T.C.
HASAN KALYONCU ÜNİVERSİTESİ
FEN BİLİMLER ENSTİTÜSÜ

STRUCTURAL EVALUATION OF THE HURMAN CASTLE AND RECOMMENDATIONS FOR STRENGTHENING

İNŞAAT MÜHENDİSLİĞİ ANABİLİM DALI

YÜKSEK LİSANS TEZİ

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DANIŞMAN

Prof. Dr. Ömer ARIÖZ

HALİL AYDIN

OCAK - 2019

FEN BİLİMLERİ ENSTİTÜSÜ MÜDÜRLÜĞÜNE
YÜKSEK LİSANS KABUL VE ONAY FORMU

İnşaat Mühendisliği Anabilim Dalı İnşaat Mühendisliği Tezli Yüksek Lisans Programı öğrencisi Halil AYDIN tarafından hazırlanan "Hurman Kalesinin Yapısal Değerlendirilmesi Ve Güçlendirme İçin Öneriler" başlıklı tez, 08/01/2019 tarihinde yapılan savunma sınavı sonucu başarılı bulunarak jürimiz tarafından Yüksek Lisans Tezi olarak kabul edilmiştir.

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Bu tez Enstitü Yönetim Kurulunca belirlenen yukarıdaki jüri üyeleri tarafından uygun görülmüş ve Enstitü Yönetim Kurulu kararı ile onaylanmıştır.

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Enstitü Müdürü

Yüksek Lisans Tezi olarak sunduğum “Hurman Kalesinin Yapısal Değerlendirilmesi Ve Güçlendirme İçin Öneriler” başlıklı çalışmanın tarafımca, bilimsel ahlak ve geleneklere aykırı düşecek bir yardıma başvurmaksızın yazıldığını ve yararlandığım eserlerin kaynakçada gösterilenlerden oluştuğunu ve bunlara atıf yapılarak yararlanmış olduğumu belirtir ve onurumla doğrularım. …./…./2019

HALİL AYDIN

ÖNSÖZ

Bu çalışmanın meydana gelmesine vesile olan ve benden gerekli yardımlarını esirgemeyen tez danışmanım sayın Prof. Dr. Ömer ARIÖZ hocama, bana her zaman destek olan aileme, çalışma arkadaşlarıma ve bu tez boyunca teknik desteklerinden dolayı M. Bülent İLİTER'e şükranlarımı sunar, teşekkür ederim.

HALİL AYDIN

ÖZET

Kahramanmaraş İli, Afşin İlçesi, Dağlıca Kasabası kırsal alanında yer alan, Hurman Kalesi'nin yapısal sorunları işbu inceleme raporunda ele alınmıştır. Bu rapor kapsamında kalenin yerinde inceleme, sistematik fotoğraflama belgeleri ve rölöve çizimlerine bağlı olarak ayrıntılı yapısal değerlendirmesi yapılmaktadır.

Değerlendirme sonucunda, yapının korunabilmesi ve gelecek nesillere aktarılması için gereken yapısal müdahale kararlarına altlık olacak veri sağlanmaktadır. Bir sonraki adımda yapısal analiz ve hesaplarla desteklenerek sorgulanacak bu ön değerlendirme sonuçları, yapısal kararların verilmesi aşamasında yönlendirici ön bilgiyi toparlayıp sunan rapor olarak sunulmaktadır.

Raporda yapısal sorun bölgeleri tariflenmektedir. Bu bölgelere ön gözlem sonucunda yapılması mümkün olabilecek koruma çözümlerinin, pratik uygulamaya yönelik, sistemli tanımları verilmektedir.

Anahtar Kelimeler: Hurman Kalesi, yapısal değerlendirme ve öneri, raporlama, kale yapısının incelenmesi.

ABSTRACT

The structural problems of the Hurman Fortress in the rural area of Dağlıca Town, Afşin District of Kahramanmaraş Province are discussed in this review report. In the scope of this report, detailed structural evaluation of the citadel is carried out according to the on-site inspection, systematic photographing documents and the drawings.

As a result of the evaluation, data are provided to support the structural intervention decisions necessary to protect the structure and transfer it to future generations. In the next step, the results of this preliminary evaluation, which will be questioned and supported by structural analysis and calculations, are presented as a report that collects and presents the leading information in the stage of giving structural decisions.

The report describes structural problem areas. These regions are given systematic definitions of protection solutions, which can be performed as a result of preliminary observation, for practical application.

Keywords: Hurman Castle, structural evaluation and recommendation, reporting, examination of fortress structure.

CONTENTS

ÖNSÖZ .... **iii**
ÖZET .... **iv**
ABSTRACT .... **v**
CONTENTS .... **vi**
LIST OF FIGURES .... **viii**
LIST OF PHOTOS .... **x**
ABBREVIATIONS .... **xi**
1. INTRODUCTION **1**
2. STRUCTURAL PROBLEMS **10**
2.1. Southern Exterior and Interior Facades 13
2.2. Northern Exterior and Interior Facades 17
2.3. Eastern Exterior Facade 19
2.4. Western Exterior Facade 20
3. STRUCTURAL ANALYSIS STUDIES WITH FINITE ELEMENT ANALYTICAL MODELS **24**
4. PROJECTS ADDED IN THE IMPLEMENTATION STAGE **38**
4.1. Retaining Accounts - For Stage Wall 38
4.1.1. Horizontal and Vertical Effects of Retaining Wall 40
4.1.2. Verifications 41
4.1.3. Additional Effects in Earthquake Case 42
4.1.4. Verification in Case of Earthquake 43
4.2. Injection Depth and Grinding Controls 44
4.3. Steel Platform Static Calculations; 48
5.1. Regulations and Specifications to be Used in Accounts 51
5.2. Materials to Use and Features 51
5.2.1. Structural Steel 51
5.2.2. Anchor Bolts 52
5.2.3. Structural bolts 52
5.2.4. Structural Resource 52
5.3. Location Features 53
6. ANALYSIS AND DESIGN METHODS **55**
7. LOADS AND LOAD COMBINATIONS **56**

7.1. Permanent Loads (DL) ... 56
7.1.1. Steel Element Weights ... 56
7.1.2. Coating Loads ... 56
7.2. Moving Load (LL) ... 56
7.3. Earthquake Load (EX, EY) ... 56
8. STEEL CONSTRUCTION DESIGN ... 57
8.1. Main Structure - Analysis and Design Model ... 57
8.1.1. Element Sections ... 59
8.1.2. Earthquake Load ... 61
8.1.3. Designs of Steel Elements ... 68
8.1.4. Steel Structure Control ... 69
8.1.5. Connection Accounts ... 72
9. RESULTS AND RECOMMENDATIONS FOR STRUCTURAL STRENGTHENING ... 76
REFERENCES ... 80

LIST OF FIGURES

Figure 1. The Plan of the Hurman Castle 2
Figure 2. Northern Front of Hurman Castle, Relief Drawing 3
Figure 3. Southern Front of Hurman Castle, Relief Drawing 4
Figure 4. The Eastern Front of the Hurman Castle 5
Figure 5. The Western Front of Hurman Castle, Relief Drawing 6
Figure 6. General representation of structural problems of southern wall 13
Figure 7. Bottom Wall Discharge at Southern Wall Wall GD-8 14
Figure 8. Sampling of Periodic Weave Differences or Repairs Detected on Southern Exterior Walls 14
Figure 9. The Downward Trench Level External Wall Loss from the GD-8 Wall to the DD-16 (Eastern Corner Wall) on the Southern Exterior Wall 15
Figure 10. Close view of the walls between the North Wall and the KD11-KD17 18
Figure 11. North Wall Exterior, West Slope Segment View 18
Figure 12. North Wall, Main Entrance Door Exterior Wall Completely Lost and Belt Destructed 19
Figure 13.DD-7(Interesting)Debris Residue which can be followed on the Eastern Outskirts 20
Figure 14. Irregular Demolition in Western Front, BD-8, 9 21
Figure 15. Non-Plane Demolition of Long Span Wall Segment in the Western Front and BD-6 Wall 22
Figure 16. Turkey Earthquake Map – Kahramanmaras Seismicity 27
Figure 17. North Front Entrance door segment a) modern geometry model b) part completion proposal model 28
Figure 18. Western Front D8,9 Sign front and side facade PORTION a) modern geometry model b) part completion proposal model 28
Figure 19. Belt Structure G + Ex Stresses a) Consolidated Structure b) Present Structure - S11 Stress (kPa) 29
Figure 20. Belt Structure G + Ex Stresses a) Consolidated Structure b) Present Structure - S22 Stress (kPa) 30
Figure 21. Belt Structure G + Ey stresses a) Consolidated structure b) Present Structure - S11 Stress (kPa) 31
Figure 22. Belt Structure G + Ey stresses a) Consolidated structure b) Present Structure - S22 Stress (kPa) 32
Figure 23. Tower Structure G + Ex Stresses a) Consolidated Structure b) Present Structure - S11 Stress (kPa) 33
Figure 24. Tower Structure G + Ex Stresses a) Consolidated Structure b) Present Structure - S22 Stress (kPa) 34
Figure 25. Tower Structure G + Ey stresses a) Consolidated structure b) Present Structure - S11 Stress (kPa) 35
Figure 26. Tower Structure G + Ey stresses a) Consolidated structure b) Present Structure - S22 Stress (kPa) 36
Figure 27. Slope slip analysis of the West Facade KD-4 Wall-1 44
Figure 28. Slope slip analysis of the West Facade KD-4 Wall-2 45
Figure 29. Slope slip analysis of the West Facade KD-4 Wall-3 46
Figure 30. Slope slip analysis of the West Facade KD-4 Wall-4 47
Figure 31. Platform Layout Plan 48
Figure 32. Platform 3-D View 48
Figure 33. Portion s Account Platform 49
Figure 34. PORTION -1- Element Numbers 49

Figure 35. PORTION -2- Element Numbers.......... 50
Figure 36. PORTION -3- Element Numbers.......... 50
Figure 37. Map of Kahramanmaras Province Earthquake Region.......... 54
Figure 38. PORTION -1- Analysis 3D Model View 57
Figure 39. PORTION -2- Analysis 3D Model View 58
Figure 40. PORTION -3- Analysis 3D Model View 58
Figure 41. PORTION -1- Structure Principal Axis Periods, T1, x = 0.07683 s.......... 61
Figure 42. PORTION -1- Structure Principal Axis Periods, T1, y = 0.08992 s.......... 62
Figure 43. PORTION -2- Structure Prime Axis Periods, T1, x = 0.12848 s 64
Figure 44. PORTION -2- Structure Prime Axis Periods, T1,y = 0.4319 s 64
Figure 45. PORTION -3- Structure Prime Axis Periods, T1,x = 0.09772 s 66
Figure 46. PORTION -3- Structure Prime Axis Periods, T1,y = 0.15513 s 66
Figure 47. PORTION -1- Steel Design Results - Impact / Capacity Ratios 68
Figure 48. PORTION -2- Steel Design Results - Impact / Capacity Ratios 69
Figure 49. PORTION -3- Steel Design Results - Impact / Capacity Ratios 69
Figure 50. PORTION -1- Vertical Displacement (G+Q) 0.18 cm < 1.06 cm (320/300) OK . 70
Figure 51. PORTION -2- Vertical Displacement (G+Q) 0.201 cm < 1.09 cm (327/300) OK 70
Figure 52. PORTION -3- Vertical Displacement (G+Q) 0.310 cm < 1.15 cm (344/300) OK 71

LIST OF PHOTOS

Photo 1. The South Side View of Hurman Castle 1
Photo 2. Hurman Castle a) Exterior wall structure in the outer wall structure, wooden vertical chain systems and internal rubble filling upper elevations of the observation b) interior vault of the eye of the eye is observed in the pattern of stone decay.Travertine type stone material is a common type of decay. 9
Photo 3. Adana Andıl Castle and Adana Bucak Castle Examples 9
Photo 4. Knitting Stone Loss Around the Entrance Wall of the South Wall 15
Photo 5. View of South Wall Interior Wall Problems 16
Photo 6. North Wall Exterior View 17
Photo 7. The View of the Interior from the West from the West. Only internal debris and partially wooden timber parts of the wall can be detected on site 23

ABBREVIATIONS

BD	**:** West Wall
BDU	**:** West Wall Top
BI	**:** West Interior
DD	**:** East Wall
DI	**:**East Interior
GD	**:** South Wall
GI	**:** South Interior
KD	**:** North Wall
BC	**:** Before Christ

1. INTRODUCTION

Afşin Hurman-Rumman fortress was built on a hill in the town of Marabuz (Dağlıca), consisting of a large rock on the north side of the district and dominating the region. The fortress on the bedrock was not built properly and was built according to the topography of the rock. The construction date is unknown. There was no inscription on the building. However, the data obtained from the architectural style, construction technique and excavations indicate that it was a fortress dating from the Roman period. We do not know the date of construction because the castle does not have an inscription.

Hurman Castle is on the Silk and Spice Road, like a dry house structure. The gate to the castle is on the west side. In fact, in Roman times, it is understood that Kangal was on the Roman road from Arabisos (Afsin-Efsus) to Sebesteia (Sivas). Kangal-Afşin-Sivas route has remained important in the Ottoman period as it was in ancient times. Caesare-Kayseri 21, Zerezde 10 Zamantu su 30 Starting from Karakilise village, there is another road to Arabissos (Afsin-Efsus), Melitene (Malatya) and the Euphrates. (Tanadaris) Karakilise and Maragos-Marabuz (Dalıca) 14 Roman Milidir (Aleppo in the Ottoman Province Salname, 2012).

Photo 1. South Side View of Hurman Castle

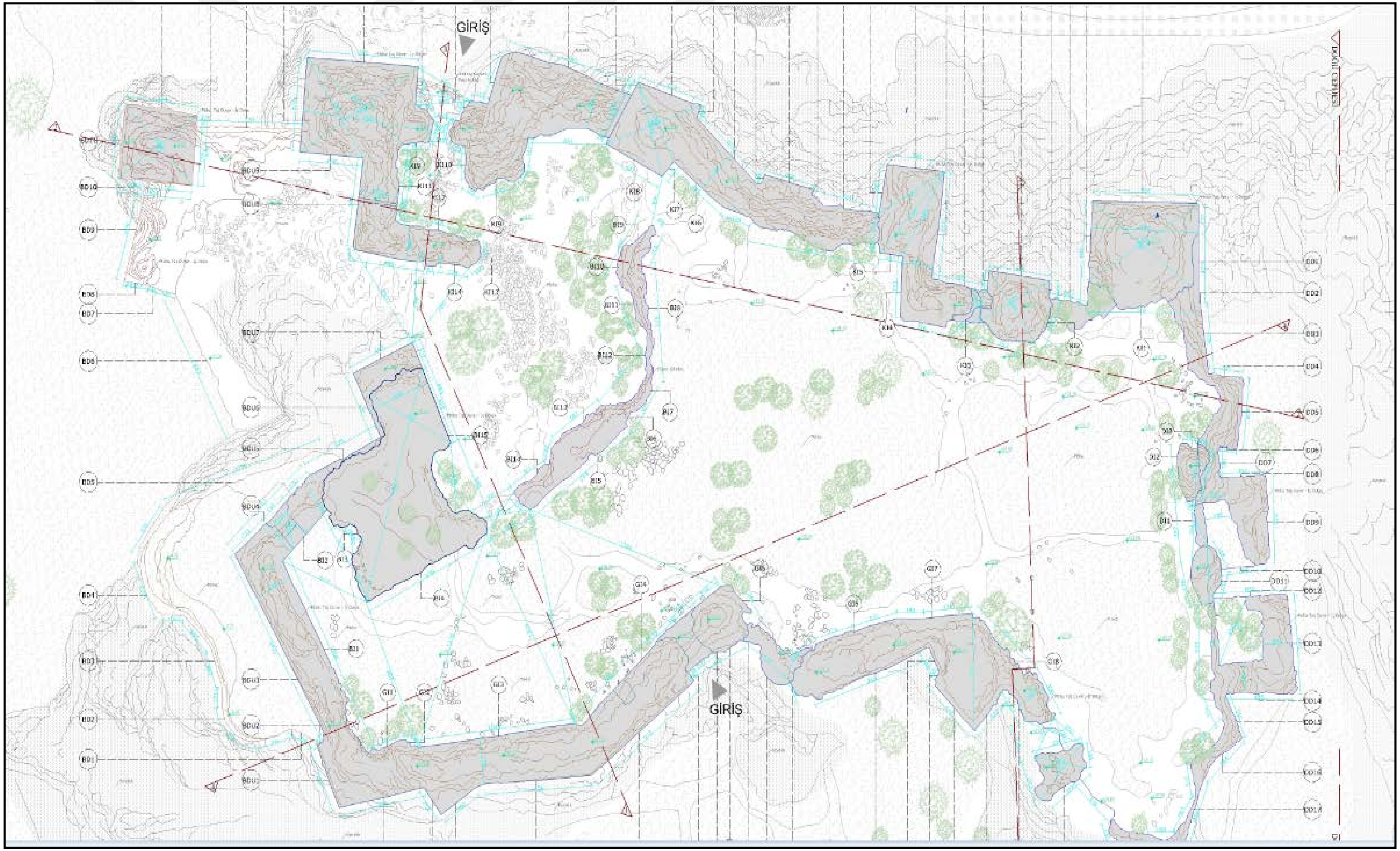


Figure 1. The Plan of the Hurman Castle

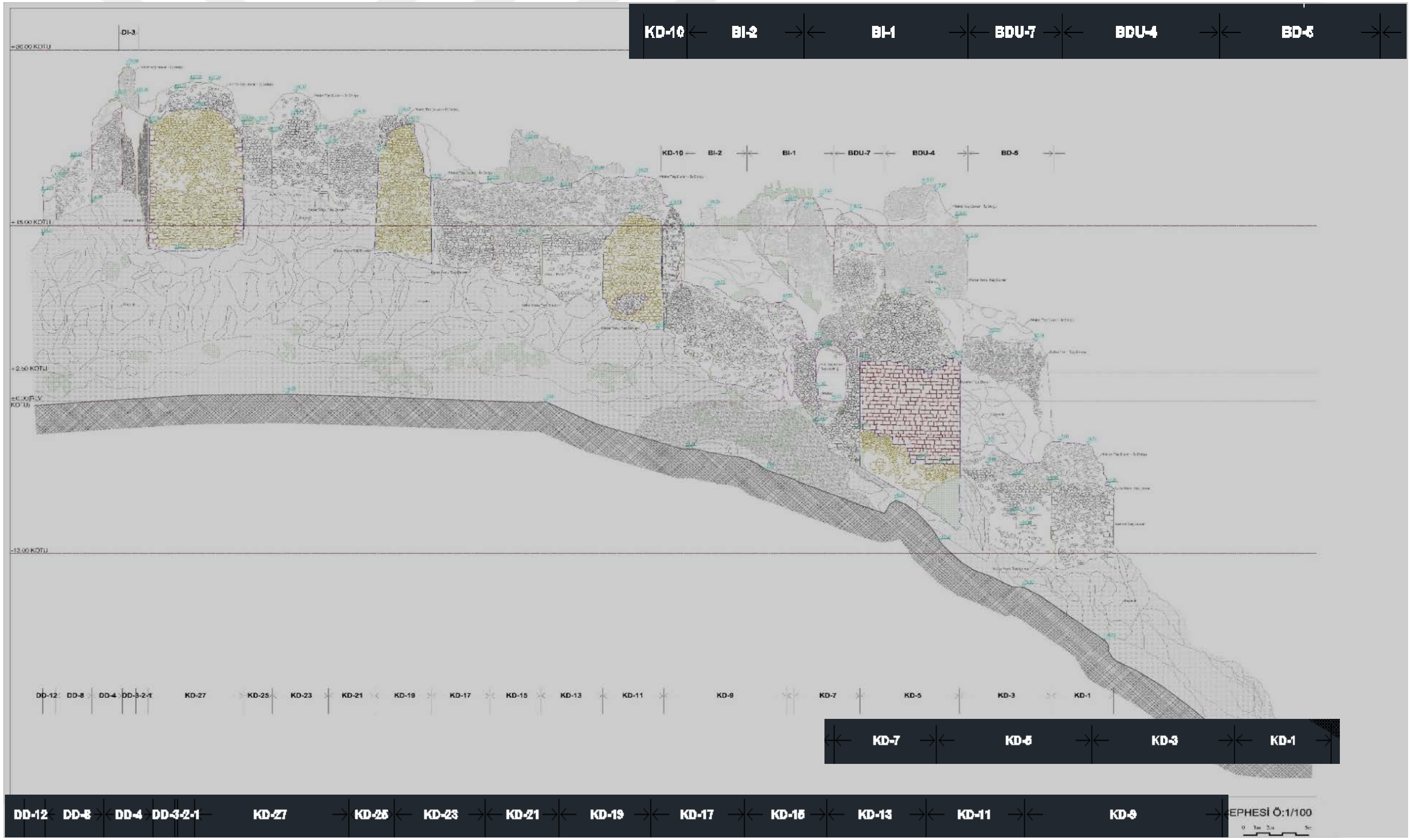


Figure 2. Northern Front of Hurman Castle, Relief Drawing

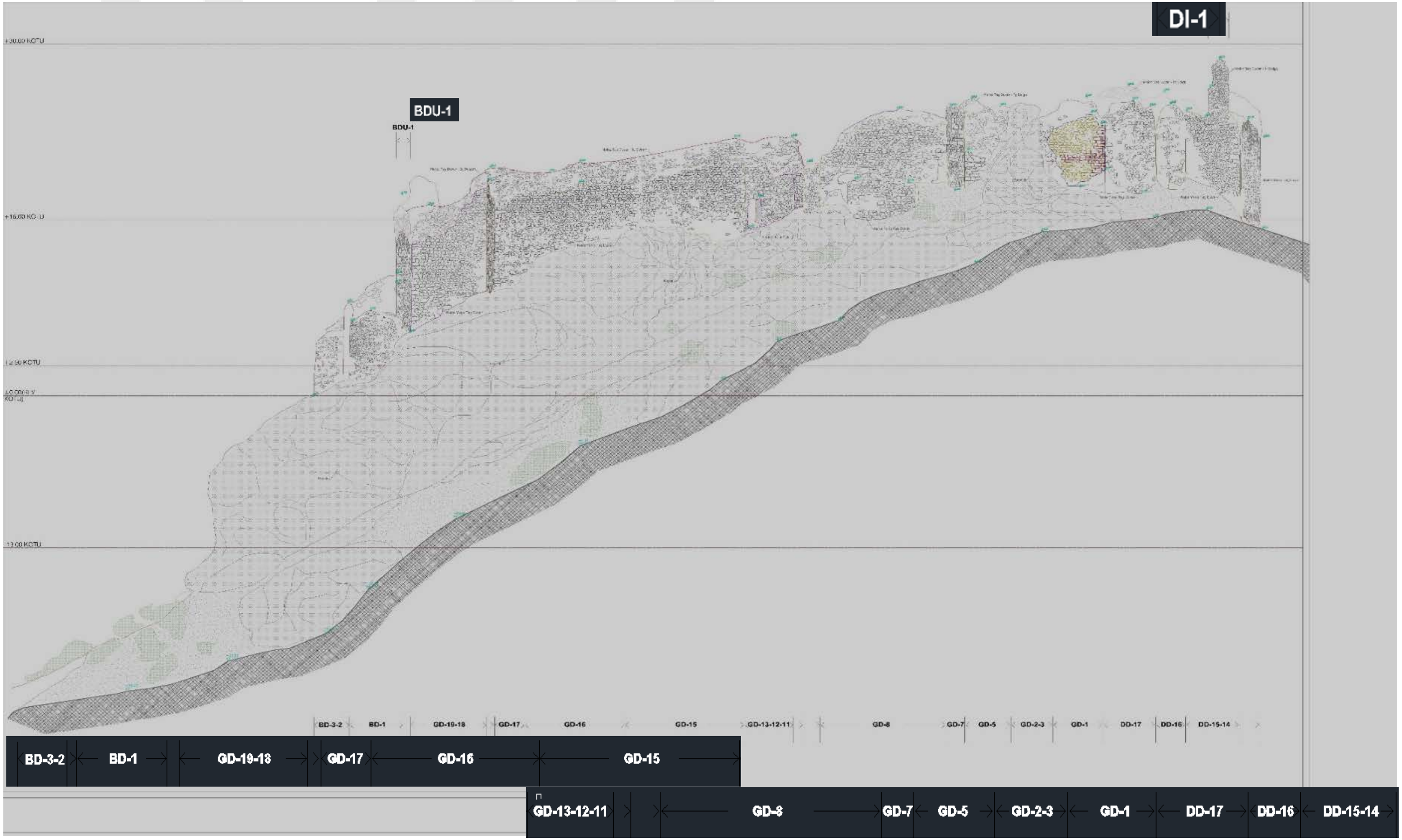


Figure 3. Southern Front of Hurman Castle, Relief Drawing

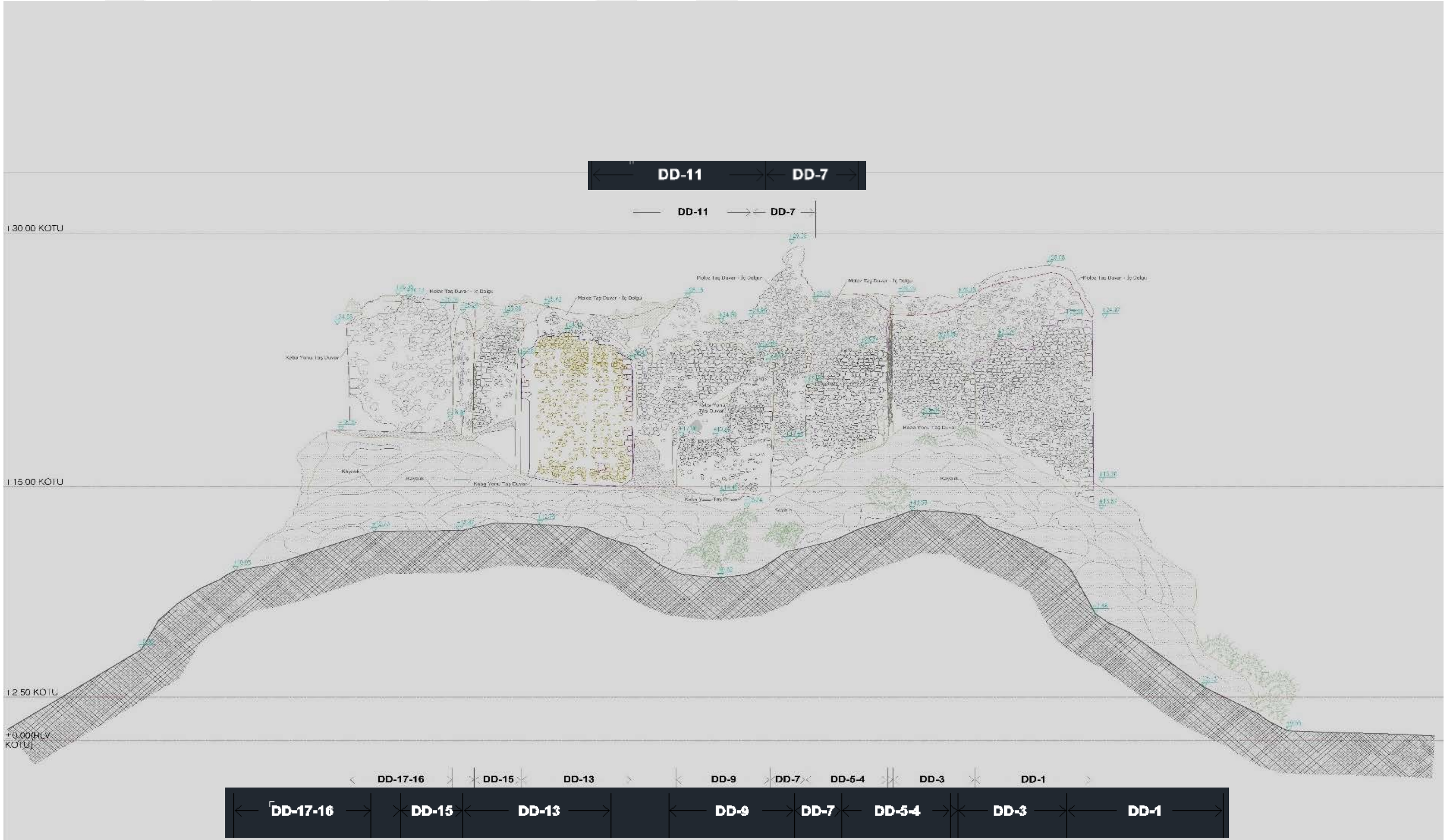


Figure 4. The Eastern Front of the Hurman Castle

Figure 5. The Western Front of Hurman Castle, Relief Drawing

The fortress structure is located on a hill that will have four facades in the direction observed today. The eastern part is the average in the east, as it appears from the surveys at the highest elevation; +22.0 m The fortress structure resting on the hill showing a slope with a difference of -4.0 m in the west; hence, it has a structure that stretches from west to east. Hence, the entrances of the building are provided from the southern and northern façades where a larger horizontal slope form is present. The structure of the castle, whose inner structure is covered by the rubble ruins of the walls that will be explained later, consists of two main sections, which are divided into three main parts which are surrounded by the outer walls of the outer walls that divide this main area into two parts. The elevation difference between these two steps forming the whole interior area is 14.0-6.75 m = ~ 7-8 m. The outer walls of the structure are approximately 60-70 m on the north and south façades and this is ~ 40 m on the eastern and western façades. However, especially in the western facade of the protruding bush structures and the steep slope of the plan formed due to the form of the uneven form due to the length of the wall in this short façade wall façade is approximately 2 times the follow-up.

Bastion numbers; on the west facade there are 4 units, on the north facade 5 pieces, on the east facade 4 pieces and on the south facade 1 pieces. The side lengths of the bushes in the plan are up to 7-8 m. The largest horoscope in the area of the castle, which occupies approximately 2300 m2, covers an area of ~ 80 m2. Considering that the average wall thickness is 2 m, it should be taken into consideration the possibility of having an interior space with a plan length of 7-8 m. Therefore, it can be assumed that the overall wall thicknesses of the structure continue at the same levels in these signs. As of today, it is not possible to follow the places in the ruins of ruined buildings. Both the towers and the rubble piles with a significant volume in the interior of the castle prevent these findings.

It is seen that the wall thicknesses are between 2-3 m in the original structure and these thicknesses have decreased to 50-100 cm especially in the upper elevations. When the wall constructions are examined, as a construction technique; It is seen that the walls are formed by the rubble mortar belonging to the low binding class and the rubble heap formed from the crushed stone. The outer walls of the walls were made of knitted stones with a depth of 20-30 cm. This mesh is located on both the inner and outer surfaces of the wall. It was observed that this wall technique of the structure was not used in dry-erected walls by using smooth cut-stone cut stone observed in similar structures of Hellenistic and Roman periods in which similar structures were used in similar structures of the Byzantine period. On the contrary, in this structure, a technique that can be more easily constructed is used. However, although similar tombstones were used in samples such as the Adana Andıl Fortress observed in the surrounding provinces from similar fortress structures, it can be observed that the thicknesses of the isodomic thicknesses are similar in a similar way. In the Hurman Fortress, although the isodomic sequences can be determined, the thickness of the sections is not the same. It is a very clear indicator that the coarse chipped row stones have more grift surfaces and the unit dimensions do not hold along with each other.

The walls were constructed both in the construction phase and in the wooden structure of the wooden scaffold. These beams are composed of rows of elements placed perpendicular to the wall structure. Circular section of ~ 20 cm diameters used at approximately 50 cm intervals were placed on a wall at 2 meters. These rows were erected and used in the walls of the bushes (corner turns), so that the bush rectangular rubble construction was built with a horizontal reinforcement in meters in 2 directions. The rarity of these rows of beams increases in flat wall constructions. The castle has thick plan dimensions such as zodiac signs. The debris removed from the building is thickened and increased the frequency of wood beams in order to increase the internal debris density and to prevent cracks under the lateral effects. As a matter of fact, in the following stages, while the state of decay is given, it has been revealed that there is a low resistance as a reason for the formation of decaying and decaying processes of the wooden timber in the lower elevations.

The basic structure of the castle structure is not in question. It is thought that the walls and bastions sitting on the main rock are rising and ending with the same thickness and knitting techniques. In fact, as the structure will not start from the basic level of the problems, as it will be mentioned later, it is known that it goes down from the upper elevations; It can be said that the performance of the foundations of the building in terms of conveying the vertical loads to the rock does not constitute a problem even if the performance of this technique is required. As a matter of fact, it is not necessary to base the structure with a special technique. However, it can be said that rubble and wall weaves may have been made by carving or filling planes during rock placement.

It is assumed that the building stone is limestone or travertine stone of medium hardness and it is thought that the original mortar with lime based pozzolonic feature is used in the inner filler. As a matter of fact, the appearance of eye-eye or beehive-type material degradation seen in stones with a travertine type heterogeneous structure can be detected on the walls. However, it is thought that the main material problems of the building are not destroyed in this wall wall elements but the interior rubble mortar is in the interior rubble mortar and it is thought that the walls have been demolished by starting from the top. In this structure, the wall thickness is relatively high, it can be said that the outer wall weave acts as a mold for the inner main carrier filling beyond the structural contribution. The wooden timber system also increases the binding by tightening this filling. Therefore, the main bearing elements of the structure can be given as a combination of the internal debris and the wood-based system.

Photo 2. Hurman Castle a) Exterior wall structure in the outer wall structure, wooden vertical chain systems and internal rubble filling upper elevations of the observation. b) In the interior vault stone lattice, deterioration is observed with the eye eye condition. Travertine type stone material is a common type of decay.

Photo 3. Adana Andıl Castle and Adana Bucak Castle Examples

2. STRUCTURAL PROBLEMS

The definitions of structural problems and their distributions are given in this section. It can be said that the problems actually correspond with one another in parallel with one another. All of the building walls have become a result of the loss of the outer wall starting from the upper elevations and then the loss of the inner debris. In some parts of the walls, while the upper elevations were degraded, it was observed that there were losses in the outer walls near the main parts. In addition, the aperture resulted in the loss of the carrier's original carrier form as part losses in the transition elements. Partial demolitions in the building elements such as the entrance gate arch and interior vaults disrupt the structure carrier form and indicate that the deformation will continue with the demolitions. Finally, there is a loss of efficiency due to both loss of material and loss of material in the wooden girder system which connects the inner and outer walls holding the internal debris together.

A) Damage to the upper walls of the outer walls of the tombs of the castle walls.

B) In the debris, mortar decay is observed, the loss is not observed. The losses are progressing in the whole structure as slope melting from the upper parts to the lower parts.

C) Horizontal timber beams, which provide loyalty in the rubble wall system, disappear as seen above. Melting with wall melting, burning, breaking, material degradation are the main reasons.

D) Structure elements such as arches and vaults are exposed to demolition and geometry degradation due to material degradation.

E) High wall fragments (formation) finished as a result of the destruction of the edges and other perpendicular walls, including the risk of collapsing out of the plane; Under some façade walls above the cantilever will occur until the carvings are demolished.

These deformations take place simultaneously in some regions; They are observed as events in which a pattern has been triggered by triggering each other and the last one causing a serious local loss has been observed. In some areas, there is a possibility of step-by-step collapse, but they exist as individual. they are seen as more likely to progress to more serious disturbances.

It is necessary to list the damage factors that the structure has been exposed to for years before the problem zones are described. The most important factor in building damage; under the influence of climatic changes (wetting drying cycles, freeze-thaw cycles) of the structure, which has lost its top cover, the possible wooden flooring where it exists (which may be thought to protect the lower wall layers from the climatic effects) or the inconvenient drainage of the wall above the walls. This is because it has been for centuries. In this case, the climatic effects of the structure, which progressed further to the inner cavity, which would show weaker resistance, in particular, improved the material degradation of the structure. As a matter of fact, material degradation (decrease in original strength) on the basis of structural deformations seen on all four sides of the building is the loss of resistance against external loading conditions and secondary loads resulting from its own dead weight and deformation occurrences occur.

Climatic effects are gül drying cycles “. The vegetation and salinization effects due to humidification were formed in the building joints and as a result the losses in the material size were the most significant cause of deformation. It is assumed that these material deformations create local minor cracks on the outer walls that do not have a homogeneous strength. (The starting of the upper wall at the top) The progress of the cracks primarily destroyed the total movement and inertia in the outer walls of the building. These single wall problems directly abolished the integrated integration of the building walls. When the outer walls were destroyed, the internal debris, which was open to climatic effects, began to crumble and into a process of rapid disappearance. This weakened the resistance against lateral effects such as earthquakes. This counter-resistance has transformed the individual wall level into an individual activity. However, the relatively high out-of-plane equilibrium in relatively thick-walled areas corresponds to the higher rate of erosion of the rubble from top to bottom, whereas the lower thicknesses of the walls are variable. (some part of the wall can be demolished to the basic level, while in some parts only the upper parts of the wall are melts.)

Although secondary destructive effects are not thought to be as effective as the first class effect, they are thought to play a role in local demolitions. These effects are wind and light seismic effects to trigger out-of-plane movements and vibrations of the walls of the structure. However, it is thought that these effects do not create any damage in the original compact form (when the structure is not under severe material degradation). It is thought that these materials were damaged gradually after loss of strength as a result of structural defects. It should be noted, however, that the internal debris filler is now vulnerable to effects such as seismic and wind effects, as a result of the destruction of the walls, resulting in the inch by inch graded melts and filling. As a matter of fact, the bevels in the inner filling are also formed under these effects.

It seems logical that the structure where the wind effects may have damaged the structure is on the slope. The castle is built on one of the high hills in the region. Therefore, it is at a relatively high point. It can be said that the wind speed acting on the tower fronts is much higher than the wind speed at the top of the hill. However, it is more likely that the seismic effects of the structure which is sufficient to withstand the thickness of the building wall height of about 4-5 m, have produced more powerful-specific degradation symptoms. It is very likely that the seismic movements have devastating effects on the local parts due to the loss of integration of the above mentioned structural inertia, cracks, internal fill and joint material. As a matter of fact, it is obvious that in defense structures where masonry type structures are relatively high, the seismic effects are more likely to occur due to the fact that the building masses are more in the defense structures (the full volume / space ratio in these structures is less than the other monumental structures). This type of lateral effects may have caused the material to be deformed, with vertical deformation or lost - under-emptied, suspended masonry components having already lost their relatively low strength.

In addition, it shows that the structure is not exposed to ground-based deformations or damages. As a matter of fact, this result can be supported and grounded by the damages that are present in the structure. The detailed description of the decay zones below and the description of this decay are more clearly understood.

2.1. Southern Exterior and Interior Facades

Figure 6. General representation of structural problems of southern wall

The southern façade of the fortress contains all of the above mentioned problems. However, from the upper elevations to the lower elevations, structural problems are described; It is seen that the outer covering of the walls disappear at approximately similar elevations along the line at the upper elevations and the internal debris is exposed to serious wear by the loss of the outer covering to the lower elevations on the GD-1 wall.

The eastern wall of the southern wall line is more problematic than the western walls. In these parts, the outer wall covering of the walls and the internal debris also have discharges at the lower elevations. In other words, the lower part of the walls left the emptied middle segments as console running parts. It is inevitable that these parts will be destroyed by progressive decomposition. The most serious losses can be detected on the GD-8 wall located to the east of the door entrance. The outer wall was destroyed to the lower elevations. On the other hand, the outer wall loss from the GD-8 wall to the DD-16 wall is detected in a horizontal line along the order of the wooden chain near the base level. It is interesting to note that this loss horizontal line is followed by the timber chain. It can be said that the loss in the fronts (burning break or vandalism) triggered the loss of this line. Periodic repair and intervention traces on the southern walls can be detected by different external walling techniques. This reveals the possibilities and traces of vandalism or repair interventions.

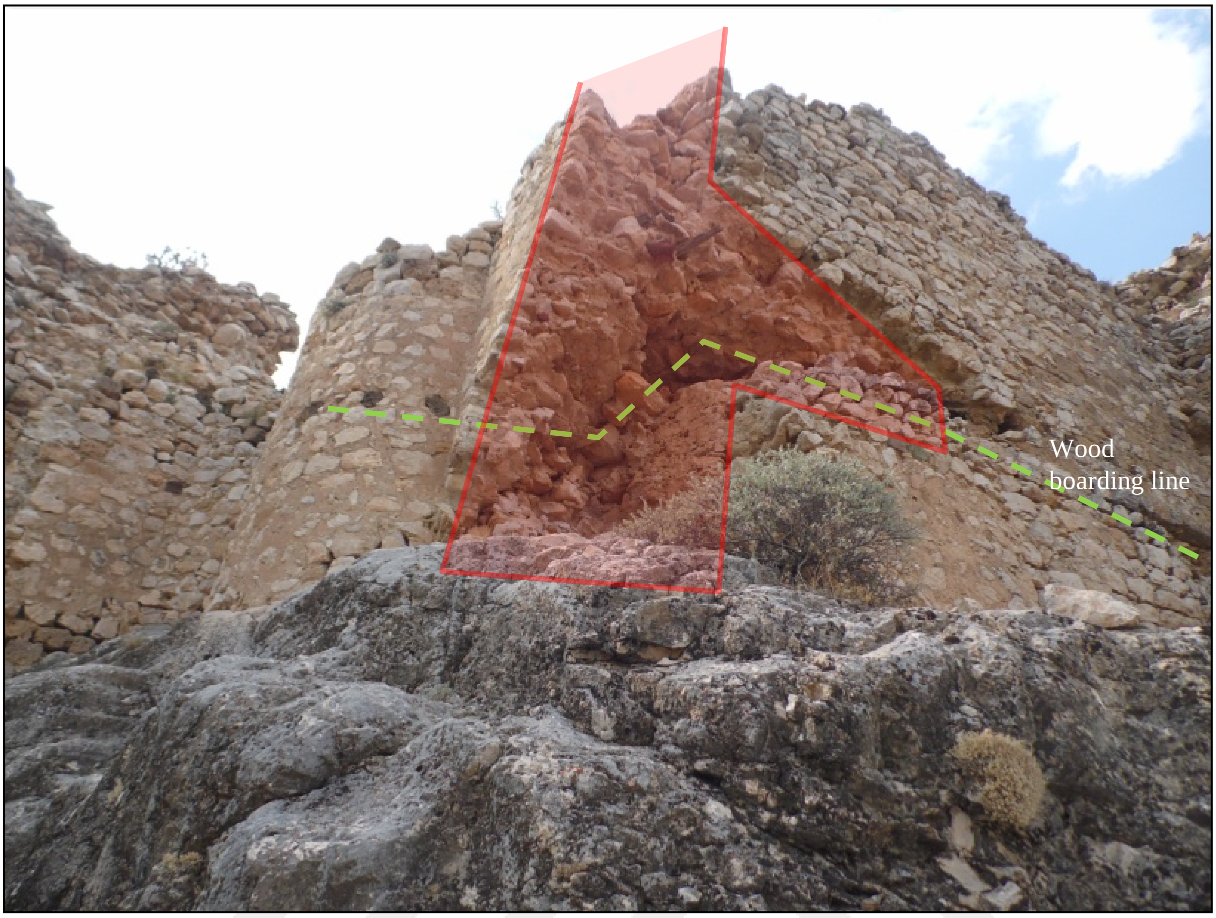


Figure 7. Bottom Wall Discharge at Southern Wall Wall GD-8

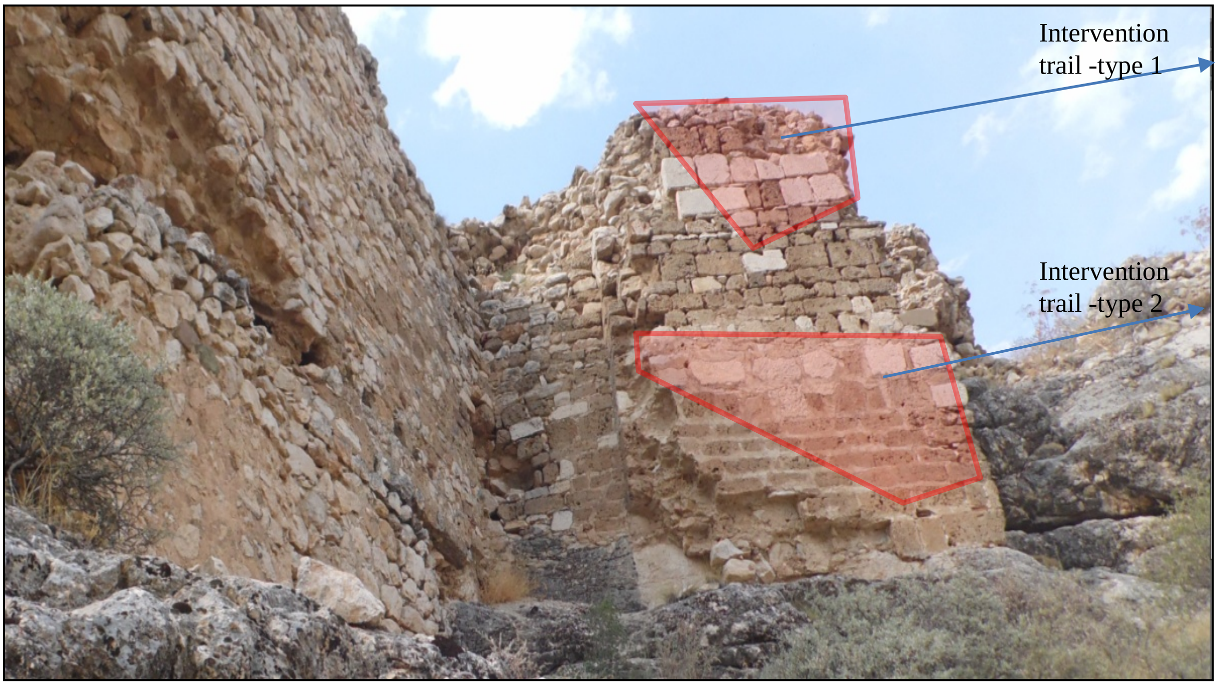


Figure 8. Sampling of Periodic Weave Differences or Repairs Detected on Southern Exterior Walls

Figure 9. The Downward Trench Level External Wall Loss from the GD-8 Wall to the DD-16 (Eastern Corner Wall) on the Southern Exterior Wall

Photo 4. Knitting Stone Loss Around the Entrance Wall of the South Wall

The entrance gate arch structure in the South Wall is still intact and appears to be carrying out the load transfer task. However, the loss of intrados stones in the outer wall segment proves that the stresses in the section of the arch are elevated relative to the original state. Therefore, the belt structure is likely to undergo an ongoing structural demolition process.

Photo 5. View of South Wall Interior Wall Problems

It can be stated that the main problem is observed when the inner walls of the South Walls are observed. The inner surface wall of the wall line has been completely destroyed and there is a serious loss in the thickness of the rubble. While the inner mortar structure of this rubble continues to crumble while the losses continue, the melting of the wall is an obvious ongoing process. On the other hand, as stated in the above section, the outer hull in the inner and outer wall segments forms the hollow form in the interior debris wall cut (as a result of abrasion). As the loss of the fragments in the inner debris of the wall continues with the part wear, the wall tries to reach the natural slope form. As of today, it is possible to say that the internal debris in the South wall survives an average of 60 ° along the line. However, it can be said that this angle is so steep that the crushed stone that forms the inner debris is not normal for the agglomeration, so when the mortars lose their binding, the form turns into shallow angles.

2.2. Northern Exterior and Interior Facades

Photo 6. North Wall Exterior View

When the North Wall was examined from the outside, there were no problems other than the southern wall at the lower elevations of the wall. It was also determined that the joints were completed with periodic repairs. The problem of the main structural threat along the length of the wall is the disappearance of the outer wall of the upper elevations and the inner filling. While this problem is present in the entire wall line, it is more effective in the western half of the wall line. Here, the outer walls have been lost to about 4-5 m below the highest elevation of the internal debris. It is assumed that the debris remains standing at a certain cross-section, although relatively narrow at the top level of the authentic wall at this point. The most obvious structural problem of this type of decay is that the inner seal is standing at an angle of almost 80-90 °. In time, these slope angles will be reduced and external debris will be lost.

This means that the angles approach the natural façade means that they will not be subject to breaks as a result of the effects of wind or earthquake. The trend of destroying the rubble is to reach this enthalpy. To ensure inaction.

Figure 10. Close view of the walls between the North Wall and the KD11-KD17

It is observed that the Outline of the Northern Front extends from the lower elevation of the bushes and the walls of the interior walls to the lower elevation when it proceeds in the western part towards the slope. In these chapters, it can be said that different period weaves are interventions to close similar losses.

Figure 11. North Wall Exterior, West Slope Segment View

Another part of the North Wall is the arched entrance door. On the exterior of the door, the arch of the door, whose knitted wall was completely lost, was demolished around the central axis. Parts on the double side of the belt stand as console. The belt no longer acts as a belt by applying a load transfer.

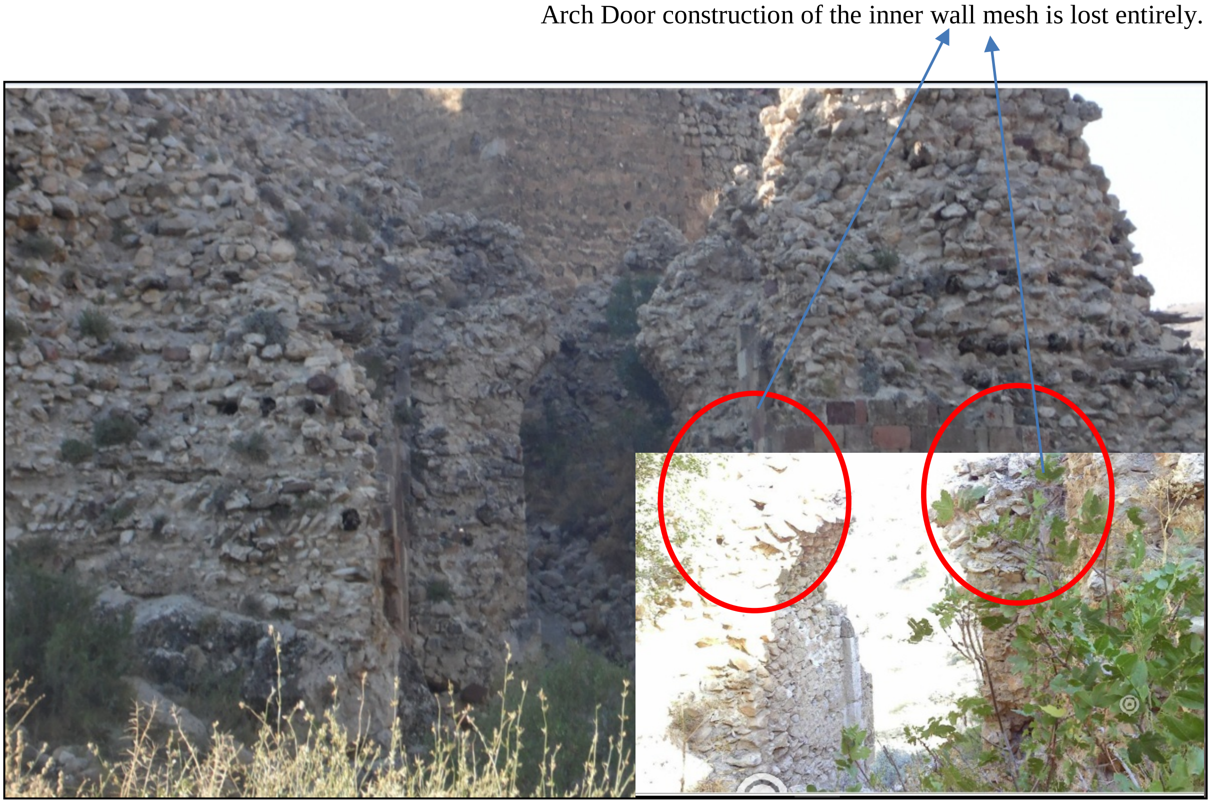


Figure 12. North Wall, Main Entrance Door Exterior Wall Completely Lost and Belt Destructed

In this case, it is inevitable for the Belt Structure to collapse the cantilever parts and even gradually lose the wearer's rubble. Vertical effects or slight seismic movements will suffice.

2.3. Eastern Exterior Facade

It can be said that the deterioration situation is not much different from the Southern Front when observations are made on the East Exterior. There is no problem of wall loss in the lower part of the building bushes and walls. On the lower part of the wall, there were losses of wall joints, while the outer wall was completely destroyed by internal debris. Slope angles are ~ 60 ° for this part. However, on this front there is a rubble residue DD-7. The residue alone remained approximately ~ 4 m with a diameter of about 1.5 m. It can be said that the remains, which were previously a wall wall corner, exhibited a much higher threat than the seizure risk under other seismic effects. On the other hand, there are no problems in dealing with problems such as internal debris disintegration / melting and outer wall losses on other fronts. It can be said that wooden beams, which are still in place on this front, remain healthier than other fronts. This is thought to be less exposed to freeze dissolution cycles and may be the effect of having a solar field.

As a result of the internal debris being exposed to both lateral loading effects and climatic negative effects, it is highly dominant on this front.

Figure 13. DD-7 (Interesting) Debris Residue which can be followed on the Eastern Outskirts

2.4. Western Exterior Facade

West Facade The most serious problems of the castle is examined from the outside. The main reason for this is that the fortress settled on the steep slope, the bushes were placed in different elevations and these bushes were connected to each other by the walls settled in a more amorphous form. Therefore, in this section mass demolitions on the walls. In some places, it is possible to see traces of walls that were destroyed by the out-of-plane deformation. In this section, it is determined that internal debris is exposed by taking serious amounts of outer areas and the inner rubble is melted in these facade bushes. As a matter of fact, both the walls of the walls of the hills and the debris of the inner debris are much more serious.

The most prominent erect internal debris residue as angle samples; It was found that these angles reached ~ 70 ° slopes in the nodes of KD-4 and BD-11-10. It can be said that these angles are in excess of the natural slope.

The outer wall loss and the internal debris melting in the upper part of the walls located at the top level of the fortress observed in other façades.

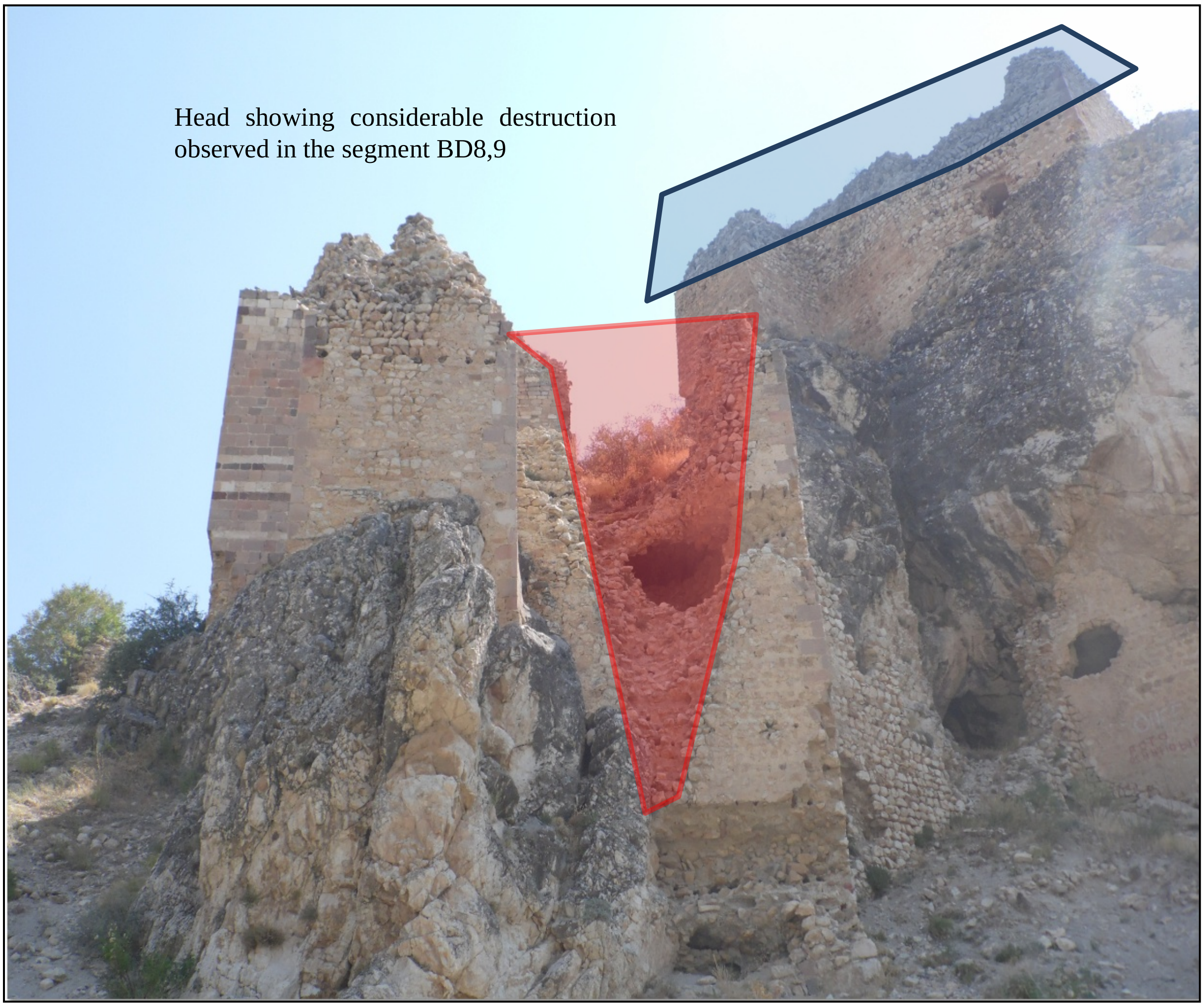


Figure 14. Irregular Demolition in Western Front, BD-8, 9

Figure 15. Non-Plane Demolition of Long Span Wall Segment in the Western Front and BD-6 Wall

Regional extinction on the walls, as mentioned above, is present in this concept. It can be assumed that the wall openings on the walls of BD 8, 9 or BD 6 are relatively out-of-plane due to the fact that the walls are relatively high due to the slope and the walls are weakened by the seismic effects due to the fact that the openings such as the crenelite or the window play a debilitating role. It is also a logical assumption that the material degradation of the wall makes an effective contribution. In the present state, these walls are similar threat because of the vertical walls of the walls or wall extensions which are perpendicular to the walls. If the original forum is caught and the walls are supporting each other, this will be an approach to consolidate the problem.

2.5. In-Castle Stepped Wall

When entered from the north side of the building, it can be said that the inner wall of the North-South axis paralleled a serious elevation difference. It can be said that the inner fill of the wall supporting a elevation of 10 m in the original is exposed to serious losses. It can be said that the wall with a thickness of about 2 m has a thickness of ~ 1.5 m at the uppermost residual elevations, but the uneven plan line is implicated in molasses which cannot be detected due to debris. It would be an approach to consolidate the backfill pressures on the wall so that this wall could not show the original retaining property and would support the front rubble heap so that it would not have the same effects.

Photo 7. The View of the Castle Interior Wall from the West. Only internal debris and partially wooden timber parts of the wall can be detected on site

3. STRUCTURAL ANALYSIS STUDIES WITH FINITE ELEMENT ANALYTICAL MODELS

It is thought that the most significant and descriptive structural evaluation of the castle, which has the same construction technique, can be done in the digital environment with linear elastic finite element models. It is thought that the mass movement of the castle towers, which have almost the same wall thickness and openness ratio in the building plan layout, will remain in the elastic margin. It can be said that the components that reach the elastic limit instead of plastic behavior show themselves with cracks and collapse problems. Therefore, instead of performing complicated finite element calculations with detailed non-linear plastic behavioral mathematical approaches, modeling studies were preferred which would yield results in approximately the same level of approach with non-linear modeling techniques.

Analytical modeling studies; The problems related to the overall structure on the sample building sections were carried out on the building elements determined in the preliminary examination of the building. These building elements; The most obvious structural questions are clearly structured and the structural problems are more likely to worsen in the future. Based on this, the structural intervention criteria should be determined by trying to solve the problems of these components; draw the frame of intervention for the fortress.

The castle analytical model was created using SAP2000 software. As the inner rubble thickness varies across the tower and the wall inertia is resistant to the lateral effects, the simplification of the shell element for the walls was found to be more suitable for the walls of the wall. It is foreseen that the contribution of the structural form to be formed as a result of the evaluation of the existing structure on the 2D plane and the possible repair approach will give more clear results specific to this structure with a higher-generalized view.

As the Hurman Fortress was subjected to severe local demolitions, models of model parts were used to examine the present state of the structure. In rugged building models defined today; respectively, in the Western Facade D8,9, 436 points, 265 solid elements; North Facade KD-7, 152 point 67 solid elements were used. In the model, for the lower supports elevations, the rigid surface grape on which the structure sits was entered as the existing surface. The existing elevation levels for the upper elevations of the wall and the intervention levels were entered for the completed models.

The ground relation of the bottom row walls of the structure is also defined by hinge elements. The relationship with the side walls (vertical walls) is defined for a square meter area with a spring constant of about 500 kN * m. On the other hand, the common wall values of different building materials with different physico-mechanical parametric-values, which represent the heterogeneity of the walls of the wall, have been removed and defined in the program as a result of hand calculation or literature search. Therefore, by creating a complete macro-analytical model in the program, more simplified / generalized results of the effect of the structural degradation regions by extracting the wall heterogeneity effect from the results are focused on these degradation regions.

The loading types on the models are limited by their own weight and the lateral effects of the earthquake. As previously mentioned, although the building is located on the slope and at higher elevations, the wind effects are not considered to have serious destruction effects except for the weaker parts of the building (except the walls that become too delicate as a result of the collapse). On the other hand, the seismic impacts which are in proportion to the mass of the building can have serious destructive levels for the masonry structure. Therefore, the results of modeling will clearly show the share of earthquakes on destructiveness.

In the case of vertical loads, no load on the building itself was taken out of its own weight (no intensive human burden exists on the existing structure). Even if the structure is already in use, the weight of its own dead weight will be much higher than the live load.

Natural vibration period calculations which are necessary for earthquake analysis and result by eigenvalue calculations - MODAL analyzes were carried out as close to 90% of the building mass participation. 1. With 100 modes in the model, 97% is exceeded for all three directions. 2. In the model, over 100 modes have been exceeded to 95% for all three directions. (It is not necessary to have mode control in completed building models.)

In the modal analysis results, the horizontal primary natural vibration periods for the North NE-7 modern structure model are 0.1941, 0.1862 sec in two directions, respectively. (These values are the values of the movement of the building walls on both sides of the belt in different directions.) (Where the first direction is x-east-west direction; the second direction is given as y-north-south direction). The West D8, in the present-day structure model, is 0.5148 s in the x-direction (x is the north-south direction) and 0.26486 s in the y direction (where y is the west-east axis).

Material properties given for the rubble wall used in the models z As a result of literature comparison study results and similar building materials parametric values;

- Natural Specific Weight; 2.56 ton/m3
- Compressive Strength; 1-2 MPa
- Elastic Modul Value = ~2000*σ = 5 GPa
- Poisson Ratio; 0.25

is assumed. In order to keep these values on the safe side; In the literature, the smallest of the values and the lowest values for the same kind of stones and mortars were used as input for these building materials.

The approach used in the earthquake analysis Turkish Standards - The Regulation on Buildings to be Constructed in Earthquake Regions was considered according to the Mod Combination method given in 2007. Kahramanmaraş - Andırın located in Turkey earthquake map 3 corresponds to the earthquake region. In this case DBYBHY 2007 requirement;

In the Mod Combination method and Response Spectrum effect analysis;

- Building Importance Factor: I=1.4
- Structure Behavior Factor: R=2.0
- Effective Ground Acceleration Coefficient; A0 = 0.2 were taken.

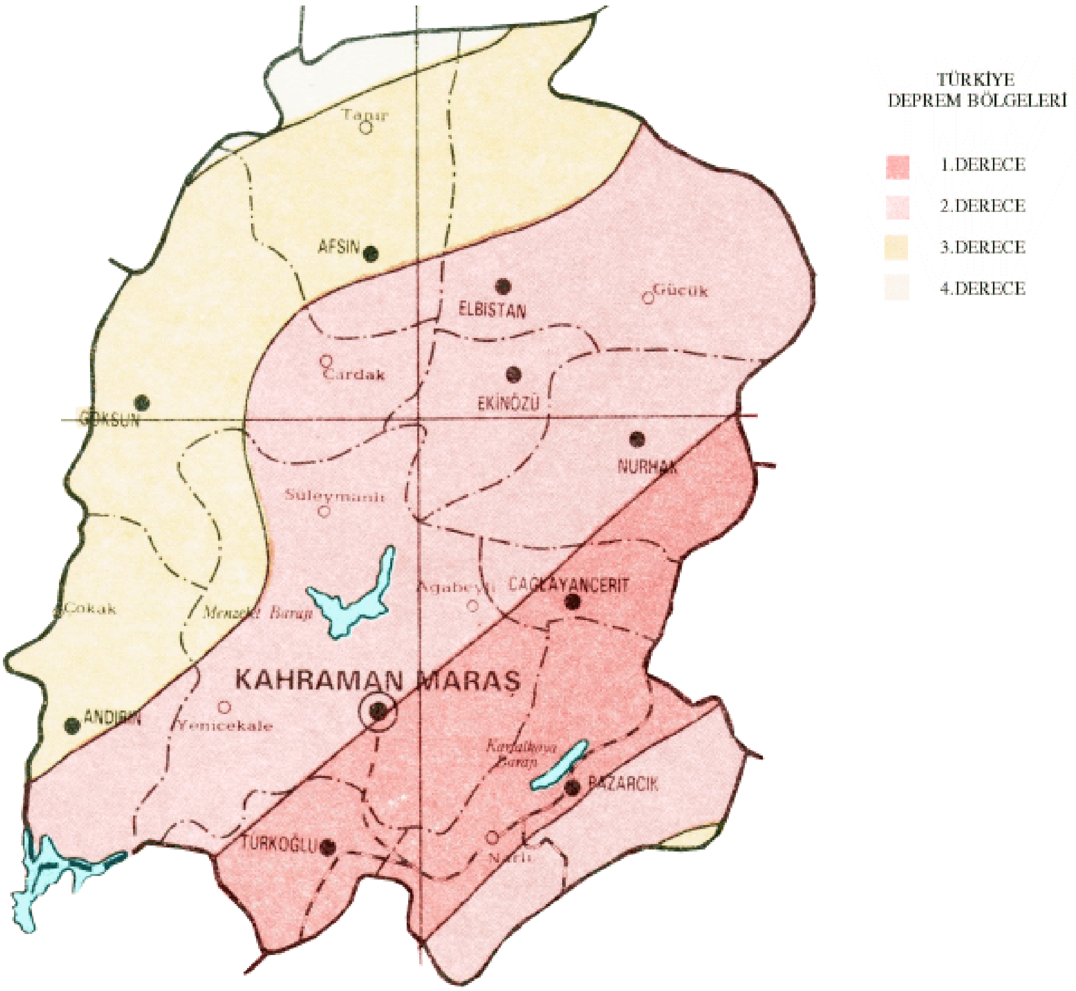


Figure 16. Turkey Earthquake Map - Kahramanmaras Seismicity

Load classes and combinations in structural analysis;

DL; Dead (Zati) Load

Ex; Earthquake Effect in X Direction

Ey; Earthquake Effect in Y Direction

Combinations: DL+Ex

DL+Ey

Example of Section Castle, Finite Element Analysis Assessment Results

The evaluation of the earthquake effects in two models for the fortress S11 is made on the horizontal and S22 vertical stress distribution maps. - negative values indicate the stresses + positive values tensile stresses.

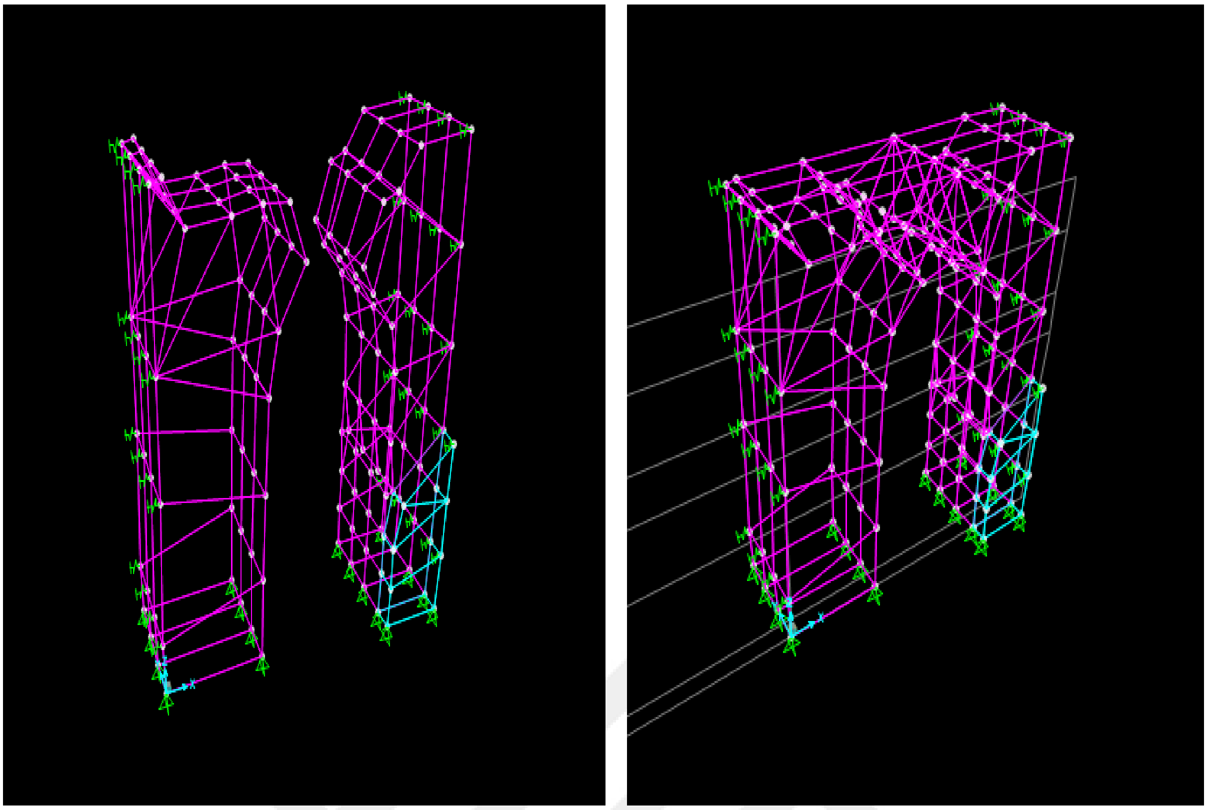


Figure 17. North Front Entrance door segment a) modern geometry model b) part completion proposal model

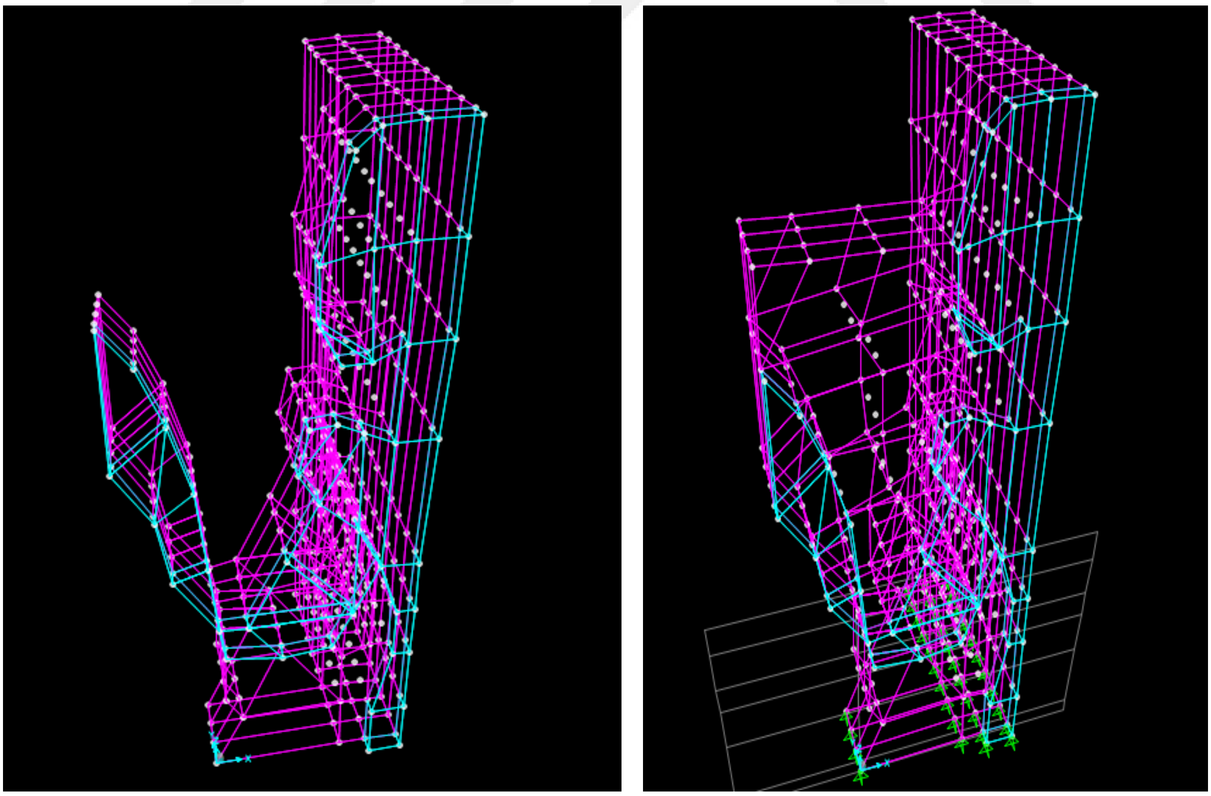


Figure 18. Western Front D8,9 Sign front and side facade a) modern geometry model b) part completion proposal model

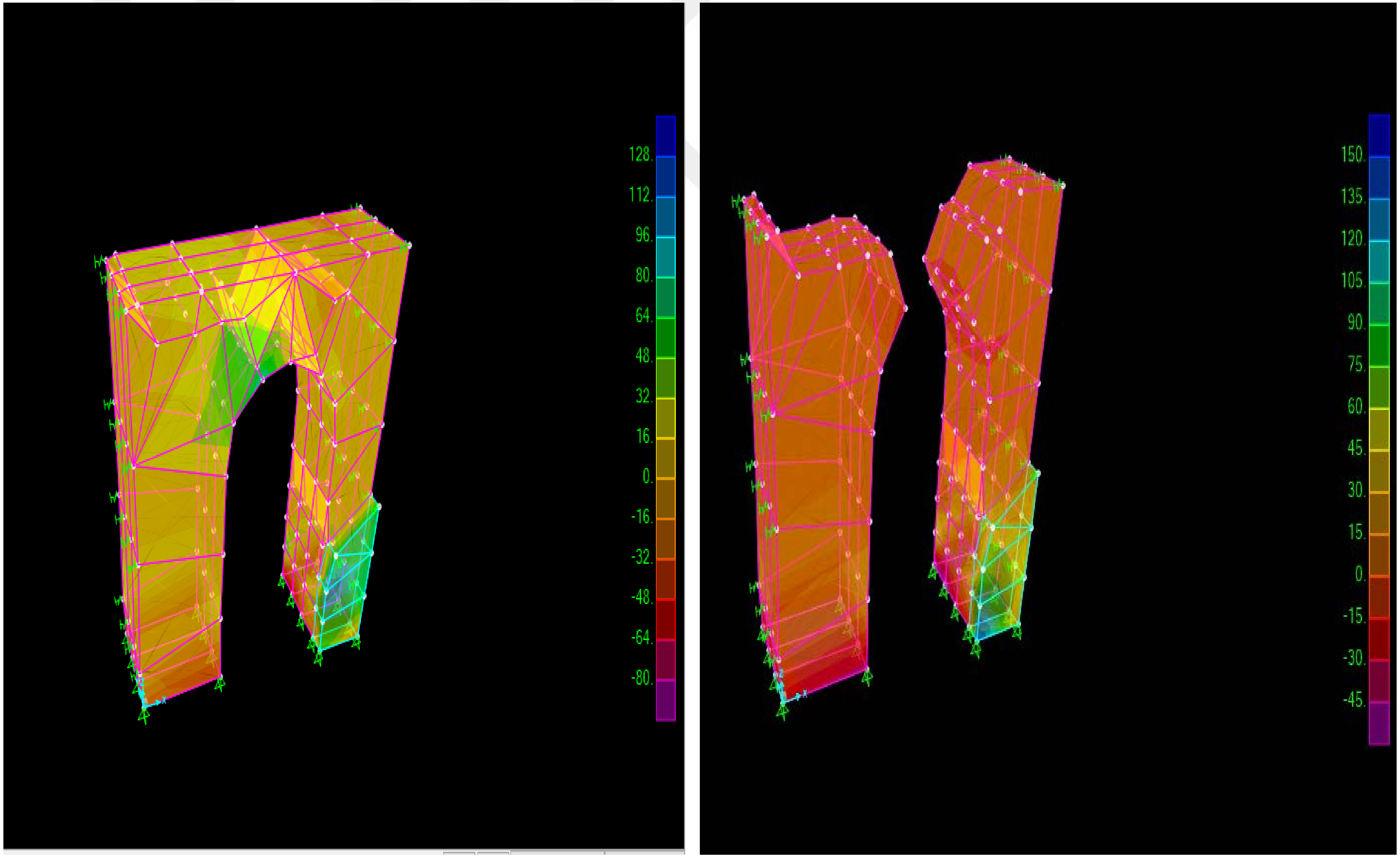
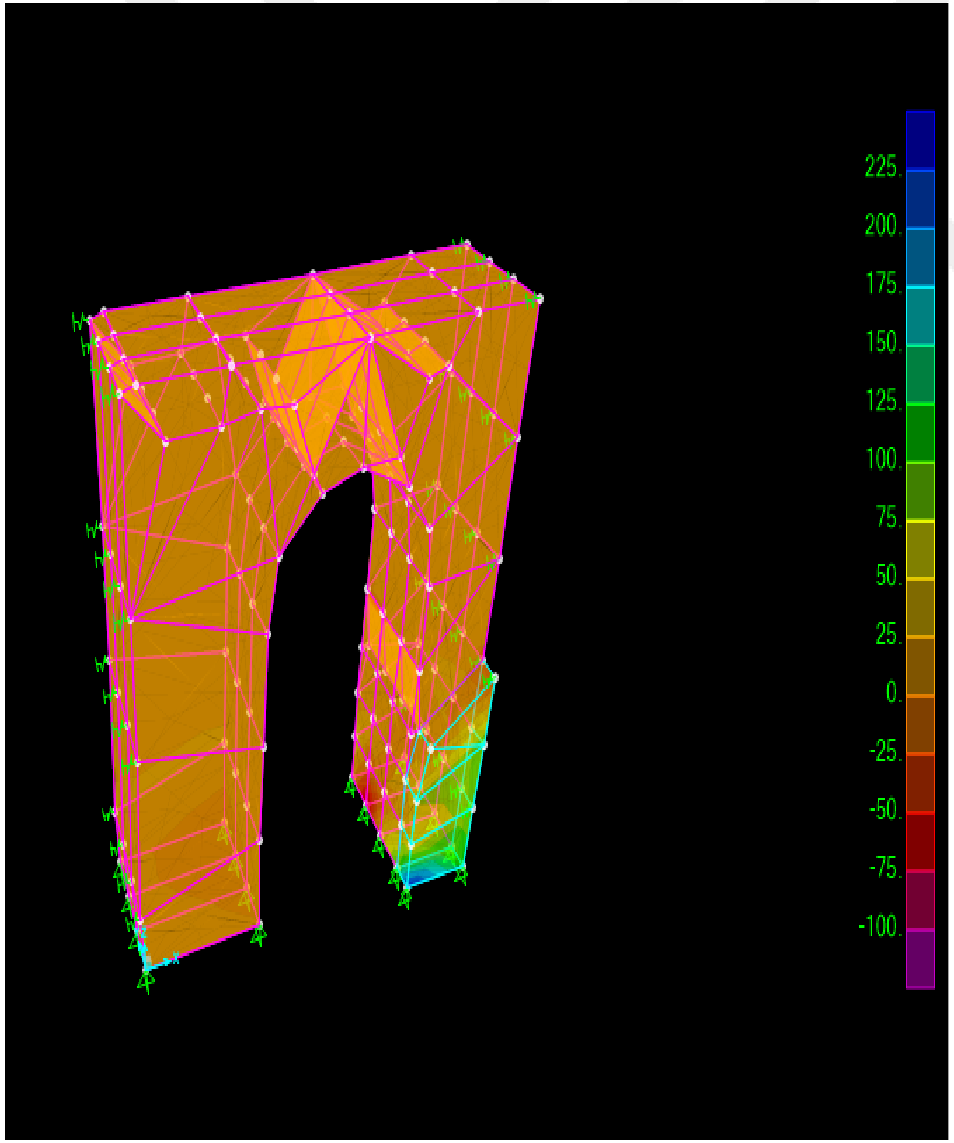


Figure 19. Belt Structure G + Ex stresses a) Consolidated structure b) Present Structure - S11 Stress (kPa)



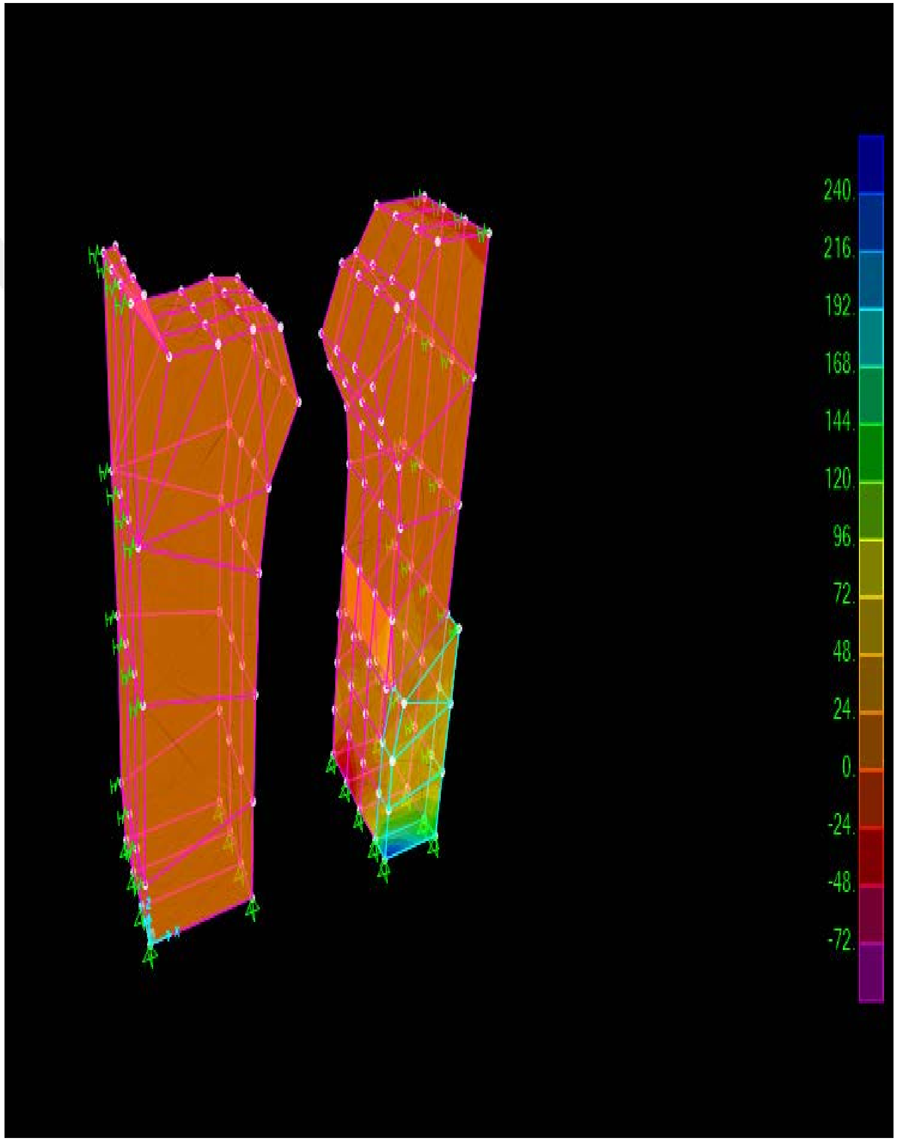


Figure 20. Belt Structure G + Ex stresses a) Consolidated structure b) Present Structure - S22 Stress (kPa)

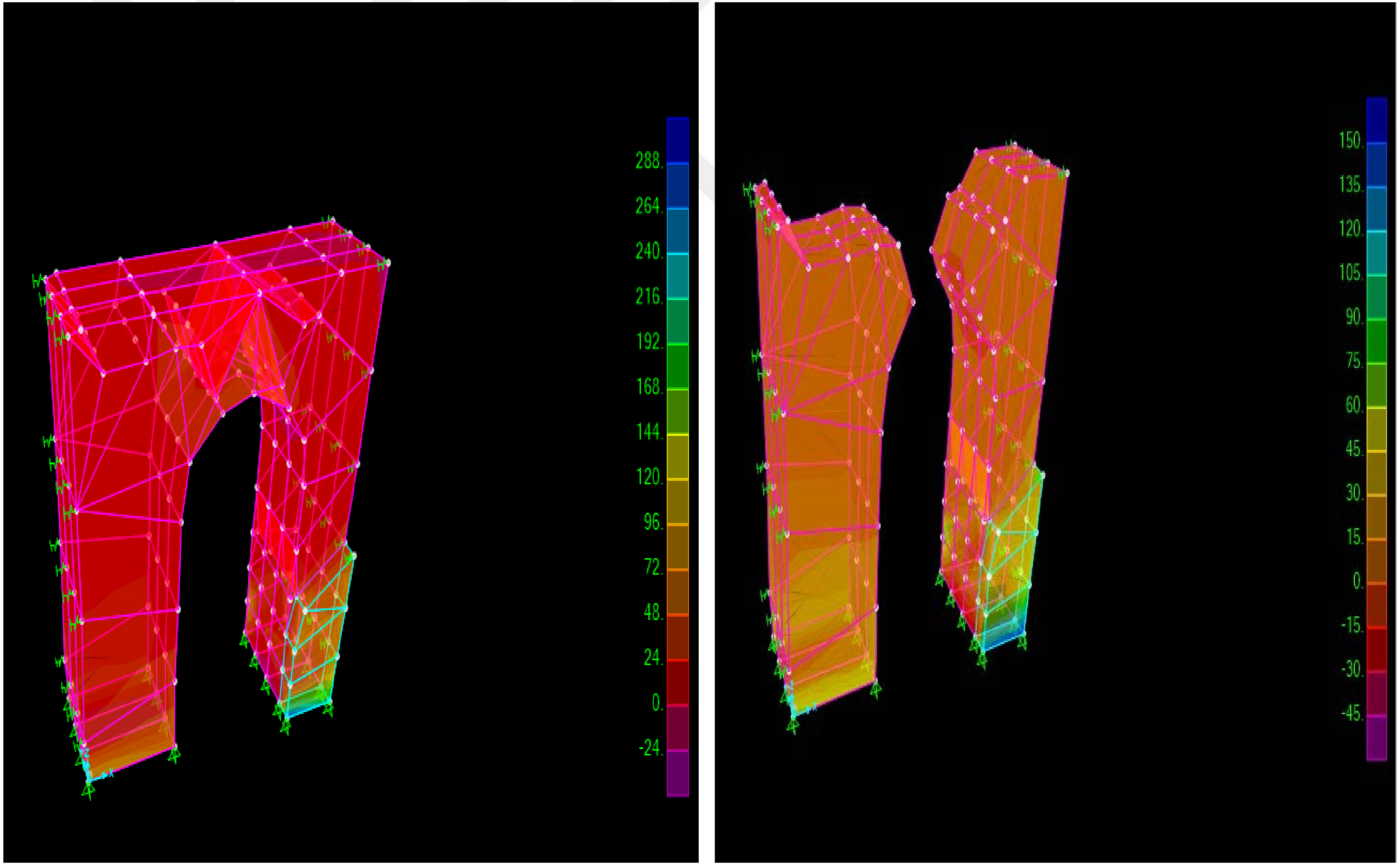


Figure 21. Belt Structure G + O stretching a) Consolidated structure b) Present Structure - S11 Stress (kPa)

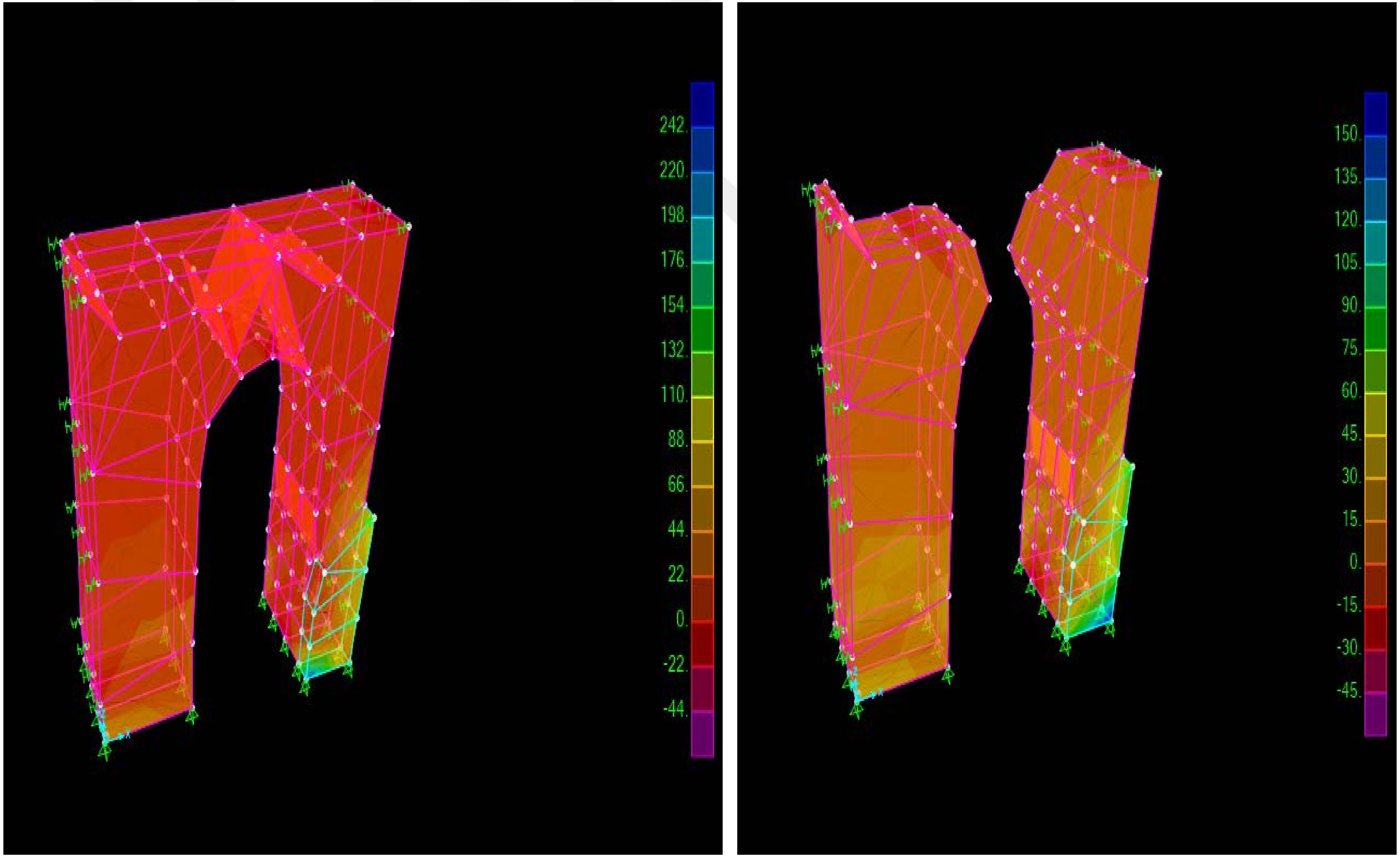


Figure 22. Belt Structure G + Ey stresses a) Consolidated structure b) Present Structure - S22 Stress (kPa)

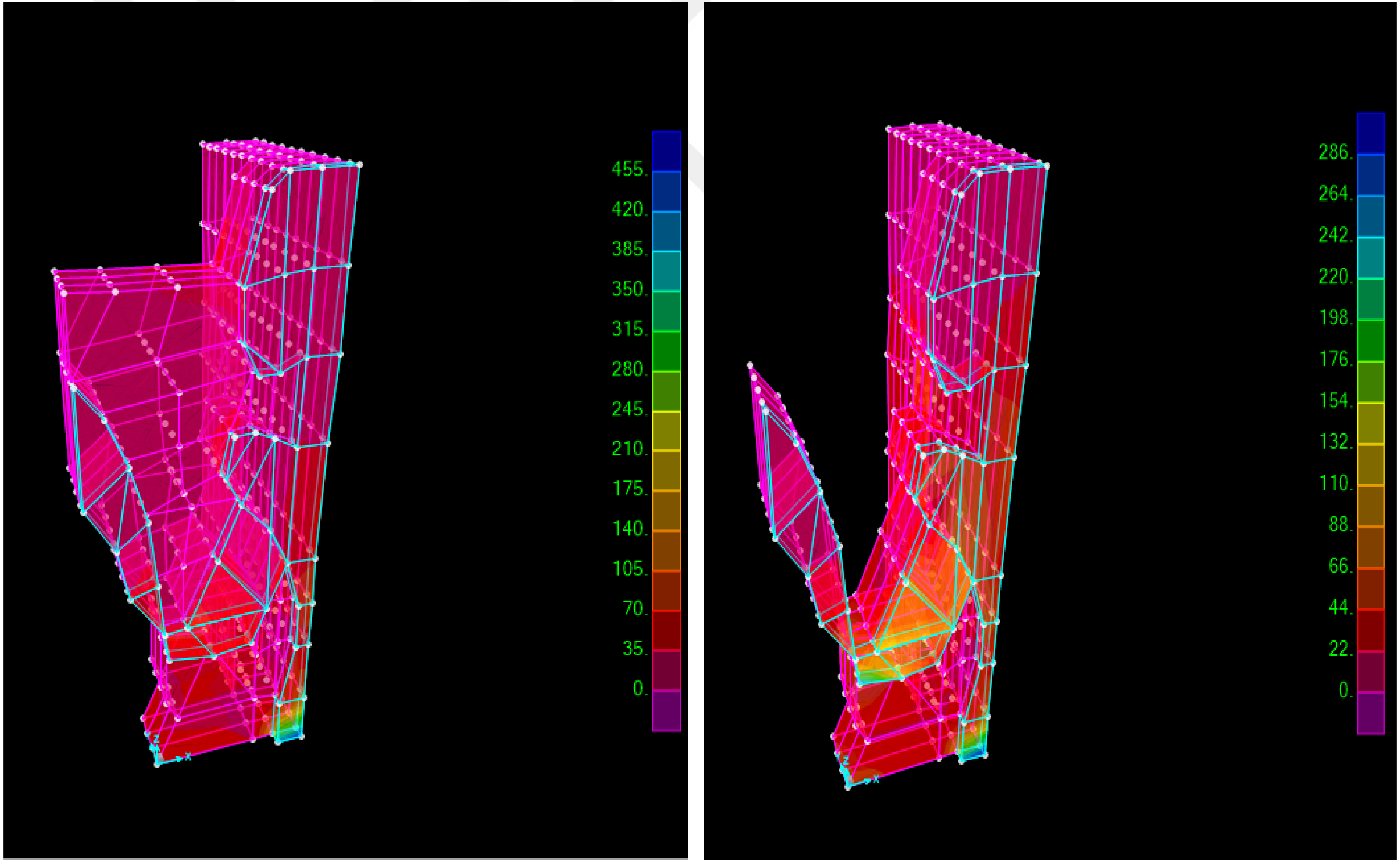


Figure 23. Tower Structure G + Ex Stresses a) Consolidated Structure b) Present Structure - S11 Stress (kPa)

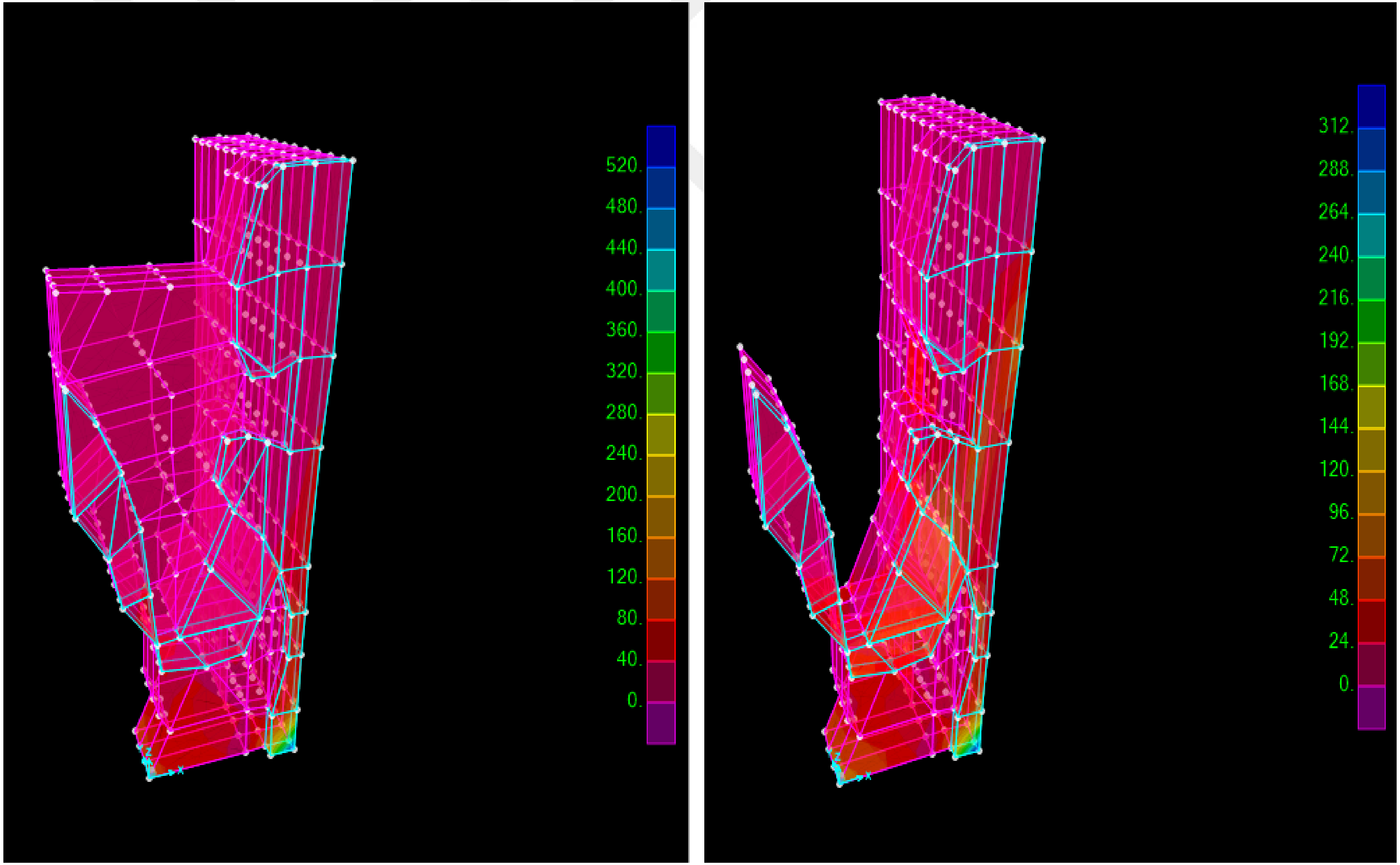


Figure 24. Figure 24. Tower Structure G + Ex Stresses a) Consolidated Structure b) Present Structure - S22 Stress (kPa)

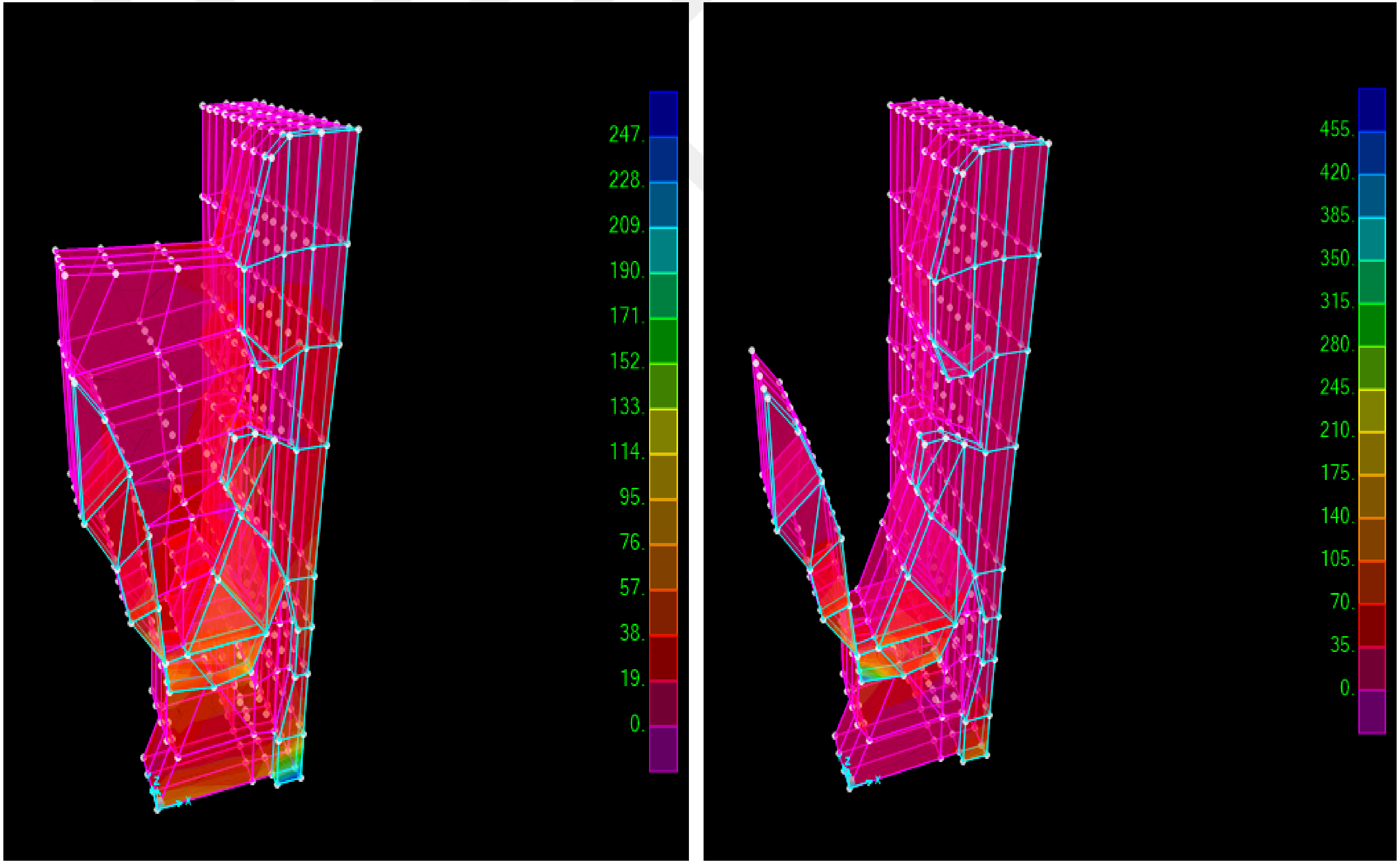


Figure 25. Tower Structure G + Ey stresses a) Consolidated structure b) Present Structure - S11 Stress (kPa)

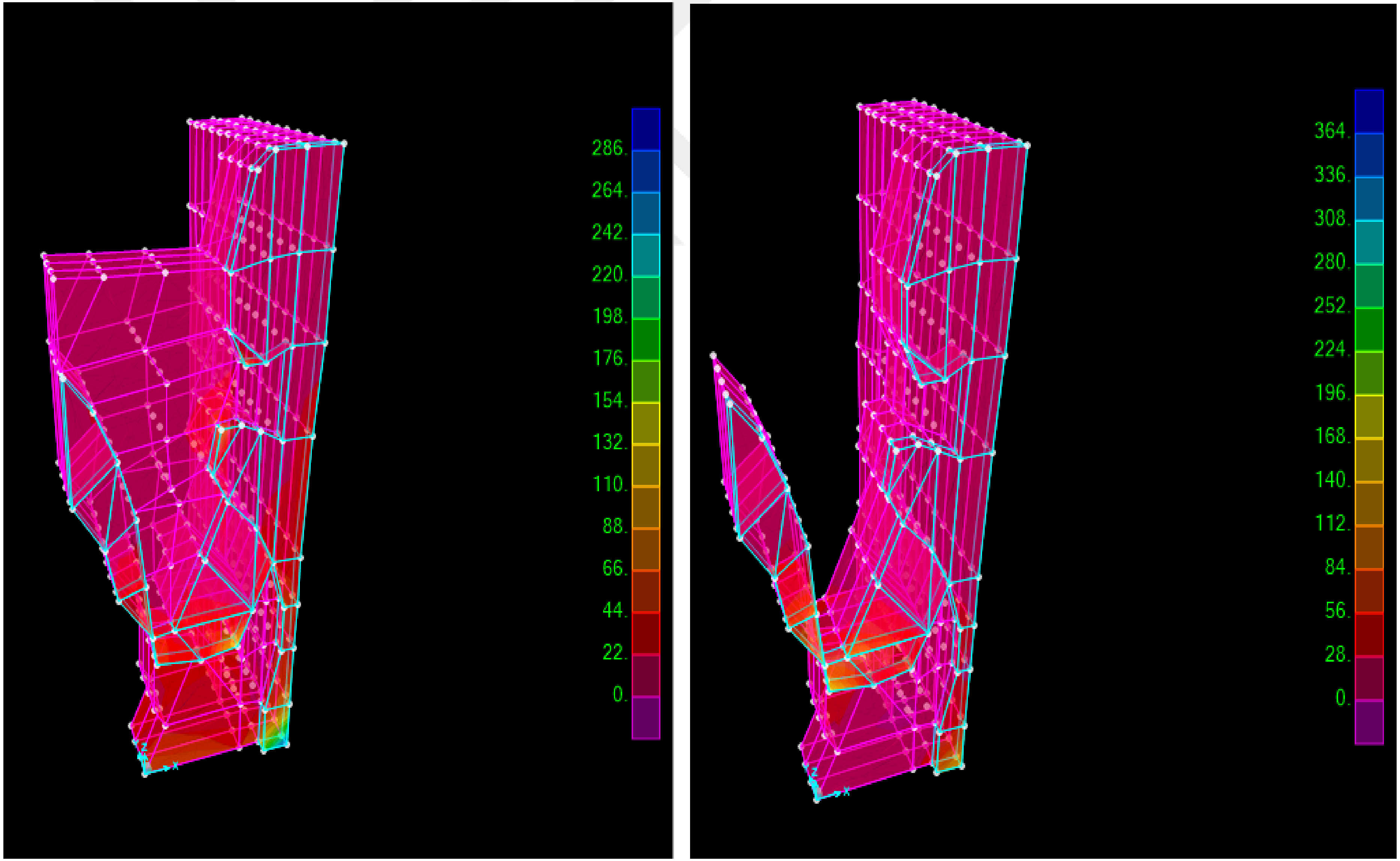


Figure 26. Tower Structure G + Ey stresses a) Consolidated structure b) Present Structure - S22 Stress (kPa)

In the light of the S11 and S22 stresses of the modeling results, the comparison of the two modeling results with DL + EQ combinations provides preliminary information on the need for intervention.

In the present geometry on the structures, the structural masses are also under stress. As a result of the laboratory analyzes carried out within the scope of this project, 1 MPa is not seen as stress on the walls even in earthquake effects. However, as a result of the relatively severe collapse of the tower structure, the staging has been observed in the model results as the accumulation of stress in the joints of the cantilever parts. Similar build-up arch structure is also present on the stirrup line in today's form. Under normal circumstances, because the structure will exhibit heterogeneous behavior (existing rubble filling structures), which are more prone to local problems than the model-homogeneous behavior, it can be said that the stresses accumulated in the models in these parts will contain problems for the real situation. As a matter of fact, the relatively small tensile stresses accumulated in these parts disappear.

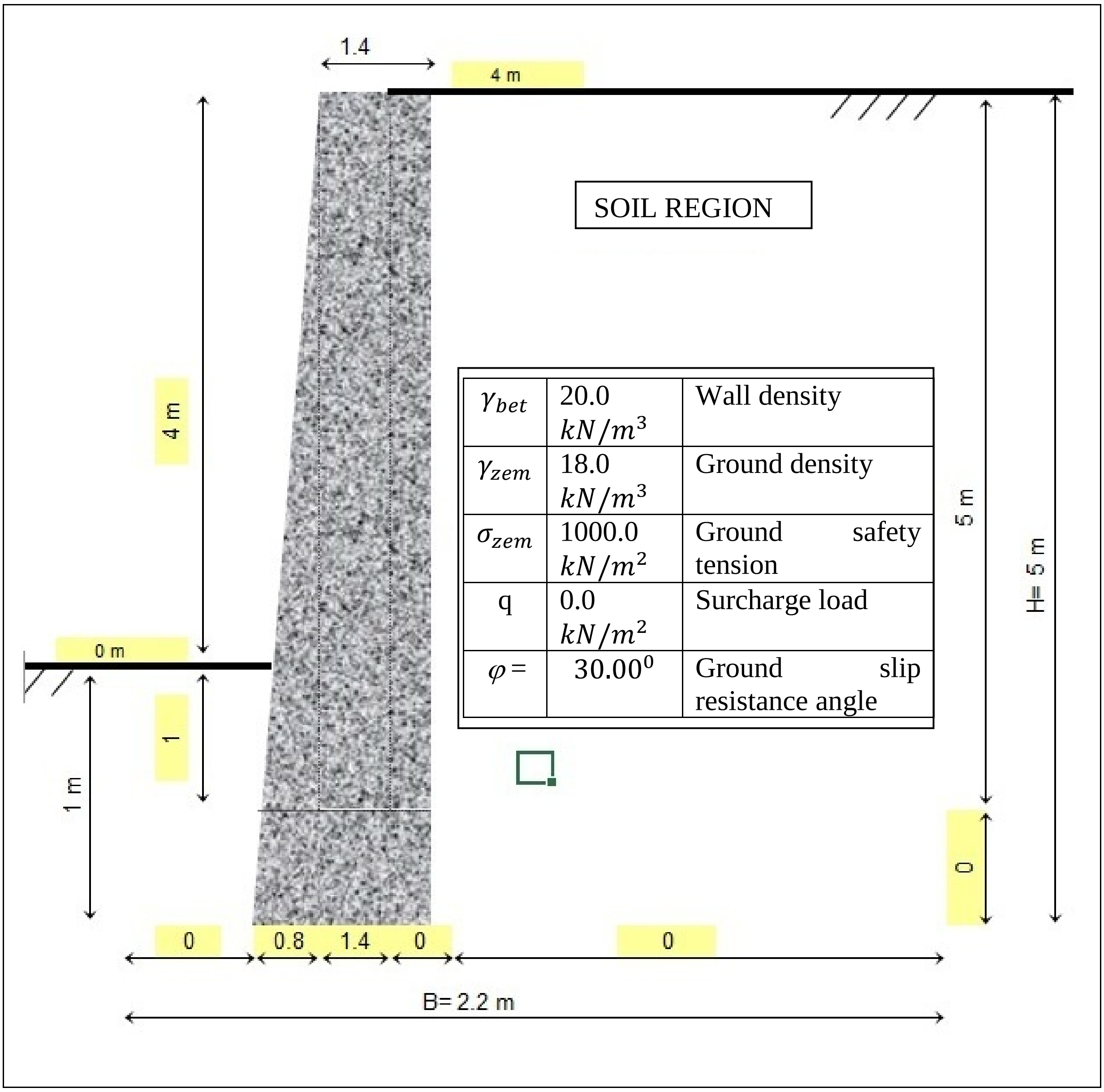
Briefly, in both horizontal and vertical stresses, the structure is consolidated by completing the parts with homogeneous distribution and healthy distribution. Therefore, analytical modeling studies give conclusions to suggest consolidation with partial completion in order to prevent further destruction of building parts throughout the castle.

4. PROJECTS ADDED IN THE IMPLEMENTATION STAGE

4.1. Retaining Accounts - For Stage Wall

The upper wall thickness of the inner wall is now 80 cm. Injection strengthened wall; The height of the wall prefill is graded ~ 4m (the difference between the backfill elevation and the pre-stage recommendation elevation). Considering that the wall is 2 m thick on the edges, the wall thickness to be used for the calculation is average when thickened by intervention; ~ 1.4 m.

Therefore, the results of the calculation of the stability of the retaining wall in accordance with the calculation;



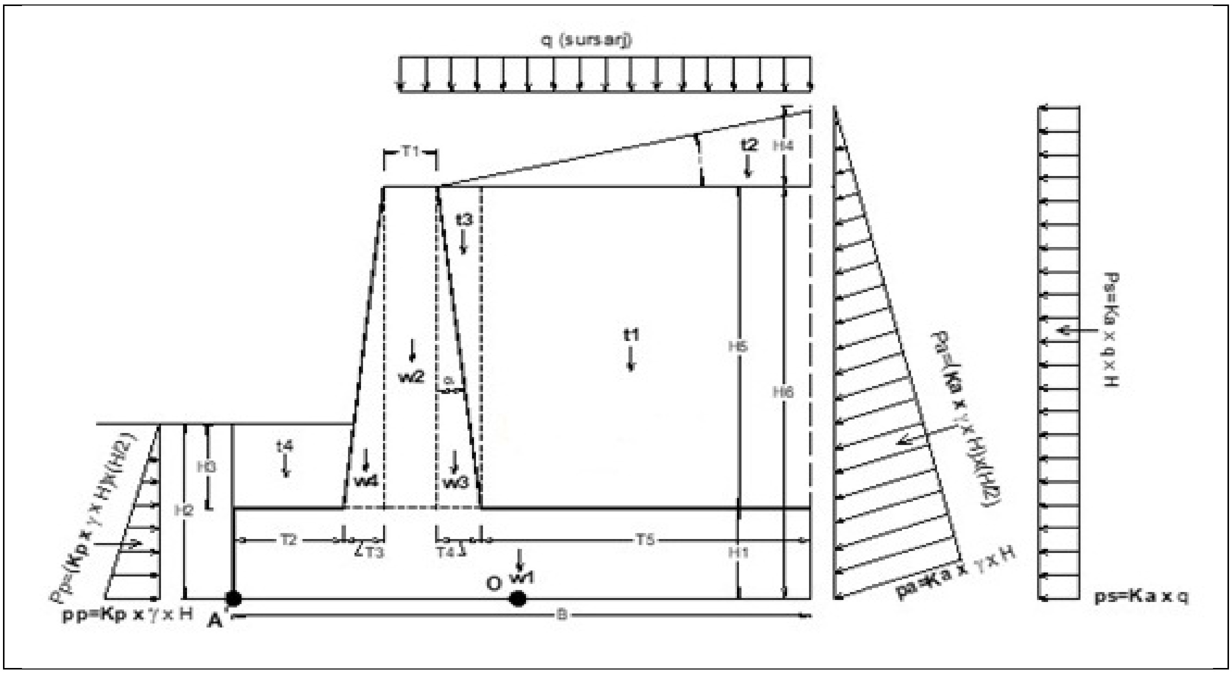
γ_{bet}	20.0 kN/m^3	Wall density
γ_{zem}	18.0 kN/m^3	Ground density
σ_{zem}	1000.0 kN/m^2	Ground safety tension
q	0.0 kN/m^2	Surcharge load
$\varphi =$	30.00^0	Ground slip resistance angle

Note: The unit of length is the meter and the shape is not scaled.

H1=	0.00 m	T1=	1.40 m
H2=	2.00 m	T2=	0.00 m
H3=	2.00 m	T3=	0.30 m
H4=	0.05 m	T4=	0.30 m
H5=	6.00 m	T5=	0.00 m
H6=	6.00 m	B=	2.00 m

i =	10.00^0	Ground bevel angle
φ =	30.00^0	Ground slip resistance angle
δ =	20.00^0	Ground wall friction angle
α =	2.86^0	Wall inclination
μ=	0.58	Coefficient of friction (μ=tan φ)

γ_{bet}	20.0 kN/m^3	Wall density
γ_{zem}	18.0 kN/m^3	Ground density
σ_{zem}	1000.0 kN/m^2	Ground safety tension
q	0.0 kN/m^2	Surcharge load
N_0	2.0	Coefficient of tipping safety
N_s	1.5	Slip safety coefficient



Under static effects			Under static effects	
Rollover Safety	√	2.7 > 2.0	Rollover Safety	√
Slip Safety	√	2.8 > 1.5	Slip Safety	√
Maximum ground stress	√	189 < 800	Maximum ground stress	√
Minimum ground stress	!!!	-25 < 0	Minimum ground stress	!!!
Earthquake case			**Earthquake case**	
Rollover Safety	√	1.5 > 1.3	Rollover Safety	√
Slip Safety	√	1.8 > 1.1	Slip Safety	√
Maximum ground stress	√	305 < 1200	Maximum ground stress	√

In this case, it can be said that the negative pressure area is not as problematic since the negative pressure value and positive pressure value are interpolated and the negative pressure area is less than 1 / 6th of the section where the negative pressure that the section can take under the risk of overturning.

4.1.1. Horizontal and Vertical Effects of Retaining Wal

i= 10.00^0 $\varphi = 30.00^0$ $\delta = 20.00^0$ $\alpha = 0.00^0$

$$K_a = \frac{\cos^2(\varphi-\alpha)}{\cos^2\alpha \times \cos(\delta+\alpha) \times \left[1+\sqrt{\frac{\sin(\varphi+\delta)\times\sin(\varphi-i)}{\cos(\delta+\alpha)\times\cos(i-\alpha)}}\right]^2} = \frac{0.75}{1\times 0.9\times 2.3} = 0.34$$

$$K_p = \frac{\cos^2(\varphi+\alpha)}{\cos^2\alpha \times \cos(\delta-\alpha) \times \left[1-\sqrt{\frac{\sin(\varphi+\delta)\times\sin(\varphi+i)}{\cos(\delta-\alpha)\times\cos(i-\alpha)}}\right]^2} = \frac{0.75}{1\times 0.9\times 0.1} = 10.90$$

1) Vertical Effects

Soil weights			Distance to point A			Distance to point O	
t1=	0.0 kN		At1=	2.20 m		Ot1=	1.10 m
t2=	0.0 kN		At2=	2.20 m		Ot2=	1.10 m
t3=	0.0 kN		At3=	2.20 m		Ot3=	1.10 m
t4=	0.0 kN		At4=	0.00 m		Ot4=	-1.10 m
Concrete weight			Distance to point A			Distance to point O	
w1=	0.0 kN		Aw1=	1.10 m		Ow1=	0.00 m
w2=	140.0 kN		Aw2=	1.50 m		Ow2=	0.40 m
w3=	0.0 kN		Aw3=	2.20 m		Ow3=	1.10 m
w4=	40.0 kN		Aw4=	0.53 m		Ow4=	-0.57 m
Surcharge load			Distance to point A			Distance to point O	
q=	0.0 kN		Aq=	2.20 m		Oq=	1.10 m

Soil moment according to point A			Wall moment according to point A			Soil moment according to point O			Wall moment according to point O	
Mt1=	0.0 kN.m		Mw1=	0.0 kN.m		Mt1=	0.0 kN.m		Mw1=	0.0 kN.m
Mt2=	0.0 kN.m		Mw2=	210.0 kN.m		Mt2=	0.0 kN.m		Mw2=	56.0 kN.m
Mt3=	0.0 kN.m		Mw3=	0.0 kN.m		Mt3=	0.0 kN.m		Mw3=	0.0 kN.m
Mt4=	0.0 kN.m		Mw4=	21.3 kN.m		Mt4=	0.0 kN.m		Mw4=	-22.7 kN.m
Mq=	0.0 kN.m					Mq=	0.0 kN.m			

2) Horizontal Effects

Active soil propulsion; P_a = 0.5x K_a x γ_{zem}x H^2 = 76.5 kN yatay P_a = 71.9 kN

Passive soil propulsion; P_p = 0.5x K_p x γ_{zem}x H^2 = 98.1 kN düşey P_a = 26.2 kN

Surcharge propulsion; P_s = K_p x q x H = 0.0 kN

	Distance to point A			Moment according to point A	
Active soil propulsion ;	A(P_a) =	1.63 m		M(P_a) =	119.8 kN.m
Passive soil propulsion;	A(P_p) =	0.33 m		M(P_p) =	32.7 kN.m
Surcharge propulsion;	A(P_s) =	2.50 m		M(P_s) =	0.0 kN.m

4.1.2. Verifications

1) Overturn Verification

Since the condition is inconvenient, the moment from the load is not included in the moment.

Protective Moment ; Mk = Mt1+Mt2+Mt3+Mt4+Mw1+Mw2+Mw3+Mw4 +(düşeyP_a x (B-(T5)/2)+M(P_p) =321.6 kN.m

Overturning Moment ; Md = M(P_a)+ M(P_s) =119.8 kN.m

Rollover safety; N_0 = Mk/Md = 321.6/119.8 = 2.7 > 2 √

2) Slip Verification

Since the load is inconvenient, the charge load is not added to vertical loads.

Resist the slip ; F_{kk} = (N x µ) + P_p= 202.1 kN N= 180 (Toplam Düşey Yük)

Slip ; F_{kay} = $P_{a.yatay}$+ P_s= 71.9 kN µ= 0.5774 (Sürtünme katsayısı)

Slip Security ; N_s = F_{kk}/F_{kay} = 202.1/ 71.9 = 2.8 > 1.5 √

3) Ground Stress Verification

Ground safety tension ; 800 kN/m^2

Momentum in base center ; M_O = 86.5 kN.m

Total vertical load ; N = 180.0 kN (Surcharge included)

Floor area ; A = 2.2 m^2 N/A = 81.8 kN/m^2

Base Strength moment ; W = <u>0.807 m^3</u> M_O/W = <u>107.2 kN/m^2</u>

Maximum ground stress ; $\sigma_{z,maks} = \sigma_z = \frac{N}{A} - \frac{M_o}{W}$ = <u>189.0 kN/m^2 < 800</u> kN/m^2 √

Minimum ground stress ; $\sigma_{z,min} = \sigma_z = \frac{N}{A} - \frac{M_o}{W}$ = <u>-25.4 kN/m^2 < 0</u> !!!

4.1.3. Additional Effects in Earthquake Case

i= 10.00^0 A_O= 0.20 λ = arctan[C_H /(1±C_v)] = 7.43^0

φ = 30.00^0 I = 1.00 Coefficient of tipping safety N_O= 1.3

δ = 20.00^0 $C_H = k_{CH}$ x (I+1) x A_O =0.12 Slip safety coefficient N_s= 1.1

α = 0.00^0 C_v = 2 x C_H / 3 = 0.08 k_{CH}= 0.3

$$K_{at} = \frac{(1 \pm C_v) \times \cos^2(\varphi - \lambda - \alpha)}{\cos\lambda \times \cos^2\alpha \times \cos(\delta + \alpha + \lambda) \times \left[1 + \sqrt{\frac{\sin(\varphi + \delta) \times \sin(\varphi - i - \lambda)}{\cos(\delta + \alpha + \lambda) \times \cos(i - \alpha)}}\right]^2} = 0.51$$

$$K_{pt} = \frac{(1 \pm C_v) \times \cos^2(\varphi - \lambda + \alpha)}{\cos\lambda \times \cos^2\alpha \times \cos(\delta - \alpha + \lambda) \times \left[1 - \sqrt{\frac{\sin(\varphi + \delta) \times \sin(\varphi + i - \lambda)}{\cos(\delta - \alpha + \lambda) \times \cos(i - \alpha)}}\right]^2} = 10.67$$

$K_{ad} = K_{at} - K_a$

K_{ad} = 0.5069 - 0.34 = 0.17 Dynamic active pressure coefficient

$K_{pd} = K_{pt} - K_p$

K_{pd} = 10.671 - 10.90 = -0.23 Dynamic passive pressure coefficient

Additional horizontal forces due to earthquakes.

P_{ad} = 0.5 x K_{ad} x γ x H^2 = 37.6 kN Active dynamic pressure compound

P_{pd} = 0.5 x K_{pd} x γ x H^2 = -2.1 kN Passive dynamic pressure composition

P_{ad} = K_{ad} x q x H = 0.0 kN Dynamic pressure resultant from the surcharge

Additional impulse moments due to earthquakes.

M_{ad} = 37.6 x 2.5 = 93.9 kN.m Active dynamic pressure

M_{sd} = 0.0 x 2.5 = 0.0 kN.m Dynamic pressure from the surcharge

4.1.4. Verification in case of earthquake

1) Overturn Verification

Since the load is inconvenient, the charge load is not added to vertical loads.

Protective Moment ; Mk = 321.6 kN.m

Overturning Moment ; Md = 119.8 + 93.9 = 213.7 kN.m

Rollover safety ; N_0 = 321.6 / 213.7 = 1.5 > 1.3 √

2) Slip Verification

Since the load is inconvenient, the charge load is not added to vertical loads.

Resist the slip ; F_{kk} = 202.1 kN

Slip ; F_{kay} = 71.9 + 37.6 = 109.4 kN

Slip Security; $N_s = F_{kk}/F_{kay}$ = 202.1/ 109.4 = 1.8 > 1.1 √

3) Ground Stress Verification

Momentum in base center ; M_O = 86.5 + 93.9 = 180.4 kN.m

Total vertical load ; N = 180.0 kN

Floor area; A = 2.2 m^2 N/A = 81.8 kN/m^2

Base Strength moment; W = 0.807 m^3 M_O/W = 223.6 kN/m^2

Maximum ground stress ; $\sigma_{z,maks} = \sigma_z = \frac{N}{A} - \frac{M_o}{W}$ = 305.4 kN/m^2 < 800 kN/m^2 √

Minimum ground stress ; $\sigma_{z,min} = \sigma_z = \frac{N}{A} - \frac{M_o}{W}$ = -141.8 kN/m^2 < 0

4.2. Injection Depth and Grinding Controls

On the western facade KD-4 Wall, a sample slope slip analysis was performed on the narrow and high debris. Bentley GEO5 slope stability module was used for analysis. Width of the rubble (for half a diameter) 2.5 m; The height of the rubble peak is ~ 5.5 m. In the case of a 30 cm deep injection of the superficial rubble, the safety coefficient was found to be 6.5 for a critical slope.

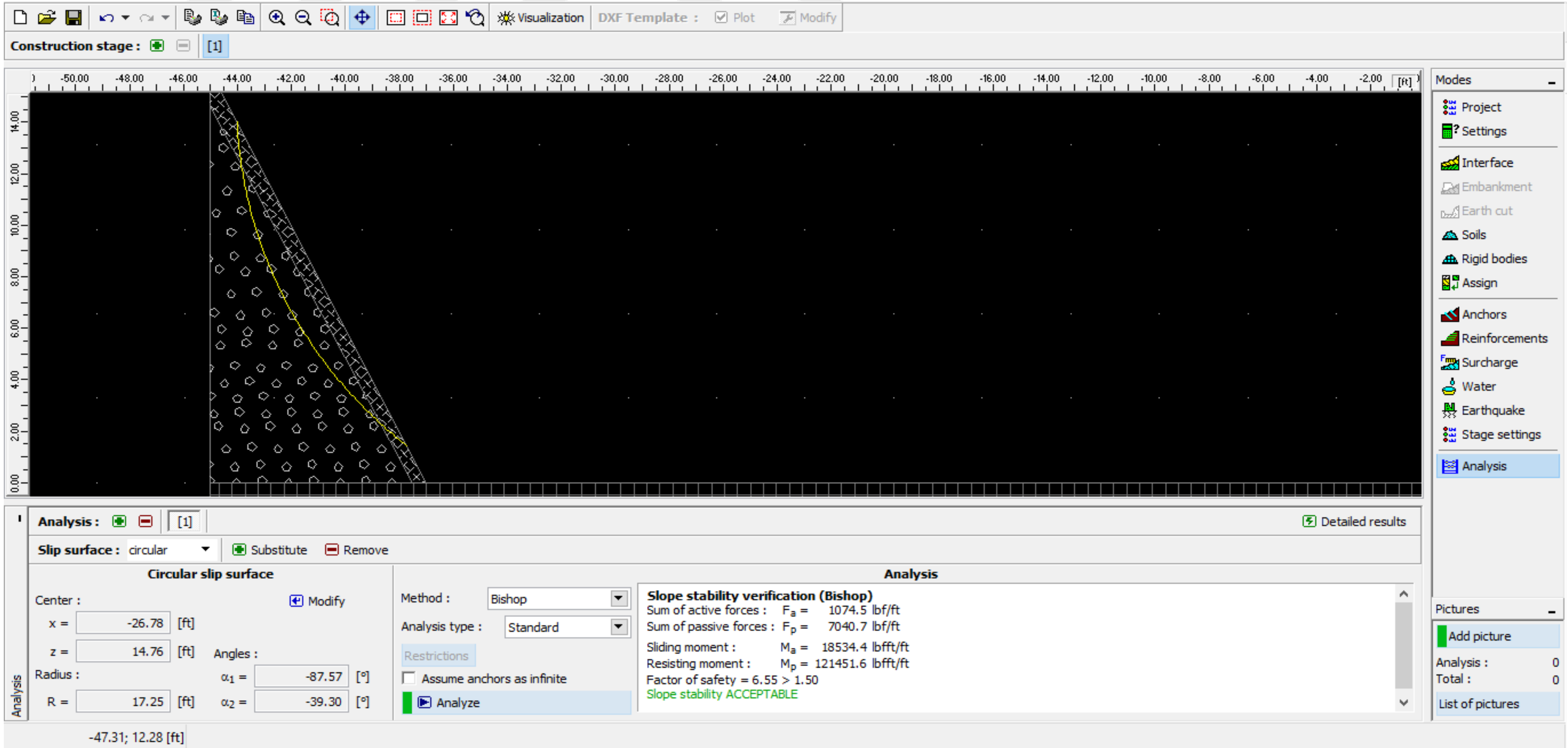


Figure 27. Slope slip analysis of the West Facade KD-4 Wall-1

When the physico-mechanical characteristics of rubble are reduced, the safety coefficient decreases to ~ 4.0.

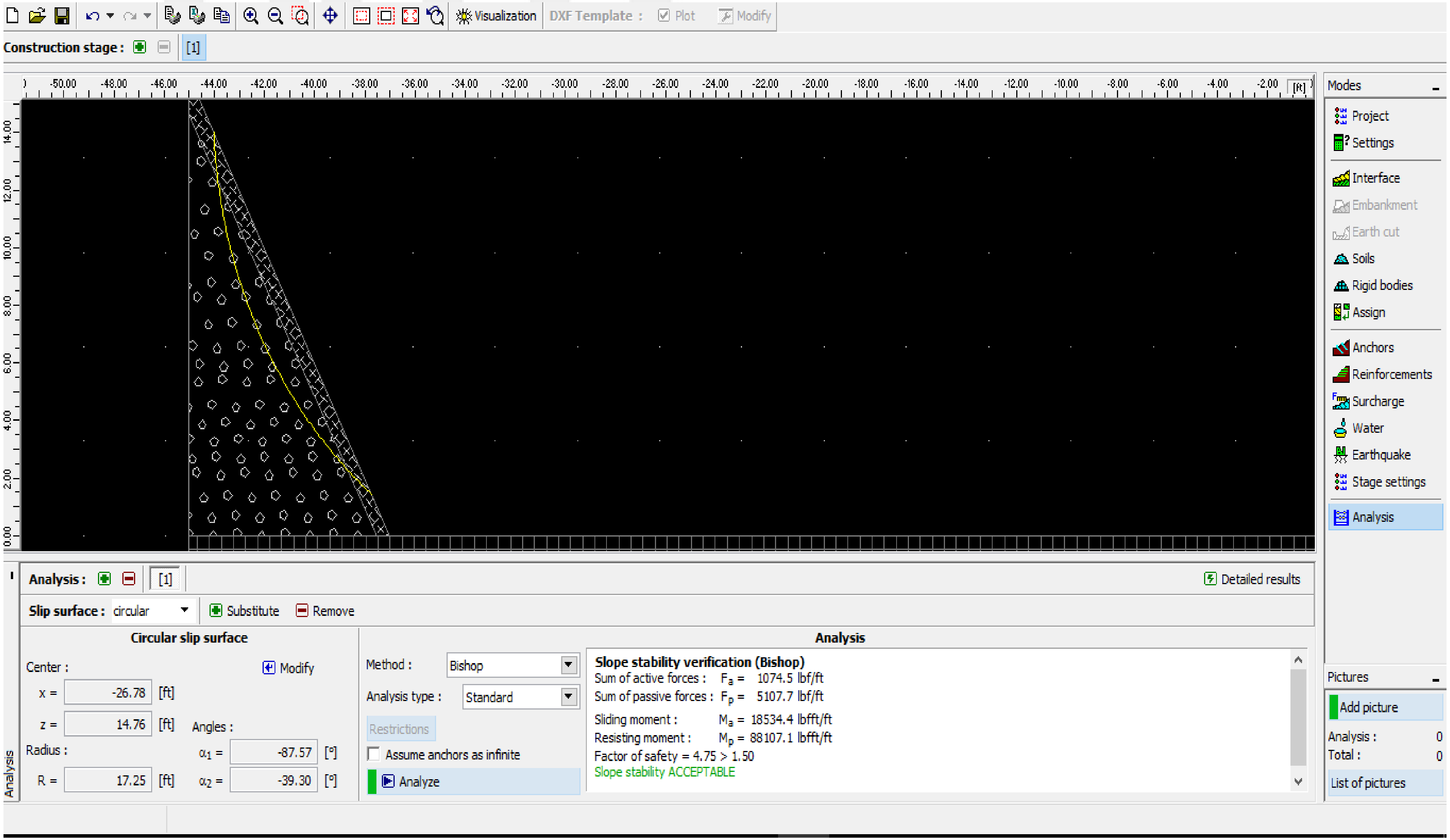


Figure 28. Slope slip analysis of the West Facade KD-4 Wall-2

When the outer shell injection is played with the values of the surface, it shows that the safety coefficient is more effective than the decrease in the internal filling. It should therefore be noted that it is advisable to perform the injection application to be used for the consolidation of the surface, to expose the wall to the desired depth, and to repeat the application until the mortar is filled to the internal cavities at regular intervals and until it cures.

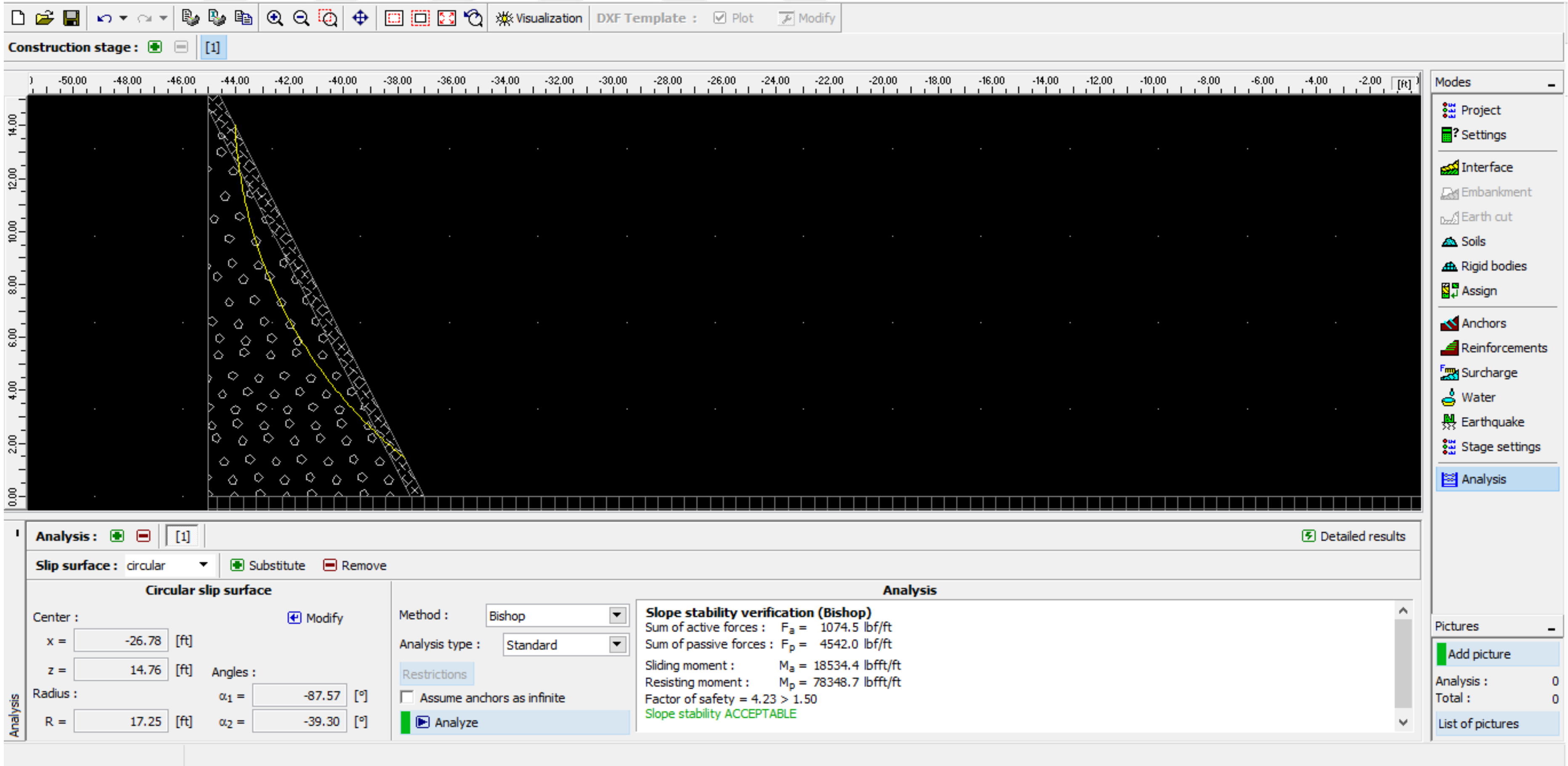


Figure 29. Slope slip analysis of the West Facade KD-4 Wall-3

When the cohesion value of the internal debris is approached to 0 degrees, it can be seen that when the external injection layer is considered 30 cm deep (with the same parametric values as in the above analyzes), the safety coefficient is very close to the limit value. Therefore, it can be said that the thickness of the need for injection should be min 30 cm for this steep slope angle (~ 70 degrees).

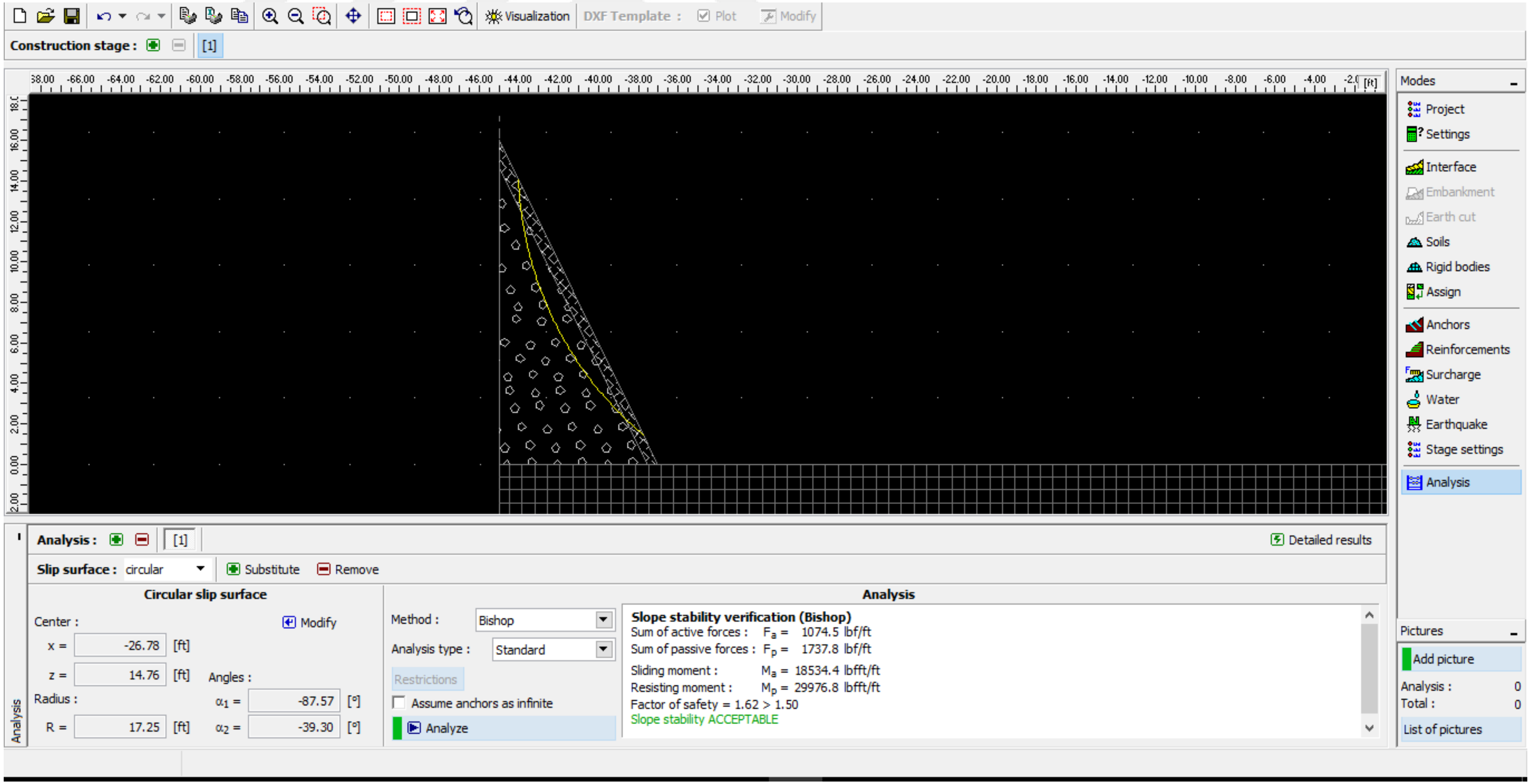


Figure 30. Slope slip analysis of the West Facade KD-4 Wall-4

4.3. Steel Platform Static Calculations;

This report contains the static project accounts of the pedestrian platform structure planned for the Hurman Fortress located within the boundaries of Afsin Municipality of Kahramanmaraş. The platform structure is steel construction and is solved by SAP2000 program.

Figure 31. Platform Layout Plan

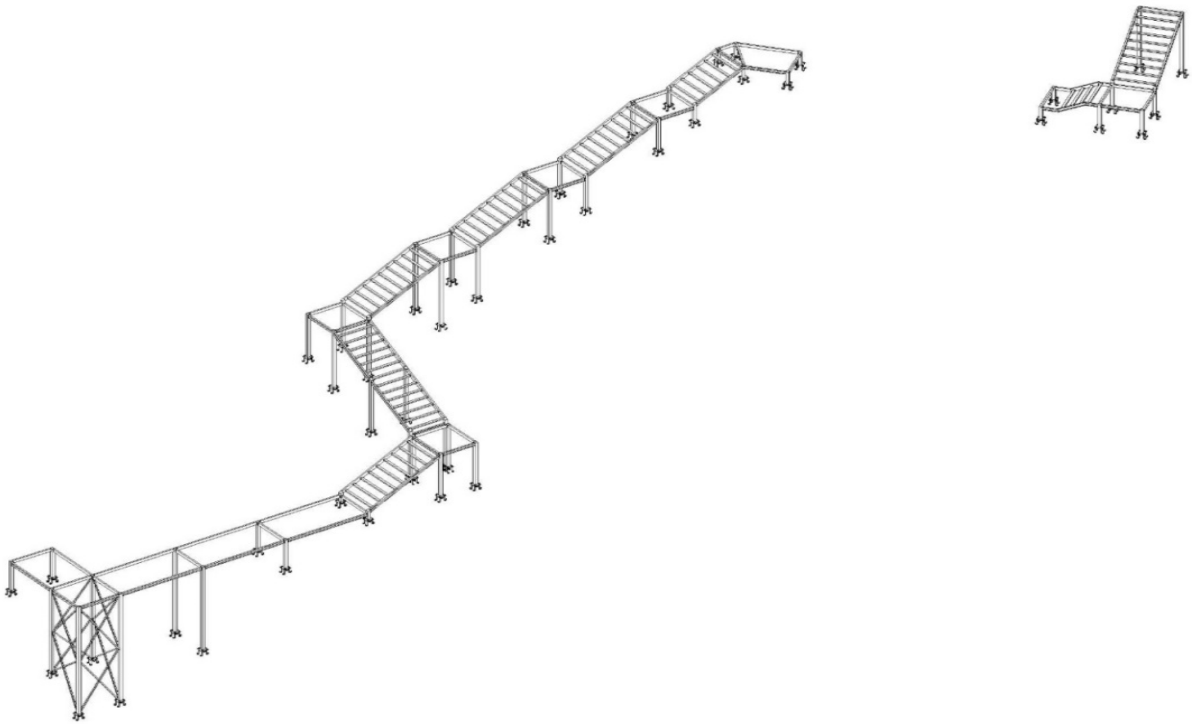


Figure 32. Platform 3-D View

Platform structure static system consists of steel column beam system on reinforced concrete foundation shoes. The platform structure, which works separately from each other, is solved in 3 separate parts..

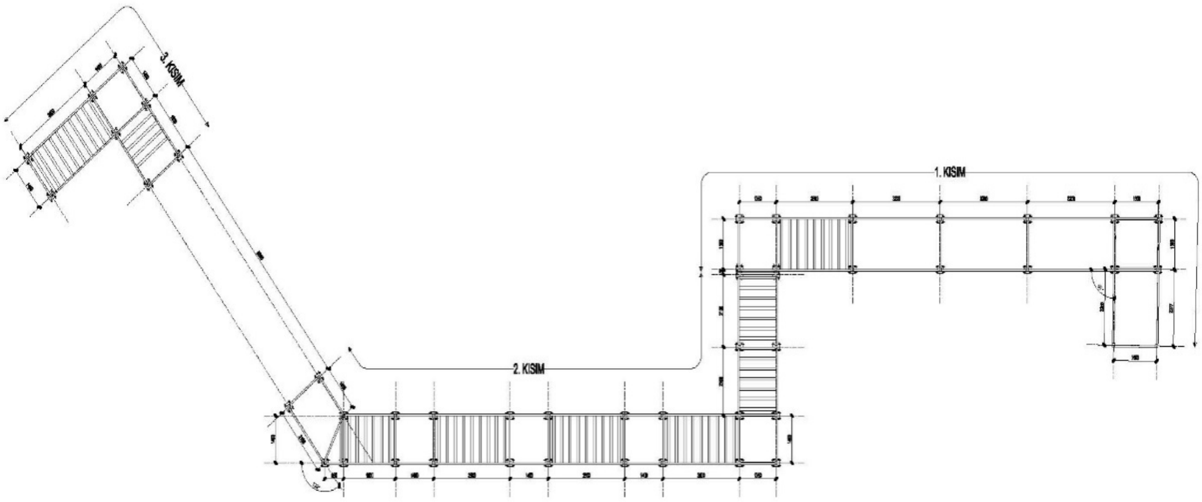


Figure 33. Portion s Account Platform

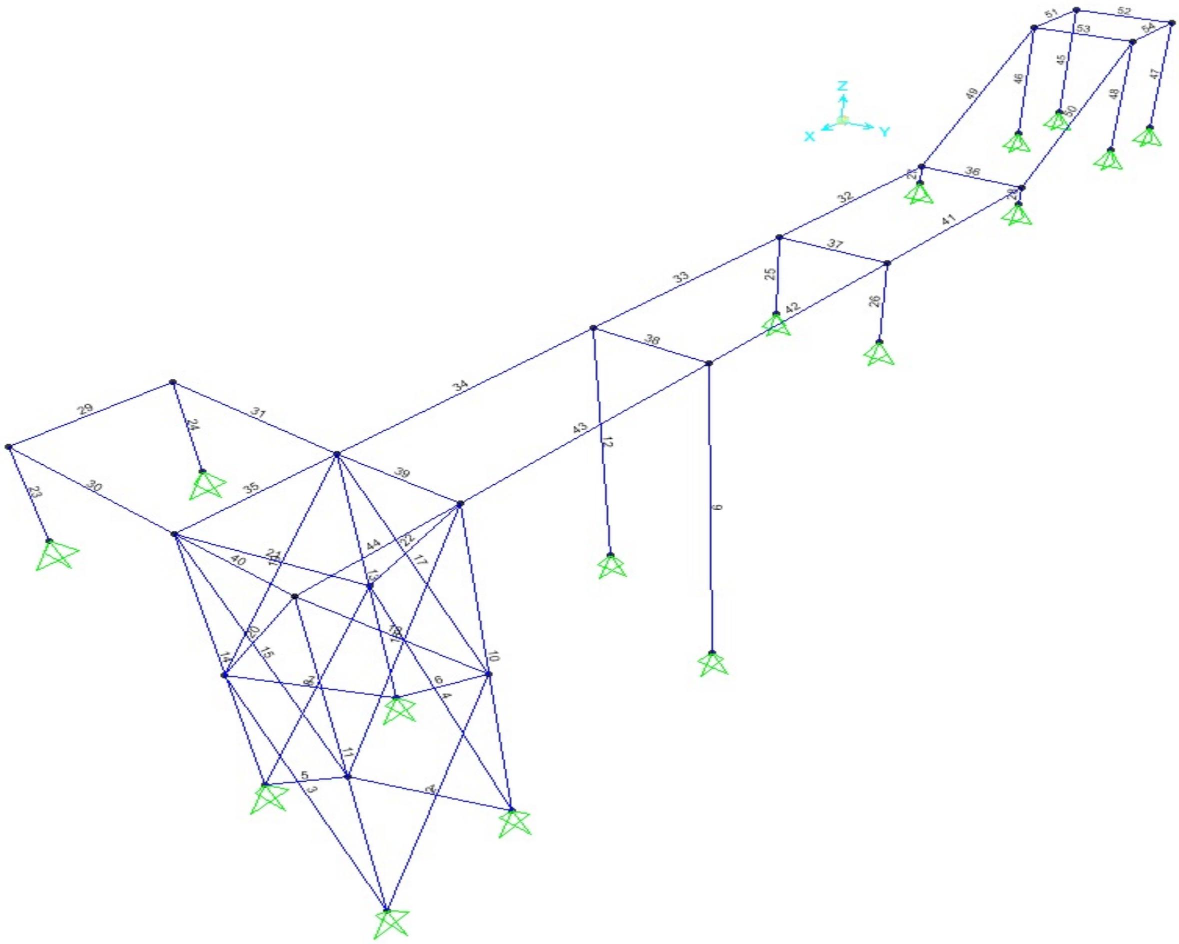


Figure 34. PORTION -1- Element Numbers

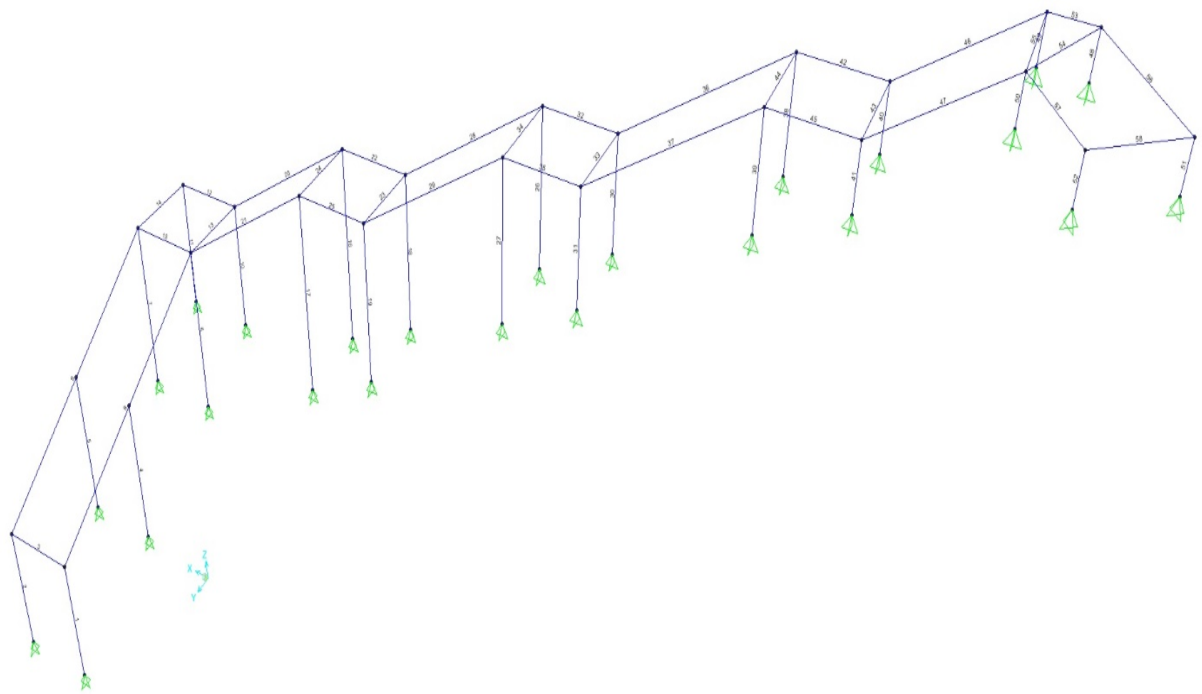


Figure 35. PORTION -2- Element Numbers

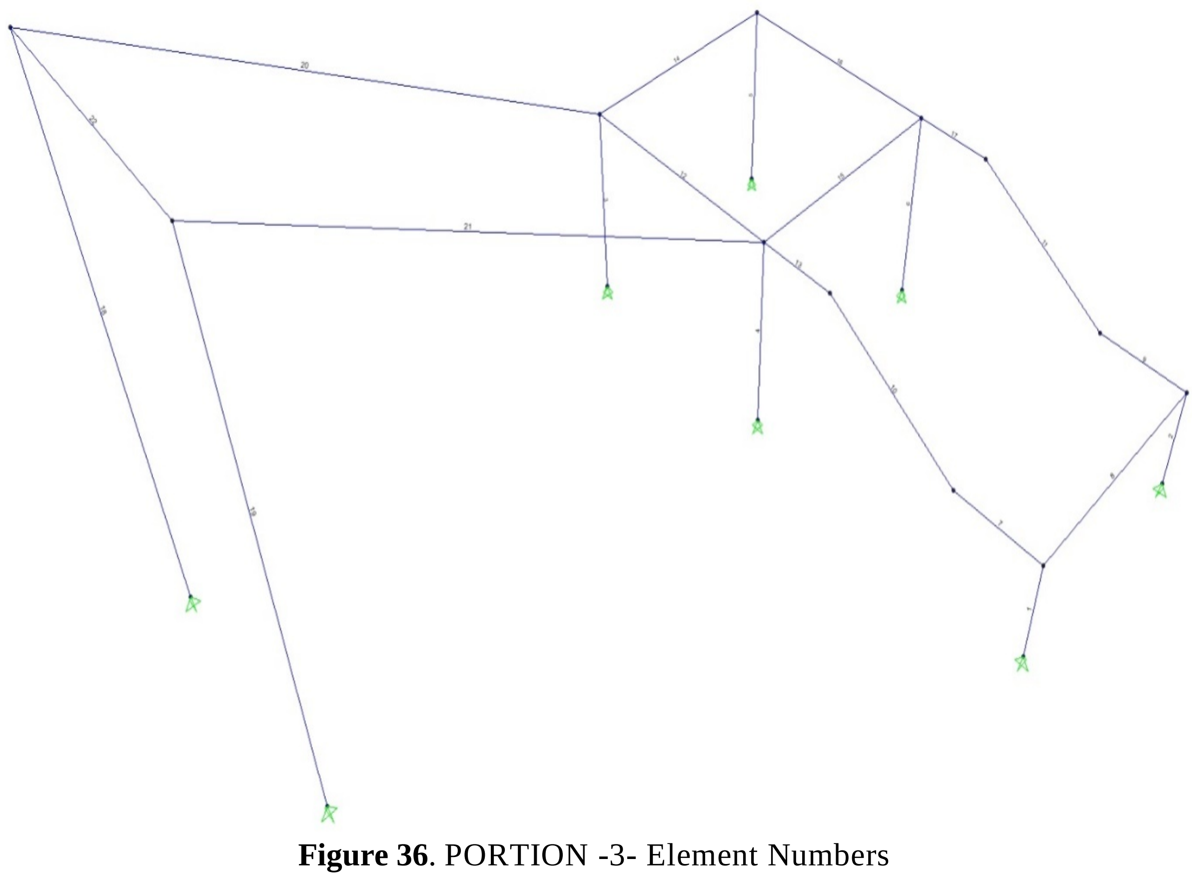


Figure 36. PORTION -3- Element Numbers

5. PRINCIPLES OF DESIGN

Scheduled for "Castle Pedestrian Platform's palm" practices to be followed in the preparation of project specifications and methods to be used are summarized below:

5.1. Regulations and Specifications to be Used in Accounts

• Regulation on the Design, Calculation and Construction of Steel Structures 2016

• TS EN 1991-1-4: Impacts on Structures - Part 1-4: General Impacts - Wind Effects (Eurocode 1)

• TS498: Account Values of Loads to be Taken in Structural Elements

• TS500: Design and Construction Rules of Reinforced Concrete Structures

• Regulation on Buildings to be Constructed in Earthquake Areas, 2007 (DBYBHY 2007)

• AISC-LRFD99: Load and Resistance Factor Design Specification for Structural Steel Buildings

5.2. Materials to Use and Features

5.2.1. Structural Steel

S235JR (St37) steel will be used in the entire bearing system. Material mechanical properties are given below.

• Rupture strength, Fu = 360 MPa

• Yield strength, Fy = 235 MPa (40 mm < t ≤ 80 mm için 215 MPa)

• Elastic module, E = 200000 MPa

• Slip module, G = 77200 t/cm^2

• Coefficient of expansion, α = 1.2e-5 /0C

5.2.2. Anchor Bolts

The structure of the Hurman Castle Pedestrian Platform will be built on the steel construction of the reinforced concrete system and the anchor rods will be placed before the concrete casting and will be left in the concrete. Anchoring materials were selected as ISO 898 Grade 4.6. Material mechanical properties are given below.

• Rupture strength, Fu = 400 MPa

• Yield strength, Fy = 240 MPa

• Characteristic Tensile Stress Strength, Fnt = 375 MPa

• Characteristic Shear Stress Strength in Crush Efficient Joints, Fnv = 225 MPa

5.2.3. Structural bolts

The bolts to be used in steel construction connections are selected as ISO 898 Grade 8.8. Material mechanical properties are given below.

• Rupture strength, Fu = 800 MPa

• Yield strength, Fy = 640 MPa

• Characteristic Tensile Stress Strength, Fnt = 600 MPa

• Characteristic Shear Stress Strength in Crush Efficient Joints, Fnv = 360 MPa

5.2.4. Structural Resource

The resources to be made in the production will be full penetration. Measurements of corner welds not specified in details

• If the thickness of the welded element is thinner than 6 mm “0.7 t”

• If the edge thickness of the welded element is 6 mm or thicker, it will be ”0.7 (t-2)“ (t: edge thickness of the welded element).

The welding electrodes (E70XX) specifications are as follows:

• Tensile strength, Fu = 480 MPa

• Yield strength, Fy = 400 MPa

5.3. Location Features

which will be held in Kahramanmaraş province of "Fortress Pedestrian Platform's palm" because of the geographic location where the structure is located, Public Works and Housing Ministry issued by the five regions separated by Turkey Earthquake Map for Grade 3 Seismic Zone is located in the zone. Continuity level is designed as structures essential to the entire seismic loads transferred by the column "Continuity normalize Systems" was obtained. Soil class "Soil Investigation Report" are listed below ground given class.

Below are some parameters to be used in the static calculations of the project.;

• Effective ground acceleration coefficient, A0 = 0.2 g

• Building factor of importance, I = 1.5

• Carrier system behavior coefficient, Rx = 5.0; Ry = 5.0

• Spectrum characteristic periods, TA = 0.10, TB = 0.30

• Earthquake spectrum equation: S(T) = 1+ 1.5 T/TA $(0 \leq T \leq TA)$

2. 5 $(TA < T \leq TB)$

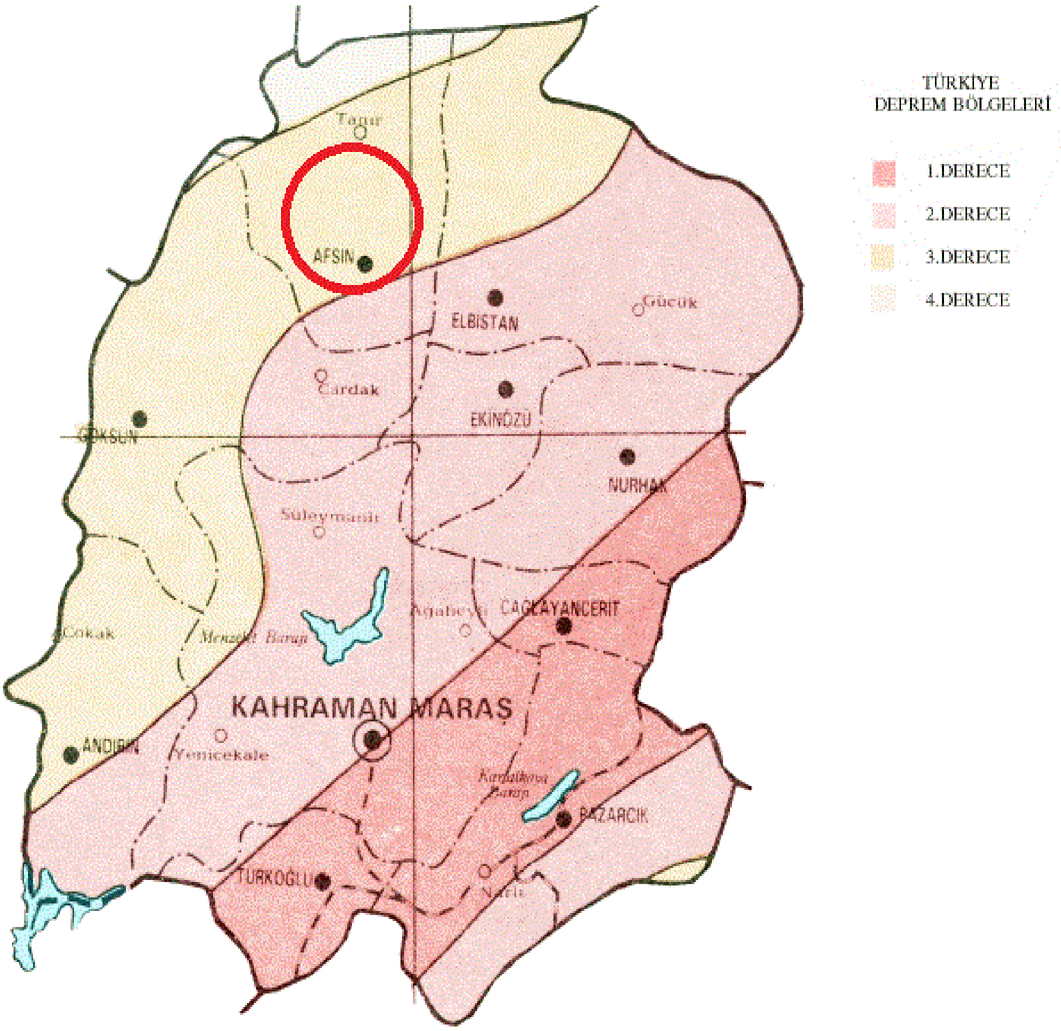
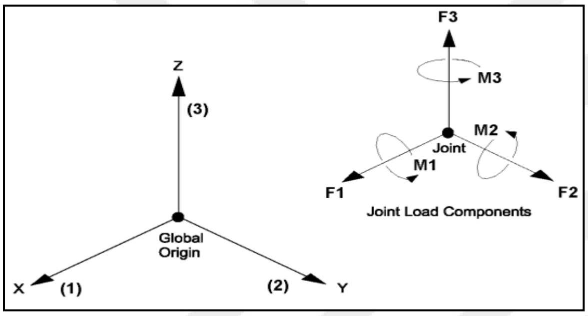


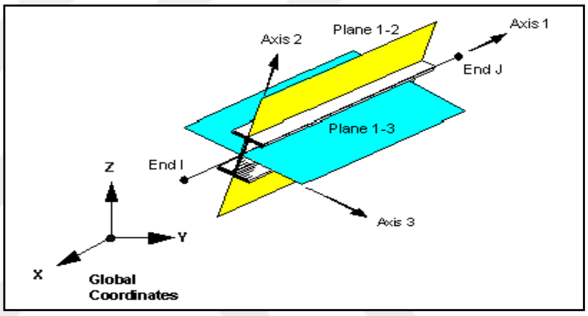
Figure 37. Kahramanmaraş Province Earthquake Region Map

6. ANALYSIS AND DESIGN METHODS

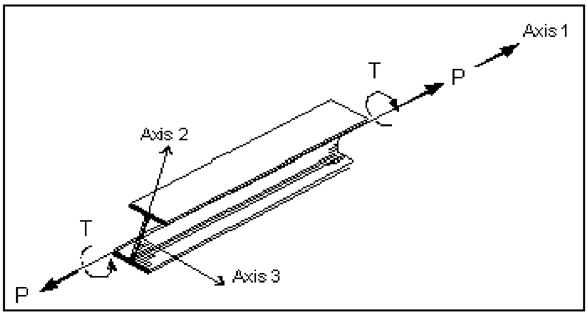
SAP2000 ın Static and Dynamic Finite Element Analysis of Structures ın structural analysis program was used in the analysis and design of the steel construction system to be constructed within the scope of the Hurman Castle Pedestrian Platform program to be constructed in the borders of the province of Kahramanmaras. AISC-LRFD 99, Steel Regulations and the Regulations on Structures to be Built in Earthquake Regions are used in the dimensioning and elaboration of the structure. It was used in the design of steel elements (Design with Load and Resistance Factors- YDKT) and design of reinforced concrete elements (Transport Force Method). All columns of the building, beams, purlins and cross elements (bar-frame) are defined as elements. Acceptances direction on the nodes and rod elements are given below.



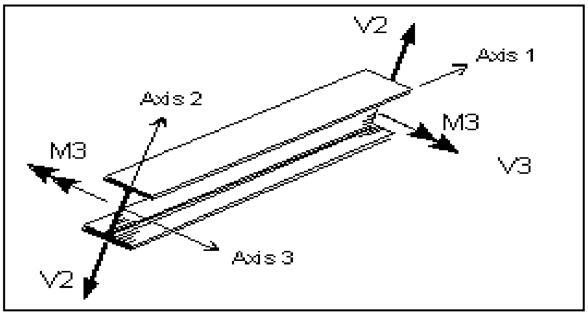
Node Points Direction Acceptances



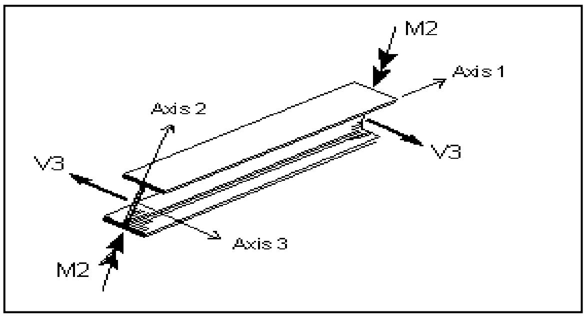
Node Points Direction Acceptances



Positive Axial Force and Torque



Positive Moment and Cutting in the Plane



Positive Moment and Cutting in the Plane

7. LOADS AND LOAD COMBINATIONS

7.1. Permanent Loads (DL)

7.1.1. Steel Element Weights

Steel material density was 7.85 tons / m3. Structural elements weights can be calculated according to profile cross-sectional areas by the analysis program..

(In the analysis program “DEAD” is defined by.)

7.1.2. Coating Loads

On the platform structure;

Wood coating weight 50 kg / m2.

(In the analysis program “DL1 ” is defined by.)

7.2. Moving Load (LL)

The moving load acting on the building elements can be calculated according to the section EM TS 498 LOADS OF LOADS THAT WILL BE TAKEN IN THE DIMENSION OF BUILDING ELEMENTS section 12.1. According to this, in analysis and design, as moving load public buildings are stated as 500 kg / m².

(In the analysis program “ LL1 ” is defined by.)

7.3. Earthquake Load (EX, EY)

Co Hurman Castle Pedestrian Platform structure which is to be constructed within the boundaries of Kahramanmaraş province was effected by the earthquake load method of User Coefficient. The earthquake load coefficients were determined according to the natural vibration periods in the building principal axes and were defined in the analysis program. In the building modal analysis, 30% of the moving loads were taken into consideration with the permanent loads in the mass participation.

(In the analysis program “EQX, EQ-X, EQY and EQ-Y” is defined by.)

8. STEEL CONSTRUCTION DESIGN

8.1. Main Structure - Analysis and Design Model

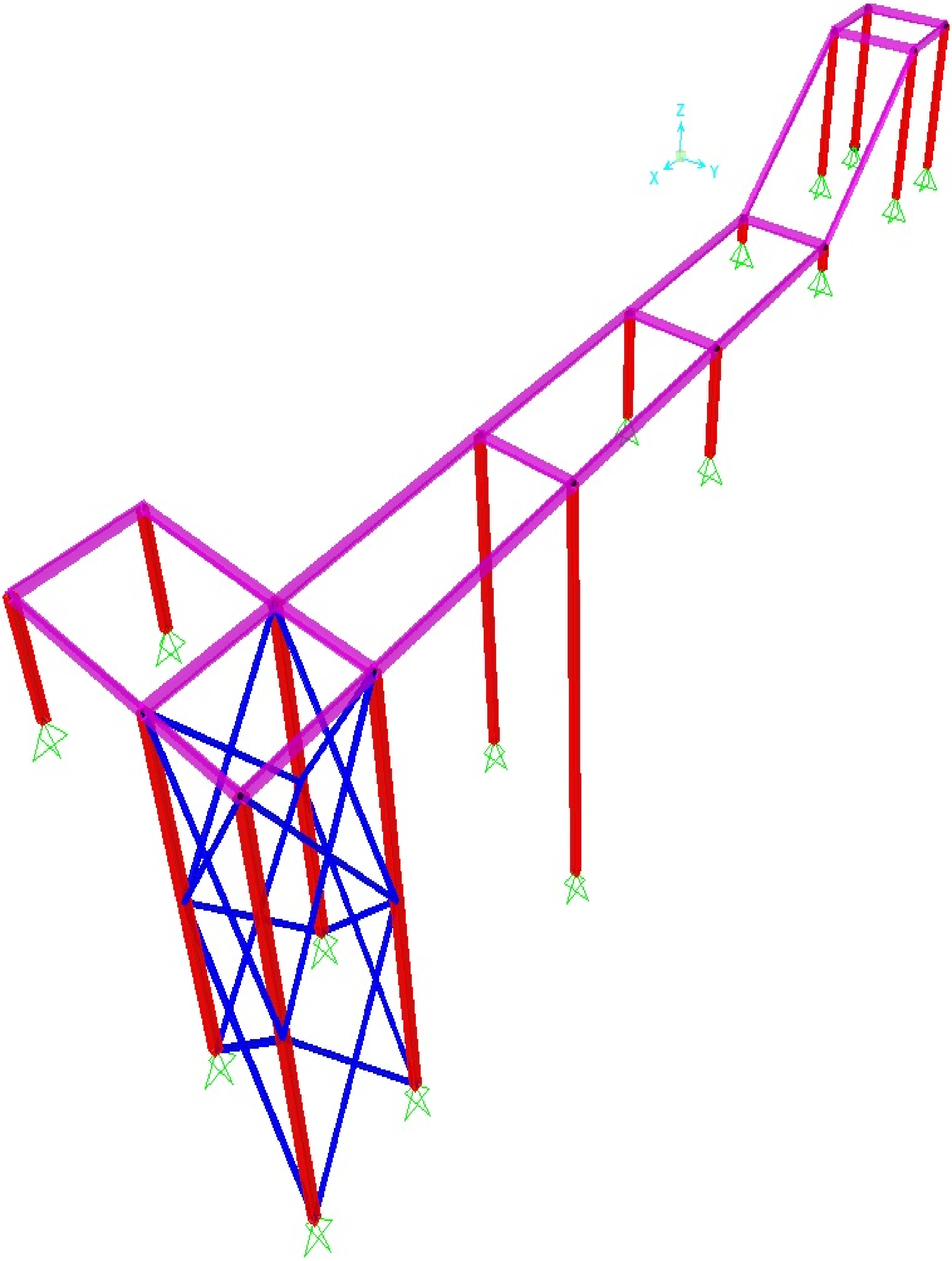


Figure 38. PORTION -1- Analysis 3D Model View

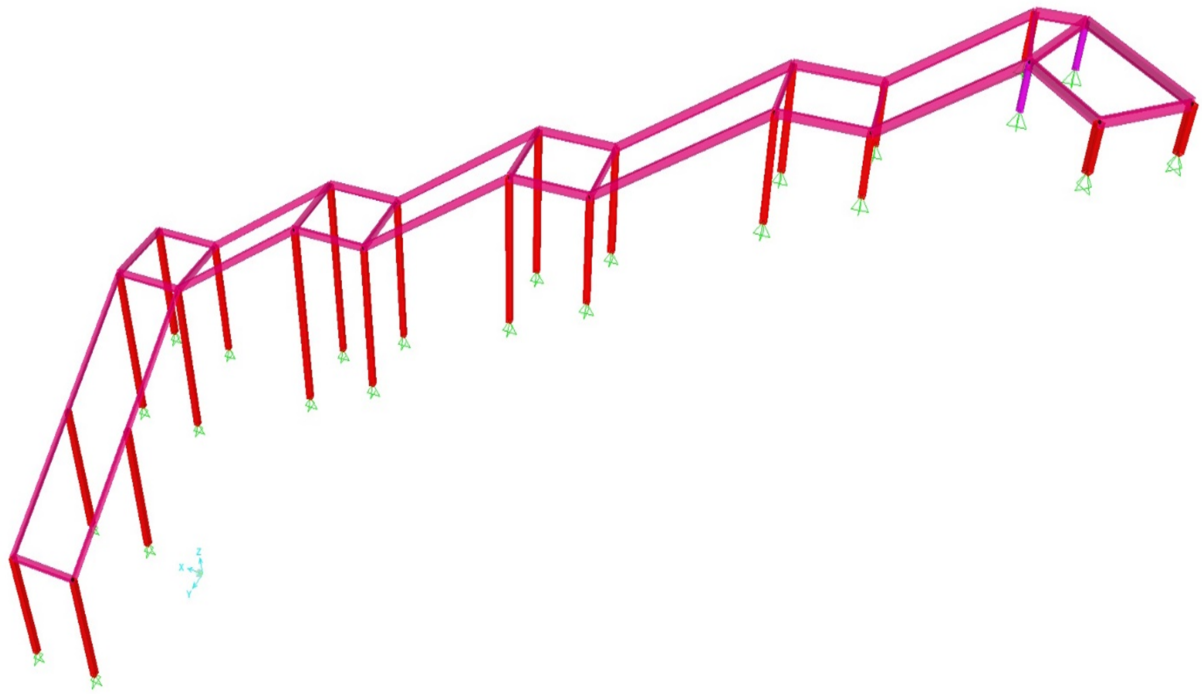


Figure 39. *PORTION* -2- Analysis 3D Model View

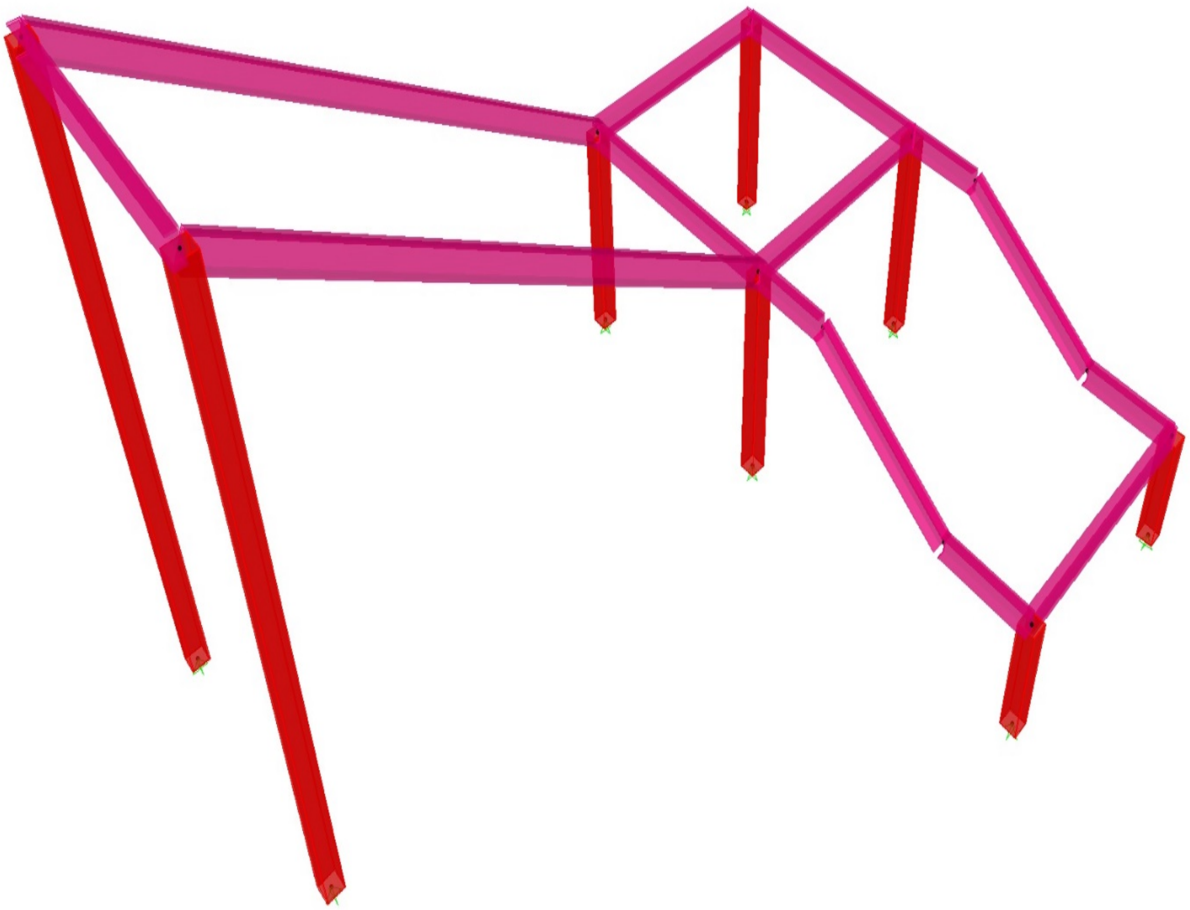
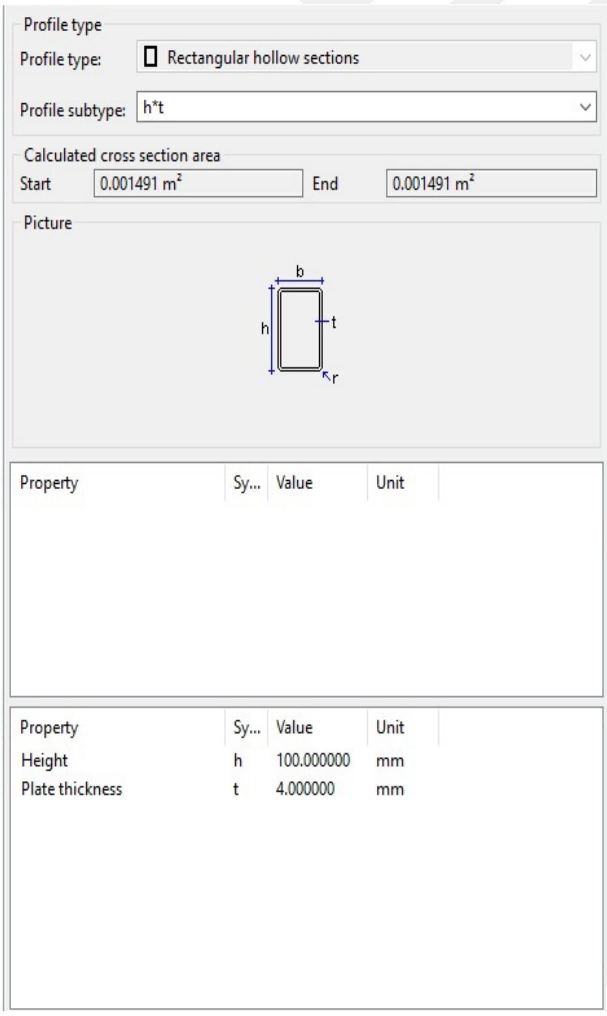


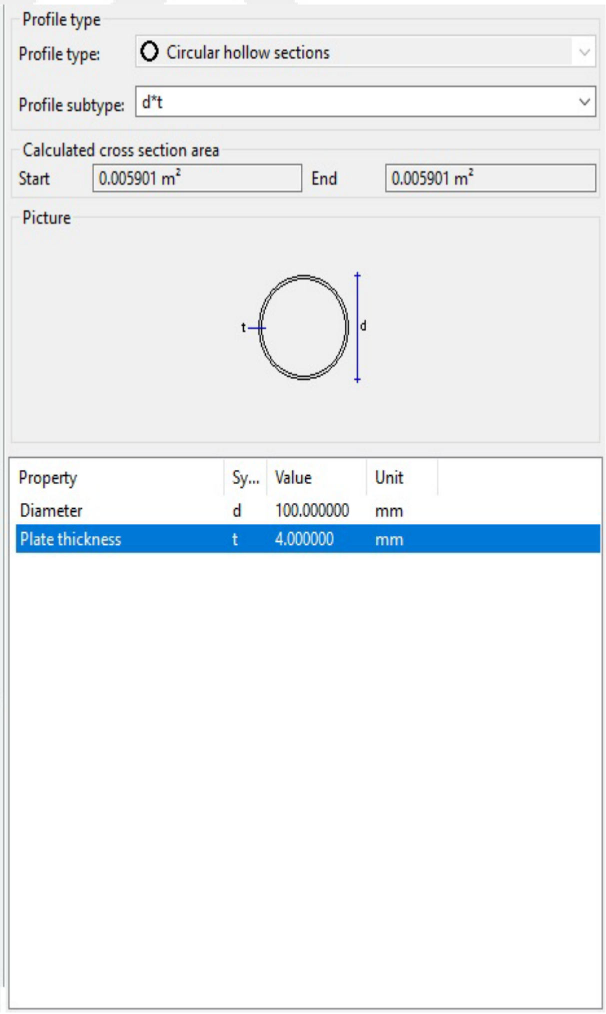
Figure 40. PORTION -3- Analysis 3D Model View

8.1.1. Element Sections

SECTION FEATURES 1									
SectionName	**Material**	**Shape**	**t3**	**t2**	**tf**	**tw**	**t2b**	**tfb**	**Area**
Text	Text	Text	cm	cm	cm	cm	cm	cm	cm2
CHS100*4	S235JR	Pipe	10			0,4			12,06
RHS100*4	S235JR	Box/Tube	10	10	0,4	0,4			15,36
RHS40*2	S235JR	Box/Tube	4	4	0,2	0,2			3,04
UPN120	S235JR	Channel	12	5,5	0,9	0,7			16,98

SECTION FEATURES 2								
SectionName	**TorsConst**	**I33**	**I22**	**I23**	**AS2**	**AS3**	**S33**	**S22**
Text	cm4	cm4	cm4	cm4	cm2	cm2	cm3	cm3
CHS100*4	278,43	139,22	139,22	0	6,04	6,04	27,84	27,84
RHS100*4	353,89	236,34	236,34	0	8	8	47,27	47,27
RHS40*2	10,97	7,34	7,34	0	1,6	1,6	3,67	3,67
UPN120	3,84	364,1	43,14	0	8,4	8,25	60,68	11,06





Profile type

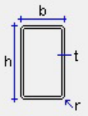
Profile type: Rectangular hollow sections

Profile subtype: h*t

Calculated cross section area

Start 0.000293 m² End 0.000293 m²

Picture



Property	Sy...	Value	Unit

Property	Sy...	Value	Unit
Height	h	40.000000	mm
Plate thickness	t	2.000000	mm

Profile type

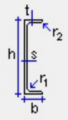
Profile type: U profiles

Profile subtype: Hot rolled

Calculated cross section area

Start 0.001701 m² End 0.001701 m²

Picture



Property	Sy...	Value	Unit
Height	h	120.000000	mm
Width	b	55.000000	mm
Web thickness	s	7.000000	mm
Flange thickness	t	9.000000	mm
Rounding radius 1	r1	9.000000	mm
Rounding radius 2	r2	4.500000	mm
Flange slope ratio	fs	0.08	°

Profile type

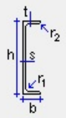
Profile type: U profiles

Profile subtype: Hot rolled

Calculated cross section area

Start 0.001104 m² End 0.001104 m²

Picture



Property	Sy...	Value	Unit
Height	h	80.000000	mm
Width	b	45.000000	mm
Web thickness	s	6.000000	mm
Flange thickness	t	8.000000	mm
Rounding radius 1	r1	8.000000	mm
Rounding radius 2	r2	4.000000	mm
Flange slope ratio	fs	0.08	°

8.1.2. Earthquake Load

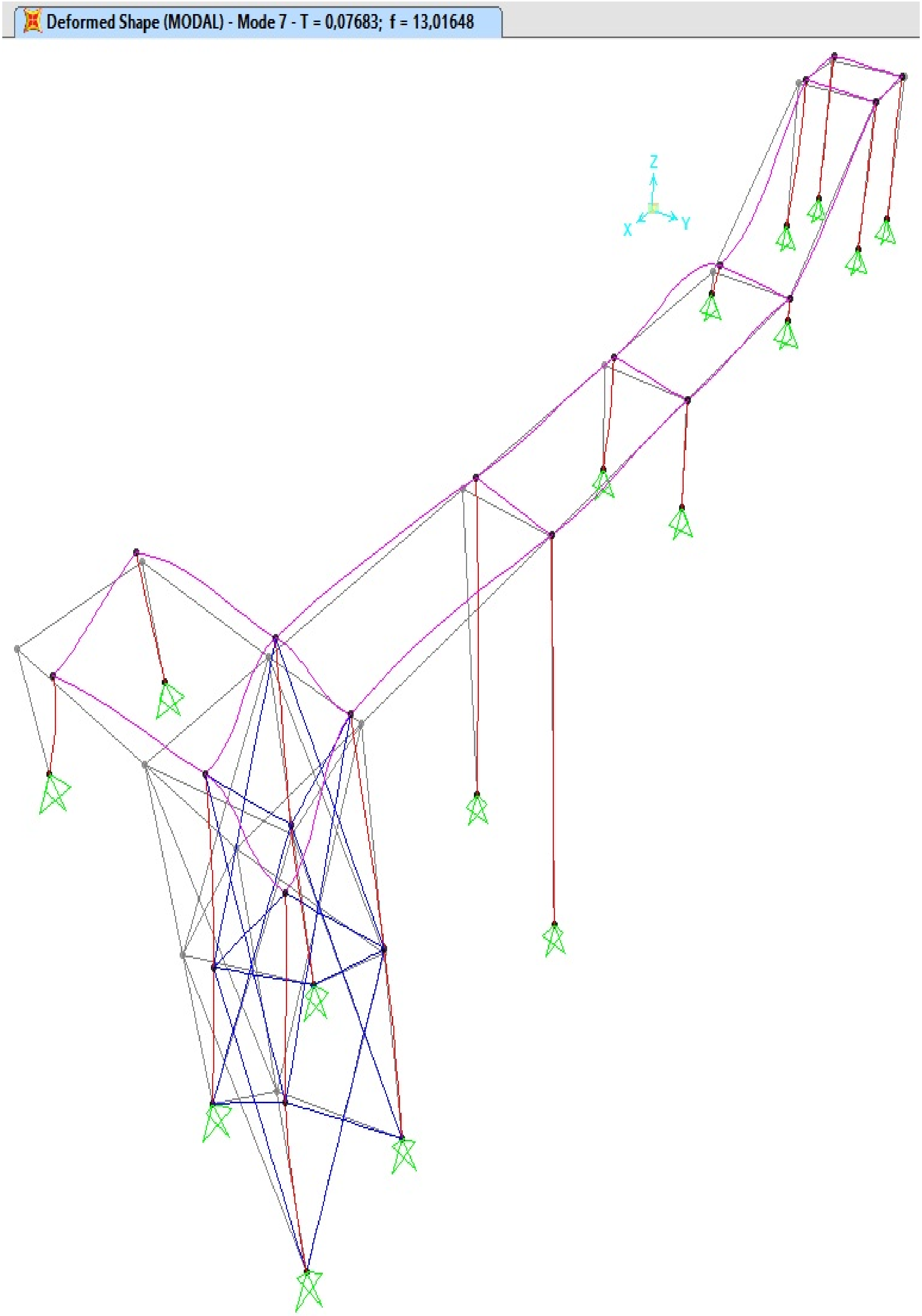


Figure 41. PORTION -1- Structure Axial Periods, T1,x = 0.07683 s

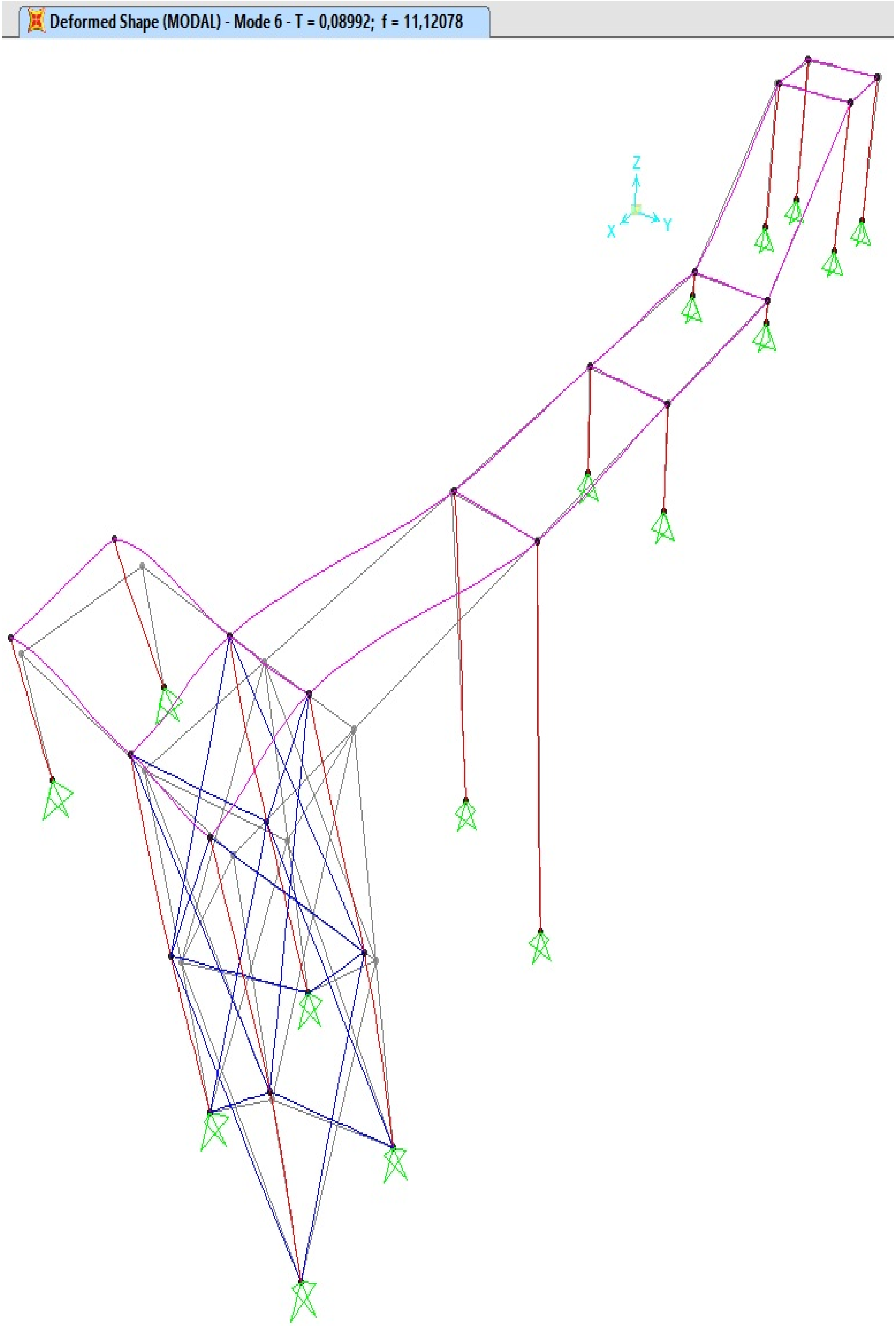


Figure 42. PORTION -1- Structure Axial Periods, T1,y = 0.08992 s

TABLE: Base Reactions								
Output Case	**CaseType**	**Step Type**	**Global FX**	**Global FY**	**Global FZ**	**Global MX**	**Global MY**	**Global MZ**
Text	Text	Text	Tonf	Tonf	Tonf	Tonf-m	Tonf-m	Tonf-m
DEAD	LinStatic		-2,9E-17	-3,7E-16	1,3571	577,731	-1317,51	-1,2E-13
DL1	LinStatic		5,15E-17	2,55E-17	1,2332	515,509	-1020,02	1,47E-14
DL2	LinStatic		0	0	0	0	0	0
LL1	LinStatic		4,76E-16	2,77E-16	12,3315	5155,088	-10200,2	1,4E-13
LL2	LinStatic		0	0	0	0	0	0
EQX	LinResp Spec	Max	-0,8726	1,36E-16	-3,1E-17	-6,8E-15	-144,71	368,641
EQ-X	LinResp Spec	Max	0,8726	-1,4E-16	3,11E-17	6,79E-15	144,71	-368,641
EQY	LinResp Spec	Max	2,12E-16	-0,8583	4,53E-16	142,336	-5,1E-13	-678,796
EQ-Y	LinResp Spec	Max	-2,1E-16	0,8583	-4,5E-16	-142,336	5,07E-13	678,796

W = 6,28975　　0,9*W = 5,660775　　Vx = 0,87　　Dx = 0,999999

Vx = 0,86　　Dy = 1,000029

Tx = 0,077 sn　Earthquake Region = 3　Effective Groud Acceleration Coefficient (Ao) =0.2

Ty = 0,090 sn　Local Floor Class= Z1　T(A) = 0,10　T(B) = 0,30

Building Importance Factor (I) = 1,5　Building Importance Factor (R) : 5

Carrier System Behavior Factor, Determination of R　R(Tx) = 4,19　R(Ty) = 4,65

Determination of Spectrum Coefficient S(T)　S(Tx) = 2,15　S(Ty) = 2,35

Determination of Spectrum Acceleration Coefficient A(T)　A(Tx) = 0,6469　A(Ty) = 0,70

Determination of Earthqueke Load V(Tx)　V(Tx) = 0,1541*W　V(Ty) = 0,1516*W

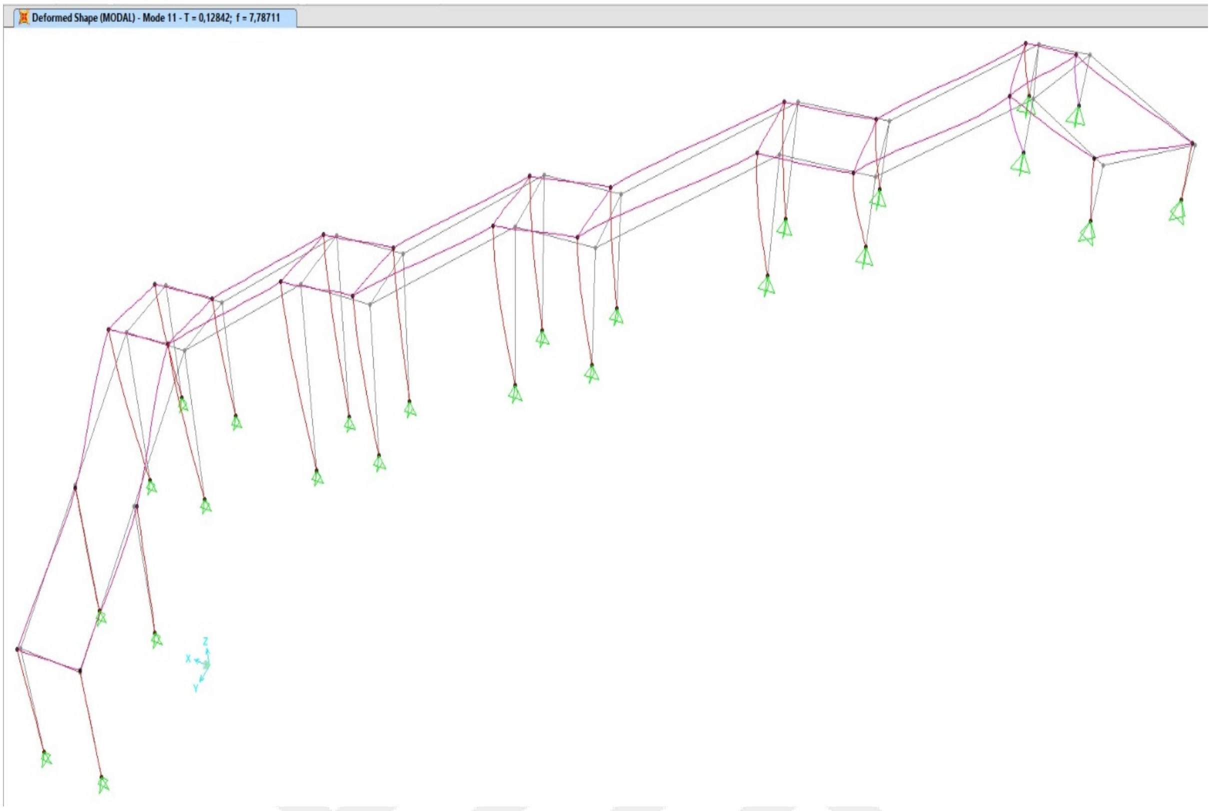


Figure 43. PORTION -2- Structure Axial Periods, T1,x = 0.12848 s

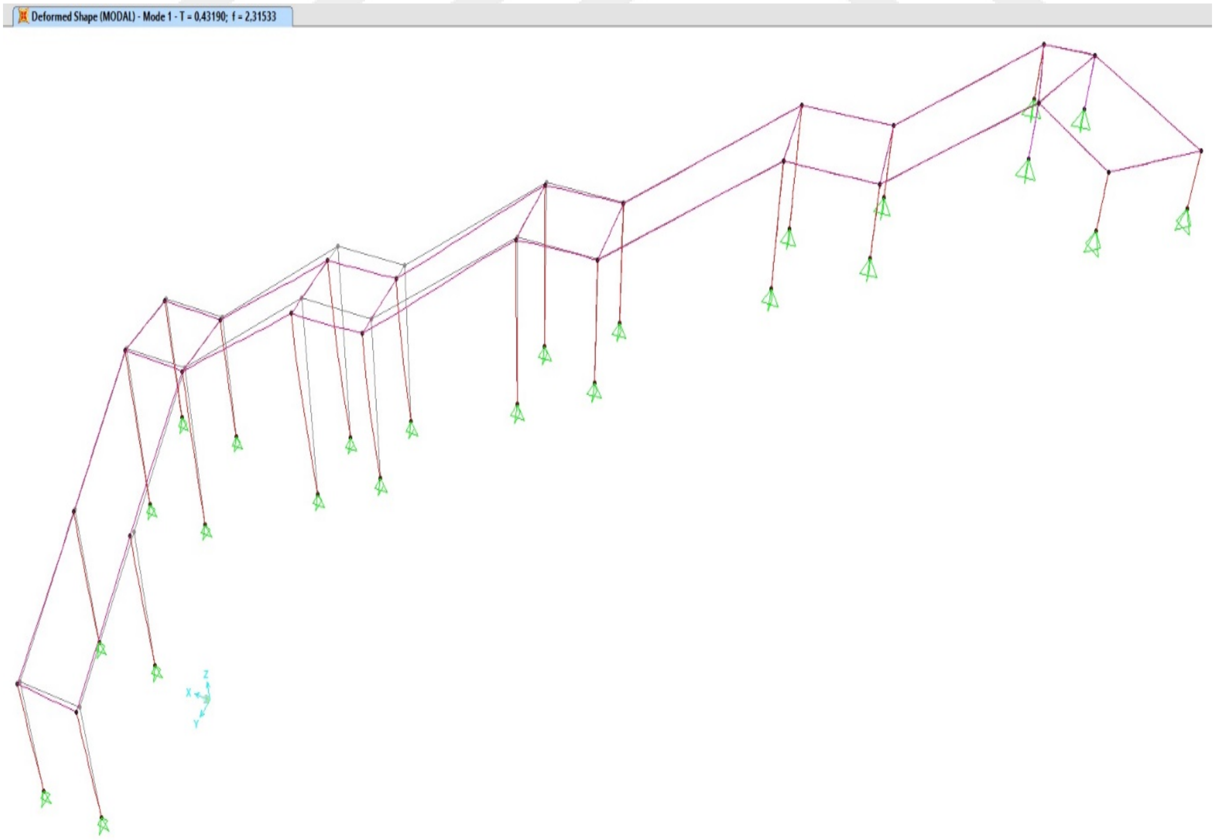


Figure 44. PORTION -2- Structure Axial Periods, T1,y = 0.4319 s

TABLE: Base Reactions								
Output Case	**CaseType**	**Step Type**	**Global FX**	**GlobalFY**	**Global FZ**	**Global MX**	**Global MY**	**Global MZ**
Text	Text	Text	Tonf	Tonf	Tonf	Tonf-m	Tonf-m	Tonf-m
DEAD	LinStatic		-2,9E-16	6,18E-16	1,5581	-101,067	853,908	-4,4E-13
DL1	LinStatic		-6,9E-16	1,42E-18	1,7738	-105,312	1021,019	-7E-14
DL2	LinStatic		0	0	0	0	0	0
LL1	LinStatic		-6,8E-15	-1,1E-17	17,7382	-1053,12	10210,19	-8E-13
LL2	LinStatic		0	0	0	0	0	0
EQX	LinRespSpec	Max	-1,1683	-1,7E-15	3,22E-15	1,75E-12	-1046,89	-115,775
EQ-X	LinRespSpec	Max	1,1683	1,77E-15	-3,9E-15	-1,7E-12	1046,889	115,775
EQY	LinRespSpec	Max	-8,5E-16	-0,8727	-9,5E-16	782,077	-1,5E-13	706,926
EQ-Y	LinRespSpec	Max	7,95E-16	0,8727	1,02E-16	-782,077	2,81E-13	-706,926

W = 8,65336 0,9*W = 7,788024 Vx = 1,17 Dx = 0,999917

Vx = 0,87 Dy = 1,000092

Tx = 0,128 sn Earthquake Region = 3 Effective Groud Acceleration Coefficient (Ao) =0.2

Ty = 0,432 sn Local Floor Class= Z1 T(A) = 0,10 T(B) = 0,30

Building Importance Factor (I) = 1,5 Building Importance Factor (R) : 5

Carrier System Behavior Factor, Determination of R R(Tx) = 5,00 R(Ty) = 5,00

Determination of Spectrum Coefficient S(T) S(Tx) = 2,50 S(Ty) = 1,87

Determination of Spectrum Acceleration Coefficient A(T) A(Tx) = 0,75 A(Ty) = 0,56

Determination of Earthqueke Load V(Tx) V(Tx) = 0,1500*W V(Ty) = 0,1121*W

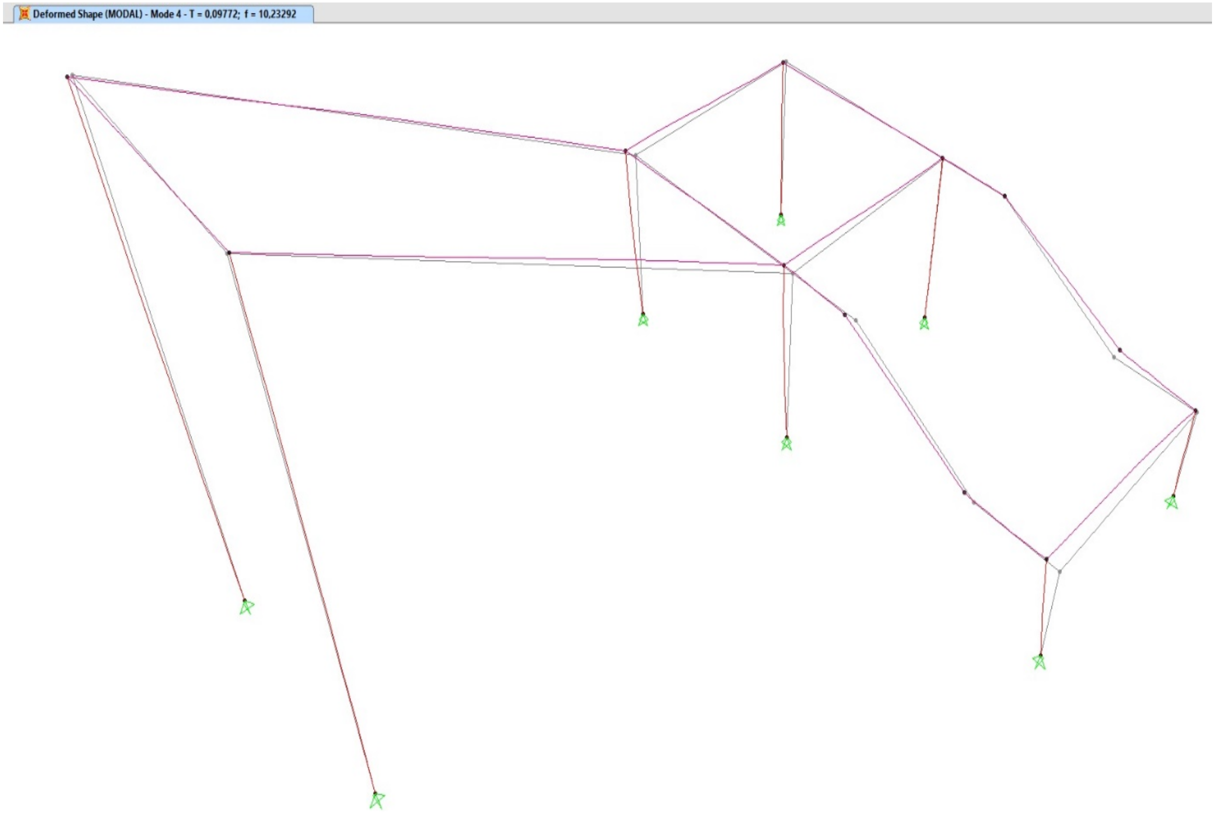


Figure 45. PORTION -3- Structure Axial Periods, T1,x = 0.09772 s

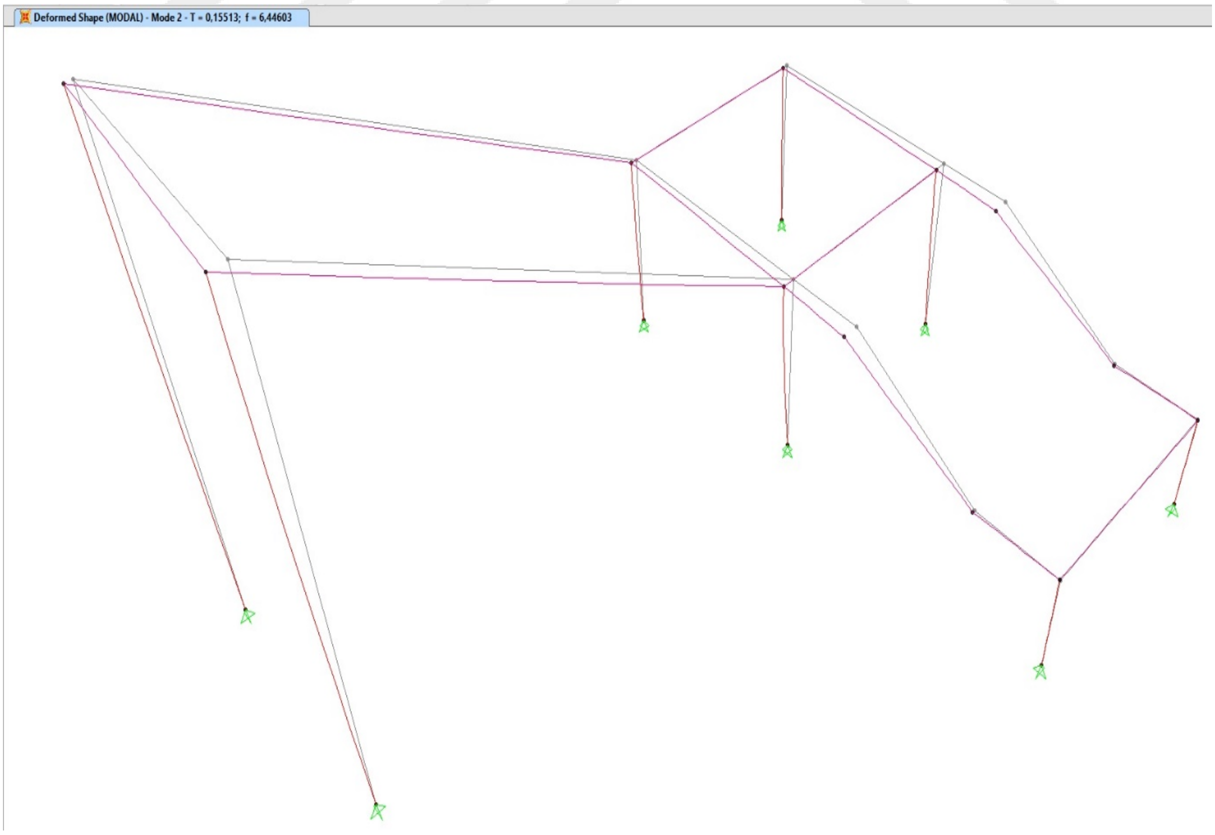


Figure 46. PORTION -3- Structure Axial Periods, T1,y = 0.15513 s

TABLE: Base Reactions								
Output Case	**CaseType**	**Step Type**	**Global FX**	**Global FY**	**Global FZ**	**Global MX**	**Global MY**	**Global MZ**
Text	Text	Text	Tonf	Tonf	Tonf	Tonf-m	Tonf-m	Tonf-m
DEAD	LinStatic		-1,9E-16	-1,3E-16	0,4309	-113,135	1141,439	1,65E-13
DL1	LinStatic		-7,6E-17	-4E-16	0,4839	-116,007	1279,558	8,51E-13
DL2	LinStatic		0	0	0	0	0	0
LL1	LinStatic		-7,6E-16	-4,2E-15	4,8389	-1160,07	12795,58	8,96E-12
LL2	LinStatic		0	0	0	0	0	0
EQX	LinRespSpec	Max	-0,3202	1,18E-15	5,89E-16	-1,6E-12	-420,686	-102,285
EQ-X	LinRespSpec	Max	0,3202	-1,2E-15	-5,9E-16	1,6E-12	420,686	102,285
EQY	LinRespSpec	Max	5,4E-16	-0,3195	-1E-15	419,803	-2,1E-12	849,947
EQ-Y	LinRespSpec	Max	-5,4E-16	0,3195	1,05E-15	-419,803	2,07E-12	-849,947

W = 2,36647 0,9*W = 2,129823 Vx = 0,32 Dx = 1,000039

Vx = 0,32 Dy = 0,999917

Tx = 0,098 sn Earthquake Region = 3 Effective Groud Acceleration Coefficient (Ao) =0.2

Ty = 0,155 sn Local Floor Class= Z1 T(A) = 0,10 T(B) = 0,30

Building Importance Factor (I) = 1,5 Building Importance Factor (R) : 5

Carrier System Behavior Factor, Determination of R R(Tx) = 4,92 R(Ty) = 5,00

Determination of Spectrum Coefficient S(T) S(Tx) = 4,92 S(Ty) = 5,00

Determination of Spectrum Acceleration Coefficient A(T) A(Tx) = 4,92 A(Ty) = 5,00

Determination of Earthqueke Load V(Tx) V(Tx) = 0,1503*W V(Ty) = 0,1500*W

8.1.3. Designs of Steel Elements

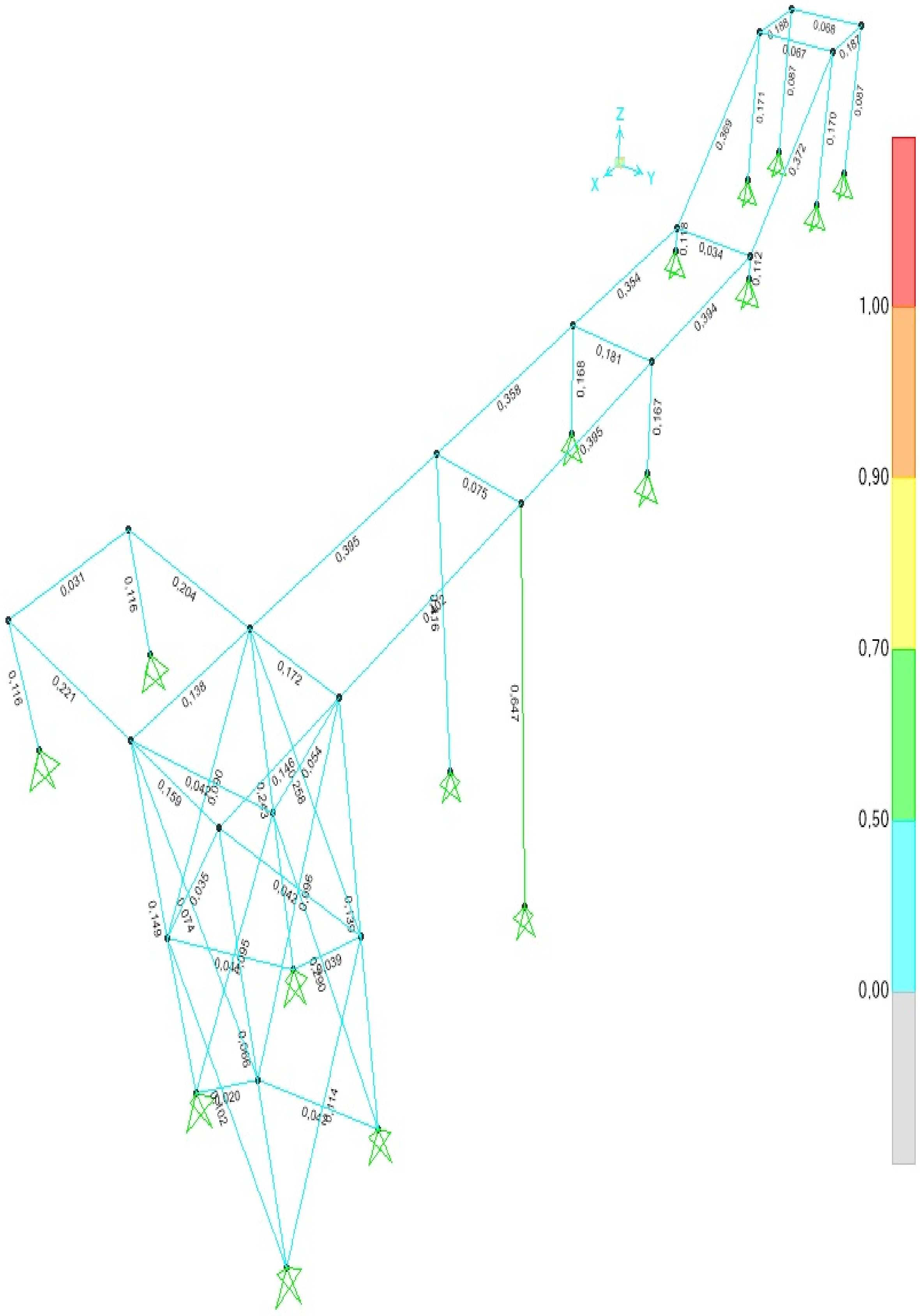


Figure 47. PORTION -1- Steel Design Results - Impact / Capacity Ratios

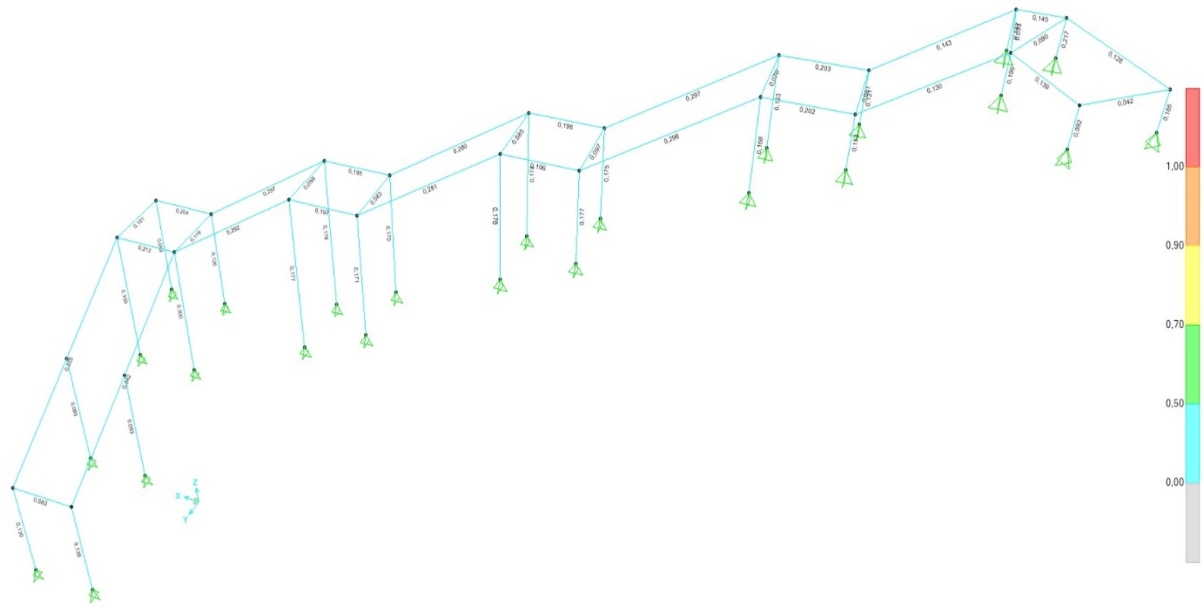


Figure 48. PORTION -2- Steel Design Results - Impact / Capacity Ratios

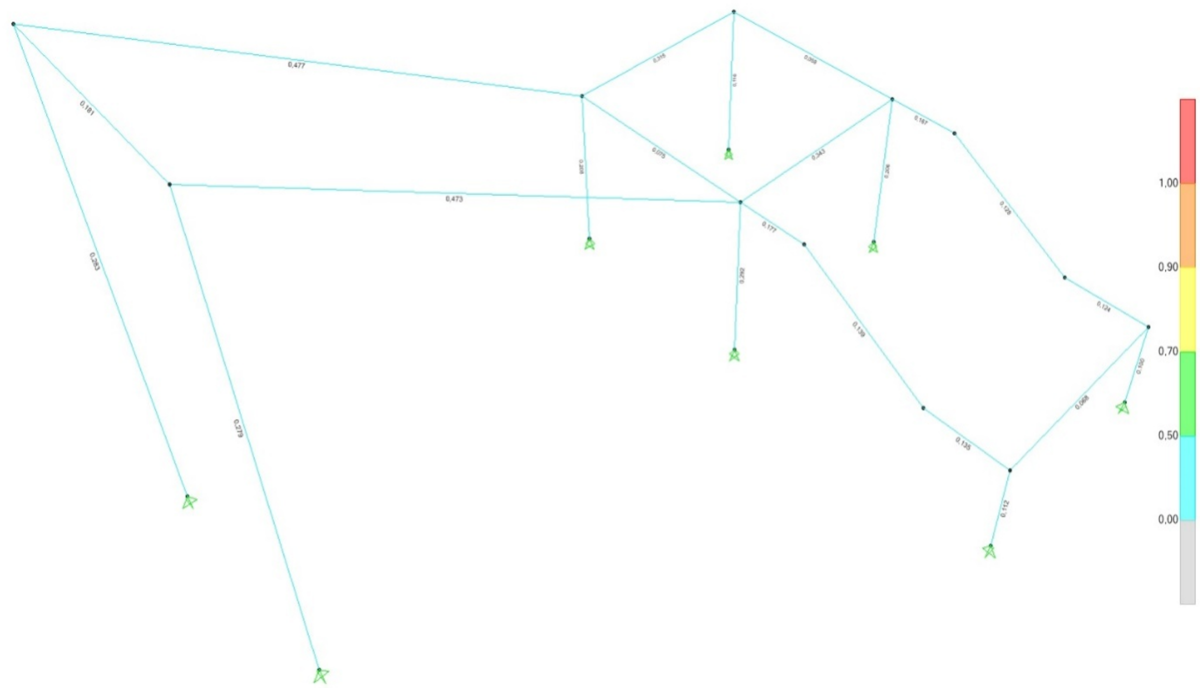


Figure 49. PORTION -3- Steel Design Results - Impact / Capacity Ratios

8.1.4. Steel Structure Control

Under the steel pedestrian platform, the vertical deflection limit L / 300 has been taken under the permanent and moving loads.

The displacement limit under the horizontal seismic force can increase by δ * R / H <0.02 ~ 0.03 (can increase by 50% in single-storey structures).

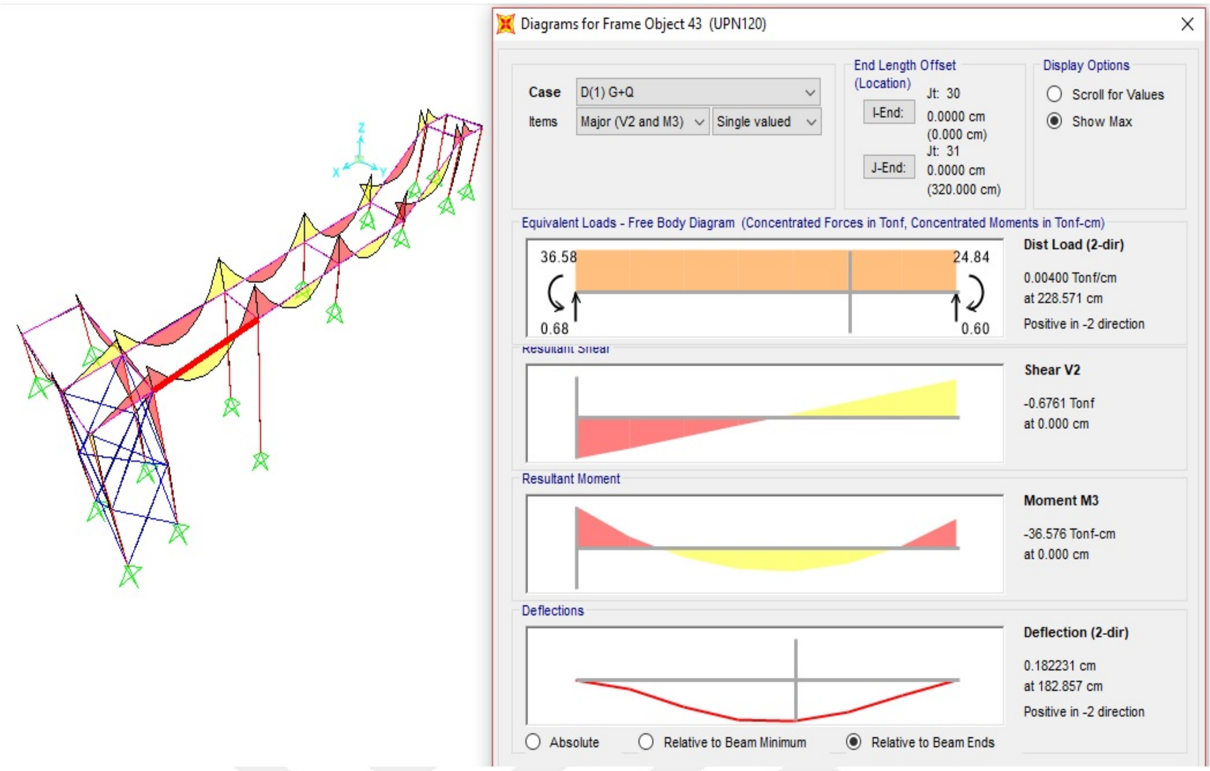


Figure 50. PORTION -1- Vertical displacement (G + Q) 0.18 cm <1.06 cm (320/300) OK

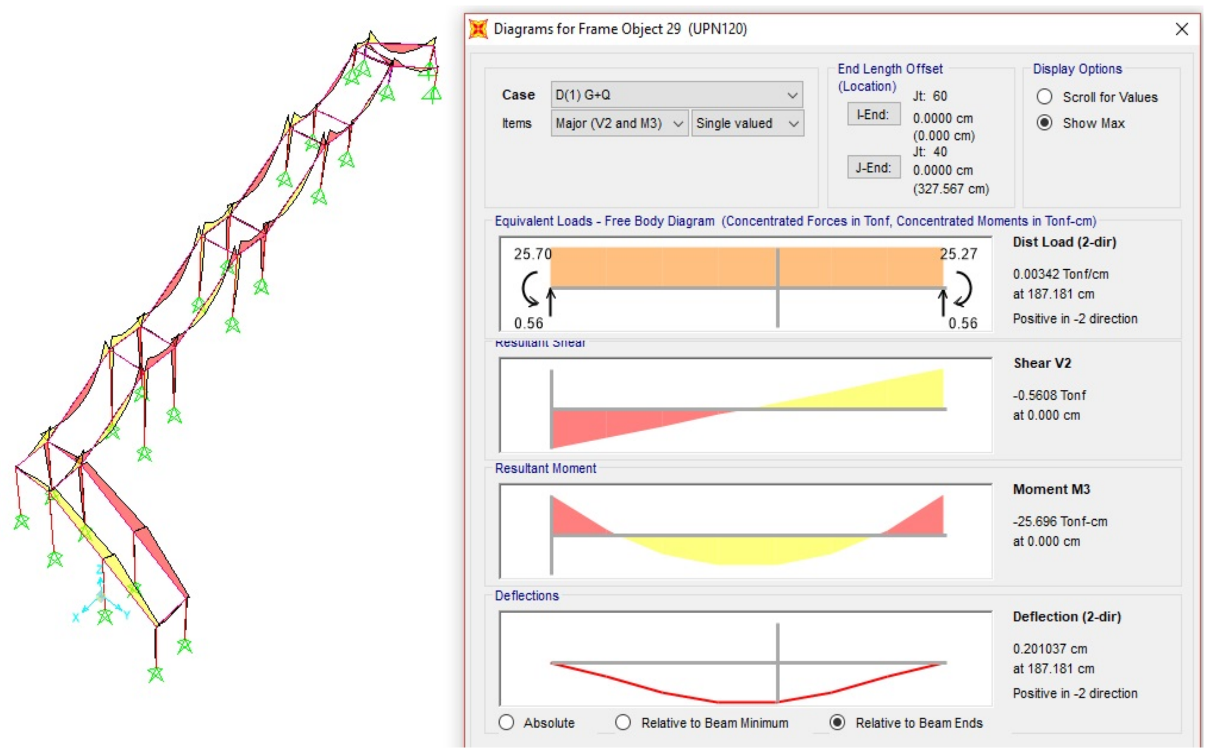


Figure 51. PORTION -2- Vertical Displacement (G + Q) 0.201 cm <1.09 cm (327/300) OK

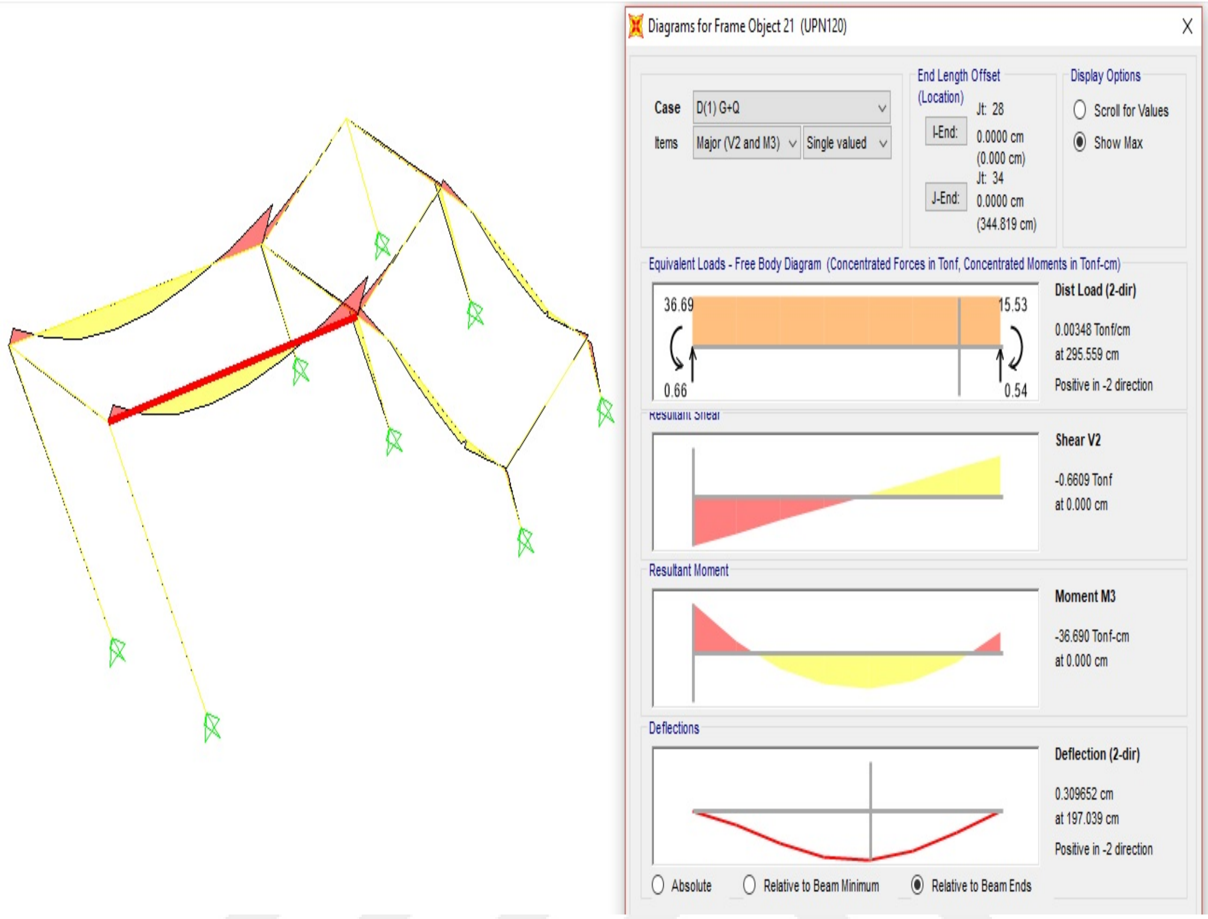


Figure 52. PORTION -3- Vertical Displacement (G + Q) 0.310 cm <1.15 cm (344/300) OK

8.1.5. Connection Accounts

Steel Column - Concrete Pin Connection

Column Section : RHS.100*4 b = 10 cm h = 10 cm

t = 0.4 cm A = 15.36 cm²

Steel Type : S235 Fy = 2.35 t/cm² Fu = 3.60 t/cm²

Concrete Class : C25 Fck = 0.25 t/cm²

Panaliz = 2.141 ton

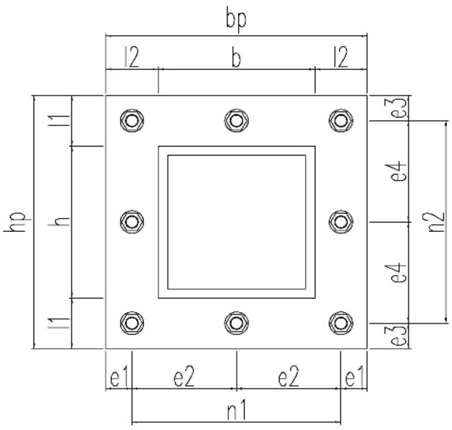
Vanaliz = 0.313 ton

Bulon Accounts:

Bulon diameter : M 12 Ag = 1.13 cm²

Bulon type : grade 4.6 Fbt = 3.00 t/cm²

Fny = 1.80 t/cm²



ϕPn,b = 0.75*Fny*Ag = 1.53 ton

nb, ger. = Vanaliz/ ϕPn,b = 0.21 adet

nb,seç. = 4 adet

hp = 20.0 cm hp = 20 cm

e1 = 1.5 cm e2 = 15.0 cm

e3 = 2.5 cm e4 = 15.0 cm

I1 = 5.0 cm I1 = 5.0 cm

Base Plate Accounts:

A1g=Panaliz/ϕ*0,85*Fck Alg = 15.50 cm² B = 20.0 cm

Reinforced concrete element dimensions: N = 20.0 cm Al = 400.00 cm²

Hb = 25 cm Bb = 25 cm A1g < A1 (1) A2 = 625.00 cm²

Pp = 0,85*Fck*A1*√(A2/A1) = 106.25 ton 1,7*Fck*A1 = 170 > Pp (1)

Ppd = ϕ*Pp = 69.06 ton Panaliz/Ppd = 0.031 < 1 (1)

Base Plate Dimensions are Suitable for Concrete Stapling.

$l=\sqrt{(l1^2+l2^2)}$ = 7.07 cm fpa=Pa/A1= 0.0054 t/cm²

tmin=l*$\sqrt{(2*fpa/0,9*Fy)}$ = 0.50 cm tseç. = 0.5 cm

1c = 1.80 cm

Rn1 = 1,2*lc*t*Fu = 3.888 ton Rn2 = 2,4*d*t*Fu = 5.184 ton

ϕRn = 2.916 ton Va/nb,seç. = 0.078 ton 0.78296 < 2.916 (1)

Selected Bulon Layout And Plate Thickness Suitable For Base plate! ...

- The base plate and bolts for pin connection can also be used for pipe section columns.
- Other column-beam, stability crosses and step profile connections; will be formed with full penetration welding in the field.

Combination Source Accounts

(Implementing Regulation on Design, Calculation and Construction of Steel Structures 2016 - Section 13.2)

All resources to be made will be made as full penetration.

<u>Steel Column - Base Plate Resources</u>

Vanaliz = 0,313 ton

Source type : E70xx F_e = 4,80 ton/cm²

a = 0,3 cm (Welding thickness)

L_w = 7,4 cm (Welding length on the axis of force(L_k-a))

L_w < 150*a = 150*0,3 = 45 cm

⇨ $\Phi L_e = L_w$

n_{we} = 2 (welding sequence)

⇨ $\Phi R_n = 0,75 * 0,6 * F_e * n_{we} * A_{we}$

= 69,05 ton

$\Phi R_n > V_{analiz}$ (Source Suitable …)

Steel Beam - Steel Column Connection Sources

Profile type : UPN120 → W_p =74,69 cm³

M_{analiz} = 56,75 ton.cm

M_n = $F_y * W_p$ = 175,52 ton.cm

M_{d1} = $(M_{analiz} + M_n)/2$ = 116,14 ton.cm

M_{d2} = $0,75 * M_n$ = 131,64 ton.cm

⇨ M_d = 131,64 ton.cm

⇨ P_d = M_d / D_p = 131,64 / 12 = 10,97 ton

Source type : E70xx F_e = 4,80 ton/cm²

a = 0,5 cm (Welding thickness)

L_w = 5,5 + 4,8 (Welding lengths in beam flange)

L_w = 10,3 cm (Welding length on the axis of force (L_k-a))

L_w < 150*a = 150*0,5 = 75 cm

⇨ ΦL_e = L_w

n_{we} = 1 (welding sequence)

⇨ ΦR_n = $0,75 * 0,6 * F_e * n_{we} * A_{we}$

= 11,12 ton

ΦR_n > Pd (Source Suitable …)

Steel Step Beam - Steel Beam Connection Sources

Profile type : UPN80 → W_p = 13,35 cm³

M_{analiz} = 2,6 ton.cm

M_n = $F_y * W_p$ = 31,37 ton.cm

M_{d1} = $(M_{analiz} + M_n)/2$ = 16,99 ton.cm

M_{d2} = $0,75 * M_n$ = 23,53 ton.cm

⇨ M_d = 23,53 ton.cm

⇨ $P_d = M_d / D_p = 23,53 / 4,5 = 5,23$ ton

Source type : E70xx $F_e = 4,80$ ton/cm²

$a = 0,4$ cm (Welding thickness)

$L_w = 8 + 6,4$ (Trunk welding lengths)

$L_w = 14,4$ cm (Welding length on the axis of force (L_k-a))

$L_w < 150*a = 150*0,4 = 60$ cm

⇨ $\Phi L_e = L_w$

$n_{we} = 1$ (welding sequence)

⇨ $\Phi R_n = 0,75 * 0,6 * F_e * n_{we} * A_{we}$

$= 12,44$ ton

$\Phi R_n > Pd$ (Source Suitable …)

<u>Steel Cross - Steel Column Connection Sources</u>

$P_d = 0,35$ yon

Source type: E70xx $F_e = 4,80$ ton/cm²

$a = 0,3$ cm (Welding thickness)

$L_w = 4$ cm (Welding length on the axis of force (L_k-a))

$L_w < 150*a = 150*0,3 = 45$ cm

⇨ $\Phi L_e = L_w$

$n_{we} = 4$ (welding sequence)

⇨ $\Phi R_n = 0,75 * 0,6 * F_e * n_{we} * A_{we}$

$= 10,368$ ton

$\Phi R_n > Pd$ (Source Suitable …)

9. RESULTS AND RECOMMENDATIONS FOR STRUCTURAL STRENGTHENING

In the light of the data obtained from structural analyzes, the order of intervention to the building parts can be described as follows;

1. The missing regions are those parts which will cause further demolition if the partial or complete completion of the parts described above is not done. Particularly long wall fragments that have been subjected to variable demolitions and lost parts by out-of-plane movements should be consolidated with this completion. It is thought that such demolitions will continue by destroying the remaining parts under any lateral effects. In order to achieve the joint movement of the building bushes or perpendicular walls on the horizontal plane, the original wall sections are reached (with completion) as much as possible in the thinned aperture sections as a result of the corner points of the horizontal loads and the collapses around them; The original wall horizontal load transfer habit should be restored to the structure. However, it is not necessary to complete the completion of the existing upper elevation levels if the internal rubble is not supported by the wall. In these parts, it is recommended to complete by using the original masonry technique (original construction techniques). However, in order to be able to recognize periodic intervention in future generations, it is possible to get away from the original knitting technique. for example, it is recommended to make it clear by using smoother cut stones. The rebuilding process in masonry rubble weave should be made with the original materials determined as a result of the material studies to be performed during the application in the structure. In the case of building openings, it is important to transfer the horizontal loads and transfer the vertical loads along the full wall sections and convey it to the foundation. As a matter of fact, as a result of the arch collapse along the wider aperture in the entrance gate region, the opening in this region has expanded over time. It is important to avoid this again and complete the original form for a healthy load transfer.

2. There was no crack formation in the building. However, structural cracks or formed capillary cracks, which were detected during the application; For the areas where the walls are located, it is recommended that the stones around the crack are replaced with longer or larger cutting stones (cutting the slab) so that the crack route is cut off,

thus eliminating the cracking route. In this way, the pieces of the walls separated by cracks will be combined with both interventions.

It is essential to add grub to the inner molos on the inner walls and to grind and suture the grindings on the outer walls in order to complete inertia or to ensure the inertia against the off-plane movements of the walls. In this way, there will be a friction-based relationship and strength between the inner rubble and the outer wall of the outer wall.

3. It should be remembered that wood beams contribute to this strength and unity. Changing woods as much as possible, it is very important to use new timber timber systems at new levels.

4. The main degradation prevailing throughout the building is the reduction of the original strength of the internal debris mortar (climatic effects, aging, climatic freezing due to the dissolution and wetting drying cycles) and the weakening of the relationship between the outer walls and the internal rubble. As a result of the attenuation, the walls destroyed by out-of-plane movement exposed the internal debris to the climatic negative effects. In this context, in order to strengthen the binding characteristic of internal rubble, the parts must be consolidated in order to maintain their position.

5. The change in saltiness caused by the weakening of the internal filler due to the weakness and internal structure of the inner filling caused the break of the bond between the walls. This backfill needs to be restored and the hollow structure must be changed to prevent water and salt transfer to the building materials. In this context, it is suggested that all of the building walls will be injected with a liquid mortar (no aggregate). Injection process; The original mortar on the inner and outer surfaces of the wall shall be made of holes 20-30 cm deep 1-2 cm in diameter corresponding to the joint parts. It is recommended that the injection be applied at a pressure of 1.5 m apart from vertical and horizontal points with a pressure of 1 bar. Through the plastic pipes placed in the holes, the aggregate-free original repair mortar (the fluidity of the water content is increased) will be injected in such a way that it does not exceed 1 bar pressure level. When injecting through a hole, it is recommended that the others be closed, and that the wall volume of the total injection applied (volume - shell volume corresponding to the injection depth) is not greater than 10%. It is thought that the injection to be applied above these levels does not contribute to the porosity of the wall which has already

reached saturation. The main purpose of strengthening the internal debris wall, which already has a thickness of 2 m, by injection is to support the solidified crust formations at a depth of 20-30 cm.

A computational representation of the depth of injection that can protect existing slopes can be found in the slope analysis section of the inserts.

6. In particular, joint restoration needs to be done in all of the structure, including the joints of the joints identified in the problems section of this report (and the river-side exterior surface walls to be held ahead). This process should be done after the injection process using joints. In the completion of the joints, the original mortar content determined by the material studies should be used during the application.

The wall fill mortars which have lost their strength due to vegetation and material degradation should be cleaned. At the same time, cement-containing mortars with a salt source should be removed. Climates where the walls of the towers become narrower into thinner sections and climatic elements acting in the wall should be destroyed in the upper elevations of the walls. Rubble stone elements with structural bearing weakness (inner and outer wall stones); should be replaced with stones by matching with original material..

7. After the completion of all processes, repairing the mortar lines on the building walls with the internal grouting of the walls is recommended at the top wall elevation. Grouting of joints will be possible by using pointing with repair mortar at about 5 cm depth of the joints. Thus, climatic effects such as rain snow will not penetrate into the interior areas of the wall. It is thought that the mortar joints will be used when the mortar joints will be used at the highest elevation and that the climatic effects will be prevented from reaching the structure internal fill. Keeping the structure away from any cement-containing repair material in the future and keeping it safe from salt effects will be one of the measures that will extend its life.

8. If it is not necessary to remove the pre-fill of the internal wall, it is recommended to terracotta. It is recommended that the wall is thickened at sections where the thickness is ~ 80 cm thick and then the wall is reinforced by injection. Because the outer wall of the wall with the present state of the mesh walls have been lost; the inner filler has emerged. In this form, it must remain in a holistic form to

maintain its retaining property. This can only be achieved by consolidating the original mortar binding wall; and it may be possible to repair the beam system that contributes to the binder. As a result of the restoration interventions of the wall as a result of the height and thickness of the pre-filling staggering stability control business is shown in this report and the project appendix.

9. These strengthening suggestions should be known to avoid further deformation of the structure; consolidation is aimed in order to preserve the present day. If the structure does not suffer more losses, these suggestions will be provided. However, since the internal debris of the structure is exposed and will be exposed to severe climatic activities and cycles, the consolidation of the debris against local losses or falling stones may only be achieved by continuous maintenance and repairs. !!! it may also show weakness after seismic and wind effects. The continuity of such repairs is essential. Especially within the scope of this project, the visitor routes are taken away from the places where there are visually destroyed or threatened parts falling in the internal debris. This principle must be taken into account during operation. Because, in order to ensure the preservation of the original structure of the building as much as possible, restitutive completions were avoided and the future generations were intended to be conveyed as much as possible today. Steel construction design calculations in the hiking routes can be found in the annex section.

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