Editorial Office:
Eurasian Journal of Science & Engineering
Ishik University, Erbil, Iraq
www.eajse.org

Eurasian Journal of Science & Engineering gratefully acknowledges the support of Ishik University.

Eurasian Journal of Science & Engineering is particularly indebted to Ishik University Research Center.

Composed by İrfan Publishing, Erbil, Iraq

Printed by Anıl Press, Gaziantep, Turkey

No responsibility for the views expressed by the authors in this journal is assumed by the editors or by Eurasian Journal of Science & Engineering.

EAJSE (Eurasian Journal of Science & Engineering) is published biannually (December, June) in both print and online versions by Ishik University.

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Geophysical Study of the Soil Surrounding Designed Submerged Concrete Tank Using Seismic Refraction Technique

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Received: February 19, 2016 Accepted: April 24, 2016 Online Published: December 1, 2016

Abstract: A shallow geophysical investigation using seismic refraction technique was conducted to specify soil layers quality surrounded a designed water storage tank as well as to detect whether any subsurface structural features like (weak zones, cavities or faults) present or not, if it is present, it will create a risk after constructing the project. For this purpose four profiles were conducted around the tank using seismic refraction technique. Interpretation results indicate the presence of three layers. The top soil layer has a velocity ranges between (200 to 665 m/sec), it refers to unconsolidated weathered layer submerged by a drift sediments along profiles No.(1) and (2), and the second layer with the velocity ranges between (800 to 1550 m/sec), which indicates of moderately cohesive soil layer. However, the third layer with the velocity ranges between (2930 to 4425 m/sec), which reveals to the highly compacted soil layer, these ranges of velocity will reflect increasing of density as well as strength of the soil layers with increasing depths.

The depth section under profile No.(1) from the first to the second layer from geophones No.(6) to (18) as well as for profile No.(2) from geophones No.(8) to (12) are revealed by the presence of the weak zone results from the submerged drift sediments including fissures and fractures. Processing of the weak zone can be done by removing of the first layer then constructing by a thick concrete foundation on the second layer as well as the supported concrete wall is necessary to avoid future risk, which may result from hydrostatic pressure on the tank.

Keywords: Seismic Refraction, P-Wave Velocity, Soil Layers, Weak Zones

1. Introduction

Seismic is most favorable method in the field of geophysics to identify the depth of subsurface without spending a huge sum of money by drilling. Accordingly seismic refraction is possibly the most important technique used in engineering geophysical investigation. It is possible to obtain information of the rock mass quality at an early stage during investigation, when specific data on jointing is lacking (Arild, 1996).

Anomohanran (2013) carried out a study to emphasize the ability of the seismic refraction method in determining the thickness of stratified layers of soil and rock. Seismic longitudinal wave velocities (Vp) were determined within four meters beneath ground surface which indirectly provided the critical subsurface information about depth of layers, morphology and stratigraphic sequence without borehole information,(Khan, 2013).Table (1) shows Velocity of primary (V_p) and Secondary (V_s) waves velocity for some soils and rock types.

Table 1. P- wave velocity (V_p) and S-wave velocity (V_s)

Materials	V_p (m/sec)	Vs (m/sec)
Top soil (dry sand)	200-1000	100-500
Top soil (wet sand)	1500-1900	200-500
Clay	1000-1800	50-300
Chalk	2100-4200	800-1300
Limestone	2000-7000	3600-max
Sand stone	2100-4500	1500-3000

2. Field Procedure

The studied area is located in western part of Sulaimani city as shown in Fig.(1). Field procedure was carried out along four profiles around the tank by using seismic refraction technique with (24) channels computerized ABEM Terraloc-Mk-II Seismic System,(ABEM, 1983). Profiles (No.1) and (No.3) were surveyed using a length of (69m) between short (A) and (B) with (24) geophones, the distance between each two geophones is (3m). Whereas profiles (No.2) and (No.4) were surveyed using a length of (115m) between shots (A) and (B), with (24) geophones, the distance between each two geophones is (5m). In all profiles also center shots were done, with off set shots in profiles (No.1), (No.2) and (No.4) for increasing depth of penetration.

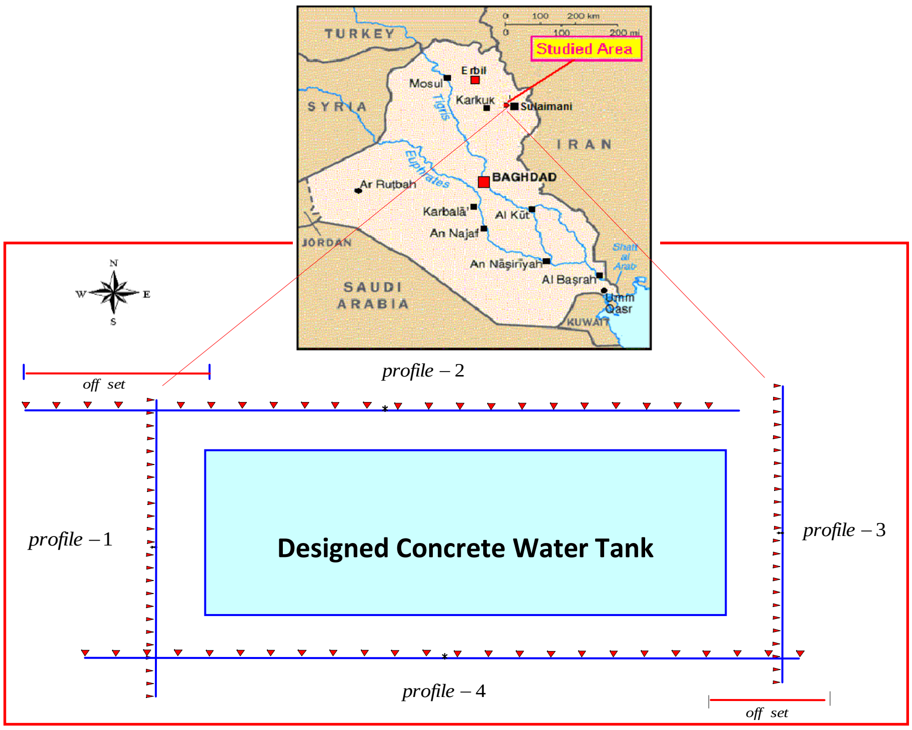

Fig.(1). Location map of the studied area with seismic refraction profiles around the tank

3.Seismic Refraction Theory

Seismic refraction involves using of seismic sources (explosive charge, weight drop or hammer blow) to generate elastic waves propagating into the subsurface with the velocity depending on the elastic properties of the material through which they travel (Griffiths & King, 1981), Fig.(2a). The horizontal variations of density and elastic properties are generally much less rapid than the variations in the vertical direction where we are going from bed to bed with consequent lithlogical changes and increasing pressure with increasing depth (Telford et al., 1990). In seismic refraction usually Time - Distance curve is drawn from arrival times against the shot point to geophone distances, the slope of the segments give the reciprocals of the true velocity (V), as shown in Fig.(2b). Through applying equations (1 to 3), the velocity of direct wave (V_o) and the refracted wave (V_1) can be calculated.

$$Where \qquad Slope = \frac{\Delta t}{\Delta x} \cdots\cdots\cdots\cdots (1)$$

$$V = \frac{1}{Slope} \cdots\cdots\cdots\cdots\cdots (2)$$

$$V = \frac{\Delta x}{\Delta t} \cdots\cdots\cdots\cdots\cdots\cdots (3)$$

Then the depth to the interface between two layers can be calculated from the intercept time arrivals of the refracted wave (t_o) and velocities can be calculated for the two successive layers (V_o) and (V_1) of horizontal regular interfaces (AlSinawi,1981), by the following equation:

$$h_O = \frac{t_O V_O V_1}{2(V_1^2 - V_O^2)^{\frac{1}{2}}} \cdots\cdots\cdots (4)$$

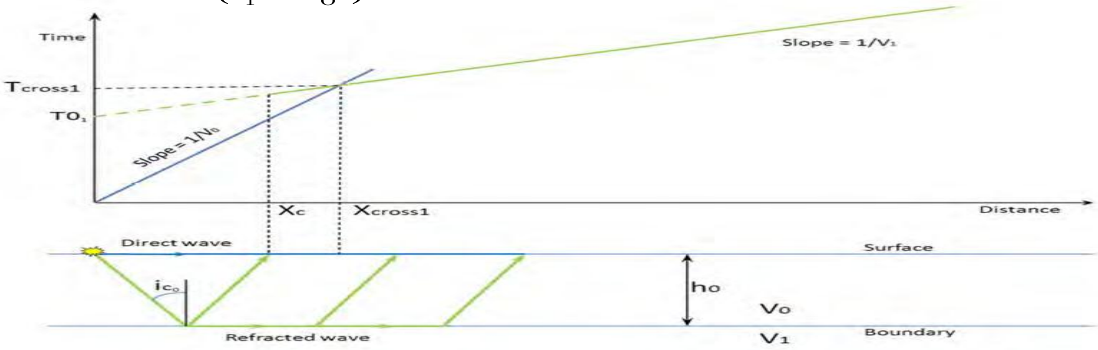

Fig.(2b). Time - Distance curve for direct and refracted waves in two horizontal layers

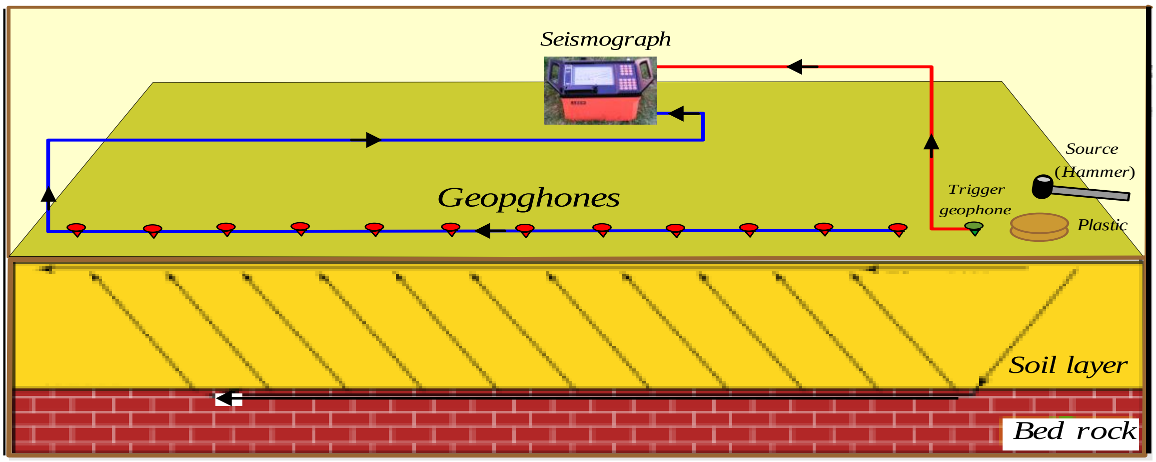

Fig.(2a). Schematic diagram of generating and detecting wave signals by spread geophones

But when the shape of interfaces is irregular the determination of the velocities and the depths become difficult and in this case (Hagedoorn’s, 1959) plus-minus method provides a simple and fast tool to interpret refraction data and calculate the geometry and velocity of the first refractor. The procedure is remarkably straightforward; the arrival times of the refracted waves from two reciprocal shots are simply added to find the depth to the refractor at all geophone stations and subtracted to find the velocity of the wave propagating through the refractor (Overmeeren, 2001).

The criterion for cavity detection delays in arriving times, when a wave front traversing a shallow cavity exhibits a measurable delay in the vicinity of the cavity. This is because acoustic impedance of the cavity is very much less than the surrounding material, this will cause a delay in arrival of direct and refracted waves, the width of the zone in which the delay occurs probably equals the greatest width of the cavity, this phenomena will cause the departure of time distance graph Fig.(3a) from its normal behavior Fig.(3b) with observed velocity being much lower than expected due to presence of the weak zone or cavity (AlKhafaji, 2004).

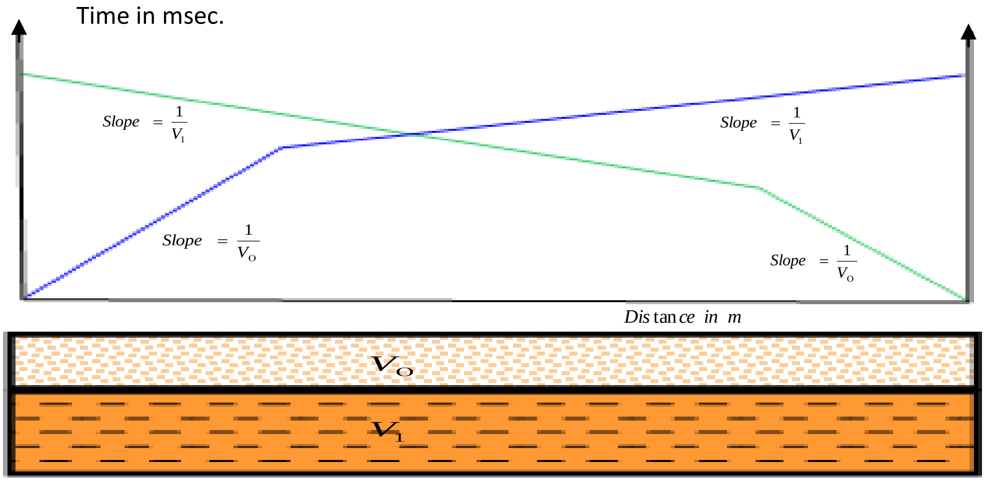

Fig. (3b). Time - distance curve for two horizontal layers

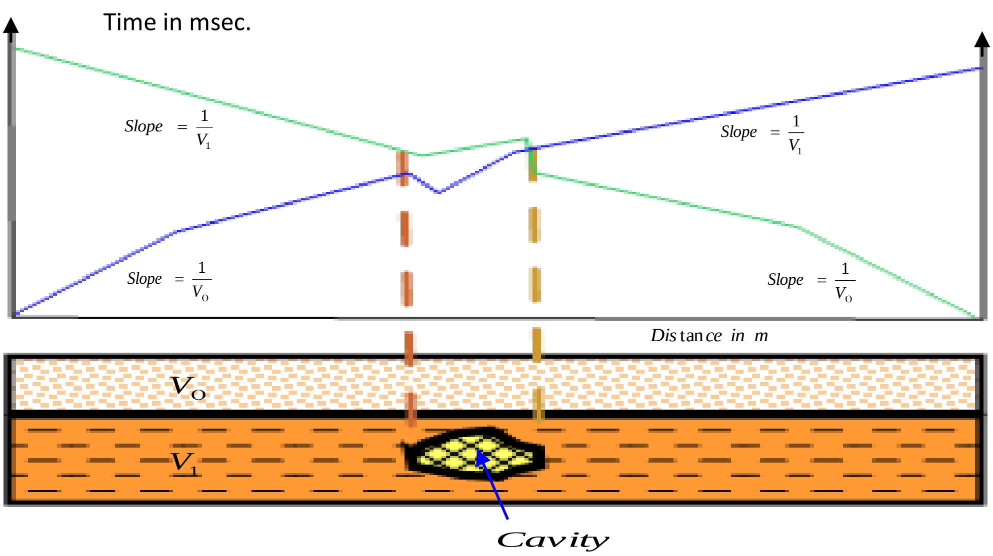

Fig. (3a). Time - distance curve for two horizontal with the presence of cavity in second layer

4. Interpretation and Results

The interpretation of the obtained data leads to the following results for the profiles:

Profile (1), Fig.(4)

Three layers under this profile are detected as follows:

1- The first layer velocity ranges between (V_o=200-600 m/sec) which reveals as unconsolidated weathered soil layer.
2- The Second layer with velocity ranges between (V_1=1545-1600 m/sec) and the depth of this layer is variable.
3- The third layer with a velocity of (V_2=2685-3525 m/sec) was recognized as a consolidated soil layer detected from the record by the offset shot. The depths under geophones are as follows:

Geophone No.	Depth (m.)
3 – 5	2 – 3
8 – 14	6 – 8
17 – 24	2 – 3

The area under geophones No.(6) to (18) indicates the presence of the weak zone due to the presence of the submerged drift sediments.

Profile (2). Fig.(5)

Two soil layers under this profile are detected as follows:

1- The first layer velocity is (V_o=540 m/sec) to the same unconsolidated soil layer of the profile - 1.
2- The second layer with the velocity of (V_1=3390-3575 m/sec) which indicates as a compacted. soil layer, the depths of the first layer is shown under geophones as follows:

Geophone No.	Depth (m.)
4 – 10	6.5 – 8
16 – 20	4 – 4.5

The second layer in this profile between geophones No.(8–12) indicates the presence of weak zone results from the presence of the same submerged drift sediments**.**

Profiles (3). Fig.(6)

There are three subsurface layers under this profile as follows:

1- The first layer with a velocity of (V_o=285m/sec) is shown as unconsolidated layer.
2- The second layer with the velocity of (V_1=1170m/sec).
3- The third layer with the velocity of (V_2=4225m/sec), which reveals to the compacted soil layer**.**

Geophone No.	Depth (m.)
9 – 10	7.5
12 – 13	4
14 – 15	3 – 3.5

Profiles (4). Fig.(7)

The surface layers under this profile were detected in addition to a thin layer:

1- The first layer with a velocity of (V_o=800m/sec) which was recognized as a natural soil layer.
2- The second layer with a velocity of (V_1=2930m/sec).

Geophone No.	Depth (m.)
2 – 8	3 – 8
10 – 16	4

Alhassan et al., 2015 obtained the basement surface varied in depth, from (10.16 m) to a maximum of (14.80 m.), weathered layer velocities ranging from (809 m/sec.) to (3612 m/sec.) and consolidated layer velocities varying between (2858 m/sec.) to (9696 m/sec.) which are coincidences with the soil layers velocity ranges of the present study.

The profiles are drawn by software Surfer mapping system (Surfer-8, 2002).

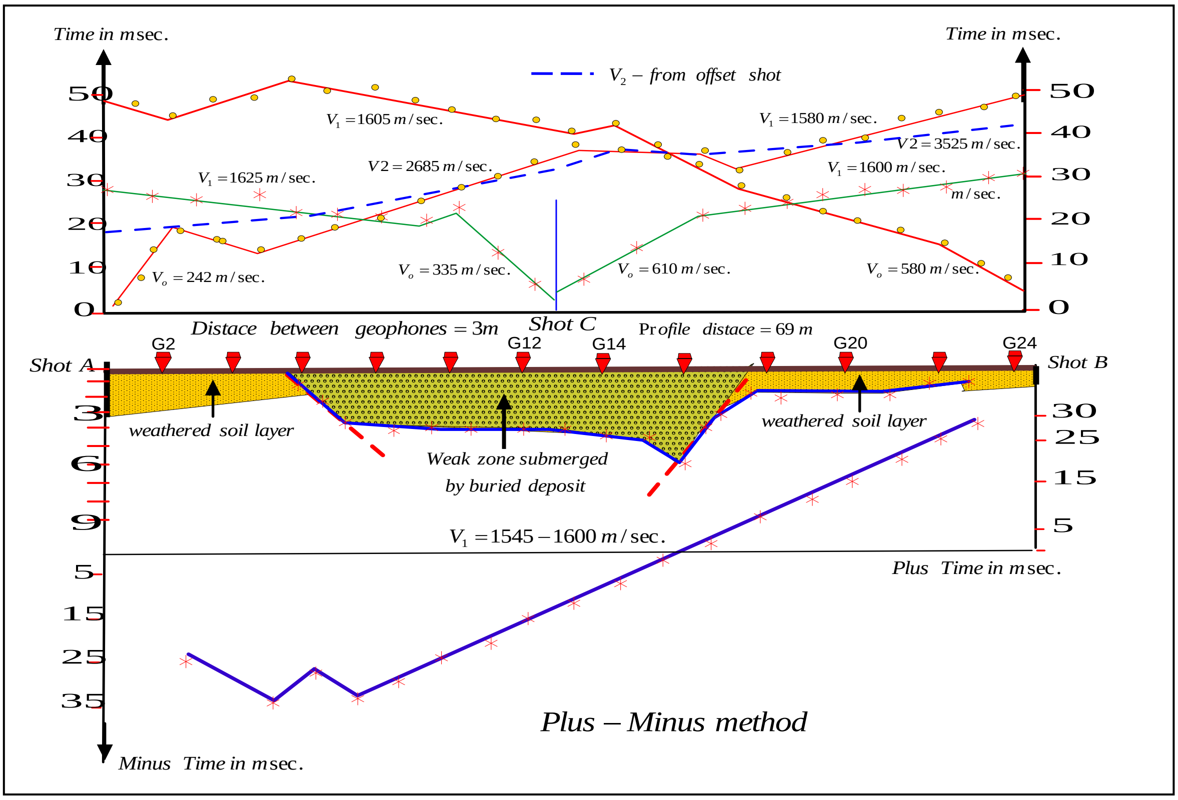

Fig. (4). Time – distance curve with applying Plus-Minus method for Profile No.1

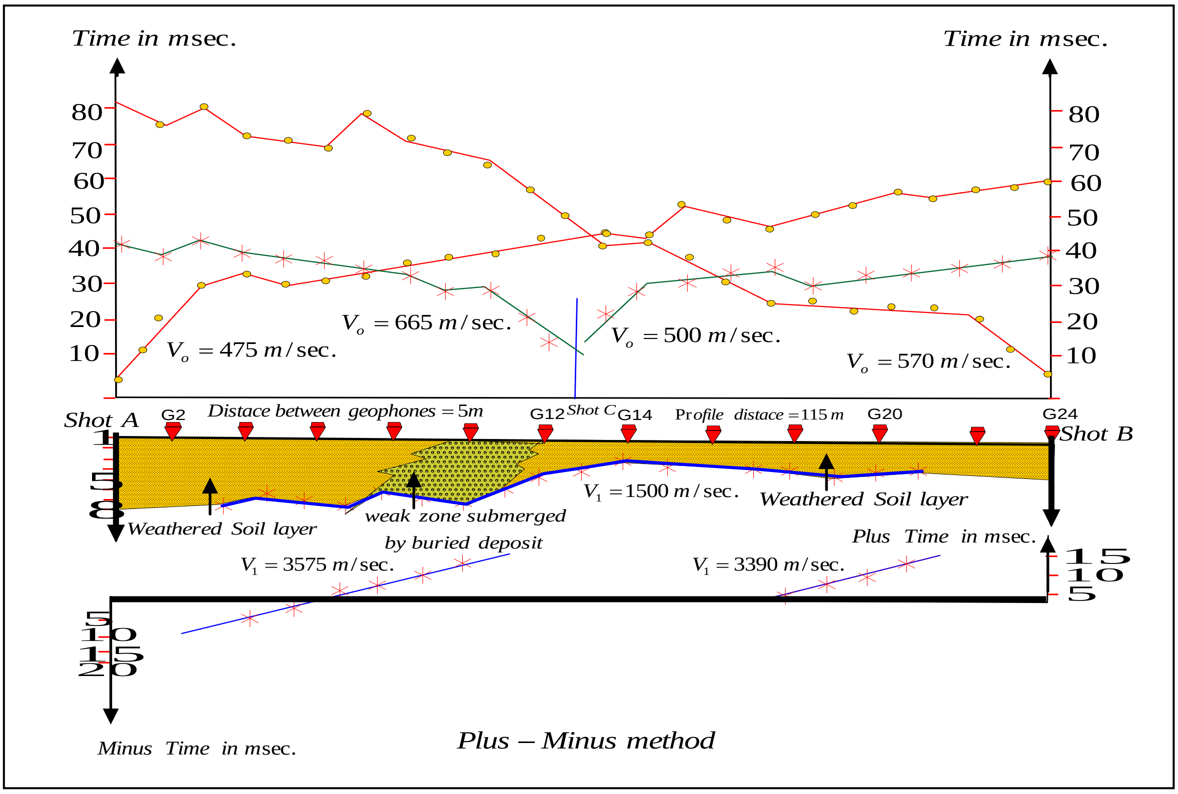

Fig. (5). Time – distance curve with applying Plus-Minus method for Profile No.**2**

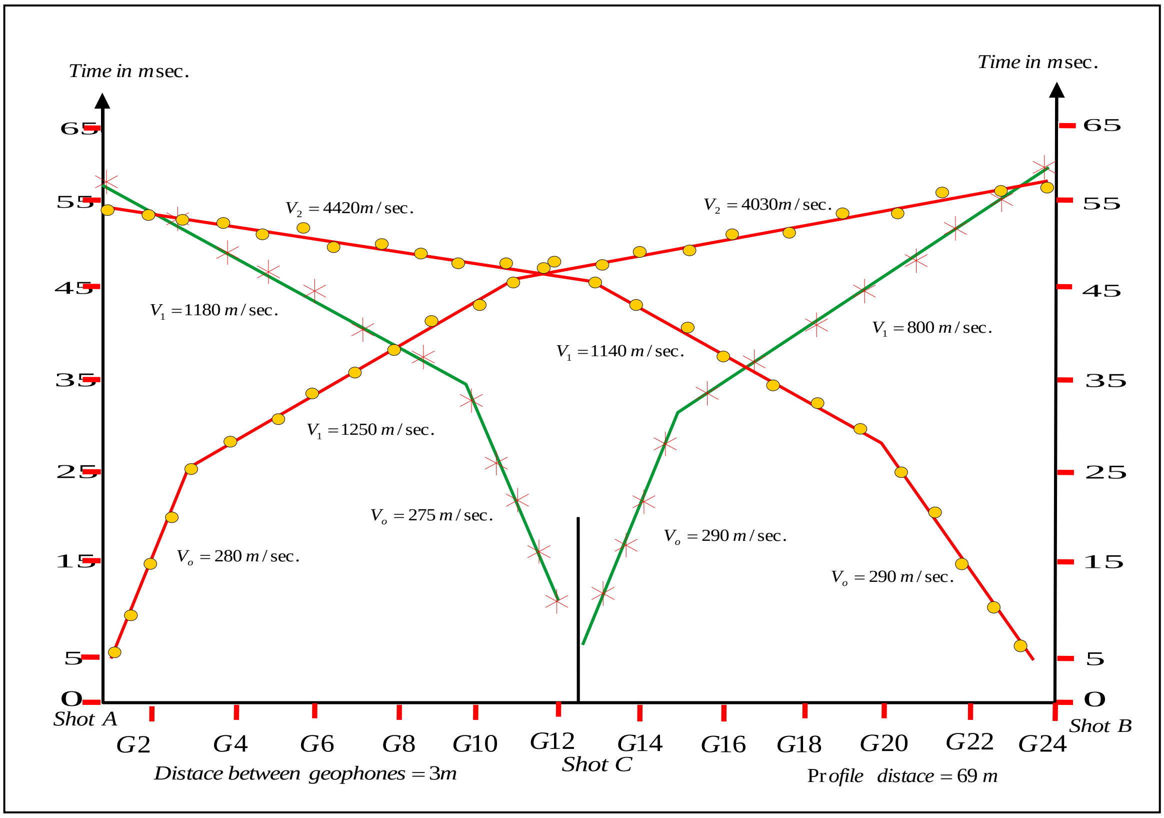

Fig. (6). Time - distance curve for Profile No. (3)

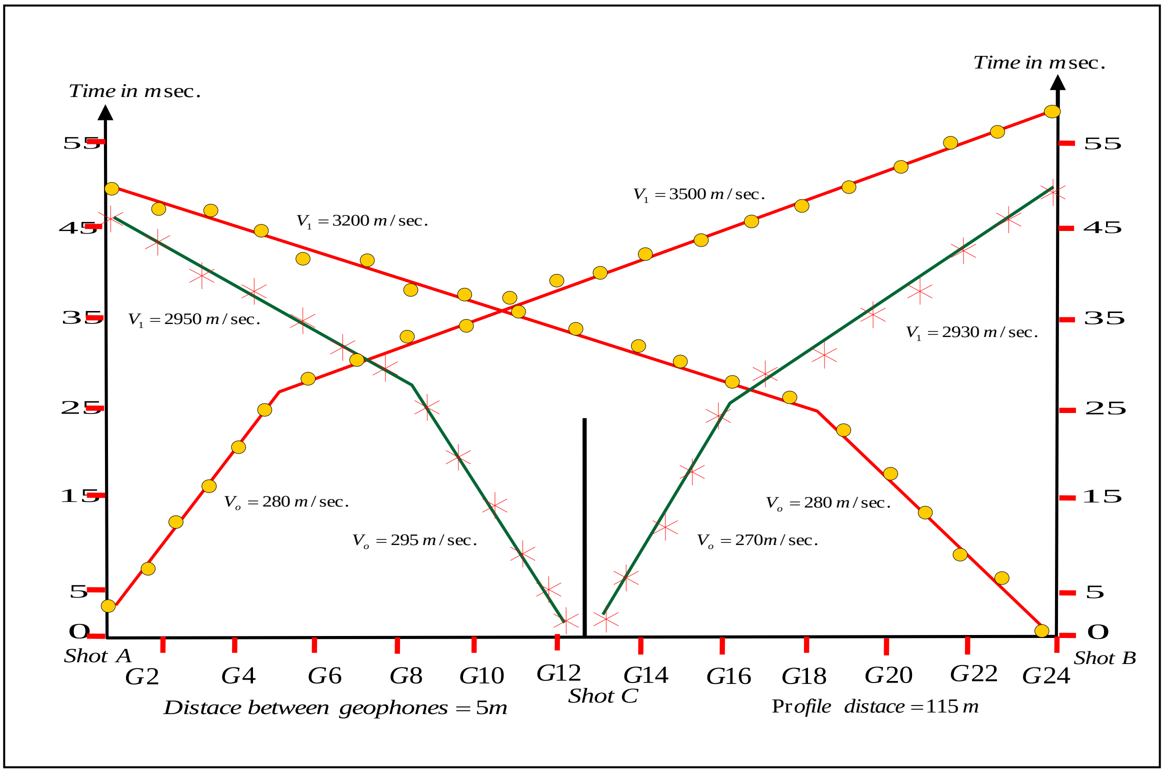

Fig. (7). Time – distance curve for Profile No.4

5. Conclusions

Interpretations of the results indicate the presence of three layers. The top soil layer has a velocity ranges between (200 to 665 m/sec), it refers to unconsolidated weathered layer submerged by a drift sediments along profiles No.(1) and (2), and the second layer with the velocity ranges between (800 to 1550 m/sec), which indicates of moderately cohesive soil layer. Whereas the third layer with the velocity ranges between (2930 to 4425 m/sec), which reveals to the highly compacted soil layer, these range values of velocity will reflect increasing of density as well as strength of the soil layers with the depth.

The depth section under profile No.(1) from the first to the second layer from geophones No.(6) to (18) as well as for profile No.(2) from geophones No.(8) to (12) are revealed by the presence of the weak zone for both profiles, which are resultant from the submerged of drift sediments including fissures and fractures. For the purpose of ensuring the presence of the weak zone as a submerged drift sediments, a hole at the mid-way of profile No.(2) was excavated, which clearly indicated unconsolidated buried of drift sediments. While the third layer is composed of well strength and compacted soil layer.

Processing of the weak zone can be done by removing of the first layer then constructing by a thick concrete foundation on the second layer as well as the supported concrete wall is necessary to avoid future risk, which may result from hydrostatic pressure on the tank.

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Effects of Soil Properties on Infiltrated Water Depth Profile

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Received: April 15, 2016 Accepted: May 18, 2016 Online Published: December 1, 2016

Abstract: Level Border system is one of the most common effective methods of applied water to the land, the border is bounded by levees on both sides and settlement horizontally without slope in the direction of flow. The end of border is blocked and no losses due to surface runoff occur. In this way the required quantity of water is delivered and allows moving forward along the border and penetrating the soil by infiltration process. And the phenomenon of infiltration is a very important factor which controls the amount of water stored in the soil, and consequently, the estimation of water distribution profile will be useful to indicate the efficiency of the application and distribution uniformity which both are required to assess the border regime. In this research, the estimating of infiltration profiles was based upon the applying of Kostiakov's Function which its coefficients and exponents results from the measurements of the double cylinder infiltrometer. Many points were selected to install the double cylinder infiltrometrs, hence, the characteristics of infiltration had been studied.

The prediction of infiltration function needs to know the flow rate, time of application, advance and exponent function. The comparison between the results obtained from the average infiltration function applied for all considered points and those estimated from direct measurements reveal a poor agreement and lead to non-reliable results for the application efficiency and distribution uniformity indications. The main conclusions derived from the study appear that the behavior of subsurface water movement is based mainly upon the soil properties and independent to some extent for amount of water and time of application.

Keywords: Surface Flow, Infiltration, Application Efficiency

1. Introduction

The level of border regime is a certain way, where is the water entry at the head of a uniform level and then pouring into two phases, the first when the front of the water that is advanced on the border along the surface and the second presents the infiltration process when the water penetrates vertically downward through the top layer of the soil. Infiltration mainly influenced by many elements, such as soil texture, quantity and time of water demand and the percentage of advanced water on the surface of the length of the field (Hadi, 1985).

2. Literature Review

Various actors have been studied in the design and operation of the border regime by many investigators. However, many works have been conducted using the equation of infiltration assessment that infiltration properties. Lewis and Milne (1938), used to balance the flow volumes to develop an expression to calculate the advance of water through the surface of the border as shown below:

$$q.T_x = d_a.X + \int_0^{T_x} z\,.\,(T_x - T_a)\,.\,X'.\,(T_a).\,dt_a \qquad (1)$$

Where

q: flow rate per unit width of border, m^3/min/m,
T_x: time of advance, min,
d_a: average depth of flow in the border ,m,
X: advance distance of water during given time, m,
Z: accumulating depth of infiltration during opportunity time, m,
t_a: applied time of water ,min, and
X': the value $\frac{dx}{dt_a}$ at $t = t_a$.
Christiansen (1966), using the form below to express accumulated of infiltration depth, the equation is so called as modified Kostiakov function for infiltrations depth:

$$Z = K.t_o^m + C.t_o \qquad (2)$$

Where
t_o: opportunity time ,min,
K, m and C: coefficients and exponent of function.

The study of the Application efficiency in the level border is carried out by Ahmed (1990). The study was based upon the following form of Kostiakov function of infiltration (1932):

$$Z = K.t_o^m \qquad (3)$$

The estimation of infiltrated depth of subsurface water has been studied by Ahmed 2014 [1], the analysis of measurements was based upon the modified infiltration function .The results were compared and with those obtained from old infiltration function and also with the actual measurements. The difference in values not in profile patterns was found in the results of infiltration functions and both them were much differ with actual measurements due to their profiles and values.

In the current research, the equation (3) was used to estimate the infiltrated depth profiles for different values of flow rates, time of applications and accordingly, the application efficiency (AE) and distribution uniformity (DU) are estimated .The results approaches of measurements are analyzed by using two approaches, average infiltration function was applied for all considered stations of measurements and direct measurement of infiltrated depth for each station. The results are compared and their effects on the values of (AE) and (DU are studied.

3. Methodology

The present research based upon many assumptions, it was considered that the soil is homogeneous across and along the border, flow delivered to the border is a constant during the application period rate, border level is absolutely prohibited in the end, so no losses due to runoff, the longitudinal surface profile during the advanced stage can be represented by the form and function of the quarter ellipse, initial soil moisture along a border was fixed, accumulated depth of infiltration can be represented by power modified infiltration function and the advance process can be estimated by power function.

The results of current research obtained from actual measurements were carried out by Ahmed 1990. The measurements were included discharge delivered to borders, advance time of water front, cutoff time, recession time and the direct measurements of infiltration depths. The depth of infiltration were based upon the results of direct measurements of double cylinder infiltrometers which were installed for many stations along the borders and hence, the average coefficients and exponents of infiltration function was estimated for each run to get average function which is used to determine the infiltrated depth for all points considered for measurements. Figure (1) shows general view of border and components of double-cylinder infiltrometer.

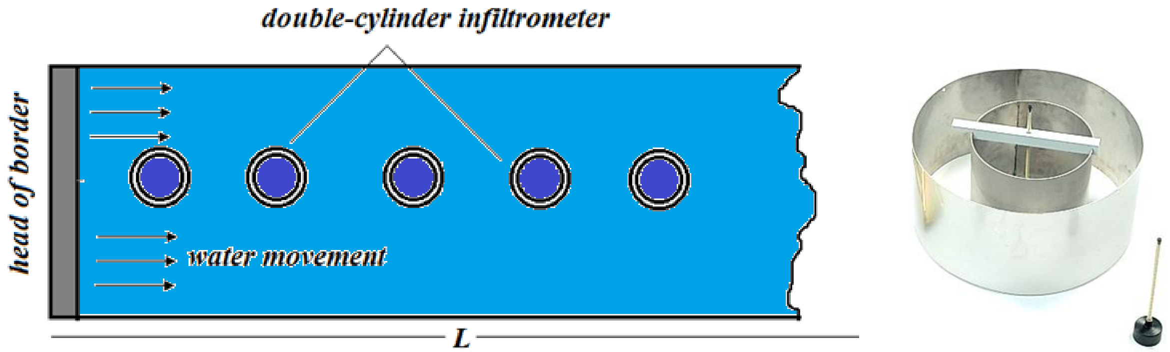

Figure (1). General view of border and Components of Double-Cylinder Infiltrometer.

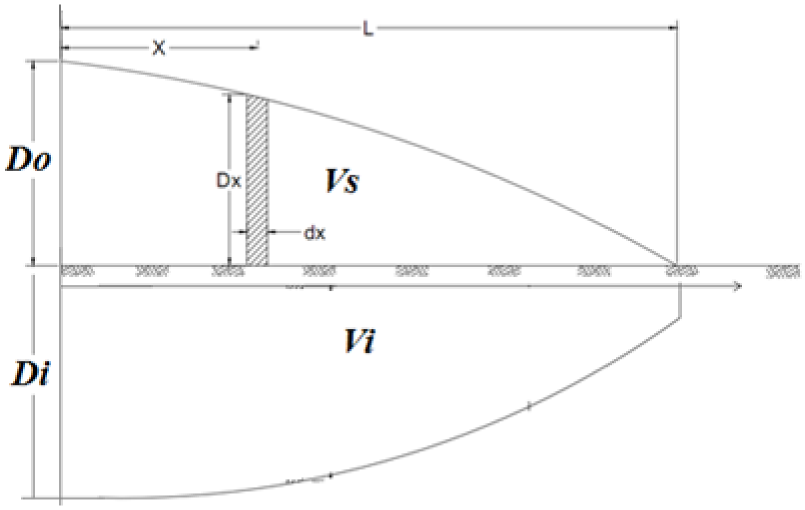

Figure (2). Longitudinal section of surface and infiltrated water volumes along the border surface.

$$D_x = \left(1 - \frac{X^2}{L^2}\right) . D_o \qquad (4)$$

Where
D_o: depth of flow at the head of border, m,
D_x: depth of flow at any distance X from the head of border, m,
X: any distance X from the head of border, m and
L: total advance length during a given time t_x, min.

The volume of surface water can be written as:

$$V_S = 0.8\, D_o\,.B.L \qquad (5)$$

Then, the volume of infiltrated water stored below the border surface can be written as:

$$V_i = K.B.T^m.L.\left[1-\frac{m}{b+1}+\frac{m(m-1)}{2(2b+1)}+\frac{m(m-1)(m-2)}{6(3b+1)}+C.B.T-\frac{1}{b+1}\right] \qquad (6)$$

$$F_1 = \left[(L-X_n)-\sum_{i=1}^{N}\frac{(-1)^i\,a^i\,[L^{ib+1}-X_n^{ib+1}]}{i\,!\,T^i\,(ib+1)}.\ \frac{m\,!}{(m-i)!}\right] \qquad (7)$$

The application efficiency can be predicted by the following equation (Ahmed, 1990):

$$AE = \frac{d_n\,.[\frac{T-\left(\frac{d_n}{K}\right)^{\frac{1}{m}}}{a}]^{1/b}+K\,T\,F_1}{q\,t_a}.\ 100 \qquad (8)$$

Where

dn: depth of water surface at the beginning of border.

a,b: are the coefficient and exponent of advance function.

$$F = \left[1-\sum_{i=1}^{N}\frac{(-1)^i\,a^i\,(\frac{3L}{4})^{ib}}{i\,!\,T^i\,(ib+1)}.\ \frac{m\,!}{(m-i)!}\right] \qquad (9)$$

The distribution uniformity (DU) can be estimated by the following expression (Ahmed, 2014):

$$DU = (4-\frac{3\,K\,T^m\,L\,F}{q\,t_a}).\ 100 \qquad (10)$$

4. Results and Discussion

In this research, the study of the infiltration properties along the level borders was carried out by using the Kostiakov function, the coefficients and exponents of this equation are based on the measurements of double cylinders installed in many locations along the border, such device is shown in figure (1). The prediction of infiltration functions for all runs are used to plot of the subsurface water front, which is seem to be uniform for the average infiltration function as shown in figure (2) which is used to derive the equation of distribution uniformity (DU), Eq.(8) and application efficiency (AE), Eq(10).

The infiltrated profiles of considered runs are shown in Figures from (3) to (8). It can be seen from these figures that the direct measurements of infiltration depths produced profiles with high rate of fluctuation, and refer to a significant effects by the soil properties variation along and across the border. Also, the figures indicate that the use of the average infiltration function led to produce a uniform varied profile from the head to the end of border. Such discrepancy between these two types of profiles indicates that the use of only one infiltration function was not enough to give representative infiltration characteristics of the soil, as may expected due to soil non- homogeneity and anisotropy.

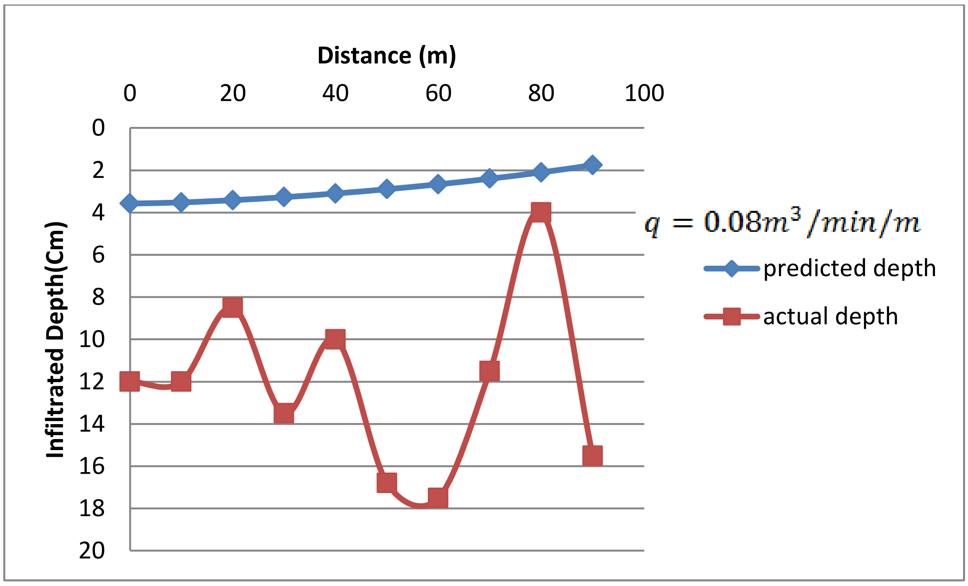

Figure (3). Variation of Infiltrated Depth Profile for Run No.1

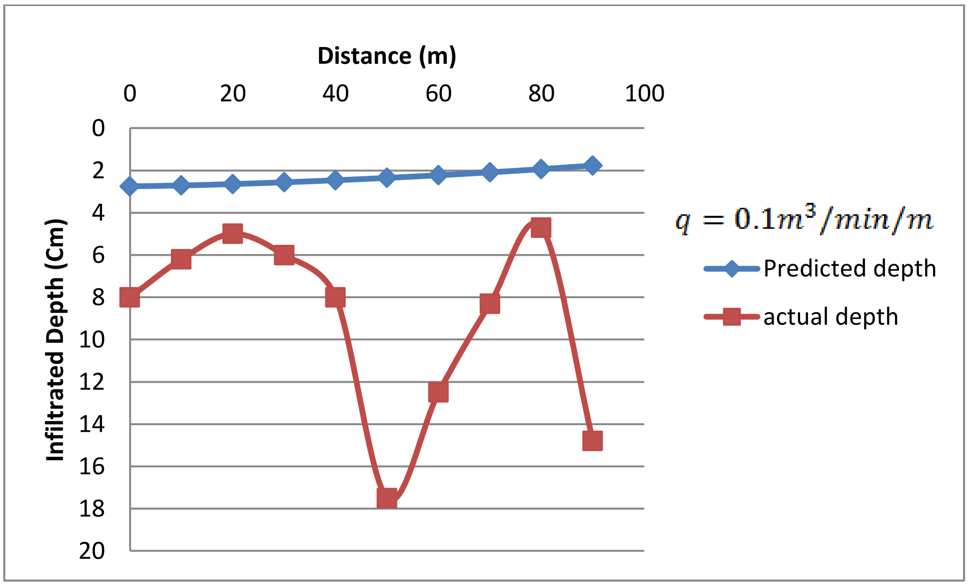

Figure (4). Variation of Infiltrated Depth Profile for Run No.2

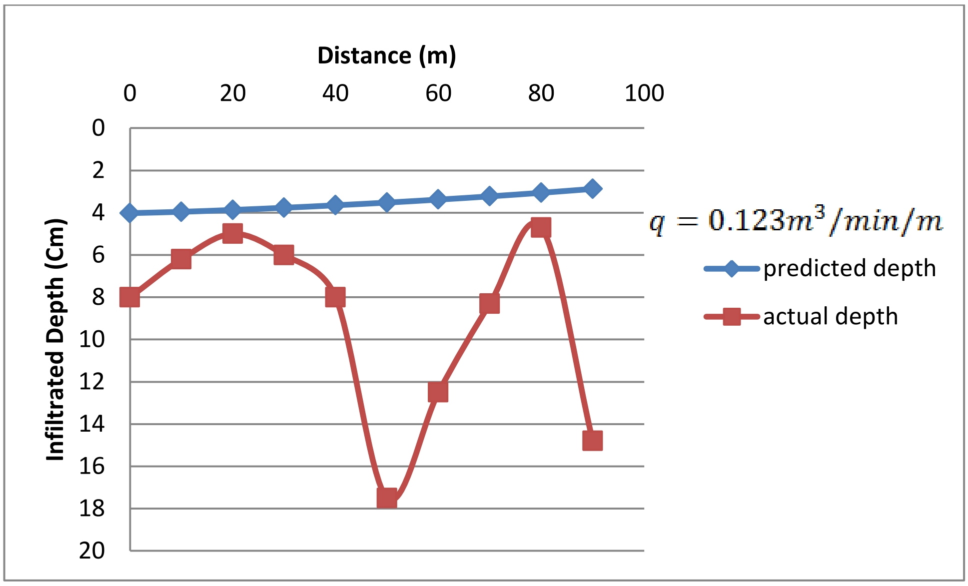

Figure (5). Variation of Infiltrated Depth Profile for Run No.3

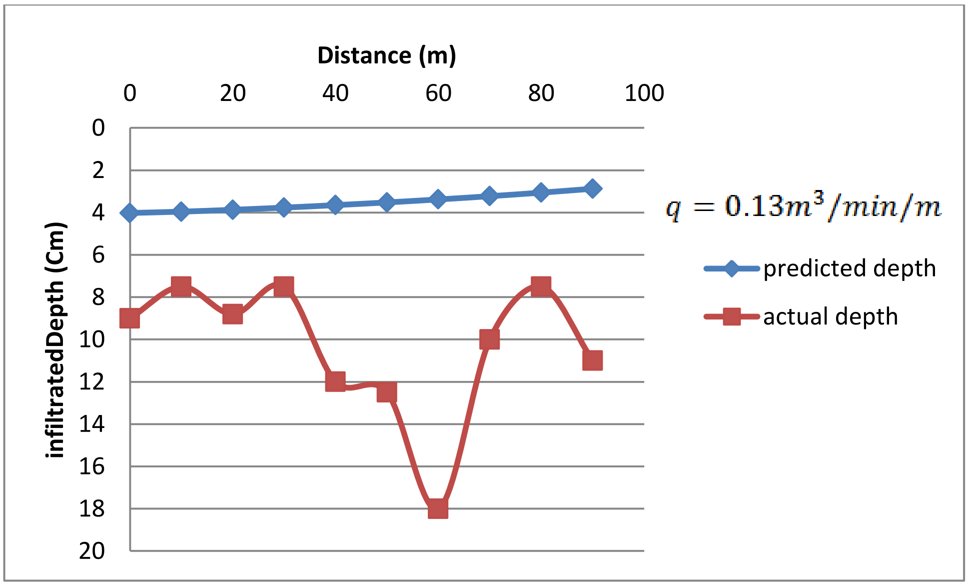

Figure (6). Variation of Infiltrated Depth Profile for Run No.4

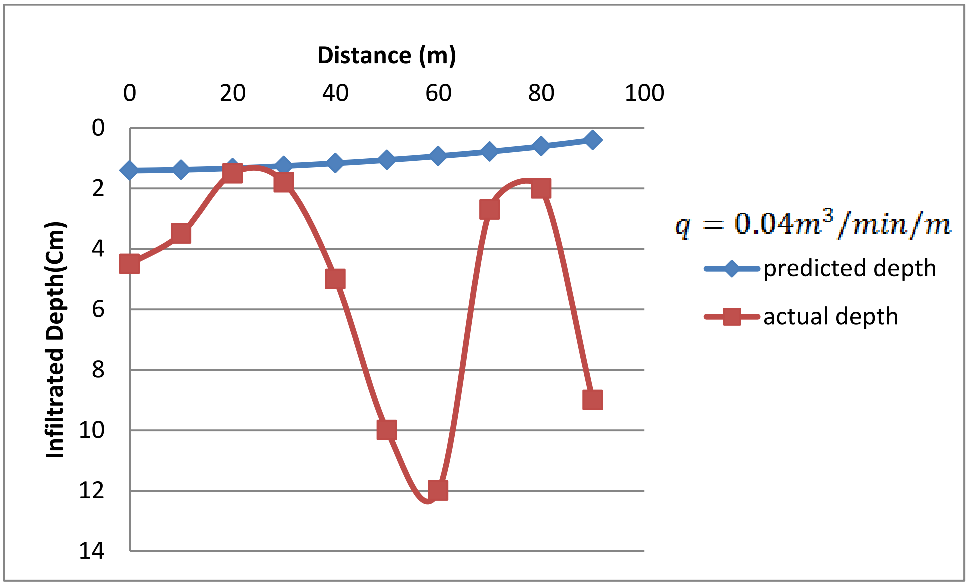

Figure (7). Variation of Infiltrated Depth Profile for Run No.9

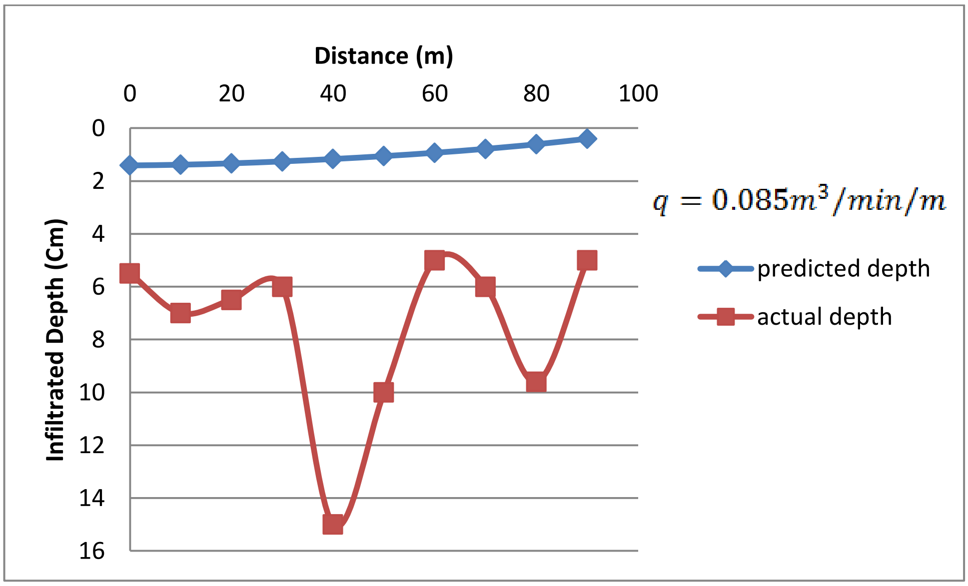

Figure (8). Variation of Infiltrated Depth Profile for Run No.10

Figure (9) shows the effects of discharges on the patterns of infiltration profiles. It can be seen from this figure, that the all profiles have followed the same trend and the changes are due to the value of discharges, so that, the increase of flow rate lead to increase the depths of infiltration accordingly.

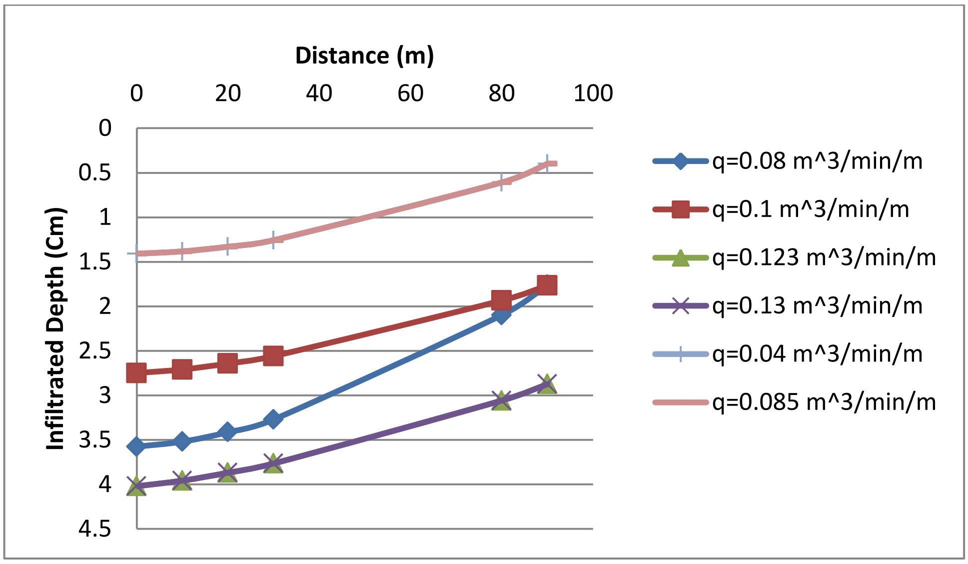

Figure (9). Variation of Infiltrated Depth Profile for many Runs

The application efficiency (AE) is one of major parameters used to evaluate the performance of the water application as it indicates the water quantity losses. It is influenced by discharge, length and slope uniformity of border, soil characteristics and time of application. Eq. (8), based upon the results of methods, average infiltration function and direct measurements, is used in current study to estimate the values of (AE) for different discharges and application times. It can be noticed from figure (10) that for average infiltration function, the values of (AE) are seemed to be has a uniform inversely variation with discharge. The measured (AE) appears a clear fluctuation. It can be observed from figure which refer to intensive effects of soil properties non-uniformity.

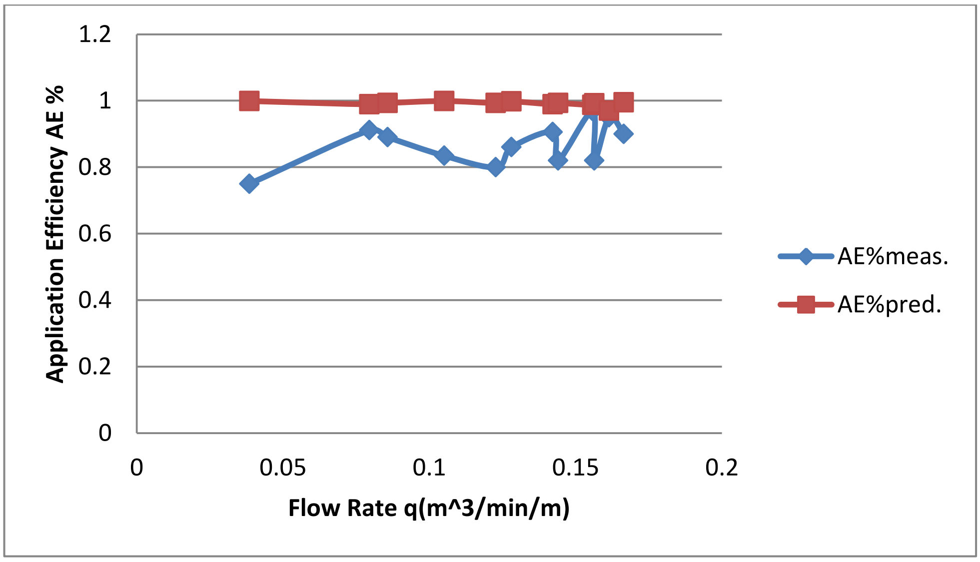

Figure (10). Predicted and Measured application Efficiency

The distribution uniformity (DU) of sub-surface water in level border systems depends upon discharge, application time, and soil and infiltration characteristics. The values of (DU) are found by using the results of average function and direct measurements of infiltrated depths. Figure (11) shows the variation of (DU) with different values of discharge. The figure indicates that for predicted infiltrated depth profile, the (DU) has varied uniformly with directly proportional changes. The measured values of (DU) are fluctuated and reflect the variation of soil properties.

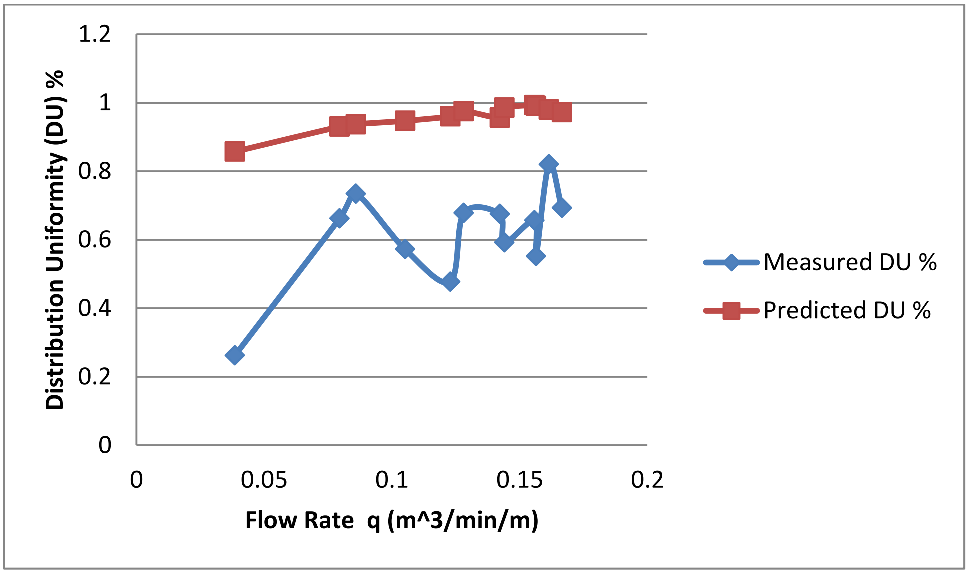

Figure (11). Predicted and Measured Distribution Uniformity.

The predicted values of application efficiency (AE) and distribution uniformity (DU) cannot be considered as accurate because of non-homogeneity and anisotropy of soil which mostly control the penetration rate and distribution of subsurface water movements. Hence, it is found from the figures (12) and (13) that if the predicted values of (AE) are multiplied by about (80% -90%), it will be very close from measured values .Whereas, if the predicted values of (DU) are multiplied by (50%-60%), it may produce more accurate results.

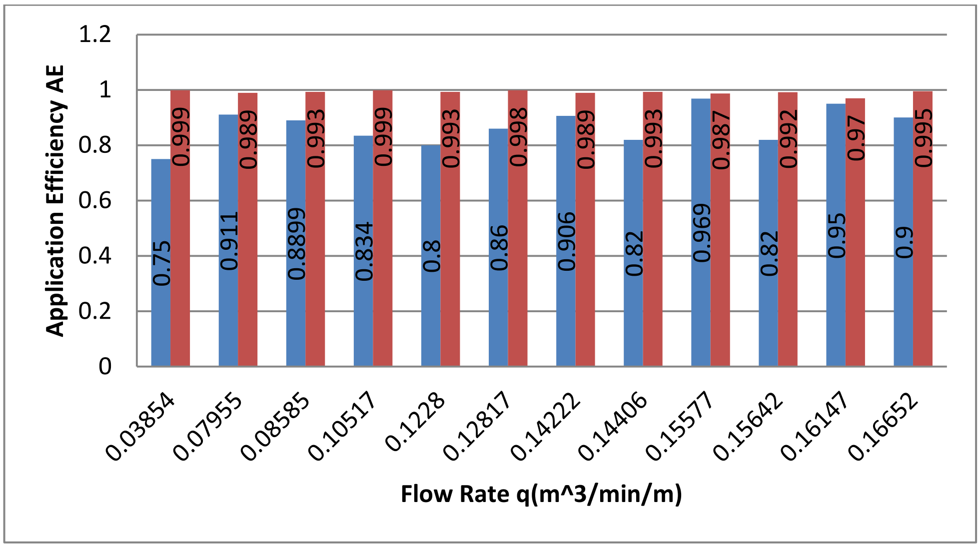

Figure (12). Predicted and Measured application Efficiency

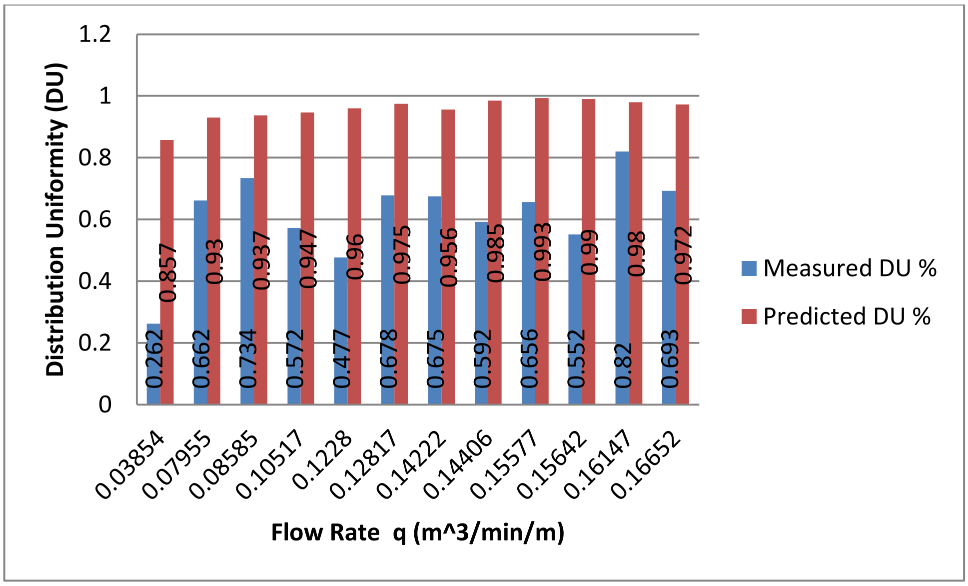

Figure (13). Predicted and Measured Distribution Uniformity

5. Conclusion

The process of infiltration is one of the main factors used to assess the level of border systems. In current research, the infiltrated depth profiles, application efficiency (AE) and distribution uniformity (DU) were studied and the results of two methods of infiltration characteristics were used, first was the direct measurements for many stations along the border and second by using the average infiltration function which is considered to be valid for all points of measurements. The following conclusions are observed from the results of study:

1-The use of only average infiltration function tend to yield large discrepancy between the measured and predicted infiltrated water profile. Such big differences between these two profiles related directly to the significant effects of the non-homogeneity of soil along and across the border.

2-The values of infiltrated depth are seemed to be influenced by the discharge, in which, when the discharge increase the infiltrated depth is consequently increased.

3-The slight effects of discharge on predicted values of application efficiency is appeared. Whereas, those obtained from the uses of average infiltration function are seem to be independent of discharge and however, only the variation of soil properties affected the values of (AE).

4-The discharge is affected the predicted distribution uniformity (DU) and whereas has no effects on measured values. Therefore, (DU) is influenced only by soil properties.

5- It is found from the general concept of analysis that if the predicted values of (AE) and (DU) are multiplied by about (80% -90%) and (50%-60%) respectively they will be very close from measured values and it will be useful in evaluation process for border system.

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Fresh and Mechanical Properties of Alum Sludge incorporated Concrete

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Received: March 9, 2016 Accepted: May 12, 2016 Online Published: June 1, 2016

Abstract: Alum sludge is a by-product material produced from' water treatment plants that use aluminum salts as a primary coagulant, and is the most widely generated water treatment residual/sludge worldwide. Disposing this material has environmental impacts. Therefore, it is necessary to reuse this waste material in such a way, to reduce its adverse effects on the environment. This research studies the effect of alum sludge powder as partial replacement of Portland cement, on the fresh and mechanical properties of high strength concrete. In this study 60 concrete samples were prepared and subjected to compression, splitting tension and flexure tests. For compression test 7, 28, 90 day moist cured 100 mm cube specimens were used while for splitting tension and flexure tests 100* 200 mm cylinders and 100*100*500 mm beams moist cured for 28 days were used respectively. For this purpose concrete mixtures containing 5, 10, 15% alum sludge as a cement replacement, beside the control Portland cement mixture were prepared. The test results showed that despite the slight reduction in strength among the replacement of cement alum sludge, high strength concrete having 28 day compressive strengths up to 72 Mpa can be produced.

Keywords: Alum Sludge, High Strength Concrete, Sustainability, Compressive Strength, Flexural Strength, Splitting Tensile Strength

1. Introduction

There is an enormous challenge towards using environment friendly materials in engineering sector. Environment is a big issue to be considered in every area especially in engineering because it has a direct effect on our life. Restudying construction materials and considering its effects in our environment is a crucial matter. For achieving this task, evaluating waste materials in construction sector is the key concern of many researchers. This research focuses on replacing cement by alum sludge which is a waste material produced from water treatment plants. The study introduces a new material that can be used in concrete for enhancing its properties.

2. Literature Review

Alum sludge is a by-product used in water treatment and discharged to the river by a massive amount, which cause pollution. Producing an environmentally friendly concrete in construction by manipulating alum sludge in it reduces both the pollution and the amount of cement used in concrete. Beside bring environmental benefit using alum sludge in concrete improves the mechanical properties as well (Owaid, 2013).

Alum sludge contains 39% aluminum after coagulation (Evuti & Lawal, 2011). Aluminum is very toxic to aquatic life and causes Alzheimer disease. Taking environmental regulation in to consideration, minimize waste products and reusing them, as secondary material is essential for saving environment from their adverse effects (Breesem et al., 2014; Lin, 2008).

A massive amount of sludge is produced by water treatment plants toxic for biotic and reusing them in partial cement replacement is suggested. Construction techniques are developing toward environmentally friendly approach (Rajin, 2015; Owaid et al., 2014).

In the last four decades, the compressive strength of commercially produced concrete has approximately tripled, from 35 MPa to 95 MPa. This significant increase in strength was largely due to the development in chemical admixture technology and the increased availability of supplementary materials, beside the increased knowledge of the principles governing the higher strength concrete (Neville, 2005). For the purpose of defining the term ‘high strength concrete’ in practice, ACI committee in its revising version specified the lower limit of high strength concrete as 55 MPa (Ibrahim et al., 2012). This definition can be considered as a reference for the HSC for the present time until new regulations will be issued.

For producing high strength concrete using mineral admixtures is essential (Ibrahim, 2011; Testing hardened concrete, 2009). Nowadays reusing waste materials instead of the industrial material is become predominant in construction sector. The aim of this research is using Alum Sludge as a partial replacement of cement for producing more environment friendly high strength concrete.

3. Materials and Methods

Ordinary Portland cement type I (42.5 Mpa) obtained from Mass Company was used in this research. The fine aggregate that was used in this research was natural river sand with a specific gravity of 2.7. Gravels with the maximum size of 12.5 mm and specific gravity of 2.67 were used as a coarse aggregate. The super plasticizer used in this research was high performance polycarboxylic based type, under the trade name of Hyperplast PC175. The alum sludge was brought from (Taq Taq water treatment plant) in a slurry form. The material was dried out in oven at a temperature of 100 ^{0}C for 24 hrs. The dried alum sludge was milled in a rotary steel ball mill, and sieved on a sieve No.200 (75µm).

Concrete mixture containing three different percentage of alum sludge as cement replacement (%5, %10 and %15) was prepared beside the control mixture. The water cement ratio for all mixture was fixed to 0.3, to keep this rate constant and super plasticizer was used for attaining proper workability. The mix proportions for 1m^3 of concrete are shown in the Table 1.

Table 1. Mix proportions for $1m^3$ of concrete.

Mix.	Cement Kg.	Gravel Kg.	Sand Kg.	Water Kg.	Alum sludge Kg.	Super plasticizer Kg.	Slump mm
P	480	750	1050	144	0	5	60
S1	456	750	1050	144	24	6.24	50
S2	432	750	1050	144	48	8	40
S3	408	750	1050	144	72	10	40

From each concrete mix 9, (100*100*100mm) cubes, 3 (100*200mm) cylinders and 3 beams (100*100*500mm) were casted. The cube specimens were put in water for 7, 28 and 90 days, while the cylinders and beams cured only for 28 days. The compression test was performed according to (Standard Test Method for Splitting Tensile Strength of Cylindrical Concrete Specimens, 2011) while indirect tension and flexure tests were performed according to (Standard test method for flexural strength of concrete (using simple beam with center-point loading, 2002; Standard Test Method for Flexural Strength of Concrete (Using Simple Beam with Center-Point Loading, 2003) respectively. The experimental part for this research was performed in materials' of construction lab Faculty of Engineering, Koya University.

4. Results and Discussion

4.1 Fresh Properties of Concrete

The slump values and the super plasticizer amounts for the control mix and that containing alum sludge is given in Table 2 and Fig.1. The slump for the control mix is 60 mm and by adding alum sludge a significant decrease in slump was recorded. The slump reduction became more significant by increasing the amount of alum sludge. For compensating a part of the slump loss, higher amounts of super plasticizer were used.

The slump loss for the mixtures containing alum sludge can be attributed to the capability of alum sludge in absorbing the mixing water of concrete.

Table 2. Slump values for concrete mixes

Mix	Slump (mm)
P	60
S1	50
S2	40
S3	40

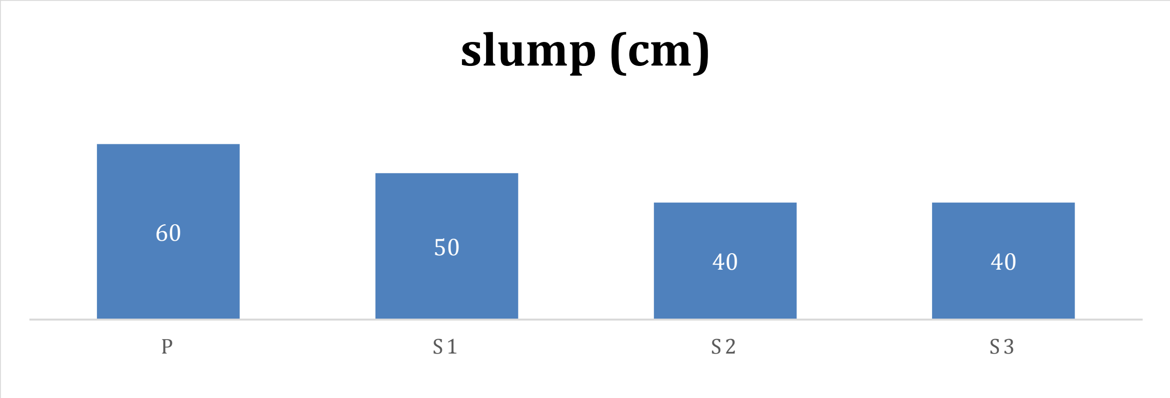

Figure (1). Relation between slump values and concrete mixes

4.2 Compressive Strength

Table 3 and Fig. 2 show the compressive strength results for concrete specimens moist cured for 7, 28 and 90 days. From the results it can be seen that the addition of alum sludge causes a reduction in compressive strength for all curing regimes compared to the control specimen. The reduction in strength increases by increasing the percentages of alum sludge. All the specimens show higher strength results by increasing the curing time.

Replacing 5% of cement by alum sludge results in a reduction of 11 Mpa in compressive strength at 7 days when it compared with the control specimen at the same age. The recorded reduction in compressive strength is nearly 5 Mpa for 90 day moist cured specimens with compared to the control specimen. By increasing the replacement percentages the same previous behavior is detected. This phenomenon is highly related to the pozzolanic activity of alum sludge which needs more time.

Table 3. Compressive strength test results

Mix	% of Alum sludge	Fć (Mpa) at 7 days	Fć (Mpa) at 28 days	Fć (Mpa) at 90 days
P	0 %	72.16	78.58	80.35
S1	5 %	61	72.58	74.94
S2	10 %	55.73	68.56	69.1
S3	15 %	49.96	66.49	68.84

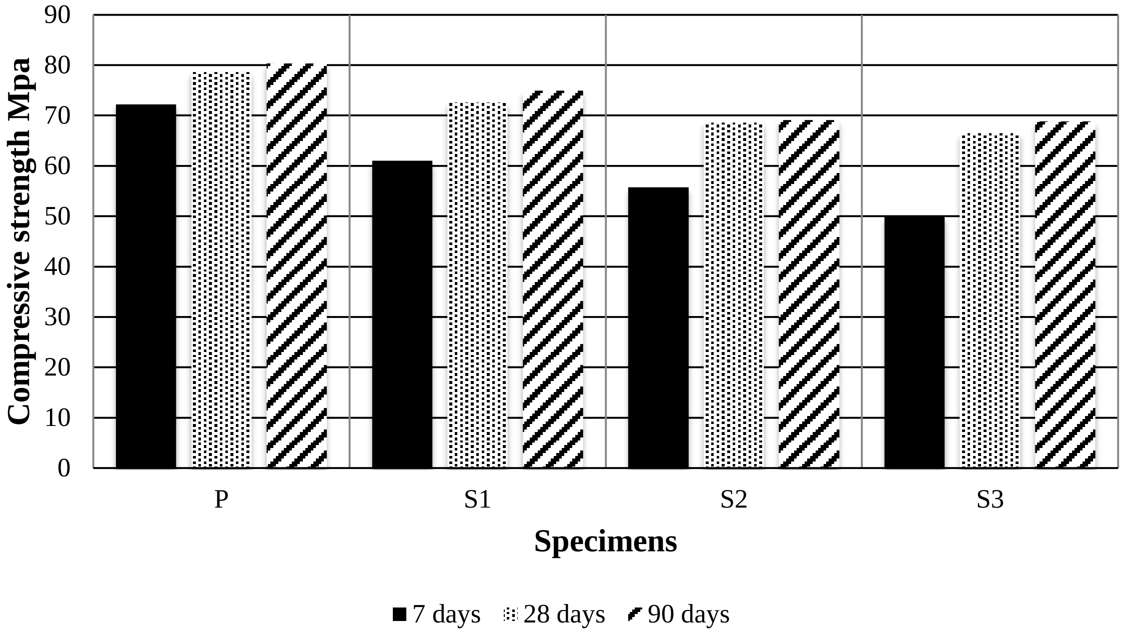

Figure (2). Compressive strength for control, S1 (5% alum sludge), S2 (10% alum sludge) and S3 (15% alum sludge contained) concrete cubes

4.3 Splitting Tensile Strength

Table 4 shows the splitting tensile strength results for 28 days moist cured cylinders. All the specimens containing alum sludge show a reduction in splitting tensile strength with compared to the control specimen. The reduction in strength is more significant when the replacement percentages are increased. This behavior is most probably due to the sensitivity of the splitting tensile strength to the amount of cement which by replacing it with alum sludge, the amount of binder material reduces.

The reduction in splitting tensile strength for the specimens is more significant than the reduction in compressive strength among the addition of alum sludge as a cement replacement. This phenomenon is due to the filler effect of alum sludge which attributes to the superior compressive strength.

Table 4. Splitting tensile test results

Mix	% of Alum sludge	Ft(MPa) at 28 days
P1	% 0	4.23
S1	% 5	3.79
S2	% 10	3.55
S3	% 15	3.375

4.4 Flexural Strength

The flexural strength results for control and alum sludge contained beams are show in Fig.3 From the figure, it can be seen that the addition of alum sludge to the mixes, as a cement replacement, results in flexural strength reduction. The reduction in the flexural strength increases by increasing the alum sludge percentages. The reduction in flexural strength is significant for 5% alum sludge when it is compared to control specimen, while the flexural strength reduction for 10 and 15% alum sludge specimens are less significant when they compared to 5% alum sludge specimens.

From the flexural strength results, it can be taken out that the alum sludge has low pozzolanic activity which is the reaction of the material with calcium hydroxide produced from the hydration process. This means that the alum sludge in this case acts as a filler in concrete.

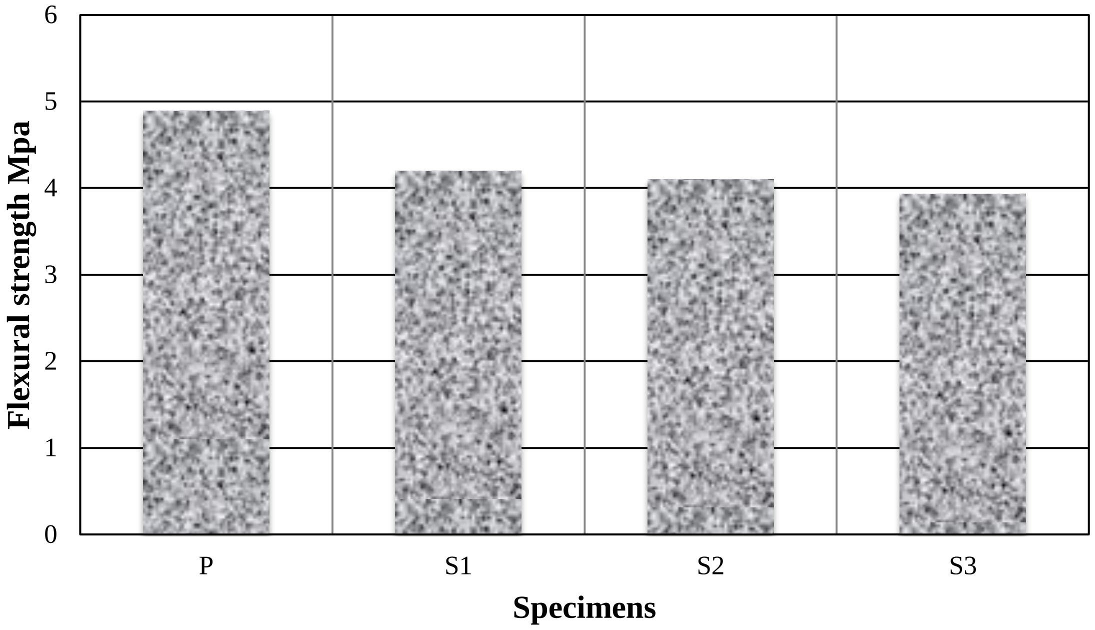

Figure (3). Flexural strength for control, S1 (5% alum sludge), S2 (10% alum sludge) and S3 (15% alum sludge contained) concrete beams

5. Conclusions

From this research study the following conclusions can be drawn:

- It is possible to produce high strength concrete having compressive strengths up to 66 Mpa by replacing 15% of cement by alum sludge; higher strengths can be obtained by replacing lower percentages.
- Although replacing cement by alum sludge causes a slight reduction in strength, the material remains in the category of high strength concrete.
- Using alum sludge causes a reduction of slump for concrete mixes, and a part of this reduction can be compensated by increasing the amount of super plasticizer.

Acknowledgement

We highly appreciate the research center of faculty of engineering at Koya University for their technical support in performing the test.

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Contemporary Approaches to Construction Management Education

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Received: February 15, 2016 Accepted: April 25, 2016 Online Published: June 1, 2016

Abstract: In today's conditions, managerial skills supply is an important competitive advantage for construction companies as well as technical skills. As a part of civil engineering department, construction management has an important role in construction technology. It covers new management techniques, economic, legal, political and environmental effects of the construction sector and similar administrative issues. As a result, it provides an important contribution on construction industry to get the right human resources profile. The purpose of this study is to determine the subjects which are supposed to be included in the construction management education by examining the training programs of international education organizations according to the needs of the construction sector. At the same time, managerial and personal skills for new graduates who will take part in the future management team are discussed.

In this context, construction management courses offered in the universities have been investigated. Some topics were identified as a result of a benchmarking studies and literature review. A questionnaire has been prepared according to these topics and some managerial and personal skills. The questionnaire has been revised by discussing it with the senior and experienced people in the construction industry. Later it was sent to people who work at the senior executive positions in the public and private companies. In the end, personal and managerial skills which are considered important for the construction management are determined and some recommendations for construction management education are made.

Keywords: Construction Management Education, Construction Industry, Managerial Skills

1. Introduction

The primary objective of construction management is supplying the analysis and design information needed for civil engineering. Beside this, providing information about the management of construction companies during construction process is one of the objectives. Social and environmental impacts can be summarized as the expansion of the engineering standpoint at construction management. In this context, the construction management education brings theory to graduates in the field of project management as much practical information and managerial skills. It aims to prepare them for a dynamic site conditions and multidisciplinary working environment as required by the construction industry. Construction management education forms its study fields according to the academic knowledge, also the applications in construction industry and the needs and expectations for the future projects. New graduates are adapted more easily to the sector by management knowledge. On the other hand, it contributes research, development and technological innovation to construction sector. In this context, it is a critical success factor for construction management field to prepare essential studying courses for students at the universities according to expectations of the construction sector.

Civil engineers use their management knowledge while they work at project management, business development, proposal development and planning and etc. The tasks of these positions are site organization, planning, labor and material supply, demand management, contract management and etc. Engineers who get a good education in these matters have competitive advantage in the sector. So it is important for civil engineering students to have managerial skills when they graduate. They may get those skills if they study right courses at the universities. As a result, quality of construction management education is increased by this way.

In this study, a survey intended to determine the information fields which increase the competitive advantage of graduates and management skills is designed. The survey was sent electronically to people who work in the construction industry in senior executive positions at the public and private companies. And 38 feedbacks were obtained. The most important information fields and skills were determined by the data collected which are not generally owned by graduates. It is argued for graduates to have right courses to improve construction management education and supply the sector's demand.

2. Prospects of Construction Management Education

As construction sector has gained an international qualification and global competition between companies has increased, they cause construction companies to expect their civil engineers to have some administrative and organizational skills. According to Russell's and Yao's observations, construction companies employ civil engineers due to their technical skills. If they are weak at public relations they are dismissed later but if they have leadership and managerial skills they may be promoted. Because of the construction industry is project based feature, it requires different skills towards to other sectors. Beside this, in the construction sector as well as other sectors, companies which combine creativity and innovation can create real value and they may have the advantage of superior competition (Harrington et al., 1998).

When literature is reviewed in this field, it can be seen that there are many studies that investigate the knowledge and skills needed by the construction sector (Egbu, 1999). As a result of these studies, as well as academic achievement, having compatible and leadership capacity for teamwork, information technology skills, foreign language skills, problem solving skills, and awareness of such features related to the business environment is understood to be significant by sector (Warszawski, 1984 ; Riggs, 1988). However, in general the graduates were reported as weak at written and oral communication, human relations, directing and managing other employees (Guthrie, 1994). Even researches were different to each other in content and purpose; all of them focused on the importance of personal and managerial skills. But these skills are being ignored in much construction management education. It is expected to provide scientific and technological knowledge as well as background based on problem solving, personal characteristics, ethics and social sciences by the civil engineering education (Liu and Fang, 2002).

According to Clough and Sears (1991), three essential features such as the use of technology for business planning, practical experience and teamwork should be given to civil engineering students. Miers (2001) states that educational institutions should have the following properties:

- Working in close cooperation with students, academics, administrators and construction sector to have common vision,
- Being communicative,
- Having feedback mechanisms for assessment and evaluation,
- Having an educational system which provides an environment to students to be good at problem solving and innovation.

In the scope of the survey which will be introduced in the following sections, current conditions and construction management knowledge areas reflect the requirements are described. Survey results are interpreted and discussed how they form the curriculum to meet the requirements of the construction management industry expectations.

3. Survey

Engineering, technology, construction techniques and operational / management issues are together in a certain balance in construction management education content. Construction management courses offered in universities located in UK and USA are shown in table 1. As observed from the table, the most abundant courses in the construction management program are project management, project planning, construction law, cost accounting, and construction management and engineering. Information fields that are included in the survey were determined along with the contents of these courses and by examining the work which Weber (2000), Miers (2001), Pietrofort and Stefani (2004), have already performed.

Table1. Construction management courses offered in universities located in UK and USA.

	Courses	**UK**	**USA**	**UK+USA**
1	Project management	9	10	19
2	Project planning	8	9	17
3	Construction Law	7	10	17
4	Cost accounting and financial	7	9	16
5	Constrution management and engineering	9	7	16
6	Engineering economy	9	3	12
7	Strategic management	7	3	10
8	Computer based planning	3	7	10
9	Structural analysis	2	7	9
10	Occupational safety and health	6	2	8
11	Risk management	5	3	8
12	Information systems in construction	4	4	8
13	Construction site and machinery	1	7	8
14	Business behaviors	3	4	7
15	Operations research	2	4	6
16	Asset management	4	I	5
17	Design management	3	I	4

18	Resource management	2	1	3
19	Facility management	2	î	3
20	Productivity	0	3	3
21	Quality management	1	1	2
22	International construction	1	0	1

Knowledge areas which are classified into 6 parts are depicted in Table 2. When Accreditation Board for Engineering and Technology (ABET)-2000 criteria are investigated, technical issues such as legislation, planning, cost control, economics, accounting, occupational safety and health, statistics and administrative issues such as ethical values, leadership, decision-making methods, engineering management are aimed to be taught in construction management programs (Abudayyeh et al., 2000). During the determining skills period, the work of Weber (2000), Miers (2001), and Love et al (2001) examined. As a result, managerial and personal skills that are presented in Table 3 were decided to be included in the survey.

4. Research Outcomes

Within this study, topics which are taught in construction management at universities and presented in table 2 have been questioned to the senior experienced administrators who worked in the public and private sectors in the construction industry. Needs of construction industry by taking into consideration the international aspects of the construction have been discussed and qualifications of graduates in these topics are examined. In addition, the evaluation of certain managerial and personal skills can be gained by construction management education shown in Table 3 has also been requested. And the importance of qualifications and skills for all the topics examined in the survey on the 1-5 scale (1: minimum, 5: being the highest) were evaluated.

4.1 Construction Management Knowledge Areas

40 knowledge areas which are formed by 6 main headings are shown in table 2. Headings are project management, financial management, company management, contract management and legal factors, technology and analytical methods. Considering the average values according to the survey results; planning, productivity and site organization and management are the most needed topics in project management. Although they are slightly lower in importance, risk management, management of international construction projects and occupational safety and health have some shortcomings. It is obtained that cost estimating and engineering economics are the most important issues in the field of financial management. On the other hand, it has been observed that graduates are insufficient in project financing which has the same importance. It is observed that strategic planning and management and bid preparation are the most needed issues in the scope of the company management and graduates are adequate in these fields. But it is noticed that there are some deficiencies in the field of enterprise resource planning-ERP issues such as innovation management and market or project selection. Settlement of disputes is discussed as the most important and striking topic with the most inadequate in contract management. Information technology and commercial software are used by a high rate of graduates needed by industry. Similarly, the much-needed analytical methods including critical path method (CPM) and PERT and other planning methods such as phase diagrams are used by graduates sufficiently. The average importance and proficiency of knowledge areas in 1-5 scale according to the survey

results are shown in Table 2.

Table 2. The average importance and proficiency of knowledge areas.

	Importance	Proficiency
Project Management		
Planning.	4.7838	2.8649
Quality management.	4.3243	2.6757
Risk management.	4.2162	2.3514
Construction site organization and management.	4.4324	2.8378
International construction projects management.	4.3056	2.4865
Supply chain management.	3.9444	2.7500
Asset management	3.7273	2.6061
Human Resources Management	3.9444	2.6000
Occupational safety and health.	4.2500	2.5714
Productivity	4.4865	3.0690
Financial Management		
Project financing	4.2162	2.3784
Cost estimating	4.3889	2.9730
Engineering economy	4.2432	2.9189
Accounting	3.8056	2.5714
Company Management		
Preparing bid	4.2162	2.8889
Strategic planning and management.	4.3784	2.6667
Enterprise resource planning (ERP).	3.9167	2.4412
Innovation management.	3.7778	2.5143
Market / project selection.	3 8889	2.5429
Marketing	3.5833	2.7143
Strategic partnership.	4.0541	2.6111
Knowledge Management, organizational learning.	4.0811	2.8056
Contract Management and Legal aspects		
Demand management.	4.1351	2.6757
Resolving disputes.	4.4324	2.4865
Standards, specifications and legislation.	4.3514	2.6486
Contract / Payment types.	4.1892	2.5676
Technology		
Construction technologies.	4.1351	3.2432
Construction machinery / equipment.	3.8378	2.9730
Construction materials	3.9444	3.1667
Automation	3.7778	2.9444
Information technologies	3.9444	3.4333
Commercial software (Primavera, etc.)	4.2500	3.3889
Analytical methods		
Critical path method.	4.3244	3.3335
Other planning methods.	3.8648	3.0336

Artificial Intelligence techniques.	3.5314	2.7502
Multi-criteria decision-making techniques.	3.5807	2.6253
Statistical analysis	3.6002	2.7408
Optimization	3.7220	2.9287
Simulation	3.3233	2.7308
Risk analysis methods (fuzzy logic, etc.)	3.6174	2.8460

4.2 Managerial and Personal Skills

Managerial and personal skills which are supposed to be had by graduates are grouped under 17 headings in table 3. According to the survey when managerial skills are examined; making decision / finding solution, leadership, taking or sharing responsibility and adopting to teamwork have been identified as the most needed skills in the sector. It is obtained that graduates are adequate in the skills mentioned above, although there are some shortcomings in negotiation technics and business development which have lower importance. Strategic thinking, written communication, professional ethics and analytical thinking are thought as the most important personal skills by the administrators. Besides this, it is observed that written communication and strategic thinking are areas where graduates are most inadequate. Most skills can be taught to many people, even if some of them are innate qualities. From this point of view, education is expected to develop both personal and managerial skills. It is come up in this study, some skills which are supposed to be important by company administrators can be taught to students by teamwork, oral and written presentations and giving various projects and assignments that require negotiations with construction companies during their education. The average importance and proficiency of managerial and personal skills in 1-5 scale are shown in table 3.

Table 3. The average importance and proficiency of managerial and personal skills.

	Importance	**Proficiency**
Managerial skills		
Time management	4.3243	2.8378
Decision making / producing solutions	4.6486	3.1351
Organization	4.4054	2.8919
Leadership	4.5135	2.8649
Taking / sharing responsibility	4.4865	2.9189
Adopting to teamwork	4.4324	3.0811
Adopting to interdisciplinary work	4.1892	2.9444
Negotiation methods	4.2432	2.6757
Business development	4.1622	2.7568
Personal Skills		
Communication - verbal	4.2703	3.2703
Communication - written	4.3784	3.0000
Professional ethics	4.3889	3.4444
Creativity	4.1081	3.2903
Analytical thinking	4.3243	3.3784
Strategic thinking	4.5676	3.1081

Ability to use information technology	4.2162	3.7027
Following innovations / improvements	4.2500	3.6757

5. Conclusion

When considering the high expectations of the sector from new graduates, it is detrimental for both graduates and industry when students finish the university without some essential skills to succeed. Regular monitoring by academics according to the expectations of the sector is required to supply high quality human resources needed for the industry to improve. It is understood that making necessary changes to make graduates better equipped is inevitable for both universities and construction industry to race in competitive environment in the world nowadays.

In this research results, it is obtained that graduates are sufficient in general but they have lack of knowledge and skills in certain subjects. Five knowledge areas and skills that are supposed as the most important by the sector and another five that the graduates are most inadequate are given in tables 4 and 5.

Table 4. The most important and the least sufficient knowledge areas.

The most important knowledge areas		**The least sufficient knowledge areas**	
Planning	(4.7838)	Risk Management	(2.3514)
Productivity	(4.4865)	Project financing (source creation)	(2.3784)
Construction organization and management	(4.4324)	Enterprise resource planning (ERP)	(2.4412)
Resolving disputes	(4.4324)	Resolving disputes	(2.4865)
Cost estimating	(4.3889)	International construction projects management	(2.4865)

Figure 1. The graphical representation of the most important knowledge areas.

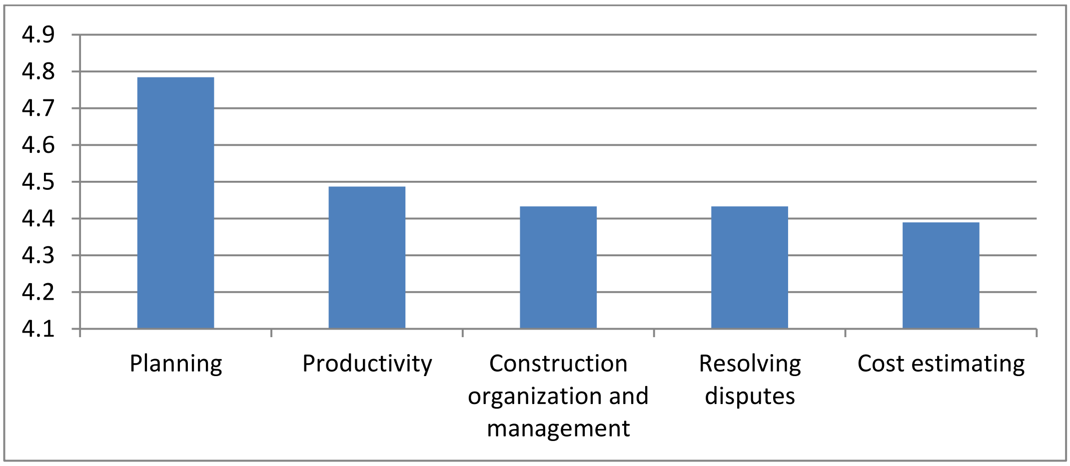


Figure 2. The graphical representation of the least sufficient knowledge areas.

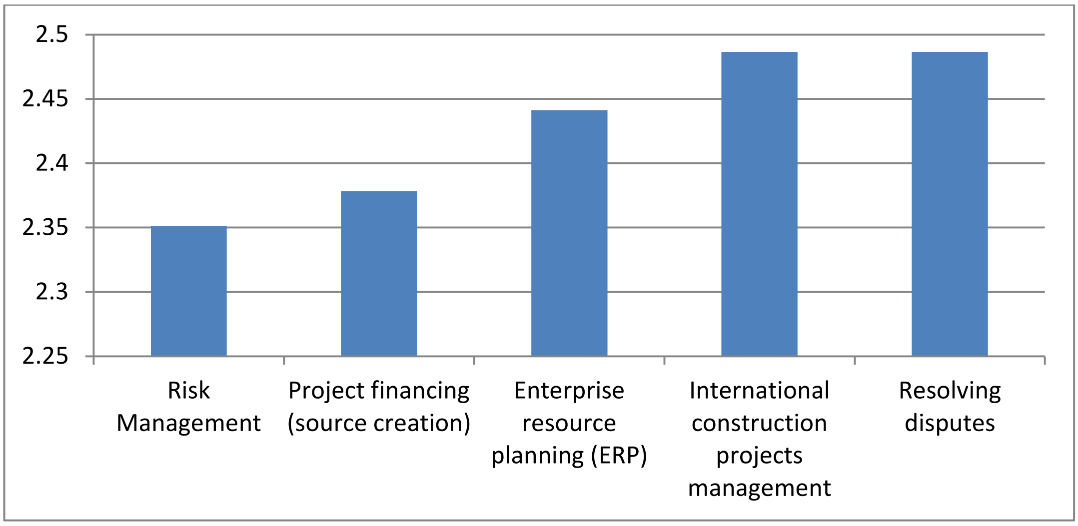


Table 5. The most important and the least sufficient managerial and personal skills

The most important managerial and personal skills		**The least sufficient managerial and personal skills**	
Decision making / producing solutions	(4.6486)	Negotiation methods	(2.6757)
Strategic thinking	(4.5676)	Business development	(2.7568)
Leadership	(4.5135)	Time management	(2.8378)
Taking / sharing responsibilities	(4.4865)	Leadership	(2.8649)
Adopting to teamwork	(4.4324)	Organization	(2.8919)

Figure 3. The graphical representation of the most important managerial and personal skills.

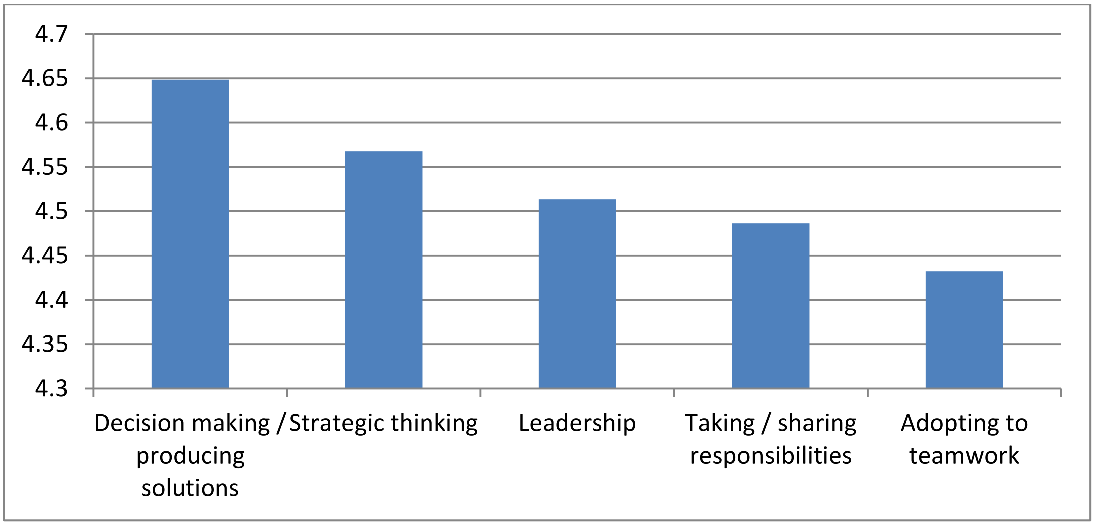
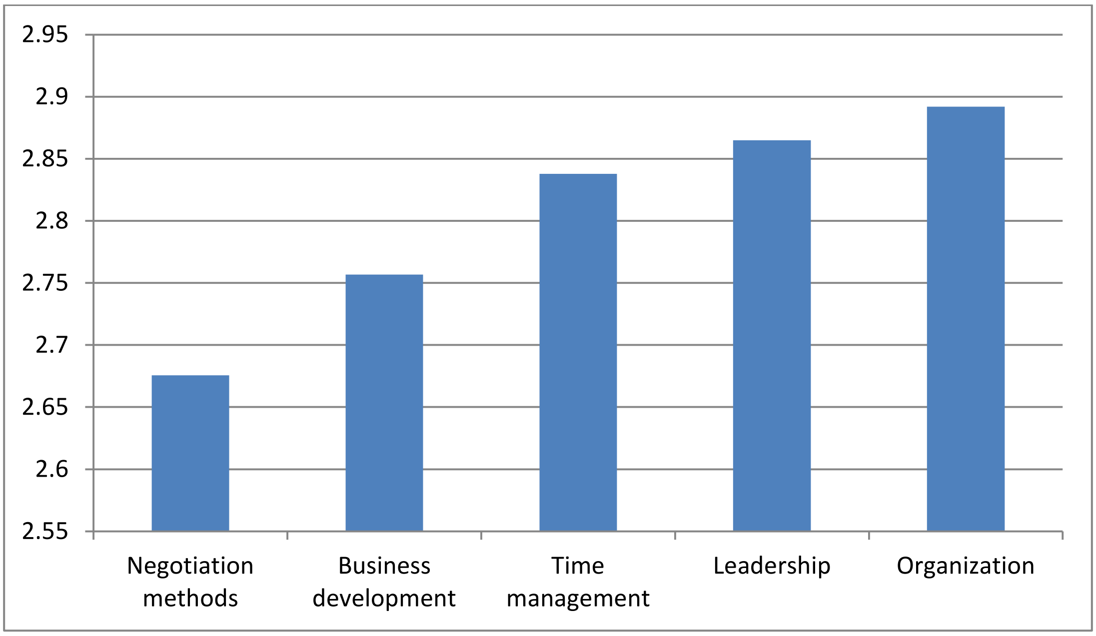


Figure 4. The graphical representation of the least sufficient managerial and personal skills.



As a result, construction management education should be redesigned to emphasize the knowledge areas and skills which are mentioned above to graduate more successful engineers. It is thought that the homework and workshops should be organized particularly the courses covering the teamwork and resolving disputes to develop their leadership skills. In order to increase the competitive advantage of graduates it is expected to correct the deficiencies in these areas. As construction management is applied field, its education should be practical. If senior and experienced engineers are integrated into the training programs of practical information, it is believed that it will be very beneficial for the prospective engineers in the construction industry.

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Load-Deflection Behavior and Failure Modes of CFRP Strengthened Reinforced Concrete Beams

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Received: April 25, 2016 Accepted: May 22, 2016 Online Published: December 1, 2016

Abstract: A total of fourteen beams, 100×150 mm in cross-section were tested in the laboratory over an effective span of 2000 mm. Two of them were used as reference beams. Twelve strengthened reinforced concrete beams were provided with externally bonded CFRP laminates at the soffit of the beam. The variables considered included number of CFRP layers, yield strength of steel reinforcement (f_y) and steel reinforcement ratio (ρ). All the beams were tested until failure. The test results showed that the load-deflection behavior, failure modes and ultimate load carrying capacity are influenced by the ratio of steel reinforcement, the steel yield strength and the number of CFRP layers.

Keywords: Load-Deflection, CFRP Laminate, Strengthening

1. Introduction

Recent studies have focused on the use of Fiber Reinforced Polymers (FRPs) as strengthening materials for the rehabilitation of existing reinforced concrete structures. FRP materials have high strength-to-weight ratio, high resistance to corrosion compared with steel plates, it is very easy and speedy to transport and installation.

The idea of strengthening concrete structures with externally bonded FRP systems were developed as alternatives to traditional external reinforcing techniques such as steel plate bonding and steel or concrete column jacketing (ACI Committee 440.2R, 2008). FRP composites, however, are of high cost, and high risk of fire and accidental damage, unless the strengthening is protected.

The addition of Carbon Fiber Reinforced Polymer (CFRP) composites affects the ductility of concrete beams strengthened with CFRP sheets. Therefore; there is a need to investigate the effect of the CFRP laminates on the overall ductility of strengthened beams (Aboutaha et al. 2003; Ali, 2015).

2. Research Significance

This paper presents results of an experimental research that focuses on the load-deflection behavior, failure modes and ultimate strength of CFRP strengthened concrete beams. The main variables are the amount of the existing reinforcing steel bars, yield strength of steel bars, and number of CFRP layers.

3. Experimental Program

The experimental program consists of casting and testing of fourteen reinforced concrete beams. Twelve beams were strengthened with CFRP laminates, the two beams were considered as control beams, and all beams have been tested under four-point loading to failure (Ali, 2015). In all beams, the cross section was 100mm wide and 150mm in depth, the overall length was 2200mm with clear span 2000mm. The beams were designed to have extra strength in shear to ensure flexural failure even after strengthening (ACI Committee 318, 2014); therefore, the shear span was reinforced with ϕ6mm @ 50mm as shear reinforcement in all beams as shown in Fig. (1) and Table (1).

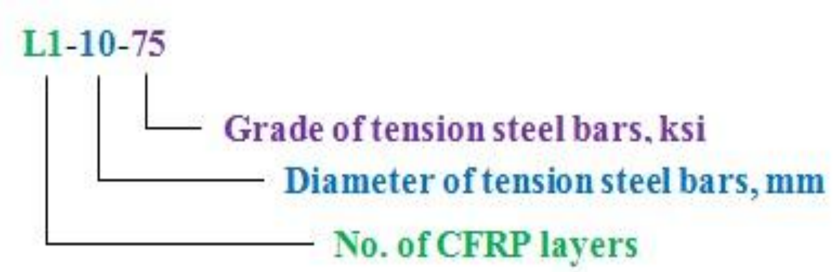
The dimensions of the CFRP laminates were constant (100mm width and 2000mm length) and applied to the bottom of the strengthened beams only by one and two layers. The concrete beam and section details are as shown in Fig. (1) and Fig. (2).

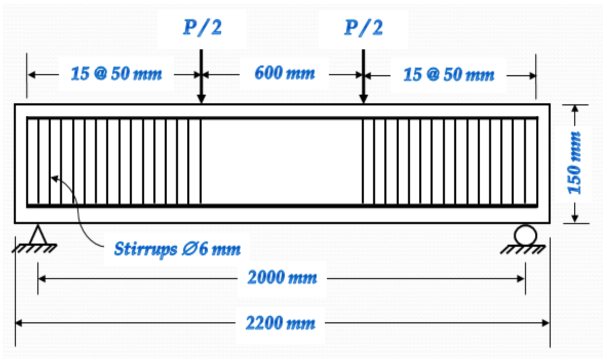
The concrete compressive strength was 45 MPa with a 100mm slump. Carbon Fiber Reinforced Polymer (CFRP) used in this study is known as SikaWrap-230 C/45 sheets. These sheets are unidirectional woven carbon fiber fabrics for the dry application process.

Table (1). Details of the reinforcement and CFRP strips of beams

Beam symbol	*Tension reinforcement*	*Yield Strength of tension bars(MPa)*	*No. of CFRP layers*
L0-10-75	**2 ϕ10 mm**	**595**	**None**
L1-10-75	**2 ϕ10 mm**	**595**	**1**
L2-10-75	**2 ϕ10 mm**	**595**	**2**
L1-12-75	**2 ϕ12 mm**	**593**	**1**
L2-12-75	**2 ϕ12 mm**	**593**	**2**
L1-16-75	**2 ϕ16 mm**	**535**	**1**
L2-16-75	**2 ϕ16 mm**	**535**	**2**
L0-10-60	**2 ϕ10 mm**	**460**	**None**
L1-10-60	**2 ϕ10 mm**	**460**	**1**
L2-10-60	**2 ϕ10 mm**	**460**	**2**
L1-12-60	**2 ϕ12 mm**	**418**	**1**
L2-12-60	**2 ϕ12 mm**	**418**	**2**
L1-16-60	**2 ϕ16 mm**	**418**	**1**
L2-16-60	**2 ϕ16 mm**	**418**	**2**

Where





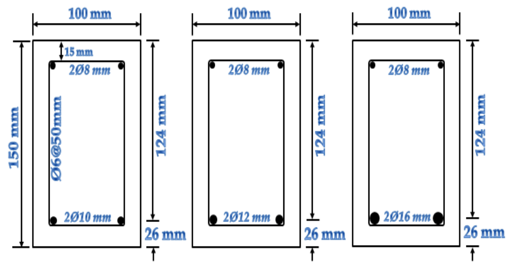
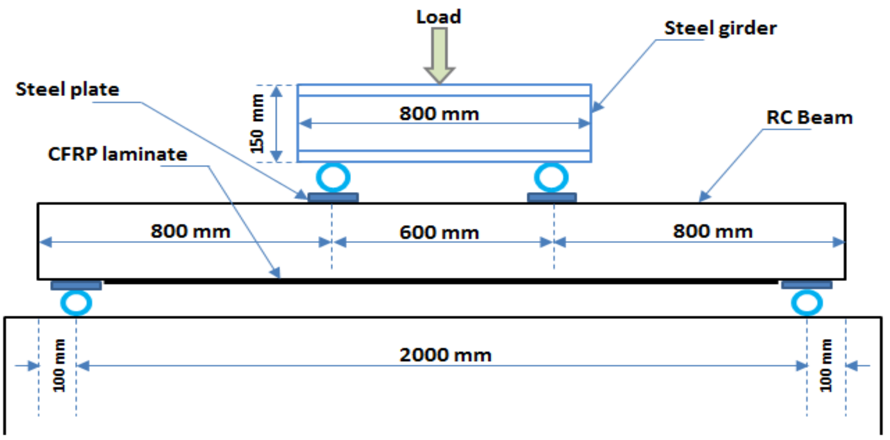


Fig. (2). Reinforcement details of the beam sections

Fig. (1). Details of the reinforcement in the beam specimen

Dry fiber properties provided by the manufacturer's guaranteed tensile strength of 4.3 GPa, modulus of elasticity of 234 GPa and elongation at break of 1.8%. The universal testing machine, which was used in this study, consists of a self-supporting steel frame with a hydraulic jack of 600 kN capacity and a computerized measuring unit connected to data logger to read load, deflection and strain as shown in Fig. (3). To generate four-point loading system, the load from the center of the universal testing machine must be transmitted to the beam deck in two point load. For this purpose, a steel girder of (150) mm depth and (0.8) m length was used.

To avoid local bearing failure during testing, steel plates (120 x 50 x 6 mm) at the point of load application and the reactions were used**.** After checking all the instruments, the vertical load was applied to the beam at two points using a steel girder which transmits the center load of the hydraulic jack to the beam deck at two points as shown in the Fig. (3). For each load stage, the deflection and strains were recorded and the cracks were noticed. The total load on the test beam specimen was taken to be equal to the applied load from the universal test machine. The self-weight of the steel girder and the beam specimen itself were ignored. As the failure was reached, the failure load was recorded and the load was removed to allow taking photographs of the final cracked beam specimens.



(a)

(b) Test arrangement

(b) Beam specimen before load application

Fig. (3). Loading system shows specimen under flexure

4. Analysis of Results

4.1 Load-Deflection Curves

The load-deflection curves showed different deformations and behaviors under load for all beams. Table (2) provides a summary of the measured loads at first cracking and measured loads and deflections at first yielding of steel reinforcement and at ultimate level for all beam specimens.

In the elastic (pre-cracking) stage, the deflection increased linearly with applied load since the strains in the steel and concrete are relatively small and both materials steel and concrete are in the elastic portion of their respective responses. Initial cracking was observed at loads ranging from 13 % for (L1-16-75) to 22.8 % for (L0-10-60) of the beam ultimate load.

It can be shown that the behavior of the control beams (L0-10-60) and (L0-10-75) is typical of an under-reinforced concrete beam specimen, showing linear behavior up to yielding of reinforcement (where deflections measured 13 mm and 11 mm, respectively), followed by a change in stiffness and increased deformation until failure (at 27.55 and 30.5 kN, with corresponding deflections of 62 mm and 50.5 mm, respectively), see Fig. (6).

The cracking load was generally not apparent from the curves for strengthened specimens, although there is a slight increase of the cracking load than that observed in the control specimen. In the post-cracking (pre-yielding) stage, there is a change of slope in the load-deflection curve due to the cracking of concrete, which in turn results in reduction of the effective moment of inertia of the beam cross section. After cracking, deflection gain increase almost linearly with load up to the point at which the tensile steel yields. Tensile steel yielding load varied between 72.2 % for (L1-10-60) to 90 % for (L1-12-75) of the ultimate load.

Yielding of steel, which is characterized by the change of the post-crack slope, is clearly apparent in the strengthened specimens, though it is more evident in the control specimen. In the one-layer and two-layer strengthened specimens, yielding occurred at a higher applied load (for instance, beams L1-10-60 and L2-10-60: 27 kN and 32 kN) and with a higher midspan deflection (16mm and 17mm), respectively than in the control specimen (23 kN, 13mm), see Fig. (5). This is attributed to the retention of the composite action at the tension face, which lowered the neutral axis, giving a greater displacement at the yielding of steel. In the post-yielding stage, the contribution of CFRP becomes very significant, since additional contribution of steel is zero in the yield plateau; post-yield part of the curve is flat for a reinforced concrete beam. The CFRP strengthened beams continue to provide strength increase because the CFRP force contribution continues at the same level.

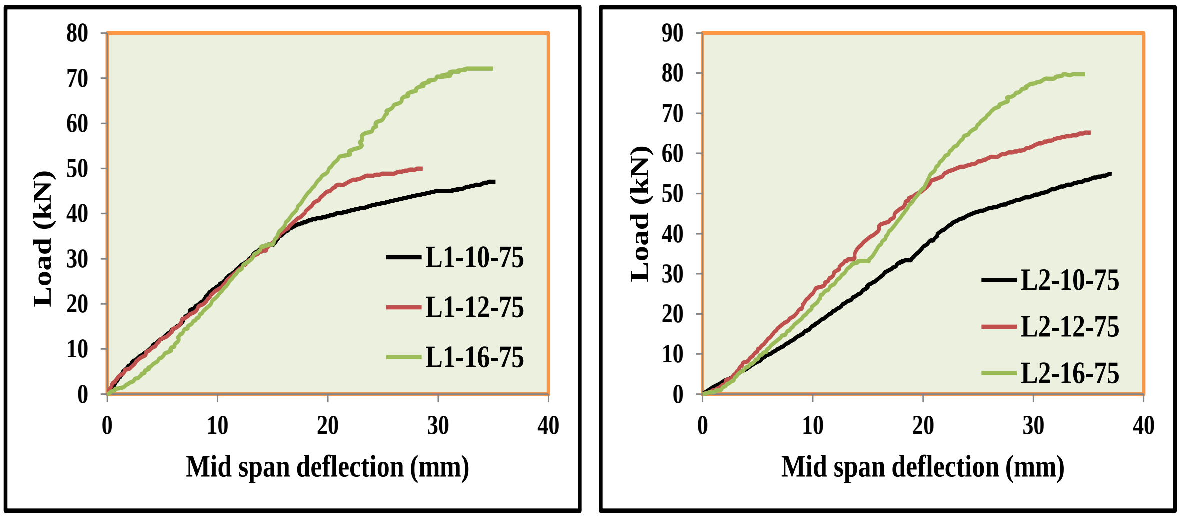
At ultimate, the unstrengthened specimens exhibited a higher midspan deflection compared with the strengthened specimens; however, the strengthened specimens achieved higher load capacities than the unstrengthened specimen, the two-layer strengthened beams being the highest. The strengthened specimens also exhibited substantially large deflections beyond the yielding of steel. Given the applied load, they showed reduced deflections and thus, increased serviceability. All the strengthened specimens exhibited an approximately bilinear load deformation response characteristic with the change in the slope of each plot occurring at a point corresponding to the yield strain of the steel. From the figures below, we observe that by increasing of CFRP layers, resulted in substantial reduction in mid span deflection and this reduction may be attributed to the increase of beam stiffness, rigidity and moment of inertia of the beam section by increasing of CFRP layers, steel yield strength and ratio of tensile steel reinforcement. These had resulted in substantial reduction in mid span deflection and this reduction which is attributed to the increase of beam stiffness and rigidity when increasing of (f_y) and tensile steel reinforcement ratio.

4.2 Failure Modes

As a result of experimental testing program, all the tested beams are designed to fail with flexural by increasing the shear strength of the beams. In all tested (reference and strengthened) beams, when the load is applied to the beam the first crack appears in the bottom of the beam face at the center of beam between the two point load and after the gradual increment of the applied load the cracks propagated to the top of beam. At higher loads, the already formed cracks are widened while new cracks started to form.

Table (2). Summary of test results for all beam specimens

Beam symbol	*First cracking load (kN)*	*Yield load (kN)*	*Yield deflection (mm)*	*Ultimate load (kN)*	*Ultimate deflection (mm)*
L0-10-75	**6.72**	**25.66**	**11.184**	**30.5**	**50.5**
L1-10-75	**8.288**	**36.73**	**16.320**	**47.04**	**35.2**
L2-10-75	**10.976**	**43.68**	**22.500**	**54.88**	**37.1**
L1-12-75	**8.736**	**45.92**	**20.462**	**49.952**	**28.6**
L2-12-75	**11.424**	**54.88**	**21.050**	**65.184**	**35.2**
L1-16-75	**9.408**	**57.34**	**21.100**	**72.128**	**35.0**
L2-16-75	**11.872**	**63.62**	**23.088**	**79.744**	**34.7**
L0-10-60	**6.272**	**22.86**	**13.170**	**27.552**	**62.1**
L1-10-60	**7.842**	**26.96**	**15.962**	**37.408**	**33.2**
L2-10-60	**8.736**	**32.15**	**15.615**	**43.456**	**33.0**
L1-12-60	**7.392**	**32.79**	**14.615**	**45.248**	**29.0**
L2-12-60	**9.184**	**40.85**	**20.965**	**52.64**	**37.1**
L1-16-60	**8.512**	**50.11**	**16.970**	**59.584**	**25.8**
L2-16-60	**11.2**	**62.94**	**20.200**	**74.816**	**36.4**



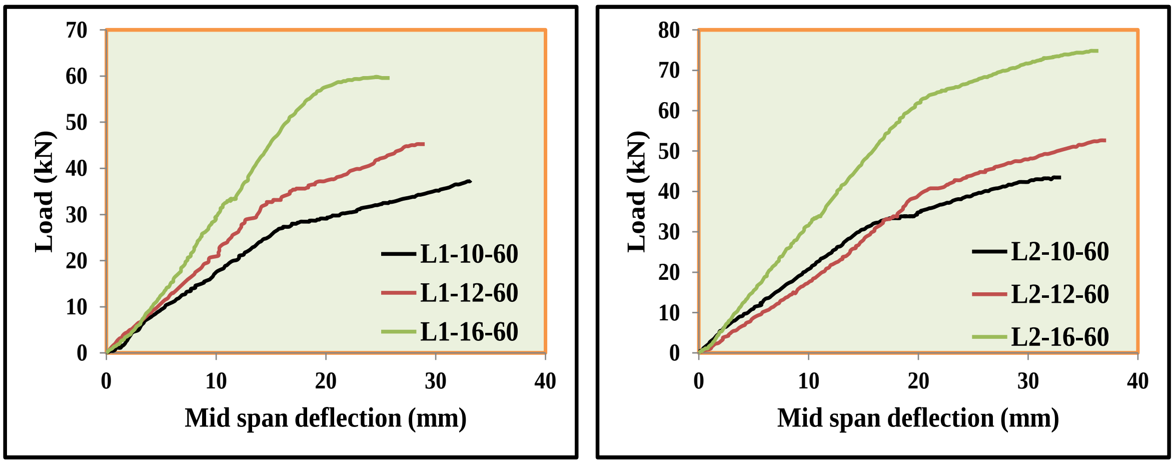
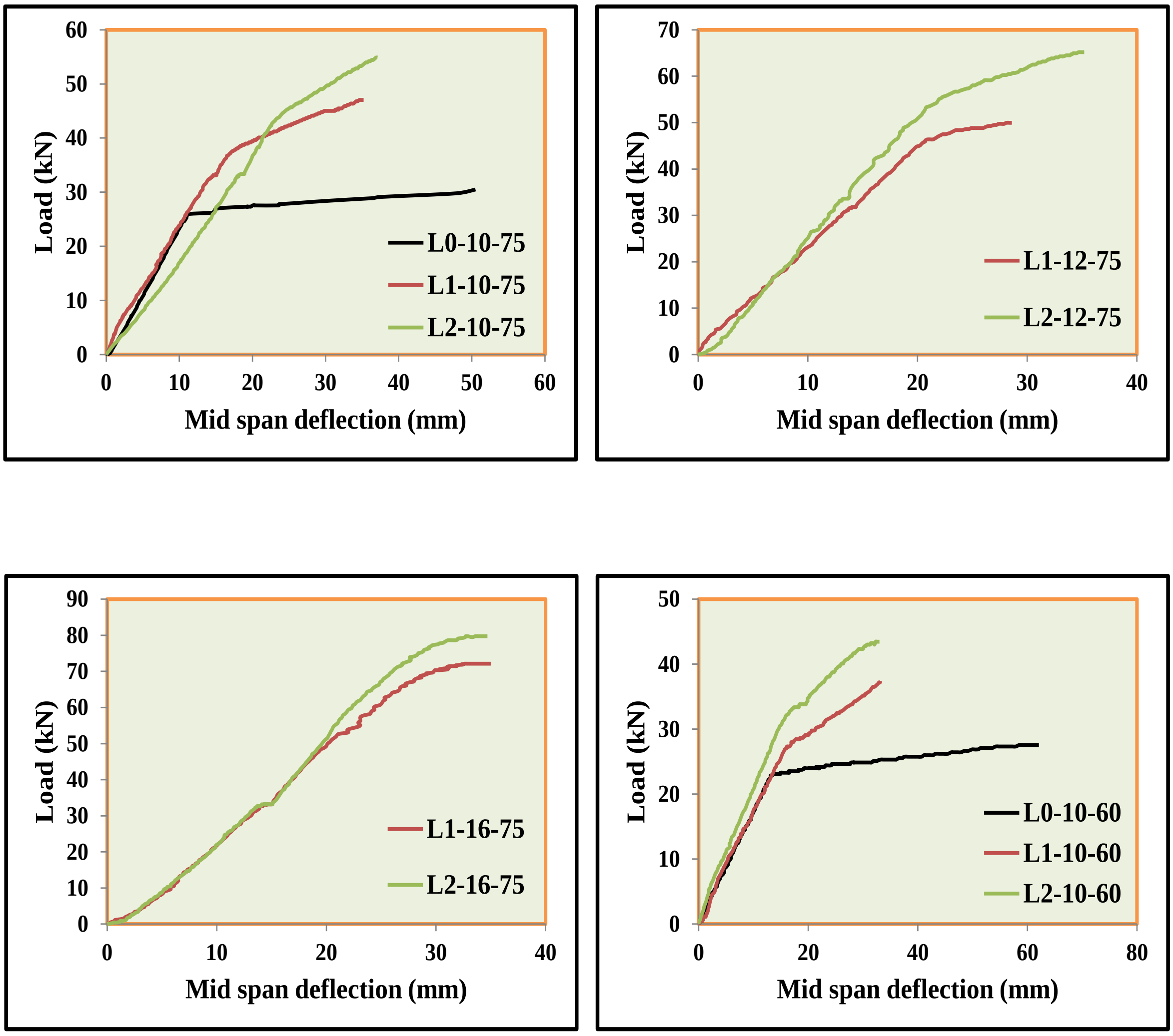


Fig. (4). Effect of tensile steel reinforcement ratio on the load – deflection curves



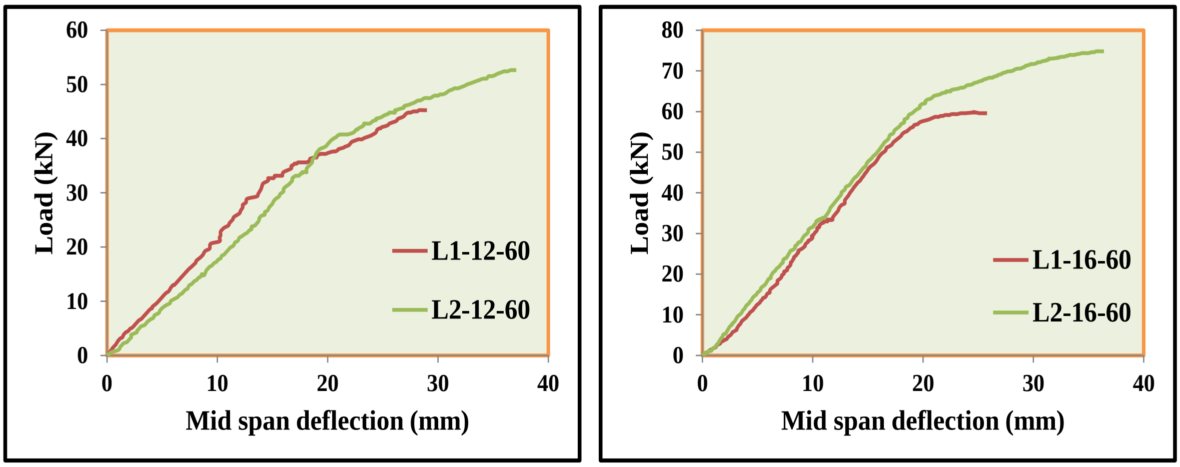
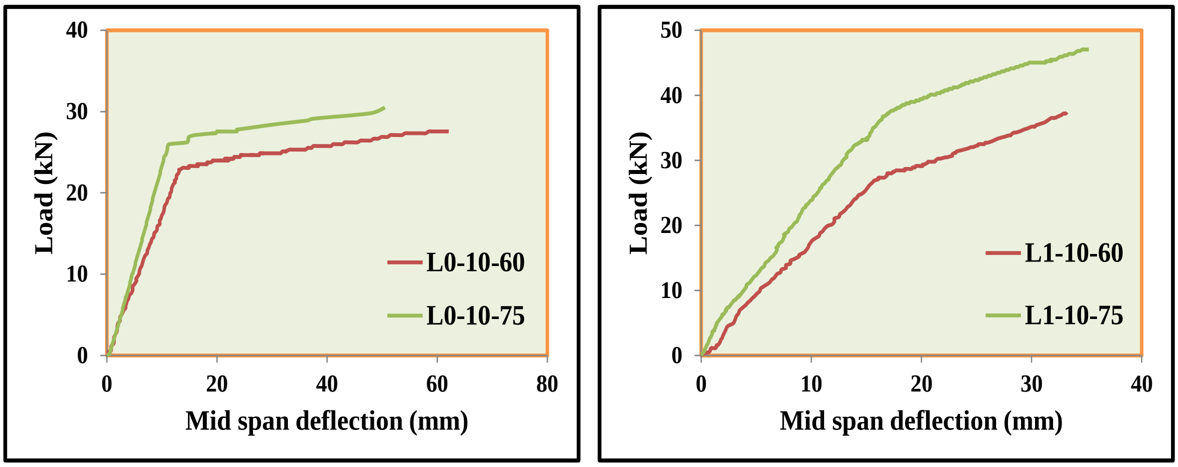
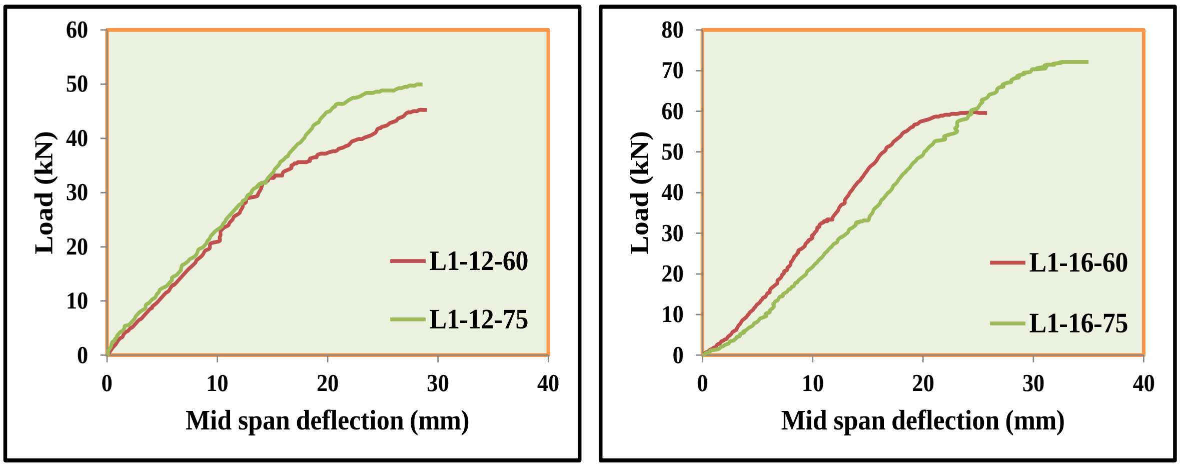


Fig. (5). Effect of CFRP layers on the load – deflection curves





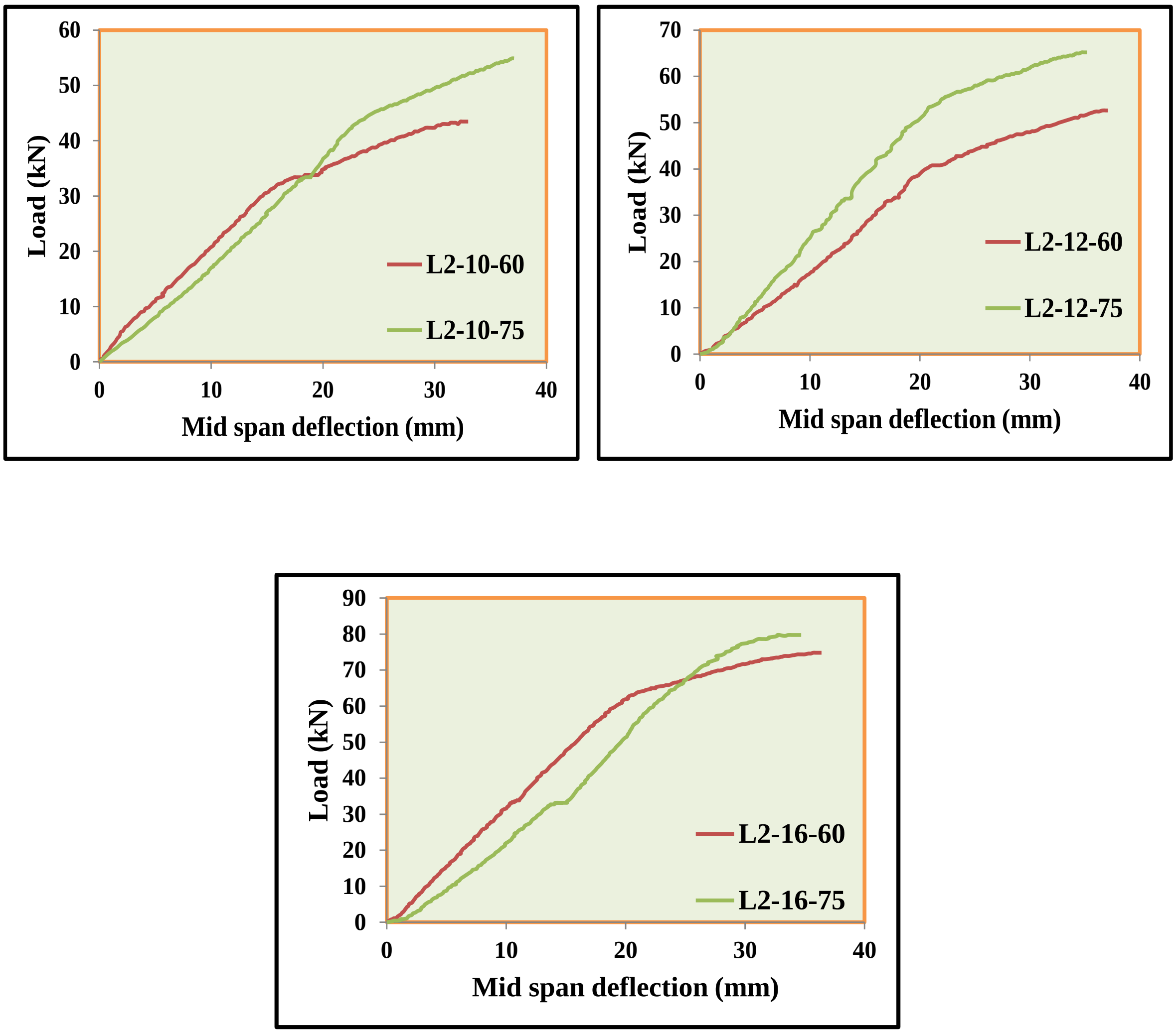


Fig. (6). Effect of tension steel bars yield strength on the load – deflection curves

Examination of failure modes suggests that all beams experienced flexural failure, and all beams except Beams L2-16-60 and L2-16-75 failed in tension. The tension failures were either by steel yielding or by debonding of the CFRP laminate from the concrete substrate. Following steel yielding, the CFRP sheets were either deboned or ruptured. No rupture of CFRP sheets occurred for the two-layer strengthened beams. See Fig. (7).

For the beams failing in compression, evidently, the fact that the beam was reinforced with highest reinforcement ratio and strengthened by the two CFRP layers may have affected the mode of failure. Some peelings in the concrete substrates adjacent to the CFRP sheets were observed. The peelings indicate that the epoxy type, coupled with the CFRP plates, was stronger than the concrete.


L0-10-75
L1-10-75
L2-10-75
L1-12-75

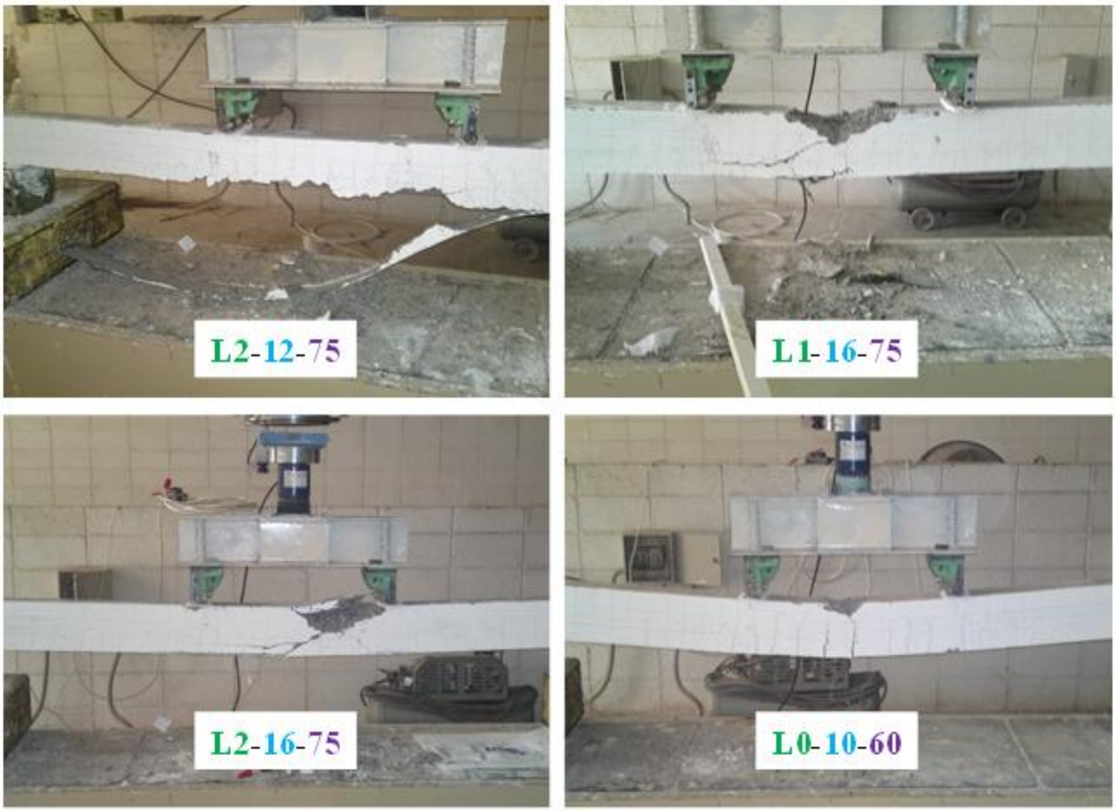

L2-12-75
L1-16-75
L2-16-75
L0-10-60

Fig. (7). Mode of failure for all beams

Failures in all strengthened beams were accompanied by the release of large amounts of energy (known as elastic energy) relative to inelastic energy. Therefore, a reasonable factor of safety should be used in the design of FRP strengthened reinforced concrete members (Oudah & El-Hacha, 2012; Ali, 2015).

5. Conclusions

Based on the experimental investigation described in this study, the following conclusions are drawn:

1. In the post-yielding stage, the contribution of CFRP becomes very significant. For instance, the ultimate load of beams reinforced with 2 No. 10 steel bars of Grade 520 MPa had increased from 30.5 kN to 47 kN (54%) when strengthened with one layer of CFRP, and to 54.9 kN (80%) when strengthened with two layers.
2. At ultimate, the unstrengthened specimens exhibited a higher midspan deflection compared with the strengthened specimens; however, the strengthened specimens achieved higher load capacities than the unstrengthened specimens, the two-layer strengthened beams being the highest.
3. For both the one-layer CFRP beams and the two-layer ones, the load-deflection curves were almost identical from beginning until the load reaches ultimate stages, where the two-layer CFRP beams showed higher strengths.
4. Examination of failure modes suggests that all beams experienced flexural failure, mostly failed in tension. The tension failures were either by steel yielding or by debonding of the CFRP laminate from the concrete substrate. No rupture of CFRP sheets was occurred for the two-layer strengthened beams.
5. Some peelings in the concrete substrates adjacent to the CFRP sheets were observed. It indicates that the epoxy type, coupled with the CFRP plates, was stronger than the concrete.
6. Failures in all strengthened beams were accompanied by the release of large amounts of energy. Therefore, a reasonable factor of safety should be used in the design of FRP strengthened reinforced concrete members.

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Experimental Study of Pressure Coefficient along Inclined Bottom Surface of Dam Tunnel Gate

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Received: April 19, 2016 Accepted: May 25, 2016 Online Published: June 1, 2016

Abstract: The major reason behind lift vertical gate installation within the dam tunnel is to provide the specific amounts of water released from the reservoir under a wide range of head variations. For such hydraulic structures, the water is flowing in two directions, below the gate which produce uplift force and above the gate through gate shaft which caused vertical downward force. The difference between these two hydrodynamic forces produces downpull force which affects the operation and stability of the gate. Moreover, the gate is subjected to different patterns of flow, especially along the bottom surface, and consequently, the attachment and reattachment of flow lead to vibrate the gate and create a significant challenge to its stability.

The analysis of the downpull forces and flow conditions effects is based upon the prediction of bottom pressure coefficients which are influenced mainly by many parameters such as pressure head distribution, jet velocity head and geometry of lip gate. In present research the results of all related measurements of physical hydraulic model were analyzed and presented by using three-dimension presentation to study the behavior of water flow pattern below the gate and hence the generation of pressure for inclined lip gate geometries (θ=55°). The measurements of upstream head, downstream head, jet velocity and peizometric heads distribution along and across the gate bottom surface are involved. The main conclusion states that the bottom pressure coefficient is significantly influenced by the lip gate shapes and gate openings.

Keywords: Coefficients of Pressure, Flow Forces, Tunnel Gate

1. Introduction

The hydrodynamic forces on vertical gates are commonly exerted by high heads of water of dam reservoir and impacted the upstream face, top and bottom surfaces of the gate. The operation and safety of the gates are directly affected by these forces and hence the prediction of such forces have been received much more attention from researchers and designers.

The parameters affecting the bottom pressure coefficient on high head gate had been studied by the Reclamation Hydraulic Laboratory Colgate Donald (1959). The pressure area computation method was used to estimate the pressure head distributions and it was found that not all parameters can be presented through a mathematical analysis with a certain extent.

2. Literature Review

The main parameters affecting the bottom pressure coefficient (Kb) were summarized by Sagar (1977) which can be expressed with same approach by the following form:

$$K_b = f\left(\frac{X}{X_o}, H_i, H_d, \frac{V_j^2}{2g}, \frac{Y}{Y_o}\right) \quad (1)$$

Where

K_b: Bottom pressure coefficient,
$\frac{X}{X_o}$: Distance ratio along bottom gate Surface,
$\frac{Y}{Y_o}$: Opening ratio,
H_i: Peizometric head on bottom gate surface,
$\frac{V_j^2}{2g}$: Jet velocity head, and
H_d: Downstream peizometric head.

The flow conditions and gate geometry were studied by Naudascher et al., (1964) to evaluate the bottom pressure coefficient. It is found that the bottom pressure coefficient can determined as follows:

$$K_b = \left(\frac{1}{\mathrm{B.d}}\right)\iint[(Hi - Ys)/(\frac{Vj^2}{2g})]dB \,.\, dx \quad (2)$$

Where:

Ys: Pressure head in the section of contracted jet,
B: Gate width, and
d: Gate thickness.

The design of the 3-leaf intake gate as a result of experimental study simulating the TVAS Melton Hill Dam was presented by Elder, R. A. and Garrison (1964). The experiments involved the examination of five proposed gate lip shapes and nine other basic shapes. Some difficulties were encountered due to the large hydraulic forces induced by lip forms, oscillation and prevention of the leaves to close.

The effects of different flow conditions on the bottom surface local pressure of intake gate were studied by Thang (1983). The study indicates that the values of downpull and discharge coefficients are sensitive due to the flow separation from the gate bottom surface.

The behavior of flow pattern and its effects on pressure fluctuations along the gate bottom surface was investigated by Bhargava and Narasimhan (1989). The study includes the measurements of many hydraulic parameters with respect to different gate openings. The study reveals that the values of hydraulic force are influenced by the pressure fluctuation and consequent vibration.

Thang (1990) studied the effects of different geometrics of bottom gates and discharge conditions on dynamic loads applied by water flow on vertical-lift gates within an open channel and at a conduit inlet. The study included the observation of vibrations which it may happened according to flow fluctuated between complete attachment and re-attachment at the gate bottom. It was concluded that the slope of the average lift curve acting on the gate bottom will contribute to the determination of critical range of gate openings with respect to potential gate vibration.

A one dimensional analysis was made by Al-Kadi (1997) to predict the effects of many hydraulic parameters on downpull forces acting on the bottom of vertical lift gate. The analysis were based upon the methodology of finite element program by considering two models, one with constant eddy viscosity, and the other of variable eddy viscosity which is important for estimating the upward downpull for large gate openings. The model was verified with actual records and gave good results.

The effects of many gate geometries with different gap width ratios on downpull force were examined by Ahmed (1999) using a hydraulic model to measure all required parameters to estimate the downpull. It is found that the downpull force is influenced mainly by gate geometries, gate openings and gap ratios of gate shaft.

3. Experimental Work

In present study, the (0.2 *0.3 * 4) m hydraulic model of gate tunnel was used to carry out all the required measurements relevant to bottom pressure coefficients. The gate was made by a thick plate with a thickness of 5 mm and provided by inclined lip shape with (θ=55°). The movement and openings of the gate within its shaft was controlled by adjustable screwed steel rode connected to the top shaft surface. Ten peizometric taps located uniformly in two lines across the bottom lip surface and connected to the manometer board were installed to measure the peizometric heads along and across the gate bottom surface (Hi). Figures (1-a) and (1-b) shows the general view of the hydraulic model, and the section of inclined gate considered which is developed for present study.

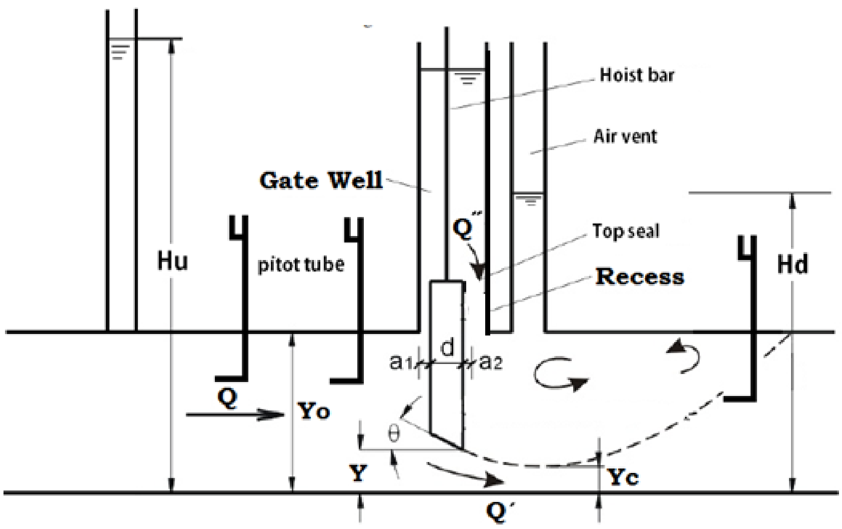


Figure (1-a). General Scheme of Hydraulic Model

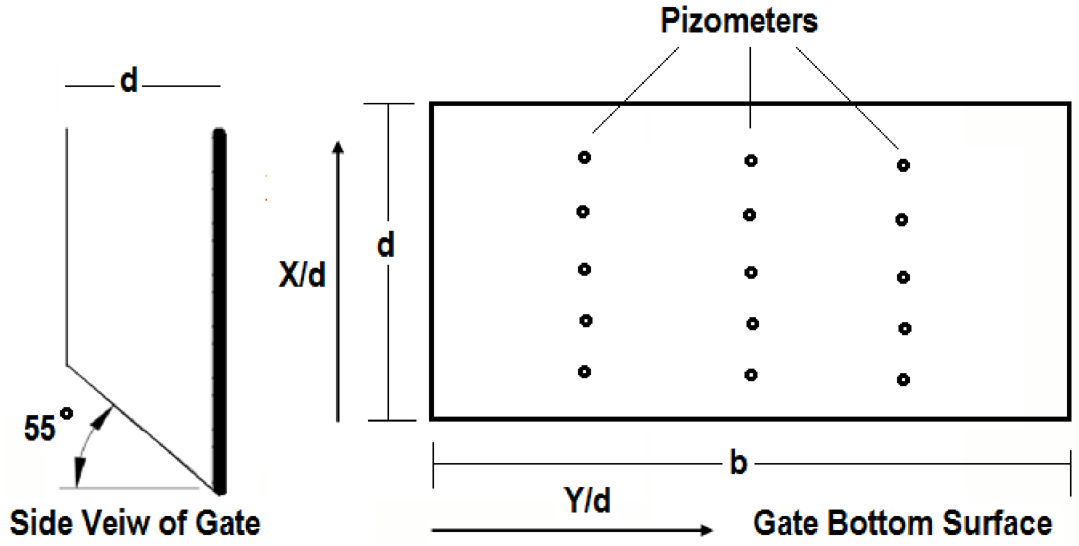


Figure (1-b). The side view of inclined gate and its bottom surface.

4. Results and Discussions

In present research, the results of the bottom pressure coefficient (Kb) values are based upon the experimental measurements and represented with aid of three dimensional model software. The inclined lip gate shape with (θ=55°) was examined. The principles of pressure fluctuations according to the values of (kb) along the gate bottom surface were used to indicate the effects of flow pattern on hydraulic forces and vibration impact on lift gate surface. However, the bottom pressure coefficient (Kb) is calculated using the following expression:

$$K_b = \frac{(H_i - H_d)}{\frac{V_j^2}{2g}} \tag{3}$$

Figure (2) indicates that for gate opening (Y/Yo=10%), the (Kb) values are generally high within the leading edge zone and decreased toward the trailing edge. It is also noticed that the pattern of variation along the middle third of gate bottom surface differs from those located on both sides of gate surface. However, such pattern of distribution revealed that the attachment and separation of flow stream lines are non-uniform across the gate bottom surface and consequently the bottom pressure coefficients are fluctuated. The peak values of (Kb) are located uniformly for X/d up to 30% and then become a little bit less for (X/d=50%). Accordingly, the attachment is occurred strongly with the first third of gate bottom surface.

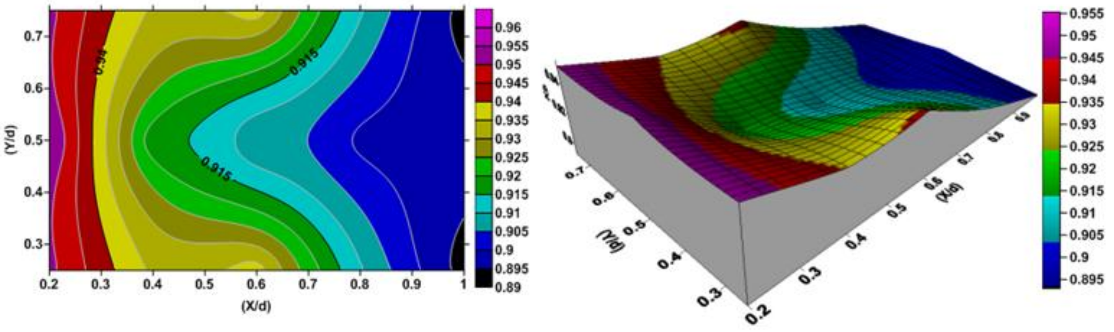


Fig. (2). Variation of (Kb) for (Y/Yo=10%)

Figure (3) indicates that for (Y/Yo=20%), the maximum values of (Kb) are concentrated in the middle third of leading gate bottom surface up to (X/d=30%), thereby making high intensity of attachment which followed by separation for remaining distance. Also, it can be seen from the figure that for (X/d=0.6), the attachment is appeared locally on both sides of gate surface due to the effects of increasing in values of (Kb).

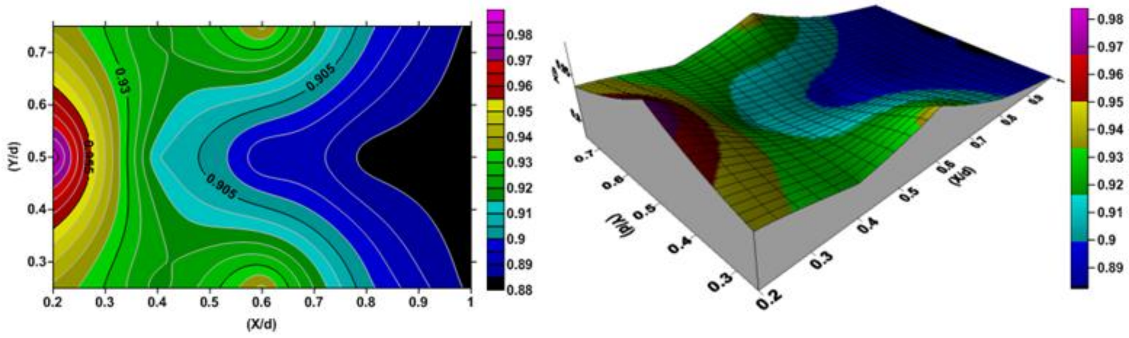


Fig. (3). Variation of (Kb) for (Y/Yo=20%)

In Figure (4), the values of (Kb) for (Y/Yo=30%) are observed high around the boundary of gate surface up to (X/d=60%). The values are then decreased throughout the major mid part from (X/d=30%) toward the trailing edge. It can be seen from this figure, that the high intensity of pressure is concentrated across the leading edge and whence, the strong attachment is established and the variation of pressure is moved gradually toward the middle part of gate surface.

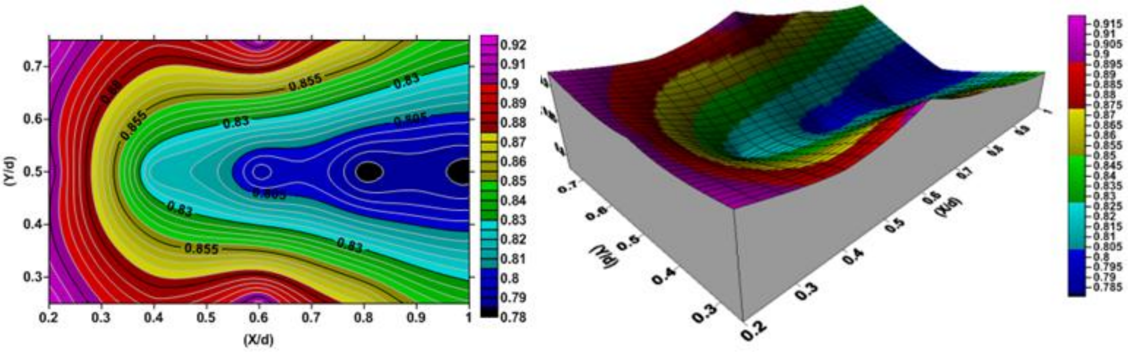


Fig.(4). Variation of (Kb) for (Y/Yo=30%)

Figure (5) represents the variation of (Kb) values for (Y/Yo=40%). It can be noticed, that the peak values of (Kb) are located on two positions, across the leading edge up to (X/d=30%) and just on both side edges of gate surface for (X/d=70%). It is appeared, that beyond these two places of peak values, the values of (Kb) start to decrease uniformly toward the trailing edge and indicate that the stream lines are being within the mode of separation.

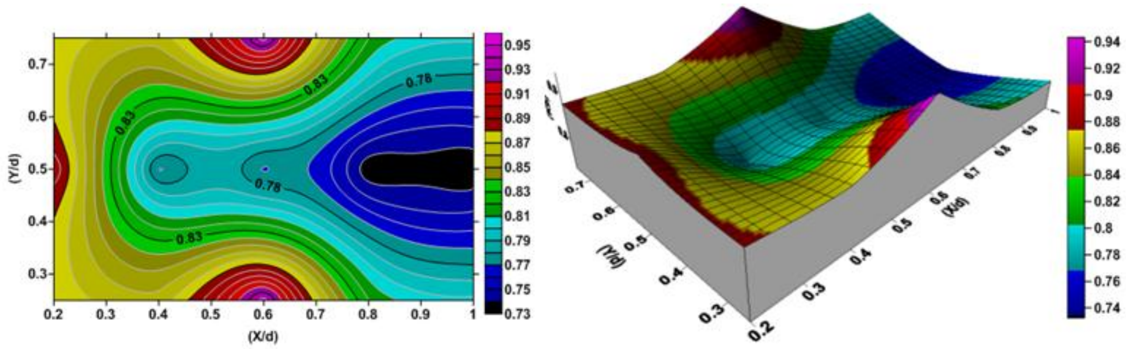


Fig.(5). Variation of (Kb) for (Y/Yo=40%)

Figure (6) indicates that for (Y/Yo=50%), the peak values of (Kb) are located on both upstream corners of gate bottom surface up to (X/d=30%) and on side edges just for (X/d=60%). Then after, the (Kb) values start to drop smoothly toward the middle part. Hereby, the strong attachment of stream lines are seem to be occurred around leading and side edges up to (X/d=60%) as a result of high values of (Kb).

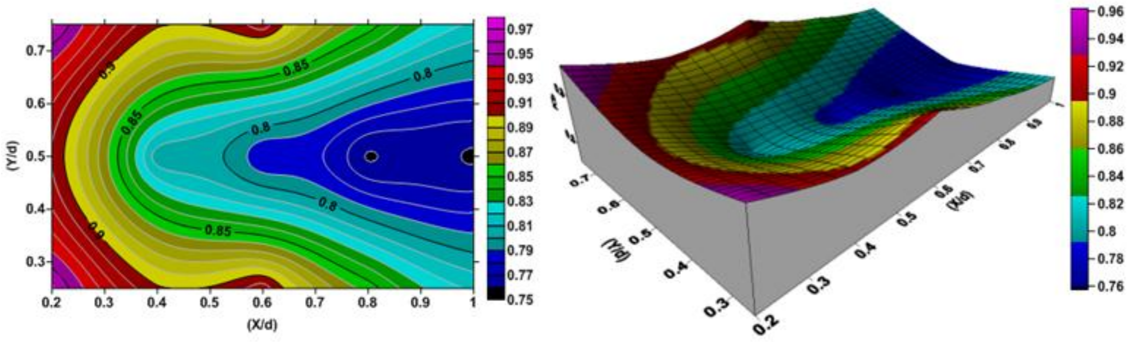
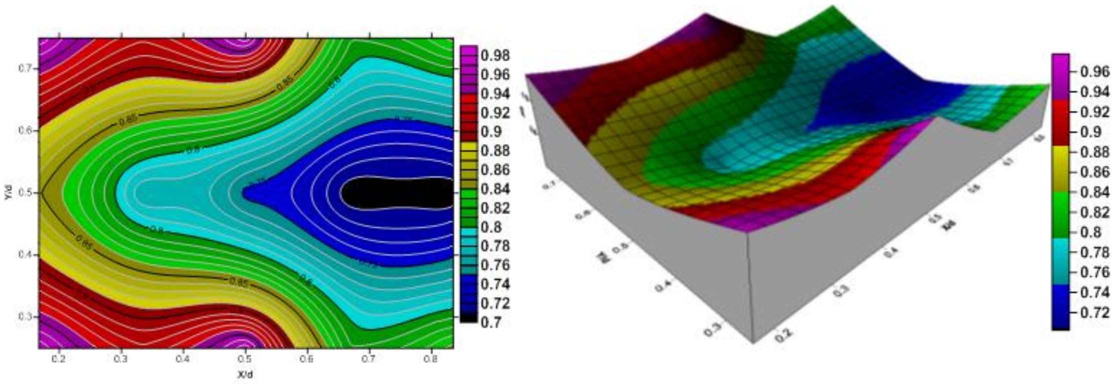


Fig.(6). Variation of (Kb) for (Y/Yo=50%)

It can be seen from figure (7), that for (Y/Yo=60%), the high values of (Kb) are observed along the both sides of gate bottom surface up to (X/d=60%) which are followed by a uniform decrease across and along its middle part toward the trailing edge. Hence, the slight fluctuation of reattachment is revealed clearly on both sides of bottom gate surface, whereas the less values of (Kb) for the remaining parts refer to the beginning of flow lines separation which mostly cover the middle part of bottom gate surface.

Fig.(7). Variation of (Kb) for (Y/Yo=60%)



For (Y/Yo=70%), the peak values of (Kb) are moved along the both side edges up to (X/d=70%) as it can be seen in figure (8) and accordingly, the flow attachment start from both sides of leading edge and then the separation seem to be take place beyond ((X/d=70%) toward the middle and the trailing edge of gate bottom surface. The minimum values of (Kb) which correspond to clear appearance of separation are located just in the center of gate bottom surface.

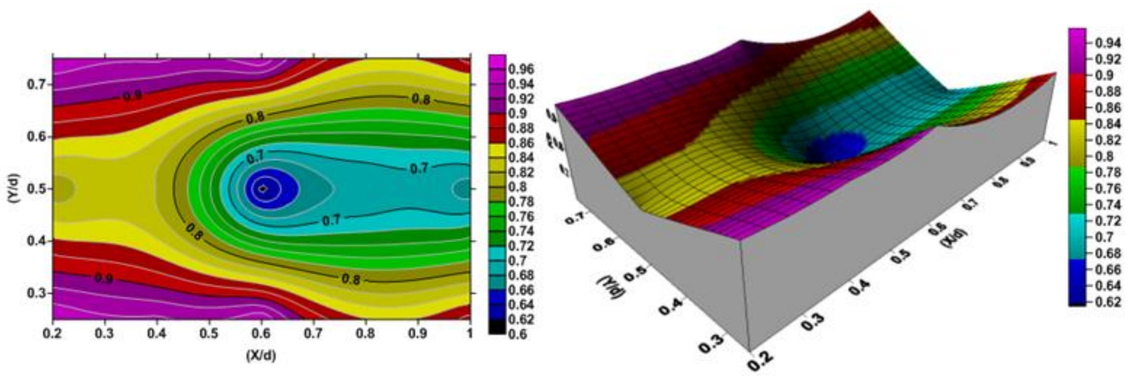


Fig.(8). Variation of (Kb) for (Y/Yo=70%)

Figure (9) indicates the variation of (Kb) for (Y/Yo=80%), It can be noticed that the peak values of (Kb) are located on both sides of gate bottom surface for (X/d=40%) and become less for the all other points. The values of (Kb) start to get low before and after (X/d=40%) and attains its lowest values within the last middle third of gate bottom surface bounded by (X/d=50%) and trailing edge. Such low values of (Kb) indicate a poor attachment of flow stream lines with gate surface.

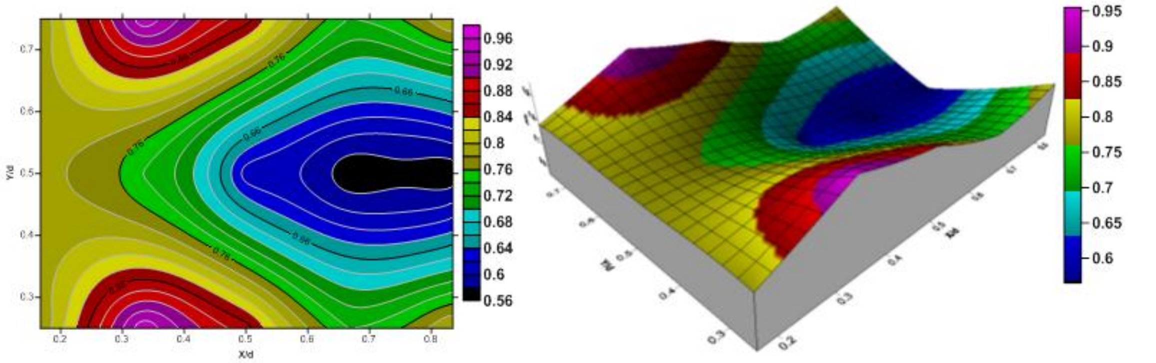


Fig.(9):Variation of (Kb) for (Y/Yo=80%)

5. Conclusions

For gate opening ratios (Y/Yo) up to 50 %, the peak values of (Kb) are mostly located at the leading edge and fluctuated slightly on both sides of gate bottom surface up to approximately (X/d=70%). Subsequently, the (Kb) values have been reduced uniformly along and across the second half of gate bottom surface toward the trailing edge of gate.

For (Y/Yo=70%), the peak values of (Kb) are distributed along both side edges and then decreased smoothly toward the middle of first half, whereas, for second middle half, the values of (Kb) became much more less and attains lowest value for (X/d=60%). The peak values of (Kb) for (Y/Yo=80%) are located on both side edges for (X/d) between 30% and 60%.

The lowest values of (Kb) are found to occupy the last middle part of gate bottom surface, such that beyond (X/d=70%) for (Y/Yo=10% and 40%) and (X/d=50%) for the remaining (Y/Yo) except (Y/Yo=70%), where they focused around (X/d=60%). The peak values of (Kb) wherever exist, assured that the phenomenon of stream lines flow attachment have been established and hence, the parts of gate bottom surface exposed to such case needs to strengthen. And accordingly, the lowest values indicate the potential for separation.

The using of three-dimensional representation reveals that the attachment and reattachment of flow lines are observed sequentially from leading and along both side edges of gate bottom surface between up to (X/d=70%) for most gate openings (Y/Yo) .For (Y/Yo= 80%), the strong impact of attachment is concentrated on (X/d=40%) for both side edges.

Such fluctuation series in pressure impact below the gate bottom surface lead to more probable occurrence of vibration and thus, instability and significant challenges to mechanical system of gate fixing and operation. Accordingly, increase the strengthen of gate is required especially for zones those exposed to high values of underneath pressure. The peak values of bottom pressure coefficients are seem to be differs slightly with gate openings (Y/Yo) and mostly it can be considered as independent to the changes in (Y/Yo).

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Derivative Spectrometric Determination of Glucose and Asparagine in Potato Samples

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Received: April 5, 2016 Accepted: May 10, 2016 Online Published: December 1, 2016

Abstract: Glucose can be detected using the first derivative spectrum depending upon the multipacks at the range 290-350 nm, while it can be determined in the concentration range (3.0-9.0) at maximum positive absorption at 260 and 320. The second derivative spectrum of the glucose can be used for the determination of glucose in the concentration range (1.0-9.0) ppm at positive peak 240 nm and in the range (2.0-9.0) at negative peak. The first derivative spectrum of asparagine which can be used for the determination of asparagine in the concentration range (4.0-20) at positive peak 245 nm and at negative peak 290 nm respectively. Asparagine can be determined using the second derivative spectrum as shown in the concentration range (4.0-14) at positive peak 230 nm and at negative peak 270 nm respectively. The method has been applied for determination of glucose and asparagines in potato samples and the results were compared with that obtained by HPLC.

Keywords: Derivative Spectrometric, Determination, Glucose, Asparagines, Potato Samples

1. Introduction

Derivative uv/visible spectrophotometry has been widely used over the last years in the analysis of multi component mixtures. Therefore, various procedures have been used to solve the problem of the overlapping the peaks. The use of digital differentiation by means of computers to obtain derivative spectra has been the usual procedure. Higher order derivatives can easily be obtained. The facility in the collection and treatment of the spectra has allowed analysis of multi component mixtures of analysis with strongly overlapping spectra. Step by step filter method was investigated to avoid such fading and providing, derivative spectra with initially good signal-to-noise ratio (Ojeda et al., 1995; El-Sayed & El-Salam, 2005).

During the photosynthesis in plants and some prokaryotes, glucose is one of the products. In animals, glycogenolysis leads breakdown of glycogen and produce glucose, while starch in plants is the source of glucose. Maize, rice, wheat, potato, cassava, arrowroot, and sago are used in various countries of the world as the source of starch (Panda, 2005). Glucose plays critical rule in the production of proteins and in lipid metabolism. Also, in plants and most animals, glucose is the main source for vitamin C (ascorbic acid) production (Nayak, 2007). Medically the lower level of glucose in blood than the normal is called hypoglycemia. Hypoglycemia is less common in non-diabetic persons, but can occur at any age, from many causes (“Hypoglycemia”n.d.). Hyperglycemia or high blood sugar is a condition in which an excessive amount of glucose circulates in the blood plasma. Long-term hyperglycemia causes many of the long-term health problems associated with diabetes, including eye, kidney, heart disease and nerve damage. A spectrophotometric method for the determination of glucose with glucose oxidase using titanium (IV)-4-(2'-pyridylazo) resorcinol reagent was suggested. This method of glucose determination was rapid, convenient and showed minimal interference from reducible substances (Ampon et al., 1994). A kinetic-spectrophotometric method for determination of glucose in solutions was described (Zaitoun, 2006). An internal standard was used for rapid analysis of potato sugars by gas chromatography. The method was based upon stearic acid as an internal standard in the analysis of sugars in potatoes by gas chromatography (Varns & Shaw, 1973). Also a rapid quantification of glucose, fructose and sucrose in potato tubers by capillary gas chromatography was investigated (Davies, 1988). A HPLC method with refractive index detection (RID) was developed for sugar determination in Royal jelly (Sesta, 2006).

One of the 20 most common natural amino acids is asparagine. Asparagine can be carboxamide as the side chains functional group. So it is considered a non-essential amino acid by scientists (Devlin, 1993). Asparagine is found in animal sources: dairy, beef, poultry, eggs, seafood and in plant sources: asparagus, potatoes, legumes, nuts, seeds, whole grains (Campbell & Farrell, 2006).

Health hazards-acute and chronic: The toxicological properties of this compound have not been thoroughly investigated, possibly toxic via multiple routes. The free amino acid asparagine and the reducing sugars glucose and fructose have been reported to serve as precursors for the heat-induced formation of potentially toxic acrylamide in a variety of plant-based food (Vianti et al., 2006). Colorimetric ninhydrin assay for was described for asparagine, the suitability of this assay for determining asparagine in food samples (Yokohira et al., 2008). Lee et al. described the genipin, a hydrolysate of geniposide from gardenia fruits, produces blue pigments on reaction with amino acids (Hurst et al., 1995). A sensitive high performance liquid chromatography method for assaying asparagine synthetase and its glutaminase activity was investigated (Lee et al., 2003). Hippe described the rapid determination of free asparagine and glutamine in potato tubers by HPLC (Unnithan et al., 1984). High-performance liquid chromatography (HPLC) analysis was used for identification of two problematic ureides, asparagine and citrulline (Hippe, 1988).

In the present work derivative spectrometric and HPLC methods were described for determination of glucose and asparagine in potato samples.

2. Reagents and Solutions of the Experiment

All chemicals were used of analytical reagent grade. Glucose stock solution (100µg/ml) (BDH) was prepared by dissolving 0.025 g of glucose in distilled water and diluting to 250 ml in a volumetric flask. Each working solution was freshly prepared by suitable dilution of the stock solution with distilled water.

Asparagine stock solution (100µg/ml) (Fluka): Was prepared by dissolving 0.025 g of asparagine in distilled water and diluting to 250 ml in a volumetric flask. Each working solution was freshly prepared by suitable dilution of the stock solution with distilled water.

Hydrochloric acid solution (0.1M) (BDH): This solution was prepared by dilution of the appropriate volume of concentrated hydrochloric acid 36% with distilled water. Sodium hydroxide solution (0.1M) (SCP): 0.4 g of NaOH was dissolved in distilled water and diluted to 100 ml.

Sodium bicarbonate solution (0.05M) (BDH): 1.05 g of $NaHCO_3$ was dissolved in distilled water and diluted to 250 ml. Buffer solution (pH=10): was prepared by mixing 18.3 ml of 0.1M NaOH with 50 ml of 0.05M $NaHCO_3$ and diluting to 100 ml in a volumetric flask (Bai & Wood, 2007).

Carrez reagent I solution (3.60%) (SCP): 3.60 g of potassium hexacyanoferrate (II), $K_4[Fe(CN)_6]$. 3 H_2O was dissolved and diluted to 100 ml with distilled water; Carrez reagent II solution (7.20%) (BDH): 7.20 g of zinc sulphate, $ZnSO_4.7H_2O$ was dissolved and diluted to 100 ml with distilled water (Lide, 2004). Perchloric acid solution (Riedel de Haen AG) (1.0M): This solution was prepared by dilution of the appropriate volume of concentrated perchloric acid 70% with distilled water ("Bochringer"n.d.) potassium hydroxide solution (2.0M) (Philip Harris): 11.22 g of KOH was dissolved in distilled water and diluted to 100 ml.

3. Apparatus

Spectral measurements were carried out on a CECIL CE 3021 UV/Visible digital double- beam spectrophotometer using 1-cm glass cells. Absorbance measurements were performed on NV 203 single-beam spectrophotometer (Invent AB), equipped with glass cell of 1-cm path length. Soxhlet extraction was used for acrylamide extraction. Rotary evaporator LABOROTA 4000(Heidolph) was used for evaporate methanol solution. A model HPLC instrument delivery system (Perkin almar-germany). Injections were made with a 20μl loop. The chromatographic separations were performed using a luna column ODS C18 (250×4.6mm /5.0 Micron), UV detector.

4. Sample Preparation

4.1 Extraction of Glucose in Potatoes

Samples of 50.0 g peeled potatoes were homogenized with 50 ml water in a homogenizer for 3.0 min and then transferred quantitatively into a 250 ml beaker. The volume was filled up to 150 ml with water, and 5.0 ml of each carrez-I- and carrez-II- solutions were added successively with mixing after each addition. The pH was adjusted to pH 7.0 to 7.5 with sodium hydroxide. The solution was transferred quantitatively into a 250 ml volumetric flask, rinsed with water and 0.3 ml of n-octanol was added with shaking until the foam was disappeared. The volume was filled up to 250 ml with distilled water the extract was filtered and first 2-3 drops was discarded ("Megazyme Assay Procedure" n.d.).

4.2 Extraction of Asparagine in Potatoes

Accurately 10.0 g of representative potato sample was weighted and transfered to 100 ml Duran bottle, then 20 ml of 1.0 M perchloric acid was added and homogenized for 2.0 min using homogeniser. The mixture was quantitatively transferred to 40 ml glass beaker and the pH was adjusted to 8.0 using 2.0 M KOH solution. The volume was completed to 100 ml in a volumetric flask and stored in ice bath for 20 min to precipitate. Potassium perchlorate and separation of any fat present. The mixture was filtered and the first 3-5 ml was discarded ("Megazyme Assay Procedure" n.d.).

5. Results and Discussion

With increasing the number of components in a mixture, derivatisation of the mormal spectrum to higher order is required for simultaneous determination of these components in the mixture with the aid of derivative uv/visible spectrophotometric methods, because derivative spectra with first, second, third and fourth orders are more complicated than zero order. This complexity of the higher order of the spectra leads to more positive and negative peaks. In derivative spectroscopy, the derivative of a spectrum is used both for qualitative and quantitative evaluation. Many of the major applications of derivative spectroscopy in the ultraviolet and visible spectral region serve to qualitatively identify and analyze samples. Specific details in the fine structures of the derivative spectra permit compounds with overlapping spectra to be distinguished. Derivative spectroscopy has also proven to be useful in the examination of analytes in the case of spectral interference. By derivation of the spectrum it is possible to suppress the background overlaid to a sample spectrum and make the specific absorption stand out more clearly (Owen, 2000).

First and second derivative spectra of glucose and asparagine were recorded as shown in Figs (1-4) respectively.

Glucose can be detected using the first derivative spectrum depending upon the multipeaks at the range 290-350 nm, while it can be determined in the concentration range (3.0-9.0) at maximum positive absorption at 260 and 320 nm as shown in the Figs (5) and (6) respectively.

The second derivative spectrum of the glucose can be used for the determination of glucose in the concentration range (1.0-9.0) ppm at positive peak 240 nm and in the range (2.0-9.0) at negative peak as shown in Figs. (7) and (8) respectively.

The first derivative spectrum of asparagine which can be used for the determination of asparagine in the concentration range (4.0-20) at positive peak 245 nm and at negative peak 290 nm as shown in the Figs. (9) and (10) respectively.

Asparagine can be determined using the second derivative in the concentration range (4.0-14) at peak 270 nm.

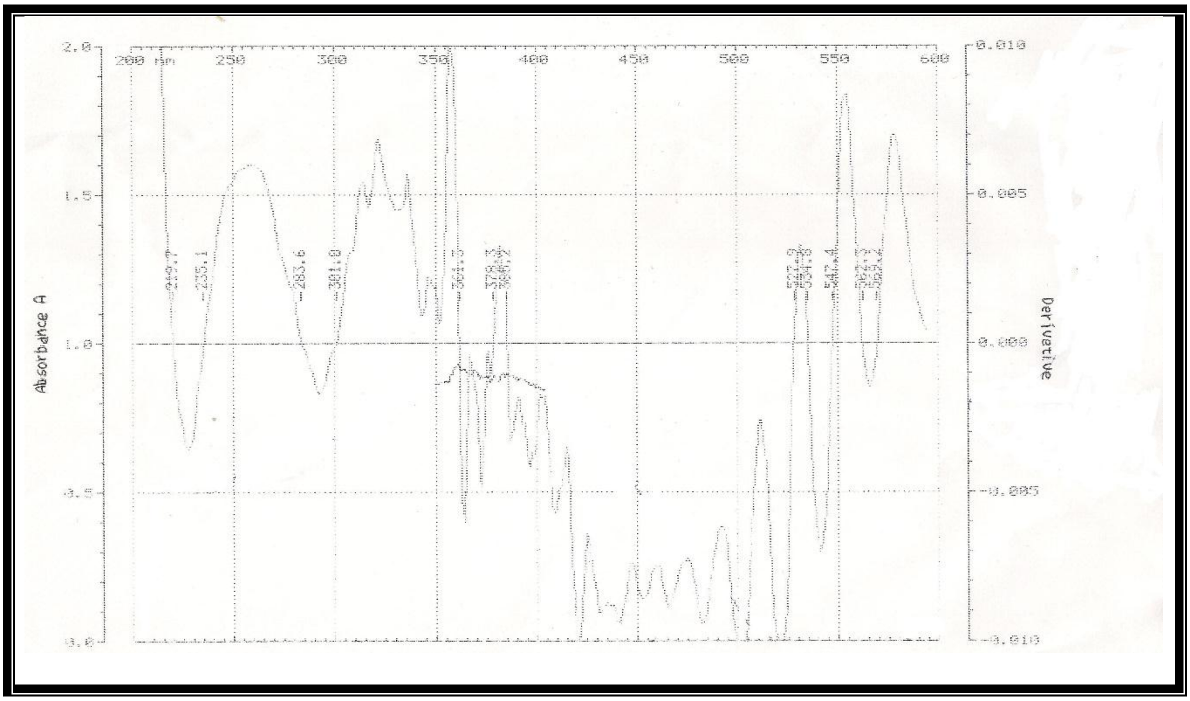


Fig.(1). First derivative spectrum of 8.0µg/ml glucose

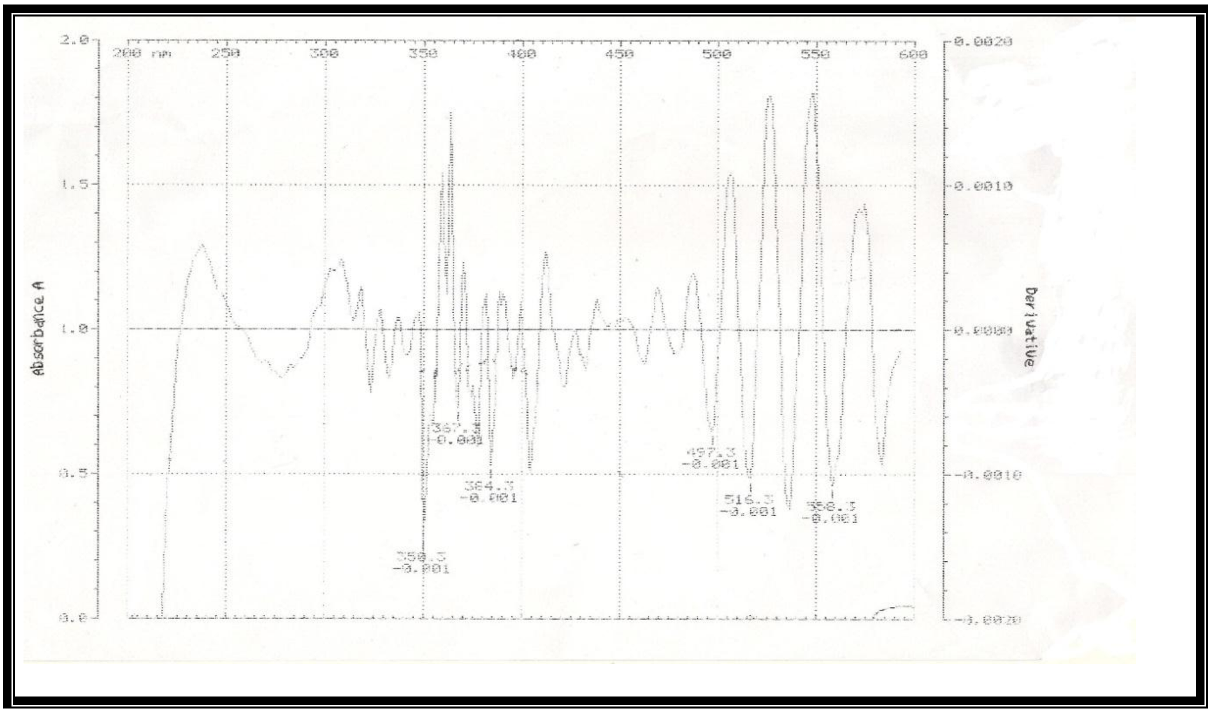


Fig.(2). Second derivative spectrum of 8.0µg/ml glucose

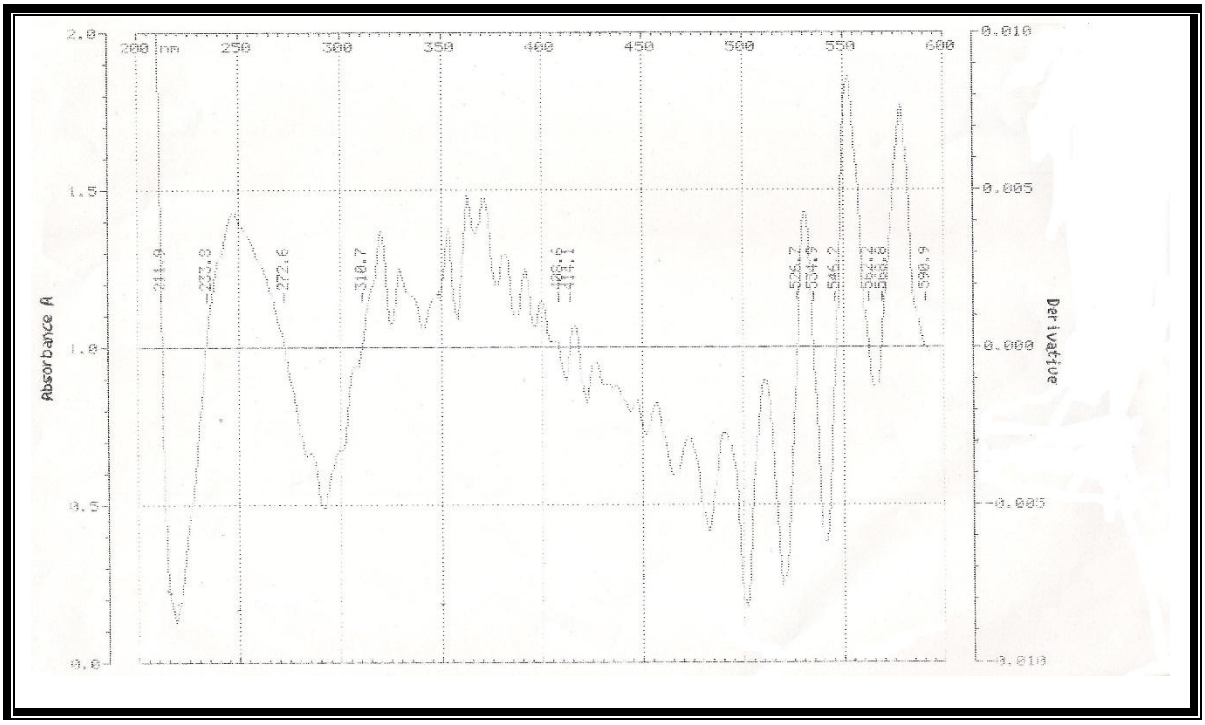


Fig.(3). First derivative spectrum of 12.0µg/ml asparagine

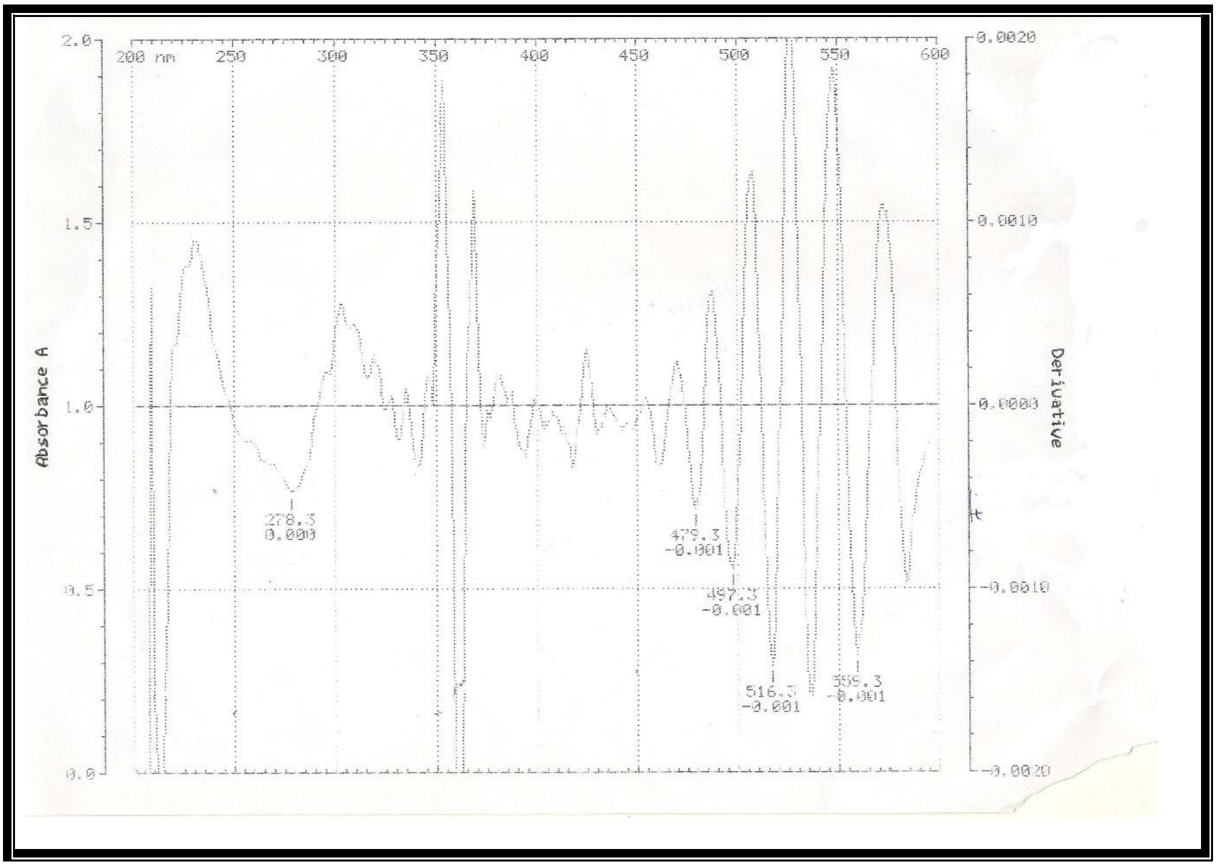


Fig.(4). Second derivative spectrum of 12.0µg/ml asparagine

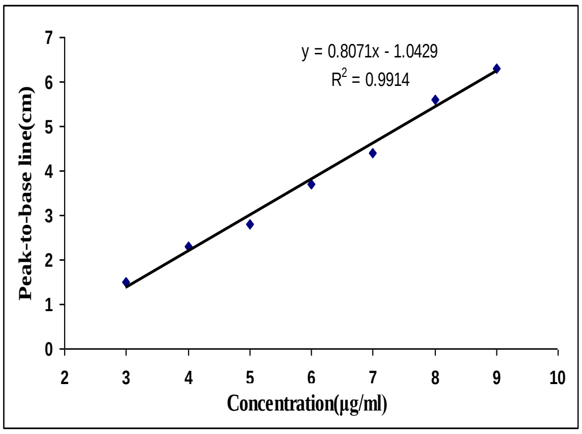
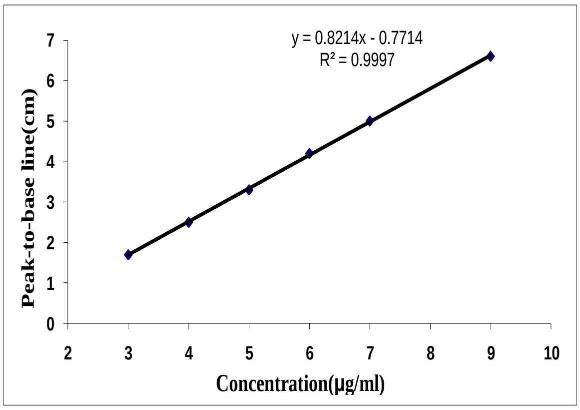


Fig. (5). Calibration curve of glucose using 1D at 260 nm



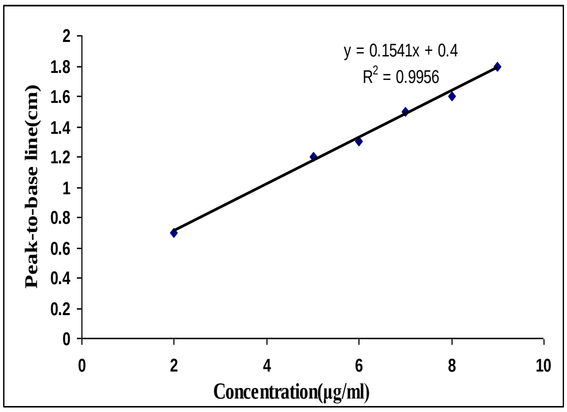
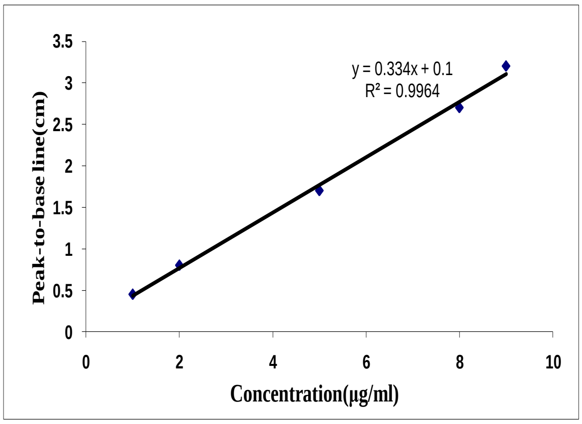


Fig. (6). Calibration curve of glucose using 1D at 320 nm

Fig. (7). Calibration curve of glucose using 2D at 280 nm



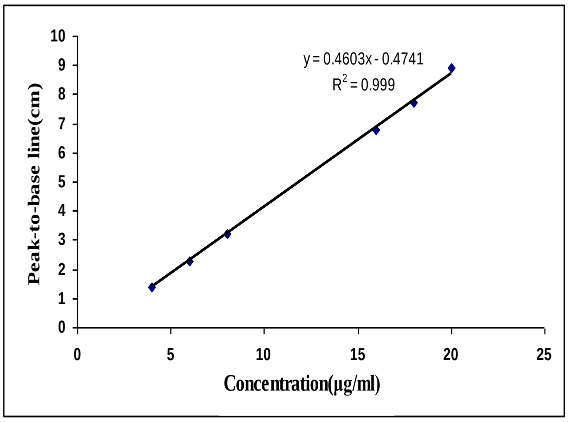
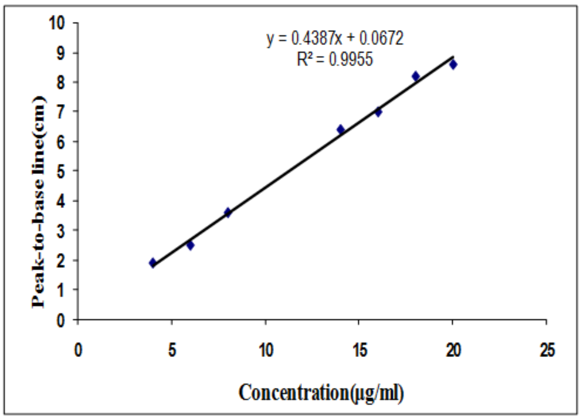


Fig. (8). Calibration curve of glucose using 2D at 235 nm

Fig. (9). Calibration curve of asparagine using 1D at 245 nm



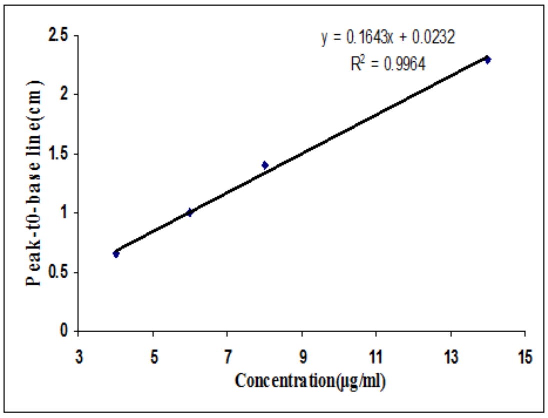


Fig. (10). Calibration curve of asparagine using 1D at 290 nm

Fig. (11). Calibration curve of asparagine using 2D at 270 nm

6. HPLC Determination of Glucose and Asparagine in Potato Samples

Glucose and asparagine were also determined in potatoes samples applying HPLC using column C18. Acetonitrile-water (75:25) was used as mobile phase with flow rate 1.8 ml/min for determination of glucose, while 50mM solution of acetate buffer (pH 5.9) and methanol were used as mobile phase with flow rate 1.0 ml/min for determination of asparagine. The detection wavelength for both determinations was at 340 nm.

The chromatograms of glucose and asparagine were compared with those of the standards of the mentioned compounds (Wilson, et al., 1981).

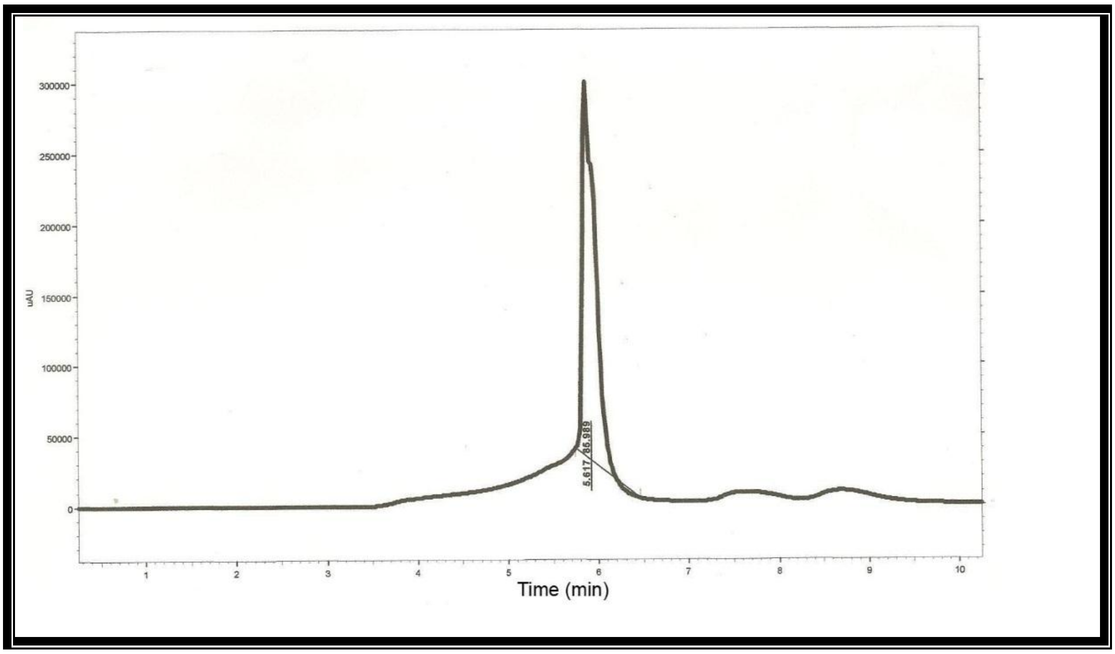


Fig. (12). HPLC chromatogram of standard glucose.

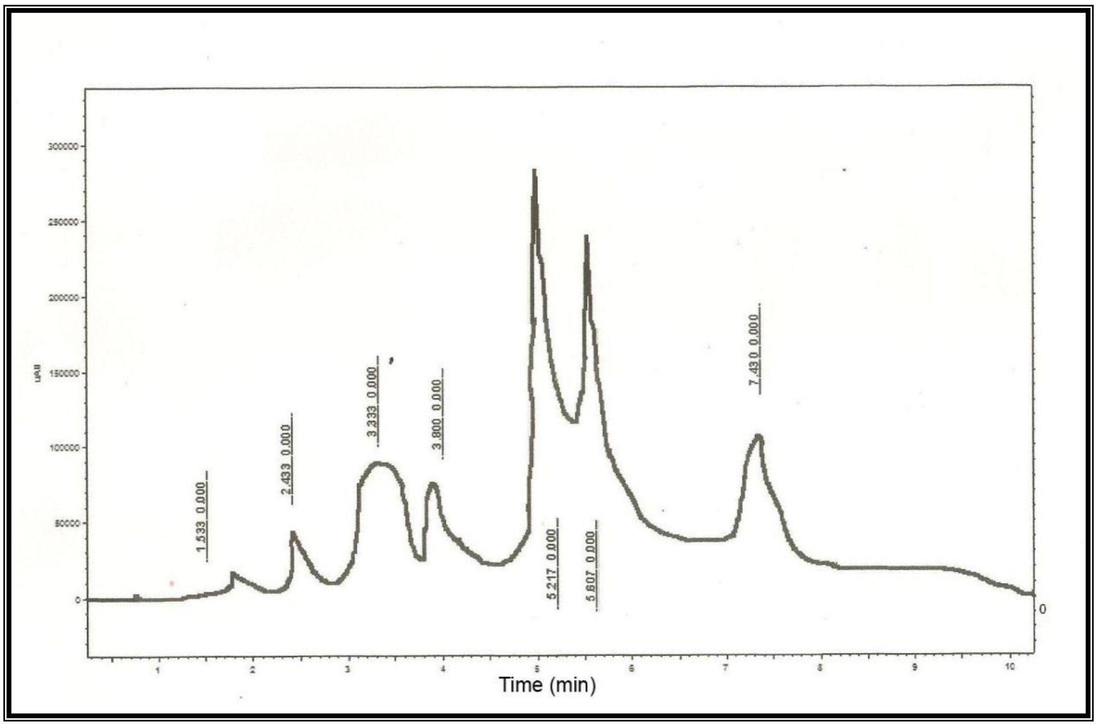


Fig. (13). HPLC chromatogram of glucose in potato sample.

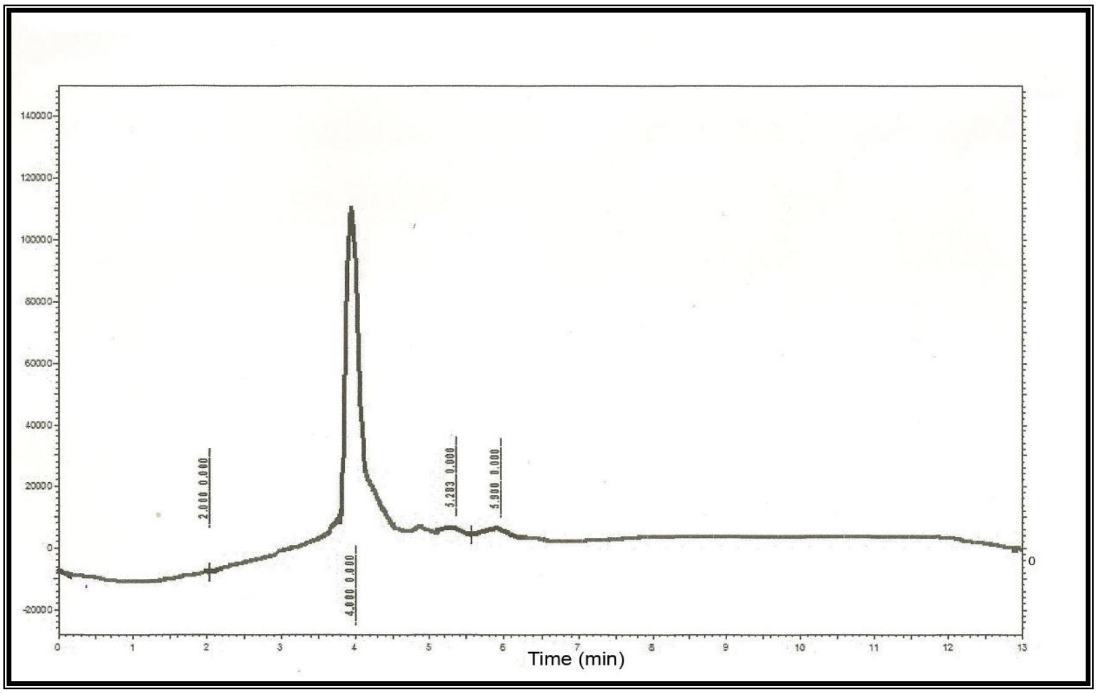


Fig. (14). HPLC chromatogram of standard asparagine.

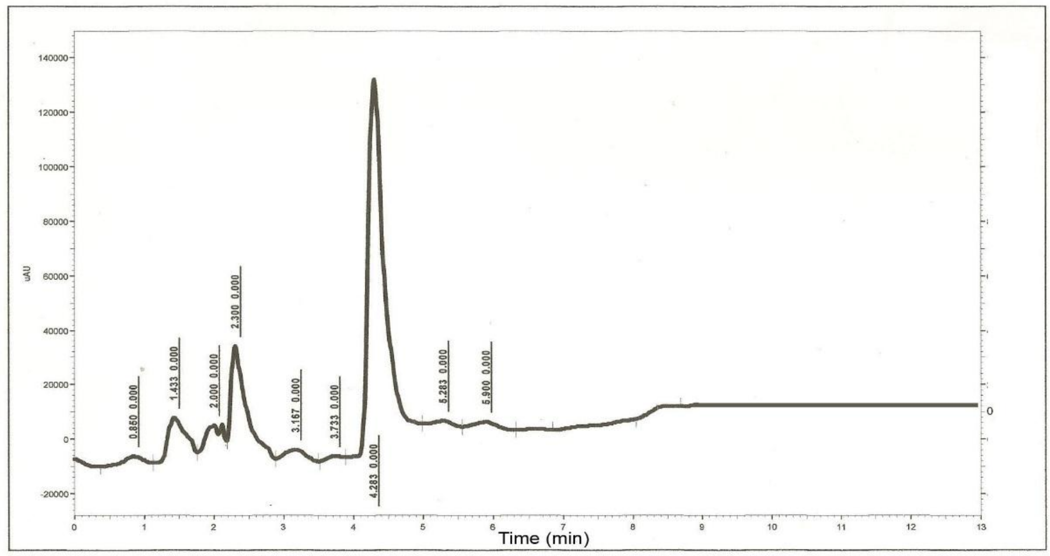


Fig. (15). HPLC chromatogram of asparagine in potato sample.

Table 1: Rtention times of acrylamide, glucose and asparagine in different potato samples

Samples	Retention time(tR)
Standard glucose	5.617
Standard asparagine	4.00
Potato sample	5.6
Potato sample	4.28

	Proposed method at 260 nm using 1D (glucose)	Proposed method at 280 nm using 2D (glucose)	HPLC method	Proposed method at 245 nm using 1D (asparagine)	HPLC method
Fresh potato	5.5	8.4	7.4	3.2	3.1

Results of 2D spectra of the extracts of the potato sample for determination of glucose shows that the result at 280 nm is of acceptable agreement with that of HPLC method.

In the case of determination of asparagine in the extracts of potato sample 245 nm of the 1D spectrum shows acceptable agreement with results obtained using HPLC method.

Table (5-1): Derivative spectrometric determination of glucose and asparagine in different potato samples

7. Conclusion

The proposed first and second derivative methods for determination of glucose and asparagine are simple, rapid (as it only requires measurements of nD values at a single wavelength).

They demonstrates the potential of first and second derivative spectrophotometry method as an analytical technique and its usefulness to accurate, rapid, simple, and quantitation of glucose and asparagine in different potato samples.

They were used for identification of the mentioned compounds depending upon characteristic peaks at certain wavelengths or ranges. The first and second derivative spectra of the mentioned compound have been used for determination of the compounds at different ranges of concentration depending upon the measurements of the heights of the peak to the baseline at certain wavelengths.

Glucose and asparagine were also determined in potatoes samples applying HPLC using column C18. The results of both methods were in good agreement.

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