

**FINITE ELEMENT SOLUTION FOR
UNCONFINED SEEPAGE PROBLEM
WITH REFERENCE TO
AL-QADISIYA DAM**

A Thesis

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By

Mohammed Abdul.Ameer Subuh

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محمد

CERTIFICATION

We certify as an examining committee that we have read this thesis entitled "*Finite Element Solution for Unconfined Seepage Problem With Reference to Al-Qadisiya Dam*", and examined the student "*Mohammed Abdul Ameer Subuh*" in its contents and in what related to it, and found it meets the standard of a thesis for the Degree of Master of Science in Civil Engineering.

Signature:

Name: Asst. Prof. Dr. Karim K. Al-Jumaily

Date: / /2002

(Supervisor)

Signature:

e: Asst. Prof. Dr. Abdul Hassan K. Shukur

Date: / /2002

(Supervisor)

Signature:

Name: Prof. Dr. Abdul-Karim H. Ghailan

Date: / /2002

(Member)

Signature:

Name: Asst. Prof. Dr. Karim R. Abed

Date: / /2002

(Member)

Signature:

Name: Prof. Dr. Rafa Hashim S. Al-Suhaili

Date: / /2002

(Chairman)

Approval of The Civil Engineering Department
Head of The Civil Engineering Department

Signature:

Name: Asst. Prof. Dr. Nameer A. Alwash

Date: / /2002

Approval of the Deanery of the College of Engineering
Dean of The College of Engineering

Signature:

Name: Prof. Dr. Abdul Amir S. Resen

Dean, College of Engineering

Date: / /2002

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Finite Element Solution for Unconfined Seepage Problem with Reference to Al-Qadisiya Dam.

ABSTRACT

Seepage study through earth dams for estimating the pore pressure distribution and locating the free surface profile, which is used in analyzing the dams stability against shear failure, comprise a very important class of unconfined flow problems.

In this research, a mathematical model is applied for analyzing two-dimensional steady state seepage through stratified an isotropic earth dams. A numerical solution using finite elements Galerkin method is employed to predict the piezometric head distribution, seepage quantity, pore water pressure and locating the free surface profile .

This analysis is performed for a practical case study, namely, AL-Qadisiya dam. Applicability of this method has been evaluated based on comparison between the numerical solutions and measured field data.

Seepage studies of AL-Qadisiya dam embankment and its foundation were tested at different stations at the left and right bank of the dam.

A set of figures are obtained to show the distribution of pore water pressure and total head, as well as, location of the free surface for all stations.

The results of application of the model on AL-Qadisiya dam, investigating some parameters, indicated that the degree of anisotropy of the soil has a considerable effect on the seepage phenomenon and soil permeability, that is, reducing permeability of the material used in earth embankment means decreasing the seepage discharge, causing an

increase in pore water pressure inside the embankment, However, using a D/s drain shall decrease the pressure in the D/s shell.

Using the grout curtain and the Asphaltic concrete diaphragm means decreasing the seepage discharge and also, the pore water pressure and the head gradient behind the curtain.

Adequacy of the grout curtain was also studied by removing the curtain and the comparison between these two cases, the results indicated that the head loss across the curtain was greater than 40% at the selected stations, while there is no sharp drop in the head in case of removing the curtain, thus the results indicated that the grout under the dam is in a good condition.

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APPENDIX A

List of symbols

Symbols	Description	Dimension
A	Problem domain	
H	Piezometric head at any point in the domain	[L]
He	Approximate solution for piezometric head distribution in The element (e).	[L]
Hi	Nodal value of head, H, of element (e).	[L]
[Hi]	The vector matrix of nodal values.	
i	Hydraulic gradient.	
k	Hydraulic conductivity.	[L/T]
Lx,Ly	Direction cosines	
n	Number of nodes in element(e)	
Ne	Total number of elements in the problem (e)	
Ni	Shape function matrix of element (e)	
[ni]	The shape function of the element(e)	
p	Surface boundaries of the element .	
p ₁	Reservoir boundaries.	
p ₂	Impermeable boundaries.	
R ^e	Element residual.	
Re	Reynolds number.	
S	Distance along the flow line.	[L]
U	Velocity in X- direction.	[L/T]
V	Velocity in Y-direction.	[L/T]
V _s	Seepage velocity through porous media	[L/T]
w _j	Weighted function.	
X,Y	Coordinates axes in the real region.	
ψ	Stream function.	
ν	Coefficient of kinematic viscosity	[L ² /T]
γ _w	Unit weigh	[M/L ² .T]

Chapter One

Introduction

1.1 General

The underground flow of water has significant practical consequences and refers to a wide range of problems. It is usual to divide underground flow problems into two categories, seepage and ground water flow.

Seepage problems are generally classified as the percolation of water due to head difference under hydraulic structures or through earth dams, river banks or into excavations. The dimensions of the areas through which seepage takes place will be measured in tens or hundreds of square meters. These areas represent porous media which may be homogenous if the hydraulic conductivity is independent of position; otherwise it is called heterogeneous, and it may be isotropic if the hydraulic conductivity is independent of direction; otherwise it is called an anisotropic. [Ruadkivi and Callander, (1976)].

Seepage flow through permeable soil may be either steady or time-variant, in the latter case, its characteristics are effected by the lapse of time; whereas the steady state occurs when the condition at the boundary of the region must remain fixed with time. Also, the seepage flow is said to be confined if the boundary of flow region is fixed otherwise it is called unconfined where one boundary is a free surface. A general-class of unconfined flow problems is the problem of seepage through earth dams [Taylor and Brown, (1967)].

1.2 Seepage through Earth Dams

Seepage of water through the embankment or foundations has been responsible for more than one third of earth dams failures [Varshney and Gupta, (1972)]. Seepage is inevitable in all earth dams and ordinarily it does not harm. Uncontrolled

seepage, may, however, cause erosion within the embankment or in the foundation which may lead to piping. Piping is the progressive erosion which develops through the dam. Seepage failures are generally caused by pervious foundations, leakage through embankments, conduit leakage and sloughing..

So, the seepage study through body of earth dams is one of the important analysis during dam design to calculate the quantity of losses from reservoir, estimating the pore water pressure distribution, locating the position of the free surface which is used in analysis of the dam stability against the shear failure, finally, studying the hydraulic gradient which give a general idea about the potential piping. [Sherard et al, (1963)].

Many methods have been developed for analyzing the seepage problems such as graphical method (The flownet), analytical methods and experimental methods of which is the electrical analogy method, viscous flow method and sand tank method. These methods of solution are well applicable when the boundary of the region of seepage is clearly defined and the soil medium is homogenous and isotropic with respect to permeability.

However, it is cumbersome for problems involving unconfined flow in nonhomogeneous soil. For solving such types of problems, the use of computer aided methods seems to be more appropriate. Three different numerical procedures have been used in computer methods for unconfined seepage analysis; the finite difference method, the finite element method and the boundary element method. [Chang, (1988); Bear and Verruijt ,(1990)].

In the present work the finite element numerical model will be used to investigate the unconfined seepage problem. (phreatic seepage problem).

1.3 Objectives of the Work

The main objectives of this research are the following: -

1. Applying the mathematical model to a real case study of “ AL-Qadisiya Dam” in order to compare the theoretical results with that available data field.
2. To investigate the effect of material properties and the degree of anisotropic on the gradient of the phreatic line and Quantity of seepage.
3. Checking adequacy of the grout curtain in satisfying the design criterion.

1.4 Scope of the Research

To meet the above objectives, this thesis is presented in six chapters:

1. Basic principles of flow through porous media review of literature are given in chapter two. Also, the methods of solution for seepage problems through earth dams and their basic assumptions were reviewed.
2. Chapter three describes the finite element formulation, which is used to solve the problem.
3. General description concerning the case study “AL-Qadisiya Dam” is given in chapter four.
4. The applications of the numerical model for case study has been presented, and its results have been analyzed and discussed in chapter five.
5. The results and conclusions, as well as some suggestions, for future studies are outlined in chapter six.

Chapter Two

Basic Principles and Review of Literature

2.1 Introduction

In order to understand the methods, which are used to solve the unconfined seepage problems and specially the seepage through earth dams the basic principles of seepage theory and most of the previous studies on the seepage through earth dams will be explained in this chapter.

2.2 Theoretical Background

The flow of ground water and seepage is concerned with the flow of fluids through porous media. It depends on many factors such as the dimensional character of the flow, its time dependency, the boundaries of the flow region or domain, and the properties of the medium and the fluid.

Materials occurring in nature are seldom isotropic and even less frequently homogenous, while many adequate general methods for solving seepage problems through isotropic media have been developed, problems of anisotropy and non-homogeneity have received scant attention due to their mathematical difficulties [Zieinkiewicz, (1966)]. In this study a general numerical model has been developed to solve seepage problem through anon-homogenous anisotropic soil.

The “Darcy’s Law of seepage” establishes a linear relationship between the seepage velocity and hydraulic head gradient. This Law, which is a simple consequence of viscous flow neglecting inertia effects, can be obviously generalized to a two dimensional situation.

2.3 The Equation of Flow through Porous Medium

The flow of water through saturated porous medium is generally governed by Darcy's Law [Harr, (1962)];

$$V_s = ki \quad \dots(2-1)$$

Where:

V_s : Seepage velocity through porous media. (m/sec)

k : Hydraulic conductivity. (m/sec)

i : Hydraulic gradient = $-\frac{\partial H}{\partial s}$

H : Piezometric head.(m)

S : Distance along the flow line.(m)

Darcy's Law is valid to flow through porous media of two and three dimensions, that's why this is considered of great importance in studying great number of practical problems.

Equation (2-1) is valid when the flow is laminar, i.e Reynolds number is taken equal to or less than one [Harr, (1962)], or.

$$Re = \frac{V_s D}{\nu} \leq 1 \quad \dots(2-2)$$

Where:

Re: Reynold's number.

V_s : Flow velocity (m/sec).

D : average diameter of soil particles (m)

ν : Coefficient of kinematics viscosity of water (m²/sec).

2.4 The General Equation of Flow

The components of seepage through porous media according to the general Darcy's Law form are [Freeze and cherry, (1979)].

$$U = kxIx = -kx \frac{\partial H}{\partial x} \quad \dots(2-3a)$$

$$\mathbf{V} = k_y \mathbf{I}_y = -k_y \frac{\partial \mathbf{H}}{\partial y} \quad \dots(2-3b)$$

Where

U: velocity component in x-direction. (m/sec)

V: velocity components in y-direction. (m/sec)

k_x, k_y : Hydraulic conductivity in x,y directions respectively (m/sec)

H: Piezometric head ($\mathbf{H} = \frac{\mathbf{P}}{\gamma_w} + \mathbf{z}$). (m)

P: pressure (kN/m²)

γ_w : Unit weight of water. (kN/m³) and

z: elevation head (m)

The continuity equation for two-dimensional and incompressible, irrotational flow is:

$$\frac{\partial \mathbf{U}}{\partial x} + \frac{\partial \mathbf{V}}{\partial y} = 0 \quad \dots(2-4)$$

Substituting Darcy's (2-3) in equation (2-4) results in:

$$\frac{\partial}{\partial x} (k_x \frac{\partial \mathbf{H}}{\partial x}) + \frac{\partial}{\partial y} (k_y \frac{\partial \mathbf{H}}{\partial y}) = 0 \quad \dots(2-5)$$

For homogenous and isotropic soil, the hydraulic conductivity is equal in all directions: $k_x = k_y$

Then equation (2-5) is going to be as:

$$\frac{\partial^2 H}{\partial x^2} + \frac{\partial^2 H}{\partial y^2} = 0 \quad \dots(2-6a)$$

Equation (2-6a) is called Laplace Equation and it is similar to Laplace Equation of velocity potential for ideal fluid flow or non-viscous, irrotational flow.

Laplace equation represents the condition of steady-state laminar flow and different methods to find the piezometric head of the flow domain can be used.

For homogenous and anisotropic soil the hydraulic conductivity is not equal in all directions, i.e.: $k_x \neq k_y$

Then Laplace equation becomes:

$$k_x \frac{\partial^2 H}{\partial x^2} + k_y \frac{\partial^2 H}{\partial y^2} = 0 \quad \dots(2-6b)$$

Where:

k_x : Hydraulic conductivity in the horizontal direction.

k_y : Hydraulic conductivity in the vertical direction.

There is another benefit function for the flow through porous media, which is called stream function.

$$\mathbf{U} = \frac{\partial \psi}{\partial y}, \mathbf{V} = -\frac{\partial \psi}{\partial x} \quad \dots(2-7)$$

U : velocity component in x-direction.

V : velocity component in y-direction.

ψ : Stream function.

From the similarity between the flow through porous media and the ideal flow, where the flow is irrotational and the vorticity equal to zero, the expression for this relation is:

$$\frac{\partial V}{\partial x} - \frac{\partial U}{\partial y} = 0 \quad \dots(2-8)$$

Substituting equation (2-7) into equation (2-8) results in :

$$\frac{\partial^2 \psi}{\partial x^2} + \frac{\partial^2 \psi}{\partial y^2} = 0 \quad \dots(2-9)$$

Which represents Laplace equation for stream function.

2.5 Boundary Conditions

In the general case of plane, steady-state flow of ground water through porous media, four types of boundaries are encountered [Harr, (1968)]. The characteristics of each of these boundaries will be considered in detail.

(1) **Impervious Boundary**

At impervious boundaries the fluid can neither penetrate the boundary nor leave gaps, thus the velocity component normal to the boundary at any point must vanish. i.e.

$$\frac{\partial H}{\partial n} = 0, \text{ Where } n \text{ is in the direction normal to the boundary.}$$

The boundary 1-5 in Figure (2-1) represents impervious boundary in the earth dams.

(2) **Boundaries of the Reservoirs**

Along the boundaries of the reservoir the pressure distribution may be taken as hydrostatic; that is, the pressure must be function of depth only; i.e., constant head. This type of boundary represents equipotential lines. The boundaries 1-2 and 4-5 in Figure (2-1).

$$H = h_1 \text{ at the boundary 1-2} \quad \dots(2-10)$$

$$H = h_2 \text{ at the boundary 4-5} \quad \dots(2-11)$$

(3) **Line of Seepage (Free Surface, Phreatic line)**

The line of seepage is the upper streamline in the flow domain. It separates the saturated region of flow from that part of the soil body

through which no flow occurs, such as 2-3 and 3-4 of Figure (2.1). The determination of its locus is one of the major objectives of unconfined seepage problems. The pressure at any point along its surface is constant and equal to atmospheric pressure. Thus, along this line: -

$$H = y \quad \dots(2-12)$$

2.6 Review of Literature

The previous studies can be classified into four directions as follows:

2.6.1 Graphical Method (the Flow Net)

The solution of Laplace equation (2-6a) in two dimensions may be presented by two families of curves intersecting one another orthogonally and forming a pattern of curvilinear squares. The two families of lines are known as equipotential lines, or lines joining points of equal total head, and streamlines. A streamline is the locus of the path of flow of an

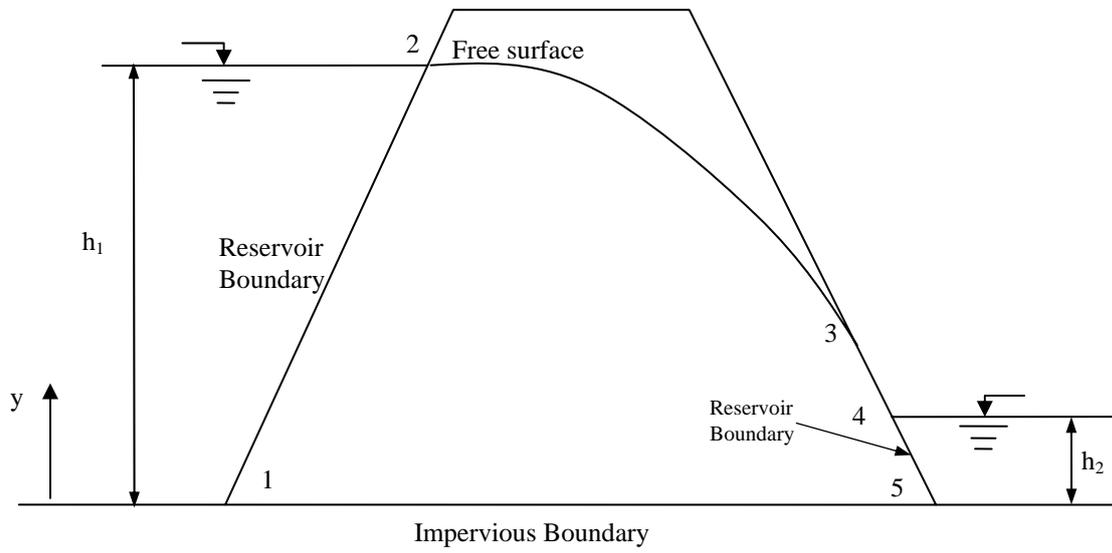


Figure (2.1) Boundary types in the earth dams

individual particle of water. The mesh formed by the intersection of these two sets of curves is known as a flow net.

A unique solution of Laplace's equation may therefore be obtained graphically by trial-and-error sketching of a flow net, which satisfies the boundary conditions, preserve right angle intersections and consists of curvilinear squares. The method may be extended to include simple anisotropic conditions by transforming the section before the flow net is constructed so that the dimensions of the cross section are shrunk in the direction of greater permeability [Lambe and Whitman, (1969) and Das, (1987)]. If a section consists of two materials having different permeabilities, the net may be drawn so that the length to width ratio of the squares is equal to the ratio of the permeabilities. Confined and unconfined flow solutions can be obtained by this method and solutions to transient advance problems are also possible. Cedergren, (1967) comprehensively describes the method and many examples on flow nets were given.

The undesirable feature of the flow net method lies in the difficulty of sketching the net. An accurate net for even a simple two-dimensional flow situation can require many hours of skilful and experienced laborer. However, the solution of two-dimensional problems is relatively insensitive to the quality of the flow net and even a poor attempt may yield suitably accurate results for the pore pressures, seepage quantities and gradients. Nevertheless, a solution by the flow net method is scarcely possible for the case of a heterogeneous section or one in which the directions of the principle axes of anisotropy do not coincide throughout the flow domain.

Non-linear problems in which the permeability is a function of pressure head are also excluded whether these are concerned with non-Darcian or unsaturated flow.

2.6.2 Analytical Methods

Exact solutions to the Laplace equation may be obtained by various analytical methods. Harr (1968) presents a number of such theoretical solutions. In general, the section must be transformed to eliminate the effect of different permeabilities and anisotropy before it can be analyzed. All the methods are, therefore, again limited to flow systems where the porous medium is relatively uniform and the boundary conditions are not too complicated.

The simplest theoretical solution are suggested by kozeny (1931) who considered the problem of seepage throughout an earth dam with a parabolic upstream face, resting on an impervious base and having a horizontal toe drain located on the down stream portion of the dam. The flow net for the section consists of a system of confocal parabolas.

Casagrane (1940) extended Kozeny's solution to include dams with trapezoidal toe drains and slope drains and suggested an adjustment to account for a straight upstream face. [Al-Qaisi, (1999)].

Unconfined flow in dams was also studied by Numereov (reported by Harr 1968) who obtained solutions for dams with various slopes and toe drains designs. He reduces all the required information to obtain a solution in a graphical form but, unfortunately, its application is still not straightforward.

Approximate analytical solutions are possible for some unconfined flow problems by involving the Dupuit theory (Das, 1987). This involve making two assumptions: -

- 1- For small inclinations of the line of seepage the streamlines can be taken as horizontal.
- 2- The hydraulic gradient is equal to the slope of the free surface and is invariant of depth.

Although the nature of these assumptions appear paradoxical, in many groundwater problems solutions based on the Dupuit assumptions compare favourably with those of more rigorous methods. Since the validity of these assumptions depends on the steepness of the free surface the method is unsuitable for many earth dam situations.

2.6.3 Experimental Methods

The following methods are rarely used in practice for flow net determination but they are of use for instructional purposes: -

2.6.3.1 Models

A flow problems can be solved by constructing a scale model and analyzing flow in the model. The model will usually be two-dimensional, formed between glass plates and dye can be injected to trace streamlines piezometer tubes can also be used. Soil models are of limited general use because of the time and effort required constructing such a model.

In addition capillarity or unsaturated flow can be of considerable importance in a soil flow model even if this is not so in the prototype.

2.6.3.2 Analogy Methods

Electrical and heat flows also occur according to the Laplace formulation and so analogues can be attempted by either approach. Electrical models are generally more useful and can be adapted to a wide range of boundary conditions. In the electrical model, voltage corresponds to total head, electrical conductivity to hydraulic conductivity and current to velocity, measuring voltage enables the equipotential to be located, which can then be used to sketch a flow pattern [Lambe and Whitman, (1969)].

2.6.4 Numerical Methods

As a result of many difficulties that appear through analytical methods, approximate and experimental methods, it was resorted to numerical methods in order to get the required results with good accuracy. These results could be compared with analytical solution by applying various boundary conditions for simple problems in addition to the development of computer systems. Numerical methods could be divided into three methods: -

2.6.4.1 The Finite Difference Method (F.D.M)

A numerical finite difference solution to steady state flow situations was first presented by Show and Southwell, (1941) but the development of the method accelerated with the advance of digital computers [Bear and Verruijt, (1990)]. Finite difference method has been used in a number of ground water applications as well as in earth dams. Many of these are described by Remson et.al, (1971).

2.6.4.2 The Finite Element Method (F.E.M)

The finite element method was first applied to boundary value problems by Zienkiewics and Cheung (1965) and their method was later extended to obtain a solution for a steady state seepage in anisotropic foundation under a concrete dam (Zienkiewicz et al, 1966). Their particular contribution was to show that solutions to anisotropic flow situations in which the principal directions of anisotropy vary from point to point could be obtained easily by the finite element method. Flow solutions for foundations with curved strata became possible.

The finite element method is more usable in solving complex problems than the finite difference method for the following reasons: -

- 1- In finite element method anisotropy and non homogenous problems are taken into account quite easily in comparison with the finite difference method.
- 2- The boundary conditions are easily handled by the finite element method, where formulas must be developed for each condition by finite difference method. [Al-Obaidi, (2001)].

2.6.4.3 The Boundary Element Method (B.E.M)

The boundary element method which could be considered as one of the new methods, which are used in analyzing the unconfined flow problems of both steady-state and non-steady state conditions. The boundary element method requires discretization only on the boundary rather than over the whole region as that required in finite element method, so, the effort of input preparation and requirement for computer are greatly reduced.

Description and application of this method in seepage analysis through earth dams can be found in references [Banerjee and Butterfield, (1981); Brebbia, (1978); Chang, (1988), Liggett, (1977)]

The applications of the numerical methods in flow through porous media especially about seepage through earth dams are concerned with proving the use of these methods and how to develop them.

Solutions to unconfined flow problems were presented by Finn (1967) and Taylor and Brown (1967) Shortly after. In the unconfined case the position of the free surface in the dam is unknown at the start of the analysis and an assumption has to be made as to its position. In both these analysis the free surface is assumed to define the upper boundary of the

mesh and an iterative procedure is adopted to adjust the position of nodes on this boundary until all boundary conditions are satisfied.

Finn's solution can be described in a number of steps: -

- i) A free surface line is assumed for the section to be analyzed.
- ii) The saturated region below that line is divided into finite elements.
- iii) The coordinate and element data are then supplied to the program with the other boundary conditions. (the pressures within the and on the free surface are assumed unknown).
- iv) A solution to the Laplace equations is obtained and the potentials printed.
- v) If the boundary condition of the pressure head (ψ) being zero on the free surface is not met at any node (to a reasonable degree of accuracy) the co-ordinate data is modified so that the second guess would satisfy the ($\psi = 0$) condition along the surface. If the first guess was poor the mesh is also modified by adding or subtracting elements to avoid any unreasonable deformation.
- vi) A second solution is obtained and iteration continues until the boundary condition on the free surface are satisfied.

A similar but improved approach is described by Newman and Witherspoon (1970). The iterative procedure is in two stages, satisfying the equations for two sets of boundary conditions. By this method the seepage face does not have to be specified as a prescribed head boundary as in the approach of Taylor and Brown or Finn. Newman and Witherspoon consider that this modification improves the stability as well as the accuracy of the solution. Examples are presented for various problems including one for a

dam with a sloping core and horizontal drain. The free surface is shown to be discontinuous and a seepage face develops at the rear of the core.

[Isaacs (1971)] employed a similar method with an automatic method for altering the position of nodes along the free surface. He also studied the Kozeny dam problem with both a coarse and fine mesh and achieved excellent agreement with the Kozeny free surface for both cases, [Al – Qaisi, (1999)].

[Liggett (1972)] studied the use of Boundary element method in the analysis of seepage through rectangular dam to locate the position of free surface and quantity of seepage. The results were good as compared with analytical results.

Unconfined flow in dams was also studied by Dunglas (reported by sing-Bharat 1976) who studied a homogenous dam (15 m height) having a vertical drain (8m height) by using the finite element method. He calculated the position of the free surface by changing the position of drain and the permeability coefficient ratio in x and y direction.

His results have been indicated that when the drain position is at the start of downstream slope, the free surface location is above the drain for $\frac{k_x}{k_y}$ greater than 10.

When position of the drain has been changed 7m from down stream slope, the $\frac{k_x}{k_y}$ ratio has been increased to 20. Figure (2.2) represents the results.

[Desai and Christian, (1977)] used the finite element method to analyze seepage flow through earth embankments in draw-down case, the results were good as compared with field measurement for Mississippi River.

[Kalkani (1983)] studied an earth dam having impervious core based on alluvial foundation with impervious cutoff by using the finite difference method for different permeability coefficient ratios for the dam and foundation. He concluded that for $\frac{k_x}{k_y}$ ratios from 10 to 100 the cutoff length must to be 35%-100% from foundation depth respectively .

[Chang, (1988)] used the boundary element method to find pore-pressure prediction, the modeled dam (Kelloq dam) made of coarse clean Ottawa sand. Seven pore pressure cells were embedded in the model at various locations so that the measured results would show the spatial variation of pore pressure. The height of the model is (38cm). The reservoir was filled to a height of (38cm) in the model , and that level was maintained to permit steady state of seepage to develop. The hydraulic conductivity used in the B.E.M prediction was (0.1 cm/s) in vertical and horizontal direction, the sides slope was (1 V : 1.7 H). The results show a close agreement between the calculated and measured potential heads at various locations (Table (2-1)).

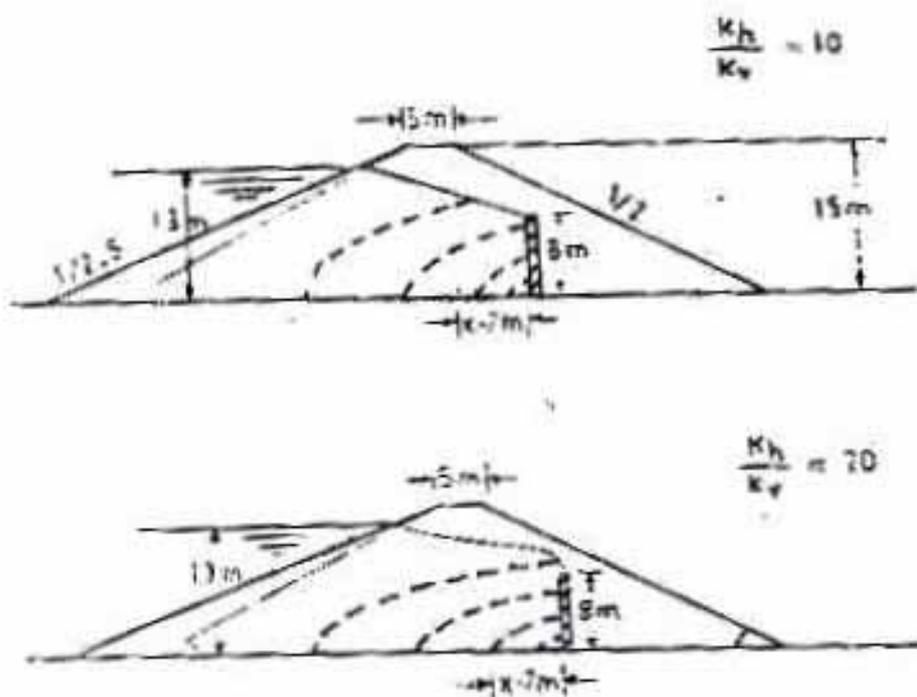


Figure (2.2) Free surface position for different values of k_x/k_y by

[Sing –Bharat , (1976)]

**Table (2-1) A comparison of measured and computed potential heads
laboratory dam model (data from Chang, 1988)**

Cell No.	Co-ordinates		Total Head Field Data (Cm)	Total Head B.E.M (Cm)
	x	y		
1	7.86	3.93	37.34	36.83
2	31.26	14.41	33.02	33.02
3	31.32	5.06	35.306	32.002
4	35.03	18.74	30.48	31.5

5	47.75	5.16	21.08	21.84
6	61.81	5.9	8.38	9.15
7	44.55	13.1	24.13	24.13

[Muhammad (1991)] had conducted a study for the seepage through earth dams by using the finite element method, first he prepared a program in Fortran language to examine the seepage through earth dams and to locate the phreatic surface position and then to find out the quantity of seepage. In order to check the program efficiency, he analyzed a rectangular dam as shown in Figure (2.3). The upstream water depth equal to 10m and downstream water depth equal to 4m, the base length equal to 10m. Assuming the soil is isotropic and the permeability coefficient equal units. The theoretical solution of this problem is known and the seepage quantity equal to $4.3\text{m}^3/\text{day}$. He operated his program many times on this case, table (2-2) represents the results that have been occurred are occurring by changing the equipotential lines number for the same number of the flow channels and the seepage quantity which results from it and the error ratio. All the results have indicated that increasing in accuracy was gained by increasing the equipotential lines number for the same number of flow channels. Also increasing the accuracy by increasing the flow channels number for the same number of equipotential lines. Figure (2.4) represents the relation between the error ratio and the equipotential lines number and for number of the flow channels, also increasing the accuracy by increasing the element numbers, figure (2.5) represents the relation between the error ratio and the elements number.

Table (2-2) The seepage quantity results by changing the equipotential lines number [data from Mohammed J.R, (1991)]

The Flow	The	The Seepage	Error Percent %
----------	-----	-------------	-----------------

Channels Number N1	Equipotential Lines Number N2	Quantity M ³ /Day	according to the theoretical value (4.3 m ³ /Day)
3	10	4.6	9.7
3	13	4.51	7.4
3	15	4.46	6.3
3	17	4.42	5.3
3	20	4.38	4.3
4	10	4.52	7.5
4	13	4.46	6.3
4	15	4.44	5.7
4	17	4.42	5.2
4	20	4.36	3.8
5	10	4.45	5.9
5	13	4.4	4.7
5	15	4.39	4.4
5	17	4.66	3.9
5	20	4.33	3.1
6	10	4.36	3.9
6	13	4.34	3.4
6	15	4.34	3.2
6	17	4.33	3.1
6	20	4.32	3.6

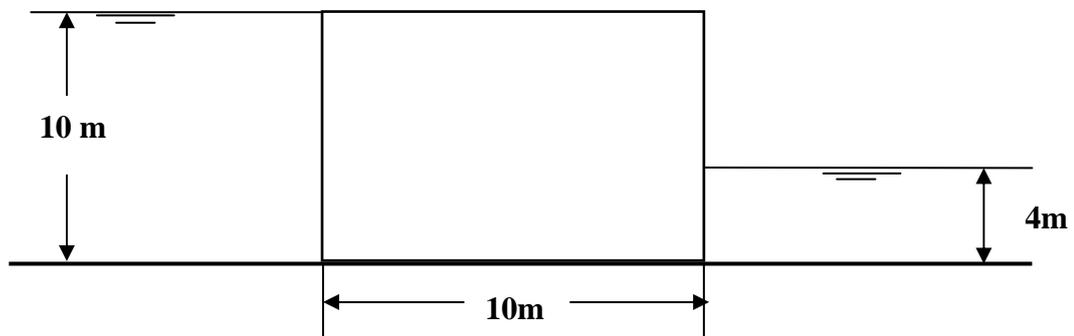
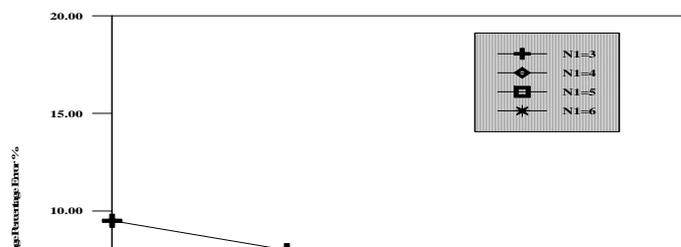


Figure (2.3) Schematic section of rectangular dam .
[after Mohammed J.R ,1991]



Seepage percentage error

N2

Figure (2.4) Relation between error ratio & equipotential lines no. for different values of flow channels [after Mohammed J.R ,1991]

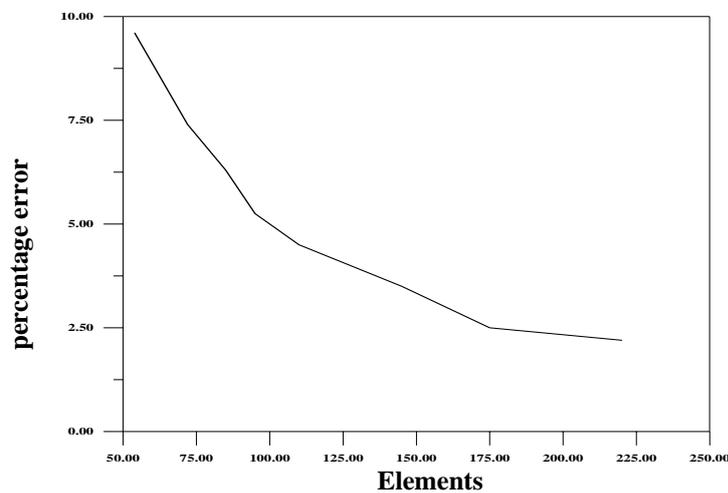


Figure (2.5) Relation between error ratio & element number [after Mohammed J.R ,1991]

[Payam C.T, (1994)] studied the use of finite difference method in the analysis of (meeks cabin earth dam). The dam consist of three different regions of shell, clay core and filters material and based on impervious rock foundation. His results were compared with the piezometric measurement which embedded in the clay core. The results show a close agreement between the field measurements and the finite difference method results. Table (2-3) represents comparison of measured and computed pore water pressure.

Table (2-3) Comparison of measured and computed pore water pressure in Meeks cabin dam. [Hussein, (2001)]

Piezometers	The Field Measurement Of Pore Water Pressure (M)	F.D.M For Pore Water Pressure (M)
1	19.39	19.25
2	13.865	13.8
3	13.767	13.77
4	8.789	8.75
5	8.063	7.9
6	3.152	3.16
7	2.679	2.63
8	0.038	0

Chapter Three

Finite Element Formulations

3.1 Introduction

In the previous chapters, the basic principles of different methods that are used for solving unconfined seepage problems have been reviewed.

Also the previous studies on seepage problems have been mentioned. In this chapter the principles of the finite element method and the finite element formulations for two dimensional flow through porous media using Galerkin weighted residual method [Zienkiewicz, (1982)] will be explained.

3.2 The Finite Element Method

This method can be clarified through the following steps: -

Step 1: discretization of the problem by selection of elements interconnected at the nodal points [Conner and Brebbia, (1976)].

Step 2: representing the behavior of the field variable on each element approximately by a continuous function depending on the nodal values of the field variable as follows:

$$H^e = \sum_{i=1}^n N_i H_i \quad \dots(3-1)$$

Where:

H^e : Approximate solution for piezometric head distribution in the element
e

N_i : Shape function of the element.

H_i : Nodal values of head, H, of the element.

n : number of nodes in the element.

It is possible to write Equation (3-1) in a matrix form as follows [Zienkiewicz, (1966)]:

$$H^e = [N_i]\{H_i\} \quad \dots(3-2)$$

Where:

$[N_i]$: Shape function matrix.

$\{H_i\}$: Vector matrix of nodal values.

The approximate solution for head variation, H , over the whole domain is given as follows:

$$H = \sum_{e=1}^{ne} H^e = \sum_{e=1}^{ne} \sum_{i=1}^n N_i H_i \quad \dots(3-3a)$$

or

$$H = \sum_{e=1}^{ne} [N_i]\{H_i\} \quad \dots(3-3b)$$

Step 3 : Differentiating the finite element matrices by using one of the following methods:

- 1- Direct approach.
- 2- Variational approach.
- 3- Weighted Residual approach.

The basic principles of these methods are given in references [Zienkiewicz, (1977); Rao, (1982)], and here the weighted residual method is used in differentiating the finite element matrix because it is easy to be implemented and it can cover a wider range than any other method.

3.2.1 Weighted Residual Method

The weighted residual method is a technique, which can be used to obtain approximate solutions to linear and non-linear differential equations.

If this method is used, the finite element equations can be derived directly from the governing differential equation of the problem [Rao, (1982)].

If A is a problem domain, and H is the field variable, then the governing equation can be written as follows:

$$F(H) = 0 \quad \text{in } A \quad \dots(3-4)$$

Then, if the approximate solution is H_a and by substituting in equation (3-4), then the right side of the equation does not equal zero, but there is a residual (R^e):

$$F(H_a) = R^e \neq 0 \quad \dots(3-5)$$

The aim is to make this residual minimum.

In order to attain this aim, it should be integrated across the problem domain after weighting by a certain function and should equal zero as follows:

$$\int_A W_j R^e dA = 0 \quad \dots(3-6a)$$

$$\sum_{j=1}^{ne} \int_{A^e} W_j R^e dA = 0 \quad \dots(3-6b)$$

Where:

W_j : Weighted functions.

R^e : Element residual.

There are some methods applied to weighted residual depending on the weighted function. One of these methods is the Galerkin method, which assumes that the weighted function is the same as the shape function, i.e.

$$W_j = N_j$$

3.2.2 Galerkin Principle

As a continuation for the last step in part (3-2) the Galerkin principle is applied to derive the elements matrix. From equation (3-1):

$$H^e = \sum_{i=1}^n [N_i H_i] \quad \dots(3-7)$$

Where:

H_i : The value of piezometric head in node i.

The general equation for seepage flow in porous media as it has been mentioned in chapter two, is for two-dimensional, anisotropic and nonhomogenous :

$$\frac{\partial}{\partial x} [k_x \frac{\partial H}{\partial x}] + \frac{\partial}{\partial y} [k_y \frac{\partial H}{\partial y}] = 0 \quad \dots(3-8)$$

Substituting equation (3-7) in equation (3-8) results:

$$\frac{\partial}{\partial x} [k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i] + \frac{\partial}{\partial y} [k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i] = R^e \neq 0 \quad \dots(3-9)$$

Now, by applying Galerkin principle and substituting equation (3-9) in (3-6b) yields:

$$\sum_1^{ne} [\int_{A^e} N_j^e [\frac{\partial}{\partial x} [k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i] + \frac{\partial}{\partial y} [k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i]] dA] = 0 \quad \dots(3-10)$$

Where:

$$dA = dx \cdot dy \quad \text{and} \quad j = 1 \text{ to } n$$

n = number of nodes for each element.

To reduce the continuity requirements for the shape function, N , from C^1 to C^0 continuities [Zienkiewicz, (1982)], integration by parts with Green's theorem is applied to the second order derivatives terms, where C^1 and C^0 are the continuity for the shape function for the first and zero stage, respectively [Burnett, (1987)].

The first term of equation (3-10) will be as follows: -

$$\int_{A^e} N_j^e \frac{\partial}{\partial x} [k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i] dA = \int_p N_j^e \frac{\partial}{\partial x} \sum_1^n N_i H_i dy - \int_{A^e} \frac{N_j^e}{\partial x} k_y \frac{\partial}{\partial x} \sum_1^n N_i H_i dA \quad \dots(3-11)$$

The second term of equation (3-10) will be as follows:

$$\int_{A^e} N_j^e \frac{\partial}{\partial y} [k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i] dA = \int_p N_j^e k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i dx - \int_{A^e} \frac{N_j^e}{\partial y} k_x \frac{\partial}{\partial y} \sum_1^n N_i H_i dA \quad \dots(3-12)$$

Substituting equations (3-11) and (3-12) in equation (3-10) results:

$$\sum_1^n \int_{A^e} [-\frac{N_j^e}{\partial x} k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i] + \frac{\partial N_j^e}{\partial y} k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i] dA + \int_p N_j^e k_n \frac{\partial}{\partial n} \sum_1^n N_i H_i dp \quad \dots(3-13)$$

Where $p = p_1^e + p_2^e$ represents the surface boundaries of the element.

The boundary conditions are:

$$H = H_0 \quad \dots(3-14a)$$

On p_1 which represents the domain boundaries, and

$$\left[k_x \frac{\partial H}{\partial x} L_x + k_y \frac{\partial H}{\partial y} L_y \right] = 0 \quad \dots(3-14b)$$

On p_2 which represents the impermeable boundaries by applying finite element method to equation (3-14b) it becomes:

$$k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i L_x + k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i L_y = R^e = 0 \quad \dots(3-15)$$

Where R^e is element boundary residual.

Using Galerkin Weighted residual method, equation (3-15) becomes:

$$\sum_1^{ne} \int_{p_2^e} \left[N_j k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i L_x + N_j k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i L_y \right] dp = 0 \quad \dots(3-16)$$

Where: $dx = L_x dp$, $dy = L_y dp$

Multiplying equation (3-13) by (-1), then adding it to equation (3-16) give:

$$\sum_1^{ne} \left[\int_{A^e} \left[\frac{N_j}{\partial x} k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i + \frac{\partial N_j}{\partial y} k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i \right] dx dy - \int_{p_1^e} \left[N_j k_x \frac{\partial}{\partial x} \sum_1^n N_i H_i L_x + N_j k_y \frac{\partial}{\partial y} \sum_1^n N_i H_i L_y \right] dp \right] = 0 \quad \dots(3-17)$$

And in matrix form:

$$\sum_1^{ne} [K^e] \{H\} = 0 \quad \dots(3-18)$$

Where $[K^e]$ represents the element matrix and its typical coefficient is :

$$K_{ij} = \int_{A^e} \left[\frac{\partial N_j}{\partial x} k_x \frac{\partial N_i}{\partial x} + \frac{\partial N_j}{\partial y} k_y \frac{\partial N_i}{\partial y} \right] dx dy - \int_{P_1^e} [K_x L_x N_j \frac{\partial N_i}{\partial x} + k_y L_y N_j \frac{\partial N_i}{\partial y}] dp \quad \dots(3-19)$$

The surface integral on P_1^e in the equation (3-19) is unknown. This condition does not cause any difficulties in solving the simultaneous equation because this equation does not form in the node, which lies in the boundaries on the surface P_1^e and the equation (3-19) does not apply in this part because the value of piezometric head is known. Therefore, and for simplifying this term in the equation of surface integral, it will be deleted from equation (3-19) because it is not active to form the element matrix. The equation (3-19) will be as follows:

$$K_{ij} = \int_{A^e} \left[\frac{\partial N_j}{\partial x} k_x \frac{\partial N_i}{\partial x} + \frac{\partial N_j}{\partial y} k_y \frac{\partial N_i}{\partial y} \right] dx dy \quad \dots(3-20)$$

It is possible to represent $[K^e]$ in matrix form as follows:

$$[K^e] = \int_{A^e} [B^e]^T [D^e] [B^e] dx dy = 0 \quad \dots(3-21)$$

Where:

$$[B^e] = \begin{bmatrix} \frac{\partial N_1^e}{\partial x} & \frac{\partial N_2^e}{\partial x} & \dots & \frac{\partial N_n^e}{\partial x} \\ \frac{\partial N_1^e}{\partial y} & \frac{\partial N_2^e}{\partial y} & \dots & \frac{\partial N_n^e}{\partial y} \end{bmatrix}$$

$$[D^e] = \begin{bmatrix} \mathbf{K}_x & \mathbf{0} \\ \mathbf{0} & \mathbf{K}_y \end{bmatrix}$$

Step 4: assembling the equations or matrices of the finite element $[K^e]$ to form the Global matrix gives:

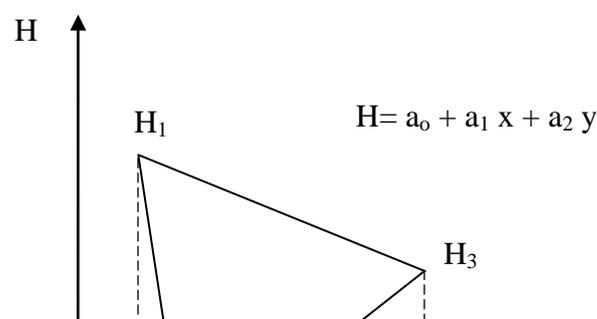
$$[K]\{H\} = 0 \quad \dots(3-22)$$

Where, $[K]$ is the global matrix $= \sum [K^e]$

Step 5: solving the assembled equation (3-22) using the frontal solution [Hinton and Owen, (1977)].

3.3 Derivation of Element Matrix (Linear Triangular Element)

The linear triangular element shown in Figure (3.1) has straight sides and three nodes, one at each corner. A constant labeling of the nodes is a necessity and the labeling here proceeds counter clockwise from node 1, which is arbitrarily specified. The nodal values of H are H_1, H_2 and H_3 , whereas the nodal coordinates are (x_1, y_1) , (x_2, y_2) and (x_3, y_3)



The interpolation polynomial is

$$\mathbf{H} = \mathbf{a}_0 + \mathbf{a}_1\mathbf{x} + \mathbf{a}_2\mathbf{y} \quad \dots(3-23)$$

To find the constant values a_0, a_1, a_2 substitute x, y and H for the three nodes results:

$$\mathbf{H}_1 = \mathbf{a}_0 + \mathbf{a}_1\mathbf{x}_1 + \mathbf{a}_2\mathbf{y}_1$$

$$\mathbf{H}_2 = \mathbf{a}_0 + \mathbf{a}_1\mathbf{x}_2 + \mathbf{a}_2\mathbf{y}_2$$

$$\mathbf{H}_3 = \mathbf{a}_0 + \mathbf{a}_1\mathbf{x}_3 + \mathbf{a}_2\mathbf{y}_3$$

In matrix form

$$\begin{bmatrix} \mathbf{H}_1 \\ \mathbf{H}_2 \\ \mathbf{H}_3 \end{bmatrix} = \begin{bmatrix} 1 & \mathbf{x}_1 & \mathbf{y}_1 \\ 1 & \mathbf{x}_2 & \mathbf{y}_2 \\ 1 & \mathbf{x}_3 & \mathbf{y}_3 \end{bmatrix} \begin{bmatrix} \mathbf{a}_0 \\ \mathbf{a}_1 \\ \mathbf{a}_2 \end{bmatrix} \quad \dots(3-24)$$

Solving the equation (3-24) for the constant and substituting the constants values in the equation (3-23) and expressed it in a matrix form results:

$$\mathbf{H} = [\mathbf{1} \quad \mathbf{x} \quad \mathbf{y}] = \begin{bmatrix} 1 & \mathbf{x}_1 & \mathbf{y}_1 \\ 1 & \mathbf{x}_2 & \mathbf{y}_2 \\ 1 & \mathbf{x}_3 & \mathbf{y}_3 \end{bmatrix}^{-1} \begin{bmatrix} \mathbf{H}_1 \\ \mathbf{H}_2 \\ \mathbf{H}_3 \end{bmatrix} \quad \dots(3-25)$$

$$= (\mathbf{a}_1 + \mathbf{b}_1\mathbf{x} + \mathbf{c}_1\mathbf{y})\mathbf{H}_1 + (\mathbf{a}_2 + \mathbf{b}_2\mathbf{x} + \mathbf{c}_2\mathbf{y})\mathbf{H}_2 + (\mathbf{a}_3 + \mathbf{b}_3\mathbf{x} + \mathbf{c}_3\mathbf{y})\mathbf{H}_3$$

$$= \mathbf{N}_1\mathbf{H}_1 + \mathbf{N}_2\mathbf{H}_2 + \mathbf{N}_3\mathbf{H}_3$$

or

$$\mathbf{H} = \sum_1^{\mathbf{I}} \mathbf{N}_i \mathbf{H}_i$$

Where:

\mathbf{I} : number of nodes in the element =3

\mathbf{N}_i : interpolation function.

\mathbf{H}_i : values of domain variable at node i.

The interpolation function

$$\mathbf{N}_i = \mathbf{a}_i + \mathbf{b}_i \mathbf{x} + \mathbf{c}_i \mathbf{y} \quad \dots(3-26)$$

$$\mathbf{a}_1 = \frac{1}{\mathbf{D}}(\mathbf{x}_2 \mathbf{y}_3 - \mathbf{x}_3 \mathbf{y}_2), \mathbf{a}_2 = \frac{1}{\mathbf{D}}(\mathbf{x}_3 \mathbf{y}_1 - \mathbf{x}_1 \mathbf{y}_3) \quad \dots(3-27a)$$

$$\mathbf{a}_3 = \frac{1}{\mathbf{D}}(\mathbf{x}_1 \mathbf{y}_2 - \mathbf{x}_2 \mathbf{y}_1)$$

$$\mathbf{b}_1 = \frac{1}{\mathbf{D}}(\mathbf{y}_2 - \mathbf{y}_3), \mathbf{b}_2 = \frac{1}{\mathbf{D}}(\mathbf{y}_3 - \mathbf{y}_1) \quad \dots(3-27b)$$

$$\mathbf{b}_3 = \frac{1}{\mathbf{D}}(\mathbf{y}_1 - \mathbf{y}_2)$$

$$\mathbf{c}_1 = \frac{1}{\mathbf{D}}(\mathbf{x}_3 - \mathbf{x}_2), \mathbf{c}_2 = \frac{1}{\mathbf{D}}(\mathbf{x}_1 - \mathbf{x}_3) \quad \dots(3-27c)$$

$$\mathbf{c}_3 = \frac{1}{\mathbf{D}}(\mathbf{x}_2 - \mathbf{x}_1)$$

$$\mathbf{D} = \begin{bmatrix} 1 & \mathbf{x}_1 & \mathbf{y}_1 \\ 1 & \mathbf{x}_2 & \mathbf{y}_2 \\ 1 & \mathbf{x}_3 & \mathbf{y}_3 \end{bmatrix} = 2\mathbf{A} \quad \dots(3-27d)$$

Where A : is the area of the triangle.

The interpolation functions derivatives with respect to x and y gives:

$$\frac{\partial \mathbf{N}_i}{\partial \mathbf{x}} = \mathbf{b}_i \quad \frac{\partial \mathbf{N}_i}{\partial \mathbf{y}} = \mathbf{c}_i \quad \dots(3-28)$$

Substituting the above values in the equation (3-20) results:

$$\mathbf{K}_{ij} = \int_{\mathbf{A}^e} (\mathbf{k}_x \mathbf{b}_i \mathbf{b}_j + \mathbf{k}_y \mathbf{c}_i \mathbf{c}_j) \mathbf{d}\mathbf{x}\mathbf{d}\mathbf{y} \quad \dots(3-29a)$$

$$\mathbf{K}_{ij} = \mathbf{A}(\mathbf{k}_x \mathbf{b}_i \mathbf{b}_j + \mathbf{k}_y \mathbf{c}_i \mathbf{c}_j) \quad \dots(3-29b)$$

The above equation represent the element matrix elements and it is rectangular matrix [3*3]

3.4 Steps of the Solution

The basic principles of the finite element method have been explained in the previous sections, Now, the general steps which are used to solve the unconfined seepage problems especially through earth dams by using the finite element method are as follows:-

- 1- Discretization the flow region into triangular elements assuming the initial position of the free surface and numbering the nodes for each element.
- 2- Evaluation the element matrix for each element by using the formulations in section (3-3).
- 3- Assemble the element matrices from (local) to (Global).
- 4- Applying the Boundary conditions in the Global matrix.
- 5- Solving the system of equations to find the head values (H) at each node.
- 6- Re assume the position of the free surface in order to satisfy the free surface boundary [y=H].
- 7- Repeating the trials until the condition in point (6) will satisfy.
- 8- Calculate the necessary parameters such as a seepage quantity and pore water pressure.

For the above steps, A general package under the name of “ Seep2d” is used to execute the above calculations.

3.5 Computation of Hydraulic Gradient

From Darcy's law,

$$U = k i_x = -k_x \frac{\partial H}{\partial x}$$

$$V = k i_y = -k_y \frac{\partial H}{\partial y}$$

from the above equations its possible to compute the velocity and hydraulic gradient from the value of piezometric head at nodes

$$H = \sum_{i=1}^n N_i H_i \quad i = 1, 2, 3.$$

then

$$\frac{\partial H}{\partial x} = \frac{\partial [N]}{\partial x} \begin{bmatrix} H_1 \\ H_2 \\ H_3 \end{bmatrix} = \frac{1}{2A} \begin{bmatrix} b_1 & b_2 & b_3 \end{bmatrix} \begin{bmatrix} H_1 \\ H_2 \\ H_3 \end{bmatrix} \quad \dots(3-30)$$

$$\frac{\partial H}{\partial y} = \frac{\partial [N]}{\partial y} \begin{bmatrix} H_1 \\ H_2 \\ H_3 \end{bmatrix} = \frac{1}{2A} \begin{bmatrix} c_1 & c_2 & c_3 \end{bmatrix} \begin{bmatrix} H_1 \\ H_2 \\ H_3 \end{bmatrix}$$

from equation (3-30) the hydraulic gradient computed in the x and y directions which also represent the velocity divided by the coefficient of permeability:

$$\frac{U}{k_x} = -\frac{\partial H}{\partial x} = i_x \quad \dots(3-31)$$

$$\frac{V}{k_y} = -\frac{\partial H}{\partial y} = i_y$$

its possible to compute the velocity resultant:

$$V_s = \sqrt{U^2 + V^2} \quad \dots(3-32)$$

if the soil layers are anisotropic, $k_x \neq k_y$, then:

$$\frac{V_s}{K_{equiv}} = \sqrt{\left(\frac{U}{k_x}\right)^2 + \left(\frac{V}{k_y}\right)^2} = i \quad \dots(3-33)$$

where, i is the hydraulic gradient, and

K_{equiv} = equivalent hydraulic conductivity = $\sqrt{k_x \times k_y}$

by knowing the hydraulic gradient in the flow domain, the discharge velocity in any point on the domain can be computed, also the quantity of seepage can also be determined.

3.6 Outline of the Computer Program

The computer program which is used in this study is entitled “Seep2d”. This program is in Fortran 77 and it’s a Two-Dimensional Finite Element Flow Model designed to compute seepage on profile such as for earthen dam and levee cross section. Seep2d can be used to model confined, partially confined, and unconfined flow situation. For unconfined flow, both the saturated and unsaturated flow is simulated and the phreatic surface determined. Seep2d can model complicated 2D seepage problems involving complex model geometries and soils that are non-homogenous and anisotropic seep2d is a steady state flow model and will compute the flow value for each node of the finite element mesh.

The input file has the property of each material and the mesh generation data for the total section.

The region is subdivided into linear triangular element with three nodes by using a general software entitled “GMS” [Ground Water Modeling System].

The structure chart of the used computer program is schematically shown in figure (3.2).

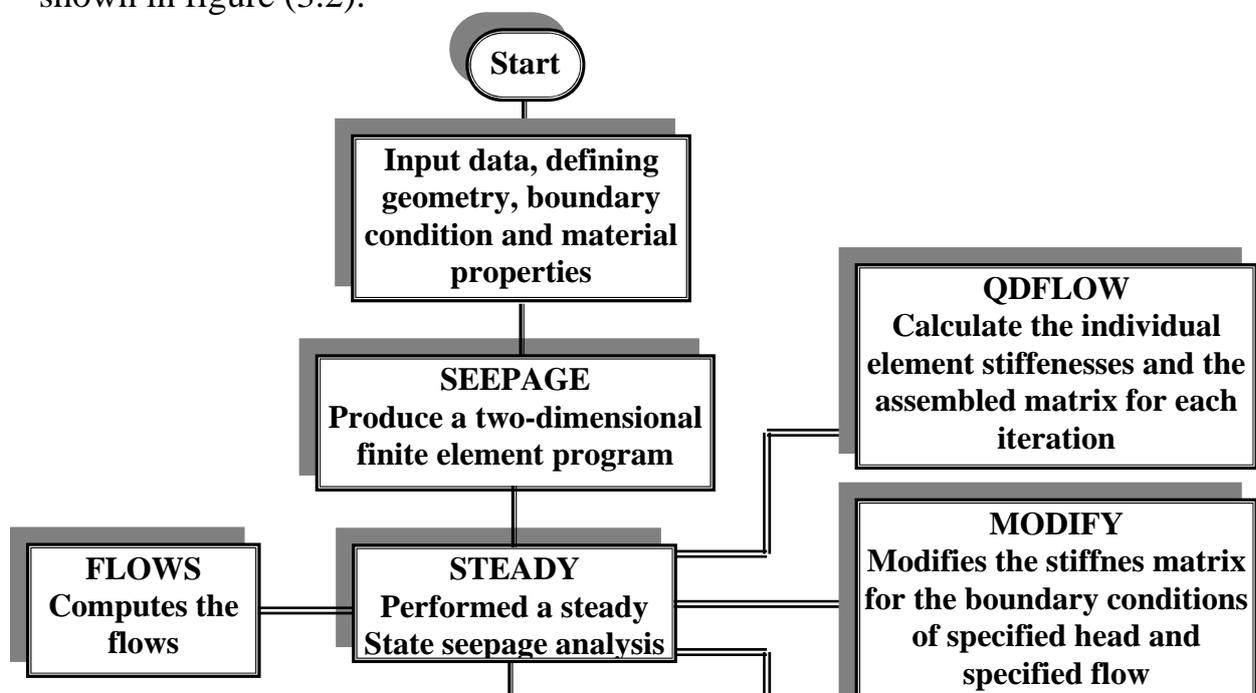


Figure (3.2) Overall structure chart of the computer program

Chapter Four

The Case Study “AL-Qadisiya Dam”

4.1 General

AL-Qadisiya dam is a multi-purpose hydro-development designed to control the Euphrates River flow in the interests of irrigation, electric power generation and for partial accumulation of extreme Euphrates River inflows into Al-Qadisiya reservoir.

AL-Qadisiya dam was constructed on the Euphrates River in the Middle West of Iraq 7km upstream from Haditha town, figure (4.1). In 1988 the project was completed. The project generates (660 Mw) of electrical power a side from performing its flood control function. Central and southern parts of Iraq gets the benefit of irrigation water from its reservoir.

The project comprises mainly of an earth dam, 9 km long, because of the considerable length of the dam and diversity of its topography and geological conditions [Irzooki, (1998)], The design of the dam embankment varies from section to section. But in general it preserves the features of the basic type which cover most of the dam length as shown in figure (4.2).

The body of the dam consists of a central dolomite core and shells made of sand/ gravel material and/or a rock muck (random rock material) [Salih, (2000)]. An asphaltic concrete diaphragm through the core was provided as an antiseepage measure through the body of the dam. A grout curtain was constructed to provide treatment for the foundation against seepage.

Various instruments were installed in the body of the dam for safety measure of the dam. Piezometers and observation wells were installed in the

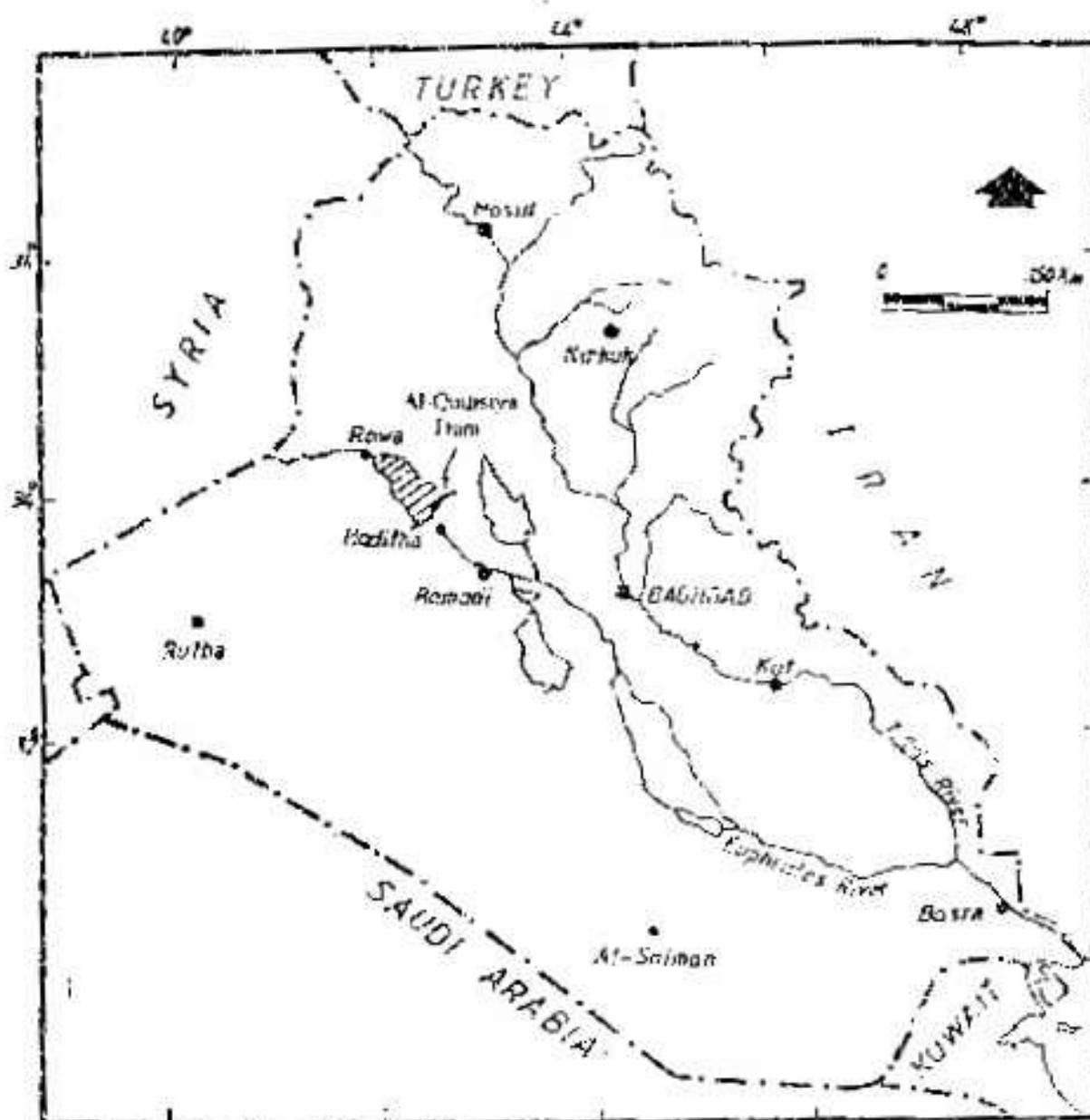
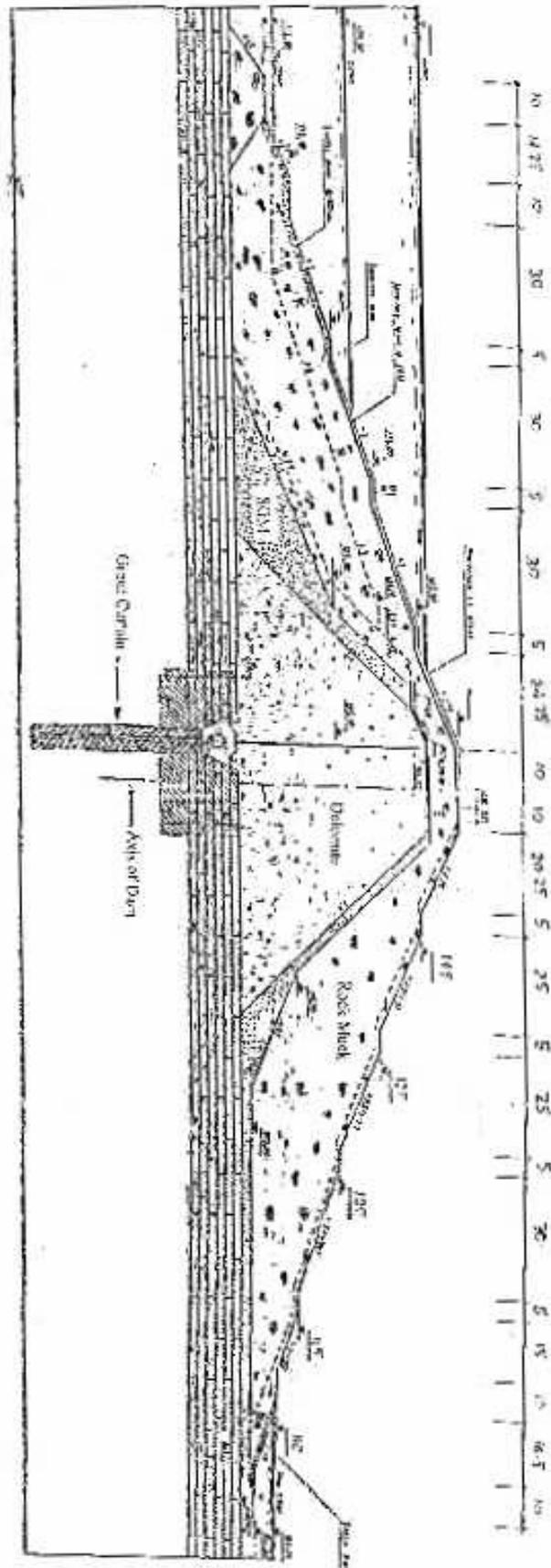


Figure (4.1) AL-Qadisiya Dam project-location map.[Salih, (2000)]



Figure(4.2) General section of AL-Qadisiya Dam (River Section). [Urzooki, (1998)]

body of the dam and adjacent area to monitor the ground water movement and for detecting any seepage that could occur.

4.2 General Features of the Dam and Reservoir

The general feature of the dam and reservoir are outlined in the following [Hydroprojekt, (1978)]: -

Dam body dimensions:

Table (4-1) Design values of the parameters of interest of Al-Qadisiya dam. (Hydroprojekt, (1978))

Body of the Dam	
MAXIMUM HEIGHT	57 m
Top width	20 m
Top elevation	154 m
Length of the right bank	3368.1 m
Length of the river section	431 m
Length of the left bank	5140 m
The Water Level in the Reservoir	
Normal water level	147 m.a.s.l
Maximum water level	150.2 m.a.s.l
Minimum water level	129.5 m.a.s.l
Dead –storage level	112 m.a.s.l

4.3 Geological Conditions of the Project Area

The area of the project is located in the Euphrates River valley ,7 km to the north of Haditha town .The geology of the dam site is rather complex .It is characteri3ed by Miocene dolomites and limestone .

Dolomites are predominate rocks in the area and are formed by the dolomization of limestone and preserve the various texture of the initial

rock [Salem, (1985)] .The upward succession of the lithological rock varieties through the geological section is as follows [Hydroprojekt , (1978)]:-

In the bottom part of the geological section, there occur the rocks of the “Baba” formation (100 m) thick in total, divided into two horizons (bench 1 and benches 2-4) and composed primarily of dolomites. The overlying rocks are coralline aphanitic limestones of the “Ana” formation, (17-24m) in thickness. The rocks are hard (bench 6) and jointly to a variable degree, cavernous and karstified with boulders and lenses of marls, clays and breccia bound with weak argillaceous cement. Total clay content (up to 20%) grows as approaching the stratum foot (bench 5) and in this case lenses and boulders of clay become (3-5m) in size. In the roof of the “Ana” formation breccia and conglomerate –breccia (bench 7), (0.2-5m), rarely up to (10m) in thickness and composed of fragments of hard aphanitic limestones bound with weak argillaceous or marly cement.

The “Euphrates” formation deposits, (80) thick in total, over the deposits of the “Ana” formation and are subdivided into three representative horizons. The lower horizon, (19-27 m) thick is represented by (bench 8-11) composed of limestones having different hardness and alternating with dolomite interlayers.

The deposits of the middle horizon, (bench 12-13), of the “Euphrates” formation (18-22 m) thick, are represented by detrital dolomites which are intensively leached. The rocks of the upper horizon of the “Euphrates” formation (18-33 m) thick, feature lithological variability and intensive crumpling, (benches 14 and 15) are singled out, figure (4.3) shows longitudinal section along axis of the dam and the geological condition of the benches.

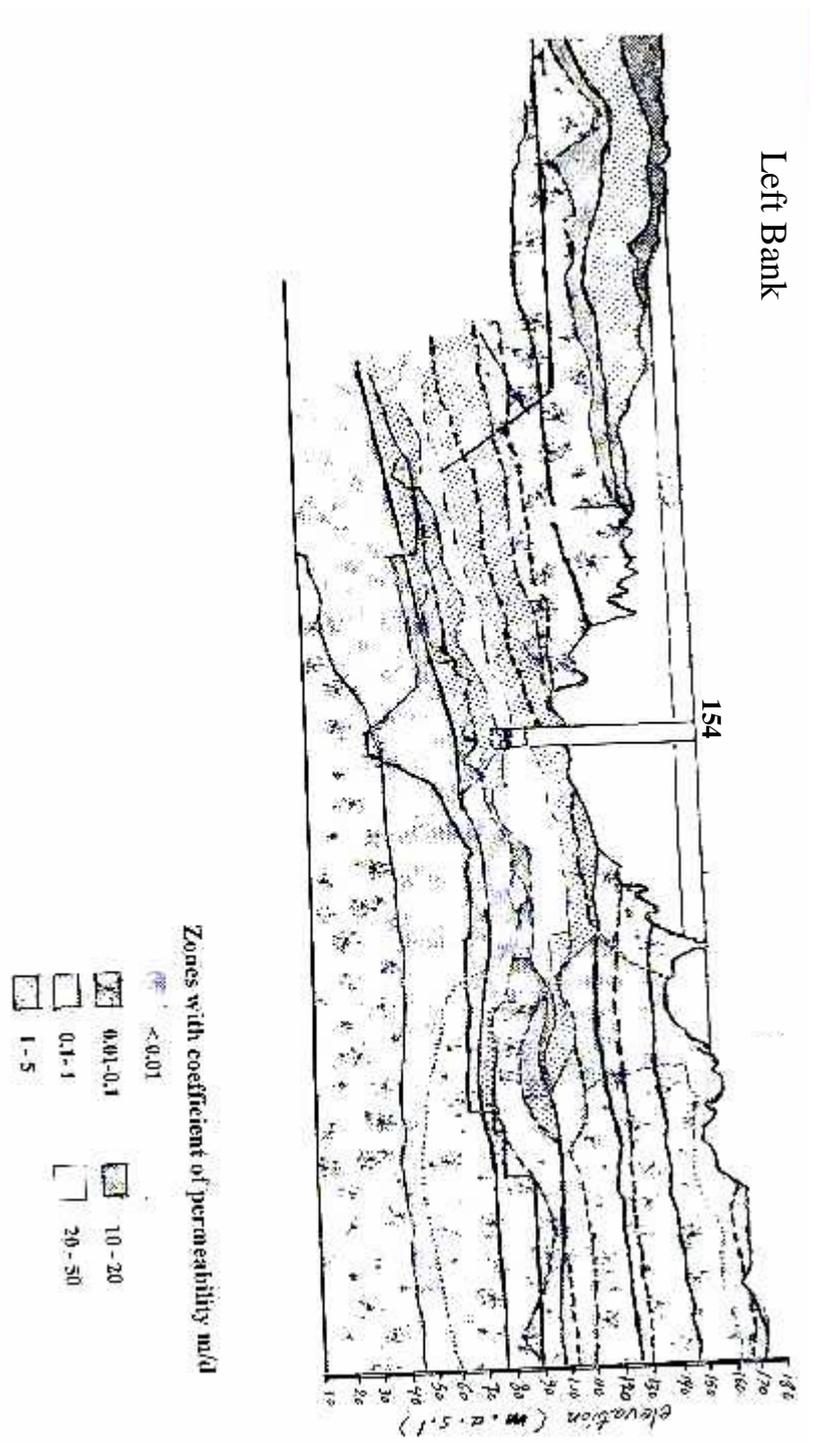


Figure (4.3) Longitudinal geological section at AL-Qadisiya Dam site. [Irzooki, (1998)].

The characteristics of rock permeability of the mean stratigraphic horizons from the bottom to the top are given in table (4-2).

Table (4-2) Coefficient of permeability of the geological sections at Dam site [Hydroprojekt, (1978)]

NO	ROCK FORMATIONS	PERMEABILITY COEFFICIENT (M/D)
1.	“Baba” Formation - The lowers part of upper Baba - The upper part of upper Baba	0.5-1 5
2.	“Ana” Formation	0.008-32.5
3.	“Euphrates” Formation -Lower horizon -Middle horizon -Upper horizon	2 0.01-0.1 0.5-5
4.	“Lower Fars” Formation -Lower horizon -Middle horizon -Upper horizon	1 1-5 0.2-38.6
5.	Alluvial deposits	0.5-50

4.4 Grout Curtain Properties

The grout curtain in the earth dam foundation is designed and being constructed as the seepage control facility necessary for decreasing the seepage losses from the reservoir through the dam foundation, for sealing the concentrated seepage paths and for providing the seepage-erosion

resistance on the contact of the detritus dolomites of the dam central portion and foundation rocks.

The maximum depth of the grout curtain is in the river channel section and under the power house where the grout curtain was extended to the lower horizon of the “Baba” formation with coefficient of permeability of about $(11.57 \times 10^{-3} \text{ cm/s})$. The deep grout curtain on the flood plain sections, 20+00-35+50 and 40+00-59+00, was extended into the “Baba” formation upper horizon with the permeability coefficient of $(11.57 \times 10^{-4} \text{ cm/s})$ [salih, (2000)].

The deep grout curtain is of two constructed rows, the distance between the rows is (1.5 m). The in-line bore holes are spaced at (3 m) centers and offset by (1.5 m) with respect to bore holes of the other row.

The seepage water passing from the dam embankment and foundation is drained into the drainage system provided in the dam embankment and foundation. The seepage water passing from the dam embankment is drained into a sandy-gravel interlayer placed on the downstream side of the central dolomite zone.

The drainage water is collected and diverted through a drainage ditch (an open drain running along the down stream slope).

4.5 Asphaltic Concrete Diaphragm Properties

In the center of the dolomite core, an Asphaltic concrete diaphragm was provided to preclude the seepage through the core. The diaphragm wall has a variable cross section, it was located (10m) upatream from the center line of the dam. The top of the diaphragm is brought to elevation of (150.8m) i.e., above the maximum flood level. From the base to elevation (125m) the diaphragm is (0.8m) thick while from elevation (125m) up to (150.8m) its thickness is (0.5m).

The asphaltic concrete mixture used for foundation sealing was the same as that for central antiseepage diaphragm. It had the properties given in the table (4-3) below [Hydroprojekt , (1978)].

Table (4-3) Properties of asphaltic concrete mixture

No.	Item	Value
1	Residual porosity	<30%
2	Compressive strength at 20 °C	>10 kgf/cm ²
3	Consistency at 150 °C	10-255
4	Bitumen excess over dry material	>1%
5	Segregation while heating to 150 °C	1.01-1.06
6	Water saturation while heating at 80 °C for 75 hr	<2.5%

Mixture of the asphaltic concrete has the following proportioning by weight:

Gravel (5-30)mm in size	49%
Sand up to 5mm in size	26%
Bitumen of (60/70) Gaira or Dora brand with	(8.6%)
100% mineral filler	

4.6 Engineering Properties of Materials Used in the Dam

AL-Qadisiya project is essentially of an earth dam with hydraulic fill shell on the left side and dry filled sand and gravel mixture on the right side.

Dolomite central core was provided by an asphaltic concrete diaphragm.

Dolomite, as semirock soils, are available in large quantities in central and western part of Iraq. Very often they are deposited close to the ground surface and considerably higher than the water table [Salih, (2000)].

Table (4-4) summarizes the physical and mechanical properties of the dolomite used [Hydroprojekt, (1988)].

Table (4-4) Physical and mechanical properties of Dolomite

No.	Item	Value	
1	Natural Moisture Content	3.5-12.5%	
2	Natural Bulk Density	1.55-2.11 Ton/M ³	
3	Swelling	0.3%	
4	Shrinkage	None	
5	Optimum Moisture Content (W)	15-18%	
6	Wax. Dry Density (γ_d)	1.76-1.85 Ton/m ³	
7	Angle Of Internal Friction ϕ	30°	
8	Cohesion (C)	0.13-0.8 Kg/Cm ²	
9	Modulus Of Deformation	1.00 Kg/Cm ²	
10	Permeability Coefficient	(1.44-0.61)*10 ⁻⁴ Cm/S	
1	Design and control parameters	γ_d	$\geq 1.8 \text{ ton/ m}^3$
2		w	18% \pm 2%
3		C	0
4		ϕ	30

Shell of the dam is from sand and gravel mixture (SGM) hydraulically filled into the dam body or into a stock piles.

The (SGM) of the stocks piles is transported to the dam body too and filled in the dry. Right side shell of the dam is filled hydraulically up to elevation (140m), while the left side shell is filled in layers and normal compaction procedures are followed.

Table (4-5) summarized the physical and mechanical properties of the (SGM) used.

Table (4-5) Physical and mechanical properties of (SGM)

No.	Item	Hydraulic Fill	Fill In Dry
1	Average Bulk density (ton/ m ³)	1.9	2.0
2	Particles >1mm	50-80%	30-70%
3	Moisture content	6-9%	2-5%
4	Permeability coefficient (cm/s)	(4-7)*10 ⁻³	(4-5)* 10 ⁻³
5	Average ϕ	(41-33) (39°)	(48-31) (45°)

In the channel sections, the rock fines was used to build the upstream prism of the shell. Table (4-6) concluded the engineering properties of the rock fines.

The upstream shell of the dam is protected by three layers of rock muck, the downstream is protected by one layer of rock muck.

Table (4-6) Engineering properties of rock fines

No.	Item	Value
1	Clay Content	≤ 5%
2	Unit Weight (γ_d)	≥ 2.15ton / m ³
3	Coefficient Of Permeability (K)	(1-10)*10 ⁻³ Cm/S
4	Angle Of Internal Friction(ϕ)	35°
5	Cohesion (C)	0.0

Chapter Five

Model Applications, Analysis and Discussion of Results

5.1 General

Wide range of techniques is available for the study of seepage problems. Much of development in recent years in the analysis of seepage flow has been by means of mathematical models. They are usually based on analog or digital computers [Rushton, and Redshaw, (1979)]. Therefore, a digital computer program technique has been developed, here, in this work. This digital computer does deal with problem of much greater complexity than is practical with electrical analog or analytical methods. On the other hand, the equations describing seepage problems can not be solved by analytical means unless major simplifying assumptions are to be introduced, however solution of these equations can often be obtained by numerical means.

There is often some hesitancy about using numerical techniques for the analysis of seepage problems. This occurs for a variety of reasons, frequently, the lack of accurate information about the soil permeability is given as reason for not attempting to obtain accurate solutions. It is also suggested that the accuracy of solutions depends on the accuracy of the data [AL-Hashamie, (2001)].

This chapter indicates the applications of the mathematical model given in chapter three and analysis of the results of its applications to the case study by solving numerically using F.E.M as mentioned in chapter three.

5.2 Application of the Finite-Element Computer Program to the Case Study

5.2.1 General

This section describes application of the finite element model to analyze seepage problem through (AL-Qadisiya) earth dam observing some involved parameters and their computed results.

The finite element model is examined and checked by comparing its results with the available field data.

5.2.2 Seepage Through and Under AL-Qadisiya Dam

Seepage through the dam embankment is controlled by the dolomite core and the asphaltic-concrete diaphragm. The core is built of mealy dolomite with low permeability of (0.1-1) m/day. It serves the purpose of the main water tight feature of the dam embankment.

Seepage studies of the dam embankment and foundation were conducted at the typical cross-sections of the right and left bank dams. The typical cross-sections of the dam at stations 29, 33+50, 42, 46 Km were used to investigate changes in the basic seepage parameters (head loss across the curtain, residual head downstream of the curtain, the seepage quantity and locating the phreatic surface for each section). Depending on the permeability coefficient of the grout curtain and foundation rock, on the actual seepage head and the position of impervious layer

5.2.3 Ground Water Observations

the seepage conditions in the dam foundation are monitored by using the network of pizometric holes arranged in the dam foundation.

The pizometric network was arranged to facilitate analysis of pizometric data and enable the view of the results obtained for separate cross sections.

Figure (5.1) and table (5.1) show a schematic section of AL-Qadisiya dam and location of piezometers at the selected left and right side of the dam.

Most of the piezometers installed at the dam are open pipe piezometers , except those installed in the river bed , which operate in artesian conditions [Hydroprojeckt , (1988)].

Table (5.1) Location of piezometers at the right and left side of dam

PIEZO. NO.	STATIONS						
	29		33+50		42	46	
	Mouth Elev. M	Bore Hole Depth	Mouth Elev.	Bore Hole Depth	Top Elev. Of Piezo.	Mouth Elev.	Bore Hole Depth
1	154.70	52.0	154.9	62.0	96.4	155.09	52
2	155.0	55.5	154.9	63.75	96.6	154.78	54.5
3	154.6	70.0	154.3	72.00	96.4	155.05	71
4	145.2	74.00	145.9	79.00	96.6	154.67	72
5	-	-	-	-	96.4	-	-
6	-	-	-	-	96.6	-	-
7	123.9	33.00	126.7	36.50	125.30	125.8	43.0
8	-	-	111.5	30.0	111.0	114.07	25.6
9	113.8	21.80	109.9	20.20		113.44	28

The readings of the piezometers located in the downstream shell (close to the asphaltic concrete diaphragm) are used to fix the location of the seepage water levels in the dam body consequently and to give the effect of the asphaltic concrete diaphragm and the grout curtain.

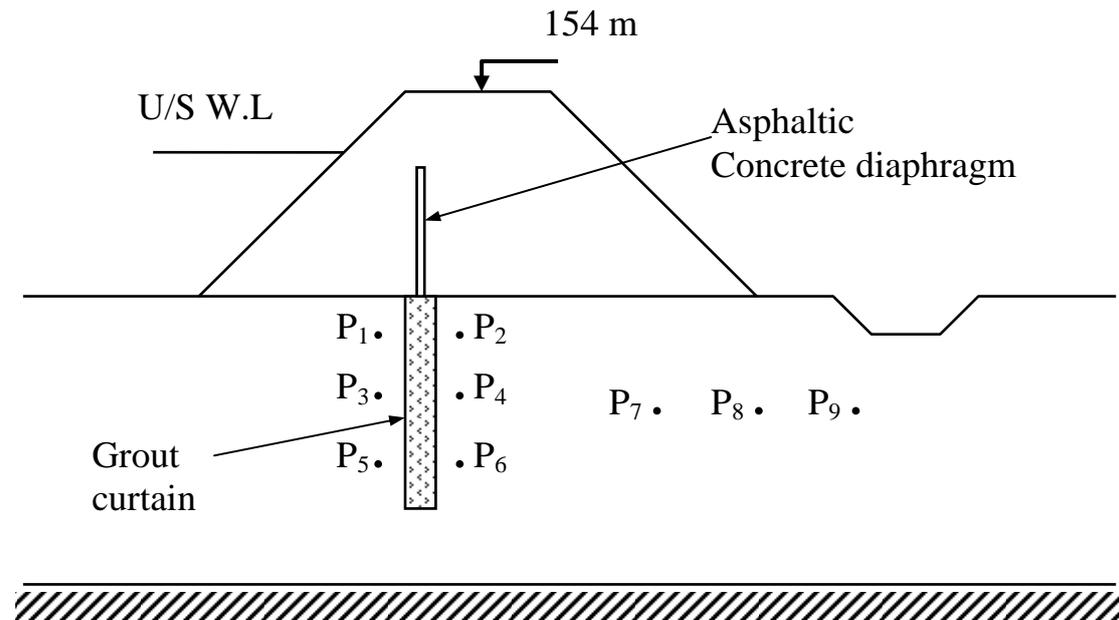


Figure (5.1). Schematic section of Al-Qadisiya Dam and locations of piezometers.

5.2.4 Application of the Program

In the finite element analysis of AL-Qadisiya dam presented here in, the computer program was applied on two stations from the left side of the dam named stations 46 and 42. Seven material types were used in the analysis of these stations, the properties of each material type are shown in table (5.2) below.

Table (5.2). Material properties used in analysis of stations 46 and 42

STATION 46							
Material type	1	2	3	4	5	6	7
Kx (m/day)	20	0.1	1×10^{-6}	0.3	3	1	0.2
Ky (m/day)	20	0.1	1×10^{-6}	0.3	3	1	0.2
STATION 42							
Material type	1	2	3	4	5	6	7
Kx (m/day)	20	0.1	1×10^{-6}	50	10	3	0.2
Ky (m/day)	20	0.1	1×10^{-6}	50	10	3	0.2

The typical cross-section of the dam at station 46 is shown in figure (5.2). The program was applied for two water levels of [135.29 and 144.53 m.a.s.l]. The mesh used in the program for representing problem domain was carefully selected so several trials have been made. These meshes have been modified many times in order to obtained acceptable results and minimize the time required for the analysis process. Figure (5.3) indicates a finite element mesh, which is used in this study. The phreatic line shown in figure (5.4). Was plotted by intersecting the elevation by the contour who has the same value of total head. This is because that, by definition, the phreatic line has a pressure head equal to zero. Figures (5.4), (5.5) and (5.6) represent the phreatic surface position, pressure head and total head contour lines for [H=144.53 m] and figures (1), (2) and (3), (in appendix A) represent the phreatic surface positions, pressure head and total head contour lines for [H=135.29 m]. The head loss in the dam central part foundation amounts to (40%) and the head gradient in the foundation of the dam central part behind the curtain does

not exceed 0.058. The seepage flow equals to 0.67 l/s/m for $H=135.29\text{m}$ and 0.73 l/s/m for $H=144.53\text{m}$.

Figure (5-2)

Figure (5-3)

Figure (5-4)

Figure (5-5)

Figure (5-6)

The finite element results was compared with that available field data in the annual report of the maintenance and operating section of the project , especially the data of the last year (2000). From table (5.3) it can be seen that the results of the finite element method are in a good agreement with the observed data.

Table (5.3) Error percentage between the calculated and observed total head at St-46

PIEZOMETER NO.	READING DATA 31/1/2000 POOL. ELEVATION =135.29 M.A.S.L			READING DATA 22/4/2000 POOL. ELEVATION =144.53 M.A.S.L		
	F.E.M Results (m)	Field Reading (m)	% Error	F.E.M Results (m)	Field Reading (m)	% Error
1	130.5	128.16	1.82	139.5	136.02	2.55
2	116.3	122.52	5.07	120.00	127.42	6.02
3	130.40	119.04	9.35	139.5	123.12	13.3
4	116.33	117.62	1.096	120.05	121.26	0.997
5		—				
6		—				
7	114.5	112	2.24	116.5	116.15	0.26
8	110.55	108.82	1.5	110.05	109.72	0.30
9	109.4	108.99	0.376	107.5	109.93	2.21

At station 42, the program was applied for two different water levels of (133.55 and 138.09 m.a.s.l). Figure (5.7) represents a typical cross-section of the dam at this station. Seepage studies in this station showed that the head loss in the dam central part foundation amounts to (50%) and the head gradient behind the curtain doesn't exceed 0.062. The seepage flow equals to 0.91 l/s/m for H=133.55 m and 1.04 l/s/m for H=138.09 m. Table (5-4) represents the error percentage for the results between the (F.E M) and observed data at different locations, the results indicate acceptable agreement between the calculated and observed data.

Table (5.4) Error percentage between calculation and observed total head at St. 42.

PEZOMETER NO.	READING DATE (3/12/2000)			READING DATE (23/1/2000)		
	Pool Elevation =133.55 m.a.sl			Pool Elevation =138.09 m.a.sl		
	F.E.M Results (m)	Field Reading (m)	Error %	F.E.M Results (m)	Field Reading (m)	Error %

1	129.05	122.4	5.3	130.41	126.4	3.5
2	112.0	110.4	1.4	114.0	111.4	2.34
3	129.05	118.9	8.5	131.0	121.4	7.9
4	112.50	110.6	1.7	114.0	111.6	2.20
5	—	—		130.40	127.40	2.3
6	—	—		114.50	107.10	7.0
7	110.4	104.23	5.9	110.9	104.66	5.96
8	105.5	103.37	1.8	106.5	103.81	2.59
9	—	—	—	—	—	—

Figure (5.8), (5.9) and (5.10) showed the phreatic line, positions, pressures heads and total contour line for H=133.55 m, and figure (4), (5), and (6) (in appendix A) represents the phreatic line positions, pressures and total heads contour line for H=138.09 m.

Two stations from the right side of the dam station (29) and station (33+50) were examined by the finite element method. Nine material types were used in the analysis of AL-Qadisiya Dam at station (29) and eight material types at station (33+50).

The material properties for each material types are shown in table (5-5) below.

Table (5-5). Material properties used in analysis of stations 29 and 33+50

STATION 29									
Material type	1	2	3	4	5	6	7	8	9
Kx (m/day)	20	0.1	1×10^{-6}	5	3	0.5	15	3	0.2

Ky (m/day)	20	0.1	1×10^{-6}	5	3	0.5	15	3	0.2
STATION 33+50									
Material type	1	2	3	4	5	6	7	8	
Kx (m/day)	10	0.1	1×10^{-6}	1	2	0.1	10		0.2
Ky (m/day)	10	0.1	1×10^{-6}	1	2	0.1	10		0.2

Figure (5-7)

Figure (5-8)

Figure (5-9)

Figure (5-10)

At station 29, figure (5.11), the finite element model was tested for two different water levels names (133.69 and 140.13 m.a.s.l) ; seepage studies in the dam at this station showed that the head loss in the foundation of the dam central part makes up 60%. The head gradient behind the curtain does not exceed 0.052. The seepage flow obtained was equal to 0.28 l/s/m for H=133.89 m and 0.30 l/s/m for H=140.13 m . The comparison between the calculated and observed head is shown in table (5-6), from this table it can be seen that the model gives acceptable results and error percentage within the agreeable limit.

Figures (5.12), (5.13), and (5.14) represents the phreatic line position , pressure and total head contour lines for H=140.13 while figures (7), (8) and (9) (in appendix A) show the phreatic line position, pressure head and total head contour lines for H=133.89 m.

Table (5.6) Error percentage between calculated and observed total head at St.29

PEZOMETER NO.	READING DATE (21/12/2000)			READING DATE (22/8/2000)		
	Pool Elevation =133.89 m.a.s.l			Pool Elevation =140.13 m.a.s.l		
	F.E.M Results (m)	Field Reading (m)	Error %	F.E.M Results (m)	Field Reading (m)	Error %
1	131.2	126.48	3.74	138.0	132.53	4.12
2	117.0	115.11	1.64	122.50	116.73	4.4
3	131.3	125.57	4.53	138.20	128.82	7.28
4	117.25	116.69	0.479	122.50	118.52	3.35
5	—	—	—			
6	—	—	—			
7	113.5	112.06	1.28	118.50	111.96	5.8
8	—	—	—			

9	110.30	110.74	0.4	114.50	110.93	3.2
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Figure (5-11)

Figure (5-12)

Figure (5-13)

Figure (5-14)

Station (33+50), figure (5.15), is also used for the evaluation of finite element method. Two different water levels are also used namely (134.9) and 144.29 m.a.s.l) . Seepage studies on the dam at this station showed that the head loss in the dam central part foundation amount to (50%) and the head gradient behind the curtain does not exceed 0.042. Seepage flow equals to 0.32 l/s/m for H=134.91 and 0.40 l/s/m for H=144.29m. Percentage of error between the calculated and measured total head is shown in table (5-7) below. This table indicates acceptable error percentage.

Table (5.7) Error percentage between the calculated and observed total head at St. (33+50)

PEZOMETER NO.	READING DATE (11/1/2000)			READING DATE (8/3/2000)		
	Pool Elevation =134.9 m.a.s.l			Pool Elevation =144.29 m.a.s.l		
	F.E.M Results (m)	Field Reading (m)	Error %	F.E.M Results (m)	Field Reading (m)	Error %
1	132.5	131.57	0.70	141.5	139.54	1.40
2	116.50	114.85	1.5	121.0	117.07	3.4
3	132.45	131.56	0.67	141.5	139.58	1.37
4	116.5	113.25	2.6	121.20	115.15	5.25
5	132.25	129.61	1.5	141.50	136.71	3.5
6	117.5	114.55	2.1	121.80	117.05	4.10
7	115.2	110.46	4.2	117.8	110.21	6.35

8	113.2	110.01	2.7	144.8	110.21	4.16
9	112.5	109.8	2.45	114.0	109.92	3.71

Figures (5.16), (5.17), and (5.18) represent the phreatic line position pressure and total head contour lines for (H=144.29 m). Figures (10), (11), and (12)(see appendix A) represent the phreatic line position, pressure and total head contour lines, for (H=134.91 m).

5-3 Effect of Degree of Anisotropic Soils

Most earth dams are anisotropy conditions because the compaction operation makes the permeability coefficient in the horizontal direction (k_x) greater than the permeability coefficient in the vertical direction (k_y). Even the taken soil from regular region, the ratio of horizontal to vertical coefficient of permeability $\frac{k_x}{k_y}$ was found to be about 4 and may reach to (20) or greater [Taylor and Brown ,(1967)].

Therefore, most of the previous analytical methods fail to solve this type of problems because the most basic theories for analytical solution assume that the soil is isotropic and the laboratory methods are found so difficult in solving this type of problems.

For these reasons, it is essential to use one of the numerical methods, especially the finite element method, which has been used in this work to solve this problem.

Al-Qadisiya Dam was tested by applying the computer program on one station of the dam (station 29) for H=133.89 m.a.s.l in U/s and zero water level in D/s for seven times, the horizontal coefficient of permeability k_x is taken constant while the vertical permeability k_y was specified so that the ($\frac{k_x}{k_y}$) ratio is reduced to (5, 10, 15, 20, 25, 30).

Figure (5-15)

Figure (5-16)

Figure (5-17)

Figure (5-18)

Two points were examined in the field media one point at $x=140.1$ m before the Asphaltic concrete diaphragm and the other is at $x=318.68$ m in the D/S shell. It has been noticed that the location of the free surface in point (1) moved up when the k_x/k_y ratio increased, while the free surface was dropped in point (2) due to the existing drain in the D/S shell, the D/S drain system is very important in drawing down the free surface and decreasing pressures inside the dam body.

Figures (5.19), (5.20), (5.21), (5.22), (5.23), (5.24) and (5.25) indicate these results. The head gradient in the D/S shell behind the curtain was reduced and seepage quantity is also reduced when the $\left(\frac{k_x}{k_y}\right)$ ratio increased, figure (5.26), this is due to that the horizontal coefficient of permeability assumed to be constant and when the vertical permeability is reduced. The permeability coefficient shall be reduced in its general meaning causing less seepage discharge.

5.4 Importance of Using the Grout Curtain and the Asphalt Concrete Diaphragm

To show the effect of existing the grout curtain and the Asphaltic concrete diaphragm , the computer program was applied to Al-Qadisiya Dam at station (46) for a head level equal to (135.29 m.a.s.l) and the same material properties of table (5.2) for the following two cases :-

A-With the existence of grout curtain and Asphaltic concrete diaphragm.

B-Without curtain and Asphaltic concrete diaphragm.

Figure (5-19)

Figure (5-20)

Figure (5-21)

Figure (5-22)

Figure (5-23)

Figure (5-24)

Figure (5-25)

Figure (5-26)

Comparing the results of the above two cases, it can be seen that the seepage quantity in case (A) equals to 0.67 l/s/m while in the case (B) is 0.81 l/s/m i.e., the seepage flow logically increased in removing the grout curtain and Asphaltic diaphragm. Also, the head gradient behind the curtain is less than that in case (B). And both the pressure distribution and total head in the dam body behind the curtain are less than that in case (B). Figures (5.27), (5.28), and (5.29) show the results.

Figure (5-27)

Figure (5-28)

Figure (5-29)

5.5 Checking Adequacy of the Grout Curtain

In order to check the adequacy and performance of the grout curtain under the dam, the following two clues may serve as indications to evaluate the significant change in the pattern of seepage flow through the dam foundation (such pattern of seepage flow is given by the designer of the dam in the maintenance and operation manual) [Hydroprojckt, (1988)]. These clues are :-

- 1-The sharp drop in water levels between piezometers 1 and 2 ; 3 and 4 ; 5 and 6 could be a good clue indicating an acceptable performance of the grout curtain.
- 2-The sharp increase in the differential water levels between piezometers 4 and 8 indicates an increase in the seepage flow velocities and its gradients in the dam foundation within the limits of the down streams shell.

Referring to table (5.1) and figure (5.1), the piezometers 1, 3 and 5 are installed at a different level adjacent to the upstream side of the grout curtain. The piezometers 2 , 4 and 6 are installed at different levels on the downstream side of the grout curtain. The piezometers 1 and 2 ; 3 and 4 ; 5 and 6 ; 4 and 8 are installed approximately at same level (Formation).

Also, as shown in figure (5.1), the path of seepage between piezometers 1 and 2 is larger than the distance between piezometers 3 and 4. Therefore, the head difference between piezometers 1 and 2 should be greater than the difference between piezometers 3 and 4.

According to the two points given above , there is a significant difference between the upstream and downstream piezometric level [measured], at the selected stations 46, 42, 29, 33+50 which observed during the year 2000. This is given in table (5.8).

Table (5-8)

Also, the grout curtain is checked by comparing the piezometric levels of the downstream shell (piezometers No. 2 , 4, 6, 7, 8, and 9) with a results of finite element method. Table (5-9) gives a comparison between the results of the finite element method and those observed in the dawn stream for the available data. The results in the above table show generally a good agreement between the F.E.M method results and the piezometric readings. This also indicates that the grout curtain is in good condition.

Table (5-9) Results of water table using F.E.M. and the piezometric reading of the selected stations for different U/s water levels

stations	Pizometr No	F.E.M		Piezometr Reading		F.E.M		Piezometr Reading	
		U/S= 135.29 m [31/1/2000]				U/S= 144.53 m [22/4/2000]			
		W.L (m)	Head percent (%)	W.L (m)	Head percent (%)	W.L (m)	Head percent (%)	W.L (m)	Head percent (%)
46	2	116.3	45	122.52	63.8	120.0	44.91	127.42	61.57
	4	116.33	46	117.62	50	120.05	45.02	121.26	47.75
	7	114.5	41	112	34	116.5	37.0	116.19	36.35
	8	110.55	30	108.82	24.9	110.05	22.56	109.72	21.82
	9	109.4	26	108.99	25.47	107.50	16.80	109.72	21.82
		U/S= 133.55 m [3/12/2000]				U/S= 138.09 m [23/9/2000]			
42	2	112.0	35.7	110.4	30.99	114.8	36	111.4	30
	4	112.5	37.25	110.6	31.59	114.0	36	111.6	30.5
	7	110.5	30.9	104.23	21.60	110.9	28.6	104.66	12.3
	8	105.5	16.39	103.57	10.64	105.5	14.4	103.81	10.0
	9	-	-	-	-	-	-	-	-
		U/S= 133.89 m [21/12/2000]				U/S= 140.13 m [22/8/2000]			
	2	117	50.1	115.11	44.58	122.5	54.30	116.73	41.68

29	4	117.5	51.6	116.69	49.29	122.5	54.30	118.52	46.18
	7	113.5	39.8	112.06	35.40	118.5	46.1	111.96	29.8
	8	-	-	-	-	-	-	-	-
	9	110.30	30	110.78	31.80	114.5	36.13	110.93	27.23
		U/S= 134.91 m [11/1/2000]				U/S= 144.29 m [8/3/2000]			
33 + 50	2	116.5	47.26	114.85	42.53	121.0	47.5	117.07	38.38
	4	116.5	47.26	113.25	37.95	121.20	47.86	115.15	34.20
	7	115.2	43.55	110.46	30	119.8	44.7	110.76	24.29
	8	113.2	37.8	110.01	28.64	114.8	33.4	110.21	23.0
	9	112.5	35.8	109.8	28.0	114.0	31.6	109.92	22.39

Chapter Six

Conclusions and Recommendations

6.1 Conclusions

In this research, a finite element method has been used for the theoretical analysis of the unconfined seepage and a comparison has been made with the field data of a case study concerning AL-Qadisiya Dam, the following conclusions have been indicated.

1. Linear triangular with three nodes is used in order to simulate 2-D seepage flow problem. It is suitable one when modeling a hydraulic structure of a multi layered system. Considering that it has been given a higher error in head calculation when there is a rapid change in the head values because the interpolation function is linear.
2. The computer program “Seep2D” and the used linear triangular element have been given acceptable results to solve the existing case.
3. In order to select the best mesh, which can be used for the finite element model for accurate results several trials must be done to comply with the existing condition of AL-Qadisiya dam. Small elements must be used in areas of rapid changes (regions of grout curtain and the asphaltic diaphragm) and large elements must be used where these changes are less severe (regions of the U/S and D/S shell).
4. The mesh used in the finite element computer program required variable time to access, the finest the mesh the more time required , therefore, a mesh configuration has presented in chapter five (in both the laboratory model and prototype dam), this technique was useful for both results accuracy and execution time.

5. The head values have more influence with the properties of each consistent material (coefficient of permeability k_x , k_y). These properties caused a various values of head in all directions around these elements and then the parameters such as (pressure head, total head ...etc.) which is produced from the values of head is not equal around these elements so that the distribution of head occurs through these materials not uniformly but represent as a contour lines which is connected the same value of head for each line.
6. By using the results of finite element procedure in describing some parameters for unconfined seepage problem (seepage through and under Al-Qadisiy Dam), The followings have been observed.
 - The effect of degree of anisotropic $\frac{k_x}{k_y}$ for the construction materials show that in reducing permeability of the materials shall cause the free surface to move up in a higher position and increase pore water pressure a case which may cause failure in dam due to piping, However, using a D/S drainage system shall reduce this danger since pore water pressure will be decreased. Also, the seepage discharge was decreased as the ratio of $\frac{k_x}{k_y}$ increased.
 - Removing the grout curtain and asphaltic concrete diaphragm, sharp drop in total head or (Gradually drop) disappear, and increasing the pressure in the D/S shell which might cause failure due to piping, Howeve, using of these antiseepage has a necessity in this aspect.
 - Analyzing the results concluded that the adequacy of the grout curtain under the dam showed generally a good condition for both left and right stations, at least for the selected stations .

6.2 Recommendations

For future researches and farther studies, the following recommendations could be made :-

1. The program seep2d may be extended to solve unsteady flow conditions.
2. Extending the present model to analyze the case of three dimensional flow.
3. Extending both the present model and the computer program to analyze the effect of earth-quake and dam stability.
4. Material Properties of the dam body of foundation are more effective on the head distribution and other parameters. Therefore, it needs a large number of experimental work and full tests to find the soil properties more accurately in order to option a correct simulation of the material in the mathematical model.

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Finite Element Solution for Unconfined Seepage Problem with Reference to AL-Qadisiya Dam.

(Finite Element Galerkin Method)

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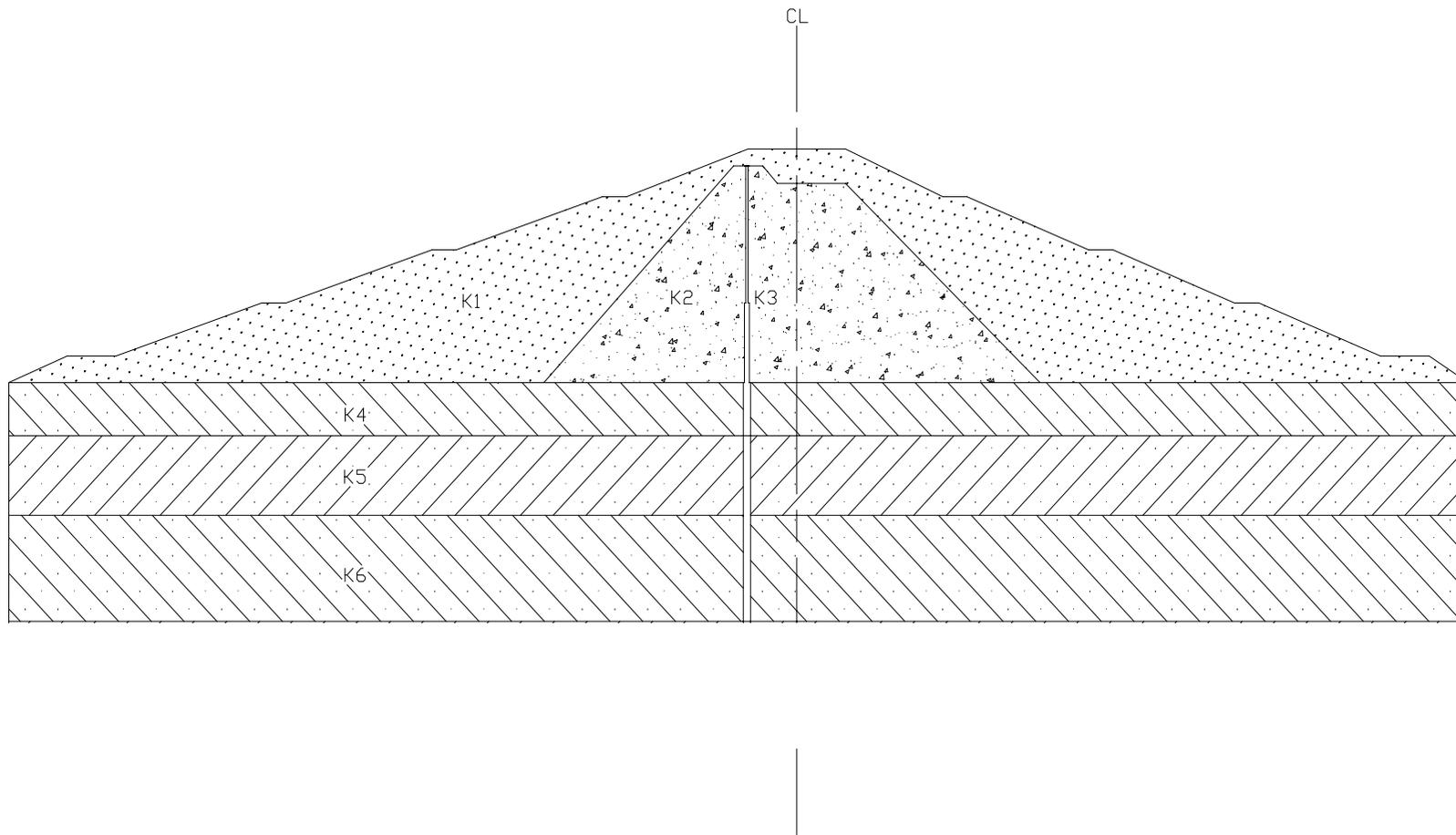


Figure (5-2); Typical cross-section of AL-Qadisiya Dam at station 46

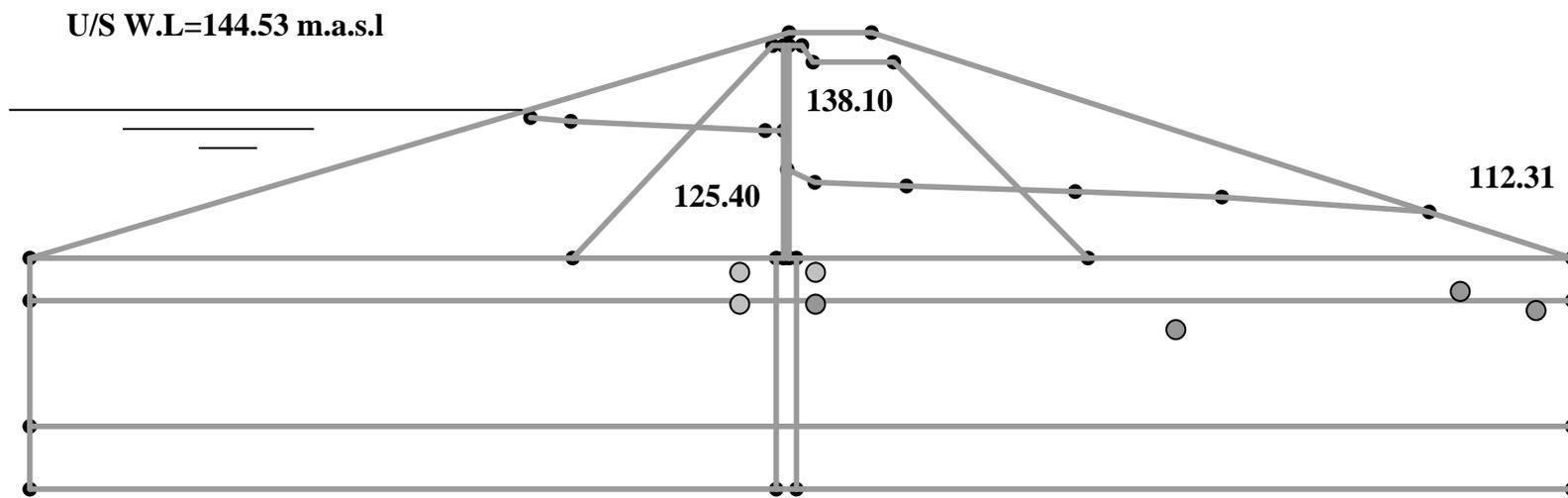
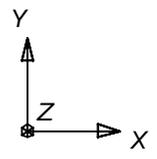


Figure (5-4). Phreatic surface position of AL-Qadisiya Dam at station 46 after 22 iteration



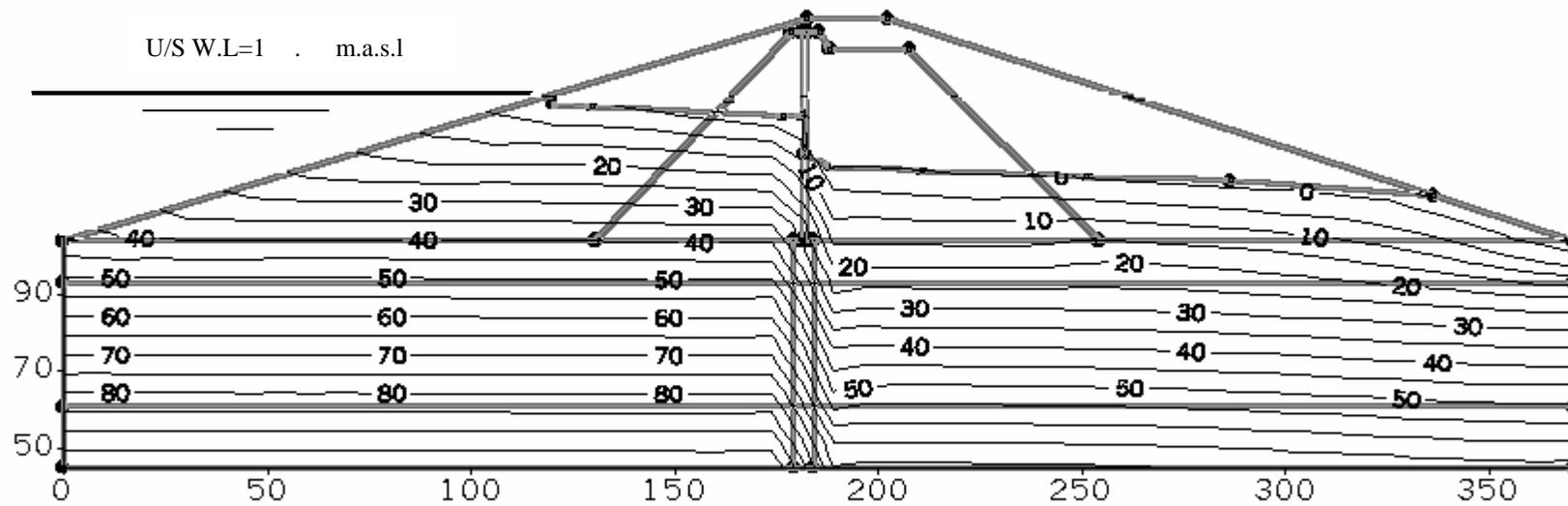
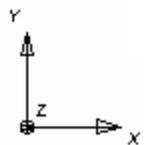


Figure (5.5): Pressure head contour lines of Al-Qadisya dam at station 46.



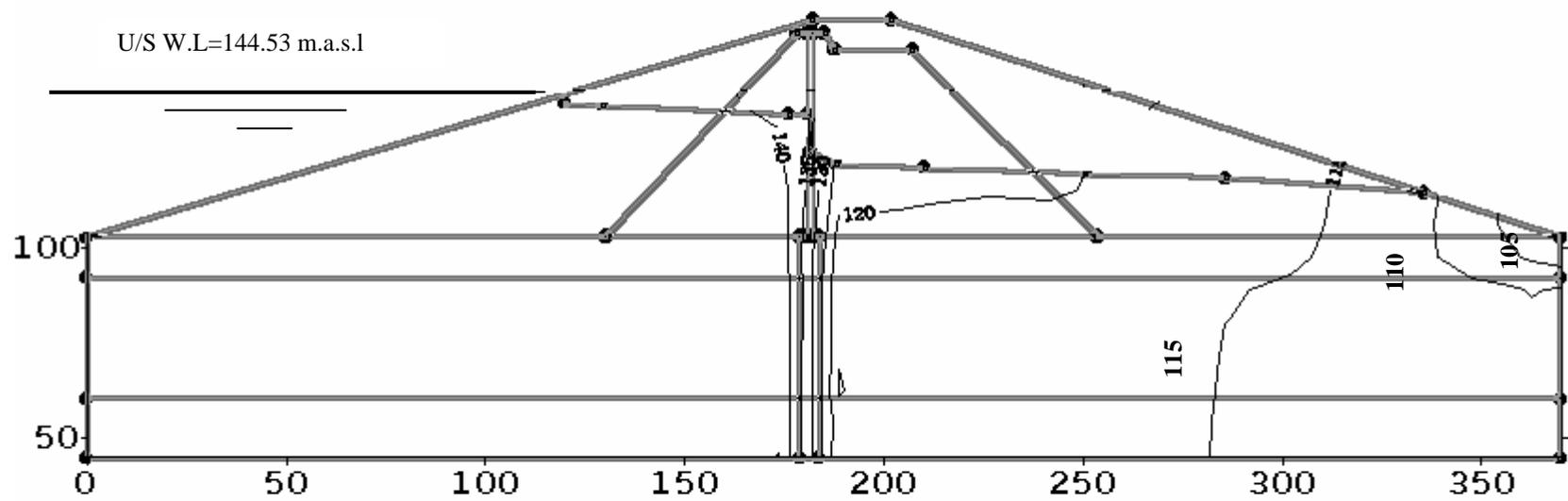


Figure (5.6) : Total head contour lines of Al-Qadisiya dam at station 46.

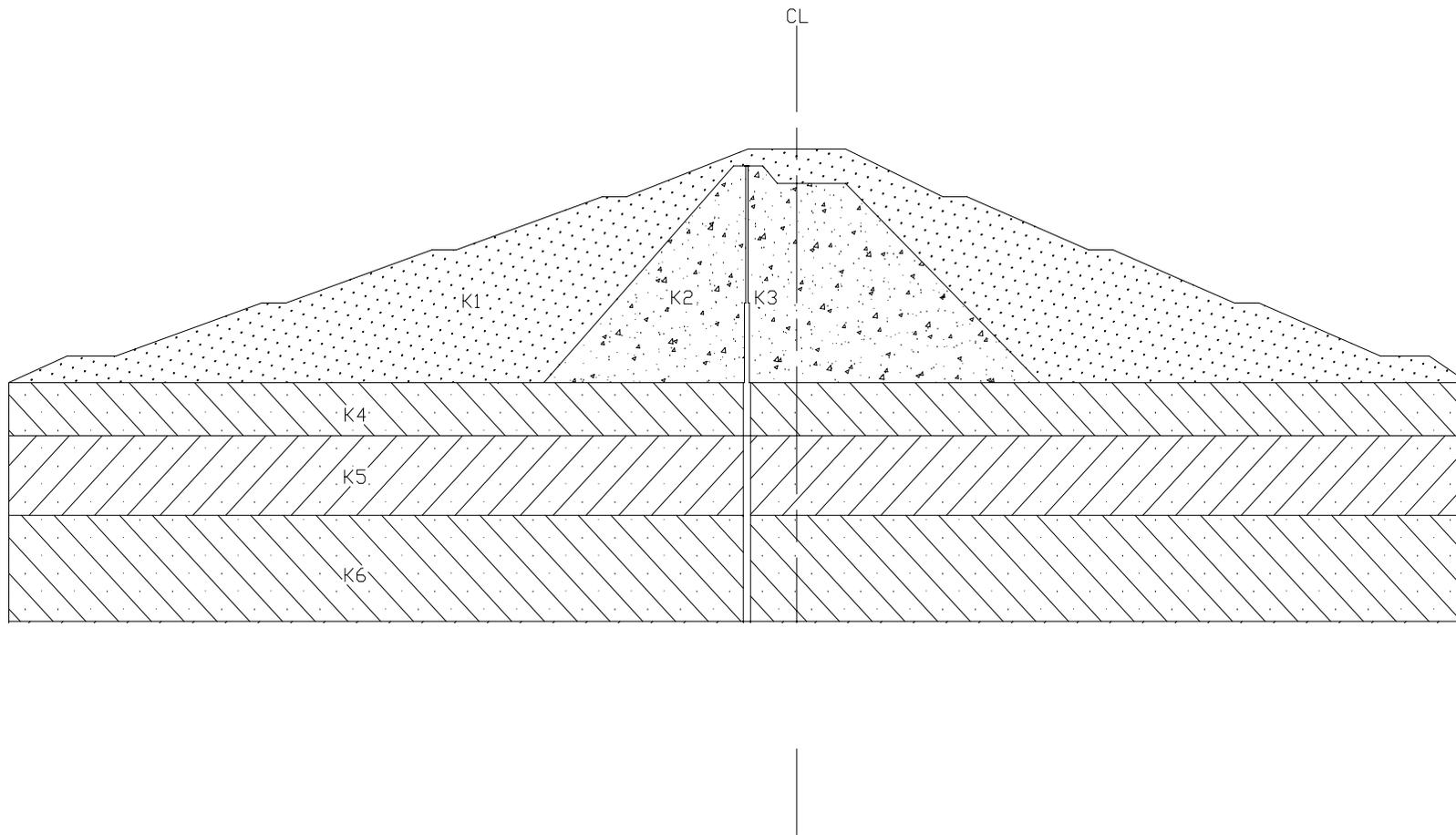


Figure (5-7); Typical cross-section of AL-Qadisiya Dam at station 42

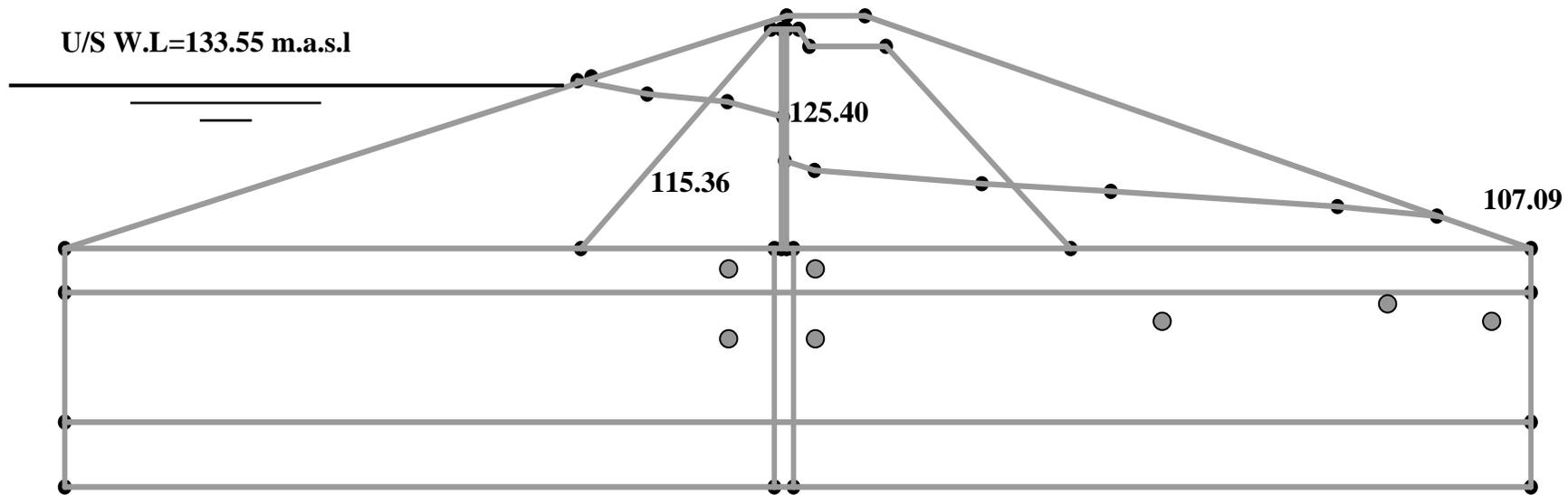
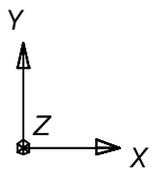


Figure (5-8). Phreatic surface position of AL-Qadisiya Dam at station 42 after 17 iteration



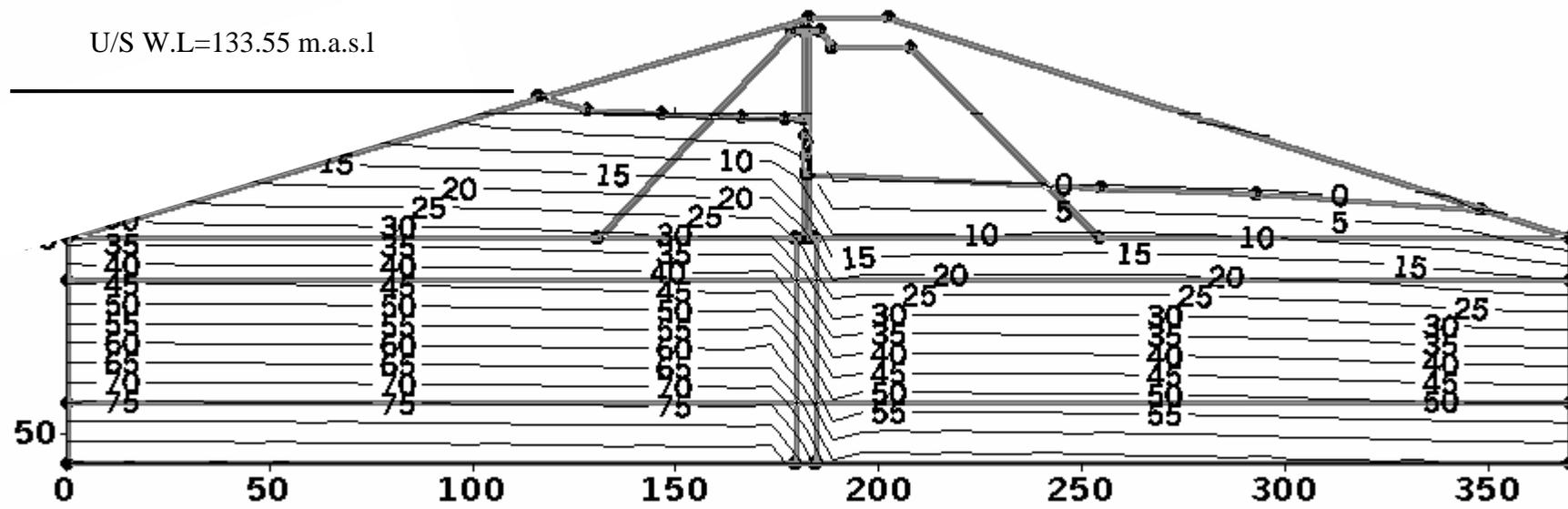
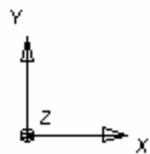


Figure (5.9): Pressure head contour lines of Al-Qadisiya dam at station 42.



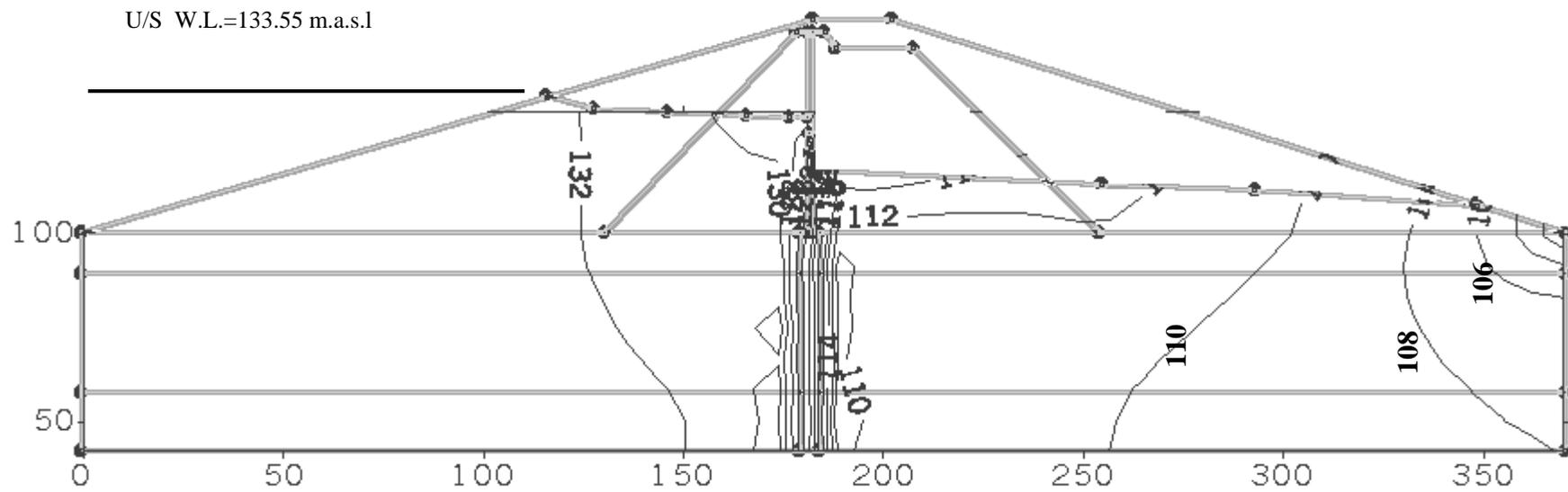
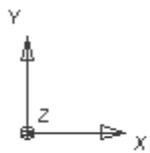
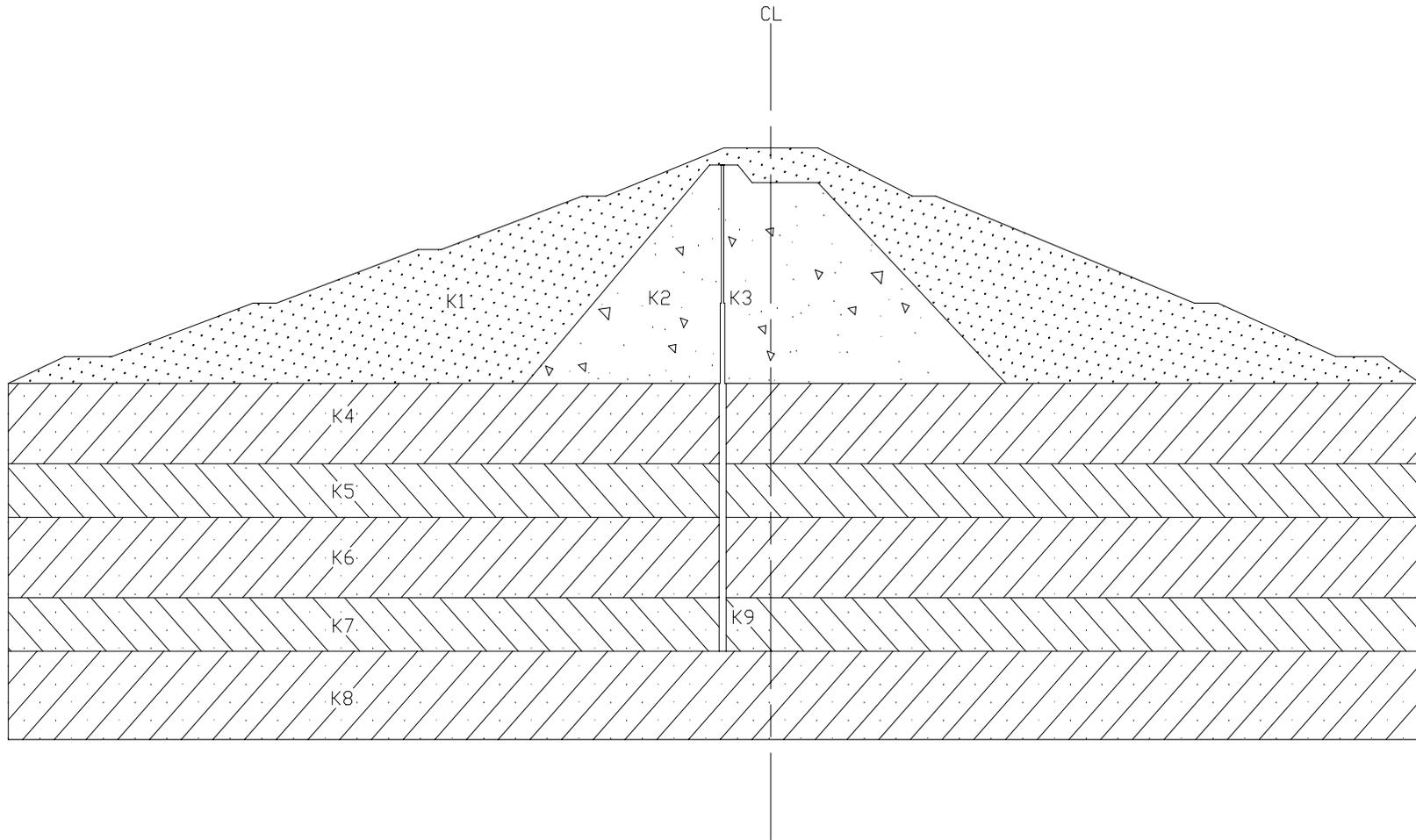


Figure (5.10): Total head contour lines of Al-Qadisiya dam at station 42.





Figure(5-11); Typical cross-section of AL-Qadisiya Dam at station 29

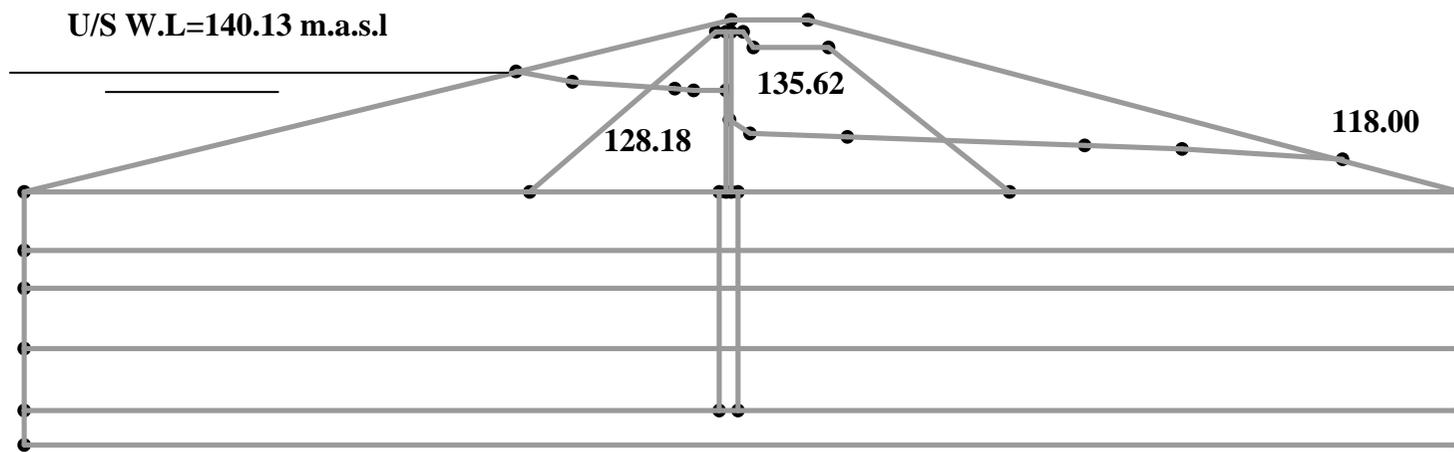
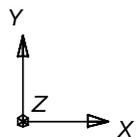


Figure (5-12). Phreatic surface position of AL-Qadisiya Dam at station 29 after 22 iteration



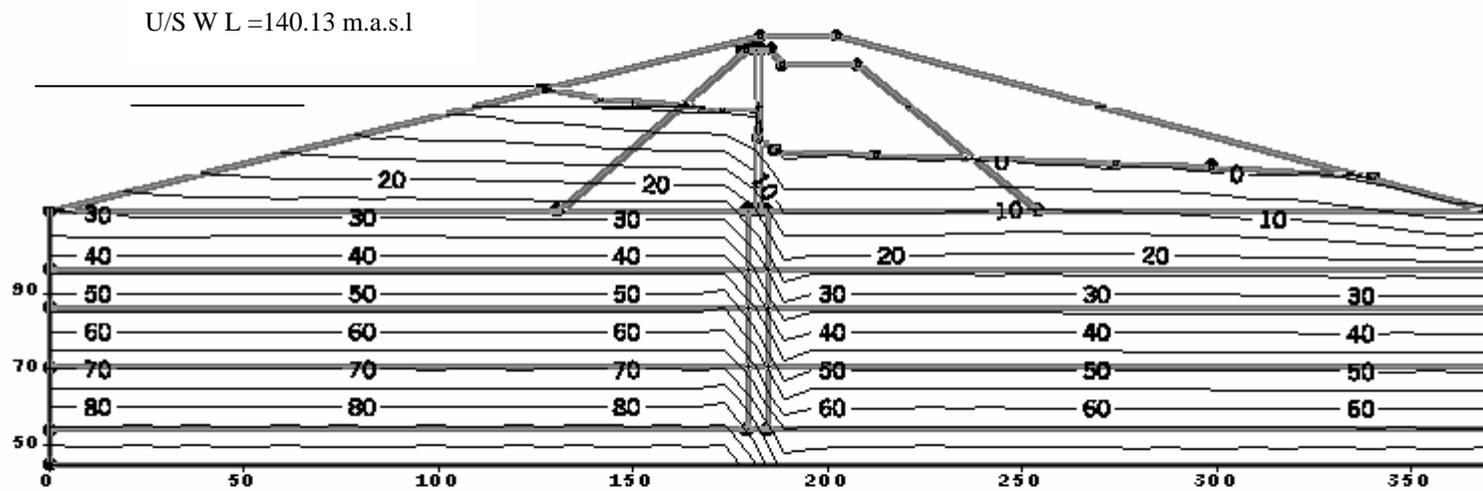
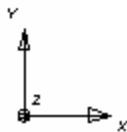


Figure (5.13) : Pressure head contour lines of Al- Qadisiya dam at station 29.



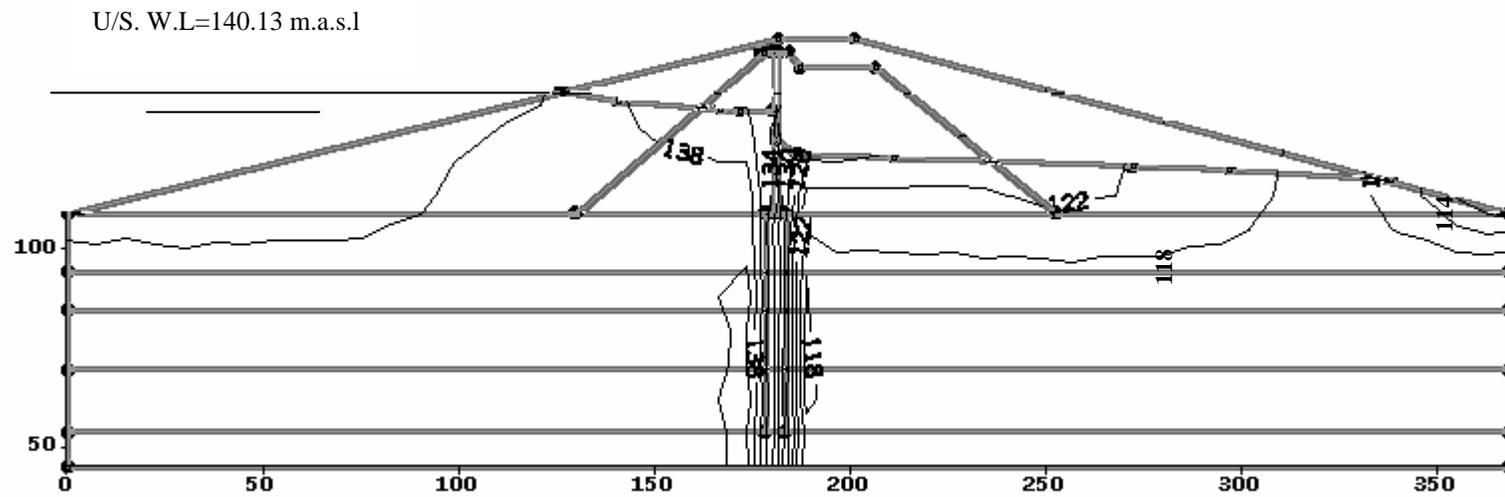
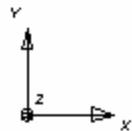


Figure (5.14) : Total head contour lines of Al- Qadisiya dam at station 29.



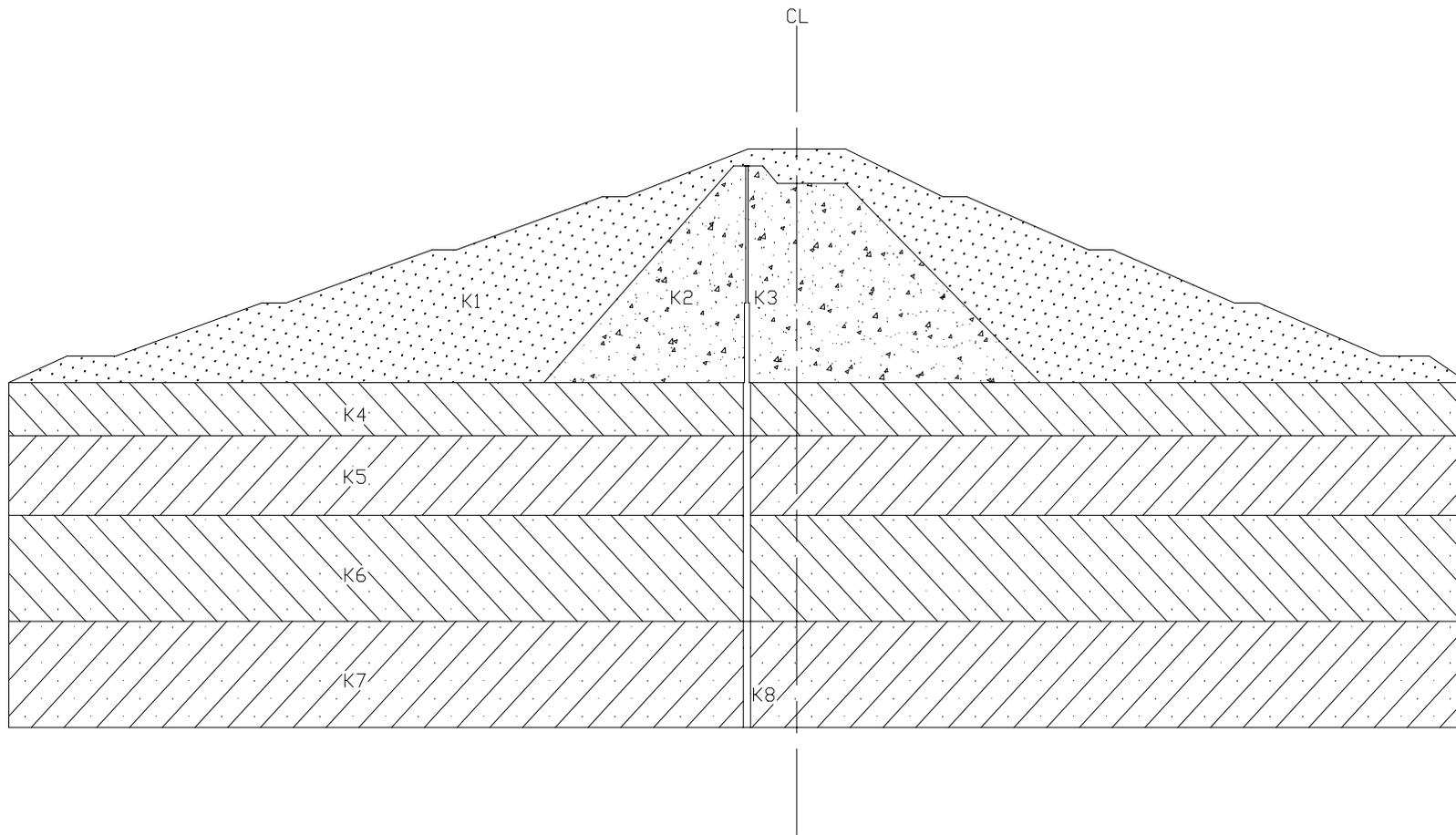


Figure (5-21); Typical cross-section of AL-Qadisiya Dam at station 33+50

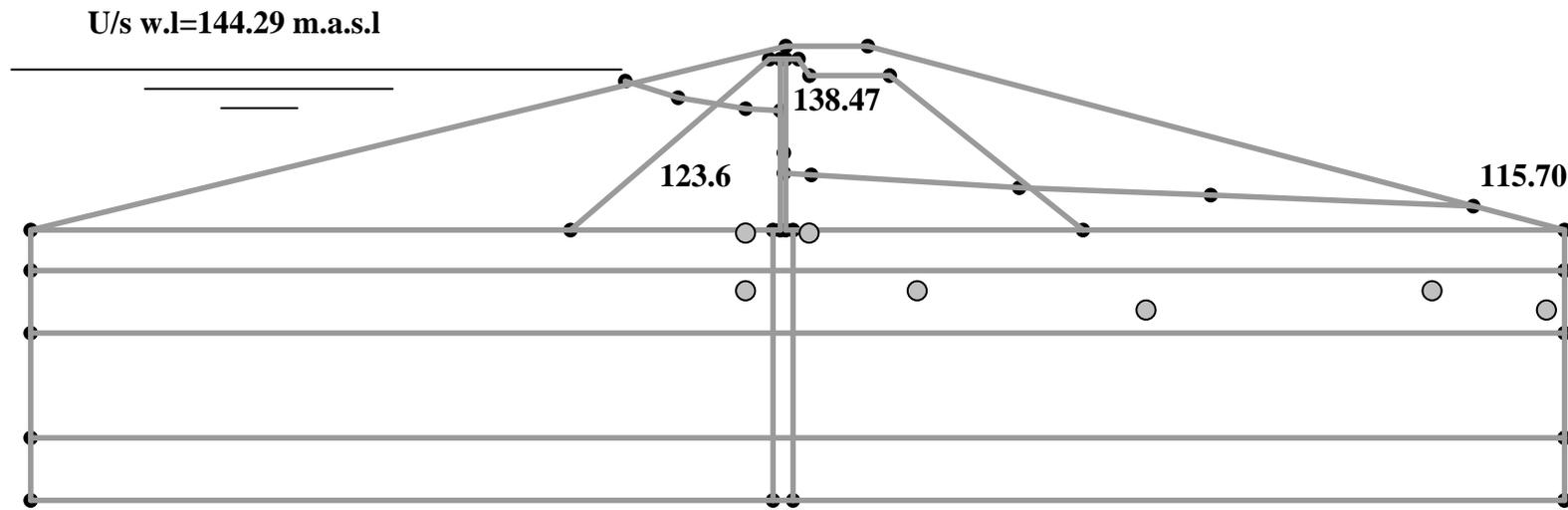
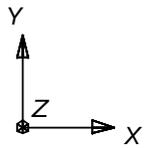


Figure (5-16). Phreatic surface position of AL-Qadisiya Dam at station 33+50 after 25 iteration



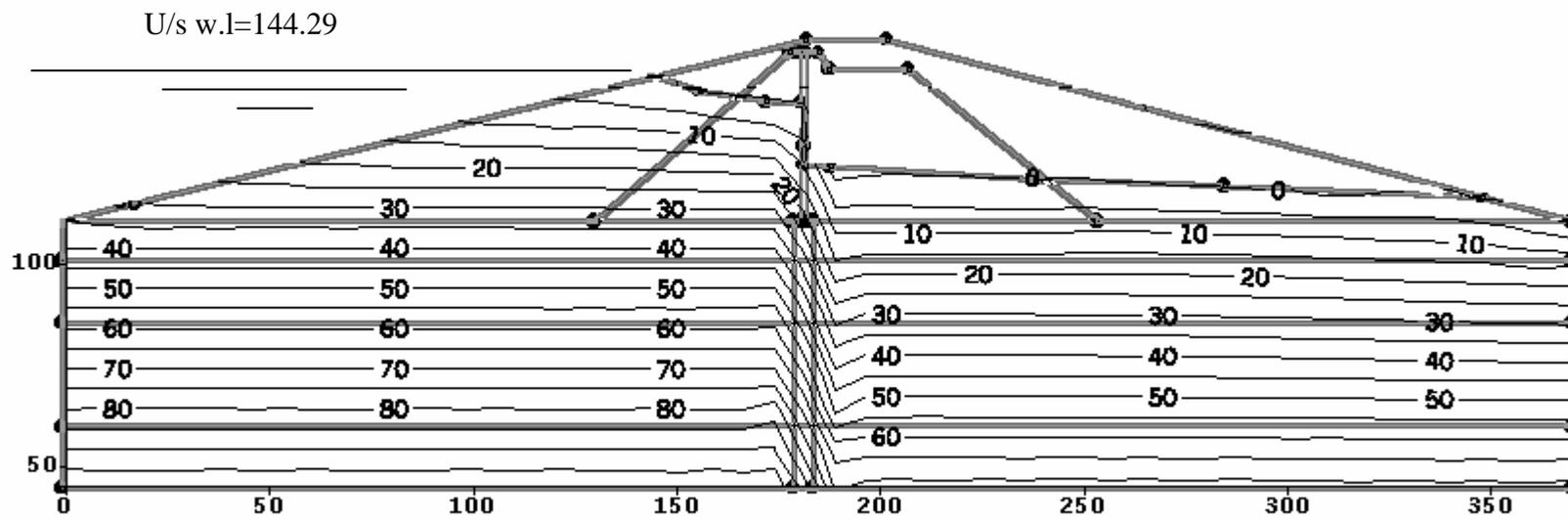
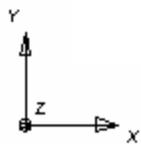


Figure (5.17) : Pressure head contour lines of Al- Qadisiya dam at station 33+50.



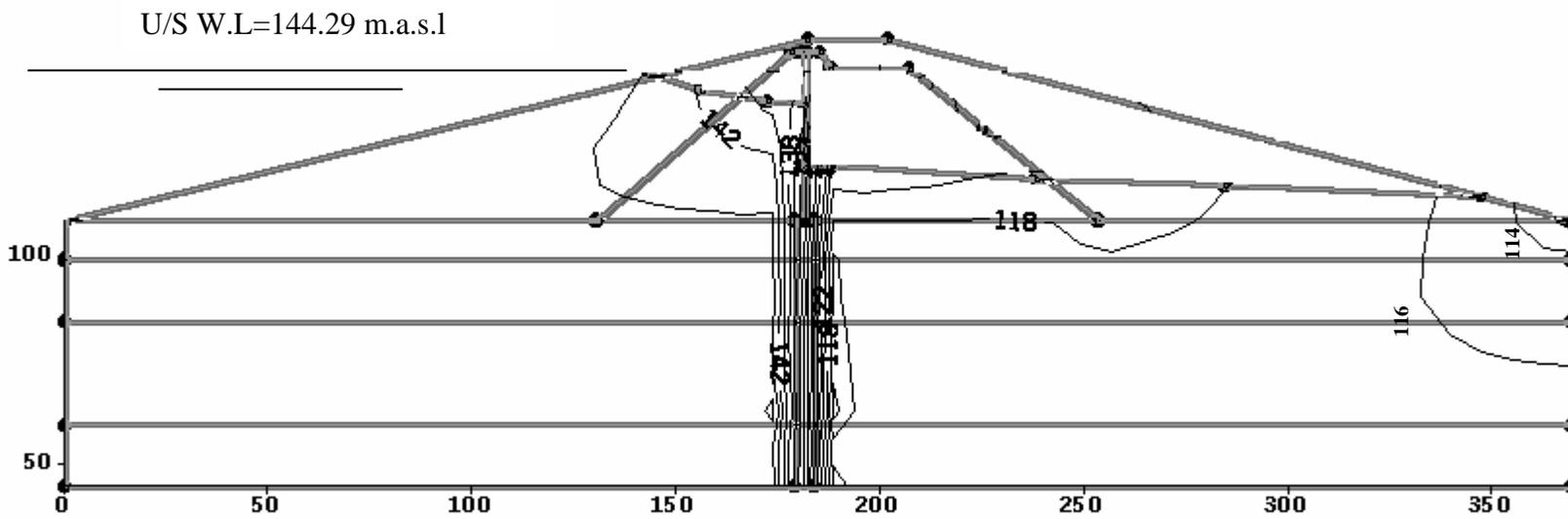


Figure (5.18): Total head contour lines of Al-Qadisiya dam at station 33+50.

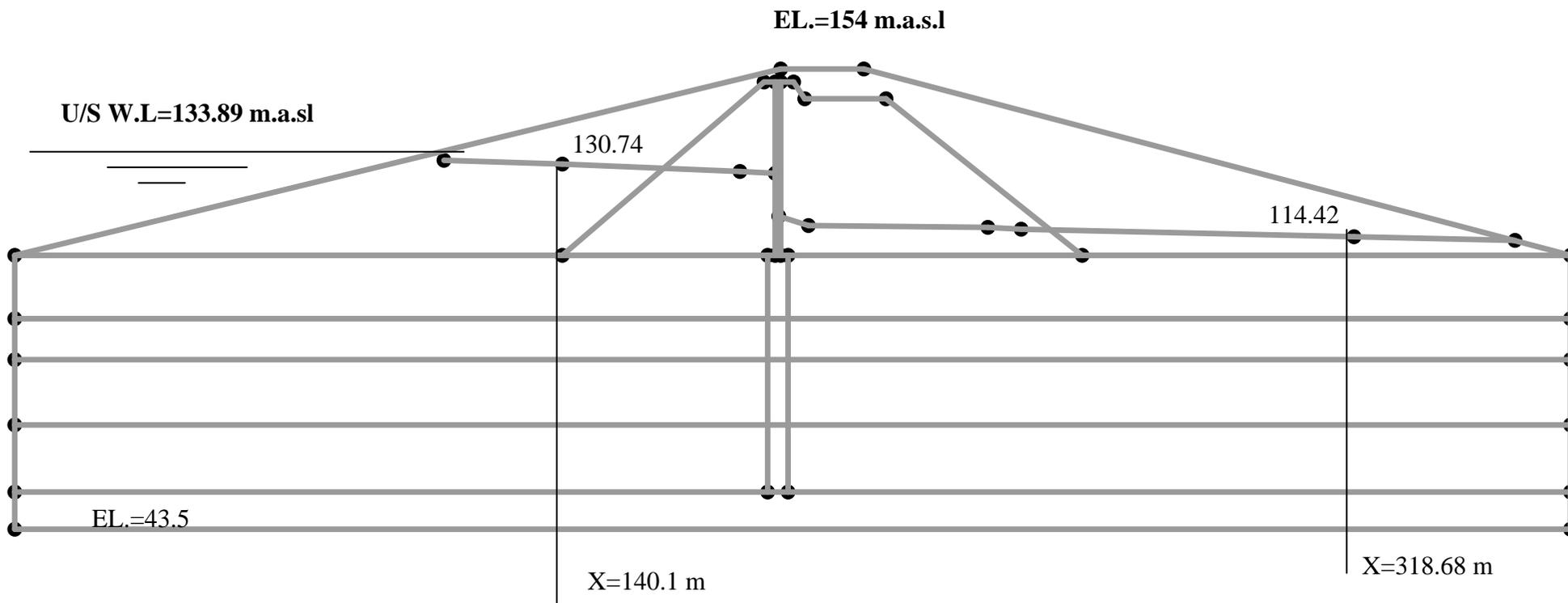
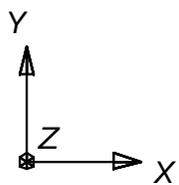


Figure (5.19) Free surface position of st.29 for $k_x/k_y=1$



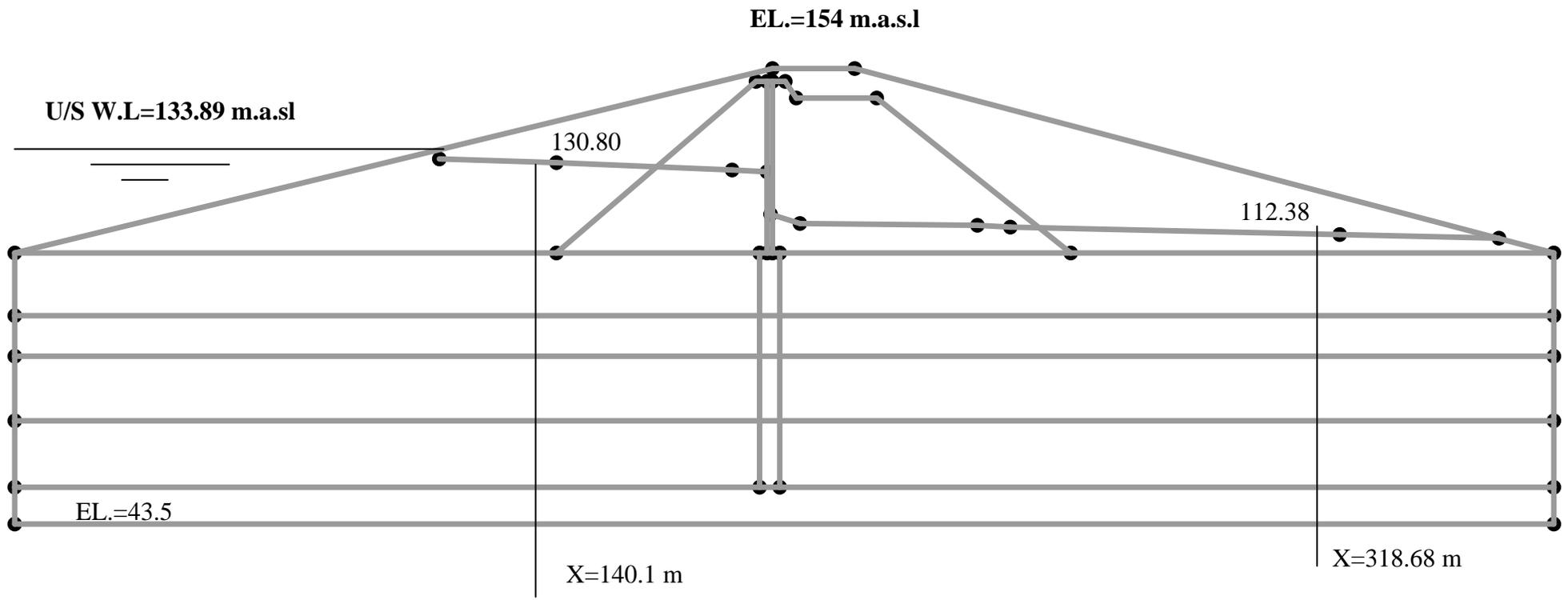
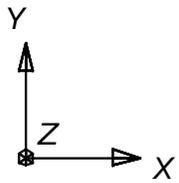


Figure (5.20) Free Surface Position of St.29 For $KX/KY=5$



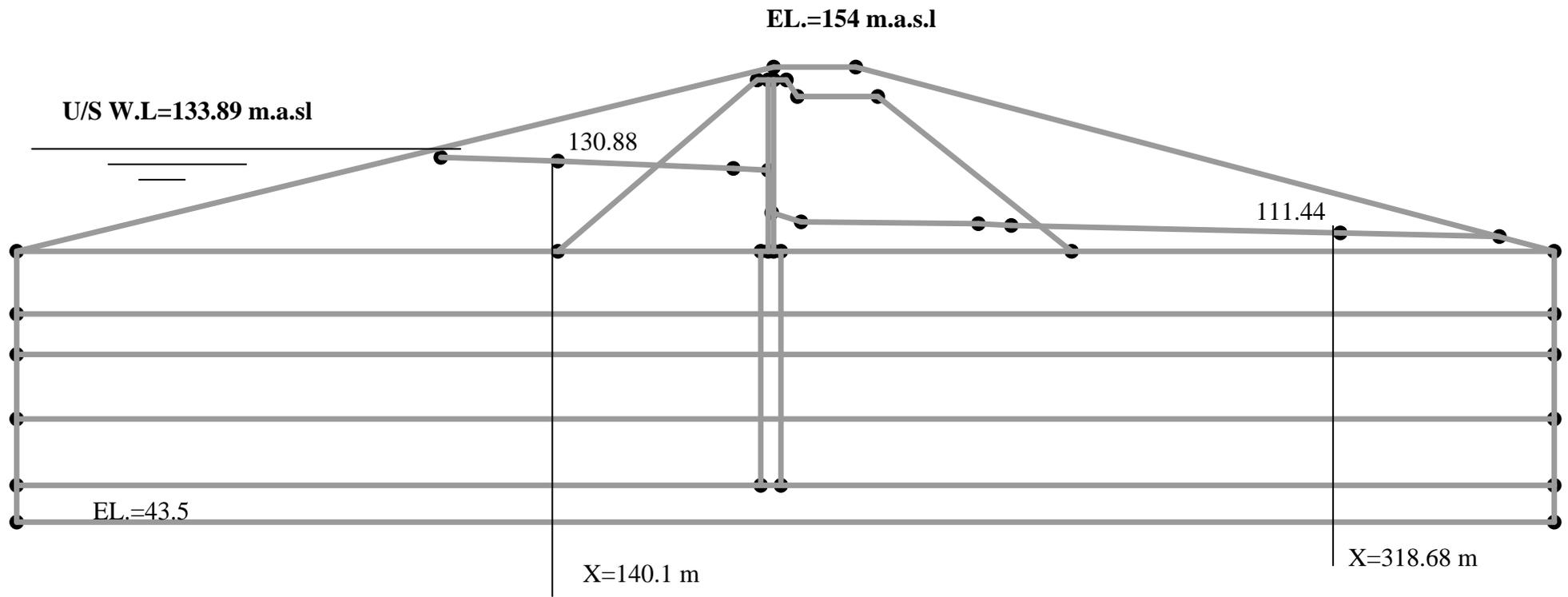
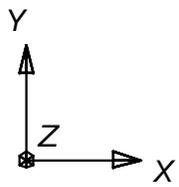


Figure (5.21) Free Surface Position of St.29 For $KX/KY=10$



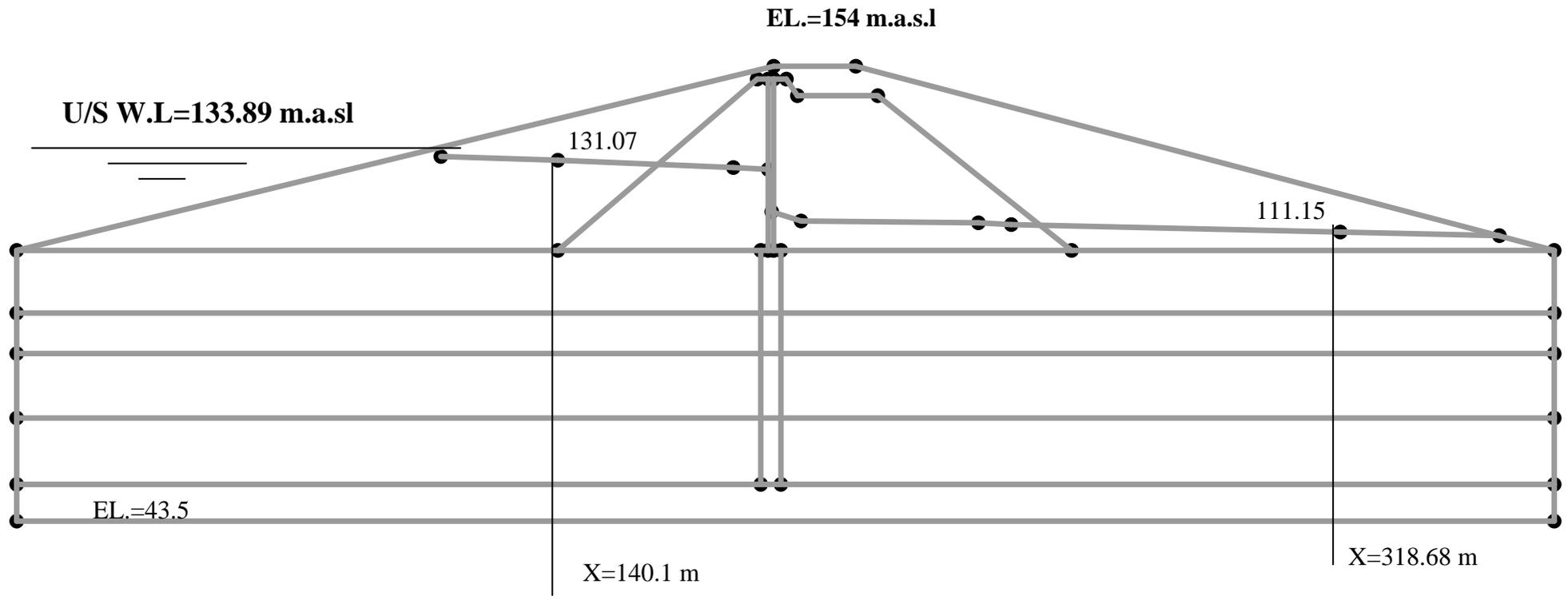
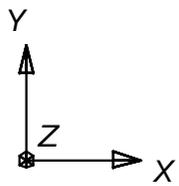


Figure (5.22) Free Surface Position of St.29 For $KX/KY=15$



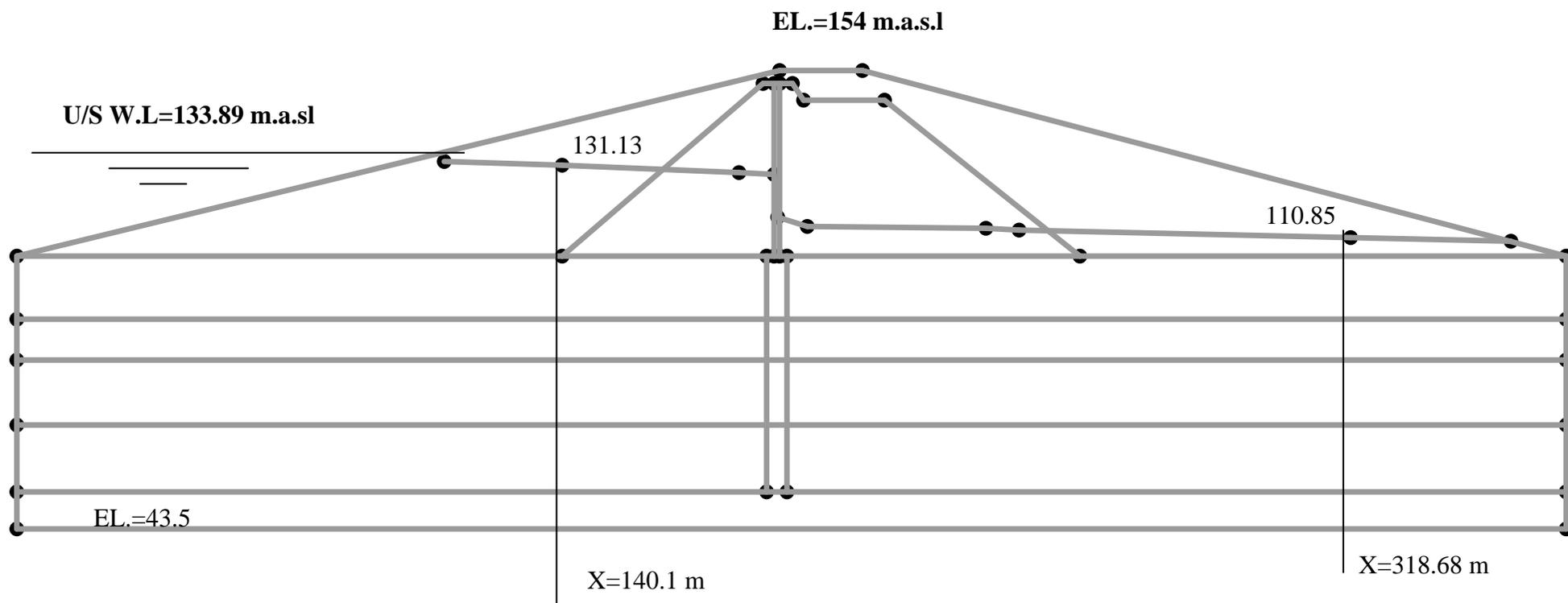
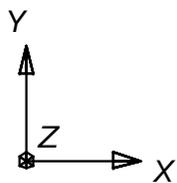


Figure (5.23) Free Surface Position of St.29 For $KX/KY=20$



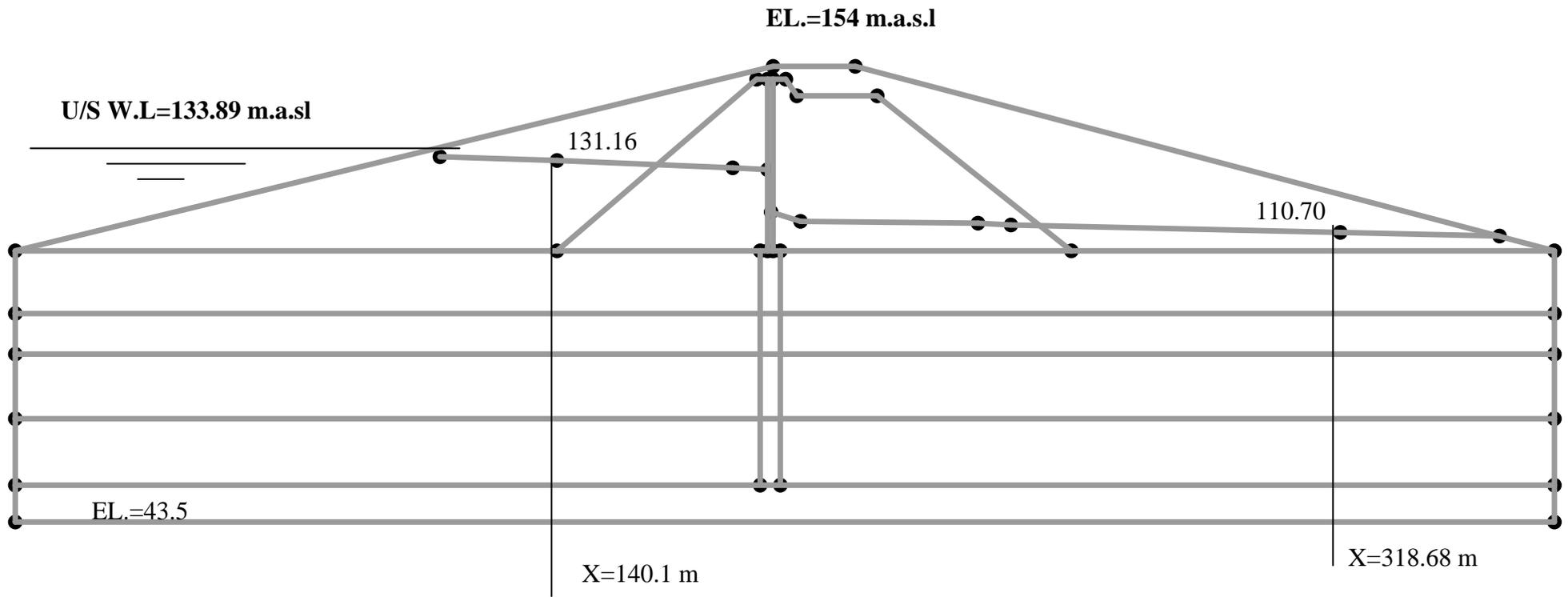
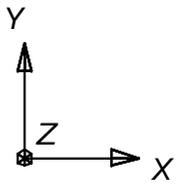


Figure (5.24) Free Surface Position of St.29 For $KX/KY=25$



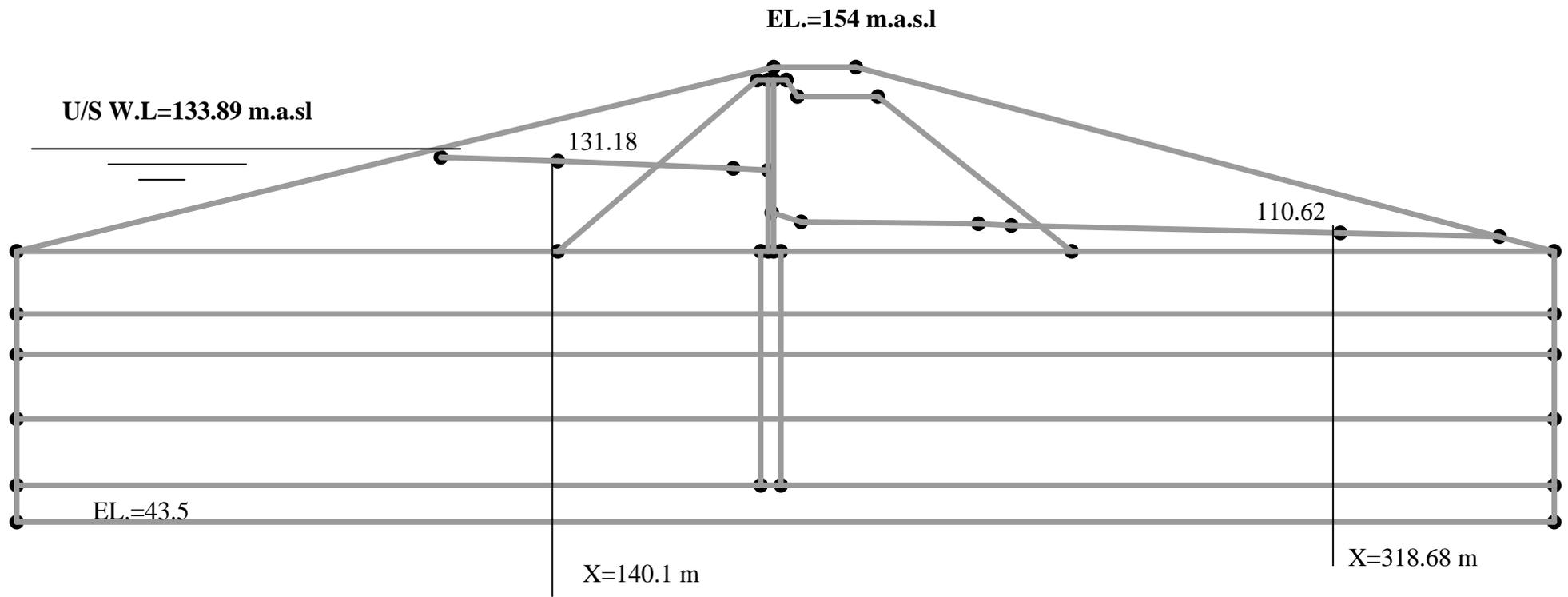
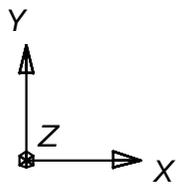


Figure (5.25) Free Surface Position of St.29 For $KX/KY=30$



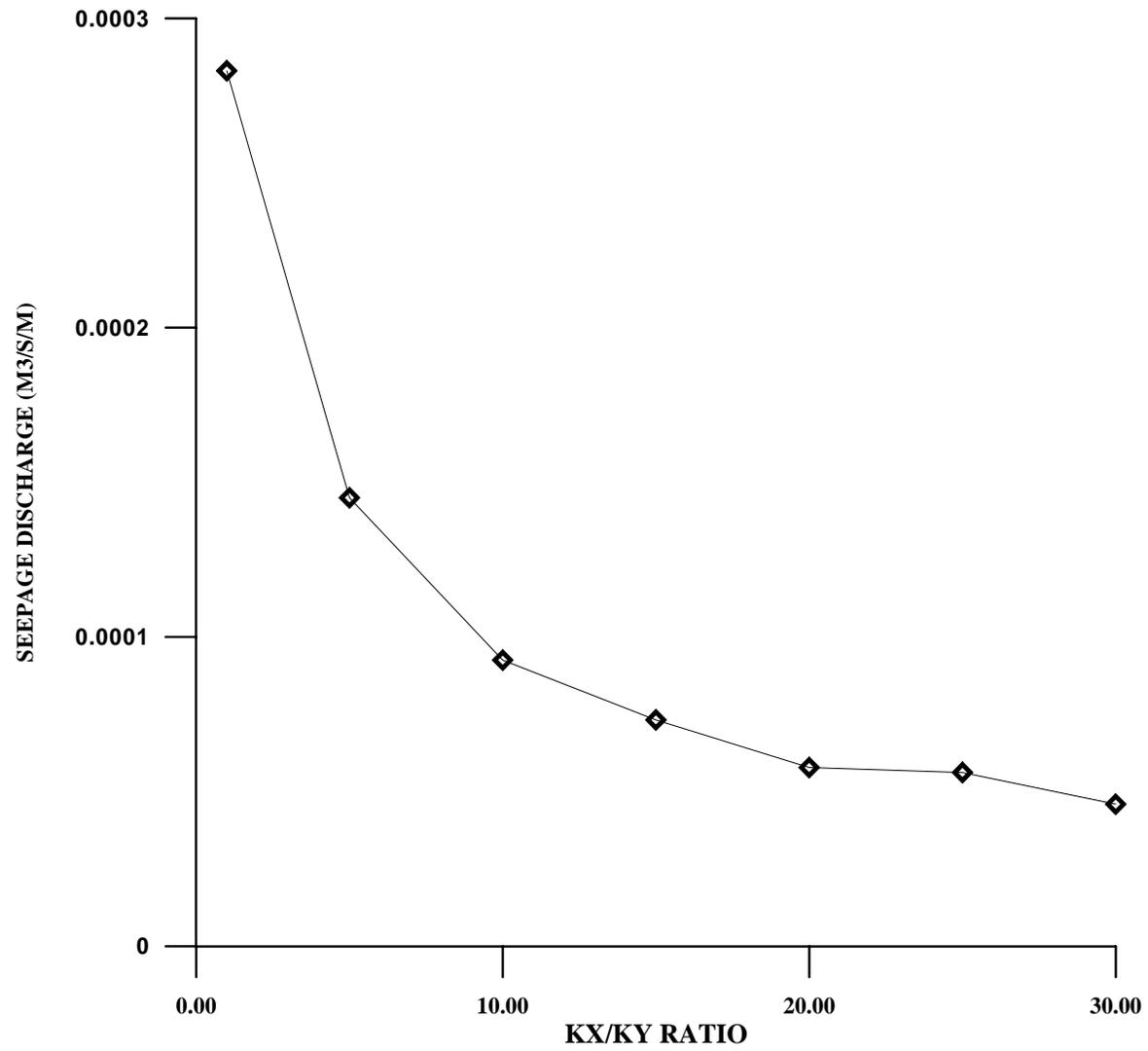


Figure (5-26).Relation between seepage discharge and kx/ky ratio

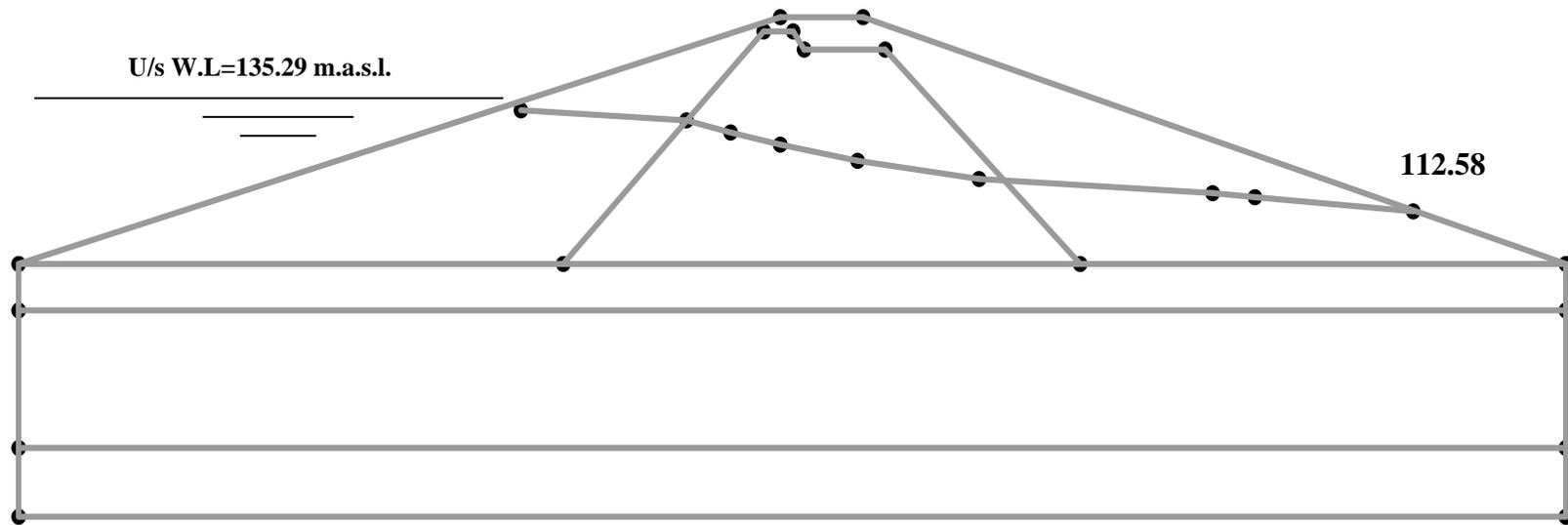
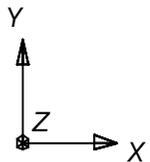


Figure (5-27). Phreatic surface position of AL-Qadisiya Dam at st. 46 without grout curtain and asphaltic concrete diaphragm



U/s w.l.=135.29 m.a.s.l

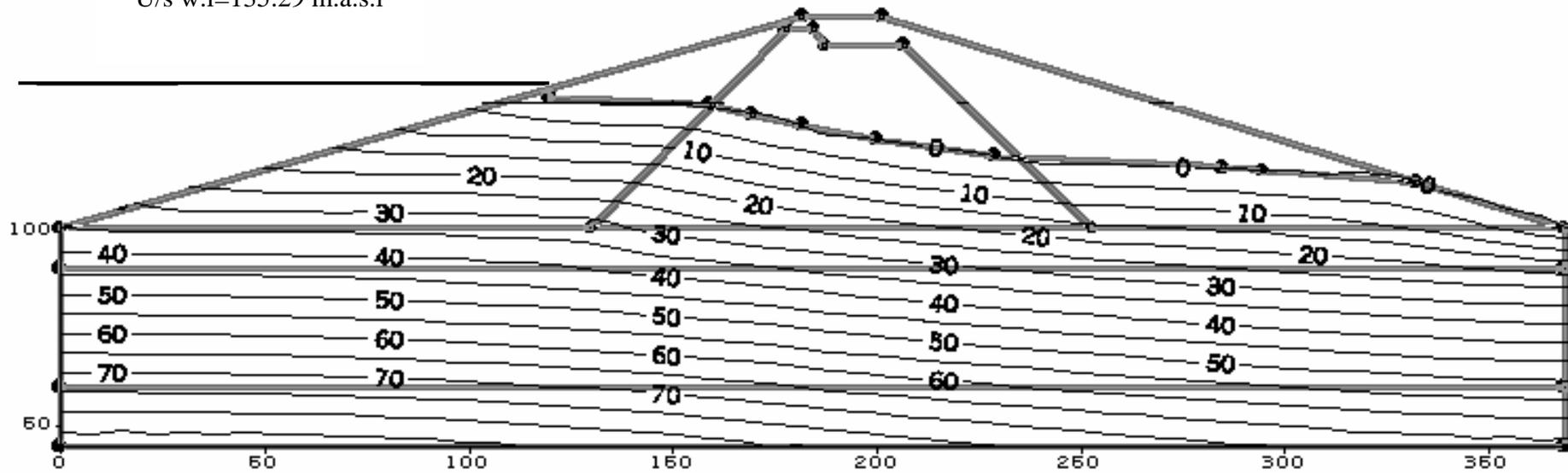
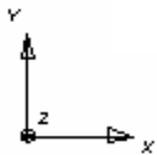


Figure (5.28) : Pressure head contour lines of station 46 without grout curtain and asphaltic concrete diaphragm



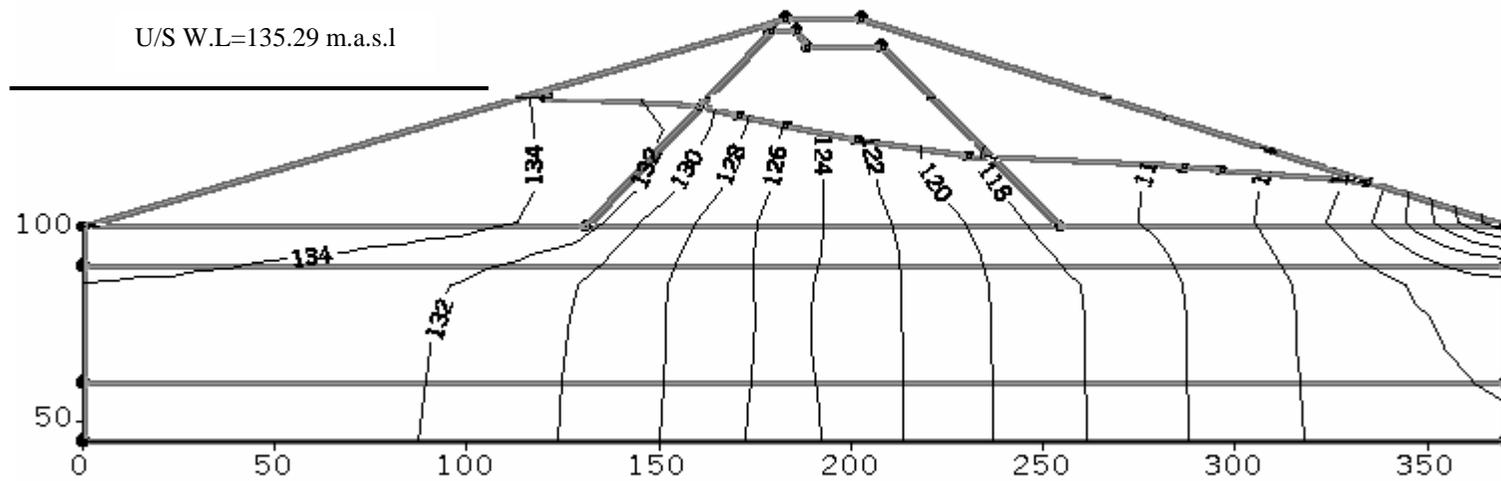
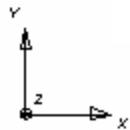


Figure (5.29): Total head contour lines of station 46 without grout curtain and asphaltic concrete diaphragm



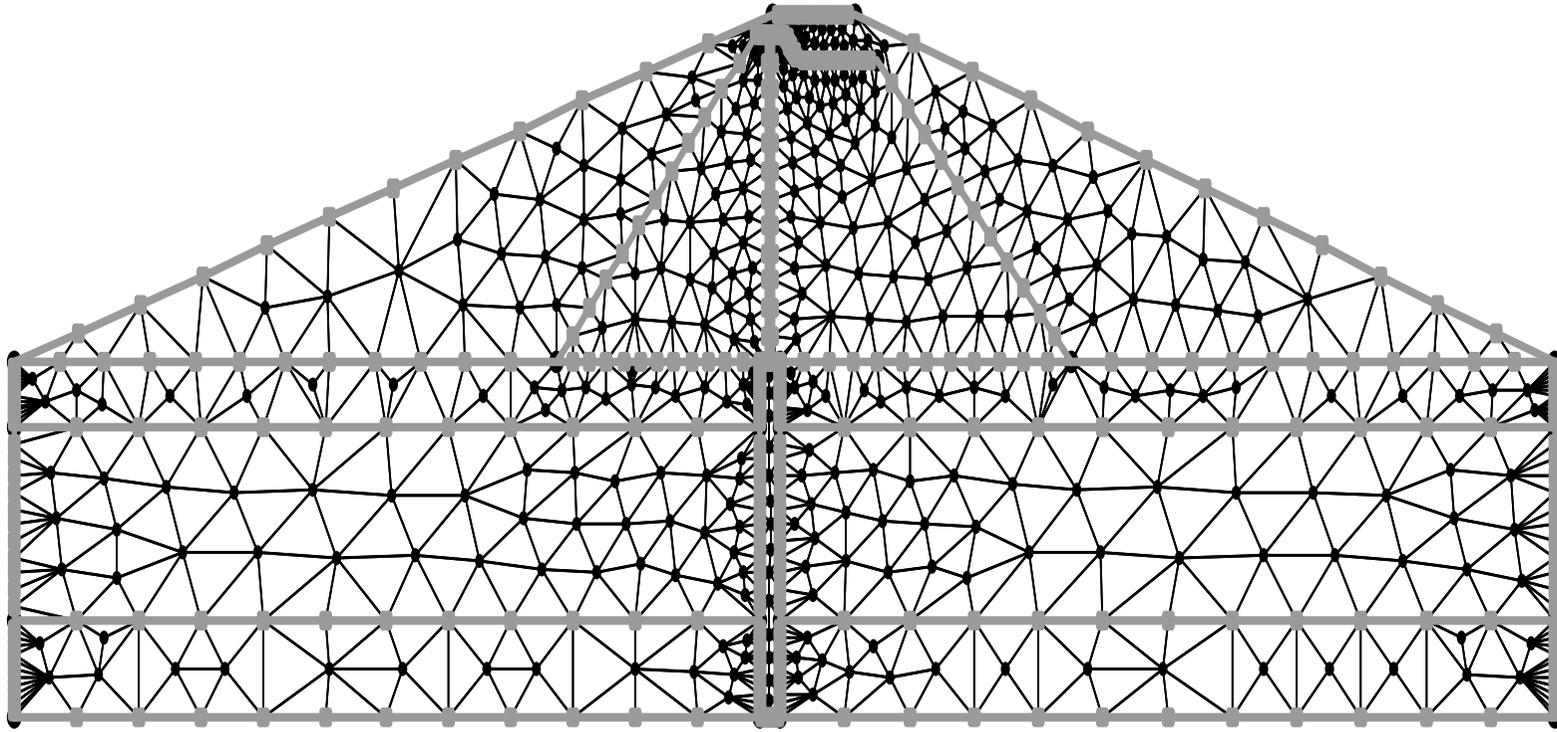
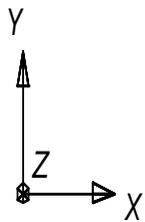


Figure (5-3) :Finite element mesh of AL-Qadisiya dam at st.46.



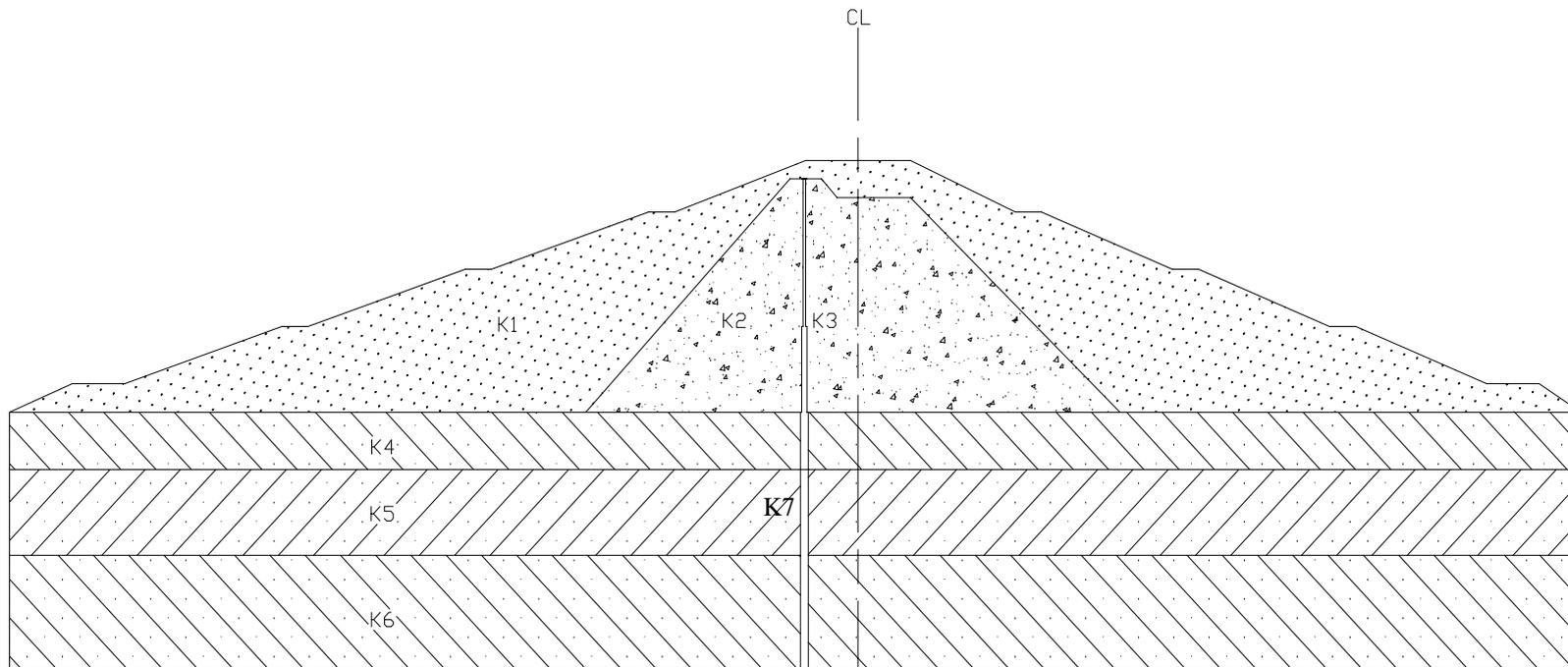


Figure (5-21); Typical cross-section of AL-Qadisiya Dam at station 33+50

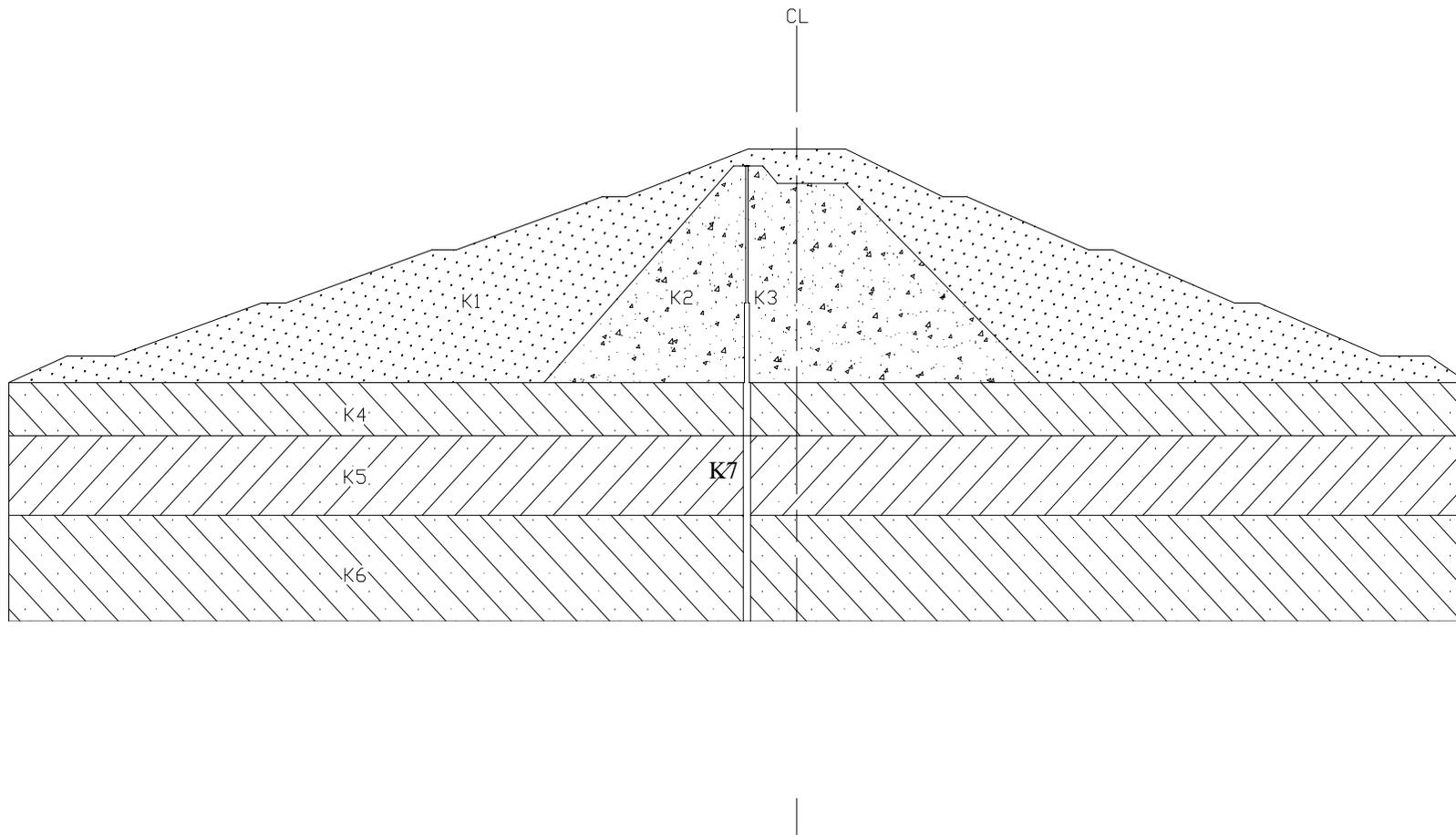


Figure (5-13) Typical cross-section of AL-Qadisiya Dam at station 42

Table (5-8): Observation reading of dam piezometers of selected station.

U/SWL	Time	Station	Piezometers									Piezometric W.L. Difference		
			1	2	3	4	5	6	7	8	9	1&2	3&4	5&6
135.29	13/1/2000	46	128.16	122.52	119.04	117.62	—	—	112	108.82	108.99	5.64	1.42	—
		42	124	112	124.4	113.4	—	—	103.14	—	—	12.0	11.0	—
135.42	5/11/2000	46	129.84	123.77	118.99	117.52	—	—	112.6	108.67	108.84	6.07	1.47	—
		42	120.4	112.4	118.9	112.9	—	—	103.28	—	—	8.0	6.0	—
135.29	7/11/2000	29	128.03	115.09	126.47	117.42	—	—	111.96	—	—	12.94	9.05	
		33+50	131.28	112.49	131.11	106.54	—	—	107.77	104.75	105.92	18.79	24.57	
136.04	25/1/2000	29	128.53	115.91	126.9	117.82	—	—	112.08	—	—	12.62	9.08	
		33+50	131.83	112.49	131.71	106.79	—	—	105.92	104.85	105.37	19.34	24.92	

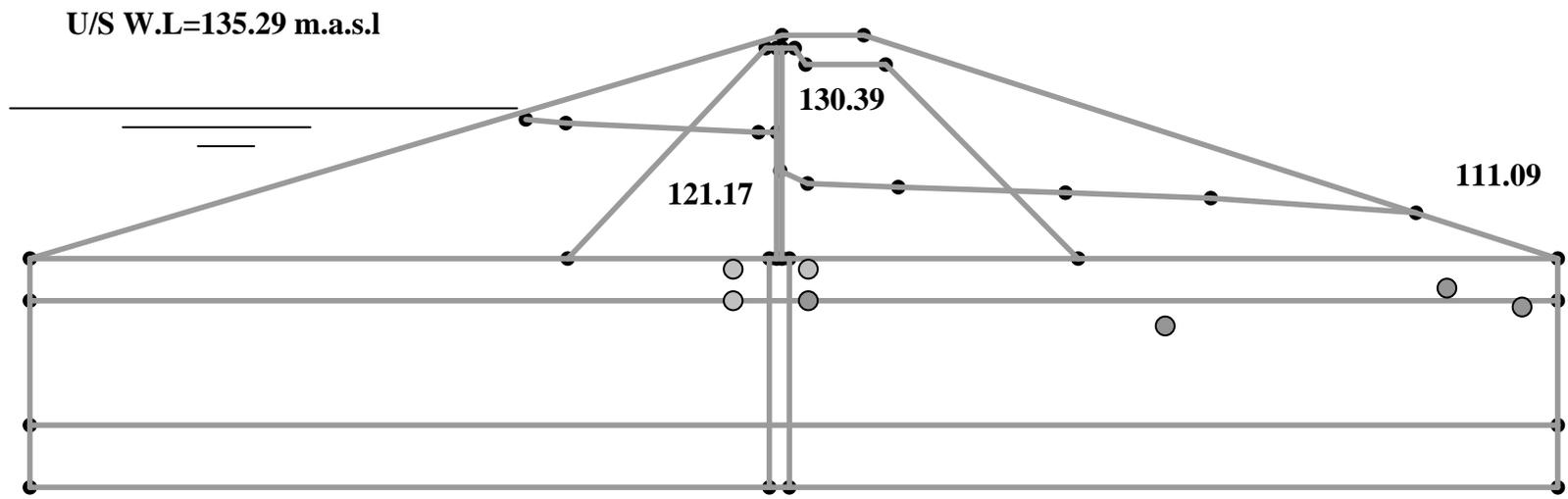
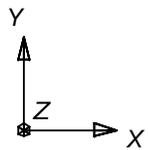


Figure (A-1). Phreatic surface position of AL-Qadisiya Dam at station 46 after 22 iteration



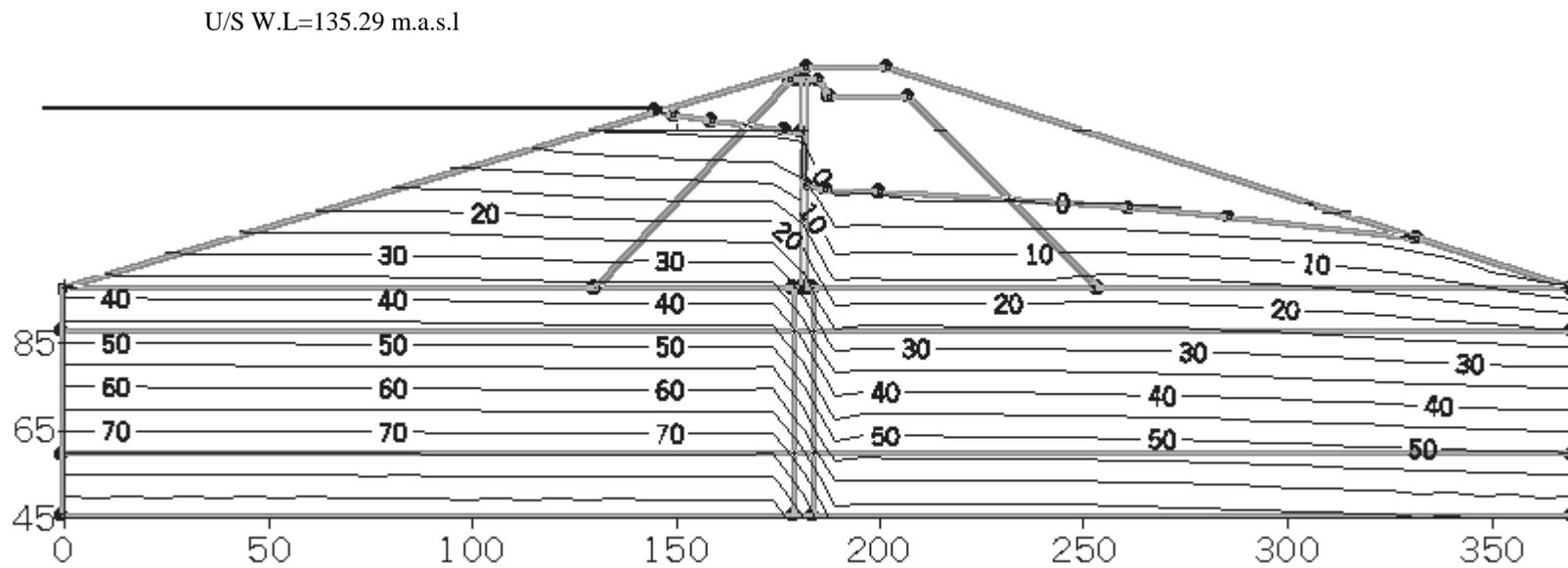
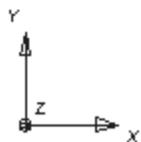


Figure (A-2): Pressure head contour lines of Al-Qadisiya dam at station 46.



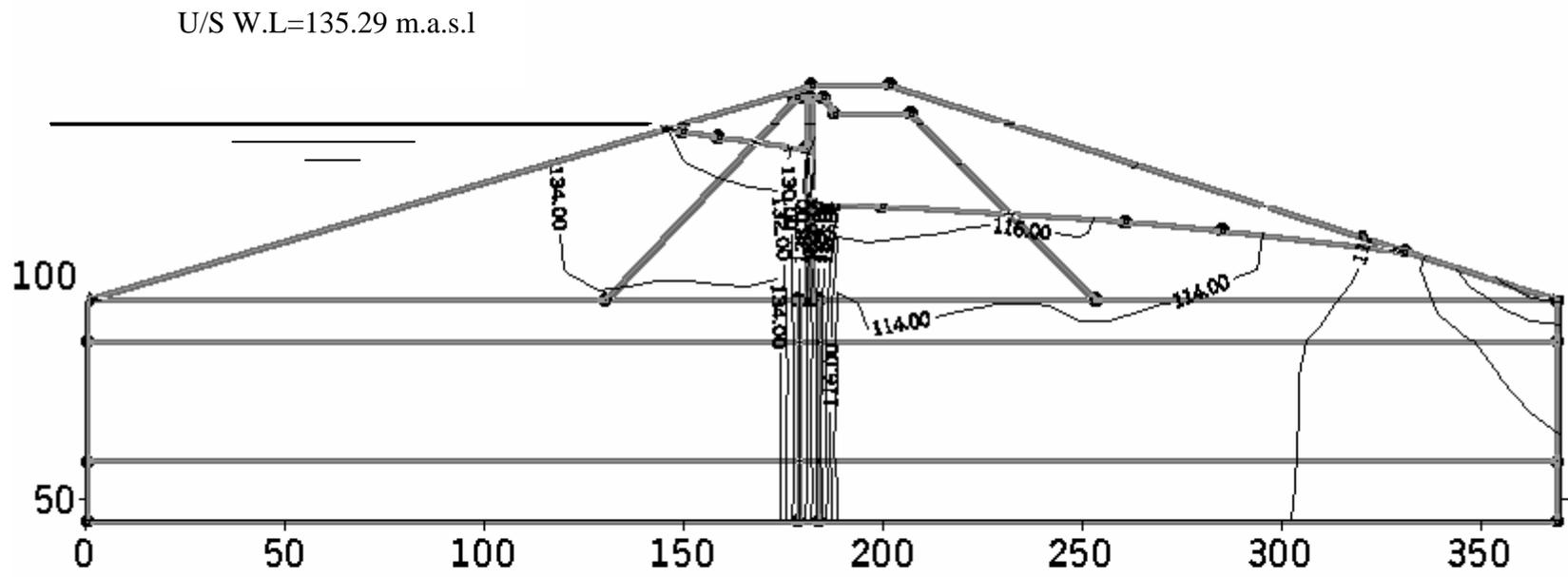
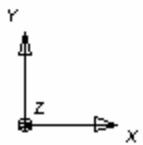


Figure (A-3): Total head contour lines of Al-Qadisiya dam at station 46.



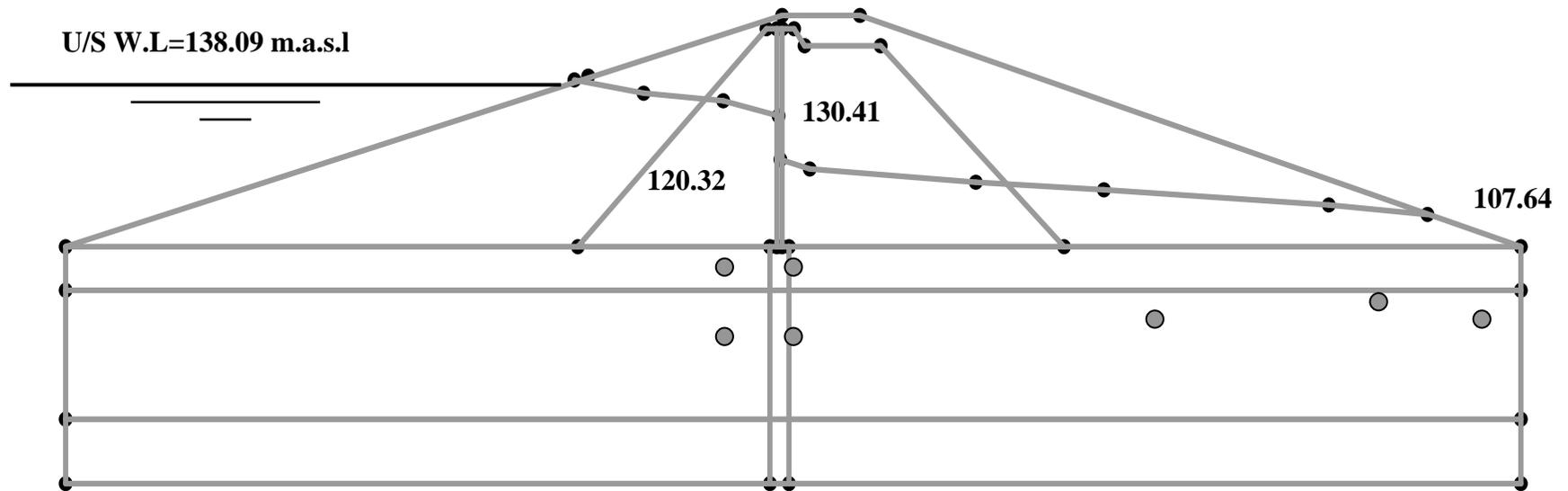
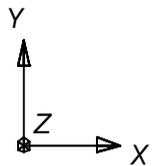


Figure (A-4). Phreatic surface position of AL-Qadisiya Dam at station 42 after 19 iteration



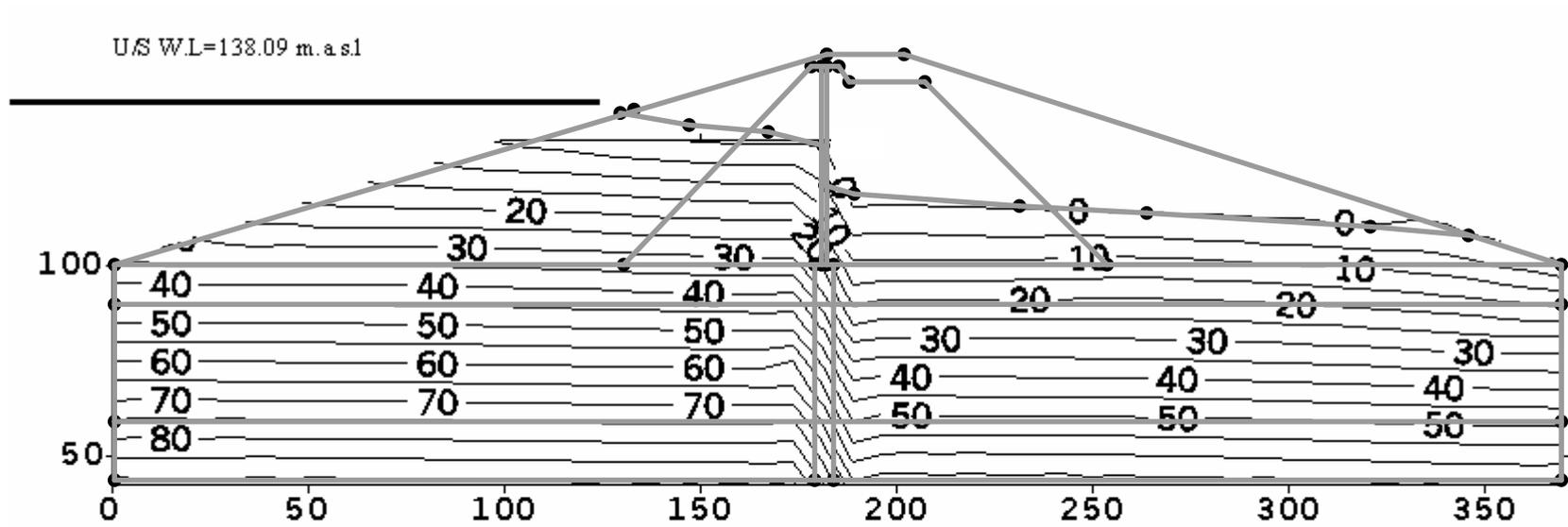
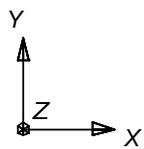


Figure (A-5): Pressure head contour lines of Al-Qadisiya dam at station 42.



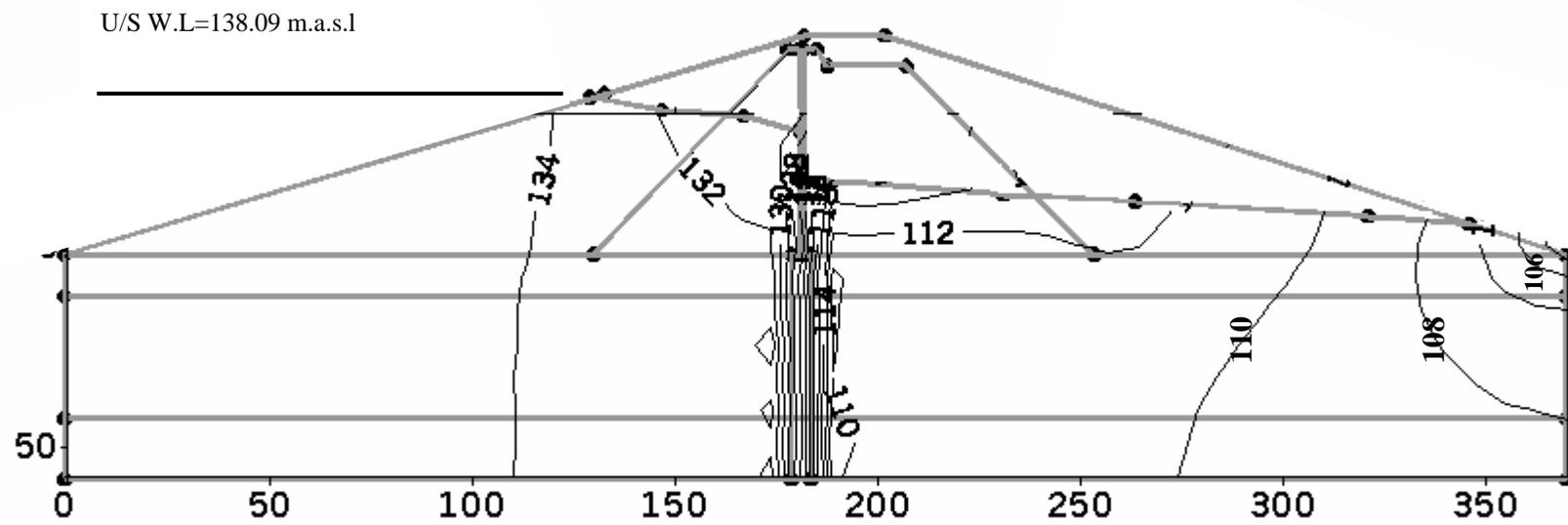
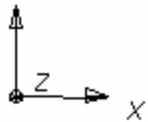


Figure (A-6): Total head contour lines of Al-Qadisiya dam at station 42.



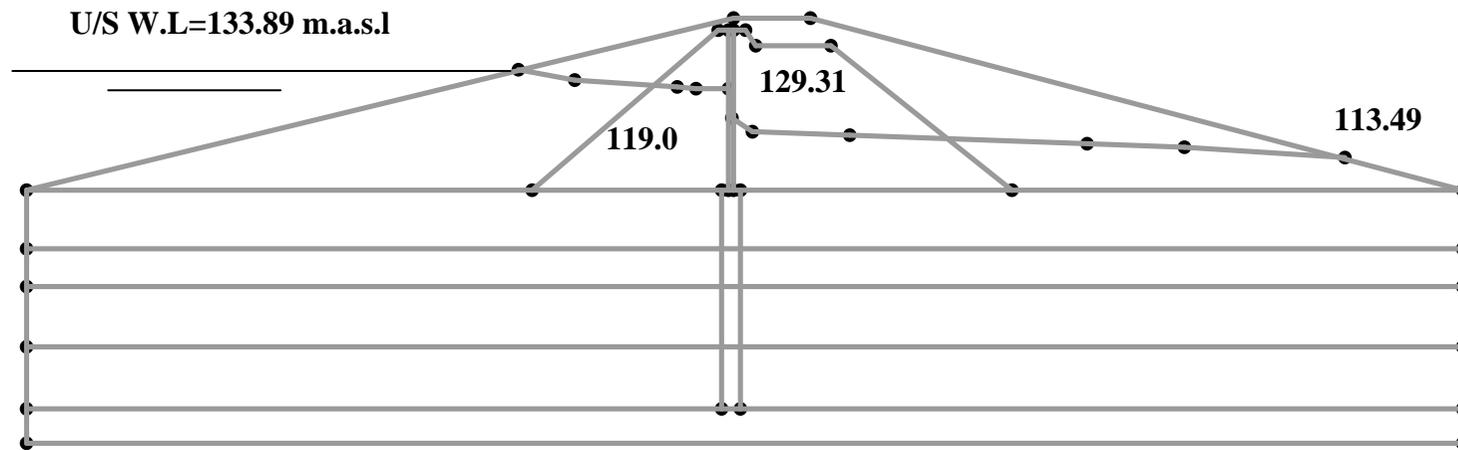
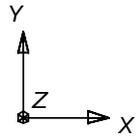


Figure (A-7). Phreatic surface position of AL-Qadisiya Dam at station 29
after 15 iteration



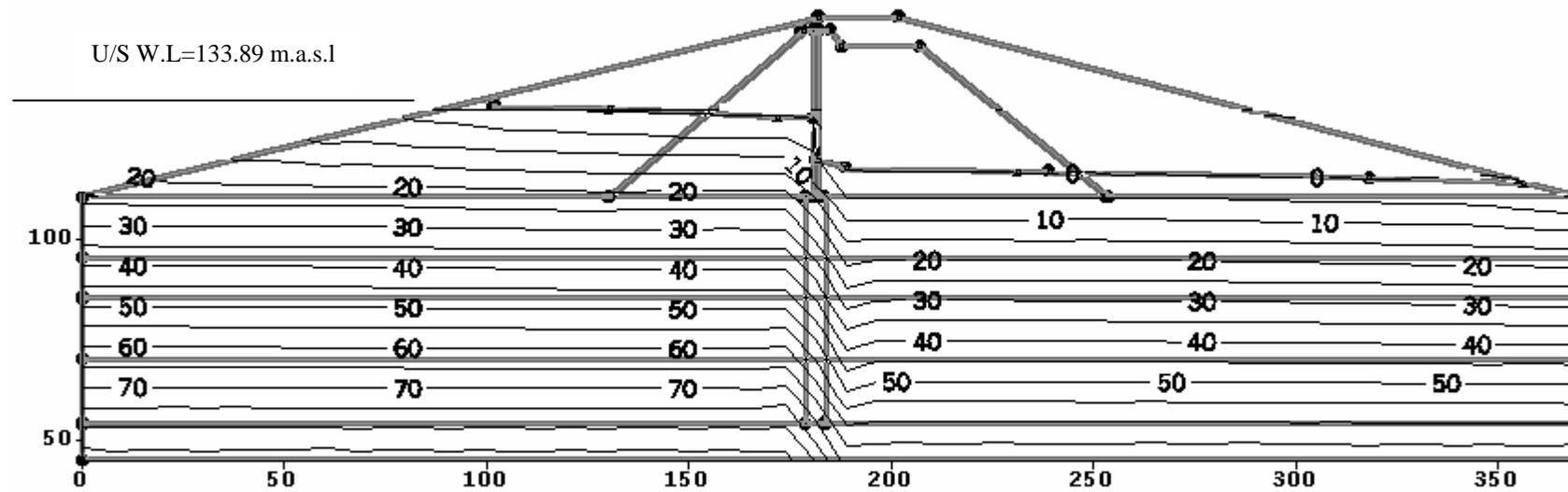
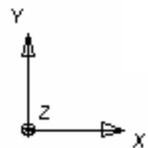


Figure (A-8): Pressure head contour lines of Al- Qadisiya dam at station 29.



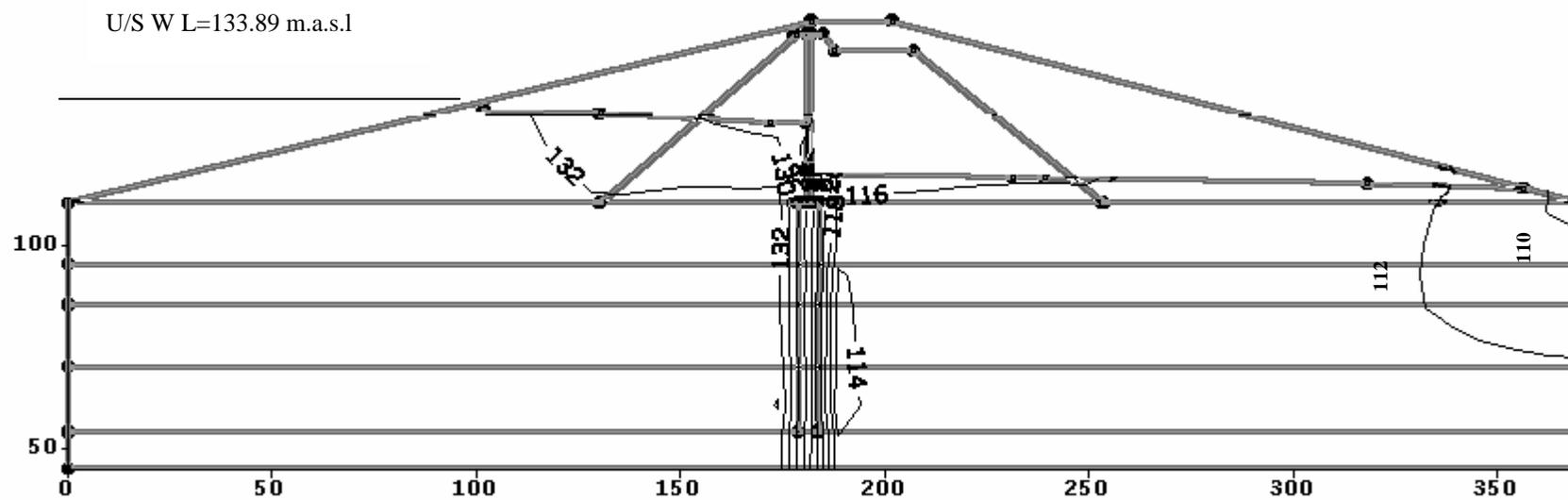


Figure (A-9): Total head contour lines of Al- Qadisiya dam at station 29.

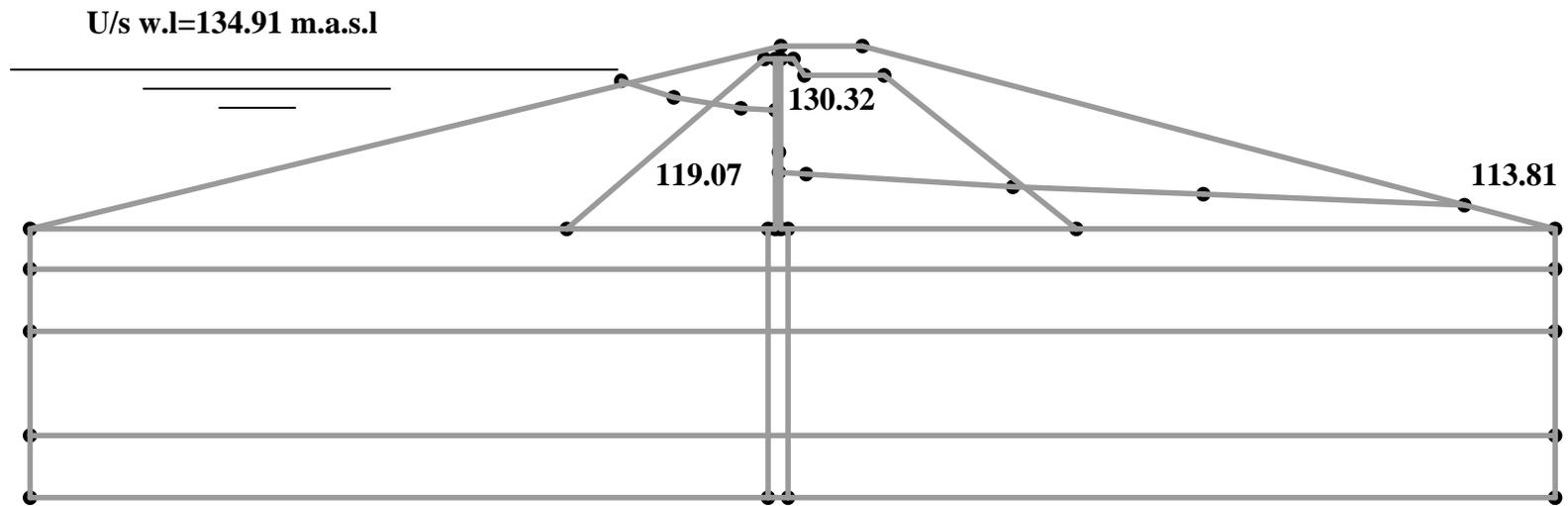
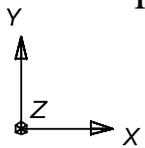


Figure (A-10). Phreatic surface position of AL-Qadisiya Dam at station 33+50 after 25 iteration



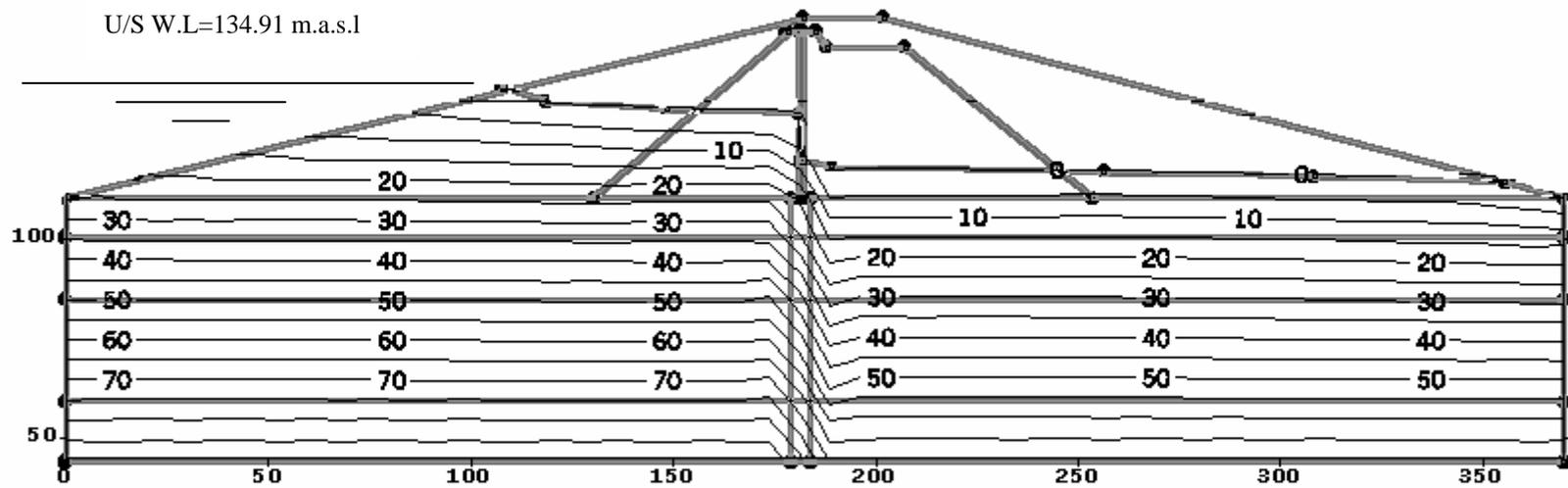
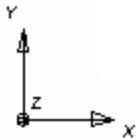


Figure (A-11): Pressure head contour lines of Al- Qadisiya dam at station 33+50.



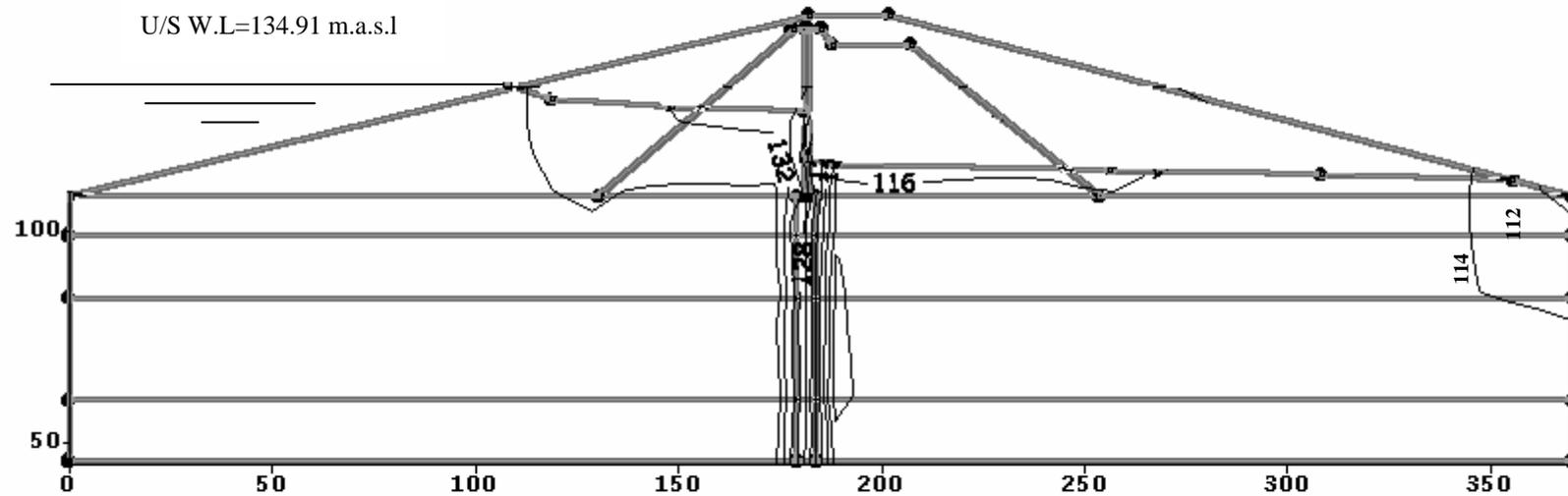


Figure (A-12) : Total head contour lines of Al-Qadisiya dam at station 33+50.

