
11	20.8
12	21.6
13	22.3
14	22.9
15	23.5



Graph 3.2 Combining Decibel Quantities

CHAPTER 4

FINITE ELEMENT MODEL OF ASPENDOS THEATRE

4.1 PURPOSES OF MODELING OF ASPENDOS THEATRE

The purposes of modeling of Aspendos theatre can be listed as follows;

- Obtaining natural frequencies**:** In order to get the natural frequencies of different parts of the theatre, modeling was necessary. The natural frequencies of these parts have been compared with the beat frequencies of sound to which the theatre is exposed to check for resonance.
- Design check under earthquake loading: In order to analyze the theatre under earthquake loading, it was necessary to model the theatre.

4.2 PROGRAM USED TO MODEL THE THEATRE

SAP2000 (version 8.15), an integrated software for structural analysis and design, have been used for modeling of the theatre.

With this fully integrated program, one can make model creation, modification, execution of analysis, and results review from within a single interface. Powerful graphical 3D model generation using plan, elevation and developed views give the user easily and effectively work with the program. The program has the analysis options of static, dynamic, linear and non-linear to fulfill the user requirements.

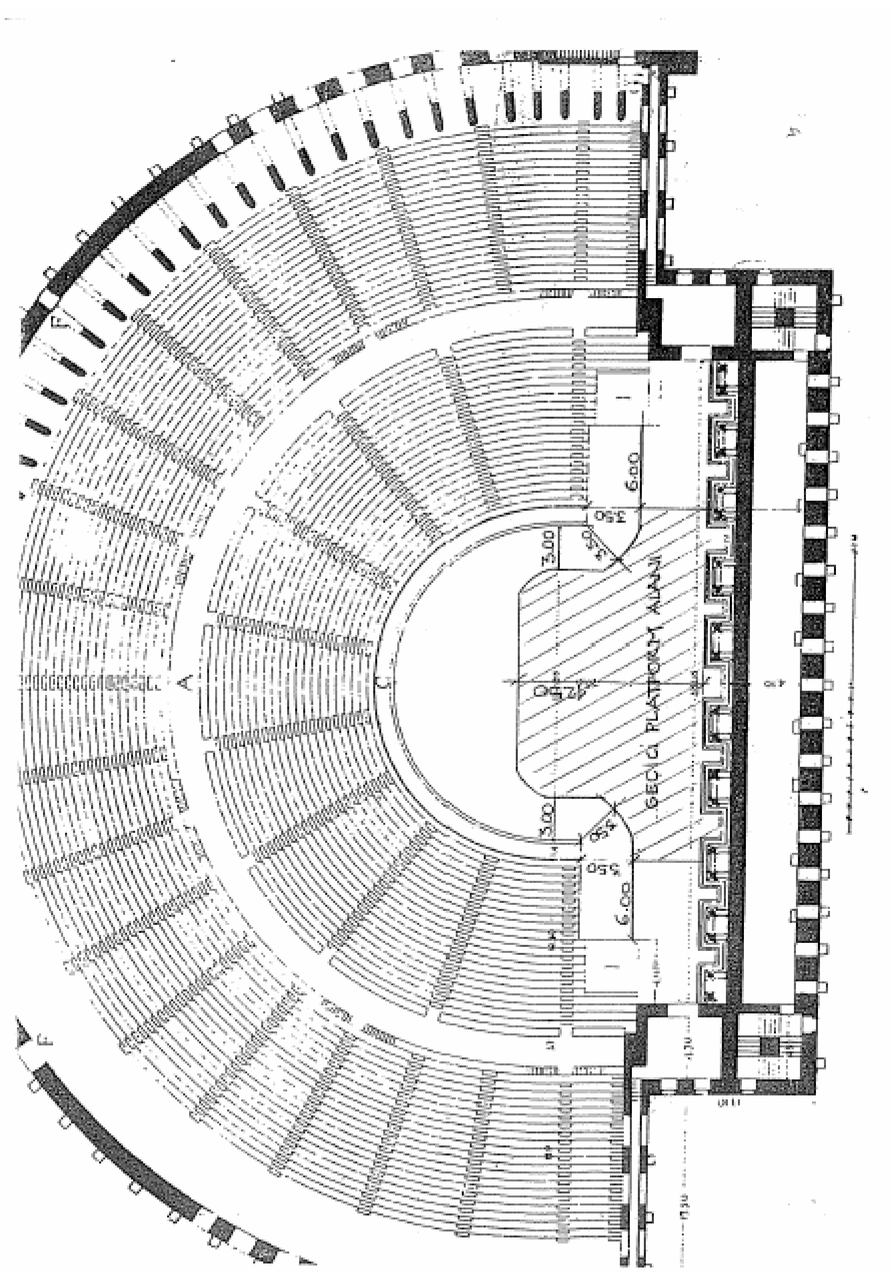
Onscreen results display such as animated display of deformed shapes, mode shapes, stress contours and time history results, user customizable tables that can be displayed onscreen or output in multiple formats, interactive custom report generator makes the user handle the modeling, design easily and effectively.

Detailed information can be obtained from the internet address of *http://www.csiberkeley.com/products_SAP.html*.

4.3 GEOMETRIC DIMENSIONS OF ASPENDOS THEATRE

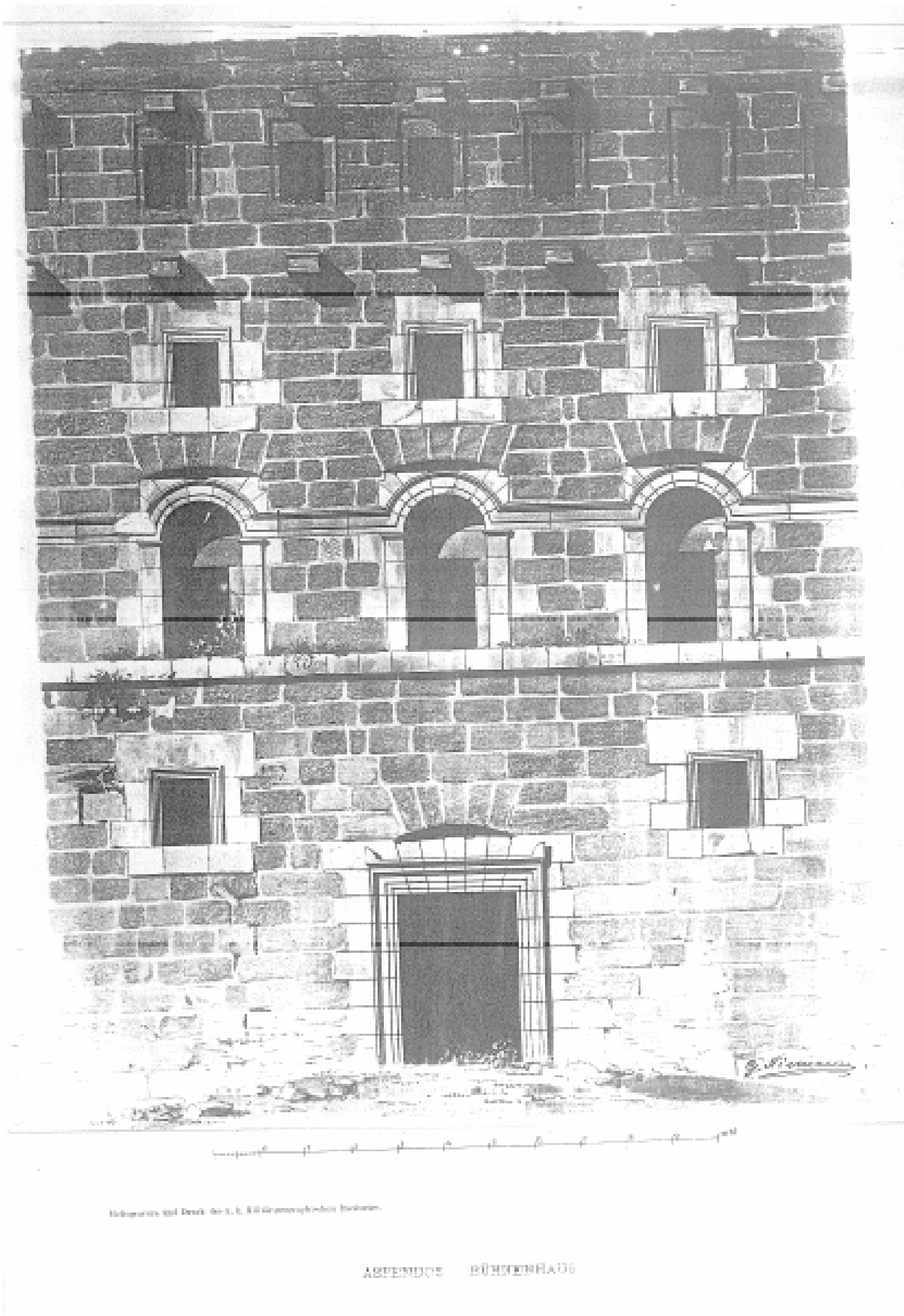
All the geometric dimensions have been obtained during the first and second visit to Antalya at the dates of 24/09/2005 and 09/04/2005, respectively.

During the first visit, the general dimensions of the theatre have been obtained from the "Museum Directory of Antalya". A bird's eye view of the theatre which is used to organize the settlement of audience and the place of scene has been obtained directly from Museum Directory of Antalya (see Figure 4.1).

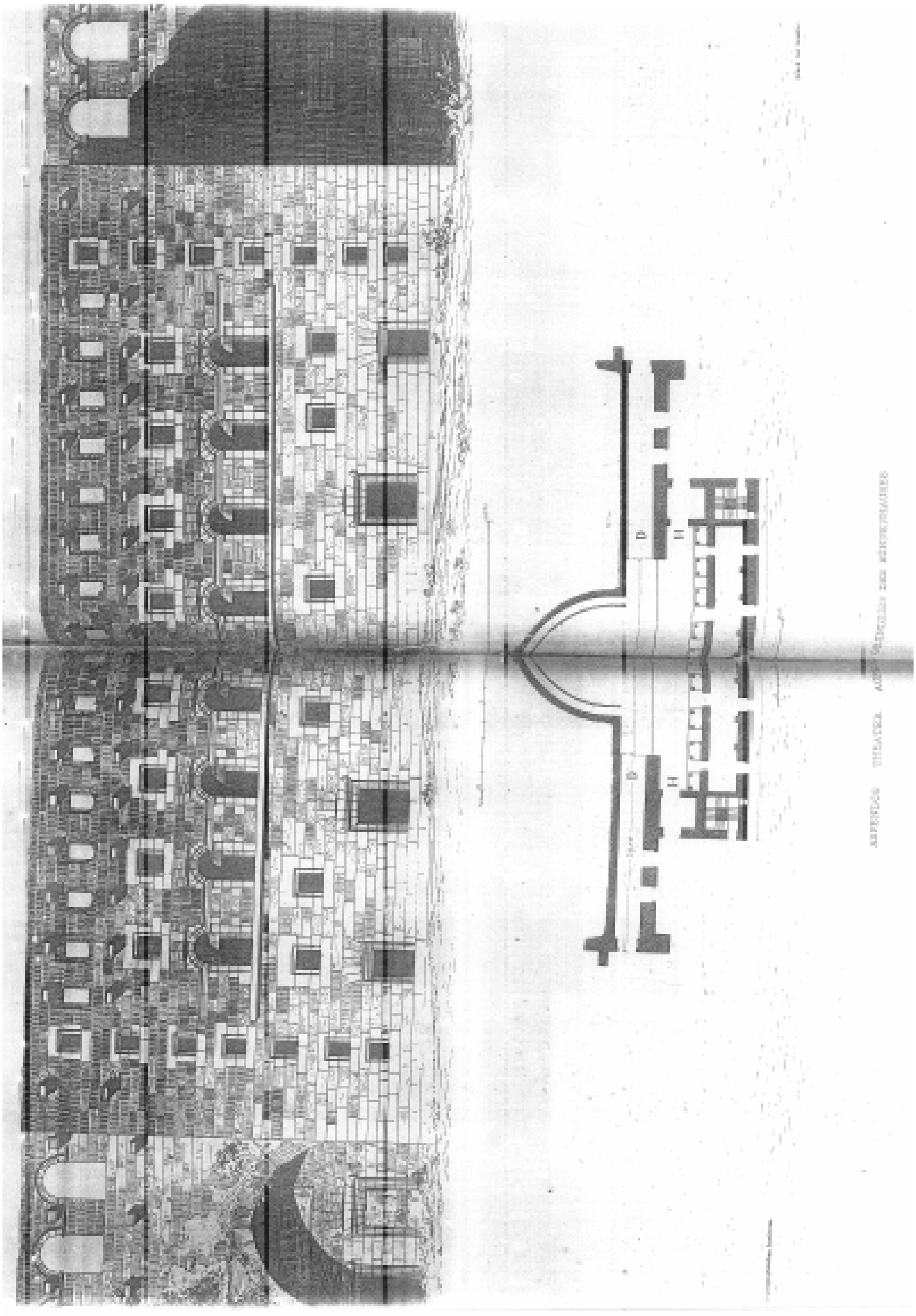


Picture 4.1 Plan view of the theatre (from Museum directory of Antalya)

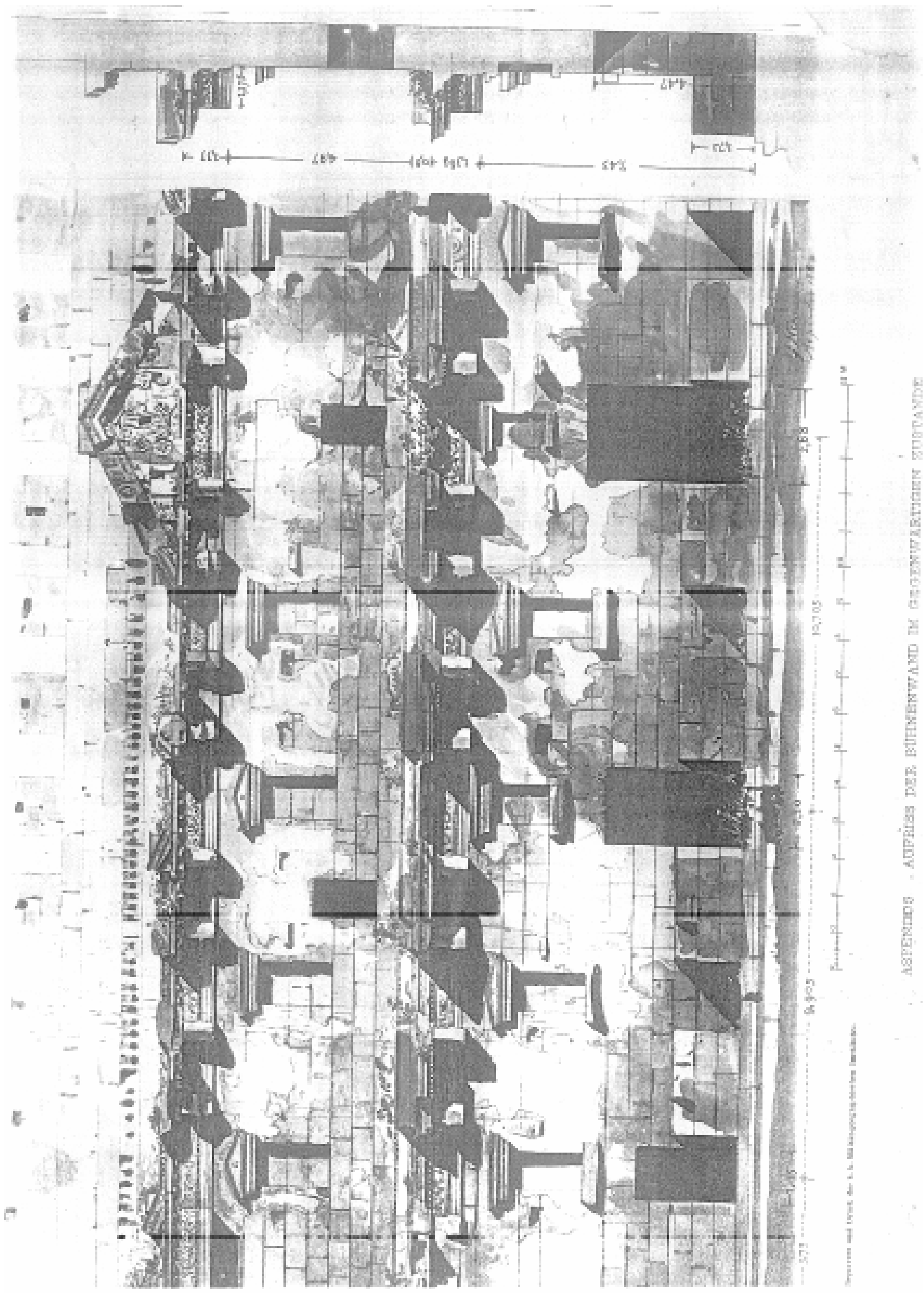
The general dimension of the outside and inside walls of the theatre has been obtained from a historical German book which was present only in the archives of Museum Directory of Antalya (Stadte Pamphyliens und PISIDIENS, 1 Band K.G. Lanckoronski, Karl Grafen Lanckoronski, G. Niemann und E. Petersen, Pamphylien, 1890). The minor dimensions on the walls such as the dimensions of the windows, thickness of the small parts have been obtained from the scaled pictures of this book (see Pictures 4.2, 4.3, 4.4).



Picture 4.2 Stage wall of the theatre (Stadte, 1890)



Picture 4.3 Front view of the theatre (Stadte, 1890)



Picture 4.4 Stage wall of the theatre (Stadte, 1890)

However, the geometrical data obtained from the Museum Directory of Antalya City was not sufficient to model the entire structure. Therefore, additional measurements have been made in the Aspendos theatre in the first visit. Also, in the second visit to Antalya, the missed dimensions from the first visit have been obtained in order to have a complete and correct modeling of Aspendos theatre.

According to the information obtained from the visits to Antalya Museum Directory and the measurements taken in Aspendos theatre, the detail drawing have been prepared by a drawing software "AutoCAD". The detail drawings of each part of Aspendos theatre have been prepared on the scaled drawings sheets to be used for the modeling of the structure.

- General View**:** A bird's eye view showing the general dimensions of the theatre has been prepared. The figure and the relevant photograph of the theatre are given below.

Photo 4.1 General view of the theatre

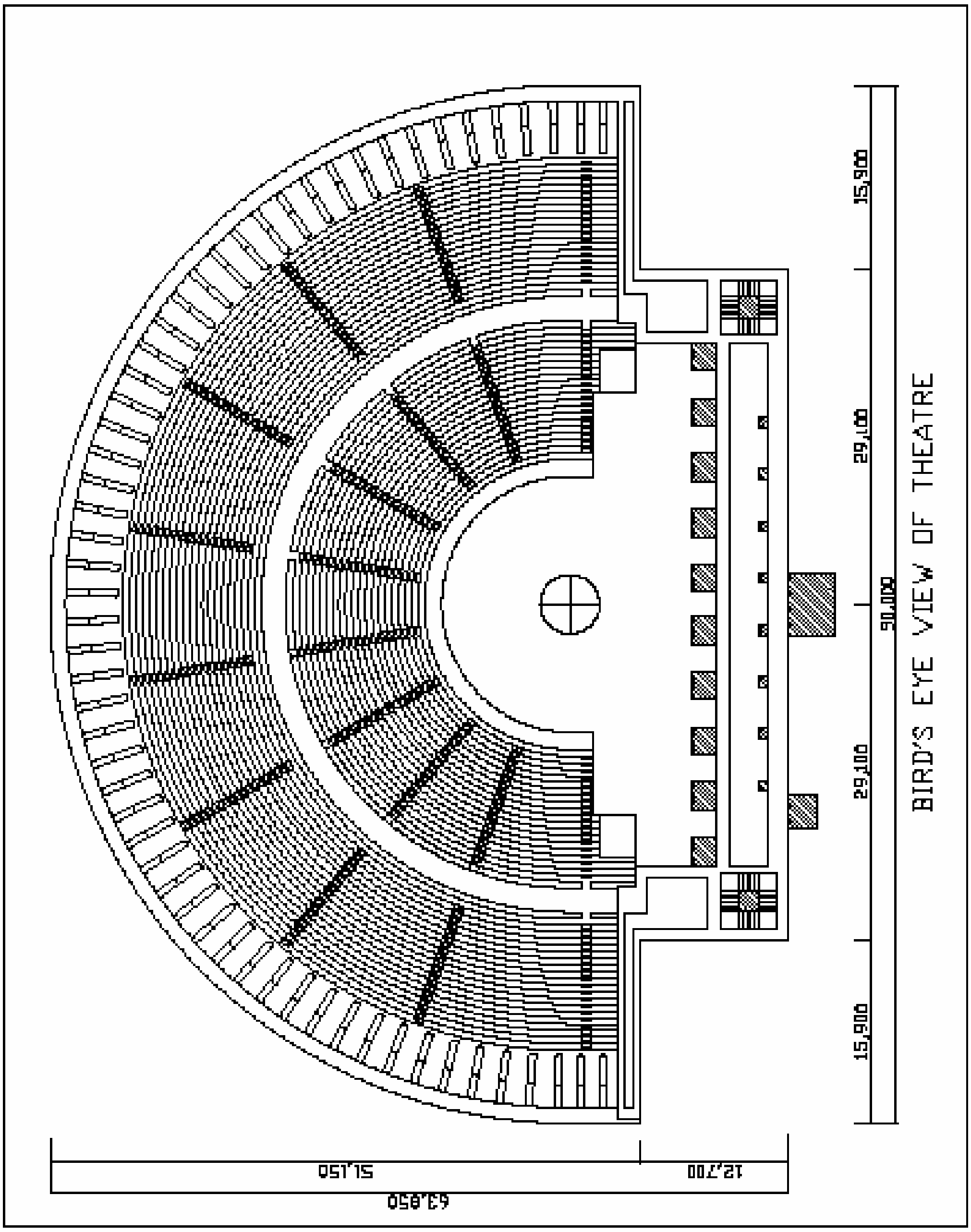


Figure 4.1 Plan view of the theatre

- Exterior wall: A scaled detail drawing for the exterior wall has been prepared and shown in Figure 4.2 below. The dimensions of all the details have been obtained from the scaled photographs

(Lanckoronski, 1890) and the measurement taken in the theatre. An appropriate photograph is also shown below (see Photo 4.2).

Photo 4.2 Exterior wall of the theatre

- Views of the side walls: Three detail drawings have been prepared for the side walls (see Figures 4.3, 4.4 & 4.5). These figures show the extended drawings of the side wall for better understanding of the views. A relevant photograph can also be found below (see Photo 4.3).
- Stage wall: The detail drawing of the stage wall has been prepared acc. to the scaled photographs (Lanckoronski, 1890) and the measurements taken in Aspendos theatre (see Figure 4.6, Photo 4.4).

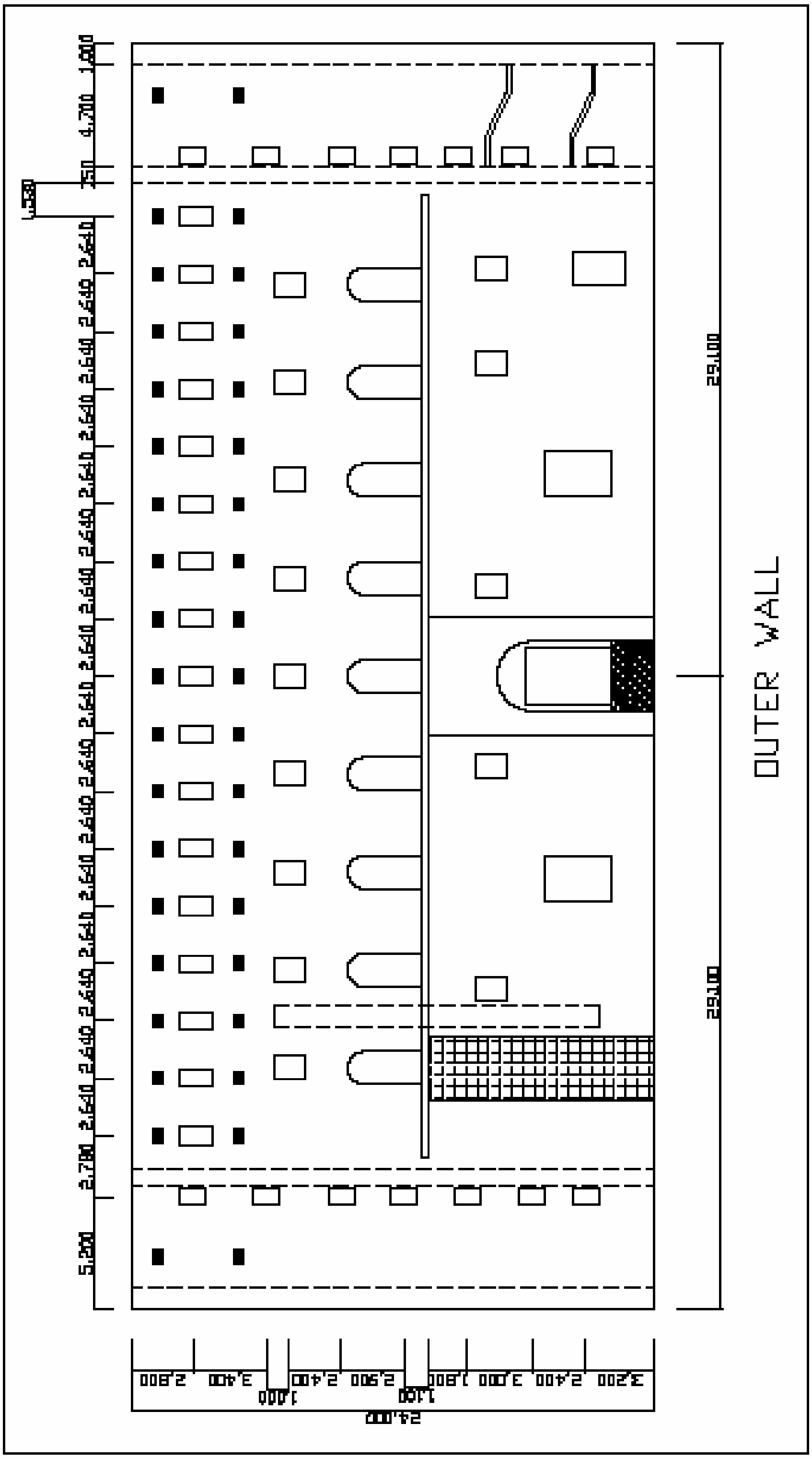


Figure 4.2 Exterior wall

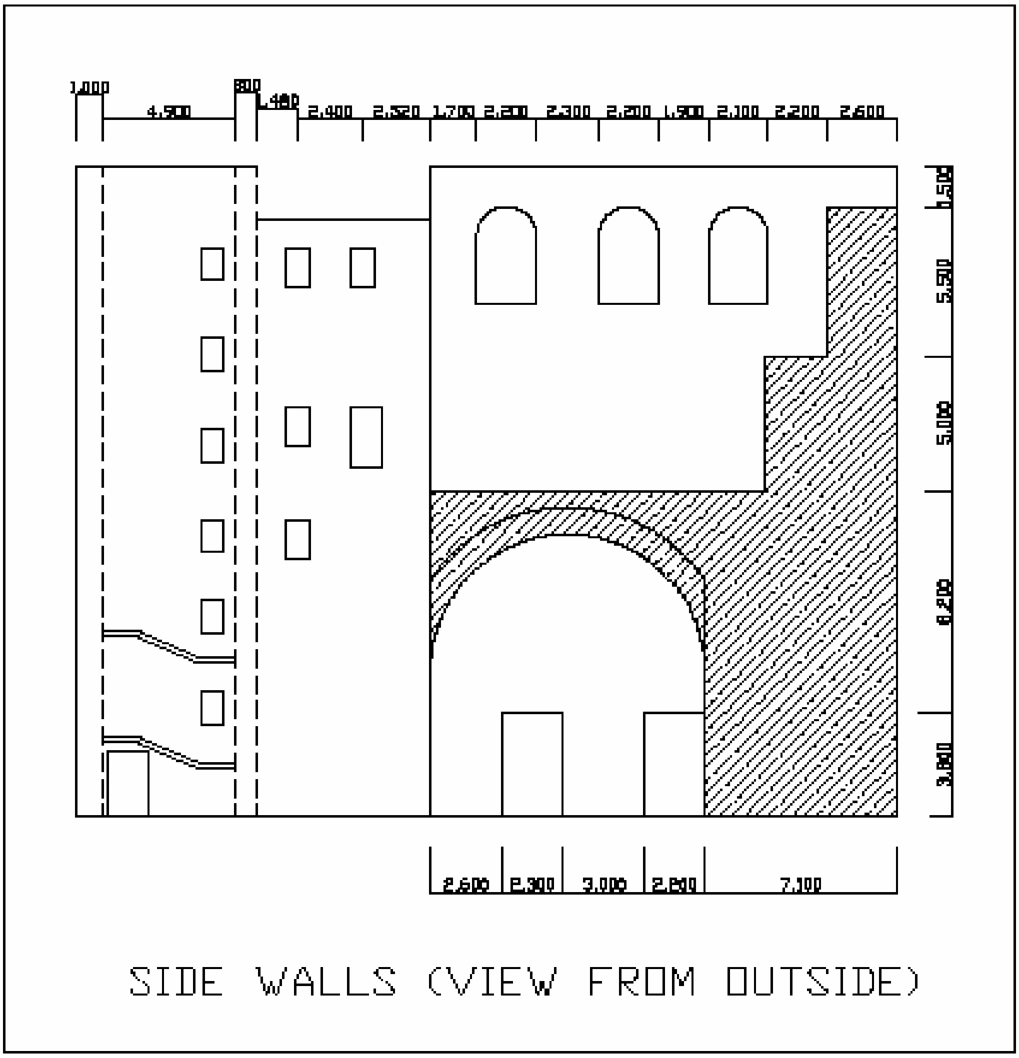


Figure 4.3 Side walls (extended view-1)

Photo 4.3 Side walls

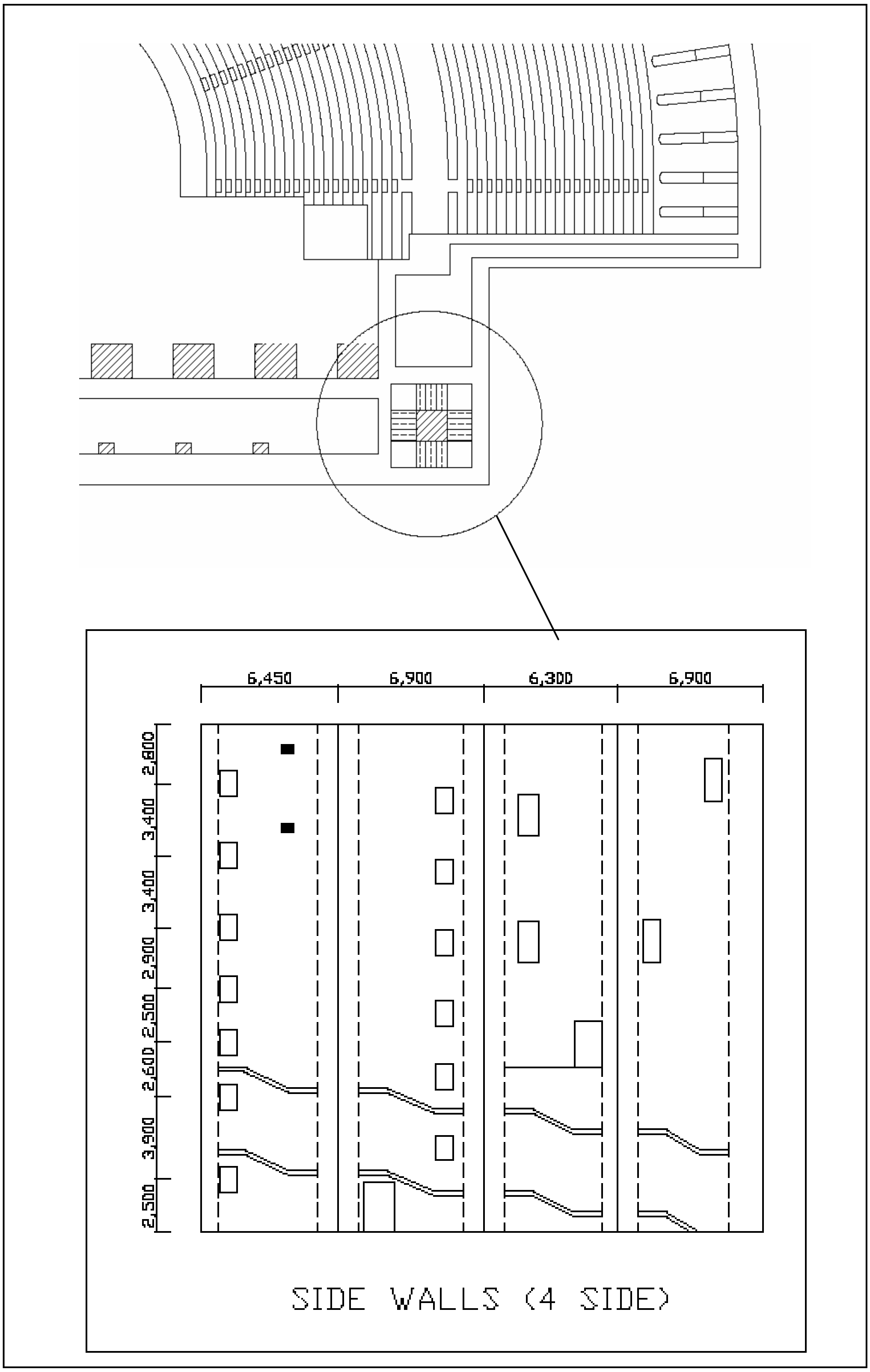


Figure 4.4 Side walls (extended view-2)

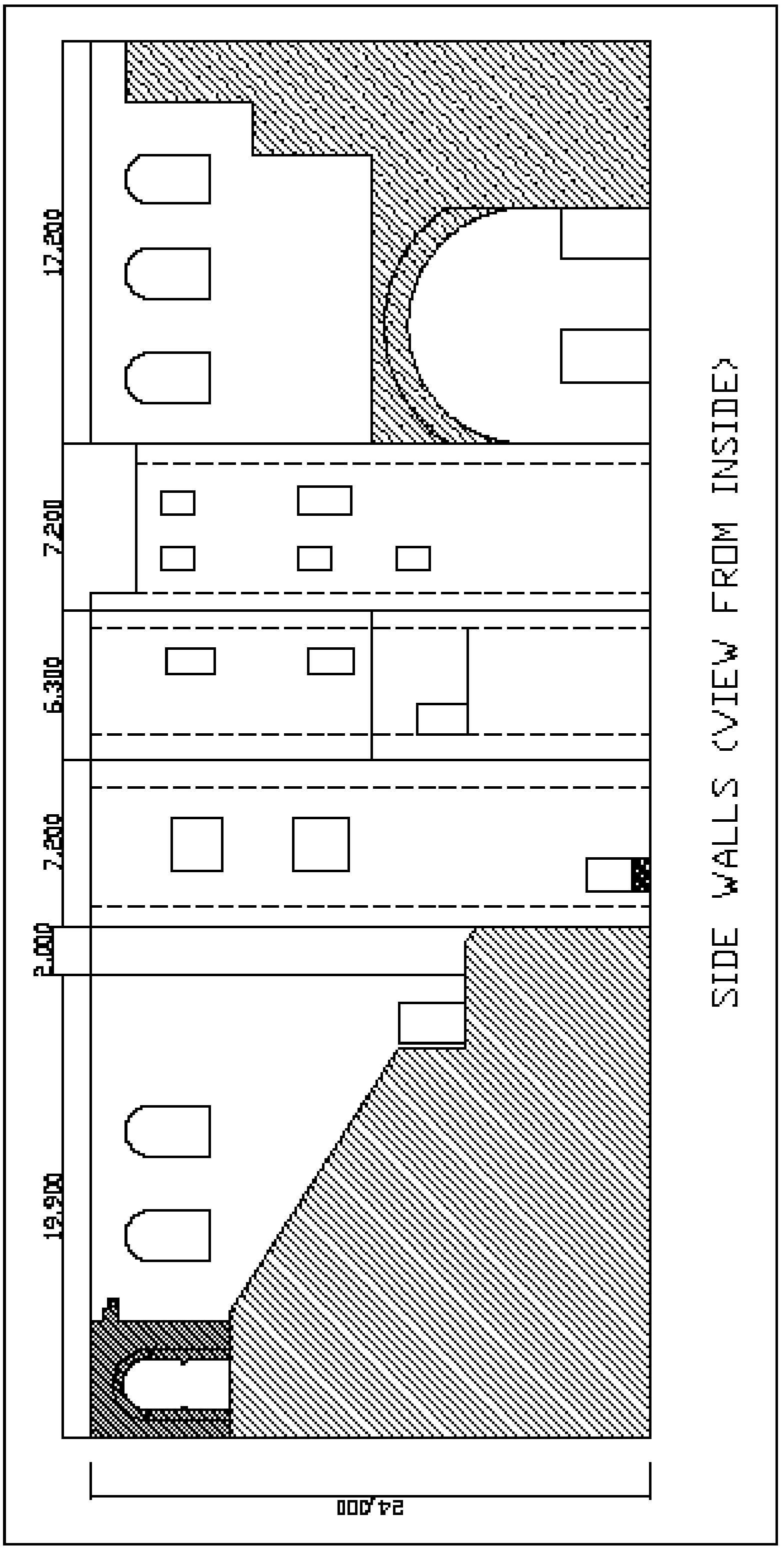


Figure 4.5 Side walls (extended view-3)

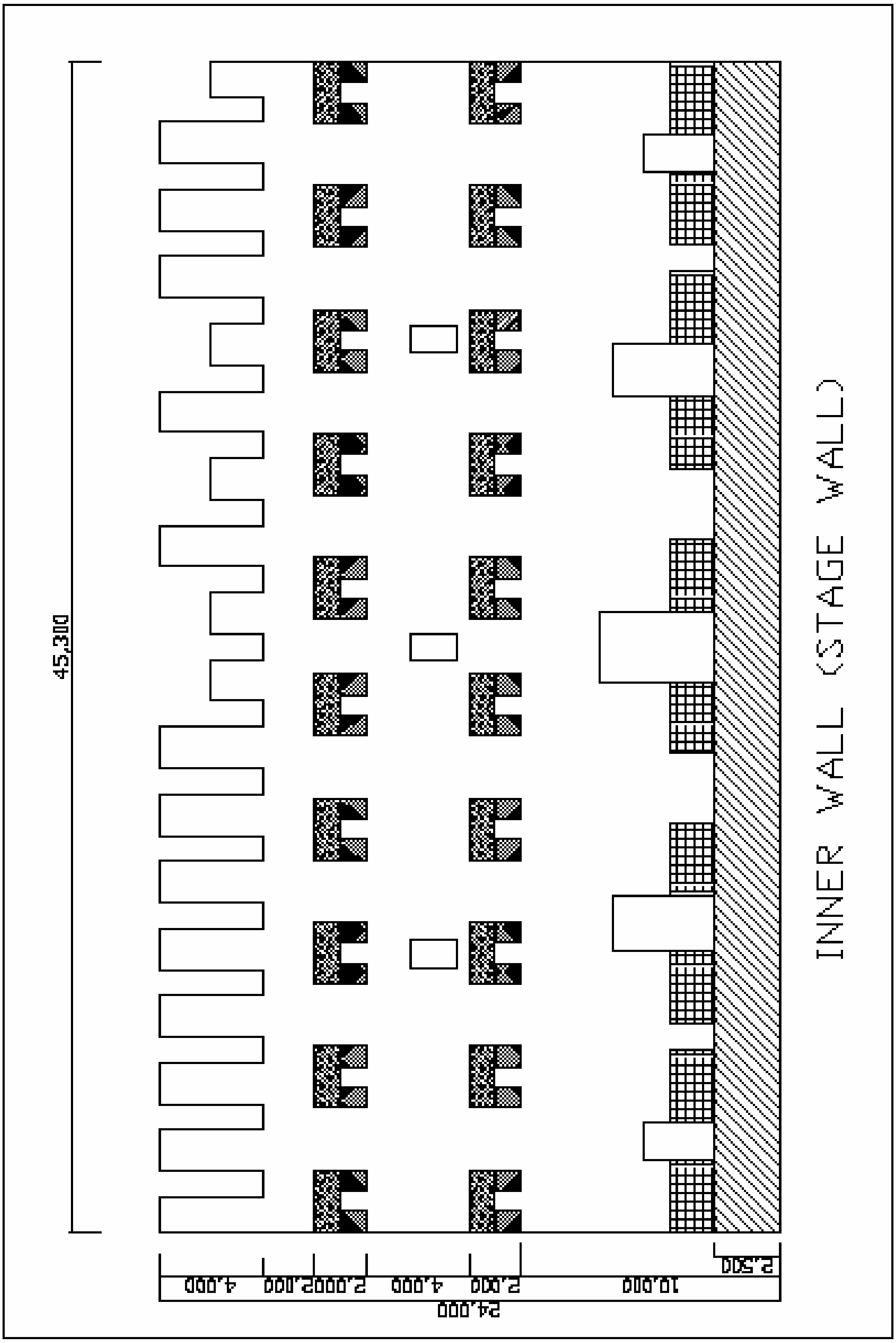


Figure 4.6 Inner (Stage) wall

Photo 4.4 Stage wall (inner wall)

- Columns: An extended detail drawing of the columns at the backside of the audience stands to be used in the model is shown in Figure 4.7 and a relevant photograph is also shown in Photo 4.5.

Photo 4.5 Columns at the backside

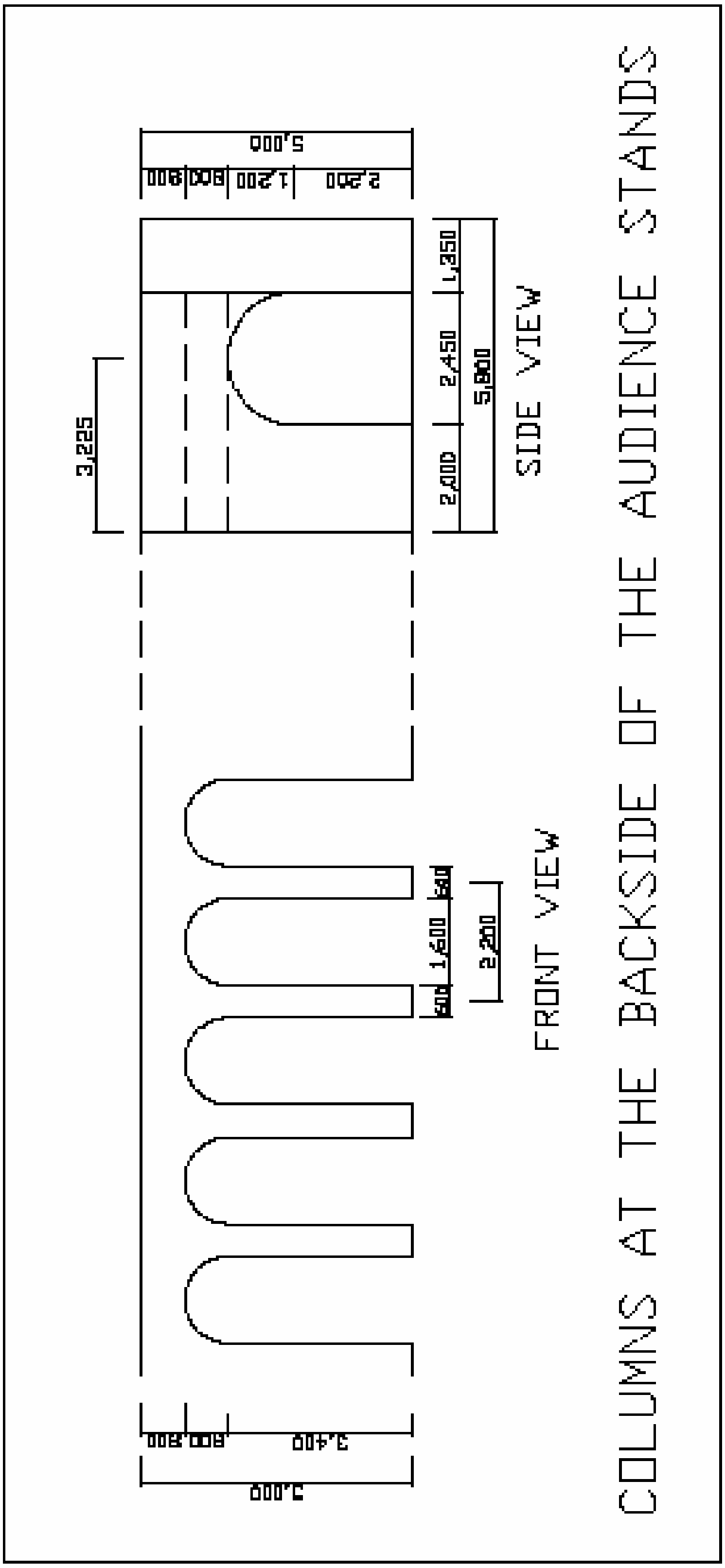


Figure 4.7 Backside peripheral columns

- Front Door: The detail drawing of the main door in front of the exterior wall has been prepared. The dimensions of this part have been obtained by the measurements taken in the theatre (see Figure 4.9 and Photo 4.6).

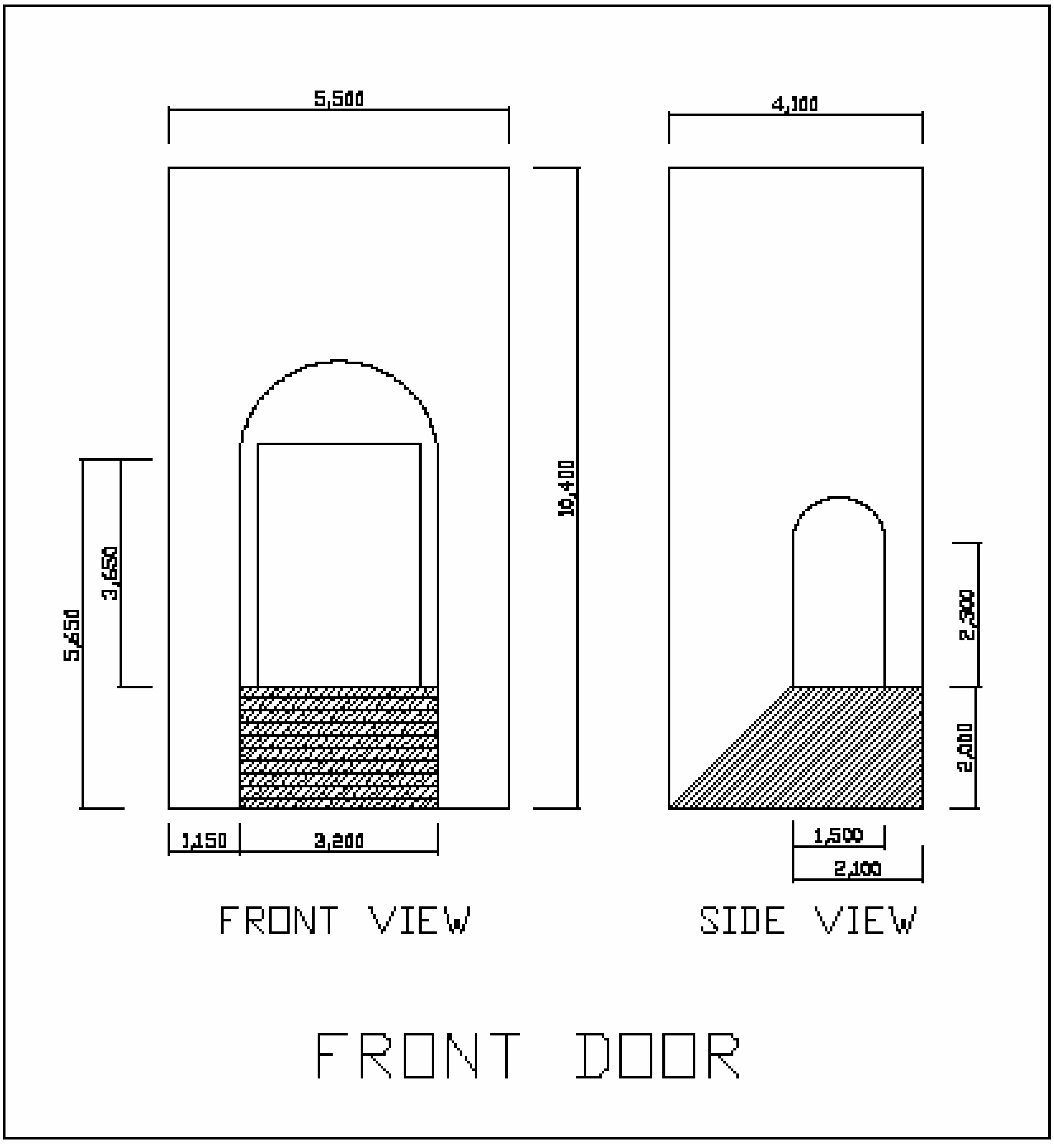


Figure 4.8 Front Door

- Stone blocks: There are big stone blocks in the theatre with various sizes. They are important for the modeling, so the detail drawings have been prepared according to the dimensions taken in the theatre. The stone blocks at the backside of the exterior wall, the stone block in front of the exterior wall, the blocks in front of the stage wall (inner wall) have been prepared (see Figures 4.10 & 4.11). The relevant photographs are also shown below (see Photos 4.).

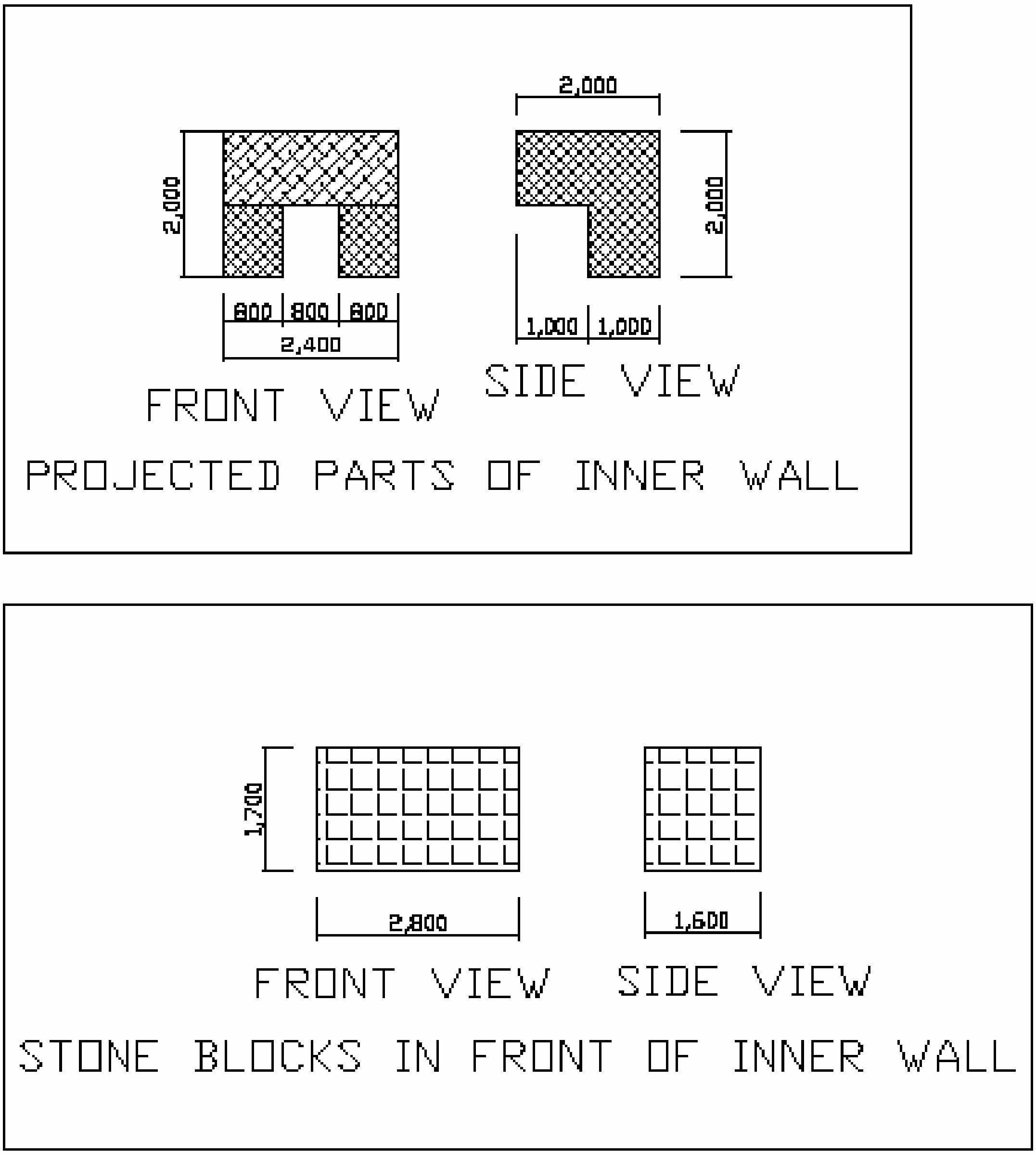


Figure 4.9 Blocks in the front of the stage wall

Photo 4.6 Blocks in front of the stage wall

Photo 4.7 Block in front of the exterior wall

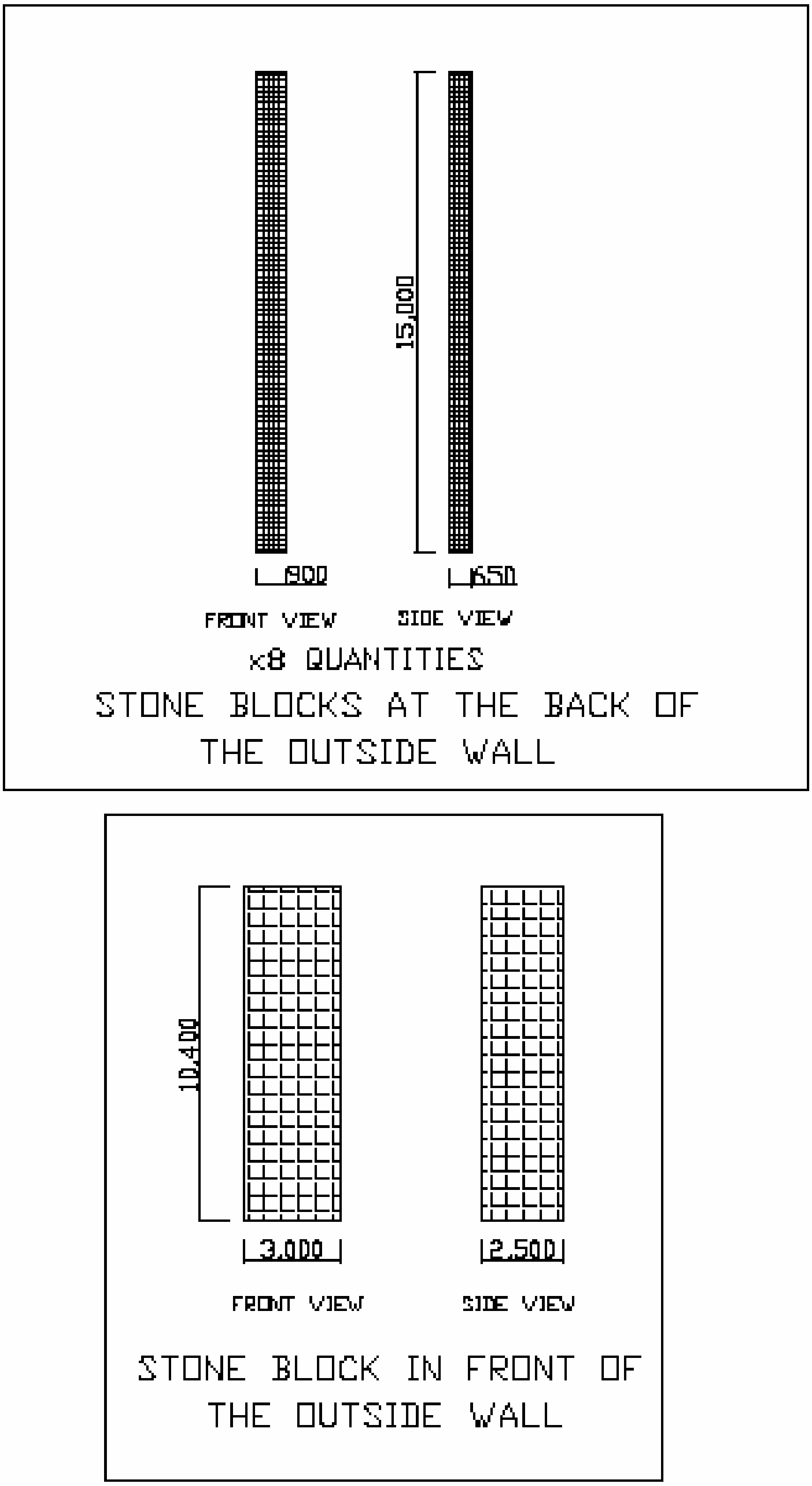


Figure 4.10 Blocks on the exterior wall

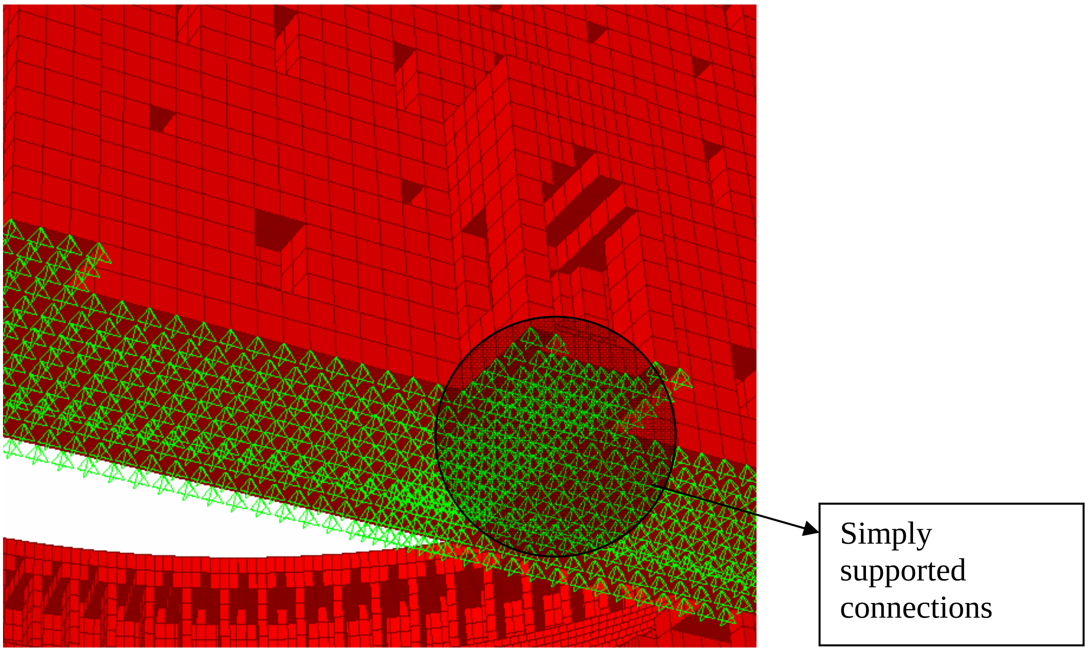
4.4 GENERATED COMPUTER MODEL OF THE THEATRE

The theatre has been modeled by using SAP2000 (ver. 8.15) program. Following applications and assumptions have been made during modeling of the structure.

- Geometric data**:** All the geometric dimensions have been taken from the references and measurements in Aspendos theatre mentioned in Section 4.3.

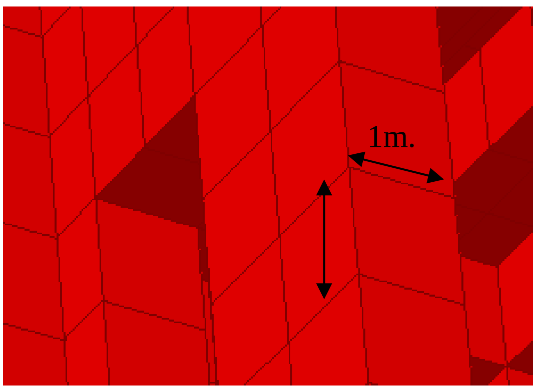
- Structure base connections: The structure has been assumed to be set on a very dense and stiff soil. The base of the theatre model has been assumed as pin-connected to the base soil. That is these joints are restrained in displacement in all directions but free to rotate in every directions. However, as there are many connection points close together at the bottom face of the fine-meshed structure model, these pin connections make the structure fixed to the ground (see Picture 4.5).

- Model elements: Solid elements have been used for the model purposes of the theatre. The solid elements have been generated by extruding the modeled areas with a depth.



Picture 4.5 Simply supported base connections

- Meshing of the model: The model formed by solid elements have been meshed (totally 25033 meshed solid elements). In order to use the speed and usage of the modeling program effectively, the solid elements have been meshed into various volumes around 1m^3 (see Picture 4.6). Each meshed solid elements have 8 points at the corners. These points are connected to the other elements by these 8 joints.



Picture 4.6 Meshed solid elements

- Material properties: The Aspendos theatre is made from mainly conglomerate material. The conglomerate is a fragmental sedimentary. It is kind of a coarse-grained rock and composed of fragments larger than 2mm in diameter cemented in a finer-grained matrix; the consolidated equivalent of gravel (see Photo 4.8), (http://mysite.wanadoo-members.co.uk/geology_revision/conglomerate.html).

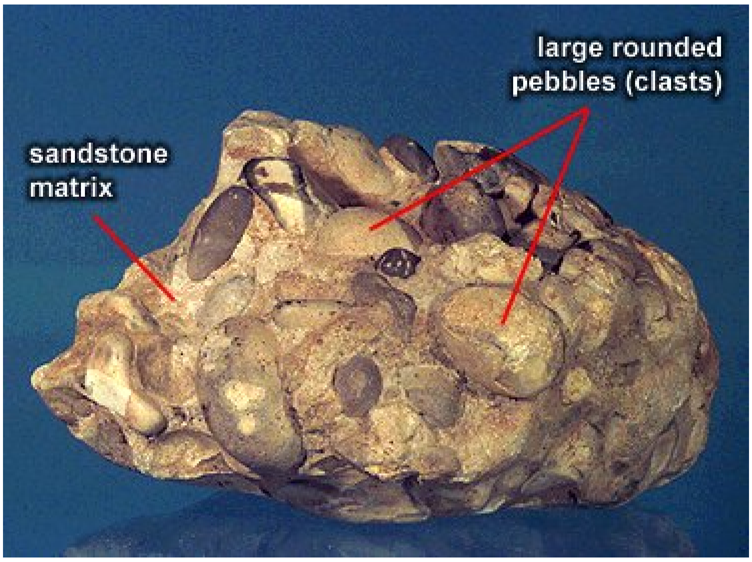


Photo 4.8 Conglomerate

The following material properties have been taken based on the measurements and previous studies, and used in the analytical model;

Unit Mass, g = 2150 kg/m^3

Unit Weight, W = 21090 N/m^3

Modulus of Elasticity, E = 2.35e+9 N/m^2

Compressive Strength, fc = 12 MPa

Tensile Strength, ft = 1.2 MPa

Poison's Ratio, v = 0.2

The unit weight of the material, by which the theatre is built, has been obtained by testing 2 samples. The weights, volumes, and calculated unit weights of the samples are shown on Table 4.1.

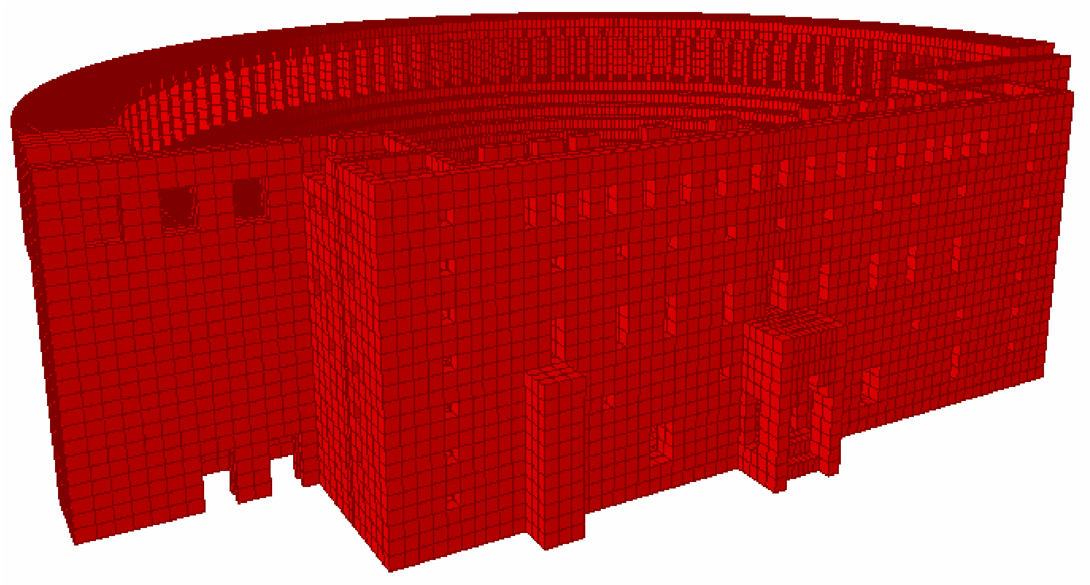
Table 4.1 unit weights of the samples

Sample #	Weight, G (kg)	Volume, V (cm^3)	Unit weight, W (kg/m^3)
1	2.285	1062	2152
2	1.580	742	2130

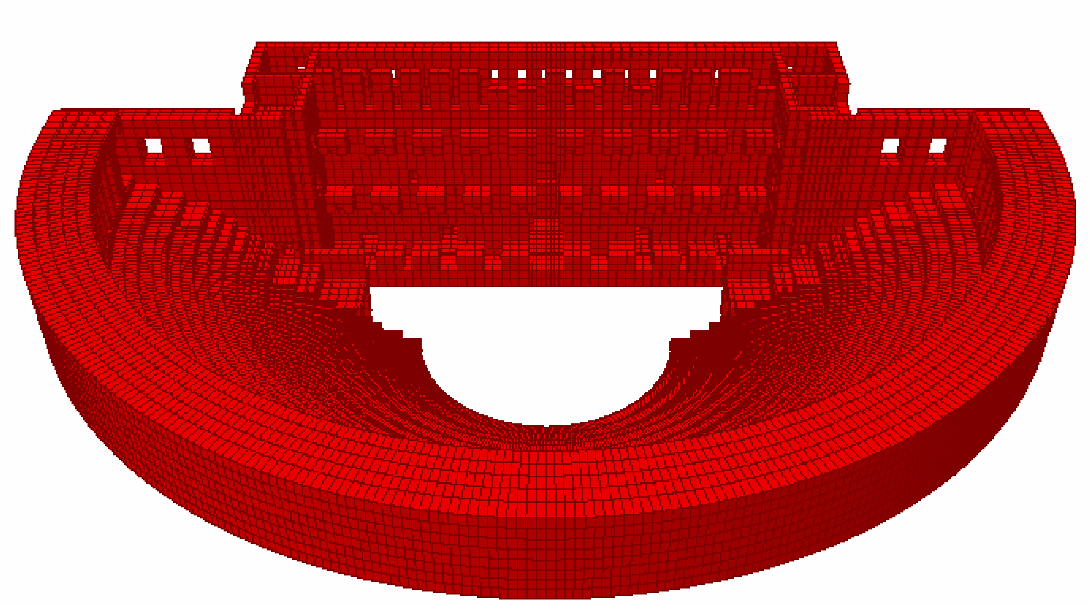
The compressive strength of the material has been obtained from a study on modulus of elasticity and compressive strengths of brick masonry mortars (Selim Sarp Tuncoku, 2001).

From this study, it is seen that, the compressive strength values are proportional to the modulus of elasticity values for stone masonry mortars. The approximate value for compressive strength of a stone masonry mortar with a 2350MPa modulus of elasticity value is found to be 12 MPa. The tension strength capacity has been assumed as 1/10 of its compression strength capacity which yields 1.2 MPa.

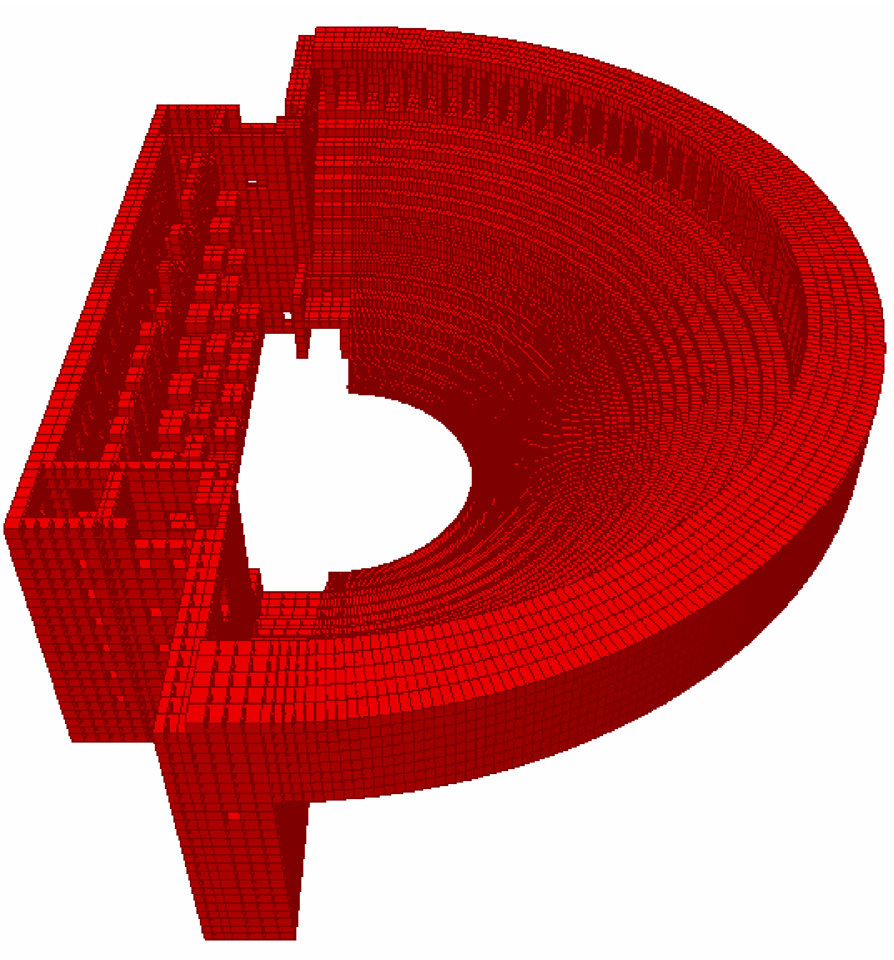
- Complete model: Several screenshots from different point of views of the model can be found below (see Pictures 4.7….4.14).



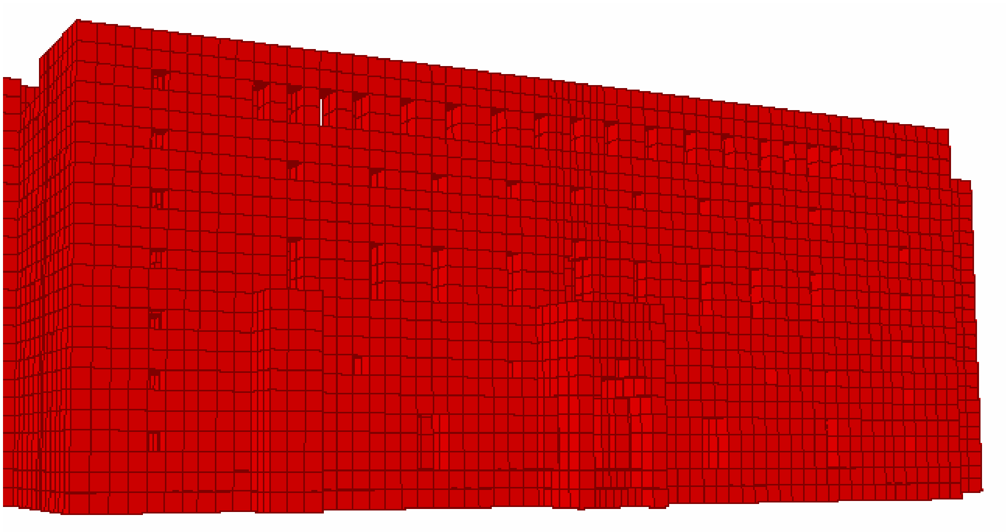
Picture 4.7 General front view



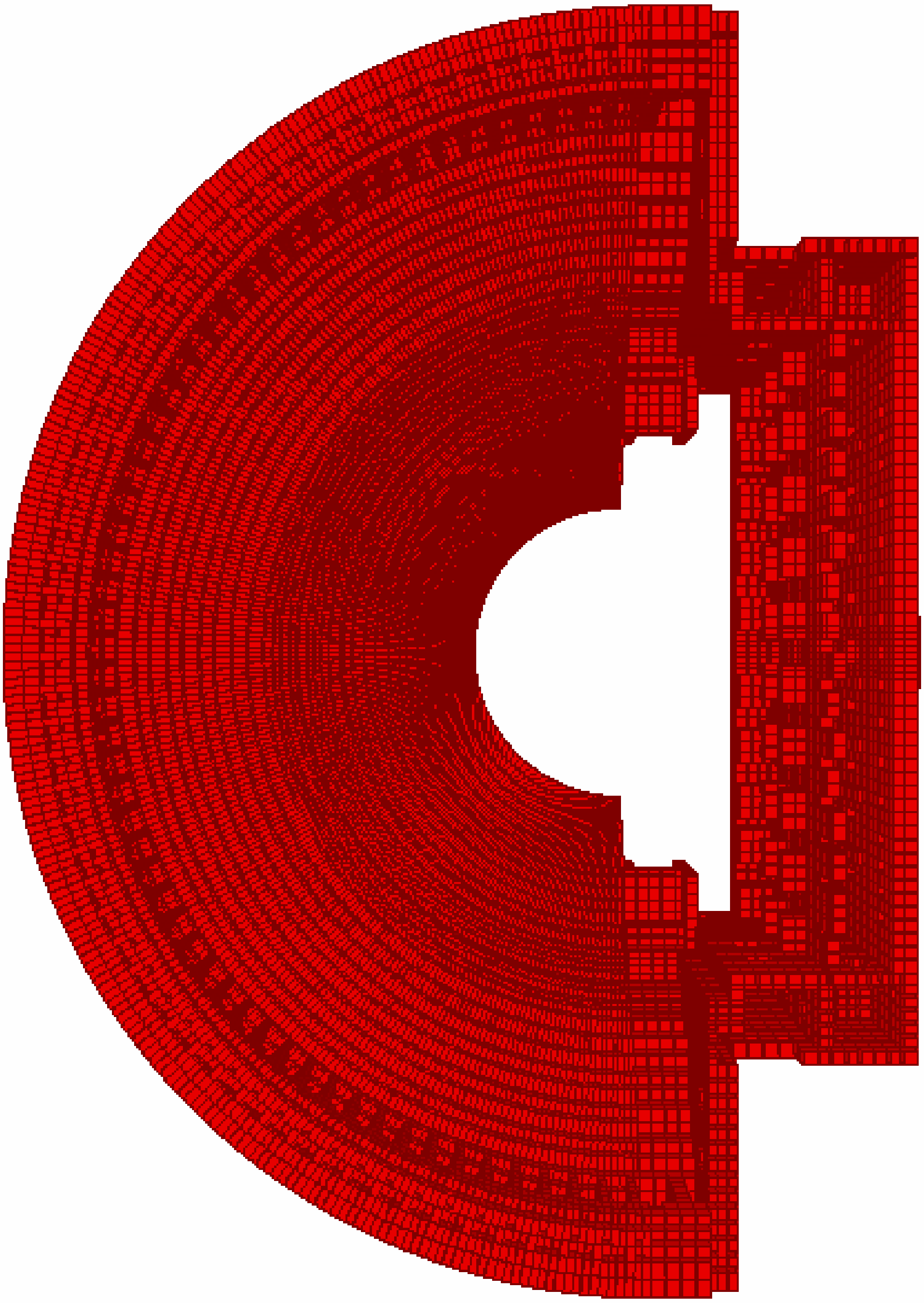
Picture 4.8 General view from behind



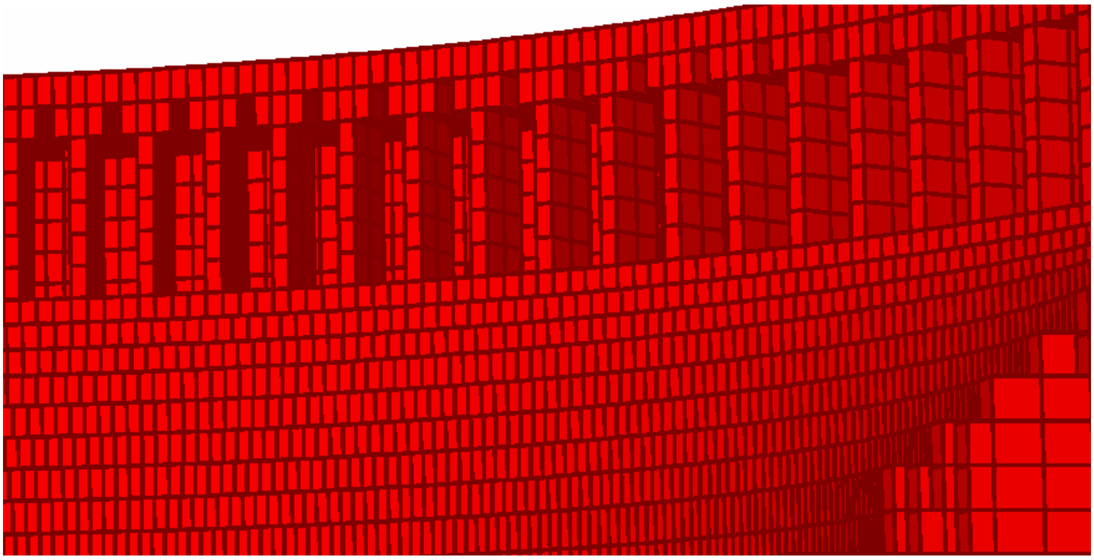
Picture 4.9 General side view



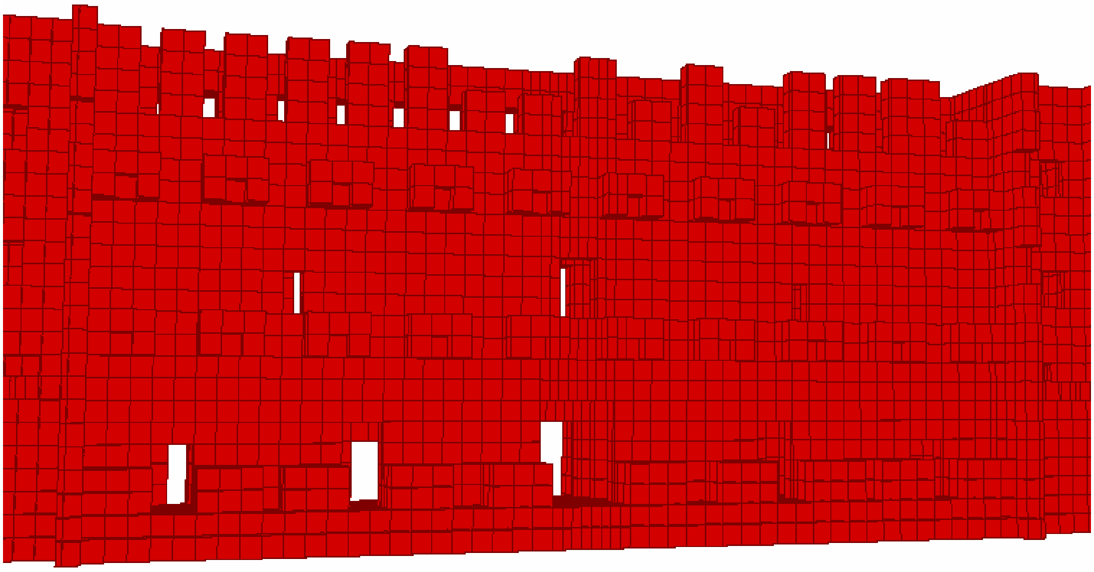
Picture 4.10 Exterior wall



Picture 4.11 Top view (plan view)



Picture 4.12 Columns



Picture 4.13 Stage wall

CHAPTER 5

DYNAMIC (IMPACT MODAL) TESTS ON ASPENDOS THEATRE

A series of natural frequency measurement has been taken from the exterior wall of the Aspendos theatre. The purpose of the tests was to investigate the natural frequencies of the theatre's exterior wall and compare the results with the ones obtained from the analytical model of the theatre.

5.1 PREPARATION AND SETUP FOR THE TESTS

The following instrumentations have been used for the purpose of natural frequency measurements;

1. A sensor (Model: PCB 393C) (accelerometer) which has been used to obtain records of acceleration over time (see Photo 5.2).
2. Data acquisition system (Guralp Systems) with built-in computer (Photo 5.4): This device has been used to obtain measurements of physical quantities using sensors.
3. A cable to connect the sensor to the data acquisition system.
4. A hammer with elastic headers (Photo 5.3).
5. A carpenter's clamp to fix the sensor to the part of the structure (Photo 5.1).

To reach the top of the exterior wall, a fire brigade has been used. The hammer, the carpenter's clamp and the sensor with the connected cable has been carried to the top of the wall by means of the basket of fire brigade (see Photo 5.5).

Photo 5.1 Sensor and the carpenter's clamp

Photo 5.2 Sensor and the carpenter's clamp

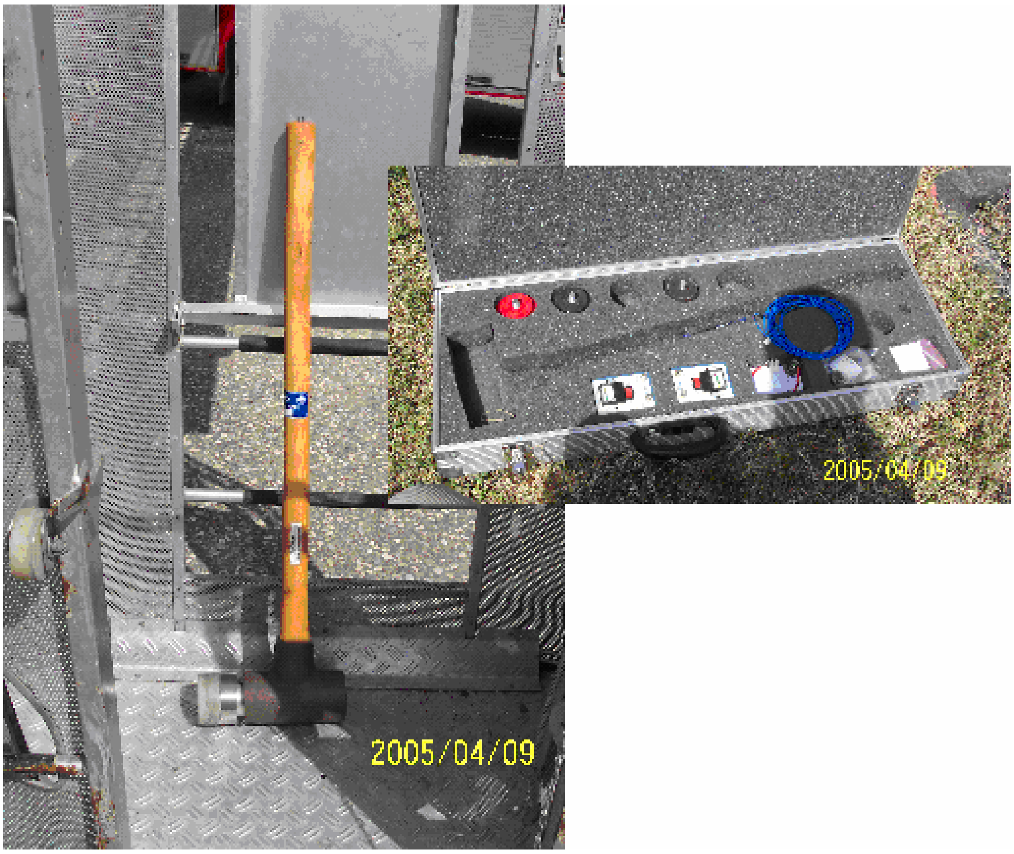


Photo 5.3 The hammer with its accessories

Photo 5.4 Data acquisition system with built-in computer

Photo 5.5 Fire brigade’s basket

2 different location on the exterior wall has been selected for the frequency measurement in order to catch the primary mode shapes;

Location 1: At a distance of 16m. from the right side of the exterior wall.

Location 2: At a distance of 29m. from the right side of the exterior wall (center of the exterior wall).

These locations for the measurements are shown on Photo 5.6.

Photo 5.6 Two locations for the measurements

5.2 IMPACT MODAL TESTS ON THE EXTERIOR WALL

The acceleration measurements have been performed on 2 locations separately on the exterior wall of the theatre. Several hits have been performed by using the hammer on the wall. Different locations from 1m. to 4m. have been selected near the sensor to hit the wall. The hits have been performed in front of the wall at the level of 1m to 2m. from top. All the hits have been performed from the basket of the fire brigade without any contact to the wall except the hammer hits. A time period (between 5sec. and 20sec.) has been left between each hits to let the wall finish its free oscillations. All the acceleration measurements have been transmitted to the data acquisition system by means of the cable. All the data has been recorded to be analyzed later. The example recordings of the acceleration (free vibration of the damped system) are shown on Figures 5.1 and 5.2.

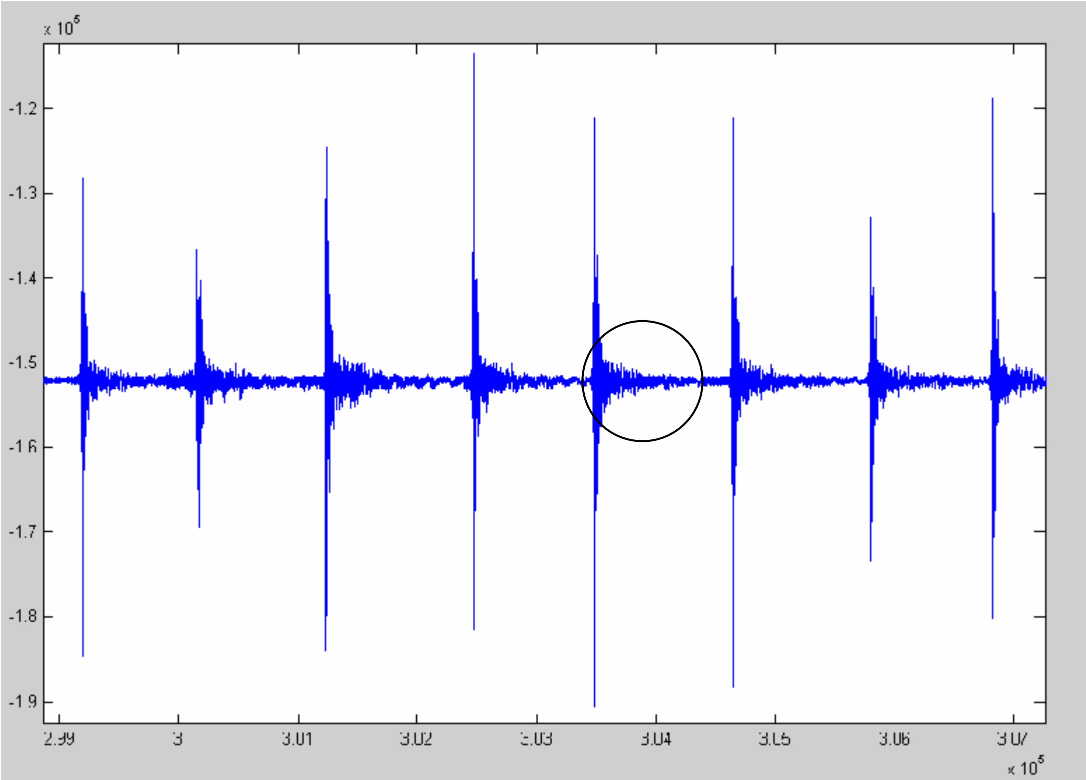


Figure 5.1 Descriptive acceleration measurements

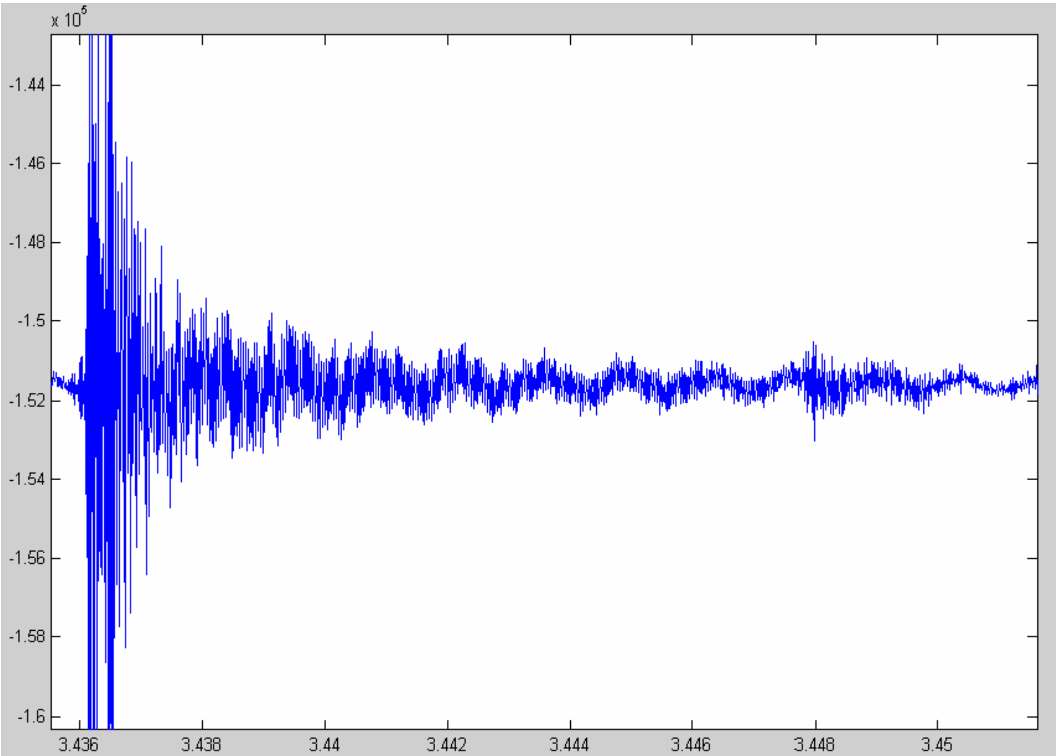


Figure 5.2 Free vibration of the exterior wall for a single impact

5.3 POST PROCESSING OF THE MEASUREMENTS

5.3.1 DETERMINATION OF DAMPING

There are two common methods used to determine the damping in structures. These methods are the logarithmic decrement method and the half power bandwidth method (CHOPRA, 1995). The logarithmic decrement technique obtains the damping properties of a structure from a free vibration test using time domain data. The half power bandwidth method uses the transfer function of the structure to determine the amount of damping for each mode.

Using free vibration data of the acceleration of the structure one may obtain the damping ratio. Figure 5.3 shows a free vibration record of a structure. The logarithmic decrement, between two peaks is defined as

$$\delta = \ln(\frac{y_1}{y_2}) \qquad \text{(Eq. 5.1)}$$

Where y1 and y2 are the amplitudes of the peaks.

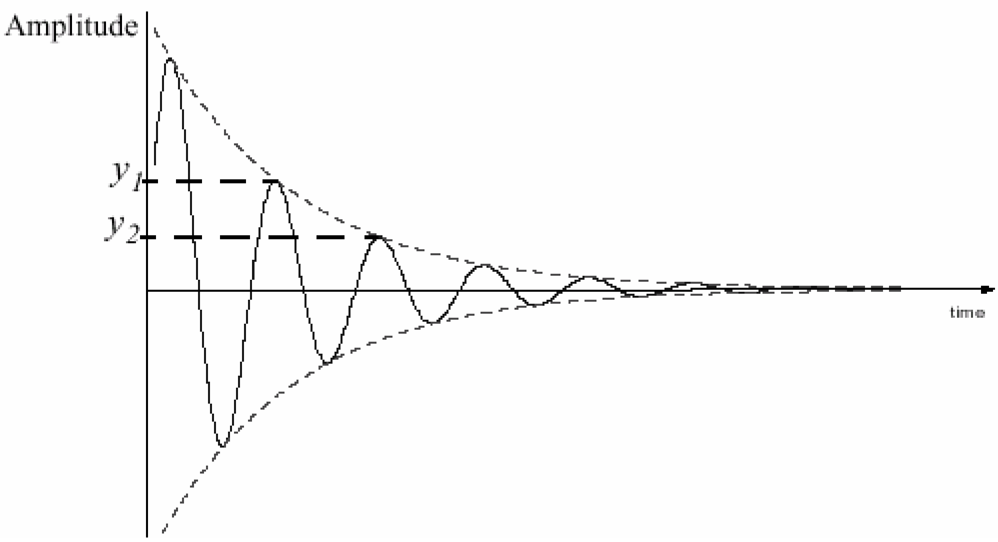


Figure 5.3 Generic free vibration of a damped system

When the damping ratio is small, ξ can be approximated as

$$\delta \cong 2\pi\xi \qquad \text{(Eq. 5.2)}$$

Solving for ξ;

$$\xi = \frac{\delta}{2\pi} = \frac{\ln(\frac{y_1}{y_2})}{2\pi} \qquad \text{(Eq. 5.3)}$$

Using the above equation, we can obtain the damping ratio of the structure using the amplitude of the signal at two consecutive peaks in a free vibration record of displacement or acceleration.

The half power bandwidth method uses the transfer function plot to obtain the damping. The method consists of determining the frequencies, at which the amplitude of the transfer function is A_2 where,

$$A_2 = \frac{A_1}{\sqrt{2}} \qquad \text{(Eq. 5.4)}$$

A_1 is the amplitude at the peak. The frequencies f_a and f_b associated with the half power points on either side of the peak are obtained, as shown in Figure 5.4. Then the damping ratio is obtained using the simplified formula for low damping ratios;

$$\zeta = \frac{f_b - f_a}{f_b + f_a} \qquad \text{(Eq. 5.5)}$$

The damping ratio associated with each natural frequency can be obtained using the half power bandwidth method.

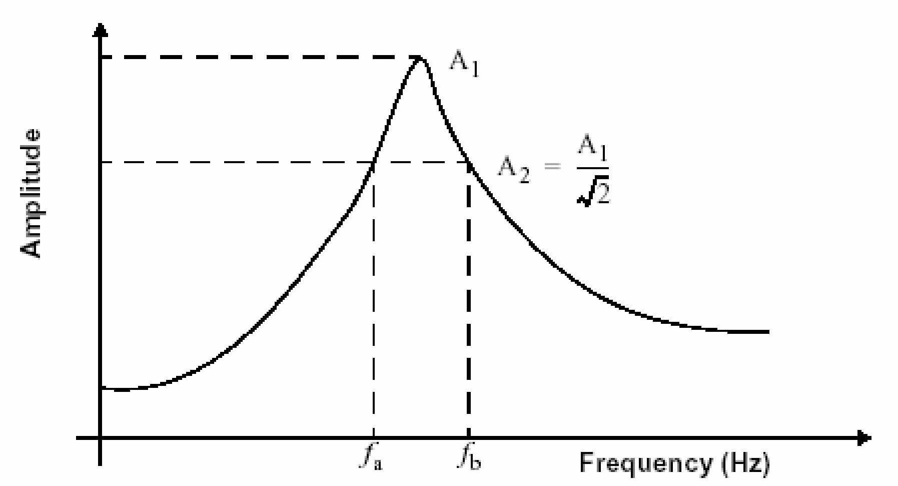


Figure 5.4 Illustration of half power bandwidth method

The half-power bandwidth technique may result in significant errors when the damping in the system is small. Because, the actual peak in the transfer function is difficult to capture and interpolation is required to estimate the half-power points. On the other hand, the decrement technique is more effective for lightly damped systems.

The damping values for the first two modes of the exterior wall have been determined by both methods.

Damping of exterior wall by half-power bandwidth method:

The mode shapes of all free vibrations, which have been gained from individual impacts on the wall, have been plotted and the A_1, A_2, f_a and f_b values have been determined on these plots. As an example, the Figure 5.5 is showing the transfer function caused by only one impact on the wall.

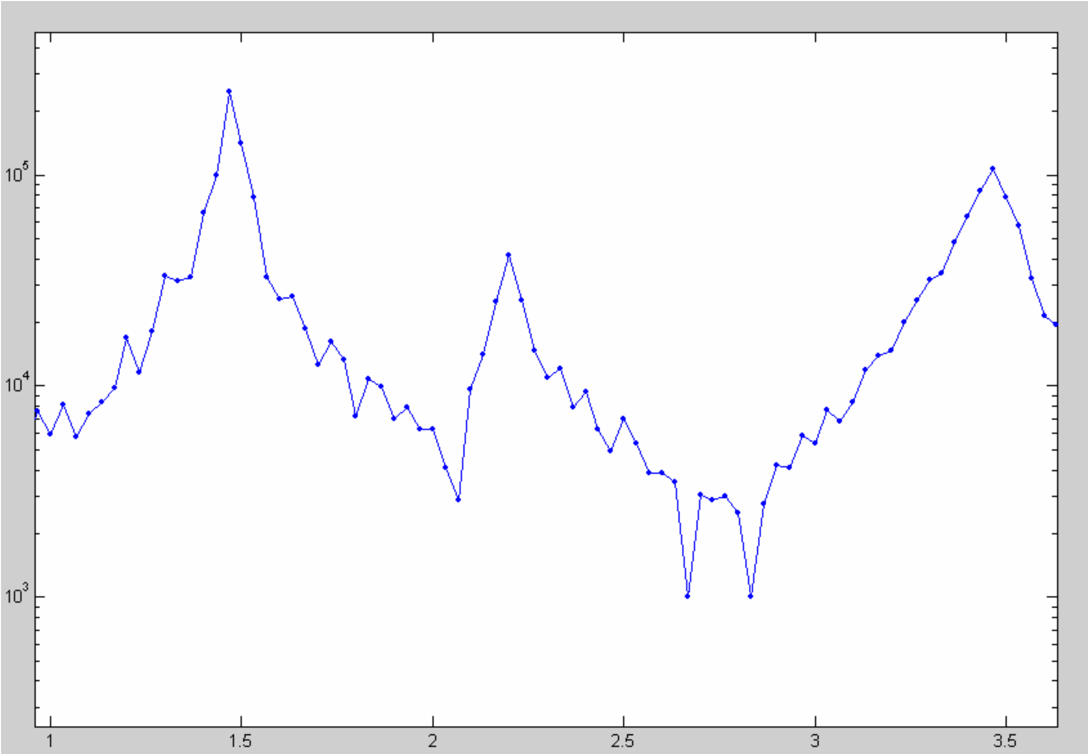


Figure 5.5 FFT of only one impact

From the figure the following values have been estimated for the first two modes:

Mode1:

A1= 2.4x10^5

A2= 1.7x10^5

fa= 1.45

fb= 1.49

By using equantion 5.2;

E= (1.49-1.45)/(1.49+1.45)=0.0136 (1.36%)

Mode2:

A1= 4.15x10^4

A2= 2.94x10^4

fa= 2.18

fb= 2.25

By using equantion 5.2;

E= (2.25-2.18)/(2.25+2.18)=0.0158 (1.58%)

Similarly, the damping ratios of the first two mode have been calculated for the transfer functions of all impacts on the wall. The range of the damping values were 1-2% for both first and second modes.

In order to eliminate the errors and to get an average transfer functions for the exterior wall, the transfer functions for the individual impacts have been combined. So one transfer function including the transfer functions of all impacts which has been applied on the exterior wall has been formed (see Figure 5.6).

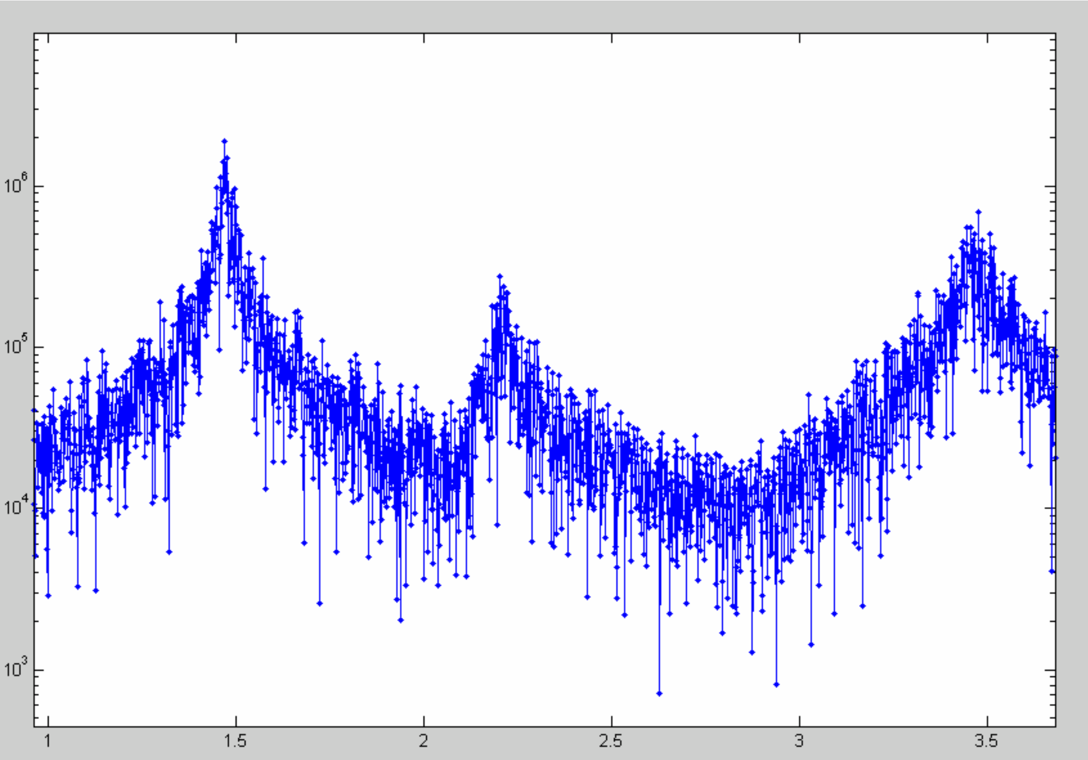


Figure 5.6 FFT of all impacts

The average transfer function and the first three modes of the exterior wall is drawn using a program called “ARTEMIS Extractor Pro” (http://www.svibs.com/products/extractor.htm). The program is the effective tool for modal identification of civil engineering structures such as buildings, bridges, dams and offshore structures. The software allows the user to accurately estimate natural frequencies of vibration and associated mode shapes and modal damping of a structure from measured responses only.

The first 2 natural frequency and the damping ratios are shown on Picture 5.1. The damping ratios for the first two modes are 0.874% and 1.08%, respectively.

Damping of Exterior Wall by Logarithmic Decrement Method:

From the free vibration Figure 5.7, the damping ratio has been calculated as 0.027 (2.7%) for one impact on the exterior wall. The damping ratios for the other free vibrations of the exterior wall have given a range of 1-3%.

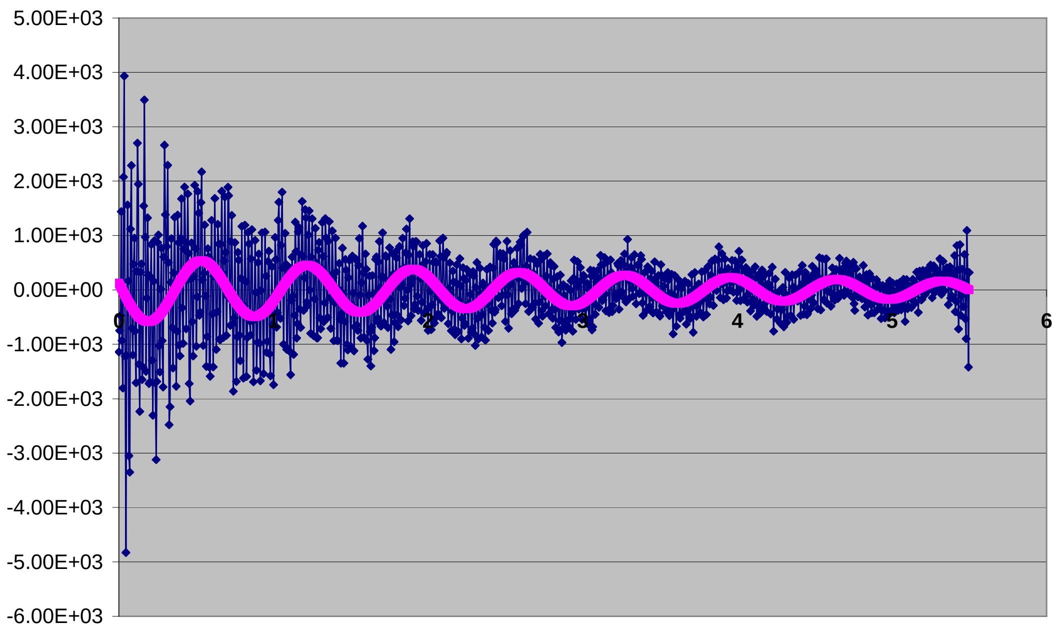
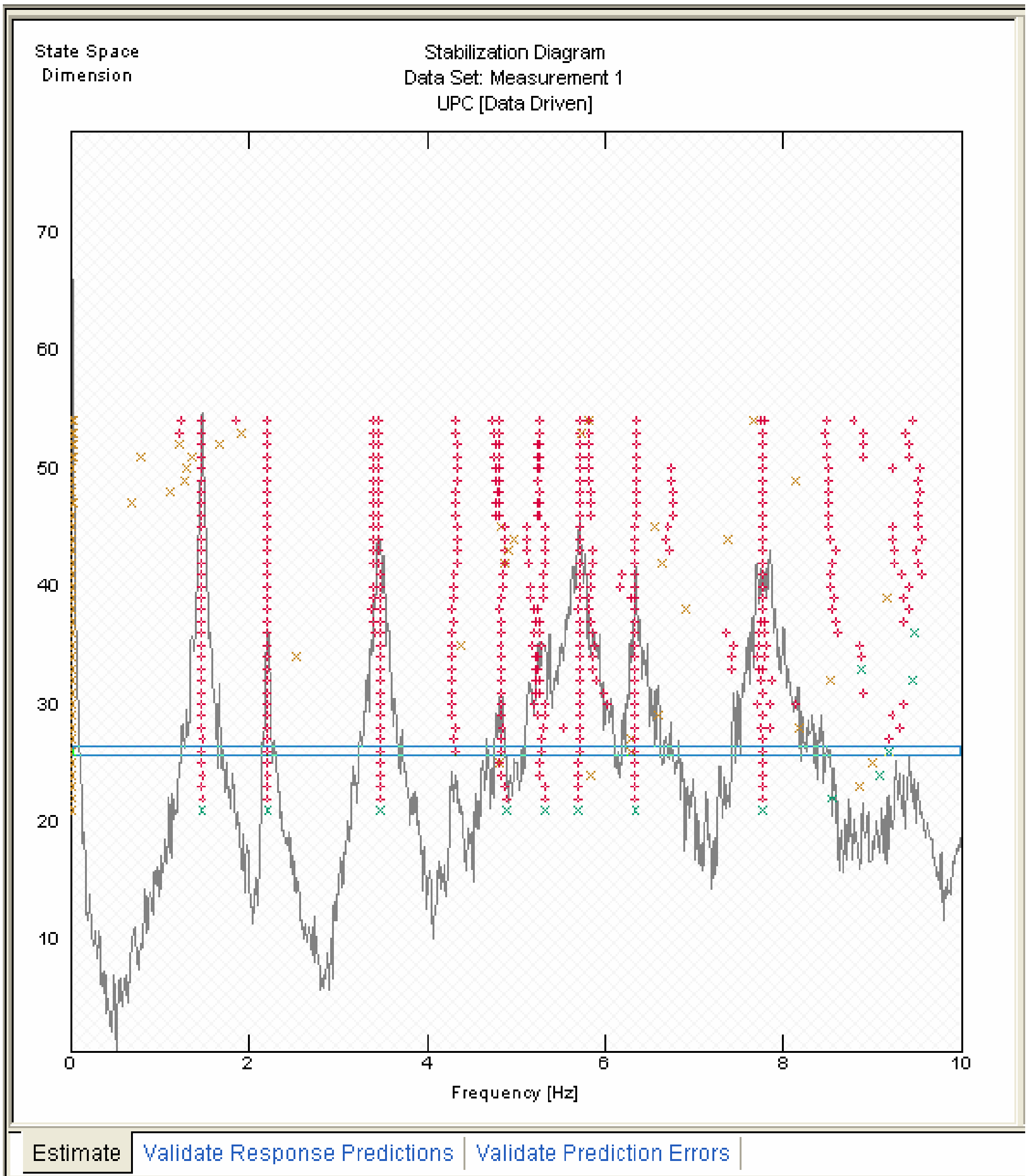


Figure 5.7 Free vibration of one impact on the exterior wall



Mode No.	Frequency [Hz]	Damping Ratio [%]	Stability
3	1.471	0.874	Stable
4	2.209	1.08	Stable
5	3.467	1.069	Stable
6	4.308	1.354	Stable
7	4.829	1.004	Stable
8	5.273	1.607	Stable
9	5.706	1.329	Stable
10	6.291	6.575	Noise
11	6.325	0.6204	Stable
12	7.762	1.308	Stable

Modal Indicators | Estimation Status | Modal Results - Cursor Model | Modal Results - Mod

Picture 5.1 Average transfer functions drawn with “Artemis” program

When the results of damping ratio values are compared with each other, it is seen that the errors are significant for both the Half-Power Bandwidth method and logarithmic decrement method. The damping ratio obtained by analyzing the average transfer function of all by the program "Artemis Extractor Pro" has given more feasible results for the damping ratios of the first two modes when compared to other methods.

In order to be on the safe side and eliminate the small errors in calculating the damping ratio, 0.008 (0.8%) has been assumed and used for the damping of exterior wall and also for the interior (stage) wall.

5.3.2 DETERMINATION OF NATURAL FREQUENCIES

The natural frequencies for the first two modes have been obtained from combined the transfer function (see Figure 5.6 & Picture 5.1) which has been formed from the transfer functions of all free vibrations of the wall. The natural frequencies of the first two modes are given on Table 5.1.

Table 5.1 Natural frequencies of the exterior wall (less than 2.5Hz)

Mode #	Natural Frequency of Exterior Wall
1	1.47
2	2.21

CHAPTER 6

STRUCTURAL ANALYSIS OF THE THEATRE UNDER SOUND INDUCED FORCES

6.1 MODAL ANALYSIS OF THE THEATRE

In order to get the natural frequencies of the theatre, a modal analysis of the model has been performed. The first five natural frequencies of the theatre is as follows;

Table 6.1 First five natural frequencies of analytical model

Mode Number	Natural Period	Natural Frequency	Part of Theatre
1	1.16	0.87	Stage Wall
2	0.68	1.47	Exterior Wall
3	0.60	1.66	Stage Wall
4	0.42	2.36	Exterior Wall
5	0.37	2.71	Stage Wall

The structure is the most critical at the first mode and less critical toward its highest mode number 5.

The mode shapes of the first 5 mode have been given on Figures 6.1, 6.2, 6.3, 6.4 & 6.5. The first, third and fifth natural frequencies of the theatre are belongs to the stage wall of the theatre. These natural frequencies of the stage wall are between 0.87 Hz and 2.71 Hz.

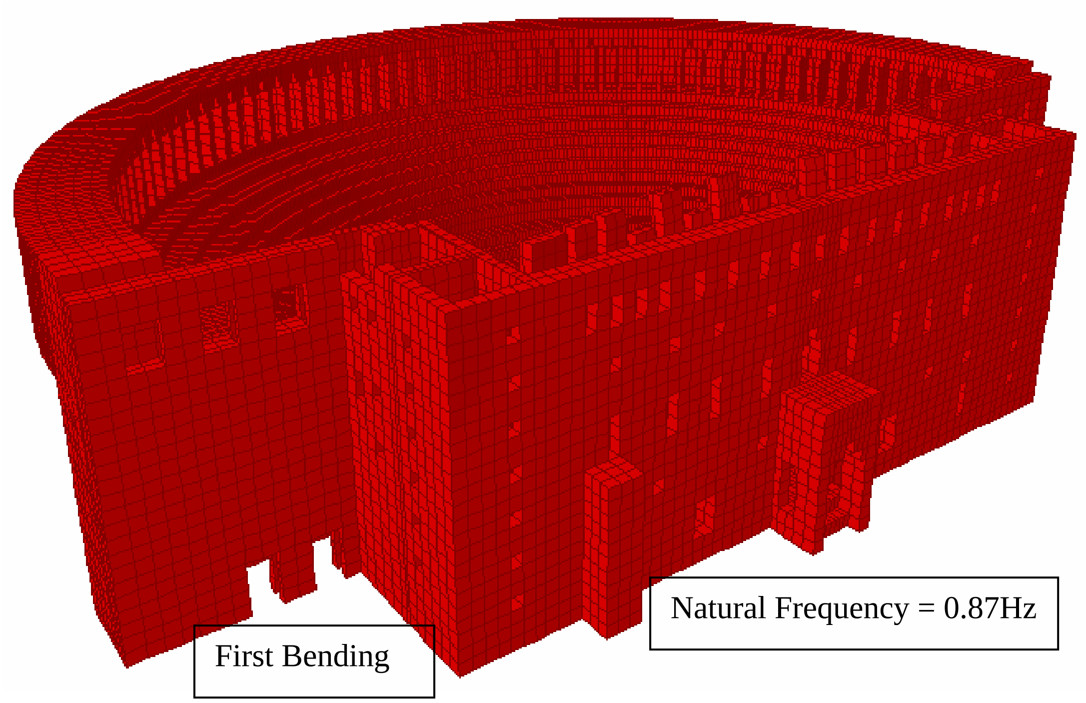


Figure 6.1 First mode shape of the model (Stage wall)

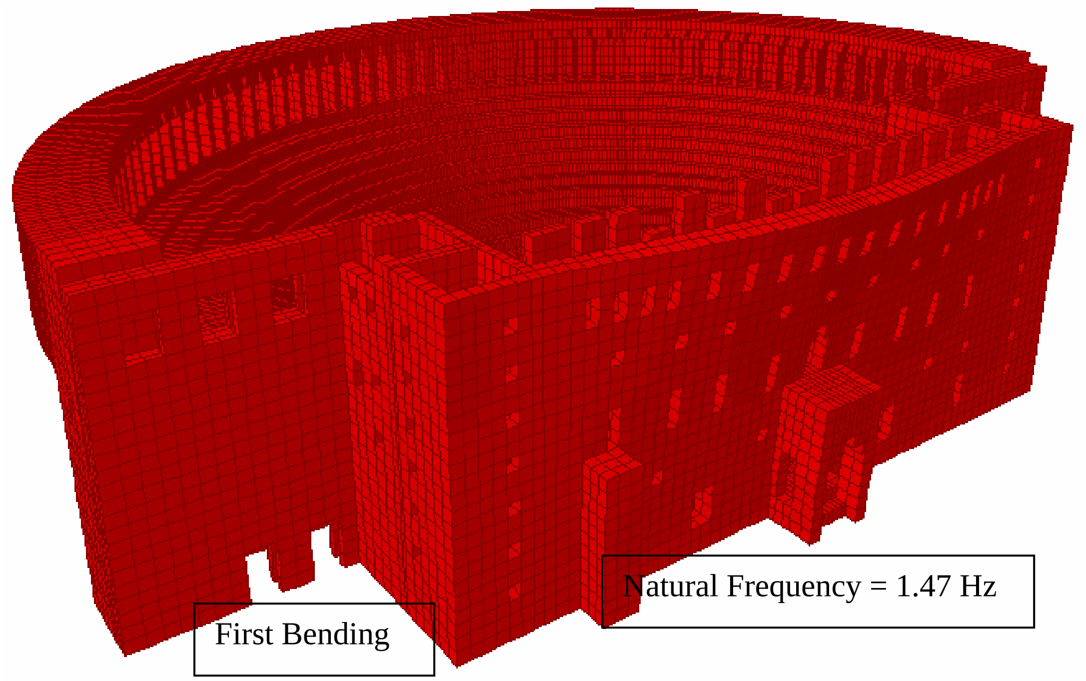


Figure 6.2 Second mode shape of the model (Exterior wall)

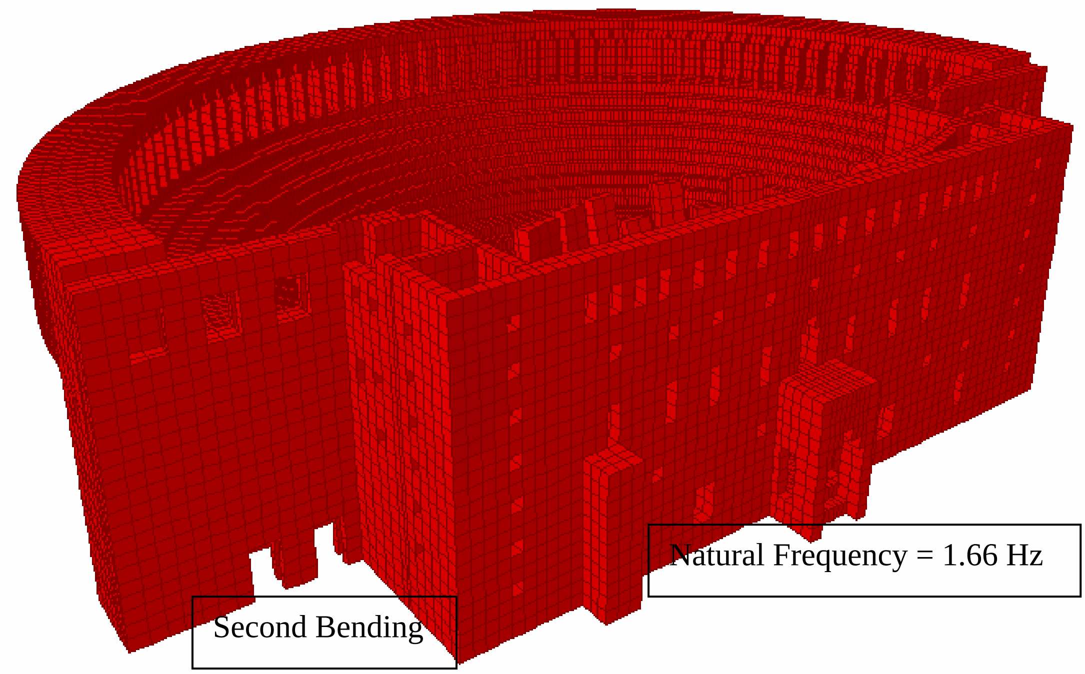


Figure 6.3 Third mode shape of the model (Stage wall)

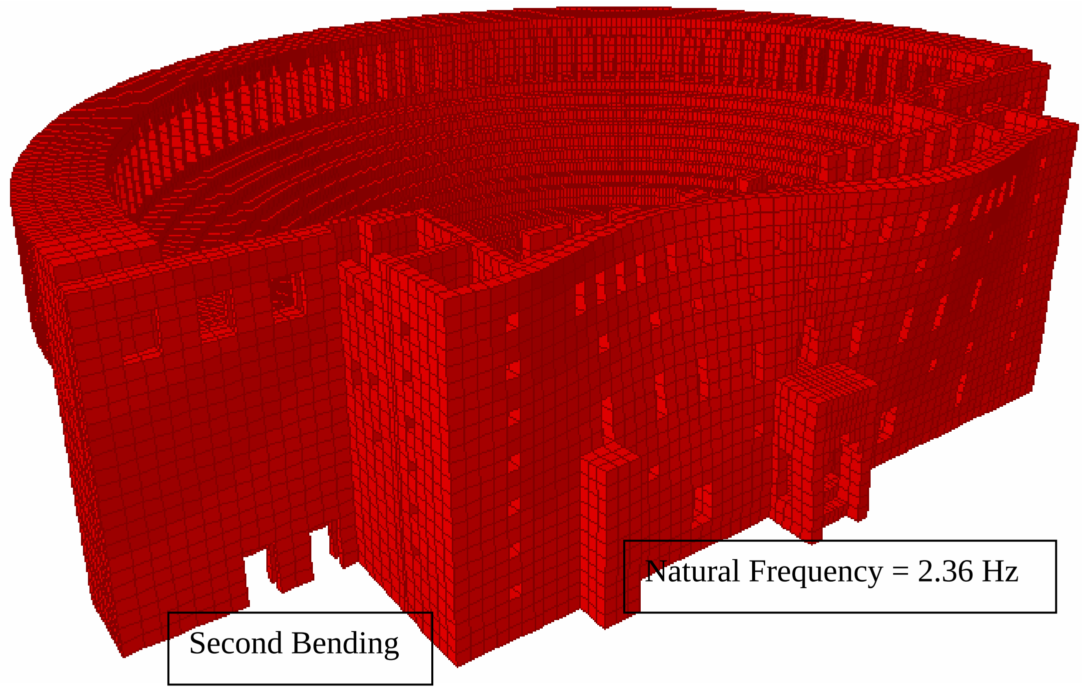


Figure 6.4 Fourth mode shape of the model (Exterior wall)

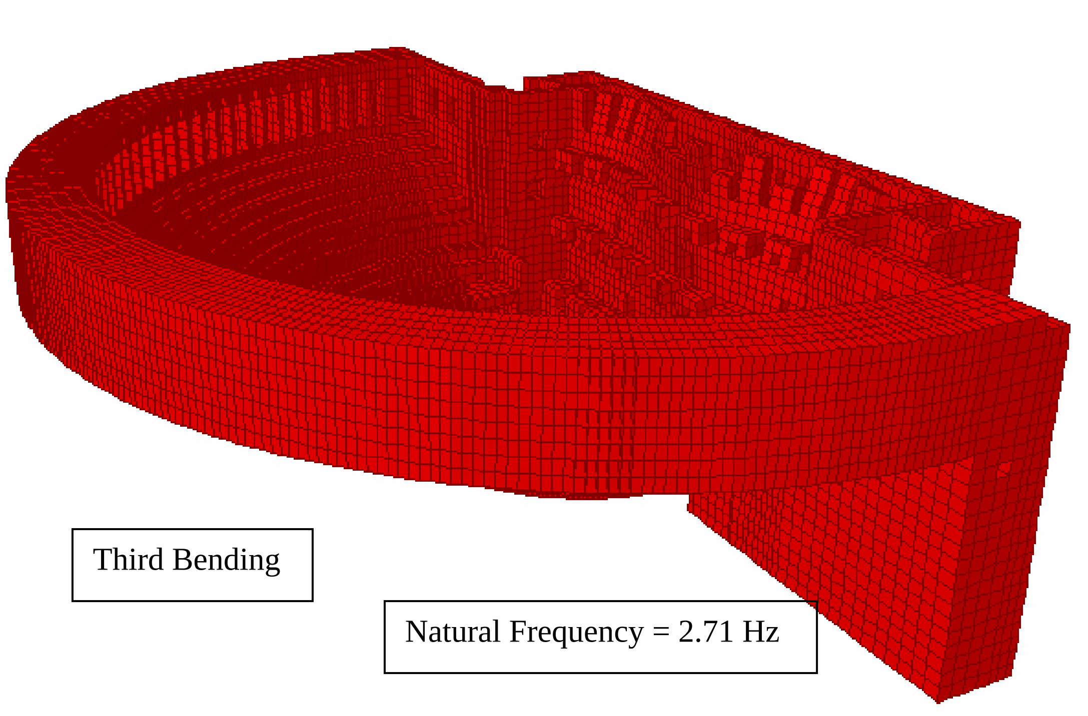


Figure 6.5 Fifth mode shape of the model (Stage wall)

6.2 BEAT FREQUENCY ANALYSIS OF SONGS

At the present time, the sound source is the speakers which are used in the various music concerts in the Aspendos Theatre. It has been observed at a number of pop concerts that structural response to dance-type loads is at the beat frequency of the music and at an integer multiple of the beat frequency. Therefore, it is suggested that the frequencies of sound source can be evaluated from the beat frequencies of music. Samples of 210 songs have been analyzed, including dance, pop and rock music. The frequency range and distribution of these songs show that 96.2% of the songs analyzed fall into the range from 1.0Hz to 2.8Hz (The Structural engineer, 2000).

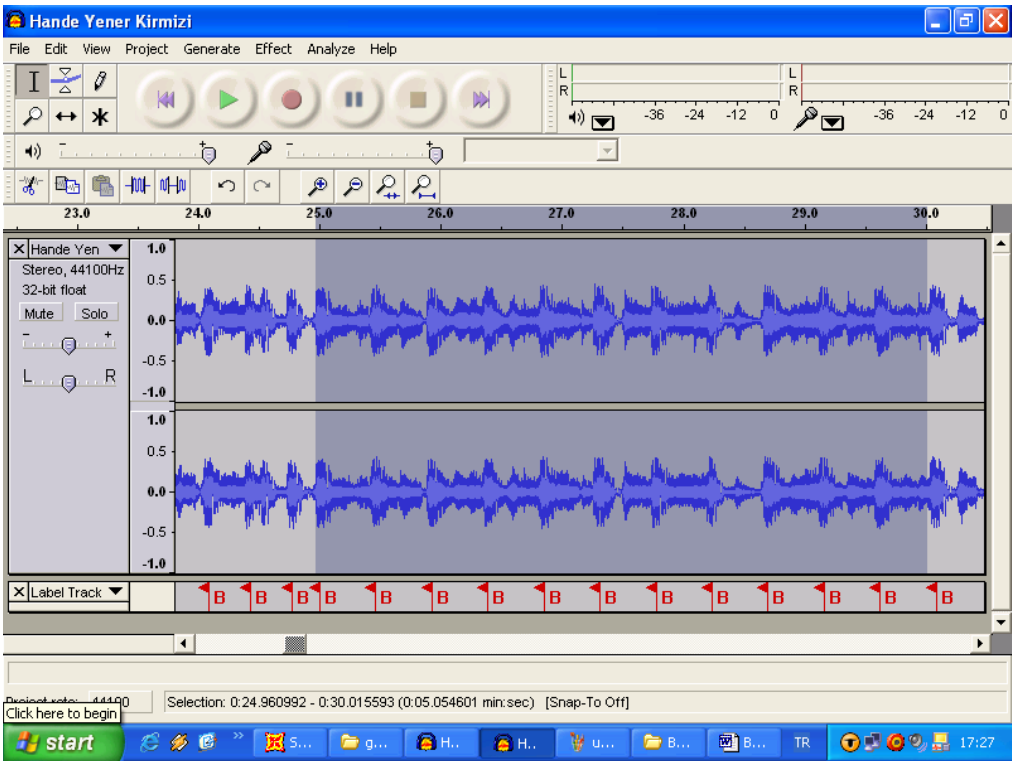
In addition to the values for the frequency range given above, a number of songs has been analyzed with a digital audio editor program called "Audacity", (http://audacity.sourceforge.net).

The aim of the analysis is to find the beats and to calculate the beat frequencies in different parts of the songs. The Table 6.2 shows the songs analyzed and the belonging beat frequencies.

Song Number	*Song Name*	*Selected time period (sec.)*	*Beats*	*Frequency*
1	Hande Yener - Kırmızı	5	11	2.2
2	Hande Yener - Acele Etme	14	28	2.0
3	Steve Wonder - Part Time Lover	15	21	1.4
4	Tarkan - Kır Zincirlerini	10	20	2.0
5	BeeGees - Staying Alive	12	20	1.7
6	Pamela - Ask Sevgiden Beter	15	18	1.2
7	Goksel - Karar Verdim	20	22	1.1

Table 6.2 Beat frequency analysis of songs

An example analysis result has been shown on Picture 6.1. The time period selected for this analysis is between 25sec and 30 sec. There are totally 11 beats in the selected time period. The beat frequency is calculated as “11 beats / (30-25sec) = 2.2 Hz”.



Picture 6.1 Analysis of Beat Frequency for a Song

6.3 COMPARISION OF NATURAL FREQUENCIES & BEAT FREQUENCIES

The frequency range for sound source (songs) has been defined to be between 1.0Hz and 2.8Hz in Chapter 6.2. The additional analysis for the beat frequencies of 7 songs has given a range of 1.1-2.2Hz which also falls into the range of 1.0-2.8Hz.

The natural frequencies of Aspendos Theatre have been found between 0.87Hz and 2.71Hz for the first 5 modes (see Chapter 6.1). As the first natural frequency of the theatre is near to 1.0 Hz, it will be assumed that the natural frequencies of the first five modes of the theatre are in the same range of the beat frequencies of the sound sources (songs).

As only the first five natural frequencies of the structure are in the frequency range of the sound source, only these five modes have been analyzed in this study. Because, the structure may only go into its resonance state if the frequencies of applied forces (sound pressure in this case) on it coincides with its natural frequencies.

6.4 SOUND INDUCED FORCES (SOUND PRESSURE)

The present limitation for the sound level which is allowed in the Aspendos Theatre is 90 decibel (dB). However, there is a no guarantee for this limitation to be kept in the future and there is also possibility that level of the sound may be increased up to a level of 110dB, which is the general sound level in the music concerts and in the discotheques. Therefore, in this study 110dB sound level has been analyzed.

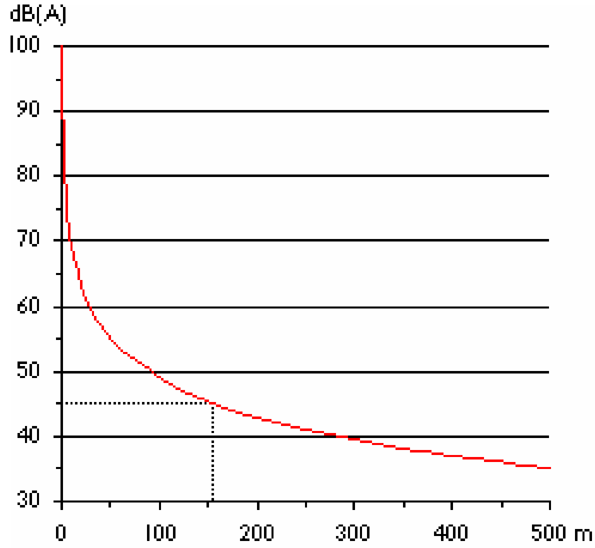
The sound pressure for 110dB sound level can be calculated as follows by using equation 3.1 :

110db = 20 log (P/2e-5)

$P = 6.325\ N/m^2$

As distance from the sound source increases, the sound pressure decreases. Therefore, it is important to include the distance from the sound source when calculating sound pressures.

The energy of sound waves (and thus the sound intensity) will drop with the square of the distance to the sound source. In other words, if sound source is moved 200 m away, the sound level will generally be one quarter of what it is 100 m away (Danish Wind Industry Association, 2003). A sample graph showing the decrease in dB with respect to the distance from the sound source is given below.



Graph 6.1 Change in sound level with respect to distance

In the Aspendos Theatre, it has been observed that the there are several alternatives for the placement of the sound sources (i.e. speakers). These alternatives can be arranged as follows:

Alternative 1: Speakers are at a distance of 0.3m from the stage wall

Alternative 2: Speakers are at a distance of 1.0m from the stage wall

Alternative 3: Speakers are at a distance of 2.0m from the stage wall

The sound level will be reduced when the distance of sound source increases from 0.3m up to 2.0m from the stage wall. For example, the 90dB sound level at a distance of 0.3m will be 74dB at a distance of 2.0m (Sound Pressure Loss with Distance Calculator, 2005)

The number of the speakers are also affective when calculating the sound pressure. In the Aspendos Theatre, it has been observed that the number of speakers near the stage wall may vary between 3-10. Therefore, in order to be on the safe side when calculating the sound level, it has been assumed that totally 10 speakers are placed in front of the stage wall. According to the P2/P1=10 ratio, the increase in dB is calculated as 20 (please see Table 3.3).

As a result, according to the sound level, number of speakers used and their distance to the stage wall, the alternatives for the calculation of the sound pressures can be written as follows:

Alternative 1.1: 10 Speakers with each 110dB sound levels are placed at a distance of 0.3m from the stage wall
Alternative 1.2: 10 Speakers with each 110dB sound levels are placed at a distance of 1.0m from the stage wall
Alternative 1.3: 10 Speakers with each 110dB sound levels are placed at a distance of 2.0m from the stage wall

The sound levels for each alternatives will be as follows:

Table 6.3 Sound levels for each alternatives

Alternative number	*Speaker Number*	*Sound Level of 1 speaker (dB)*	*Sound Level of 10 speakers (dB)*	*Distance from Stage wall (m)*	*Final Sound level (dB)*
1.1	10	110	130	0.3	130
1.2	10	110	130	1.0	121
1.3	10	110	130	2.0	115

Consequently, the sound pressures for sound levels of each alternatives are calculated according to the equation 3.1 and they are shown on Table 6.4.

Table 6.4 Sound pressures for each alternatives

Alternative number	*Final Sound level (dB)*	*Sound Pressures (N/m2)*
1.1	130	63.25
1.2	121	22.44
1.3	115	11.25

According to Table 6.4, the sound pressure range varies from 11.25N/m^2, which is for Alternative 1.3, to 63.25N/m^2, which is for Alternative 1.1. It is seen that, the minimum sound pressure (11.25N/m^2) arises when the distance of the speakers are far from the stage wall. Also, it is seen that the maximum sound pressure (63.25N/m^2) arises when the distance of the speakers are at the minimum distance to the stage wall.

6.5 SOUND INDUCED FORCES UNDER RESONANCE CONDITION

When the structure enter its resonance state, the deflection will be increased linearly by X_m multiplier (Chopra, 1995). Xm multiplier is calculated by

$$X_m = \left(\frac{1}{2 * Dampingratio}\right) \qquad \text{(Eq 6.1)}$$

The sound pressures for each alternative loading is shown on Table 6.5.

Table 6.5 Sound pressures for each alternatives

Alternative number	*Sound Pressures (N/m2)*	*Xm Multiply*	*Sound Pressures with Xm (N/m2)*
1.1	63.25	62.5	3953
1.2	22.44	62.5	1403
1.3	11.25	62.5	703

6.6 ANALYSIS OF THE THEATRE UNDER SOUND INDUCED FORCES

It has been assumed that the sound pressure is applied uniformly on the surface of the whole stage wall regardless the distance change of the speakers from every point of the stage wall. That is, the distance will be kept as it is for its alternative number for the whole surface of the stage wall. For example, the sound pressure will be 3953 N/m^2 for the whole surface of the stage wall for alternative number 1.1 (Table 6.5).

The sound pressure loading on the interior (stage) wall is shown on Figure 6.6. Please note that, because of the capabilities of the program, the loads on the small parts on the wall and the loads on the wall is displayed separately although they are applied at the same time.

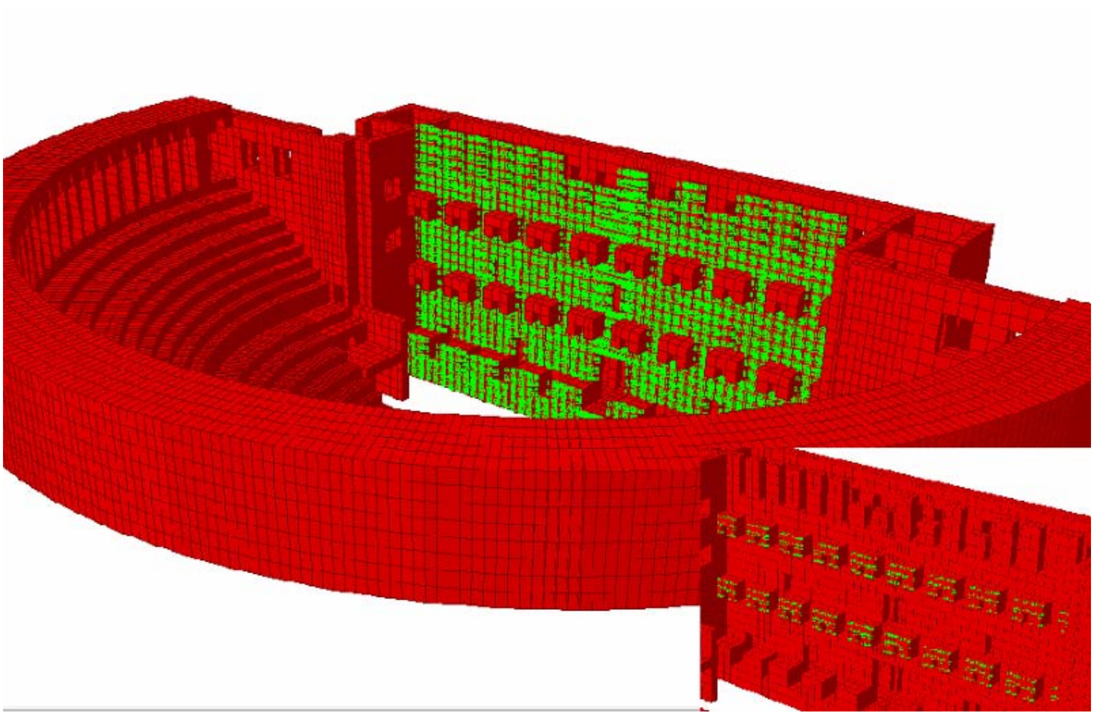


Figure 6.6 Sound pressure loading on the stage wall

6.7 FEM ANALYSIS RESULTS

The analysis results have been grouped into 4;

1. The maximum compressive and tensile stresses due to the dead weight of the theatre,
2. The maximum compressive and tensile stresses due to the sound induced forces,
3. The maximum compressive and tensile stresses due to both the sound induced forces and dead weight of the theatre,
4. The maximum deflections at the top part of the stage wall

As, only the stage wall is exposed to the sound induced forces, the analysis results will be shown for this part of the entire theatre. Also, the stage wall will be extracted from the model view while showing the analysis results for clarity. The stress results will be shown on both front view and the rear view of the stage wall.

The locations (Location F1, F2, F3, F4, R1, R2, R3. R4), where the maximum compressive and tensile stresses occurred, are shown on Figures 6.7 & 6.8 for front and rear view of the stage wall, respectively..

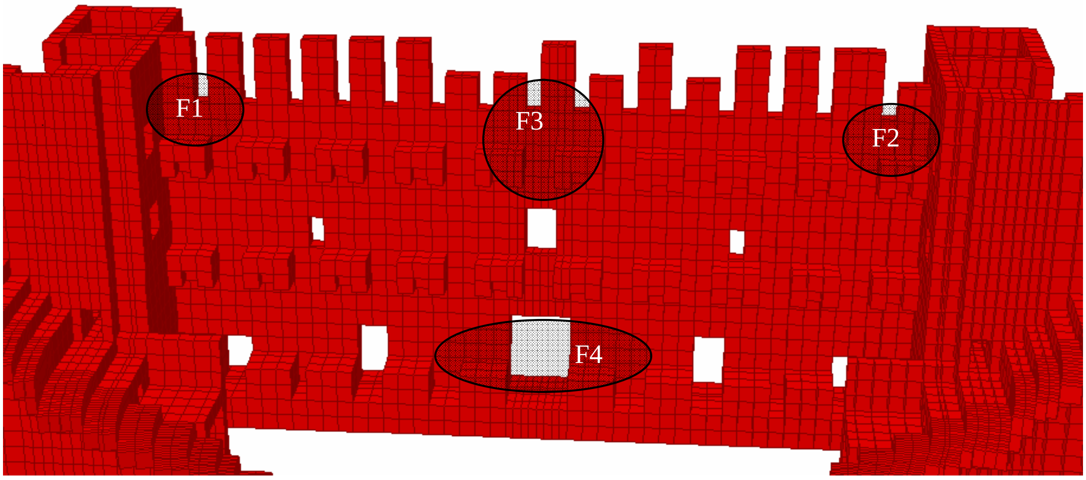


Figure 6.7 Locations of maximum stresses in front view

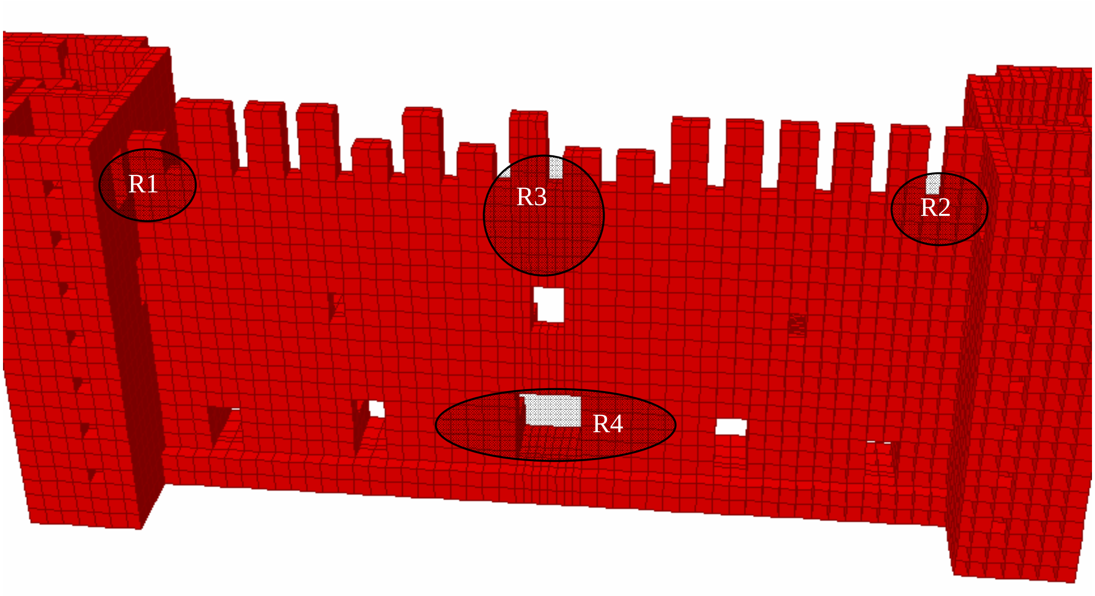


Figure 6.8 Locations of maximum stresses on rear view

1. Stresses due to the dead (own) weight of the theatre is shown on Figure 6.9.

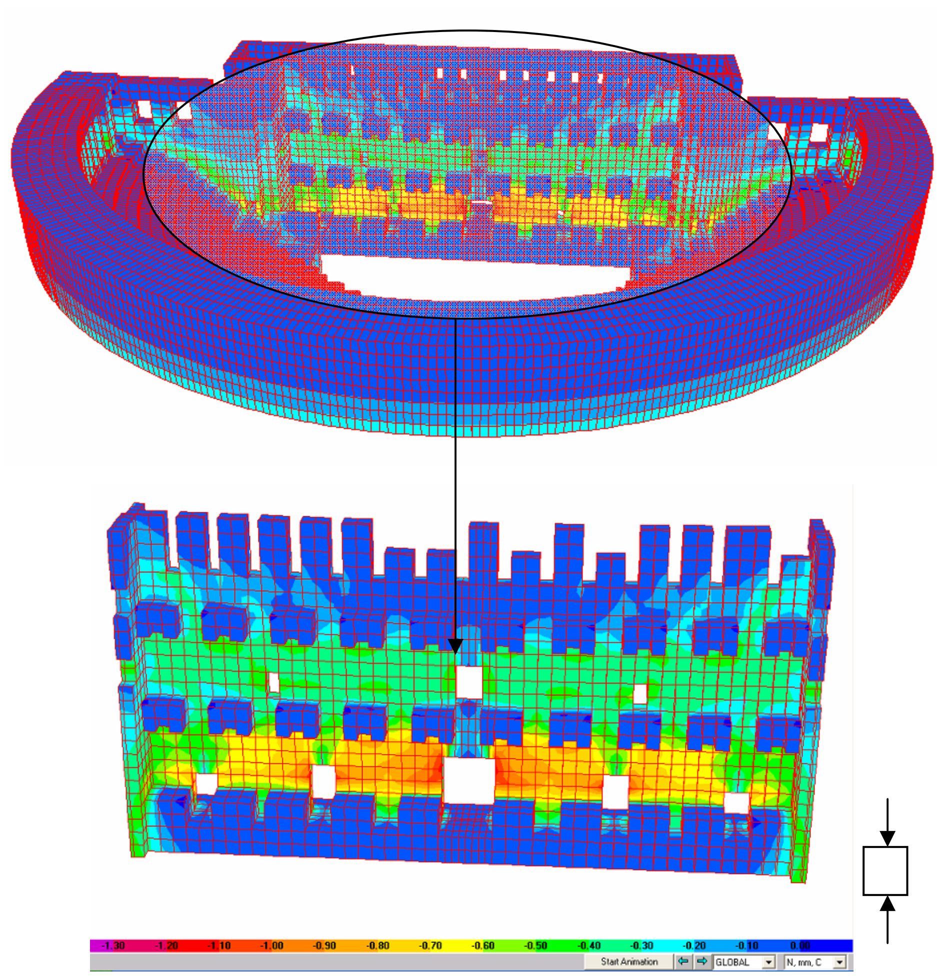


Figure 6.9 Compressive stresses due to dead weight (on front view)

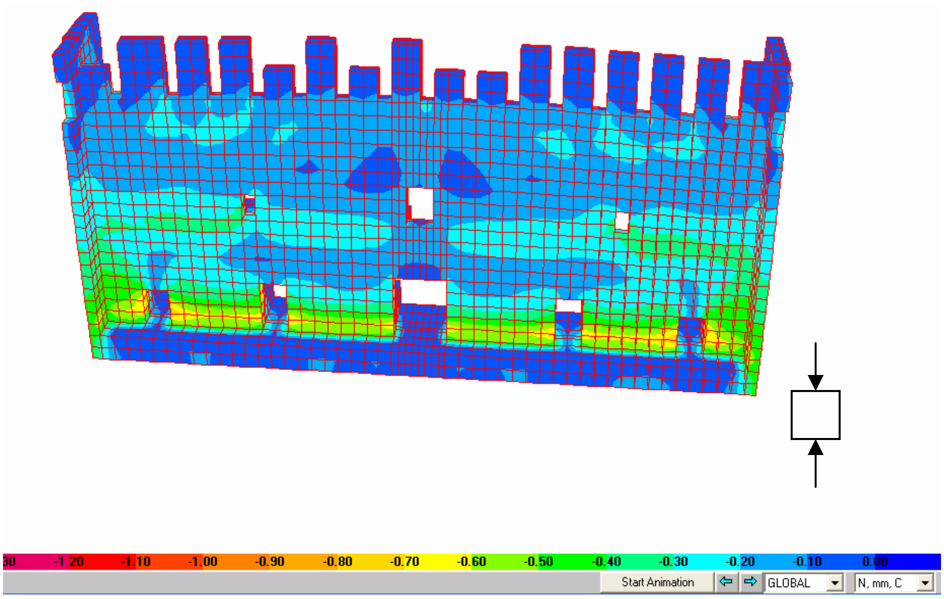


Figure 6.10 Compressive stresses due to dead weight (on rear view)

The maximum compressive stress (“-“ denotes compression) on the front view of the stage wall is around 1.1 MPa at the F4-location (Figure 6.9). The maximum compressive stress at the rear view is around 0.7 MPa along the bottom line of the stage wall (Figure 6.10).

2. The stresses due to the sound induced forces (alternative 1.1, sound pressure=3953N/m^2 from Table 6.5) are shown on Figures 6.11, 6.11a, 6.12 and 6.12a). The stress directions are also shown on the figures. The governing stresses are the out of plane stresses due to bending of stage wall under the sound induced forces. Therefore, all the sketches below show the out of plane stresses only.

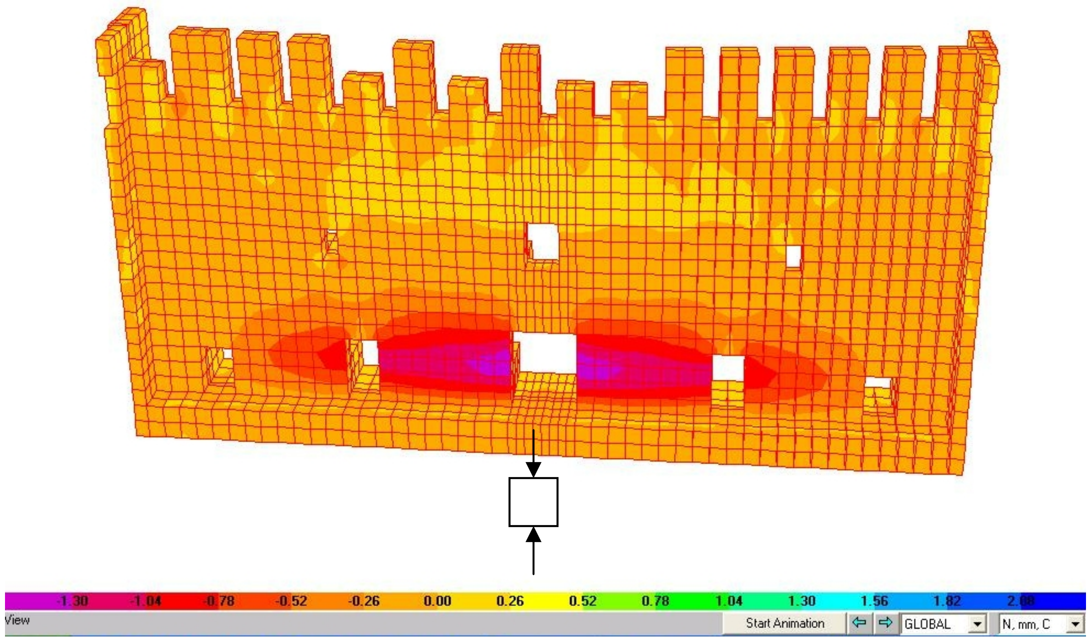


Figure 6.11 Stresses due to sound induced forces (on rear view) – 1

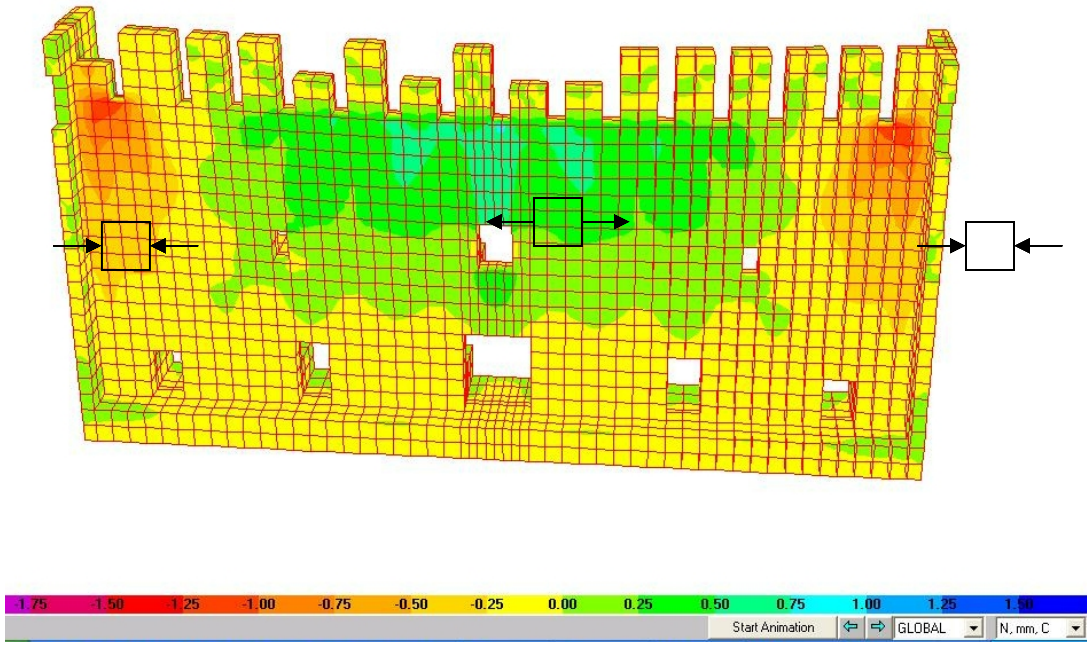


Figure 6.11a Stresses due to sound induced forces (on rear view) – 2

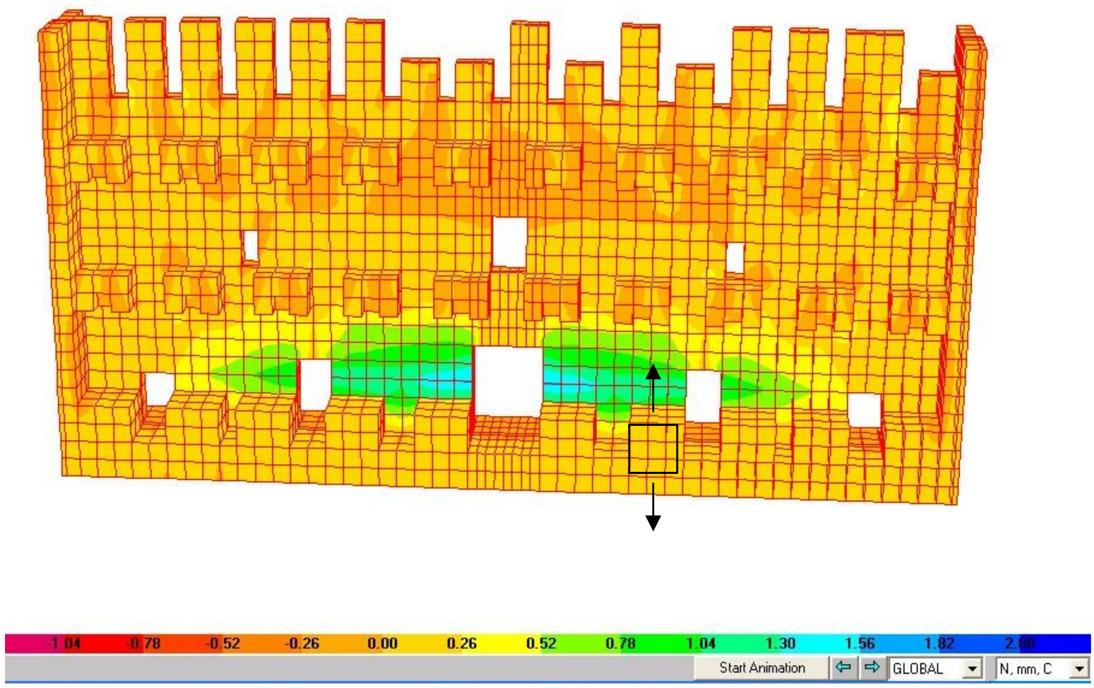


Figure 6.12 Stresses due to sound induced forces (on front view) – 1

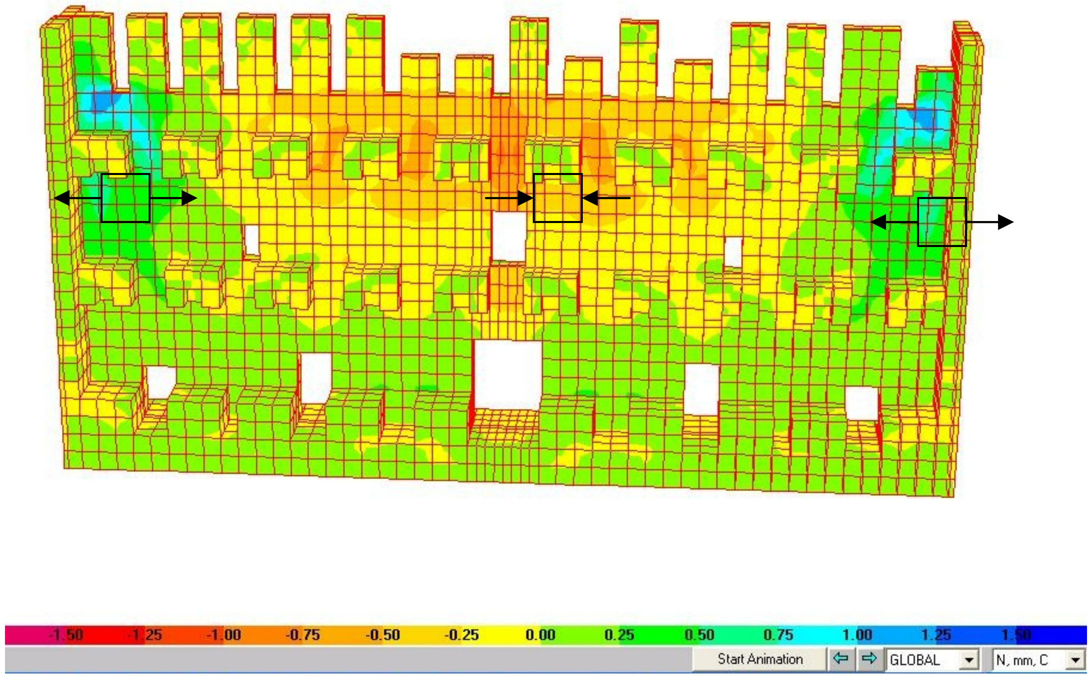


Figure 6.12a Stresses due to sound induced forces (on front view) - 2

It is seen that, the maximum compressive stress is around 1.6 MPa at R4-location (Figure 6.11). Also, the maximum tensile stress is found to be around 1.5 MPa at F4-location (Figure 6.12). As the loading direction of the sound induced forces is toward the front view of the stage wall, the stresses are compressive on rear view and tensile on the front view of the stage wall. In reality, the stage wall will be in its resonance state and both compressive and tensile stresses will be formed on both view of the wall. Therefore, it can be thought that the stresses on Figures 6.11, 6.11a, 6.12 and 6.12a are in both compression and in tension.

3. When the analysis results of dead weight and the sound induced forces are combined, the maximum compressive stress will be 1.6 MPa + 1.1 MPa = 2.7 MPa at F4-location and the maximum tensile stress will be 1.5 MPa + 0.3 MPa = 1.8 MPa at R1-location.

4. The maximum deflections at the top part of the stage wall are 24.4mm for dead weight loading, 92.5mm for loading of sound induced forces.

Therefore, the total deflection is 24.4mm + 92.5mm = 116.9mm at the top-middle of the stage wall.

The maximum compressive & tensile stresses and the deflections at the top of the stage wall is summarized on Tables 6.6 & 6.7.

Table 6.6 Maximum compressive and tensile stresses

	Due to Dead Weight	Due to Sound induced forces	Due to Combination of Dead Weight and Sound induced forces	Location on the Stage Wall
Compressive Stress on front view (MPa)	-1.1	-1.6	**-2.7**	F4
Tensile stress on front view (MPa)	-0.3	1.1	0.8	F1, F2
Compressive Stress on rear view (MPa)	-0.7	-1.5	-2.2	R4
Tensile stress on rear view (MPa)	0.3	1.5	**1.8**	R1, R2

Table 6.7 Maximum deflections at the top part of stage wall

Loading	Maximum Deflection (mm) *
Due to Dead Weight	24.4
Due to Sound induced forces	92.5
Due to Combination of Dead Weight and Sound induced forces	116.9

* Horizontal direction, perpendicular to the wall (U2 direction, Figure 7.1)

CHAPTER 7

STRUCTURAL ANALYSIS OF THE THEATRE UNDER EARTHQUAKE LOADING

The theatre has probably seen many earthquakes during its lifetime of 2200 years. However, there is no information regarding its demolition or re-construction in the history. It seems that, the theatre has not been destroyed fully or partially under the earthquakes in the region of the theatre during the last 22 centuries. However, during the inspection in the theatre, it has been learned that the theatre had some minor repairs on various parts of the building; some of them in the near future, some of them centuries before. 5 or 6 columns at the backside of the stands have been repaired in 1968 according to the information gained from the staff of the theatre. Also, there is an information that the Seljuk Turks have restored the theatre to use it as a caravanserai.

In order to determine the behavior of the Aspendos Theatre under earthquake loading, the analytical model of the theatre has been analyzed using the concepts obtained from “Specification for Structures to be Built in Disaster Areas” with appropriate modifications for the damping ratio and return period.. The linear analysis results have been examined for expected damage condition during/after a possible earthquake.

7.1 CALCULATION OF THE EARTHQUAKE LOADS

The earthquake analysis of Aspendos Theatre has been conducted using Turkish code, “Specification for Structures to be Built in Disaster Areas”. The

Aspendos Theatre, which is located in Antalya city, is in the second seismic zone. The earthquake loads have been calculated as follows;

7.1.1 Load calculations for design purpose

Effective ground acceleration coefficient, Ao = 0.30 (seismic zone of 2),
Building importance factor, I = 1.4 (museum)
Spectrum characteristic periods; Ta & Tb = 0.15 & 0.40, respectively (Z2 soil conditions)
Reduction factor, R = 1.5
The spectrum coefficient S(T) has been calculated using the equations below:

S(T) = 1 + 1.5*T/Ta ……. $0<T\leq Ta$, (Eq. 7.1)
S(T) = 2.5 ……. $Ta<T\leq Tb$, (Eq. 7.2)
S(T) = 2.5*(Tb/T)^0.8 … $T>Tb$, (Eq. 7.3)

The spectral Acceleration Coefficient, A(T) has been calcualted using the equation 7.4:

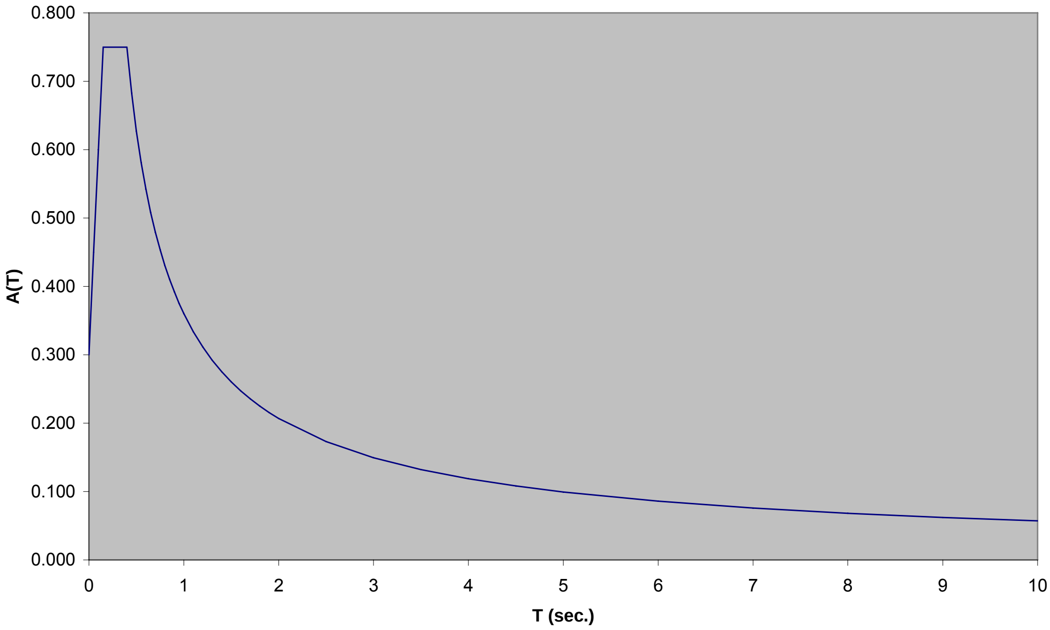
A(T) = Ao*I*S(T) (Eq. 7.4)

7.1.2 Load calculations for damage assessment/evaluation purpose

As the Aspendos Theatre is not being re-designed, the following values should be used for evaluation purpose;

Building importance factor, I = 1.0
Reduction factor, R = 1.0

The table and the graph for the response spectrum are shown below (Table 7.1 & Graph 7.1).



Graph 7.1 Response spectrum for 5% damping

The performance evaluation of the Aspendos Theatre is the main aim of the study. Therefore, the philosophy of the code requirements and formulas are modified and used to calculate the EQ demand on the structure. The evaluation studies reveals if the Aspendos Theatre will undergo any structural damage during an EQ. The exceedence of the linear range of the materials (i.e. tensile capacity) is the damge criteria. A rating factor (i.e. Factor of Safety; F.S) is calculated for tensile and compressive capacities indicating the total capacity in terms of applied loads as a unitless quantity.

Table 7.1 Response spectrum values

	T (sec.)	S(T)	A(T) = 0.3*S(T)
	0	1	0.300
	0.05	1.5	0.450
	0.1	2	0.600
TA	0.15	2.5	0.750
	0.2	2.5	0.750
	0.25	2.5	0.750
	0.3	2.5	0.750
	0.35	2.5	0.750
TB	0.4	2.5	0.750
	0.45	2.275	0.683
	0.5	2.091	0.627
	0.55	1.938	0.581
	0.6	1.807	0.542
	0.65	1.695	0.509
	0.7	1.598	0.479
	0.75	1.512	0.454
	0.8	1.436	0.431
	0.85	1.368	0.410
	0.9	1.307	0.392
	0.95	1.251	0.375
	1	1.201	0.360
	1.1	1.113	0.334
	1.2	1.038	0.311
	1.3	0.974	0.292
	1.4	0.918	0.275
	1.5	0.868	0.261
	1.6	0.825	0.247
	1.7	0.786	0.236
	1.8	0.751	0.225
	1.9	0.719	0.216
	2	0.690	0.207
	2.5	0.577	0.173
	3	0.499	0.150
	3.5	0.441	0.132
	4	0.396	0.119
	4.5	0.361	0.108
	5	0.331	0.099
	6	0.286	0.086
	7	0.253	0.076
	8	0.228	0.068
	9	0.207	0.062
	10	0.190	0.057

7.2 THE STRUCTURAL ANALYSIS UNDER EQ LOADS

The analytical model has been analyzed under the earthquake loads, which have been calculated in Chapter 7.1, by using SAP2000 software. The theatre has been analyzed in three global directions; U1, U2 & U3 (see Figure 7.1).

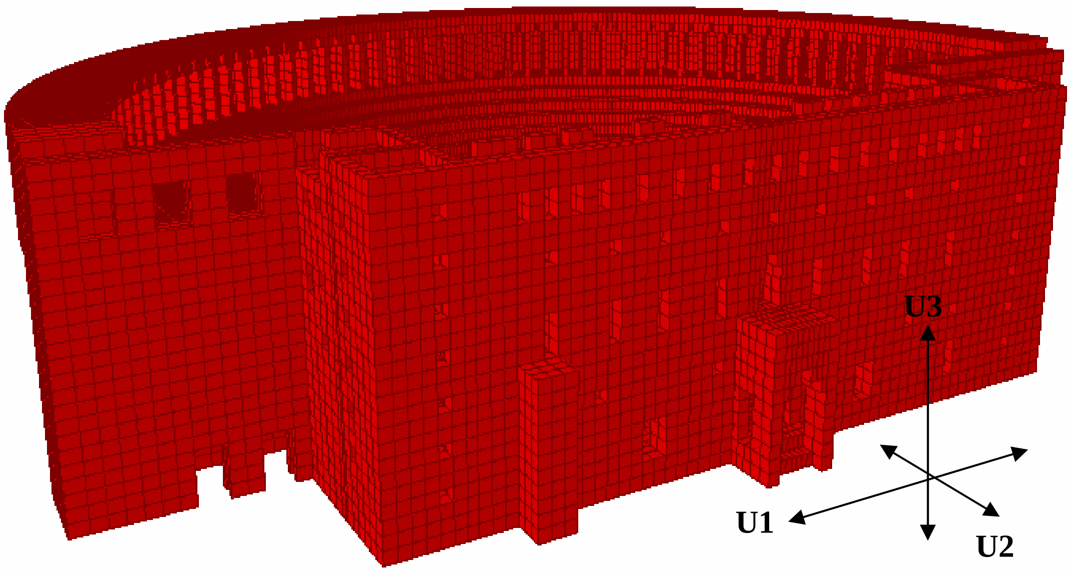


Figure 7.1 Directions of earthquake loadings

7.3 THE ANALYSIS RESULTS

The analysis results have been calculated for the maximum compressive and tensile stresses due to both the earthquake forces and dead weight of the theatre and maximum deflections at the top part of the stage and exterior walls.

The analyses results showed that the stage wall and the exterior wall are critical parts of the theatre. Therefore, the analysis results will be shown for these parts of the entire theatre. Also, the stage wall and the exterior wall will be extracted from the model view while showing the analysis results for clarity. The stress results will be shown on both front view and the rear view of these two walls.

The locations are labeled with letters F and R for the front and rear views, respectively. These locations are F1, F2, F3, F4, R1, R2, R3, R4, F5, F6, F7, F8, F9, R5, R6, R7, R8), where the maximum compressive and tensile stresses occur as shown in Figures 7.2, 7.3, 7.4 & 7.5 for front and rear views of stage walls.

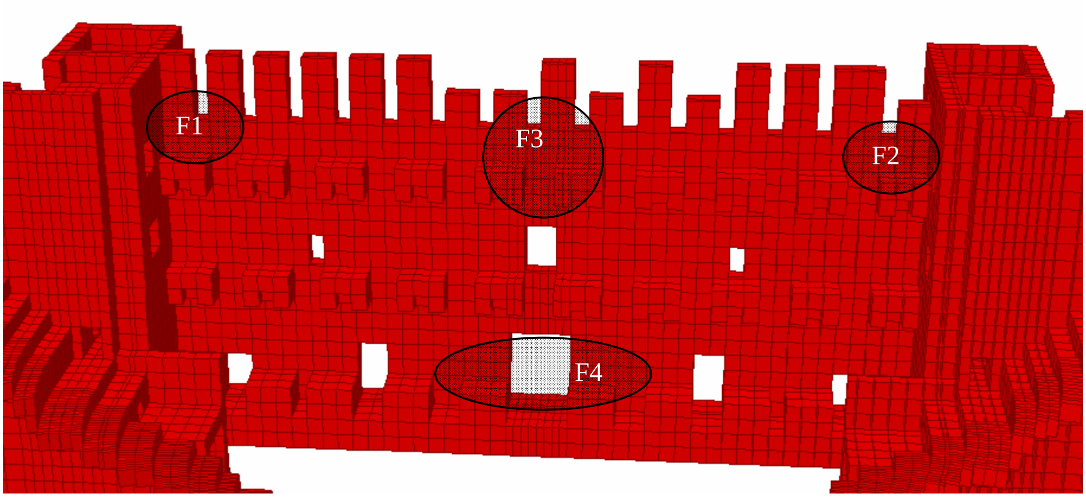


Figure 7.2 Locations of maximum stresses in front view of stage wall

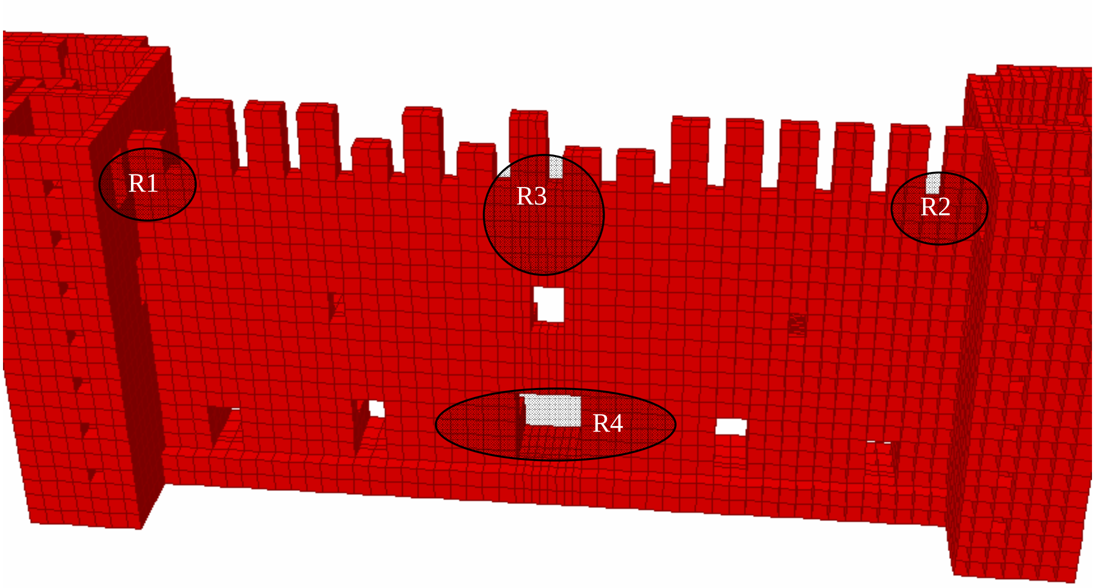


Figure 7.3 Locations of maximum stresses on rear view of stage wall

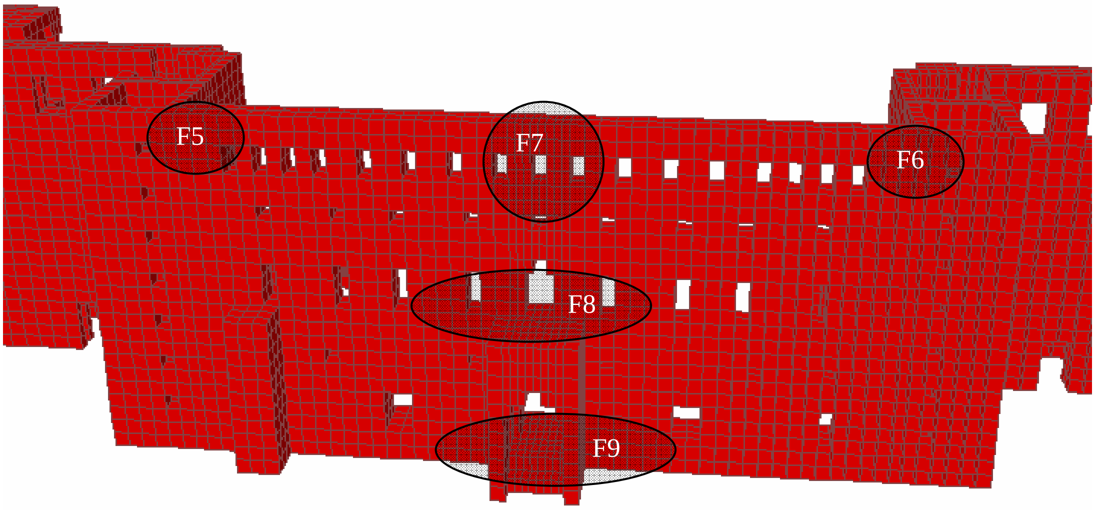


Figure 7.4 Locations of maximum stresses on front view of exterior wall

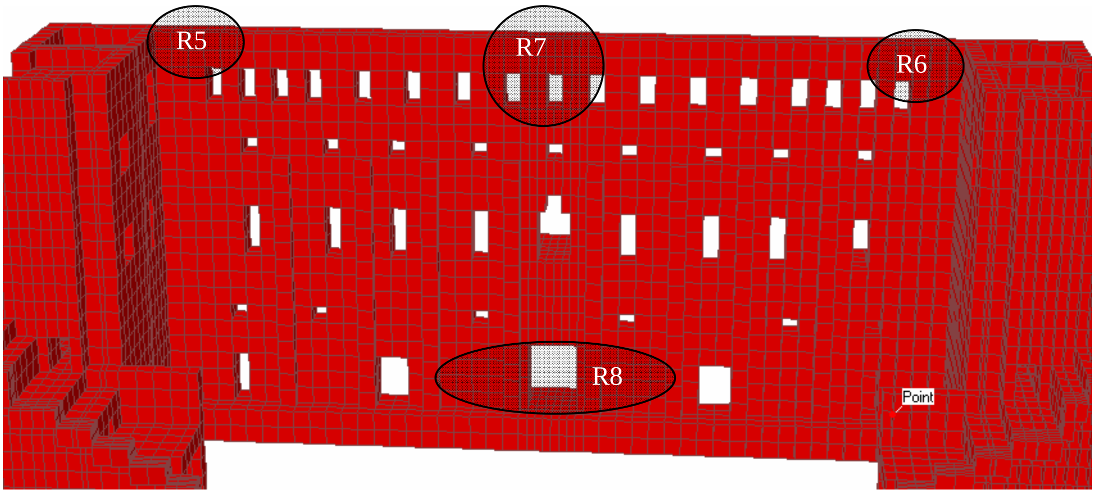


Figure 7.5 Locations of maximum stresses on rear view of exterior wall

The analysis results have shown that the maximum stresses under earthquake loads in U1 & U3 directions are insignificant when compared to the results of earthquake loading in U2 direction. Two examples for the stress distribution of U1 & U3 directional earthquake loading are shown on Figures 7.6 & 7.7, respectively. The governing stresses are the out of plane bending stresses of stage and exterior walls under earthquake loads.

Therefore, all the following figures show the out of plane bending stresses only. Also, EQ and DL denotes earthquake and dead load, respectively.

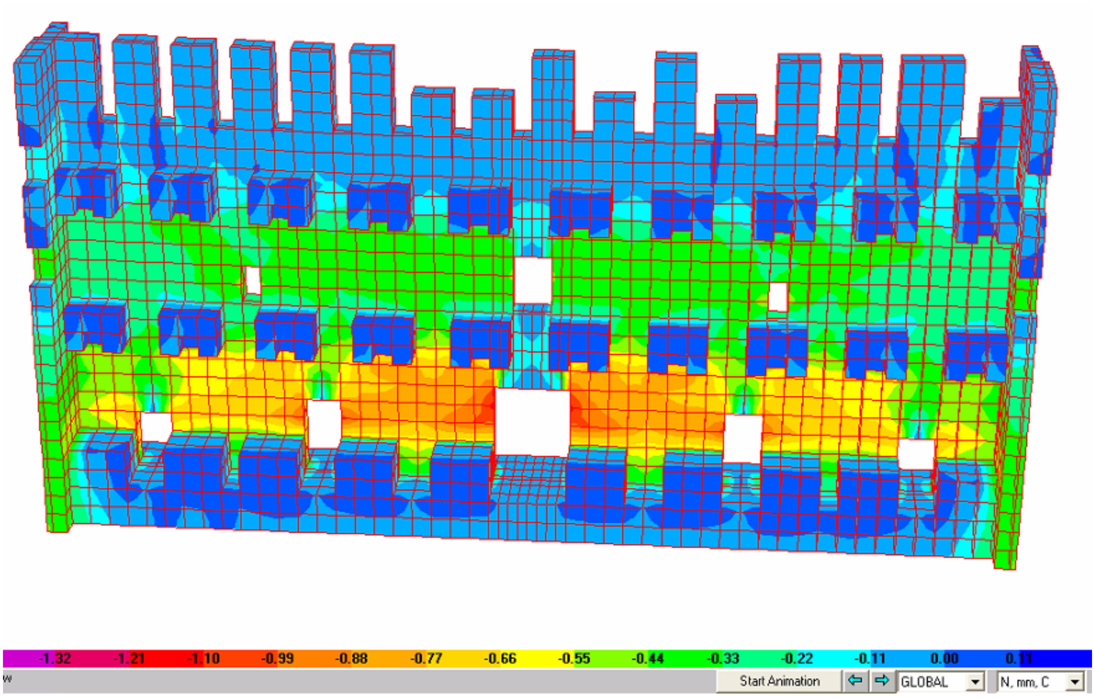


Figure 7.6 EQ (dir-U1) + DL, stage wall, front view

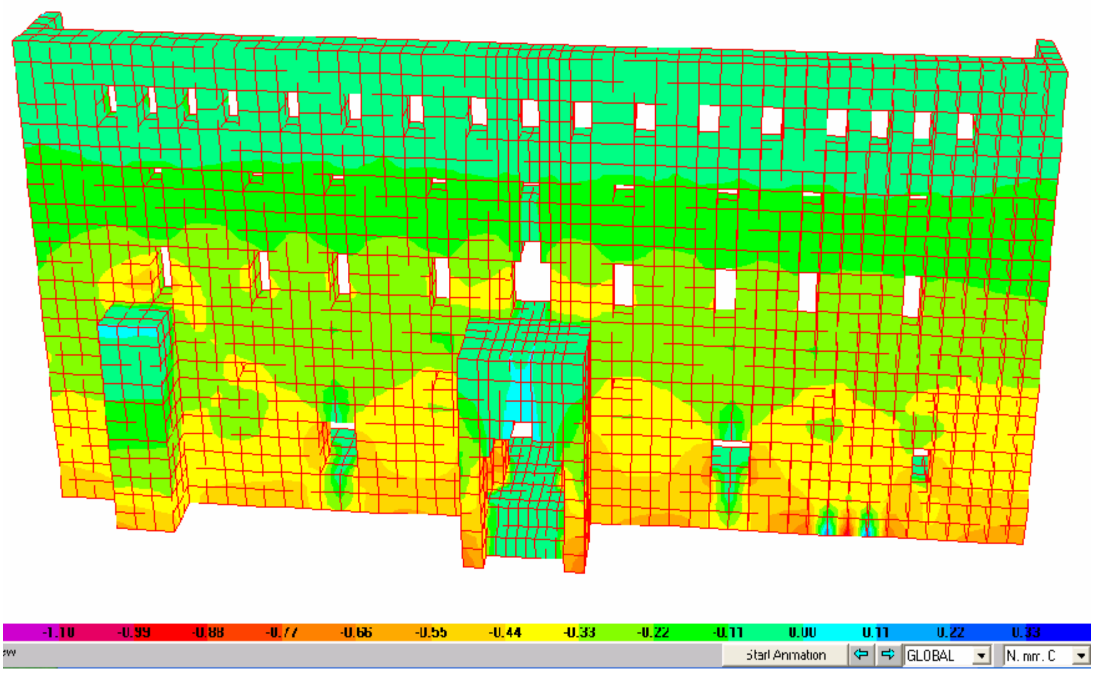


Figure 7.7 EQ (dir-U3) + DL, exterior wall, front view

1. The maximum compressive and tensile stresses are calculated and plotted considering both of the earthquake forces in U2 direction and dead weight of the theatre (Figures from 7.8 through 7.23). The directions for stress contours are also shown on the right bottom corner of the figures.

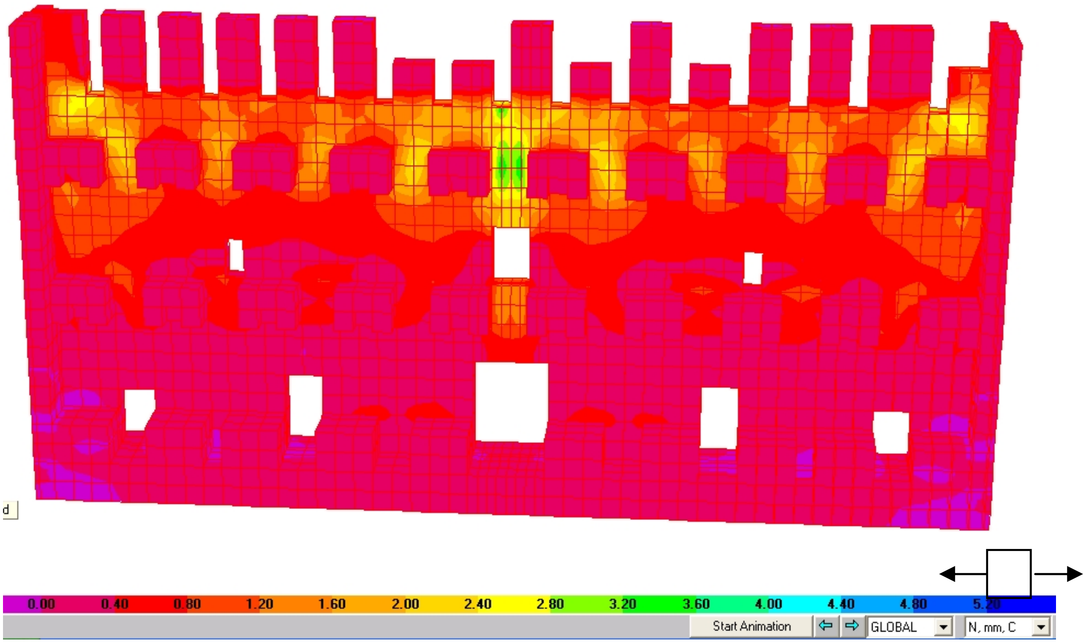


Figure 7.8 EQ (dir-U2) + DL, stage wall, front view, tensile stresses - 1

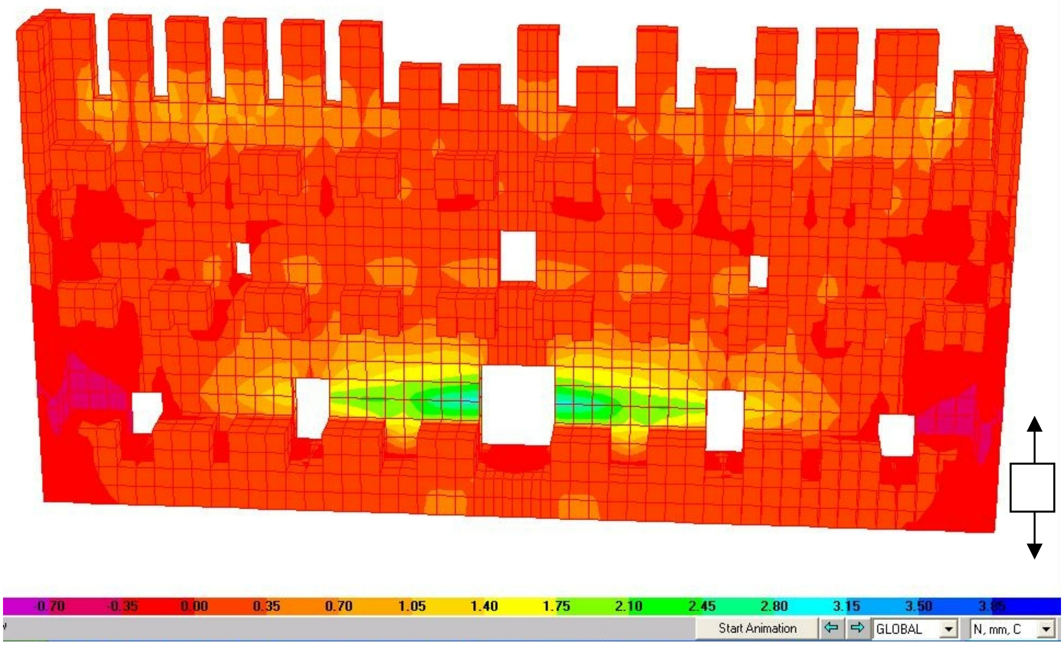


Figure 7.9 EQ (dir-U2) + DL, stage wall, front view, tensile stresses - 2

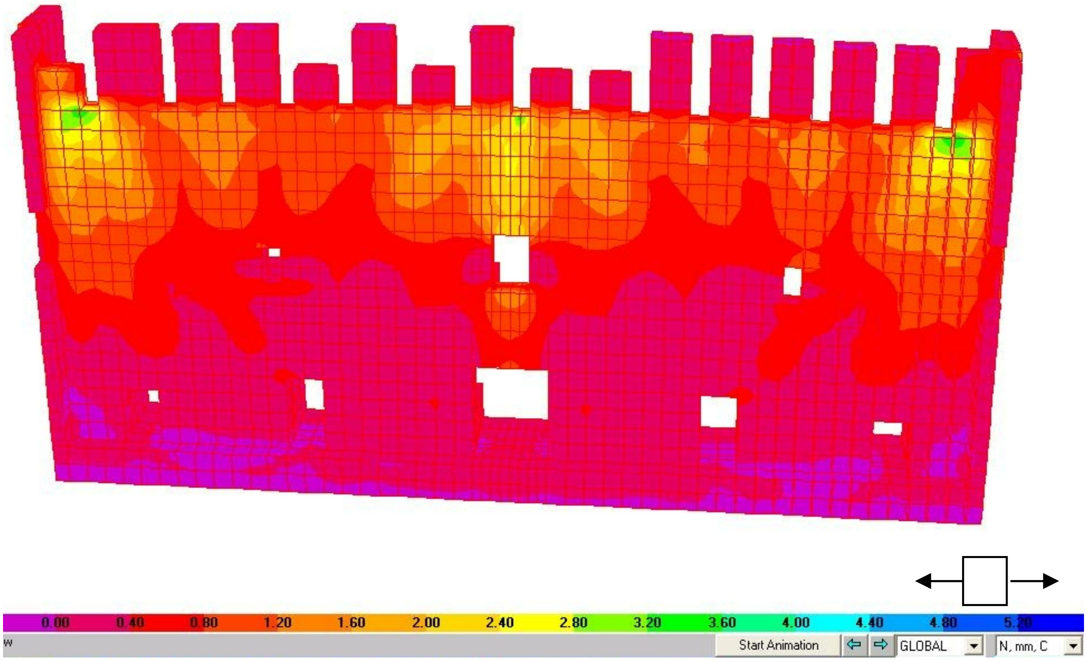


Figure 7.10 EQ (dir-U2) + DL, stage wall, rear view, tensile stresses - 1

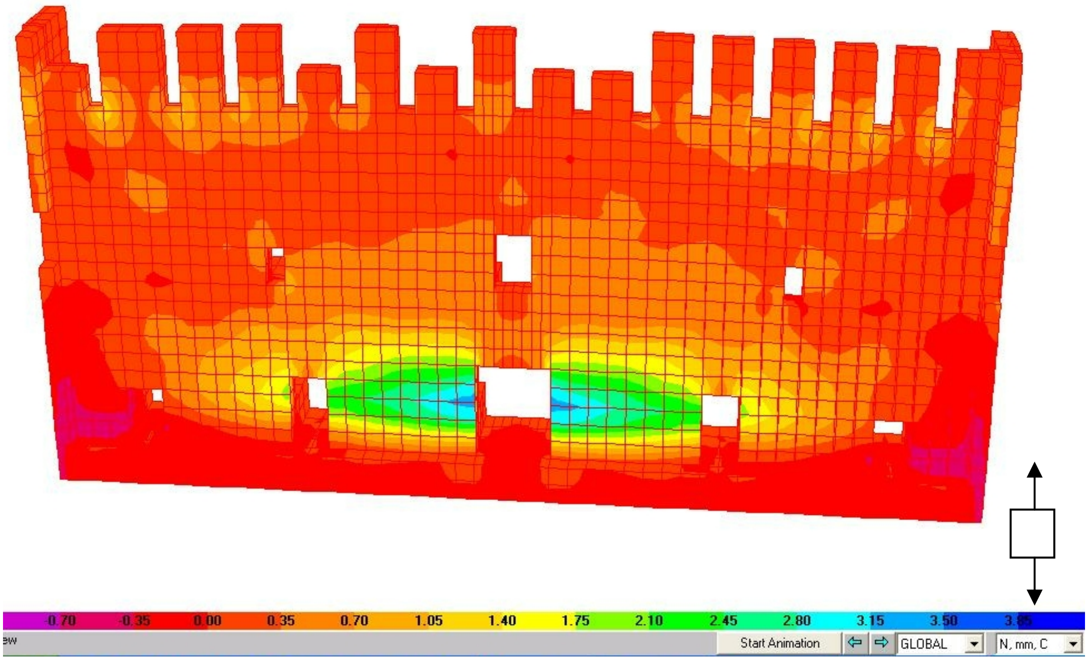


Figure 7.11 EQ (dir-U2) + DL, stage wall, rear view, tensile stresses - 2

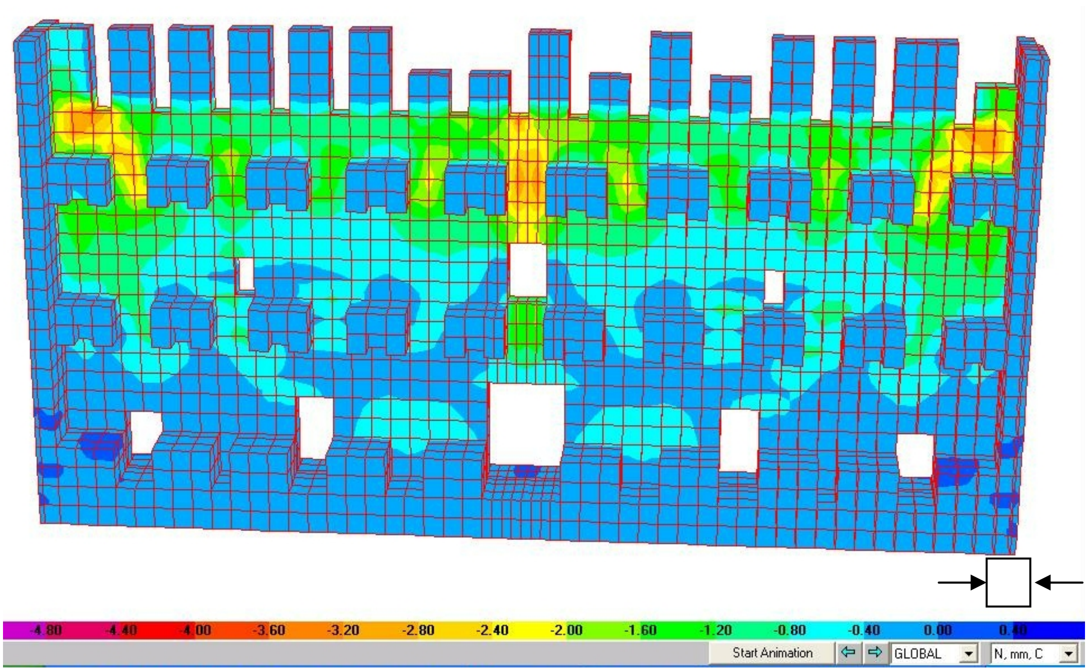


Figure 7.12 EQ (dir-U2) + DL, stage wall, front view, compressive stresses - 1

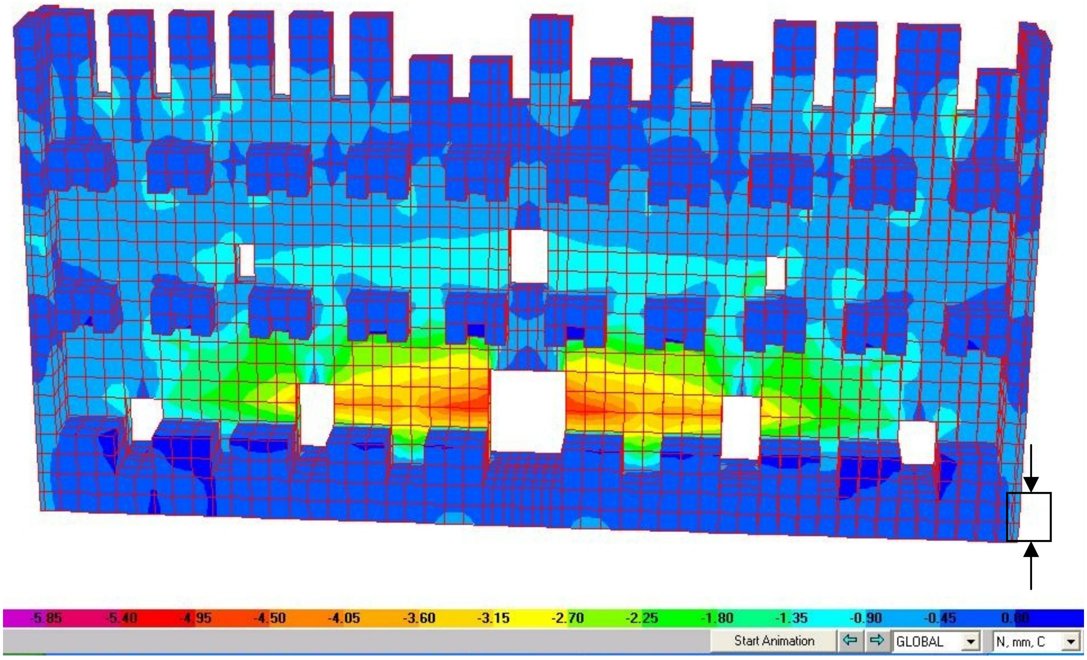


Figure 7.13 EQ (dir-U2) + DL, stage wall, front view, compressive stresses - 2

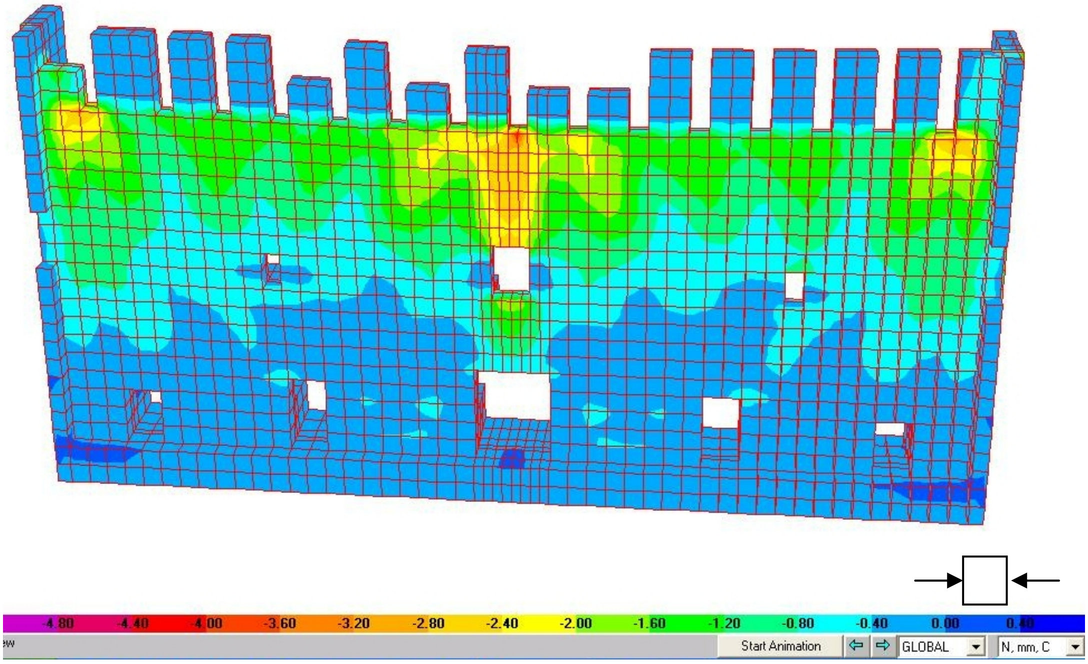


Figure 7.14 EQ (dir-U2) + DL, stage wall, rear view, compressive stresses - 1

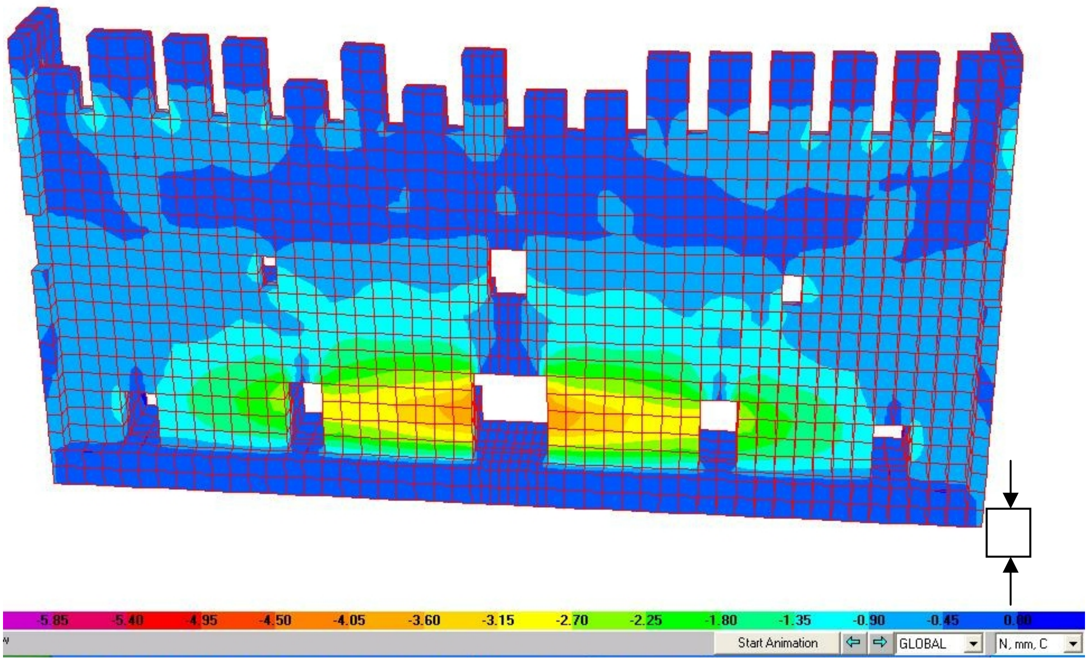


Figure 7.15 EQ (dir-U2) + DL, stage wall, rear view, compressive stresses - 2

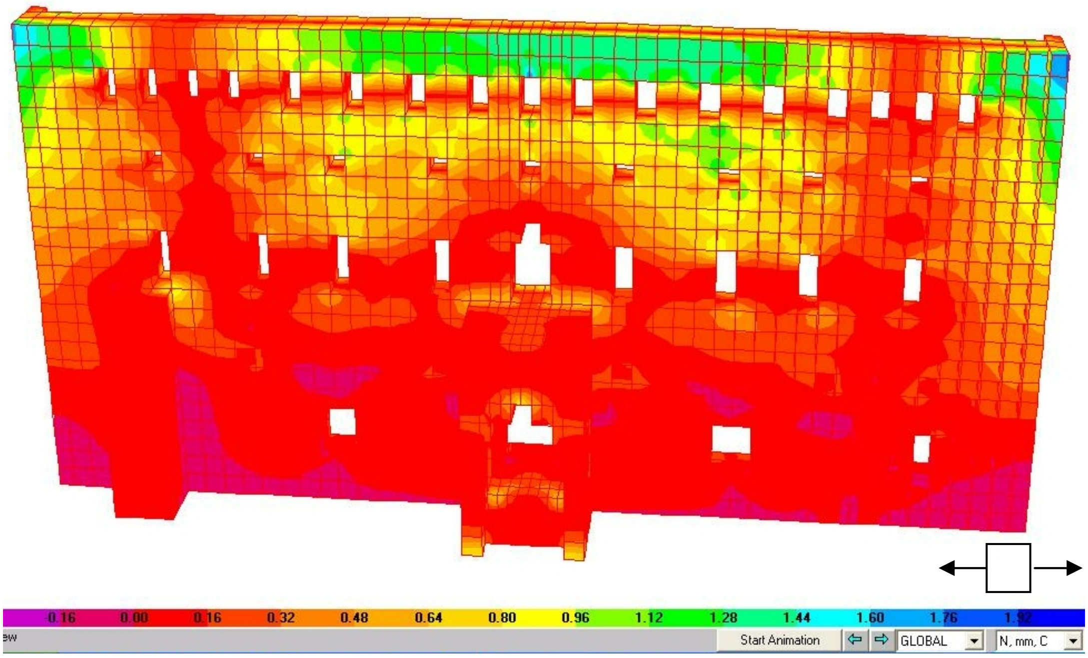


Figure 7.16 EQ (dir-U2) + DL, exterior wall, front view, tensile stresses-1

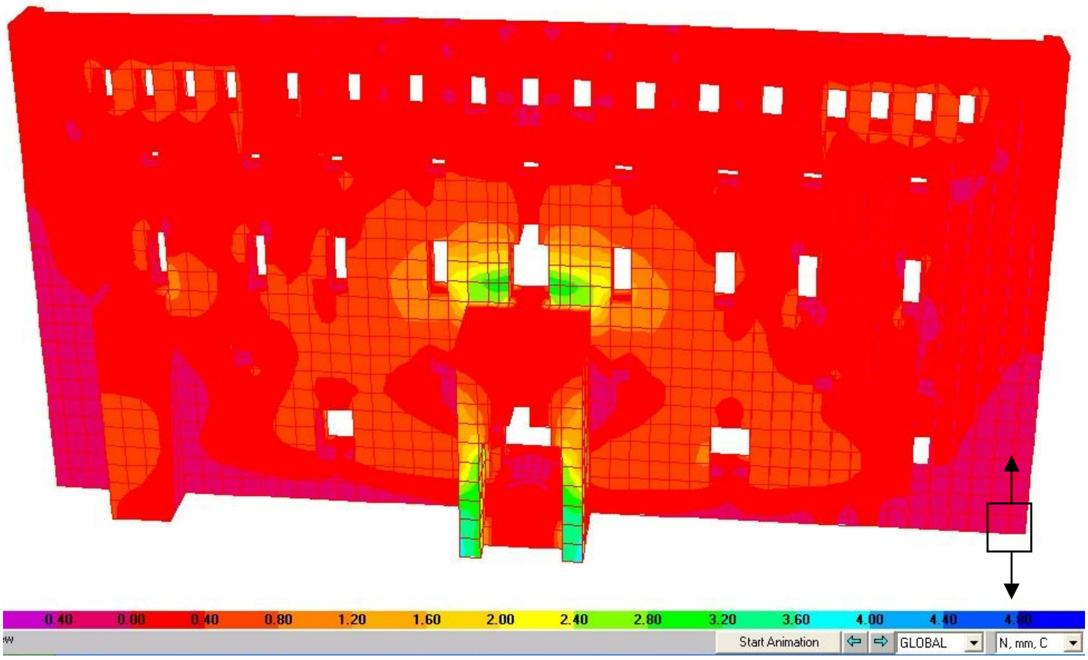


Figure 7.17 EQ (dir-U2) + DL, exterior wall, front view, tensile stresses-2

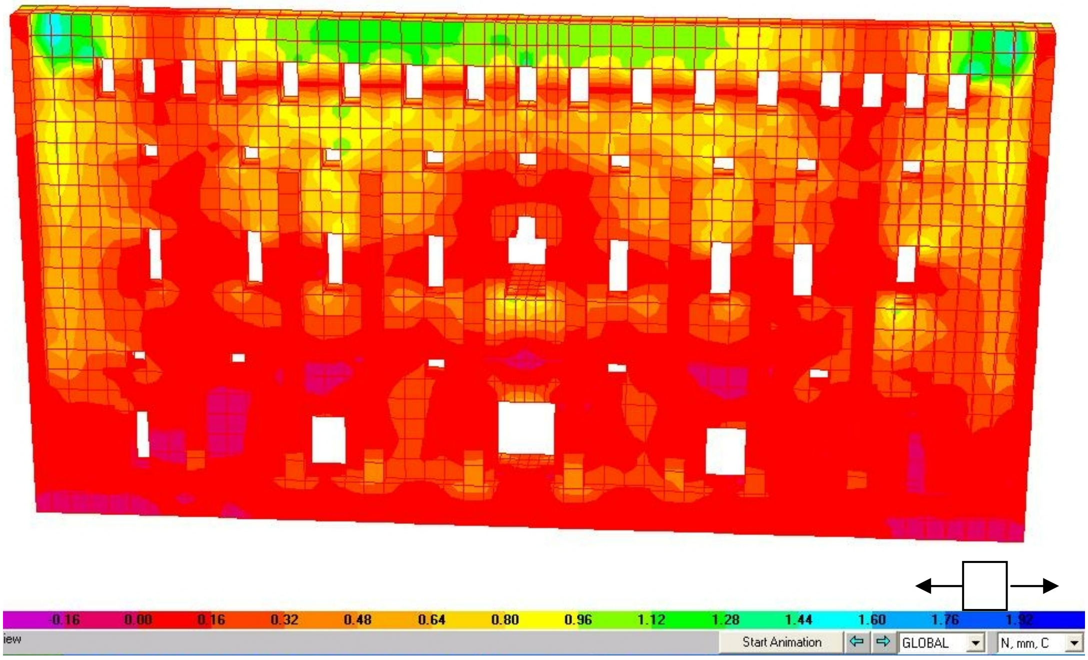


Figure 7.18 EQ (dir-U2) + DL, exterior wall, rear view, tensile stresses-1

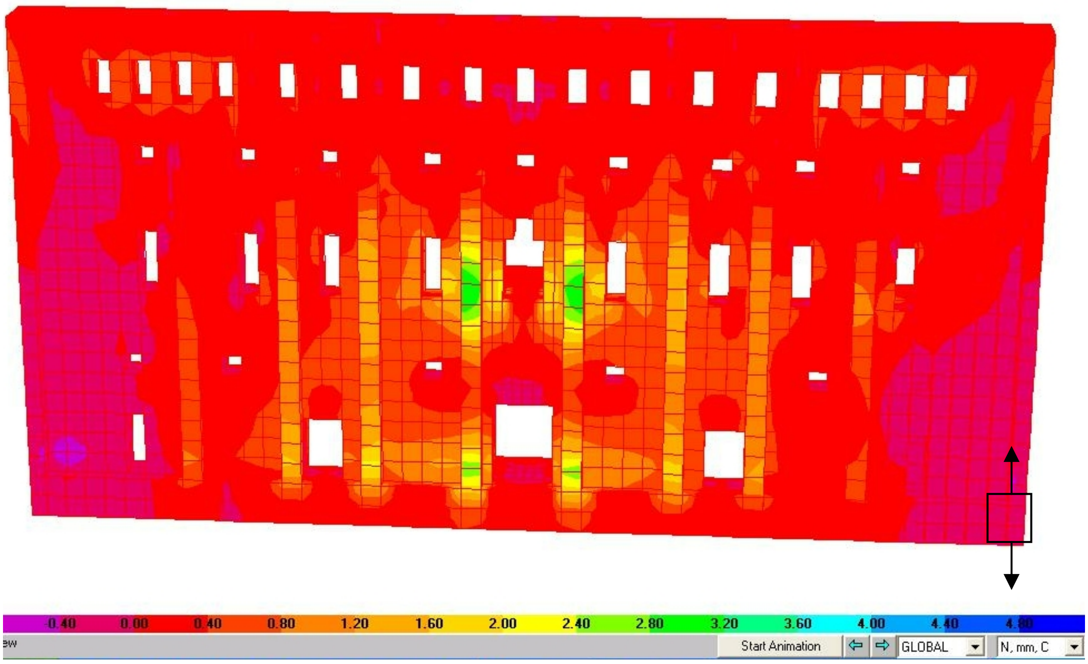


Figure 7.19 EQ (dir-U2) + DL, exterior wall, rear view, tensile stresses-2

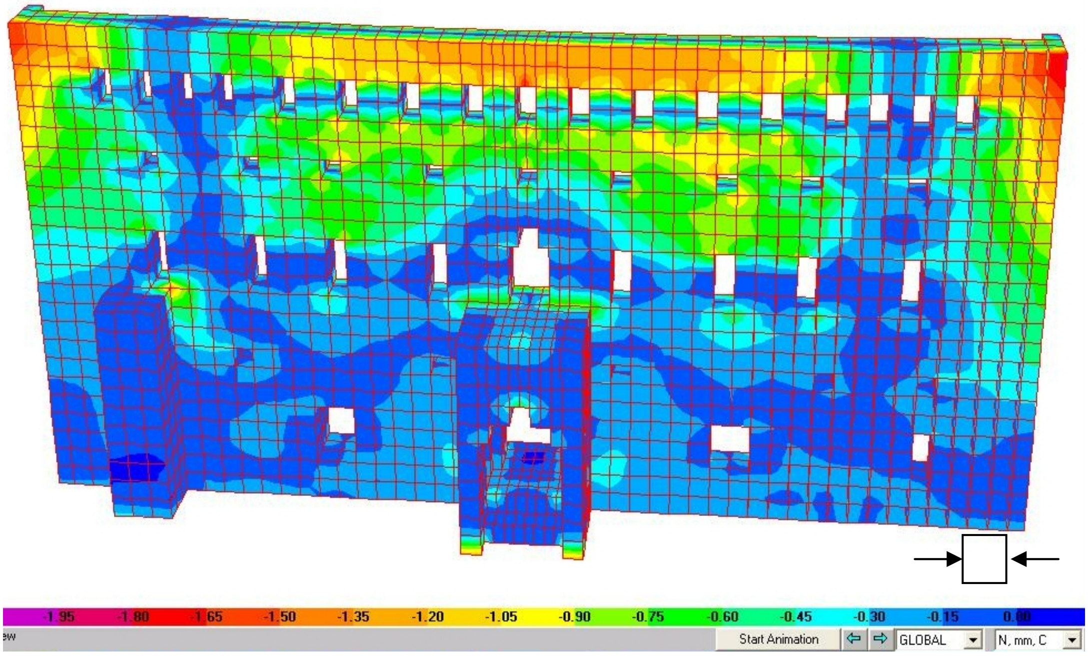


Figure 7.20 EQ (dir-U2) + DL, exterior wall, front view, compressive stresses - 1

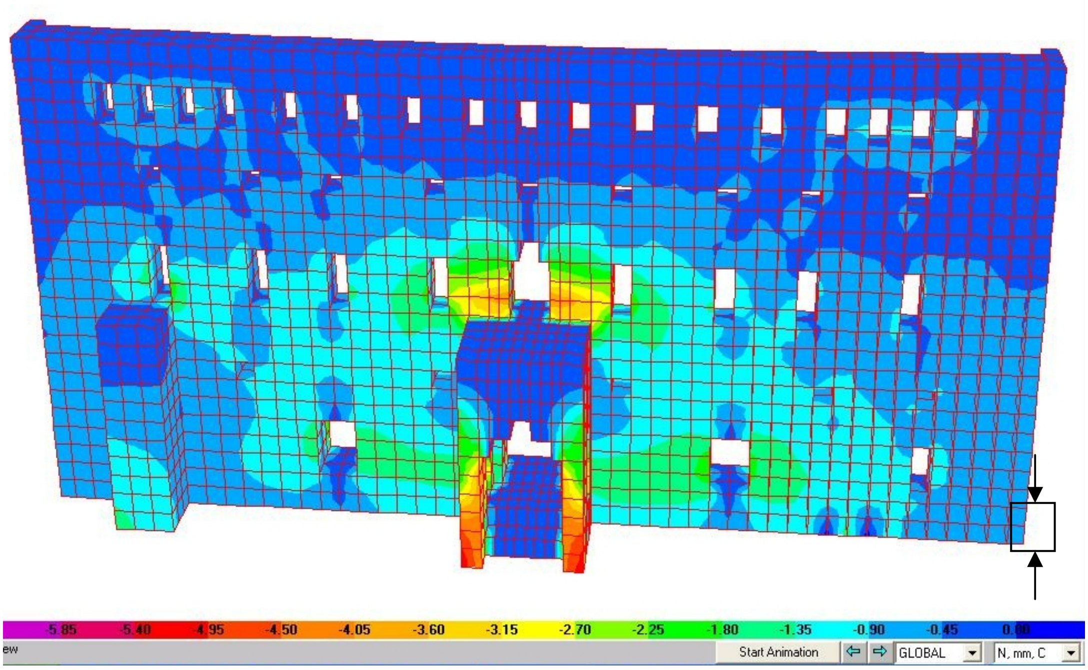


Figure 7.21 EQ (dir-U2) + DL, exterior wall, front view, compressive stresses - 2

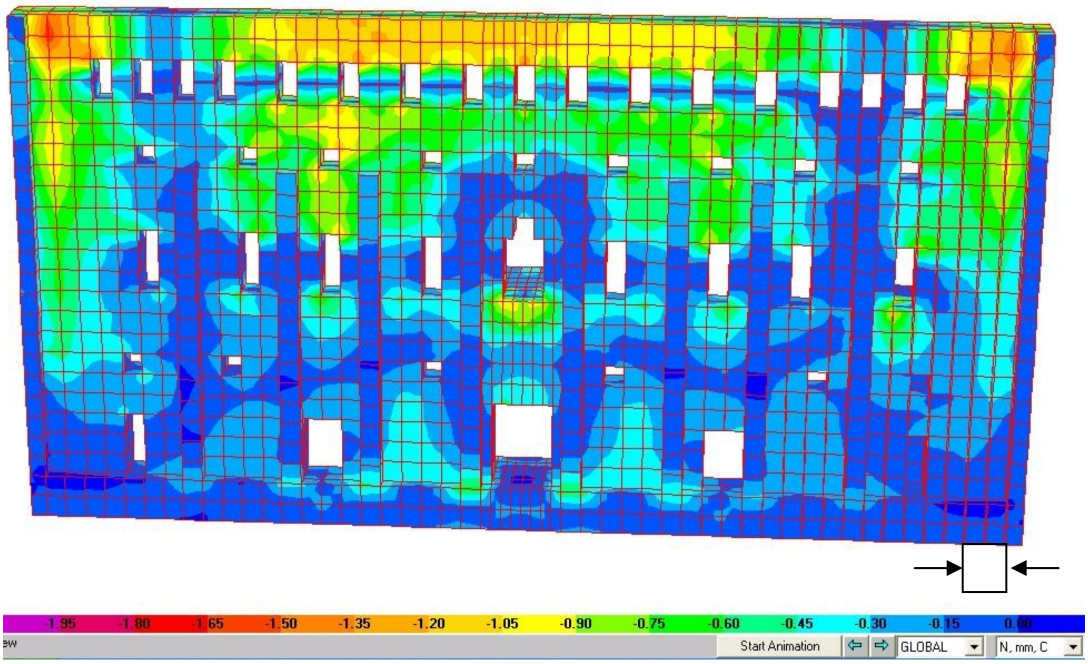


Figure 7.22 EQ (dir-U2) + DL, exterior wall, rear view, compressive stresses – 1

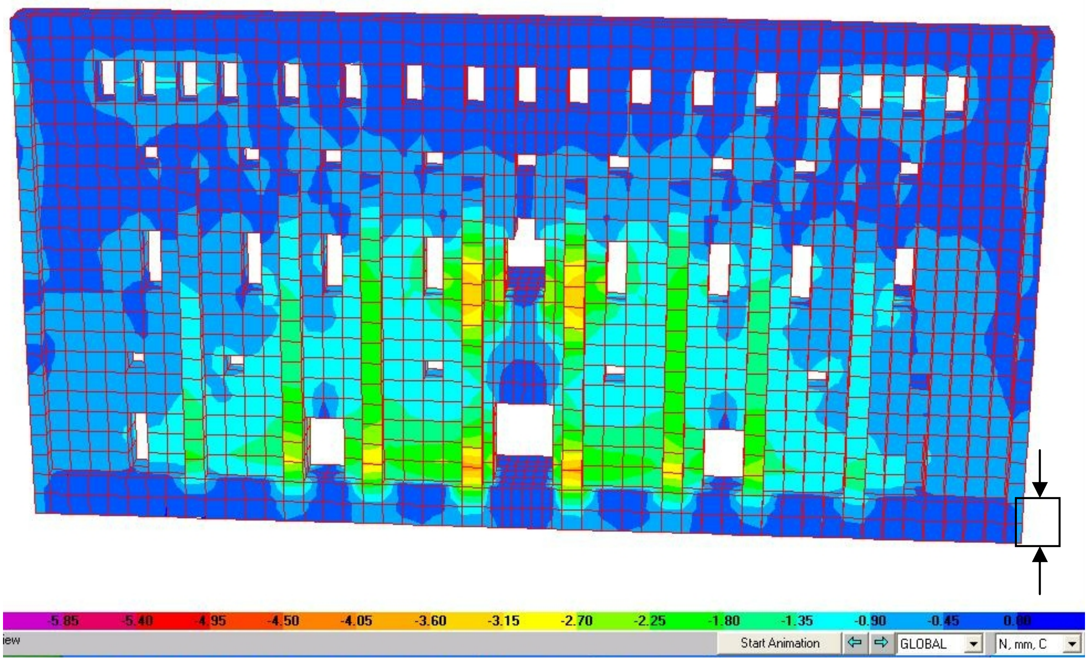


Figure 7.23 EQ (dir-U2) + DL, exterior wall, rear view, compressive stresses - 2

According to the analysis results, compressive stresses are found negative. Therefore, the maximum compressive stresses would be equal to the absolute minimum stresses. The maximum compressive stress on the stage wall is on the front view (Figure 7.13, F4 locations) and it is around 3.5 MPa. The maximum tensile stress on the stage wall is 2.0 MPa at the locations of F3, R1, R2, R3 & R4 (Figures 7.8, 7.9, 7.10 & 7.11).

The maximum compressive stress on the exterior wall is 3.5 MPa at the location of F9 (see Figure 7.21). Also, the maximum tensile stress on the exterior wall is 2.2 MPa and it is also located at F9 (see Figure 7.17).

The maximum horizontal deflections in perpendicular direction to the walls (in U2 direction) are at the top of the stage wall and are equal to 24.4mm for dead weight loading only, and 253.5mm for earthquake loading only. Therefore, the total deflection of the wall is 24.4mm + 253.5mm = 277.9mm at the top-middle of the stage wall using theory of superposition.

Also, the maximum horizontal deflections (in U2 direction) at the top part of the exterior wall are 1.3mm for dead weight loading, 128mm for earthquake loading in U2 direction. So, the total deflection is 1.3mm + 128mm = 129.3mm at the top-middle of the exterior wall.

The maximum compressive & tensile stresses and the deflections at the top of the stage & exterior walls are summarized on Tables 7.2 & 7.3.

Table 7.2 Maximum compressive and tensile stresses

Maximum Stresses		Due to Combination of Dead Weight and Earthquake forces *	Demand/Capacity (%)	Location on the Stage Wall
STAGE WALL	Compressive Stress on front view (MPa)	**-5.0**	42	F4
	Tensile stress on front view (MPa)	**3.0**	246	F3
	Compressive Stress on rear view (MPa)	-3.7	31	R4
	Tensile stress on rear view (MPa)	**3.2**	262	R1, R2, R3, R4
EXTERIOR WALL	Compressive Stress on front view (MPa)	**-5.0**	42	F9
	Tensile stress on front view (MPa)	**3.6**	295	F9
	Compressive Stress on rear view (MPa)	-3.6	30	R8
	Tensile stress on rear view (MPa)	2.8	230	R8

Table 7.3 Maximum deflections at the top part of both stage & exterior walls

Location	Loading	Maximum Deflection (mm) *
Stage Wall (Top-Middle part)	Due to Dead Weight	24.4
	Due to Earthquake Loading	253.5
	Due to Combination of Dead Weight and Earthquake Loading	277.9
Exterior Wall (Top-Middle part)	Due to Dead Weight	1.3
	Due to Earthquake Loading	128
	Due to Combination of Dead Weight and Earthquake Loading	129.3

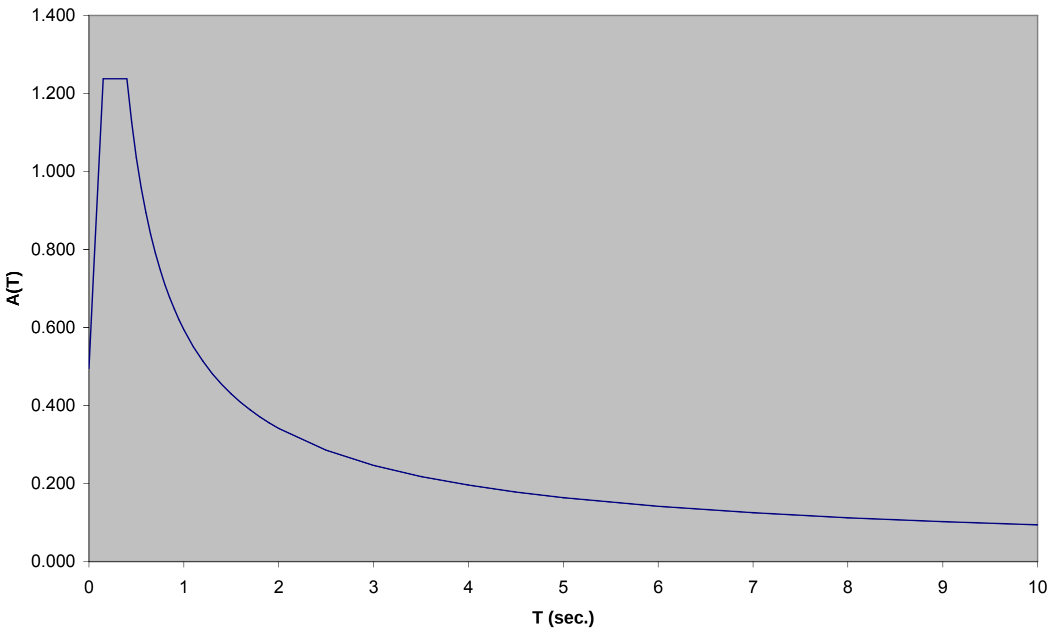
* For 5% damping, Zone 2, I=1, Return period of 475 years, R=1

7.4 DISCUSSION OF RESULTS

In the Turkish code of “Specification for Structures to be Built in Disaster Areas”, the response spectrum is given according to 5% damping. However, the damping ratio of Aspendos Theatre has been found as 0.8% after modal impact tests. Therefore, the effective ground acceleration coefficient should be increased with a factor of “1.65” (Chopra, 1995) to count for structural response for 0.8% damping. According to the actual damping of Aspendos Theatre, the response spectrum was revised as follows;

Effective ground acceleration coefficient, Ao = 0.30 * 1.65 = 0.495

The other parameters are kept constant. The revised response spectrum graph is shown in Graph 7.2.



Graph 7.2 Response Spectrum for damping of 0.8%

Because a linear analysis is performed, all of the stress and deflections are scaled using the modification factor of 1.65. The following stress and deflection values are obtained for 0.8% structural damping.

Table 7.4 Maximum compressive and tensile stresses (475 years)

Maximum Stresses		Due to Combination of Dead Weight and Earthquake forces *	Demand/Capacity (%)	Location on the Stage Wall
STAGE WALL	Compressive Stress on front view (MPa)	**-7.0**	58	F4
	Tensile stress on front view (MPa)	**5.0**	410	F3
	Compressive Stress on rear view (MPa)	-5.4	45	R4
	Tensile stress on rear view (MPa)	**5.5**	451	R1, R2, R3, R4
EXTERIOR WALL	Compressive Stress on front view (MPa)	**-7.2**	60	F9
	Tensile stress on front view (MPa)	**6.6**	541	F9
	Compressive Stress on rear view (MPa)	-5.7	48	R8
	Tensile stress on rear view (MPa)	5.0	410	R8

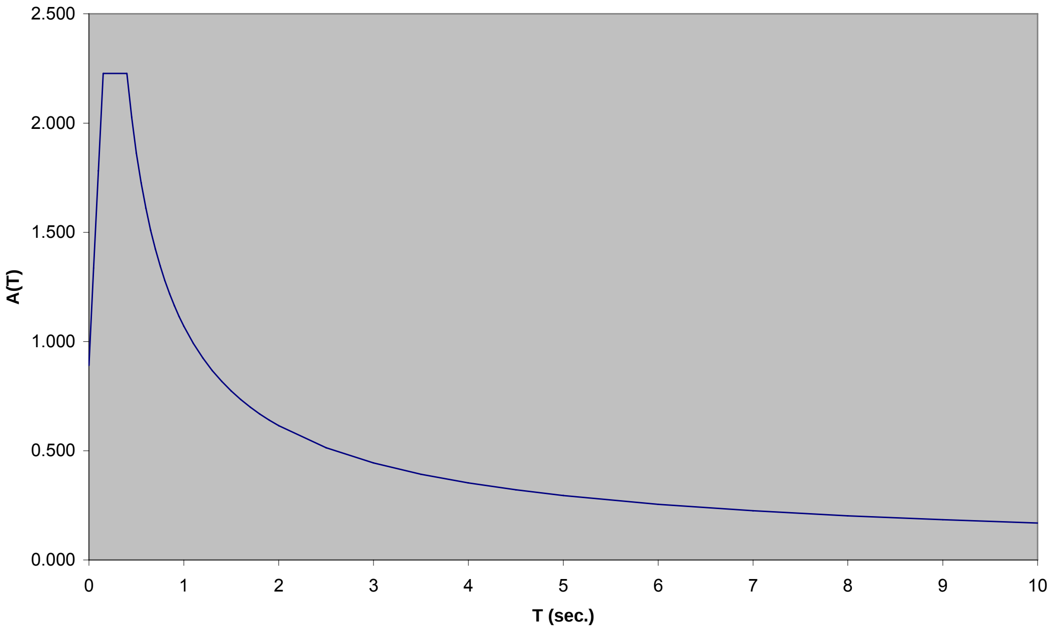
Table 7.5 Maximum deflections at the top part of both stage & exterior walls

Location	Loading	Maximum Deflection (mm) *
Stage Wall (Top-Middle part)	Due to Dead Weight	24.4
	Due to Earthquake Loading	418.3
	Due to Combination of Dead Weight and Earthquake Loading	442.7
Exterior Wall (Top-Middle part)	Due to Dead Weight	1.3
	Due to Earthquake Loading	211.2
	Due to Combination of Dead Weight and Earthquake Loading	212.5

* For 0.8% damping, Zone 2, I=1, Return period of 475 years, R=1

Also, in the Turkish code of “Specification for Structures to be Built in Disaster Areas”, the response spectrum is given according to 475 years of return period. The Aspendos Theatre has been also analyzed under 2500 years of return period. For this purpose, the effective ground acceleration coefficient has been increased with a factor of “1.8” for the difference of return period between 475 and 2500 years. The revised response spectrum and the analysis results are shown on Graph 7.3 and Table 7.4, respectively.

Effective ground acceleration coefficient, Ao = 0.495 * 1.8 = 0.891



Graph 7.3 Response Spectrum for 2500-years of return period

Table 7.6 Maximum compressive and tensile stresses (2500 years)

Maximum Stresses		Due to Combination of Dead Weight and Earthquake forces *	Demand/Capacity (%)	Location on the Stage Wall
STAGE WALL	Compressive Stress on front view (MPa)	**-11.0**	92	F4
	Tensile stress on front view (MPa)	**8.9**	730	F3
	Compressive Stress on rear view (MPa)	-9.6	80	R4
	Tensile stress on rear view (MPa)	**10.0**	820	R1, R2, R3, R4
EXTERIOR WALL	Compressive Stress on front view (MPa)	**-12.2**	102	F9
	Tensile stress on front view (MPa)	**12.0**	984	F9
	Compressive Stress on rear view (MPa)	-10.0	83	R8
	Tensile stress on rear view (MPa)	8.8	721	R8

Table 7.7 Maximum deflections at the top part of both stage & exterior walls

Location	Loading	Maximum Deflection (mm) *
Stage Wall (Top-Middle part)	Due to Dead Weight	24.4
	Due to Earthquake Loading	752.9
	Due to Combination of Dead Weight and Earthquake Loading	777.3
Exterior Wall (Top-Middle part)	Due to Dead Weight	1.3
	Due to Earthquake Loading	380.2
	Due to Combination of Dead Weight and Earthquake Loading	381.5

* For 0.8% damping, Zone 2, I=1, Return period of 2500 years, R=1

An equation to compute EQ load cracking deflection was derived using simple cantilever approach as shown below;

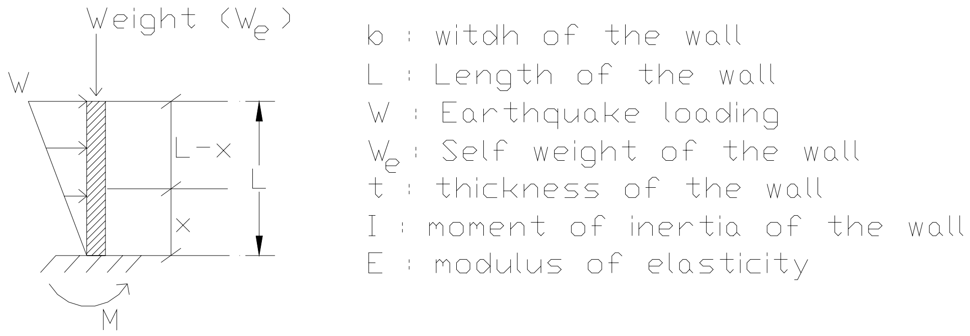


Figure 7.24 Cantilever wall earthquake and self weight loading on a cantilever wall

Fixed end moment (M) and critical moment (M_{cr}) are calculated as follows;

$$M = (W \times b \times L/2) \times (2 \times L/3) = W \times b \times L^2/3 \qquad \text{(Equation 7.4)}$$

$$M_{cr} = \sigma_{cr} \times (b \times t^2/6) \qquad \text{(Equation 7.5)}$$

$\sigma_{tension}$ is calculated as follows;

$$\sigma_{tension} = (M \times y/I) \qquad \text{(Equation 7.6)}$$

When Equation 7.4 is substituted in Equation 7.6;

$$\sigma_{tension} = (W \times b \times t \times L^2)/(6 \times I) \qquad \text{(Equation 7.7)}$$

and W is calculated as;

$$W = (\sigma_{tension} \times I \times 6)/(b \times L^2 \times t) \qquad \text{(Equation 7.8)}$$

Moment at a x distance from the fixed end is calculated as;

$$M(x) = ((L-x)^2 \times W \times x/2 + (L-x)^3 \times W/3)/L \quad \text{(Equation 7.9)}$$

From Equation 7.9, the deflection (δ) is obtained at x = L as follows;

$$\delta = (11/120) \times L^4 \times W/(E \times I) \quad \text{(Equation 7.10)}$$

δ is obtained by substituting W (from Equation 7.8) into Equation 7.10;

$$\delta = 0.55 \times (L^2/t) \times (\sigma_{tension}/E) \quad \text{(Equation 7.11)}$$

δ_{cr} is obtained for the stage wall of the Aspendos Theatre (L = 20m, t =1.2m, $\sigma_{tension}$ = 1.2 Mpa, E = 2350 Mpa) as 0.094m.

However, when the weight of the cantilever wall is considered, $\sigma_{tension}$ should be revised as follows;

$$\sigma_{tension2} = (\sigma_{tension2}) + (W_e / b \times t) \quad \text{(Equation 7.12)}$$

When $W_e = (b \times t \times L \times \rho)$ is substituted in Equation 7.12;

$$\sigma_{tension2} = (\sigma_{tension}) + (b \times t \times L \times \rho)/(b \times t) \quad \text{(Equation 7.13)}$$

where ρ is the unit weight of the cantilever wall material.

δ is obtained by substituting $\sigma_{tension2}$ (from Equation 7.13) into Equation 7.11 as follows;

$$\delta = 0.55 \times (L^2/t) \times (\sigma_{tension} + L \times \rho)/E \quad \text{(Equation 7.14)}$$

δ_{cr} is obtained for the stage wall of the Aspendos Theatre (L = 20m, t =1.2m, $\sigma_{tension}$ = 1.2 MPa, E = 2350 MPa, ρ =2150kg/m^3) as 0.127m.

P-Δ effect of the vertical load (weight of the cantilever wall) in Equation 7.14 is not considered; therefore, δ_{cr} will be between 0.094m and 0.127m. As a conservative assumption, the critical lateral deflection is taken as the average of equation 7.14 and 7.11 by considering half of the wall weight. Therefore δ_{cr} is obtained as 0.110m. The modified equation for δ is shown in Equation 7.15.

$$\delta = 0.55 \times (L^2 / t) \times (\sigma_{tension} + L \times \rho / 2) / E \qquad \text{(Equation 7.15)}$$

The result of the Equation 7.15 obtained for the stage wall (0.11m) is found to be in the same range with the 3D FEM EQ analysis results when the maximum deflection is divided to the ratio of maximum tensile stress to the tensile capacity of the wall material (0.095m). The difference between 3D FEM and Equation 7.15 approaches can also be affected by lateral supports of the stage wall in 3D FEM. Therefore, Equation 7.15 can be used as an approximation for EQ deformation capacity of any laterally unsupported wall.

CHAPTER 8

DISCUSSION OF SOUND & EQ INDUCED LOADING ANALYSIS RESULTS

8.1 COMPARISON OF STRESSES GENERATED BY EARTHQUAKE AND SOUND LOADS

The earthquake analysis results show that the maximum compressive stress on both the stage and exterior walls is 5.0 MPa (see Table 7.2). Because the compressive strength of the material is 12 MPa, the structure can be said to be safe under compressive stresses. The factor of safety (FS) in compression is about 12/5=2.4 and the demand is at about 42% of the capacity (Demand Capacity Ratio, DCR=42%).

The tensile strength of the material accepted for the theatre walls is 1.2 MPa. However, the earthquake analysis results show that the maximum tensile stresses of about 3.2 MPa on stage wall (see Table 7.2) and 3.6 MPa on the exterior wall are developing and exceeding the capacity (FS=1.2/3.6 = 0.33 < 1.0; DCR=300%). The tensile stresses exceeding the capacity of the material are at the critical locations as shown on Figures from 7.2 to 7.5 can cause crack formation and damage to the theatre. The level of the damage may vary because the maximum stresses are limited to a relatively small area at shown locations (see Figure from 7.6 to 7.23).

When the response spectrum is revised according to the damping of 0.8%, the factor of safety against compressive and tensile stresses have been found as 12/7.2 = 1.67 > 1.0 and 1.2/6.6 = 0.18 < 1.0, respectively. It is clear that the tensile stresses under 475 year return period and 0.8% damping ratio is critical and exceeding the tensile stress limit of the structure.

The Aspendos Theatre has also been analyzed under earthquake loads for 2500 years return period. The analysis results have shown that the factor of safety values against compressive and tensile stresses are 12/12.2 = 0.98 < 1.0 and 1.2/12 = 0.1 < 1.0, respectively. Exceedance of tensile capacity guarantees crack formation but does not always necessitate total collapse or heavy damage to a structure. However, a factor of safety too low (about 0.1) for tension shows that the structure will not remain linear and experience some level of damage. The crack formation will lead to redistribution of forces. Furthermore, factor of safety in compression is also low (about 0.98) and poses danger of compressive stresses exceeding the capacity, especially after redistribution. Based on the calibrated linear FE analysis results, the Aspendos Theatre is expected to experience heavy damage and partial collapse of the stage and exterior walls for a 2500 years return period earthquake.

The analysis results of the sound induced forces for 130dB combined pressure at resonant frequency show that the maximum compressive and tensile stresses are calculated as 2.7 MPa and 1.8 MPa, respectively. However, the compressive and tensile strengths are 12 MPa and 1.2 MPa, respectively for the material. The factor of safety in compression and tension are equal to 12/2.7=4.44 and 1.2/1.8=0.67, respectively.

The earthquake and sound analysis results show that both compressive and tensile stresses due to the earthquake forces are more than the stresses generated by the sound induced forces.

The factor of safety of the material under compression is above 0.98 when the theatre is under either earthquake loads or sound induced forces. However, the factor of safety values in tension are 0.18, 0.10 and 1.5 for earthquake with 475 years of return period, earthquake with 2500 years of return period, and 130 dB sound analysis, respectively.

During the inspection of the Aspendos Theatre, it has been seen that there are small cracks on the exterior and stage walls near the corner of windows. The source of these cracks can be thought as stress concentrations generated by previous small to medium scale earthquakes.

8.2. MAXIMUM PERMISSIBLE SOUND PRESSURE LEVEL AT RESONANCE STATE

Maximum levels of sound pressure are calculated for two cases; 1) Material stress capacities and 2) Earthquake induced stress levels. These two cases are explained under each heading below.

8.2.1. ALTERNATIVE 1: WITH RESPECT TO STRESS CAPACITIES

It is possible to determine the limit level of the sound pressure by comparing the compressive and tensile stresses generated by sound resonant loading with the material's tensile and compressive stress capacities.

Table 6.6 shows that the stresses resulting from;

1. Dead weight only,
2. Sound induced forces only,
3. Both dead weight and sound induced forces together.

As the dead weight of the structure will not change under any sound pressure level, the compressive stress due to the dead load will be constant. The maximum compressive or tensile stresses generated by the sound induced forces are 1.6 MPa and 1.5 MPa for front and rear views of the stage wall, respectively. In order to reach the compressive and tensile stress capacity levels of the material, the following equations can be used to find the multipliers of the sound induced force multipliers as m1, m2, m3 and m4;

1. Front view of the stage wall;

For compressive; 12 MPa = 1.1 MPa + m1 * 1.6 (Eq. 8.1)

For tensile; 1.2 MPa = -0.3 MPa + m2 * 1.1 (Eq. 8.2)

2. Rear view of the stage wall;

For compressive; 12 MPa = 0.7 MPa + m3 * 1.5 (Eq. 8.3)

For tensile; 1.2 MPa = 0.3 MPa + m4 * 1.5 (Eq. 8.4)

where;

m1 = 6.8125

m2 = 1.3636

m3 = 7.5333

m4 = 0.6000

The minimum multiplier for the sound induced forces is found to be m4 which is equal to 0.6. That is, if the sound induced forces are multiplied by 0.6, the tensile stress on the material (1.2 MPa) will reach the tensile capacity of the material.

As seen on Table 6.5, the sound pressure level which had has been used for the analysis is 63.25 N/m2 for the final Sound level of 130 dB (alternative 1.1, Table 6.3). When the pressure level is multiplied with m4, the limit sound pressure can be found as follows;

Plimit = 63.25 N/m2 * 0.6

Plimit = 37.95 N/m2

The sound level can be calculated for Plimit by using equation 3.1;

Sound Pressure Level (SPL) = 20 log (P/Po)

SPL = 20 log (37.95/2e-5)

SPL = 125 dB

Therefore, the limit value causing tensile failure for the sound level is 125 dB. That is, when the sound level reaches to 125 dB at resonant frequency, the sound pressure level will create tensile stress of 1.2 MPa on the stage wall at location R1 (Figure 6.8).

The exterior wall is kept outside the calculations since sound pressure is assumed from inside only. Nevertheless, the exterior wall is expected to have smaller stress at resonance due to its larger thickness and sloped ground conditions causing speakers to be located farter away from the wall.

8.2.2. ALTERNATIVE 2: BY COMPARING AGAINST 475 YEAR EQ ANALYSIS

Another alternative way to find the limit sound level is to compare the stresses of earthquake and sound analyses.

The theatre is considered to be standing after several possible big earthquakes during its 2200 years life time. Therefore, the theatre may be considered as safe against earthquakes with 475 years of return period. Therefore, the maximum compressive and tensile stresses of the material can be selected as the limit for the sound induced forces.

The maximum compressive and tensile stresses under earthquake loading are 7.0 MPa and 5.5 MPa at the locations of F4 and R4 of the stage wall (see Table 7.4). Also, the compressive and tensile stresses under sound induced forces are only 1.6 MPa and 1.5 MPa at the same locations of the stage wall (see Table 6.6).

In order to reach the compressive and tensile strength levels of the material under earthquake loading, the following equations can be used to find the multipliers of the sound induced forces;

For compressive; 7.0 MPa = 1.1 MPa + m5 * 1.6 (Eq. 8.1)

For tensile; 5.5 MPa = -1.1 MPa + m6 * 1.5 (Eq. 8.2)

where;

m5 = 3.6875

m6 = 4.4

The minimum multiplier for the sound induced forces is found to be m5 which is equal to 3.6875. That is, if the sound induced forces are multiplied by 3.6875, the compressive stress on the material (7.0 MPa) will match the compressive stress of the material under earthquake loading.

As seen on Table 6.5, the sound pressure level which had has been used for the analysis is 63.25 N/m2 for the final Sound level of 130 dB (alternative 1.1, Table 6.3). When the pressure level is multiplied with m5, the limit sound pressure can be found as follows;

Plimit = 63.25 N/m2 * 3.6875

Plimit = 233.2 N/m2

The sound level can be calculated for Plimit by using equation 3.1;

Sound Pressure Level (SPL) = 20 log (P/Po)

SPL = 20 log (233.2/2e-5)

SPL = 141.3 dB

So, the limit value for the sound level is 141.3 dB. That is, when the sound level reaches to 141.3 dB, the sound pressure level will create compressive stress of 7.0 MPa on the stage wall when it is in its resonance state.

However, as the compressive strength of the material is supposed to be higher than 7.0 MPa, Plimit value should be calculated by m6 multiplier and checked with its compressive strength as 1.1 + 4.4 * 1.6 = 8.14 MPa which is still much less than 12 MPa. So,

Plimit = 63.25 N/m2 * 4.4

Plimit = 278.3 N/m2

The sound level can be calculated for Plimit by using equation 3.1;

Sound Pressure Level (SPL) = 20 log (P/Po)

SPL = 20 log (278.3/2e-5)

SPL = 142.9 dB *

* 140 dB is equivalent to jet engine sound at 50 meter away from the source (See Table 3.2).

Therefore, the limit value for the sound level is 142.9 dB. That is, when the sound level reaches to 142.9 dB, the sound pressure level will create tensile stress of 5.5 MPa on the stage wall when it is in its resonance state (which is equivalent to 475 years EQ induced tensile stresses). 142.9 dB is the total effect of 10 speakers with even distribution in front of the stage wall. Each speaker should generate 122.9 dB to match the stresses generated by a 475 years return period earthquake. Even then, the theatre is not expected to collapse since it has been standing since 200 B.C.

CHAPTER 9

DAMAGE ASSESMENT OF PERIPHERAL BACKSIDE COLUMNS

9.1 AIM OF THE DAMAGE ASSESMENT

The theatre is surrounded by 58 columns at the backside of the stands (Photo 9.1). The top parts of these columns are connected with semi-circular arches which support a continuous slab in transverse and longitudinal directions (Photo 9.2). They provide walking area around the stands and a pleasing view of the Theatre (Photo 9.1 & 9.2).

Photo 9.1 Backside columns

Photo 9.2 Backside columns

In order to determine the structural condition of these columns, a visual inspection and non-destructive testing on these columns have been performed. Natural frequencies of all 58 columns have been measured and compared with each other. As the geometrical dimensions and the material properties used for these columns are almost the same, any difference between the natural frequencies of them may be an indication of damage. The columns have been numbered from #1 to #58, starting from the right side when standing at the stage and looking towards the stands and columns.

In addition, a computer model of the columns has been prepared and FEM modal analysis has been performed. The analysis results have been compared against the average value of the natural frequencies measured from the theatre columns.

9.2 NATURAL FREQUENCY MEASUREMENT OF THE COLUMNS

The measurements have been taken for all 58 columns. The procedure for taking the measurements of a column's natural frequency is as follows:

1. The accelerometer is located on the column by using a carpenter's tool as shown on photo 9.3.
2. The accelerometer is connected to the datand to the built-in c acquisition system.
3. The hammer with its elastic header is hit to the wall creating an impact pulse. The impact is repeted at least 7 times for each column.

Photo 9.3 Setup for a Measurement

4. The measured vibrations for each hit are recorded to the computer at 200Hz.

5. The measured vibrations are transferred to the frequency domain by using of Fast Fourier Transformation (See Chapter 3.1.6).

As an example, column #45's recorded accelerations and the frequency response function are shown on Figures 9.1 and Figure 9.2, respectively.

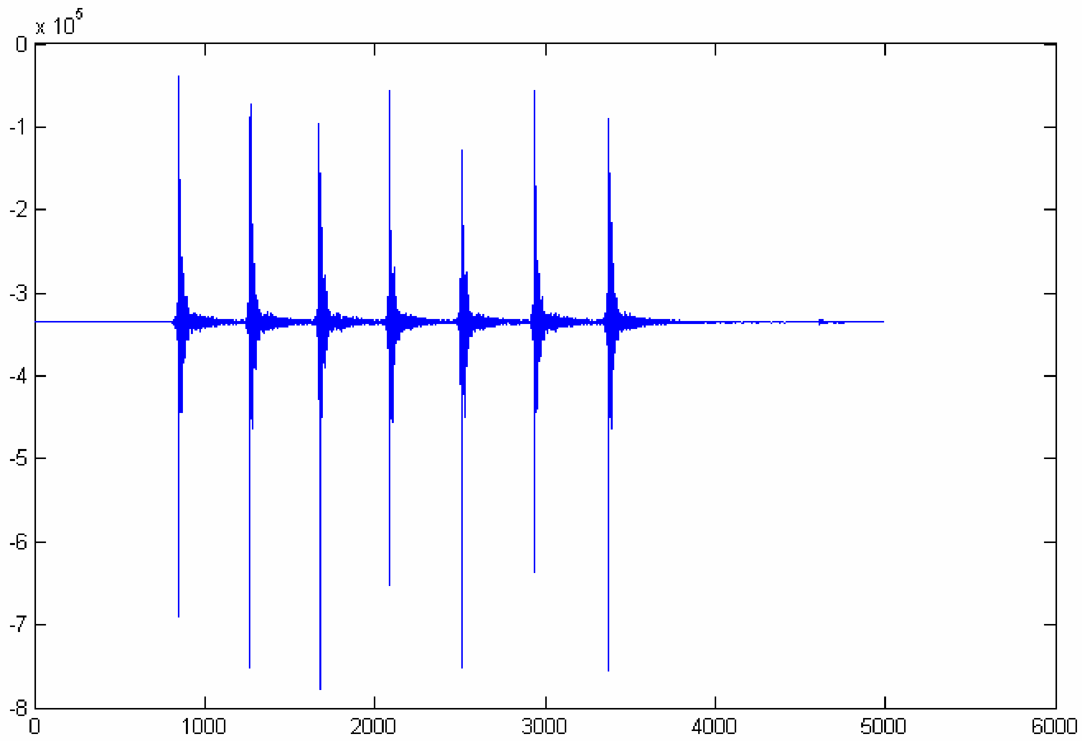


Figure 9.1 Acceleration record of column #45

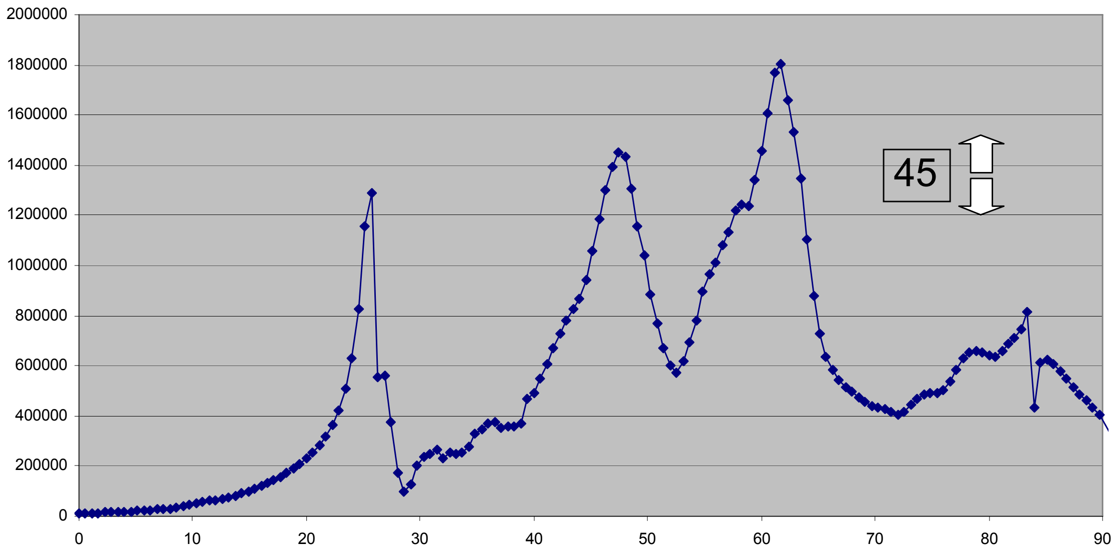
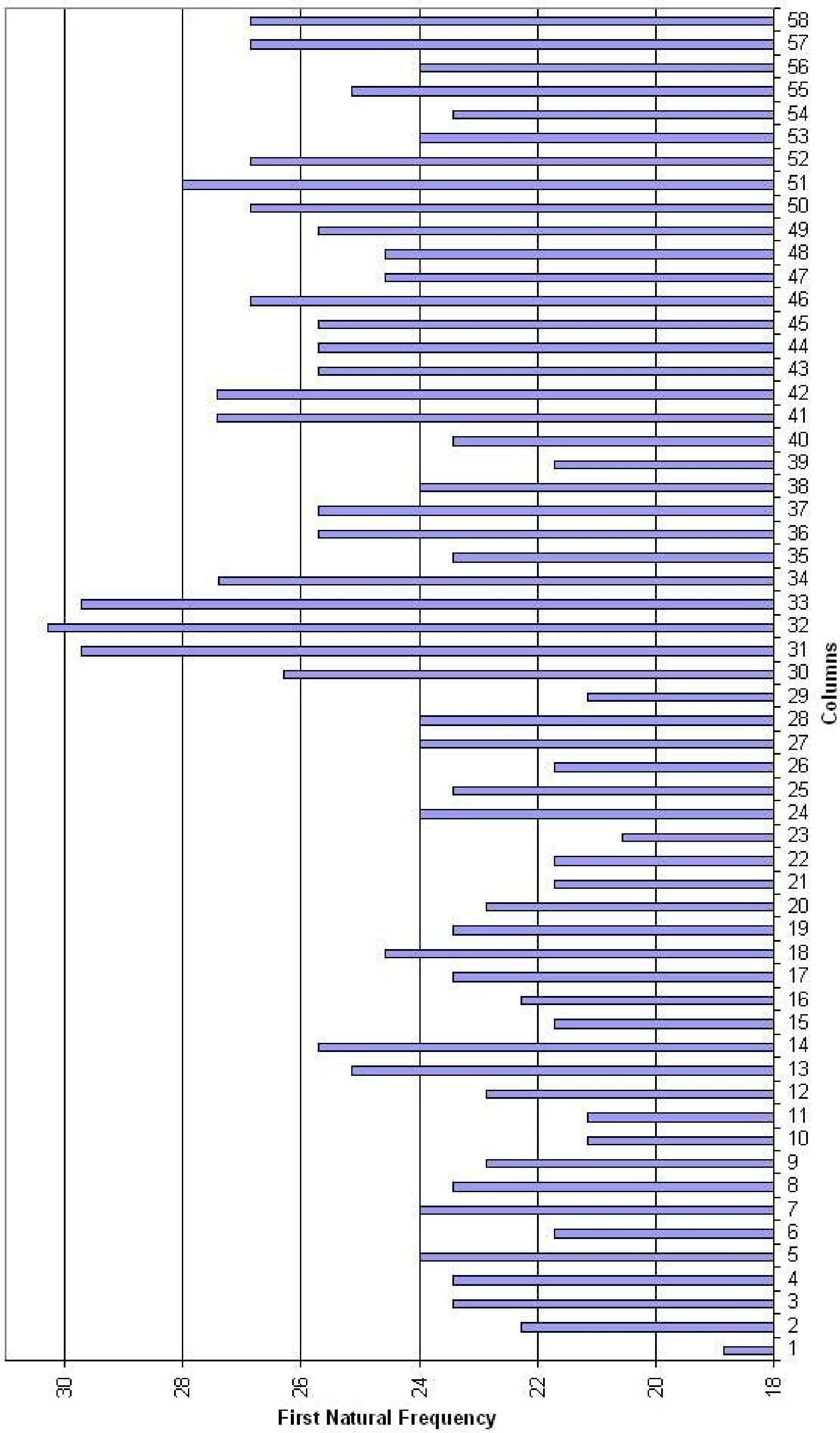


Figure 9.2 Frequency function Column #45

The same procedure has been performed for all 58 columns. The first frequencies of the columns are given on Table 9.1 and on Graph 9.1.

Table 9.1 First natural frequencies of columns

Column #	First natural Frequency	Column #	First natural Frequency
1	18,9	**30**	26,3
2	22,3	**31**	29,7
3	23,4	**32**	30,3
4	23,4	**33**	29,7
5	24,0	**34**	27,4
6	21,7	**35**	23,4
7	24,0	**36**	25,7
8	23,4	**37**	25,7
9	22,9	**38**	24,0
10	21,1	**39**	21,7
11	21,1	**40**	23,4
12	22,9	**41**	27,4
13	25,1	**42**	27,4
14	25,7	**43**	25,7
15	21,7	**44**	25,7
16	22,3	**45**	25,7
17	23,4	**46**	26,9
18	24,6	**47**	24,6
19	23,4	**48**	24,6
20	22,9	**49**	25,7
21	21,7	**50**	26,9
22	21,7	**51**	28,0
23	20,6	**52**	26,9
24	22,3	**53**	24,0
25	21,1	**54**	23,4
26	21,7	**55**	25,1
27	24,0	**56**	24,0
28	24,0	**57**	26,9
29	21,1	**58**	26,9



Graph 9.1 First natural frequencies of columns

9.3 VISUAL INSPECTION & CONDITION EVALUATION OF THE COLUMNS

The 58 columns have been inspected visually. During the inspection, the cracks, the demolished parts etc. have been noted and a grading assigned to each part of the structure. The inspection methodology and the grading system have been described below:

1. The columns: Two faces of each column has been checked and graded. The grading has been based on 5 point scale; 1 being worst and 5 being best condition. Photo 9.4 shows two examples for column #40 and #33 with grades of 5 and 2, respectively. The average grade has been calculated for columns.

Photo 9.4 Column #40 and column #33

2. Arches over the walkway (perpendicular to the stands): Aches have been checked and graded. The grading has been based on 3 point scale; 1 being worst and 3 being best condition. Photo 9.5 shows the beam # 25 and # 31 with grades of 1 and 3, respectively.

Photo 9.5 Beam #25 and beam #31

3. Ceiling: The ceiling of the slab supported by arches have been checked and graded. The grading has been based on 3 point scale; 1 being worst and 3 being best condition. Photo 9.6 shows the top floors of Column # 31 and # 21 with grades of 3 and 2 points, respectively. The average value for the grade has been calculated.

Photo 9.6 Top floor for column #31 and beam #21

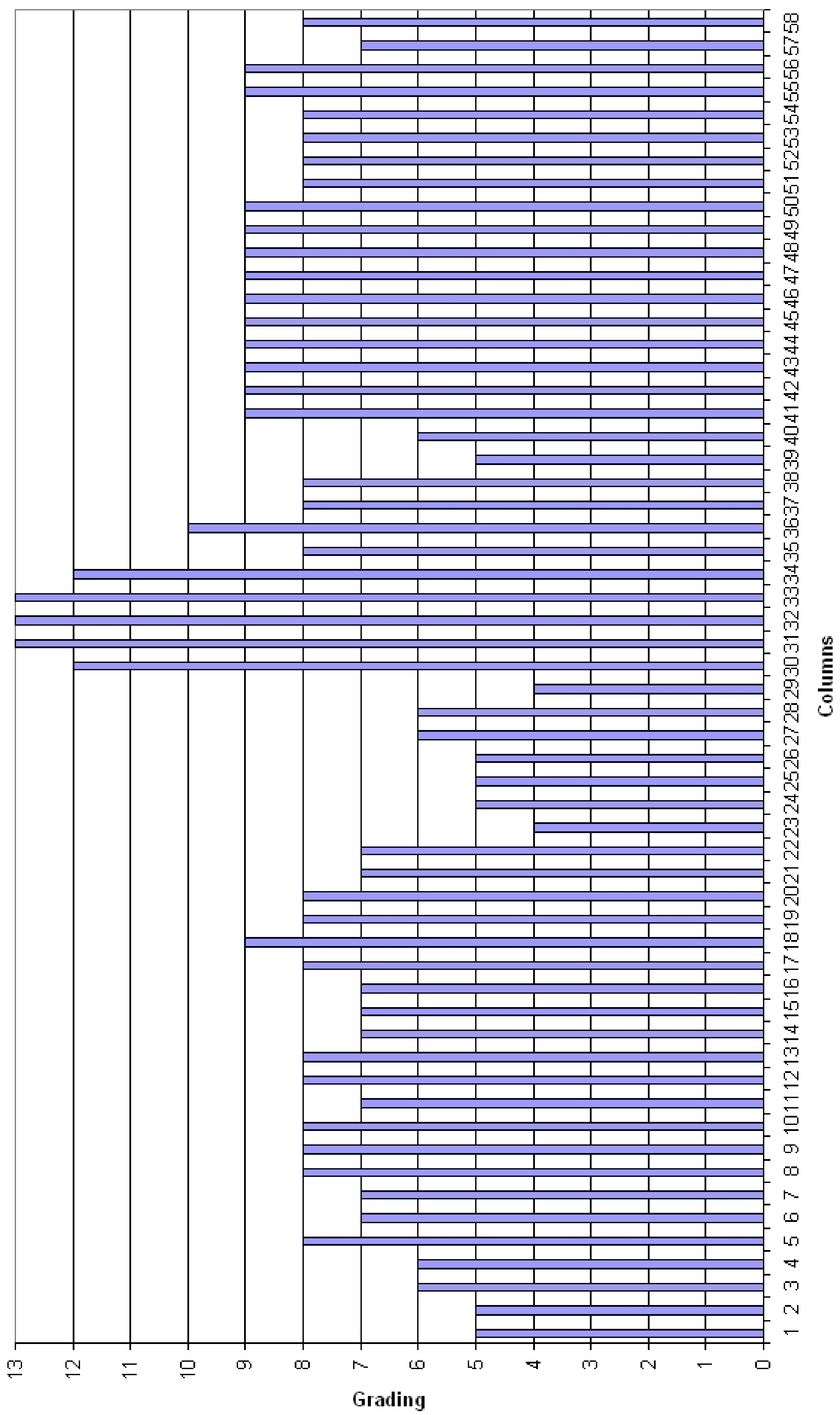
4. Façade of columns: The front parts of all columns have been checked and graded. The grading has been based on 2 point scale; 0 being worst and 2 being best condition. Photo 9.7 shows the front parts of Column # 33 and # 4 with grades of 2 and 0, respectively.

Photo 9.7 Front part for Column #33 & Column #4

After grading every four parts of all columns, a total grade has been calculated by summing all grades. The total grade ranges from 3 to 13 points as sum of above mentioned evaluation grade points. The Table 9.2, Table 9.3 and the Graph 9.2 show the grading of all parts of the column structures.

Table 9.2 Visual grading of the columns

Column ID No.	Visual Grading Points Assigned to Different Parts of the Backside Columns								
	Column Side 1	Column Side 2	COLUMN (/5) Average	ARCH (/3)	Ceiling (/3) (Right)	Ceiling (/3) (Left)	Ceiling (/3) Average	Façade (/2)	Column overall grade
1	3	3	3	1	1	1	1	0	5 / 13
2	2	2	2	1	1	2	2	0	5 / 13
3	2	2	2	2	2	2	2	0	6 / 13
4	2	2	2	2	2	2	2	0	6 / 13
5	4	4	4	2	2	1	2	0	8 / 13
6	4	4	4	1	1	2	2	0	7 / 13
7	4	4	4	1	2	2	2	0	7 / 13
8	3	4	4	2	2	2	2	0	8 / 13
9	3	3	3	2	2	2	2	1	8 / 13
10	3	4	4	1	2	1	2	1	8 / 13
11	4	4	4	2	1	1	1	0	7 / 13
12	4	4	4	2	1	2	2	0	8 / 13
13	4	4	4	2	2	2	2	0	8 / 13
14	4	4	4	1	2	1	2	0	7 / 13
15	3	3	3	1	1	2	2	1	7 / 13
16	4	4	4	1	2	2	2	0	7 / 13
17	4	4	4	2	2	2	2	0	8 / 13
18	3	4	4	2	2	2	2	1	9 / 13
19	4	4	4	2	2	2	2	0	8 / 13
20	4	4	4	2	2	2	2	0	8 / 13
21	4	4	4	1	2	2	2	0	7 / 13
22	3	3	3	2	2	2	2	0	7 / 13
23	1	1	1	1	2	2	2	0	4 / 13
24	1	2	2	1	2	2	2	0	5 / 13
25	2	2	2	1	2	2	2	0	5 / 13
26	2	2	2	1	2	2	2	0	5 / 13
27	2	1	2	2	2	2	2	0	6 / 13
28	2	2	2	2	2	2	2	0	6 / 13
29	1	1	1	1	2	2	2	0	4 / 13
30	5	5	5	3	2	2	2	2	12 / 13
31	5	5	5	3	2	3	3	2	13 / 13
32	5	5	5	3	3	3	3	2	13 / 13
33	5	5	5	3	3	2	3	2	13 / 13
34	5	5	5	3	2	2	2	2	12 / 13
35	3	3	3	1	2	2	2	2	8 / 13
36	4	3	4	2	2	2	2	2	10 / 13
37	4	4	4	2	2	2	2	0	8 / 13
38	3	4	4	2	2	2	2	0	8 / 13
39	1	1	1	2	2	2	2	0	5 / 13
40	2	1	2	2	2	2	2	0	6 / 13
41	4	4	4	2	2	2	2	1	9 / 13
42	4	4	4	2	2	2	2	1	9 / 13
43	4	4	4	2	2	2	2	1	9 / 13
44	4	4	4	2	2	2	2	1	9 / 13
45	4	4	4	2	2	2	2	1	9 / 13
46	4	4	4	2	2	2	2	1	9 / 13
47	4	4	4	2	2	2	2	1	9 / 13
48	4	3	4	2	2	2	2	1	9 / 13
49	4	4	4	2	2	2	2	1	9 / 13
50	4	4	4	2	2	2	2	1	9 / 13
51	4	4	4	2	2	2	2	0	8 / 13
52	4	4	4	2	2	2	2	0	8 / 13
53	4	4	4	2	2	2	2	0	8 / 13
54	4	4	4	2	2	2	2	0	8 / 13
55	4	4	4	2	2	2	2	1	9 / 13
56	4	4	4	2	2	2	2	1	9 / 13
57	4	4	4	1	2	2	2	0	7 / 13
58	4	4	4	2	2	2	2	0	8 / 13



Graph 9.2 Visual Grading of all columns

9.4 COMPUTER MODEL OF THE COLUMNS

The analytical model of the backside columns have been prepared by SAP2000 program (Figure 9.3). For the modeling purpose of the columns, three column portions have been selected instead of modeling and analyzing the entire set of columns. Half of the columns at the edges are modeled and pin supports have been used. The columns at the edges and the base connections have been modeled as fixed to the surfaces which they are connected. For this purpose, multi pin-connections have been used (see Figure 9.4).

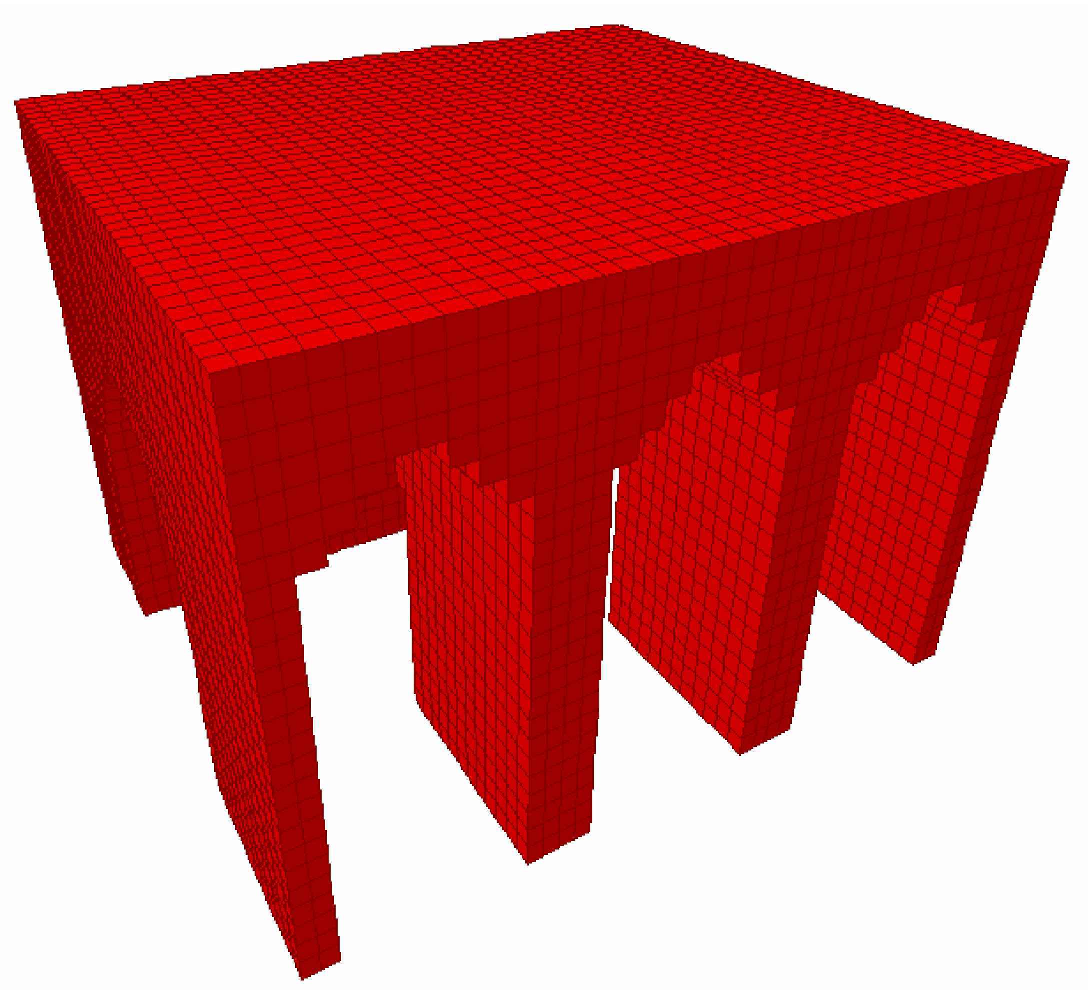


Figure 9.3 General view of the model

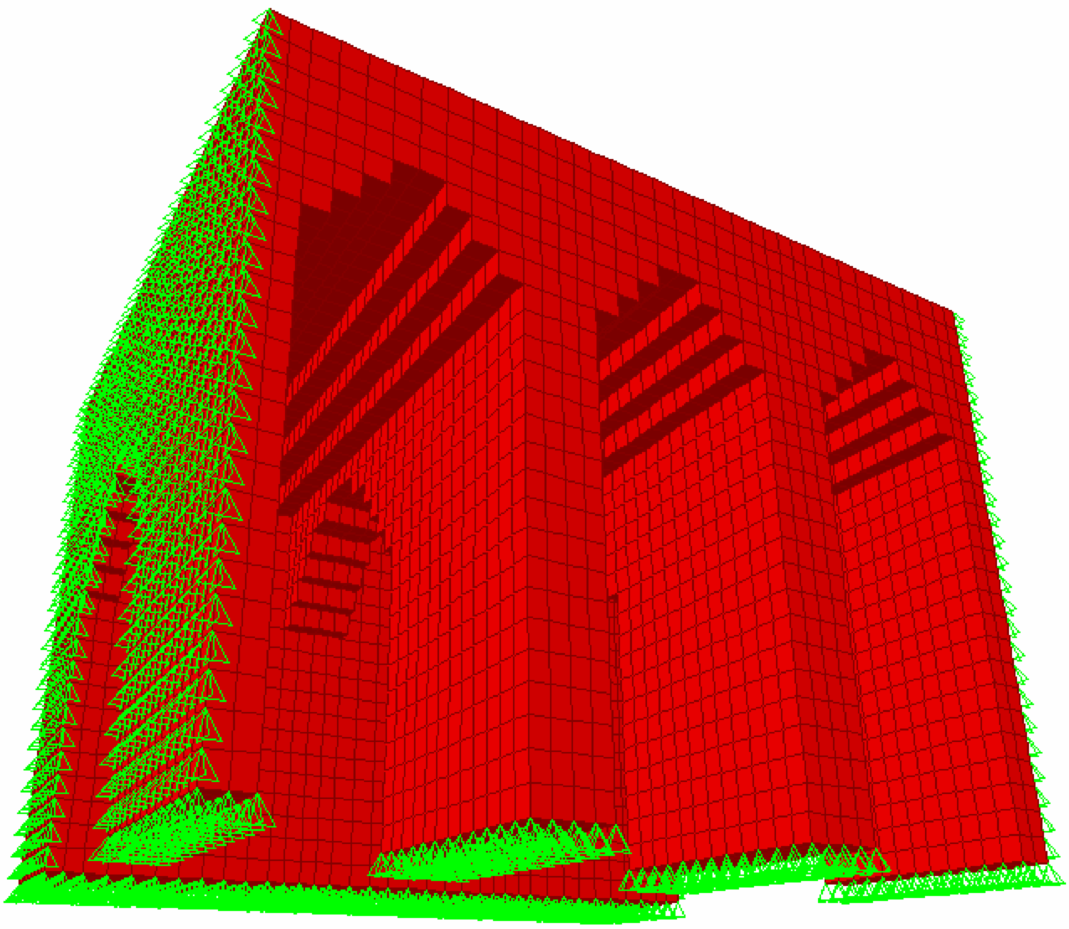


Figure 9.4 Side and base connections of the model

The material properties and the analysis options are defined same as the theatre main building model analysis (See Chapter 4).

A modal analysis has been performed to obtain the first natural frequency of the column model. The mode shape of the first mode is shown on Figure 9.5. The natural period and the natural frequency of the first mode are 0.0298 second and 33.6 Hz, respectively. Therefore, the measured natural frequencies of the columns are expected to be around 33 Hz.

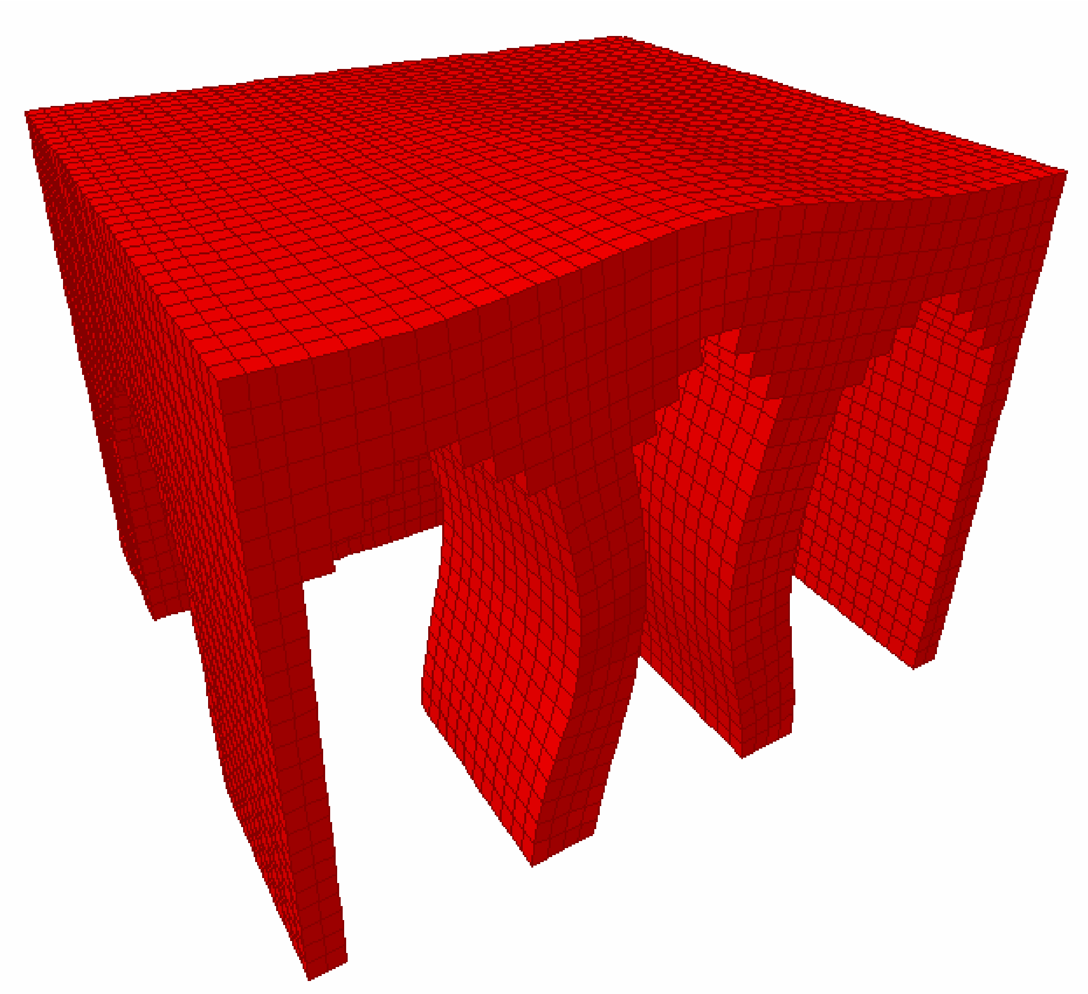


Figure 9.5 First mode shape of the model

9.5 COMPARISON & DISCUSSION OF THE RESULTS

The backside columns in the Aspendos Theatre have been modal tested and the measured frequencies are compared with analytical model results.

The measured natural frequencies of the columns are also compared against visual inspection results. As a column is deficient, it is expected to have a smaller stiffness. As the main mass of all columns are assumed to be similar, a column with low stiffness is expected to have low (natural) vibration frequency. The visual inspection results assign a similar index for a column in bad visual condition and a larger index for a better visual condition.

Assuming constant mass, the natural frequency becomes a function of square root of stiffness. The visual inspection results are calculated as the sum of four subcategories and plotted against frequency square times a scaling constant. The stiffness of a column is assumed to be a linear function of the visual inspection grade. The frequency square is plotted against visual inspection grade in Graph 9.3. The overall good agreement between the two lines in Graph 9.3 can be further improved by optimizing for the subcategory weighing coefficients. Furthermore, assuming that each visual inspection subcategory grade may have an error of about ±0.2 points, a second round of optimization can be implemented to have a better correlation between measurements and visual inspection grading.

The visual inspection points for each subcategory (i.e. column, arch, ceiling and column façade) are modified by adding a value which is smaller than 0.2 and larger than -0.2. The modification values for each subcategory are listed in Table 9.3. The improved correlation between the C*freq^2 and visual grading after modified values is given in Graph 9.3. The weighing coefficient for each subcategory is further optimized for a better match. The optimized coefficients and weighing factors are listed in Table 9.3. The results show that the most significant visual inspection subcategory is condition of ceiling (weighing factor = 7.362) followed by the condition of the column (weighing factor = 2.838), which is rather interesting.

Table 9.3 Optimized coefficients and overall new weighing factors

	Column	Arch	Ceiling	Facade
Optimization Coefficient	0.567	0.500	2.454	0.386
Overall new Weighing factors	2.838	1.500	7.362	0.773

Assuming that the visual inspection grades are subjective and maybe corrected by ±0.2 points over a range of 3 to 5, the grading values shown on

Table 9.2 were modified to better match measurements and presented in Table 9.4 and optimized coefficients are plotted in Graph 9.3. The correlation coefficient (R) for the “modified values and optimized coefficients” shown in Graph 9.3 is obtained to be 0.9685 while R value for “modified values only” was 0.8846. The R coefficient for the original visual inspection grading without modified values or optimized coefficients was 0.7814 which is still satisfactory. The comparison of data presented in Graph 9.3 is also shown as a X-Y plot in Graph 9.4.

After optimization studies, the condition evaluation of the backside columns are obtained using Equation 9.1. The new total column condition index values change between 3.522 and 12.474.

$C_{EI} = 0.567 * C_I + 0.5 * A_I + 2.454 * S_I + 0.386 * F_I$ (Equation 9.1)

Where C_{EI} is the total Condition Evaluation Index.

The same values can also be obtained experimentally by using Equation 9.2).

$$C_{EI} = (f_1)^2 / 78.43 \quad \text{(Equation 9.2)}$$

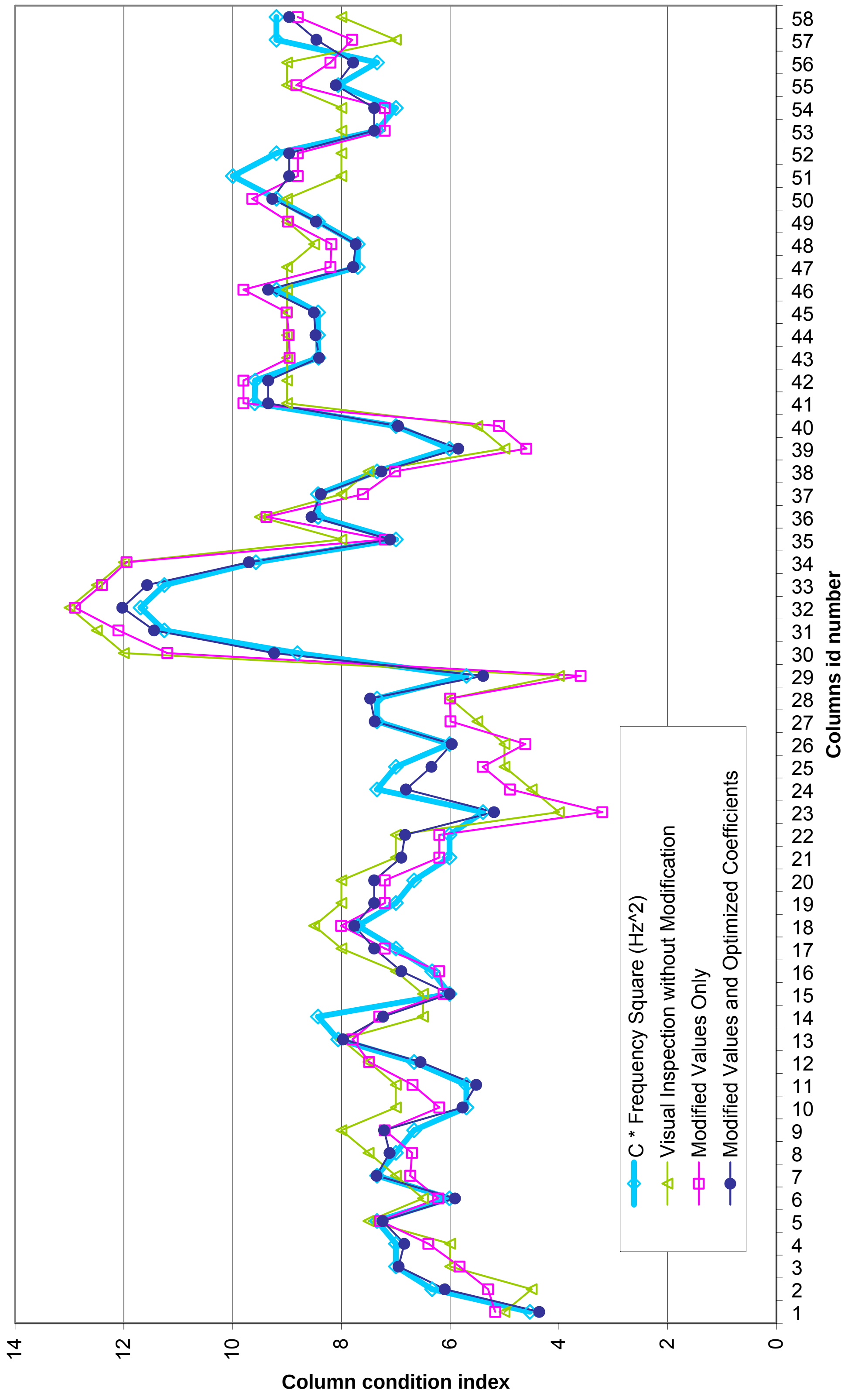
The column index values are sorted to group column condition thresholds (Graph 9.5). The column condition index intervals are defined as shown in Table 9.4.

Table 9.4 Column condition index intervals

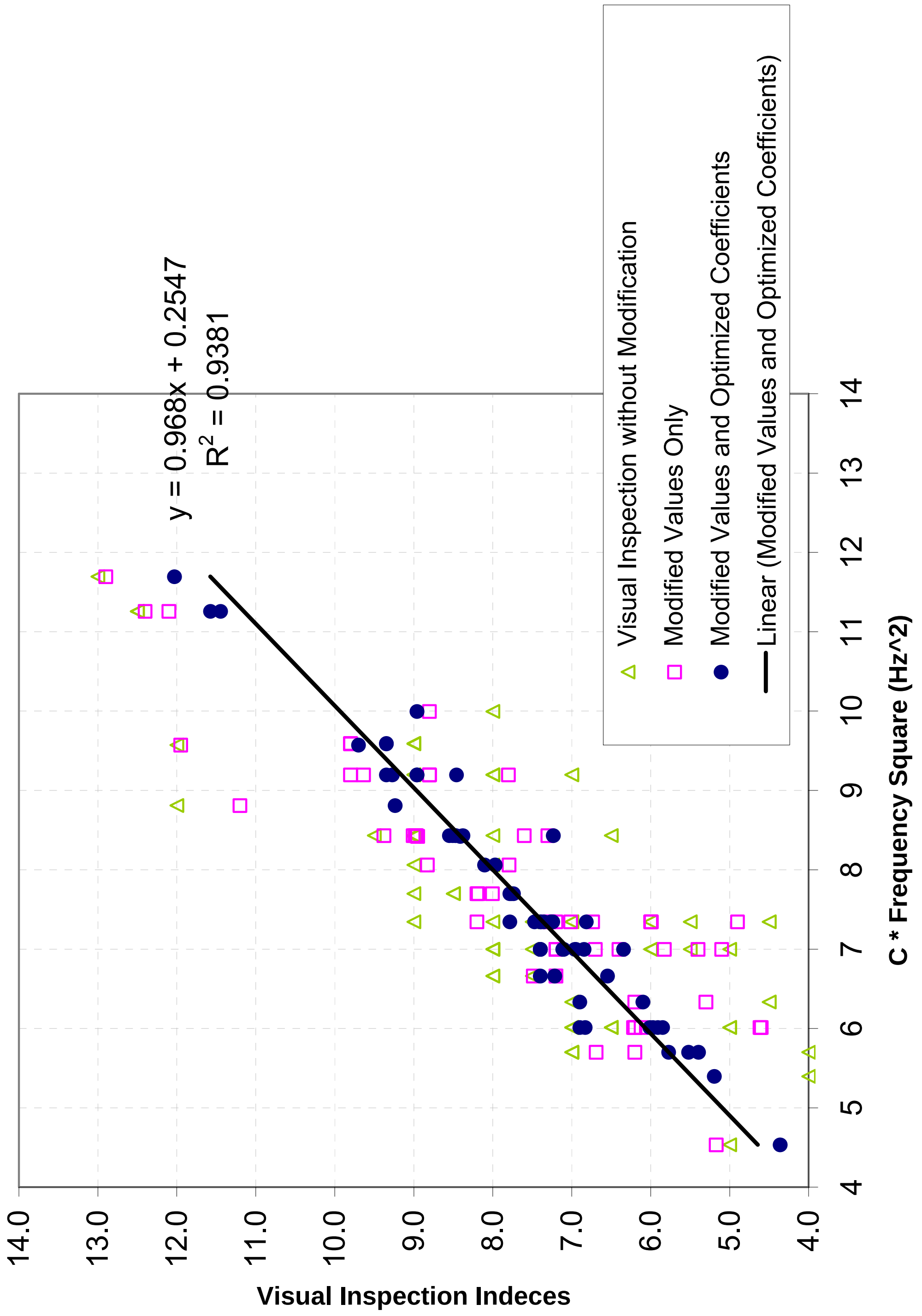
Column Condition	Column Index Interval
Excellent	$10 < C_{EI}$
Good	$7.5 < C_{EI} \leq 10$
Fair	$6.6 < C_{EI} \leq 7.5$
Bad	$C_{EI} \leq 6.6$

Table 9.5 Modified visual grading of the columns

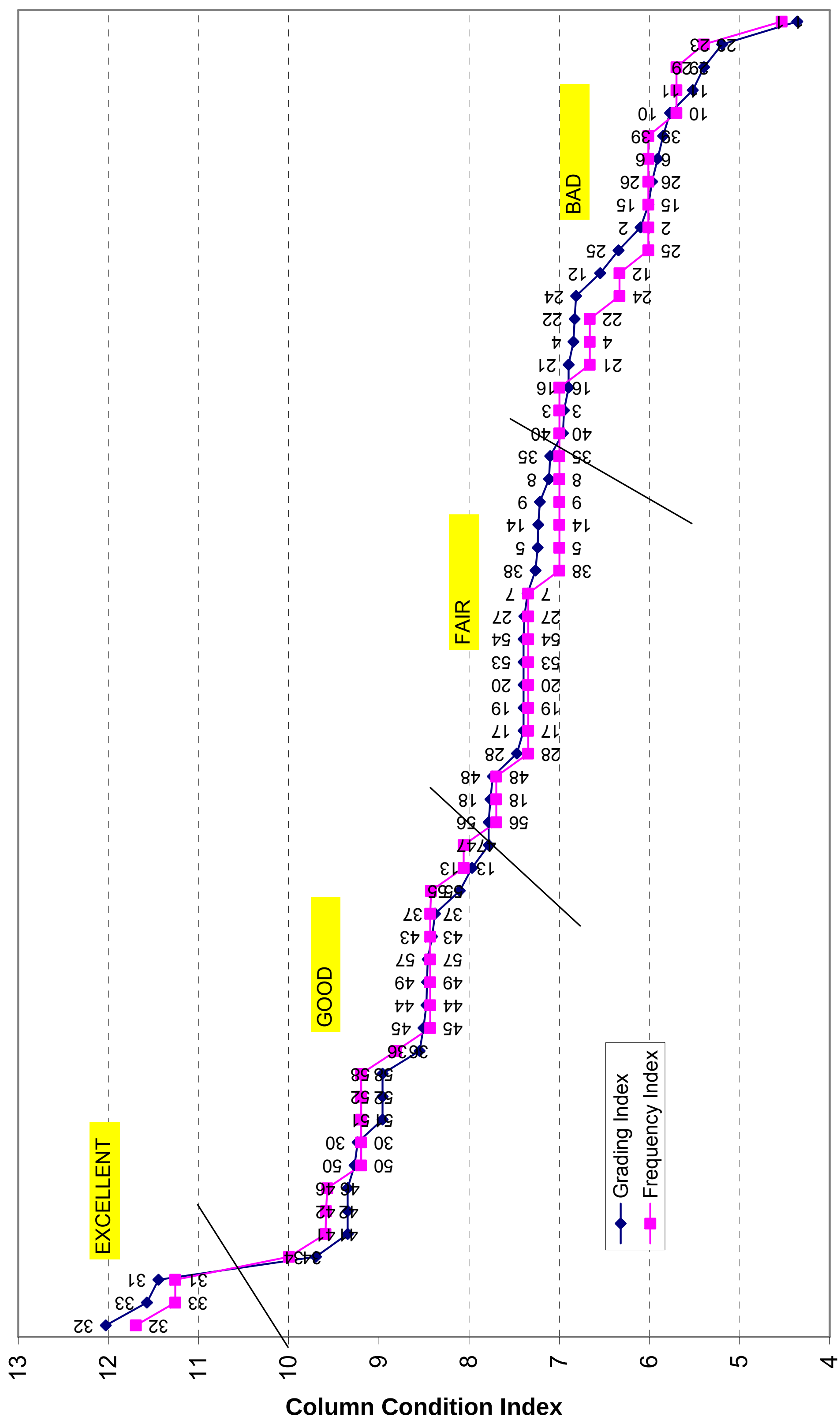
Column ID No.	Modified Visual Grading Points Assigned to Different Parts of the Backside Columns				Column overall grade		
	COLUMN (/5)	ARCH (/3)	CEILING (/3)	Façade (/2)	orig.	val.	final
1	3.2	1.2	0.8	0.0	5.0	5.2	4.4
2	2.2	1.2	1.7	0.2	4.5	5.3	6.1
3	1.9	2.0	2.0	0.0	6.0	5.8	6.9
4	2.2	2.2	1.8	0.2	6.0	6.4	6.8
5	3.8	1.9	1.7	-0.1	7.5	7.3	7.2
6	4.0	0.8	1.3	0.1	6.5	6.2	5.9
7	3.9	0.9	1.9	-0.1	7.0	6.7	7.4
8	3.3	1.8	1.8	-0.2	7.5	6.7	7.1
9	2.8	1.8	1.8	0.8	8.0	7.2	7.2
10	3.3	0.8	1.3	0.8	7.0	6.2	5.8
11	3.9	1.9	1.0	-0.1	7.0	6.7	5.5
12	4.2	1.8	1.3	0.2	7.5	7.5	6.5
13	4.1	1.9	1.9	-0.2	8.0	7.8	8.0
14	4.2	1.2	1.7	0.2	6.5	7.3	7.2
15	3.0	0.8	1.5	0.9	6.5	6.1	6.0
16	3.8	0.8	1.8	-0.2	7.0	6.2	6.9
17	3.8	1.8	1.8	-0.2	8.0	7.2	7.4
18	3.4	1.9	1.9	0.9	8.5	8.0	7.8
19	3.8	1.8	1.8	-0.2	8.0	7.2	7.4
20	3.8	1.8	1.8	-0.2	8.0	7.2	7.4
21	3.8	0.8	1.8	-0.2	7.0	6.2	6.9
22	2.8	1.8	1.8	-0.2	7.0	6.2	6.8
23	0.8	0.8	1.8	-0.2	4.0	3.2	5.2
24	1.3	1.2	2.2	0.2	4.5	4.9	6.8
25	2.2	1.2	1.8	0.2	5.0	5.4	6.3
26	2.0	1.0	1.8	-0.1	5.0	4.6	6.0
27	1.7	1.9	2.2	0.2	5.5	6.0	7.4
28	2.2	1.8	2.2	-0.2	6.0	6.0	7.5
29	0.8	1.2	1.8	-0.2	4.0	3.6	5.4
30	4.8	2.8	1.8	1.8	12.0	11.2	9.2
31	4.8	2.8	2.7	1.8	12.5	12.1	11.4
32	5.1	3.0	2.8	2.0	13.0	12.9	12.0
33	4.8	2.9	2.7	2.0	12.5	12.4	11.6
34	4.9	3.1	1.9	2.1	12.0	11.9	9.7
35	2.8	0.8	1.8	1.8	8.0	7.2	7.1
36	3.4	1.9	2.0	2.1	9.5	9.4	8.5
37	3.8	1.8	2.2	-0.2	8.0	7.6	8.4
38	3.4	1.9	1.8	-0.1	7.5	7.0	7.3
39	0.8	1.8	1.8	0.2	5.0	4.6	5.8
40	1.3	1.8	2.2	-0.2	5.5	5.1	7.0
41	4.2	2.2	2.2	1.2	9.0	9.8	9.3
42	4.2	2.2	2.2	1.2	9.0	9.8	9.3
43	3.9	2.0	1.9	1.1	9.0	9.0	8.4
44	4.0	2.0	2.0	1.1	9.0	9.0	8.5
45	4.0	2.0	2.0	1.0	9.0	9.0	8.5
46	4.2	2.2	2.2	1.2	9.0	9.8	9.3
47	3.8	1.8	1.8	0.8	9.0	8.2	7.8
48	3.6	1.8	1.8	1.0	8.5	8.2	7.7
49	3.9	2.1	2.0	1.0	9.0	9.0	8.5
50	4.1	2.2	2.2	1.1	9.0	9.6	9.3
51	4.2	2.2	2.2	0.2	8.0	8.8	9.0
52	4.2	2.2	2.2	0.2	8.0	8.8	9.0
53	3.8	1.8	1.8	-0.2	8.0	7.2	7.4
54	3.8	1.8	1.8	-0.2	8.0	7.2	7.4
55	4.1	2.0	1.8	0.9	9.0	8.8	8.1
56	3.8	1.8	1.8	0.8	9.0	8.2	7.8
57	4.2	1.2	2.2	0.2	7.0	7.8	8.5
58	4.2	2.2	2.2	0.2	8.0	8.8	9.0



Graph 9.3 Comparison of visual grading and first natural frequencies



Graph 9.4 Comparison of visual inspection data



Graph 9.5 Column condition index intervals

During the visual inspection, it has been observed that; five columns (Column #30, 31, 32, 33 & 34) look like new or repaired recently. According to the information gained from the staff of the Aspendos Theatre, these columns have been completely rebuilt in 1968 because they were demolished until this year (photo 9.8).

Photo 9.8 New columns

As well as the high grades awarded for these columns, the first natural frequencies are also comparatively high (See Graph 9.1). As the higher first natural frequency indicates higher rigidity, these 5 columns have been considered as they are in good to excellent condition; and consequently have more structural stiffness when compared to the other columns.

The analytical model of the columns has given result of 33.6Hz for the first natural frequency. This value is very close to ones obtained from the frequency measurements for new columns (#30…#34). This shows that the new columns can be considered as they are in good structural condition.

It has been also observed that the structural condition of the first 40 columns, except for the recently repaired five columns (30…34), is in worse condition than the remaining columns according to both first natural frequencies and visual grading results. The columns that are in poor structural condition are dominantly facing south and aging can be a function of sunlight deterioration as well as vegetation growing due to sunlight. It has been learned that there were large trees growing on the stands close to the mid section and restored columns.

The structural condition for column numbers 1, 10, 11, 23 and 29 have been considered as the worst among all columns because they have relatively low first natural frequencies when compared with other columns.

According to the natural frequency measurement, the best and the worst columns can be considered as column #32 and column #1, respectively. The first natural frequency of column #1 is 18.9Hz, whereas it is 30.3Hz for column #32.

CHAPTER 10

CONCLUSIONS

Since the analysis is based on many assumptions and subjective decisions, the following conclusions should be used with wise care.

In this thesis work, the historical Aspendos Theatre has been experimentally and analytically evaluated using impact modal testing and calibrated three dimensional (3D) finite element modeling (FEM). The theatre is evaluated under earthquake forces (475 and 2500 years return period) and sound induced forces that may be generated during concerts and shows that use amplified speakers and large drums. The frequency content of songs is compared with measured and simulated natural vibration frequencies of the structure. The calculated stress levels are compared against material capacities in tension and compression. Approximate deflection capacity equations are derived and deflection demands are evaluated. Peripheral columns at the backstage are also evaluated visually and based on non-destructive testing. Three-dimensional analytical model of the columns are used to estimate healthy condition vibration frequency of the columns. Conclusions are drawn based on findings.

The dynamic tests and modal analysis of the Aspendos Theatre has shown that the natural frequencies of the first 5 modes of the structure are in the range of the general beat frequency range of songs (1 Hz – 2.8 Hz). The songs played using amplified speakers at concerts and similar shows in the theatre may generate cyclic pressure loads on the walls which are evaluated to see if the theatre may go into resonance state. Field measurements and calibrated 3D finite element models are used to calculate stress levels that may develop during resonance state. The stage wall, which is directly exposed to the speakers, has the first 3 natural frequencies within the beat frequency range of songs: 0.87 Hz., 1.66 Hz. and 2.71 Hz. (1.36 Hz. half frequency).

The limiting sound level on the stage wall was calculated using two approaches: 1) maximum accepted material stress capacities and 2) maximum stresses developing in the structure when 475 year return period earthquake analysis is conducted. Since the structure survived earthquakes occurred during the last 2200 years, it is assumed that the structure would be safe if the sound induced stresses remain lower than the 475 year earthquake analysis stress results. The sound level limit has been found as 125 dB (at 0.3 m distance to the wall = 86 dB at 25 m away from the wall) when the stress capacity of the material is considered. However, the sound level limit is found to be 142.9 dB (at 0.3 m distance to the wall = 104 dB at 25 m away from the wall) when the strength limit is taken as the maximum stresses developing on the structure under 475 years return period earthquake loads. The concerts, music and dance shows that will be conducted in the future are not likely to cause any significant structural damage on the main walls of the structure since the song beat frequency should exactly match the natural vibration frequency of the stage wall at more than 140 dB which is equivalent to the jet engine sound at 50 m distance. On the other hand, songs with dominant beat frequencies at 0.87Hz (first bending natural frequency of the stage wall based on calibrated FE model), 1.47Hz (first bending measured natural frequency of the exterior wall), 1.36Hz (half of the third bending mode of the stage wall based on FE model), and 2.71 Hz should be limited to 125 dB (combined) at 0.3m distance to the stage wall or 85 dB (combined) at 25m away from the stage wall to eliminate any potential damage to the structure.

The analysis results of the Aspendos Theatre under sound induced pressure forces and dead load have shown that the factor of safety coefficients for compression stresses developing at the theatre walls are lower than the compressive capacity of the material (12 MPa). The factor of safety for compression is found as 4.5 for sound induced forces (combined 130 dB at 0.3 m away from the wall) at resonant frequency. The factor of safety under tension has been found to be 0.67 under sound induced forces under worst conditions (130 dB at 0.3 m distance from the stage wall and at resonant frequency).

The factor of safety values for compression stress under earthquake loading with return periods of 475 years and 2500 years are calculated as 1.67 and 0.98, respectively. However, the F.S. in tension has been calculated as 0.18 and 0.10 under earthquake loading with return periods of 475 years and 2500 years, respectively for 0.8% measured damping. During the inspection of the Aspendos Theatre, it has been seen that there are small cracks on the exterior and stage walls near the corner of windows. Information on reconstruction or restoration of the main building is not available; however, some level of damage is expected during 475 year earthquakes. The source of cracks at the window corners can also be thought to occur due to stress concentrations generated by previous small to medium scale earthquakes.

The factor of safety in tension calculated to be 0.1 for 2500 year return period EQ shows that tensile stresses generated by such an earthquake 10 times exceeds the accepted tensile stress capacity. A large tensile stress of 12 MPa developing at the vicinity of front wall and 5 MPa tension developing at the upper left and right corners of the wall, although appears to be concentrated locally on a linear model, are too high to be tolerated by masonry and most likely would generate heavy local structural damage at the corners and around the door or lead to structural collapse. The maximum wall deflection calculated by linear analysis is in the order of 80 cm which is close to the wall thickness of 120 cm. On the limit state, if the maximum wall deflection reaches to the wall thickness, the wall would become unstable and collapse. The linear analysis results for deflection that cause tensile failure of the material is obtained as 9.5 cm which is much smaller than the calculated demand deflection of 80 cm. An equation derived to compute capacity under earthquake load was derived using simple cantilever approach (Equation 7.15 on page 112). The maximum deflection calculated using linear earthquake analysis (about 8 cm) is in close with the critical deflection capacity (11 cm) calculated by the simplistic approach defined by Equation 7.15. The lateral support of the Aspendos Theatre walls would cause the critical deflection capacity to be calculated slightly smaller than the one calculated using Equation 7.15. Nevertheless, the maximum deflection capacity calculated using Equation 7.15 can be used as a conservative approximation for any laterally unsupported masonry wall.

Structural strengthening of the theatre stage and exterior walls using horizontal and vertical direction post tensioning (to the foundation rock) is highly recommended to prevent heavy structural damage to the theatre during a possible 2500 year return period earthquake. Slab floor constructed between the stage and exterior walls, where it used to be in the original design, might also improve the stability of the structure.

The theatre's peripheral backside columns were also examined. Vibration measurements were taken from all 58 columns and the first natural frequencies of each column were identified. The computer model for these columns was also prepared and modal analysis was performed. Condition assessment of columns was made by visual inspection. Grading values were assigned indicating the condition of each portion of the columns for overall condition evaluation. The first natural frequencies from measurements, modal analysis results, and visual grading results have been compared. The structural condition (level of deterioration) levels of columns were linked to square of the first natural frequency of columns through different optimization studies. A condition evaluation index was developed for the columns (Equations 9.1 and 9.2 on page 136) and ranges for bad, fair, good, and excellent condition states were defined. The columns with relatively low first natural frequency were considered to be in worse structural condition which also agrees with the visual inspection. From this comparison, the columns #30, #31, #32, #33, and #34, which has been recently restored, were found to be in very good condition. It was also determined that the first 40 columns (except for restored columns #30…34) are in worse condition than the others. Direct sunlight exposure on columns #1 through #40 due to the orientation of the theatre may be a factor for relatively faster deterioration and aging for these columns. The structural condition for column numbers 1, 10, 11, 23, and 29 have been considered as in the worst structural condition among all columns. The columns #32 and #1 have been found to be in the best and the worst conditions, respectively.

Recommendations for future studies:

- Since the available 2500 year earthquake analysis results show large tensile stresses 4 to 10 times the accepted tensile stress capacity, strengthening of the stage and exterior walls are strongly recommended. A post-tensioning approach through the thickness of the walls in vertical and horizontal directions would be preferred as the strengthening would not be visually exposed to visitors.
- Dynamic non-destructive tests, similar to the exterior wall tests, should be conducted for the stage wall which did not have access using the fire truck bucket for testing.
- The columns located above the stage wall are in bad condition and may become unstable in an earthquake. Likewise, cantilever stone blocks located on the front face of the stage wall have cracks and potentially unsafe. Each cantilever stone block on the front face and on the top of the stage wall that poses instability or vulnerability should be locally checked by non-destructive testing and removed or strengthened if necessary.
- The cracks at the window corners and walls should be documented and monitored over time.
- The vegetation growing on the walls of the theatre imposes threat to the structural integrity by their roots. The vegetation should be periodically

removed and preferably regrowth should be prevented by chemicals (which are not harmful to the structural material).

- The soil characteristics of the theatre foundation should be investigated in detail.
- History for the restoration of the theatre should be investigated in detail. Therefore, the previous damages and possible reconstruction stages of the structure can be used to better evaluate current analysis results.

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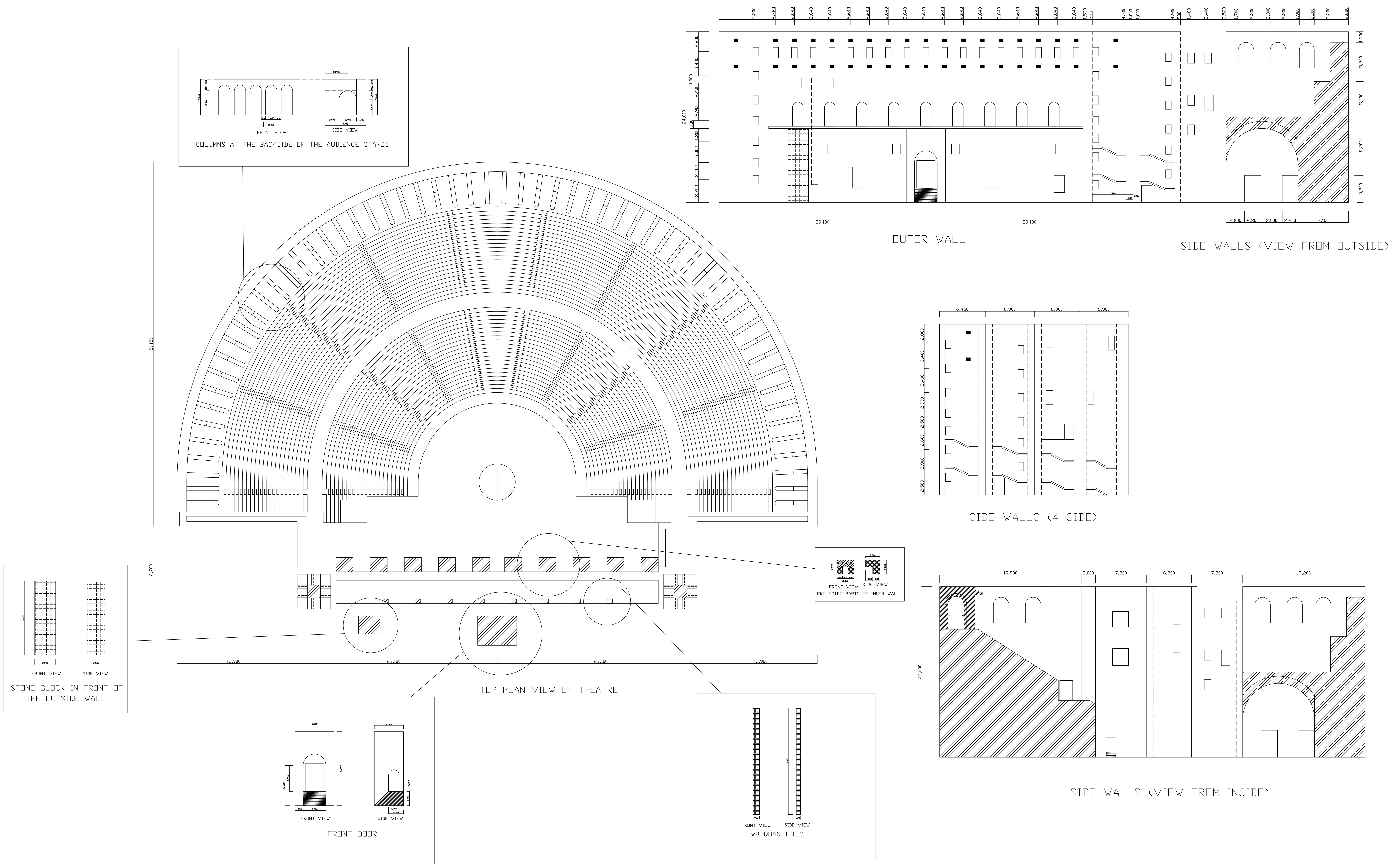

APPENDIX A
DETAILED DRAWING OF
ASPENDOS THEATRE

Prepared by BERK BOZ

APPENDIX B

ALL STRESSES AT EACH LOCATION DUE TO BOTH EQ and SOUND INDUCED FORCES

All the following stresses include stresses which are due to dead load of the structure.

ALL STRESSES ON STAGE WALL for SOUND INDUCED FORCES			
Location	**Direction**	**Compressive (MPa)**	**Tensile (MPa)**
F1		-1.2	0.8
F2		-1.2	0.8
F3		-1.1	0.9
F4		**-2.7**	0.3
R1		-1.2	**1.8**
R2		-1.2	1.8
R3		-1.0	0.6
R4		-2.2	0.8

ALL STRESSES ON EXTERIOR and STAGE WALLS for 475 YEAR RETURN PERIOD EQ with 5% damping			
Location	**Direction**	**Compressive (MPa)**	**Tensile (MPa)**
F1	↔ □ ↔	-3.3	2.5
F2	↔ □ ↔	-3.3	2.5
F3	↔ □ ↔	-3.2	3.0
F4	↕ □ ↕	**-5.0**	3.0
F5	↔ □ ↔	-1.7	1.5
F6	↔ □ ↔	-1.7	1.5
F7	↔ □ ↔	-1.7	1.8
F8	↕ □ ↕	-3.6	2.8
F9	↕ □ ↕	**-5.0**	**3.6**
R1	↔ □ ↔	-2.8	3.2
R2	↔ □ ↔	-2.8	3.2
R3	↔ □ ↔	-3.6	3.2
R4	↕ □ ↕	-3.7	3.2
R5	↔ □ ↔	-1.7	1.5
R6	↔ □ ↔	-1.7	1.5
R7	↔ □ ↔	-1.5	1.2
R8	↕ □ ↕	-3.6	2.8

ALL STRESSES ON EXTERIOR and STAGE WALLS for 2500 YEAR RETURN PERIOD EQ with 5% damping

Location	Direction	Compressive (MPa)	Tensile (MPa)
F1	↔ □ ↔	-5.9	4.5
F2	↔ □ ↔	-5.9	4.5
F3	↔ □ ↔	-5.8	5.4
F4	↕ □ ↕	**-8.0**	6.4
F5	↔ □ ↔	-3.1	2.7
F6	↔ □ ↔	-3.1	2.7
F7	↔ □ ↔	-3.1	3.2
F8	↕ □ ↕	-5.9	5.4
F9	↕ □ ↕	**-8.0**	**6.9**
R1	↔ □ ↔	-5.0	5.8
R2	↔ □ ↔	-5.0	5.8
R3	↔ □ ↔	-6.5	5.8
R4	↕ □ ↕	-5.9	6.0
R5	↔ □ ↔	-3.1	2.7
R6	↔ □ ↔	-3.1	2.7
R7	↔ □ ↔	-2.7	2.2
R8	↕ □ ↕	-5.7	5.4

ALL STRESSES ON EXTERIOR and STAGE WALLS for 475 YEAR RETURN PERIOD EQ with 0.8% damping

Location	Direction	Compressive (MPa)	Tensile (MPa)
F1	↔□↔	-5.4	4.1
F2	↔□↔	-5.4	4.1
F3	↔□↔	-5.3	5.0
F4	↕□↕	-7.0	5.9
F5	↔□↔	-2.8	2.5
F6	↔□↔	-2.8	2.5
F7	↔□↔	-2.8	3.0
F8	↕□↕	-5.6	4.8
F9	↕□↕	**-7.2**	**6.6**
R1	↔□↔	-4.6	5.5
R2	↔□↔	-4.6	5.5
R3	↔□↔	-5.9	5.5
R4	↕□↕	-5.4	5.5
R5	↔□↔	-2.8	2.5
R6	↔□↔	-2.8	2.5
R7	↔□↔	-2.5	2.0
R8	↕□↕	-5.7	5.0

ALL STRESSES ON EXTERIOR and STAGE WALLS for 2500 YEAR RETURN PERIOD EQ with 0.8% damping

Location	Direction	Compressive (MPa)	Tensile (MPa)
F1	↔□↔	-9.8	7.4
F2	↔□↔	-9.8	7.4
F3	↔□↔	-9.5	8.9
F4	↕□↕	-11.0	10.2
F5	↔□↔	-5.0	4.5
F6	↔□↔	-5.0	4.5
F7	↔□↔	-5.0	5.3
F8	↕□↕	-10.1	8.6
F9	↕□↕	**-12.2**	**12.0**
R1	↔□↔	-8.3	10.0
R2	↔□↔	-8.3	10.0
R3	↔□↔	-10.7	10.0
R4	↕□↕	-9.6	10.0
R5	↔□↔	-5.0	4.5
R6	↔□↔	-5.0	4.5
R7	↔□↔	-4.5	3.6
R8	↕□↕	-10.0	8.8