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Effect of Pore Water Pressure Parameters on The Stability of AL-Ad'daim Earth Dam

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Abstract

The construction and *operation of earth dams (*homogeneous and those of clay cores) are normally controlled by the pore water pressure generated during these stages. These pore water pressures are the main reason behind the dam deformations, settlements, and instability. Throughout the experimental part of the study, a large number of classification tests, physical tests, and $\overline{B}$ -stress path tests were carried out. In the theoretical part of the study, the finite element analysis was adopted to assess the effect of the pore water pressure parameters of the marl (dam core) and the water contents of the compacted core layers on the expected deformations and stability of an earth dam throughout the construction and operation stages

تأثير معاملات ضغط ماء المسام على استقرارية سد العظيم

الخلاصة: -

يتم ضبط انشاء وتشغيل السدود الترابية (ذات المقاطع المتجانسة منها والتي تحتوي على قلب طيني) عادة من خلال السيطرة على ضغط ماء المسام الذي يتولد اثناء مراحل التشييد والتشغيل . يعتبر ضغط ماء المسام السبب الرئيسي وراء حصول الازاحات الجانبية والهبوط وبالتالي وراء مسألة استقرارية السد .

من خلال الجانب العملي لهذه الدراسة اجريت فحوصات كثيرة لتصنيف التربة للحصول على الخواص الفيزيائية ومجموعة من فحوصات القص لغرض تحديدمعاملات ضغط ماء المسام .

في الجانب النظري من هذه الدراسة أعتمدت طريقة ألعناصر ألمحددة لدراسة تأثير معاملات ماء المسام لتربة قلب السد وكذلك دراسة محتوى الرطوبة المعتمدة أثناء حدل تربة قلب السد على الازاحات الافقية والشاقولية للسد اضافة الى حساب درجة إستقراريته خلال مرحلتي الانشاء والتشغيل

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1. Introduction

Earth dams are considered as the oldest and the most famous establishment used in water reservoir. The beginning of their usage goes back to several decades before the message of the prophet Mohammed. Mea'arb dam in Yaman is a clear example of that.

Design engineers are finding it necessary in certain circumstances to utilize areas, which are not very suitable for construction of a dam because of some required economic development. With suitable zoning and compaction, the dam can be constructed from a wide range of naturally occurring soils. The relative quantities in the different zones are determined chiefly by economic considerations.

The design of an earth dam must be adopted to the available construction materials(USSD-2007). At most sites both pervious and impervious materials can be obtained. Under these circumstances, the dam is made up of a relatively impervious internal membrane known as core, and of outer zones that provide the earth structure the required stability. Such dams are called "zone earth dams".

Before 1940, the design of embankment-dams was essentially empirical, the height of these types of structures ranged from a few meters to 50 and, with some exceptions, most of the dams were located in rather wide valleys having good foundation conditions. The behavior of these structures was evaluated upon visual observations of cracks, deformations, and water flow through the foundation and the embankment. Settlements of dam-crest were systematically measured after construction.

2. Dam Site and Purpose

The central cross section perpendicular to the river direction, Fig.(1), reveals the main geological features of the site. The dam-embankment was founded on inclined layers 22.50 with the horizon of marl and sandstone bed rocks. The width of the river at the selected dam site is about 100 m in flood season, but this width shortens to about 30 m in dry season.

The construction of the AL-Ad'daim dam started in 1995 in a site known as Damir Kapu in the Jabil Hamrin. The river forms where the two branches Tuz Chai and Tauq Chai merge at about 1.5 Km upstream of the dam, Fig.(2).

The length of the dam is about 3.1 Km. It consists of main embankment spillways and tunnels pass through, and many small embankments, which connect the embankment with left and right shoulders. The foundation level is at 93.0 m above mean sea level (msl) and the crest level is 146.5 m above mean sea level.

Bienne and Partners-1988 specified that clay core material must be derived from neighboring marl quarries, then moistened and stockpiled in advance of placement in the dam, to ensure consistent moisture content.

The main purpose behind constructing the dam is to control the Tigris river flood, to control the flood of the AL- Ad'daim river which may threaten Baghdad city, to produce electricity, to cover the increased demand of water for irrigation of large agricultural areas, and to sediment large amount of fine sediments carried by water that cause problem to the water establishment in Baghdad.

3. Experimental Program

The local geology of the site is well established by the investigation of Smith and Partners-1988. A detailed review of the local geology of the AL-Ad'daim dam site was presented by Al-Abdullah-1996.

In addition to the original testing program carried out by the designer, the Engineering Bureau of the College of Engineering/Baghdad University-1994, performed an extensive laboratory program to investigate the shell, filters, core, and foundation properties. Engineering Consulting Bureau/College of Engineering-Al Nahrain University-1994, performed a testing program in N.C.C.L. of Baghdad. Al-Abdullah-1996, investigated the properties of the marl material used in zone C (clay core). The effect of gypsum content on the shear strength, compressibility, and permeability of the clay core material was studied.

The investigation carried out in this study was directed to assess the pore-water pressure evaluation by evaluating Skempton parameters A & B for the core material in the partially saturated condition, as well as to make classification, compaction, and shear strength tests. The pore water pressure generated within the core of the dam can be calculated as :

$U=B\,\Delta\sigma3 + A(\Delta\sigma1 - \Delta\sigma3)$ where :-

U: is the value of the pore water pressure.

A,B: are pore water pressure parameters.

$\Delta\sigma3$: is the increase in minor principal stress.

$\Delta\sigma1$: is the increase in major principal stress.

4. Summary of Results of Experimental Program

Results of the classification and compaction tests carried out during the present study are summarized in Fig.(3), Fig.(4) and Table (1), while the results of the shear strength parameters are summarized in Fig.(5) and Table (2).

The $\bar{B}$-stress path tests were conducted according to Bishop-1954. Sample preparation, testing procedure, and apparatus used are well explained by Al-Marsoumi-1997.

The results of the $\bar{B}$-stress path tests are summarized in Table (3) and (4), while the variations of A & B parameters with moisture content are presented in Figs.(6).

5. Theoretical Analysis of the Dam

Engineering structures are usually constructed in a definite sequence of operations. A conventional linear analysis of such structures is performed by assuming that the entire construction takes place in a single operation. In other words, stresses and deformations are computed by considering loads on completed structures (Desai and Abel-1972). Therefore in order to simulate the construction sequences of the dam, the following procedure was followed in the analysis during construction stage:

1. As in the standard incremental analysis, the full height of the dam is divided into many construction

layers. The height of each layer is selected so that it corresponds approximately to the level of overall cell pressure chosen in the laboratory tests conducted during experimental work. The details of these layers are given in Fig.(7).

2. Bienne and Partners-1988, specified in their design to use a rate of construction not more than 0.15 m/day. Later in 1994, CEB/Al Nahrain University stated that "during the construction of earth fill dam structure, the rate of loading is of great importance. A fast rate of construction can easily lead to collapse. A dam could be built up to any higher level before any possibility of failure(Gens and Alonso-2006) by slowing down the rate of construction". Several sets of F.E. analysis were conducted by CEB/Al Nahrain University to investigate the appropriate rate of construction that may be seen suitable to be implemented in the construction of the Great earth dam, see Table (5). In Table (5) the ratio (Ru) is defined as the excess pressure head in m to the embankment height (m), i.e. $\Delta h/\Delta H$.

As seen from Table (5) the first four rates of construction can lead to Ru>1.0, a value to which attention must be paid to avoid failure(SCDNR-2007) due to reduction in factor of safety as the shear strength of the dam materials is reduced to values equal to or less than resulting shear stresses.

It must be mentioned here that two selected percentages of moisture content are used in the present F.E. analysis. They are chosen to represent approximately the specified lower and upper limits of moisture decided by the designer. Therefore, using a rate of construction of 0.15 m/day seems to be very reasonable for the first four layers. In the 5th layer, the rate of construction was increased to 0.29 m/day. The volume of this constructed lift is much less than other lifts (as shown in Fig.(7)). For this rate (0.29 m/day), it was found that the generated pore water pressure was not highly affected.

3. A finite element analysis is performed to obtain pore pressures in the shell and core of layer (1) due to self weight using elastic module (E,v) and γ. The pore pressure parameters (A & B) are calculated from Fig.(6) and the material properties are taken from Table (6), where a pressure equals (γ.H) is considered to correspond to the cell pressure (σ3). The pore pressures are determined at every node in the dam. Deformations due to undrained condition are also calculated.

4. Consolidation analysis is performed with the following considerations:

a. The generated pore water pressures resulting from the undrained analysis are now considered as the initial pore water pressures in entire nodes of the dam.

b. The unit weight of the constructed layer and different materials of the foundation are considered equal to zero.

c. The Cvx and Cvy for each material are defined
d. With the specified number of time steps and their sizes required to cover the duration of constructing the first layer, drained analysis is performed to evaluate the pore water pressure and deformations due to consolidation.

5. At the end of (nth) time step specified for the duration of construction of (1st) layer, the total deformations are found by summing the deformations resulting in (3) plus deformations calculated in (4-d).
6. A second layer is then added. Unit weight of all materials of foundation and (1st) layer is taken equal to zero, except materials of the newly added layer. Pore water pressures due to undrained condition are then calculated, and deformations are calculated as in (3) above. These pore pressures are summed with the residual pore pressures resulting from consolidation analysis (4-d) and then considered being initial pore pressures for conducting consolidation analysis as in step (4) with the same considerations in (4-(a-d)). At the end of this stage, the total deformations are also found and the calculations in (3-5) are performed.
7. For each additional layer, procedure described in (3) through (5) is repeated to evaluate the pore pressures and incremental deformations in all underlying layers. Complete (accumulative) deformations for every node of the entire dam are shown in Fig.(8) and Fig.(9).

The analysis procedure for filling and draw down stages are presented in Fig.(10). The analysis procedure shown in Fig.(10-c) for every additional draw down level, the only changing parameter is the time required for consolidation analysis, for example:

Time required to draw down water level from 143 to 115 is:

Tf+15 days

And so on for draw down from level 143

Tf+20 days

Where Tf = time for flood level + two months after loading.

The finite element mesh for the idealized section, Fig.(7), is shown in Fig.(11). The dam is zoned earth structure with an inclined impervious core. The core is comprised of brown silty clay and the shells are sandstone. The impervious core was founded directly on the inclined marl bedrock layer.

A comprehensive testing programs were performed to investigate the geotechnical properties of the selected materials used in the construction of the dam. Summary of the tested materials and their properties are presented in Table (6).

6. Results of the Theoretical Analysis

Two percentages of moisture content were selected to represent the desired percentages of moisture specified by the designer (+1 and +3) of optimum moisture contents. Their corresponding values of pore water pressure parameters (A & B) were derived from Figs.(6).

The material properties that have been used in this analysis during construction stage are tabulated in Table (6), where values in shaded zones only are considered in the analysis.

The boundary conditions were assumed to be impervious and restricted in both vertical and horizontal directions in the lower of the descritized domain of the dam foundation. Boundaries at the sides of the descritized domain were assumed to have free draining conditions and only the horizontal movement was assumed to be restricted. On the other hand the upper surface of dam foundation was assumed to be free draining and could move in both directions.

The rate of construction selected to analyze the dam was 0.15 m/day for first four lifts and 0.29 m/day for the last lift. Table (7) presents the thickness of lift versus time for dam construction.

During the analysis, at every time step there was a check for the generated excess pore water pressure. This check was concerned with updating the ratio between the value of pore water pressure of the last and present time interval. Once this ratio approached 1.0, a stoppage of construction had to be considered to allow for the generated pore water pressure to dissipate.

The current analysis of the dam showed that during the suggested first and second lifts (Fig.(8)) no effect was account for by the Mandle-Cryer phenomenon as seen from Fig.(12, 13, 14, 15), where contours of excess pore pressure are presented in.

It was noticed that during these two lifts for both percentages of moisture, that the fields of maximum pore pressures are concentrated within the dam foundation.

As the third layer is placed, the pore pressure starts to develop. At the end of time required to place this lift (Table (7)), the excess pore pressure is 1.4 times greater than the summed pore pressures at the beginning of this lift when the construction water content is 20.5%. On the other hand this ratio decreases to 1.17 for an 18.5% of moisture content.

It was explained that 82 and 37 days are required to minimize this excess pore pressure to a level less than (γ soil * height of placed fill) for 20.5% and 18.5% of moisture contents respectively.

With the placement of the fourth lift, the pore pressure starts to increase, and for higher percents of moisture content (here 20.5%) it was again found that at the end of construction of this lift 29 days are required to allow for the excess pore pressure to dissipate to a level less than the initial pore pressure as this fill is started to be placed.

From contours plotted for generated pore pressures for both percentages of fill moisture, it is noticed that:

At early stages of construction (i.e. 1st and 2nd lifts), fields of high excess pore water pressure were concentrated in the marl stone layer within dam foundation. As the construction proceeded, these fields shifted up and concentrate within the mid-way points of the upper part of the lower third of the core. This may be attributed to the fact that at node (69) of Fig.(11), the drainage conditions are restricted as it is compared to that provided at the base of the dam, where the (L) shape filter would accelerate the dissipation of excess pore pressure.

Horizontal movement was found to be small enough so that there is no lateral bulging might occur during construction stage. Maximum fields of horizontal movements are noticed (17.5 cm) near the core downstream face towards the downstream side for lower percents of fill moisture content. Maximum horizontal displacements towards the upstream side was found at the middle third of the dam height in the upstream portion when higher percents of fill moisture content were used.

From contours of vertical displacements depicted, the following remarks can be raised:

1. In the early stages of construction, these movements are concentrated at the base of the dam within the core portion. As construction proceeds, the maximum displacements are noticed at the middle third of the core points.
2. Maximum vertical displacements are predicted at lower percentages of fill moisture contents near optimum (on the wet side).

When first filling of the dam reservoir is commenced, a process of accommodation of the structure to the newly imposed state of stresses began.

The rapid increase of pressure with reservoir filling is mostly a matter of pressure distribution from the upstream portion in contrast to water transport.

Regular analysis of recorded movements for earth dams indicates that under normal operating conditions the dam settlement increases proportionally to settlement increments caused by filling of successive layers and loading of both the reservoir bowl and dam with water (Harbowski and Razadkowski-1988). Under usual consolidation (without peat), the dam construction and reservoir filling, settlements should not exceed (1 to 2)% of the dam height.

In this analysis, it was assumed that after two months of completion of dam construction, the reservoir is started to be filled. The analysis of the dam reservoir filling achieved by raising water level through three levels, 115, 131, and 143.5.

Figures (16,17) show the pore water pressure contours after rising water level up to elevation 115. These contours show that fields of high pore pressures are concentrated at the dam foundation, as the excess pressure in the constructed core is dissipated.

Contours of horizontal movements show slight increase occurring towards the downstream side with no movements appearing to occur towards the upstream side.

Vertical displacement contours (Fig.(18,19)) show the decrease in vertical displacement when the dam is constructed with moisture near the O.M.C. while it increases when dam is constructed with a more moisten fill material.

The time suggested to cover this level of reservoir filling is 1.5 months. Similar trend of PWP concentration was also noticed after raising water level in the reservoir up to ordinary operation and flood levels (131 & 143.5 respectively).

Contours for horizontal displacements show that slight reduction occurs in displacement as they are compared with that after raising water level up to elevation 115, when the fill moisture content is near the O.M.C.. the direction of this movement is still towards the downstream side. With higher percent of fill moisture, this movement increases towards the downstream side.

Vertical displacements due to this raising of water level in reservoir are presented by ISO-settlement contours in Figs.(20, 21). For both percentages of moisture content, the vertical displacements increase slightly with increasing water level and maximum fields of this movement is concentrated at the middle third of the dam height near the upstream core face. When the water level increases to flood level (143.5 msl), large horizontal displacements are noticed in the upstream face, where values of more than 1.4 m are detected for dam constructed with higher moisture content. During this stage care must be taken because this level of movement may cause the concrete blocks stocked at the upstream face to be moved, keeping the dam exposed to erosion.

Again 1.5 months, was suggested to satisfy each level of reservoir filling. It must be mentioned that maximum horizontal displacement in the downstream surface is 0.32 m for the dam constructed with lower moisture of fill material, when water level is raised in the reservoir up to flood level. This level of displacement causes the downstream side to be bulged, and longitudinal cracks might be initiated in the downstream surface. Draw down case was studied after permitting the flood level to be maintained for 2 months, and then three suggested levels of draw down were selected as shown in Table (8).

Draw down to levels 131 and 115 are considered to be partial draw down conditions, while complete draw down is considered to be satisfied if pool level decreases to elevation 100.

Contours for pore water pressures show that maximum values occur at mid-way point of lower third of core (point 59, Fig.(11)). This was also noticed when the water level is lowered from elevation 143 to 115 msl.

Lowering the water level from flood to ordinary operational level causes slight increase in pore water pressures and maximum fields remain within the marl stone layer below the dam.

Contours for horizontal displacements show that during linear draw down from level 143.5 to levels 131, 115 when the construction moisture content was 18.5%, and linear draw down from level 143.5 to 131 for construction moisture content equal to 20.5%, maximum values are obtained at the upstream surface, oriented towards the downstream side. For all remaining cases, maximum horizontal fields were noticed to be concentrated at the middle third of clay core zone with higher values calculated for draw down from level 143 to 100 when construction moisture content was 18.5%.

ISO-settlements contours presented for draw down case, the decrease in settlement encountered for flood level was due to linear draw down for all cases studied. This may be attributed to the swelling that occurs due to the release of load applied as the reservoir was being emptied. Fields of highest level of vertical displacements are shown to be concentrated within the middle third part of the core.

7. Conclusions

The following conclusions could be drawn from the results of the laboratory tests and the theoretical analyses of AL- Ad'daim dam:

1. Strains for soils tested in a partially saturated condition are found to increase as the compaction moisture content tends to increase from the dry side of optimum towards the wet side of optimum. Until a certain level of moisture content, which may be termed as (critical moisture content), strains, are then reduced as compaction moisture content increases. This level was found to be approximately+2% of optimum.
2. Pore water pressure parameters (A & B) are found to increase with the increase of fill moisture content.
3. Zones of high generated pore water pressures are found to concentrate at the upper part of the lower third of the core.
4. Maximum vertical displacements occur in the core middle third during construction stage.
5. During full reservoir filling, maximum horizontal displacements in upstream surface occur at the middle third of the surface, while maximum values in downstream surface are noticed at the upper third.
6. A differential deflection between the upstream and downstream edges of the crest seems to indicate a progressive widening of the crest and thus, a possible development of longitudinal cracks.
7. The research show that it is possible to construct the dam core with clays from quarries with a compaction moisture content not more than +3% of optimum to minimize the generated pore water pressures and settlements.
8. The increase of moisture content of fill material is found to increase the shutdown periods required to dissipate the generated pore pressures.

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Table (1) Summary of Index Properties and Compaction Results

Quarry No.	4	10
Gs	2.75	2.77
LL	37	48.9
PL	21	24.5
PI	16	19.4
OMC%	17.74	17.35
Max γ_d (kN/m³)	17.15	17.25

Table (2) Summary of Shear Strength Parameters

Quarry No.	C_v (m²/year)	t (hr)	rate of strain (%/min)	C'	φ'
4	[illegible]	6.7	0.020	8	31.5
10	[illegible]	7.2	0.028	30	24.8

Table (3) Variation of P.W.P Parameters (A&B) with Water Content, Minor Principal Stress and Deviator Stress for Quarry No. (4)

Average w.c %	Over all σ_3=200kpa				Over all σ_3=600kpa				Over all σ_3=900kpa			
	σ_1	$\sigma_1-\sigma_3$	B	A	σ_1	$\sigma_1-\sigma_3$	B	A	σ_1	$\sigma_1-\sigma_3$	B	A
10.84	284	590	.330	.002	498.6	336	.384	.01	892.3	455	.381	.023
11.45	283	166.7	.340	.003	423.5	361	.47	.013	861	472	.540	.000
13.35	164	119.24	.236	.006	228.4	216.46	.042	.013	411.7	282.1	.620	.017
15.20	168.4	173.25	.236	.006	361.6	165.6	.061	.02	364	248	.718	.027
21.03	102.5	72.1	.391	.07	154.2	121.6	.749	.061	323.7	103.3	.770	.068
22.42	85.2	46.3	.786	.327	171.3	56	.827	.363	322.7	105.8	.846	.306

Table (4) Variation of P.W.P Parameters (A&B) with Water Content, Minor Principal Stress and Deviator Stress for Quarry No. (10)

Average w.c %	σ_3=200kpa				σ_3=400kpa				σ_3=[illegible]kpa			
	σ_1	$\sigma_1-\sigma_3$	B	A	σ_1	$\sigma_1-\sigma_3$	B	A	σ_1	$\sigma_1-\sigma_3$	B	A
13.70	222.3	128.26	.232	.023	471.4	311.35	.408	.006	1003.7	488	.488	.022
17.43	174.8	186.2	.432	.010	314.8	296.23	.517	.024	458	223.9	.44	.031
18.50	183	118.74	.042	.024	283.1	216.63	.022	.039	326.5	376	.081	.062
22.10	118.4	69.8	.081	.070	186	147.24	.277	.081	264.6	128.1	.268	.068
25.87	86.1	61.8	.784	.082	126.1	86.8	.706	.107	184	126.5	.822	.148
32.4	83.7	32.4	.030	.144	87.8	65.3	.840	.102	146.3	90.8	.891	.262

Table (5) Pore Pressure Development Using Various Rates of Construction (After AL-Nahrain Univ.–1994)

Rate of Const. (m/day)	Average Pore Pressure Ratio r_u		
	end of const.	After (6) months	After (1) year
1275	1.13	0.98	-
0.662	1.12	0.98	0.9
0.318	1.11	0.97	0.88
0.15	1.07	0.86	0.88
0.150	0.89	0.60	0.596

Table (6) Summary of E, ν, γ Values for AL-Ad'daim Earth Dam Materials

Material	E (kN/m²)	ν	γ (kN/m³)	Reference
Sandstone (Foundation)	[illegible]	[illegible]	-	[illegible]
	[illegible]	-	-	AL-Kafaghi Bureau
	[illegible]	[illegible]	[illegible]	[illegible]
	[illegible]	0.40	[illegible]	Current analysis
Marl Stone-Salt Sandstone (Foundation)	[illegible]	-	-	BSP
	[illegible]	0.3–0.45	–	AL-Kafaghi Bureau
	[illegible]	[illegible]	[illegible]	Current analysis
Marl (Foundation)	[illegible]	–	–	[illegible]
	[illegible]	-	-	AL-Kafaghi Bureau
	[illegible]	0.3	21	[illegible]
	[illegible]	0.3	19	Current analysis
Sandstone (Shell)	[illegible]	0.3	[illegible]	[illegible]
		0.2	[illegible]	[illegible] Nahrain. U.
		0.25	18	Current analysis
Clay Core Material	[illegible]	0.2–0.4	[illegible]	[illegible]
	[illegible]	0.3	18	[illegible] Nahrain. U.
	[illegible]	0.25	Variable	Current analysis

Table (6-b) Summary of Coefficient of Consolidation
Cv for AL-Ad'daim Dam Materials

Material	C_v (m^2/sec)	Reference
Sandstone (Foundation)	$C_{vx}=0.716\times10^{-6}$ $C_{vy}=0.67-0.89$ (Cm^2/min)	CEB/Bag . U.
	$C_{vx}=C_{vy}=0.8\times10^{-6}$	AL-Kafaghi Bureau
	$C_{vx}=0.716\times10^{-6}$; $C_{vy}=1.25\times10^{-6}$	Current analysis
Marl (Foundation)	$C_{vx}=C_{vy}=0.516\times10^{-6}$	CEB/Bag . U.
	$C_{vx}=C_{vy}=0.1-1.0\times10^{-6}$	AL-Kafaghi Bureau
	$C_{vx}=C_{vy}=0.516\times10^{-6}$ (ave.)	Current analysis
Clay Core	$C_{vx}=0.8\times10^{-6}$ $C_{vy}=0.3\times10^{-6}$	CEB/Bag . U.
	$C_{vx}=C_{vy}=(0.1-0.6)\times10^{-6}$	Al-Abdullah ph .D Thesis
	$C_{vx}=0.6\times10^{-6}$; $C_{vy}=0.3\times10^{-6}$	Current analysis
Marl stone –soft sandstone (Foundation)	$C_{vx}=C_{vy}=(0.1-0.6)\times10^{-6}$	AL-Kafaghi Bureau
	$C_{vx}=C_{vy}=0.2\times10^{-6}$	CEB/AL Nahrain U.
	$C_{vx}=C_{vy}=0.3\times10^{-6}$ (ave)	Current analysis
Sandstone (Shell)	$C_{vx}=0.716\times10^{-6}$ $C_{vy}=0.67-0.89$ (Cm^2/min)	CEB/Bag . U.
	$C_{vx}=C_{vy}=0.8\times10^{-6}$	AL-Kafaghi Bureau
	$C_{vx}=0.716\times10^{-6}$ $C_{vy}=1.25\times10^{-6}$	Current analysis

Table (6-c) Summary of Coefficient of permeability For
AL- Ad'daim Dam Materials .

Material	K(m/sec)	Reference
Sandstone (Foundation)	$5\times10^{[illegible]}-5\times10^{-7}$	NCCL
	$Kx=10^{[illegible]}-10^{-7}$ $Ky=10^{-6}-10^{-7}$	CEB/Bag . U.
	$Kx=Ky=10^{-6}$	Current analysis
Marl stone –soft sandstone (Foundation)	$1.22\times10^{[illegible]}$ (m/day)	CEB/AL-Nahrain U.
	$1.22\times10^{[illegible]}$ (m/day)	Current analysis
Marl (Foundation)	$Kx=Ky=10^{-10}$	CEB/Bag . U.
	$Kx=Ky=10^{-10}$	Current analysis
Sandstone (Shell)	$Kx=Ky=1.25\times10^{[illegible]}$	AL-Kafaghi Bureau
	$K=10^{[illegible]}-10^{-5}$	CEB / AL-Nahrain U.
	$Kx=Ky=1.25\times10^{-6}$	Current analysis
Clay Core	$Kx=2.25\times10^{-10}$ $Ky=1.0\times10^{-10}$	CEB/Bag . U.
	$K=10^{-10}-10^{[illegible]}$	CEB / AL-Nahrain U.
	$Ky=1.0\times10^{-10}$; $Kx=2.25\times10^{-10}$	Current analysis

Table(7)Height of Lift Versus Time to Construct Each Lift.

Lift No.	Height(m)	Time (day)
1st	7	47
2nd	12	80
3rd	12	80
4th	12	80
5th	10.5	37

Table(8) Suggested Draw Down Levels Versus Time

Moisture Content %	18.5	20.5
Draw Down Level	Time for draw down (days)	Time for draw down (days)
143-131	10	10
143-115	15	15
143-100	20	20

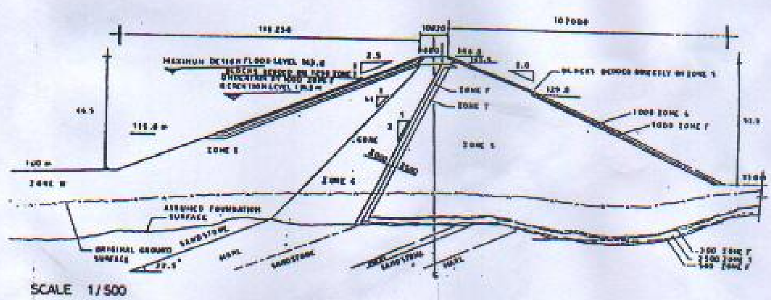

Fig. (1) Central Cross Section of Embankment Along The Crest Earth Dam.

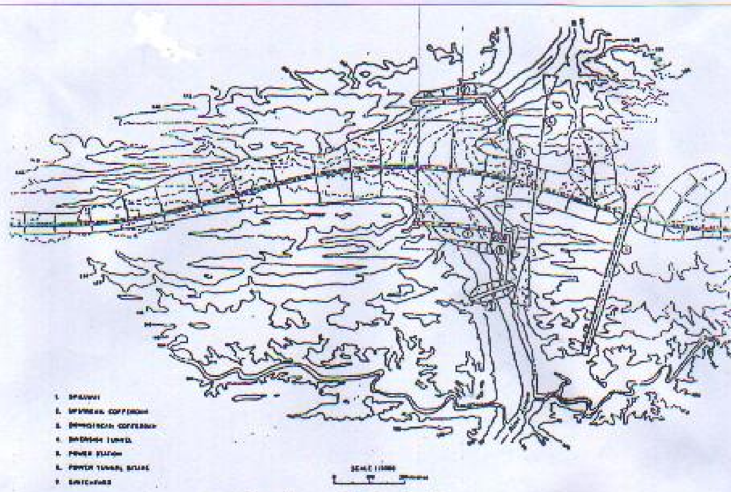

Fig. (2) General Layout of The Great Earth Dam.

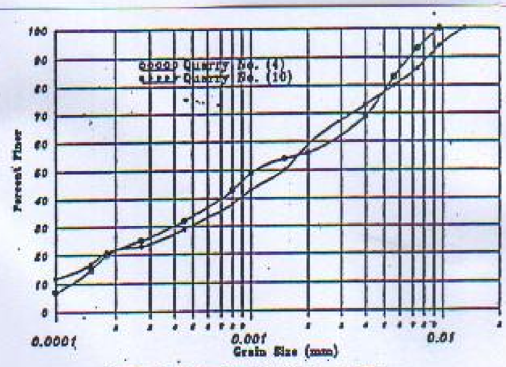

Fig. (3) Grain Size Distribution Curve For Marl Material.

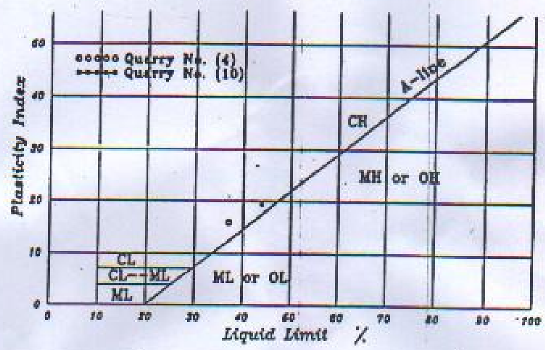

Fig. (4) Elasticity Chart : Unified System.

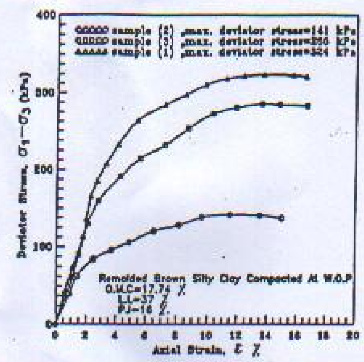

Fig. (5-a) Consolidated Undrained Triaxial Test
With P.W.P. Measurements, Quarry No. (4).

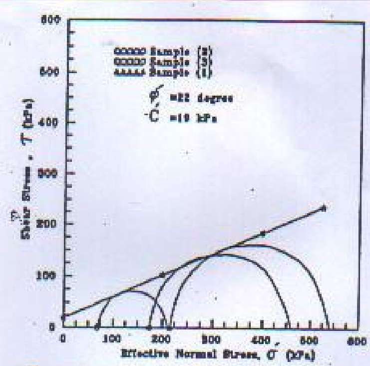

Fig. (5-b) Mohr-Circle Envelop For Clay Core
Material, Quarry No. (4).

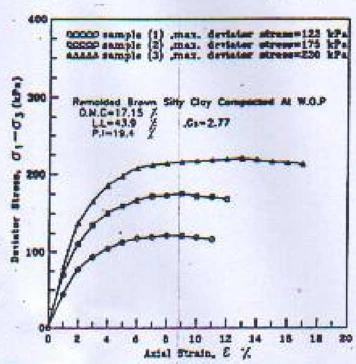

Fig. (5-c) Consolidated Undrained Triaxial Test
With P.W.P. Measurements, Quarry No. (10).

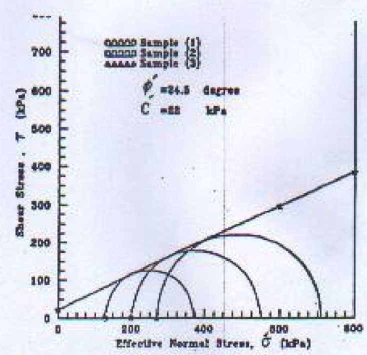

Fig. (5-d) Mohr-Circle Envelop For Clay Core
Material , Quarry No. (10)

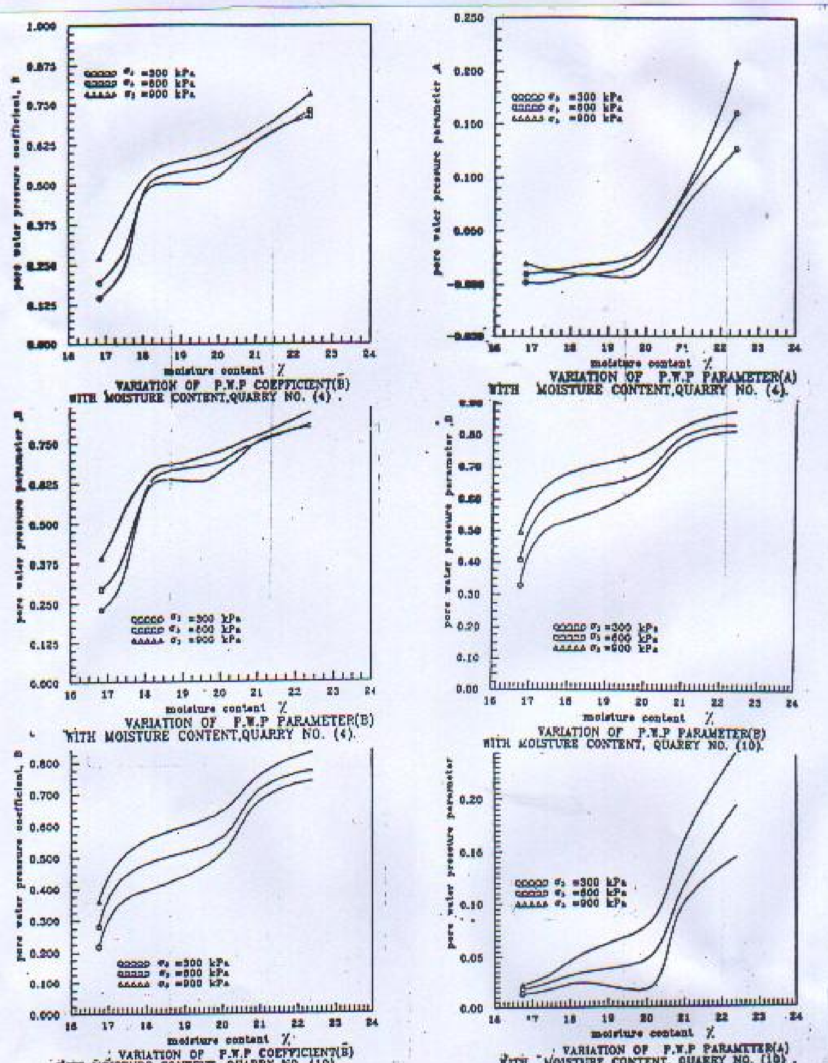

Fig. (6) Variation of P.W.P. Parameters With Moisture Content for Quarry No. (4) and (10).

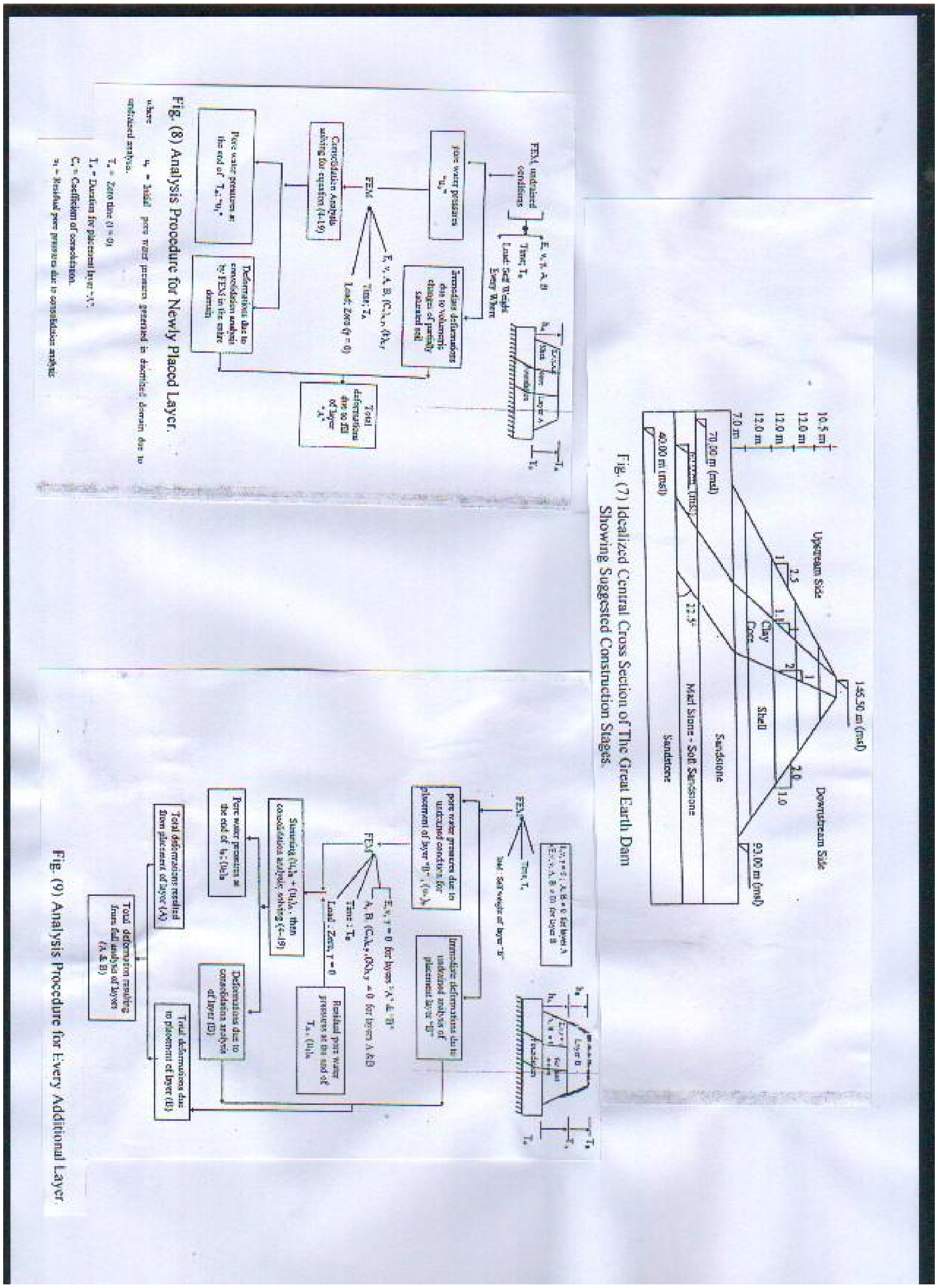

Fig. (7) Idealized Central Cross Section of The Great Earth Dam Showing Suggested Construction Stages.

Fig. (8) Analysis Procedure for Newly Placed Layer.

Fig. (9) Analysis Procedure for Every Additional Layer.

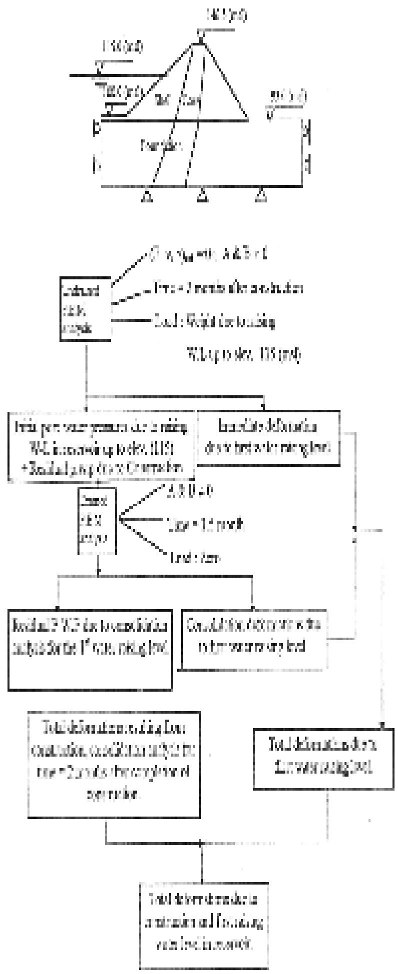

Fig. (10-a) Analysis Procedure Due to Raising W.L. Up to Elev. (115.00)

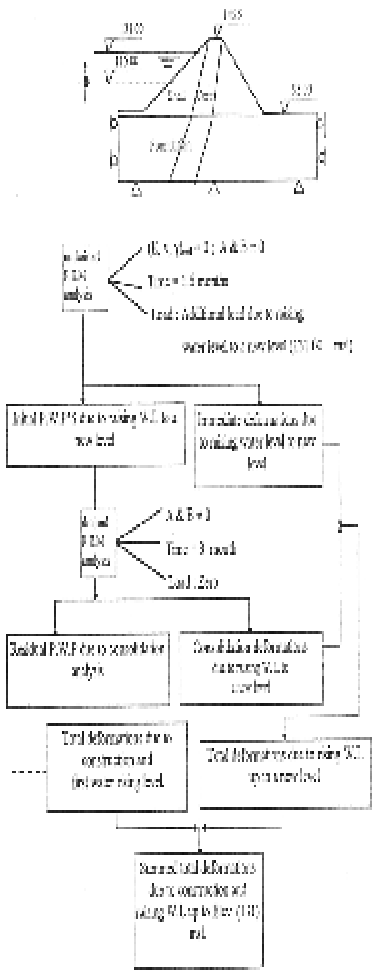

Fig. (10-b) Analysis Procedure for Every Additional Water Raising Level in Reservoir

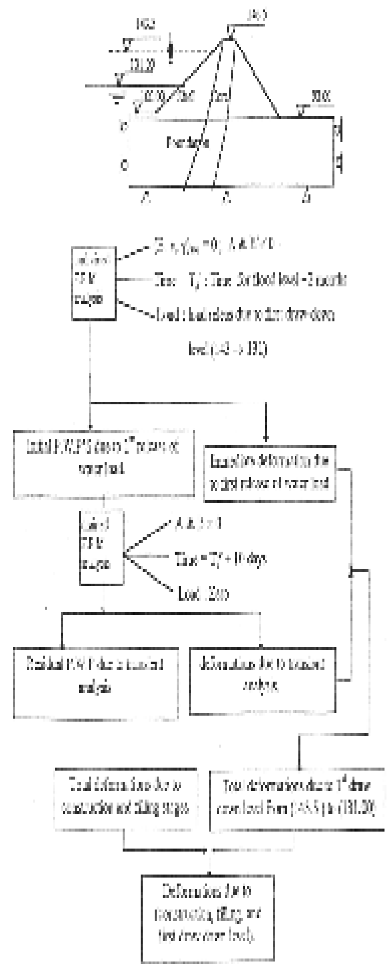

Fig. (10-c) Analysis Procedure for The First Draw Down Level

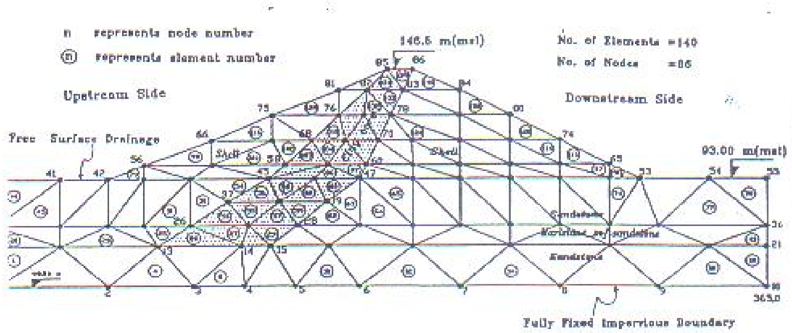

Fig. (11) Finit Element Mesh Used to Analyze The Great Earth Dam.

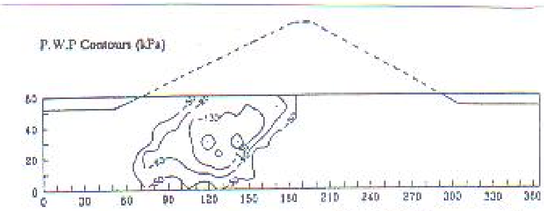

Fig. (12) PWP Contours,Const. with w.c=18.5 / ,1st lift,(-ve compression).

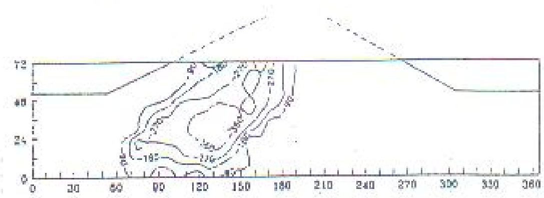

Fig. (13) PWP contours ,Const. w.c=18.5 / ,2nd lift,(-ve comp.)

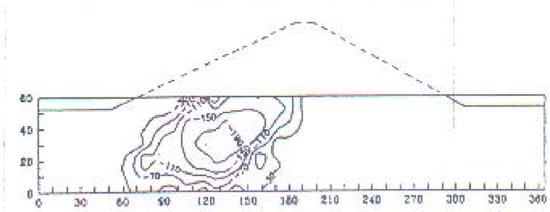

Fig. (14) PWP Contours,Const. w.c=20.5 /,1st lift,(- comp.).

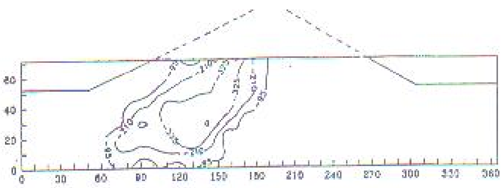

Fig. (15) PWP Contours,Const. w.c=20.5 /,2nd lift,(- comp.).

P.W.P Contours (kPa); -ve means compression

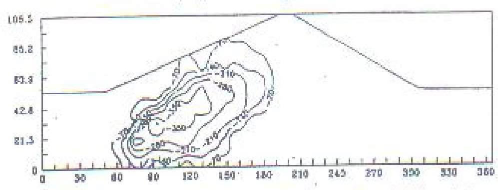

Fig. (16) PWP Contours ,Const. w.c=18.5 /,W.L at Elev. 115(filling stage).

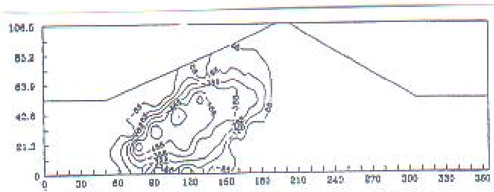

Fig. (17) PWP Contours,Const. w.c=20.5,W.L at Elev. 115.0 (-ve comp.).

Vertical Displacement (- ↓) (in meters)

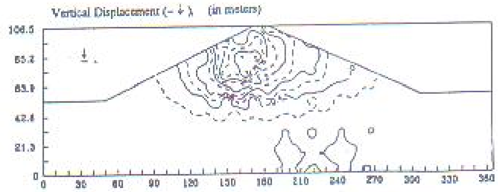

Fig. (18) Ver. Disp. Contours,Const. w.c=18.5 /,W.L at Elev. 115.

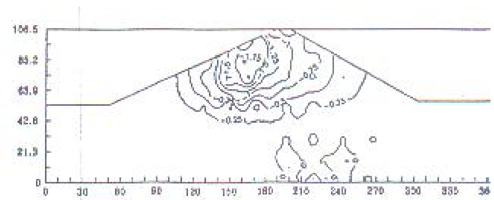

Fig. (19) Ver. Disp. Contours,Const. w.c=20.5,W.L at Elev. 115.0(-ve ↓)

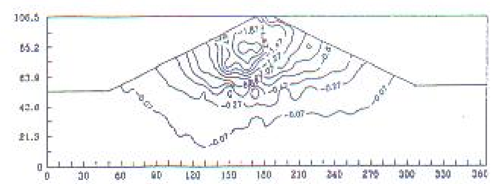

Fig. (20) Ver. Disp. Contours,Const. w.c=18.5/,W.L at Elev. 131(-ve ↓).

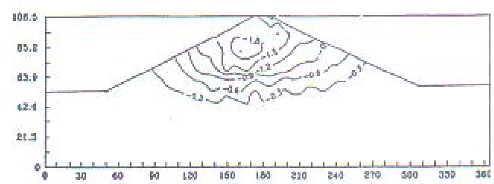

Fig. (21) Ver. Disp. Contours,Const. w.c=20.5,W.L at Elev. 131(-ve ↓).