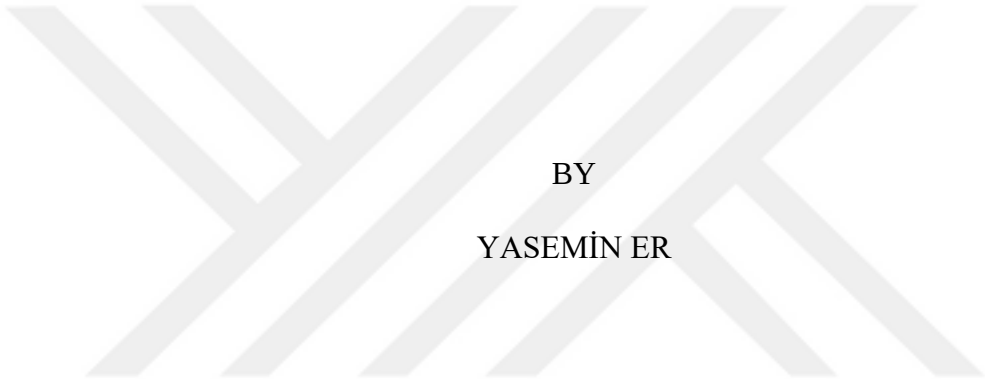


PREDICTED AND OBSERVED DEFORMATION BEHAVIOUR OF DERİNCE
DAM

A THESIS SUBMITTED TO
THE GRADUATE SCHOOL OF NATURAL AND APPLIED SCIENCES
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DERİNCE DAM**

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ABSTRACT

PREDICTED AND OBSERVED DEFORMATION BEHAVIOUR OF DERİNCE DAM

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Concrete face rockfill dams (CFRDs) have gained global popularity due to their stability and efficient construction, particularly in regions lacking impermeable materials and in humid environments. In CFRDs, alluvial materials can be used as the main fill instead of traditional rock materials, leading to the evolution of concrete face sand-gravel dams (CFSGDs). These CFSGDs are modified versions of CFRDs, incorporating design variations to accommodate different fill materials. This study examines the behavior of the Muğla Milas Derince Dam, constructed on approximately 15-meter-deep alluvial deposits, during its construction and impoundment stages. Two-dimensional plane strain finite element analyses using Plaxis v20 are conducted to investigate the dam behavior. The hardening soil model is utilized to represent the inelastic, non-linear, and stress-dependent behavior of the sand-gravel fill material and alluvium strata. The Hoek-Brown failure criterion is adapted for the bedrock to estimate foundation parameters. Preliminary analyses include evaluating previous studies of similar dams to select embankment parameters. After calibration of material parameters with settlement data from

settlement cells in the dam body, the dam's behavior during the impoundment stage is predicted. The observed values align well with the settlements for both stages.

Keywords: Concrete Face Sand-Gravel Dams, Finite Element Method, Dams Constructed on Alluvial Deposits, Settlement, Stress



ÖZ

DERİNCE BARAJI'NIN TAHMİN EDİLEN VE GÖZLEMLENEN DEFORMASYON DAVRANIŞI

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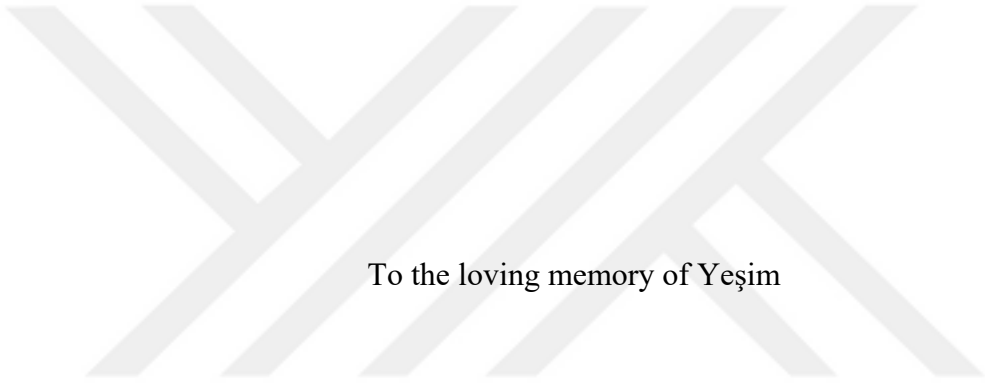
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Ön yüzü beton yüzü kaya dolgu barajlar (ÖYBK), özellikle geçirimsiz malzemelerin az olduğu ve nemli çevrelerin bulunduğu bölgelerde, stabilite ve verimli inşaat süreçleri nedeniyle dünya çapında popülerlik kazanmıştır. Ön yüzü beton kaplı kaya dolgu barajlarda geleneksel kaya malzemeleri yerine gövde dolgusu olarak alüvyonal malzemeler de kullanılabilir, bu da ön yüzü beton kaplı kum-çakıl barajların (ÖYBKÇ) ortaya çıkmasına yol açmıştır. ÖYBKÇ'ler, farklı dolgu malzemelerini barındıracak şekilde tasarım varyasyonlarını içeren ÖYBK'ların modifiye edilmiş versiyonlarıdır. Bu çalışma, yaklaşık 15 metre derinlikteki alüvyon üzerine inşa edilen Muğla Milas Derince Barajı'nın, inşaat ve dolgu aşamaları arasındaki davranışını incelemektedir. Barajın davranışını incelemek için Plaxis v20 kullanılarak iki boyutlu düzlem gerilme sonlu analizleri gerçekleştirilmiştir. Kum-çakıl dolgu malzemesinin ve alüvyon tabakasının elastik olmayan, doğrusal olmayan ve gerilime bağlı davranışını temsil etmek için sertleşen zemin modeli kullanılmıştır. Anakayanın parametreleri tahmin etmek için Hoek-Brown Yenilme Kriteri kullanılmıştır. Ön analizlerde, benzer barajların önceki çalışmalarını değerlendirilerek dolgu gövde parametreleri seçilmiştir. Baraj gövdesindeki hidrolik

oturma ölçerlerden alınan oturma verileri ile malzeme parametrelerinin kalibre edilmesinin ardından, dolgu aşaması sırasında barajın davranışı tahmin edilmiştir. Hesaplanan değerler, her iki aşama için gözlemlenen oturmalar ile iyi bir uyum içinde bulunmuştur.

Anahtar Kelimeler: Ön Yüzü Beton Kaplı Kum-Çakıl Barajlar, Sonlu Elemanlar Yöntemi, Alüvyon Üzerine İnşa Edilmiş Barajlar, Oturma, Gerilme





To the loving memory of Yeşim

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Finally, a note to myself, from the past to the future: Γνώθι σεαυτόν (Know Thyself).



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LIST OF ABBREVIATIONS

ABBREVIATIONS

CFRD	: Concrete face rockfill dam
CFSGD	: Concrete face sand-gravel fill dam
DSI	: General Directorate of State Hydraulic Works
ECSGD	: Earth core sand gravel dam
EOC	: End of Construction
FEM	: Finite Element Method
ICOLD	: International Commission on Large Dams

CHAPTER 1

INTRODUCTION

Dam construction dates to ancient civilizations, and it has played a crucial role in the development of human civilizations. Initially, they were built for water storage, irrigation, and flood control, but with time, their functions have expanded to include energy generation, recreation, and even aesthetics.

The need for new dam types has arisen from both construction and environmental challenges. Traditional dams primarily use impermeable materials like clay as their core to prevent seepage. However, such traditional designs are often ineffective in areas that lack these impermeable materials or that experience humid conditions, including extreme cold and humidity.

Additionally, the constraints of tight schedules and budgets necessitate solutions that are both efficient and robust. These combined pressures have pushed engineers to innovate and design alternative types of dams. Concrete face rockfill dam (CFRD) is a type of dam where a concrete slab on the upstream side ensures water tightness and rockfill material provides structural stability. Recently, there has been a global increase in the construction of CFRDs. This rise in popularity is attributed to their inherent stability and efficient construction methodologies. Furthermore, the requirements and improvements for the foundation of CFRDs are less tight and more straightforward than those for concrete gravity or arch dams.

Starting in the 19th century, the designs of CFRDs relied heavily on the engineer's judgment. Instrumentation in these dams is crucial. The data from instruments along with observed behaviors during both the construction and impoundment stages provide design engineers with essential guidance for upcoming projects.

With the requisite modifications, sand-gravel materials can also be utilized to construct CFRD dams. These variants are known as concrete face sand-gravel dams (CFSGDs). Despite this distinction, they are often still classified under the broader category of CFRDs. In practice, while some refer to them specifically as CFSGDs, it is common for them to be generally identified as CFRDs.

CFSGD offers several advantages over CFRD. These benefits include being more economical, primarily due to lower excavation and loading costs, as well as reduced wear on tires and rollers. Additionally, CFSGDs have a higher modulus of elasticity, resulting in less displacement, making them particularly suitable for higher dams. However, if a significant amount of fines content is present, adjustments such as incorporating chimney drains, abutment drains, filters, and intermediate drainage layers are necessitated.

Since the 1960s, the finite element method has been employed for dams. As with all types of dams, this method is crucial in predicting the static and dynamic behavior of CFRDs. Engineers using this method have enriched their design expertise through the verification of data from instrumentation in dams.

1.1. Objective of the Study

The objective of this study is to align the observed and calculated settlements of the Concrete Face Sand-Gravel Derince Dam. This will be achieved by adjusting the constitutive models and determining the optimal parameters for this dam. Additionally, the study aims to contribute to the academic literature by proposing new parameters for sand-gravel fill materials used in similar dam constructions.

1.2. Scope of the Study

This study focuses on the behaviour of the 62.5-meter-high Derince Dam in Muğla Province, constructed on approximately 15-meter-deep alluvial deposits and primarily intended for irrigation. The analyses presented cover both the construction and impoundment stages of the dam. During the construction phase, data from settlement cells is utilized to analyze and calculate the material parameters of the

dam body. These parameters are then used to predict the dam's behavior during the impoundment stage. Two-dimensional analyses are conducted using the finite element method with Plaxis v20 software. In these analyses, the *Hardening Soil Model* is applied to all sand-gravel fill and alluvium strata, representing their inelastic, nonlinear, and stress-dependent behavior. The Hoek-Brown failure criterion is employed to estimate the bedrock parameters in this study. Finally, the results from these analyses are compared with observed data collected from the instrumentation.





CHAPTER 2

CONCRETE FACE ROCKFILL DAMS AND CONCRETE FACE SAND- GRAVEL DAMS

Concrete face rockfill dams (CFRDs) have gained popularity worldwide due to their stability and efficient construction, especially in regions with scarce impermeable materials and humid environments. Recently, the decision to select a dam type has often taken into consideration economic factors and safety concerns. However, it is essential to note that modern Concrete Face Rockfill Dams (CFRDs) are designed to balance both these aspects effectively. They are recognized for their cost-efficiency while also adhering to rigorous safety standards, making them a popular choice in diverse environmental and geographical conditions.

2.1. General

One of the earliest definitions of rockfill dams specified in the 1939 ASCE Symposium on Rockfill Dams is as follows: *“a dam consisting of loose (dumped) rockfill with slopes on both faces closely approximating natural slopes, with an impervious facing on the upstream side between which and the rockfill there should be placed a cushion of dry rubble”* (Galloway, 1939).

In 1960, the ASCE Symposium on Rockfill Dams has adopted the present-day definition of rockfill dams, according to which a rockfill dam is *“a dam relies on rock, either dumped in lifts or compacted layers, as a major structural element.”*

The impermeability element of rockfill dams is achieved by using an impervious material placed on the upstream face or by incorporating an impervious core (Singh et al., 1995).

The choice of the type and placement of the impervious component in dam construction is influenced by various factors. These include the availability of materials for construction, the prevailing climatic conditions, the construction timeline, and the level of seismic activity in the area (Sherard et al., 1963).

Dams made of rockfill and equipped with upstream face membranes are built using rockfill that includes different sizes of gradation. The waterproof aspect of these dams is achieved through a facing on the upstream side, which is typically made of cement-concrete, or materials based on asphalt. Occasionally, membranes made of steel or timber have also been used (Singh et al., 1995).

The earliest documented example of a rockfill dam with a impervious membrane is the La Granjilla Dam, built in Spain circa 1660. Standing at a height of 13 meters, it employed a mix of mortar and lime to create its impervious facing (Cruz et al., 2009).

2.2. Historical Summary of Rockfill Progress

The subsequent section of this thesis, following an overview of rockfills, delves into a comprehensive historical overview of rockfill dam technology. This part meticulously examines the evolution of Rockfill Dams, segmenting their historical progression into three specific stages initially delineated by Cooke in 1984.

The development of rockfill dams can be divided into three distinct periods: the early stage spanning 1850 to 1940, the intermediate phase between 1940 and 1965, and the modern era, beginning in 1965 and continuing up to the current day (Cooke, 1984).

1-) Early Period (1850-1940)

Rockfill dam construction in the United States originated in California's Sierra Nevada Mountains. Initially, these dams were constructed for the purpose of water storage, predominantly to support operations in gold mining, encompassing exploration and processing activities. Comprised of dumped rockfill and equipped with timber facings, these structures achieved heights of up to 25 meters and were characterized by notably steep slopes, ranging from 0.5H:1V to 0.75H:1V. Although

the overall performance of these concrete-faced dumped rockfill dams was satisfactory, they began to experience increasing leakage issues, particularly as their heights were extended. The Chatsworth Park Dam, established in California in 1895, is the earliest documented example of a rockfill dam incorporating a concrete face for impermeability. Despite facing leakage challenges, the Salt Springs Dam, operational since 1931, has maintained its functionality (Cooke, 1984).

2-) Transition Period (1940-1965)

The widespread use of dumped rockfill with a concrete face revealed several limitations and potential issues, particularly when constructing high dams (those exceeding 91 meters in height). During this period, it was believed that dumped rockfill needed to possess high unconfined compressive strength. The layers of rockfill used in construction typically ranged in thickness from 18 to 60 meters. Severe leakage issues were common in these kinds of dumped rockfill dams, often resulting from a significant amount of settlement during the initial filling of the reservoir (Cooke, 1984).

3-) Modern Period (1965-)

Between 1955 and 1965, the demand for higher and more sophisticated concrete face rockfill dams (CFRDs) precipitated a transition from dumped to compacted rockfill. This shift was catalyzed by introducing vibratory rollers and using less durable materials, leading to more compact rockfill layers, thinner concrete facings, reduced settlement amounts, and reinforced concrete faces. In this period, the compaction of rockfill emerged as a standard practice in dam construction. However, dumped rockfill continued to be used to construct outer shells and cofferdams (Cooke, 1984). Table 2.1. presents significant dams categorized according to their early, transitional, and modern historical periods.

Table 2.1. Historical Summary of CFRDs (Cooke, 1984)

Periods in Evaluation of Dam Technology	Year	Height (m)	Number or Names
1-) Early Period (1850-1940)			
Dumped rockfill	1850	22.86	Many timber face
	1925	30.48	8 dams
	1925	83.82	Dix River
	1930	100.58	Salt Springs
	1930-1940	60.96	Many
2-) Transition Period (1940-1965)			
Dumped rockfill	1950-1955	60.96	A number
		60.96-91.44	PG&E dams
	1955	112.776	Paradela
	1965	149.35	Exchequer
Compacted	1955-1965	60.96	Several
3-) Modern Period (1965-1984)			
Compacted rockfill	1965-1970	60.96-91.44	Many
	1971	97.536	Cethana
	1974	140.21	Anchicaya
	1980	161.55	Areia

2.3. Design of Concrete Face Rockfill Dams

As previously mentioned, the design of Concrete Face Rockfill Dams (CFRDs) has traditionally been predominantly based on empirical methods. These methods draw from practical experience and expert judgment. In this section, detailed information regarding the design of CFRDs will be provided.

2.3.1. Toe Slab (Plinth)

The plinth, also known as the toe slab, serves as the integral link between the foundation and the face slab. Essential aspects of the plinth's design include its dimensions, stability, construction techniques, and the treatment applied to the foundation. Typically, the dimensions of the plinth are informed by historical precedents and may vary in response to the water levels in the reservoir and the conditions of the foundation. For foundations constituted of solid rock that are amenable to grouting and exhibit erosion resistance, the widths of plinths have ranged from 3 meters to in excess of 10 meters. Such variations in width are designed to create a hydraulic gradient beneath the foundation slab, generally ranging between 15 to 20. Commonly, the minimum thickness of a plinth in design spans from 0.3 to 0.4 meters. The following formula is used to specify the thickness of the plinth.

$$T=0.3+0.003H \quad (2.1)$$

where T (m) is the thickness of plinth and H (m) is reservoir head. To simplify the construction process, generally a uniform thickness is selected.

Foundation conditions are categorized into four types, ranging from highly competent rock to completely decomposed residual soil. The width of the plinth varies from 1/18 of the reservoir's head above the plinth in the most favorable ground conditions, to 1/3 in the least stable ground conditions. Table 2.2. gives the selection criteria for toe slab width.

Table 2.2. Foundation Criteria for Selection the Width of Toe Slab (ICOLD, 141)

<i>A</i>	<i>B</i>	<i>C</i>	<i>D</i>	<i>E</i>	<i>F</i>	<i>G</i>	<i>H</i>
I	Non-erodible	1/18	> 70	I to II	1 to 2	< 1	1
II	Slightly erodible	1/12	50 to 70	II to III	2 to 3	1 to 2	2
III	Erodible	1/6	30 to 50	III to IV	3 to 5	2 to 4	3
IV	Highly Erodible	1/3	0 to 30	IV to VI	5 to 6	> 4	4

A = Foundation Type

B = Foundation Class

C = Minimum Ratio: Plinth Width/Depth of Water, full reservoir

D = Rock Quality Designation, RQD, in %

E = Weathering Degree: I equals sound rock; VI equals residual soil

F = Consistency Degree: 1 equals very hard rock; 6 equals friable rock

G = Weathered Macro Discontinuities per 10 m

H = Excavation Classes:

1. Needs explosive blasting.
2. Demands heavy-duty rippers; occasional blasting required.
3. Excavatable using light rippers.
4. Can be dug out using a bulldozer blade.

2.3.2. Concrete Face Slab

In a concrete face rockfill dam (CFRD), the essential water barrier is constituted by concrete face slabs poured on the rockfill's support zones. These slabs rest entirely on the rockfill below and predominantly experience compressive forces due to the load of the reservoir. However, areas near the dam abutments may experience tensile strains. As a result, recent design strategies for face slabs have shifted focus towards enhancing watertightness and long-term durability rather than primarily emphasizing structural strength. Additionally, there has been a growing emphasis on detecting and managing crack formation in the face slabs (ICOLD, 2010).

Figure 2.1. depicts the cross-sectional deformation of a CFRD concrete face slab under reservoir loadings. The figure shows that the deformation of the CFRD face slab is consistent with the deformation of the rockfill body beneath it. This highlights the necessity of carefully selecting, positioning, and compacting the rockfill materials that support the face slabs to minimize extreme deformations and prevent cracks. (ICOLD, 2010).

Modern Concrete Face Rockfill Dams (CFRDs) benefit from a layer of well-compacted and well-graded crushed rock, providing consistent support under the load of the reservoir. This characteristic has enabled designers of CFRDs to opt for more cost-effective thicknesses in the design of the face slabs. The current practice regarding face slab thickness is detailed in Table 2.3.

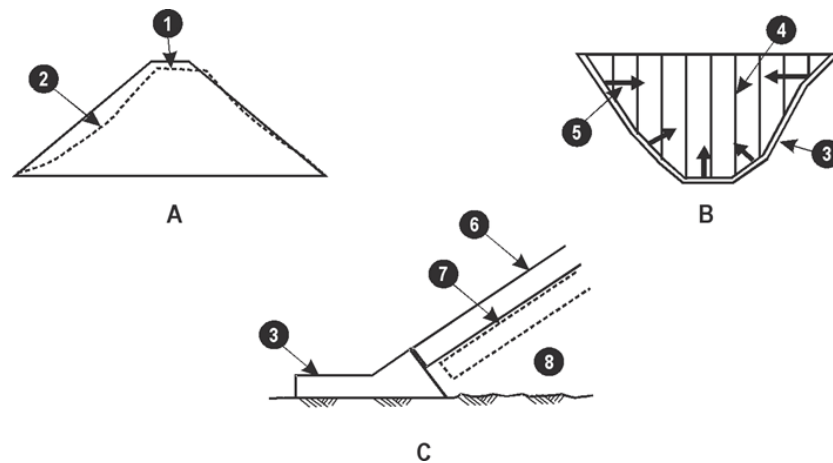


Figure 2.1. CFRD Fill and the Behaviour of Concrete Face Slab (ICOLD, 2010)

A	Embankment deformations under water load	1	Crest settlement
B	Movements in plane of face slab	2	Face settlement
C	Face displacement at perimetric joint	3	Plinth
		4	Face joints

In CFRDs, different types of joints, such as the perimeter joint, vertical joints, and horizontal construction joints, play a key role in dividing adjacent panels of the face slab. The perimeter joint distinguishes the plinth from the face slab, while vertical joints split panels along the dam's axis. Horizontal joints are employed to either segregate multiple pours within a single slab or to demarcate starter slabs from the principal face slab (ICOLD, 2010).

Table 2.3. Concrete Face Thickness (ICOLD, 2010)

Head of Water	Face Slab Thickness
Over 100 meters	30 cm + 0.2H* to 30 cm + 0.4H*
50-100 m	30 cm
Less than 50 meters	30 cm

* H = Head of water above toe slab in meters

2.3.3. Perimeter Joints

In CFRDs, the perimeter joint serves as the vital connection between the concrete face slab and the toe slab, thus completing the upstream water barrier of the dam. This joint is crucial in maintaining a watertight seal against the entire reservoir load. It also allows for the anticipated movements between the toe and face slabs (ICOLD, 2010).

The toe slab is securely anchored to the rock or, in some instances, constructed on compressible alluvium, providing a stable base. The face slab, positioned on the dam's rockfill, undergoes movement and deformation due to the settling of the rockfill beneath it. Figure 2.2. illustrates the potential movements of the face slab about the plinth in three ways: normal to the perimeter joint, leading to an opening; normal to the slab itself, causing settlement; and parallel to the perimeter joint, resulting in shear. Any movement in these directions could lead to the detachment of the face slab from the toe slab (ICOLD, 2010).

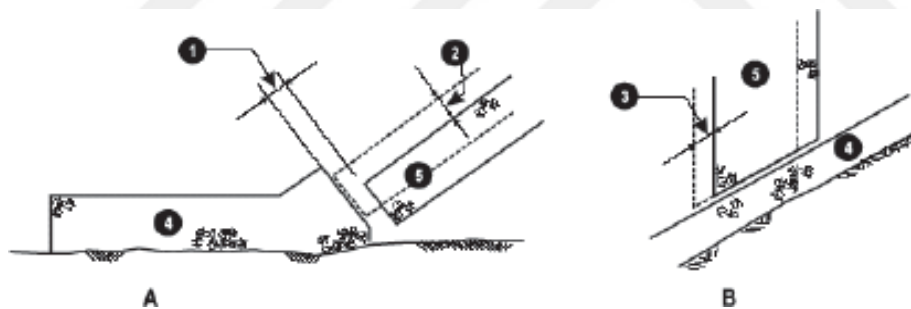


Figure 2.2. Movements of Perimeter Joints in Three Ways (Pinto & Mori, 1988)

- | | | | |
|---|---------------------------------|---|-----------|
| A | Section through perimeter joint | 3 | Shear |
| B | Profile along perimeter joint | 4 | Toe Slab |
| 1 | Opening | 5 | Face Slab |
| 2 | Settlement | | |
| 3 | Shear | | |

2.3.4. Foundation

The ideal foundation rock for construction should be erosion-resistant; in cases where it is not, specific measures are required to prevent piping and erosion

(Fitzpatrick et al., 1985). For foundations consisting of hard, solid, non-erodible rock, the standard excavation process for the plinth involves several steps:

- Removing any soft materials found in the rock's fissures and openings,
- Creating a detailed geological map of the foundation area for the plinth as a construction record,
- Cleaning the excavated surfaces with air and high-pressure water, or using only air if there is a risk of water damaging the rock surface,
- Filling in cracks, joints, fractures, and voids with concrete, either during the placement of the foundation concrete below the plinth or concurrently with the placement of the toe slab itself.

2.3.5. Parapet Wall

Unlike earth core rockfill dams, CFRDs commonly include a concrete parapet wall at the upstream edge of the crest. The primary purpose of this wall is to reduce the requirement for rockfill. This decrease in rockfill volume becomes more pronounced as the height of the dam increases, especially in regions where the valley expands downstream from the dam's axis. While a single parapet wall is standard in these dams, further reduction in rockfill can be achieved by installing a double parapet wall, with one located at both the upstream and downstream edges of the crest.

2.3.6. Embankment Zones and Properties

Figure 2.3. illustrates the sectional layout of the CFRD and the zone numbering system introduced by Cooke and Sherard (1987). This zoning scheme has been widely adopted in many countries. A concise description of these zones is given in Table 2.4.

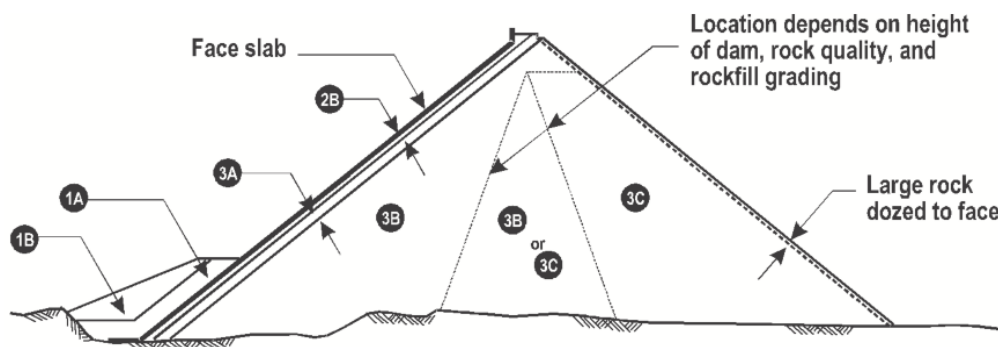


Figure 2.3. Sectional Layout of CFRD (ICOLD, 2010)

- | | | | |
|----|--------------------------------|----|----------------------------------|
| 1A | Cohesionless fine-frained soil | 3A | Selected rockfill - 0.4 m layers |
| 1B | Random fill | 3B | Quarry run rockfill - 1 m layers |
| 2A | Perimeter zone filter | 3C | Quarry run rockfill - 2 m layers |
| 2B | Processed minus 75 mm | | |

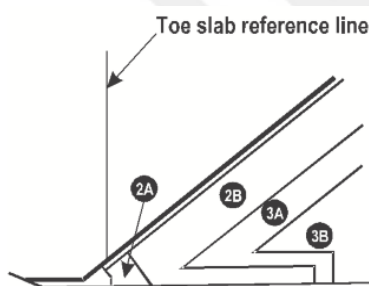


Figure 2.4. Riverbed Transverse Section

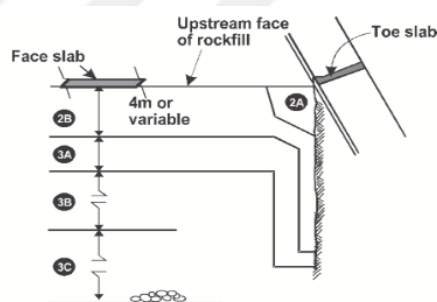


Figure 2.5. Abutment Plan Section

Zone	Material	Grading	Layer
2A	Filter	Minus 36 mm	0.4 m
2B	Crushed rock	Minus 75 mm	0.4 m
3A	Rockfill	Minus 0.4 m	0.4 m
3B	Rockfill	Minus 1.0 m	1.0 m
3C	Rockfill	Minus 2.0 m	2.0 m

Table 2.4. Typical Zoning of CFRD (ICOLD, 2010)

Zone Numbers	Description
Zone 1A	The purpose of this zone, if required, is to migrate through cracks in the face slab. It consists of fine-grained cohesionless silt and fine sand without gravel and cobble-sized rock particles.
Zone 1B	This zone protects Zone 1A and consists of silts, clays, sands, gravels, and cobbles.

Table 2.4. (cont'd)

Zone 2A	Located within two or three meters of the perimeter zone (Zone 1A), this zone acts as a defense mechanism in case In the case of waterstop failures at Zone 1A. The sand and gravel in Zone 2A will stop the movement of silt-sized particles through the zone, serving as secondary defense against leakage.
Zone 2B	Supporting the face slab and this zone consists of sand-gravel-sized particles.
Zone 3A	A rockfill zone that acts as a transition area between Zone 2B and Rockfill Zone 3B with a maximum particle size of 400 mm.
Zone 3B	The maximum particle size in this zone is 1000 mm. Approximately 10% to 20% of the rockfill should be mixed with water. Adding water achieves weak rockfills with satisfactory density. In fillings that use sand and gravel, such as in Aguamilpa Dam, the layer thickness is reduced to 600 mm.
Zone 3D, 3E, etc. (not shown in Figure 2.3.)	This zone facilitates drainage within the embankment. In dam fill constructions that include sand and gravel, an additional chimney drain is built in this area. High-capacity internal drainage is a crucial feature for the safety of Cemented Fill Rockfill Dams (CFRDs).

2.4. Materials and Grading for Rockfill Dams

The rockfill is the principal structural component of a dam, essential for maintaining slope stability and controlling seepage from the foundation and the overlying concrete slab. Its mechanical properties should be evaluated independently from those of granular soil materials such as gravel and sand since rockfill uniquely undergoes fracturing and crushing under relatively low-pressure conditions (Cruz et al., 2009).

In contrast to concrete aggregates, the selection criteria for materials in rockfill dams are more diverse. The chosen rock for dam construction should be durable and resistant to weathering. Commonly selected materials include igneous solid or metamorphic rocks, while rocks that fracture into flat shards upon blasting are typically avoided. Granite, diorite, gneiss, basalt, dense sandstone, limestone, and dolomitic quartzite are frequently used in rockfill dams. In some cases, softer rocks like siltstone, schist, and argillite have also been effectively utilized (Singh et al., 1995).

The unconfined compressive strength of rockfill usually varies from a low of 100-200 kg/cm² to more than 2,500 kg/cm², with the majority ranging between 500-1,500 kg/cm². Rocks with an unconfined compressive strength exceeding 300 kg/cm² are typically categorized as hard rock. Nonetheless, using hard rock in rockfill is not mandatory, as it can lead to wear on rubber-tired machinery and incur higher quarrying and handling expenses (Cooke & Sherard, 1987).

Sluicing, or adding water to rockfill during construction, primarily aims to moisten the material. Moistening the fines in the rockfill reduces its compressive strength, decreasing post-construction settlements in the dam. However, if the rock has a low water absorption rate, the enhancement in compressibility is often insignificant and can be overlooked, particularly in moderately high dams and Zone 3C (Cooke & Sherard, 1987).

Adding water to rockfill results in the softening of the finer components and a reduction in the unconfined compressive strength of the larger rock particles, thereby minimizing post-construction settlement (Cooke & Sherard, 1987).

2.5. Use of Sand-Gravel Fill as Rockfill in CFRDs

Sand-gravel, although not typically classified as rockfill, has proven effective when used as shell material in dam construction. It offers a more cost-efficient handling process and can be combined with traditional rockfill. The compressibility modulus of gravel is about 5 to 10 times higher than that of compacted rockfill and nearly 40 times greater than that of dumped rockfill. As the movements of the face slab in rockfill dams tend to increase with the square of the dam's height, the use of gravel-shell in concrete face dams allows for construction heights surpassing the current highest, the 160-meter Areia Dam (Cooke, 1984).

CFRDs utilizing gravel fill, also known as Concrete Face Sand Gravel Dams (CFSGDs), have internal designs that slightly differ from typical CFRDs (Cooke, 1984).

For instance, the typical vertical compression modulus of gravel at the Oroville Dam is 365 MPa, while it ranges between 551-689 MPa at the W.A.C. Bennett Dam. The Oroville Dam used a 3.5-ton, static-weight, smooth drum vibratory roller for compaction, in contrast to a 6-ton vibratory roller at the Bennett Dam, based on the roller sizes available during their construction periods. The layer thickness for compacted gravel usually ranges from 0.3 to 0.9 meters, depending on the dam's height, gravel size, and the amount of material smaller than no. 200 mesh. Typically, no water is added during the compaction process. However, when dealing with 'dirty gravel,' which contains 7-12% of material finer than no. 200 mesh, excess moisture can lead to a springy fill, although satisfactory density is still achievable. In such cases, the permeability of the gravel becomes a critical design factor, necessitating the use of features like chimney and abutment drains (Cooke, 1984).

According to Cruz et al. (2009), the optimal position for a drainage layer is generally near the central axis of the dam. (Mori, 1999) suggests using alluvial gravel in the upstream third of the dam to minimize deformation and cracking of the face slab. (Fell, 2015).

When a sand-gravel fill includes considerable material, it is finer than no. 200 mesh, permeability becomes a critical factor in its design. Therefore, the internal layout of Concrete Face Sand Gravel Dams (CFSGDs) should include chimney drains, abutment drains, filters, and intermediate drainage layers. Figure 2.6. illustrates the standard cross-section of a CFSGD (Fell 2015).

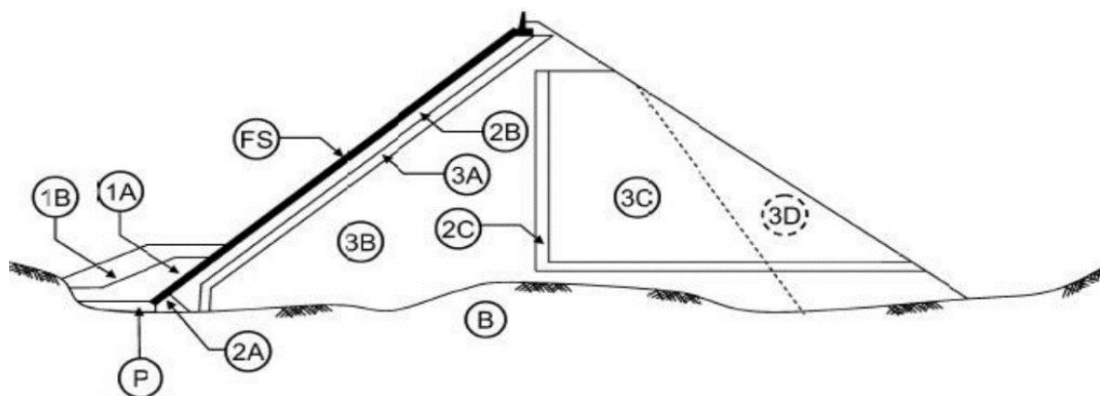


Figure 2.6. Typical Cross-section of CFSGD (Fell et al., 2015)

Zone	Material	Zone	Material
3A	Filter Zone	2A	Perimeter Zone
3B	Sand- Gravel Fill	2B	Cushion Zone
3C	Sand-Gravel Fill (dirty)	2C	Filter
3D	Rockfill	P	Plinth
1A	Self- Healing Fill	FS	Concrete Face Slab
1B	Protection Fill	B	Bedrock

Fitzpatrick et al. (1985) and Mori (1999) propose that, owing to the significant modulus achieved with compacted gravel fills, these materials are more suitable for constructing very high dams. They advise exercising caution when considering the use of rockfill for dams that exceed the heights of those currently constructed.

2.6. Numerical Analyses of CFRDs

Historically, studies on concrete-faced rockfill dams (CFRDs) predominantly utilized linear elastic models, primarily focusing on two-dimensional analyses. However, recent developments have led to a notable shift towards non-linear analysis, now the more commonly accepted approach. At present, the finite element method (FEM) and the finite difference method are the leading techniques for such analyses.

In the concrete-faced rockfill dams, understanding the stress and deformation properties of the rockfill and the concrete face slab is vital for ensuring the dam's safety and operational efficiency. As CFRDs have become higher recently, the topographical and geological complexities at dam sites have also become more pronounced. These complexities present various challenges that demand advanced theoretical modeling and rigorous application of analytical methods in CFRD numerical analysis.

Numerical analysis is essential in the design process of CFRDs. It plays a key role in determining the stress and deformation behaviors of CFRDs under various conditions and assists in predicting potential deformation patterns or failure modes. The results from numerical analyses provide critical guidance for these dams' structural design and construction phases.

2.6.1. Constitutive Models for Rockfill Material

In the numerical study of concrete-faced rockfill dams (CFRDs), selecting an appropriate constitutive model for rockfill is crucial. Comprehensive stress and deformation analyses of CFRDs require careful consideration of several factors, including the stress-deformation relationship and the strength and deformation characteristics of the materials.

Rockfill materials exhibit complex constitutive behaviors, which include non-linear strain hardening, volumetric changes due to shear stress, plastic yielding, anisotropy, and creep. External factors, such as the methods of stress application, historical stress exposures, initial stress conditions, and the granular structure of the rockfill, also influence these behaviors.

Modern theoretical approaches to constitutive modeling of rockfill include non-linear elastic, elastoplastic, and visco-elastoplastic theories. In numerical studies of CFRDs, the most commonly used constitutive models for rockfill are non-linear elastic and elastoplastic models.

Given rockfill materials' significant non-linear stress-strain relationship, the constitutive model chosen for CFRD numerical analyses must accurately represent this non-linearity. Moreover, volumetric changes in rockfill during shearing, which involve both contraction and dilation, critically impact the stress distribution on the concrete face. Therefore, elastoplastic models that use multiple yield surfaces and a non-associated flow rule are theoretically optimal for capturing these changes.

Subsequent sections will provide a concise overview of non-linear material models, highlighting the constitutive principles applied in finite element analyses of dams. This includes a discussion of Duncan and Chang's hyperbolic and hardening soil models, with the latter being the preferred method for representing rockfill behavior in this study. A summary of linear elasticity theory is also included.

2.6.1.1. Linear Elasticity

Elastic materials are characterized by their ability to return to their original form once external forces are removed, thus exhibiting reversible deformation. All materials possess, to varying extents, the property of elasticity. In civil engineering, the most fundamental model for this property is linear elasticity. Hooke's Law governs the constitutive relationship that links applied force to the resulting deformation. The behavior of an elastic material is described using two key parameters: Poisson's ratio (ν) and the modulus of elasticity (E). The stress-strain relationship in the x-direction is demonstrated as follows.

$$\epsilon_x = \sigma_x / E \quad (2.2a)$$

$$\epsilon_y = -\nu \cdot \sigma_x / E \quad (2.2b)$$

$$\epsilon_z = -\nu \cdot \sigma_x / E \quad (2.2c)$$

$$\gamma_{yz} = G \cdot \tau_{yz} \quad (2.3)$$

The symbol σ_x denotes the normal stress in the x-direction, while ϵ_x , ϵ_y and ϵ_z represent the strains in the x, y, and z directions, respectively. The terms γ_{yz} and τ_{yz} indicate the shear strain and shear stress in the y-z plane, respectively. G is the shear modulus. The relationship between the modulus of elasticity (E) and the shear modulus (G) is described as follows.

$$G = E \cdot [2 \cdot (1 + \nu)] \quad (2.4)$$

The curve depicting the stress-strain relationship for a linear-elastic material during both loading and unloading is shown in Fig 2.7 (a). Figures 2.7 (b). and 2.7 (c). illustrate other possible responses.

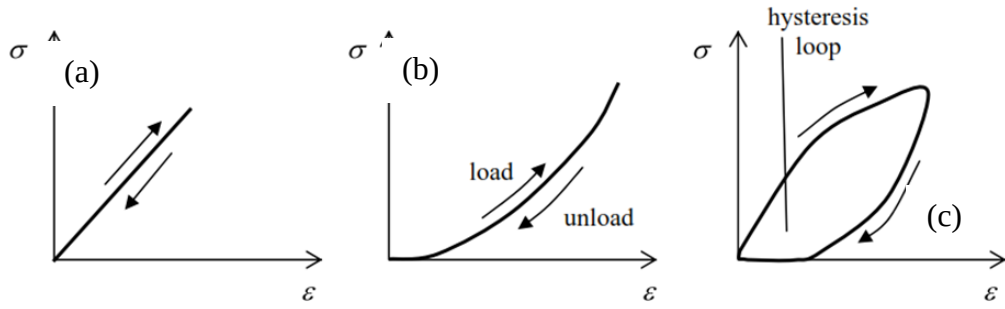


Figure 2.7. Various stress-strain relationship types include: (a) linear-elastic, (b) non-linear elastic, and (c) viscoelastic, viscoplastic, or plastic.

2.6.1.2. Duncan Chang's Hyperbolic Model

The stress-strain curves for both clay and sand, which are nonlinear, can be described using a hyperbolic function. The equation representing this hyperbola is depicted below, as described by Kondner (1963) and Kondner and Zelasko (1963).

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{a + b \cdot \varepsilon} \quad (2.5)$$

In Eq. (2.5), σ_1 , σ_3 , represent major and minor principal stresses, respectively, while ε denotes the axial strain. Coefficients a and b , essential for the analysis, can be ascertained through traditional triaxial tests, as depicted in Figure 2.8 (a). Nevertheless, employing the transformed axes method, as illustrated in Figure 2.8 (b)., offers a more straightforward approach.

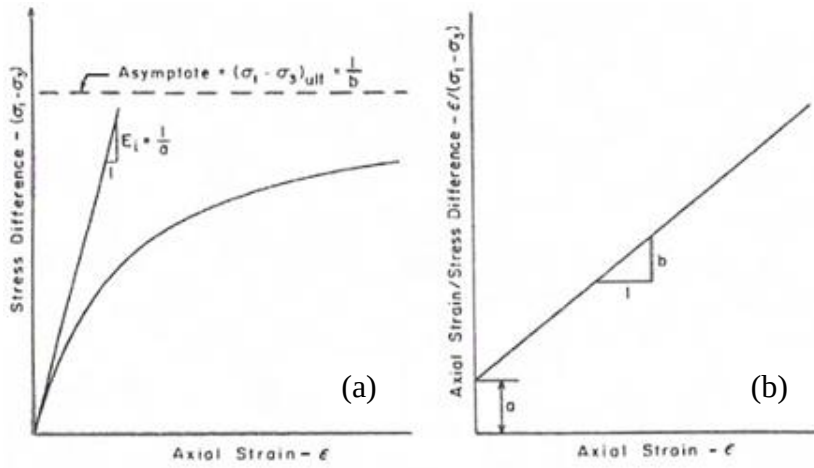


Figure 2.8. Hyperbolic Stress-Strain Curve According to Duncan and Chang (1970)

The asymptotic value of the stress difference, located above the hyperbola, is displayed in Figure 2.8 (a).

$$(\sigma_1 - \sigma_3)_f = R_f (\sigma_1 - \sigma_3)_{ult} \quad (2.6)$$

In Equation (2.6), R_f represents the failure ratio, $(\sigma_1 - \sigma_3)_f$ denotes the compressive strength, and $(\sigma_1 - \sigma_3)_{ult}$ represents the asymptotic value of the stress difference, which is closely associated with the strength of the soil. It is important to note that the value of $(\sigma_1 - \sigma_3)_{ult}$ is consistently higher than the compressive strength of the soil.

Kondner (1963) pointed out that the confining stress does not influence R_f and falls within the range of 0.75 to 1.00, as mentioned by Duncan and Chang in 1970. The general equation can be expressed in the following manner by utilizing the initial tangent modulus and the compressive strength.

$$(\sigma_1 - \sigma_3) = \frac{\varepsilon}{\frac{1}{E_i} + \frac{\varepsilon \cdot R_f}{(\sigma_1 - \sigma_3)_f}} \quad (2.7)$$

Duncan and Chang's hyperbolic model is based on Kondner's study (1963). Kondner's study effectively represents the non-linear behavior of soils. In most soils, except fully saturated ones, an increase in confining pressure leads to a steeper stress-strain curve. Janbu established a relationship between E_i and confining pressure in 1963, as shown in Eq. (2.8) (Janbu, 1963).

$$E_i = K \cdot P_a \cdot (\sigma_3 / p_a)^n \quad (2.8)$$

In Eq. (2.8), σ_3 represents the minor principal stress, and p_a denotes atmospheric pressure. K and n are dimensionless parameters, with K being the modulus number and n indicating the rate of change of E_i concerning σ_3 . These values, K and n , can be determined by referring to Figure 2.9.

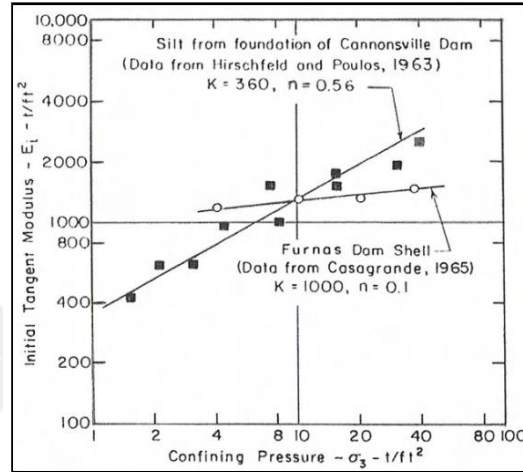


Figure 2.9. Determination of K and n (Duncan & Chang, 1970)

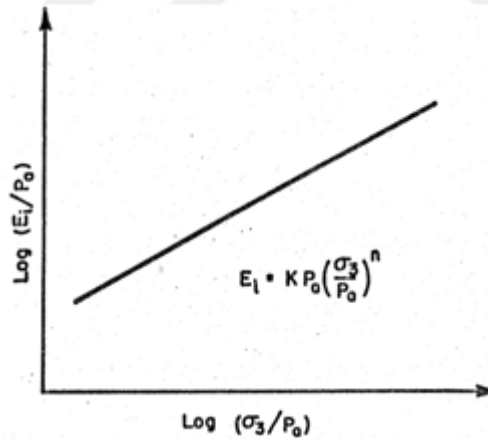


Figure 2.10. The Change in the Initial Tangent Modulus as a Function of Confining Pressure (Duncan & Chang, 1970)

Duncan and Chang (1970) have observed that when the minor principal stress remains consistent, one can determine the tangent modulus using Equation (2.9).

$$E_t = \frac{d(\sigma_1 - \sigma_3)}{d\epsilon} \quad (2.9)$$

E_t denotes tangent elastic modulus. By integrating variables Eq. (2.7) and Eq (2.9), Eq. (2.11) is obtained.

$$E_t = \left[\frac{1}{E_i} / \left(1 + \frac{R_f \cdot \epsilon}{(\sigma_1 - \sigma_3)_f} \right)^2 \right] \quad (2.10)$$

It should be observed that Equation 2.10 incorporates both the stress differential and strain, which may be based on varying reference points. To rectify this discrepancy, Duncan and Chang revised Equation 2.10, thus removing the strain variable from Equation 2.7.

$$\epsilon = \frac{\sigma_1 - \sigma_3}{E_i \left[1 - \frac{R_f(\sigma_1 - \sigma_3)}{(\sigma_1 - \sigma_3)_f} \right]} \quad (2.11)$$

Duncan and Chang (1970) incorporated the well-established Mohr-Coulomb failure criterion into their hyperbolic model, which is represented as follows:

$$(\sigma_1 - \sigma_3)_f = \frac{2c \cdot \cos\phi + 2\sigma_3 \sin\phi}{1 - \sin\phi} \quad (2.12)$$

In this context, ' c ' represents the soil's cohesion and ϕ denotes the angle of internal friction. By integrating Equation 2.12 with Equations 2.10 and 2.11, Equation 2.13 is derived.

$$E_t = \left[1 - \frac{R_f(1 - \sin\phi)(\sigma_1 - \sigma_3)}{2c \cdot \cos\phi + 2\sigma_3 \sin\phi} \right]^2 \text{ K. Pa.} \left(\frac{\sigma_3}{p_a} \right)^n \quad (2.13)$$

In hyperbolic stress-strain relationship, E_{ur} , unloading-reloading modulus, is used for both unloading and reloading. The value of E_{ur} depends on confining stress.

$$E_{ur} = K_{ur} \cdot p_a \left(\frac{\sigma_3}{p_a} \right)^n \quad (2.14)$$

K_{ur} denotes the modulus number relevant to unloading and reloading situations, as described by Duncan and Chang (1970). The value of K_{ur} is always greater than that of K , which is the loading modulus number. For hard soils, such as dense sand, K_{ur} may be about 20% higher than K . Conversely, K_{ur} could be up to three times larger than K for softer soils like loose sands. The exponent n maintains a consistent value throughout the loading, unloading, and reloading processes, as noted by Duncan and Chang (1970).

During triaxial testing, if the specimen is unloaded at a certain point, the stress-strain curve for unloading is observed to be steeper than the loading curve, as demonstrated in Figure 2.11. When the specimen is reloaded, the stress-strain curve becomes steeper than that for the initial loading and almost identical to the unloading curve. This phenomenon indicates the inelastic behavior of the soil, where only a part of the strains incurred during loading is partially recoverable.

Regarding the hyperbolic model introduced by Duncan Chang, it lacks a direct correlation with Poisson's ratio. Kulhawy et al. addressed this limitation in 1972. They developed a method to calculate the tangent Poisson's ratio, consistent with the hyperbolic model's framework. The proposed formula is as follows:

$$v_t = \frac{G - F \log\left(\frac{\sigma_3}{p_a}\right)}{1 - \left[\frac{d(\sigma_3 - \sigma_1)}{K \cdot p_a \cdot \left(\frac{\sigma_3}{p_a}\right) 1 - \left(\frac{R_f(1 - \sin(\phi)(\sigma_1 - \sigma_3))}{2cc \cos(\phi + 2\sigma_3 \sin(\phi))}\right)} \right]} \quad (2.15)$$

Where v_t represents the tangent Poisson's ratio, and the constants G , F , and d are parameters that can be ascertained from the outcomes of triaxial tests which include measurements of volume change, as established by Kulhawy et al. (1972).

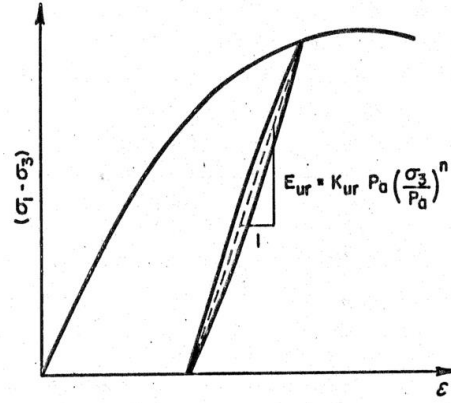


Figure 2.11. Unloading-Reloading Elasticity Modulus (Duncan&Chang, 1970)

Duncan et al. (1980) revisited the subject and introduced a bulk modulus parameter labeled B . They observed that this parameter fluctuates with confining pressure and is related to the variations in stress that occur with volumetric strains. The equation for the bulk modulus is as follows.

$$B = K_b \cdot p_a \left(\frac{\sigma_3}{p_a} \right)^m \quad (2.16)$$

In Equation (2.16), K_b represents the number for the bulk modulus, and m is the exponent associated with the bulk modulus.

In soils without cohesion, like sands, gravels, and rockfills, choosing a single friction angle (ϕ) does not accurately cover the entire range of pressures of interest. For instance, at the base of a dam, where rockfill is subjected to high pressure, the friction angle may be lower than at the dam's toe. Selecting a ϕ value appropriate for the dam's central area could significantly underestimate the true strength behavior at the toe, and the reverse is also true. The friction angle tends to decrease with the logarithm of the confining pressure. An equation has been proposed to describe this relationship.

$$\phi = \phi_0 - \Delta\phi \log \left(\frac{\sigma_3}{p_a} \right) \quad (2.17)$$

When σ_3 is equals to p_a , ϕ will be equal to ϕ_0 . $\Delta \phi$ is the reduction in ϕ for a ten-fold increase in σ_3 .

In the hyperbolic stress-strain and volume change models, nine parameters are delineated, as indicated in Table 2.5. The study's findings about the hyperbolic model include the following observations:

- a) The tangent modulus is affected by both the confining pressure and the degree of strength mobilization.
- b) The bulk modulus is subject to change with confining pressure but does not depend on the degree of strength mobilization.

Table 2.5. Key Parameters in Hyperbolic Model

Parameter	Name	Function
K, K_{ur}	Modulus number	Relate E_t and E_{ur} to σ_3
n	Modulus exponent	
c	Cohesion intercept	Relate $(\sigma_1 - \sigma_3)_f$ to σ_3
Φ	Friction angle	
R_f	Failure ratio	Relates $(\sigma_1 - \sigma_3)_{ult}$ to $(\sigma_1 - \sigma_3)_f$
G	Poisson's ratio parameter	Value of v_t at $\sigma_3 = p_a$
F	Poisson's ratio parameter	Decrease in v_t for a ten-fold increase in σ_3
d	Poisson's ratio parameter	Rate of increase of v_t with strain

The limitations of the fundamental hyperbolic model are as follows:

- Derived from a generalized form of Hooke's Law, the hyperbolic stress-strain model is adept at analyzing stress and deformation up to failure. It adeptly predicts the non-linear correlation between load and deformation, which is applicable until some aspects of the soil fail. Its predictive reliability, however, needs to improve when considering the behavior of soil masses influenced by elements that have already failed, particularly in forecasting

behavior during and after failure. This model is primarily effective in estimating movements in stable soil conditions.

- A significant limitation of the model is its failure to account for volume changes resulting from shear stress or 'shear dilatancy.' This shortcoming leads to inaccuracies in predicting deformations in dilatant soils, such as dense sands under low confining pressure. Sometimes, the computed tangent Poisson's ratio may exceed 0.5, but most computer applications using these parameters limit it to a maximum of 0.49.
- It is important to note that the parameters in the hyperbolic model do not represent inherent soil properties. Instead, they describe the soil's behavior under specific testing conditions. These parameters are subject to soil density, moisture content, the range of pressures applied during tests, and drainage conditions. To ensure that the parameters accurately reflect field conditions, laboratory tests must be carefully designed to emulate the real-world scenarios as closely as possible.

2.6.1.3. Hardening Soil Model

The hardening soil model has been adopted to effectively model rockfill material's behavior effectively, as previously discussed. This model serves as a modified version of Duncan Chang's hyperbolic model. While foundational, Duncan Chang's hyperbolic model presents certain limitations, as highlighted earlier. To overcome these constraints, the hardening soil model has been developed within an elastoplastic framework. Unlike the original hyperbolic model, the hardening soil model applies the principles of plasticity instead of elasticity, incorporates soil dilatancy, and introduces a yield cap feature (Schanz et al., 1999).

In the hardening soil model, the yield surface is not fixed in the principal stress space; instead, it can enlarge due to plastic straining. The theory of this model will be explained using the drained triaxial loading condition, where $\sigma_3' = \sigma_2'$ and σ_1' is the major principal stress.

In simple terms, the hardening soil model is formulated based on the relationship between vertical strain and deviatoric strain in primary triaxial loading. When primary deviatoric loading is applied to the soil, the soil's stiffness decreases, and plastic strains occur. Previously, in 1963, Kondner et al. first described the stress-strain behavior of soil using a hyperbolic curve in a drained test. The hyperbolic behavior in the hardening soil model is depicted using the following equation, as shown in Figure 2.12.

$$\varepsilon_1 = \frac{q_a}{2E_{50}} - \frac{q}{1 - \frac{q}{q_a}} \text{ for } q < q_f \quad (2.18)$$

In equation (2.18), q represents the deviatoric stress, q_a stands for the shear strength at the asymptotic limit, q_f denotes the maximum deviatoric stress achievable, and E_{50} represents the stiffness modulus that varies with stress.

$$E_{50} = E_{50}^{ref} \left(\frac{ccot\phi - \sigma'_3}{ccot\phi + p^{ref}} \right)^m \quad (2.19)$$

The reference stiffness modulus, denoted as E_{50}^{ref} , is associated with the reference confining pressure, p^{ref} . Within the framework of the hardening soil model, the actual stiffness is influenced by the minor principal stress, denoted as σ'_3 , representing the confining pressure in a triaxial test. It is important to note that σ'_3 assumes negative values during compression. The parameter m plays a pivotal role in governing the degree of stress dependency, much like the exponent n in the Duncan-Chang model. Similar to the Duncan-Chang model, the Mohr-Coulomb failure criterion is utilized to assess q_f in the hardening soil model.

$$q_f = (c + \sigma'_3 \tan\phi) \frac{2\sin\phi}{1 - \sin\phi} \quad (2.20)$$

$$q_a = \frac{q_f}{R_f} \quad (2.21)$$

The term ‘failure ratio’ is similar to the Duncan-Chang model to illustrate the analogous relationship between q and q_f . Another stiffness modulus, which depends on stress levels, is utilized in the context of unloading-reloading conditions.

$$E_{ur} = E_{ur}^{ref} \left(\frac{c \cdot \cot \phi - \sigma_3'}{c \cdot \cot \phi + p^{ref}} \right)^m \quad (2.22)$$

E_{ur}^{ref} refers to a reference stiffness modulus associated with unloading-reloading conditions, aligned with the reference confining pressure p^{ref} . In numerous cases, it is common practice to consider E_{ur}^{ref} as equivalent to $3E_{50}^{ref}$, which is the default setting in the hardening soil model. E_{oed} is employed for one-dimensional compression and can be computed using Eq. 2.23.

$$E_{oed} = E_{oed}^{ref} \left(\frac{c \cdot \cot \phi - \sigma_3'}{c \cdot \cot \phi + p^{ref}} \right)^m \quad (2.23)$$

E_{oed} represents the tangent stiffness modulus, as depicted in Figure 2.13. It is crucial to emphasize that the one-dimensional compression behavior is exclusively represented by σ_1 . When a test result similar to the one illustrated in Figure 2.13. is unavailable, $E_{oed}^{ref} = E_{50}^{ref}$ relationship can be employed as an alternative approach.

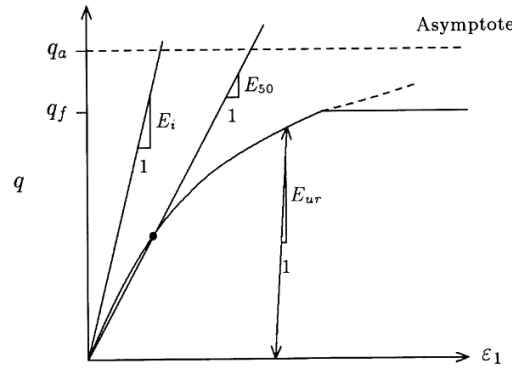


Figure 2.12. Hyperbolic Stress-strain Relation in Primary Loading for a Standard Triaxial Test (Schanz et al., 1999)

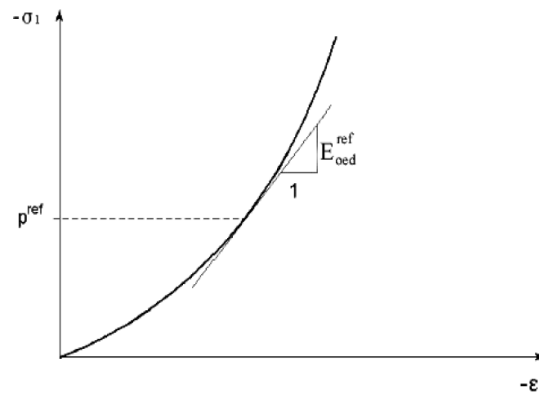


Figure 2.13. E_{oed}^{ref} in Oedometer Test (Schanz et al., 1999) with Negative Values Values Signifying Compression

2.6.2. Shear Strength Characteristics of Sand-Gravel Fills

This chapter introduces some innovative approaches from literature review for assessing the static strength and deformation characteristics of sand and gravel, especially those containing particles too large for standard laboratory testing. In recent developments within civil engineering, there has been an increasing preference for fill materials composed of coarse-grained materials due to cost-effectiveness and material availability. This shift towards coarser granular materials presents specific challenges for engineers in determining their strength and deformation properties, which are critical for stability analysis.

This section presents some valuable significant studies on the shear strength of sand-gravel fills.

Leps (1970) compiled and presented, in a single chart (Figure 2.14.), the available large-scale test data on gravels and rockfills up to that point. This data encompassed 15 different materials, which Leps categorized into three groups based on their gradation and compressive strength. He defined weak rock particles as those with strengths ranging from 500 to 2500 psi, average rock particles with strengths from 2500 to 10,000 psi, and strong rock particles as having strengths from 10,000 to 30,000 psi (1 Mpa is equivalent to 145 psi).

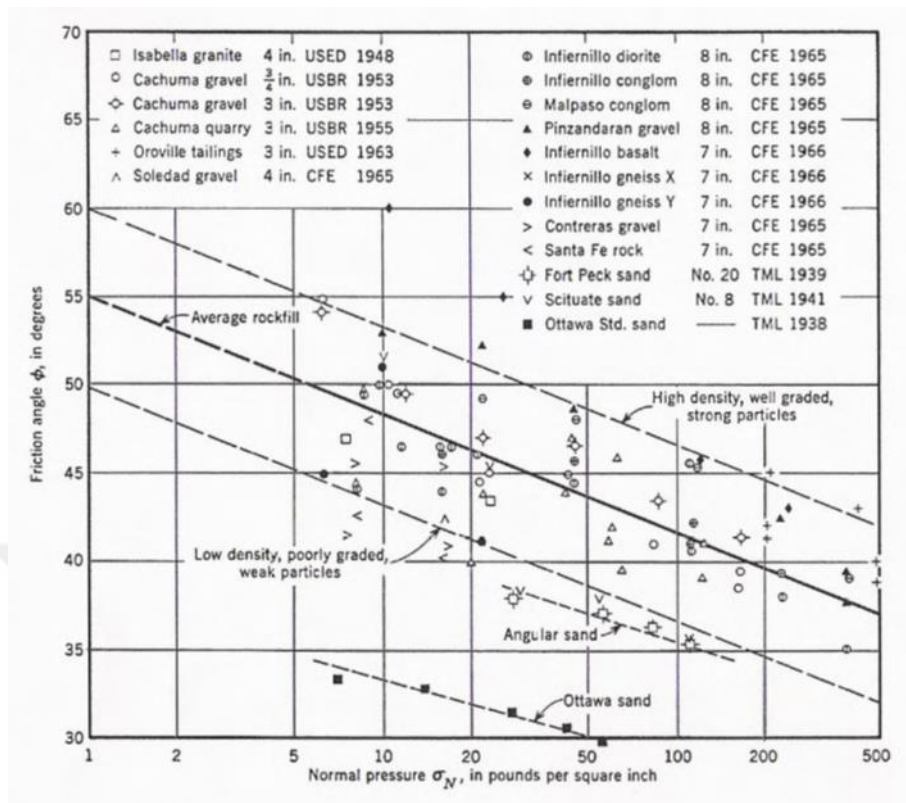


Figure 2.14. Investigating the Influence of Confining Pressure on the Peak Friction Angle in Rockfill Specimens (Leps, 1970)

Fragazsy et al. (1992) focuses on a new method for evaluating the strength and deformation behavior of gravelly soils. This method, known as the matrix modeling method, is designed for soils containing particles too large for conventional laboratory equipment. The general rule for this testing dictates that the soil particles should not exceed one-sixth of the diameter of the test specimen. Consequently, for fill materials that include particles larger than about 0.5 inches (12.7 mm), standard triaxial testing is not suitable if the test specimen's maximum size is only 2.8 inches (71.1 mm). In such cases, it becomes necessary to either remove the oversized particles from the sample or seek alternative testing methods to accurately assess the material properties.

Up until that time, two primary methods had been used estimate the static strength of soils containing oversized particles: the parallel gradation method and the scalp-and-replace method. The parallel gradation method involves removing oversized

particles and supplementing the soil with finer materials to achieve a grain size distribution that aligns with the original soil. This method's effectiveness is evidenced in studies like those conducted on Oroville gravel, as reported by Marachi et al. in 1969. On the other hand, the scalp-and-replace method, detailed by Donaghe and Townsend in 1976, involves replacing the scalped oversized particles with an equal mass of smaller particles that comply with the maximum size limits of standard testing procedures. However, it's important to note that both these methods do not consider the impact of grain boundary effects on the soil's density.

Fragazsy et al. (1992) presented a new method for evaluating the static strength and deformation behavior of gravelly soils containing particles too large for conventional laboratory testing. This method, known as the matrix modeling method, is based on the assumption that large particles in a matrix of finer material do not significantly affect the strength and deformation characteristics of the mixture. Thus, the behavior of a soil containing oversized particles can be modeled by testing the matrix portion alone. To better explain this method some definitions need to be provided. The term 'oversized' in this context refers to soil particles that are too large for a specific test apparatus, without a fixed size definition. Figure 2.15 (a) and Figure 2.15 (b) show the schematic diagram of a soil containing oversized particles. The matrix soil is further categorized into two components: the near-field matrix, comprising particles close to the oversized ones, and the far-field matrix, consisting of soil grains located several diameters away from the oversized particles. Fig 2.15 (a) demonstrates oversized particles in a floating state, constituting about 40% of the mix. This means that the oversized particles constitute a minor portion (around 40% by mass) of the prototype material (by mass), with minimal or no contact between them.

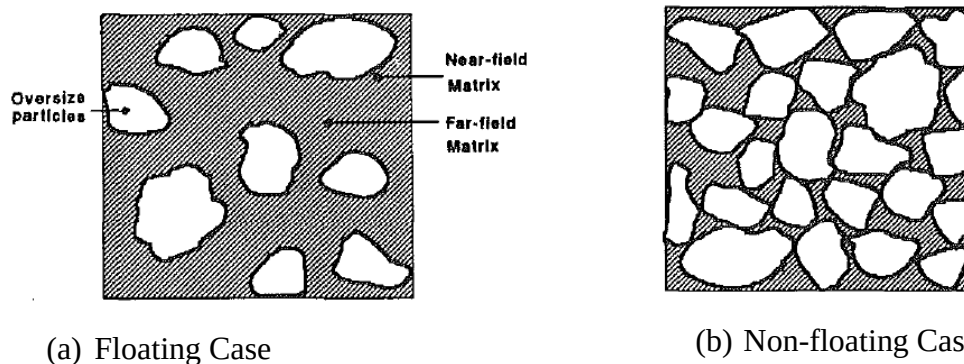


Figure 2.15. Illustration Depicting Soil Containing Larger-Sized Particles (Fragaszy et al., 1992)

The hypothesis is that in soils where oversized particles are in a floating state, their strength and deformation properties are primarily governed by the matrix material. In some cases, soils may comprise a large proportion (about 65% or more) of the total material, leading to direct contact between these particles. Here, the matrix material merely fills the gaps formed by the oversized particles' bridging action, a condition known as the non-floating state. In this state, illustrated in Fig. 2.15 (b), the soil's strength and deformation characteristics are largely influenced by the oversized particles. In situations between these two states, both the matrix and oversized particles notably influence the soil's behavior.

In this study, the material was sourced from Lake Valley Dam in Northern California. The embankment material was predominantly coarse-grained, including sizes up to cobbles and boulders. Particles larger than 1.0 inch (25.4 mm) were removed, resulting in a prototype material primarily composed of sandy gravel with a small silt content. The specific gravity of these materials varied between 2.81 and 2.84 across different size groups. Particles sized 0.5 to 1.0 inches (12.7-25.4 mm) were classified as oversized, while those smaller than 0.5 inches (12.7 mm) formed the soil matrix. In the assumed prototype material, oversized particles made up 30% by mass and were generally smooth and rounded.

To validate the matrix method, two series of comparative tests were carried out on the Modified Lake Valley gravel. Each series involved consolidated drained tests on model specimens with a diameter of 2.8 inches (71.1 mm) and prototype specimens

measuring 6 inches (152 mm) in diameter. These specimens were compacted to relative densities varying between 13.3% and 64%. All prototype specimens included 30% oversized particles by mass. The tests were conducted under two different effective confining pressures, specifically 75 and 150 kPa.

Table 2.6. The Results of Triaxial Tests on Prototype and Model Specimens (Fragazsy et al., 1992)

Test Number	Diameter of sample (in)	Percent of oversize particles	Relative Density (%)	Dry Density (Mg/m ³)	Confining Pressure (kpa)	Peak deviator pressure (kpa)	Axial Strain ^a (%)	Volumetric Strain ^a (%)
PD1	6	30	13.3	1.93	150	633	9.10	- 0.18
PD2	6	30	29.1	1.98	150	715	6.40	- 0.07
PD3	6	30	47.1	2.04	150	772	6.50	- 0.47
PD4	6	30	52.8	2.06	150	795	5.10	- 0.48
PD5	6	30	64.0	2.10	150	860	4.20	- 0.14
MD1	2.8	0	14.6	1.83	150	640	9.60	-
MD2	2.8	0	26.8	1.86	150	706	6.50	-0.08
MD3	2.8	0	35.1	1.90	150	770	5.50	-0.43
MD4	2.8	0	40.5	1.92	150	781	5.20	- 0.31
MD5	2.8	0	53.7	1.97	150	842	5.00	- 0.62
MD6	2.8	0	61.2	2.00	150	863	4.50	- 0.52
SP1	6	30	29.0	1.98	75	388	4.10	- 0.10
SP2	6	30	41.2	2.02	75	393	3.50	- 0.70
SP3	6	30	50.0	2.05	75	408	3.70	- 0.82
SP4	6	30	61.2	2.09	75	423	3.80	- 1.20
SM1	2.8	0	38.0	1.91	75	370	4.29	0.02
SM2	2.8	0	50.0	1.96	75	426	4.30	- 1.20
SM3	2.8	0	60.0	2.00	75	437	3.90	- 1.00
^a At Peak deviator stress								

The study's findings lead to these conclusions:

- In a prototype soil, large, smooth, subrounded-to-rounded grains floating within a finer matrix have minimal impact on the soil's peak strength and deformation properties.
- The peak strength of such a soil is effectively predicted by testing the matrix soil without the oversized particles, ensuring the void ratio aligns with that of the far-field matrix in the prototype.

- Up to peak strength, the deformation characteristics of the soil are accurately represented by the matrix material alone, provided it's tested at the same void ratio as the prototype's far-field matrix.

Singh et al. (1995) stated that the shear strength of silt, sand and gravel depends on their friction angle. Thus, it can be said that their strength characteristics is frictional. These soils have high coefficient of permeability and are non-cohesive. They dissipate pore pressure nearly simultaneously as the load is applied. Stability problems are related with their drained shear strength.

(Yagiz, 2001) conducted a study to investigate how the shape and percentage of gravel affect the shear strength and internal friction angle of sand and gravel mixtures. He performed direct shear tests on angular and angular and rounded limestones with maximum diameter of 6.3 mm. Experiments were conducted under various normal pressures (0.5, 1.0, and 1.5 kN) with a loading speed set at 0.1 mm/min. The selection of gravel, including both angular and rounded types, was done through visual examination. Medium-grade sand, ranging in size from 0.2 to 0.4 mm, was utilized in these tests. Gravel of both angular and rounded shapes was chosen through visual evaluation according to standart visual characteristic criteria (Figure 2.16 and 2.17).



Figure 2.16. Categorization of Gravel Grain Shapes (Cudoto, 1999)

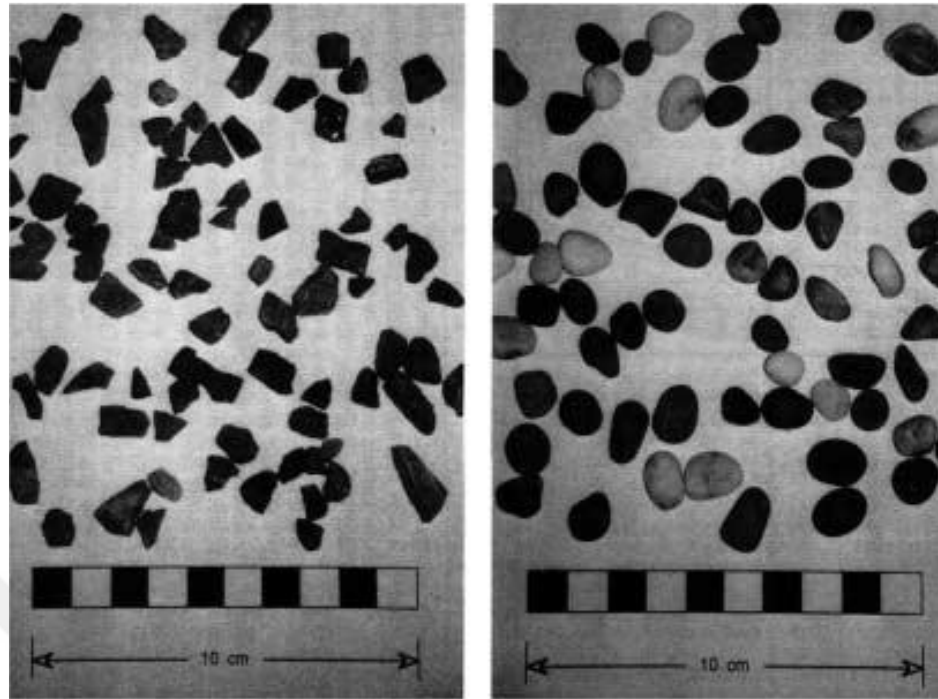


Figure 2.17. View of Angular and Rounded Gravels (Yagiz, 2001)

Table 2.7 presents the results of direct shear tests conducted on mixtures of angular gravel and sand. It shows the influence of varying percentages of angular gravel in the sand on key parameters such as normal stress, shear strength, friction angle, and density. The data is organized to demonstrate how the inclusion of different proportions of angular gravel (ranging from 10% to 50%) impacts these parameters, with a control group of 100% sand for comparison. The results indicate an increase in shear strength and friction angle with the increase in the percentage of angular gravel.

Table 2.7. Results from Direct Shear Tests on a Combination of Angular Gravel and Sand (Yagiz, 2001)

Normal Stress (kPa)	<i>Shear</i>		
	<i>Strength</i> (kPa)	<i>Friction Angle</i> (degrees)	<i>Density</i> (g/cm ³)
Shear strength test results for 100% sand			
158	107		
237	136	32	1.62

Table 2.7. (cont'd)

316	212		
Shear strength test results for 90% and 10% angular gravel			
158	118		
237	158	34	1.62
316	224		
Shear strength test results for 80% and 20% angular gravel			
158	145		
237	171	35	1.66
316	244		
Shear strength test results for 70% and 30% angular gravel			
158	161		
237	222	41	1.66
316	297		
Shear strength test results for 50% and 50% angular gravel			
158	200		
237	306	45	1.66
316	340		

Table 2.8, on the other hand, focuses on mixtures of rounded gravel and sand. Similar to Table 2.7, it displays the results of direct shear tests, but with varying percentages of rounded gravel. It includes data on normal stress, shear strength, friction angle, and density for mixtures with 10% to 50% rounded gravel.

Table 2.8. Results from Direct Shear Tests on a Combination of Rounded Gravel and Sand (Yagiz, 2001)

Normal Stress (kPa)	<i>Shear</i>		
	<i>Strength</i> (kPa)	<i>Friction Angle</i> (degrees)	<i>Density</i> (g/cm ³)
Shear strength test results for 90% and 10% rounded gravel			
158	129		

237	162	35	1.62
Table 2.8. (cont'd)			
316	222		
Shear strength test results for 80% and 20% rounded gravel			
158	144		
237	179	38	1.66
316	242		
Shear strength test results for 70% and 30% rounded gravel			
158	157		
237	213	39	1.66
316	250		
Shear strength test results for 50% and 50% rounded gravel			
158	179		
237	258	43	1.66
316	287		

As conclusion Yagiz (2001) found:

- The shape and percentage of gravel significantly influence the shear strength properties of the mixture.
- Mixtures with more than 20% angular gravel exhibit higher friction angles, making it more difficult for grains to slide past one another.

Simoni & Houlsby (2006) focused on understanding the strength and dilatancy of sand-gravel mixtures using direct shear tests. The aim of study is to find a relationship between the grain-size characteristics of the materials and the shearing resistance. For this reason, 87 direct shear tests in large direct shear box apparatus were done to investigate strength and dilatancy of sand-gravel mixtures.

The materials used in this study were sourced from Double Arches Quarry in Leighton Buzzard, UK. These materials include: a silica sand that is well-graded and has medium roundness to sub-angular in shape, (a) a gravel that is poorly graded

with medium roundness to sub-angular characteristics (as depicted in Figure 2.18), and (b) another gravel that is well-graded and varies from rounded to sub-angular in shape (also shown in Figure 2.18). Throughout the study, these materials are referred to as sand, G06 gravel, and G20 gravel, respectively.

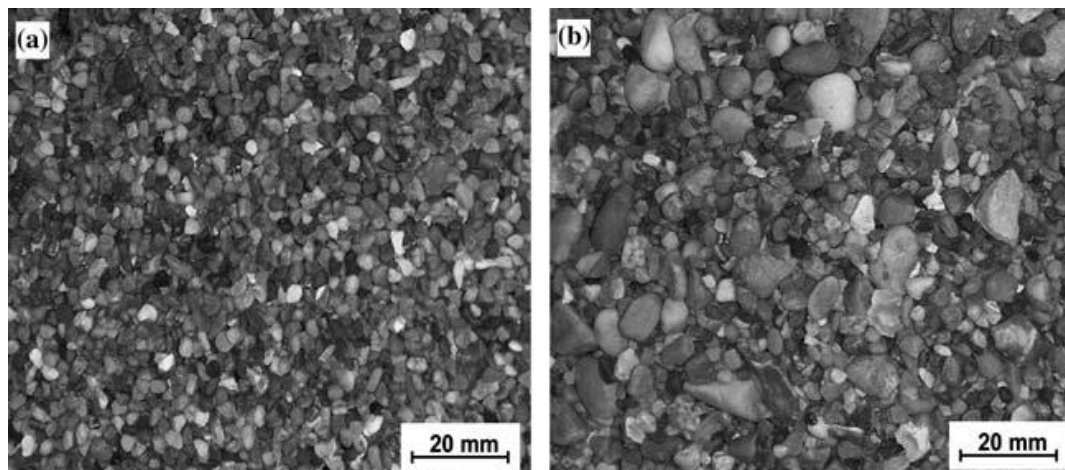


Figure 2.18. Gravels categorized as G06 (a) and G20 (b) (Simoni & Houlsby, 2006)

The key properties of these three materials are summarized in Table 2.9, and their grain size distributions are illustrated in Figure 2.19.

Table 2.9. Properties of Sand, Gravel (G06) and Gravel (G20) (Simoni & Houlsby, 2006)

Soil Type	<i>Maximum grain size (mm)</i>	<i>D₆₀ (mm)</i>	<i>D₁₀ (mm)</i>	<i>Coefficient of Uniformity, C_u</i>	<i>Specific gravity, G_s</i>
Sand	2	0.81	0.21	3.87	2.65
Gravel G06	6	5.30	3.10	1.61	2.63
Gravel G20	20	9.10	3.90	2.33	2.63

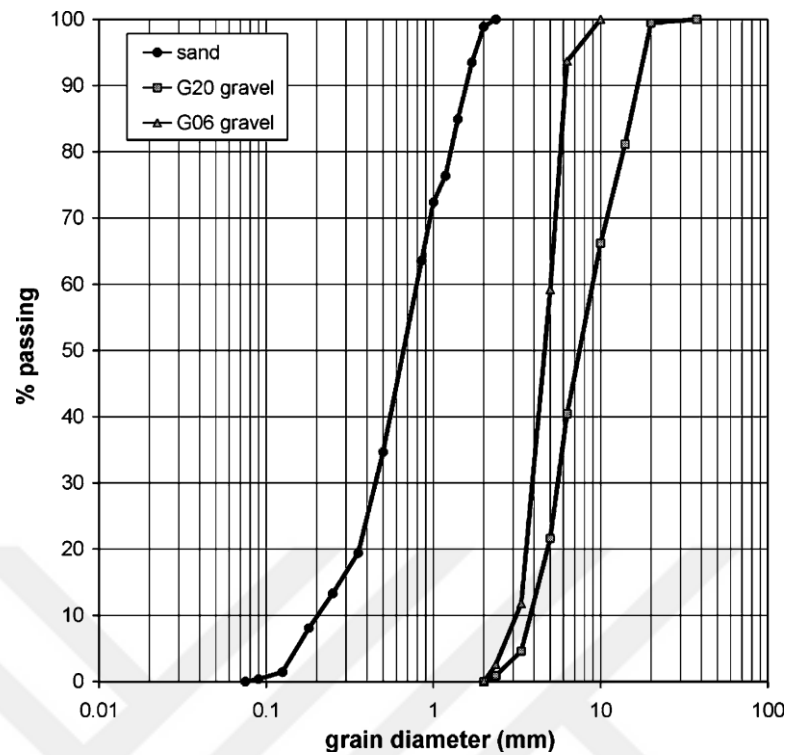


Figure 2.19. Grain Size Distribution Curves for Tested Sand and Gravel Mixtures with Varying Proportions (Simoni & Houlsby, 2006)

Figure 2.20 displays the limit states of compactness (e_{min} and e_{max}) for G06 and G20 sand-gravel mixtures, plotted against the gravel fraction. The graph shows that as gravel content increases, void ratios initially decrease, reaching their lowest at gravel fractions of about 0.5 for G06 and 0.6 for G20 mixtures. Beyond these fractions, the void ratios begin to increase once more. This trend highlights the impact of gravel addition on the mixture's compactness, which is crucial for understanding the mechanical behavior of these mixtures. Theoretical curves for floating and non-floating states, which are essential for interpreting the mixtures' behavior based on their components, are also depicted for comparison.

A material with a well-graded composition, which includes a wide range of grain sizes, can attain greater densities compared to a poorly graded counterpart. Additionally, a consistent gradation, signifying no gaps in the grading curve, is conducive to higher densities within a given grain-size range. Research by Fuller et

al. (1907) and Rothfuchs (1935) has shown that optimal densities in an aggregate are achieved when its grain-size distribution adheres to a specific law:

$$p = 100 \sqrt{\frac{a}{d_{\max}}} \quad (2.24)$$

Here, p represents the proportion of the material that passes through a sieve with an opening of size a , and $d_{\max}$ denotes the largest grain size present in the granular aggregate.

Figure 2.21. depicts the relationship between the limit states of compactness and the coefficient of uniformity (C_u) for the tested sand-gravel mixtures. The graph reveals a distinct and consistent correlation between these two parameters, suggesting that the formation of dense packings in granular mixtures is significantly influenced by their grain-size distribution. The coefficient of uniformity, a measure of the breadth of particle sizes in a mixture, is observed to markedly affect compactness, with higher C_u values generally leading to denser packings. This observation is crucial for understanding how the physical properties of mixtures contribute to their overall mechanical behavior, especially regarding shear strength and dilatancy.

For each base material or mixture, a minimum of five direct shear box experiments were conducted at varying relative densities D_r , all under a constant normal pressure of 90.8 kPa, totaling 87 tests.

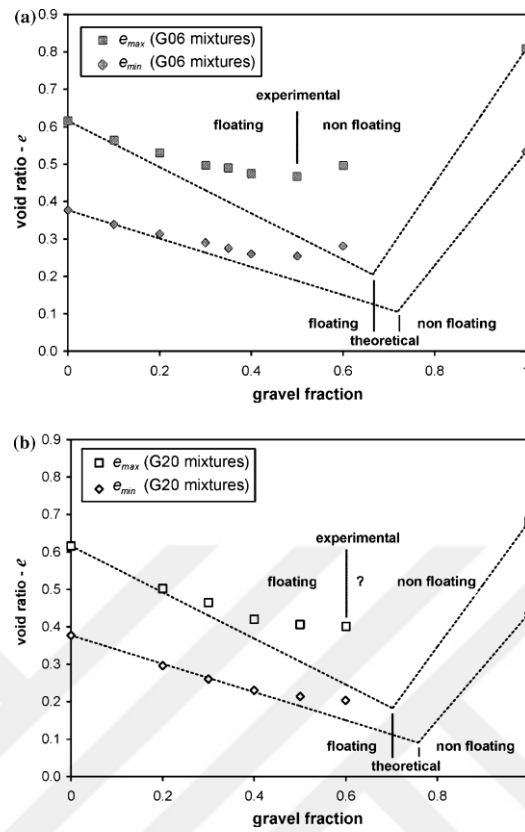


Figure 2.20. Comparing Compactness Limit States for G06 (a) and G20 (b) Mixtures Across Different Gravel Fractions (Simoni & Houlsby, 2006)

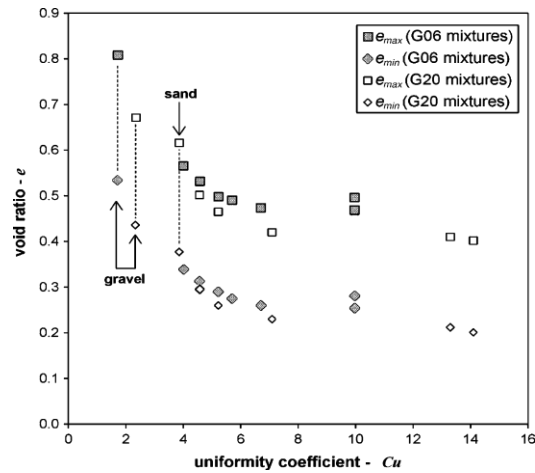


Figure 2.21. Compactness Limit States in Relation to the Coefficient of Uniformity (Simoni & Houlsby, 2006)

Figure 2.22 offers a comparative analysis of the shearing behavior across different densities of mixtures. It presents the outcomes from direct shear box tests on medium dense sand, a loose sample of the same sand, and a medium dense sand–gravel mixture (G0640). The figure effectively emphasizes the influence of gravel addition on the peak shear stress ratio and the dilation properties of the mixtures. Notably, the inclusion of gravel leads to a marked increase in the peak shear-to-normal stress ratio, an effect largely due to improved dilatancy which bolsters the mixture's resistance to shear.

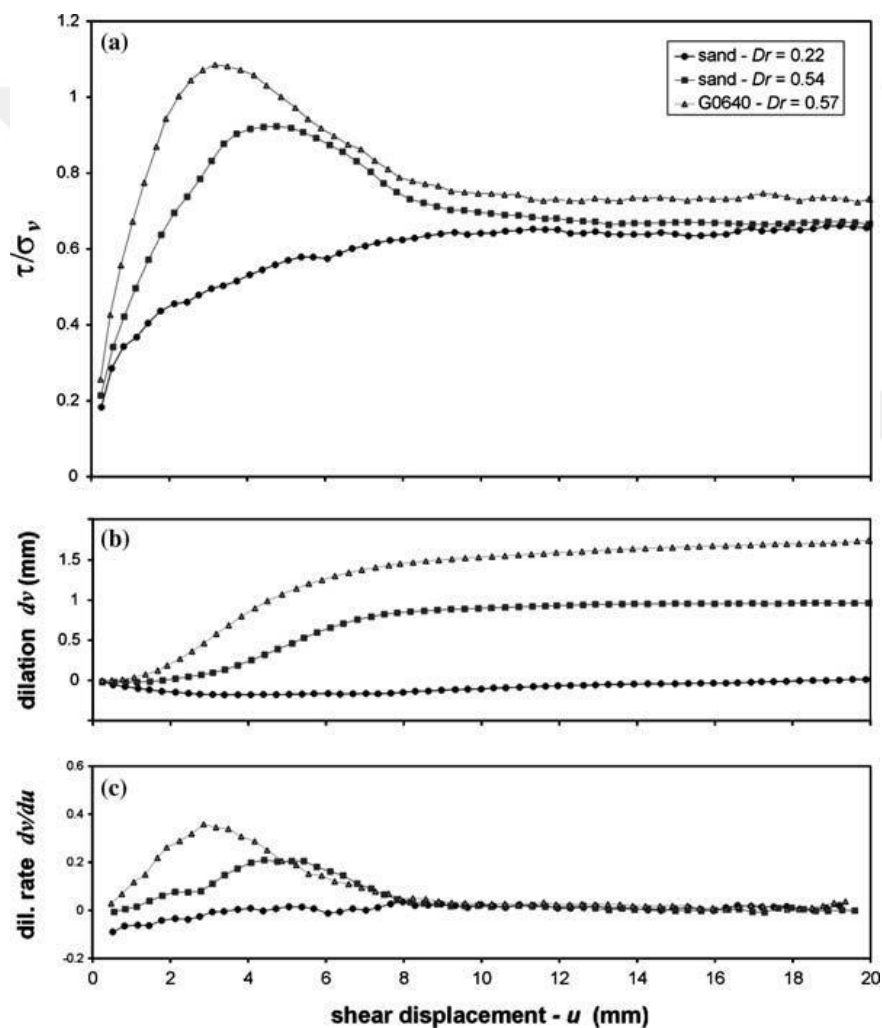


Figure 2.22. Example Results of Direct Shear Box Tests

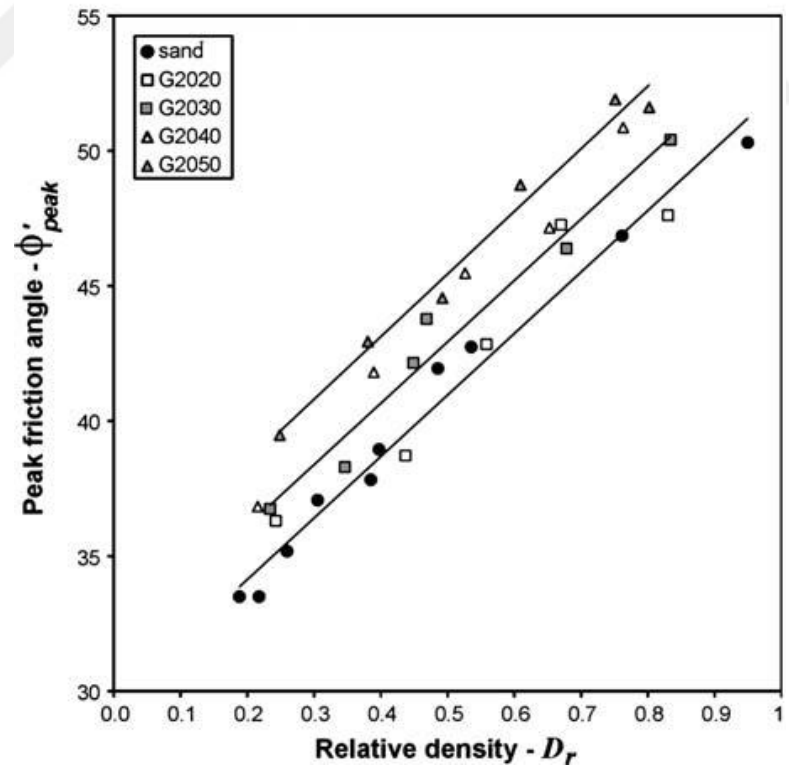
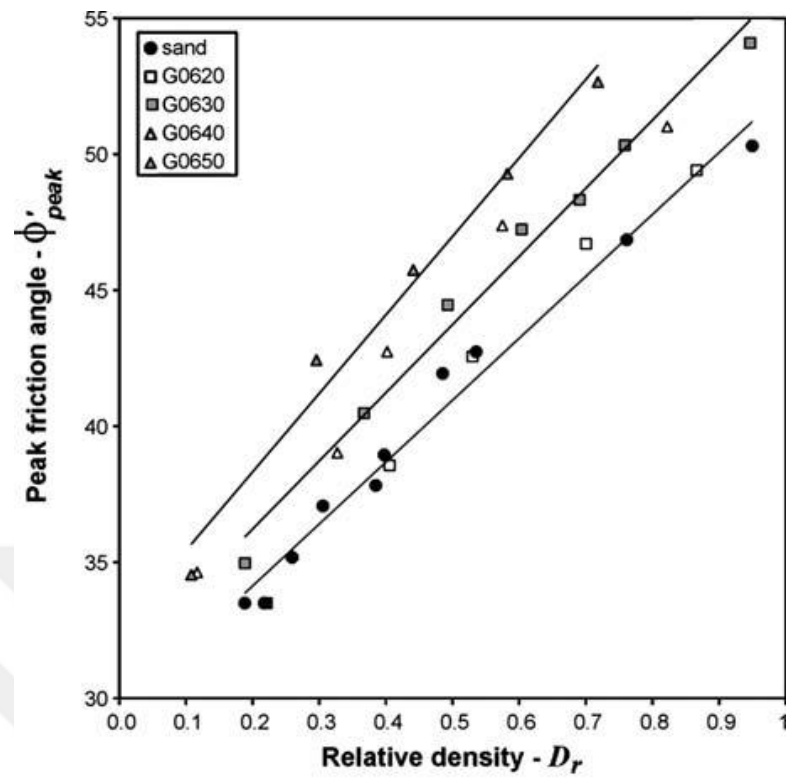


Figure 2.23. Example Results of Direct Shear Box Tests

Figure 2.23 illustrates the correlation between the peak friction angle (Φ_{peak}) and the relative density (D_r) for various sand-gravel mixtures, with a specific focus on sand, G06, and G20 mixtures. The graph displays Φ_{peak} as a function of D_r across different gravel fractions, with an emphasis on the trends at gravel fractions of 0.3 and 0.5 by weight. It reveals that an increase in relative density is associated with an increase in the peak friction angle for each mixture. Significantly, the data suggest that mixtures with a higher gravel content have increased peak friction angles at comparable relative densities, underscoring the influence of gravel content on the mixtures' mechanical strength.

Hamidi et al. (2009) conducted an extensive study on the shear strength of sandy soils containing different gravel contents. A critical aspect of this research addressed a gap in previous studies where the effects of oversized particles were often neglected due to limitations in experimental sample sizes. They performed a series of laboratory experiments using a large-scale direct shear apparatus, testing samples with varying gravel contents (20%, 40%, and 60%) and under different conditions of density (35%, 60%, and 85%) as well as surcharge pressures (150, 300, and 450 kN/m²). The findings revealed that increases in gravel content and relative density enhanced shear strength and dilation in the mixtures. The influence of gravel content on shear strength was more pronounced than relative density.

The study explored the effects of gravel size and shape, overburden pressure, and relative density on the shear behavior of these mixtures. The research demonstrated that the shear strength of mixtures is significantly influenced by the frictional resistance of the gravel grains, particularly when the gravel content exceeds 70%. For gravel contents below 40%, the frictional resistance of sand grains predominates in determining the shear strength (Hamidi et al., 2009).

Vallejo et al. (2014) studied the shear strength of mixtures of sand and dispersed rock fragments commonly found in natural slopes and rockfill structures. Utilizing Ottawa sand and fine gravel in their experiments, they conducted direct shear tests under two normal stresses of 52 kPa and 103 kPa. They maintained a relatively constant initial

void ratio of the matrix at 0.8, while the gravel concentration by weight in the mixtures varied between 0% and 30%.

The research findings indicated that the mixtures' internal friction angle increased with gravel concentration. Specifically, the friction angle rose from 40.4 degrees for sand alone to 46.9 degrees for a sand sample with a 20% gravel concentration by weight (12.1% by volume).

Vallejo et al. (2014) proposed that the shear strength of the sand-gravel mixtures could be determined from the shear strength of the sand matrix alone, combined with the volume concentration of the gravel, using the equation: $S_c = S_m(1 + 2.5C)$. In this equation S_c denotes the shear strength of the sand-gravel mixture, S_m is the shear strength of the sand matrix, and C represents the gravel's volume concentration.

2.7. Literature Review of CFRDs and CFSGD

The efficacy of concrete face rockfill dams is assessed based on factors such as their vertical and horizontal movements, long-term creep behavior, rate of leakage, and the deflection observed in their concrete faces. Except for two documented instances of failure—the Gouhou Dam and the Upper Taum Sauk Dam—the issues arising in CFRDs have generally been repairable (Cruz et al., 2009).

The Gouhou Dam in Qinghai Province, China, was breached on August 27, 1993, resulting in 320 fatalities and significant property damage. Standing at 71 meters high, this dam represents the first known instance of a CFRD failure, a dam built with modern construction methods (Mei et al., 2023).

On December 14, 2005, the Upper Taum Sauk Dam, located in Reynolds County, Missouri, USA, underwent a breach. This breach, caused by overtopping flow due to faulty reservoir stage sensors, resulted in nine fatalities and significant property damage. The construction of the project was completed in 1962 (Mei et al., 2023).

The performance of CFRDs is primarily influenced by two factors: the dam's height and the valley's shape. The stresses accumulating in the rockfill lead to vertical and horizontal displacements. As the pressures on these materials increase, their

deformability modulus decreases, resulting in more significant proportional deformations in higher dams. Additionally, the valley's shape significantly affects the arching effects, which become more pronounced in narrower valleys—a well-established fact.

The vertical deformation in rockfill bodies generally follows a logarithmic trend, marked by high settlement rates soon after construction. Over time, these rates decrease, leading to extended deformation due to secondary breakages in the rockfill, a process referred to as creep. Although impoundment initially results in increased settlements, the creep primarily dictates the long-term settlement of CFRDs post-impoundment.

During the construction phase, horizontal movements in the rockfill typically occur towards both the upstream and downstream faces, as is typical in standard embankment construction, and are influenced by the rockfill's construction timetable. In the impoundment stage, the embankment undergoes horizontal deflection towards the downstream side due to the exertion of hydraulic pressure. Following the impoundment phase, the horizontal deformations stabilize into a creep phase, characterized by reduced deformation rates. A challenge in monitoring these horizontal displacements lies in establishing reliable reference points, as the process of sealing and affixing monitoring instruments can compromise the accuracy of the measurements.

Leakage in CFRDs is an essential consideration for operational efficiency, though it usually does not pose a significant threat to stability. There have been cases reported where even leakage rates in the thousands of liters per second have not caused instability. The peak leakage rates typically occur during the impoundment phases, but in some CFRDs, increased leakage has been observed years post-impoundment. Acceptable leakage rates for CFRDs generally range from 30 to 400 liters per second, varying based on the project's unique features. Literature indicates that leakage rates surpassing 800-2000 liters per second should trigger investigations and remedial

measures. Reinforcing the rockfill, incorporating fine materials, and using sand-mortar mixtures are among the solutions frequently employed in such scenarios.

2.7.1. Golillas Dam (Colombia, 1978)

It is a concrete face gravel fill dam with height of 125 m. Golillas Dam is located on very narrow valley with a ratio of 0.86 (H/V). Since the shape of valley is quite narrow, concrete slab and exclusively perimeter joint has behaved different from other dams. This different behavior has caused leakage which does not affect the dam safety. The use gravel material as rockfill causes flatter slopes and requirement of a chimney drain. The degree of hydraulic gradient of the plinth in the foundation and the rock properties in the abutments necessitated placing zone 1 (semi-pervious) between the concrete face and chimney drain along the abutments. The cross-section of Golillas Dam is given in Figure 2.24.

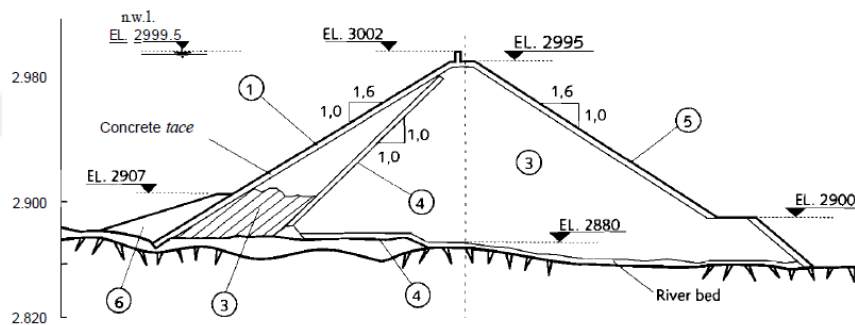


Figure 2.24. Cross-section View of Golillas Dam

Some characteristics of dam are L (crest length): 108 m, $L/H = 0.86$, $A/H^2 = 0.92$, upstream and downstream slopes are $1.6H/1V$ (Amaya & Marulanda, 1985).

2.7.2. Salvajina Dam (Colombia, 1983)

Salvajina Dam is a 148 m high gravel/rockfill dam with an impervious reinforced concrete face. Maximum section of the dam body is given in Figure 2.38. It was built for producing energy and flood control. The construction was started on July 1982 and finished on November 1984.

Some characteristics of dam are L (crest length): 362 m, L/H - 2.44, A/H^2 -2.62, upstream slope 1.5H/1V (gravels); downstream slope 1.3-1.4 H:1V (rockfill) and the volume is 3395000 m³ (Sierra et al., 1985).

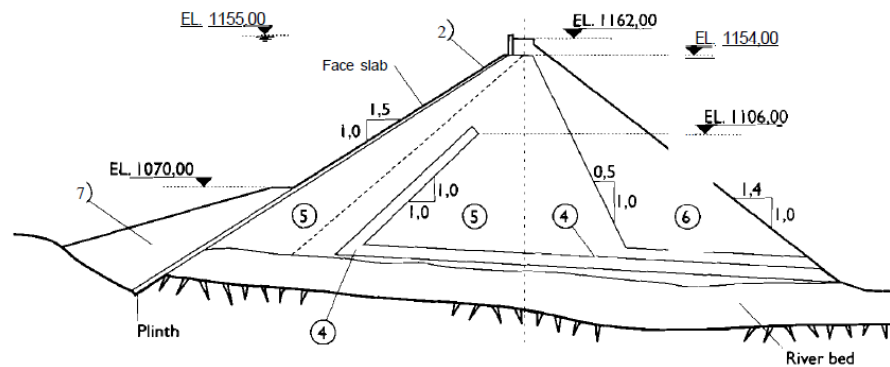


Figure 2.25. Cross-section View of Salvajina Dam

2.7.3. Aguamilpa Dam (Mexico, 1993)

Aguamilpa dam is a 187 m high CFRD situated in central Mexico. The dam is part of a multipurpose project including irrigation, flood control and electricity generation with an installed capacity of 960 MW.

Aguamilpa Dam was the highest CFRD in the world until 2006 and alluvial deposits were used in the upstream embankment as filling material. The dam's volume amounts to approximately 13000000 m³. Some characteristics of dam are L (crest length): 642 m, L/H - 3.43, A/H^2 -3.92, upstream slope 1.5H/1V (gravels); downstream slope 1.4 (H):1V (rockfill) (Montanez, Haelas&Castro, 1993). The crosssection of Aguamilpa Dam is given in Figure 2.26.

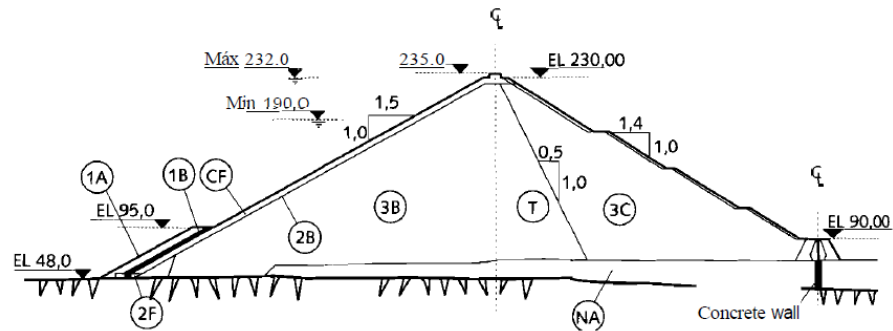


Figure 2.26. Cross-section View of Aguamilpa Dam

2.7.4. Mazar Dam (Ecuador, 2008)

The dam constructed in an asymmetric valley with a notably steep right abutment, having a slope of 0.60H:1V. The upstream slope 1.4H/1V and a downstream slope of 1.5H/1V, encompassing a volume of 5000000 m³. Some characteristics of dam are H (height)-166 m, L (crest length)- 340 m, L/H- 2.05, A/H²-1.7.

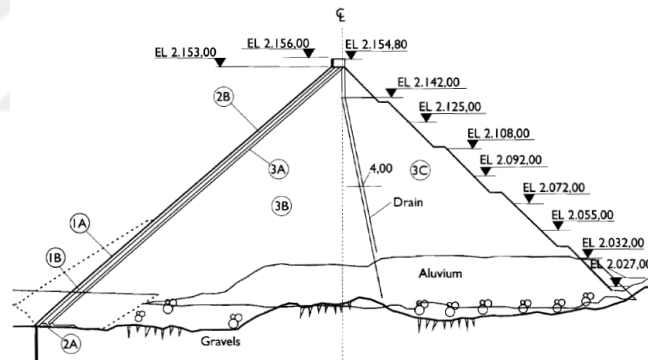


Figure 2.27. Cross-section View of Mazar Dam

2.7.5. Shuibuya (China, 2009)

The Shuibuya Dam, the world's tallest CFRD at 233 meters, was constructed between 2002 and 2007, as (Zhou et al., 2011). Built on shale and limestone, the dam exhibited most of its total settlement during construction, with over 85% occurring in this initial phase. As the reservoir filled, settlement rates increased more rapidly upstream than downstream, likely due to lower infiltration rates. Some characteristics of dam are L (crest length): 660 m, L/H- 2.83, A/H²-2.21, upstream

and downstream slopes are 1.4H/1V. The cross-section of Shuibuya Dam is given in 2.28.

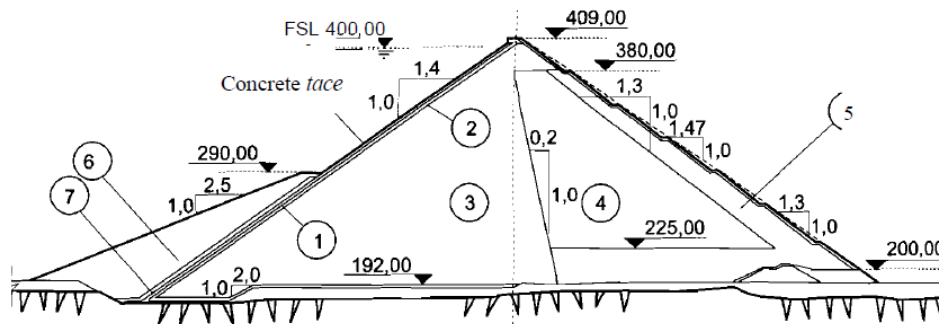


Figure 2.28. Cross-section View of Shuibuya Dam

CHAPTER 3

DERİNCE DAM

3.1. General Information

Derince Dam is a concrete face rockfill dam (CFRD), utilizing sand-gravel material as the rockfill component with a maximum height of 62.50 m and crest a length of 360 meters. The dam is located in the Milas District, within the province of Muğla, and is situated 20 km northwest of the Milas District and 2 km north of Derince. It was constructed on Derince Stream. This dam has ensured the provision of irrigation water for 830 hectares of agricultural land.

In tender design, the body type of Derince Dam was an earth core sand-gravel dam. Nevertheless, challenging weather conditions characterized by heavy rainfall, the difficulty in sourcing impervious soils, and the increasing prevalence and expertise in Concrete-Faced Rockfill Dam (CFRD) implementations in Turkey led to its redesign as a CFRD.

The dam's location in Türkiye map, general layout view, maximum cross-section, longitudinal section, and photos from construction and impoundment stages are given in Figures 3.1., 3.2., 3.3., 3.4., 3.5., and 3.6., respectively.

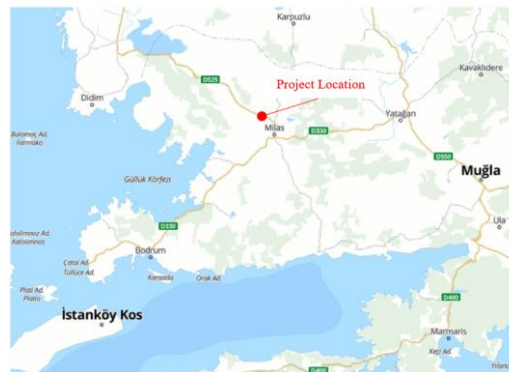


Figure 3.1. Location of Derince Dam on Türkiye's Map

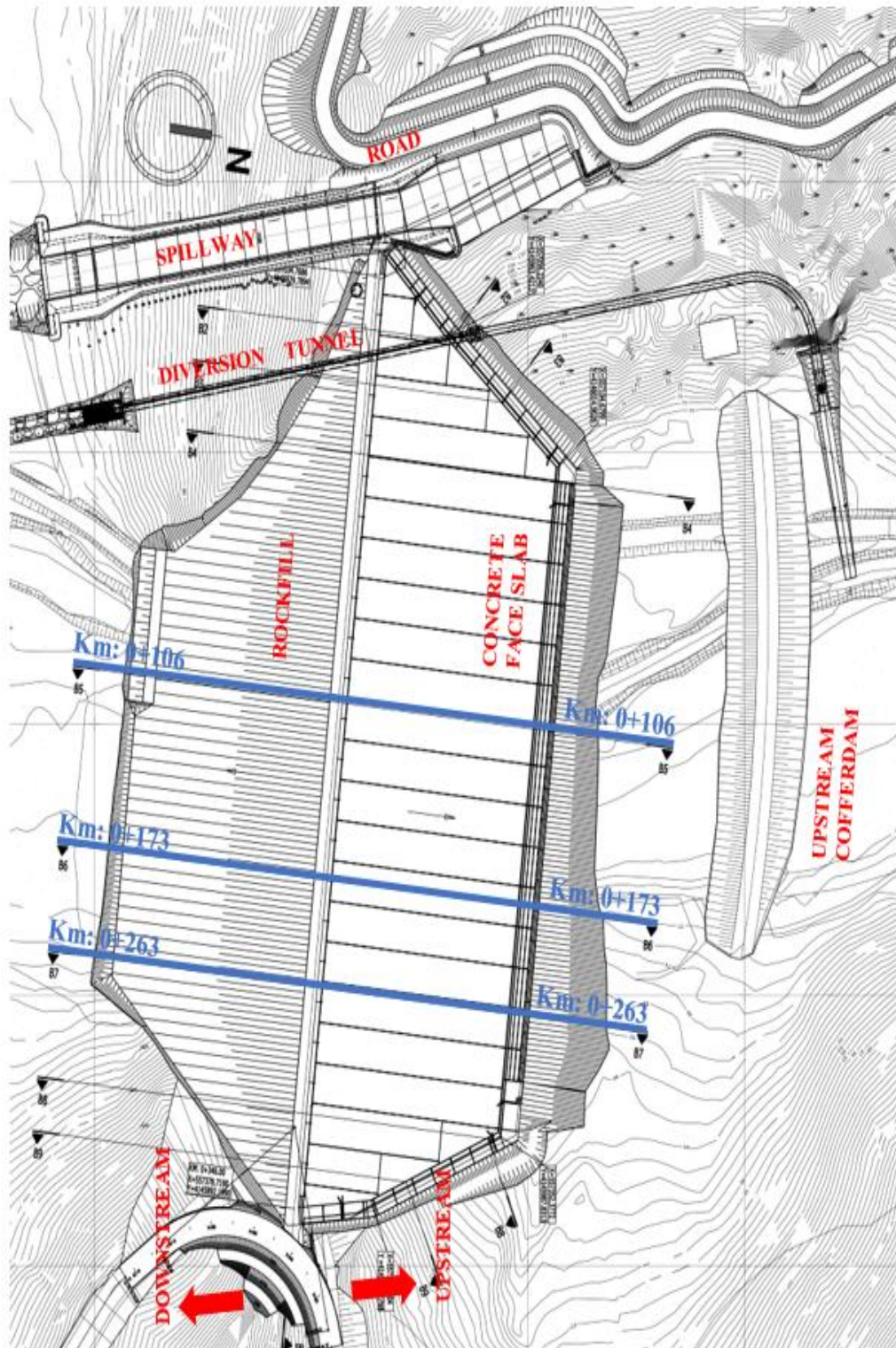


Figure 3.2. General Layout View of Derince Dam Showing Instrumented Cross-Sections at Km: 0+106, 0+173, and 0+263 (not to scale)

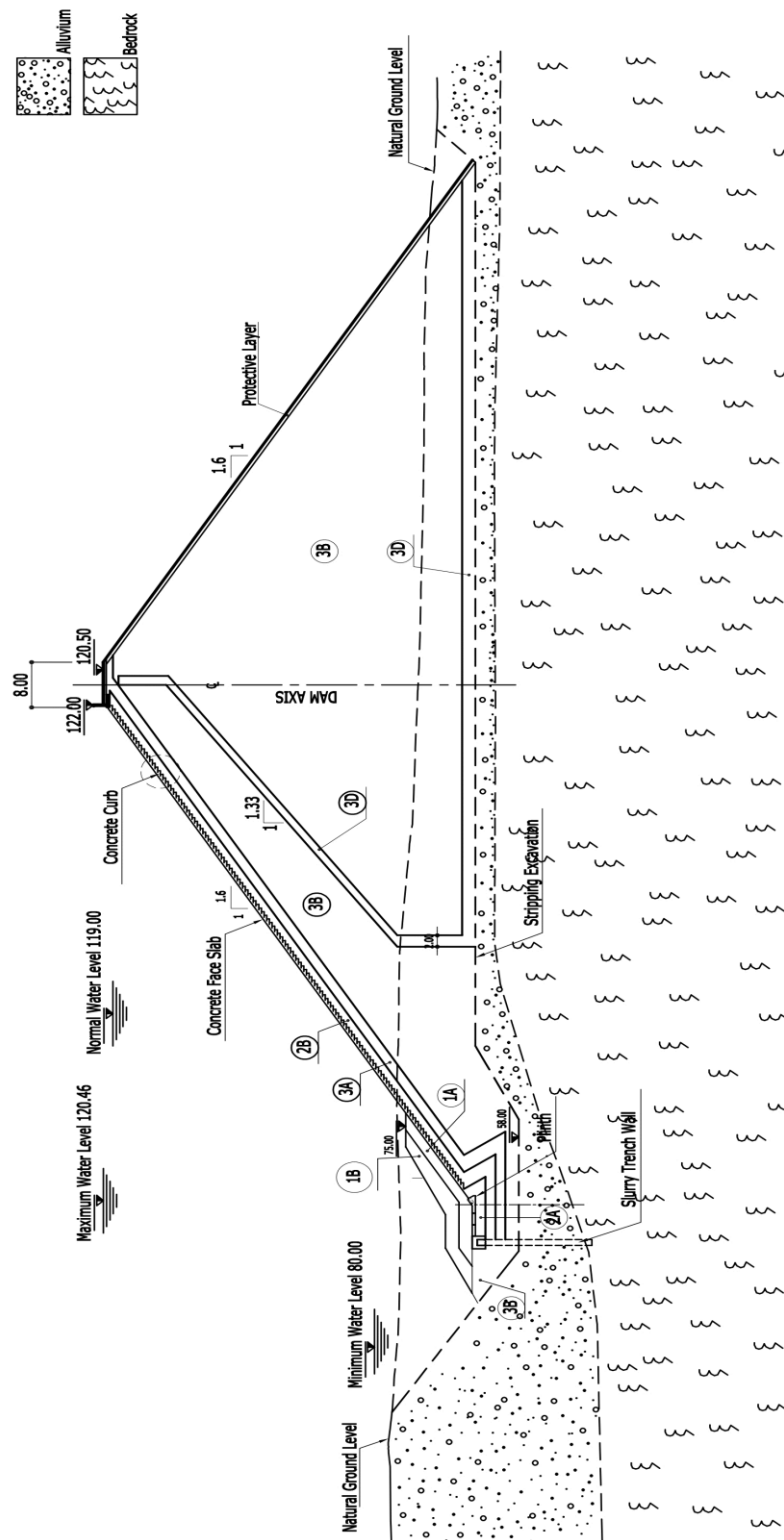


Figure 3.3. Maximum Cross-Section of Derince Dam (not to scale)

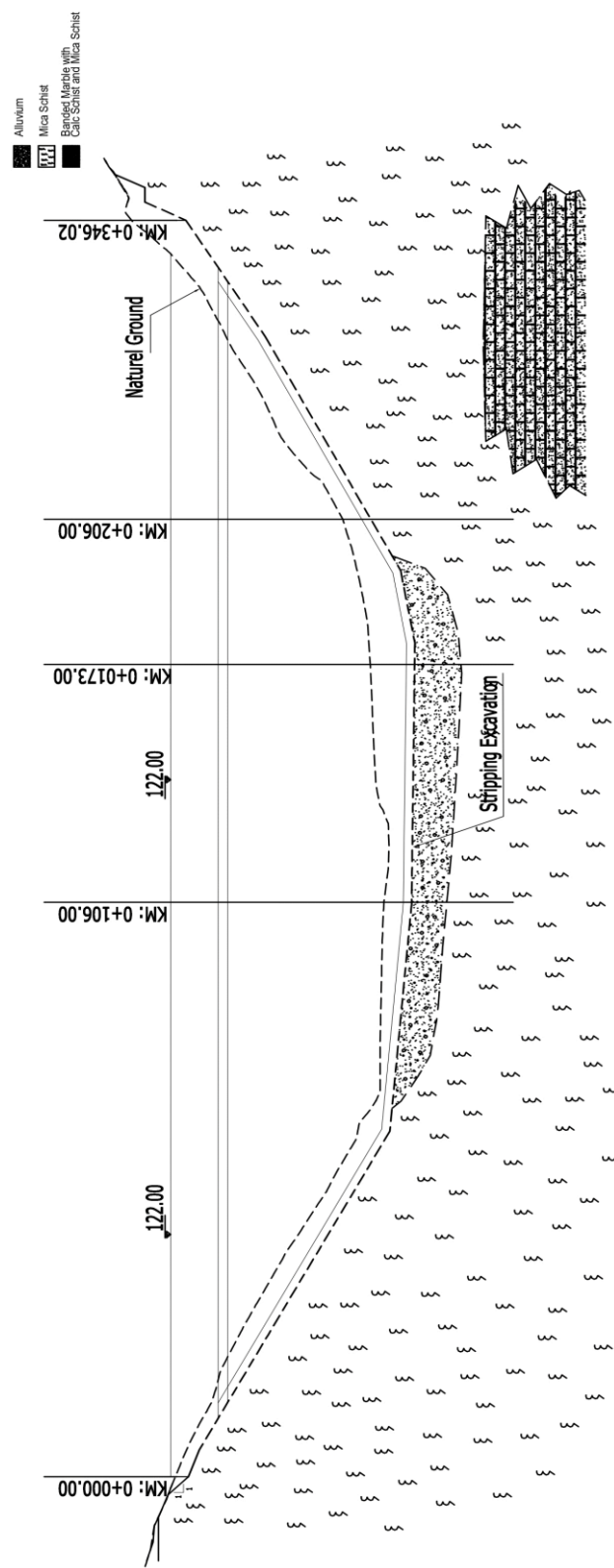


Figure 3.4. Longitudinal Section View from Dam Axis (not to scale)



(a)



(b)



(c)

Figure 3.5. a) Rockfill Construction, b) Concrete Curb Installation, and c) Placing of Concrete Face Slab



(a)



(b)

Figure 3.6. a) Concrete Face Slab upon Completion of Construction, and b) Derince Dam After Impoundment

Construction of the dam project commenced in 2011, with the rockfill embankment completed on December 16, 2013. Subsequently, after the completion of the rockfill construction, a waiting period of five months was observed to allow the dam to settle.

However, contrasting this approach, the Kürtün Dam, as documented by Özkuzukıran (2005), experienced a different process. After completing the rockfill

construction at Kürtün Dam, a deliberate waiting period of 1.5 years was implemented before the construction of the concrete facing commenced.

After the data obtained from the hydraulic settlement cells indicated that the settlement of the dam was complete, construction of the concrete facing on the upstream facing started, given the significant increase in the construction of CFRDs in Turkey at that time and the use of sand-gravel as rockfill, there was no prolonged waiting period after completing the embankment construction.

Following the completion of the concrete facing, the construction of the parapet wall along the dam's crest and the fill behind it were finalized. The reservoir impoundment commenced on November 14, 2014, and the water level reached an elevation of 111.59 meters by June 01, 2015. The characteristics of Derince Dam are given in Table 3.1.

Table 3.1. Characteristics of Derince Dam

Construction Purpose	Irrigation (830 ha)
Dam Type	Concrete Face Sand-Gravel Dam
Elevation From Thalweg	52.00 m
Elevation from Foundation	62.50 m
Thalweg Level	70.00 m
Crest Level	120.50 m
Crest Level of Parapeth Wall	122.00 m
Crest Length	346.00 m
Slopes	80.00 m
Upstream and Downstream Slope	1.6H : 1V and 1.6H : 1V
Total Rockfill Volume	1.30 hm ³
Normal Water Level	119.00 m
Maximum Water Level	120.46 m
Minimum Water Level	80 m
Volume of Reservoir at NWL	20,60 hm ³

The dam is constructed on a wide, alluvial valley, and the river bed is approximately 150 m. The average slopes of the right and left abutments are 27° .

Metamorphic rocks (such as mica schist, biotite schist, calc schist, and garnet-bearing schists) are the main rock foundations in the dam site. The mica schists exhibit characteristics ranging from impermeable to slightly permeable. At depths of 8-15 meters, these schists are highly weathered and clayey, while at greater depths, they are less weathered, exhibiting fresh rock properties, and are of weak to medium strength.

The alluvium's thickness beneath the plinth's axis reaches up to 15 meters. Alluvial consists of slightly clayey silt, sandy coarse gravel, and blocks. Seven meters of the alluvium found at the dam's base has been removed.

Alluvium of varying depths is present under the toe slab and dam axis. Additionally, different thicknesses of alluvium are found both in the upstream-downstream direction and from the right bank to the left bank. In this study, alluvium exists beneath each of the three analyzed sections.

To provide impermeability on the dam's foundation, an impermeable element has been created in the alluvium using the slurry trench method, and this element has been socketed 2 m into the bedrock. In the right and left abutments, which consist of mica-schist, and beneath the alluvium in the riverbed, curtain grouting has been performed to ensure impermeability. An impermeable unit, composed of a grout curtain and a slurry trench wall, formed with a maximum depth of 45 m.

In Derince Dam, the dam zones are established following an upgraded standard zoning definition, which involves the use of sand-gravel fill instead of rockfill, and a chimney drain. These designated zones – 1A, 1B, 2A, 2B, 3A, 3B, and 3D – are strategically positioned, as depicted in the representative cross-section in Figure 3.3. The compaction specifications, detailed in the construction drawings, are provided in Table 3.2. Table 3.3. gives the particle size distributions for 2A, 2B, and 3A.

Table 3.2. Zoning of Derince Dam

Zone	Material	Dmax (cm)	Layer Thickness (cm)	No. of Passes
1A	Cohesionless fine sand+silt fill	2.00	20-30 cm	-
1B	Unsorted pervious fill	20.00	20-30 cm	-
2A	Perimeter joint filter zone	3.81	20	4 + 150 L/m ³ water
2B	Cushion layer zone	7.62	40	Min.4 +150 L/m ³ water
3A	Transition zone between 2B and 3B	25.00	40	Min.4 +100-250 L/m ³ water
3B	Sand-Gravel Fill	35.00	60	4-6 +100-250 L/m ³ water
3D	Drainage Zone	35.00	60	Min.4 +100-250 L/m ³ water
Protective Layer	Placed by machine			

Table 3.3. Particle Size Distributions of 2A, 2B and 3A

Sieve Aperture (mm)	Percentage By Weight Passing		
	2A	2B	3A
250.00			100
150.00			65-100
76.20		100	55-95
38.10	100	70-100	35-70
19.10	85-100	55-80	25-50
10.00			18.-37
4.76	50-75	35-60	10-27
1.19	25-50	18-40	

0.59		6-18	5-15
0.297	10-25		0-10
0.074 (cohesionless)	0-5	0-7	

3.2. Instrumentation

In terms of dam safety, data collected from instruments play a crucial role during the construction and impoundment phases. This data provides information about the behavior and performance of the dam body. It is vital for the early detection of unexpected or abnormal conditions. This aids in examining any detected abnormal conditions, helps to verify satisfactory performance, provides feedback after any repairs, and offers data for future projects.

In the context of CFRD construction in Turkey, the State Hydraulic Works (DSI) has established guidelines necessitating the inclusion of various monitoring tools such as hydraulic pressure cells, hydraulic settlement devices, in-place and servo inclinometers, jointmeters, and vibrating wire piezometers. The number and locations of dam instrumentation are specific to each project. However, the Authority provides detailed stipulations regarding their strategic placement and the required accuracy of these instruments (DSI, 2014).

Derince Dam, a concrete face sand-gravel dam, has been extensively instrumented to observe its behavior. This instrumentation was vital due to some of the dam's unique structural features. While it is uncommon to find alluvium beneath the toe slab of such dams, this particular case presented a rare example where alluvium was present. This atypical geotechnical situation necessitated the strategic placement of settlement devices under the toe slab. These instruments were crucial for monitoring any potential settlement, providing essential data to understand the dam's performance under these unusual conditions. The types and numbers of instruments used in the Derince Dam are given in Table 3.4.

Table 3.4. General Information About Instrumentation of Derince Dam

#	Name of Instrument	Abbreviation	Number
1	Hydraulic Settlement Devices	SC	36
2	Hydraulic Pressure Cells	TPC	36
3	Jointmeters	JM	20
Table 3.4. (cont'd)			
4	External Settlement Gauge	SM	18
5	Strainmeters	SG	3
6	In-place Inclinometers	INC	3
7	3D Jointmeters	PJM	13
8	Vibrating Wire Piezometer	TP	27
9	Strong-Motion Accelerograph	AC	2

The instruments to be discussed in this study, hydraulic settlement devices and hydraulic pressure cells, are located at three stations: Km: 0+106, Km: 0+173, and Km: 0+263. The dam's cross-sectional views at these stations are presented in Figures 3.7, 3.8, and 3.9, where the instruments are marked at their designated locations are given in Table 3.4 and depicted in Figure 3.2.

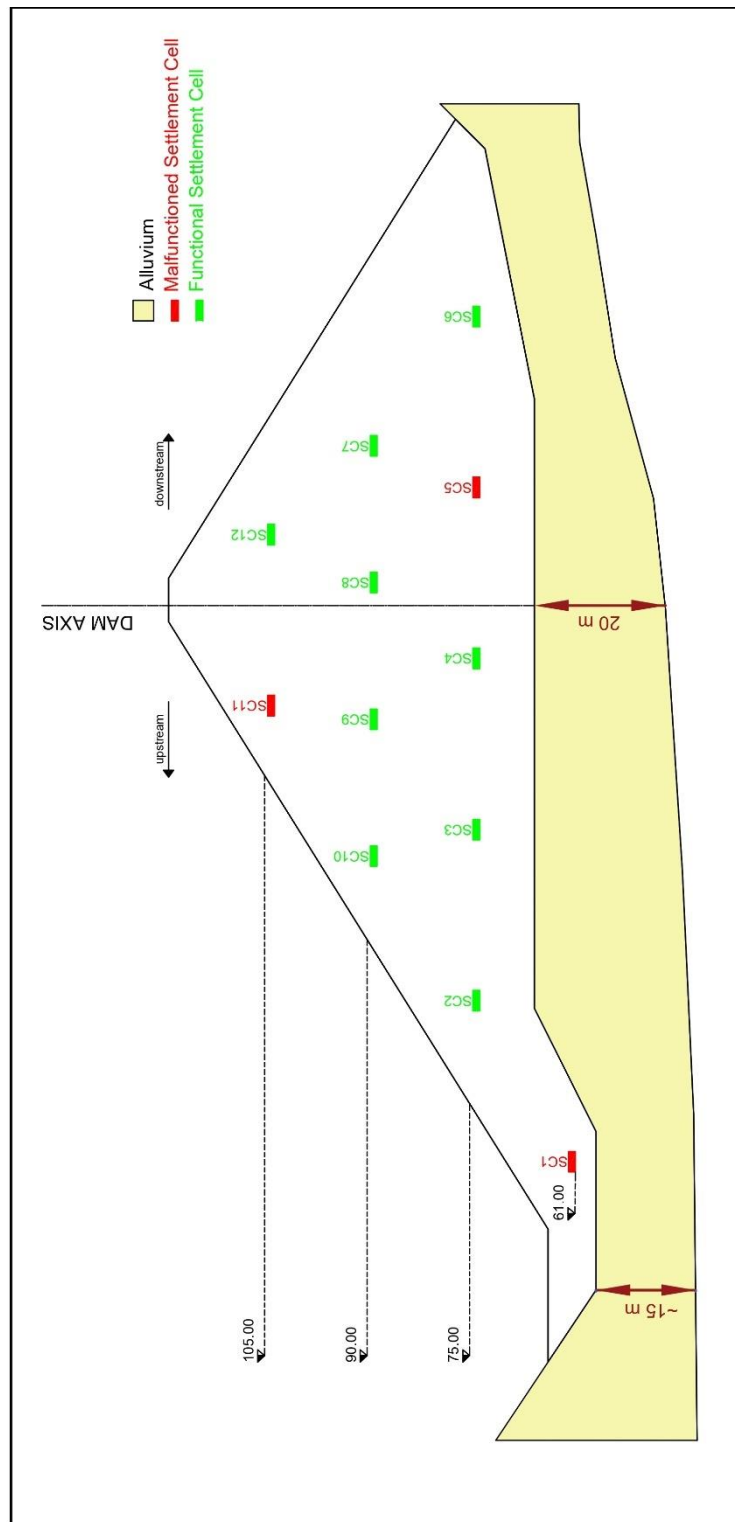


Figure 3.7. Location of Hydraulic Settlement Cells in cross-section Km: 0+106

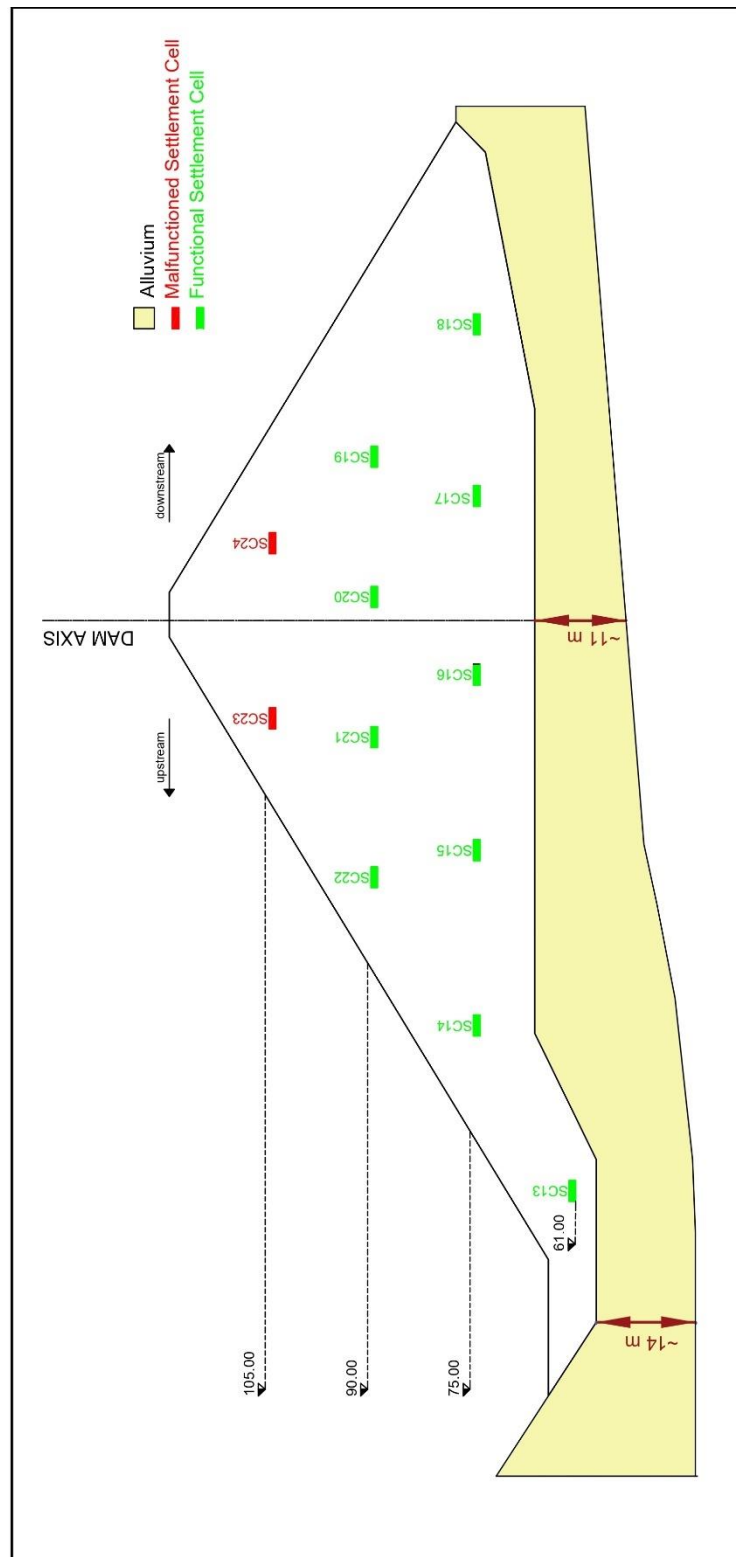


Figure 3.8. Location of Hydraulic Settlement Cells in cross-section Km: 0+173

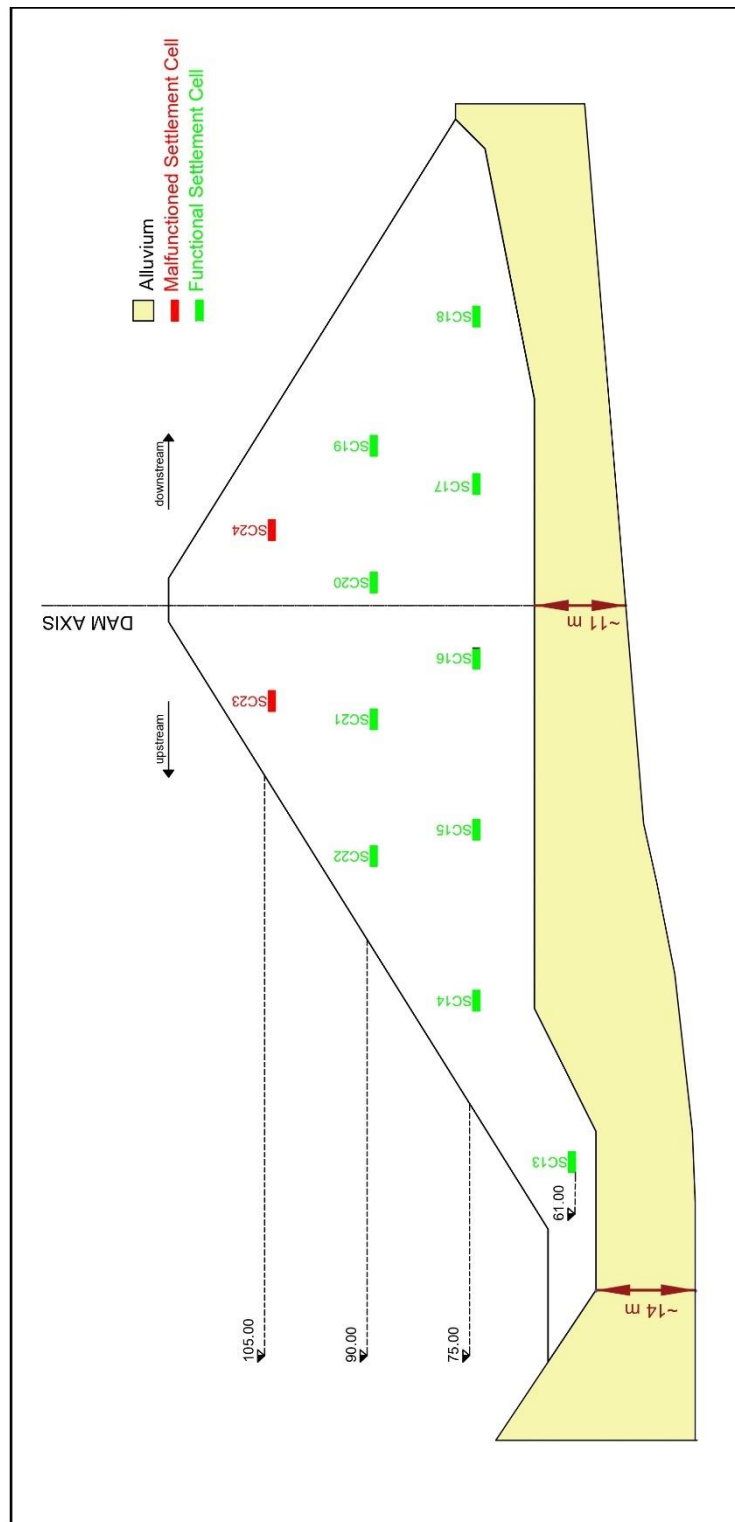


Figure 3.9. Location of Hydraulic Settlement Cells in Cross-Section Km: 0+263

3.2.1. Hydraulic Settlement Cells

In all embankment structures, the phenomenon of settlement is a natural occurrence due to the significant weight of the body. However, these deformations must remain within acceptable parameters to ensure the structural integrity and safety of the dam. Hydraulic settlement devices are strategically installed within CFRDs to monitor and evaluate the rate and amount of settlements. These devices act as indispensable tools for real-time settlement monitoring, crucial in contributing to the overall safety and performance assessment of CFRDs.

The settlement device has three main components: a liquid-filled tube, a pressure transducer, and a liquid reservoir. One end of the tube is connected to the pressure transducer, which is embedded in the soil. In contrast, the other end is connected to the reservoir at a higher elevation on stable ground, away from construction activities. The pressure transducer measures the pressure exerted by the liquid column inside the tube. The height of this column corresponds to the difference in elevation between the transducer and the reservoir. The column height increases as the transducer settles with the surrounding soil, resulting in an elevated pressure reading. To determine the settlement, changes in pressure are converted into units such as millimeters or inches of liquid head. This measurement provides valuable data for evaluating ground settlement and its impact on construction projects.

These devices are placed directly at the elevations specified in the body project and appropriately shielded. Transmitter cables are housed within protective pipes to safeguard them against potential damage from water or the dam's embankment material. Figure 3.10. shows the placement of hydraulic cells in Derince Dam, and Figure 3.11. shows the scheme of these devices.



Figure 3.10. Hydraulic Settlement Device Placement in Derince Dam

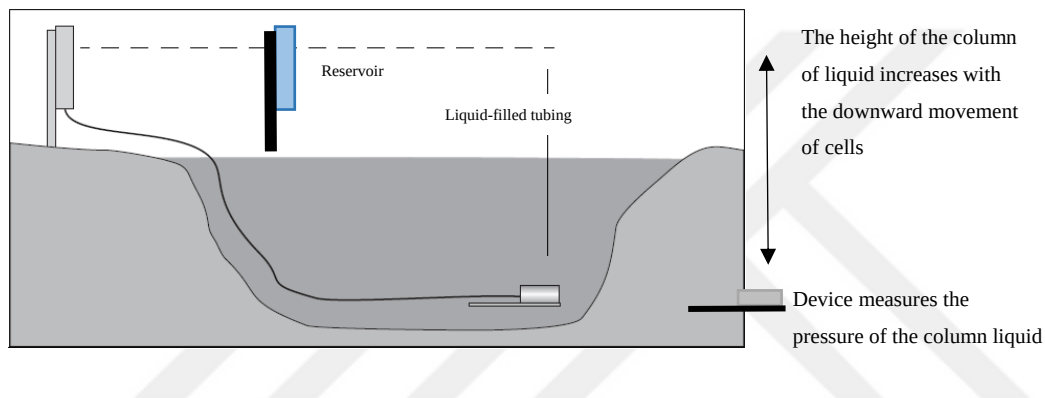


Figure 3.11. Scheme of Hydraulic Settlement Device

3.2.2. Hydraulic Pressure Cells

These devices measure the pressure caused by the embankment and the pore water pressure. They are also installed in dam bodies like hydraulic settlement devices. The total pressure devices used in the project are hydraulic systems consisting of two welded circular stainless steel plates enclosing hydraulic fluid along their circumference. Then, a transducer is attached to the plates. The pressure on this circular hydraulic fluid chamber is transferred to the vibrating wire reading unit based on the principle that liquids cannot be compressed. Pressure variations are then observed in the tension of the vibrating wire mechanism, leading to changes in its frequency. Figure 3.12. shows a hydraulic pressure device.



Figure 3.12. Hydraulic Pressure Devices

3.3. Evaluation of Observed Settlement Behavior of Derince Dam

As mentioned before, construction of the dam project started in 2011, with the rockfill completed on December 16, 2013. In the construction period, the performance of the dam had been observed by hydraulic settlement cells placing at different cross-sections, which are shown in Figures 3.7, 3.8, and 3.9 (the placement of cross-sections on the longitudinal section is given in Figure 3.2).

The settlement values recorded by the hydraulic settlement cells are presented in Appendix A. These values correspond to key dates: December 16, 2013, marking the end of construction (EOC), and June 07, 2015, indicating the reservoir impoundment condition (RF).

However, it is important to note that normal water level specified in the project is 119.00 m. The maximum level that the reservoir has reached thus far, which is 111.59 m. According to information provided by the General Directorate of State Hydraulic Works, the reservoir has not operated at a level higher than 111.59 m as of the current date.

3.3.1. Construction Period

Upon examining the data presented in Appendix A, it was observed that at the location Km: 0+106, the instruments SC-1, SC-5, and SC-11 were not functioning properly. Similarly, at Km: 0+173, instruments SC-23 and SC-24 and at Km: 0+263, instrument SC-33 were found to be malfunctioning. Consequently, these instruments have been excluded from the calculations. This exclusion is necessary to ensure the accuracy and reliability of the findings within the scope of this study. In the cross-sections at Km 0+106, Km 0+173, and Km 0+263, the height from the plinth axis of the dam is consistently 62.5 meters. However, the heights from the dam axis at these sections are 53.5 m, 56 m, and 56 m, respectively. It is important to note that all these cross-sections are located in the riverbed rather than near the abutments. Consequently, this study does not include any observations or comments regarding the abutments.

When assessments are based on dam heights, it was expected that the most significant settlements would occur in the Km 0+173, and Km 0+263, owing to their similar heights. Contrary to expectations, the largest settlement was observed in the Km 0+106 section, which contains approximately 14.57 m of alluvium below the dam axis. In contrast, the Km 0+173, section has 8 meters of alluvium beneath the dam axis, while the Km 0+263 section has no alluvium under it. Consequently, the least settlement was recorded in the Km 0+263.

According to the hydraulic settlement cells in the sections at Km 0+106, Km 0+173, and Km 0+263, the settlement values are presented in Figures 3.13, 3.14 and 3.15. Maximum settlement value is 516 mm which is recorded at SC4 at El. 75 m. In standard conditions, the maximum settlement is expected at the mid-height of the dam body. However, due to the presence of alluvium under the foundation of dam, the maximum settlement shifts to approximately 27% of the dam's total height ($H \times 0.27$), and it is observed in the Km:0+106. An evaluation is made by using the SC27 measuring instrument at Km:0+263, strategically positioned at the same elevation and distance from the dam axis as the projected point of maximum

settlement, revealed a settlement value of 310 mm. This discovery is very important because it confirms how alluvium affects the location and the amount of settlement. It gives us important information about how the dam behaves structurally in these special conditions.

In Derince Dam, measurement instruments (SC1, SC13, and SC36) were installed beneath the plinth at sections Km:0+106, Km:0+173, and Km:0+263 to monitor settlement patterns. SC1, however, consistently produced readings that were not consistent with expected outcomes throughout the construction phase. In contrast, the instruments in the other two cross-sections revealed significant settlements, with SC13 registering 230 mm and SC36 showing 112 mm of settlement. These measurements appear to be directly influenced by the alluvium beneath the dam, displaying a trend of decreasing settlement with reducing alluvium thickness.

3.3.2. Impoundment Period

At this stage, the most significant settlements were observed in the measurement instruments closest to the concrete face of the dam (SC2, SC10, SC14, and SC22). Generally, the values recorded by these instruments tended to decrease from the upstream towards the downstream. Instruments showing contrary trends are considered to be providing inaccurate readings. Additionally, at this stage, SC1, SC5, SC11, SC23, SC24, and SC33, which were previously malfunctioning, along with SC8, SC9, and SC34, were excluded from the analysis due to their failure to yield logical values. Upon examining Figures 3.13, 3.14 and 3.15, it was observed that the settlement values measured by the instruments inversely varied with the water load.

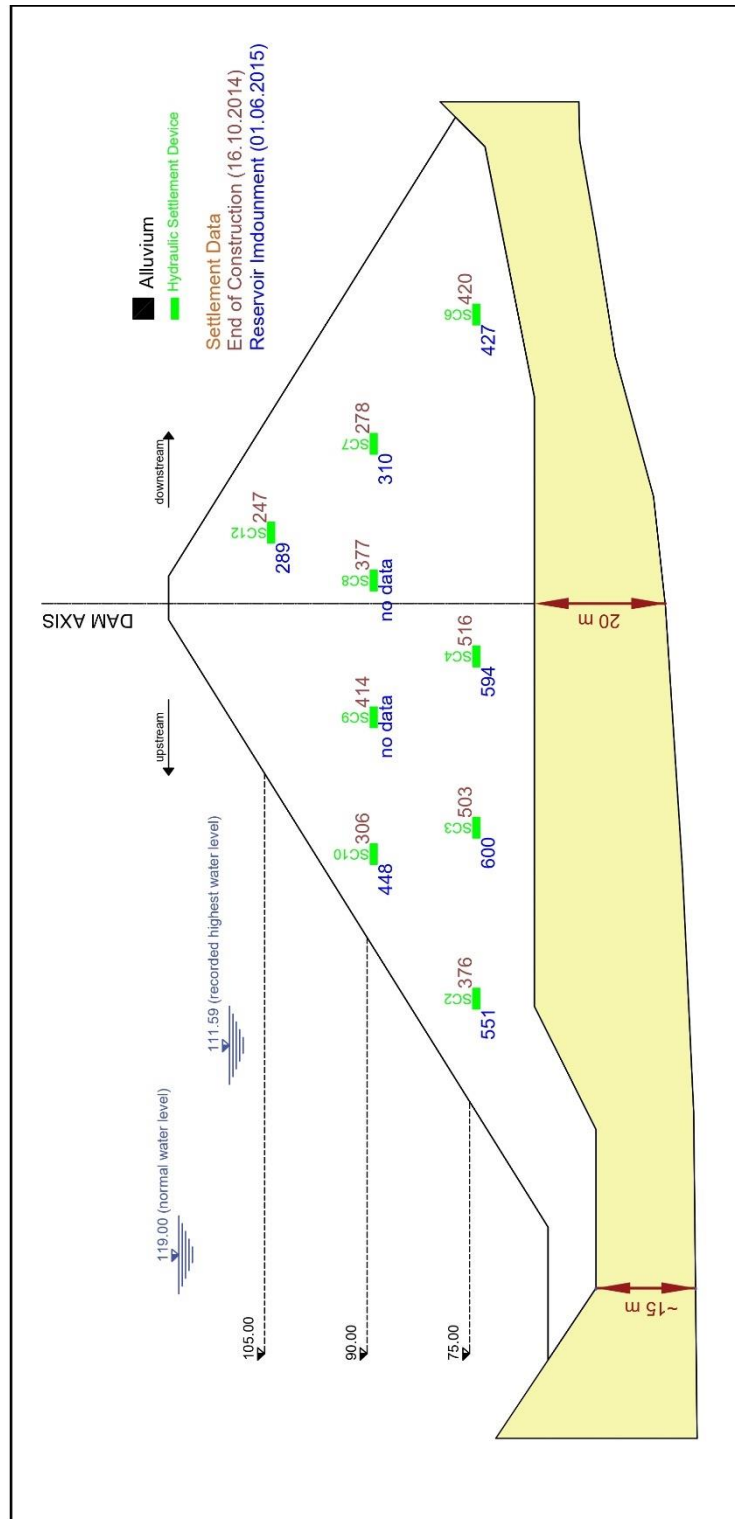


Figure 3.13. Recorded Settlements During End of Construction and Reservoir Impoundment at Km:0+106

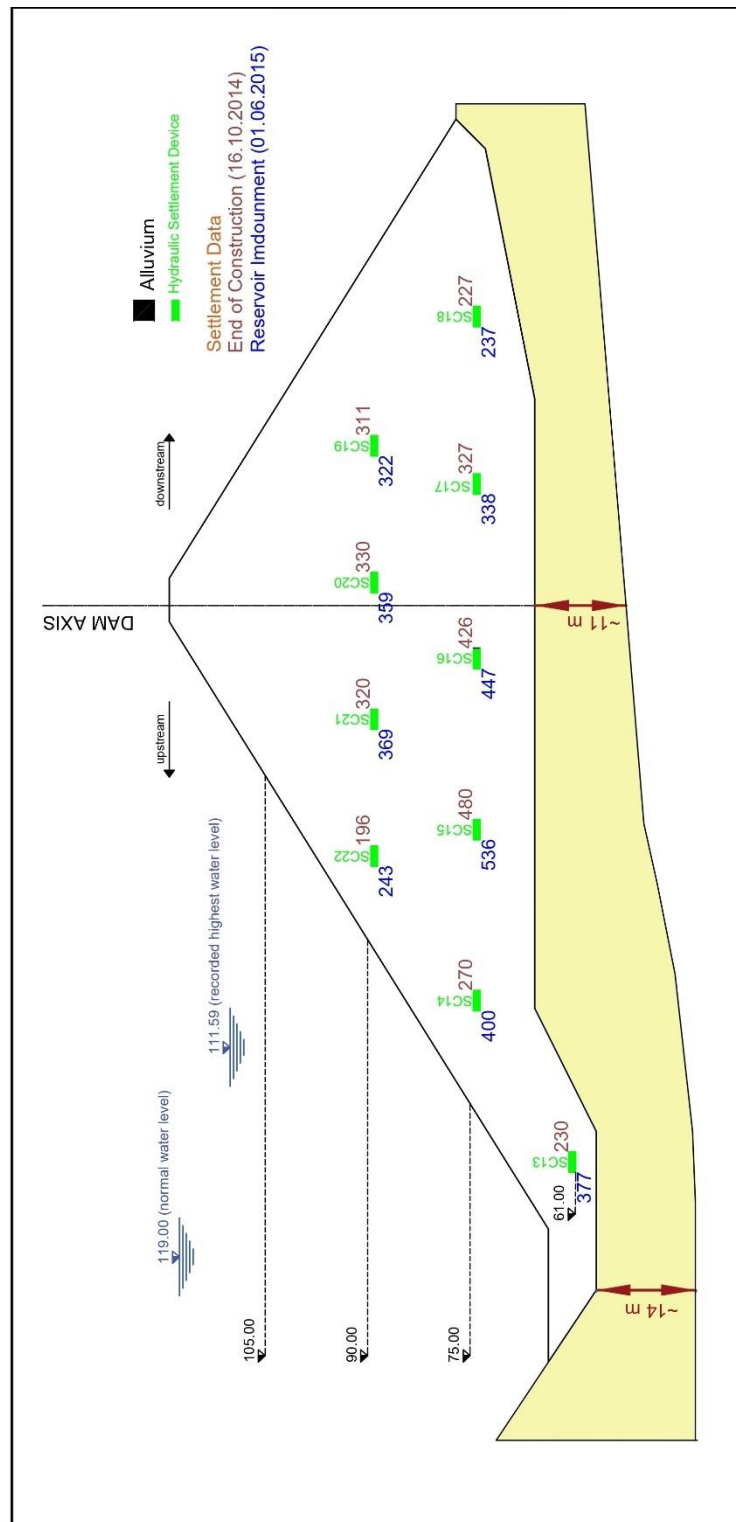
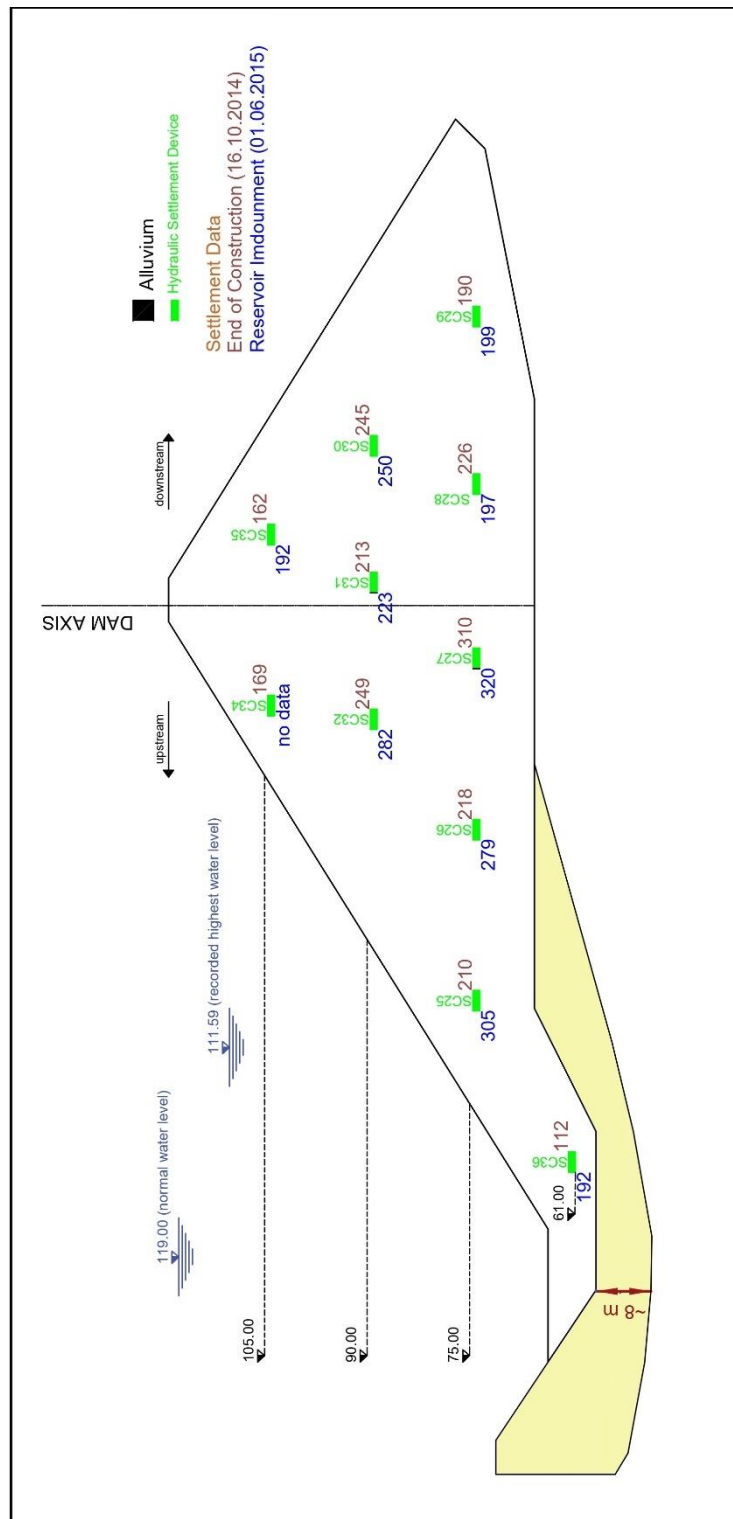


Figure 3.14. Recorded Settlements During End of Construction and Reservoir Impoundment at Km:0+173



3.4. Stress-Deformation Analysis of Derince Dam

For deciding the suitable type of analysis (two-dimensional or three-dimensional analyses) for accurate deformation assessment, four methods were evaluated in literature.

Among these, Pinto and Marques Filho (1998) introduced a metric known as the *valley shape factor*, derived from historical data, calculated as $SF = A/H^2$. In this equation, 'H' stands for the embankment height, and 'A' represents the area of the concrete face slab, with measurements in meters and square meters, respectively. This factor aids in categorizing valleys as either *narrow* or *wide*. Valleys with a shape factor above 4 are categorized as wide, while those below 3.5 are considered narrow. In wide valleys, the valley's shape has a minimal impact on deformations.

For Derince Dam, the height of the dam is 62.5 m from foundation. For a more accurate assessment, the height of the dam was calculated by accounting for both the rockfill and the alluvium beneath it. This approach led to a combined height measurement, consisting of 62.5 m from the rockfill and an extra 14.57 m from the alluvial layer. Therefore, the overall height of the dam is taken to be 77.07 m in this calculation. The area of the concrete facing of Derince Dam is 27,000 m². With using these parameters Derince Dam's *valley shape factor* is calculated as 4., which means the dam constructed on *wide* valley. This categorization suggests that a two-dimensional plane strain analysis is sufficient for Derince Dam, thereby the arching effect is excluded in this study.

Guidici et al (2000) performed a three-dimensional study on a model CFRD, adjusting both the width of the riverbed and the inclines of the abutments to explore their effects on deformation, as depicted in Figure 3.16. As mentioned before, Derince Dam, was constructed on a wide, alluvial valley, and the width of riverbed is approximately 150 m. The average slopes of the right and left abutments are 27 ° (1V/1.75 H).

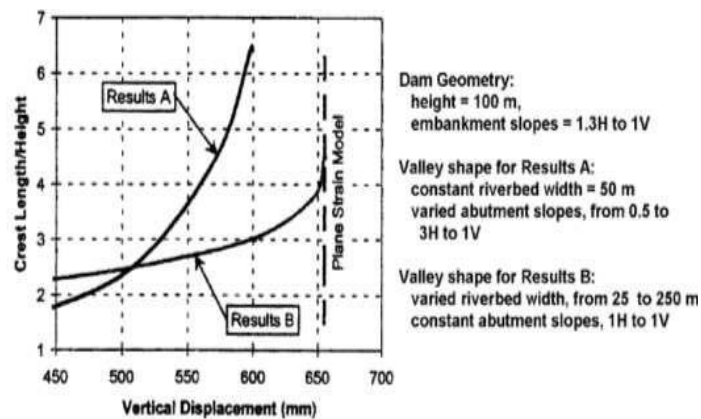


Figure 3.16. Effect of Valley Shape on Vertical Displacement (Guidici et al., 2000)

The length of crest of Derince Dam is 346 m, and to be on safe side the height of dam is taken as 77.07 m. The crest length/height Ratio of Derince Dam is found as 4.48. Based on the data presented in Figure 3.16, it is considered that Results B is more suitable for Derince Dam. Although, the abutment of slopes of Derince Dam are much flatter. According to crest length/height ratio, plane strain model is deemed sufficient for this study.

It's important to recognize that the analysis conducted by Guidici et al. (2000) is predicated on the assumption that the embankment is constructed in a singular stage. According to Hunter and Fell (2002), this assumption potentially results in an underestimation of stresses within the rockfill.

Hunter and Fell (2002) examined the impact of valley shape using a two-dimensional layered finite difference analysis on a hypothetical 100-meter-high embankment. Their findings reveal that:

- Pronounced cross-valley arching, marked by a decrease in vertical stress of more than 20%, is primarily found in narrow valleys where the river's width is less than 30-40% of the dam's height. This is accompanied by steep abutment slopes, often exceeding 50 degrees, although this can vary depending on the river's width. This phenomenon is typically observed in the bottom third to half of the embankment.

- In scenarios where the river's width is about half of the embankment's height, the impact of cross-valley arching on stress levels is moderate, with a 10 to 20% reduction in vertical stress. This is particularly the case for abutment slopes steeper than about 45 degrees. The effect is generally restricted to the lower portion of the embankment.
- When the river's width is roughly equal to the embankment's height, the arching effect is negligible, regardless of the steepness of the abutment slope.

Sukkarak et al. (2019) stated that the valley shape factor, typically represented as A/h^2 , has limitations in its ability to accurately reflect the complex influences of dam geometry, particularly in various valley conditions. Thus, they presented modified valley shape factor as S_β , which is defined as the product of A/h^2 and $(\tan \beta)^{-0.5}$, where β is the slope of the abutments. The modified shape factor is calculated as 6.37, 6.49 and 7.6 for Km: 0+106, Km: 0+173, and Km: 0+263. S_β being less than 3 indicates that the valley is *narrow*. When S_β being greater than 7, indicates *wide valley* configuration where the impact of valley shape is not significantly important. This analysis can be simplified as a 2D plane strain problem.

For Derince Dam, the choice to not conduct a 3D analysis is supported by two main factors: first, the evaluation of two crucial cross-sections shows that their modified valley shape factors, while slightly below the key value of 7, are still close to it. This proximity indicates that the variations in valley shape are within acceptable parameters, suggesting that the dam's structural stability does not require a more intricate analysis. Second, this decision is further justified by the approach used to calculate the dam's height, which combines the heights of both the alluvial deposits and rock fill.

This study primarily investigates two critical scenarios impacting settlement: the condition at the end of construction (EOC) and the condition when the reservoir is impounded or full (RF). In the EOC situation, the settlement is due to the inherent weight of the rockfill, whereas in the RF case, it's influenced by the pressure of the

water. The analyses utilize a two-dimensional (2-D) plane strain model, focusing on the dam's cross-section at Km: 0+106, Km:0+173 and Km: 0+263.

3.4.1. Staged Construction

In models that consider embankments being constructed in a single stage, the highest settlements are seen at the crest and upper third, a finding that does not align with actual deformations in dams. In real situations, the step-by-step construction of the embankment helps to balance out the initial settling of each layer (due to its own weight) with the material added in the next layer. In effect, this process neutralizes the initial settlements, while the overall heightening of the embankment leads to an increase in deformations.

Duncan et al. (1980) noted that for two-dimensional finite element analyses, the embankment needs to be modeled with at least 8 layers to accurately calculate deformations. This requirement generally aligns with the findings presented by Kulhawy (1972) for both linear elastic and stress-dependent hardening soil models.

To achieve a highly realistic simulation of dam behavior, a staged construction model is adopted. An essential element of this model is the specification of layer thicknesses, with the understanding that a greater number of thinner layers enhances the precision of the analysis. Nonetheless, this precision comes at the cost of increased computational time and resources. The study adopts this multi-layered construction method. In the preliminary analyses, 5 m, 2.5 m, and 1 m thick layers are used. When the analyses results compared with data from hydraulic settlement cells, it has been decided to use 1 m thickness.

For the End of Construction (EOC) phase, 62 stages were implemented. With the onset of the 63rd stage, the settlements were recalibrated, and the concrete face was specified. At the commencement of the 64th stage, the settlements were readjusted, the reservoir loading was predicted, and the resulting displacement and stress values from the reservoir load were calculated and compared against the measured results.

For the calibration of the model, the value at the point represented by the measuring instrument is calculated by subtracting the value read at the level where the measuring instrument is placed from the value measured at the final level of the fill.

3.4.2. Elements and Finite Mesh Used in the Analyses

In the Plaxis software, users can choose between 6-node and 15-node triangular elements for modeling soil layers and other volumetric features. These elements are illustrated in Figure 3.17.

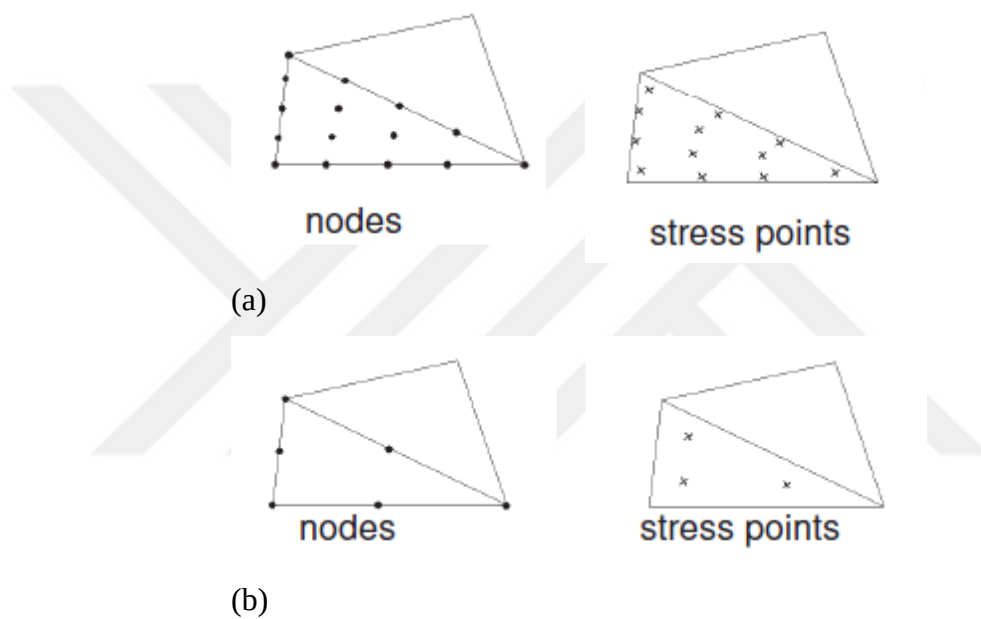
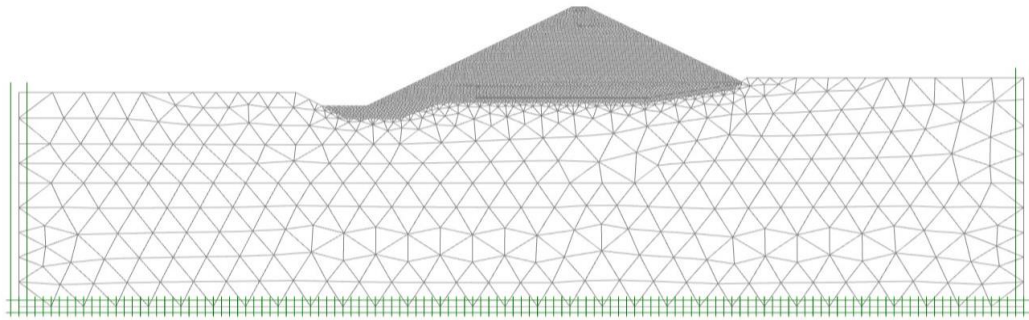


Figure 3.17. (a) 15-noded elements, (b) 6-noded elements

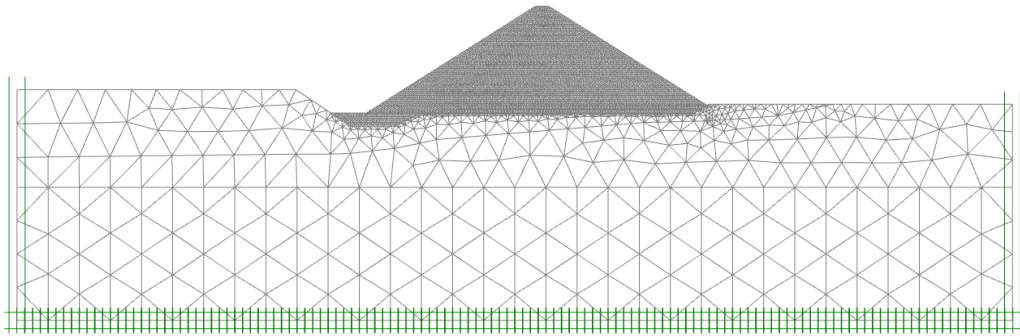
The default choice for modeling is the 15-node triangular element, which offers fourth-order displacement interpolation and utilizes twelve Gauss points for numerical integration of stresses. This element type is automatically aligned with the selected soil element type for both structural elements and interfaces. The 15-node triangle is highly accurate, consistently delivering superior stress analysis results in complex scenarios. However, its use results in increased memory usage and slower computation and operational performance.

For Derince Dam, Plaxis 2D v21 has been utilized for conducting stress-strain analysis studies, reflecting a targeted approach in the investigation of structural

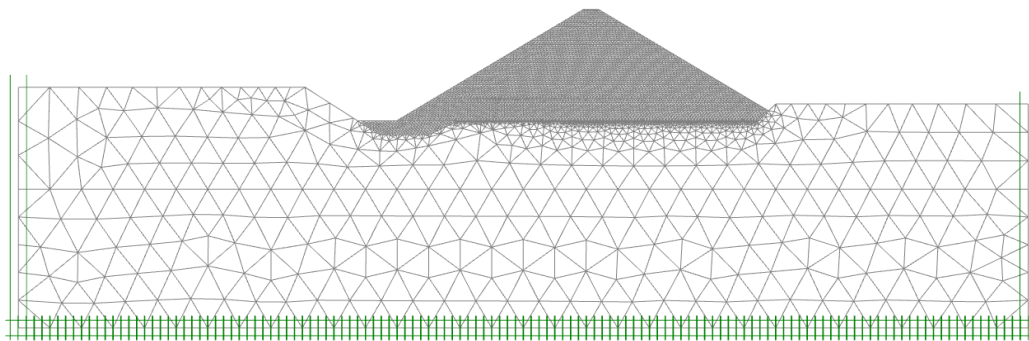
responses. The configurations of the finite element mesh, including the total number of elements, and nodes, across all cross-sections, are illustrated in Figure 3.18.



(a) Km: 0+106 with 7968 elements, 64137 nodes



(b) Km: 0+173 10079 elements, 81099 nodes



(c) Km:0+260 9432 elements, 75865 nodes

Figure 3.18. Configuration of Mesh for All Cross-sections

3.4.3. Foundation of Derince Dam

In the context of these analyses, the dam's foundation consists of mica schist, weathered mica schists, and alluvial layers. The soil parameters for these layers were

obtained from the Engineering Geology Report of the Muğla Milas Derince Dam, compiled in July 2003.

In the analysis, the alluvium layer is modeled using the hardening soil model, while for both mica schist layers, the Hoek-Brown model is employed.

The geological report in question, while providing a detailed description of the alluvium layer at the Derince Dam site, regrettably lacks specific geotechnical parameters for this layer. It delineates that the alluvium, located in the riverbed of the Derince River, is primarily constituted of coarse gravel and blocks. This material exhibits a heterogeneous composition, comprising slightly clayey, silty, and sandy elements. The grading of the material is described as poor, and it encompasses a diverse array of geological constituents, namely mica schist, garnet-bearing mica schist, quartz schist, calc schist, quartz, and gneiss. In certain areas, the composition is exclusively silt, clay, and sand. The description of the gravel particles notes them to be sub-rounded to sub-angular in shape, with an average grain size oscillating between 3-4 mm and 1-4 cm. The maximum block size recorded in the report is approximately 20 cm. The permeability of alluvium and both mica-schists are 10-5 cm/s and $3 \cdot 10^{-7}$ cm/s, according to permeability and lugeon tests. And the water level is measured 5.7 m below foundation level in thalweg. Thus, the parameters of alluvium strata are taken from literature and hardening soil model is used.

Table 3.5. Hardening Soil Parameters of Alluvium

$E_{ref}^{50} = E_{oed}^{ref}$ (kPa)	$E_{ur} = 3 \cdot E_{ref}^{50}$ (kPa)	m	R_f	c (kPa)	ϕ (°)	ψ (°)
25000	75000	0.5	0.7	10	36	6

The Hoek-Brown Failure Criterion, initially proposed by Hoek in 1994 and further developed by Hoek and colleagues in 1995, is a methodology used to assess the strength of rock masses (Hoek, 1994) (Hoek et al., 1995).

The generalized Hoek-Brown failure criterion is expressed as a nonlinear correlation between the principal major and minor stresses, where tension is regarded as positive and compression as negative.

$$\sigma'_1 = \sigma'_3 - |\sigma_{ci}| \left(m_b \frac{-\sigma'_3}{\sigma_{ci}} + s \right)^a \quad (3.1)$$

Where m_b is a reduced parameter of the intact rock parameters m_i , which also depends on the Geological Strength Index (GSI) and the Disturbance Factor (D):

$$m_b = m_i \exp \left(\frac{GSI - 100}{28 - 14D} \right) \quad (3.2)$$

s and α are auxiliary material constants for the rock mass, that can be expressed as:

$$s = \exp \left(\frac{GSI - 100}{9 - 3D} \right) \quad (3.3)$$

$$\alpha = \frac{1}{2} + \frac{1}{6} \left[\exp \left(-\frac{GSI}{15} \right) - \exp \left(-\frac{20}{3} \right) \right] \quad (3.4)$$

σ_{ci} is the uni-axial compressive strength of intact rock material (defines as a positive value) From this value, the uni-axial compressive strength of the specific rock under consideration σ_c obtained by:

$$\sigma_c = -|\sigma_{ci}| s^a \quad (3.5)$$

The rock mass Young's modulus E_{rm} can be calculated by (Hoek&Diederichs, 2006)

$$E_{rm} = E_i \left(0.02 + \frac{1 - \frac{D}{2}}{1 + e^{(60+15D-GSI)/11}} \right) \quad (3.6)$$

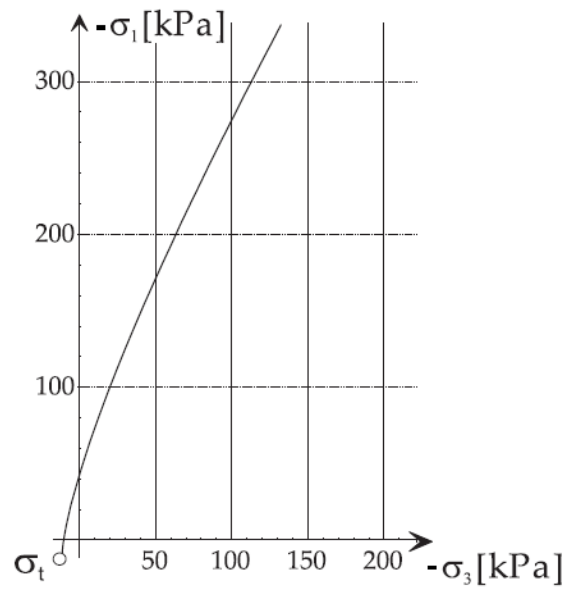


Figure 3.19. Hoek-Brown Failure Criterion in Principal Stresses

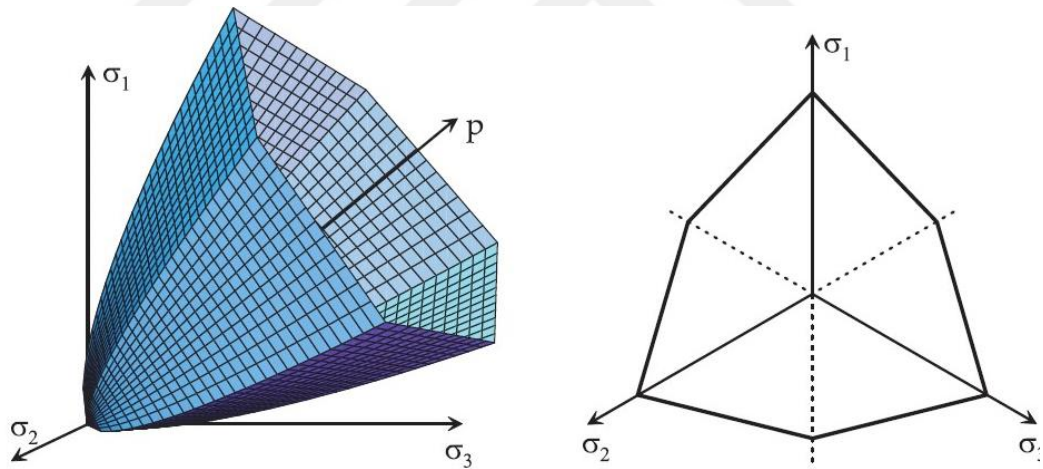


Figure 3.20. Hoek-Brown Failure Criterion in Principal Stresses

This criterion stands as a highly regarded empirical criterion for assessing the strength of fractured rock masses. Essential in geotechnical engineering, this model plays a crucial role in evaluating the stability of rock slopes, underground tunnels, foundations, and rock excavations. It effectively captures the non-linear and stress-responsive characteristics of both solid rock and rock formations.

Key features of the Hoek-Brown model include:

- **Formulation:** The model is expressed through a formula that relates the stress at failure to the major and minor principal stresses, incorporating material constants that reflect the properties of the rock.
- **Parameters:** It involves parameters such as the uniaxial compressive strength of the intact rock and the Hoek-Brown material constants, which describe the rock mass quality.
- **Adaptability:** The model is adaptable to various types of rock masses and can account for different degrees of jointing, weathering, and other geological conditions.
- **Applications:** It's applied in the design and analysis of underground excavations, slope stability, and other structures in rock engineering.
- **Evolution:** Since its introduction, the model has undergone several refinements to enhance its accuracy and applicability to a broader range of rock types and conditions.

During excavations in rock masses, like those for tunnels, slopes, or foundations, rock removal leads to stress relief, permitting the surrounding rock mass to decompress and dilate. The goal of effective design is to regulate this dilation and the resultant displacements to minimize the risk of rock failure. Achieving this objective involves careful selection of the excavation's shape and method, as well as the implementation of appropriate reinforcement and support systems when necessary (Hoek&Brown, 2019).

Mechanical or manual excavation in rock masses of inferior quality results in minimal disturbance to the adjacent rock mass, as Hoek and Brown (2019) noted. For such low-quality rock, Hoek and Brown recommend assigning a disturbance factor ranging from 0 to 0.5 (Hoek & Brown, 2019).

In their 2023 study, Demirdogen and Yildirim investigated the correct determination of the disturbance factor in dam foundations and the importance of assessing mechanical excavation and blasting techniques in the jointed rock

masses found in dams. They suggested that when excavating with mechanical equipment in rock masses having a Geological Strength Index (GSI) value below 40, which indicates poor rock quality, the disturbance factor should be set at 0.2.

The foundational characteristics of Derince Dam was determined using the Hoek-Brown Failure Criterion. This data, covering all three layers, is detailed in Table 3.6. The Engineering Geology Report for the Muğla Milas Derince Dam (2003) states that the most intact mica-schist layer, situated above a firmer bedrock, was up to 15 meters thick in the boreholes. To ensure safety, it is presumed that this 15-meter layer of weathered mica schist is uniformly present across all the dam's specified cross-sections.

Table 3.6. Rock Mass Parameters of Derince Dam

Material	Hoek-Brown Classification					Hoek Brown Criterion			
	<i>Rock Mass Class (Bienawski 1989)</i>	σ_{ci} (Kpa)	m_i	GSI	m_b	s	α	D	E_{rm} (kpa)
Weathered Mica-schist	Very poor rock	3000	10	22	0.453	0.0001	0.558	0.2	311030
Mica-schist	Poor rock	30000	10	39	0.889	0.0007	0.512	0.2	2617000

3.4.4. Material Model and Material Properties

Selecting a suitable material model is pivotal for accurately capturing the stress-strain characteristics of materials in finite element analysis. Rockfill, as indicated by triaxial tests, displays stress-dependent, non-linear, and inelastic behavior, a finding supported by Saboya et al. (1993). The hyperbolic model, frequently cited in literature, is commonly used to represent this behavior. Initially developed by Duncan and Chang in 1970 and later refined by Kulhawy et al. in 1972, the

hyperbolic model assumes incrementally elastic behavior but does not account for the plastic behavior of rockfill, a crucial factor in this study. As a result, the finite element software PLAXIS, which adopts the hardening soil model based on the hyperbolic model principles, was selected for this analysis. For Derince Dam, since no triaxial tests were conducted on the fill material, the parameter values for the material were taken from the literature. Since Derince Dam utilizes sand-gravel fill, preliminary analyses were performed using material parameters from a dam of similar characteristics and height. Table 3.7. lists a range of dams that exhibit features comparable to those of Derince Dam, including height, type of fill, and foundation.

Table 3.7. Some Characteristics of Some Worldwide CFRDs

Name	Rockfill Source	Foundation	Height (m)	Length (m)	Slab Face Area ($10^3 m^2$)	Main Rockfill (Comments)	E_{rc} Mpa	E_{rf} Mpa
Golillas	gravels, unprocessed	Rock	125	107	17	Clean gravels were used as a chimney filter and positioned in the central section of the fill.	107	250
Salvajina	natural gravels	Rock and soil, gravels in river section	148	362	86	The alluvial gullies were in a dense condition.	130	500
Mackintosh	graywacke, some slate	Rock	75	465	27.5	Zone 3B not used.	39	63
Kangaroo Creek	schist	Rock and some gravels	60	178	-	Zone 3A constitutes the primary rockfill area Zone 3B serves as a foundational layer of exceptionally strong rockfill designed for drainage.	-	140

In Table 3.8. the hardening soil parameters are given for the preliminary analysis of Derince Dam.

Table 3.8. Hardening Soil Model Parameters for Preliminary Analysis

Material	γ (kN/m ³)	E_{50}^{ref} (Kpa)	m	R_f	C (Kpa)	ϕ (°)	ψ (°)
Sand-Gravel Fill	21.00	30000-150000	0-1	0.6-0.7	1	35-40	5-10
Chimney Drain	21	100000-30000	0-1	0.6-0.7	1	30-40	5-10

It is known that the cohesion of sand-gravel fill material is 0. However, to avoid convergence problems in the analyses, this value was assumed to be 1 kPa.

3.4.5. Determination of Parameters for Sand-Gravel Fill and Chimney Drain

The examined hardening soil model parameters for sand-gravel fill and chimney drain are given in Table 3.8. In the analyses conducted to determine material parameters, the section at Km: 0+106, which is considered to be the most unfavorable and where the maximum settlement occurs, has been selected. After deciding on the body material parameters, analyses for other sections have been conducted, and the decided parameters have also been taken into consideration for the operational phase.

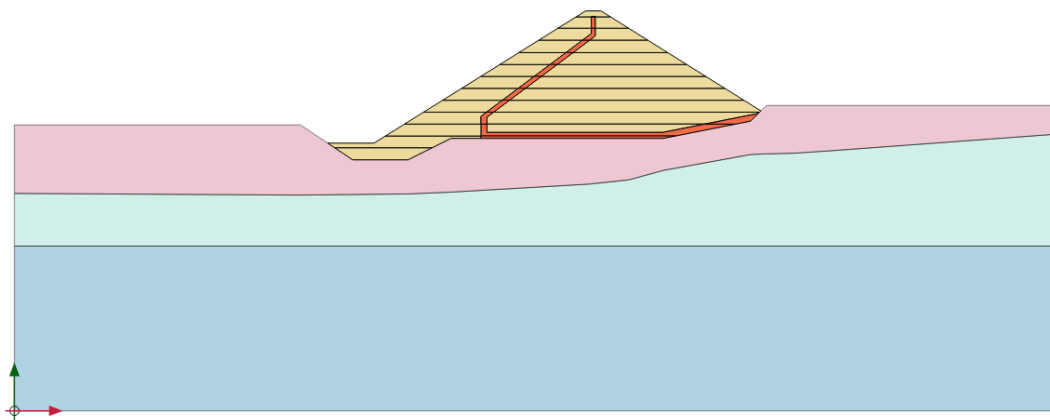


Figure 3.21. Finite Element Model of Derince Dam

In 2021, Sukkarak and colleagues undertook a study using Plaxis software to establish hardening soil parameters for rockfill material. They introduced a technique to determine the necessary parameters for predicting rockfill material behavior with the Hardening Soil (HS) model in PLAXIS, specifically targeting six types of Nam Ngum 2 rockfill materials. The study used data from triaxial and oedometer tests, and selected rockfill materials were tested to validate the HS model simulation. Key findings include:

R_f value is typically between 0.6 and 0.7 for alluvial materials.

In the case of alluvial rockfill materials, the reference oedometric modulus of elasticity (E_{oed}^{ref}) is approximately 0.9 to 1.2 times the reference secant modulus (E_{50}^{ref}).

For future analysis of materials used in rockfill dams, especially in preliminary designs of small to moderate-sized dams, they suggest the guidelines in Table 3.9.

Regarding Derince Dam, many different assumptions were made to identify the characteristics of sand-gravel fill materials, as listed in Table 3.10. The resulting settlement values calculated under these assumptions are presented in Table 3.11.

An examination of Table 3.11. indicates an inverse correlation between the modulus of elasticity values and settlement. It also shows that the chimney drain has a negligible impact on structural behavior.

Table 3.9. Parameters of the Hardening Soil Model for Rockfill Materials as Proposed by Sukkarak (2021)

Source	Materials	c (Kpa)	ϕ (°)	ψ (°)	E_{50}^{ref} (Mpa)	E_{oed}^{ref} (Mpa)	E_{ur}^{ref} (Mpa)	m	ν_{ur}	K_0^{NC}	Rf	Pref (kpa)	OCR
NN2 – RF1	Sandstone	0	39.0	2	62	54.5	186	0.25	0.3	0.371	0.82	100	1
NN2 – RF2	Sandstone	0	39.0	1.2	60	47.2	180	0.35	0.3	0.371	0.75	100	1
NN2 – RF3	Sandstone	0	39.0	1	78.5	66.4	235.5	0.15	0.3	0.371	0.78	100	1
NN2 – RF4	Sandstone 85% Siltstone 15%	0	35.6	1	33.5	22.1	100.5	0.42	0.3	0.418	0.8	100	1
NN2 – RF5	Sandstone 70%, Siltstone 30%	0	35.3	-3	19.3	15	57.9	0.45	0.3	0.422	0.6	100	1
NN2 – RF6	Sandstone 55%, Siltstone 45%	0	32.4	-1	12.5	8.5	37.5	0.5	0.3	0.464	0.65	100	1
Kol dam	Limestone	0	43.0	7	61	53	183	0.47	0.3	0.318	0.78	100	1
Purulia dam	Hornblende-quartz-schist	0	39.0	11	42	37.5	126	0.55	0.3	0.371	0.75	100	1
Parbati dam	Quartzite	0	42.6	1	100	95	300	0.5	0.3	0.323	0.6	100	1
Cihaxia dam	Unknown	0	42.2	2	100	85	300	0.3	0.3	0.328	0.8	100	1
Xiaolangdi dam	Unknown	0	42.0	2	100	87	300	0.3	0.3	0.331	0.8	100	1
KNIAC	Limestone	0	37.5	3.5	30	20	90	0.22	0.3	0.391	0.7	100	1
Noa Dehing dam	Riverbed	0	37.0	4	33	35	99	0.48	0.3	0.398	0.6	100	1
Shah Nehar	Micaceous sandstone, quartzite	0	36.0	4	25	25	75	0.6	0.3	0.412	0.65	100	1
Ranjit Sagar	Quartzite, jasper, shale, conglomerate, claystone, sandstone, grits of chart	0	48.5	-5	64	52	192	0.47	0.3	0.251	0.62	100	1

Table 3.10. Hardening Soil Model Parameters used in Preliminary Analyses

Material	Sand-Gravel Fill							Chimney Drain						
	E_{50}^{ref} (Kpa)	E_{oed}^{ref} (Kpa)	E_{ur}^{ref} (Kpa)	m	Rf	ϕ (°)	ψ (°)	E_{50}^{ref} (Kpa)	E_{oed}^{ref} (Kpa)	E_{ur}^{ref} (Kpa)	m	Rf	ϕ (°)	ψ (°)
Assumption-1	100000	90000	300000	0.35	0.7	39	-2	100000	90000	300000	0.5	0.7	35	-2
Assumption-2	100000	90000	300000	0.35	0.7	35	-2	150000	135000	450000	0.5	0.7	35	-2
Assumption-3	100000	90000	300000	0.35	0.7	35	-2	66000	59500	198000	0.5	0.7	35	-2
Assumption-4	70000	63000	210000	0.35	0.7	35	-2	70000	63000	210000	0.5	0.7	35	-2
Assumption-5	60000	42000	180000	0.35	0.7	35	-2	60000	42000	180000	0.5	0.7	35	-2
Assumption-6	40000	40000	120000	0.35	0.7	35	-2	50000	45000	150000	0.5	0.7	35	-2

Table 3.11. Hydraulic Settlement Cells Data vs Calculated Settlements

Measured Settlements		Calculated Settlements (mm)						
Hydraulic Settlement Cell	Elevation (m)	EOC (mm)	Assumption-1 (mm)	Assumption-2 (mm)	Assumption-3 (mm)	Assumption-4 (mm)	Assumption-5 (mm)	Assumption-6 (mm)
SC-2	75	376	179	172	180	190	213	203
SC-3	75	503	260	278	267	287	308	311
SC-4	75	516	289	278	289	295	333	342

SC-6	75	420	196	190	201	209	226	210
SC-7	90	278	183	177	184	208	254	264
SC-8	90	377	221	212	214	250	308	302
SC-9	90	414	213	207	211	243	294	300
SC-10	90	306	154	153	159	178	224	217
SC-12	105	247	118	117	120	153	189	197

3.4.6. Analysis for End of Construction Stage

In the research, the following methodology is adopted:

- Initially, the parameters of the material are identified to optimally match the settlement distribution under the End of Construction (EOC) stage,
- Effects of alluvium strata are examined,
- An analysis of the Reservoir Full Condition (RFC) is conducted,
- Lastly, stress and displacement contours are illustrated to provide an overview of the general behavior under both EOC and RFC conditions.

The hardening soil model parameters applied to the modeling of sand-gravel fill, chimney drain and alluvium is give in Table 3.12.

Table 3.12. Hardening Soil Model Parameters used in Plaxis Analysis

Material	γ (kN/m^3)	E_{50}^{ref} (kpa)	m	R_f	C (Kpa)	ϕ ($^\circ$)	ψ ($^\circ$)
Sand-Gravel Fill and Chimney Drain	22.00	37500	0.35	0.6	1	39	9
Alluvium	21.00	25000	0.5	0.6	10	36	6

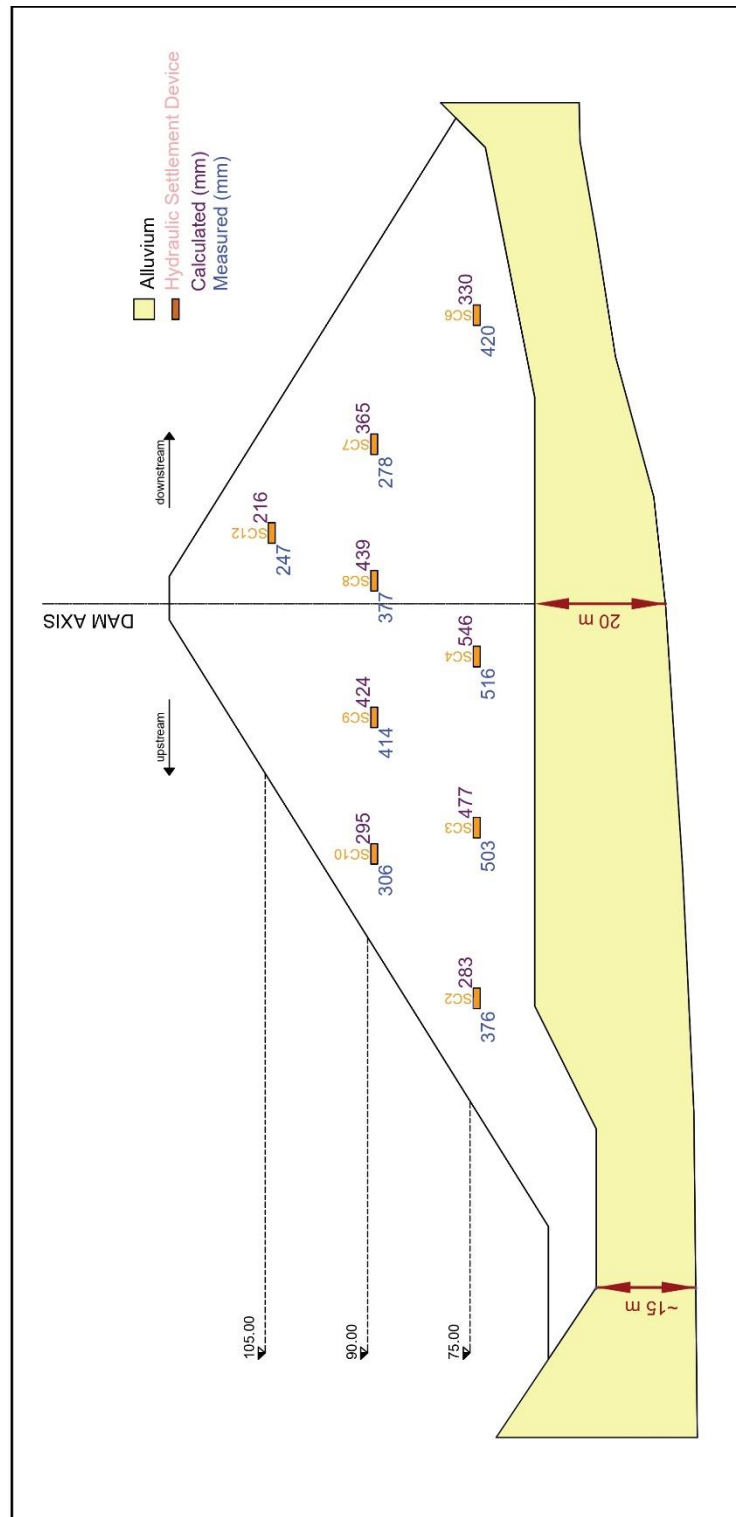


Figure 3.22. Calculated and Measured Settlements for End of Construction Stage at Km:0+106

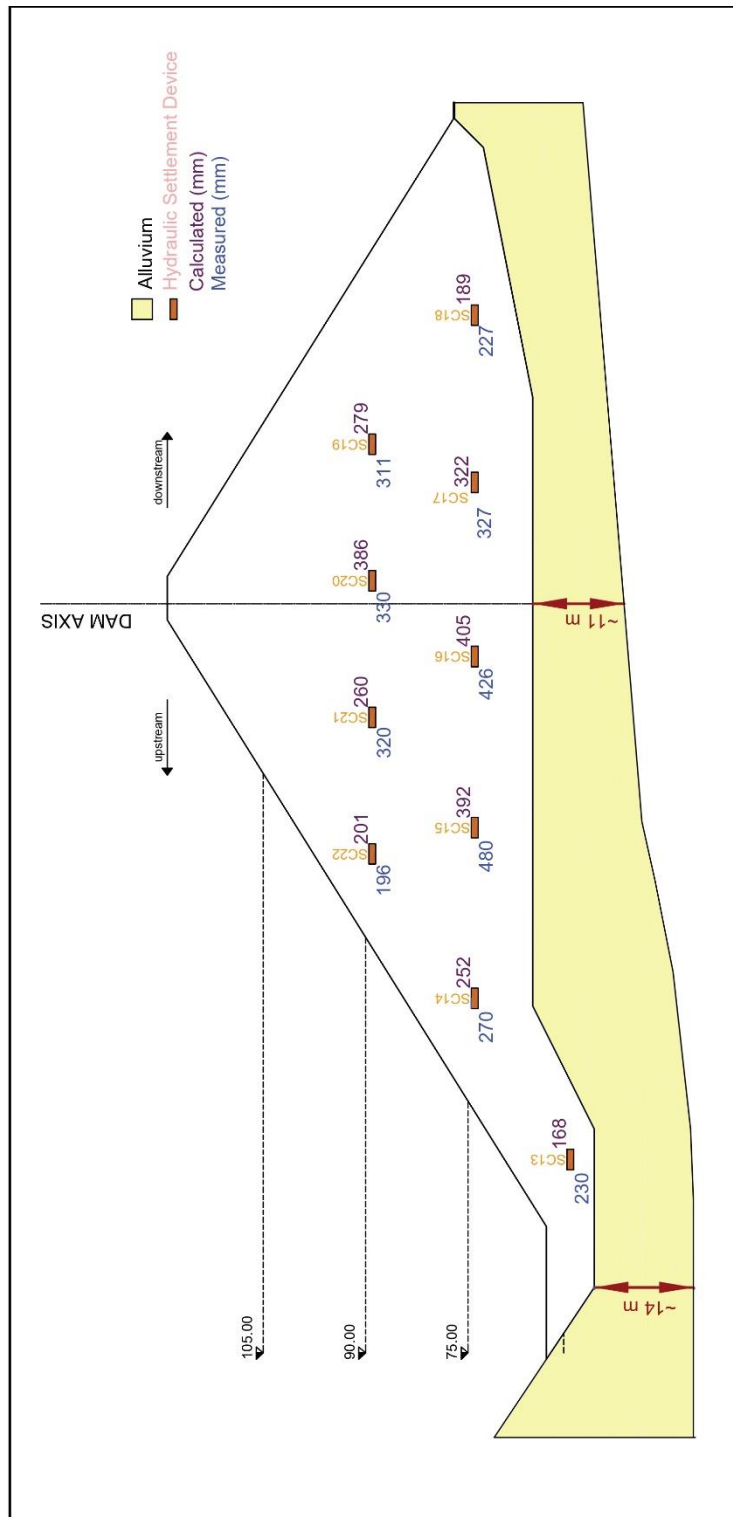


Figure 3.23. Calculated and Measured Settlements for End of Construction Stage at Km:0+173

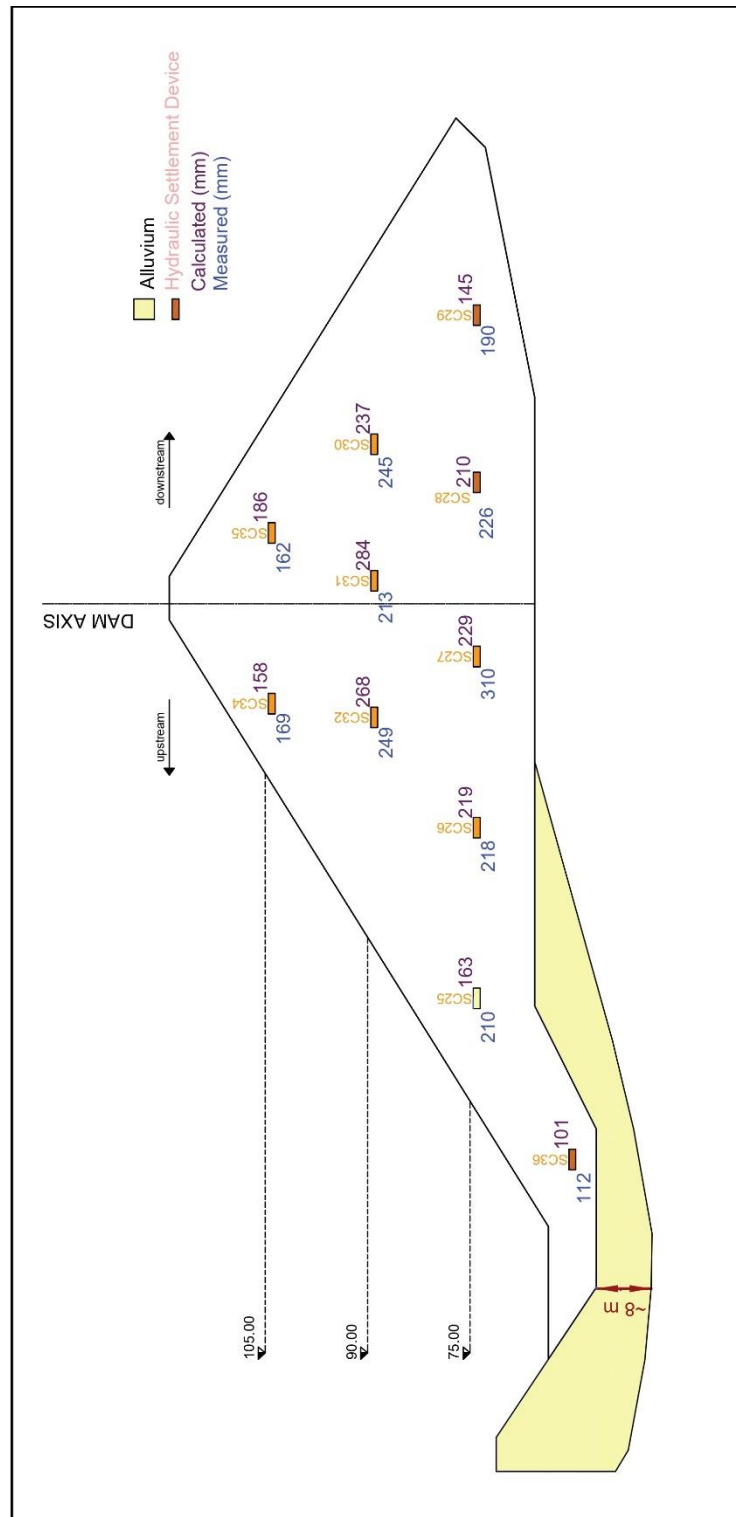


Figure 3.24. Calculated and Measured Settlements for End of Construction Stage at Km:0+263

Table 3.13. Measured and Calculated Settlements for End of Construction Stage

Hydraulic Settlement Cell	Location	Elevation (m)	Measured	Calculated	Difference	Difference (%)
SC-2	0+106	75	376	283	-93	-24.73404255
SC-3	0+106	75	503	477	-35	-5.168986083
SC-4	0+106	75	516	546	-84	5.813953488
SC-6	0+106	75	420	330	60	-21.42857143
SC-7	0+106	90	278	365	-110	31.29496403
SC-8	0+106	90	377	439	-98	16.44562334
SC-9	0+106	90	414	424	-36	2.415458937
SC-10	0+106	90	306	295	-21	-3.594771242
SC-12	0+106	105	247	216	6	-12.55060729
SC-13	0+173	61	230	168	32	-26.95652174
SC-14	0+173	75	270	252	-16	-6.666666667
SC-15	0+173	75	480	392	47	-18.33333333
SC-16	0+173	75	426	405	-5	-4.929577465
SC-17	0+173	75	327	322	-1	-1.529051988
SC-18	0+173	75	227	189	40	-16.74008811
SC-19	0+173	90	311	279	24	-10.28938907
SC-20	0+173	90	330	386	-43	16.96969697
SC-21	0+173	90	320	260	-63	-18.75
SC-22	0+173	90	196	201	-39	2.551020408
SC-25	0+263	75	210	163	23	-22.38095238
SC-26	0+263	75	218	219	-6	0.458715596
SC-27	0+263	75	310	229	95	-26.12903226
SC-28	0+263	75	226	210	28	-7.079646018
SC-29	0+263	75	190	145	46	-23.68421053
SC-30	0+263	90	245	237	28	-3.265306122
SC-31	0+263	90	213	284	-50	33.33333333
SC-32	0+263	90	249	268	-13	7.630522088
SC-34	0+263	105	169	158	-1	-6.50887574
SC-35	0+263	105	162	186	-18	14.81481481
SC-36	0+263	61	112	101	-18	-9.821428571

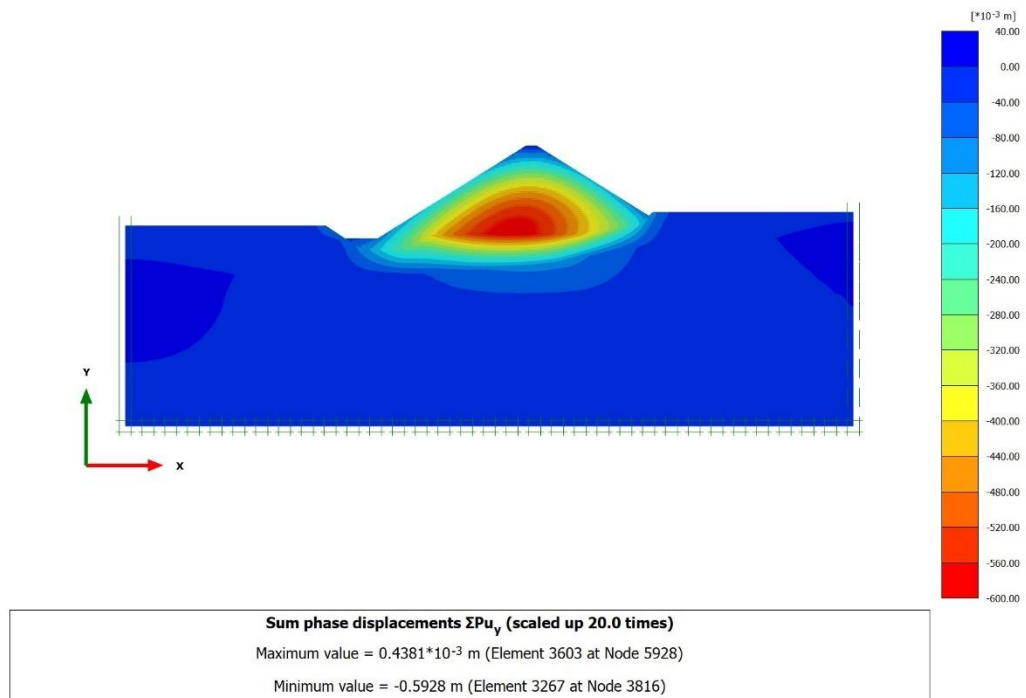


Figure 3.25. Vertical Displacements at Km:0+106 for EOC

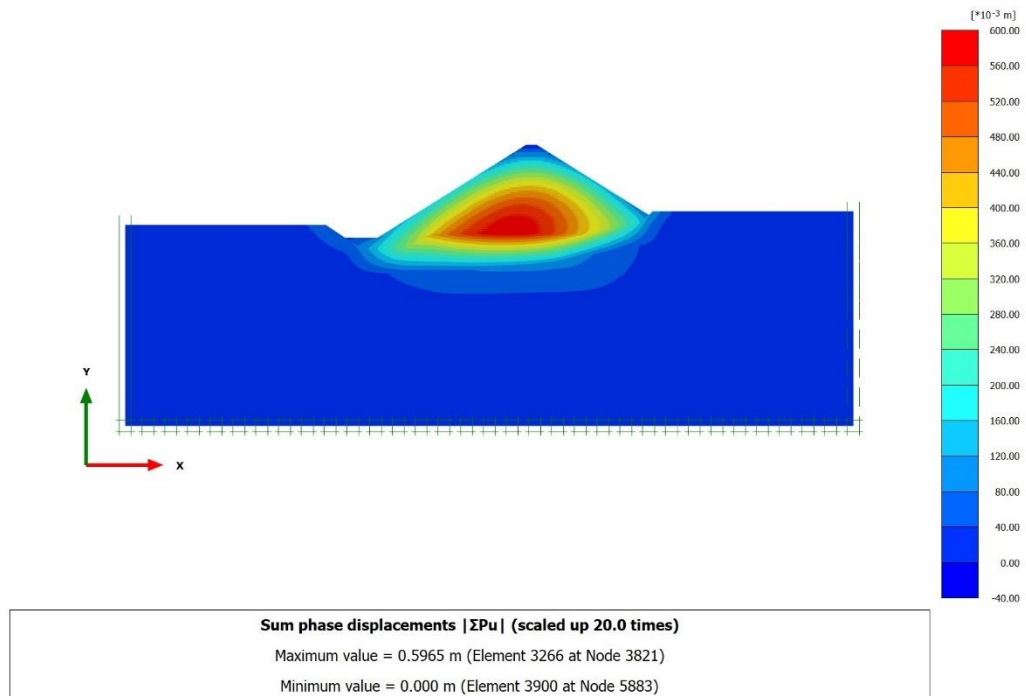


Figure 3.26. Total Displacements at Km:0+106 for EOC

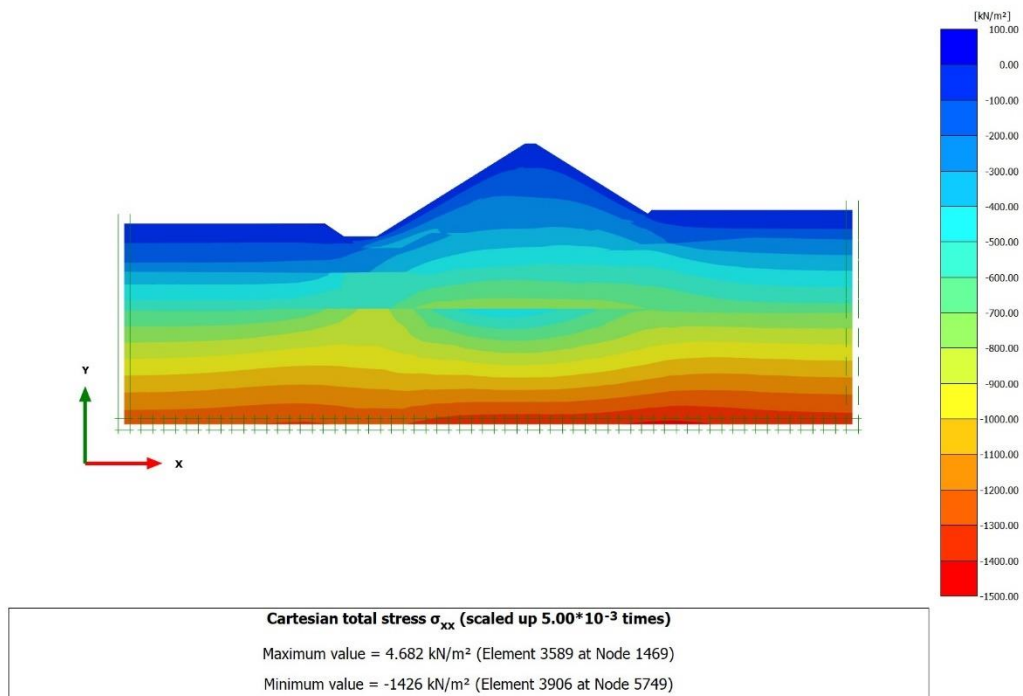


Figure 3.27. Horizontal Stresses at Km:0+106 for EOC

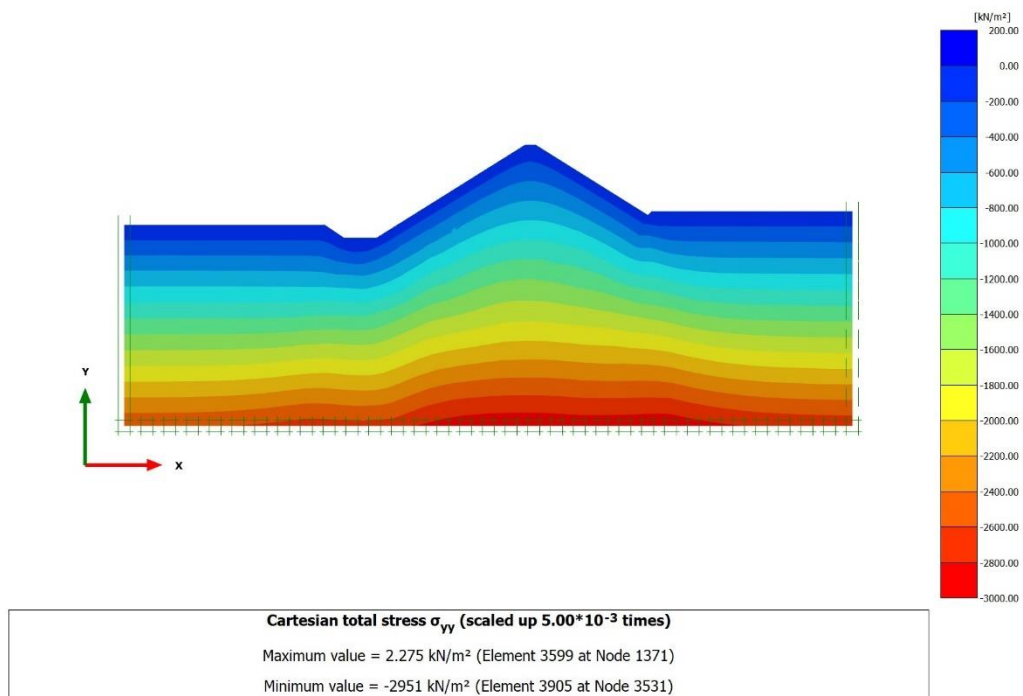


Figure 3.28. Vertical Stresses at Km:0+106 for EOC

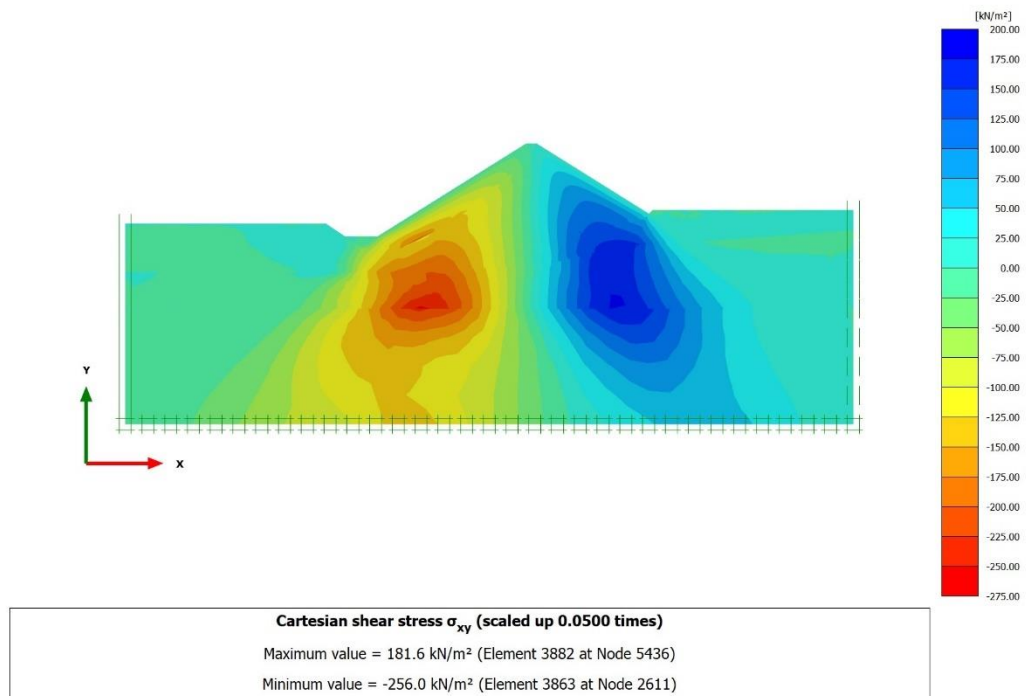


Figure 3.29. Shear Stresses at Km:0+106 for EOC

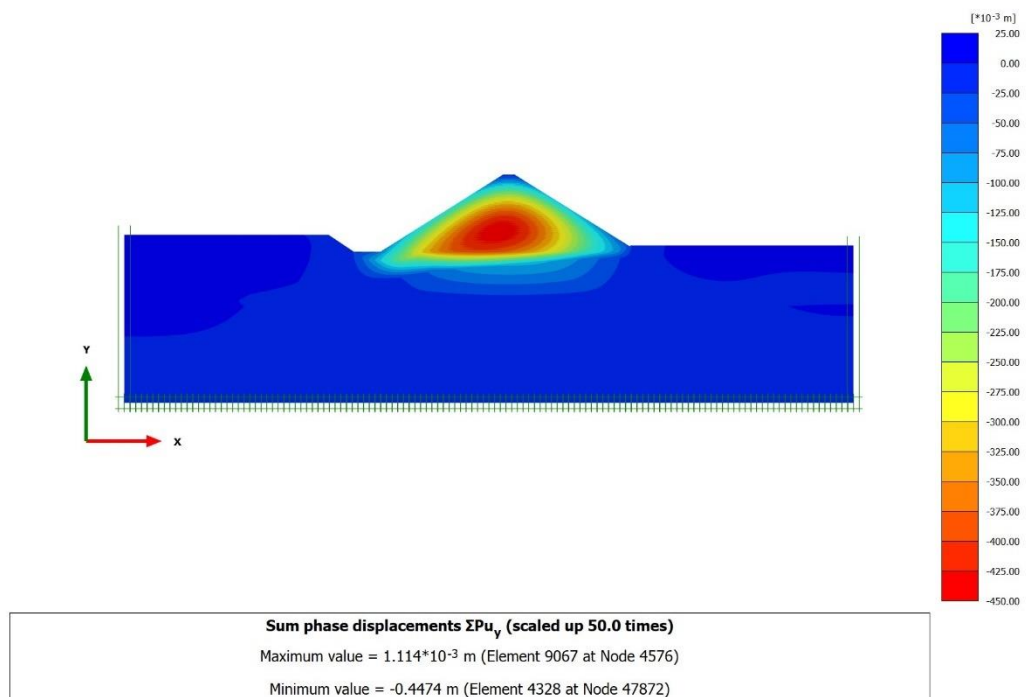


Figure 3.30. Vertical Displacements at Km:0+173 for EOC

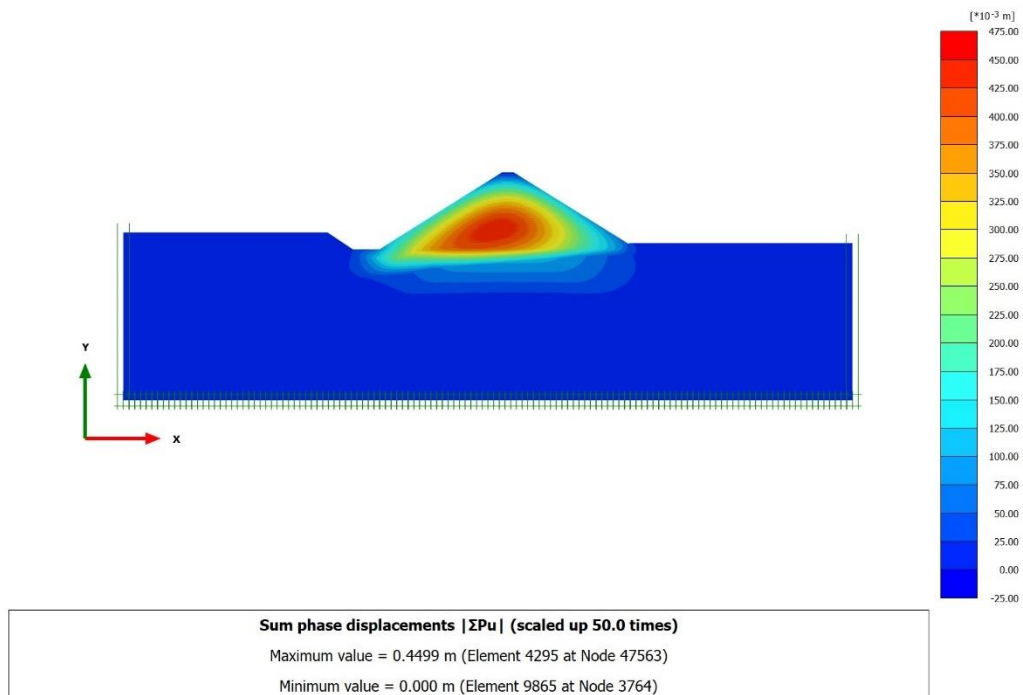


Figure 3.31. Total Displacements at Km:0+173 for EOC

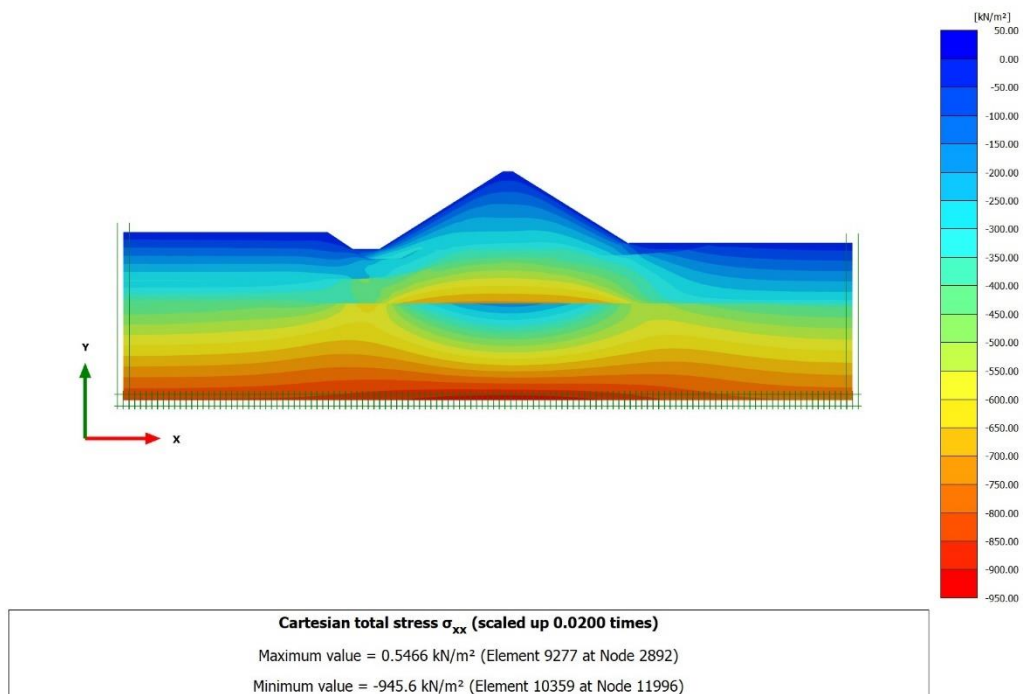


Figure 3.32. Horizontal Stresses at Km:0+173 for EOC

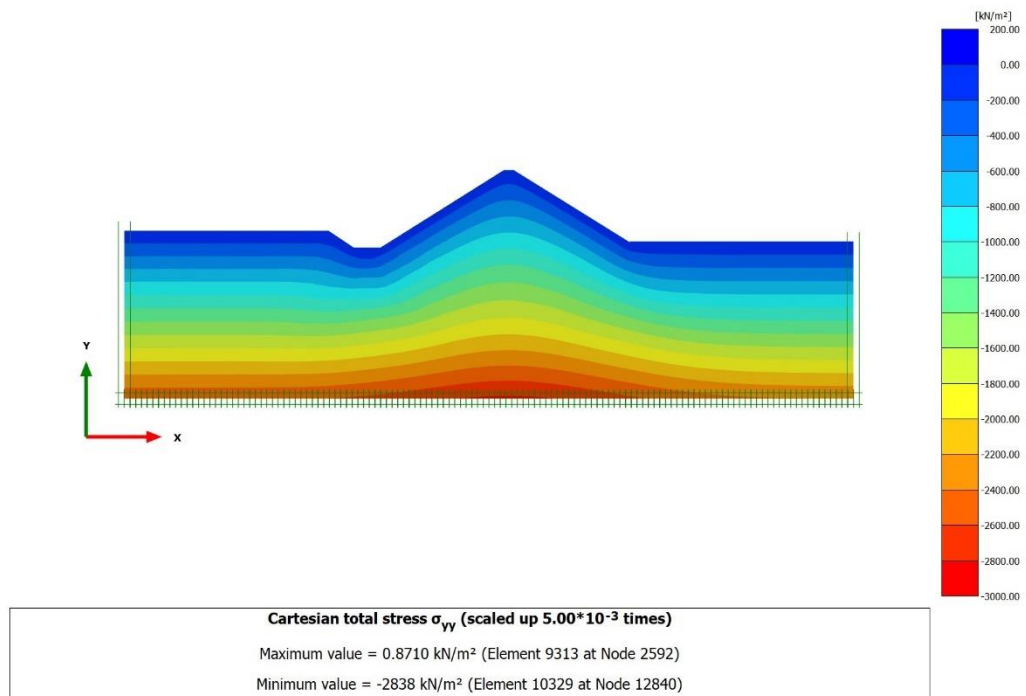


Figure 3.33. Vertical Stresses at Km:0+173 for EOC

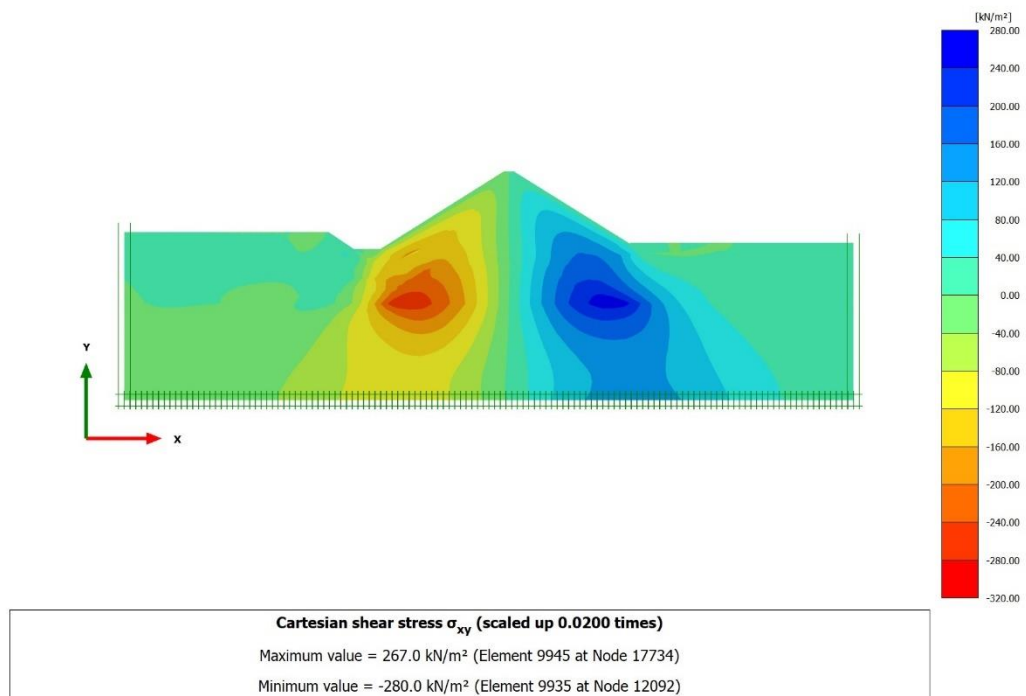


Figure 3.34. Shear Stresses at Km:0+173 for EOC

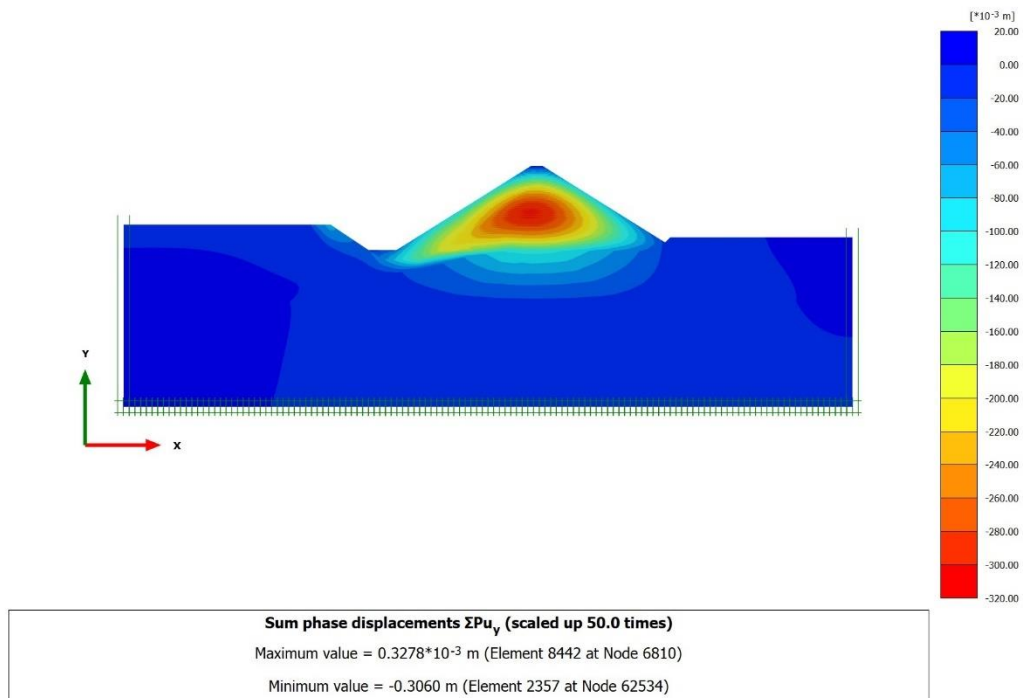


Figure 3.35. Vertical Displacements at Km:0+263 for EOC

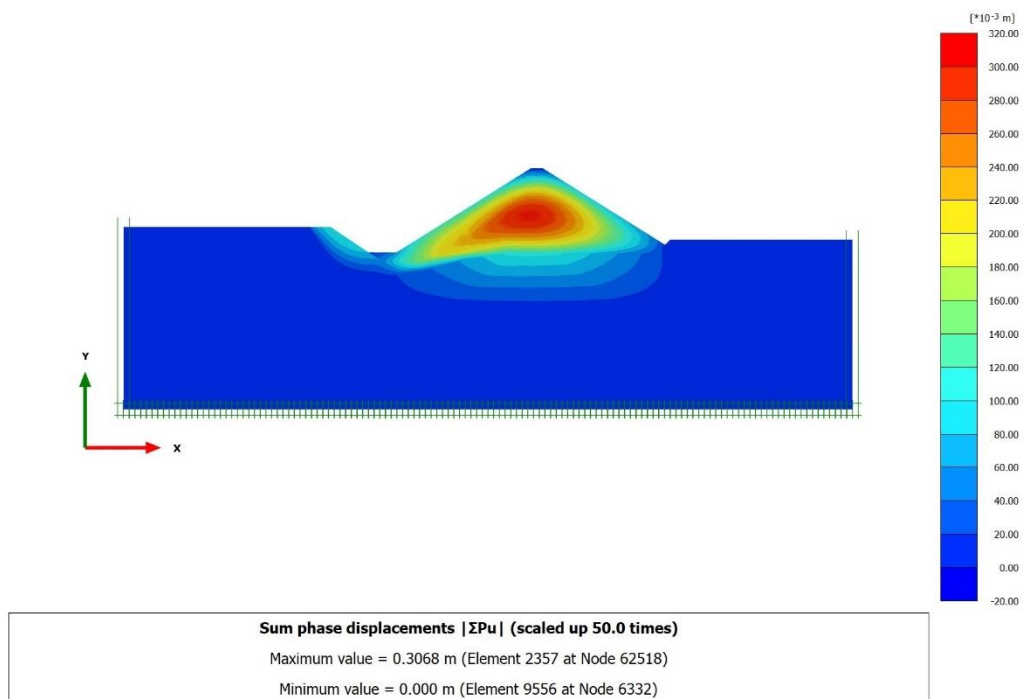


Figure 3.36. Total Displacements at Km:0+263 for EOC

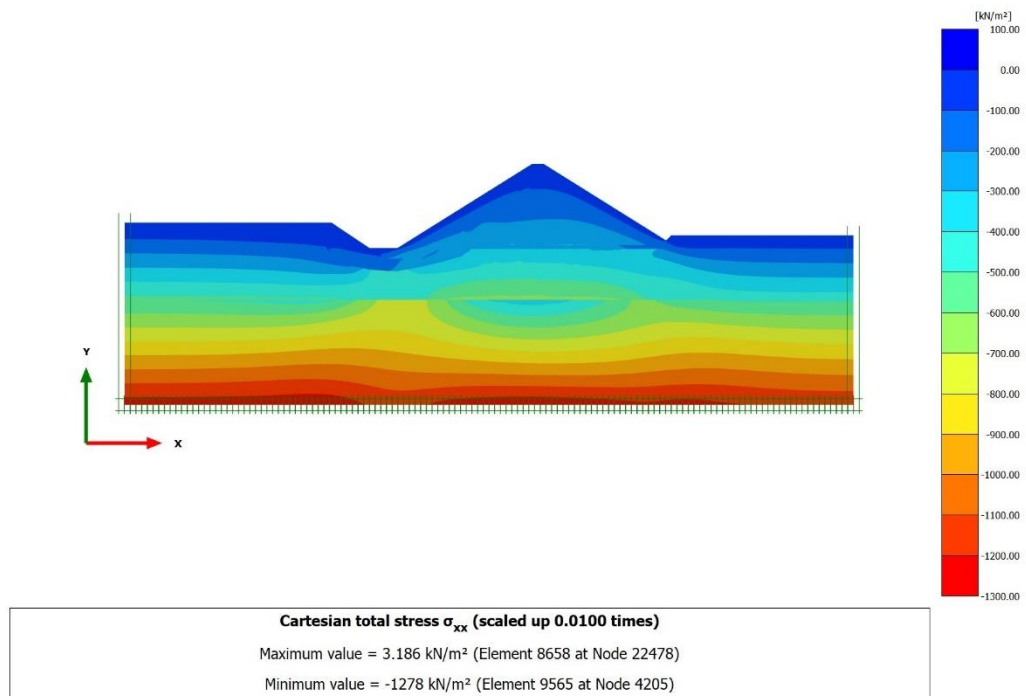


Figure 3.37. Horizontal Stresses at Km:0+263 for EOC

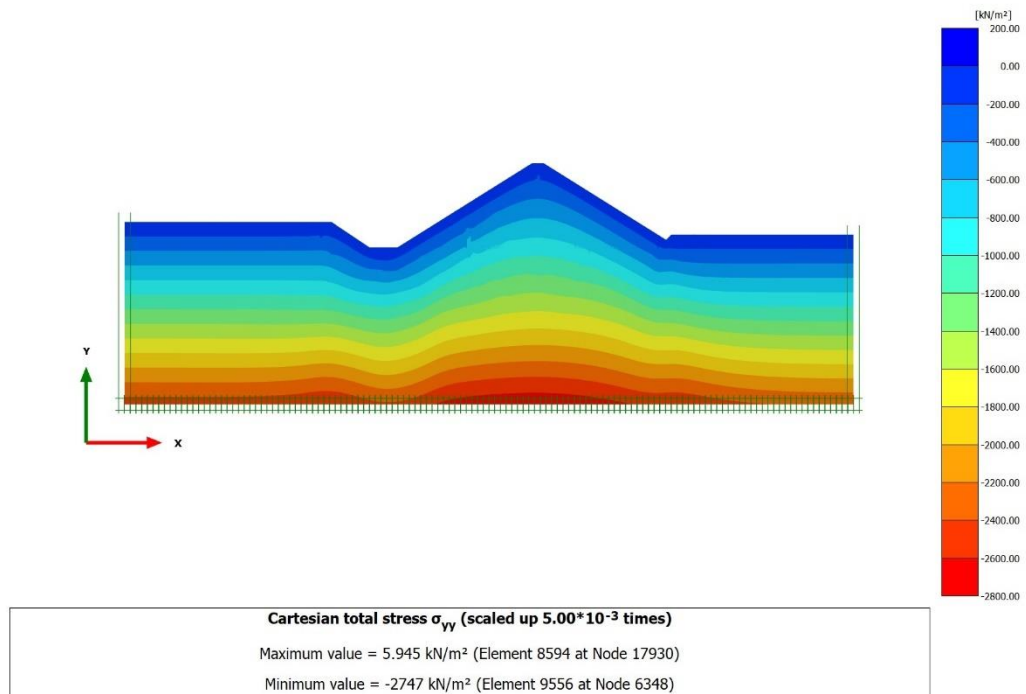


Figure 3.38. Vertical Stresses at Km:0+263 for EOC

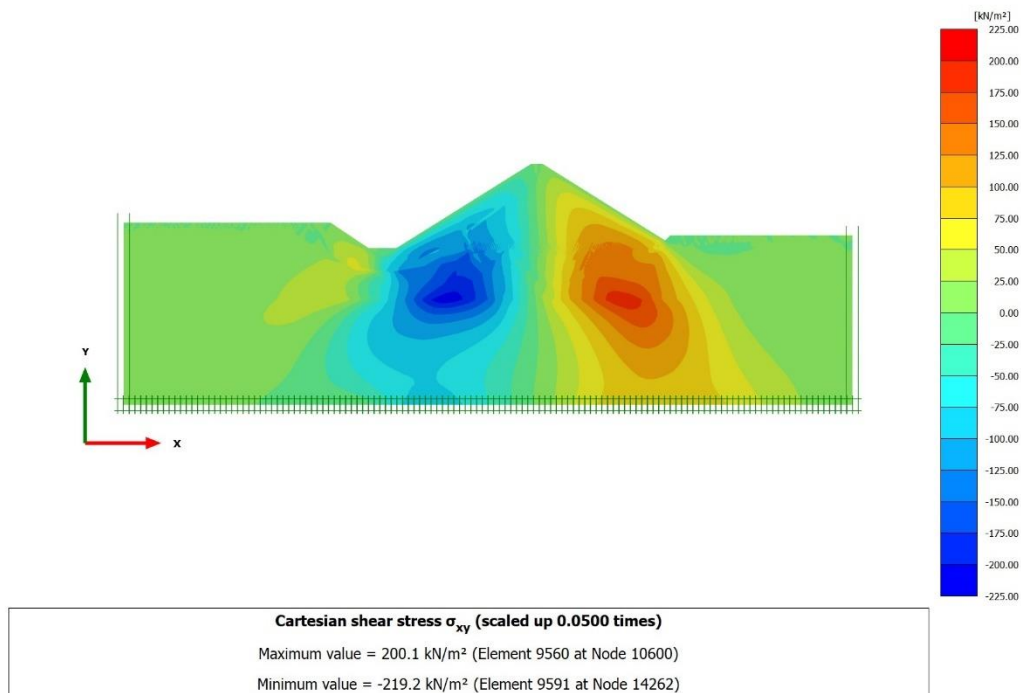


Figure 3.39. Shear Stresses at Km:0+263 for EOC

3.4.7. Reservoir Impoundment Analyses

Impounding, a significant stage in dam performance, significantly contributes to post-construction settlements. When a reservoir's water level increases, it leads to both horizontal and vertical shifts in the rockfill embankment, directly impacting the dam's functionality. Excessive displacements can cause cracks in the concrete membrane, leading to leakage issues that are costly to repair. The rate of impounding is a key factor; a rapid increase in water level results in greater settlements and horizontal movements.

For Derince Dam, the reservoir impoundment started on November 14, 2014, and the water level reached an elevation of 111.59 by June 01, 2015.

It is important to note that the term full condition does not refer to the normal water level specified in the project. Instead, it represents the maximum level that the reservoir has reached thus far, which is 111.59 m. According to information provided

by the General Directorate of State Hydraulic Works, the reservoir has not operated at a level higher than 111.59 m as of the current date.

The hydraulic settlement cells provided settlement values, as depicted in Appendix A, with the dates 16.12.13 and 01.06.15 corresponding to the end of construction (EOC), and the reservoir impoundment (RF), respectively.

The analysis for impoundment stage is only performed with cross-section: K:0+106. FEM and reservoir water load applied to upstream face is shown in Figure 3.40 reservoir load is 470 kN/m^2 .

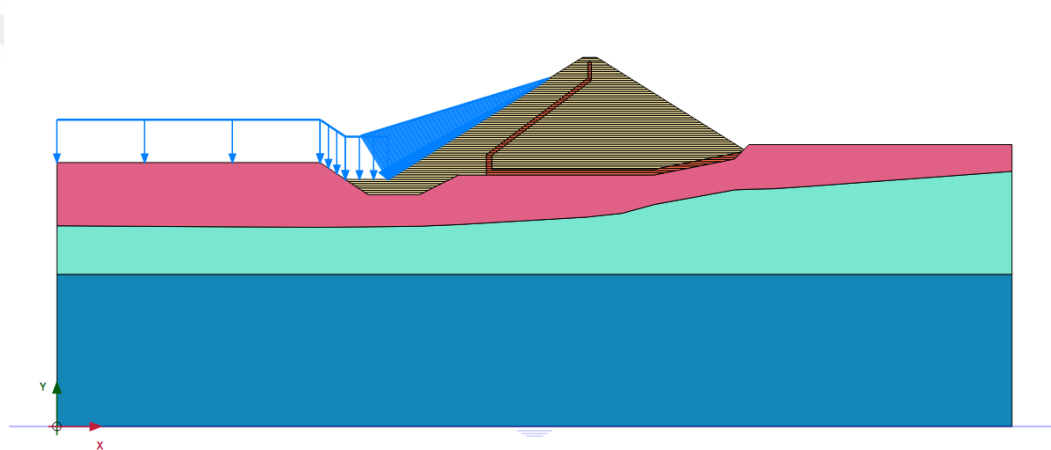


Figure 3.40. Reservoir Water Load Applied On Upstream Face

Table 3.14. Measured vs Calculated Settlement Values for Reservoir Impoundment

Hydraulic Settlement Cell	Location	El. (m)	1.06.15-16.10.14	Calculated
SC-2	0+106	75	175	120
SC-3	0+106	75	97	60
SC-4	0+106	75	78	30
SC-6	0+106	75	7	10
SC-7	0+106	90	32	2
SC-10	0+106	90	142	90
SC-12	0+106	105	42	20

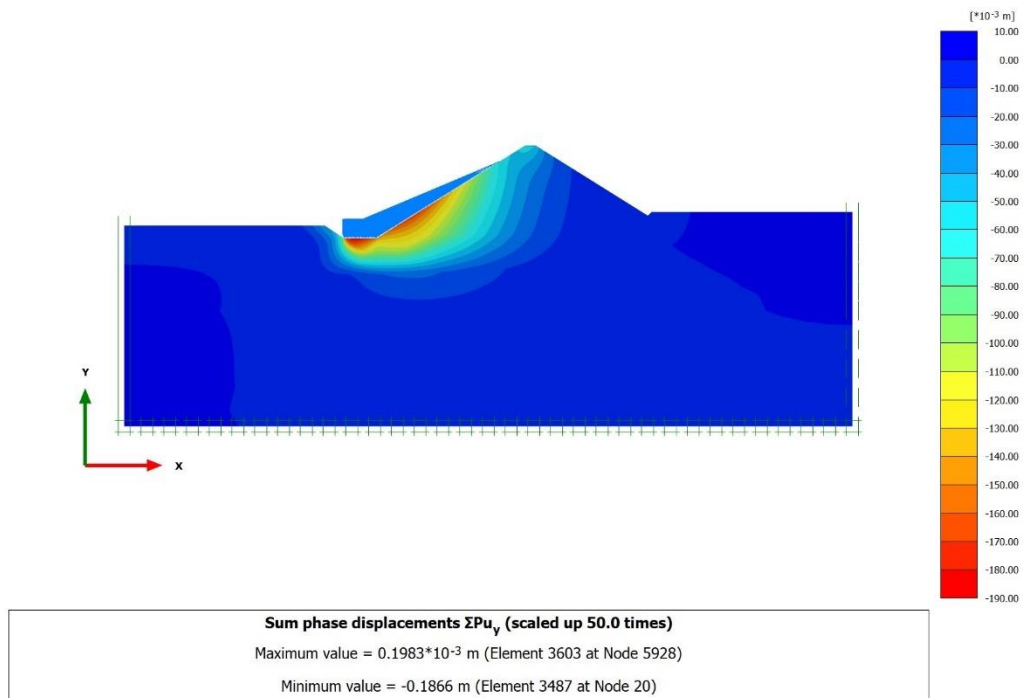


Figure 3.41. Vertical Displacements at Km:0+106 for Reservoir Impoundment

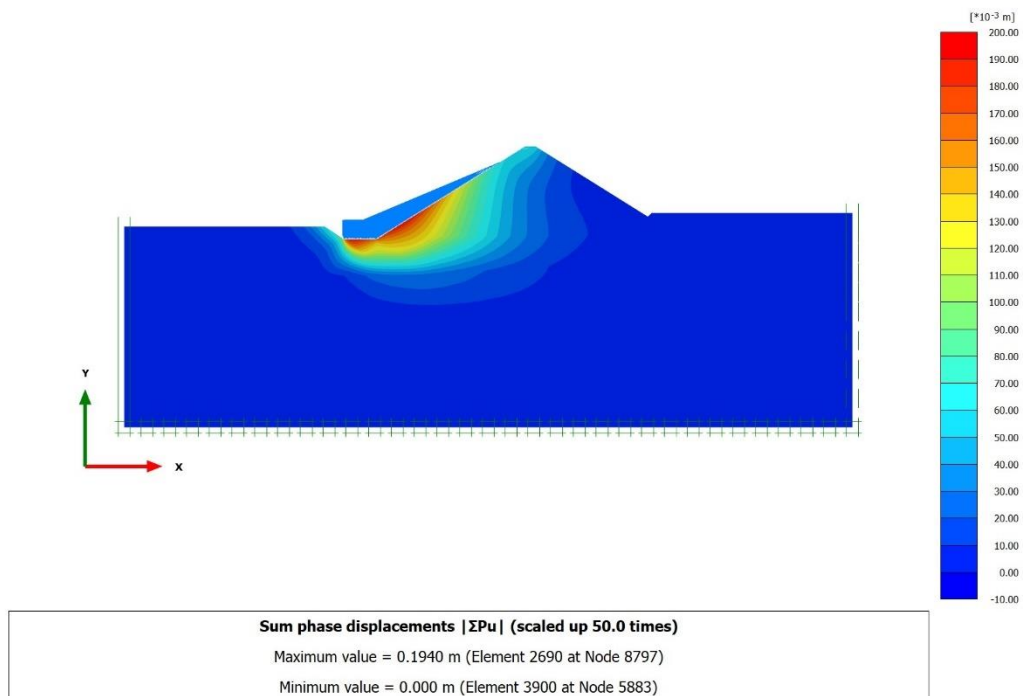


Figure 3.42. Total Displacements at Km:0+106 for Reservoir Impoundment

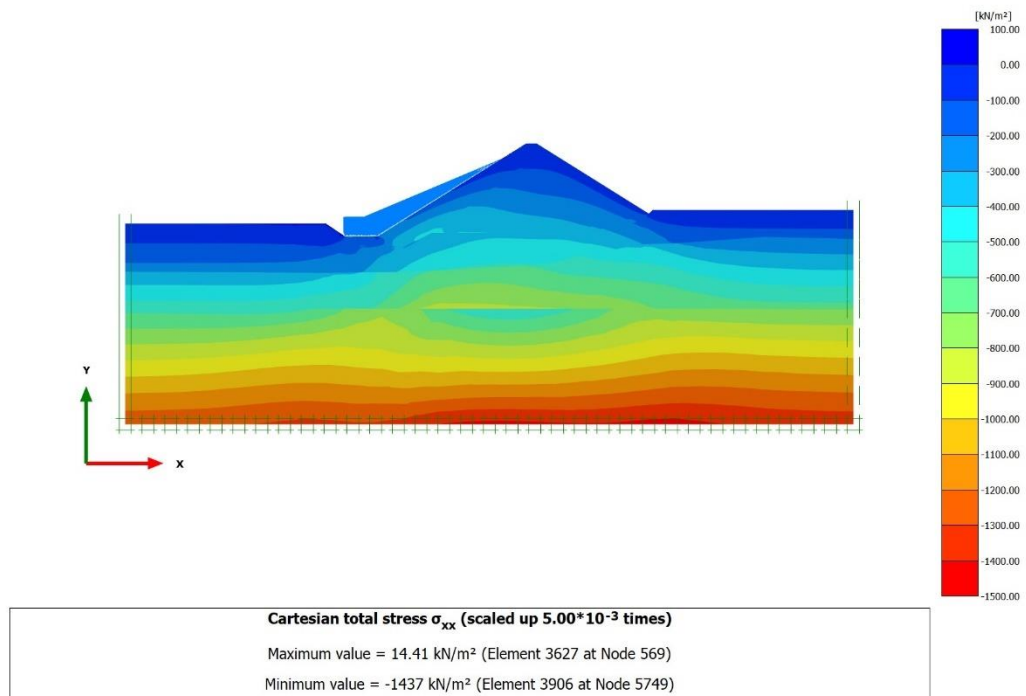


Figure 3.43. Horizontal Stresses at Km:0+106 for Reservoir Impoundment

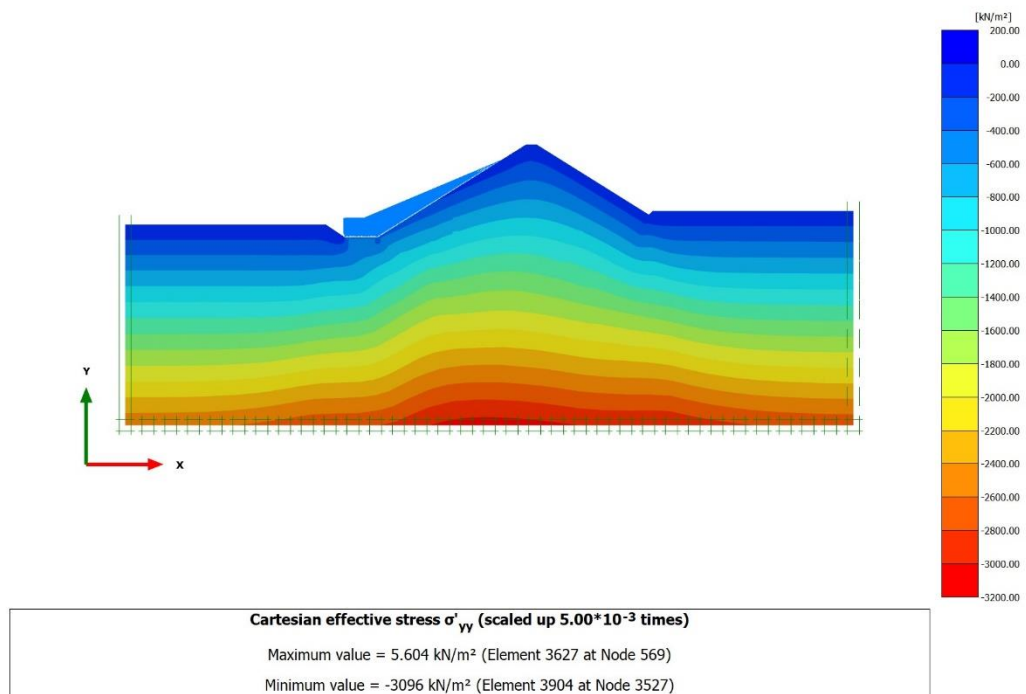


Figure 3.44. Vertical Stresses at Km:0+106 for Reservoir Impoundment

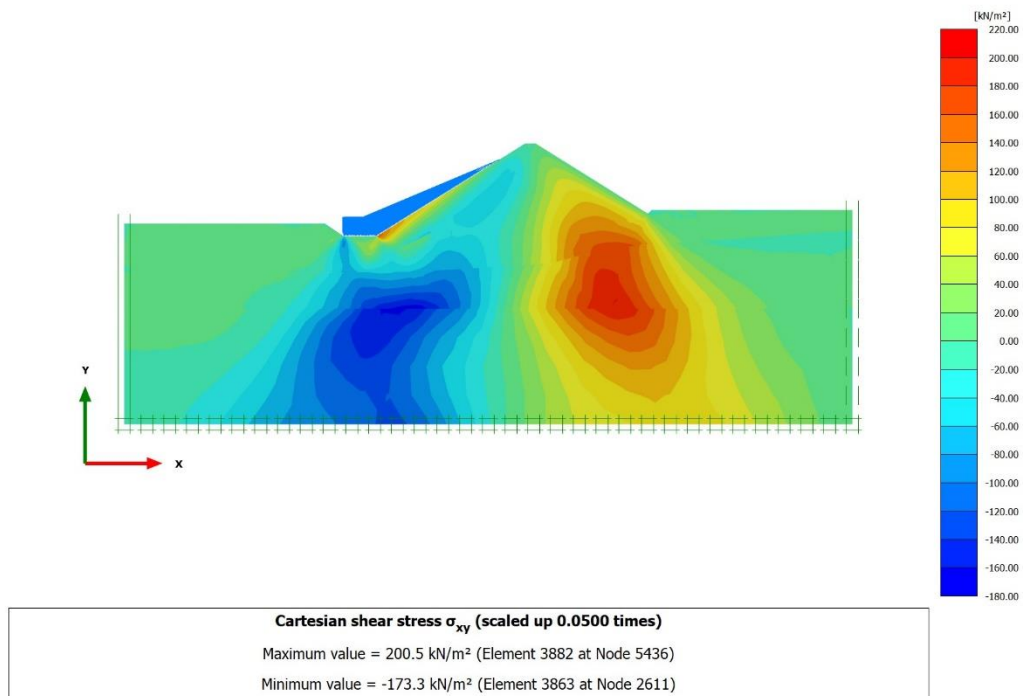


Figure 3.45. Shear Stresses at Km:0+106 for Reservoir Impoundment

CHAPTER 4

SUMMARY AND RESULTS

In this research, two-dimensional finite element analyses were conducted on Turkey's Derince Dam, a Concrete Face Sand-Gravel Dam (CFSGD) situated on 15-meter deep alluvial deposits. Key assumptions in the analysis include:

- The upstream membrane was presumed impermeable and free of cracks.
- The sand-gravel fill and alluvial deposits were simulated using the hardening soil model, derived from Duncan and Chang's renowned hyperbolic model.
- For the bedrock composed of mica schist and weathered mica schists, Hoek-Brown Failure Criterion were employed to find foundation parameters.
- Model parameters were sourced from literature on CFSGDs with similar deformation characteristics to Derince Dam, due to the absence of triaxial test results.

The study yielded several conclusions:

- When comparing calculated stresses with observations, it becomes apparent that the cross-valley arching effect is not significant for the end of construction condition.
- The highest settlement, measuring 516 mm, was noted at SC4 at El. 75 m in Km:0+106. Owing to the alluvium beneath the dam's foundation, the maximum settlement shifted to about 27% of the dam's total height ($H \cdot 0.27$).
- Using the SC27 instrument at Km:0+263, a settlement of 310 mm was recorded, indicating how alluvium influences settlement location and magnitude, and providing insights into the dam's structural behavior under specific conditions.

- The settlement tends to move back towards the dam's central point when the modulus of elasticity of both the alluvium and the embankment material become more similar.

In summary, the outcomes of the two-dimensional plane strain analysis appear promising. The placement of instruments appears suitable, as evidenced by the alignment of observations with calculations. However, monitoring the horizontal displacement, which is crucial in such structures, is recommended. The analyses should be renewed for the normal water level shown in the project.



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A. Settlement Cell Locations for Derince Dam

Name	Cross-section	Elevation (m)	Distance to Dam Axis(m)
SC1	Km: 0+106	61	81.35
SC2	Km: 0+106	75	57.79
SC3	Km: 0+106	75	32.79
SC4	Km: 0+106	75	7.73
SC5	Km: 0+106	75	17.27
SC6	Km: 0+106	75	42.27
SC7	Km: 0+106	90	23.36
SC8	Km: 0+106	90	3.36
SC9	Km: 0+106	90	16.64
SC10	Km: 0+106	90	36.64
SC11	Km: 0+106	105	14.64
SC12	Km: 0+106	105	10.36
SC13	Km:0+173	61	81.37
SC14	Km:0+173	75	57.75
SC15	Km:0+173	75	27.75
SC16	Km:0+173	75	2.25
SC17	Km:0+173	75	27.25
SC18	Km:0+173	75	52.25
SC19	Km:0+173	90	28.34
SC20	Km:0+173	90	8.34
SC21	Km:0+173	90	16.59
SC22	Km:0+173	90	36.59
SC23	Km:0+173	105	14.66
SC24	Km:0+173	105	10.34
SC25	Km:0+263	75	57.86
SC26	Km:0+263	75	32.86

SC27	Km:0+263	75	7.86
SC28	Km:0+263	75	17.14
SC29	Km:0+263	75	42.14
SC30	Km:0+263	90	23.31
SC31	Km:0+263	90	3.31
SC32	Km:0+263	90	16.61
SC33	Km:0+263	90	36.69
SC34	Km:0+263	105	14.69
SC35	Km:0+263	105	10.31
SC36	Km:0+263	61	81.4

B. Settlement Data at Cross-sections 0+106, 0+173 and 0+263 (settlement values are in mm)

			Settlement (mm)				
Cell	Location	Elevation (m)	16.12.2013	16.05.2014	16.10.2014	1.06.2015	1.06.2015-16.10.2014
SC-2	0+106	75	313	311	376	551	175
SC-3	0+106	75	365	476	503	600	97
SC-4	0+106	75	388	456	516	594	78
SC-6	0+106	75	411	436	420	427	7
SC-7	0+106	90	170	285	278	310	32
SC-8	0+106	90	219	363	377	No Data	-
SC-9	0+106	90	204	347	414	No Data	-
SC-10	0+106	90	168	243	306	448	142
SC-12	0+106	105	148	232	247	289	42
SC-13	0+173	61	243	276	230	377	147
SC-14	0+173	75	338	339	270	400	130
SC-15	0+173	75	502	430	480	536	56
SC-16	0+173	75	420	416	426	447	21
SC-17	0+173	75	350	298	327	338	11
SC-18	0+173	75	222	198	227	237	10
SC-19	0+173	90	211	239	311	322	11
SC-20	0+173	90	209	314	330	359	29
SC-21	0+173	90	189	241	320	369	49
SC-22	0+173	90	101	139	196	243	47
SC-25	0+263	75	191	265	210	305	95
SC-26	0+263	75	197	246	218	279	61
SC-27	0+263	75	134	272	310	320	10
SC-28	0+263	75	232	219	226	197	234
SC-29	0+263	75	176	160	190	199	9

SC-30	0+263	90	111	121	245	250	5
SC-31	0+263	90	158	264	213	223	10
SC-32	0+263	90	107	149	249	282	33
SC-34	0+263	105	154	149	169	No Data	-
SC-35	0+263	105	95	115	162	169	7
SC-36	0+263	61	113	125	112	192	80

