NUMERICAL MODELING OF THE AUGUSTUS TEMPLE

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by

RIDVAN DOGER

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GCRIS

NUMERICAL MODELING OF THE AUGUSTUS TEMPLE

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Approval of the Graduate School of Natural and Applied Sciences, Atilim University.

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ABSTRACT

NUMERICAL MODELING OF THE AUGUSTUS TEMPLE

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This thesis reviews the numerical modeling of the Augustus Temple. “3DEC” software by Itasca and “PLAXIS” software by PLAXIS B.V. were used to perform the numerical analysis. Recommendations for future testing and monitoring were discussed. Analysis methods and properties of dry stone structures were also discussed in this thesis.

Keywords: Numerical modeling, Augustus Temple, 3DEC, PLAXIS

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ÖZ

AUGUSTUS TAPINAĞININ SAYISAL MODELLEMESİ

Döğer, Rıdvan

Yüksek Lisans, İnşaat Mühendisliği Bölümü

Tez Yöneticisi: Doç.Dr. Yasin Dursun Sarı

Ocak 2010, 97 Sayfa

Bu çalışma, Augustus Tapınağının sayısal modellemesini incelemektedir. Itasca firmasının 3DEC yazılımı ve PLAXIS B.V. firmasının PLAXIS yazılımı sayısal analiz için kullanılmıştır. Gelecek testler ve gözlemlemeler için öneriler verilmiştir. Analiz metodları ve taş yapıların özellikleri de bu tez de incelenmektedir.

Anahtar Kelimeler: Sayısal Modelleme, Augustus Tapınağı, 3DEC, PLAXIS

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CHAPTER 1

INTRODUCTION

Located in the interior Anatolian plateau of central Turkey, capital city Ankara, the Augustus Temple has been affected by environmental changes occurring over 2,000 years. The altered climatic conditions with human effects such as re-construction, excavation and conservation hold specific implications for understanding and interpreting past and current conditions of the structures and site. Understanding climate, soil-structure interaction and the region's seismicity is critical in diagnosing and predicting the temple walls' responses to ground movement and in developing a strategy to stabilize the structure and prevent further collapse.

"3DEC" software by Itasca and "PLAXIS" software by PLAXIS B.V. were used to analyze the wall at the east side of the Augustus Temple. East side wall was selected, because the condition of the wall was better than the west side wall. Also, the foundation excavation was done close to the east side wall and this gives more accurate information about the wall's overall structure.

The analysis was focused on how east wall of the Augustus Temple has still been standing after more than 2,000 years. The foundation structure is selected as the most critical parameter that can affect wall's stability. Eight different 3DEC analyses and nine different PLAXIS analyses were performed in order to show the effect of the foundation and earthquake parameters to the Augustus Temple's wall stability.

CHAPTER 2

PROPERTIES OF THE AUGUSTUS TEMPLE

2.1. General Information

Augustus Temple was built in about 25 BC in Ankara. This monument celebrated the glory of Augustus, the first roman Emperor; after his death the temple was adorned with colored engraved letters, the most famous epigraph of the roman world.

The engraved marble with its remarkable political heritage is now rapidly deteriorating because of the severe atmospheric conditions and heavy pollution. Only the two longitudinal walls together, with the entrance door, can be seen. The temple was transformed in a byzantine church. The transversal wall was then destroyed, producing weakening of the whole structure. Below the base there is a crypt.

The temple was changed into a basilica and three windows were annexed on the south wall of the Cella (inner part of the temple) with Cryptos (vaulted place) beyond the Naos (internal complexes). The plan of the temple is a rectangular shape and the axis directed north-west.

The main entrance faces to the large square in front of the Hacı Bayram mosque. It is a 15th-century mosque with minaret and perhaps the Turkish capital's most important Islamic centre. The Cella, Pronaos and the temple have survived, though the North West wall of the Cella was destroyed in 1834 [1].

The general conditions of the remains of the temple are indeed very poor, from the static and general point of view, because of the very hard environmental state, produced by the severe atmospheric conditions and mainly the air pollution. The condition of the Augustus Temple can be seen in an ancient print in Figure 1.

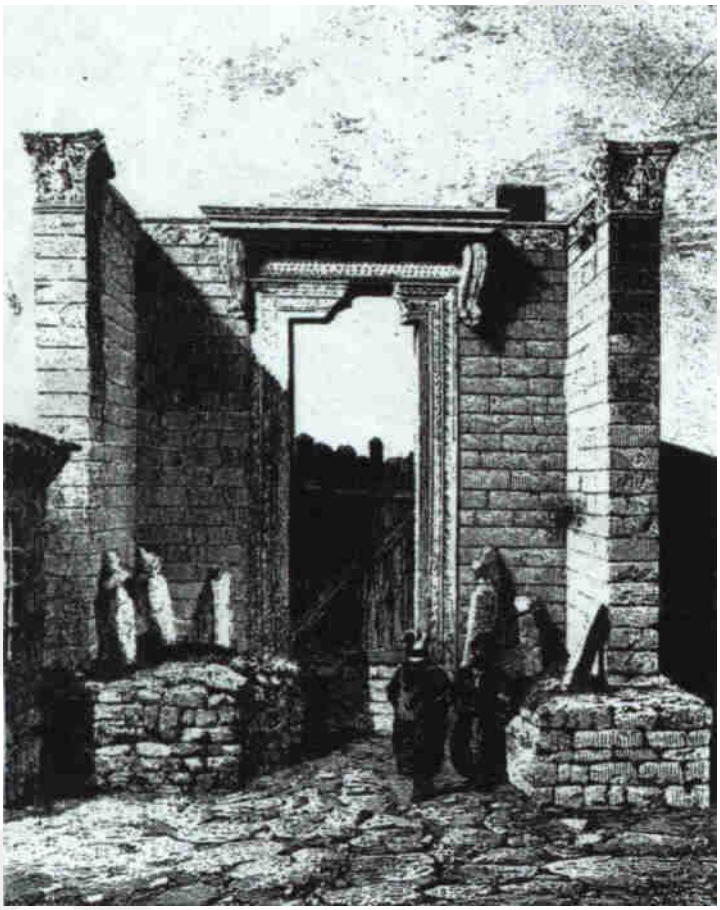


Figure 1 - The entrance of the Augustus Temple in an ancient print

Figure 2 shows the temple and the minaret of the near mosque during the restoration works. Little can be perceived as modern urban development has drastically altered the original setting. A wide pavement surrounds the mosque and parts of the temple whilst a road open to traffic runs parallel to the lateral side of

Roman monument. General view of the surrounding around Augustus Temple can be seen in Figure 3.



Figure 2 - Augustus Temple view during restoration of the structure



Figure 3 - Augustus Temple and Hacı Bayram Mosque from Ankara Castle

Only the walls of the cella, the inner part of the temple, are still standing. Large scaffolding surrounds the wall. A colonnade originally bordered the temple. No one column is still there. Closer view of the Augustus Temple's still standing walls can be seen in Figure 4, 5 and 6.



Figure 4 - Standing wall with foundation underneath of Augustus Temple



Figure 5 - Standing wall of Augustus Temple from archaeological site



Figure 6 - An inside view of cella of Augustus Temple

The Marble Temple measuring 36x54.82 meter used to stand on a pediment of several steps about 2 meters high. It faces the west, is planned in pseudodipteros (one line columns) with 8 ionic pillars on the short side and 15 on the long side (See Figure 7) [1].



Figure 7 - A scaled model of original Augustus Temple

Georadar investigation was done by Culture and Tourism Ministry of Turkey to have an idea about the soil structure underneath the wall structure and can be seen in Figure 8. Foundation investigation was done by excavation in front of the east wall and details of the foundation investigation can be seen in Figure 9,10 and 11.



Figure 8 - Georadar investigation at cella of Augustus Temple



Figure 9 - Excavation for foundation investigation



Figure 10 - The foundation excavation in front of the east wall of cella



Figure 11 - A view from the foundation excavation work

The excavation work was ended at 6 mt below the standing wall foundation and the detail of the foundation underneath the standing east wall of cella is shown in Figure 12.

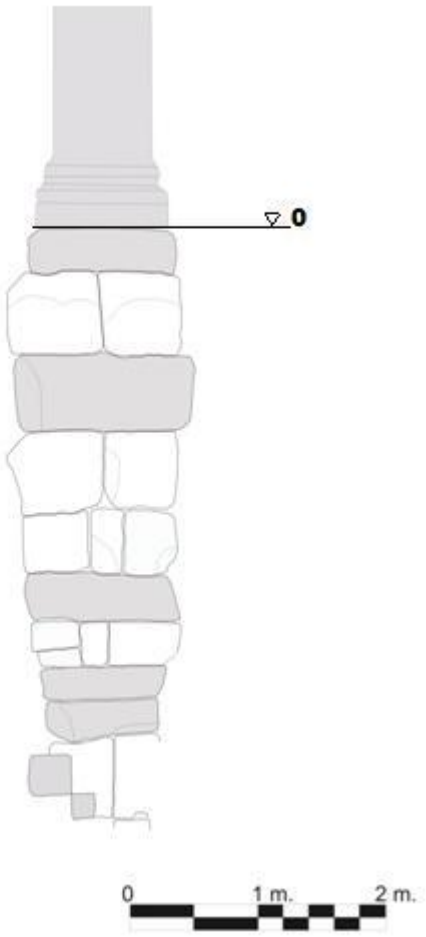


Figure 12 - A scaled foundation detail of the standing east wall

2.2. Climate

Because of its location within Central Anatolia, Augustus Temple does not have the same humid, mild conditions of Turkey's Mediterranean coast. The region experiences somewhat harsher conditions with more significant temperature extremes. The climatic disparity occurs from topographical differences between the coastal and inland regions.

Mountains generally run parallel to the coastline and prevent any substantial precipitation from reaching the plains. As a result, the Central Anatolian region is characterized as semi-dry and receives only about 400mm of precipitation annually, as compared to the average accumulation of 1,100mm in the coastal regions [2]. The

little precipitation that reaches Central Anatolia occurs mostly in winter in the form of snow, since temperatures average -2°C during the winter months. The dry summer season average is 23°C [3]. Even with low accumulation of precipitation, the climate provides the necessary conditions for freeze/thaw cycling.

2.3. Characterization of Soils

The soil-structure interaction has significant implications relating to the stability of the Augustus Temple. Knowledge of the bedrock and composition of the soil not only provides some indication of the structure's general stability but also increases the capacity to predict the standing east and west wall's behavior during seismic events. The assumed values of the wall, foundation and soil properties for the analysis are listed in Chapter 5.

2.4. Seismic Conditions

Located at the center between active fault lines, the Augustus Temple walls experience frequent ground movements from various plates. A comparison of maps shows the correlation between major earthquakes and the North Anatolian Fault. Figure 13 illustrates the extent of Turkey's seismic area and shows the highly active region surrounding the Central Anatolian plateau. Because Turkey is situated on a wedge of continental crust at the convergence of multiple plates, a lot of seismic events have occurred in the region surrounding Augustus Temple over the past century; many of these earthquakes have registered at 5.0 or above in magnitude and caused extensive damage to the building environment [4].

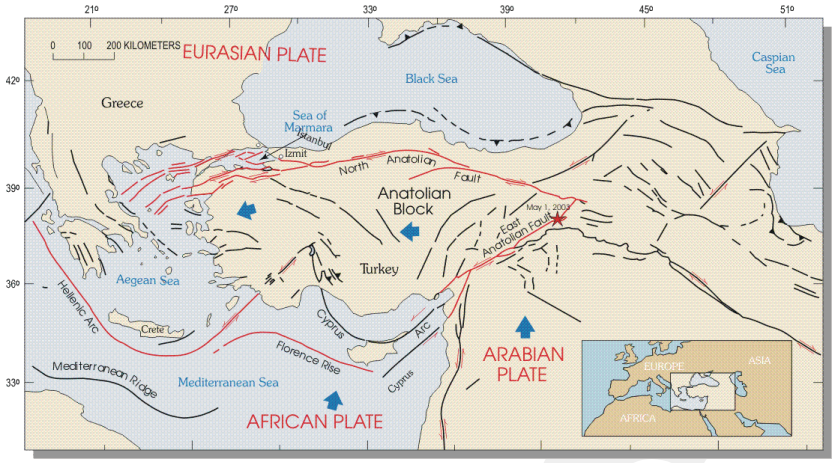


Figure 13 - Map of the fault lines around the Central Anatolia (www.usgs.gov)

The Augustus Temple is most affected by activity along the 1,500 kilometer long North Anatolian fault line. Built up stresses are found to be released approximately every twenty years; the Izmit segment ruptured in 1999 [5]. The frequency and severity of earthquake activity along this critical North Anatolian fault line leaves the West Wall of Augustus Temple susceptible to large displacements, which could eventually lead to collapse if unsupported (See Figure 14).



Figure 14 - The west wall of Augustus Temple

CHAPTER 3

DRY STONE STRUCTURES

3.1. Literature Review

A review of literature concerning dry stone masonry indicates that only within the past few years, there has been significant interest in large-scale testing for quantifying properties and predicting failure. These tests mainly apply to the future construction of retaining walls and are limited in their application to historic structures [6].

The first experiment performed on a full-scale dry stone masonry test wall was conducted in 1834 by British Lieutenant General Sir John Burgoyne as part of the Corps of Engineers research [7]. Burgoyne acknowledged previous work that attempted to calculate dimensions needed to construct stable walls, but he was the first to empirically determine factors contributing to failure through a trial and error process.

Early literature provided information on causes of failure and was used specifically for diagnosing existing problems and implementing treatment plans. Literature specified for the conservation field conflict in intervention techniques to stabilize structures. However, these early conservation studies unanimously recognized the importance of material compatibility and rejected the use of Portland cement as a binding agent to improve load distribution. Typical application was superficial, so it could not relieve points of stress and offered little additional cohesion [8].

However, conflict arose in various intervention techniques. Some interventions valued function, stability, safety over fabric, while other noninvasive techniques placed significance on existing fabric. Interventions which cause various degrees of material loss require pinning, soil nails or the insertion of ties to stabilize the structure.

Other options researched during this period that did not require loss of original fabric included grout injection and supporting; though these methods retain fabric, they introduce material to the original structure. Grout injections irreversibly alter the properties and form of the dry stone wall, while supporting may aesthetically diminish the appearance.

Alternatively, practices at certain Southern African sites included careful documentation and recordation of each stone to allow for reconstruction after collapse. The conservation program at these sites focuses on maintaining a high degree of integrity through minimal intervention, since current technologies for stabilization adversely impact the form and properties of the dry stone structures [9]. The lack of knowledge of failure mechanisms and relatively low number of conservation publications has proven to be an area of weakness in the conservation field.

Engineering standards are necessary for current building practices in order for any structure meet code specifications. Many studies quantify Coulomb's friction in an effort to establish the amount of overall cohesion and predict the amount of friction necessary to prevent shear failure.

Tests for dry stone wall stability include computing Coulomb's friction and using a limit equilibrium analysis to predict failure. Additionally, testing to determine stability under cyclic loading was conducted to increase knowledge of shear properties. Though loading can compress stones and create deformation, the results of the test found that shear strength increased during the first few loading cycles and then stabilized during successive cycles [10]. The test also determined that irregular surfaces ultimately contain the most friction between interfaces. Even with the progress made through the engineering studies, failure under realistic conditions remains unquantifiable. However, establishing parameters for failure under ideal conditions does lend useful knowledge to the conservation field.

Computer-generated modeling has become popular in engineering publications to represent and predict failure modes in dry stone constructions. However, these models assume several factors when making these predictions: the wall reacts as one unit and the stones are homogenous. When considering archaeological sites such as Augustus Temple, many irregularities emerge in the structure and material. Though the models compute age and weathering by lowering the amount of friction by decreasing the amount of stone interface, the wall is still represented in the calculations as a homogenous unit.

The model simulations fail to consider the innate, heterogeneous quality of stone and any irregularities in the structure, which may result from original construction methods or uneven weathering patterns. These irregularities contribute to localized points of weakness and greatly alter structural performance. Limitations on testing methodology arise through several factors. The tests conducted generally measure friction, compression, sliding and shearing on a limited scale. Most tests rely on a single interface between two stones to calculate cohesion and then apply that figure to the larger wall. Large-scale testing is relatively uncommon and does not utilize weathered stones. However, certain tests have accounted for weathering by reducing the contact area between stones.

Another important limit in these experiments is time. Many studies note the period of settlement that occurs just after construction or reconstruction of the wall. Generally, observations show that after a year of settling, the masonry wall stabilizes, though it slowly and steadily compresses and moves in response to load patterns and the environment [6].

3.2. Observations on Failure

As evidenced in the recent engineering tests examined in the literature review, block interfaces generate the friction integral to wall stability. Because friction is produced through surface contact, the weathering of stone surfaces greatly impacts the overall cohesion and, ultimately, wall stability. Though drainage reduces

the risk of bulging or toppling failure in retaining structures, environmental factors may still cause deterioration of individual stones.

Both internal and external components are susceptible to weathering from water ingress. Though the degree of tolerable deterioration cannot be quantified in rubble masonry constructions, the structure's flexibility allows a great deal of decay before failing. As dry stone structures weather, they may deflect several centimeters before regaining stability. This type of movement from water ingress can result in sliding/shear movement and bulging.

Seismic activity also presents the potential for structural movement and can produce instability by weakening tie connections and reducing the amount of stone interface. Weakened connections decrease overall strength and create localized areas vulnerable to bulging or other failure. Vegetation growth can similarly impact stability by rooting in and cracking stones and accelerating weathering by holding moisture.

3.3. Conservation Methods for Dry Stone Structures

Several techniques have been developed to stabilize dry stone walls, including soil nailing, pinning and injection grouting. The other interventions can be applied to either freestanding or retaining structures. Each system affects the appearance and performance of the wall to varying degrees. While injection grouting introduces additional material into the wall system, pinning and soil nailing result in localized losses of fabric. There are other interventions exist such as reconstruction and geotextiles. However, soil nailing, pinning and grouting are common interventions at historic sites.

Though already considered as an intervention to stabilize structures in seismic regions, injection grouting is also used as a general measure to prevent incremental displacements produced by all dry stone structures. Developed for the purpose of increasing bond strength and redistributing loads, grouts injected into the dry stone wall fill voids and increase continuity [11].

The injected material varies and can include resins and polymeric and cementitious grouts. Efficiency depends on material compatibility, penetration within the wall to fill voids, and durability in a particular environment [12]. Though injection grouting has been found to improve strength, the process remains invasive as are the other interventions by changing the wall properties and aesthetic.

3.4. Seismic Response of Historic Stone Structures

Predicting seismic behavior in historic masonry structures presents innumerable challenges due to the variations in construction technique, existing conditions, the long history of adaptation and additions in many buildings and each structure's unique load patterns. Increasing accuracy of predicted behavior requires extensive knowledge of each factor. However, understanding general properties of masonry constructions allows for the anticipation of certain deformations or failure modes.

These failure mechanisms, largely the result of in-plane movement and out-of-plane bending, are informed by in-field assessments of a specific structure combined with knowledge of failures during past events or model simulations of the actual building system.

Field assessments and computer-generated models aid in identifying structural weaknesses that lead to failure under seismic loads. Predicting behavior allows for interventions prior to damage or collapse, and requires some accuracy in classifying possible failure modes. In-field assessments are limited by the correlation of past damage with the type of construction, prior weakness within the structure and magnitude of the seismic event. Similarly, computer simulations require inputs of force and geometries to calculate failure modes; existing conditions are a necessary component when quantifying damage mechanisms.

Measures to overcome limitations from computer-generated models involve assessments of existing conditions, nondestructive methods for identifying unknown load patterns and critical conditions, and modeling multiple failure mechanisms,

since failure modes are generally produced by dynamic actions during seismic events.

The ability to predict behavior has led to preventive actions that attempted to inhibit the failure mechanisms of buildings under seismic loads. Traditionally, engineers and conservators have formulated strengthening programs based on conforming to modern code which have allowed them to resist seismic loads in past events. Instead, engineers and conservators projected modern design standards onto these structures and molded the buildings into rigid, monolithic constructions without consideration of historic form, material compatibility or inherent resistance to seismic loads.

Two distinct approaches have been employed to understand seismic behavior of stone masonry structures with field analyses of failure modes and laboratory based numerical modeling and experimentation. The current research has advocated for an integrated methodology as the most accurate and effective process of predicting behavior and ultimately failure of masonry systems. Research is focused primarily in European countries of high seismic activity, such as Italy, Greece and Turkey where both monumental and vernacular structures are used as case studies.

A review of recent literature illustrates how in-field and laboratory techniques have transitioned to this more integrated approach. Through early in-field research, typologies of earthquake damage have been established to provide a basis for understanding and assigning causality to historic masonry in seismic regions. The laboratory process of determining seismic behavior has the use of discrete and/or finite element methods (DEM/FEM) to graphically represent structures and also the use of shaking tables to simulate the response of large-scale structures.

More recently, risk assessments have been developed as a means of both identifying construction weaknesses of buildings in the field and facilitating DEM and FEM representations by increasing the accuracy of the simulated models [6].

3.4.1. In-Field Observations of Seismic Damage

Developing damage typologies for stone structures in seismic conditions proved to be the simplest approach to understand the behavior. Langenbach's 1990 study analyzed construction techniques of masonry systems which survived previous seismic events [13].

The information provided insight into seismic-resistant construction for poor, rural regions where strengthening or retrofitting is not financially viable. Langenbach along with Erdik and his examination of stone buildings in Turkey formulated a vocabulary for assessing earthquake related damage in masonry systems, which was then used to interpret prior damage and differentiate seismic damage from general weathering patterns and deterioration [14]. Because of the accessibility of this research, studies have continued to develop more advanced damage typologies and expanded the research to include other masonry systems, such as adobe and brick [15], [16].

Several site investigations were made recently in order to get more information about the Augustus Temple surrounding (See Figure 15 and 16).

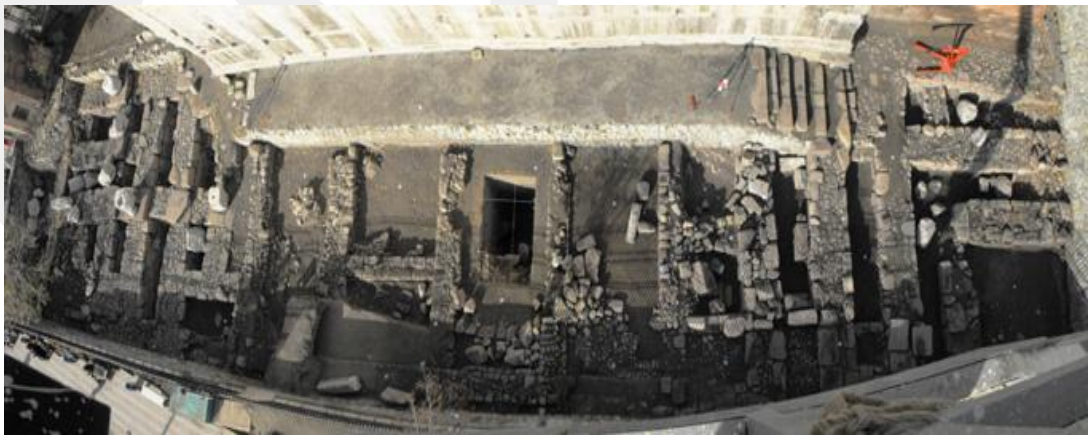


Figure 15 - Top view of the site investigation of the Augustus Temple near east wall



Figure 16 - Georadar investigation of Augustus Temple in front of the east wall

The Georadar output of the Augustus Temple site investigation revealed out a discontinuity at the ground underneath the west wall (See Figure 17).

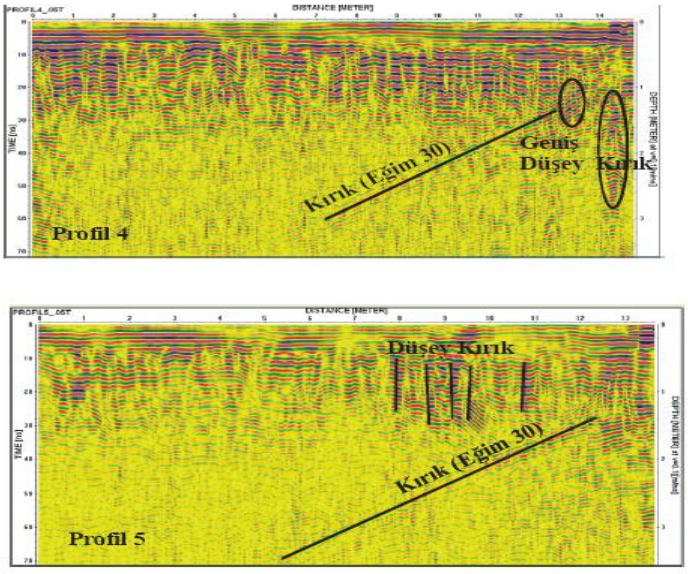


Figure 17 - The georadar output of the Augustus Temple site investigation

3.4.2. Numerical Modeling of Historic Stone Structures

The discrete element method (DEM) and finite element method (FEM) allow researchers to numerically model idealized or existing structures and chart seismic behavior. The DEM technique was originally applied by Peter Cundall in the 1970s to model the behavior of granular assemblies [17]. The engineering field adopted the technology to numerically model new and historic structures. Though DEM accurately simulates properties of new structures (since construction techniques and materials are known) there are limitations in its application to historic structures due to the complexities inherent in masonry assemblies. To overcome DEM's limitations, knowledge of the construction methods, weathering patterns and material properties must be ascertained.

Numerically modeled columns exhibited the complex geometries created by fluting of typical Greek columns, but failed to account for irregularities from weathering, material deficiencies or past interventions. As the DEM/FEM process developed, monumental structures were modeled with actual conditions shown in the simulation [18], [19]. Research focused on Greek temples to predict seismic behavior of unreinforced columns and also illustrated the altered behavior of the structure after seismic strengthening [20].

More recent published research on computer-generated modeling of seismic behavior has attempted to establish a multi-scale approach to simulation [21]. This approach requires identifying failure mechanisms at a macro or structural and micro (individual block) level. The initial construction of each model simulates behavior at corresponding scales and then allows for the macro and micro models to be combined into one multi-scale model.

Several large-scale experiments were conducted to assess properties of structural assemblies using shaking tables. Watabe et al. simulated the impact of seismic activity on the Parthenon columns to identify failure due to weathering [22]. Weathering significantly affects performance and can increase possible displacements caused by decreased interface between blocks, which lowers static friction.

Though the shaking table experiments provide invaluable insight into the actual performance of masonry systems, they are limited in scale and scope. In an effort to validate both simulation methods, Pagnoni applied a DEM model to a constructed wall, which was also subjected to a shaking table test. Pagnoni confirmed that the discrete element method was able to predict the actual behavior produced by the shaking table test [23]. Though Pagnoni verified the accuracy of the methods with known constructions; historic masonry systems maintain some limitations and require extensive research of construction techniques and materials to produce accurate results from DEM models.

During the past decade, the field of earthquake engineering has shifted some focus from computer-generated and laboratory modeling to in-field vulnerability assessments of individual structures as a measure of predicting potential failures. Binda et al. first argued for the need to understand actual building assemblies and current conditions before any modeling or interventions could be applied [24].

As a result, engineers and conservators designed several assessment methodologies to record construction methods and existing conditions; measured drawings were produced to examine geometries, past interventions and crack patterns of masonry walls. Nondestructive testing also served as a tool for investigating unknown building assemblies.

The vulnerability assessments are intended not only to assign safety values to existing structures but also to increase the accuracy of DEM models by supplying a much greater amount of information for each structure subjected to seismic simulations. Examples of vulnerability assessments to Augustus Temple's east wall were made by Culture and Tourism Ministry of Turkey can be seen in Figure 18 and 19.



Figure 18 - An archaeometric application (nanapool) to Augustus Temple's east wall



Figure 19 - Taking rock samples from the Augustus Temple

Mistler et al. have integrated the in-field assessments with the computer-generated DEM models of Aachen Cathedral to predict the behavior of the complex system under seismic conditions. Because the cathedral is composed of several different stone assemblies and exhibits various crack patterns, an accurate DEM model must simulate the behavior of each type of construction and project the anticipated changes and affects of the cracking [25].

The progress in computer simulations of seismic events indicates the potential of DEM in the field of conservation. However, research completed at Massachusets Institute of Technology suggests that seismic behavior is still not wholly understood. However, the current methods of assessing and simulating monumental buildings through DEM will allow professionals to design monitoring programs, calculate potential failures and determine the need for seismic strengthening.

3.4.3. General Properties of Historic Stone Structures

Seismic behavior varies according to building construction (e.g. single leaf/ multiple leaf structures, mortared/dry systems). Stone constructions carry certain general characteristics when subjected to seismic loads; however, these characteristics hinge on building type. Bell towers, aqueducts, free-standing walls, columns and arches exhibit unique behavior, since the design of these structures differs from basic building construction.

Typical stone masonry structures were historically built with extra thick walls to compensate for seismic loads and, as a result, up to 90% of the mass is contained within the walls [26]. Because of the incredible mass, many stone structures endure high amounts of deformation before failing. These structures experience the greatest susceptibility from horizontal loads, which causes out-of-plane bending; irregular or projecting components of a building such as L- or U-shaped areas of the building plan suffer more damage from horizontal loads [27].

Though rigid and well connected floors strengthen stone constructions, buildings with multiple stories show less resistance to seismic loads. Surveys

conducted following high magnitude events indicate the general resistance of historic structures to large seismic waves, likely due to their ability to deform heavily before collapsing. Modern buildings can be hindered by the rigidity of construction, which produces an inelastic, monolithic structure, and experience failure when subjected to severe vibration [13]. This difference is due to the extent of elasticity inherent in historic stone structures.

Dry stone walls exhibit unique mechanical properties that differentiate them from mortared systems. Rather than relying on mortared joints to lend cohesion, dry stone structures gain stability and cohesion through the friction of the joint contact interfaces. As potential energy stored in the system is released, small movements occur until the structure equilibrates [28].

Energy releases occur slowly over time and can be independent of seismic events; however, seismic waves can intensify the process and cause a sudden collapse. Cohesion in a dry stone wall is greatly reduced by large, vertical accelerations during seismic activity and more greatly affects the friction level of the joint interfaces than horizontal accelerations. This stick-slip action produced by either gravity or seismic waves may result in structural instability, deformation or collapse.

Results from large-scale testing of a dry stone wall illustrate the cracking patterns and failure mechanisms associated with ground movements. Shaking table experiments used to simulate seismic waves show the general behavior of the test wall. Stepped flexural cracks form under cyclic loads with inelastic sliding of the stones along the bed joints, which can cause a rocking mechanism to occur or can lead to shear failure [29].

Multiple leaf dry stone constructions add to the structural complexity and produce a relatively unpredictable behavior when subjected to ground movement. Typical components include outer butted or bonded veneers, a rubble core and tying mechanisms of stone or other material like wood or metal. Seismic waves affect the system by producing out-of-plane movement or bending of the wall which tends to separate the veneer from the core. The level of damage depends, in part, on the effectiveness of the tying mechanisms and pre-existing damage [30].

The critical element which enables historic masonry systems to survive earthquake loads is the tying mechanism. Connections between the various components provide greater stability in preventing displacement or collapse. For multiple leaf dry stone structures, proper tying mechanisms significantly increase stability during seismic events by aiding in energy dissipation.

Typical tying components include timber or iron cramps or through stones, which increased resistance to lateral loads. Conditions observed following seismic activity in Turkey indicate different levels of construction quality affect damage patterns. Because the tying mechanisms in multiple leaf walls increase stability, they have been identified in the field as being a critical component in a structure's ability to withstand ground movement [31].

High quality constructions are defined as stone systems containing regular stone courses laid in cement mortar with concrete tie beams demonstrate a significantly higher resistance to ground motion than irregularly shaped, random rubble structures constructed with low quality mortar and no tying element [14].

Though mortar increases binding properties and helps dissipate energy, dry stone systems require proper tying mechanisms to provide stability. Unlike dry masonry walls with no tying mechanism, dry walls which utilize timber beams or through stones endure much greater deformation before failing; without a tying element, the dry stone walls do not dissipate high enough levels of energy and can catastrophically fail [32].

3.4.4. Analyzing and Diagnosing Damage from Past Events

Even with knowledge of certain properties exhibited by stone structures, an element of unpredictable behavior remains during seismic events. Analyzing and classifying damage after a seismic event increases knowledge of behavioral patterns in masonry systems. Several studies have documented conditions resulting from earthquake damage and have made correlations between intensity of the event and

resultant damage. Non-destructive testing has also been employed to predict behavior by assessing existing conditions, such as crack patterns (See Figure 20).

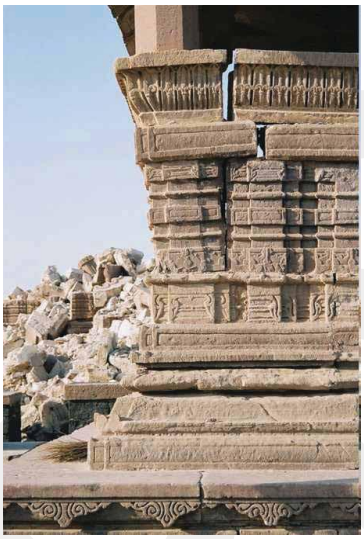


Figure 20 - Vertical crack produced by seismic action. (www.conservationtech.com)

In-field analysis of failed and damaged systems has provided insight into the vulnerabilities of certain structures and identified architectural elements most susceptible to damage. Common damage observed in many masonry structures includes vertical and stepped cracks and open joints. Vertical fractures generally occur around large openings like windows, doors and arches [15]. Diagonal stepped cracks form in dry stone and mortared masonry systems and result from inelastic sliding [29]. This cracking pattern corresponds with horizontal loads, which cause the linear deformation.

Other structural failures are specific to certain construction methods. Lateral loads applied to multiple leaf masonry systems emphasize any structural deficiencies. As previously mentioned, when not properly tied, the wall can separate or collapse with horizontal and even vertical forces.

These out-of-plane bending failures typically occur in brittle systems under horizontal forces. However, all buildings are subject to partial or full collapse due to inadequate connections or anchoring. Wall deformation or separation generally occurs in corners of poorly connected load bearing structures (See Figure 21).



Figure 21 - Failed corner of a multiple leaf system. (www.world-housing.net)

Roofing systems can also heavily influence behavior during seismic events. Both poor connection of roof and walls and thrust of the roof can lead to failure. Roof failure occurs from inadequate connection of the supporting walls. Inadequate connection of walls and roof can also lead to additional thrust on the walls and cause out-of-plane failure. Non-structural architectural elements are also susceptible to damage from out-of-plane mechanisms. The seismic effects on walls, especially cracking, can detrimentally affect parapets, cornices and spandrels and result in severe cracking or localized or total collapse of the elements.

Establishing correlations between damage types and seismic events requires prior knowledge of each structure's initial conditions. Records of the fabric prior to

an earthquake eliminate false correlations, since similar damage may occur from unrelated events, such as differential settlement or general neglect. The possible effects of differential settlement including large-scale cracking and tilting resemble are the products of seismic activity. Without records of preexisting conditions, assumptions must be made based on other evidence. Buildings constructed on solid bedrock allow for more accurate interpretation; if fractures extend through the building and bedrock, the condition likely results from seismic activity [16].

Understanding conditions due to prior damage coupled with knowledge of existing fabric offers an indication of seismic performance. However, sites with an extensive history or unique circumstances, present some complexity in using past and existing conditions to predict seismic behavior.

For instance, when analyzing seismic damage at archaeological sites, other factors are considered to determine whether damage occurred from a seismic event or from general aging. Geological evidence of past earthquakes, burial of biological elements, historical records from texts or images and complete destruction of settlements can indicate catastrophic seismic events with related structural damage.

Failure occurs not only from large-scale structural flaws or inconsistencies but also from problematic design of or changes in smaller details. Investigations of a building's connections, existing conditions and load patterns reveal the defects which produce failure during seismic events.

Existing conditions inform changes in load distribution or failure modes; cracks can indicate areas of weakness, which may not have existed previously, and are evidence of crucial structural changes. Identifying certain characteristics in structure geometries, construction techniques and physical, chemical and mechanical properties of materials is required to ascertain critical failure mechanisms [24].

Field studies of historic structures prior to seismic events have been established as a method of preventive action. After diagnosing and understanding parameters established to quantify damage potential, buildings are categorized by vulnerability. Assessments rank vulnerability based on a historic structure's relation to building codes, such as Eurocode 8, in order to define a standard safety factor.

Data collection regarding in-plan area, area to weight, number of stories, regularity of plan and length of walls and openings informs the vulnerability level and determines the safety factor [33], [34]. Factors or indices offer a low-cost prediction method to understand which buildings are susceptible to failure during a seismic event. Understanding possible failure mechanisms prior to an earthquake enables engineers to identify weaknesses and correct poor connections or implement larger strengthening programs; these assessments are sometimes augmented by computer generated modeling software to project vulnerabilities and further understand seismic behavior.

3.4.5. Discrete Element and Finite Element Methods of Modeling Historic Structures under Seismic Loads

Numerical methods for computing movements and the behavior of masonry structures emerged after Peter Cundall's 1971 development of a discrete element method (DEM) of monitoring the contact and motion of grains. This method of understanding grain interaction was later applied to larger structures, such as masonry columns, and modeled using computer simulations to understand movement when subjected to seismic activity.

Advancements in technology during the past decade have enabled engineers to construct complex computer-generated models, which simulate seismic conditions and predict the behavior of entire buildings during an event of a certain magnitude. Due to the unpredictable nature of earthquakes, the system is based on controlled conditions, such as wave type and magnitude. Many of these inputs relate to actual conditions observed and recorded from past events.

Anticipating seismic behavior offers two advantages: one corresponds to preventive conservation, which would allow conservators to anticipate possible damage and strengthen structural systems. The other advantage is to avoid unnecessary seismic strengthening, which in itself can be damaging to historic structures. The development of discrete and finite element modeling enables conservation engineers to simulate seismic strengthening of computer-generated

models to determine changes in or improvement of behavior during ground movement.

The modeling process requires certain known characteristics of the masonry structure before simulation. When considering basic forms of construction an accurate model can be generated by applying a finite element (FE) mesh (See Figure 22). The FE analysis is generated from inputs representing the geometries, materials, loading and boundary conditions of the wall construction [35].

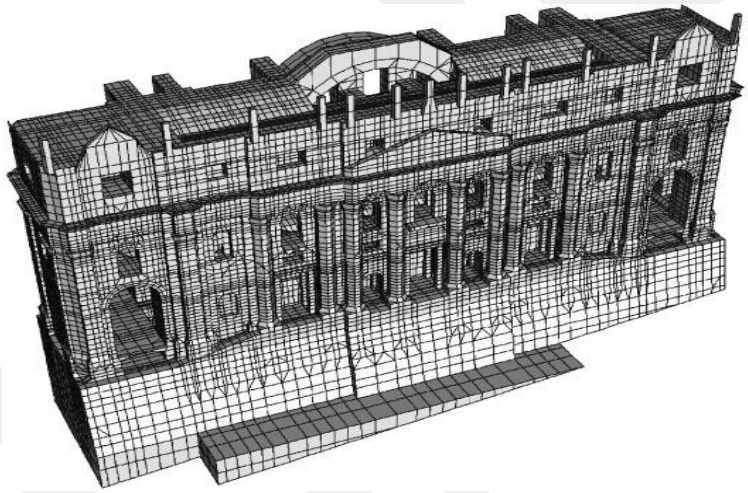


Figure 22 - Application of FE mesh to a historic structure. (Lourenco, 2002)

The accuracy depends not only on the ability to properly represent the material and conditions but also on grid density. Because of the complexities in even basic structures (e.g. presence of joints, variations in block sizes, heterogeneity of the material, etc.), some simplification occurs in each simulation. However, inaccuracies develop from poor data inputs. It is usually a condition of variations in wall width, unknown load distributions or intersections, use of composite materials and complicated geometries. Using a linear analysis, the shear and friction failure modes can be modeled at the macro level to show overall structural movement. Ainsworth

et al. generated a series of models to demonstrate linear failures in a simple dry stone construction [21].

A 1.00 meter square test wall with a thickness of 0.20 meters was clamped on a horizontal rigid surface; with uniform in-plane vertical and horizontal forces applied to the top of the wall surface, the model yielded a tensile failure. The failure occurred as diagonal stepped cracks - typical of dry stone walls. This commonly used macro model, which predicts the broad structural movements of the entire object, does not illustrate small-scale behavior of the individual components (See Figure 23).

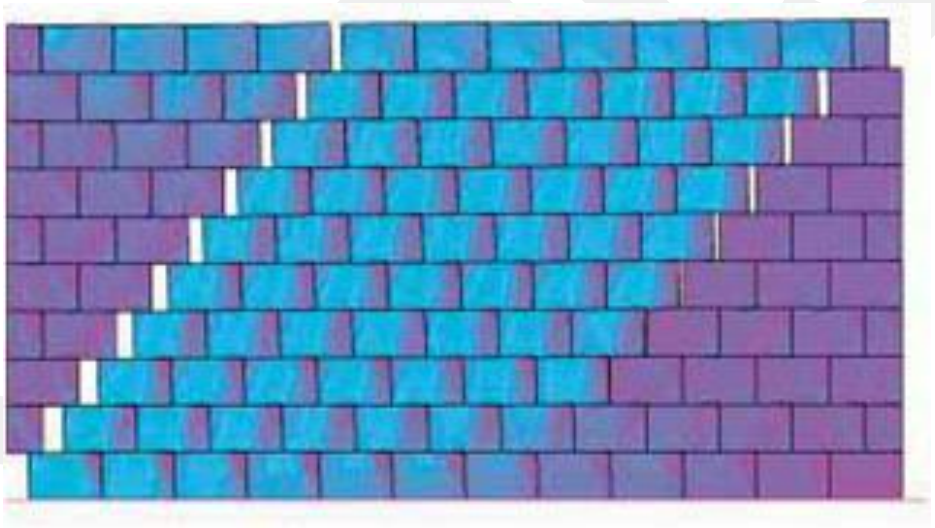


Figure 23 - FE model showing shear failure in a dry stone wall (Ainsworth et al.,2008)

In order to increase the accuracy of predicted behavior, Ainsworth et al. created a multi-scale approach, which combines overall structural movement and individual behavior of the stone units. The multi-scale approach better represents stress distributions and apparent dislocations, because the mesoscopic or micro-structural scale identifies movement, such as cracking and displacements, in the joint

interfaces. Though unrealistic to model an entire structure unit by unit, mesoscopic behavior of a limited sample area can be applied to the larger structure.

The combined macro/micro modeling systems utilize a homogeneous structure to understand displacement during seismic activity. Though this type of model generally applies to dry stone structures, mortared constructions contain different materials, each with distinct properties. The interaction of the differing materials is not always known, and as a result, can only be generalized through FEM modeling.

The development of micro analysis has enhanced the capabilities of modeling composite systems by allowing simulations to individually model the component materials. By modeling each unit, mortar face and interface separately enables methods limited to homogeneous constructions to predict behavior of composite systems when combining the results of each component. However, limitations exist in the practicality and accuracy of this method due to the difficulty in understanding wall geometries [25].

3.4.6. Application of DEM/FEM Analysis to Historic Stone Structures

Though more sophisticated modeling techniques have emerged in the past few years, models of historic structures require a more simplistic design to simulate behavior due to the restrictive size of most buildings. The amount of information needed for and received from simulation when using the intensive micro-scale approach is currently too excessive for large-scale structures. To compensate for the excess of information, simplifications in geometries are required to reduce the output while still achieving valid data.

When analyzing the accuracy of results generated by FE models, test walls were constructed and seismic waves simulated by a shaking table or other means. The same test methods were employed to confirm the accuracy of FEM results when applied to historic structures. Large-scale tests conducted in conjunction with FE

models enabled researchers to establish correlations between real movement and modeled movement.

Simple ashlar wall (dressed stone work) constructions, arches and columns have been built and modeled to understand the relationship between numerically predicted and actual behavior. The tests indicate that the simple dry stone wall constructions and arch systems have very predictable failure mechanism when subjected to rocking and harmonic shaking; these large-scale experiments were accurately modeled using a DEM/FEM analysis [23]. However, freestanding columns contain more complicated geometries and are more susceptible to changes in performance due to slight variations in inputs including geometry, structural properties or force from seismic load.

Many studies have been conducted to investigate the seismic performance of ancient Greek columns at the Parthenon. The results of these studies have contributed to both the understanding of column behavior and the limitations of numerical modeling. Though the numerical models accurately represent the types of possible failure mechanisms produced by rocking and sliding, too much variability exists to correlate peak ground acceleration with collapse.

Psycharis et al. analyzed the response of a simplified classical column subjected to seismic events of varying magnitudes. The study demonstrated changes in behavior due to different seismic inputs. Typical involves displacements and rotation of the lower joints due to rocking with the upper blocks moving as a single unit [20]. The highly non-linear nature of movement infers that response varies greatly with differing force. Changing the earthquake input shows an altered behavior where sliding occurs at the top of the column rather than at the base (See Figure 24).

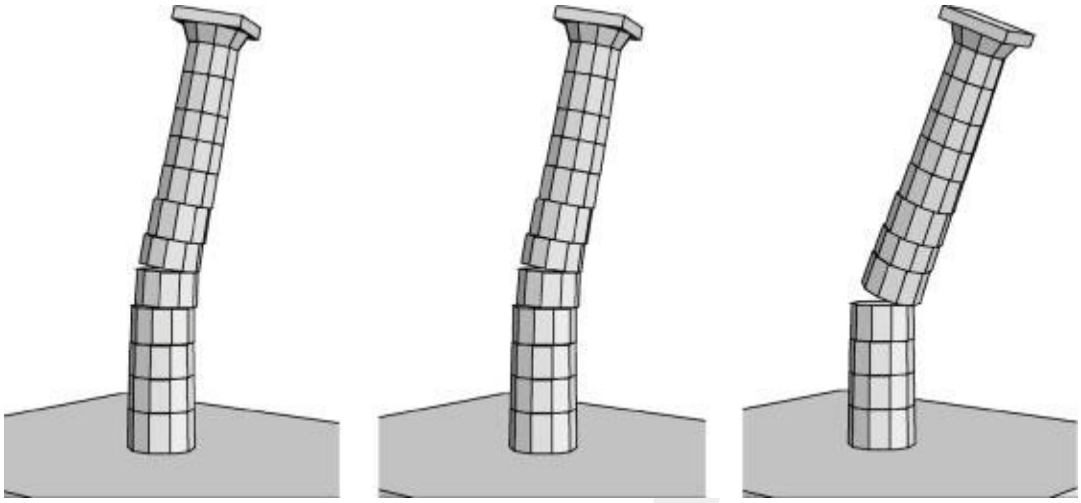


Figure 24 - Simulation of column showing failure at the base (Psycharis et al.,2003)

More important to understanding the behavior of historic structures is the difference in response due to weathered surfaces and existing conditions. Models represent these conditions through simplifications, such as reducing joint interface by rounding corners or splitting blocks into multiple units to characterize cracks [36].

The simplifications account for the basic deterioration of masonry units, but cannot accurately represent the weathering patterns caused by innumerable mechanisms during years of exposure to environmental conditions. However, simplifying weathered surfaces does largely affect the failure mode in seismic simulations and better approximates where failure will occur. Lowered joint interface reduces the necessary friction required to minimize sliding; simulations replicate joints displaying conditions from loss or cracks and demonstrate the resultant structural failure [20].

When applied to historic building systems, simplifications of weathering patterns and existing conditions pose multiple complexities but still allow for the rendering of generalized failure mechanisms. Producing a historic model requires several inputs: geometry of internal and external elements, construction (including tying mechanisms), core material (if present), crack patterns and other existing conditions. This initial state requires intensive assessment and is difficult to accurately represent due to the many unknown conditions of historic masonry

systems. Many load patterns have shifted since their initial construction; structural beams do not always carry the apparent load. Loads tend to shift to exterior walls, which cannot be easily detected, but greatly impact the structure's seismic behavior (See Figure 25).

Understanding geometries presents a further challenge, since original construction methods may not be known. In-field observations have proven the importance of tying mechanisms (particularly in multiple leaf constructions), so knowledge of the structural system and the component connections increases the accuracy of historic models.

Improperly tied systems provide significant points of weakness in buildings, which subsequently leads to failure at those intersections; strength of these connections is necessary prior to generating models. As mentioned previously, modeled simulations largely rely on homogeneous systems to predict material behavior, which can be problematic in historic structures. Many historic structures contain numerous materials, sometimes resulting from multiple additions.

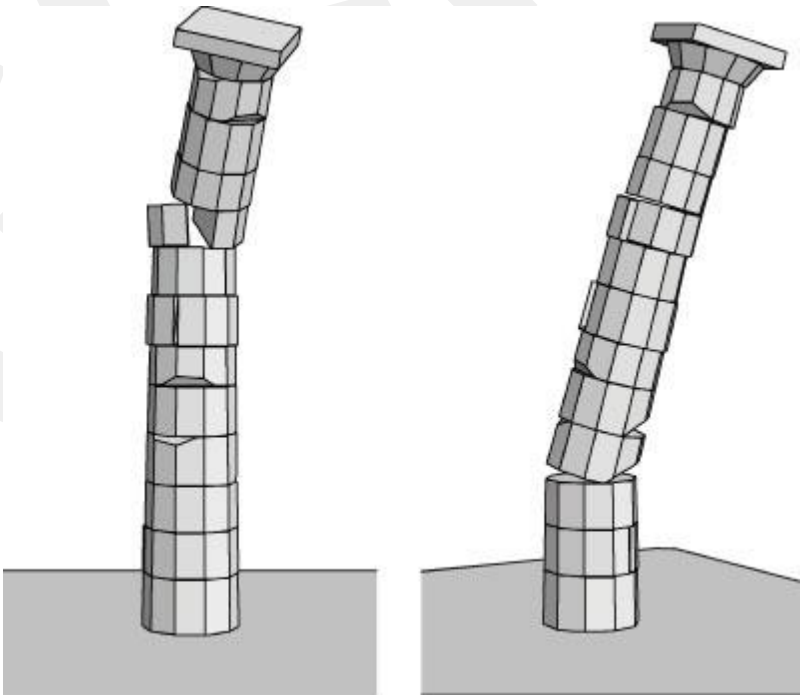


Figure 25 - Changes in failure due to existing conditions (Psycharis et al., 2003)

One method to account for varying construction periods and materials requires modeling individual sections of a building. Simplifying each section by homogenizing the materials allows a more accurate system to be modeled; each modeled component can then be synthesized into a single structure to identify overall movement of the building (See Figure 26).

Even without precise data relating to construction, general failure mechanisms emerge and provide an awareness of the building's structural strengths and weakness. This data enables engineers to adjust the inputs like seismic load, material, structural load, etc. to experiment with a building subjected to various conditions. For historic buildings that exhibit failure mechanisms resulting from poor construction, deteriorated structural materials or emerging conditions, models can simulate the effects of seismic strengthening techniques, which greatly alter seismic response.

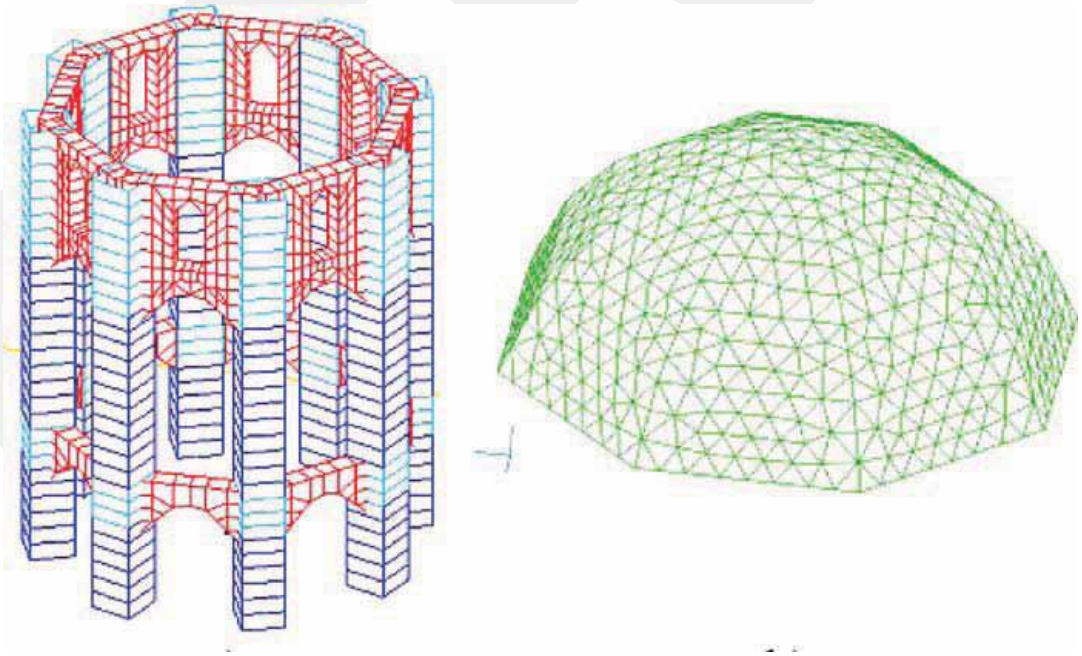


Figure 26 - Individual meshes applied to building components of the Aachen Cathedral, Germany (Mistler et al., 2006)

The limitations in accuracy of computer-generated simulations create a need for more complete data in order to predict behavior of historic stone structures. Nondestructive testing of connections, load patterns and materials would greatly reduce the unknown properties and constructions in historic systems. Nondestructive testing methods such as thermography, sonic, radar, X-ray, flat-jack, hardness, penetration and pull-out tests can supply information on voids within the system, load distributions, connective components and basic internal construction techniques.

The benefit of revealing those undefined factors that have been typically associated with failure during seismic events is the pronounced increase in accuracy when predicting behavior and formulating intervention strategies. Eliminating unknown factors within historic structures improves the ability to prevent future catastrophic failure. [6]

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CHAPTER 4

MODELING

4.1. Overview

The 3DEC (3-Dimensional Distinct Element Code) program is the three-dimensional extension of Itasca's two-dimensional code UDEC. 3DEC is a numerical modeling code for advanced geotechnical analysis of soil, rock, and structural support in three dimensions. 3DEC simulates the response of discontinuous media such as wall blocks that is subject to either static or dynamic loading.

In contrast with classical methods like Finite Element and Boundary Element Methods that treat the medium as a continuum, the Distinct Element Method treats the medium as a discontinuum. The emphasis of Distinct Element Method is therefore in reproducing the mechanics of contacts and impacts between distinct blocks. The method is well suited to analyze diverse problems at various scales in civil engineering.

PLAXIS is a Finite Element software for geotechnical applications in which soil models are used to simulate the soil behaviour. Development of PLAXIS began in 1987 at the Technical University of Delft as an initiative of the Dutch Department of Public Works and Water Management. Because of continously growing activities, a company named PLAXIS B.V. was formed in 1993. In 1998, the first PLAXIS version for Windows was released.

PLAXIS is intended to provide a tool for practical analysis to be used by geotechnical engineers who are not necessarily numerical specialists. Practising

engineers consider non-linear finite element computations cumbersome and time-consuming. Many geotechnical engineers world-wide have adopted the product and are using it for engineering purposes. PLAXIS is a well-known software and there is no need to give detailed information about it. Only the main features of 3DEC software are given in section 4.2 to give a better idea about the software's capabilities.

4.2. 3DEC Main Features

- Discontinuous medium modeled as an assemblage of convex polyhedra or concave polyhedra formed by joining together convex polyhedra; blocks may be rigid or deformable
- Discontinuities treated as boundary conditions between blocks
- Motion along discontinuities governed by linear and non-linear force-displacement relations for movements in both the normal and shear direction
- Material models include: elastic, anisotropic, Mohr-Coulomb, Drucker Prager, bilinear plasticity, strain softening, creep, and user-defined
- Modeling attributes may include absorbing boundaries and wave input for fully dynamic response; "null" blocks for excavation and backfill simulation; structural elements with general coupling to blocks
- Extensive, fast interactive menu-based 3D OpenGL on-screen graphics. Available plots include: joint structure; block structure; displacement and velocity vectors and contours; joint plane displacements and stresses; structural element displacements and stresses; and, fluid flow apertures and velocities. Hard copy graphics are available in several industry standard formats
- Utilities include a preprocessor that reads AutoCAD files and outputs 3DEC data files for polyhedra generation; tunnel generator and statistically-based joint-set generator

- Built-in programming language (FISH) provides powerful flexibility to customize analyses
- Explicit time marching solution that provides realistic path-dependent post peak failure behavior in joints and plastic zones
- Modeling "infinite domain" problems using inner/outer region coupling and automatic radially-graded mesh generation
- Effective stress calculation performed using pore-pressure gradients
- Joint fluid flow with thermal and mechanical coupling
- Analytic and numerical thermal calculations with mechanical and fluid coupling
- Structural elements with cables, beams, and liners [37]

Dynamic option models the full dynamic response of a system in the time domain. Capabilities added to standard 3DEC include: specification of velocity or stress-wave input; quiet boundaries; free-field conditions; and damping (mass-proportional, stiffness-proportional, and Rayleigh). Problems such as seismic loading, explosive loading, seismic release of energy, and flow of particles may be modeled.

Discontinuities are treated as boundary conditions between blocks. Large displacement on the discontinuities such as slip and opening is simulated in a discontinuous medium. Relative motion along discontinuities is governed by linear and non-linear force-displacement relations for movement in both the normal and shear directions. The program uses an explicit solution scheme, which gives a stable solution to unstable physical processes.

3DEC is particularly well suited to simulate blocky structures, such as stone masonry arches. Assessment of the safety conditions of old masonry bridges and the seismic behavior of stone masonry arches has been done using 3DEC. It has been successfully employed to simulate the behavior of a concrete arch dam constructed on a jointed rock foundation and also to perform stability analysis of underground powerhouse station.

4.3. Comparison of PLAXIS and 3DEC

PLAXIS software seems more user friendly than 3DEC software and that is maybe why PLAXIS is more popular than 3DEC in geotechnical engineering projects. However, 3DEC seems more accurate because it lets users to input more realistic structure and it accounts for the interaction between the parts within the structure.

Getting graphics and results are also easier in PLAXIS. However, 3DEC can give 3D view of a result of any paramater and this helps to visualize the end results. The real earthquake loading scenario should be entered and worked on it carefully. PLAXIS lets user to enter the earthquake loading simply, but the representation of real earthquake is somewhat questionable. 3DEC software may give closer results to the real life situations, but user should be an expert to enter and use the commands needed to run the software.

Both softwares have weaknesses and strengths. Therefore, geotechnical engineers should select the software to use in their projects or researches according to their own engineering judgments.

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CHAPTER 5

ANALYSIS RESULTS OF NUMERICAL MODELING

The static state is an initial condition for the dynamic analysis. The non-linear analysis performed with 3DEC offers an interesting insight into various failure mechanisms that can develop as the blocks slide, slip and open during static and dynamic loading. The 3DEC results were used in the DISPLAY program to plot deformed shapes. The analysis was done by using 3DEC Version 4.00 to show the effect of the foundation and earthquake parameters to the more than 2,000 year old and still standing historical Augustus Temple's east side wall.

Damping was used to simulate real earthquake effect acting on the wall and it gives more deflections comparing with no damp analysis. Stiffer structure is absorbing the forces better, but gives more deflections comparing with no damping analysis. Damping formula was taken from a dam foundation example in FLAC3D manual. As a result, analysis has to be done by using damping. The analysis boundaries (edge coordinates) are close to the wall and selected using similar examples in 3DEC manuals.

The earthquake loading was performed by using the commands in Figure27 for 3DEC analyses. There several factors that were changed and compared to see the effect on "y-displacement" values. These values are frequency, amplititude, attack-decay time and damp values. The files have ".dat" extension to run. "call" function is used to run the file and "plot" function is used to see the analysis results as graphically.

```
def setup

omega = 2.0 * pi * freq

o_env = 2.0 * pi / env_time

end

def wave

if dytime > env_time

wave = 0.0

else

wave = 0.5 * (1.0-cos(o_env*dytime)) * ampl * sin(omega*dytime)

endif

end

set freq=1.0 ampl=8.0 ; (0.4 g)

set env_time= 3.0 ; (6 sec attack & decay)

setup

damp 0.05 200 stiff
```

Figure 27 – An example of commands section used to simulate earthquake loading for 3DEC analysis

0.2-0.3g values of frequency and amplititude were used to show an earthquake. In the analysis, the three components of the earthquake are applied simultaneously. A quiet period of two seconds and six seconds were used to check that the relative movement to joints had stopped after the earthquake. The input acceleration record was integrated to obtain the velocities. The velocities were used as the primary seismic input for the 3DEC analysis.

Some of the assumed values of the wall, foundation and soil properties for the analysis are listed below in Figure 28. The way of inputting the values differs between PLAXIS and 3DEC. Therefore, closer values of material properties are selected in order to compare both analysis results.

	PLAXIS	3DEC
MATERIAL MODEL	Mohr-Coulomb	Mohr-Coulomb
WALL	γ_{unsat} = 30 kN/m^3 γ_{sat} = 30 kN/m^3 k_x = 1m/day k_y = 1 m/day E_{ref} = 5*10^7 kN/m^2 C_{ref} = 1,000 kN/m^2 $\Phi = 30^o$	dens = 0.0027 kiloton/m^3 k = 20,833 MPa G = 22,727 MPa kn = 50,000 MPa/m ks = 20,000 MPa/m fric = 30^o
FOUNDATION	γ_{unsat} = 30 kN/m^3 γ_{sat} = 30 kN/m^3 k_x = 1m/day k_y = 1 m/day E_{ref} = 1*10^7 kN/m^2 C_{ref} = 1,000 kN/m^2 $\Phi = 30^o$	dens = 0.0027 kiloton/m^3 k = 15,000 MPa G = 16,000 MPa
SOIL	γ_{unsat} = 20 kN/m^3 γ_{sat} = 22 kN/m^3 k_x = 0.01 m/day k_y = 0.01 m/day E_{ref} = 1*10^7 kN/m^2 C_{ref} = 100 kN/m^2 $\Phi = 30^o$	dens = 0.0020 kiloton/m^3 k = 10,000 MPa G = 11,000 MPa

Figure 28 – Assumed values of the wall, foundation and soil properties for PLAXIS and 3DEC analysis

Figures 29, 30, 31 and 32 are given as examples for the 3DEC software inputs and outputs.

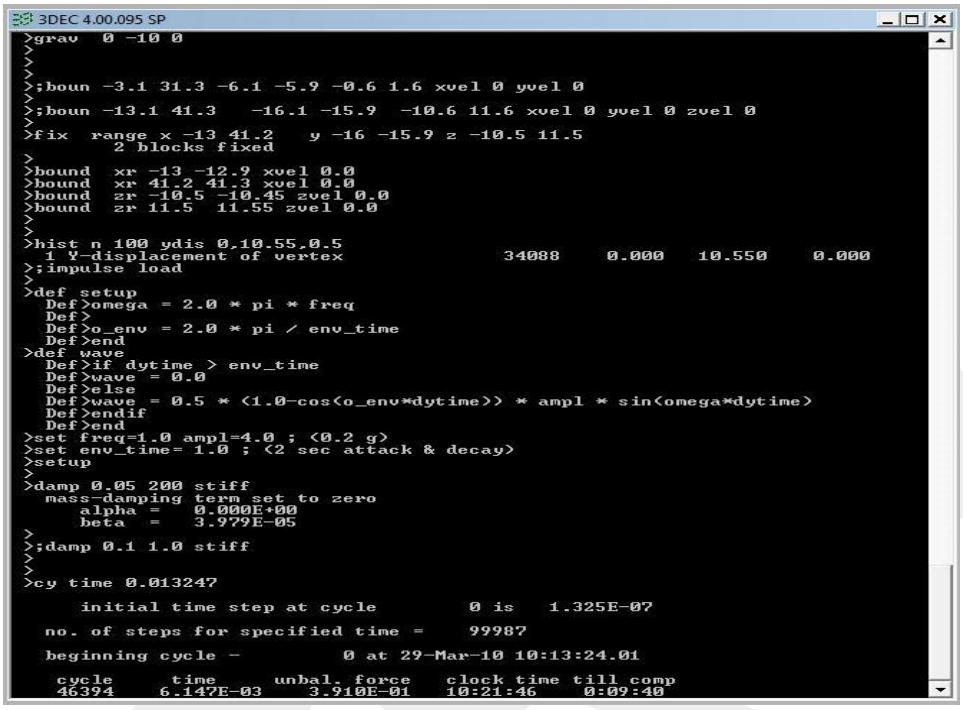


Figure 29 – A screen-shot of 3DEC Software Run

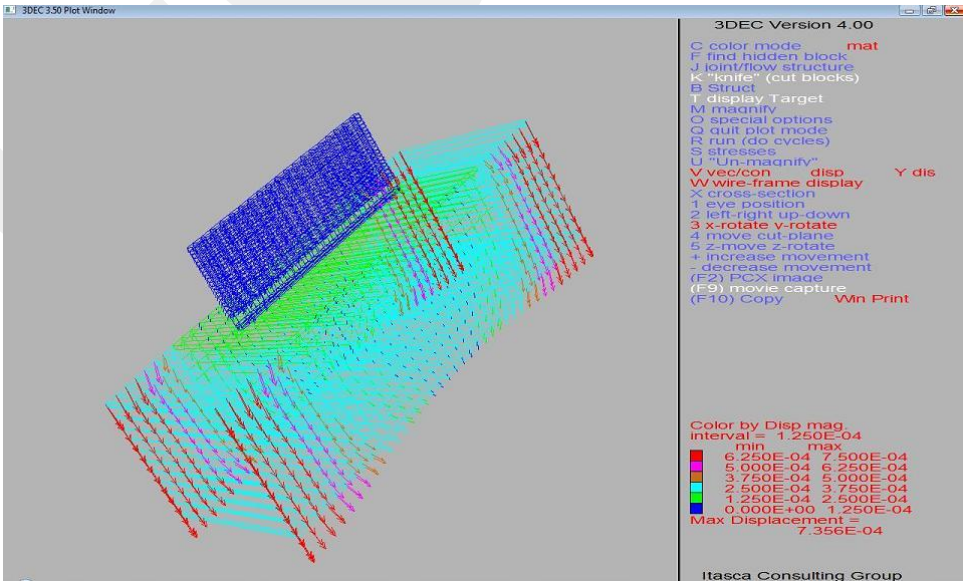


Figure 30 – A screen-shot of 3DEC Software Plot Option

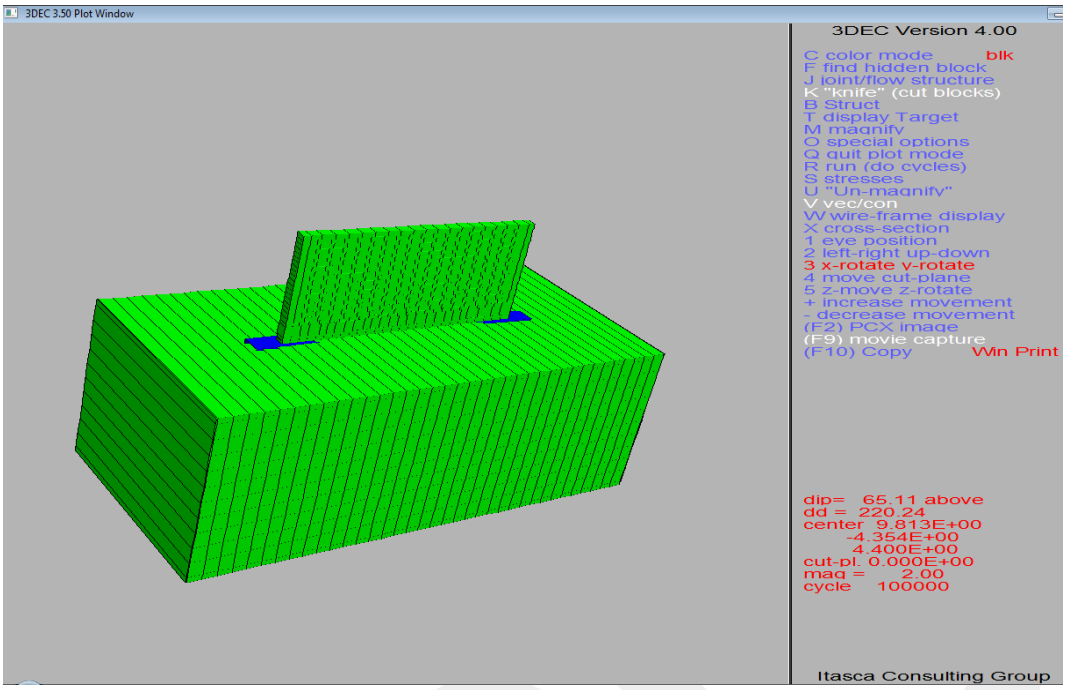


Figure 31 – General view the east side wall and surrounding boundary (3DEC)

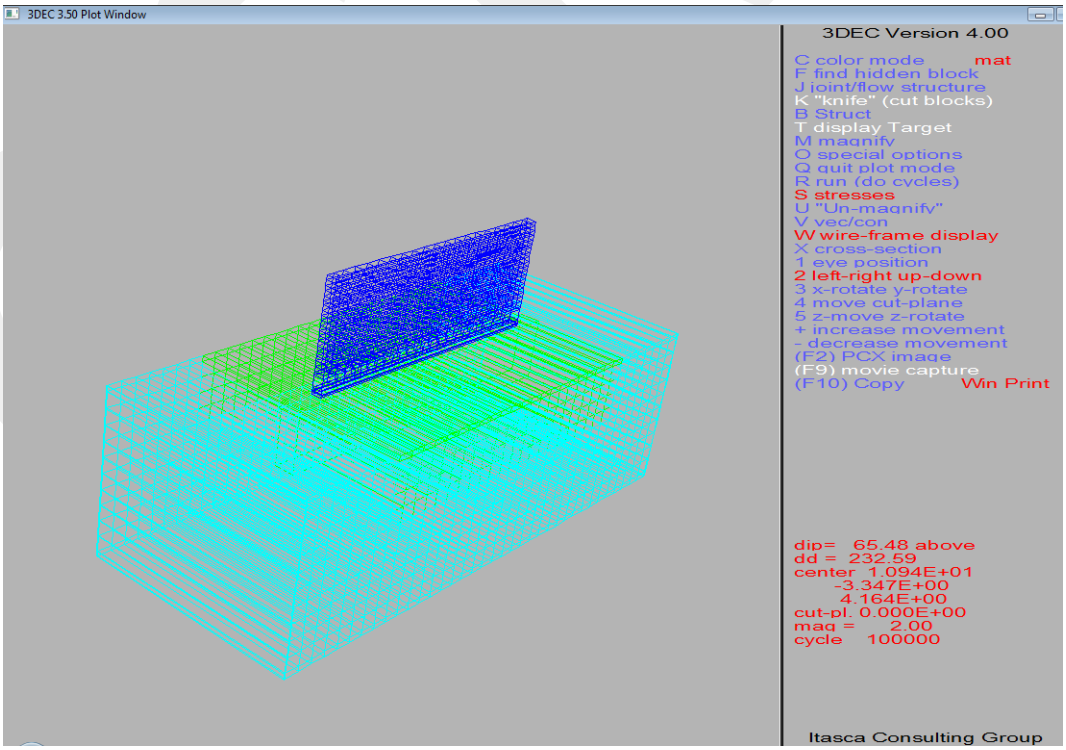


Figure 32 – General view of the east side wall and surrounding boundary with foundation (3DEC)

The analysis was done by using PLAXIS Version 8.2. The earthquake loading was performed by using amplititude and frequency factors for PLAXIS software. A value of 4 for amplititude and a value of 1 for frequency are used to compare the results of PLAXIS results with 3DEC results.

One direction and both direction earthquake loading scenarios were also performed to get a real earthquake effect on the structure. Figures 33, 34, 35, 36, 37 and 38 are given as examples for the PLAXIS software inputs and outputs.

The results for PLAXIS and 3DEC software analysis are listed below in Figure 39 and 40. The comparison and discussion of the results are given in the Conclusion Chapter.

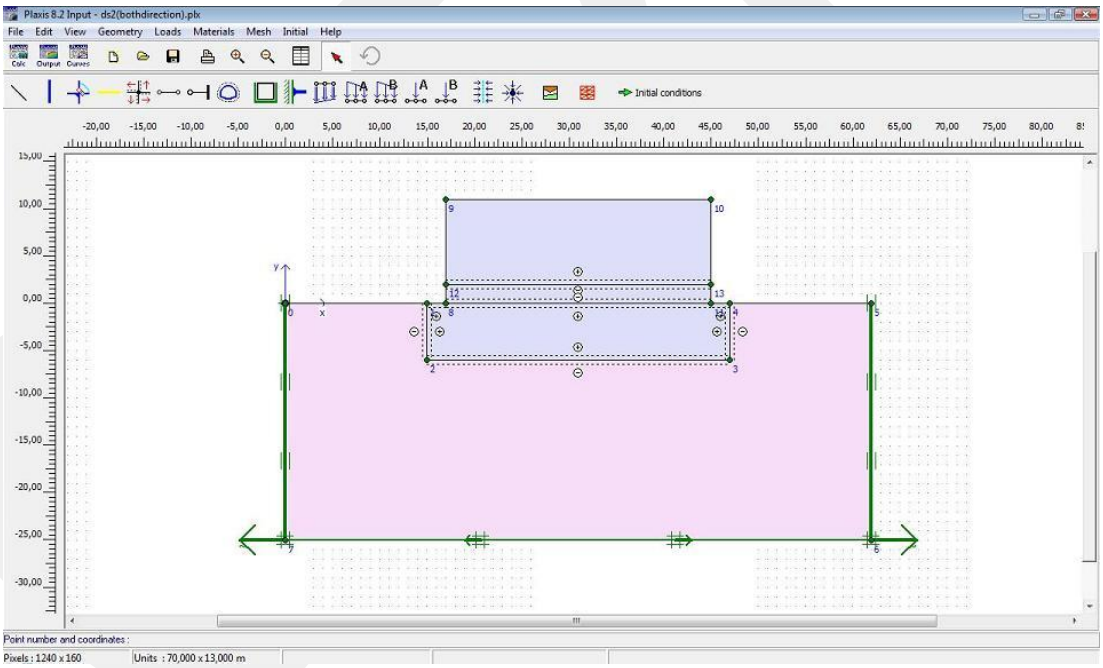


Figure 33 – General view of both direction earthquake loading (PLAXIS)

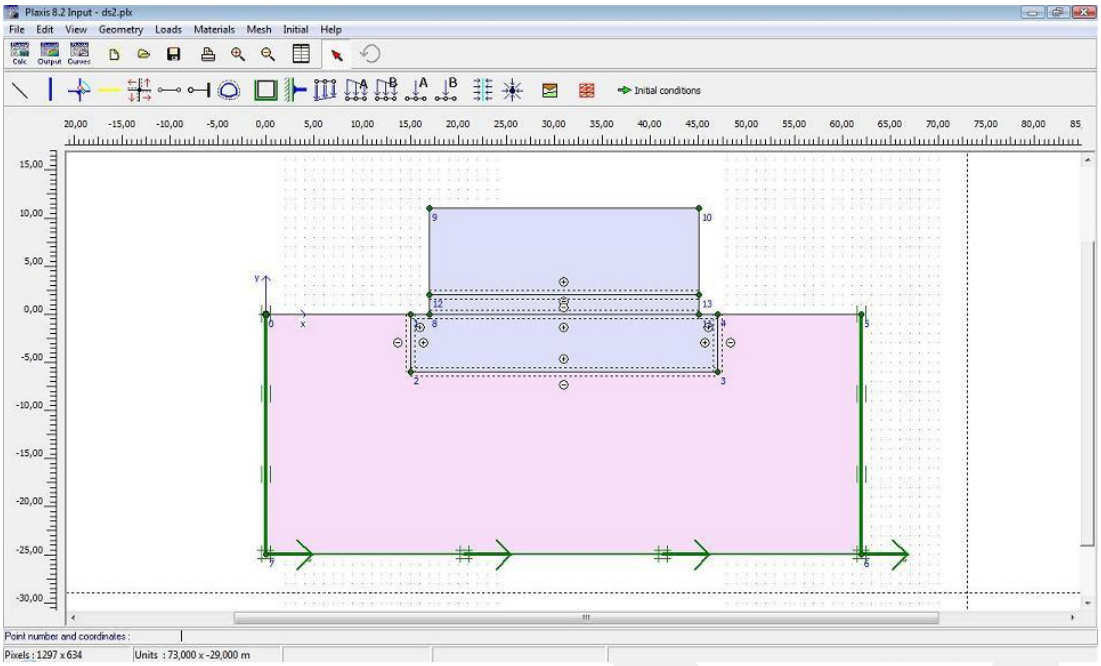


Figure 34 – General view of one direction earthquake loading (PLAXIS)

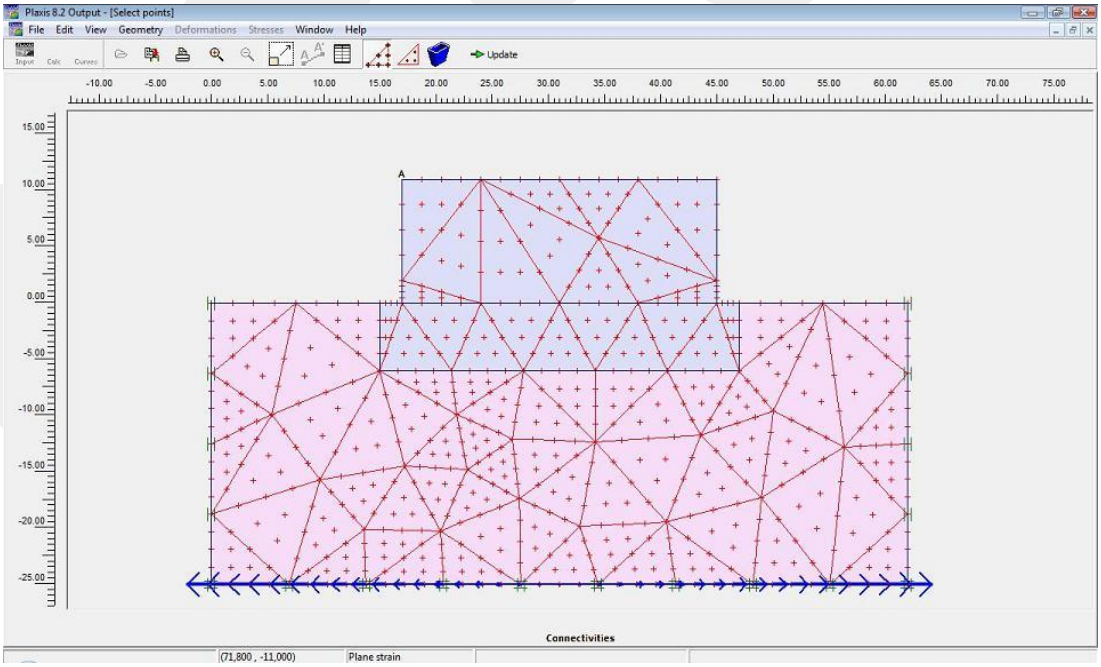


Figure 35 –Mesh view of the structure (PLAXIS)

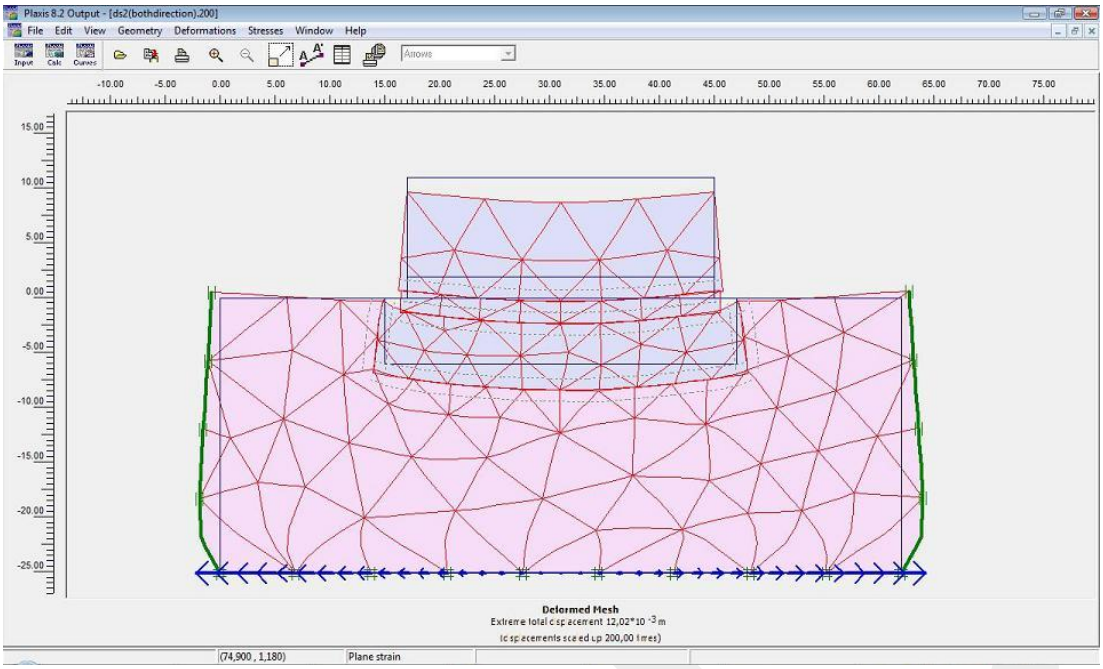


Figure 36 – An example for deformed structure after analysis run (PLAXIS)

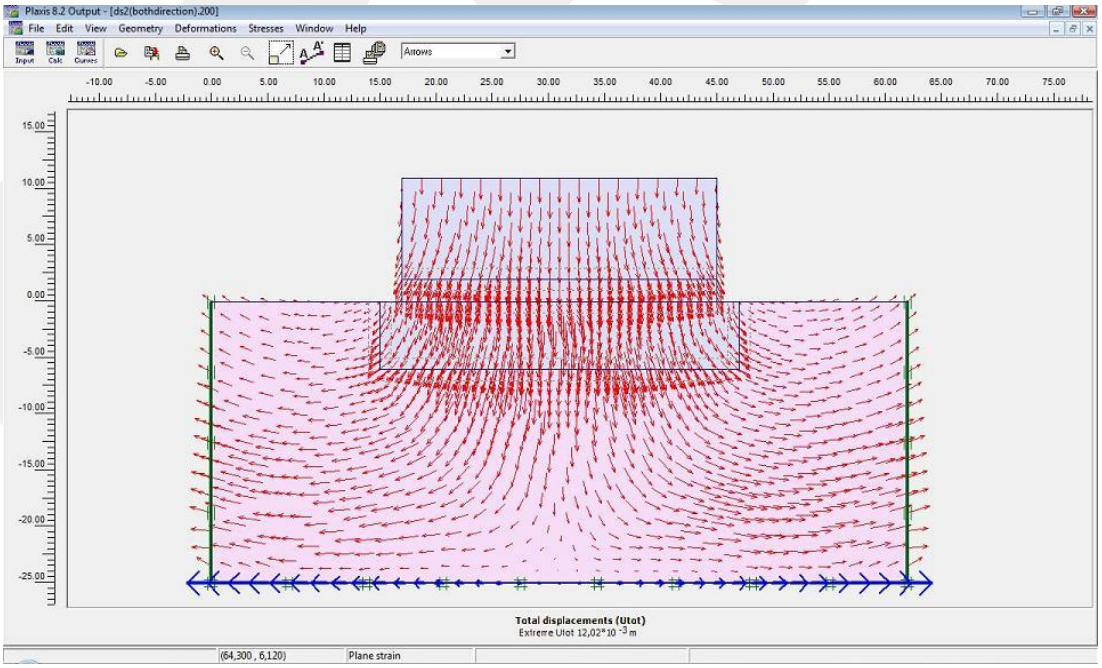


Figure 37 – Screenshot of total displacement representation with arrows (PLAXIS)

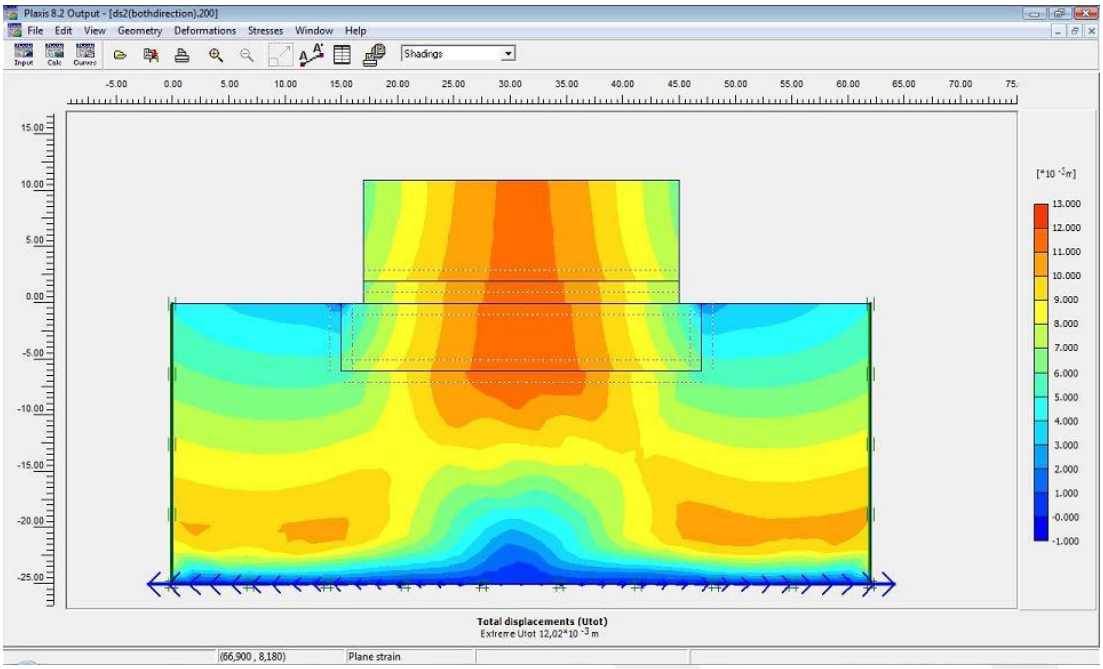


Figure 38 – Screenshot of total displacement representation with contours (PLAXIS)

PLAXIS ANALYSIS	File name	Amplititude	Frequency	Earthquake Load	y displacement (m)
Interface	ds2(both)	4	1	both directions	23.69*10-3
No interface	ds(both)	4	1	both directions	25.03*10-3
Interface	ds2(both)	6	1	both directions	31.16*10-3
No interface	ds(both)	6	1	both directions	32.34*10-3
Interface	ds2(both)	4	2	both directions	33.86*10-3
No interface	ds(both)	4	2	both directions	36.09*10-3
Interface	ds2	4	1	one direction	188.39*10-3
No interface	ds	4	1	one direction	56.99*10-3
Interface	ds2(both) large	4	1	both directions	25.36*10-3

Figure 39 – Summary of PLAXIS Software Results

3DEC ANALYSIS	File name	Amplititude	Frequency	Env time	damp value	y displacement (m)
with foundation	1-4amp	4	1	1	0.05 and 200	7.356*10-3
no foundation	1-4amp-nofound	4	1	1	0.05 and 200	8.777*10-4
with found.(rigid)	1-4amp-rigid	4	1	1	0.05 and 200	6.478*10-5
with foundation	1-6amp	6	1	1	0.05 and 200	7.356*10-3
with foundation	1-4amp-fre2	4	2	1	0.05 and 200	7.356*10-3
with foundation	1-4amp-4env	4	1	3	0.05 and 200	7.356*10-3
with foundation	1-4amp-damp01	4	1	1	0.1 and 200	2.246*10-4
with foundation	1-4amp-damp400	4	1	1	0.05 and 400	6.135*10-4

Figure 40 – Summary of 3DEC Software Results

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CHAPTER 6

RECOMMENDATIONS FOR FUTURE TESTING AND MONITORING

Recommendations will be offered to inform future investigations and conservation work on the Augustus Temple. Based on the background research and condition assessment presented in this thesis, unknown elements pertaining to wall construction, stone properties and wall behavior have been identified and require further examination. In order to increase available knowledge and implement an effective conservation program for the Augustus Temple, the following research should be conducted:

- Laboratory testing of stone and soil samples
- Structural monitoring
- Seismic modeling to predict structural behavior

The wall's vulnerabilities mainly relate to load distributions, weathering and seismic activity. The recommended areas of research will reduce or eliminate many of the current uncertainties concerning stone strength, material loss due to weathering, response to ground movement, displacement and soil-structure interaction.

6.1 Laboratory Testing

Testing of the wall materials to ascertain porosity, density, elasticity and compressive strength, will allow for the quantification of certain behavior.

The following tests should be conducted to determine material properties:

- Water absorption/desorption test
- Freeze/thaw test
- Compression and three-point bending tests of stone wet and dry, parallel and perpendicular to the rift
- Soil volume expansion

The general aging and weathering process alters material properties by increasing surface permeability, which accelerates weathering in stone. Weathered surfaces more quickly absorb moisture through the pores. These properties determine the material's susceptibility to water-related damage and also provide some indication of the degree of change possible in the stones' frictional properties. Because water decreases friction, testing should be conducted to ascertain the effect of water on stone cohesion and quantify the reduction of friction at the stone interface. Understanding conditions contributing to material loss are critical in the wall's stability. The conditions relating to material loss which were recorded during the survey included spalling and split faces.

Mechanical tests, such as three-point bending and compression, determine the elasticity and strength of the stones. Most risks related to strength of the individual stone units and the larger structure. Since being exposed to environmental conditions, the stones remain vulnerable to compressive loads in localized areas, but are also now subjected to mechanical and chemical weathering.

6.2 Monitoring Structural Changes

The diagnosis of current conditions serves as the basis for implementing a future monitoring program. Data collected from monitoring devices requires

interpretation; however, effectively implementing the system and analyzing data will allow correlations between current conditions and their causes to be identified or confirmed. Though many conditions were found to be historic, monitoring in these areas is necessary to determine whether displacements remain active.

The requirements of the monitoring device are location specific. However, general criteria necessary to determine the monitoring system include:

- Identify what needs to be measured
- Establish the range or area of the measurement
- Determine the resolution
- Identify where to position the device(s)
- Ascertain the accuracy required
- Ensure that the measurement is repeatable [38]

Identifying what is to be measured is the first step in implementing a monitoring system. Several factors affecting the wall's structural stability - displacement and material loss - require different monitoring programs. Also, multiple factors contributing to displacement in a single area may also necessitate the use of several devices to record type, direction and rate of movement. Devices to measure erosion or other detectible material loss may be necessary for a more complete assessment of emerging or current structural instabilities, since material loss in the core affects settlement and, consequently, displacement of the veneer.

The range and resolution refer to specific factors related to the size of the measured area in distance and the increment of measurement needed to determine the type of device necessary to capture change. Dry stone structures experience very small, gradual displacements as the static friction inherent in the system periodically changes to a dynamic state.

The brief transitions from static to dynamic friction can result in sub-millimeter displacements. However, areas within the multiple leaf system may experience larger movements, such as those recorded during seismic events or when the loss of material within the rubble core causes larger settlements. A monitoring system must be capable of capturing the long-term, sub-millimeter and larger, seismically-induced movements to correlate cause with amount of movement.

Device placement is critical for obtaining the necessary data to confirm cause-effect relationships. Measuring movement at open joints, existing cracks and bulges typically provides information related to shear and localized out-of-plane displacements. Rotation presents another measurable type of displacement noted at the site. Measurements detecting change in the wall's angle will determine whether the historic movement remains active.

The types of measurements needed warrant more advanced devices in addition to the relatively simple and low-cost methods of analysis. However, it is important to note that though the low-cost systems already implemented have lower resolution and limited range, all monitoring devices maintain some source of error.

High-cost monitoring technologies do not guarantee accurate results, nor are they more easily repeatable. The use of multiple devices and methods reduces inherent error to record more accurate and repeatable measurements at the proper range and resolution. Even with a program designed to measure various factors contributing to movement, limitations exist in using monitoring to delineate cause and effect relationships in multiple leaf structures. Several factors which produce similar conditions can be difficult to isolate.

The complexity of multiple leaf walls underscores the necessity for a more sophisticated monitoring program to understand the critical factors governing the wall's stability. Because failure in dry stone constructions is not well understood or predictable, the amount of displacement tolerated cannot be quantified. However, obtaining the rates of current displacements provides some indication of degree of stability and urgency of intervention to prevent collapse. Using monitoring systems to calculate the rate and causes of displacement will inform future conservation programs of the wall.

6.3 Modeling Seismic Behavior

Once general properties and long-term behavioral patterns are established through testing and monitoring, computer generated modeling for seismic behavior is

recommended to increase knowledge of structural movement caused by varying degrees of ground movement. Though the models contain various inaccuracies and require specific inputs relating to current conditions, construction, and magnitude of force, they illustrate general areas of vulnerability within a structure and serve as a basis for understanding the effects of past and future interventions.

The limitations of modeling accuracy and unknown factors of wall construction must be considered when employing this method as a predictive tool. Modeling provides a valuable demonstration of the structure's response to possible future interventions. Because the conservation program at Augustus Temple operates with the goal of minimal intervention, the ability to predict behavioral changes from structural modifications can eliminate inefficient methods of stabilization. [6]

6.4 Further Research

The tying mechanism has been identified as a critical component in multiple leaf constructions, and although lead connectors were discovered between blocks, their frequency and binding strength remains unknown. The high frequency of ties within a system greatly improves stability; non-destructive techniques to investigate the number and placement of the lead connectors should be conducted and further research completed to assess the integrity of the connectors.

CHAPTER 7

CONCLUSION

The results of PLAXIS and 3DEC analysis are compared by using the "y displacement" (vertical displacement) values. There are also other factors that are collected from the analysis, but "y displacement" values give a brief idea to the engineer how the structure behaves under different conditions. Interfaces are inserted between wall and foundation to show the isolation effect of structure.

One directional earthquake force gives more displacement values than both directional earthquake force. However, when interface and one directional earthquake force applied together, the displacement values are lower. Displacement value should have been higher for that scenario. Thus, applying both directional earthquake force gives more realistic values (ex: $23.69*10^{-3}$ m = 23.69 mm).

To understand the effect of the earthquake parameters to the structure, the amplititude and frequency values are changed. Increasing amplititude of the earthquake increases "y displacement" value. Also, increasing frequency of the earthquake increases "y displacement" value as expected.

Applying interface between foundation and the wall always decreases the displacement values. This shows how the foundation helps to reduce the displacement values. The boundary limits are also changed in one of the analysis and larger boundary limits give more displacement values. Because, larger boundary limits give more area to the movement of the structure under earthquake loading.

3DEC analysis was done in three different scenarios. Including foundation of the structure, not including foundation and rigid boundary scenarios are the ones to check the effect to the structure. All scenarios were done with the same earthquake loading to observe the differences.

No foundation run gives less "y displacement" values comparing with foundation run. This is because the extra foundation weight acting on the soil and the displacement values are higher with more force on the soil. Rigid foundation boundary also gives less displacement as expected.

3DEC gives the designer to input the structure in three dimension. The interaction between the units in the structure can also be observed. This is very helpful to the designer and the accuracy of the calculation increases with detail that the designer can enter to the software.

3DEC analysis are done by using earthquake scenario taken from the 3DEC manual. However, changing amplititude, frequency and env time do not make any change on displament values. The only parameters that can change the displacement values are found to be the damp values (ex: damp 0.05 200 stiff). Changing these values effect the displacement values directly. Thus, finding a real earthquake effect at the analysis needs a further research on 3DEC input.

The "y displacement" values are somewhat closer when comparing two completely different software (ex: 24mm for PLAXIS and 7mm for 3DEC). The computation procedures and assumptions are all different for both softwares. Also, the soil parameters, structure properties and earthquake conditions are entered in a different way. Including all these differences, these acceptable displacement values show the assumptions that were made are quite close the real conditions. Thus, both analysis gives an idea about the real structural behaviour.

The depth of the foundation is around 6 meters deep. The deep foundation helps the wall to absorb forces better than walls with shallow foundations. The foundation isolates the forces acting on the wall very well.

Dry stone multiple leaf buildings exhibit unique behavior under both normal and seismic conditions. Understanding and analyzing these structures involves

acquiring a detailed knowledge of construction techniques, material properties, site history and environmental conditions.

Without the ability to calculate and totally predict failure modes in dry stone structures, a thorough assessment of all known factors and present conditions delineates areas of weakness. Identifying unknown aspects of construction and enabling factors proves just as important as understanding failure mechanisms.

Unknown elements exist at every site. However, these elements contribute the quantifiable data needed to establish cause and effect relationships of current conditions. They should be thoroughly tested and measured.

Once understood, conditions resulting from weathering, external force and structural movements can be correlated to specific enabling factors. Because the moment of failure in dry stone constructions is relatively unpredictable, finding rates of change and types of displacement provide the data necessary to determine the level of intervention required.

By isolating and reducing the deteriorative elements and monitoring critical structural displacements, possible failure mechanisms can be identified and preventive measures can be implemented.

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APPENDICES

A. PLAXIS OUTPUTS

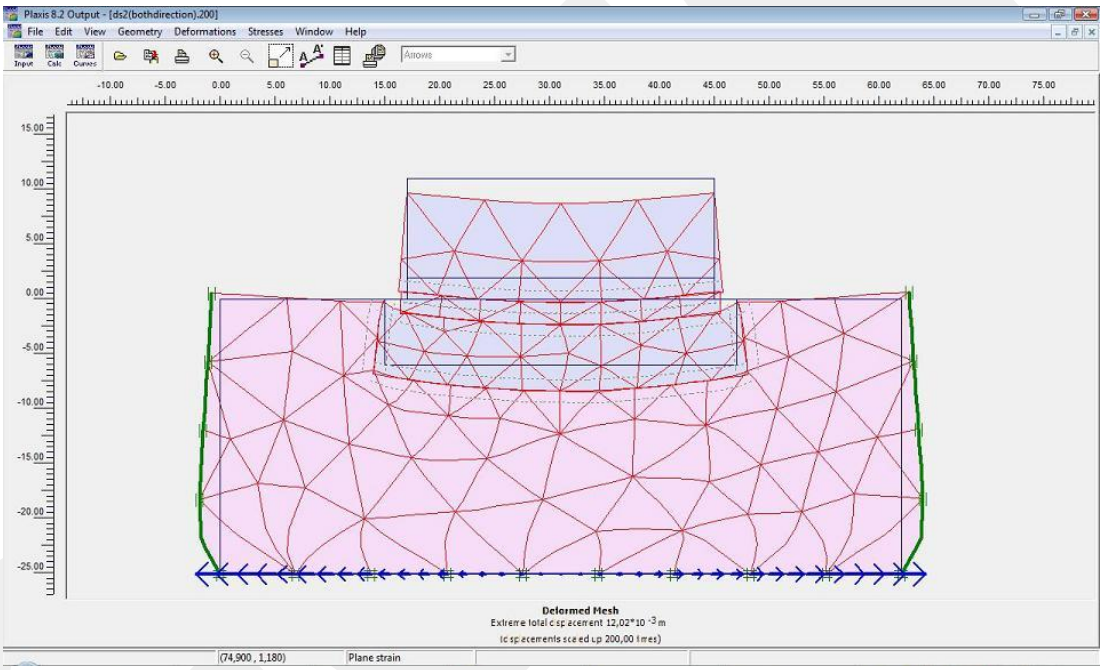


Figure A1 - Deformed Mesh (With interface – both direction earthquake load – Magnititude 4)

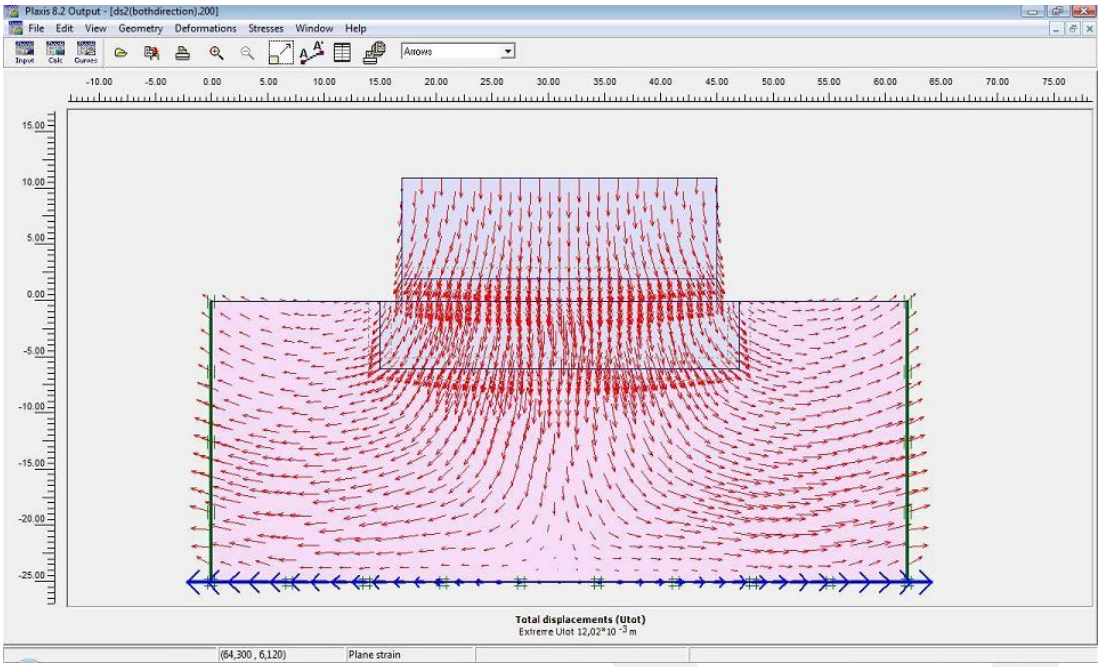


Figure A2 - Total Displacement (With interface –both direction earthquake load– Magnititude 4)

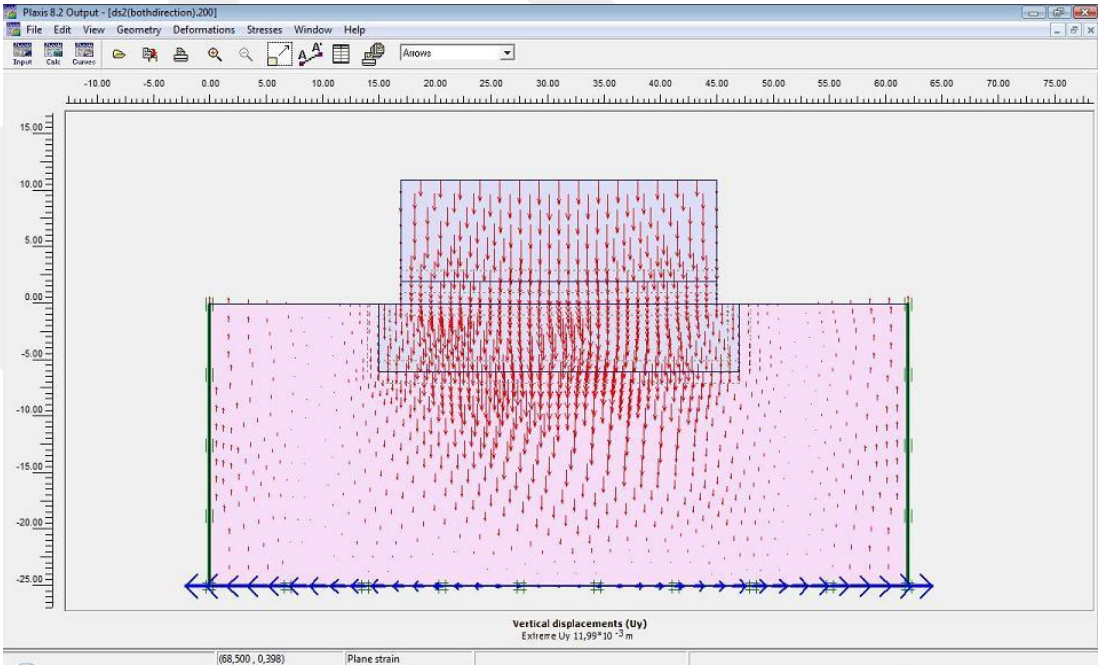


Figure A3 - Vertical Displacements (With interface – both direction earthquake load – Magnititude 4)

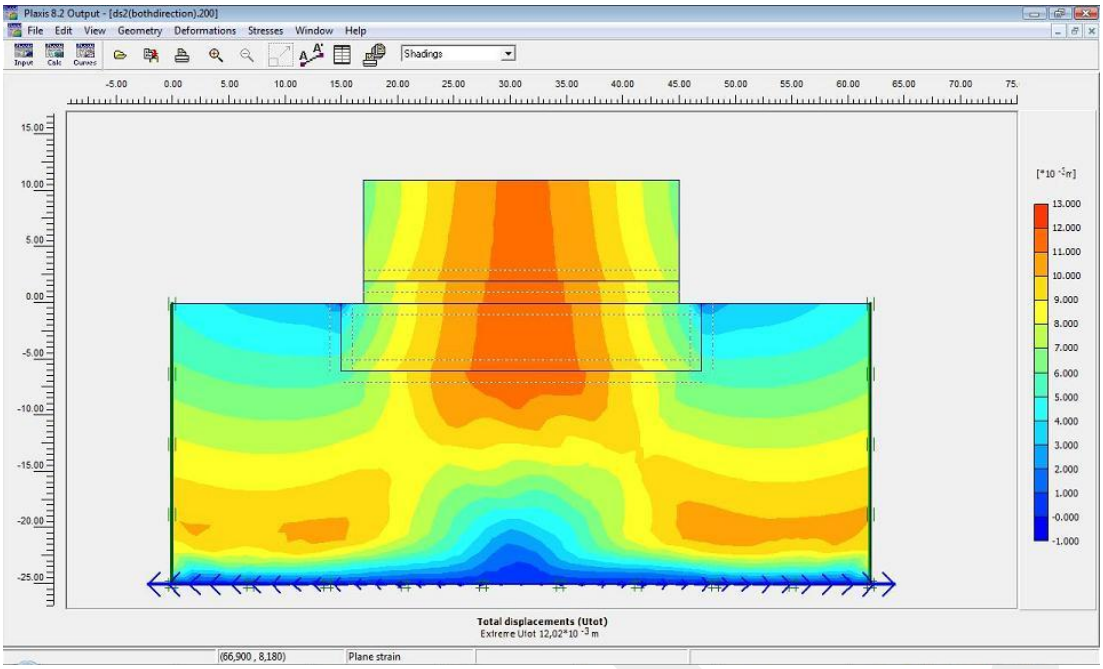


Figure A4 - Total Displacement (With interface –both direction earthquake load– Magnititude 4)

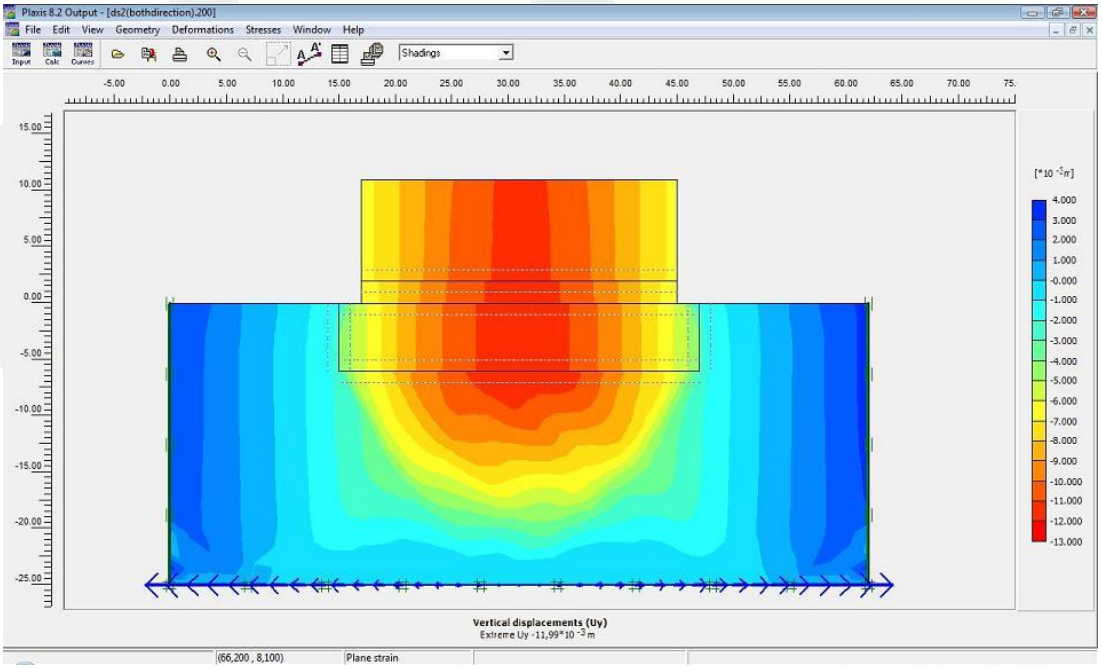


Figure A5 - Vertical Displacements (With interface – both direction earthquake load – Magnititude 4)

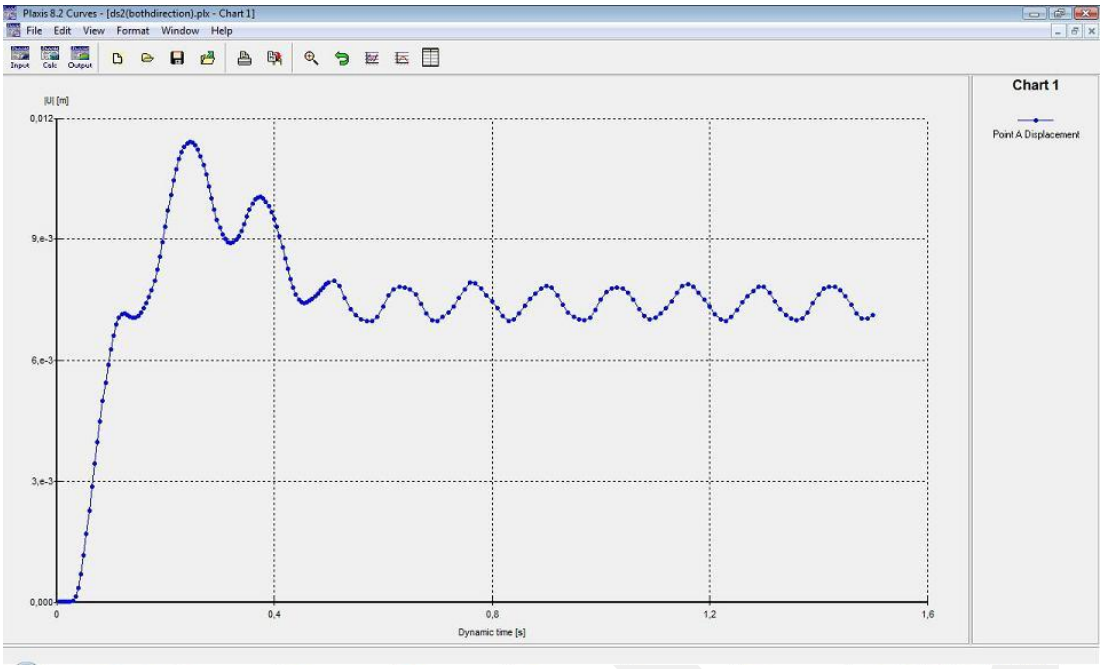


Figure A6 - Point A Displacement (With interface – both direction earthquake load – Magnititude 4)

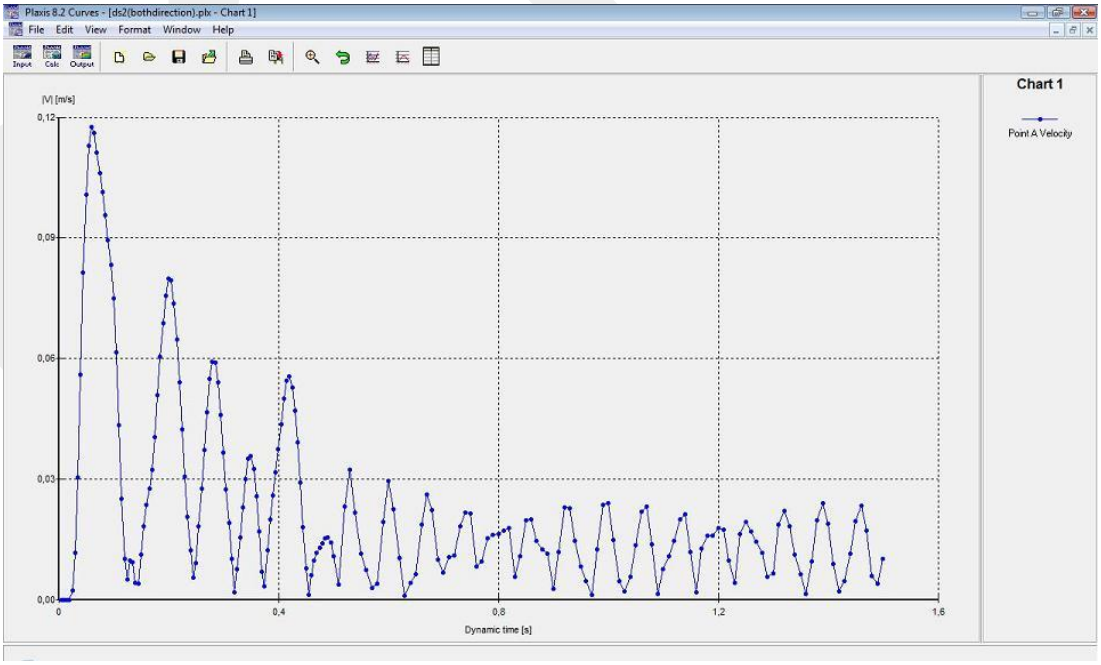


Figure A7 - Point A Velocity (With interface – both direction earthquake load – Magnititude 4)

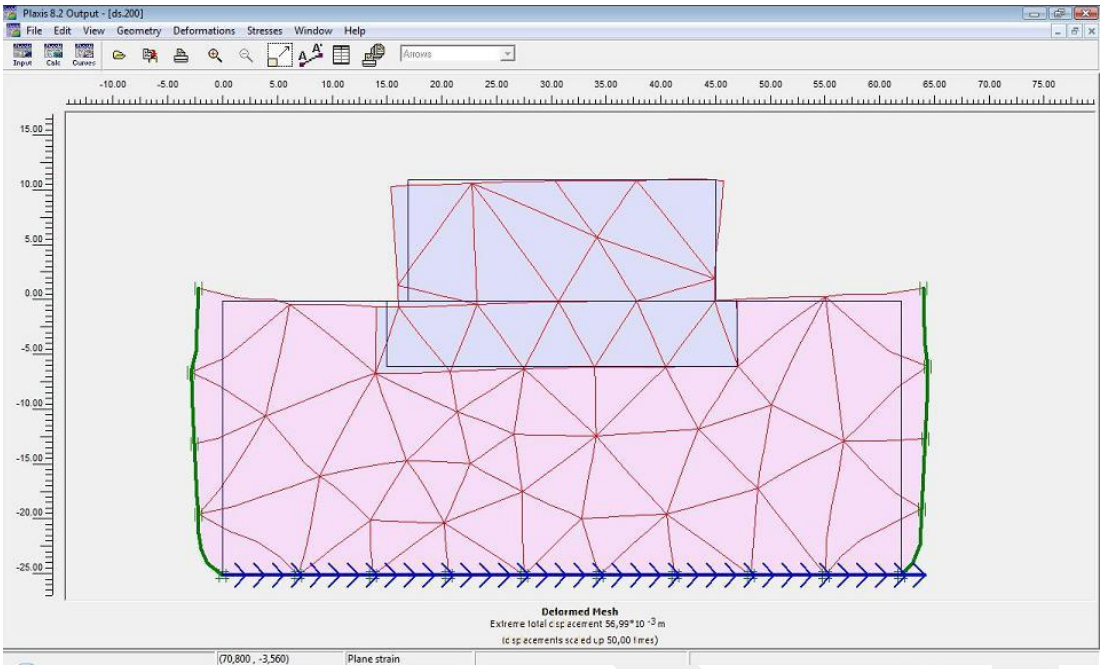


Figure A8 - Deformed Mesh (No Interface – One direction earthquake load – Magnititude 4)

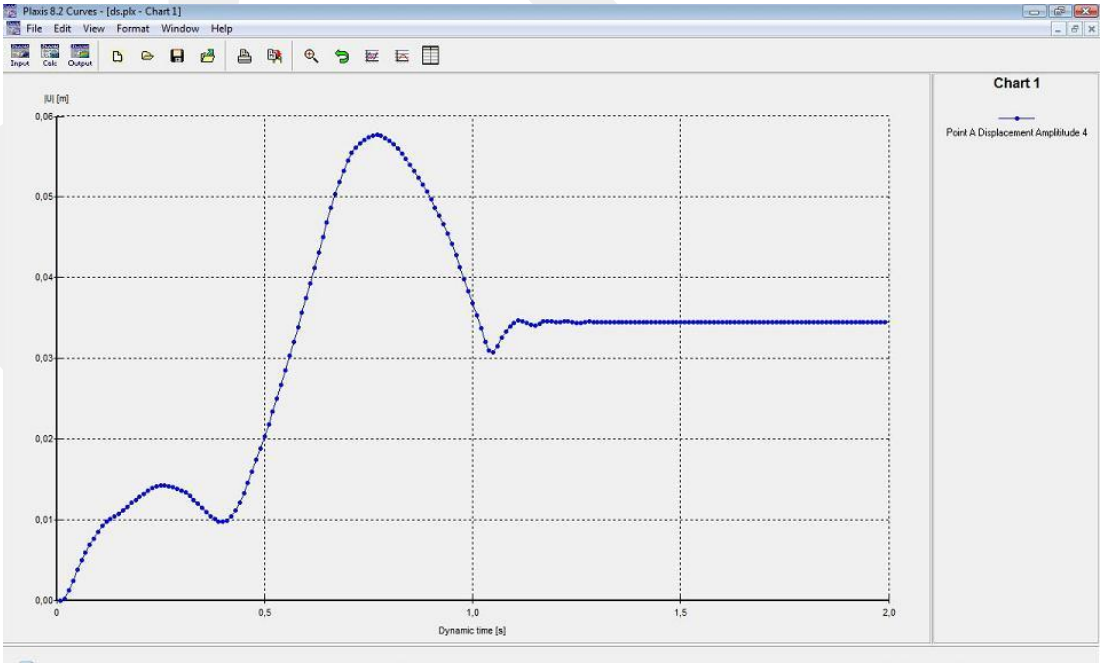


Figure A9 - Point A Displacement (No Interface –One direction earthquake load– Magnititude 4)

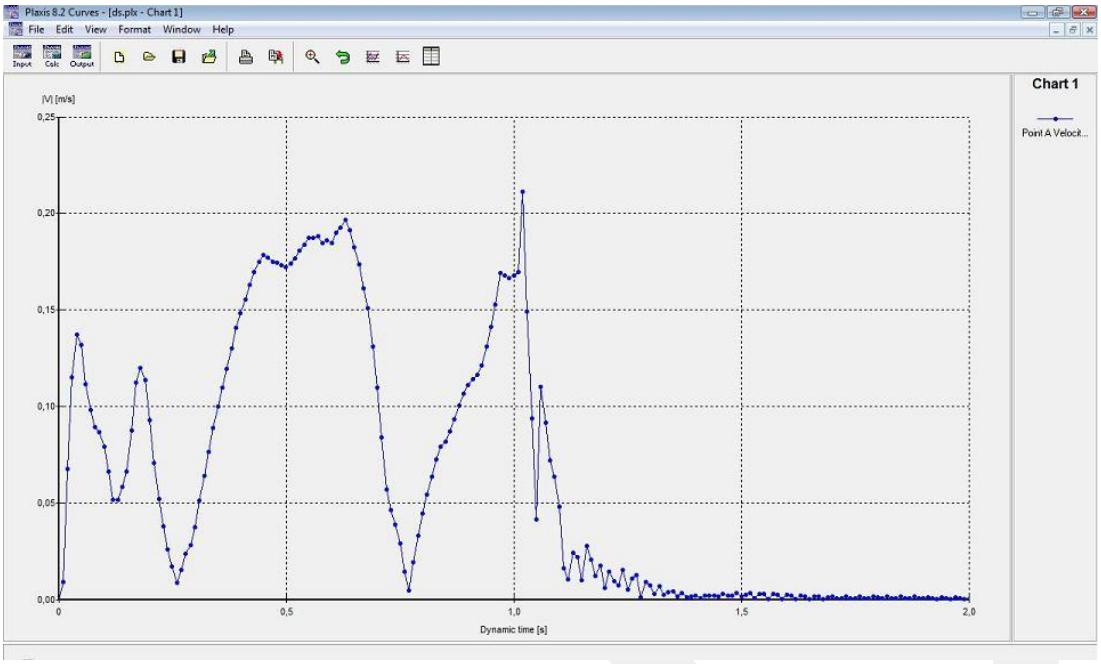


Figure A10 - Point A Velocity (No Interface – One direction earthquake load – Magnititude 4)

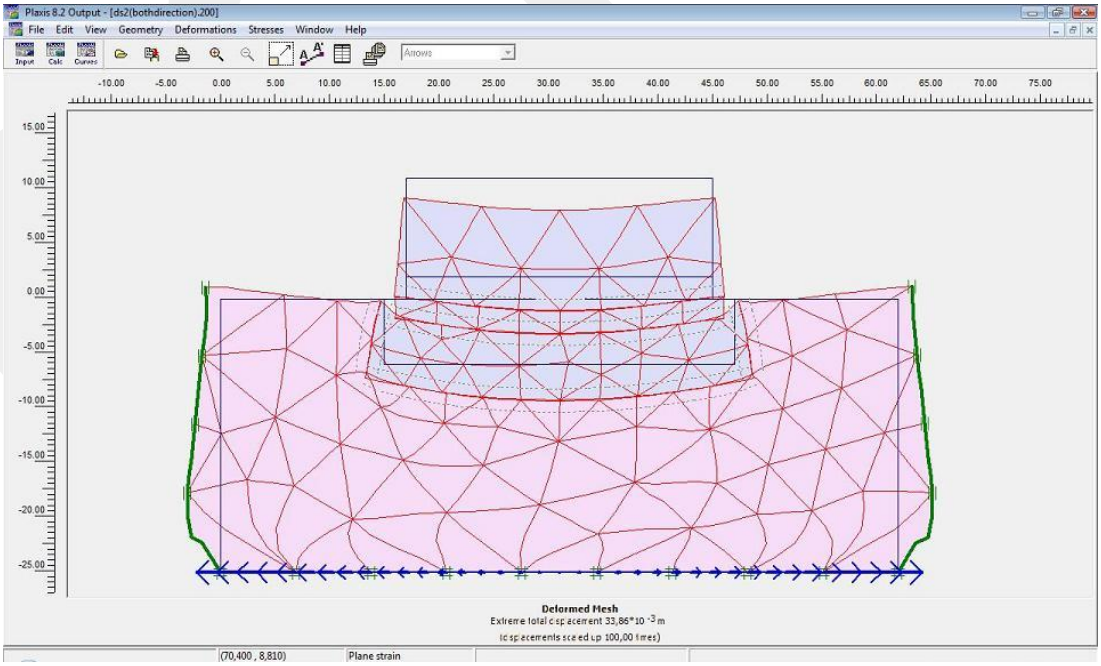


Figure A11 - Deformed Mesh (With interface – both direction earthquake load – Frequency 2)

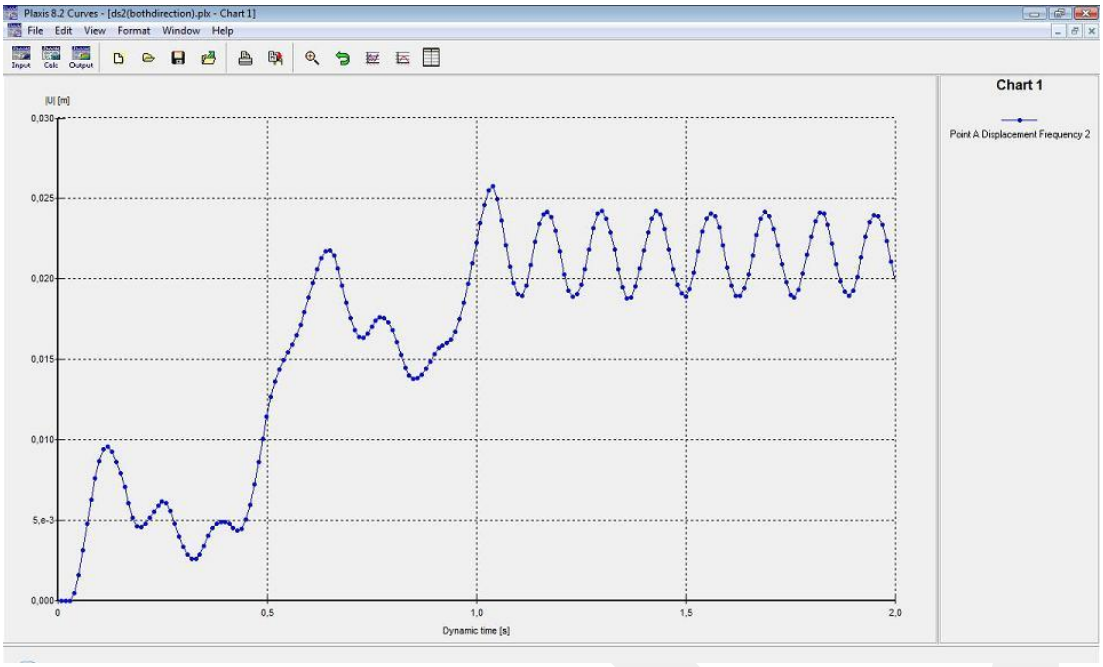


Figure A12 - Point A Displacement (With interface–both direction earthquake load–Frequency 2)

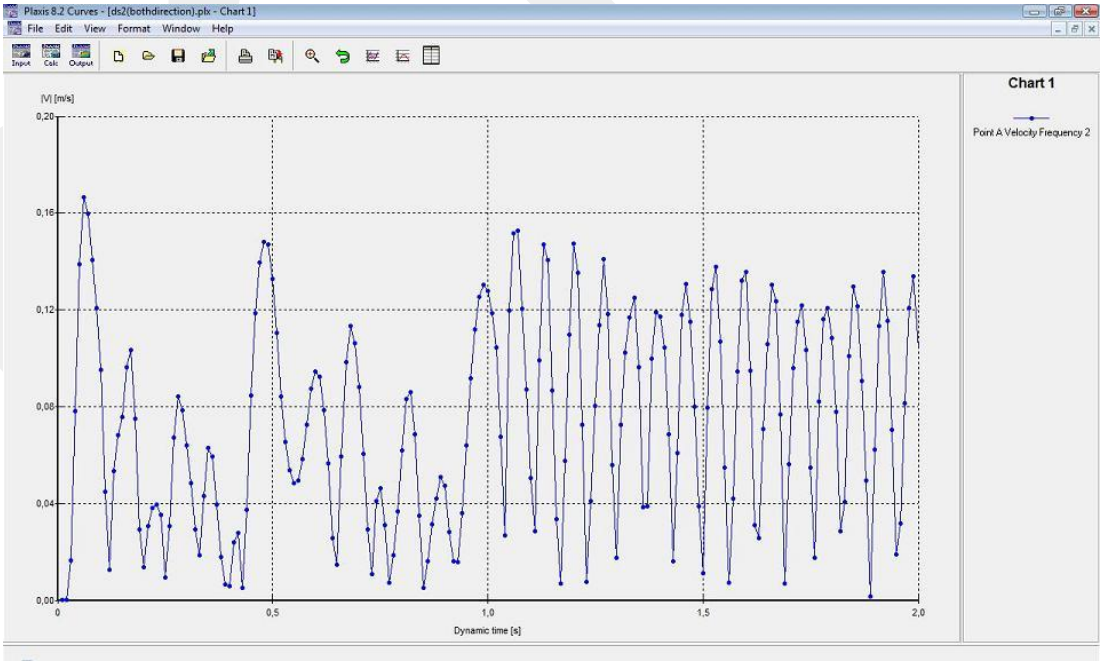


Figure A13 - Point A Velocity (With interface–both direction earthquake load–Frequency 2)

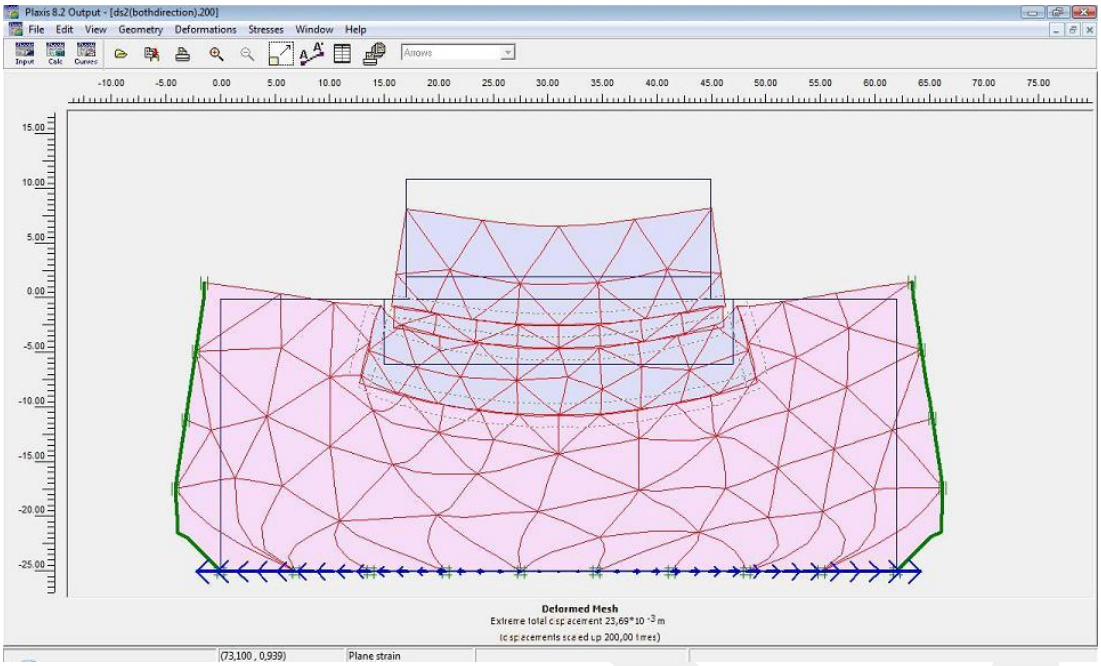


Figure A14 - Deformed Mesh (With interface – Both direction earthquake load – Magnititude 4)

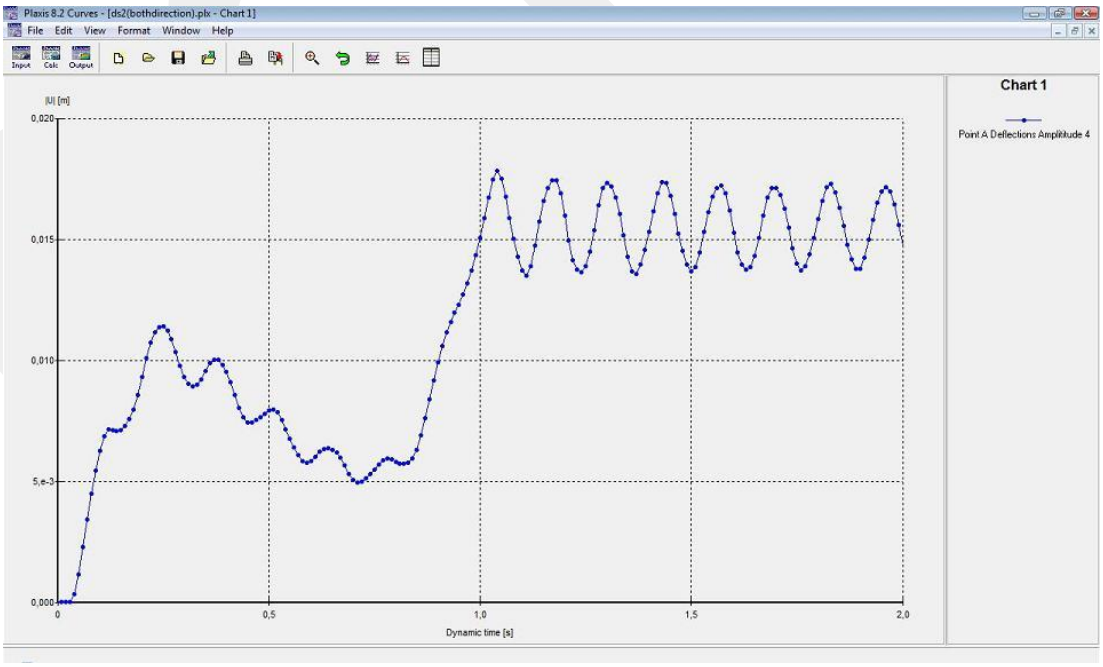


Figure A15 - Point A Displacement (With interface – Both direction earthquake load – Magnititude 4)

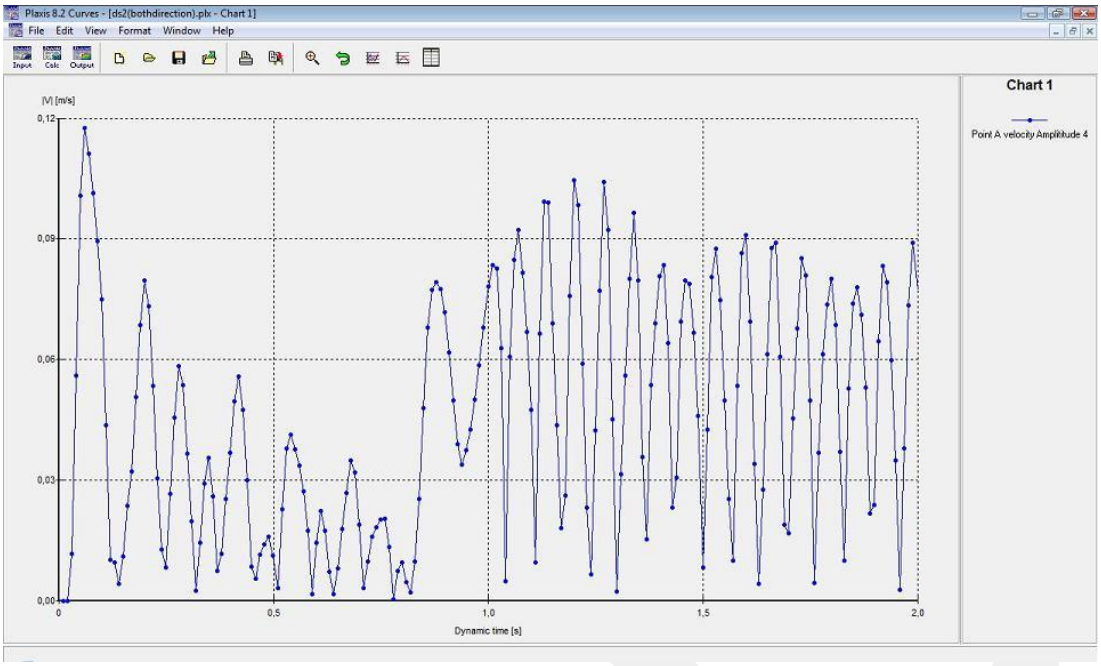


Figure A16 - Point A Velocity (With interface – Both direction earthquake load – Magnititude 4)

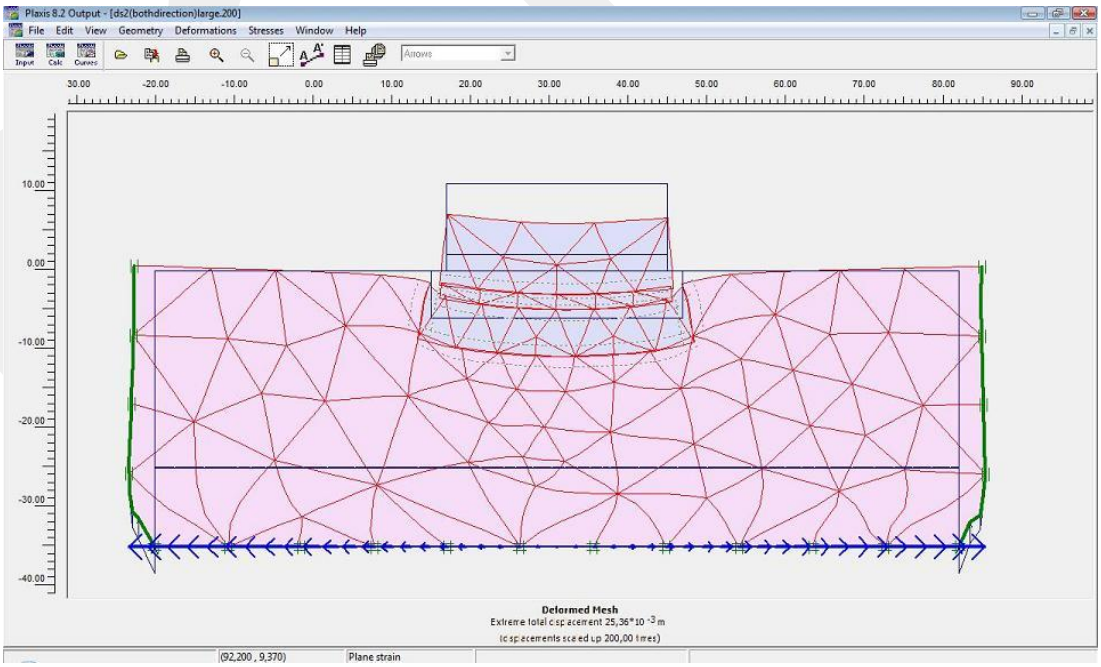


Figure A17 - Deformed Mesh (With interface – Both direction earthquake load – Magnititude 4 – Large Boundary)

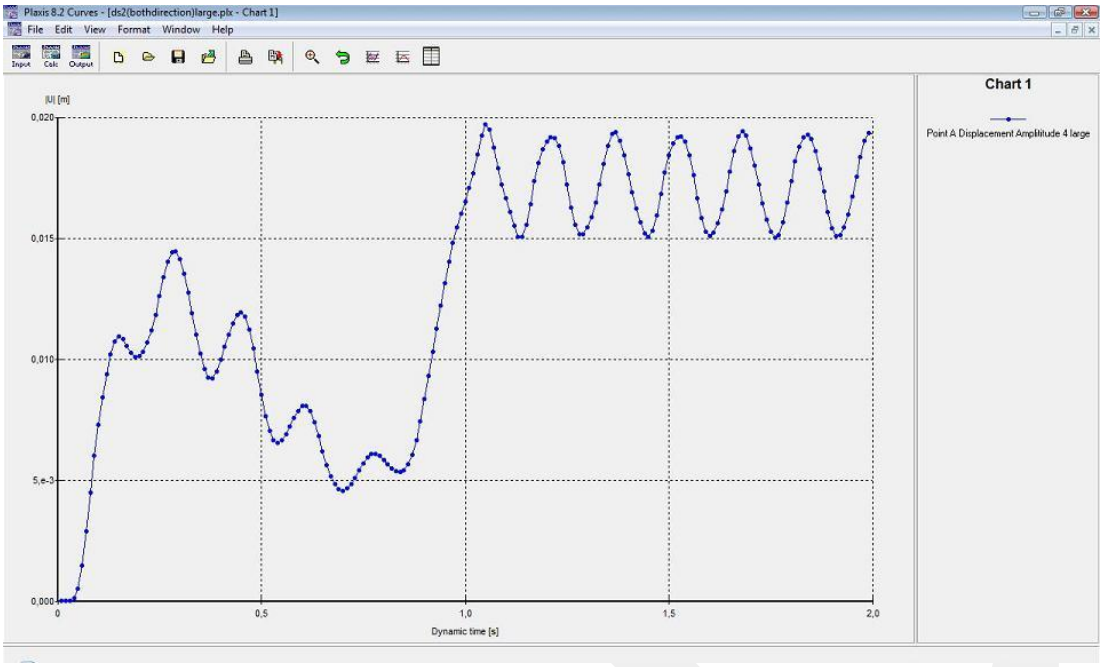


Figure A18 - Point A Displacement (With interface – Both direction earthquake load – Magnititude 4 – Large Boundary)

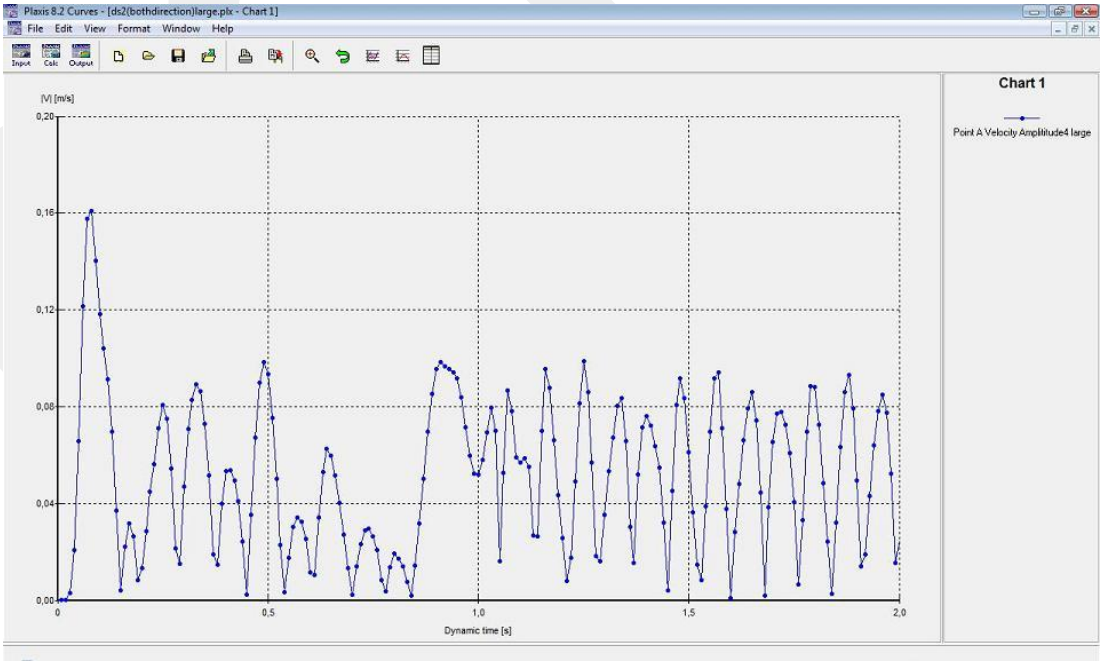


Figure A19 - Point A Velocity (With interface – Both direction earthquake load – Magnititude 4 – Large Boundary)

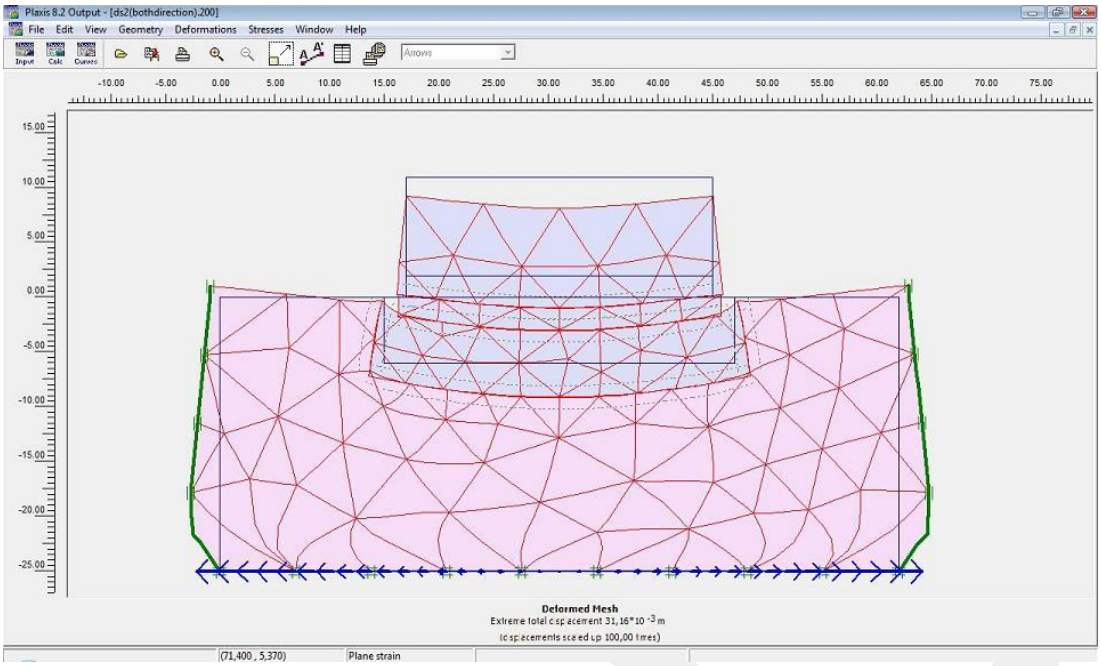


Figure A20 - Deformed Mesh (With interface – Both direction earthquake load – Magnititude 6)

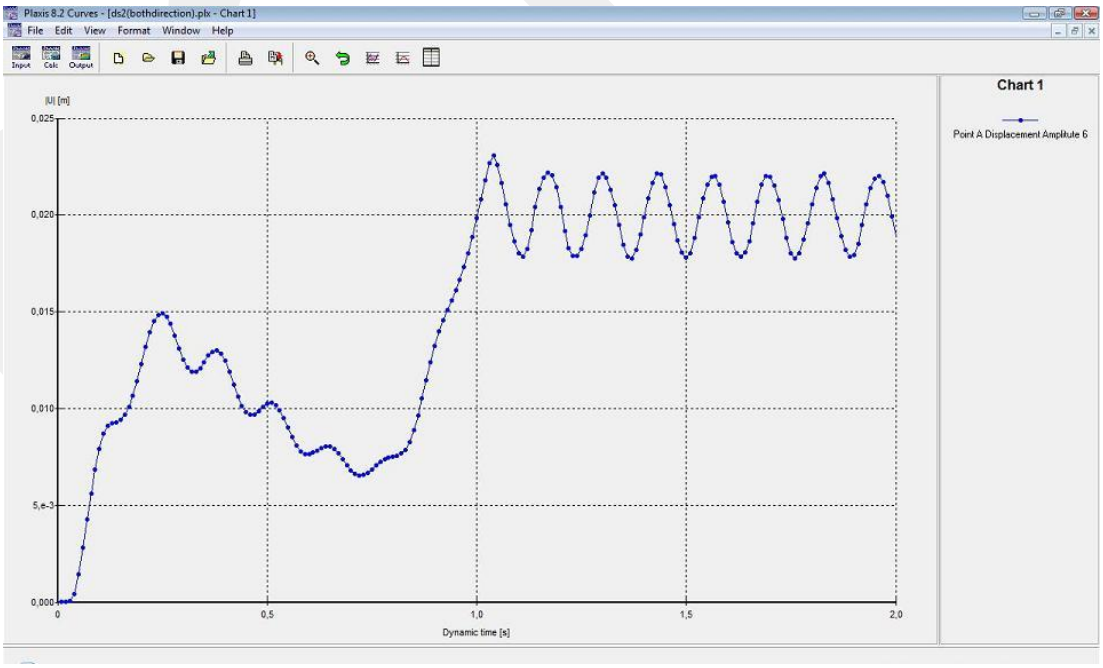


Figure A21 - Point A Displacement (With interface – Both direction earthquake load – Magnititude 6)

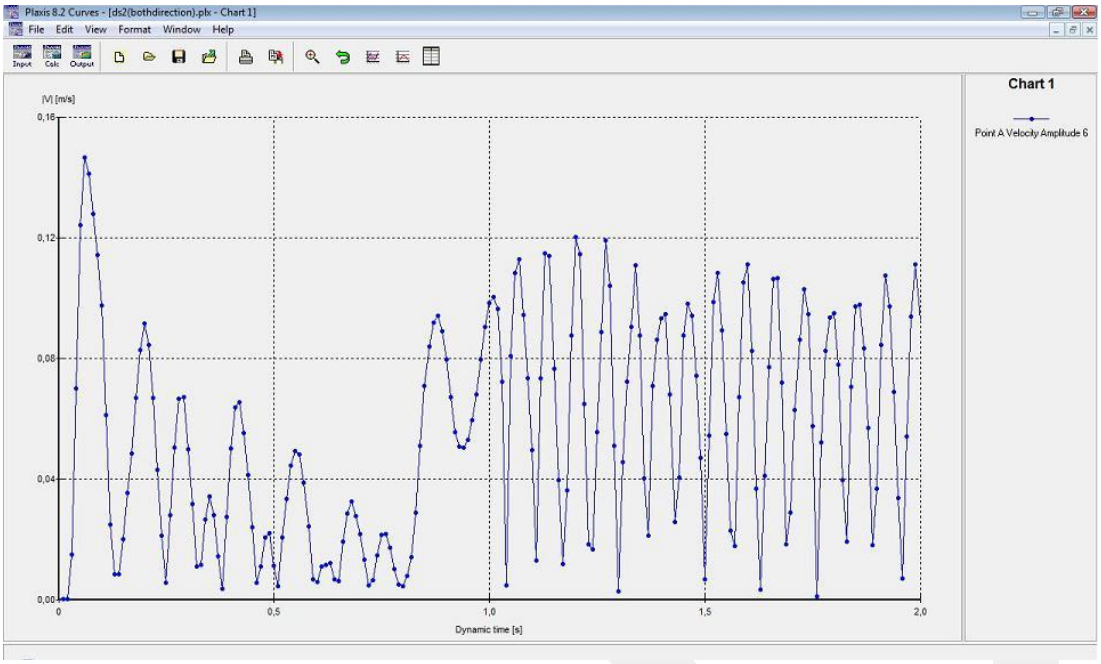


Figure A22 - Point A Velocity (With interface – Both direction earthquake load – Magnititude 6)

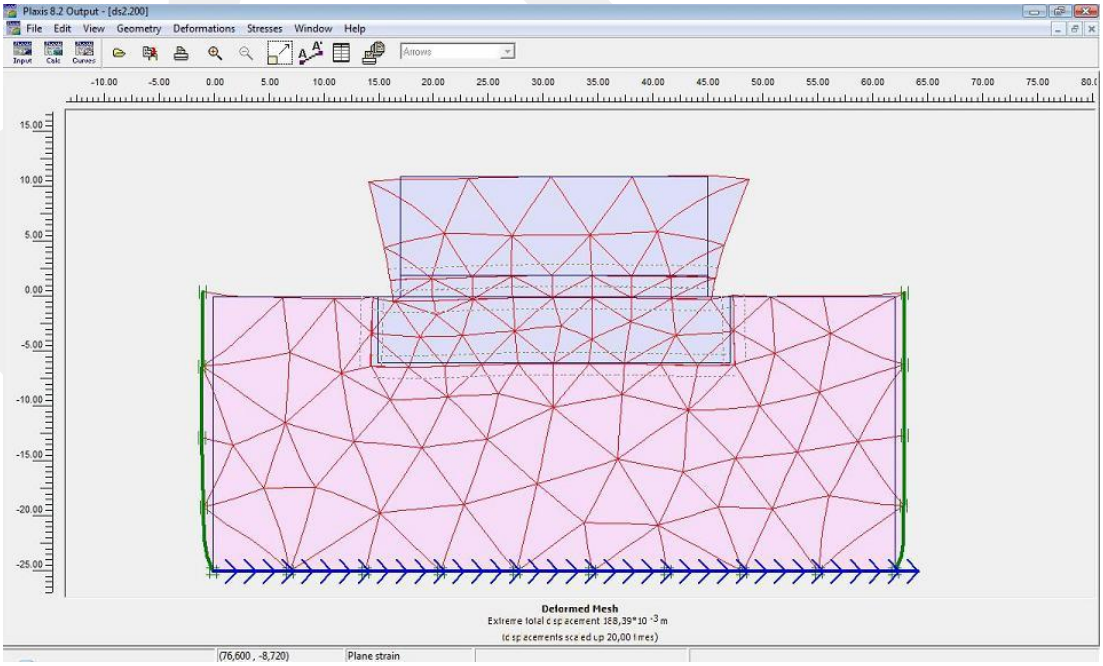


Figure A23 - Deformed Mesh (With interface – One direction earthquake load – Magnititude 4)

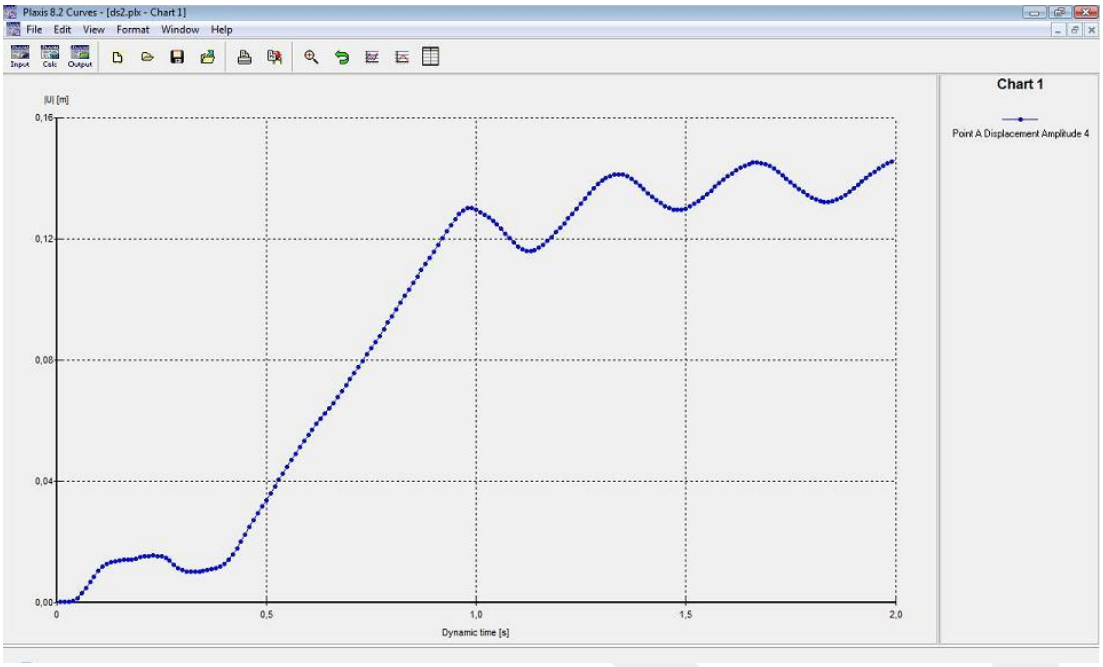


Figure A24 - Point A Displacement (With interface – One direction earthquake load – Magnititude 4)

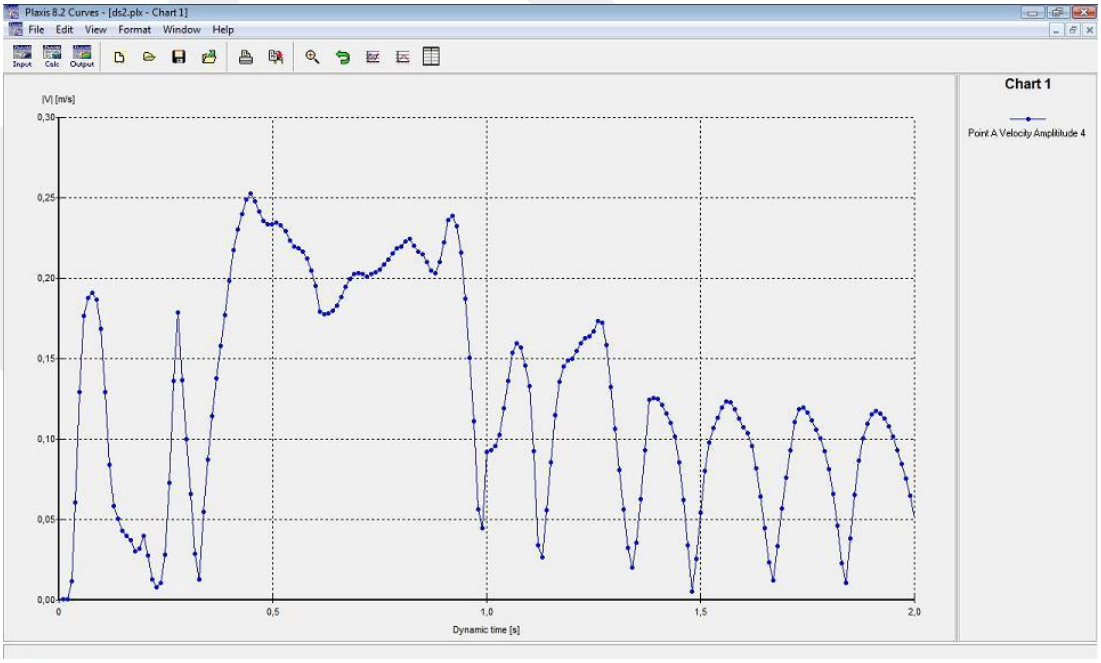


Figure A25 - Point A Velocity (With interface – One direction earthquake load – Magnititude 4)

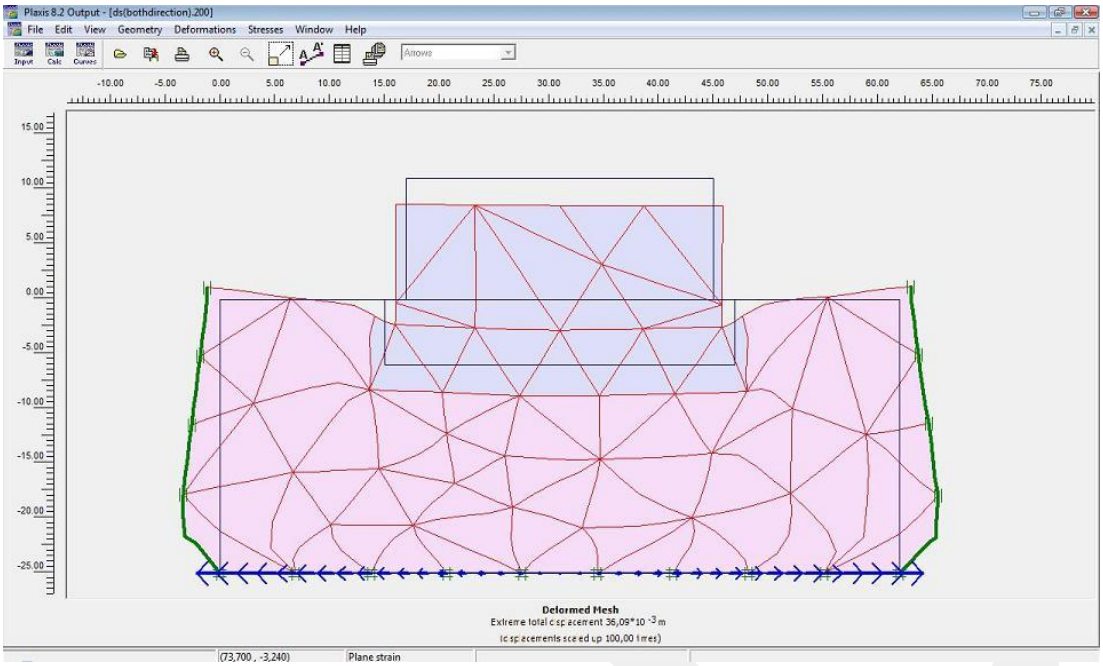


Figure A26 - Deformed Mesh (No Interface – Both direction earthquake load – Frequency 2)

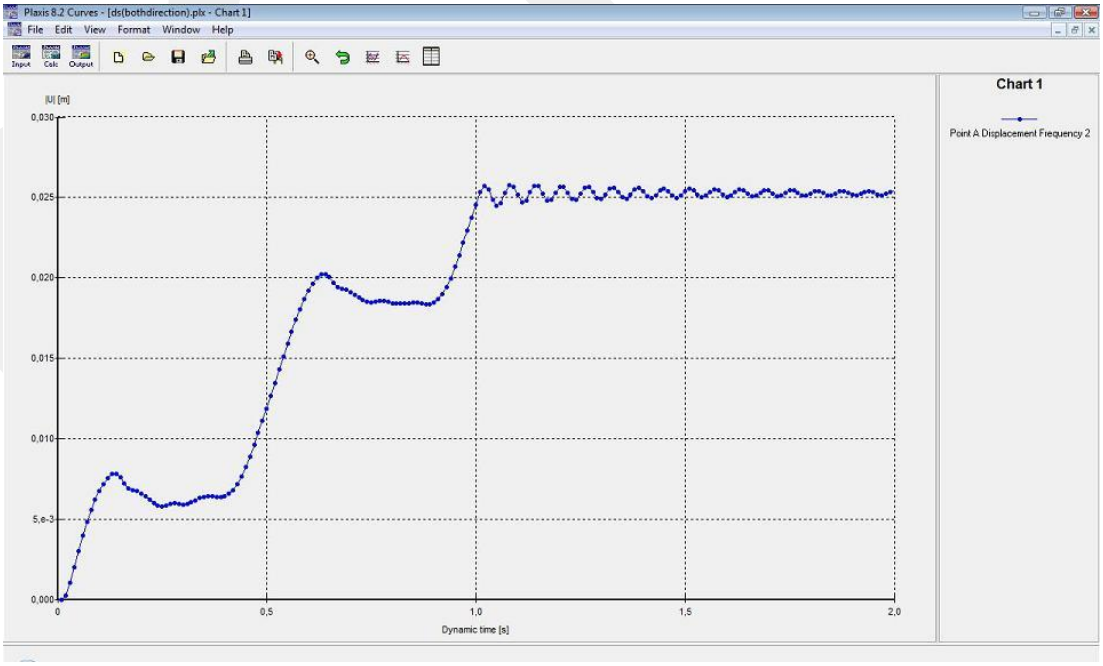


Figure A27 - Point A Displacement (No Interface –Both direction earthquake load– Frequency 2)

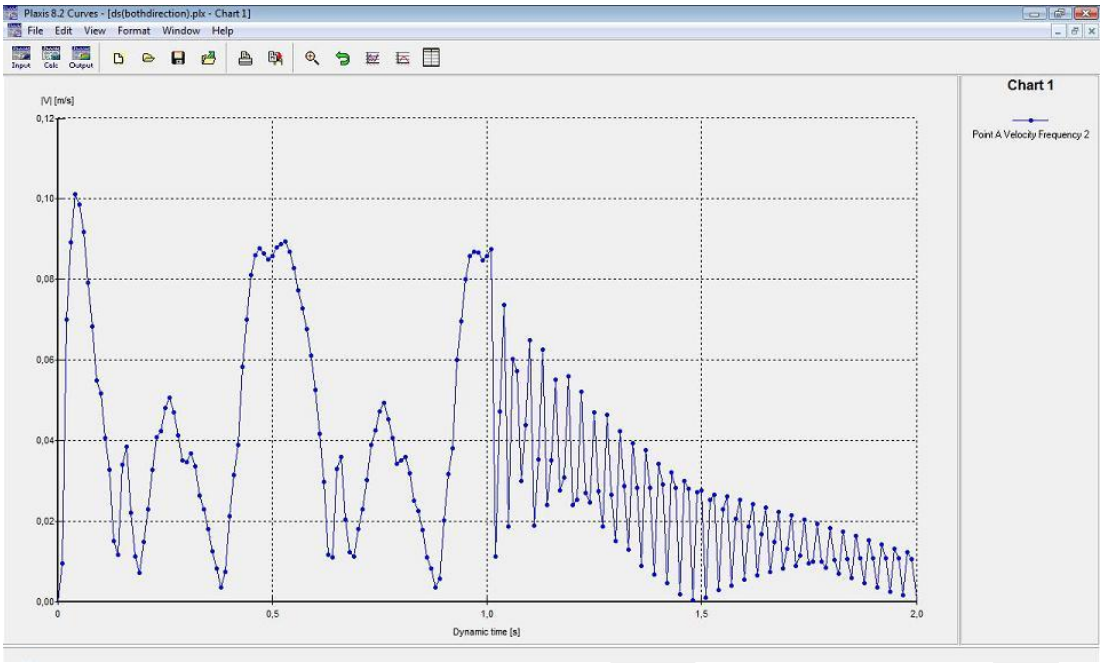


Figure A28 - Point A Velocity (No Interface –Both direction earthquake load– Frequency 2)

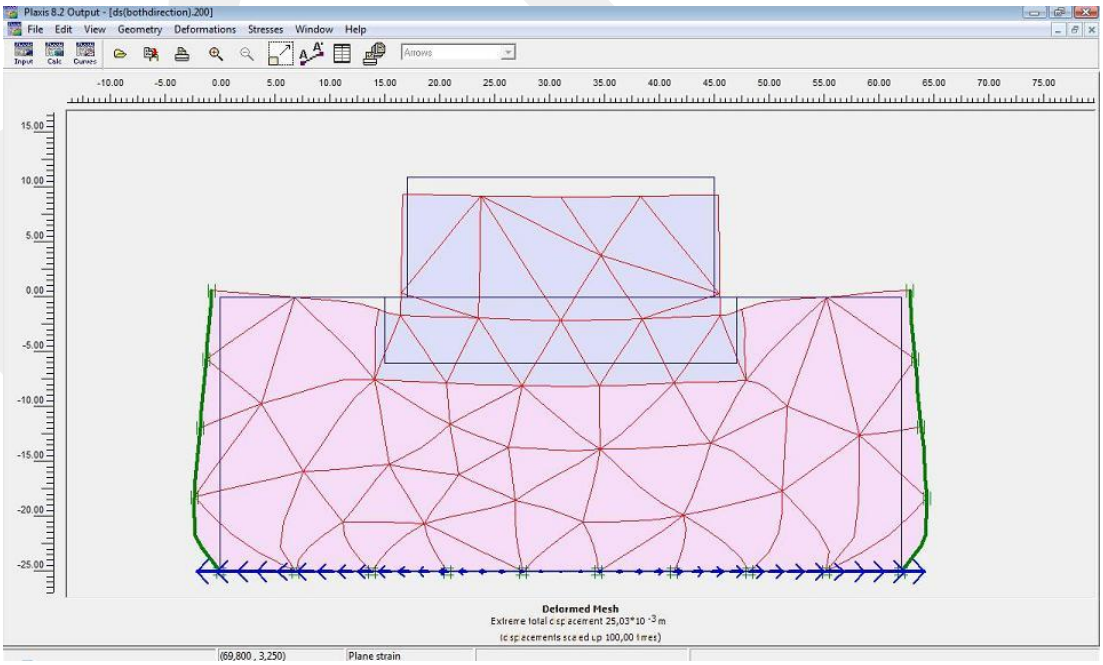


Figure A29 - Deformed Mesh (No Interface – Both direction earthquake load – Magnititude 4)

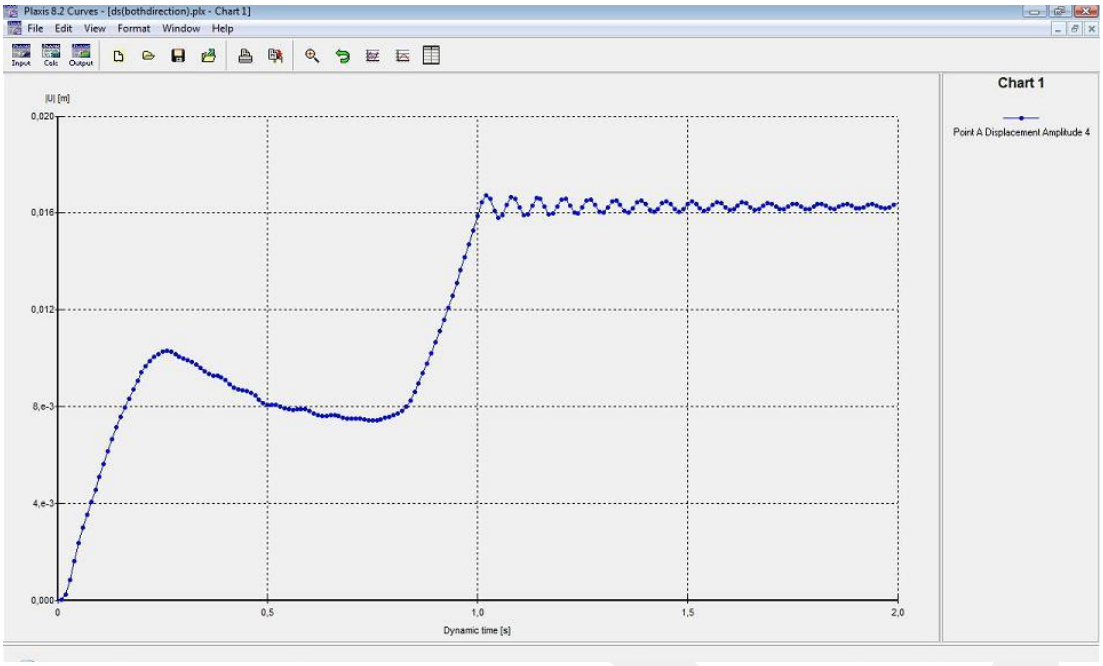


Figure A30 - Point A Displacement (No Interface–Both direction earthquake load–Magnititude 4)

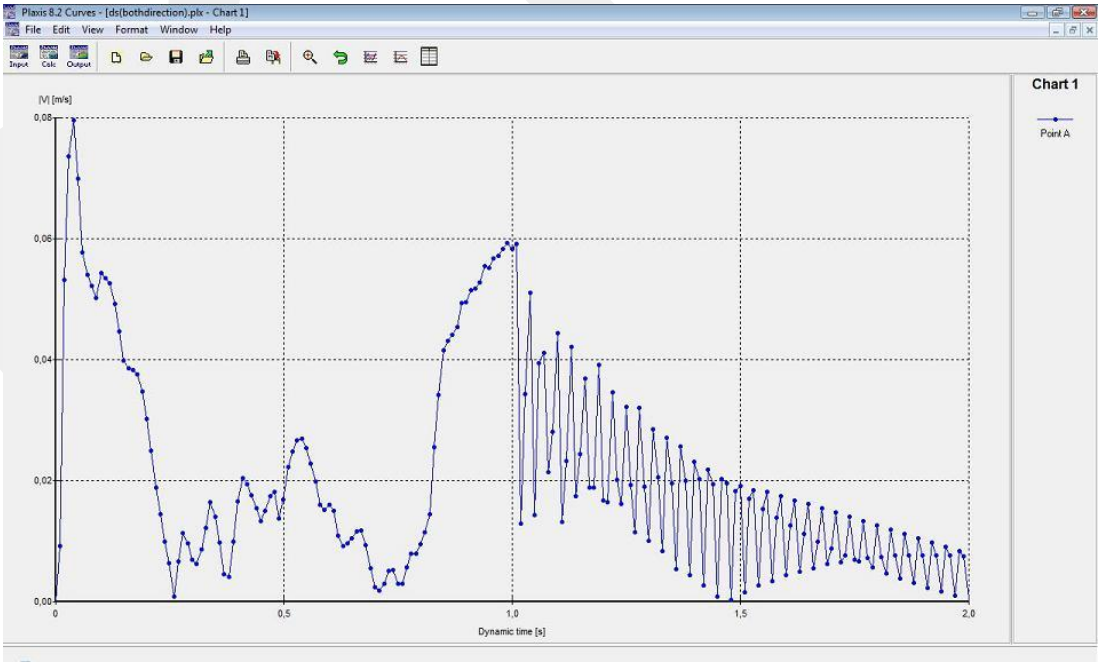


Figure A31 - Point A Velocity (No Interface–Both direction earthquake load–Magnititude 4)

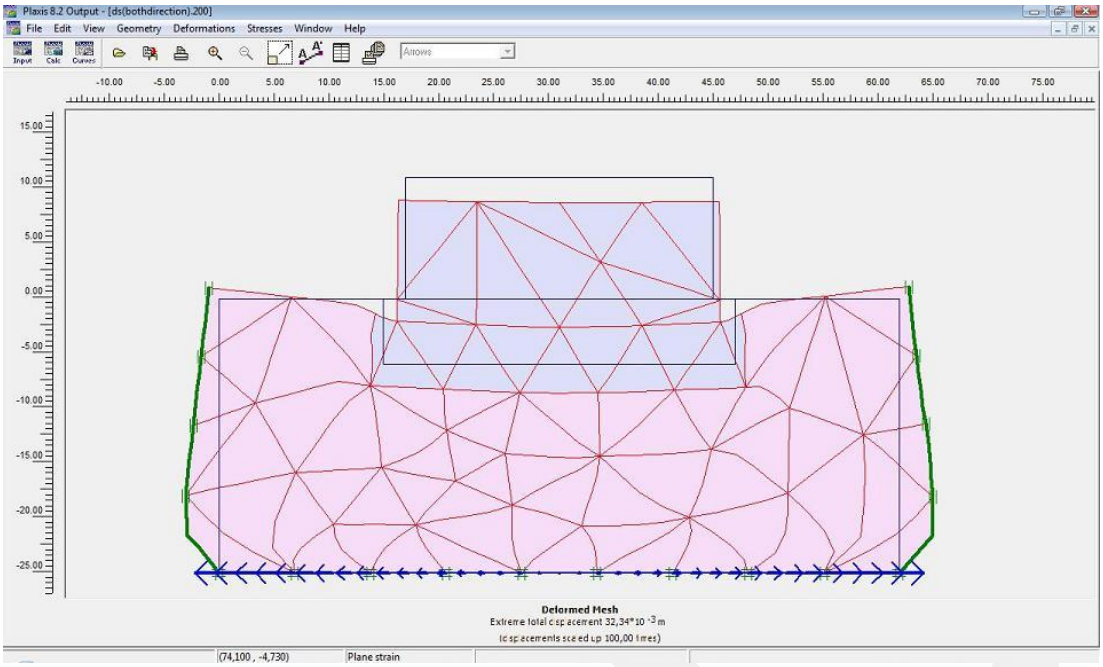


Figure A32 - Deformed Mesh (No Interface – Both direction earthquake load – Magnititude 6)

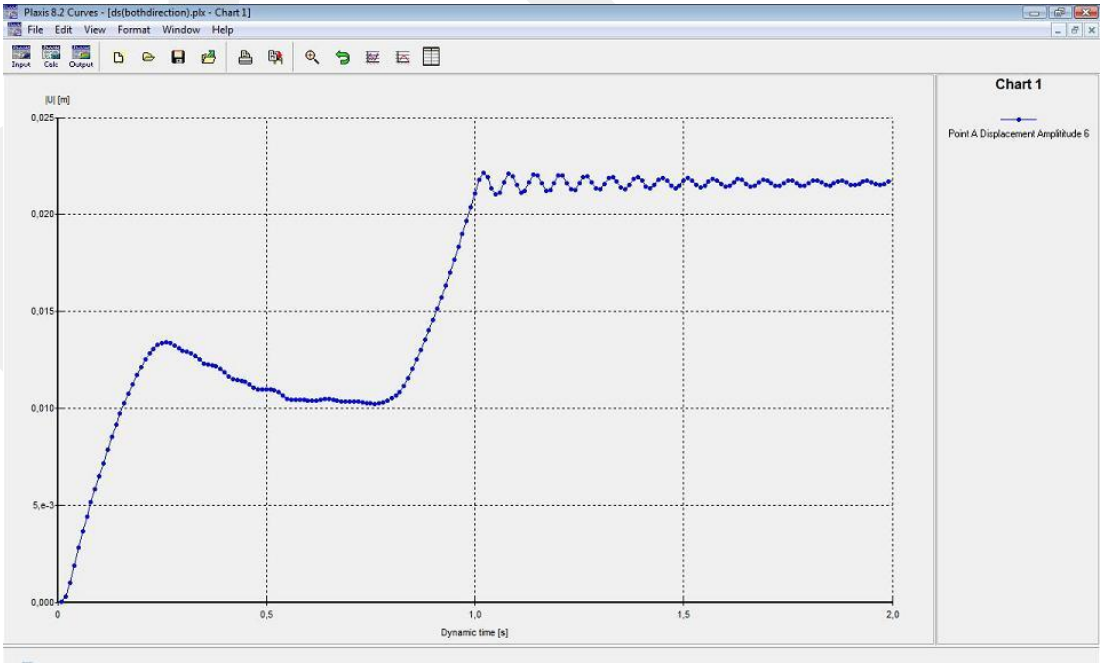


Figure A33 - Point A Displacement (No Interface–Both direction earthquake load– Magnititude 6)

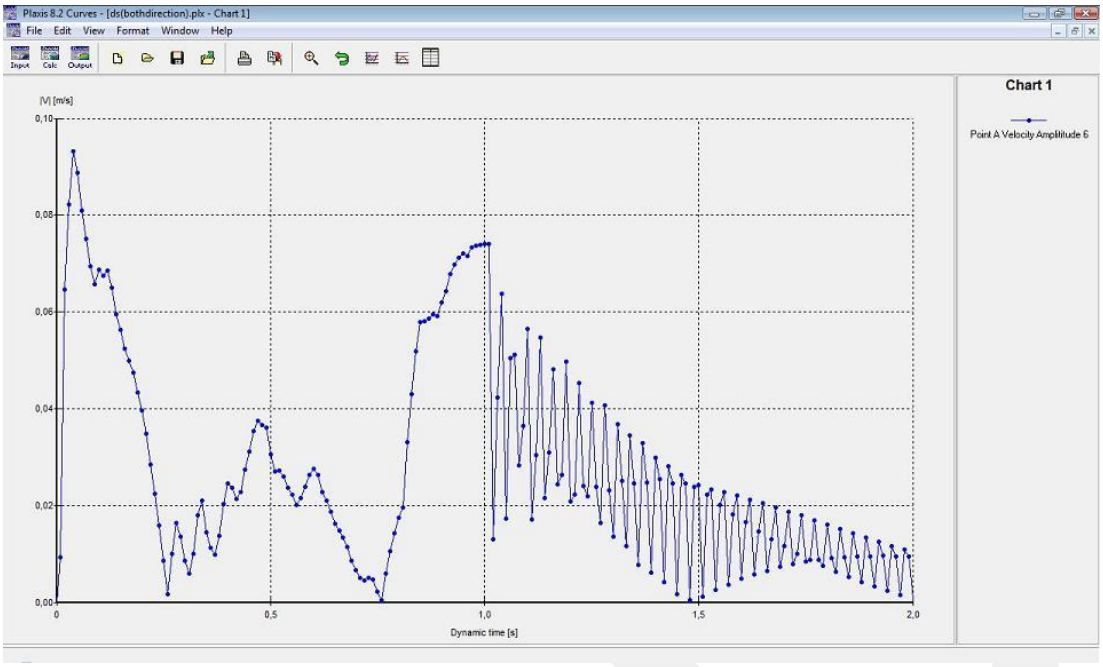


Figure A34 - Point A Velocity (No Interface–Both direction earthquake load–Magnititude 6)

GCRI

B. 3DEC INPUT (SAMPLE FILE)

File name: 1-4amp-nofound.dat

Purpose of file: To see the foundation effect on the structure

```
new
config dynamic
;  masonry WALL
po reg 0 28.2  0 1.7  0 1
jset dip 0 org 0 .2 0
jset dip 0 org 0 .3 0
jset dip 0 org 0 .6 0
jset dip 0 org 0 1.7 0
mark yr 0 0.6 reg 2
hide reg 2
jset dip 90 dd 90 orig 1.8 0.6 0
jset dip 90 dd 90 orig 2.75 0.6 0
jset dip 90 dd 90 orig 4.05 0.6 0
jset dip 90 dd 90 orig 4.8 0.6 0
jset dip 90 dd 90 orig 5.75 0.6 0
jset dip 90 dd 90 orig 6.85 0.6 0
```

jset dip 90 dd 90 orig 7.9 0.6 0

jset dip 90 dd 90 orig 9.1 0.6 0

jset dip 90 dd 90 orig 10 0.6 0

jset dip 90 dd 90 orig 11.05 0.6 0

jset dip 90 dd 90 orig 12.25 0.6 0

jset dip 90 dd 90 orig 13.35 0.6 0

jset dip 90 dd 90 orig 14.35 0.6 0

jset dip 90 dd 90 orig 15.45 0.6 0

jset dip 90 dd 90 orig 16.8 0.6 0

jset dip 90 dd 90 orig 18 0.6 0

jset dip 90 dd 90 orig 19.2 0.6 0

jset dip 90 dd 90 orig 20.3 0.6 0

jset dip 90 dd 90 orig 21.35 0.6 0

jset dip 90 dd 90 orig 22.2 0.6 0

jset dip 90 dd 90 orig 23.75 0.6 0

jset dip 90 dd 90 orig 25.55 0.6 0

jset dip 90 dd 90 orig 26.7 0.6 0

find reg 2

po reg 0 28.2 1.7 2.0 0 1

jset dip 0 org 0 2 0

mark yr 0 1.7 reg 12

hide reg 12

jset dip 90 dd 90 orig 14.1 1.7 0 sp 1.15 num 23

find reg 12

po reg 0 28.2 2.0 10.55 0 1

jset dip 0 org 0 2.5 0

jset dip 0 org 0 3 0

jset dip 0 org 0 3.45 0

jset dip 0 org 0 3.9 0

jset dip 0 org 0 4.35 0

jset dip 0 org 0 4.8 0

jset dip 0 org 0 5.25 0

jset dip 0 org 0 5.7 0

jset dip 0 org 0 6.15 0

jset dip 0 org 0 6.6 0

jset dip 0 org 0 7.05 0

jset dip 0 org 0 7.5 0

jset dip 0 org 0 7.95 0

jset dip 0 org 0 8.3 0

jset dip 0 org 0 8.75 0

jset dip 0 org 0 9.2 0

jset dip 0 org 0 9.65 0

jset dip 0 org 0 10.1 0

mark yr 0 2.0 reg 13

mark yr 2 2.5 xr .8 27.4 reg 6

mark yr 2.5 3 xr .8 27.4 reg 7

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mark yr 3 3.45 xr .8 27.4 reg 6

mark yr 3.45 3.9 xr .8 27.4 reg 7

mark yr 3.9 4.35 xr .8 27.4 reg 6

mark yr 4.35 4.8 xr .8 27.4 reg 7

mark yr 4.8 5.25 xr .8 27.4 reg 6

mark yr 5.25 5.7 xr .8 27.4 reg 7

mark yr 5.7 6.15 xr .8 27.4 reg 6

mark yr 6.15 6.6 xr .8 27.4 reg 7

mark yr 6.6 7.05 xr .8 27.4 reg 6

mark yr 7.05 7.5 xr .8 27.4 reg 7

mark yr 7.5 7.95 xr .8 27.4 reg 6

mark yr 7.95 8.3 xr .8 27.4 reg 7

mark yr 8.3 8.75 xr .8 27.4 reg 6

mark yr 8.75 9.2 xr .8 27.4 reg 7

mark yr 9.2 9.65 xr .8 27.4 reg 6

mark yr 9.65 10.10 xr .8 27.4 reg 7

mark yr 10.10 10.55 xr .8 27.4 reg 6

;mark yr .2 2.0 xr .8 27.4 reg 36

hide reg 13

hide reg 7

;hide reg 36

jset dip 90 dd 90 orig .8 0.2 0

jset dip 90 dd 90 orig 27.4 .2 0

GCRIS

```
jset dip 90 dd 90  orig 14 2 0 sp 1.5 NUM 17

find reg 7


;hide reg 13

hide reg 6

jset dip 90 dd 90 orig .8 2 0

jset dip 90 dd 90 orig 27.4 2 0

jset dip 90 dd 90 orig 13.6 2.5 0 sp 1.5 num 17

find reg 13 6

;jset dip 90 dd 90 orig .8 0 0

;jset dip 90 dd 90 orig 27.4 0 0

;

;Temel geometrisi

;

gen edge 2

po reg -13 41.2   -16.0 0  -10.5 11.5

change  -13 41.2   -16.0 0  -10.5 11.5 mat 3

join on

;jset dip 0 org -3 -3 0

mark xr 0 28.2 yr 0 10.55 zr 0 1 reg 25

hide reg 25

jset dip 90 dd 90  orig 20.1 -16 0 sp 1.5 num 50

jset dip 0  orig 20.1 -8 0 sp 1.5 num 40
```


GCRIS

```
seek
;po reg -3 31.2   -6.0 0  -0.5 1.5
;change -3 31.2 -6.0 0  -0.5 1.5 mat 2
gen edge 20
;hide -3 31.2   -6.0 0  -0.5 1.5
prop mat=1 dens=0.002700 k=20833 G=22727
prop jmat=1 kn=50000 ks=20000 fric=30 ten 0
;prop mat=2 dens=0.002700 k=5000 G=6000
prop mat=3 dens=0.002700 k=5000 G=6000
;boun -3 31.2 -6.0 0  -0.5 1.5 mat 2
;
grav  0 -10 0
;boun -3.1 31.3 -6.1 -5.9 -0.6 1.6 xvel 0 yvel 0
;boun -13.1 41.3   -16.1 -15.9  -10.6 11.6 xvel 0 yvel 0 zvel 0
fix  range x -13 41.2   y -16 -15.9 z -10.5 11.5
bound  xr -13 -12.9 xvel 0.0
bound  xr 41.2 41.3 xvel 0.0
bound  zr -10.5 -10.45 zvel 0.0
bound  zr 11.5  11.55 zvel 0.0
hist n 100 ydis 0,10.55,0.5
;impulse load
def setup
omega = 2.0 * pi * freq
```

```
o_env = 2.0 * pi / env_time

end

def wave

if dytime > env_time

wave = 0.0

else

wave = 0.5 * (1.0-cos(o_env*dytime)) * ampl * sin(omega*dytime)

endif

end

set freq=1.0 ampl=4.0 ; (0.2 g)

set env_time= 1.0 ; (2 sec attack & decay)

setup

damp 0.05 200 stiff

;damp 0.1 1.0 stiff

cy time 0.10

pl hist 1 hold
```

GCRIS

C. 3DEC OUTPUTS

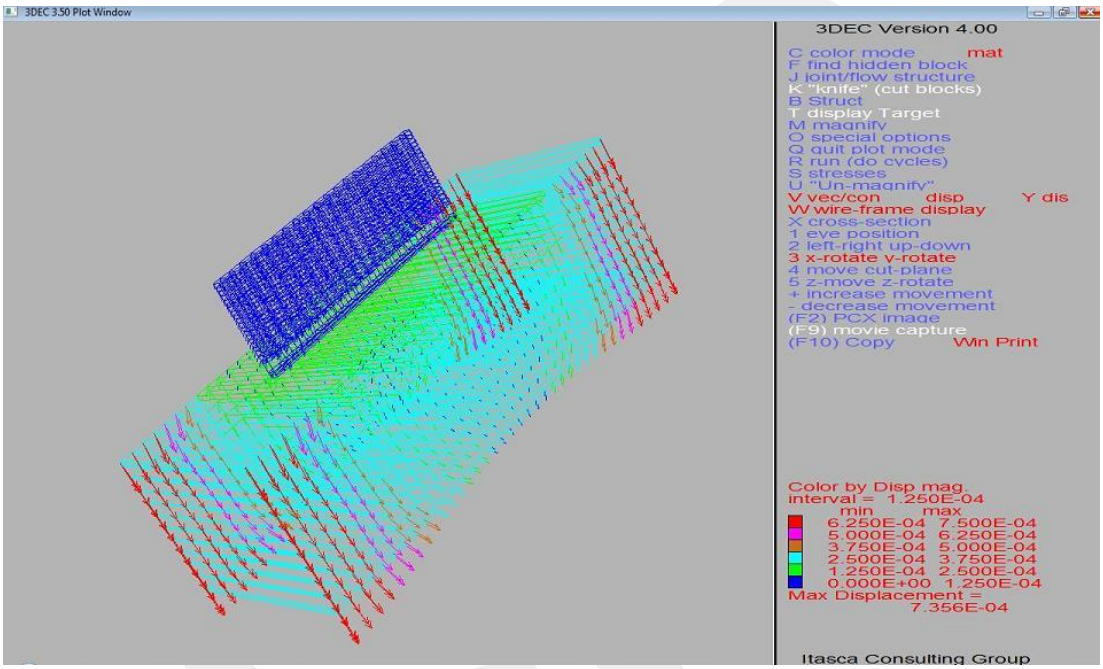


Figure C1 - 3D view of the structure (Frequency 1 – Amplititude 4 – Env time 1)

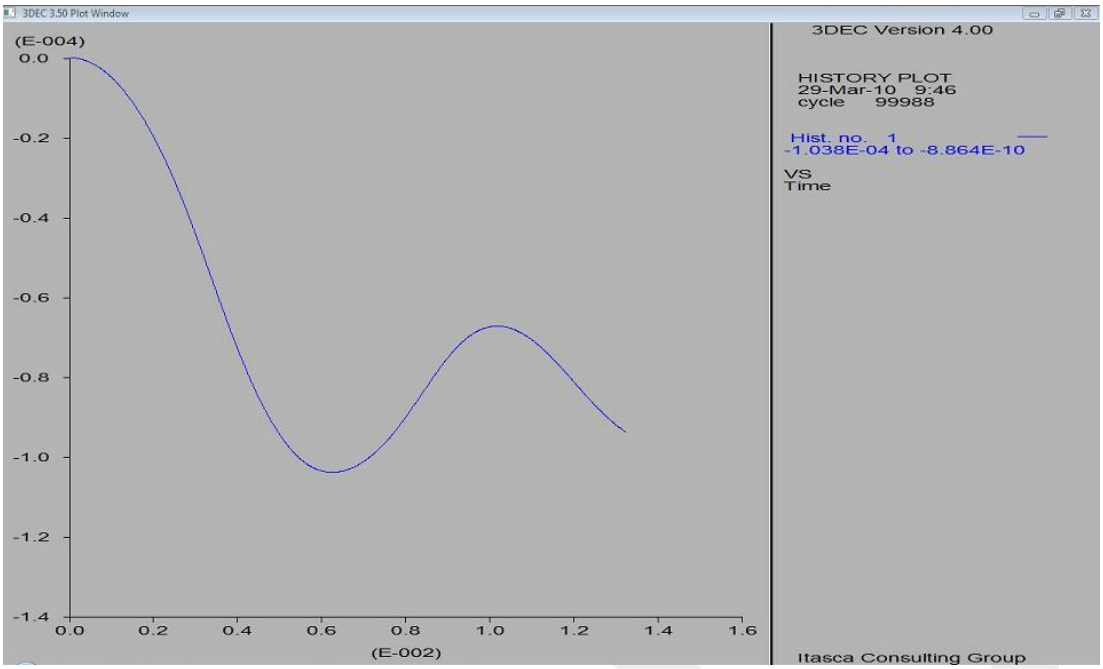


Figure C2 - History Plot (Frequency 1 – Amplititude 4 – Env time 1)

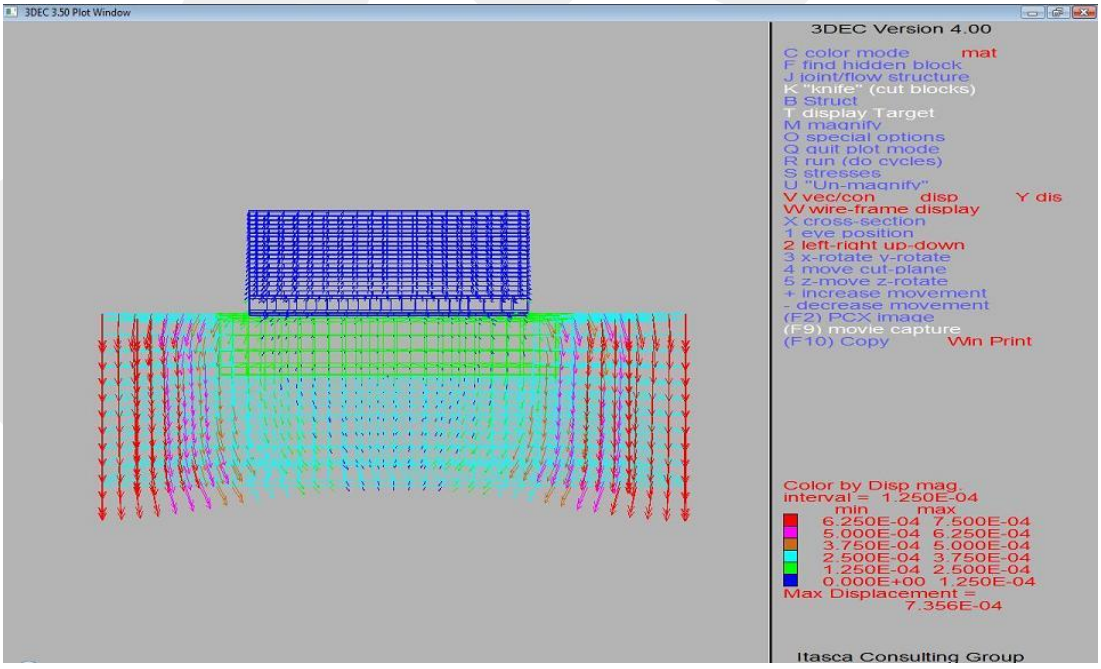


Figure C3 - 2D view of the structure (Frequency 1 – Amplititude 4 – Env time 1)

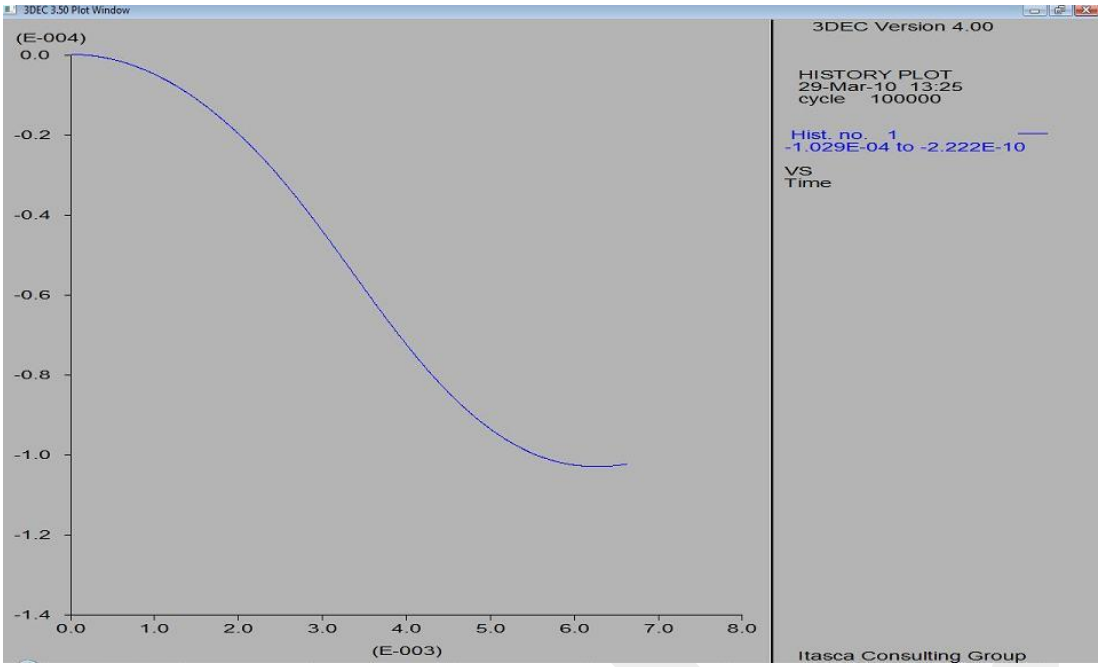


Figure C4 - History Plot (Frequency 1 – Amplititude 4 – Env time 1 – Damp 0.1)

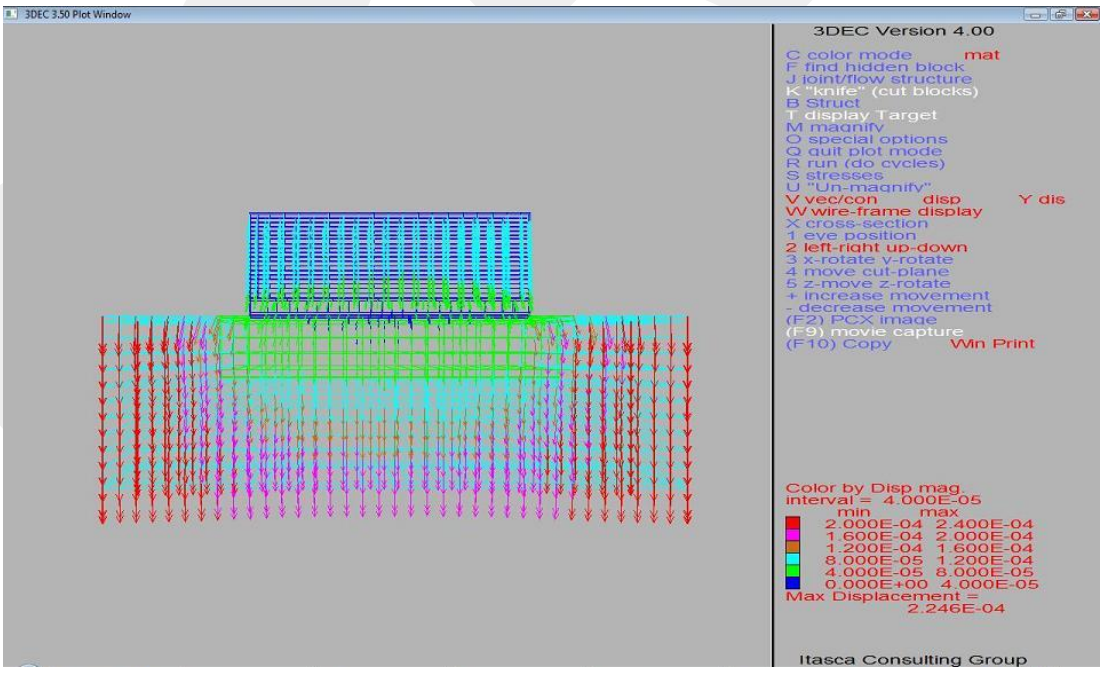


Figure C5 - 2D view of the structure (Frequency 1 – Amplititude 4 – Env time 1 – Damp 0.1)

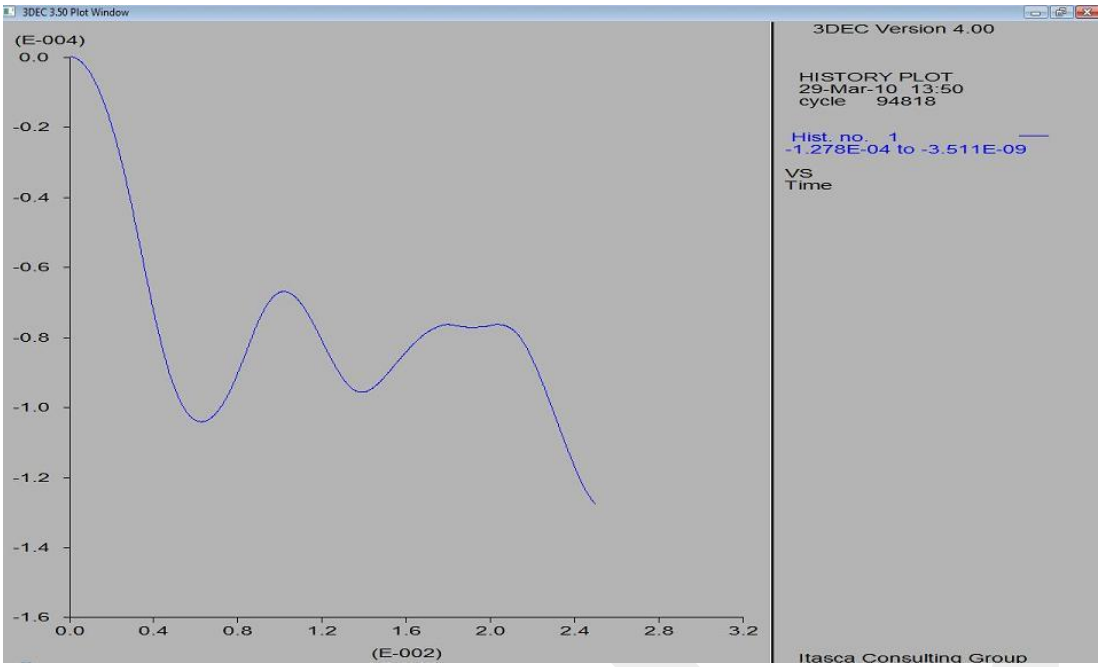


Figure C6 - History Plot (Frequency 1 – Amplititude 4 – Env time 1 – Damp 400)

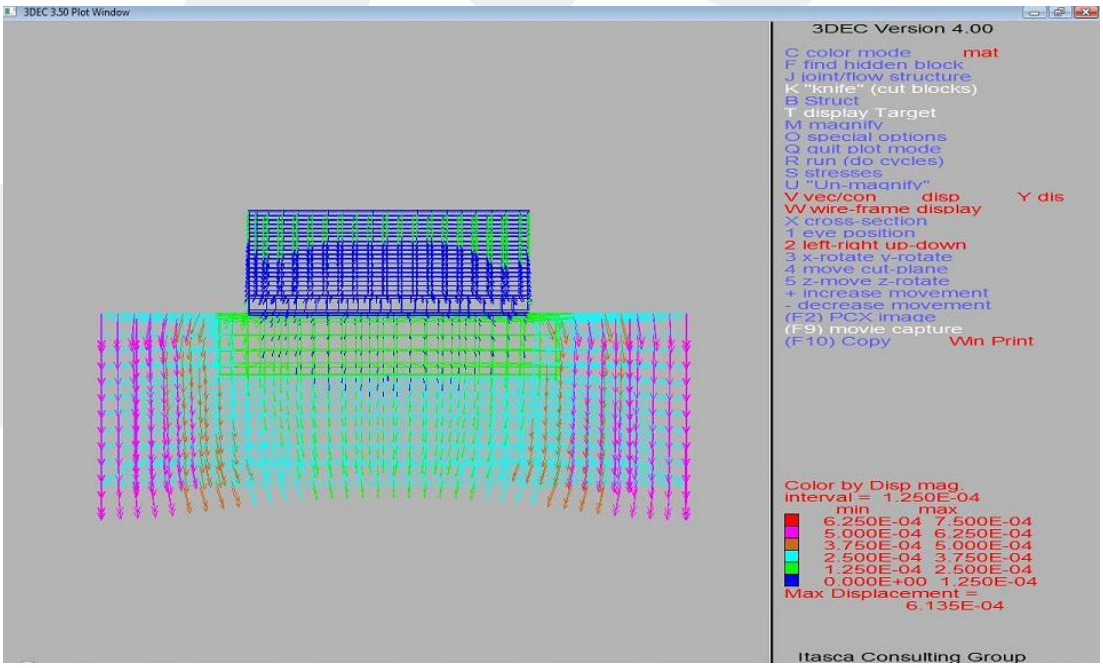


Figure C7 - 2D view of the structure (Frequency 1 – Amplititude 4 – Env time 1 – Damp 400)

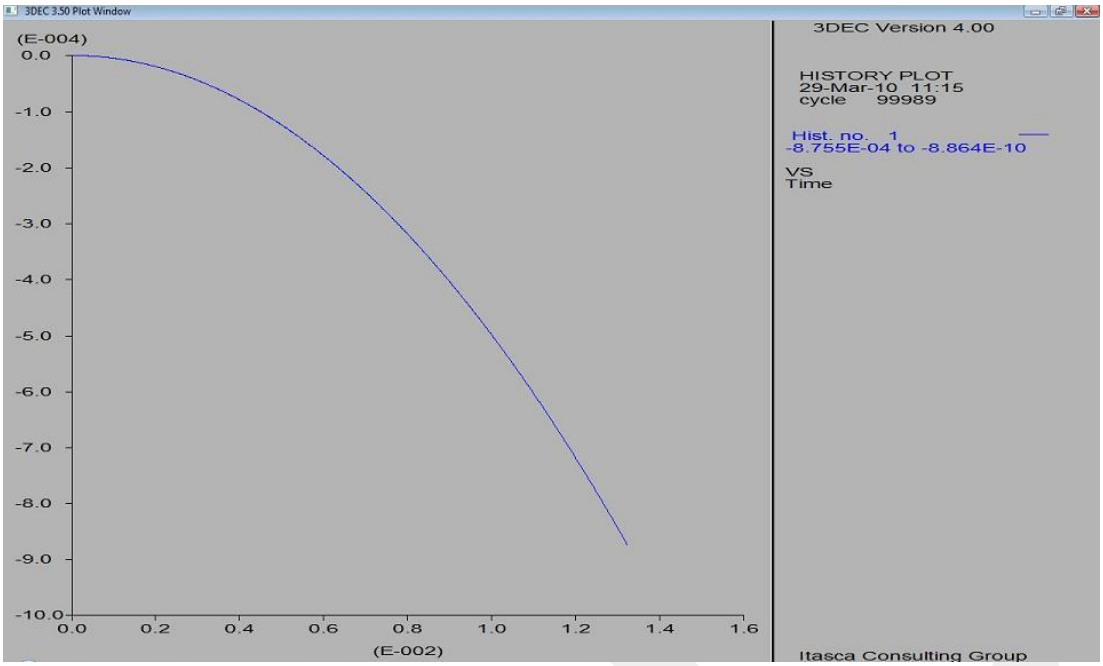


Figure C8 - History Plot (Frequency 1 – Amplititude 4 – Env time 1 – No foundation)

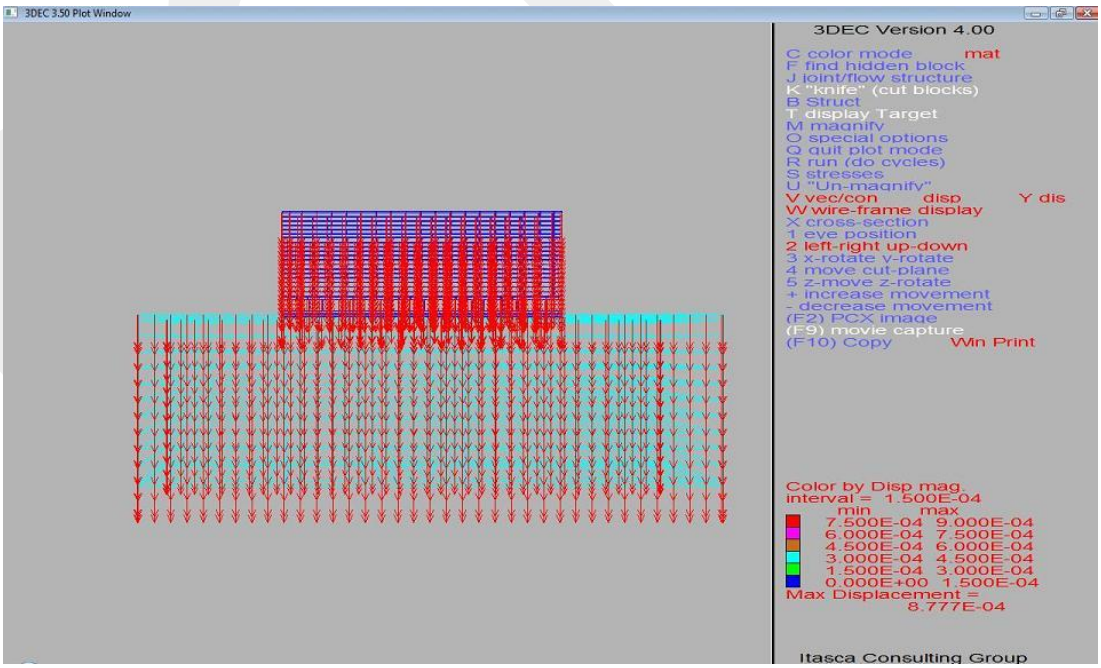


Figure C9 - 2D view of the structure (Frequency 1 – Amplititude 4 – Env time 1 – No foundation)

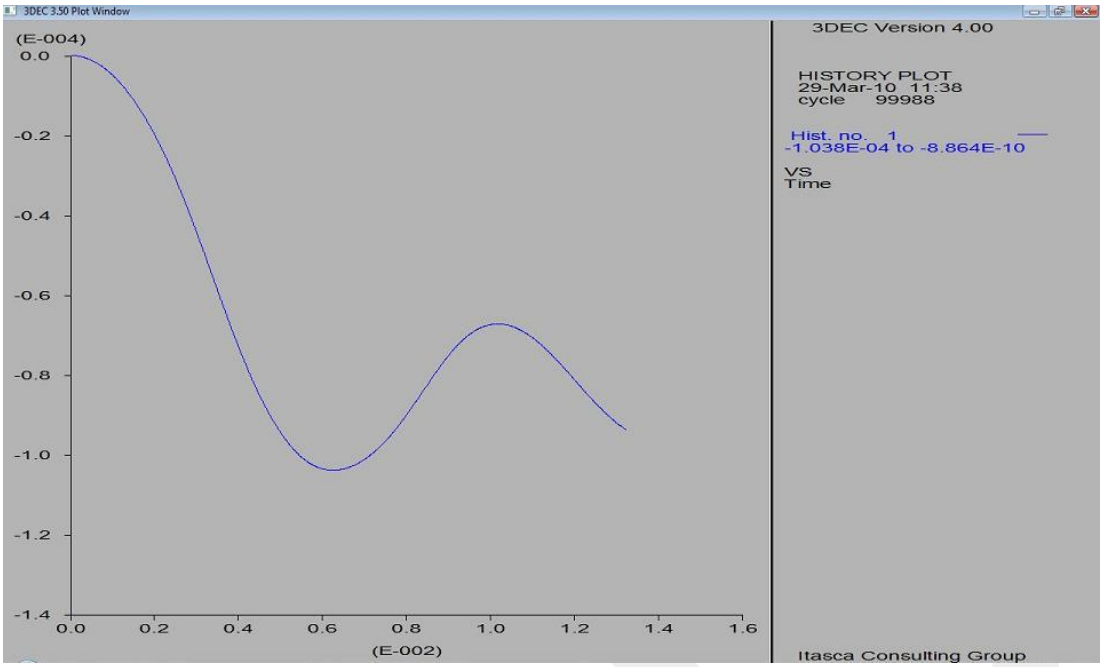


Figure C10 - History Plot (Frequency 1 – Amplititude 6 – Env time 1)

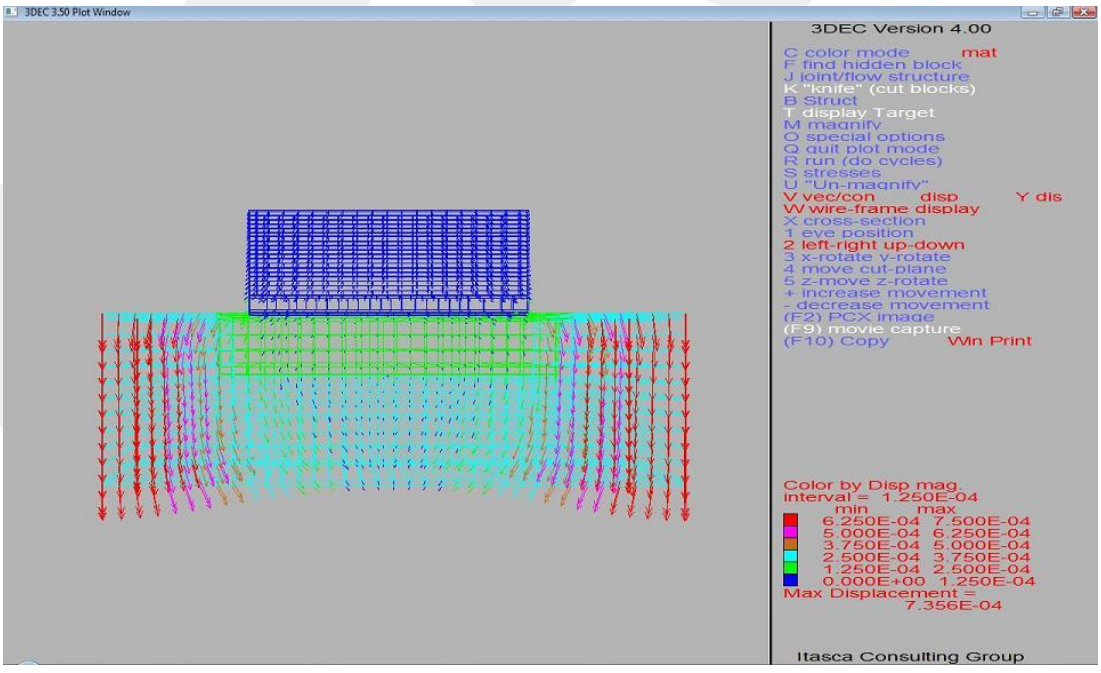


Figure C11 - 2D view of the structure (Frequency 1 – Amplititude 6 – Env time 1)

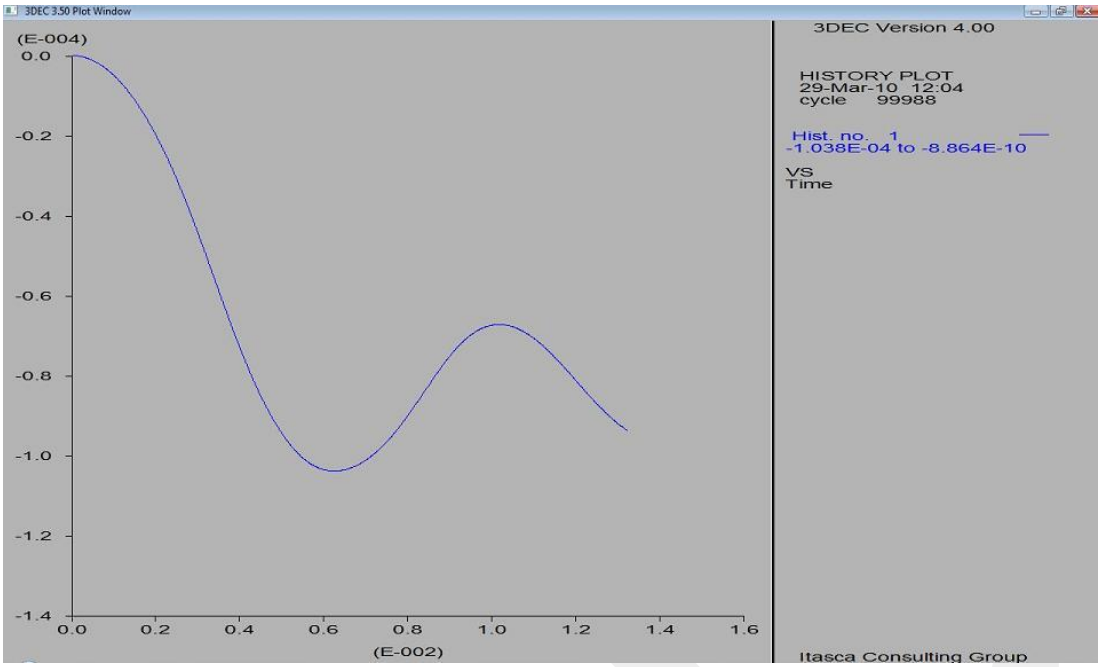


Figure C12 - History Plot (Frequency 2 – Amplititude 4 – Env time 1)

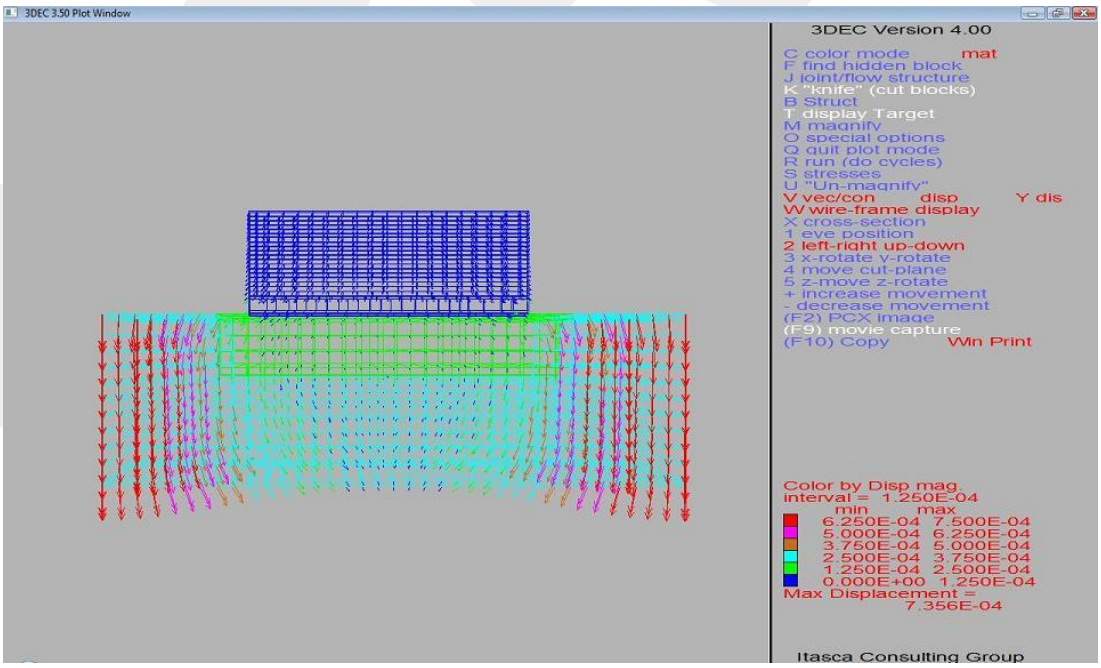


Figure C13 - 2D view of the structure (Frequency 2 – Amplititude 4 – Env time 1)

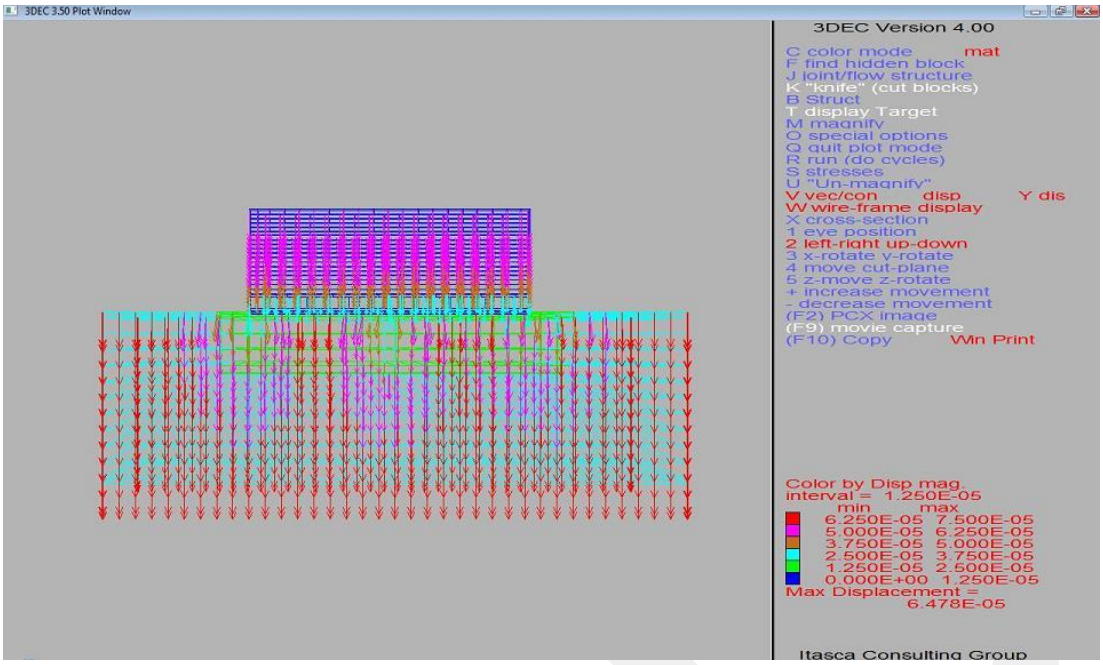


Figure C14 - 2D view of the structure (Frequency 1 – Amplititude 4 – Env time 1 – Rigid)

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