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ZASNOVA IN STATIČNA ANALIZA LOČNE PREGRADE Z UPORABO PROGRAMSKEGA ORODJA SAP2000

Magistrsko delo št.:33/II.GR

DESIGN AND STATIC ANALYSIS OF ARCH DAM USING SOFTWARE SAP2000

Master Thesis No.: 33/II.GR

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Ljubljana, 20.08.2016

CORRECTIONS

Page of mistake	Line of mistake	Mistake	Replace by

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Izvleček

Predmet obdelave v magistrski nalogi je izdelava statične analize ločne pregrade. V analizi smo izbrali hipotetični primer ločne pregrade, ki smo jo zasnovali po priporočilih za načrtovanje ločnih pregrad, ki so v uporabi v Združenih državah Amerike. V prvem delu naloge so podane osnovne definicije in pojmi v povezavi z ločnimi pregradami. Sledi splošni opis osnovnih izhodišč in kriterijev za načrtovanje ločnih pregrad in izdelavo statično-stabilitetne analize. V drugem delu naloge je prikazan opis zasnove ločne pregrade na hipotetični lokaciji, ki po vseh projektnih parametrih ustreza postavitvi ločne pregrade. Sledi prikaz izdelave izhodiščnega modela v grafičnih okoljih AutoCad in SketchUp. Statična analiza je izdelana v programskem okolju SAP2000, ki temelji na metodi končnih elementov. Za analizo so bili uporabljeni že vgrajeni tridimenzionalni končni elementi. Materialni parametri so bili določeni na podlagi izbranih priporočenih vrednostih iz literature, ker z dejanskimi vrednostim nismo razpolagali. V sklopu statične analize smo obravnavali obtežni primer glavne obtežbe, ki jo predstavlja lastna teža pregrade in hidrostatični tlak zaradi akumulirane vode. V računu je upoštevanih več obtežnih kombinacij osnovnih dveh obtežb z največjo frekvenco pojavljanja v obdobju eksploatacije pregrade. V nalogi je obravnavan tudi primer vpliva osončenja na zračno lice pregrade - zgolj s predstavitvijo osnovnih podatkov o tipičnih porazdelitvah temperatur in temperaturnih spremembah v telesu pregrade. V računskem modelu smo upoštevali dva načina kontakta konstrukcije pregrade s temeljno podlago. Prvi način predvideva podprtje pregrade s togimi podporami, v drugem načinu pa je predpostavljeno, da je pregrada vpeta v elastično podlago, ki je v modelu opisana s ploskovno porazdeljenimi elastičnimi vzmetmi. Togost vzmeti je določena glede na privzete materialne lastnosti osnovne podlage. Rezultat statične analize so grafični prikazi pomikov in porazdelitve napetosti po telesu pregrade za izbrane obtežne primere in načina izvedbe kontakta pregrade s temeljno podlago.

BIBLIOGRAPHIC-DOCUMENTALISTIC INFORMATION AND ABSTRACT

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Abstract

This thesis presents the layout and the analysis of an arch dam.

The dam was layout and designed at an optional location. We followed two US manuals for design of arch dams and designed our dam accordingly. These two manuals are published by United States Army Corps of Engineer (USACE) and Interior Department of United States Bureau of Reclamation (USBR).

The thesis starts with some basic information about arch dams. A review of references which are used in thesis is provided in chapter two. The static analysis of arch dam and the description of the selection procedures are presented with a brief description about finite element method (FEM) and its application. Layout and initial modeling of the dam were done by two graphical software; AutoCad and SketchUp.

Analysis of the dam is based on FEM as implemented in SAP2000 software. 3D solid elements are used to model the dam. The material properties are defined based on typical values which are chosen on the recommendations found in literature.

Two load cases were studied; hydrostatic pressure and concrete self-weight and various load combinations were applied to the structure. Some basic description about the temperature distribution and temperature load are also discussed, but they are not included into the finite element method.

Special attention is dedicated to modeling of boundary conditions. In the first case the foundation is assumed to be rigid and in the second case elastic springs are used to model the foundation and part of dam body which is under abutment rocks. The stiffness of spring is defined on the bases of typical properties of foundation rocks.

The results of analysis are presented by diagrams of displacements and stresses. We show, compare and discuss the response of both models under various load combinations.

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For advices, assistance and their helps, I would like to thank my supervisor Doc. dr. Andrej Kryžanowski and co-supervisor Prof. dr. Dejan Zupan.

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1. Introduction

Arch dams are solid concrete dams that are constructed for water storage and power producing purposes in narrow canyons with steep abutments. All arch dams are curved in upstream direction. Unlike the gravity dams which mainly resist loads by their weight, the arch dams resist only part of the loads by their own weight and transfer the remaining hydrostatic and other loads to the canyon walls and dam foundation due to their curved geometry.

The dam which is under study in this thesis is based on optional data. Location, water reservoir data, climate condition and other necessary data for dam layout and design were assumed. Typical values were chose for defining the materials properties.

The main goal here is to evaluate stress distribution over the dam as well as deflection of the dam. Two methods for modeling the boundary conditions (contact area between the dam and the foundation) were considered. In first case the dam is assumed to be rigidly supported and in the second case elastic springs were used for modeling the contact area between the dam and the abutments.

We will first gather some general information about arch dams.

Arch dams can be classified based on their thickness (b) as well as their curvatures and height (h). Arch dams are divided into three categories based on their thickness; thin, medium thick and thick arch dams. An arch dam with b/h ratio 0.2 or less is thin; ratio 0.2 to 0.4 is for medium thick and higher for thick arch dams. It should be noted that curved dam with b/h ratio more than 0.65 is classified as curved gravity dam in new classification methods. (4) Here b denoted the base thickness of cantilever and h is dam height from highest elevation to base.

Arch dams can be low, medium high and high based on their height. Dams with less than 30 m are low, 30 to 91m medium high and higher than 90 m are high arch dams.

An arch dam can be classified based on its curvature as single curved when it is curved about one axis only, and double curved when it is curved in two directions.

Some terms that will be used frequently in this thesis are defined as follow: (All definitions are taken from USACE, EM 1110-2-2201)

A section in dam can be an arch or a cantilever section:

Arch (Arch unit): A portion of dam bounded by two horizontal planes with one unit thickness.

Cantilever (Cantilever unit): A portion of dam contained between two vertical radial planes, 1 unit apart.

Extrados and intrados: Are terms which will be used for upstream and downstream of dam respectively in horizontal plane.

Crown cantilever: Maximum high vertical cantilever which is always located in the streambed.

Reference plane: Is a vertical radial plane usually based on the streambed. A reference plane contains the crown cantilever and shows the location of centers of arches.

Line of centers: Is a line in space that shows the location of circular arc centers which are used to define the dam faces. An arch dam can have one or more centers. A single centered dam is dam that has one set of centers lines on the reference plane.

Crest: Is the top of dam.

Site shape: A dam site can have U or V shape. Each of these shapes then classify to narrow and wide shapes. The classification of dam site is based on the crest length to high ratio (cl: h) of dam site.

Narrow-V is a site with cl: h ratio equal or less than 2:1. The streambed is narrow and the canyon wall is almost straight. This type of site is perfect for arch dam, because the action on the dam can be easily transferred to canyon walls by arch action.

Wide-V site would have cl: h ratio equal or more than 5:1. This type of arch dams should be thicker than the one in narrow-V sites.

In Narrow-U site the canyon walls are near to vertical in the upper half of the canyon. The streambed is wide and loads will be resisted by cantilever action toward the lowest point as well as by the arch action.

And finally Wide-U sites which are most difficult sites for arch dam, because most of arches will be long and cannot transfer load by their arch action. The loads will be mostly carrying by cantilever sections. These canyons are suitable for a gravity arch dam.

2. Literature review

In the thesis we used as main references two US Government Manuals that were prepared for the design of arch dams. USBR and USACE both introduced manuals for the design of arch dams in 1977 and 1994 respectively. USACE EM 1110-2-2201, which is our main reference, presents some general information and definitions of arch dams. The second chapter of manual includes some consideration for the design of arch dams. Third chapter is considered to introduce different parts of arch dams. Next chapter present loads and their combination in arch dam design. Fifth chapter introduces method to layout an arch dam. Latter chapters provide information about static, earthquake and temperature analysis of arch dam. Structural properties of dams and foundation study are followed latter. The last three chapters are providing information about analysis criteria, and construction instruments and methods.

It should be mentioned that following sections in this chapter are referred to both US manuals for arch dam design, and if any other reference is used it will be mentioned.

2.1. Design Consideration

To design an arch dam there are many issues that should be considered properly. Studying of all these issues in details are beyound the scope of this thesis, but some of the most important will be discussed here.

The load resisting of arch dam stem from transferring the load to dam abutment as well as by their self-weight, therefore the site geometry of dam is utmost important in selection of arch dam. A narrow valley or canyon with abutments with sufficient strength is most favorable site for an arch concrete dam. Best canyon for an arch dam is a canyon with length-height ratio of 3 or less. For ratio greater than 4 many serious factors shall be considered.

Another important consideration is to provide arch dam profile as smooth as practicable. The abutment surface naturally can have many irregularities, but with the excavation these irregularities should be removed. Also the weathering surface should be removed to reach the sound rock for dam foundation.

Arch dam contact angle with the abutment is also critical, because the contact surface should be capable to transfer the loads to abutments. EM 1110-2-2201 suggests the value of α (shown in figure 2.1), which is the angle between the average rock contours and the resultant stress in dam, should be greater than 30 degrees. Smaller value can cause concentration of shear stress near the rock surface and can be unsafe. This value can be obtained after stress analysis, but a value of greater than 40 for angle β during the primary design can help to reach a reasonable α angle.

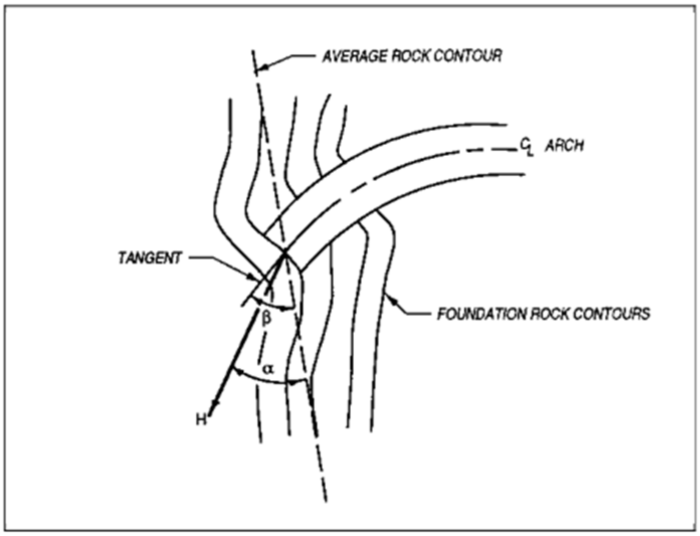


Figure 2.1: Contact angle with the abutment [1]

Arch dams have smaller dam-foundation contact area than other dams, so they insert a larger bearing pressure on the foundation. An arch dam requires a competent rock foundation and abutment with sufficient strength to withstand the imposed load of reservoir.

Deformation behavior of foundation has a direct effect on the stresses within the dam. A lower foundation modulus will reduce stresses at the base of dam, and higher modulus causes less foundation yielding and higher stresses along the dam base. The information about deformation modulus of foundation can be more critical when it is found that deformation modulus is different for various parts of abutments.

An important step is to study the local conditions, which yeilds the costs and and the choise of the method of construction and dam latter effects on nature and human living.

Geometrical survey, map and topography of area are also important. Final loaction, geometry, type of dam and some other dam parameters are related to dam site geometry. It is noticeble that the topography of site should be extended at least 150 m to both sides of estimated location of the dam.

Another important step is to prepare the hydrological data. Streamflow records, discharge, flood studies, water requirement for project, sedimentation and water quality, ground water data and other hydrological studies should be done for the design of a dam.

The design of a dam and the reservoir criteria also depend on reservoir capacity and anticipated reservoir conditions. Spillway and outlet works are also strongly related to reservoir conditions and capacity.

By studying the reservoir conditions we shall find these criteria:

Maximum water level, flood exclusive control level, top of active conservation capacity, top of dead capacity and streambed level.

Each of these elevations is depended on a capacity which is defined by the Technical Dictionary on Dams (ICOLD) [10] as follows:

1- Surcharge capacity: The volume of reservoir between the retention water level (normal operating level) and the maximum water level. This capacity flow over the spillway until the retention water level is reached.
2- Gross (total) capacity: This is capacity of the reservoir from the streambed up to the retention water level. It includes both live and dead storages of the reservoir.
3- Live capacity: This capacity includes both active and inactive capacities. The active capacity is the volume of water which is available for use. Inactive capacity is the volume of water between the invert level of the lowest outlet and the minimum operating level.
4- Dead capacity: This is volume of the reservoir below the invert level of lowest outlet.

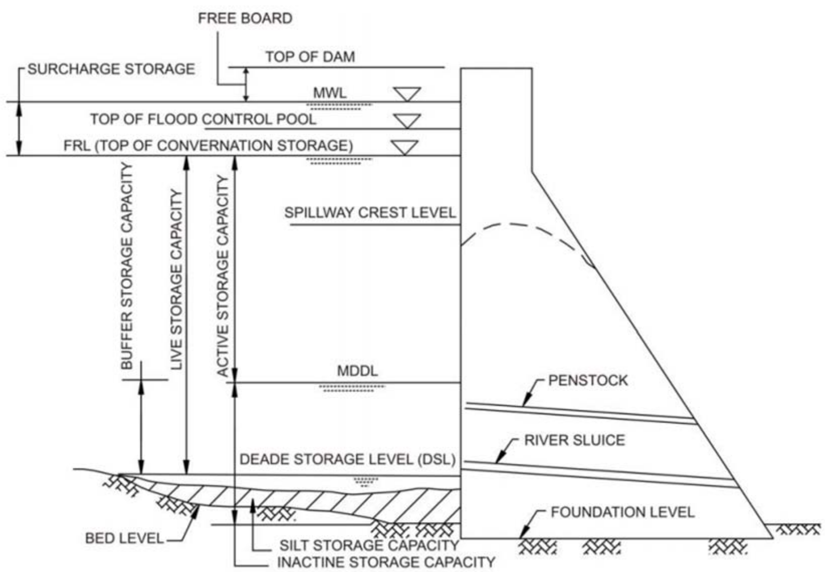


Figure 2.2: Capacity zones of dam [3]

Climate condition is another important issue that should be studied before the final design of a dam.

Site selection of a dam should be done by considering some factors like topography, geology, access, and local condition of area. For an arch dam narrow site is favorable and the abutment should also be massive to resist the acted loads on dam. Also the foundation of a dam should be free of fault and shears. Local conditions such as presence of roads, railways, canals, etc., should also be considered in the design of a dam.

2.2. Spillways and outlets

Information about dam spillway, outlet work and appurtenances is not included in this thesis. These data are provided by many references as well as USACE Design of Arch Dam Manual. The third chapter of manual is dedicated to these structural parts.

In this thesis we focus on the static analysis of an arch dam. In static analysis, effect of spillway and other voids in self-weight and hydrostatic analysis can be neglected, except for large spillway or tunnel spillway which should be analyzed separately. Generally the weight of spillway is much smaller than the dam body and that it is not need to be take into account.

2.3. Design layout

The design of an arch dam is an iterative process. Four steps are included in the preliminary design:

1) layout
2) analysis
3) evaluation
4) modification.

The first step is to prepare a layout of tentative shape of arch dam due to some given data and assumptions. The previous dams design can also be good references to choose the dam layout based on the site condition.

The next step is to provide a preliminary static analysis on the prepared layout. Then the results of strain and stress distribution over the dam should be evaluated and checked.

The last step in first phase is to redefine the layout based on the static analysis.

This process will be repeated to reach an acceptable layout of the dam shape, and only then the final analysis under the imposed load, temperature and dynamic loads will be done.

USBR provided the latest method for a layout of an arch dam, which is based on the 1966 Engineering Monograph (EM) NO.36. The layout method, which is briefly described below, is taken from USBR EM36-86-68110-2012-01.

The following primary data are needed to start the procedure for the layout of an arch dam:

The structural height of a dam (H), which is the high of a dam from the elevation below the streambed on sound rock to the crest elevation of a dam;
The cross canyon distance between the ground surfaces of abutment (L1);
Distance between ground surfaces of abutment at the elevation of 0.15H (L2), this is the elevation where the arching action starts to dominate the shearing resistance.

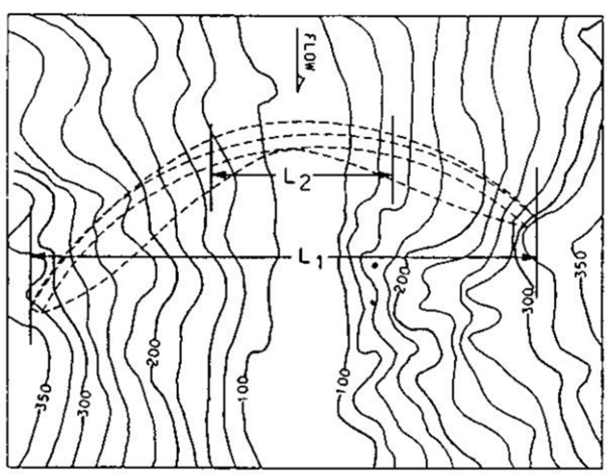


Figure 2.3: Determination of empirical values L1 and L2 [1]

With H, L1, and L2 in hand we can assume the dam thickness, upstream and downstream faces projections at the crest, at 0.45H and at the base of the dam. Also the circular arches through 3 points can be estimated.

Then we can determine the axis radius Raxis which is the distance between the crest of dam and the line of centers.

The location of the line of centers and the shape of crown cantilever will be determined by the given data and after that arches at various elevations can be determined.

The complete procedure for layout of arch dam is described in the next chapters for the dam under our consideration.

2.4. Materials

Concrete: Concrete is the main material of an arch dam. Strength, durability, elastic properties, thermal properties, poison's ratio, unit weight and other properties of concrete shall be considered properly.

Concrete strength should be determined and tested at an early age during construction and specific age that designer can assign. Also the tensile strength of concrete should be determined.

Temperature effects on dam are strongly related to thermal properties of concrete such as thermal expansion and conductivity.

At preliminary state, where the type of concrete is not yet chosen, some typical values can be used as follows:
Specific compressive strength: 21 to 34.5 MPa
Tensile strength: 4 to 6 percent of compressive strength.
Concrete cohesion: 10 percent of compressive strength.
Coefficient of internal friction: 1
Modulus of elasticity for static analysis: 21000 MPa
Modulus of elasticity for dynamic analysis or short time load: 34500 MPa
Poison ratio: 0.2

It should be noted that the modulus elasticity of concrete, which is used in dam design, is sustained modulus of elasticity. This modulus is used in static analysis to take into account the effect of creeps in dam. The value can be found by standard test or by using reduced instantaneous modulus (60 to 70 percent of modulus).

Foundation: The foundation is crucial for stress and strain distribution over the dam. The foundation deformation behavior shall be determined over all contact area with the dam. The elastic modulus and deformation modulus of foundation shall be evaluated. (Deformation modulus is the ratio of stress to the elastic plus inelastic strain).

In modeling the foundation we should use the deformation modulus of foundation instead of elastic modulus, to consider the effect of joints, shears, and faults. Deformation modulus can be obtained from in-situ test or estimated by reducing the elastic modulus of the rock using a reduction factor.

Foundation always contains variable materials; therefore the investigation of foundation should consider properly the composite behavior of these materials. Of course, more uniform and homogeneous foundation needs less complex investigation. Deformation of foundation that has the following criteria can be evaluated by stress-strain relationship:

a) Presence of only one or two types of rock,
b) Closely spaced and regular patterns between rock, that a jack test can evaluate the deformability of site, (Jack test is used to determine the mass deformability of rock in design of a large project)
c) Presence of no major low modulus zone.

For the site which doesn't exhibit above properties more complex investigation should be done.

The compressive strength of foundation is its most important property. Tensile strength of foundation is rarely needed, because the unhealed joints and discontinuity of rock prevent transferring tensile stress.

Shear resistance within the foundation and contact area between dam and foundation is related to the cohesion and internal friction of foundation materials, or concrete rock contact of dam and foundation. The formula which is suggested to find the shear resistance is:

$R = CA + N \tan\emptyset$

Where:

R = shear resistance,
C = unit cohesion,
A = contact area,
N = Normal force,
$\tan\emptyset$ = tangent of angle of friction

Laboratory and site tests show that this linear relation between shear resistance and normal force is realistic for most of intact rocks.

In our analysis the first model assumes the dam body to be fixed to the rigid foundation. In the second model the elastic modulus of foundation was estimated, and based on that value the foundation was modeled with elastic springs.

2.5. Loading

Several loads effect on dam. In additional to dam self-weight and hydrostatic load, temperature load, earthquake load and miscellaneous loads should be considered. The static analysis here is limited to influence of water reservoir and self-weight of the dam.

Dead load: The dead load is due to self-weight and appurtenant structures. In static analysis appurtenant weight can be neglected, because this weight is very small compare to the weight of concrete body of the dam. Sometimes appurtenant influence is significant and needs to be taken into account. Some of them such as large spillway openings should be considered in a model of the dam geometry. If spillway is designed to be tunnel, it should be analyzed separately. Other large appurtenant are usually not modeled as a part of finite element mesh, but still considered as concentrated loads applied to the nodal points.

Temperature loads: In contrast to gravity dams, temperature has significant effects on arch dams. The reference temperature for dam design is the closure temperature. This term is related to mean temperature of concrete during the grouting of contraction joint between concrete blocks. Actually this is the time that the dam is assumed to be monolithic body and the arch action begins. In an arch dam temperature load results from the difference between closure temperature and concrete temperature during its operation. To take into account this temperature we consider the temperature during grouting as free stress temperature in dam. It means as long as temperature of concrete in a dam remains the same as grouting temperature there will be no temperature influences in the dam. If concrete temperature increases more than closure temperature then it causes compression and the dam will deflect to upstream face, if temperature decreases less than closure temperature then tension stresses will occur and the dam will defect into downstream side.

The temperature of concrete in a dam is depended on air temperature and water reservoir temperature. The concrete and reservoir temperatures vary along the dam elevation as well as in upstream and downstream faces. Temperature is lower at the base and higher on crest. Air temperature in all months of the year shall be determined. Shading and sun radiation also effect on concrete temperature of the dam. Pavel Žvanut studied the heat transfer through the dam concrete for an arch gravity dam in Moste, Slovenia. (5) The paper describes the effect of changing surrounding condition in thermal analysis of an arch gravity dam.

Chapter 8 of USACE EM 1110-2-2201 also describes the conditions of temperature in an arch dam.

Temperature analysis of concrete structures shows that temperature distribution in a dam body is related to the thickness of the dam. For the dam with thin thickness the temperature distribution is approximately linear from the water reservoir temperature at upstream face to the air temperature on the downstream face. Distribution of temperature in relative thick dam is different and middle part of the dam is not so sensitive to the boundary temperature. The concrete temperature near to the contact boundary is depended on the adjacent temperature, but the middle part of the dam experiences almost constant temperature distribution.

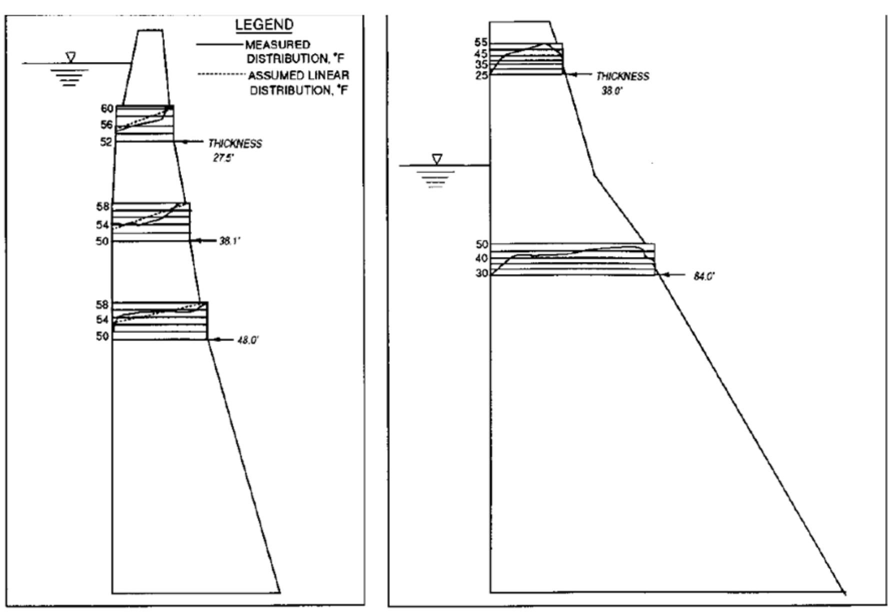


Figure 2.4: Measured temperatures in a dam for a thin and a relatively thick dam [1]

In this thesis the temperature load analysis was not included and it can be done by further studies.

Hydrostatic load: Theses loads are based on reservoir pressure on dam faces. In gravity dams the higher reservoir level always causes higher stresses, but for an arch dam a lower level of reservoir water can produce high tensile stress on the downstream face. Therefore, arch dams should be analyzed for both maximum and minimum water level.

Other important effects we need to study are the frequency, duration and time of the year when different level of water occurs. Based on these data we can also estimate the temperature of concrete and use a correct combination of hydrostatic load and temperature.

Earthquake loads: Earthquake analysis of a dam is based on two scenarios, Operational Basis Earthquake (OBE) and Maximum Design Earthquake (MDE). OBE is defined as a ground motion with a 50 percent chance of exceedance in 100 years. During OBE it is assumed that dam has elastic behavior (continues monolithic action along entire volume). MDE is maximum level of ground motion for which the arch dam should be analyzed. This analysis allows nonlinear behavior of dam, which might result in a significant damage but not failure in term of life or economy losses. In this thesis the earthquake analysis of dam is not considered.

Miscellaneous loads: These loads are due to ice and silt in reservoir. For ice when no data is present a load of 72 KN per linear meter can be applied, along the axis-water contact.

Uplift loads: Another load that effects on a dam is the uplift load due to pressure from saturated ground below the dam foundation. This force is negligible for thin arch dams, but for thick concrete arch dam it should be considered. In this thesis we won't consider it.

Loading combination: two loads combinations are applied in arch dam design. The first one is a combination of all static loads and the second one considers also the earthquake loads. Load combinations on an arch dam are divided based on their probability of occurrence into usual, unusual and extreme load cases. Because each dam is a unique structure the choice of load combination in a dam is depended on dam properties, climate condition, purpose of dam, spillway usage and other factors.

USACE EM 1110-2-2201 suggests some combination for static and dynamic loading combination as minimum loads that should be taken into account. In this thesis the dynamic load will not be considered, therefore the combinations for static loads are defined as follow:

Static Usual Loads Combination (SU)	
SU1	Minimum usual concrete temperature Reservoir elevation occurring at that time Dead load
SU2	Maximum usual concrete temperature Reservoir elevation occurring at that time Dead load
SU3	Normal operating reservoir condition Concrete temperature occurring at that time Dead load
Static Unusual Loads combination (SUN)	
SUN1	Reservoir at spillway crest elevation Concrete temperature at that time Dead load

SUN2	Minimum design reservoir elevation Concrete temperature occurring that time Dead load
SUN3	End of construction condition Structure completed and empty reservoir Temperature load
Static Extreme Loads Combination (SE)	
SE1	Reservoir at parabolic maximum flood (PMF) elevation Concrete temperature occurring at that time Dead load

Table 2.1: Load combinations for static analysis [1]

It is very important to classify the type of load combination in the prior cases of design based on the dam performance and other considerations. Each category affects the dam design and shape specifically. The classification which was presented in table 2.1 is not always valid for all dams. For example the combination of maximum water level, temperature at that time and self-weight of dam for which was classified as unusual combination shall be classified as usual combination for an arch dam which is used only for power producing, because usually the level of water will remain high at that kind of dam.

The maximum and minimum operating water levels were studied here, as shown in figure A10 of Annex.

2.6. Analysis criteria

EM 1110-2-2201 presents the criteria for static and dynamic results. We will present some of these criteria for the stresses obtained by static analysis.

As it was mentioned in Section 2.5 three categories of loads combination are studied in the design of arch dam; usual, unusual and extreme combination.

The following table shows the allowable stresses result from different combination categories and safety factor for each case.

Terms/ Load combination	Static usual	Static unusual	Static extreme
$\boldsymbol{f_c}$	$f'_c/4$	$f'_c/2.5$	$f'_c/1.5$
$\boldsymbol{f_t}$	f'_t	f'_t	f'_t
$\boldsymbol{F.S_s}$	2	1.3	1.1

Table 2.2: Allowable stresses and safety factor [1]

Where:

f_c : Allowable compressive stress
f_t : Allowable tensile stress
f'_c : Design compressive stress ≥ 4000 Psi
f'_t : Design tensile stress
$F.S_s$: Factor of safety against sliding

2.7. Static analysis

The static analysis of an arch dam will be based on linear analysis and effects of nonlinear materials or geometrical behavior such as large displacements, strain softening, and cracks will not be considered in this method.

Yusof Ghanaat, in Theoretical manual for analysis of arch dams, described two methods for design of an arch dam. The first method is Trial Load Method, which has been traditionally used and the second is the Finite Element Method (FEM). Latter is employed here using the elements implemented in SAP2000 software.

The crucial idea of FEM is in the division of a body into smaller parts where the governing equations could be simplified. This simplification is strongly related to the choice of the discrete primary variables. We will comply here the three dimensional elements, where the primary unknowns are the nodal displacements. The strain and stress field is then obtained by post processing.

The general procedure of FEM can be described as follows:

1- Divide the structure into finite elements.
2- Prepare the equations at the element's level; evaluate the stiffness matrix of each element and the right hand side the load vector.
3- Assemble the equations at the structural level, form the structural stiffness matric and right hand side.
4- Define the boundary conditions.

The general structure equation after these four steps is:

$$f = k.u$$

Here:

k denotes the stiffness matrix of the structure, u is the vector of unknown nodal displacements, and f the vector of nodal external forces and moments.

5- Solve the governing equations for displacements at the structural level.
6- Calculate the strains and stresses from known displacements for each element.

The details on our implementation of the arch dam into finite element software is presented next.

2.7.1. Idealization and modeling

The first step is to define the model of a structure with sufficient accuracy with respect to the real structure. This step can be done by graphic interface in FEM software or with specialized graphic software. Our goal was to define a model very accurately; therefore the exact model was created in AutoCAD software and then exported to SAP2000 for static analysis.

2.7.2. Mesh generation

Next, we divided the geometry of the dam into a large number of finite elements. The result of this procedure is called the finite element mesh. Different density of meshes can be chosen. In general denser meshes are more accurate, but computationally more demanding. In SAP2000 software we used the implemented Solid elements to define our meshes. Each Solid element in SAP2000 is determined by 8 nodes at which the unknown displacements are sought.

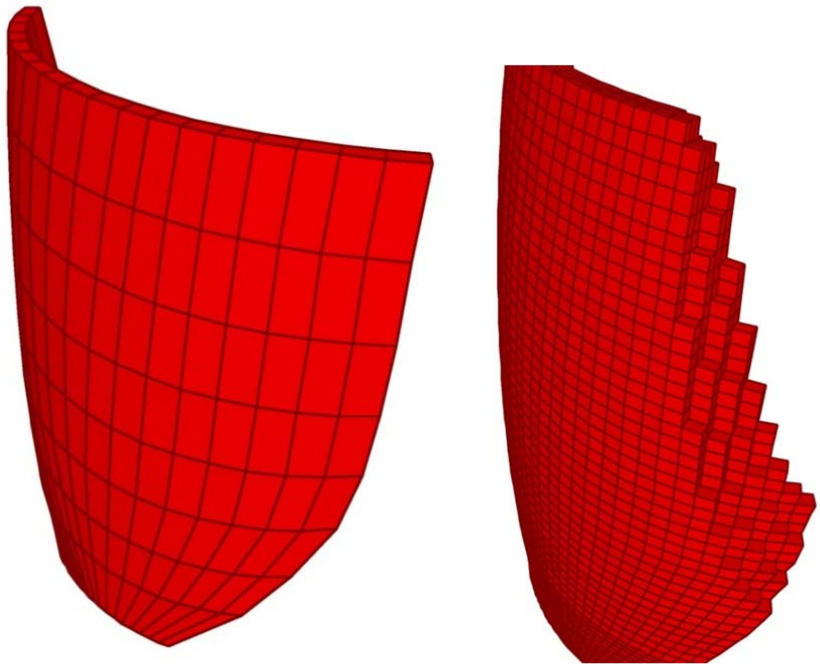


Figure 2.5: Left: 3D model of dam before discretization ; right: Solid element meshes on dam body

Reference manual of SAP2000 and Wiki.CSI define Solids as eight-node objects used to model 3D structural systems. "Each solid has six quadrilateral faces with a joint at each corner. Nodes may be collapsed to form wedges, tetrahedra, and other irregular volumes. An isoparametric formulation offers nine optional incompatible bending modes which improve bending behavior. Material, temperature-dependent and anisotropic properties may be assigned, and gravity loads, surface pressures, pore pressures, and thermal loads may be applied." [6]

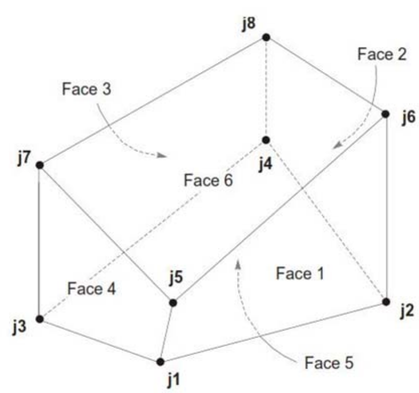


Figure 2.6: Solid element joint connectivity and face definition [7]

The local coordinate system at each solid is denoted by 1, 2 and 3 while X, Y and Z denote the global coordinate system. Both global and local coordinate systems in SAP2000 are right hand systems. It is also possible in SAP2000 to define a local axis in Advanced Local Axis menu or to relocate, rotate and replace axis. This property was used here to define the local coordinates that follow the curved geometry of the dam.

2.7.3. Evaluation

Some elementary information on the crucial steps for obtaining the results are presented. We will limit ourselves to isoparametric solid elements by implemented in SAP2000. The shape functions employed are spaced on four nodes, therefore the terms quadrilateral is used.

Only a brief overview will be presented here, more details can be found in textbook Theoretical manual for analysis of arch dams (Yusof Ghanaat) which provides detailed information about FEM applying in Arch dams and textbook Introduction to Finite Elements Method (Carlos A. Felippa).

2.7.3.1. Shape functions

The geometry of each element is defined by coordinates of its 8 nodes and the chosen shape function N_i. Following formula describes the arbitrary point on the element in term of nodal values.

$$\begin{bmatrix} 1 \\ x \\ y \\ z \end{bmatrix} = \begin{bmatrix} 1 & 1 & \dots & 1 \\ x_1 & x_2 & \dots & x_8 \\ y_1 & y_2 & \dots & y_8 \\ z_1 & z_2 & \dots & z_8 \end{bmatrix} = \begin{bmatrix} N_1 \\ N_2 \\ \vdots \\ N_8 \end{bmatrix} \qquad \text{(eq. 2.1)}$$

$$1 = \sum_{i=1}^{8} N_i \qquad x = \sum_{i=1}^{8} x_i N_i \qquad y = \sum_{i=1}^{8} y_i N_i \qquad z = \sum_{i=1}^{8} z_i N_i \qquad \text{(eq. 2.2)}$$

Here:
x, y and z are global coordinates of an arbitrary point within the element;
N_i is the i-th interpolation shape function;
and x_i, y_i and z_i are coordinates of the i-th node.

The interpolation shape function will be described by natural coordinate system as follow:

$$N_i = \frac{1}{8}(1 + \xi_i \xi)(1 + \eta_i \eta)(1 + \zeta_i \zeta) \qquad i = 1,2,3 \dots \dots 8 \qquad \text{(eq. 2.3)}$$

Where ξ_i, η_i and ζ_i are the coordinates of i-th node expressed in the natural coordinate system. See figure 2.7.

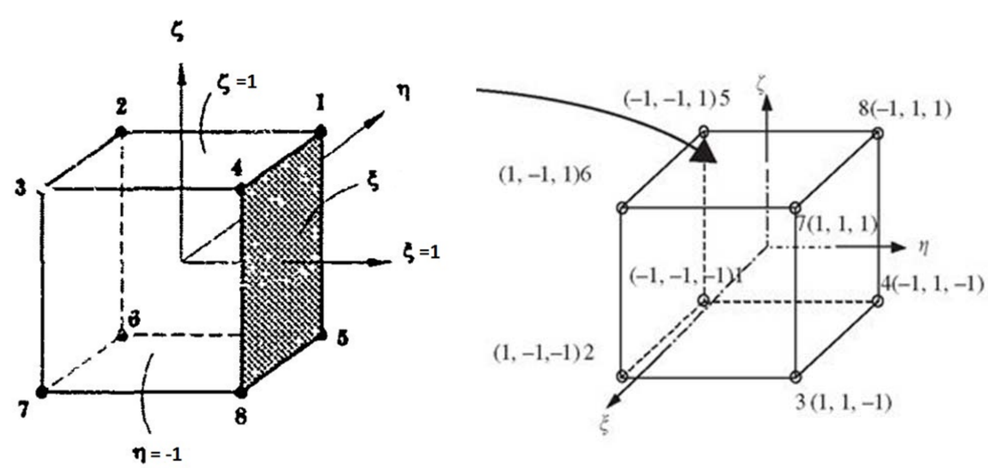


Figure 2.7: The natural coordinate system for 3D solid elements

2.7.3.2. Displacement field

Displacements of an element are also expressed with nodal displacement in the same manner as coordinates.

$$\begin{bmatrix} u_x \\ u_y \\ u_z \end{bmatrix} = \begin{bmatrix} u_{x1} & u_{x2} & \cdots & u_{xn} \\ u_{y1} & u_{y2} & \cdots & u_{yn} \\ u_{z1} & u_{z2} & \cdots & u_{zn} \end{bmatrix} \begin{bmatrix} N_1 \\ N_2 \\ \vdots \\ N_n \end{bmatrix} \quad \text{(eq. 2.4)}$$

$$u_x = \sum_{i=1}^{8} u_{xi} N_i \qquad u_y = \sum_{i=1}^{8} u_{yi} N_i \qquad u_z = \sum_{i=1}^{8} u_{zi} N_i \quad \text{(eq. 2.5)}$$

Here:
u_x, u_y and u_z are displacements in the global coordinate system,
N_i is the i-th interpolation shape function,
u_{xi}, u_{yi} and u_{zi} are displacements of i-th node.

2.7.3.3. Strain and stress equation

Strain and stresses are described by tensors, but for computational purpose it is often convenient to rearrange their components to six dimensional vectors.

$$e = \begin{bmatrix} e_{xx} \\ e_{yy} \\ e_{zz} \\ 2e_{xy} \\ 2e_{yz} \\ 2e_{xz} \end{bmatrix} \qquad \sigma = \begin{bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} \\ \sigma_{xy} \\ \sigma_{yz} \\ \sigma_{xz} \end{bmatrix} \quad \text{(eq. 2.6)}$$

Where:

e_{xx}, e_{yy}, e_{zz} are normal strains and e_{xy}, e_{yz}, e_{xz} denoted shear strains. Similarly $\sigma_{xx}, \sigma_{yy}, \sigma_{zz}$ denote normal stresses and $\sigma_{xy}, \sigma_{yz}, \sigma_{xz}$ the shear stresses.

When eq. 2.4 is considered in the kinematic equations of a solid body, we have:

$$\mathbf{e} = \begin{bmatrix} \frac{\partial u_x}{\partial x} \\ \frac{\partial u_y}{\partial y} \\ \frac{\partial u_z}{\partial z} \\ \frac{\partial u_y}{\partial x} + \frac{\partial u_x}{\partial y} \\ \vdots \end{bmatrix} = \begin{bmatrix} N_{x1} & 0 & 0 & \cdots & N_{xn} & 0 & 0 \\ 0 & N_{y1} & 0 & \cdots & 0 & N_{yn} & 0 \\ 0 & 0 & N_{z1} & \cdots & 0 & 0 & N_{zn} \\ N_{y1} & N_{x1} & 0 & \cdots & N_{yn} & N_{xn} & 0 \\ 0 & N_{z1} & N_{y1} & \cdots & 0 & N_{yn} & N_{xn} \\ N_{z1} & 0 & N_{x1} & \cdots & N_{zn} & 0 & N_{xn} \end{bmatrix} \begin{bmatrix} u_{x1} \\ u_{y1} \\ u_{z1} \\ \vdots \\ u_{xn} \\ u_{yn} \\ u_{zn} \end{bmatrix} = \mathbf{Bu}, \quad \text{(eq. 2.7)}$$

N_{xi}, N_{yi}, and N_{zi} are the partial derivatives of shape function N_i with respect to x, y and z.

Assuming the linear elastic material the relationship between stress and train reads:

$$\boldsymbol{\sigma} = \begin{bmatrix} \sigma_{xx} \\ \sigma_{yy} \\ \sigma_{zz} \\ \sigma_{xy} \\ \sigma_{yz} \\ \sigma_{zx} \end{bmatrix} = \begin{bmatrix} E_{11} & E_{12} & E_{13} & E_{14} & E_{15} & E_{16} \\ & E_{22} & E_{23} & E_{24} & E_{25} & E_{26} \\ & & E_{33} & E_{34} & E_{35} & E_{36} \\ & & & E_{44} & E_{45} & E_{46} \\ & & & & E_{55} & E_{56} \\ symm & & & & & E_{66} \end{bmatrix} \begin{bmatrix} e_{xx} \\ e_{yy} \\ e_{zz} \\ 2e_{xy} \\ 2e_{yz} \\ 2e_{zx} \end{bmatrix} = \mathbf{Ee}. \quad \text{(eq. 2.8)}$$

The equations Eij are depended on the material used.

The directions of the components of stress tensor is utmost important for their proper interpretation. They are shown in figure 2.8.

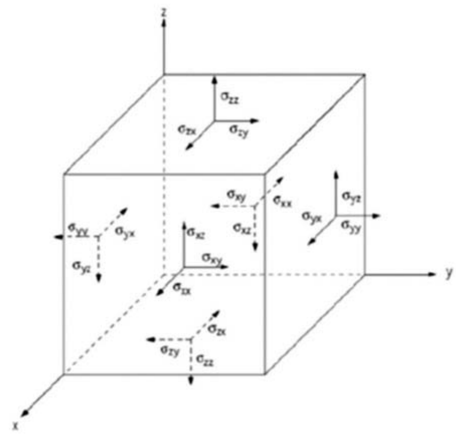


Figure 2.8: Stresses distribution in a 3D solid element

2.7.4. Modeling

It is very important that numerical model of the dam be capable to represent its real behavior accurately. Dams are 3D structures which are laid on a flexible foundation; therefore it is required to include the foundation rock into model or define such boundary conditions that represent the effect of foundation behavior on dam. EM1110-2-2201 suggests using the deformation modulus of foundation rock instead of elasticity modulus to consider the geological formation of rock in foundation. Following figure shows a typical model of dam and foundation.

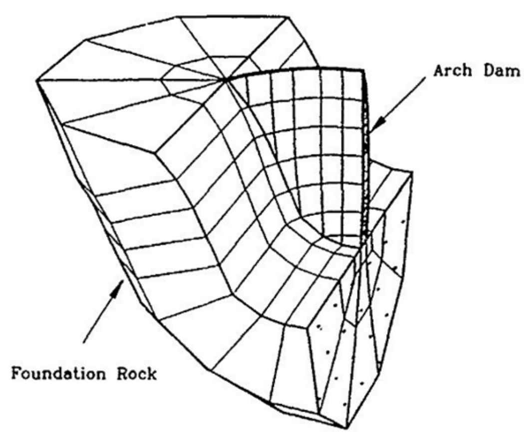


Figure 2.9: A typical model of dam and foundation [1]

The foundation flexibility has direct effect on the stresses and strain within the dam body. In should be mentioned that actual foundation is not completely uniform and there will be several zones with different deformation modulus. In modeling the foundation the variability of deformation and elasticity modulus shall be considered.

The following table presents the elasticity modulus of different materials of foundation. In this thesis we used 30000 MPa as modulus elasticity of foundation, which is used as average elasticity modulus for the overall rocks of foundation.

In our model two assumptions were used:

I) A rigid foundation,
II) An elastic foundation modeled with linear springs.

Moduli of elasticity for several rocks are shown in the table 2.2.

Rock Type	Modulus of Elasticity
-	(MPa x 1000)
Limestone	3-27
Dolomite	7-15
Limestone (very hard)	70
Sandstone	10-20
Quartz-sandstone	60-120
Greywacke	10-14
Siltstone	3-14
Gneiss - fine	9-13
Gneiss - coarse	13-23
Schist - Micaceous	21
Schist - Biotite	40
Schist - Granitic	10
Schist - Quartz	14
Granite - very altered	2
Granite - slightly altered	10-20
Granite - good	20-50
Quartzite - Micaceous	28
Quartzite - sound	50-80
Dolerite	70-100
Basalt	50
Andesite	20-50
Amphibolite	90

Table 2.3: Modulus elasticity for different rocks
(Durham University, UK)

The effect of openings in dam also should be considered. While spillways, outlets, and other openings are present in dam, the stresses tend to be excessive in the area near to these opening and this should be considered on final design of dam, but for static analysis when these opening are small the effect of them on overall structure behavior can be neglected.

3. Layout

To layout the dam there are some data which should be provided in primary studies. In this thesis data are not based on some actual values, but we followed the recommendations from the literature with typical quantities. The data which are required for the dam layout are listed below:

1- Topographic map of purposed location.
2- Geological data of the site. The rock location, type of material, depth of excavation to rock and other geological properties of site shall be cleared.
3- Reservoir information. Reservoir water elevation, reservoir capacity, surcharge flow amount, probable sediment amount and other information about reservoir should be estimated.
4- Location and size of dam spillway, outlet and other openings should be estimated.
5- Any available data about the dam which are located nearby or they are similar to purposed site should be taken into consideration.

The location which was chosen for dam is a hypothetical location. Topography of location was presented on a scaled JPG image format which is shown in Annex A.

The first step was to model the topography by graphical software. SketchUp is an easy usable software to model the site with a good precision. The software has the capability to export model to other graphic software such as AutoCad. The topography lines were drawn by free hand option in SketchUp and then transferred to their exact elevation. 3D model of the site was prepared.

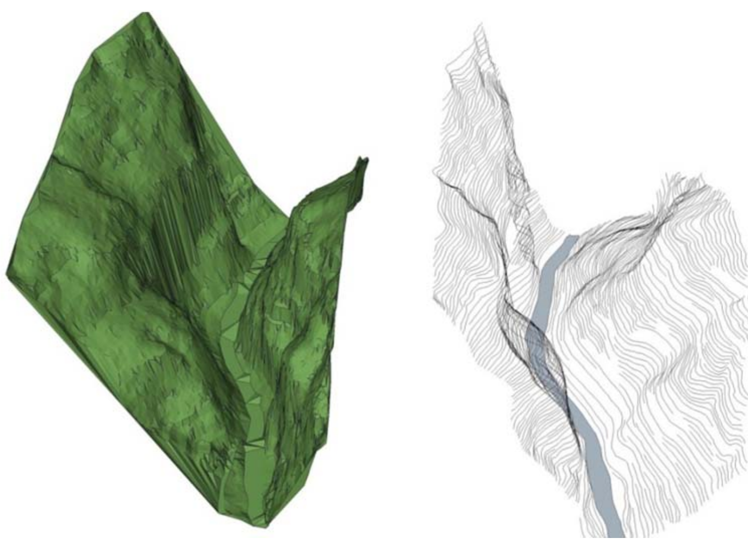


Figure 3.1: Site area 3D model (prepared by SketchUp Pro. Software)

Map topography was converted to terrain surface in 3D space with SketchUp software. In addition to site and material data, 3D model of topography enabled us to choose an appropriate location for the dam construction.

The figures show that canyon can be classified as narrow-V shape and is suitable for an arch dam design.

The location which was chosen for the dam is shown in figure 3.2. One of the important reason of this location is that abutment is almost vertical and smooth with rock near surface of ground that can resist the load on the dam.

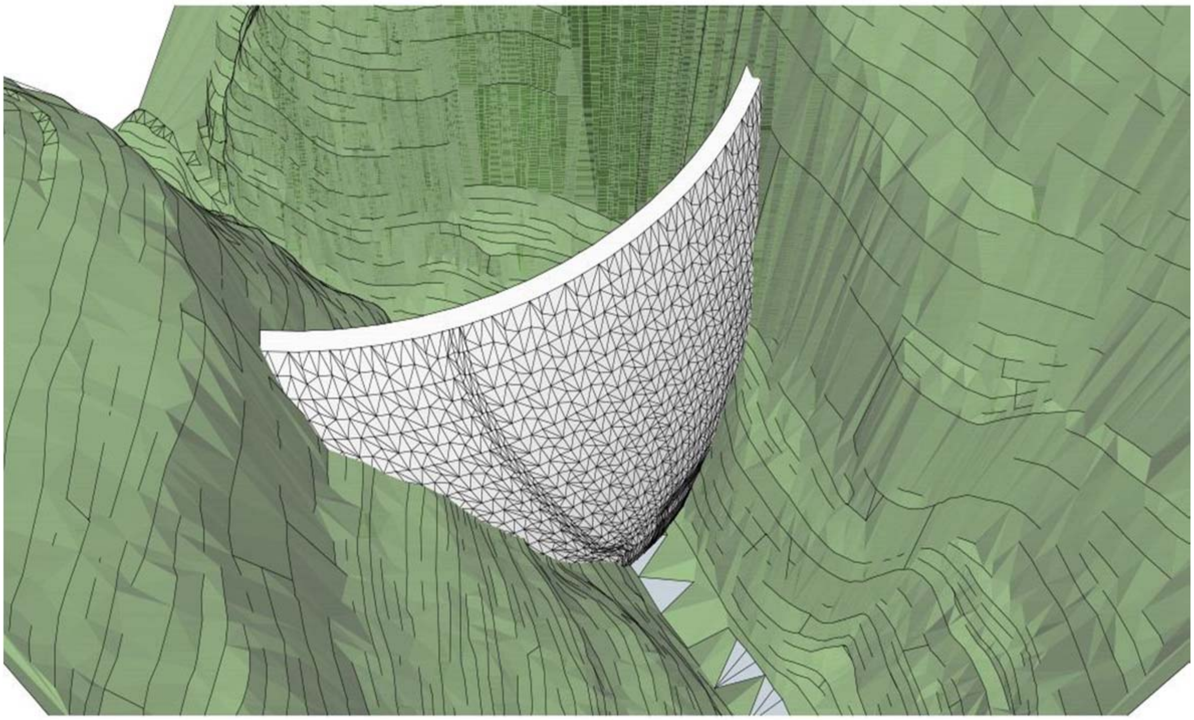


Figure 3.2: Dam in canyon (Prepared by SketchUp)

As it was mentioned, the design and the layout are iterative process, but here we will just discuss about the last layout that was prepared for final analysis.

The first step is to choose the type of an arch dam based on conditions. We chose a single-center, uniform thickness arch dam. It should be mentioned that canyon is not symmetric at the chosen location, so the dam will not have symmetry arcs and whole structure need to be modeled. The other important information in this part is the depth of sound rock. If it is not available at the primary analysis the designer can assume it at the primary layout and design.

Maximum water level from hydrological study was assumed to be at elevation 816 m, so we consider the crest elevation of dam to be 820 m. The streambed elevation is 665 m, so we assume the high of dam (H) with considering excavation and foundation pad as 166 m.

By having the dam location and assuming excavation to sound rock, we can find the straight line distance between to excavated abutments L1 at the crest elevation of dam and L2 at 0.15H elevation from base of dam.

H, L1 and L2 are required data to determine axis radius of dam and crown cantilever layout. These data were provided by having the site topography and choosing the dam location and properties.

H = 166m

L1 = 164m

L2 = 52.4m

The dam layout procedure is described in the following.

3.1. Determination of axis radius and the central angle

The first step is to draw the dam axis which is also the upstream face line of crest. Because the canyon is not symmetry, therefore the angle in right side and left side of reference plane is not equal. The axis arc will continue to contact point of sound rock in abutments. Based on USACE EM 1110-2-2201 suggestion the central angle should not be bigger than 120 degrees. USACE manual also suggest an empirical formula for the radius of axis.

Raxis = 0.6 L1 (eq. 3.1)

The above formula is just a guide for designer to choose the axis radius, but some other adjustments shall be done by designer to reach the appropriate value for a purposed site. Figure 3.3 shows the axis and the central angle for the chosen site.

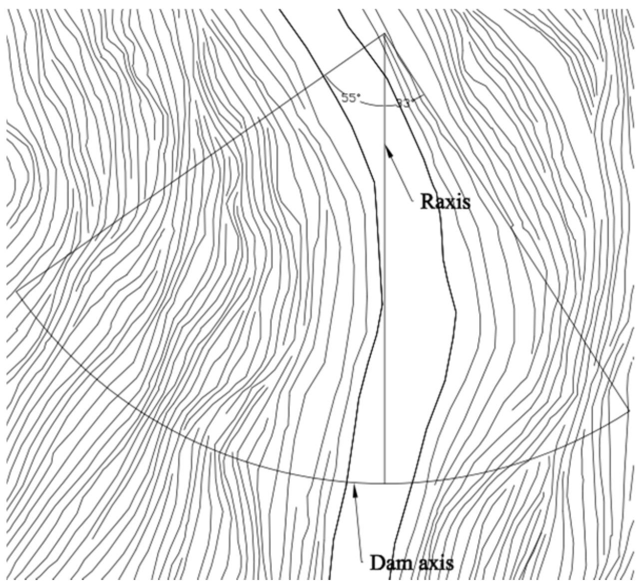


Figure 3.3: Dam axis and central angle

3.2. Crown cantilever and reference plane

Crown cantilever is located at the intersection of the dam axis and the lowest point on streambed. Reference plane also passes from this line and axis center. Centers of other arcs will also be determined in the reference plane. The centers of arches in the reference plane shall be along smooth curves or straight lines.

The crown cantilever geometry is the most important part of dam layout, because it will control the shape of entire dam and then the distribution and magnitude of stresses within the body. In defining the cantilever one important point which should be considered is the smoothness of both extrados and intrados faces of dam. An abrupt change in geometry of a dam can cause excessive stress concentration on that location.

Some suggestion of USACE manual for cantilever derivation is as follow: (all dimensions in formulas are in feet but then the result converted to meters)

Crest thickness; $T_C = 0.01\ (H + 1.2L_1)$ (eq. 3.2)

Base thickness; $T_B = \sqrt[3]{0.0012\ HL_1L_2(\frac{H}{400})^{\frac{H}{400}}}$ (eq. 3.3)

Thickness at 0.45H; $T_{0.45} = 0.95\ T_B$ (eq. 3.4)

These values can be used as initial assumptions for the dam layout. To provide access for vehicles on top of the dam, we considered a width of 5 m for crest and the other values were increased proportionally by this width. The final values which were used in this dam are:

$T_C = 5\ m, T_B = 22\ m, T_{0.45} = 15\ m.$

USACE also suggested values for projection of extrados and intrados faces at base, and at 0.45H.

$USP_{Crest} = 0.0$ (Upstream face distance to axis in crest)

$USP_{base} = 0.67\ T_B$ (Upstream face distance to axis in base)

$USP_{0.45H} = 0.95\ T_B$ (Upstream face distance to axis in elevation 0.45H)

$DSP_{crest} = T_C$ (Downstream face distance to axis in crest)

$DSP_{base} = 0.33\ T_B$ (Downstream face distance to axis in base)

$USP_{0.45H} = 0.0$ (Downstream face distance to axis in elevation 0.45H)

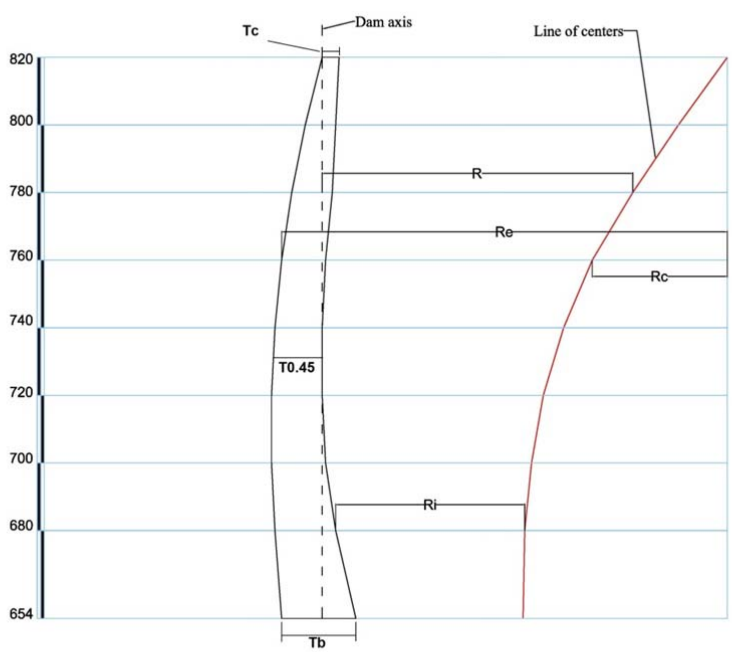


Figure 3.4: Crown cantilever geometry in reference plane

The next step is to define the upstream and downstream faces of dam. It is possible with drawing circular arc or combination of arcs and straight lines. These guides for drawing of crown cantilever were followed and the last geometry of crown cantilever was obtained as shown in figure 3.4.

3.3. Estimating the dam foot print

The axis and crown cantilever were layout in the last part, the intrados face of crest was drawn from the same center of axis with axis radius that reduced by crest thickness. In this thesis the extrados and intrados faces for each elevation were drawn from the same center.

The other parts of dam the can be design with several arcs. In this dam in addition to the crest arcs, we drew arcs for 8 other elevations for the extrados face of the dam. Arcs that we draw should meet three stated below conditions:

a) The arc center should be located along reference plane,
b) The arcs should pass the upstream face of cantilever at the crown cantilever which is already drawn,
c) Both ends of each arc should meet the foundation in the abutment elevation or deeper.

The intrados faces can then be drawn from the same center, with a radius that decreases by cantilever thickness at that elevation.

Because the intrados and extrados faces were drawn from same elevation the arch is an uniform thickness Figure 3.4 shows the crown cantilever geometry in the reference plane.

Now we can show the line of centers in the reference plane and plan view.

Three drawings of a dam should be prepared:

1- Drawing of the reference plane that includes also line of centers, (Shown in figure 3.4).
2- A plan view of the dam which shows the arches, (Shown in figure 3.5)
3- A profile of the dam that looking downstream of the axis shows site surface and foundation of the dam. This profile view is a developed view of the dam, rather than projection of upstream face of dam onto a flat plane. The profile view is used to estimate the amount excavation. (Shown in figure 3.5)

Table 3.1 presents the information on the layout of the cantilever and arch geometry.

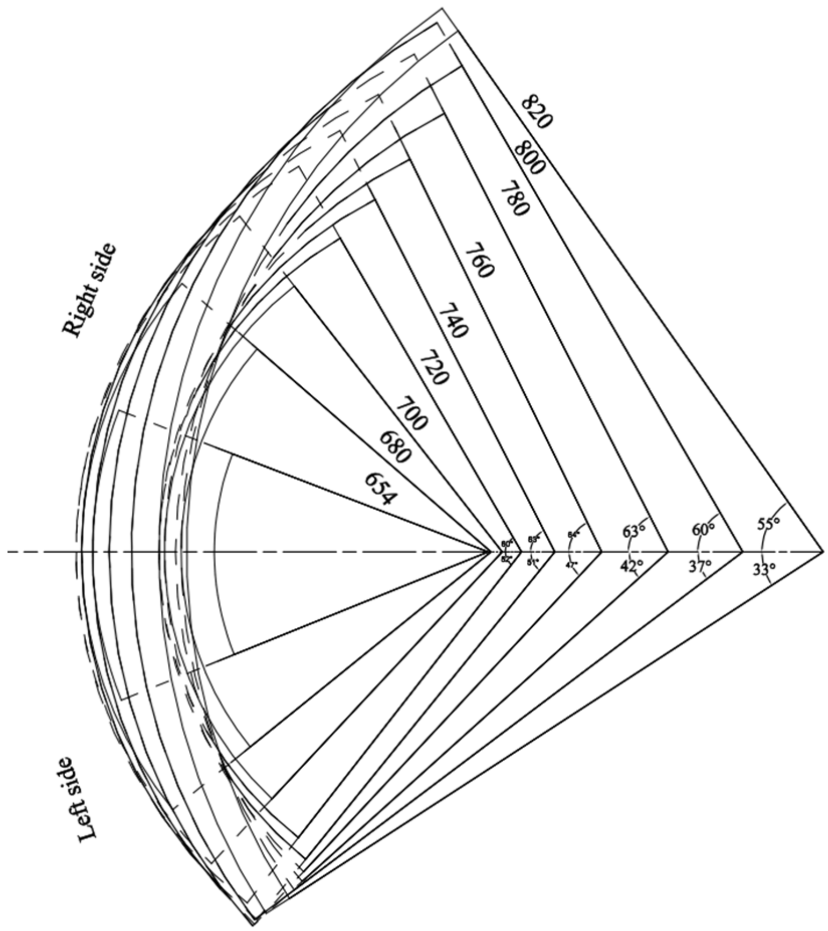


Figure 3.5: Plan view of the dam

All information about figure 3.4 and 3.5 are provided in the following table:

Elev.	Re [m]	Ri [m]	Rc [m]	$\emptyset_{right}$	$\emptyset_{left}$	R[m]
820	120	115	0	55	33	120
800	125	116	14.5	60	37	105.5
780	129	117	28	63	42	92
760	132	119.5	40	64	47	81
740	134	120	48.5	63	51	71.5
720	135	120	54.5	60	52	65.5
700	135	119	58	52	47	62
680	134	116	60	41	39	61
654	132	110	60.5	21	22	59.5

Table 3.1: Crown cantilever and arch geometry parameters

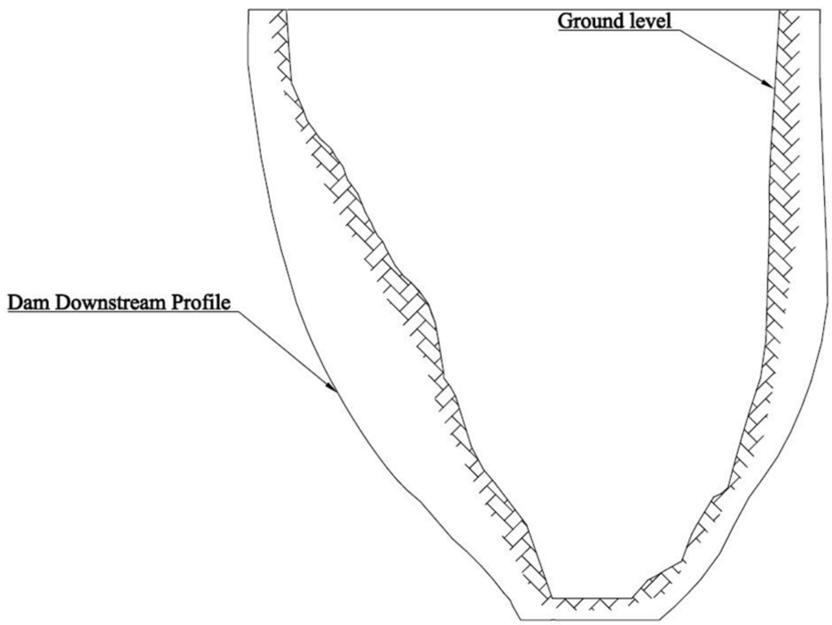


Figure 3.6: Profile view of the dam and the foundation rock

4. Modeling

4.1. Dam structure

The numerical analysis was done using SAP2000 software. As it was mentioned in previous chapters, spillway and other openings on the dams were not considered in static analysis and therefore they were not included in the model. Modeling and analysis of an arch dam by SAP2000 has two major challenges, one is the choice of the model and another is the result representing.

SAP2000 doesn't have any specialized template for arch dams. Arch dam has variable thickness in elevation and of course different centers of arches and radius for different elevation. Therefore, modeling of such a demanding geometry with the tools in SAP2000 is a heavy task. Besides that, SAP2000 software doesn't refine the model into meshes automatically. Therefore in modeling and discretization steps we need to construct model and finite elements meshes manually.

To avoid these problems we used AutoCAD software to model the dam and then exported the geometry into SAP2000 environment.

SAP2000 is capable to import data from AutoCAD as shown in the following table:

	DWG entity	SAP2000 object
1	Point	Joint
2	Line	Frame
3	Point	Link (one-point)
4	Line	Link (two-point)
5	3D Face	Shell
6	N/A	Solid

Figure 4.1: AutoCAD to SAP2000 [6]

We have chosen to model the dam in SAP2000 with solid elements, but as it can be seen in the figure 4.1 it is not possible to import model as solid elements from AutoCAD into SAP2000. Therefore, we decided to import the geometry by defining the points and inserting them into SAP2000 as joints.

To determine points on arcs in AutoCAD, each horizontal arch was divided into 20 line , to get a sufficient accuracy of pricewise linear approximation.The next step was to export theses points into SAP2000 as joints. This can be done by two methods. The first one is to save points in an Excel file and then export Excel file to SAP2000.

The alternative method, which was used here, is to import joints directly from AutoCad to SAP2000. For this purpose all points on dam model were defined in a specific layer in AutoCAD and AutoCAD file was saved as .dxf file which is the required format for SAP2000.

Th points in the special layer were imported into SAP2000 as special joints and the specified layer of joints in AutoCAD was chosen in front of special joints in DXF import window.

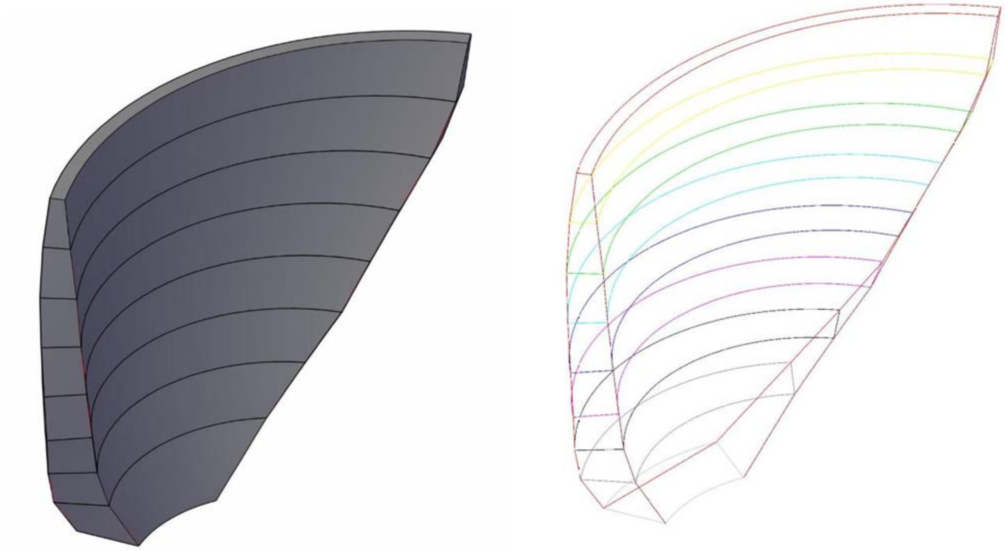


Figure 4.2: 3D model of the dam in AutoCAD
(each curve is a combination of 20 straight lines and 21 points)

After having the joints in SAP2000 we needed to define the solid elements and mesh elements to prepare the model for analysis. This step was most time consuming part of the modeling process.

3D solid elements could be created directly from joints, but we used another method which is described below:

In SAP2000 the shell area can be converted to the solid element by extrude option. Shells are 3 or 4 node elements with constant thickness which are used to model wall, floor and bridge deck.(7) The following figure shows a shell element with 4 nodes.

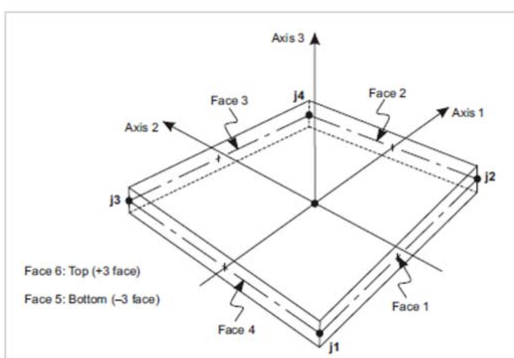
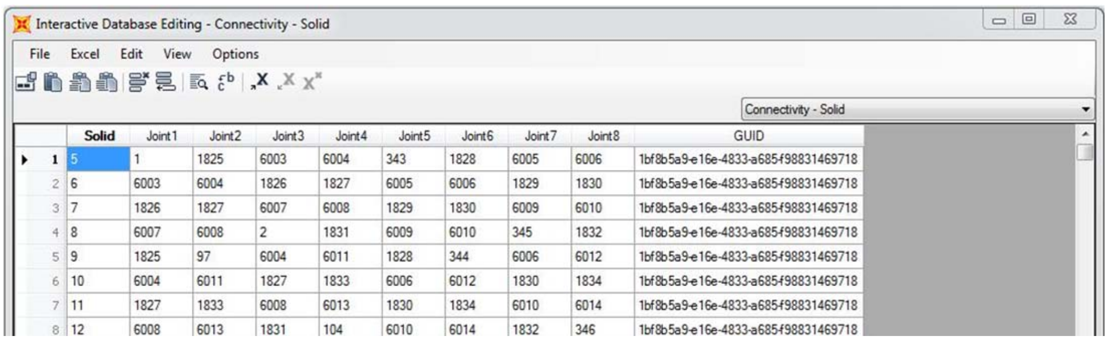


Figure 4.3: Shell elements with 4 nodes [6]

We used this property of software to provide 3D solid elements. Each 4 nodes surface in extradus was clicked and converted to the shell area. These areas extruded to solid elements with 8 nodes. The obtained solid elements were not positioned in their exact location on the dam body, because the extrude option is accessible only for a constant depth. The direction of extrude was along the thickness of dam. Therefore the 4 nodes of solid elements were not replaced exactly on intrados surface of dam.

The next step was to adjust these 4 nodes of solid on their exact location on imported joints. For this purpose, we edited the database of SAP2000. Software gives access to edit the database in Edit window. In this window database editing is possible for all parameters of the model.

In Display menu the joint labels were activated to see the number of each joint. Then the 4 nodes of solid which were not located on their exact location were replaced by right joints. This procedure was followed for all other solid elements which were created by shell extruding. Database editing menu provides access to copy data in the excel file. Solid connectivity data were copied in an excel file where the editing was done. The corrected data was then imported to SAP2000 to obtain the exact model.



	Solid	Joint1	Joint2	Joint3	Joint4	Joint5	Joint6	Joint7	Joint8	GUID
1	5	1	1825	6003	6004	343	1828	6005	6006	1bf8b5a9-e16e-4833-a685-f98831469718
2	6	6003	6004	1826	1827	6005	6006	1829	1830	1bf8b5a9-e16e-4833-a685-f98831469718
3	7	1826	1827	6007	6008	1829	1830	6009	6010	1bf8b5a9-e16e-4833-a685-f98831469718
4	8	6007	6008	2	1831	6009	6010	345	1832	1bf8b5a9-e16e-4833-a685-f98831469718
5	9	1825	97	6004	6011	1828	344	6006	6012	1bf8b5a9-e16e-4833-a685-f98831469718
6	10	6004	6011	1827	1833	6006	6012	1830	1834	1bf8b5a9-e16e-4833-a685-f98831469718
7	11	1827	1833	6008	6013	1830	1834	6010	6014	1bf8b5a9-e16e-4833-a685-f98831469718
8	12	6008	6013	1831	104	6010	6014	1832	346	1bf8b5a9-e16e-4833-a685-f98831469718

Figure 4.4: A view of solid element nodes database (SAP2000)

Each solid element was then divided into several solid meshes. The density of these meshes should be accurate enough to describe geometry of the dam. Besides that the density of meshes has direct effect on the results. More accurate result can be obtained by finer meshes.

In the SAP2000 the mesh was created by dividing the solids into several smaller solid elements that match criteria for discretization.

A fine mesh with 9990 joints and 7632 solid elements was obtained as shown in figure 4.5.

Another challenge in SAP2000 is the presentation of results. SAP2000 software presents stresses with respect to the global and local axis, but the local axis that is automatically defined is equal to the global one. For the stress analysis a more natural local axis that follows the curved shape of the dam is more appropriate. For this purpose the local axis of elements was manually rotated.

The local axis of solid elements needs to be rotated to find the stress along the dam arch direction. We rotated the local axis in a way that coordinates 1 and 2 become vertical axis and tangential axis to the surface face of the dam. This was done by assigning cylindrical coordinates in SAP2000. Figure 4.6 shows the local and global axis of the dam model.

These two steps were most time consuming parts of arch dam modeling in SAP2000.

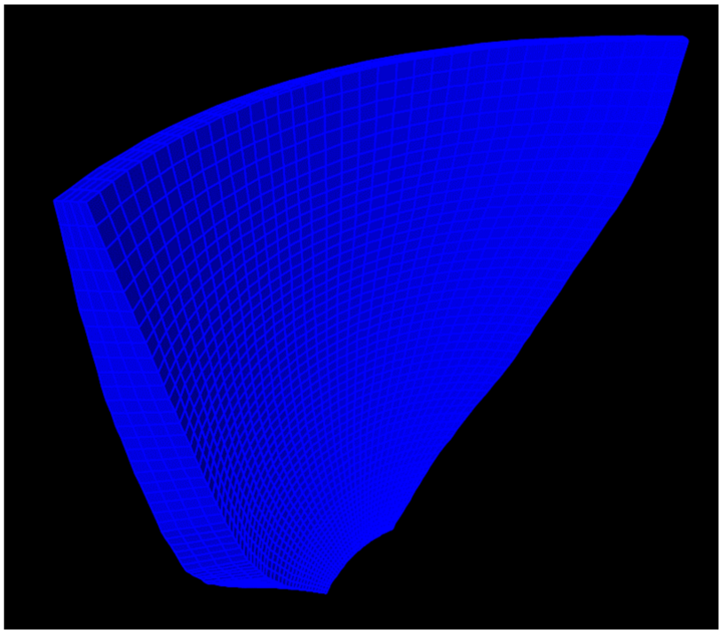


Figure 4.5: 3D view of meshed model in SAP2000

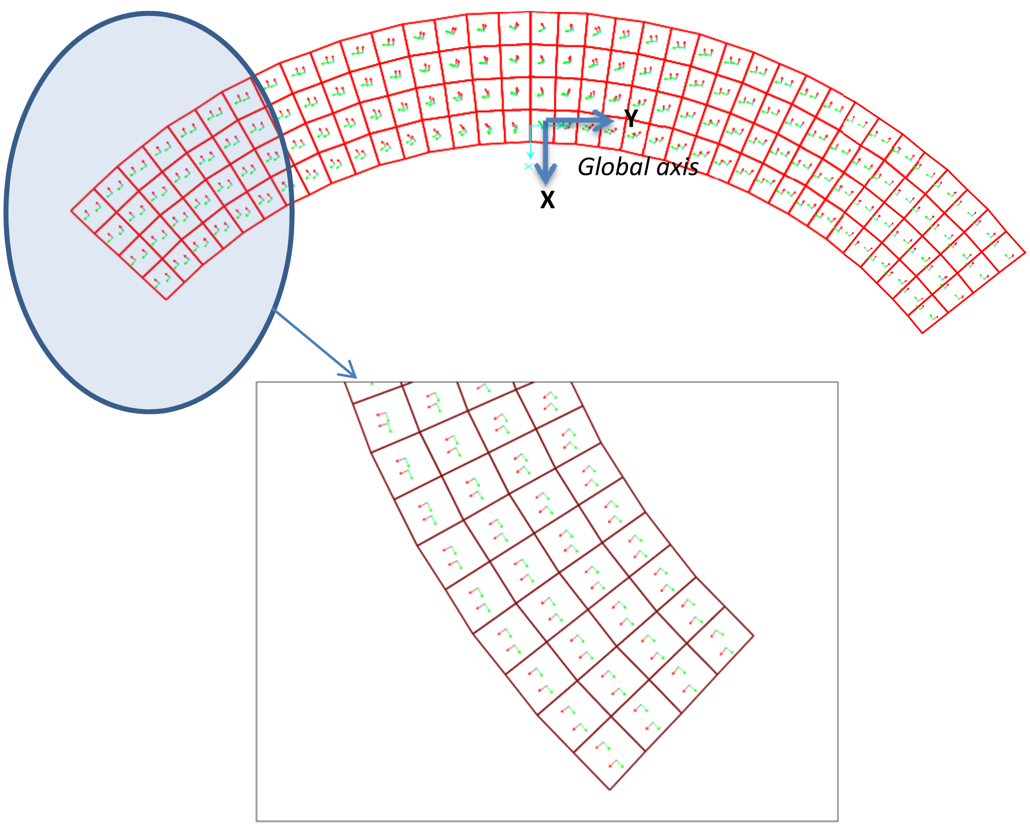


Figure 4.6: Local and global axis directions

As it can be seen from figure 4.6 the rotated local axis 2 displayed in green color presents the direction normal to the foundation rocks at the abutments. Red colored axis presents axis 1 which is orthogonal to the dam faces and blue color presents vertical axis 3 (not shown in figure) which is normal to the base foundation of the dam.

The next step in SAP2000 was to define the material properties and the boundary conditions. The main material of an arch dam is concrete, thus the typical values for concrete material were defined as shown in the next table. The deformation modulus of sound rock are shown in the table was used to define the boundary condition of the dam at foundation.

Table 2.2 presented modulus of elasticity for different foundation materials. The value chosen and shown in Table 4.1 for foundation rock is based on an assumption as a typical quantity.

Parameter		Symbol [unit]	Used value
Concrete	Modulus of elasticity	E [MPa]	31000
	Poison ratio	μ	0.2
	Concrete unit weight	γ [KN/m3]	25
	Shear modulus	G [MPa]	12917
	Bulk modulus	B [MPa]	17222
	Coefficient of thermal expansion	α [1/K]	1.00E-5
	Compressive strength	f_c[MPa]	34
	Nominal compressive strength	f_{ck}[MPa]	30
	Tensile strength	f_{ctm}[MPa]	3.5
Rock	Modulus of elasticity	E_s [MPa]	40000
	Deformation modulus	E_{ds} [MPa]	20000

Table 4.1: Material properties

4.2. Boundary conditions

As it was mentioned in the previous chapters, the site area of a dam should be considered in the model. Modeling of the site and the rock foundation in SAP2000 is a quite demanding and time consuming process and is therefore simplified for the purpose of this work.

We used two models of the foundation influence on the dam.

1) rigid foundation
2) elastic springs.

4.2.1. Rigid foundation

In this model the stiffness of sound rock is assumed to be much larger than the concrete. The sound rocks thus assumed to be rigid and elasticity and flexibility of rocks were neglected. By this method the displacements at the contact joints between the dam and the foundation were not allowed.

All joints at the base of dam and in contact with the abutment were restricted to move in any direction or rotate around any axis.

4.2.2. Surface spring

The second method to define the boundary condition was to assign the surface springs at the contact area of dam and sound rocks. The dam is in contact with sound rock along its foundation and abutments; at both sides of the canyon the ends cantilevers of the dam are clamped into the foundation rocks. When the contact between the dam and the rock is not rigid, the normal and shear forces occur. Due to limitations in SAP2000 we modeled contact area by surface springs which act only normally on surface and the influence of shear stiffness was not included. It was mentioned that EM1110-2-2201 suggests using deformation modulus of rock for considering effect of cracks and joints of foundation rocks in static analysis. Since the springs are placed on the orthogonal directions, the shear effect can be modeled by increasing their stiffness. Thus 30000MPa was used as appropriate spring stiffness.

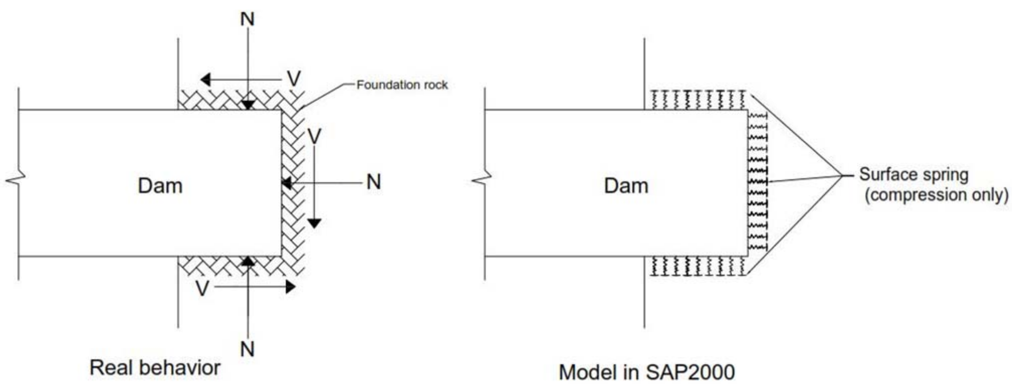


Figure 4.7: Modeling abutment of dam

4.3. Loading

The loads which were considered here are self-weight and hydrostatic pressure load. The self-weight of a dam is computed automatically by software from material properties which were assigned in program.

For hydrostatic load two elevation of reservoir water considered.

1- Water at elevation 816m (162m from dam foundation) which is the maximum operating water level of reservoir.
2- The minimum operating water level of reservoir at elevation of 760m (106m from dam foundation)

Figure A10 in Annex shows applying of these two operating water level on the dam.

It is important to note that the streambed elevation is at 660 m and that the dam base is at elevation 654m, the soil between these two elevations is saturated and water pressure will have effect on the dam; therefore we applied water pressure down to the base of the dam (base elevation 654m).

The load patterns which were defined in SAP2000 are self-weight and hydrostatic load at elevation 816m and at the elevation 760 m. In load case menu all loads were assigned as linear static type.

Water pressure should be applied on the upstream face of the dam as a live load. The pressure on the dam face is increasing from the top level of reservoir to the base of reservoir. To present this property of hydrostatic load, we assigned joint patterns with unit weight of water. SAP2000 allow us to assign surface pressure load on solid elements. The load for each elevation was assigned on corresponding face of the dam using joint patterns.

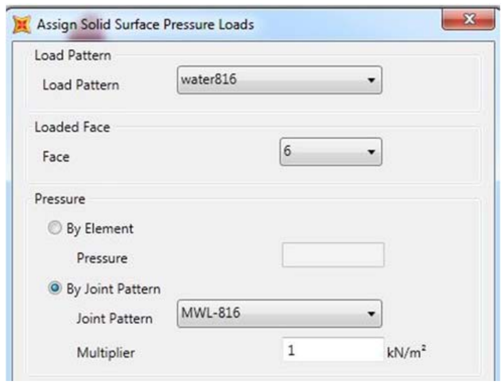


Figure 4.8: Assigning hydrostatic loads

Loading diagram for both maximum and minimum water level is shown in Appendix A, figure A1 and A2.

For each support model three load cases were studied:

Self-weight in the case of dam after construction, when the reservoir is empty.

1- Self-weight combined with maximum water level (SUN1 in table 2.1 without considering the temperature)
2- Self-weight combined with minimum water level (SUN2 in table 2.1 without considering the temperature)

5. Results

The basic results that should present for static analysis of arch dams are nodal displacement and stresses at different locations of the dam. In this thesis displacements are shown by deformed shape graphs for crown cantilever and several arches at different elevations.

The stresses are presented by stress diagrams for upstream and downstream faces of dam as well as crown cantilever.

The dam response was studied for both cases of dam boundary conditions, and for each foundation, three loads combinations were applied.

1- SUN1: Dead load + Maximum water level at elev. 816m.
2- SUN2: Dead load + Minimum water level at elev. 760m.
3- Dead load: SUN3 which is at the case while the dam construction phase is finished and dam is only subjected to its self-weight and temperature loads. Temperature load is not included in this thesis, therefore this combination contain only self-weight, shown under name of "Dead".

5.1. Displacements

5.1.1. Rigid foundation

Let us start with deformation of the dam when the foundation is defined to be rigid. Figures 5.1 through 5.3 present the deformations for each of three load combinations defined above. For each load combination the displacement in crown cantilever section and 8 arch sections were shown. Also to present the deformed dam better, 9 points on cantilever sections and 15 points on arch sections are show with their initial and deformed position. Tables 5.1 and 5.2 present the components of displacement vector at these points.

Rigid foundation – Crown cantilever deformation									
load →	Dead load			SUN1 combination			SUN2 combination		
Joints↓	Displacement[mm]			Displacement[mm]			Displacement[mm]		
[Elev.]	U1	U2	U3	U1	U2	U3	U1	U2	U3
1[820m]	7.28	2.64	-14.34	16.76	1.79	-10.47	7.39	2.78	-12.66
2[800m]	4.60	1.94	-13.59	17.39	1.50	-10.64	5.07	2.05	-12.03
3[780m]	2.39	1.66	-14.08	18.02	0.39	-10.33	3.67	1.61	-12.14
4[760m]	0.52	1.16	-13.10	17.96	-0.17	-9.98	3.36	1.02	-11.03
5[740m]	-1.09	0.57	-10.80	16.60	0.45	-8.10	3.80	0.57	-10.11
6[720m]	-2.08	0.30	-9.42	14.37	0.24	-6.12	4.29	0.29	-8.40
7[700m]	-2.00	0.25	-7.16	11.09	-0.28	-6.98	4.22	0.05	-6.57
8[680m]	-1.02	0.07	-3.63	6.11	-0.16	-4.94	2.83	-0.04	-4.12
9[654m]	0	0	0	0	0	0	0	0	0

Table 5.1: Displacements of the crown cantilever for rigid foundation
(Refer to figure 5.1 through 5.3 for cantilever)

All displacements in Table 5.1 and 5.2 are based on global coordinate system as shown in figure 4.6.

Since the foundation is defined to be rigid, the displacements at the boundary nodes of the dam vanish.

If the dam is subjected only to its self-weight, which acts vertically, the displacement in vertical direction 3 or z is higher as for the other two load combinations. Vertical displacements are at the highest magnitudes for joints on crest and decrease to zero at elevation 654m at the foundation of the dam. Deformed crown cantilever section shows that the dam is bend vertically due its self-weight. Such bending results in displacements in the direction of local axis 1 which are positive displacements for the joints at the upper level (moving toward downstream) and negative displacements (moving toward upstream) for lower joints on cantilever section. (Table 5.1)

The deformation in the horizontal plane stems from the nonsymmetric shape of dam arches. It is shown in figure 5.1. For upper arches, it can be seen that the left side of the dam (defined in figure 3.5) and crown cantilever move toward downstream, while right side is moving toward upstream in negative x-direction. As the elevation of arch decreases when moving in the x-direction displacement decreases as well and reaches zero value at foundation.

Rigid foundation –Arches deformation									
Load →	Dead load			SUN1 combination			SUN2combination		
Joints ↓	Displacement[mm]			Displacement[mm]			Displacement[mm]		
[Elev.]	U1	U2	U3	U1	U2	U3	U1	U2	U3
1[820m]	-1.59	4.70	-17.11	3.77	3.92	-13.98	-2.04	5.28	-15.61
2[820m]	7.28	2.64	-14.34	16.77	1.24	-10.38	7.39	2.78	-12.66
3[820m]	4.58	2.52	-13.56	6.35	1.46	-9.76	4.39	2.46	-11.81
4[780m]	-0.80	2.28	-16.29	8.17	-2.09	-13.49	-0.52	2.34	-14.97
5[780m]	2.33	1.38	-12.81	17.82	1.11	-10.33	3.60	1.44	-11.54
6[780m]	1.41	1.35	-12.49	8.35	3.44	-9.89	1.79	1.45	-10.98
7[740m]	-1.04	1.57	-13.82	9.17	-3.74	-10.90	1.46	0.25	-12.81
8[740m]	-1.09	0.57	-10.80	16.60	0.45	-8.10	3.80	0.57	-10.11
9[740m]	-0.46	-0.07	-10.90	8.07	3.51	-8.64	1.88	0.97	-9.87
10[700m]	-1.55	1.37	-9.95	6.24	-2.74	-6.65	2.11	-0.59	-8.41
11[700m]	-2.31	0.11	-7.68	10.63	0.10	-3.78	3.84	0.10	-6.05
12[700m]	-1.29	-0.91	-8.12	5.69	2.48	-5.44	2.10	0.78	-6.73
13[654m]	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
14[654m]	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
15[654m]	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00

Table 5.2: Displacements of arches for rigid foundation (Refer to figure 5.1 through 5.3 for arches deformation)

When the dam is subjected to hydrostatic load as well as self-weight, the displacements toward downstream are observed due to horizontal water pressure loads. For maximum water level (figure 5.2) the horizontal displacements are higher as for the other two combinations. This stems from the high water pressure load, which starts from elevation 816 m. In this case the hydrostatic load, which acts normal to self-weight of the dam, decreases the vertical displacements as well as the dam vertical bending due to dead load.

In the case of minimum water level at elevation 760 m displacements due to water pressure toward downstream are smaller as for the maximum water level, but still larger as in the dead load case. A small bending in crown cantilever section of dam is also observed. In both cases of hydrostatic combinations the displacements of arches are in the downstream direction, except at the end of the right side which has negative displacement due to nonsymmetrical shape of the dam. Displacements on the abutment and at the foundation are vanished.

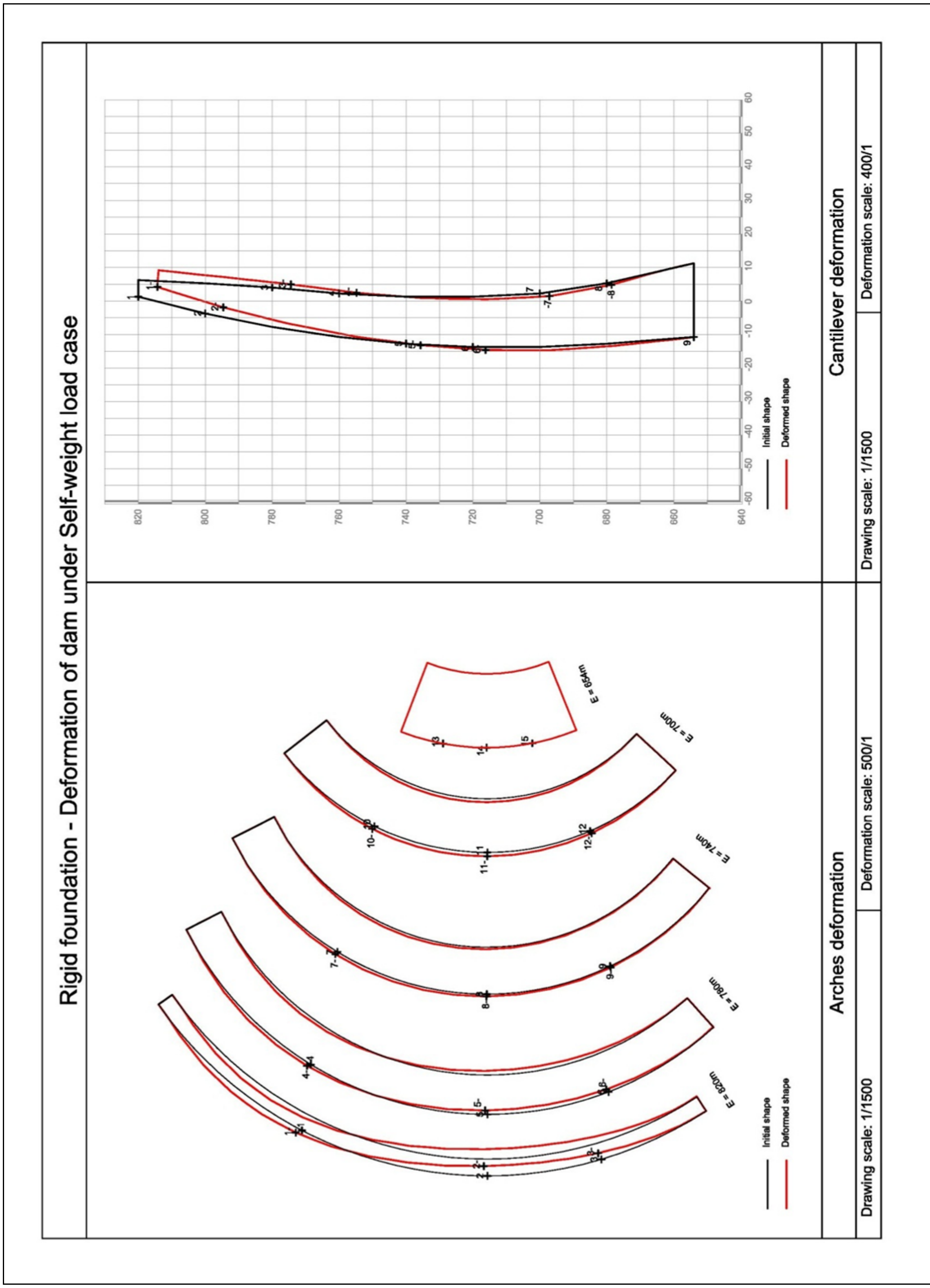


Figure 5.1: Deformation of rigid foundation dam subjected to dead load only

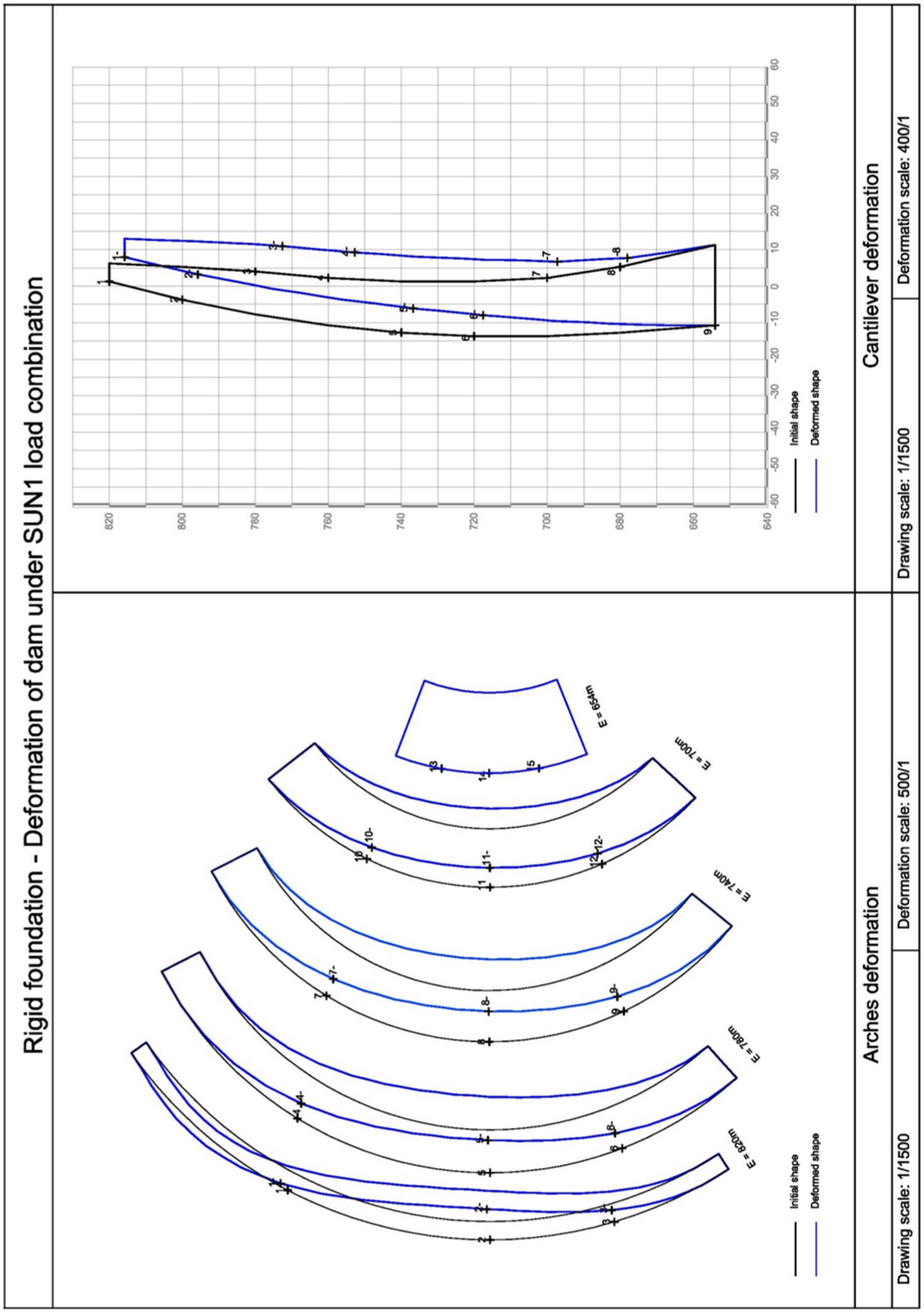


Figure 5.2: Deformation of rigid foundation dam subjected to SUN1 loads combination

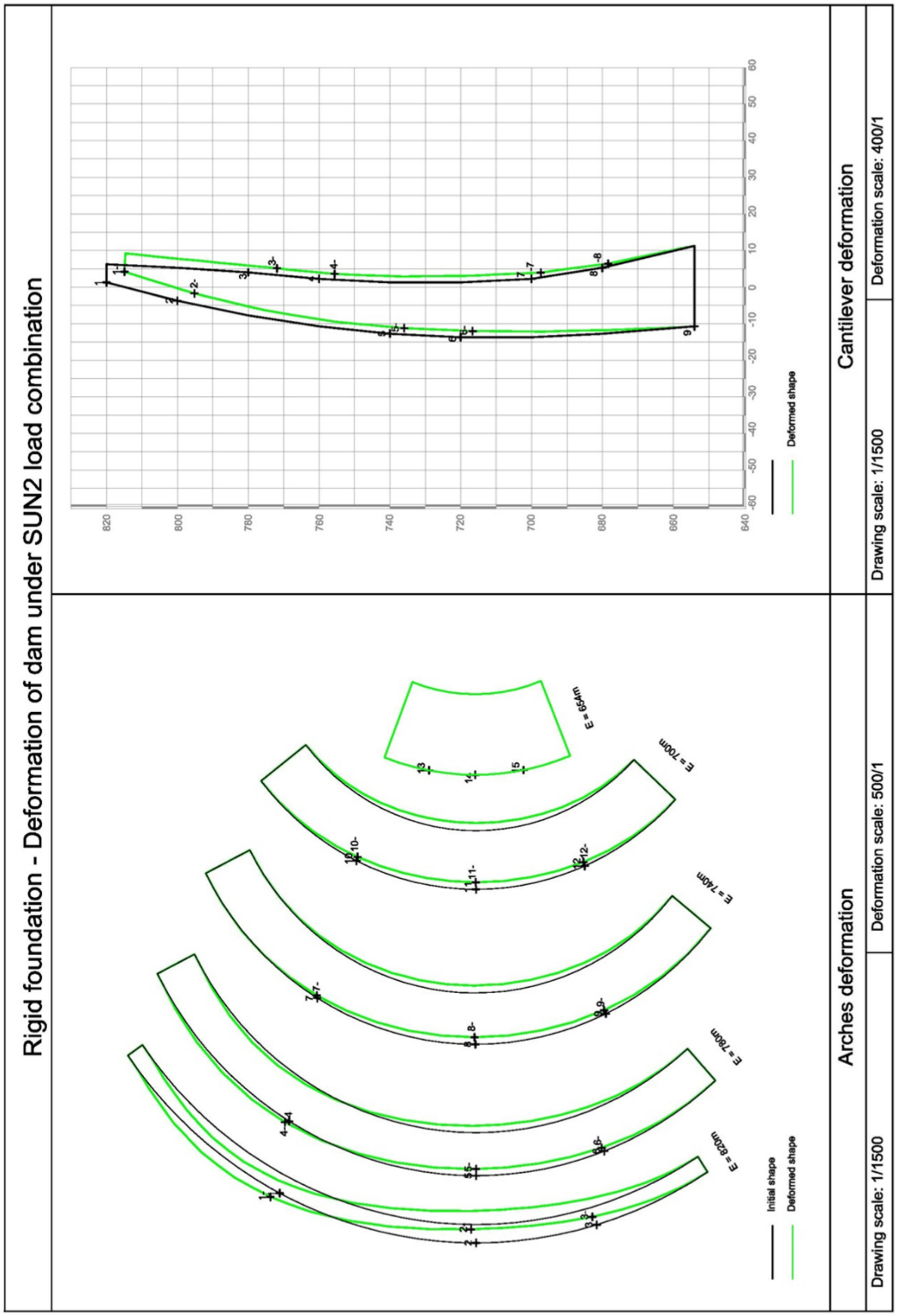


Figure 5.3: Deformation of rigid foundation dam subjected to SUN2 loads combination

5.1.2. Flexible foundation

In the case of flexible foundation, nodes on foundation and abutment experience small displacements in all directions. To evaluate displacement on foundation, 6 additional nodes are defined on arches as shown in figures 5.4 to 5.6.

The general behavior when the displacements are compared for the three load combinations remains similar as it was found for rigid foundation. When we compare the load cases in between, we are able to draw the same conclusion. However the displacements are now in general smaller due to the flexible boundary, but we have nodal displacement at the foundation and at the abutments.

Table 5.3 presents the displacements for nodes which are shown in figure 5.4, 5.5 and 5.6.

Flexible foundaion - Cantilever deformation									
load	Dead load			SUN1 combination			SUN2 combination		
joints	Displacement [mm]			Displacement [mm]			Displacement [mm]		
[Elev.]	U1	U2	U3	U1	U2	U3	U1	U2	U3
1[820m]	-0.95	-0.37	-7.48	10.12	-1.94	-3.93	-0.65	-0.34	-6.30
2[800m]	-1.75	-0.63	-7.17	12.61	-1.30	-4.53	-0.96	-0.55	-6.14
3[780m]	-2.58	-0.22	-7.23	14.48	-2.42	-3.86	-0.94	-0.37	-5.84
4[760m]	-3.49	-0.19	-6.63	14.89	-2.13	-3.95	-0.42	-0.47	-5.18
5[740m]	-4.21	-0.71	-5.52	13.59	-0.35	-2.50	0.51	-0.61	-5.10
6[720m]	-4.20	-0.61	-5.02	11.15	-0.08	-1.24	1.38	-0.49	-4.15
7[700m]	-3.05	0.03	-3.18	7.77	-0.88	-3.02	1.80	-0.46	-2.95
8[680m]	-1.33	-0.03	-1.53	3.82	-0.38	-2.25	1.33	-0.33	-1.89
9[654m]	-0.01	-0.30	-0.16	0.07	0.21	0.01	0.04	-0.22	-0.08
Flexible foundation - Arches deformation									
1[820m]	-2.25	-0.75	-7.33	4.02	-2.33	-2.64	-2.02	-0.48	-6.06
2[820m]	-0.95	-0.37	-7.48	10.12	-1.94	-3.93	-0.65	-0.34	-6.30
3[820m]	-0.06	0.23	-7.10	1.61	-1.47	-3.78	-0.16	0.10	-5.91
4[780m]	-2.52	-0.47	-7.09	5.77	-3.96	-2.43	-1.86	-0.41	-5.88
5[780m]	-2.64	-0.76	-6.65	14.27	-0.93	-4.37	-1.01	-0.66	-5.89
6[780m]	-0.62	-0.55	-6.60	4.38	0.75	-4.08	-0.30	-0.48	-5.55
7[740m]	-2.71	-0.45	-6.17	5.46	-3.94	-1.56	-0.74	-1.23	-5.07
8[740m]	-4.21	-0.71	-5.52	13.59	-0.35	-2.50	0.51	-0.61	-5.10
9[740m]	-0.95	-0.36	-5.56	3.09	1.31	-3.09	0.07	0.03	-4.66
10[700m]	-1.88	-0.62	-4.53	2.78	-2.55	-0.39	0.11	-1.62	-3.09
11[700m]	-3.31	-0.50	-4.35	7.60	0.13	-0.20	1.57	-0.36	-2.94
12[700m]	-0.69	0.22	-3.92	1.04	1.38	-1.75	0.13	0.66	-2.96
13[654m]	-0.14	-0.63	-0.16	0.01	-0.25	0.00	-0.09	-0.62	-0.08
14[654m]	-0.01	-0.30	-0.16	0.07	0.21	0.01	0.04	-0.22	-0.08
15[654m]	-0.03	0.09	-0.14	-0.08	0.62	-0.03	-0.02	0.21	-0.09
16[820m]	-0.12	-1.78	-5.97	0.01	-0.51	-1.80	-0.10	-1.45	-4.87
17[780m]	-0.04	-0.10	-6.34	0.02	-0.13	-3.81	-0.03	-0.09	-5.19
18[740m]	-2.03	-1.94	-6.18	-0.52	-0.55	-1.89	-1.56	-1.50	-4.82
19[740m]	-0.14	0.83	-5.70	-0.03	0.50	-3.74	-0.10	0.66	-4.61
20[700m]	-1.44	-2.00	-4.13	-0.43	-0.61	-1.46	-1.08	-1.49	-3.17
21[654m]	-0.23	0.52	-0.26	-0.31	0.78	-0.20	-0.20	0.52	-0.22

Table 5.3: Displacements of the dam on the flexible foundation (Refer to figure 5.3 through 5.6)

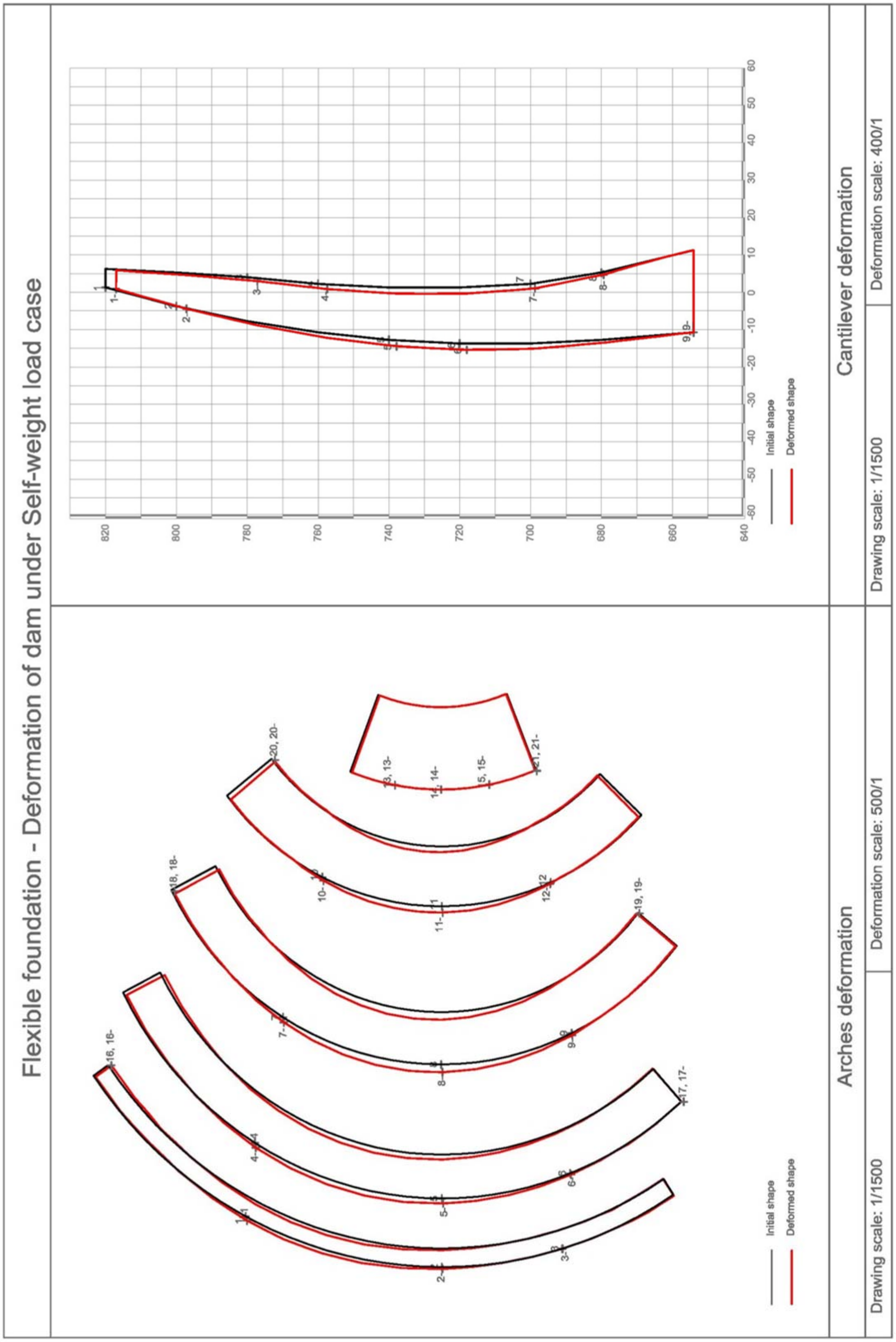


Figure 5.4: Deformation of flexible foundation dam subjected to dead load only

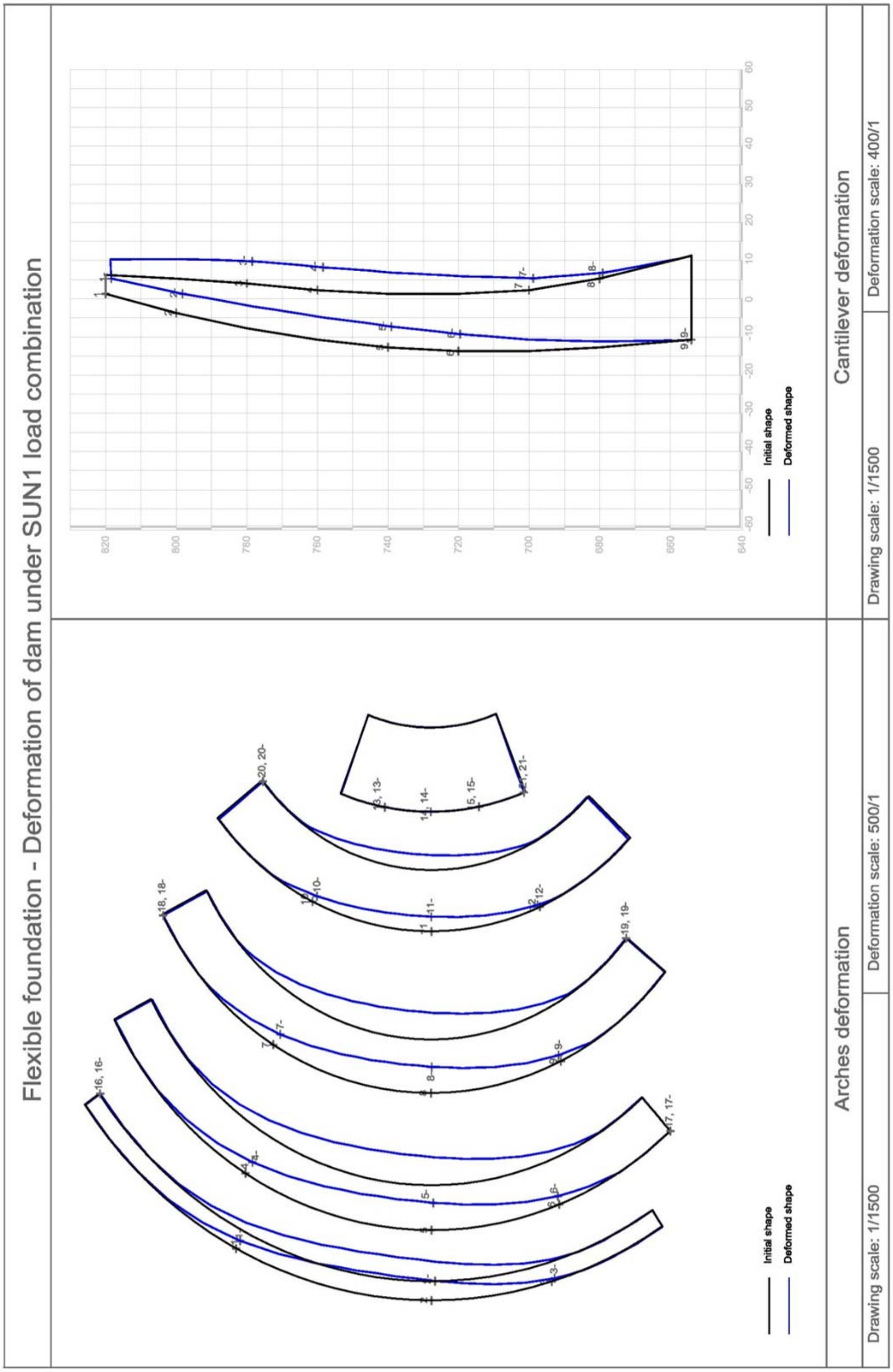


Figure 5.5: Deformation of flexible foundation dam subjected to SUN1 loads combination

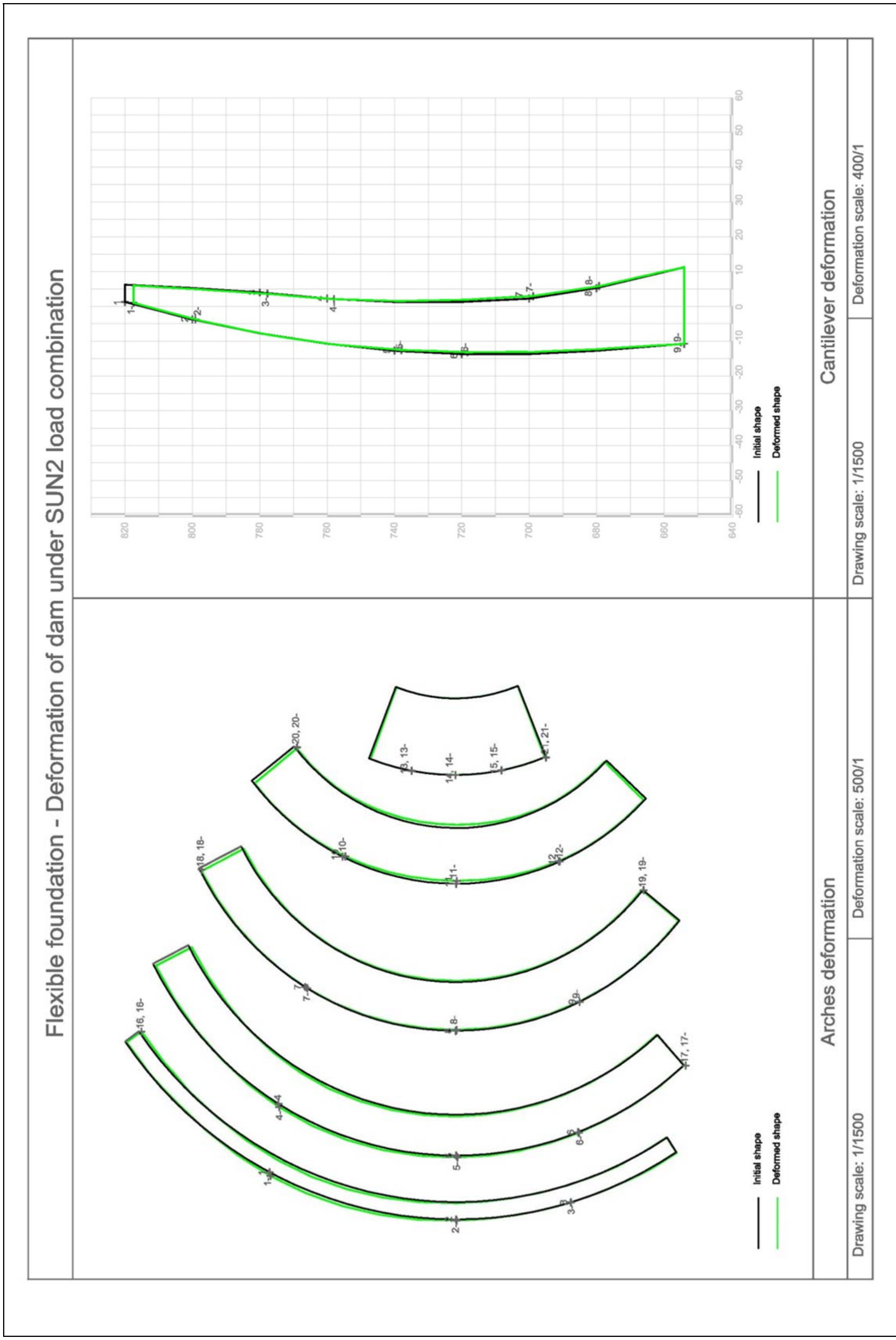


Figure 5.6: Deformation of flexible foundation dam subjected to SUN2 loads combination

In next three figures we compare the displacements for both support models for each load combination separately.

The flexible foundation allows the displacements of the dam at the abutment and foundation rocks, thus small displacements occur also at the boundary nodes. This results in the overall decrease of displacements in nodes through the entire dam.

In crown cantilever graph for dead load, the vertical bending of the dam is evident for the rigid foundation, but for flexible foundation dam the initial shape remains almost the same.

The deformation shapes for arches, especially for crest arch, show that the dam has a non-uniform behavior when the foundation is modeled as rigid, while in the case of flexible foundation the displacements are closer to symmetry.

EM 1110-2-2201 mentions that loads deflection pattern in a dam should vary smoothly from point to point. This means there should not be any evident change in curvature of deformed arches. This property can be observed in the case of flexible foundation while the deflection for rigid foundation mostly follows a non-smooth pattern.

Generally in the case of flexible foundation, the displacements of the dam are more uniform and the dam shows better monolithic behavior. This can be seen for both crown cantilever and arches in the graphs comparing the displacements for both support models.

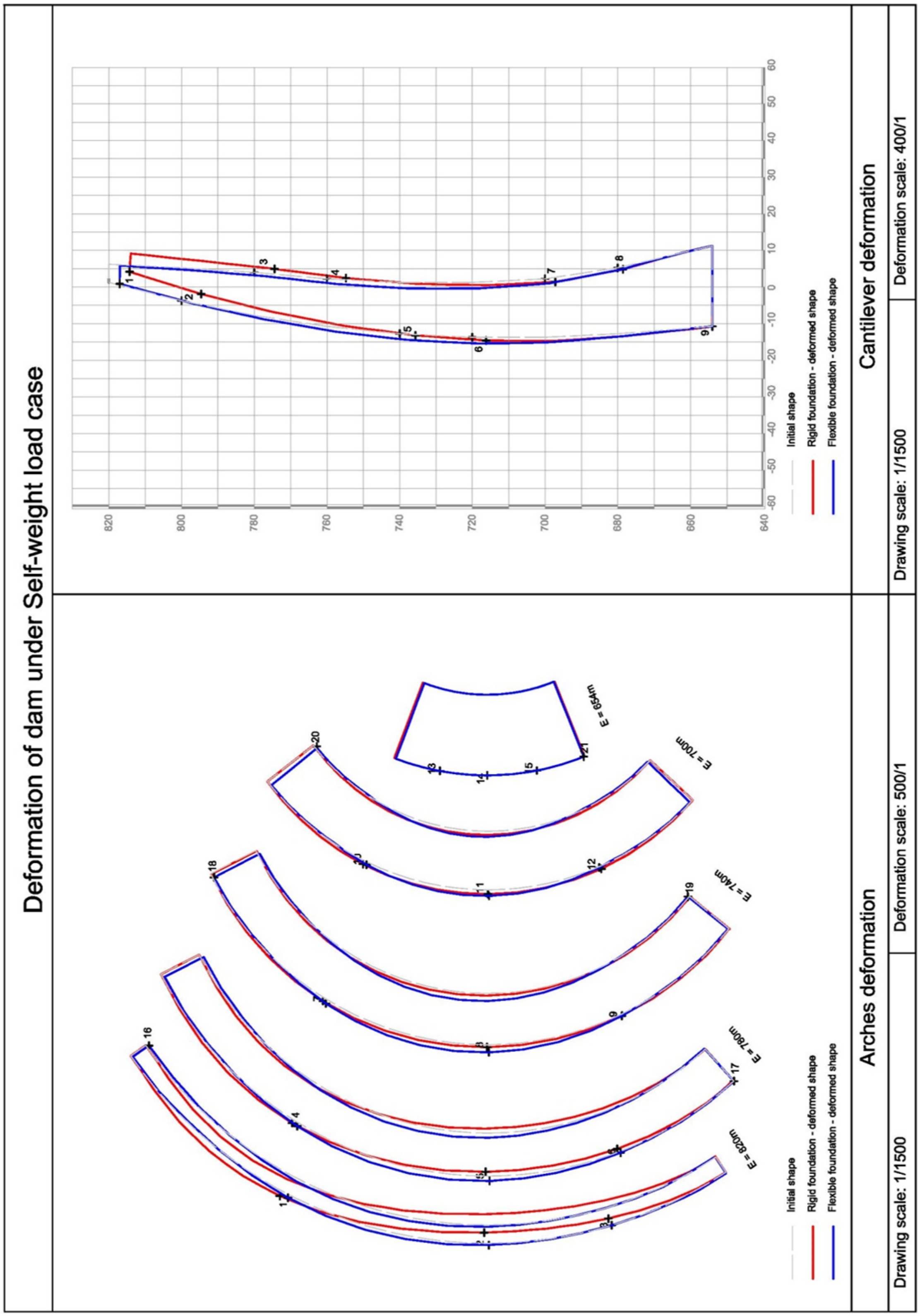


Figure 5.7: Deformation of rigid and flexible foundation dam subjected to dead load only

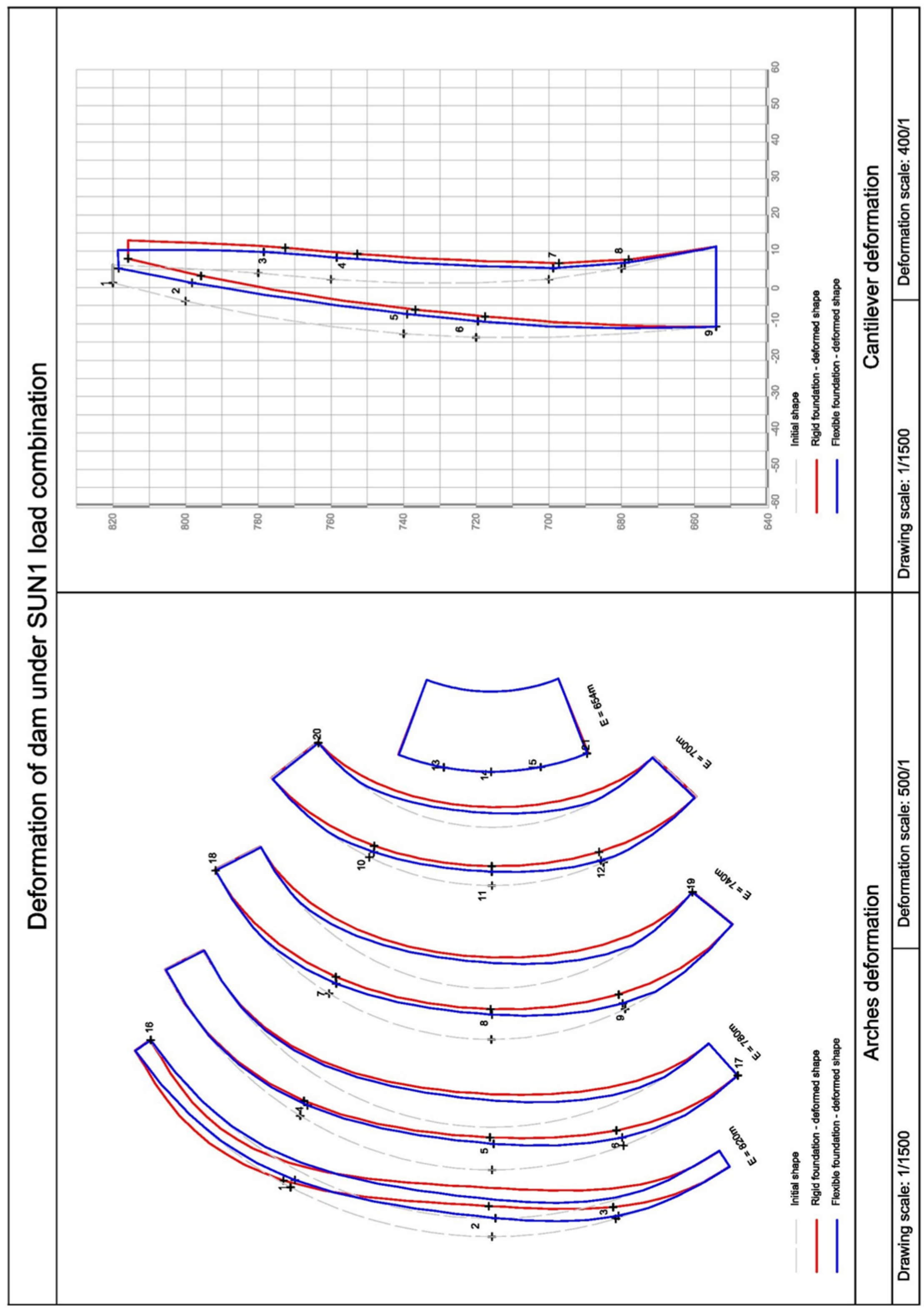


Figure 5.8: Deformation of rigid and flexible foundation dam subjected to SUN1 loads combination

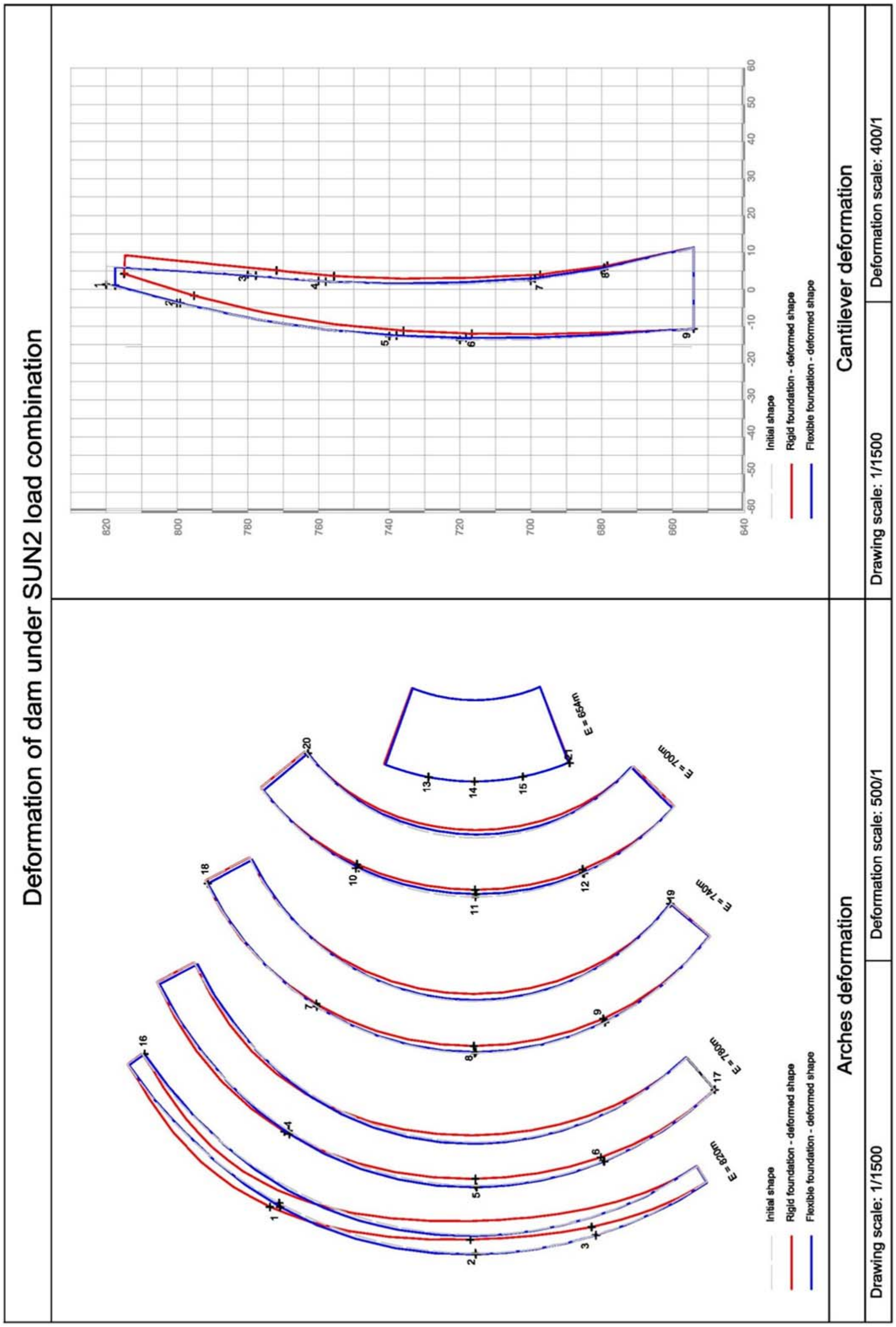


Figure 5.9:Deformation of rigid and flexible foundation dam subjected to SUN2 loads combination

5.2. Stresses

In the design of structures the stress field under the prescribed load combinations is highly important. We obtain the stresses using the methods implemented in SAP2000.

These stresses will be compared with the allowable stresses, which were specified in section 2.6. The largest compressive and tensile stress should be less than compressive and tensile stress of concrete by safety factor provided in section 2.6 according to the combination categories. Besides that we will be able to analyze if any unusual stresses distributions or magnitude are present in the dam.

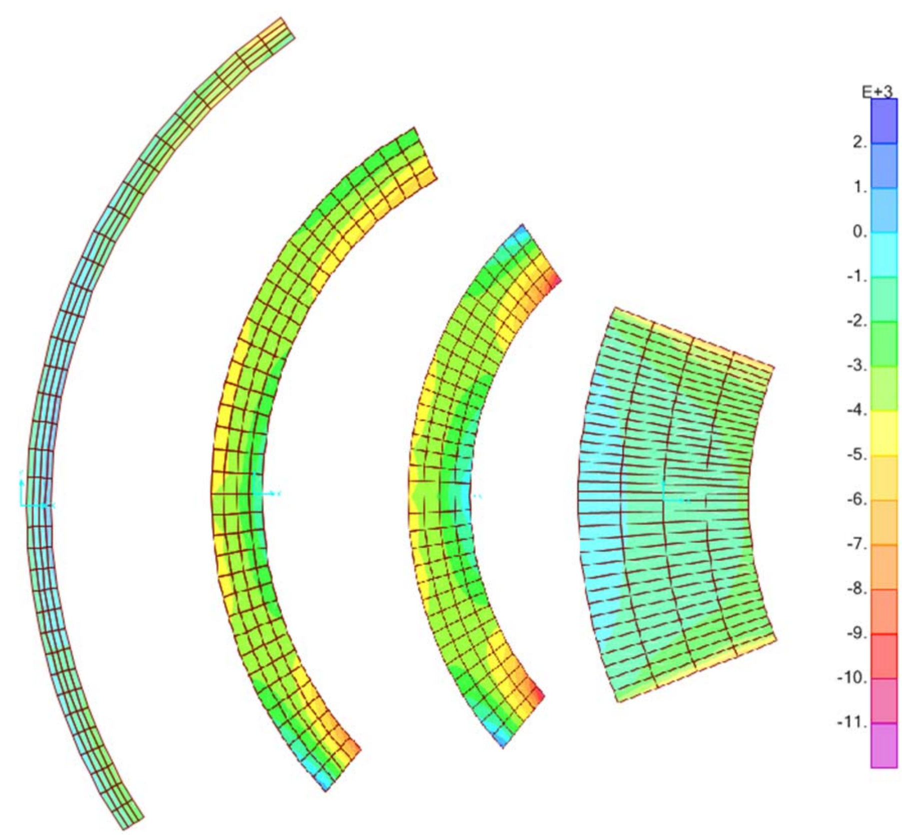
Due to the properties of concrete, it is important to avoid the tensile stresses or at least to keep it at the minimum level. The presence of large tensile stress could results in tension cracking and joints openings should be considered. Minimizing the vertical tension stress is possible by vertical arching and overhanging the crest.

Tensile stress in the downstream face of dam at crown is unacceptable. If the preliminary results present this stress there are two ways to resolve this issue: We can reduce the thickness of concrete at downstream face at the crown while thickness remains the same at abutment. Another possibility is to increase the horizontal curvature that increases the rise of arch.

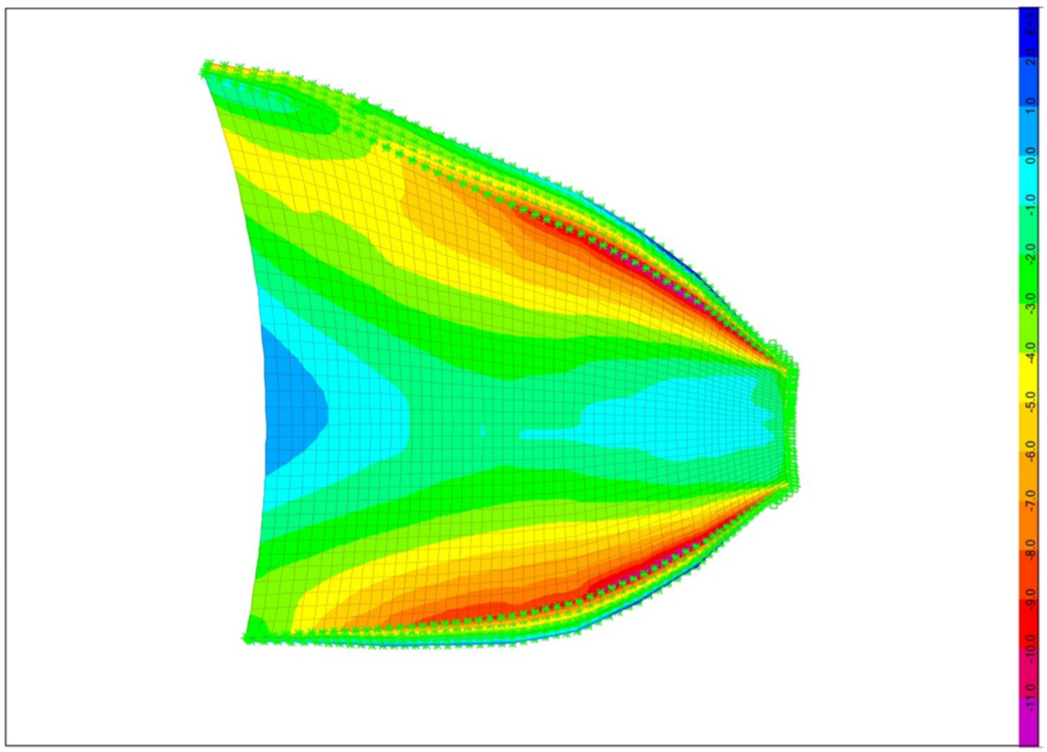
It was mentioned that arch dam is designed to transfer horizontal hydrostatic loads to abutment and foundation by arch action, therefore high compression stress at the abutment throughout the dam is expected.

In the case of empty reservoir or low water level, some horizontal zones of tensile stress are observed in upstream and downstream faces. The tensile stress on upstream face of the dam is also expected while reservoir has water, because hydrostatic load is acting horizontally and normally to the upstream face of dam. The magnitudes of these stresses are much smaller than allowable tensile stress as it can be seen from figures. With some additional loads, such as earthquake or temperature these tensile stresses can open vertical contraction joints of dam, because contraction joints are designed to have little or no tensile strength. After opening contraction joints loads will be redistributed to cantilever action.

In general "the results of a linear elastic analysis are valid only if the cracking or joint openings that occur in the dam are minor and the total stiffness of the structure is not affected significantly."[1]

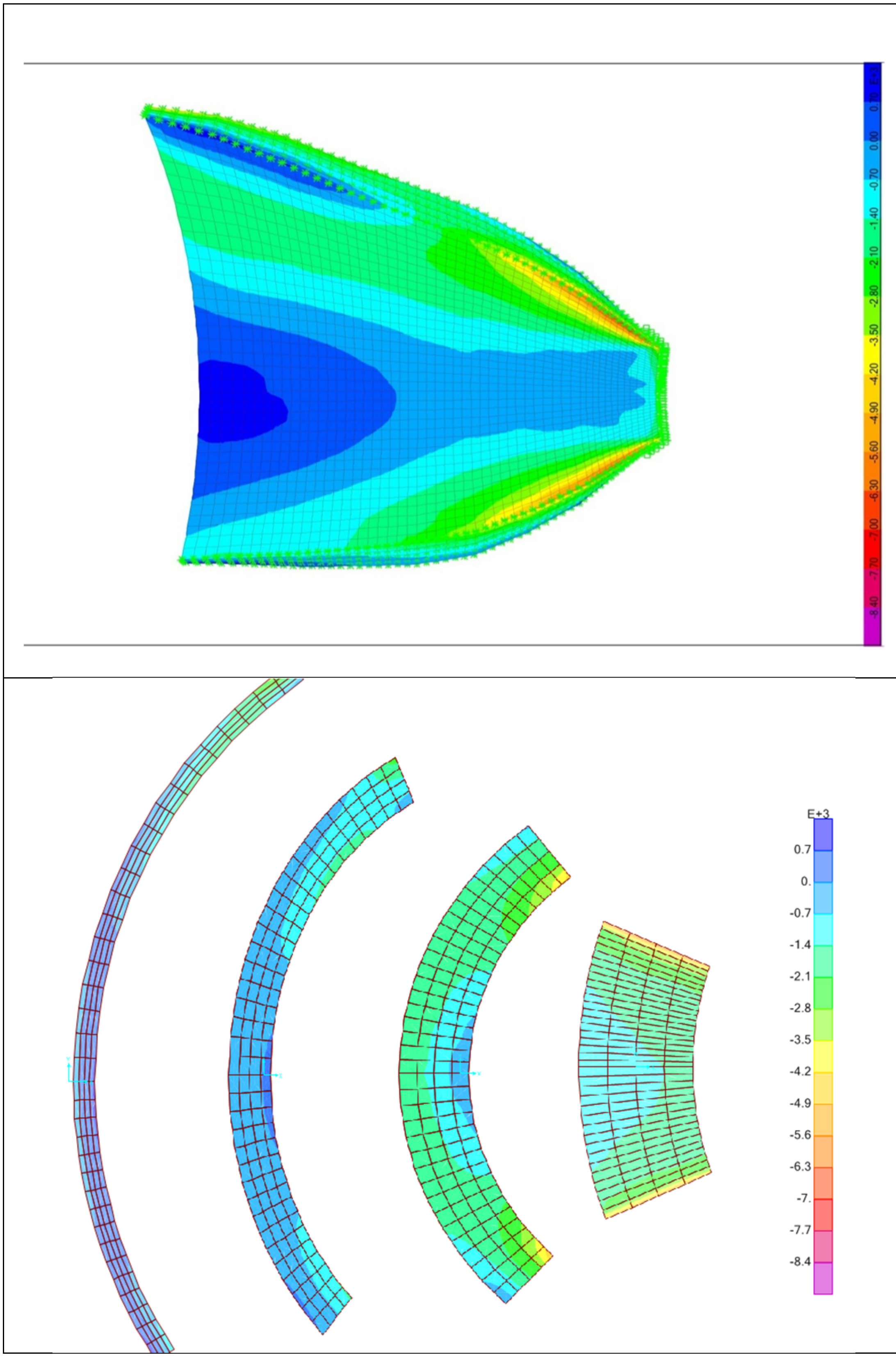


Plan view: Normal stresses with respect to local axis 2



Downstream view: Normal stresses with respect to local axis 2

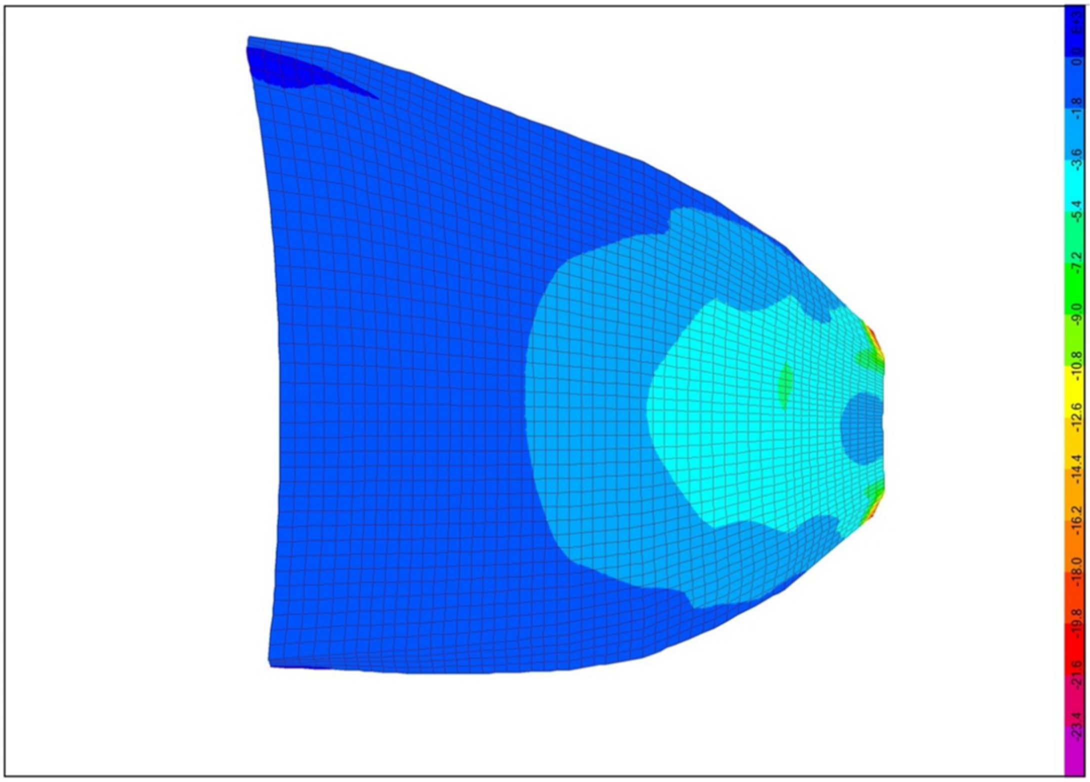
Figure 5.10: Rigid foundation, S22 due to SUN1 combination



Downstream view: Normal stresses with respect to local axis 2

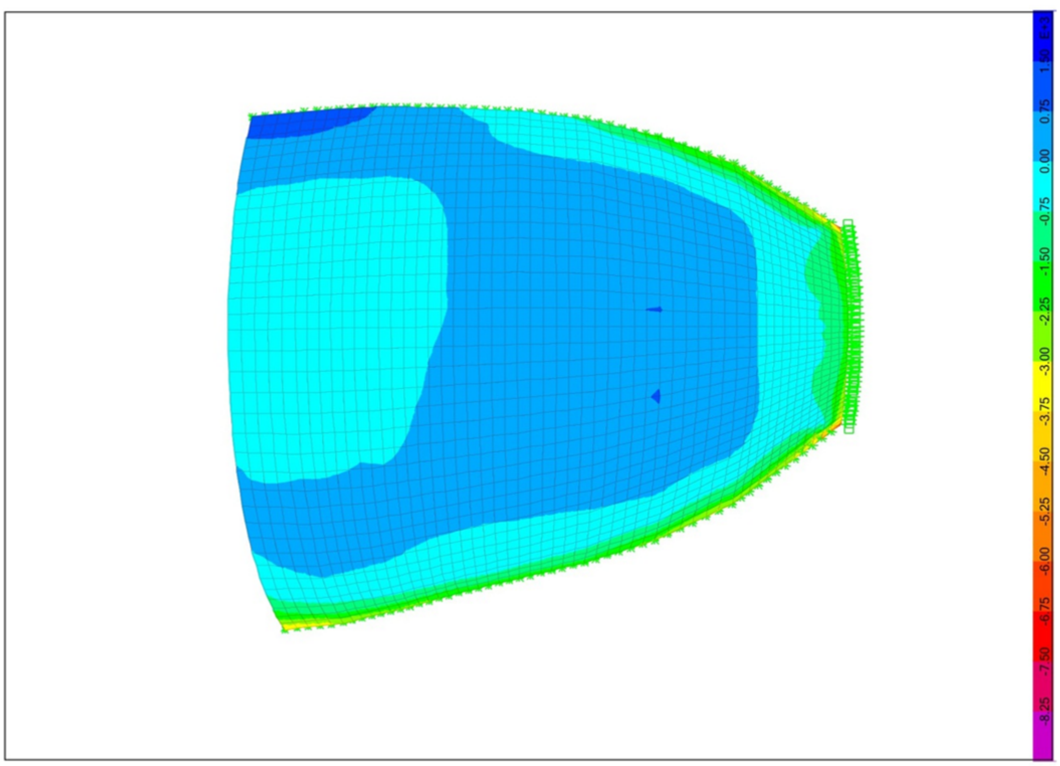
Plan view: Normal stresses with respect to local axis 2

Figure 5.11: Rigid foundation, S22 due to SUN2 combination



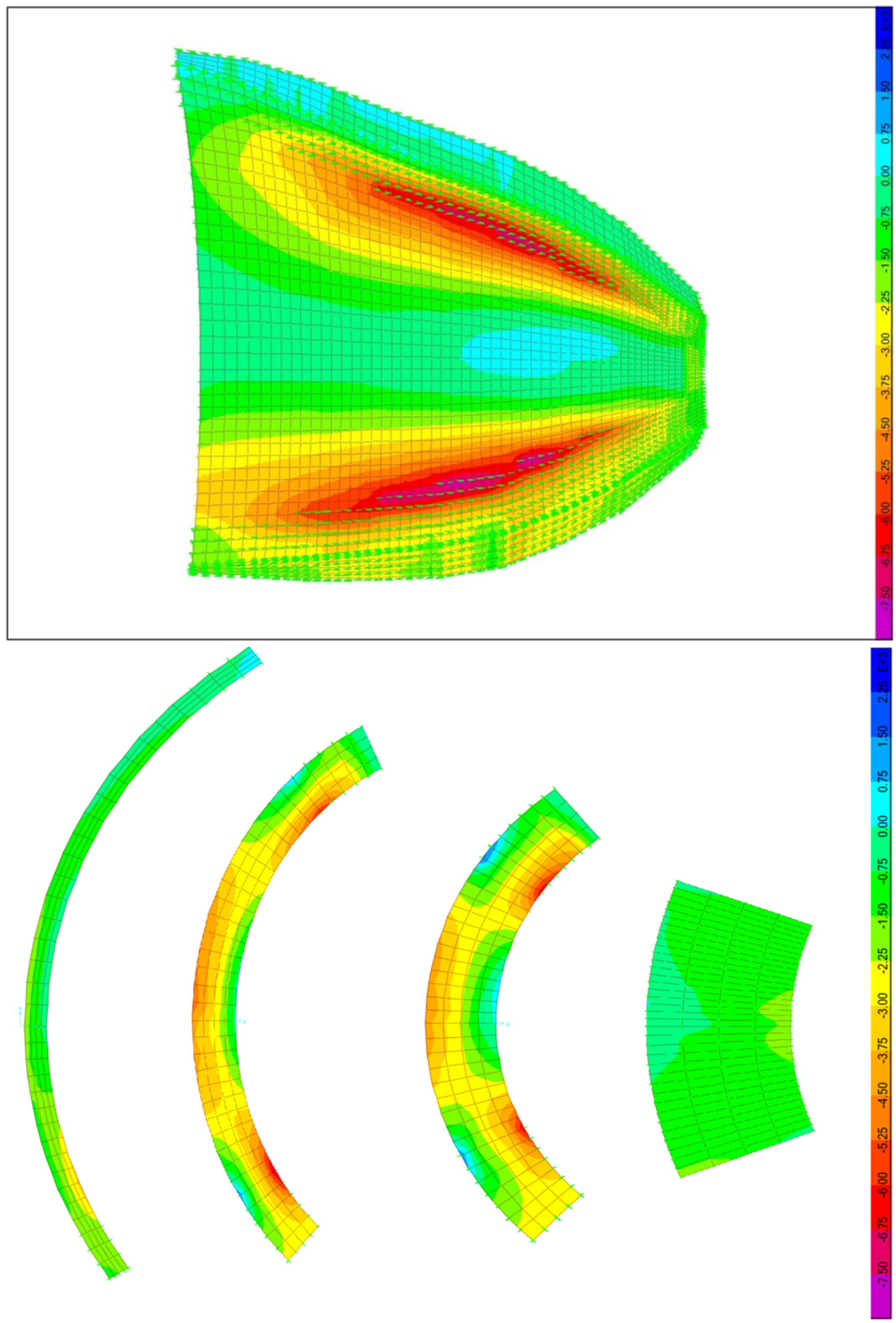
Downstream view: Normal stresses with respect to local axis 3

Figure 5.12: Rigid foundation, S33 due to self-weight



Upstream view: Normal stresses with respect to local axis 2

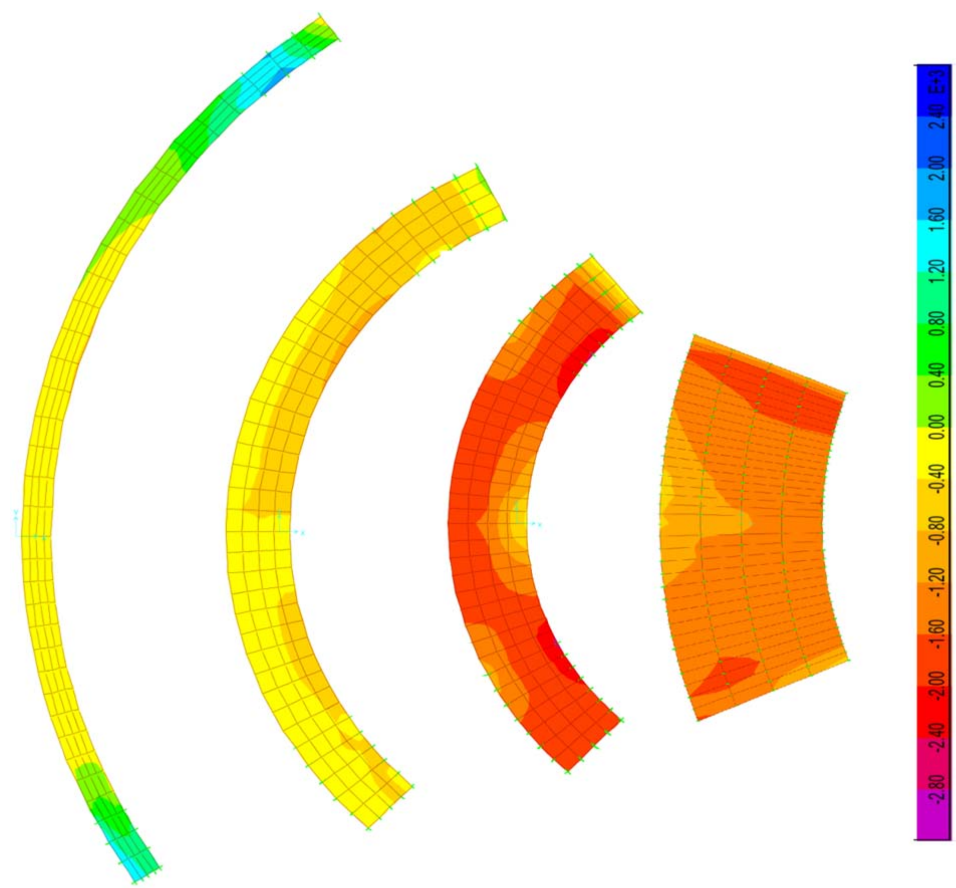
Figure 5.13: Rigid foundation, S22 due to self-weight



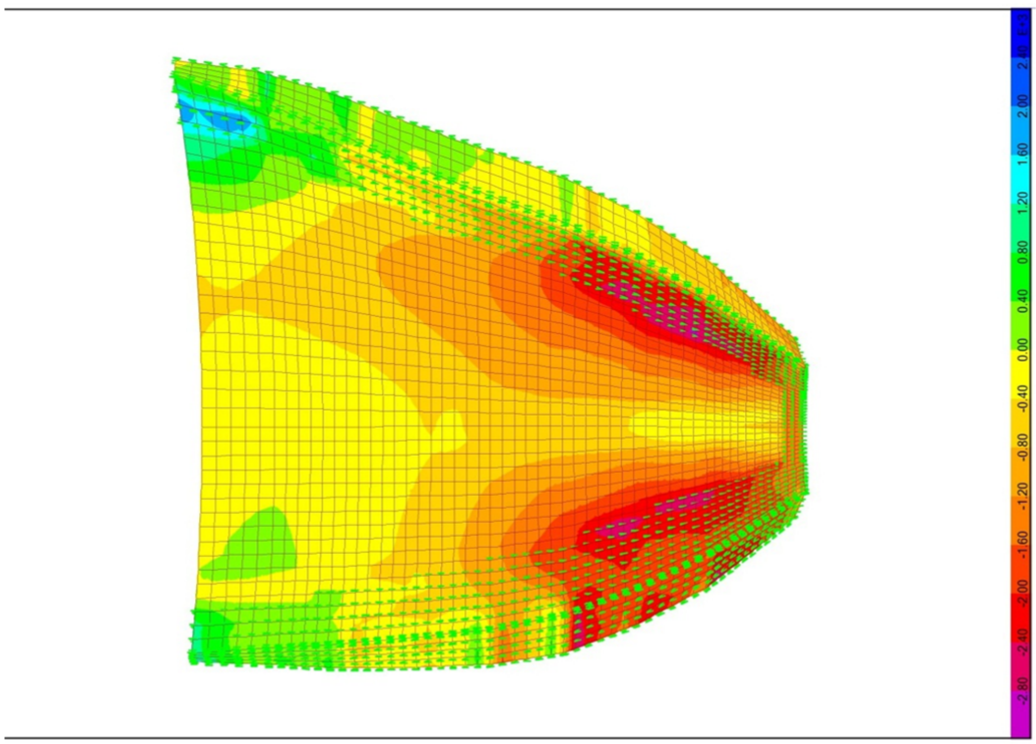
Plan view: Normal stresses with respect to local axis 2

Downstream view: Normal stresses with respect to local axis 2

Figure 5.14: Flexible foundation, S22 due to SUN1 combination

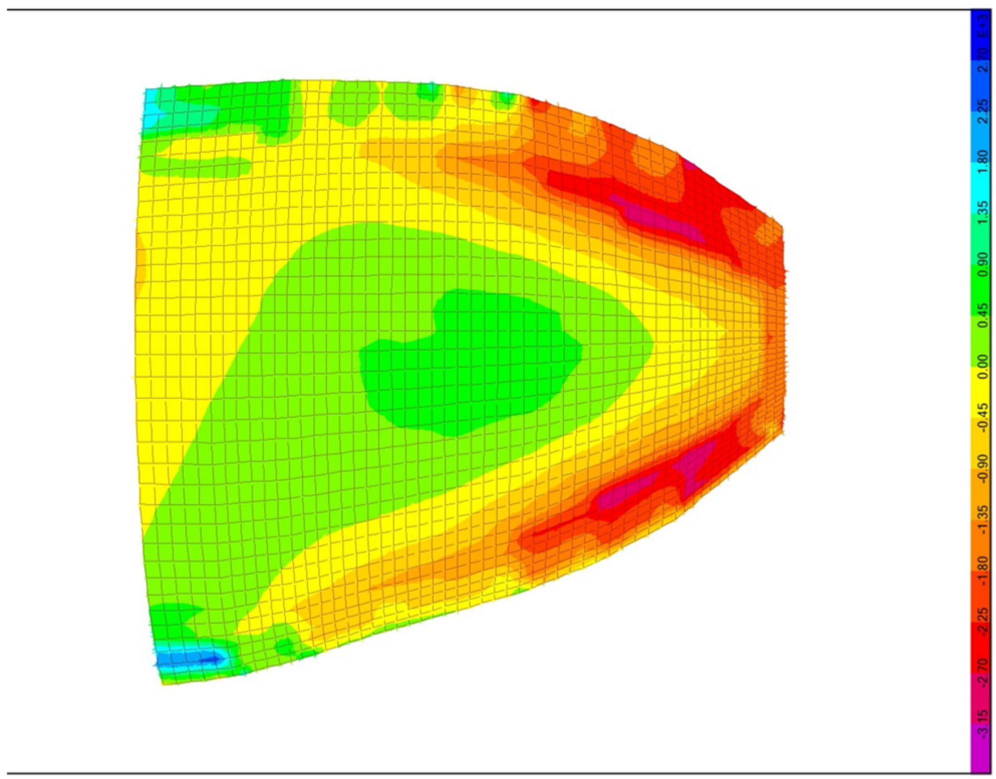


Plan view: Normal stresses with respect to local axis 2



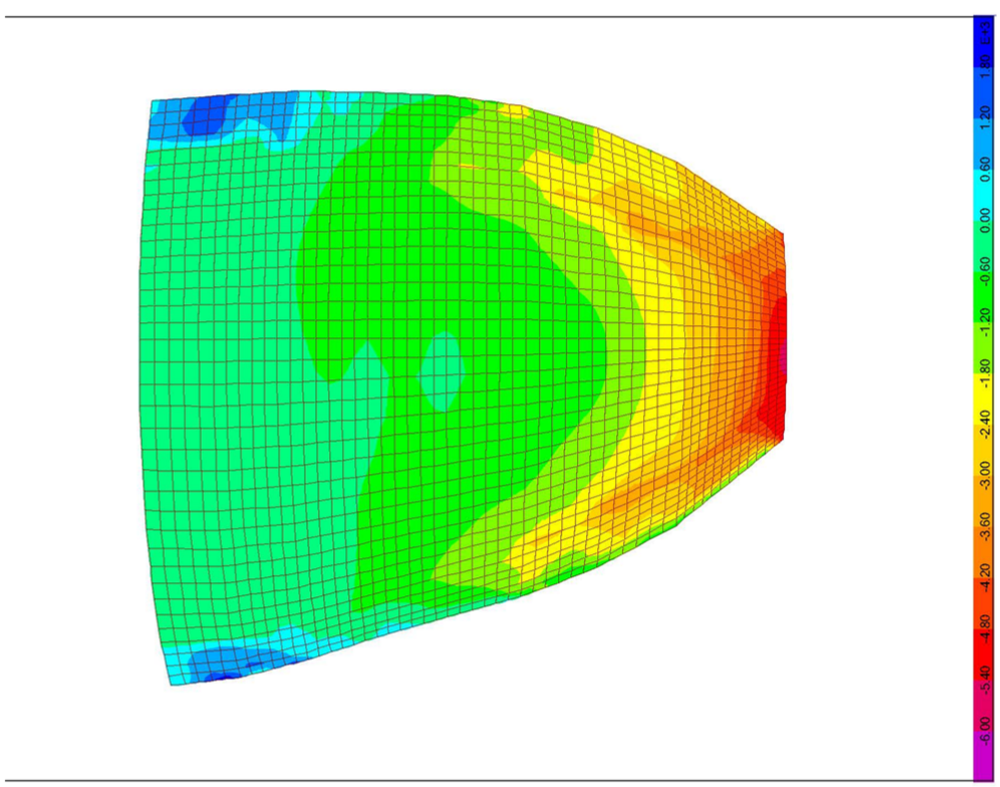
Downstream view: Normal stresses with respect to local axis 2

Figure 5.15: Flexible foundation, S22 due to SUN2 combination



Upstream view: Normal stresses with respect to local axis 2

Figure 5.16: Flexible foundation, S22 due to dam self-weight



Upstream view: Normal stresses with respect to local axis 3

Figure 5.17 :Flexible foundation, S33 due to dam self-weight

As it was expected high compression zones are located near the abutment foundation of the dam, which is due to the arch action which transfers loads to the abutment rocks. If the compression zones at the abutment were not obtained then it is required to increase the horizontal curvature of dam. Maximum normal stresses due to self-weight and hydrostatic loads are in the direction normal to the abutment. As natural local axes were defined to follow the shape of the dam, the stresses are expressed in local bases. The results for S22 and S33 are presented in the above figures. To compare these results with S11 some stress diagrams are shown in Annex.

The maximum compression stresses S22, normal to the abutment rocks, are obtained due to SUN1 loads combination. In this combination the dam is subjected to the full reservoir at maximum water level. Figures 5.10 and 5.14 present these stresses for rigid and flexible foundation, respectively. It can be seen that tensile stresses at the upper arches for this case smaller than for the other two load combinations.

In the case of minimum water level some tensile stresses S22 are obtained at the upper arches, which can be seen at figures 5.11 for rigid foundation and figure 5.15 for flexible foundation. The main compression zones of S22 are located below elevation 760 m which is the initial elevation of water level.

The maximum vertical normal stresses S33 is obtained for the self-weight load case.

The results show a high decrease in the magnitudes of obtained stresses in the case of flexible foundation. It proves that stresses distribution and magnitude within the dam is highly depended on the foundation stiffness. The more rigid foundation will cause higher stresses in the dam.

The results which are obtained for flexible foundation are of course more realistic. For the case of flexible foundation the stresses are in the acceptable ranges that were presented in section 2.6. Therefore the primary design can be accepted, but further analysis of temperature and earthquake loads is still required.

6. Conclusions

The design of an arch dam is an iterative procedure where many issues should be considered. It is important to study these issues carefully, because each of them can influence the design and results and might even cause the need to changing the shape of the dam.

In this thesis we focused on the behavior of the dam which is subjected to its self-weight and hydrostatic load of reservoir. The static analysis of the dam was done using linear elastic analysis based on finite elements implemented in SAP2000. The results of analysis allow us to observe the response of the dam in terms of deflections and stresses along the entire structure.

Arch dams are defined as structures which transfer the loads to the abutments by the arch action. This property of arch dams can be observed from the obtained stresses which were calculated for several load combinations. These results are in accordance with the expected behavior of an arch dam. The compression zones are located in both abutments of the dam. The maximum normal stress direction is perpendicular to the foundation rocks, which shows load transfer by horizontal arch to abutments.

Two boundary conditions models used in the thesis show high importance of foundation model and foundation properties on the response of an arch dam. The study of foundation rock is crucial in the design of a dam. Defining foundation as rigid will cause non-realistic results; therefore the stiffness of rocks in different zones of foundation shall be determined by field tests or geological data of the site. For accurate analysis of a dam it is recommended to model the foundation rocks together with the dam structure. Using surface springs to define the foundation flexibility in this thesis has proven as a good approximation in primary static analysis.

The design in this thesis can be used as a primary step to determine the shape of dam and if unacceptable results will be obtained from further temperature and earthquake studies, then the reshaping of dam or changing the materials properties are required.

Povzetek

Ločne pregrade so masivni betonski objekti, ki jih praviloma postavljamo v ozkih soteskah s strmimi bregovi. Zasnovane so v ločni obliki, ki omogoča da se obtežbe na pregrado prenašajo direktno v boke pregrade. Na ta način lahko s sorazmerno tanko konstrukcijo prevzemajo velike obremenitve in so zato tudi od vseh tipov pregrad najbolj ekonomične. V nalogi obravnavamo ločno pregrado, ki smo jo zasnovali na hipotetičnih podatkih. Lokacija, podatki o akumulaciji, hidrološke razmere in ostali podatki so izmišljeni, vendar izbrani smiselno in realistično. Za materialne parametre so izbrane tipične vrednosti, ki ustrezajo pogojem za postavitev ločne pregrade. V nalogi je postavljen računski model ločne pregrade, ki je temeljena na dva tipa podlage:. Ločimo povsem togo idealno podlago in realistično podajno podlago. Za oba modela je določeno deformacijsko in napetostno stanje v ločni pregradi za različne obtežne primere.

Pri načrtovanju pregrade smo se oprli na dva priročnika za načrtovanje pregrad, po priporočilih USBR (United States Bureau of Reclamation) in USACE (United States Army Corps of Engineers). Pri načrtovanju ločnih pregrad moramo upoštevati mnogo dejavnikov. Za izgradnjo ločnih pregrad so najprimernejše ozke doline ali soteske, t.i. kanjonski tip rečne doline. Pogoj za temeljenje ločne pregrade je zadostni trdna in nosilna temeljna podlaga, ki je sposobna prevzeti obtežbo vodnega tlaka. Pri tem ne smemo zanemariti podajnosti temeljene podlage in njen vpliv na napetostno stanje v pregradi. Pozornost moramo nameniti detajlu naklonskega kota stika pregrade in brežine, saj je pomemben pri razporeditvi napetosti. Višina zajezitve in pričakovane razmere v akumulaciji, skupaj z lokalnimi klimatskimi razmerami so ravno tako pomembne pri načrtovanju tovrstnih objektov.

Načrtovanje ločne pregrade je iterativen postopek, ki ga sestavljajo štirje koraki:

1. *zasnova,* kjer izberemo grobo obliko pregrade na osnovi pridobljenih podatkov, lokacije in dosedanjih izkušenj s podobnimi objekti;
2. *analiza*, kjer za izbrano zasnovo določimo napetostno in deformacijsko stanje;
3. *vrednotenje*; kjer preverimo in ovrednotimo rezultate analiz;
4. *modifikacija*; kjer spreminjamo obliko pregrade glede na dobljene napetosti.
   Postopek ciklično ponavljamo dokler ne dobimo zadovoljive geometrije objekta. Za takšno konstrukcijo potem izvedemo podrobnejše analize obnašanja pod statično, dinamično in temperaturno obtežbo ter njihovimi kombinacijami.

   Zasnova pregrade je pripravljena v programu AutoCAD na osnovi priporočil USBR in USACE. Za zasnovo moramo najprej določiti tri vrednosti: višino pregrade (H), dolžino med brežinama na dnu soteske (L1) in dolžino med brežinama na višini 0,15 H (L2). S temi podatki lahko iz empiričnih formul določimo zasnovo prečnega prereza pregrade.

Poglaviten material, iz katerega so zgrajene vodna pregrada, je beton. Zanj je potrebno določiti osnovne lastnosti, kot so trdnost, elastični modul, Poissonov koeficient, gostota, temperaturni razteznostni koeficient in druge. Natančneje jih lahko določimo eksperimentalno, za načrtovanje pa zadošča že poznavanje tipičnih lastnosti, kot jih predstavljamo v razdelku 2.4

Na ločno vodno pregrado delujejo različne obtežbe: lastna teža, hidrostatična obtežba, temperaturna obtežba, potresna obtežba, … Poleg lastne teže na pregrado deluje sila teže različnih pomožnih objektov na pregradi (hidromehanska oprema, prekladne konstrukcije, strojna oprema, …), ki pa jo

vsaj pri statični analizi lahko zanemarimo. Temperatura ima pomemben vpliv na ločne pregrade. Temperatura v betonu je odvisna od temperatur zraka in vode v akumulaciji. Porazdelitev temperatur prečno čez telo pregrade pa je odvisna od debeline pregrade. Hidrostatične obtežbe so posledica tlakov zaradi vode v akumulaciji. V ločni pregradi lahko nižji gladini vode v akumulaciji povzroči neugodna napetostna stanja na dolvodni strani. Zaradi tega je potrebno pri statični analizi, poleg maksimalne gladine obravnavati tudi minimalno gladino vode v akumulaciji, ki je običajno minimalna obratovalna gladina. Obtežbe sestavljamo v obtežne kombinacije glede na njihovo verjetnost. Ločimo običajne, neobičajne in ekstremne obtežne kombinacije. Priročnik USACE EM 1110-2-2201 predlaga nekaj obtežnih kombinacij za statično analizo kot minimalne obtežbe, ki jih je potrebno upoštevati.

V naši nalogi analiziramo obnašanje ločne pregrade pod statično obtežbo z geometrijsko in materialno linearnimi računskimi modeli. Uporabimo tridimenzionalne končne elemente, kot so vgrajeni v programskem okolju SAP2000. Zasnova geometrij objekta je v okolje SAP2000 prenesena neposredno iz zasnove pregrade v programu AutoCAD. Objekt razdelimo na mrežo končnih elementov. Osnovna ideja metode končnih elementov je namreč razdelitev zahtevnejše konstrukcije na manjše, preprostejše dele, ki jih lahko opišemo s preprostejšimi enačbami. Na nivoju vsakega elementa tako pripravimo osnovne enačbe, ki jih sestavljata matrika koeficientov in stolpec desnih strani. Zaradi njene narave matriko koeficientov imenujemo tangentna togostna matrika. Enačbe posameznih elementov potem združimo na nivoju konstrukcije, kjer upoštevamo še robne pogoje. Z reševanjem teh enačb določimo deformacijsko in napetostno stanje v konstrukciji. Za modeliranje so uporabljeni tridimenzionalni elementi z osmimi vozlišči.

Posebno pozornost namenimo modeliranju robnih pogojev. Ločimo dva modela: v prvem, preprostejšem ločno pregrado togo podpremo; v drugem, natančnejšem modelu uporabimo ploskovno porazdeljene elastične vzmeti za opis podlage. Togosti vzmeti določimo glede na lastnosti obnašanja skalne podlage. Obravnavamo tri obtežne kombinacije: (1) lastno težo brez obtežbe vode, (2) lastno težo in maksimalno gladino vode v akumulaciji ter (3) lastno težo in minimalno gladino vode v akumulaciji. Spremljamo pomike konstrukcije in napetostno stane v konstrukciji za oba računska modela podpor. Rezultati kažejo enakomernejšo porazdelitev pomikov pri podajni podlagi ob manjših napetostih ob podporah.

Dobljene napetosti primerjamo z dopustnimi vrednostmi. Za obravnavane obtežne kombinacije obravnavana pregrada zadošča zahtevam. Tako rezultati, predstavljeni v temu delu zadoščajo kot prvi korak k načrtovanju ločne pregrade. Analizo temperaturnih vplivov in potresne obtežbe lahko izvedemo na tako zasnovani konstrukciji, ki jo v primeru neustreznosti spremenimo ter ponovimo postopke analize.

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Annex

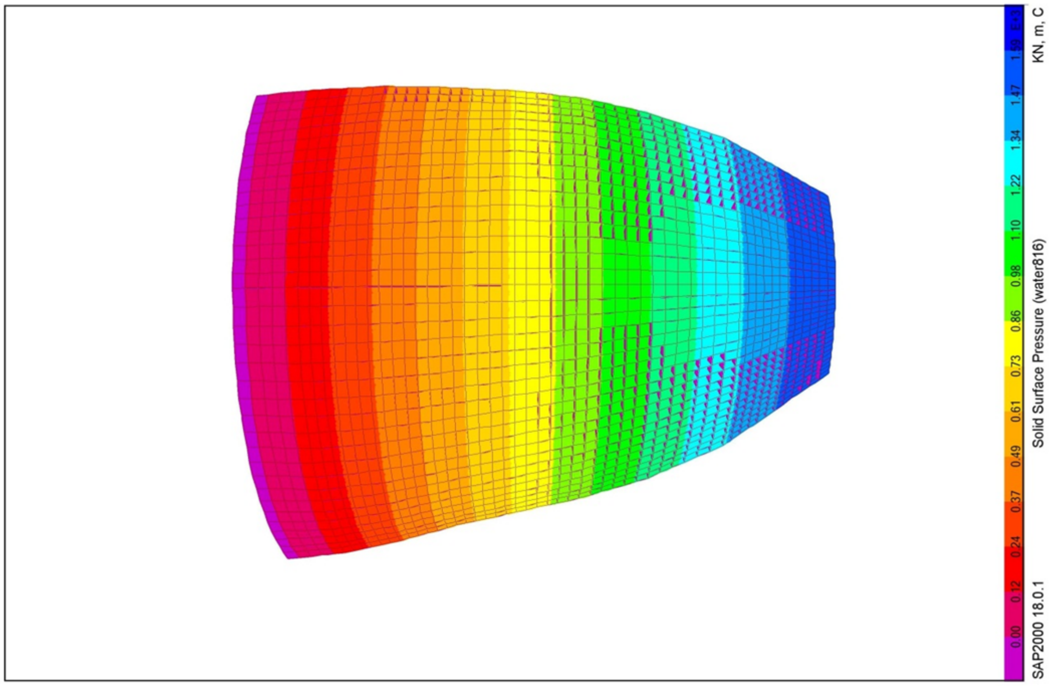

Figure. A1: Water pressure diagram on upstream face for maximum water level

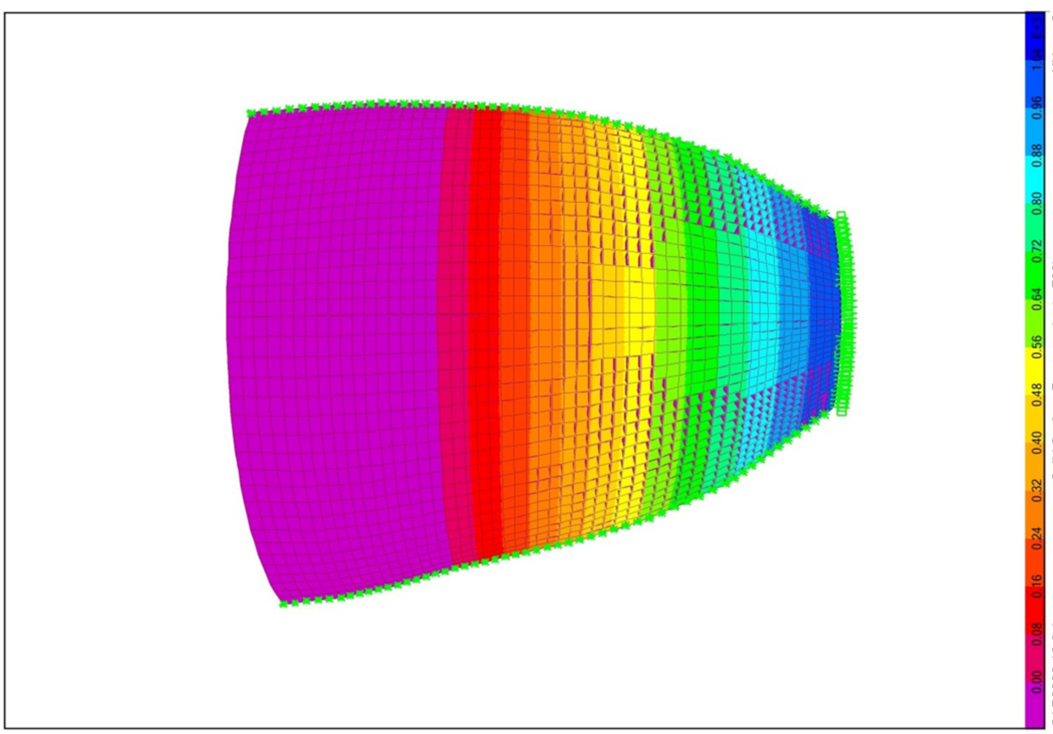

Figure. A2: Water pressure diagram on upstream face for minimum water level

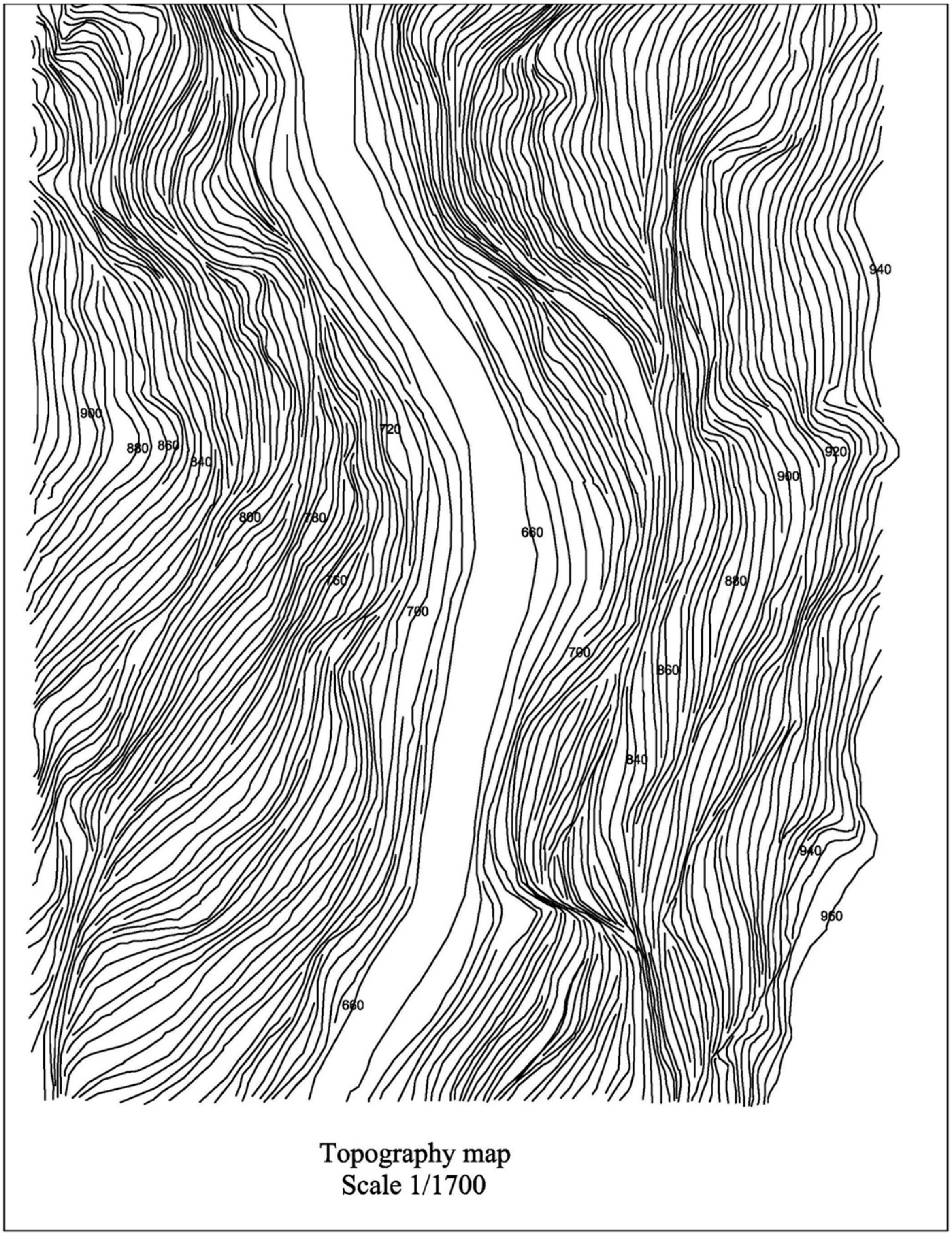

Figure A3: Topography map of the canyon

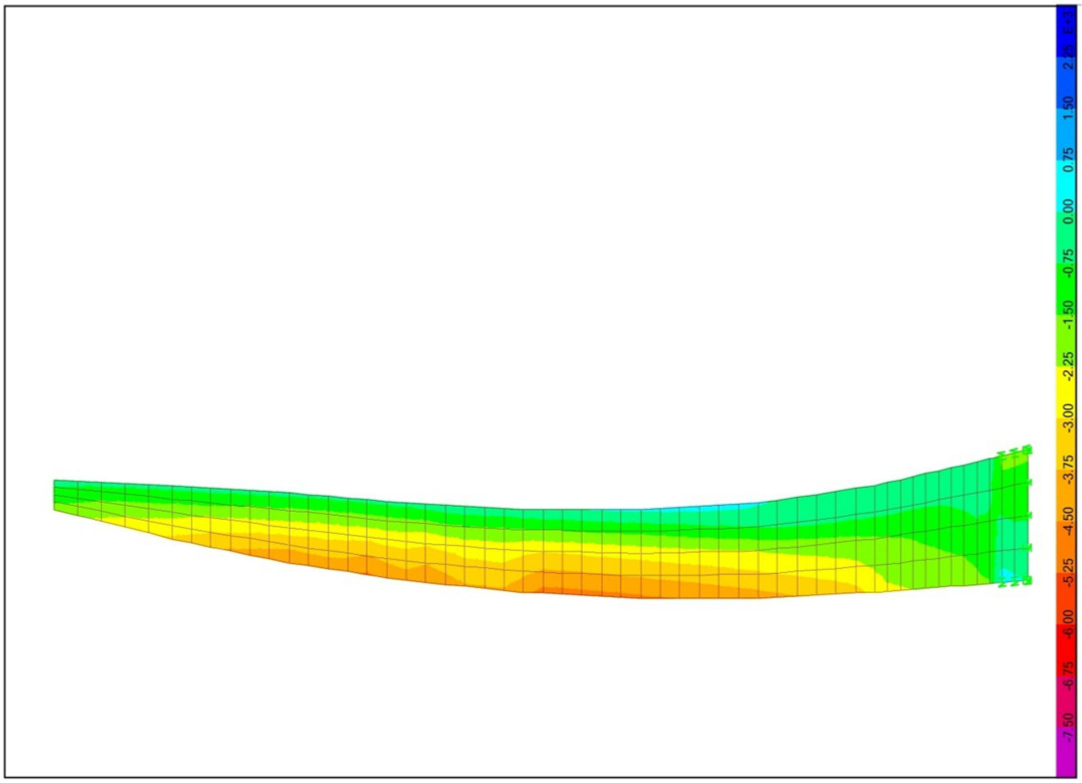

Figure. A4: Flexible foundation, S22 due to SUN1 combination

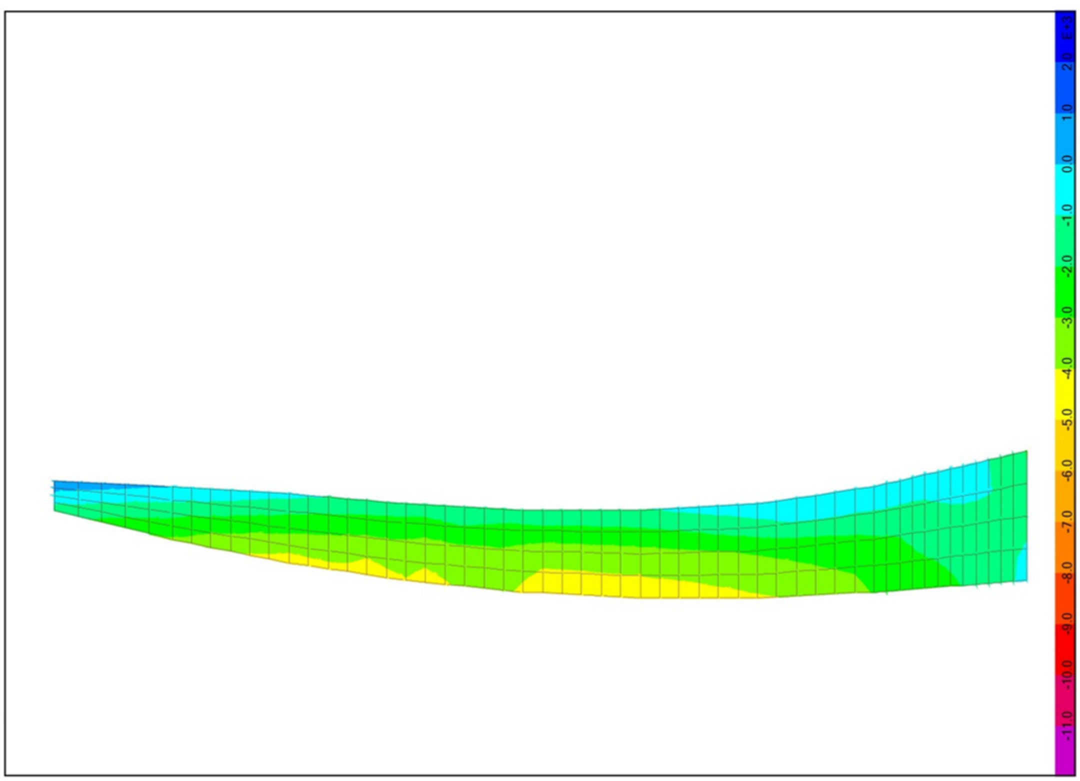

Figure. A5: Rigid foundation, S22 due to SUN1 combination

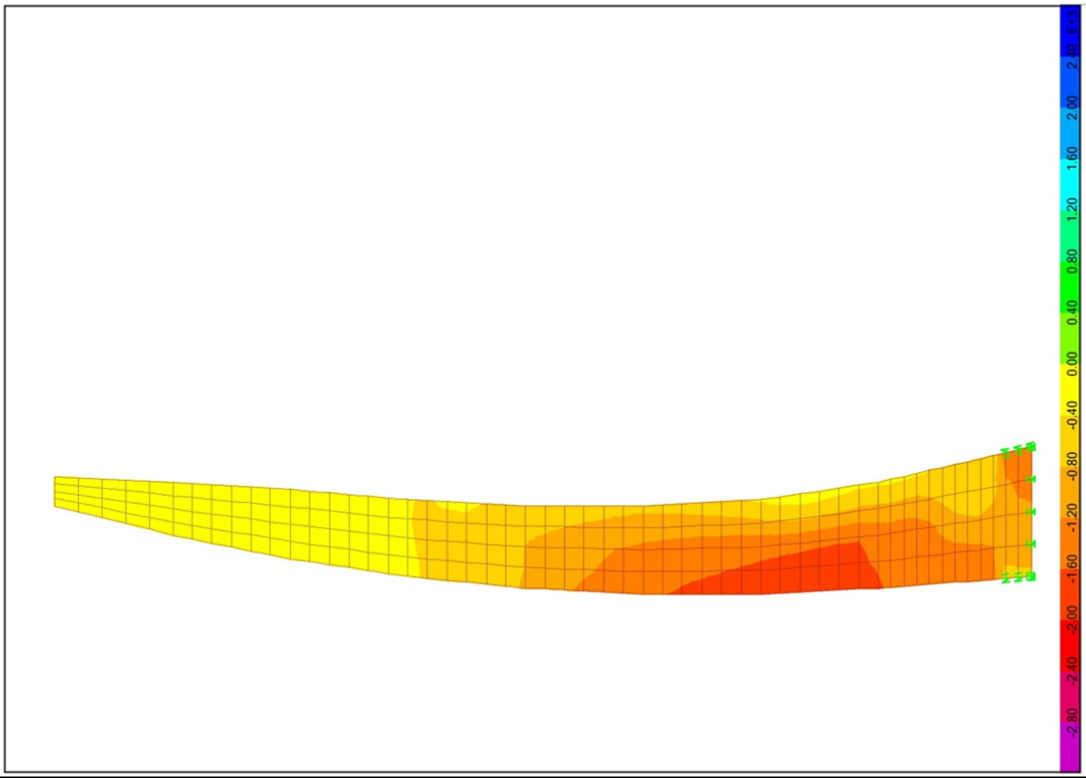

Figure. A6: Flexible foundation, S22 due to SUN2 combination

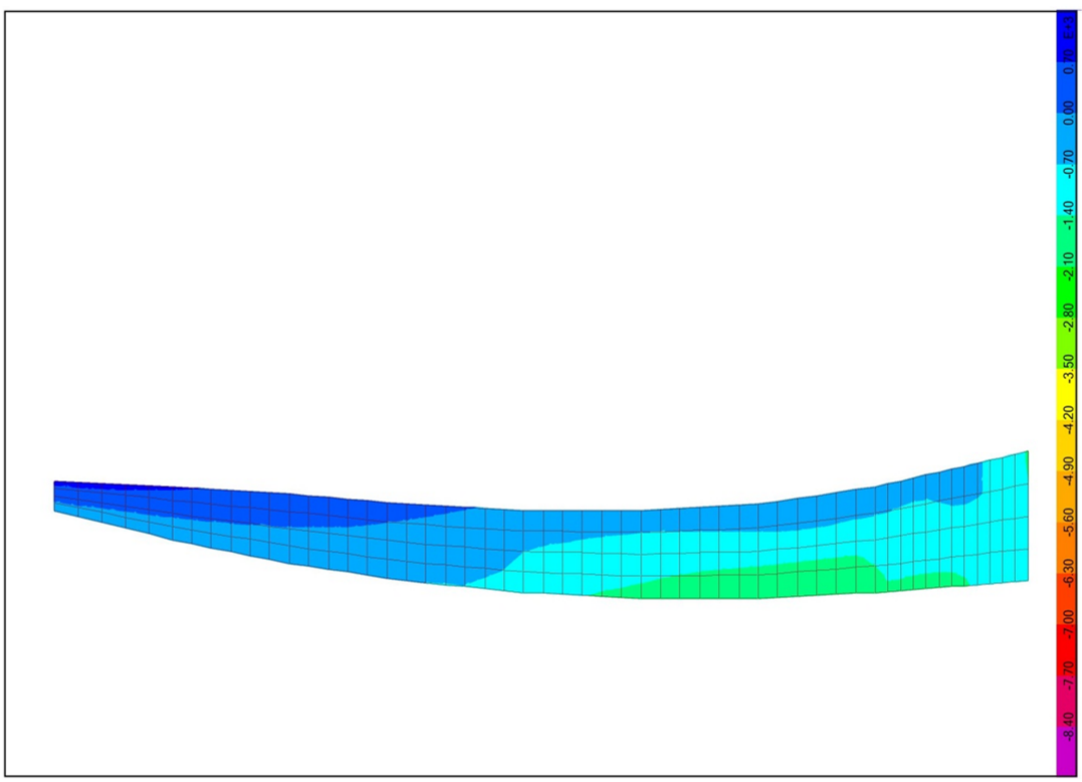

Figure. A7: Rigid foundation, S22 due to SUN2 combination

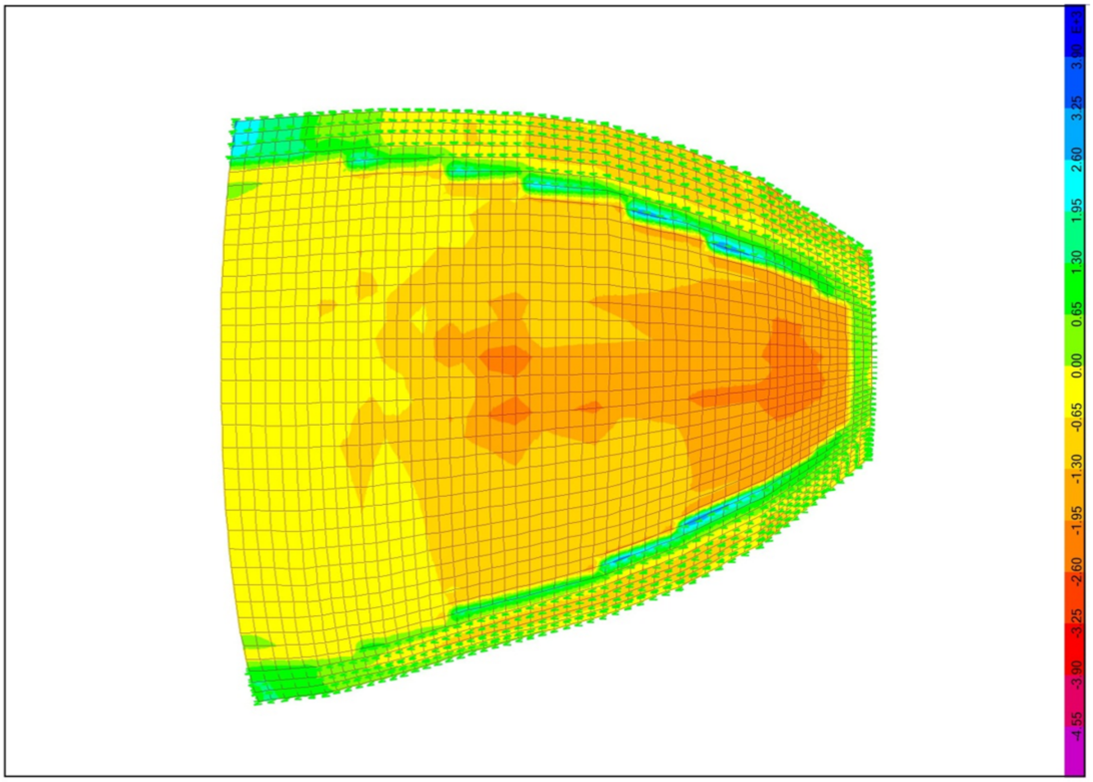

Figure. A8: Flexible foundation, S11 due to SUN1 combination

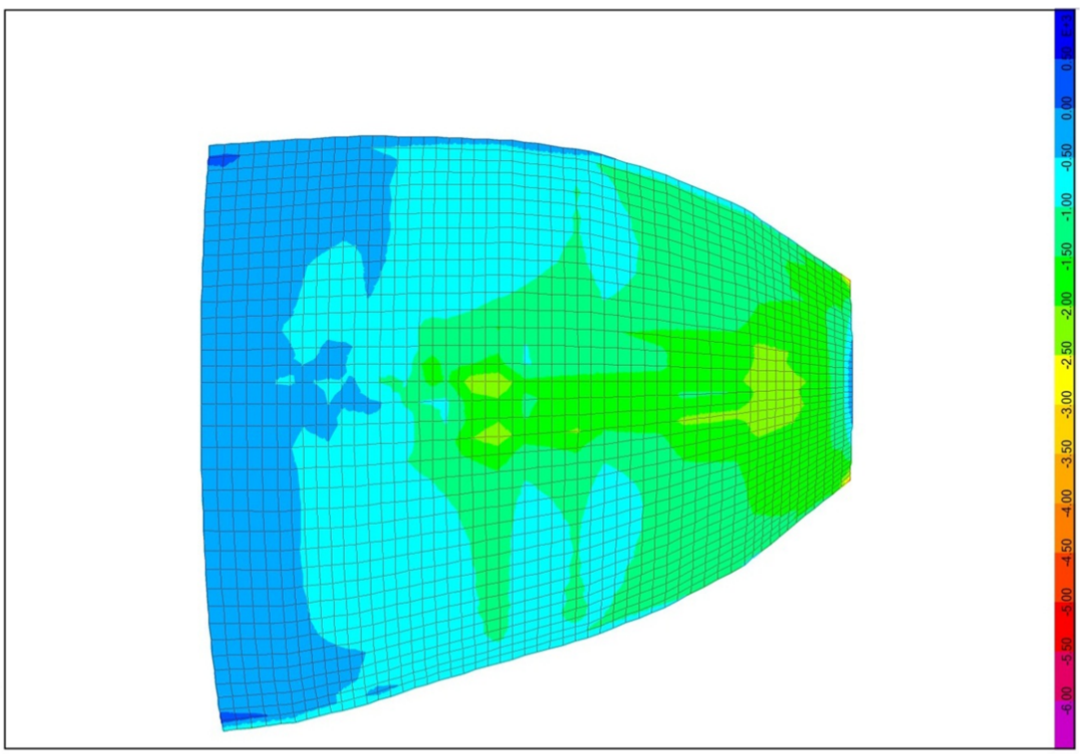

Figure. A9: Rigid foundation, S11 due to SUN1 combination

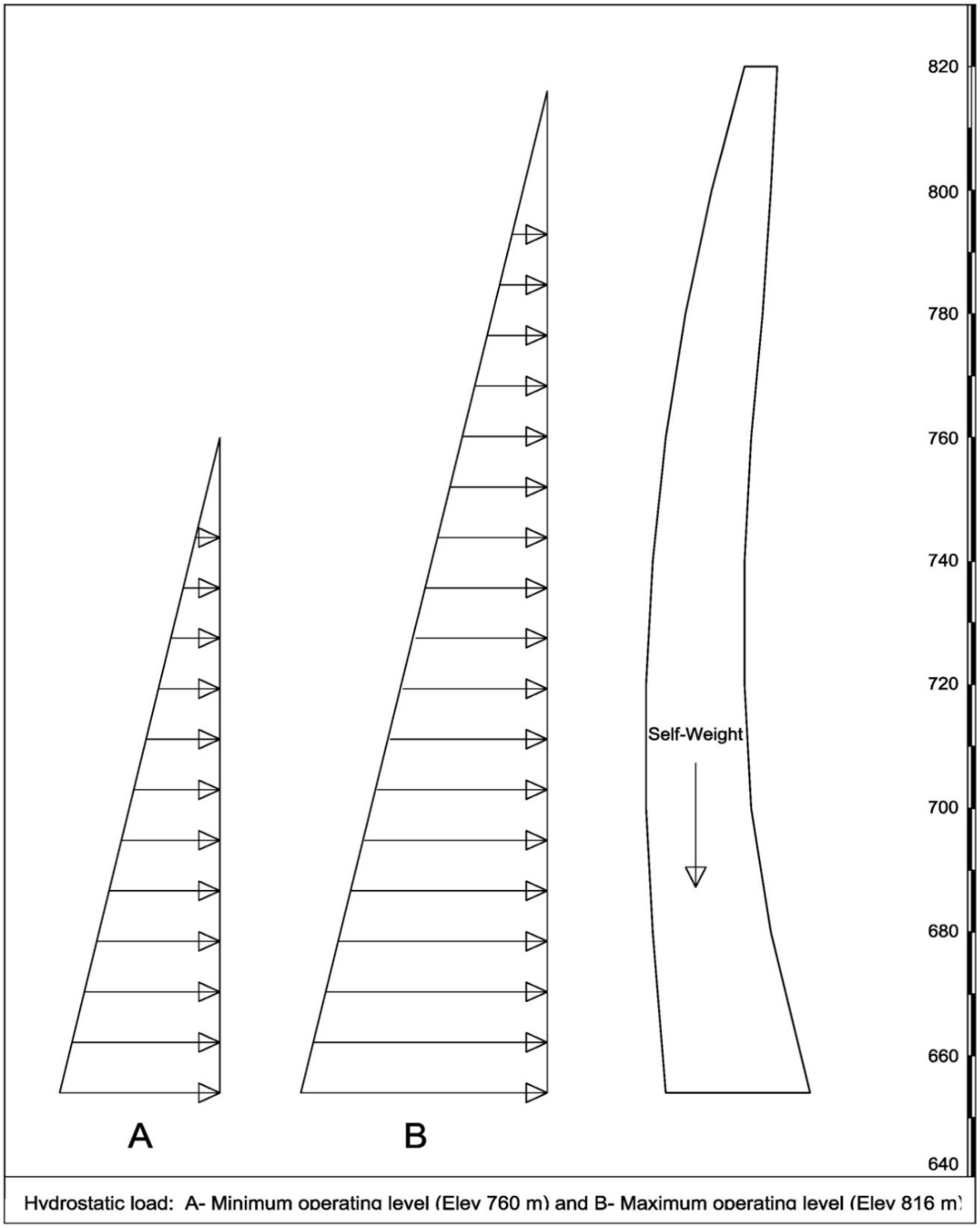

Figure A10: Hydrostatic loads on the arch dam