بِسۡمِ ٱللَّهِ ٱلرَّحۡمَٰنِ ٱلرَّحِيمِ

SUDAN UNIVERSITY OF SCIENCE AND TECHNOLOGY

COLLEGE OF GRADUATE STUDIES

INTERACTION OF IRRIGATED AREAS POWER AND SEDIMENT IN AL ROSEIRES DAM

التداخلات في المساحات المروية الطاقة والأطماء في خزان الروصيرص

A DISSERTATION SUBMITTED IN FULFILLMENT FOR THE DEGREE OF DOCTOR OF PHILOSOPHY IN WATER RESOURCES ENGINEERING

BY

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بِسۡمِ ٱللَّهِ ٱلرَّحۡمَٰنِ ٱلرَّحِيمِ

قال تعالي:

(وَلَئِنْ سَأَلْتَهُمْ مَنْ نَزَّلَ مِنَ السَّمَاءِ مَاءً فَأَحْيَا بِهِ الْأَرْضَ مِنْ بَعْدِ مَوْتِهَا لَيَقُولُنَّ اللَّهُ ۚ قُلِ الْحَمْدُ لِلَّهِ ۚ بَلْ أَكْثَرُهُمْ لَا يَعْقِلُونَ)

صدق الله العظيم

سورة العنكبوت ﴿٦٣﴾

DEDICATION

I dedicate this work to my friends, brothers, sisters, and in a special way to soul of my father Hamed Elamein and my mother Solha Khalid and my Husband Mahamed Zeidan and my elder brothers Mohamed Hamed,Kamal Hamed, Esam Hamed, Gasem Hamed,Hashem Hamed,Atif Hamed, and Neamat Hamed who through their efforts spiritually, materially, physically and financially had made me to complete my studies peacefully and successfully.

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First of all, Thanks be to Al-mighty Allah; the most beneficial, the most merciful and most glorified, the Creator and master at the last day of judgment whom by his will I was able to conduct this research.

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Finally, my heartfelt thanks to my **Husband** who always trusted me and gave me his unconditional love and full Support.

الملخص

الخزانات وبحيراتها يتم انشاؤها على الأنهر لأغراض متعددة. التوليد المائى والرى تحظى بالأسبقية الأولى.بالرغم من أن الزراعة المرويه فى السودان تنتج حوالى 50 % من كل المحاصيل، لكن يلازمها المعاناه المتمثله فى ازالة الطمى من شبكة أنظمة الرى.مشكلة الاطماء تعرض تشغيل بحيرة خزان الرصيرص للخطر.أضافة الى ذلك فأن بحيره خزان الروصيرص تعانى من الانجراف على الجسور خلف بحيرة الخران ،ونقصان فى امدادات مياه الرى للمشاريع الزراعية ويصاحب ذلك مشاكل فى التوليد المائى.

الهدف من هذه الدراسة هو تقييم العلاقات بين الطمى والمياه المستعمله فى الرى.كما أن الهدف يشمل التدليل على العلاقات المتداخله بين التصرف والطمى والتوليد المائى.

لحل هذه المشاكل وتحقيق الاهداف قام الباحث باجراء الدراسات البعديه مستخدما نظرية

(باى π) لباكنجهام ونماذج ال (SPSS) باستعمال كل من التوافق البسيط والمتعدد المتغيرات.أسم ال (Software) يدلل على الموسوعه الاحصائيه للعلوم الاجتماعية. التوافق البسيط اعطى نتائج معامل بقيمة توافقيه عالية.تم الحصول على مجموعات كاملة غير بعدية باستخدام قوانين التحليل غير البعدى ونظرية النماذج فى التحليل التوفقى المتعدد.

دللت النتائج بوضوح العلاقات المتشابكه بين الأربعه مجموعات بمعادلة واحدة. ولتوضيح العلاقات المتشابكة بين التصرف والطمى والتوليد المائى والمساحات الروية ،كل واحد من المتغيرات غير البعدية وضع كمتغير رئيسى بالجانب الايسر من المعادلة والثلاثة الاخرى على اليمين لتوضيح العلاقات المتشابكة ودرجة الاعتماد.أختبار النتائج دلل على توافق جيد مع خطأ مقبول.

Abstract

Dams and reservoirs are constructed in rivers for multipurpose. Hydropower, and irrigation, are predominant. Although irrigated agriculture in Sudan produces about 50 % of the total crop production, yet it is associated with painstaking of removing sediments from the irrigation network system. Sedimentation problem endangers operation performance of Reseires Dam Reservoir. Furthermore Roseires Dam Reservoir is suffering from downstream river bank erosion, insufficient irrigation water for the agricultural schemes, with problems in power generation.

The objective of this study is to assess the relations of sediment and water used for irrigation. The objective is extended to indicate the interaction among discharge sediment and power generation.

To solve these problems and fulfill these objectives the researcher conducted dimensional analysis using Buckingham pi theorem and SPSS models, using both simple and multiple correlation. The software name of SPSS originally stood for Statistical Package for the Social Sciences. The simple correlation gave acceptable results with high correlation coefficients. Complete set of dimensionless groups were obtained using the rules of dimensionless analysis and theory of models in the multiple regression correlation. The results obtained clearly defined the interationship among the four groups in a single equation.

To reveal the interrelationship among discharge, sediment, generated power and irrigated areas, each dimensionless variable was put as independent variable on the left hand side of the equation and the other three on the right hand side to reveal the interaction and the degree of dependence. Examination of the results indicated good correlation with acceptable error.

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LIST OF ABBREVIATIONS

FAO	**Food and Agriculture Organization of the United Nations**
FERC	**Federal Energy Regulatory Commission**
FHWA	**Federal Highway Administration**
HRI	**Hydraulics Research Institute**
ICOLD	**International Commission on Large Dams**
IWRM	**Integrated Water Resources Management**
MRSS	**Model Residual Sum of Squares**
MSS	**Model Sum of Squares**
NTPP	**New Tebbin Power Plant**
NWRC	**National Water Research Center**
RCC	**Roller-Compacted Concrete**
SPSS	**tatistical Package for the Social Sciences**
TSS	**Total Sum of Squares**
WCD	**World Commission on Dams**

CHAPTER ONE

1. INTRODUCTION

1.1. Background:-

Dams and reservoirs are constructed in rivers for flood control, hydropower generation, irrigation, navigation, water supply, fishing and recreation. Among multipurpose dams, hydropower and irrigation dams are predominant. Environmental impacts and long-term morphological changes of the natural water course due to human intervention are inevitable. Sedimentation is one of the major problems which endangers the performance and sustainability of reservoirs. It reduces the effective flood control volume. It poses hazards to navigation, changes water stage and underground water conditions. It also affects the operation of low-level outlets gates and valves, reduces stability, water quality and recreational benefits.

Alarming rates of storage depletion have been reported world-wide and especially in drought prone areas. Mean yearly losses of 0.3% to 1.3% were reported in USA, 0.7 % to 1.5% in Turkey (Bechteler 1997), 1.67% in Sudan and 1% worldwide (Mahmood 1987).

Sedimentation is a complex hydro-morphological process and is difficult to predict. It has been underestimated in the past and was perceived as a minor problem which could be controlled by sacrificing a certain volume of the reservoir for accumulation of the sediment (dead zone). However, nowadays it is of paramount importance to take design and implementation of sediment control measures into consideration in the planning, design, operation and maintenance phases of the reservoirs. The current state of the art in combating this problem of reservoir sedimentation ranges from, measures which used to reduce sediment influx into reservoirs by bypassing, trapping or by watershed management, to measures which use artificial methods (dredging) or utilize natural forces (flushing and sluicing) to clear or release incoming sediment along with the flow. The application of one measure or the other depends on many factors such as the geometry of the reservoir, operational rules, characteristics of the sediment its distribution, and the possibility of the measure. The mostly practiced techniques are sediment flushing and sluicing which is the dredging limited to clearing of blocked hydropower intakes or sediment removal (Loman1994). However, with exception of few cases, these methods are either claimed to be inefficient or cost prohibitive. For example the cost of restoring lost storage by conventional dredging, without the additional cost of providing disposal areas and containment facilities, varies from $2 to $3 per cubic meter (Mahmood 1987). It is well known that sedimentation has often greatly reduced and endangered the live storage of many existing reservoirs coupled with the limitations of the existing sediment control measures. Attention of researchers is focused on the subject of reservoir sedimentation area in water resources engineering. This is because it became clearly apparent that the mechanics of reservoir sedimentation are not yet fully understood.

1.2. Sedimentation Problems in the Agricultural Sector:-

Agriculture is the most significant element in Sudanese economy. Most of the important crops such as cotton, wheat, sorghum, and groundnut are produced by irrigation. Irrigated agriculture produces about 50 % of the total crop production. In the year 1990, due to siltation and reduced water availability, the combined Gezira-Managil scheme probably the largest irrigation complex in the world (760 000 ha), was dropped to 57% as 326800 ha were taken out of production (FAO report 1995a). Figure (1.1) represents painstaking of removing sediments from the Gezira and Managil canals network system. The siltation problem in irrigation networks and pumping station intakes are well known in the field of water resources management, since the early part of this century. Depending on the annual sediment yield, quantities ranging from 10 to 20 million m3 of sediment were removed from the irrigation canals before the beginning of the then coming season (Adam, A.M. 1997). In brief, the rate and magnitude of sedimentation in the new reservoirs and its impact on surface water management was received as a shock (Loman1994). This impact has manifested in progressive reduction in the irrigated areas of main crops and lead to reversal and postponing of some of the ambitious agricultural development schemes (For example; New Halfa and Rahad extension schemes).

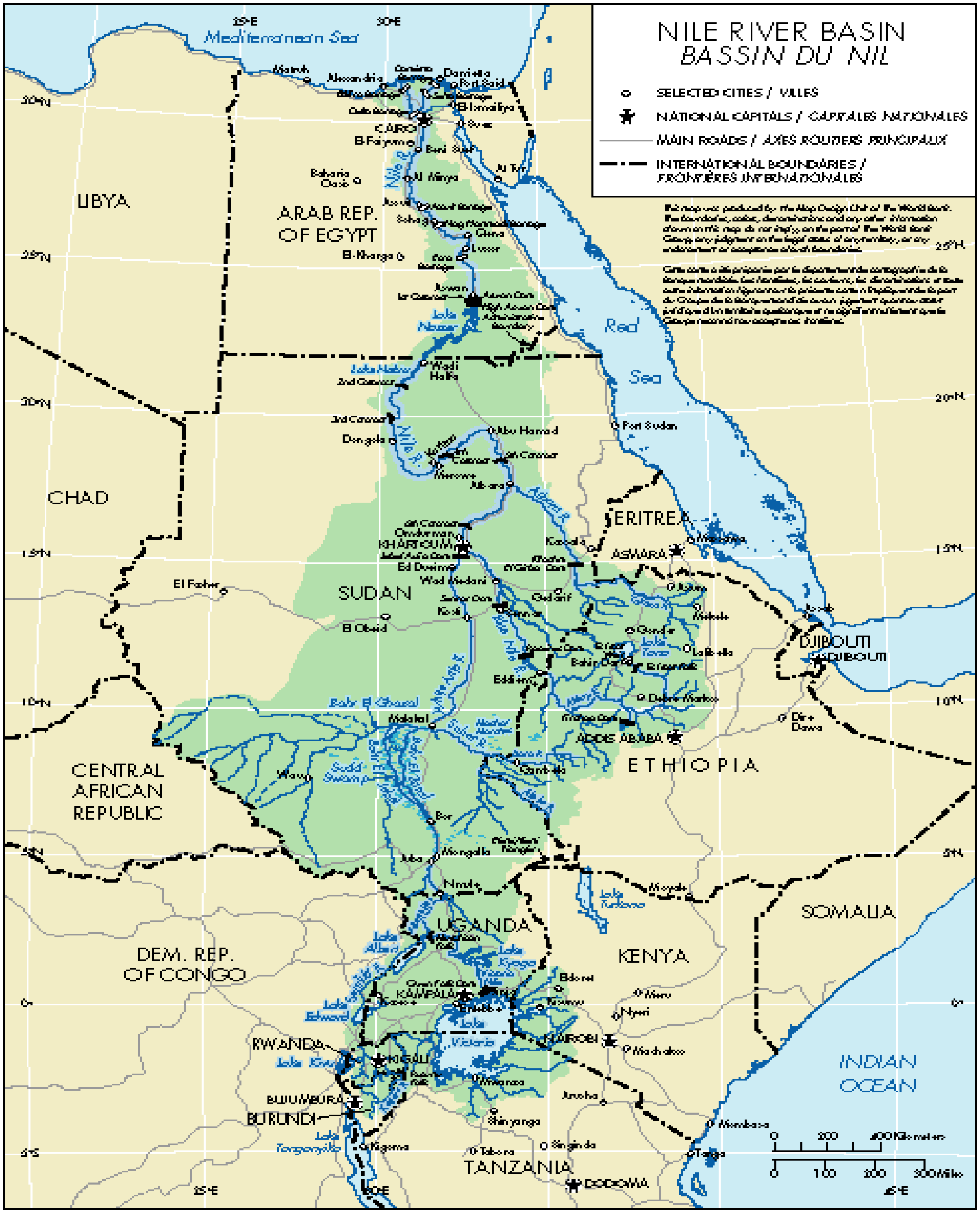
Fig. No. (1.1):Sediment in Irrigation Channels

1.3 Area of Study:-

The area of study is located in the Roseires Dam 550 km south of Khartoum and 110 km from the Sudanese Ethiopian border. The storage capacity of

Roseires Dam reservoir is 3.3 milliards ($3.3 \times 10^9 m^3$) at 481 R.L. (reduced level) in 1966.In the year 2007 it was found that about 42.3 % of its storage capacity was Lost. Figure (1.2), shows the Sudan boundaries and Roseires Dam geographical location.Figure (1.3), shows detailed boundaries in the vicinity of Roseires Dam. An overview of the area of study reveals that the Nile Basin encompasses northeastern and central Africa which is about 1/10th of Africa area. It connects 11 riparian countries:Burandi, Democratic Republic of Congo, Ethiopia, Kenia, Rowanda,Tanzania and Uganda in the upstream (7), Sudan, Eritrea South Sudan in mid-stream (3). Egypt is most downstream country.

The results of this study will help in understanding the situation of river system and the impact of sediment on irrigation water. It will help in planning better way of thinking for decision makers and clear management lines.



<u>Fig.No.(1.2):Sudan Boundaries and Roseires Dam Geographical</u>

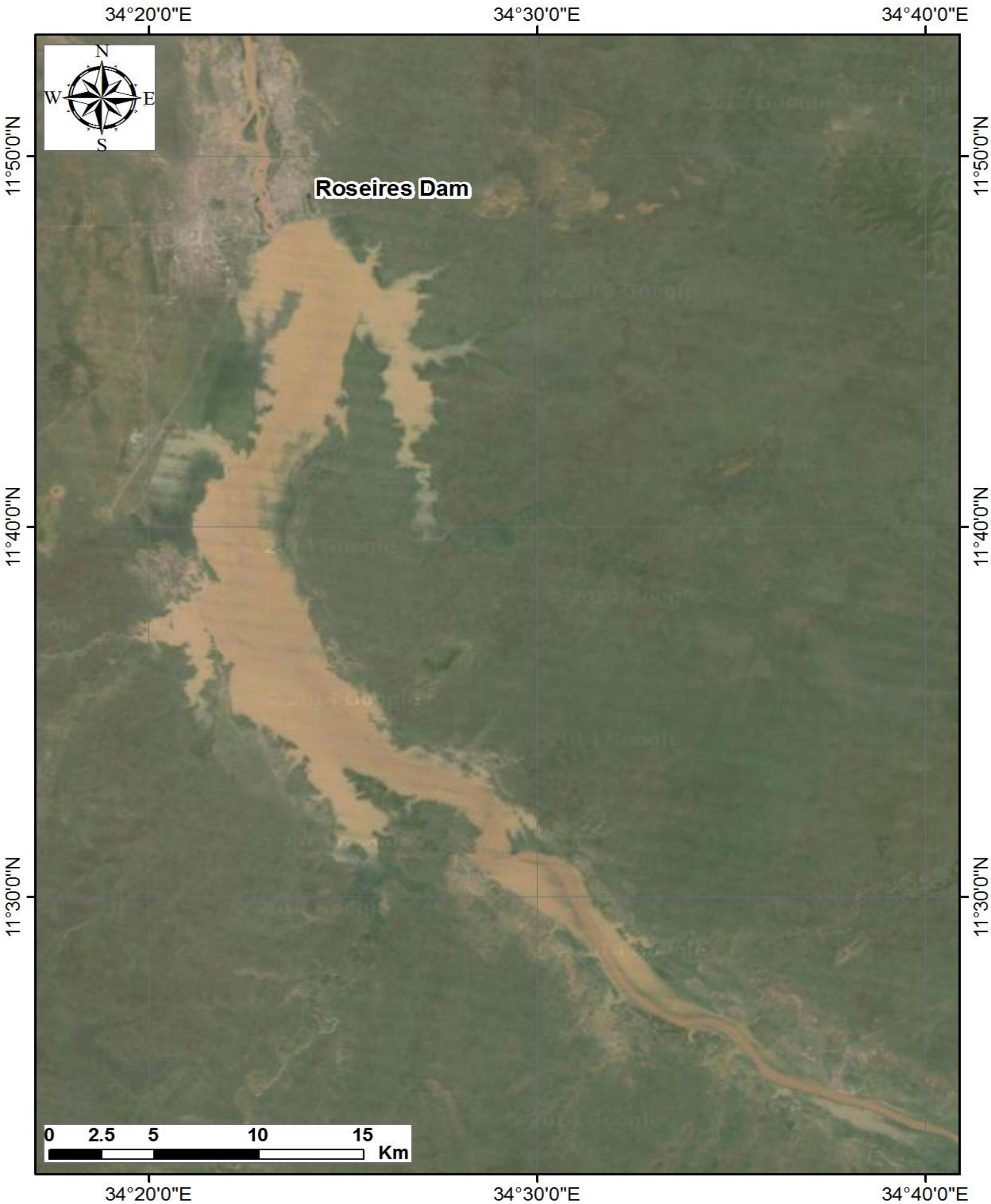


Fig.(1.3):Detailed Boundaries in the Vicinity of Roseires Dam Reservoir

1.4. Statement of the Problems:-

The prevailing problems affecting Roseires Dam are several. They are mainly:-

- Intakes reduced hydropower as well as power outage.
- Upstream downstream sediment transportation.
- Diminishing downstream releases associated with decreasing and fluctuating cultivated areas.
- Complexity in reservoir operation and maintenance coupled with downstream the dam river bank erosion.

1.5. Objectives:-

1.5.1.Main Objective:-

The main objective is to assess the impact of sediment on irrigation water and optimization of use and consumption of water for irrigation.

1.5.2.Specific objectives:-

- Indicate the interaction among discharge sediment and power.
- Determine impact of the Reservoir operation on the schemes of the Gezira, Suki, Rahad, and North West Sennar Sugar Factory.
- Indicate the erosion occurring during flood season.
- Suggest some recommendations to measures that can be taken to reduce the amount of deposited sediment.

1.6 Layout of the Thesis:-

Chapter one gave brief Introduction and cited the problem and objectives of the study with an overview of the significance of the study. It was concluded with this layout of the thesis.

Chapter two presented the literature review cited and described with the necessary figures and tables.

Chapter three is dedicated for the discussions of methodology adopted in the study complemented by materials and equipments used.It included both desk studies citing previous studies and field studies of the study area. Chapter four presented collecting the data, and analysis.

Chapter five issues of the study area were discussed including the analyzed data and results obtained.

Chapter six covered the conclusions and recommendations.

At the beginning of this thesis the abstract in both English and Arabic languages were presented. At the end of the thesis the Appendix and references are presented.

CHAPTER TWO

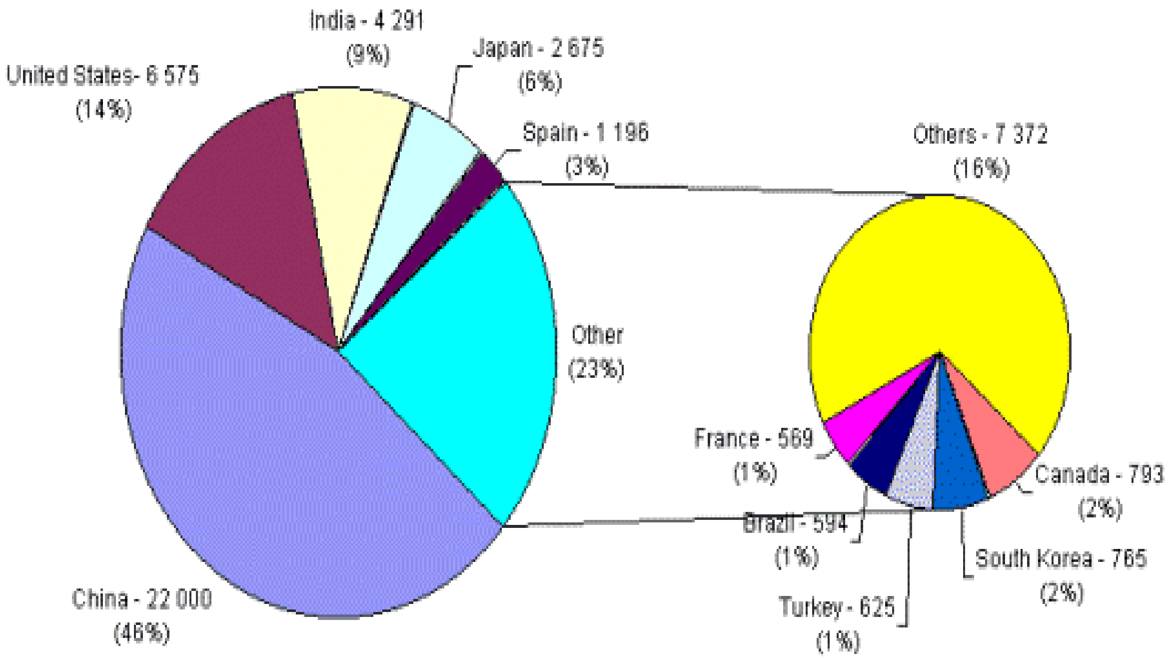
2. LITERATURE REVIEW

2.1. Dams:-

Dams have been promoted as an important means of meeting the need for water and energy services and as long-term, strategic investments with the ability to deliver multiple benefits.

Some of these additional benefits are typical of all large public infrastructure projects, while others are unique to dams and specific to particular projects. Regional development, job creation, and fostering of an industry base with export capability are most often cited as additional considerations for building large dams. Other goals include creating income from export earnings, either through direct sales of electricity or by selling cash crops or processed products from electricity-intensive industry such as aluminum refining. Clearly, dams can play an important role in meeting people's needs (WCD, 2000) Hydropower accounts for more than 90% of the total electricity supply in 24 countries, such as Brazil and Norway. Half of the world's large dams are built exclusively for irrigation, and dams are estimated to contribute to 12-16% of world food production. In addition, in at least 75 countries large dams have been built to control floods. For many nations, dams remain the largest single investment project in the country. These hydropower, irrigation, water supply and flood control services were widely seen as sufficient to justify the significant investments made in dams, and other benefits were often cited as well. These included the impact of economic properties on a region due to multiple crops, rural electrification and the expansion of physical and social infrastructure such as roads and schools. The benefits were seen as self-evident. When balanced with the construction and operational costs - in economic and financial terms -

these benefits were seen to justify dams as the most competitive option (WCD, 2000).Figure (2.1) is WCD estimates, based on ICOLD.



<u>Fig.No.(2.1): WCD Estimates, Based on ICOLD</u>

There are 45 000 large dams in the world today. A large dam is defined as a dam with a height of 15 m or more from the foundation, or a height of 5 m or more but with a reservoir volume of more than 3 millions cubic meters (WCD, 2000). Dam construction is one of few attractive options of many nations and investors in the last decades. Their contribution to economic growth cannot be denied, but the impacts were also recognized on environment and human beings.

The two principal poles in the debate illustrate the range of views on past experience with large dams. One perspective focuses on the gap between the promised benefits of a dam and the actual outcomes. The other view looks at the challenges of water and energy development from a perspective of 'nation building' and resourced allocation. In terms of the social impacts of dams, the

WCD found that the negative effects were frequently neither adequately assessed nor accounted for. The range of these impacts is substantial, including those on the lives, livelihoods and health of the affected communities dependent on the riverine environment (WCD, 2000). Among all dams that were built during the last century, the vast majority of irrigation and hydropower dams are single purpose. In the early 1970s, the environmental movement, notably in the USA, claimed that water resources projects, single or multipurpose, should encompass multi objective analysis. Economic efficiency should be considered at national and regional level and environmental impacts and mitigation measures should be part of the decision making process. More recently, a full multi objectives and multipurpose analysis with public participation is required for water resources development. Multi objectives include consideration of economic efficiency at national and regional levels, equity, environmental conservation, social and political acceptance and national security (Annandale, G.W., 1984). The concept of Integrated Water Resources Management (IWRM) provides a holistic approach- based framework for water management, that places due emphasis on equitable social welfare, poverty reduction and sustainability of ecosystems. IWRM is defined as "... a process which promotes the coordinated development and management of water, land, and related resources in order to maximize the resultant economic and social welfare in an equitable manner without compromising the sustainability of vital ecosystems.

2.2. Classification of Dams Types:-

Dams may be classified into a number of different categories, depending upon the purpose of the classification. For the purposes of this study, it is convenient to consider three broad classifications. Dams are classified according to their use, their hydraulic design, or the material of which they are constructed.

2.2.1. Classification According to Use:-

Dams may be classified according to the broad function they serve, such as storage, diversion, or detention. Refinements of these classifications can also be made by considering the specific functions involved.

Storage dams are constructed to impound water during periods of surplus supply for use during periods of deficient supply. These periods may be seasonal, annual, or longer. Many small dams impound the spring runoff for use in the dry summer season. Storage dams may be further classified according to the purpose of the storage, such as water supply, recreation, fish and wildlife, hydroelectric power generation, irrigation, etc. The specific purpose or purposes to be served by a storage dam often influence the design of the structure and may establish criteria such as the amount of reservoir fluctuation expected or the amount of reservoir seepage permitted. Figure (2.2) shows a small earth fill storage dam, and figure (2.3) shows a concrete gravity structure serving both diversion and storage purposes.

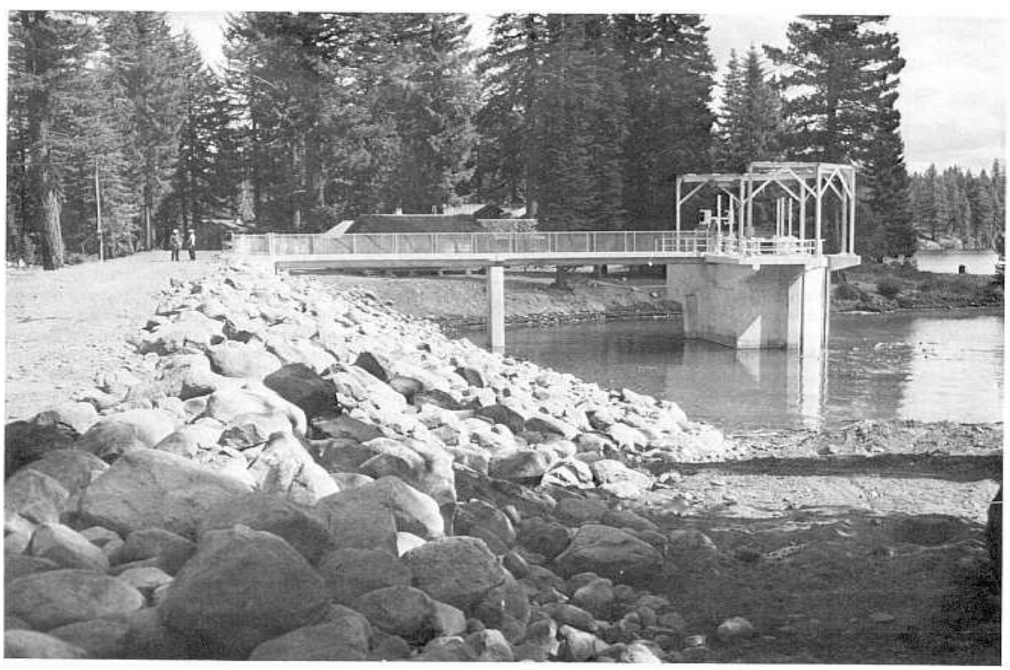


Fig.No. (2.2):Crescent Lake Dam, a Small Earth Fill Storage Dam (Design of small dams, Third Edition 1987)

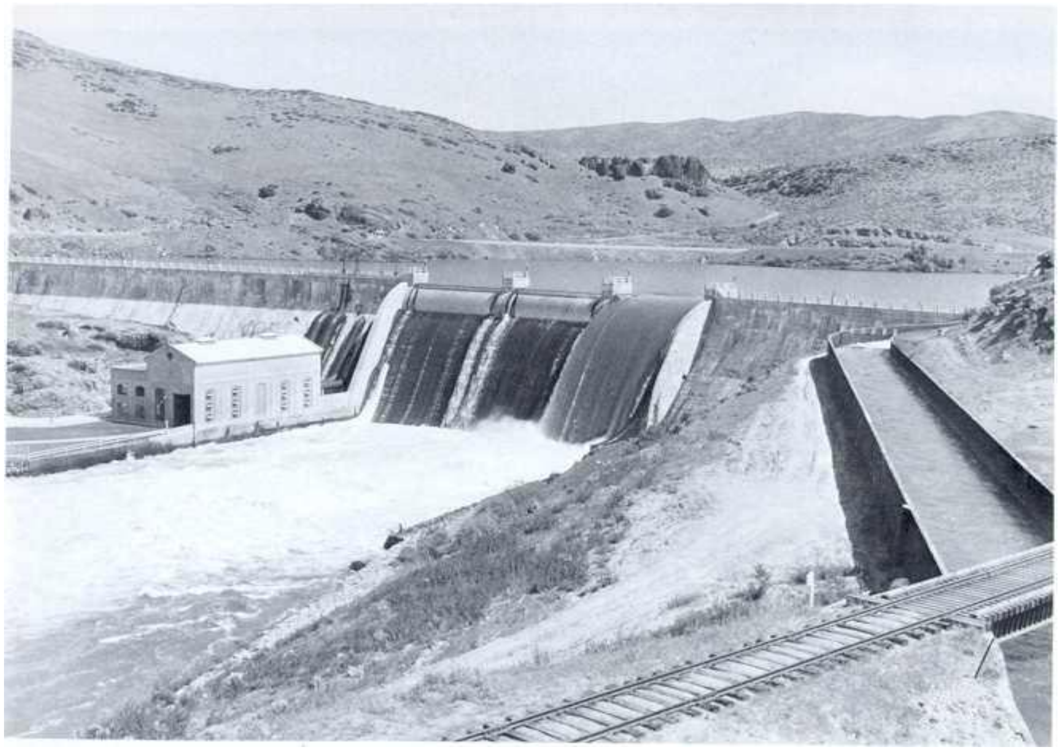
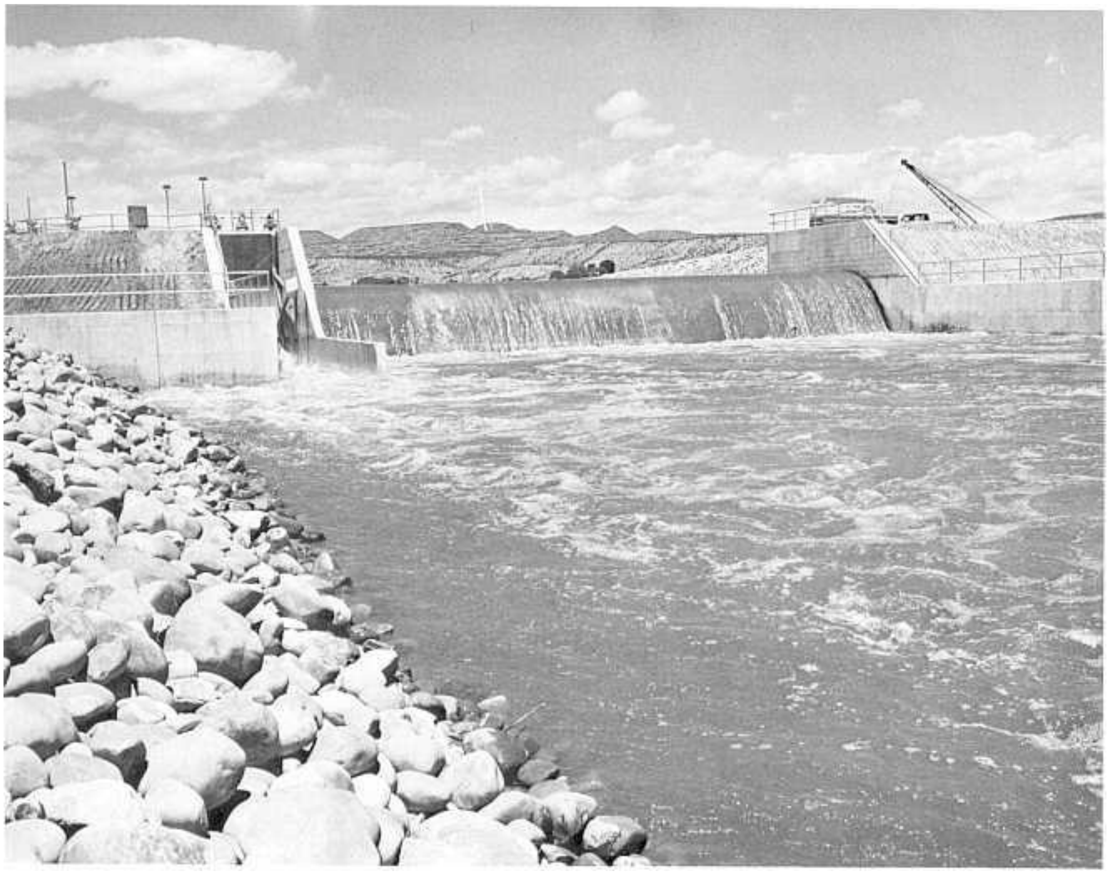


Fig. No.(2.3):Black Canyon Dam, a Concrete-Gravity StorageAnd Diversion Structure

(Design of small damsThird Edition 1987)

Diversion dams are ordinarily constructed to provide head for carrying water into ditches, canals, or other conveyance systems. They are used for irrigation developments, for diversion from a live stream to an off-channel-location storage reservoir, for municipal and industrial uses, or for any combination of the above. Figure (2.4) shows a typical small diversion dam.



<u>Fig. No. (2.4): Knight Diversion Dam- a Small Diversion Structure (Design of small dams, Third Edition 1987)</u>

Detention dams are constructed to retard flood runoff and minimize the effect of sudden floods.

Detention dams consist of two main types. In one type, the water is temporarily stored and released through an outlet structure at a rate that does not exceed the carrying capacity of the channel downstream. In the other type, the water is held as long as possible and allowed to seep into pervious banks or into the foundation. The latter type is sometimes called a water-spreading dam or dike because its main purpose is to recharge the underground water supply. Some detention dams are constructed to trap sediments; these are often called debris dams. Although it is less common on small projects than on large developments, dams are often constructed to serve more than one purpose.

Where multiple purposes are involved, a reservoir allocation is usually made to each distinct use. A common multipurpose project combines storage, flood control, and recreational uses.

2.2.2. Classification by Hydraulic Design:-

Dams may also be classified as overflow or no overflow dams. Overflow dams are designed to carry discharge over their crests or through spillways along the crest. Concrete is the most common material used for this type of dams. Non overflow dams are those designed not to be overtopped. This type of design extends the choice of materials to include earth fill and rock fill dams. Often the two types are combined to form a composite structure consisting, for example, of an overflow concrete gravity dam with earth fills dikes. Figure (2.5) shows such a composite structure built by the Bureau of Reclamation, while figure (2.6) shows an earth fill dam.

2.2.3. Classification by Materials:-

The most common classification used for the discussion of design procedures is based upon the materials used to build the structure. This classification also usually recognizes the basic type of design, for example, the “concrete gravity” dam or the “concrete arch” dam. This include the more common types of dams constructed today; namely, earth fill, rock fill, and concrete gravity dams. Other types of dams include concrete arch, concrete buttress, and timber dams.

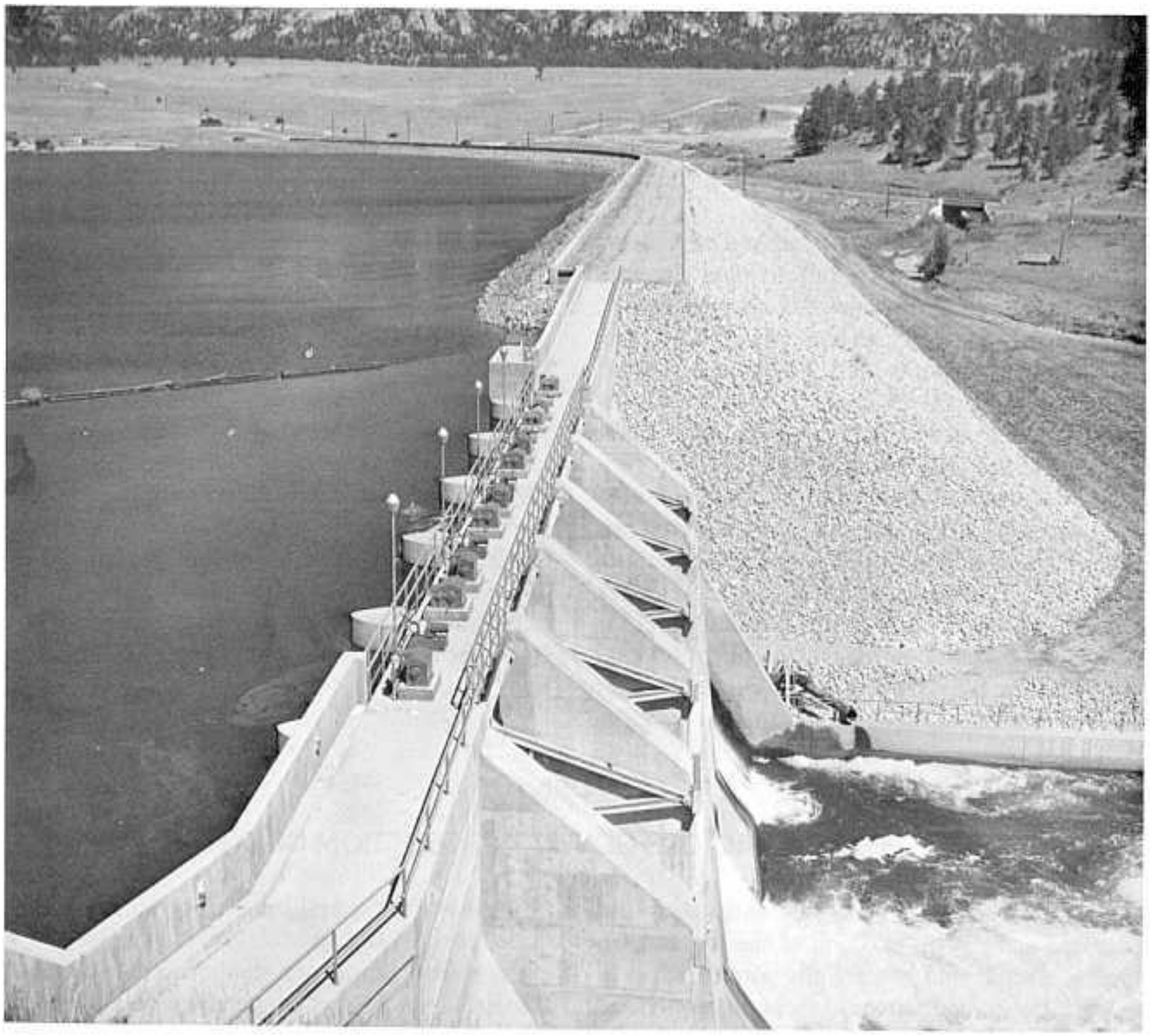


Fig. No.(2.5):Oiympus dam, a Combination Earth Fill And Concrete-Gravity Structure (Design of small dams, Third Edition 1987)

1- Earth fill Dams:-

Earth fill dams are the most common type of dams, principally because their construction involves the use of materials from required excavations and the use of locally available natural materials requiring minimum processing.

Using large quantities of required excavation and locally available borrow are positive economic factors related to an earth fill dam. Moreover, the foundation and topographical requirements for earth fill dams are less stringent than those for other types.

It is likely that earth fill dams will continue to be more prevalent than other types for storage purposes, partly because the number of sites favorable for concrete structures is decreasing as a result of extensive water storage development. This is particularly true in arid and semiarid regions where the conservation of water for irrigation is a fundamental necessity. Although the earth fill classification includes several types, the development of modern excavating, hauling, and compacting equipment for earth materials has made the rolled-fill type so economical as to virtually replace the semi hydraulic- and hydraulic-fill types of earth fill dams. This is especially true for the construction of small structures, where the relatively small amount of material to be handled precludes the establishment of the large plant required for efficient hydra'ollic operations.

Earth fill dams require appurtenant structures to serve as spillways and outlet works. The principal disadvantage of an earth fill dam is that it will be damaged or may even be destroyed under the erosive action of overflowing water if sufficient spillway capacity is not provided. Unless the site is off stream, provision must be made for diverting the stream past the dam site through a conduit or around the dam site through a tunnel during construction. A diversion tunnel or conduit is usually provided for a concrete dam; however, additional provisions can be made for overtopping of concrete blocks during construction. A gap in an embankment dam is sometimes used for routing the river through the dam site during construction of portions of the dam on either or both sides of the gap.

Fig.No.(2.6): EARTH DAMS

2- Rock fill Dams:-

Rock fill dams use rock of all sizes to provide stability and an impervious membrane provide water tightness. The membrane may be an upstream facing of impervious soil, a concrete slab, asphaltic-concrete paving, steel plates, other impervious elements, or an interior thin core of impervious soil. Like the earth embankments, rock fill dams are subject to damage or destruction by the overflow of water and so must have a spillway of adequate capacity to prevent overtopping. An exception is the extremely low diversion dam where the rock fill facing is designed specifically to withstand overflows. Rock fill dams require foundations that will not be subject to settlements large enough to rupture the watertight membrane. The only suitable foundations, therefore, are rock or compact sand and gravel.

The rock fill type dam is suitable for remote locations where the supply of good rock is ample, where the scarcity of suitable soil or long periods of high rainfall make construction of an earth fill dam impractical, or where the construction of concrete dam would be too costly. Rock fill dams are popular in tropical climates because their construction is suitable for long periods of high rainfall.

3- Concrete Gravity Dams:-

Concrete gravity dams are suitable for sites where there is a reasonably sound rock foundation, although low structures may be founded on alluvial foundations if adequate cutoffs are provided. They are well suited for use as overflow spillway crests and, because of this advantage, are often used as spillways, for earth fill or rock fills dams or as overflow sections of diversion dams. Gravity dams may be either straight or curved in plan. The curved dam may offer some advantage in both cost and safety. Occasionally the dam curvature allows part of the dam to be located on a stronger foundation, which requires less excavation. The concept of constructing concrete dams using RCC (roller-compacted concrete) has been developed and implemented. Several RCC dams have been constructed in the United States and in other countries. The technology and design procedures, however, are relatively new and are still being developed.

Concrete gravity dams depends on its own weight for stability. Figure (2.7), shows a concrete gravity dam.

Fig No (2.7) Concrete Gravity Dam North of Irkutsk, Russian Federation (www.hydroelecritc.energy.blogspot.com).

4- Concrete Arch Dams:-

Concrete arch dams are suitable for sites where the ratio of the width between abutments to the height is not great and where the foundation at the abutments is solid rock capable of resisting arch thrust. Two types of arch dams can be defined namely single and multiple arch dams. A single arch dam spans a canyon as one structure and is usually limited to a maximum crest length to height ratio of 10:1. Its design may include small thrust blocks on either abutment, as necessary, or a spillway some where along the crest. A multiple arch dam may be one of two distinct designs. It may have either a uniformly thick cylindrical barrel shape spanning 50 feet or less between buttresses, such as Bartlett Dam in Arizona, or it may consist of several single arch dams supported on massive buttresses spaced several hundred feet on centers. The dams purpose whether it be a permanent major structure with a life expectancy of 50 years or a temporary cofferdam with a useful life of 5 years, will directly influence the time for design and construction. The quality of materials in the dam and foundation, the foundation treatment, and the hydraulic considerations re also to be considered.

Structural and economic aspects prohibit the design of an arch dam founded on stiff soil, gravel, or cobblestones. Uplift usually does not affect arch dam stability because of the relative thinness through the section, both in the dam and at the concrete rock contact. Figure (2.8), shows an arch dam in Zernez, Switzerland.

Fig.No.(2.8): Arch Dam in Zernez, Switzerland, View From Site (www.commandatastorage.googleapsis.com).

5- Concrete Buttress Dams:-

Buttress dams are comprised of flat deck and multiple arch structures. They require about 60 percent less concrete than solid gravity dams, but the increased formwork and reinforcement steel required usually offset the savings in concrete. A number of buttress dams were built in the 1930's, when the ratio of labor costs to material costs was comparatively low. The cost of this type of construction is usually not competitive with that of other types of dams when labor costs are high. Figure (2.9), shows one of the buttresses in the Manic-Ceng Buttress Dam in Québec, Canada.

The design of buttress dams is based on the knowledge and judgment that comes only from specialized experience in that field.

Fig.No.(2.9):One of the Buttresses in the Manic-Ceng Buttress Dam in Québec, Canada (www.dappolonia.com).

2.3. Forces on Gravity Dam:-

1. Gravity (weight of dam):-

$$W = V \times \gamma --- (2.1)$$

Where:-

$V = Volume \quad \mathrm{ft}^3$

$\gamma =$ Specific weight of material $\frac{\mathrm{Lb}}{\mathrm{ft}^3}$

2. Hydrostatic pressure:-

$$H_h = \frac{\gamma h_2}{2} --- (2.2) \rightarrow (Horizental \text{ component})$$

Where:-

h = Depth of water at the section (ft)

$\gamma =$ Specific weight of material $\frac{\mathrm{Lb}}{\mathrm{ft}^3}$

$$H_V = \frac{\gamma V}{h} --- (2.3) \rightarrow (Vertical \text{ component})$$

Where:-

V = Volume of the dam at that point ft^3

3. Uplift Pressure:-

Dams are subjected to uplift force (U) under their bases. The uplift acts upward.

$$U = \frac{1}{2}\gamma_\omega hB --- (2.4)$$

Where:-

B = The width of the base of the dam.

$\gamma_\omega =$ Specific weight of water.

4. Wave Pressure:-

The upper part of the dam (above the water level) is subjected to the impact of waves. The maximum wave pressure P_V per unit width is:-

$$P_V = 2.4\gamma_{\omega} h_w ---(2.5)$$

Where:-

h_w = The wave height

5. Body Force:-

Body force P_{em} acts horizontally at the center of gravity and is calculated as:

$$P_{em} = \alpha \text{ W--------} \quad (2.6)$$

Where:-

α = Earthquake coefficient, taken as 0.2 for practical reasons.

W = Weight of the dam.

6. Ice pressure:-

Pressure created by thermal expansion exerts thrust against upstream face of the dam. This is not applicable in arid regions.

2.4. Hydropower Characteristics:-

The water of the oceans and water bodies on land are evaporated by the energy of the sun heat and gets transported as clouds to different parts of the earth. The clouds traveling over land and falling as rain on earth produces flows in the rivers which returns back to the sea. The water of rivers and streams flows down from places of higher elevations to those with lower elevations, and loose their potential energy and gain kinetic energy.

The energy is quite enrichment in many rivers which have caused them to etch their own path on the earth's surface through millions of years of continuous erosion. In almost every river, the energy still continues to deepen the channels and migrate by cutting the banks, though the extent of morphological changes varies from river to river. Much of the energy of a river flowing water gets dissipated due to friction encountered with its banks or through Loss of energy through internal turbulence. Nevertheless, the energy of water always gets replenished by the solar energy which is responsible for the eternal circulation of the Hydrologic Cycle (Peter F., March, 1999, pp.55-47).

Hydropower engineering tries to tap this vast amount of energy available in the flowing water on the earth surface and convert that to electricity. There is another form of water energy that is used for hydropower development: the variation of the ocean water with time due to the moon pull, which is termed as the tide. Hence, hydropower engineering deals with mostly two forms of energy and suggest methods for converting the energy of water into electric energy. In nature, a flowing stream of water dissipates throughout the length of the watercourse and is of little use for power generation. To make the flowing water do work usefully for some purpose like power generation, it has been used to drive water wheels to grind grains at many hilly regions for years. It is necessary to create a head at a point of the stream and to convey the water through the head to the turbines which will transform the energy of the water into mechanical energy to be further converted to electrical energy by generators. The necessary head can be created in different ways of which two have been practically accepted (Mosonyi, Emil (1991).

2.4.1. Components and Structure of a Hydropower System:-

In hydropower production a water system, typically a river or part of it is harnessed to electricity generation. The main components of a hydropower system are power plants, water reservoirs and the water channels between, which form a network-type system. The system often comprises a main river and its tributaries. The reservoirs in the hydropower system can be divided into two categories; seasonal reservoirs and plant reservoirs. A seasonal reservoir is a reservoir which can store a significant part of the annual water inflow. Seasonal reservoirs are typically relatively large; they can be either natural or artificial lakes. A plant reservoir is a smaller reservoir with minor storing capability situated directly upstream of a power plant (Vilkko, M. 1999).

A power plant situated directly below a seasonal reservoir is regarded as fully controllable. The main constraints bounding its usage are the plant maximal amount of discharge and the availability of water. Being part of a network structure, the usage of the plant can be actively restricted by the current and future flexibility of the reservoirs situated upstream and downstream of the plant. If situated in the river with no seasonal reservoirs directly above, the plant is called a run-of-river plant. The water inflow to this kind of plants has to be discharged or passed by (spilled) immediately or at least rather quickly, depending on the storage capability of the related plant reservoir. Spilling the water means that its potential energy is lost and, furthermore, the inflow can not be taken advantage of. Spillage occurs typically when it can not be avoided, i.e. when the water level of the related reservoir is at the maximum limit. During a period with high spot prices it might be an optimal decision to spill water by some plants in order to get additional water to downstream plants. The spillage in the run-of-river plants is especially relevant during the spring flood period (Antila, H. (1997).

A power plant is composed of one or more generators or units. All the generators of a plant do not have to be running simultaneously. Different unit scheduling schemes result in different economical results especially in terms of required maintenance costs. Therefore the unit scheduling is an important part of hydropower production planning. Another important element in the hydropower system is the water channel. The water channel can represent the main water route between reservoirs and power plants or the spillage route of a power plant. Each main water route consists of one or more parallel water channels. The water channels can be used in order to take into account possible time delays in water flows in the modeling of the system. In real life, the time delays depend on the volume of water running along the water route. However, volume dependent delays would cause difficulties in the modeling of the system. Thus, time delays in the system are usually regarded as volume independent (P. M. Anderson, 1977).

The characteristics of a hydropower generator define the maximum discharge through it and furthermore, its maximum power generating capacity. Generators under maintenance or revision have temporarily lower maximum discharge restrictions. In addition, minimum discharge restrictions are introduced at times. A specific minimum limit of production for the whole hydropower system may be defined at any given time period, as well as a maximum feasible spillage for a plant, and a maximum allowed change in reservoir water levels in consecutive time periods, to mention some additional constraints (D. B. Arnautovic and D. M. Skataric, September 1991).

2.4.2. Conversion From Water to Energy:-

Hydropower generation is based on the potential energy of water. This potential energy is converted into mechanical energy by a hydro turbine. The mechanical energy is converted further to electrical energy by a generator. In the final stage, the electrical energy is transmitted and distributed to customers via a transmission and distribution network (Hydropower Energy, July 2002).

The mechanical power of a hydro turbine is usually represented as a function of discharge, efficiency factor, and turbine head. The discharge is the water flow through the turbine. As the discharge increases, the amount of generated power increases (Curtis C. Ebbesmeyer, 1998).

2.4.2.1 Spillway:-

A spillway is a vital appurtenance of a dam. Frequently, its size and type and the natural restrictions in its location are the controlling factors in the choice of the type of dam. Spillway requirements are dictated primarily by the runoff and stream flow characteristics, independent of site conditions or type or size of the dam. The selection of specific spillway types should be influenced by the magnitudes of the floods to be passed. Thus, it can be seen that on streams with large flood potential, the spillway is the dominant structure, and the selection of the type of dam could become a secondary consideration. The cost of constructing a large spillway is frequently a considerable portion of the total cost of the project. In such cases, combining the spillway and dam into one structure may be desirable, indicating the selection of a concrete overflow dam. In certain instances, where excavated material from separate spillway channels can be used in the dam embankment, an earth fill dam may prove to be advantageous Figure (2.10) shows a small spillway in operation.

Small spillway requirements often favor the selection of earth fill or rock fill dams, even in narrow dam sites. The practice of building overflow concrete spillways on earth or rock embankments has generally been discouraged because of the more conservative design assumptions and added care needed to forestall failures. Inherent problems associated with such designs are unequal settlements of the structure caused by differential consolidations of the embankment and foundation after the reservoir loads are applied; the need for special provisions to prevent the cracking of the concrete or opening of joints that could permit leakage from the channel into the fill, with consequent piping or washing away of the surrounding material; and the requirement for having a fully completed embankment before spillway construction can be started. Consideration of the above factors coupled with increased costs brought about by more conservative construction details, such as arbitrarily increased lining thickness, increased reinforcement steel, cutoffs, joint treatment, drainage, and preloading, have generally led to selection of alternative solutions for the spillway design. Such solutions include placing the structure over or through the natural material of the abutment or under the dam as a conduit.

One of the most common and desirable spillway arrangements is the use of a channel excavated through one or both of the abutments outside the limits of the dam or at some point removed from the dam. Where such a location is adopted, the dam can be of the non overflow type, which extends the choice to include earth fill and rock fill structures. Conversely, failure to locate a spillway site away from the dam requires the selection of a type of dam that can include an overflow spillway. The overflow spillway can then be placed so as to occupy only a portion of the main river channel, in which case the remainder of the dam could be either of earth, rock, or concrete. Olympus Dam of figure (2.5) is an example of this type of dam.

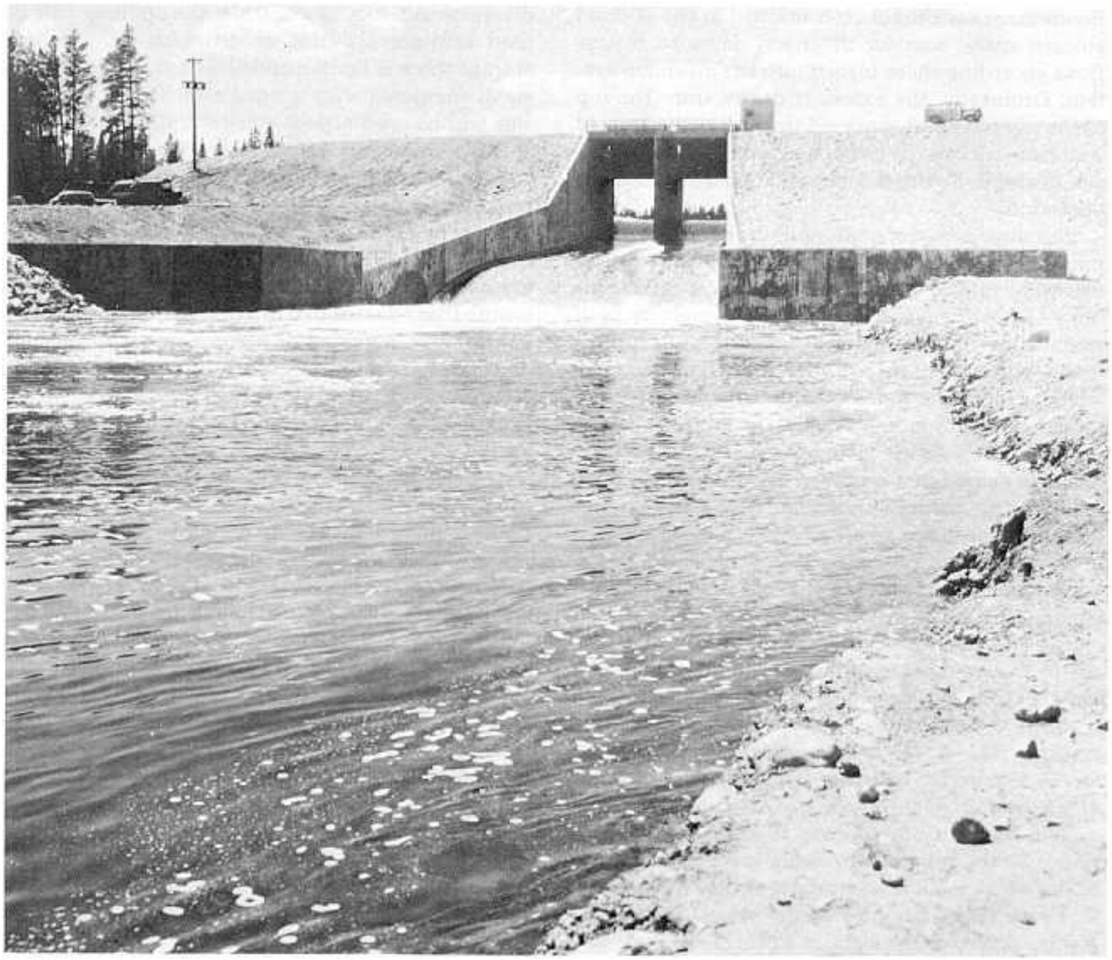


Fig.(2.10): Small Chute Spillway In Operation

(Design of small dams, Third Edition 1987)

2.4.2.2 Hydraulic Turbines:-

Hydraulic turbines may be considered as hydraulic motors or prime movers of a water-power development, which convert water energy (hydropower) into mechanical energy (shaft power). The shaft power developed is used in running electricity generators directly coupled to the shaft of the turbine, thus producing electrical power. Turbines may be classified as impulse- and reaction-type machines (Fontane, D. G., J. W. Labadie, et al. (1981). In the former category, all of the available potential energy (head) of the water is converted into kinetic energy with the help of a contracting nozzle (flow rate controlled by spear-type valve) provided at the delivery end of the pipeline (penstock). After impinging on the curved buckets the water discharges freely (at atmospheric pressure) into the downstream channel (called the tail race). The most commonly used impulse turbine is the Pelton wheel). Large units may have two or more jets impinging at different locations around the wheel.

In reaction turbines only a part of the available energy of the water is converted into kinetic energy at the entrance to the runner, and a substantial part remains in the form of pressure energy. The runner casing (called the scroll case) has to be completely airtight and filled with water throughout the operation of the turbine. The water enters the scroll case and moves into the runner through a series of guide vanes, called wicket gates. The flow rate and its direction can be controlled by these adjustable gates. After leaving the runner, the water enters a draft tube which delivers the flow to the tail race.

2.4.2.3 Power House:-

The power house structure can be divided in two sections, a substructure supporting the hydraulic and electrical equipment and a superstructure housing the equipment. The substructure is usually a concrete block with all the necessary waterways formed within it. The scroll case and draft tube are usually cast integrally (especially in large low-head plants) with the substructure with steel linings. The superstructure usually houses the generating units and exciters, the switch board and operating room. Vertical-axis units (whose turbines are placed just below the floor level) generally require less floor space than those mounted on horizontal axes. The cost of the superstructure can be reduced considerably by housing individual generators only (outdoor power house), although it has the disadvantage that maintenance works have to be restricted to good weather conditions only. Under certain topographic conditions, particularly when the power plant is situated in narrow canyons with no convenient site for a conventional type of power house, this may be located underground. Many examples of this type exist in Europe and elsewhere (e.g. the Cruachan and Dinorwic plants in the UK).

It is essential to equip a power house with a crane to lift and move equipment for installation and maintenance purposes. Travelling cranes spanning the width of the building and capable of traversing its entire length is normally used. The crane rail elevation depends on the maximum clearance required when the crane is in operation which, in turn, determines the overall height of the superstructure (Fontane, D. G., J. W. Labadie, et al. (1981).

2.4.2.4Turbine Governors:-

The governor is a mechanism controlling the rotational speed of the turbo generator unit; constant speed must be maintained in order to obtain the a.c. supply with constant frequency. As the turbine and hence its interconnected generator tend to decrease or increase speed as the load varies, the maintenance of an almost constant speed requires regulation of the amount of water allowed to flow through the turbine by closing or opening the gates (or nozzles) of the turbine automatically, through the action of a governor. A rapid closing or opening of the nozzle or guide vanes (gates) is undesirable, as serious water hammer problems may result in the penstocks.

Sudden changes may be avoided in the case of a Pelton turbine if a deflector is activated in front of the jet, thus diverting part of the flow away from the turbine. Similarly, in the case of a reaction-type turbine a relief valve may allow a part of the discharge to flow directly to the tail race without entering the runner.

2.5. Technical Aspects of Reservoirs:-

Small reservoirs (size approximately 0.01 km3) (SRP Website, 2006) are a result of small dams constructed on streams or rivers to impound water and are mostly constructed with earth materials such as clay, gravels, rocks or sometimes boulders. Most reservoirs normally use clay which has been well compacted as the water proofing element which prevent loss of water through seepage.

The banks of the reservoirs can be of gravel or boulders depending on the condition of the existing soil type and the purpose of the reservoir. The use of good earth material is a prerequisite for long lasting reservoirs and also for efficient utilization. The efficiency of the reservoir is dependent on the materials used in the construction.

Proper surveying and field studies should be conducted before the construction begins. Field studies involves the selection of site and the determination of slope of the reservoir which is another important steps since this is required to ensure constant flow into and out of the reservoir. The spillway is the part that is used to empty the reservoirs when too much water has been stored in the reservoir. It normally consists of an outlet pipe made of either caste concrete or PVC pipes.

The lining of the reservoir could be of well compacted clay or a water proofing material such as polyethylene. The riverbed could be gravels bed with a good slope to ensure adequate flow into the reservoir.

2.5.1Reservoirs Operation of Storage Dams:-

Storage dams should be operated to provide as many benefits as feasible. They should be operated and releases made to provide optimum benefits considering contractual requirements and primary benefits. Dams may be used to store water to achieve benefits related to irrigation, power, municipal, industrial, recreation, fish, wildlife, flood control, and water quality. Obviously, not all dams will provide all of these benefits, but an evaluation should be made to determine the potential benefits from a dam and how management of the dam and reservoir can best achieve optimum benefits. The operational requirements should be based on these studies and on experience, and these requirements should be documented in the written instructions for dam operation. Multiple uses of reservoirs often results in conflicts among the potential beneficiaries. For example, optimum power production may result in reduced irrigation water supply and fewer recreation benefits. These conflicts should be evaluated keeping in mind the primary purposes of the facility. Votruba, L. and Breza, V. (1989) Quite often, multiple benefits can be achieved without significant loss to primary beneficiaries. Obtaining accurate and timely hydrological data is critical to the proper and safe operation of a dam and reservoir. A reliable means of determining the potential water supply is essential to safe operation, i.e., reservoir evacuation to pass flood flows, and maximum storage for given conditions. To obtain accurate and useful data for deterring the potential water supply, the entire hydrologic cycle should be considered on a basin-wide scale. Pertinent information required for efficient operation include quantity of precipitation, distribution over time, and relative uncertainty. The technology is now available to telemeter hydro meteorological phenomena and to relay that information through ground or satellite links to computers to improve forecasting capabilities.

Computer models are available to determine river flow forecasts from that data. Before installing a data-acquisition system, a study should be made to determine what other equipment is already installed in the area. A compatible telemetry scheme should be used where possible to enhance existing capabilities and to avoid redundancy.

Considerations in developing a hydro meteorological telemetry and forecasting scheme include:

- Defining issues and requirements for improved water management service and operations, and identifying how these improvements may be addressed through enhancements in measurement systems, data handling, and research

- Selecting the appropriate research in hydrologic and meteorological areas to understand the key elements of the hydrologic cycle, and defining corresponding goals and approaches.
- Providing program guidance and concepts as input to complement the existing plans and programs
- Using the information gathered to govern operational processes.

Special precautions should be taken in the operation of a dam and reservoir during periods of potential high inflows. Management during these periods should be governed by forecasted inflows, potential runoff, reservoir elevation, and downstream condition. The dam should normally be attended continuously during these periods; however, discretion based on conditions should be practiced. The written procedures for dam operations should not be arbitrarily changed without consideration of the effect of such a change. However, these procedures should not be inflexible when conditions suggest alternative operations.

The stimulation and protection of vegetation to retard erosion on the slopes of the reservoir and on the slopes of earth fill dams not otherwise protected is an important aspect that should be given frequent attention. This vegetative cover is essential to protect against erosion and sloughing of banks, which can result in costly maintenance and safety problems.

Expert advice on suppression of algal growth in reservoirs should be obtained and followed, and no chemicals should be introduced into a reservoir without competent advice.

Periodic inspections of the reservoir area should be made to detect slide areas and to monitor their progress. Corrective action should be taken in these areas at an early stage to minimize problems. Posting warring signs for slide areas should be considered if they pose a safety problem to boaters or recreationists or could lead to liability for the operator or owner.

Safety buoys should be constructed upstream of overflow spillways if there is a potential danger to boaters or others. Log booms are sometimes necessary to preclude blockage of spillways during high water periods in reservoirs that have a high volume of debris. Debris should be cleared from the reservoir areas periodically (annually if large amounts) and burned in a safe area. Burning of debris on rip rapped surfaces should be avoided because it leads to rapid deterioration of the riprap. Instructions for operating mechanical equipment should be followed closely to prevent damage to the installations through improper operation. Instructions for the control of spill way gates during flood flows into the reservoir should be followed in detail as outlined in the written operating procedures. Deviations from these instructions should not be made without approval from higher management.

Dams that are operated remotely or that depend on remote readings for proper and safe operation require periodic inspection of their facilities to ensure proper operation. For example, equipment that read reservoir elevation remotely should be checked by means of a staff gauge periodically or when a problem is suspected.

The degree of attendance needed at a dam should be determined by evaluating such aspects as size, complexity, prior history, and downstream hazard.

4 2.6 Sediment:-

The sediments are the result of rocks decomposition. As individual particles, the sediments have their own characteristics. The knowledge of such characteristics is needed to model their behavior when in water and when deposited. Therefore, these characteristics are important for reservoir sedimentation as the rate of entrainment, transport, deposition and compaction are functions of the properties of the sediment particles.

The properties of the sediments can be classified basically into two types: individual and the bulk properties.

Individual Properties:-

1. Size

Size is the most basic and readily measurable property of sediment. Size has been found to sufficiently describe the physical property of a sediment particle for many practical purposes. The size of particle can be determined by sieve size analysis or size distribution curve. Figure (2.11), is a typical size distribution curve. Table (2.1) shows the particle size distribution.

TABLE 2. Laser particle-size distribution measurements (means and standard deviations) for Tahoe Basin disturbed soils

Soil type	n	D_{10}	D_{30}	D_{60}	D_{90}	Sand	Silt	Clay
		µm				%		
Granitic mean	16	70.4*	294.8a	785.6a	1,589a	90.7a	7.82a	1.52a
Std. dev.	16	30.2	91.9	146.4	83.5	3.19	2.90	0.55
Volcanic mean	48	3.98b	41.3b	390.1b	1,227a	64.9b	28.2b	6.92b
Std. dev.	48	2.06	26.0	175.7	342.9	7.43	4.82	2.97
Tahoma mean	4	8.67b	66.0b	297.8b	1,194a	74.0c	21.8c	4.20ab
Std. dev.	4	3.06	6.39	54.2	245.6	2.11	1.45	0.85

* Mean values followed by different letters differ significantly ($P < 0.05$).

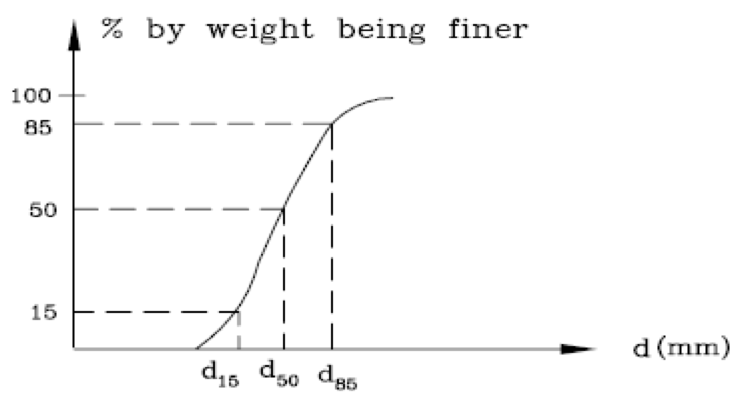


Fig. No. (2.11):A Typical Size Distribution Curve

2. Shape

Shape refers to the form or configuration of a particle regardless of its size or composition. Corey shape (Put equation) factor is commonly used to describe the shape.

$$S_{Cored} = \frac{d_c}{\sqrt{d_a \times d_b}} ---(2.7)$$

Where:-

d_c = short diameter .

d_a = long diameter .

d_b = intermediate diameter.

3. Fall Velocity

The fall velocity of sediment particle in quiescent column of water is directly related to relative flow conditions between sediment particle and water during conditions of sediment entrainment, transportation and deposition. It reflects the integrated result of size, shape, surface roughness, specific gravity, and viscosity of fluid. Fall velocity of particle can be calculated from a balance of buoyant weight and the resisting force resulting from fluid drag. Figure (2.12) shows the drag and submerged forces.

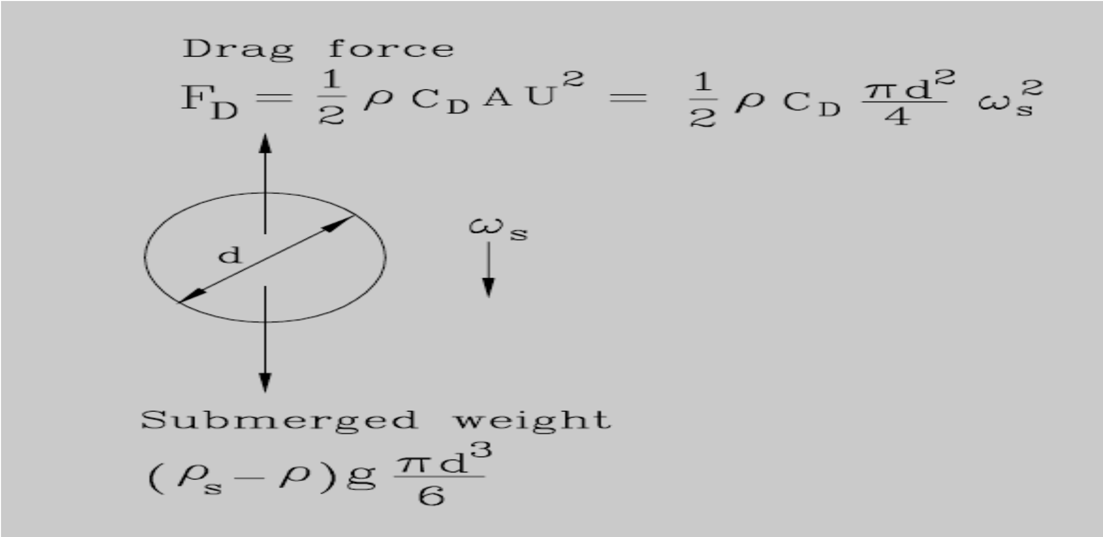


Fig. No.(2.12) The Drag and Submerged Forces

The balance between the drag force and the submerged weight give:-

$$\frac{1}{2}\rho C_D \frac{\pi d^2}{4} W_s^2 = (\rho_s - \rho)\, g \frac{\pi d^3}{6} --- (2.8)$$

$$W_s = \sqrt{\frac{4(s-1)\, gd}{3C_D}} --- (2.9)$$

5- Bulk Properties:-

1. Specific Weight

The volumetric unit or specific weight of the sediments determines the deposits of sediments. It is a very simple obtainable property computed simply by weighing an amount of sediment filling a vessel with known volume. The division of the sediment weight by the volume of the vessel gives γ_s, the sediment specific weight.

2. Grain Size Distribution

Owing to the non-uniformity of the sediment particle, it is necessary to know the particle size distribution. This process is made by subjecting a sample of sediments to a grain size analysis. The most common methods for particle fractioning are the direct measurement, method appropriate for large particles, the shifting method more appropriate for sand, medium and small gravel particles and the sedimentation method for fine materials.

The direct measurement is the individual measurement of the particle. The particle diameter can be assumed to be the average of its three orthogonal axes. Particles larger than 16 mm are suitable for this kind of analysis.

The sieve analysis is made taking a sediment sample and passing it through a sieve series. The sieves must be fitted one over the other with the mesh size decreasing in the downward direction. There are several sieve series and a sieve is generally known by its mesh opening. After the sample is sieved, the retained sample fraction in each sieve is taken and weighted. The weighted fractions can be represented by a cumulative frequency curve and a frequency curve (Gaussian-type), previously shown in figure (2.11). Both curves are made by putting the sieve openings on the ordinate and the accumulated per cent finer or coarser on the abscissa.

The sedimentation method for fine and very fine particles is made by separating the fine fraction of a sample with a sieve. Some standards consider that fine fractions are those that pass through the sieve 0.074 mm and others in the sieve 0.062 mm. basically; the method is made putting the fine fraction of the sample in a cylinder with distilled water. After that, the cylinder is agitated and at determined times the density and a particle fall velocity equation, generally Stokes law, the particle diameter can be determined.

5 2.6.1.Sediment Load and Transport in Rivers:-

The material of the sediment load of the rivers originates basically from two sources. The first is the river bed material that is displaced by the stream shear stress and the second, main source of sediments, is the material resulting from the basin erosion. The eroded soils are brought to the rivers through the basin run-off.

The knowledge of the process of sediment transport is fundamental for modeling reservoir sedimentation since transport equations are at the heart of sedimentation models. The mechanisms of transport lead to the sediment transport governing equations of the phenomenon which are used in the model.

2.6.2The Sediment Process:-

Sediment is the material originated from decomposition of the rocks. They become fluvial sediments when are transported by flowing water.

The first sediments to be transported are those that compose the river bed. Despite this, the bed erosion generally is not a very important source compared with the basin erosion. Often, bed erosion in most rivers contributes with less than 20% of the annual sediment discharge. The main source of fluvial sediments is the river catchment erosion. It is in the catchment that the sediment transport process starts yielding sedimentary material to the streams.

The catchment sediments journey towards the rivers begins when the soil particles are detached from their place by any means. That is, something that provokes soil erosion which can be a natural or a human action. Depending on the presence of soil protection, such as vegetation canopy, the impact of the rain drops may cause high soil erosion. Many human activities, such as construction of roads or other earth works can also increase hugely the soil erosion.

Once freed, the soil particles are transported to the rivers by the catchment run-off. The runoff is originated mainly from rainfall or snow melting. It is able to displace soil particles still in their repose state and transport them. A part of the runoff transported particles can be deposited in land depressions before reaching the streams.

When the sediments arrive in a river, they are carried to a depositional site which can be the river bed, a reservoir, a lake or the sea. If the river has not sufficient capacity to transport totally a determined sediment load, part of it will settle immediately. Whilst a sediment particle is transported in water, it is basically under the influence of two forces. The first is the particle submerged weight and the second is the uplifting force due to the flow turbulence. The balance between the vertical forces which act on the particles, i. e. the vertical component of the force due to turbulence and the particle submerged weight, makes possible their transport or deposition.

2.6.3The Fluvial Transport of Sediments:-

Sediments can be transported in two different modes, the bed load transport, which is the transport mode of the coarser sediments, moving along the bed, and the finer transport mode, which is the suspended load transport. The bed load takes place near the stream bottom with the particles sliding or leaping on the bed. The suspended load occurs when the turbulence of the flow puts the finer matevrials into suspension and the particles travel at a velocity similar to that of the flow. Then, the suspended load is considered an advanced stage of the bed load transport.

2.6.4 Bed-Load Transport:-

A bed particle is displaced when the flow shear stress overcomes the particle resistance to motion, which is determined by its weight and the friction between the particle and its neighbors on the bed. This incipient transport, called bed-load transport, occurs near by the bottom of the river and the particles are transported sliding on the bed or by small jumps (saltation). Often, this is the mode of transport of the coarser sediment particles.

Bed-load is the sediments which are in successive contacts with the stream-bed and the transport is driven by gravity. Einstein (1950), considered that the bed transport come to its end at a distance (a).

$$a = 2d \quad ---(2.10)$$

Where:-

$d =$ The sediment characteristic diameter.

Bed-Load in the Stream Flow Direction:-

Many researchers have proposed methods to predict bed-load transport in rivers or channels. Among them are Einstein (1950) and more recently van Rijn

(1984a). These authors have produced some of the best known functions for bed-load transport.

According to Yang (1996) the Meyer-Peter-Muller equation, is the most commonly used empirical bed-load function in Europe. Meyer-Peter-Muller equation has the form:-

$$\left(\frac{n_s}{n}\right)^{3/2}\frac{\gamma_f R_b S}{(\gamma_s-\gamma_f)} = 0.047 + 0.25\left(\frac{\gamma_f}{g}\right)^{1/3}\left(\frac{q_B}{\gamma_s}\right)^{2/3}\frac{1}{\left(\gamma_s-\gamma_f\right)d_a} ---(2.11)$$

$$n_s = \frac{d_{90}^{\frac{1}{6}}}{26.0} ---(2.12)$$

Where:-

$n_s =$ Is rugosity due to the sediment grain as below.

$d_{90} =$ the size for which 90% of the sediment is finer in m.

Van Rijn (1984a) proposed a bed-load function based on the numerical solution of the equations of motion for saltation particle. Using new proposed relationships for particle characteristics and measured bed-load data, the concentration of bed load particles was computed and represented by a simple function. The van Rijn model equation is for particles in the range 200-2000. van Rijn equation has the form.

$$\frac{q_B}{[g\Delta]^{\frac{1}{2}}\, d_{50}^{\frac{3}{2}}} = 0.053\frac{\tau^{2.1}}{D_*^{0.3}} ---(2.13)$$

Where:-

q_B =The rate of bed-load transport per unit width in the main flow direction.

g = The acceleration of the gravity

d_{50} The sieve diameter for which 50% of the sediment is finer.

Δ = Computed as follows:-

$$\Delta = \frac{\rho_s - \rho}{\rho} --- (2.14)$$

Where:-

ρ_s = The sediment density.

ρ = The fluid density.

T = The transport stage parameter computed by:-

$$T = \frac{(u'_*)^2 - (u_{*cr})^2}{(u_{*cr})} --- (2.15)$$

Where:-

u_{*cr} = The shear stress according to the Shields diagram or the relations that represent the diagram.

u'_* = Supplied by van Rijn and is the effective bed-shear velocity computed by

$$u'_* = \frac{g^{0.5}\overline{u}}{C'} --- (2.16)$$

Where:-

$\overline{u}$ =The average flow velocity.

C' = Chèzy coefficient related to the surface roughness of the sediment bed defined as:-

$$C' = 18\log\left(\frac{12R_b}{3d_{90}}\right) --- (2.17)$$

Where:-

g = The gravitational acceleration.

In Which:-

R_b = The hydraulic radius related to the bed.

d_{90} = The sediment diameter finer than 90%.

Still in Equation (2.19), the particle parameter D_* is computed according to the following expression:-

$$D_* = d_{50}\left[\frac{(s-1)g}{v^2}\right]^{\frac{1}{3}} --- (2.19)$$

Where:-

$$s = \rho_s / \rho --- (2.20)$$

v = The fluid kinematics viscosity.

Bed-load in The Transversal Direction:-

In contrast to the equations described in the previous section, functions to compute the bed-load in the direction transverse to the main flow are not so common. The bed-load formulae are essentially one-dimensional but the transport on the bed is two-dimensional, making necessary the use of other means to compute the transversal component of bed-load. This can be done using Hasegawa's equation (Shimizu et al., 1990) as follows.

$$q_{Bx} = q_{By}\left(\frac{u_B}{v_B} - \sqrt{\frac{\tau_{*c}}{\mu_s \mu_k \tau_*}}\frac{\partial \eta}{\partial x}\right) --- (2.21)$$

Where:-

q_{Bx} is the sediment flux transversal direction of the main flow,

q_{By} is the sediment flux in the main flow direction (from van Rijn),

q_{Bx} is the sediment flux in the flow perpendicular direction,

μ_s is the static friction factor (=1.0)) and

μ_k Kinetic friction factor (=0.45).

The flow velocities near the bed u_B and u_B in the transversal direction and in the direction of the main flow respectively are taken from the flow field calculations.

τ_{*c} is the critical shear stress computed with Iwagaki's diagram from the expressions as follows:

671.0≤ R_* τ_{*c} =0.05

162.7≤ R_* ≤671.0 τ_{*c} =0.00849R3/11

54.2≤ R_* ≤162.7 τ_{*c} =0.034

2.14≤ R_* ≤54.4 τ_{*c} =0.195R-7/16

R_* ≤2.14 τ_{*c} =0.14

Where:-

$$R_* = \frac{\sqrt{\left[\frac{\rho_s}{\rho} - 1\right] g d^3}}{v} --- (2.22)$$

Where:-

g is the gravity,

ρ_s and ρ are the specific weight for the sediment particles and the fluid respectively and

d is the characteristic particles diameter.

τ_* is the dimensionless bed shear stress.

2.6.5 Suspended Load Transport:-

In this section, the suspended load transport is introduced. The boundary between the bed and the suspended load transport zone is not very clear. When the turbulence of the flow reaches a certain level, the fine material which is transported near the bed is put into suspension. This mode of transport is governed by the Advection-Diffusion equation and is composed of materials of both origins of sediments: the bed material and the basin eroded material. According to van Rijn (1984a), Bagnold said that a solid is able to remain in suspension when turbulence eddies have dominant vertical velocity components which exceed particle fall elocity.To separate the bed material from the material originated by erosion of the basin the wash load concept was introduced. Wash load is a generally polemical definition. Possibly the most known definition for this part of the suspended transport is the fine material not found in the river bed and banks in appreciable quantity. The words "appreciable quantity" does not help a lot in the wash load definition. Wash load is actually the sediments originating solely from the catchment erosion. Despite the definitions, it is difficult in practice to separate wash load from the bed material load after they have been transported by the rivers. The use of isotope tracers in the soil of the basins can help to distinguish the suspended sediment originated in the river bed from those originated in the catchment.

2.6.6 Sediment Concentration Profile:-

To estimate the suspended load in rivers, one must know the sediment concentration profile. There are several relations for the concentration profile, which are the result of the integration of the one-dimensional diffusion equation.

$$\omega C + \varepsilon_s \frac{dC}{dz} = 0 - - - (2.23)$$

Where:-

ω is the particle fall velocity,

C is the sediment concentration and

ε_s is the sediment diffusion coefficient.

One of the best know integrations of Equation (2.22) is that made by Rouse (1937) as follows

$$\frac{C}{C_a} = \left[\frac{D - z}{z} \frac{a}{D - a} \right]^z - - - (2.24)$$

Where:-

D is the local depth,

C_a is the reference concentration, the concentration at a distance a from the bed and Z is

$$Z = \frac{\omega}{k u_*} - - - (2.25)$$

Where:-

k=0.4 is the von Karman coefficient and

u_* is the bed-shear velocity.

There are other relations for the vertical sediment profiles such as those proposed by van Rijn (1984b, 1986), Shimizu et al. (1990).

As shown in figure (2.13), Van Rijn sediment profile is a complete model providing expressions for all values involved in the computations of the profile. Van Rijn model is as follows

$$\frac{C}{C_a} = \left[\frac{a(d-z)}{z(d-z)}\right]^{z} if\ \frac{z}{d} \le 0.5 --- (2.26a)$$

$$\frac{C}{C_a} = \left[\frac{a}{(d-a)}\right]^{z} e^{-4z(d-0.5)} if\ \frac{z}{d} \ge 0.5 --- (2.26b)$$

Where:-

$Z' = Z + \varphi$ .The value of Z can be obtained with the Equation (2.28), d is the local depth,

a is as defined previously and

φ is

$$\varphi = 2.5\left[\frac{\omega^{s}}{u_*}\right]^{-0.8}\left[\frac{C}{C_0}\right]^{-0.4} for 0.01 \angle \frac{\omega_s}{u_*} \angle 1 --- (2.27)$$

Where:-

$C_0 = 0.65$ is a constant,

ω_s is the particle fall velocity and u_* is the bed-shear velocity

$$Z = \frac{\omega_s}{\beta k u_*} --- (2.28)$$

Where:-

k =0.4 is the von Kàrman coefficient,

β is

$$\beta = 1 + 2\left[\frac{\omega_s}{u_*}\right] for 0.1 \angle \frac{\omega_s}{u_*} \angle 1 --- (2.29)$$

All values used in Equation (2.23) are as defined previously and

u_* is the overall bed-shear velocity.

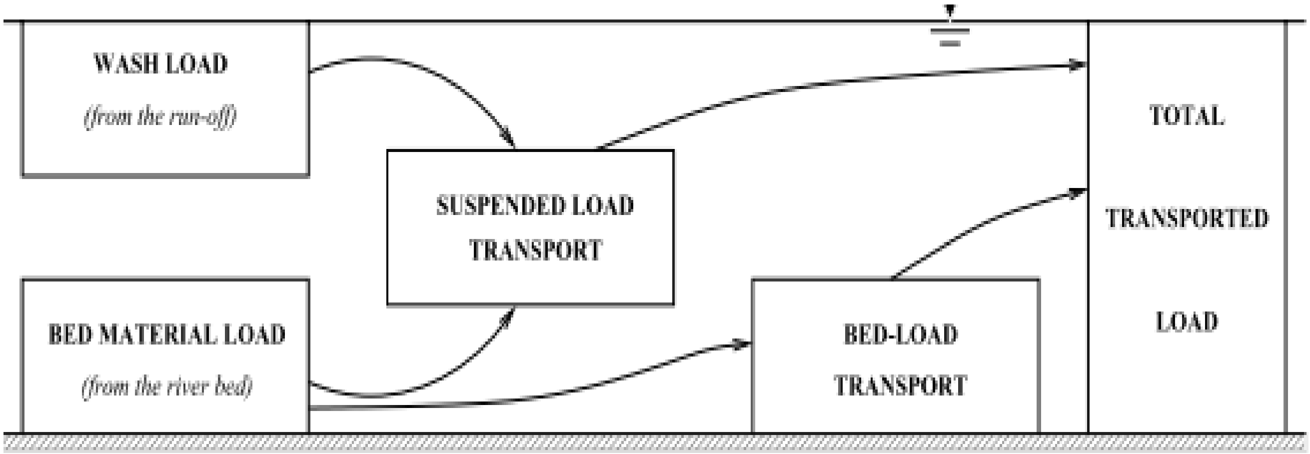


Fig. No.(2.13):Details of Total Sediment Load

Transport must be simulated separately according to their modes. Because of this, the total load transport will not be as precisely depicted as its components. Figure (2.13) is a general scheme of the sediment transport in rivers. It links the sediment origin and the modes of transport. For reservoirs, the bed-load transport is important only in the affluent river backwater reach or in the reservoir tail. The suspended load goes through all the reservoir extension, a part of it is trapped inside the lake and another is released through the reservoir spillway, sluicing gates or turbine water inlet.

2.7.Sedimentation in Reservoirs:-

The construction of a dam across a river affects the physical and hydraulic characteristics of that river reach between the section where the dam stands and the water level in the point upstream corresponding to the stage of the operational structures(spill way) .Indeed, the sedimentation process begins in the backwater reach of the reservoirs.

2.7.1 Sedimentation Deposition Process:-

Owing to the dam, the affluent river flow velocity, and consequently, the turbulence, is reduced because of the river cross section increase. This velocity reduction makes possible the sediment deposition.

In a general way, the responsibility for the particle lifting, transport and deposition can be credited to the flow turbulence. As the turbulence ceases, the particles cannot remain in suspension and begin to settle. That is, when the river competency diminishes, the particles tend to be deposited. Competency is the capacity of river to transport sediments. Its definition is based on the competent velocity concept that is the “mean velocity which is just able to move material of a given size and given specific weight” (Grade and Raju, 1985).

According to Borland (1971), the following occurs when the particles are deposited in the reservoir:

“As the flow enters the reservoir, the cross-sectional channel area of the inflowing stream increase and this is accompanied by a decrease in the velocity and dampening of the turbulence until either or both components become ineffective in the transporting the sediment and the particles begin to deposit”.

Note that, depending on the reservoir size, the finer particles can continue in suspension for long period. Sometime these form a colloid that never allows them to settle.

2.7.2 Reservoir Sediment Deposits:-

The coarser particles are the first ones to be deposited, and generally, they are laid in the backwater formed before the reservoir, in the reservoir headwater and tail. This last deposit, called the delta deposit, is composed principally of gravel and sand. The particles of median size are those next to be deposited and finally, the fine particles fall in the reservoir to form the bottom deposit.

2.7.3 Specific Weight of Deposited Sediments:-

The knowledge of the weight of deposited sediments is necessary to compute more precisely the volume of the sediments stored in the reservoir. However, it is not easy to compute this value as sediments settled into reservoirs are most of the time inaccessible because of water level .This makes material sampling difficult. There are some sediment deposits that are not submerged all the time, which is the case of the small or medium generally empty reservoirs. To compute accurately the deposited sediments specific weight, it is necessary to take undistributed samples, which are not easy taken. There exist equipment (core samplers) used to do such work but the lack of sample disturbance cannot be entirely guaranteed.

When penetrating the sediment deposit, the core samplers use their own weight or percussion that probably causes the sample disturbance.

Lane and Koelzer (1943) presented an empirical equation to estimate the bulk density of sediment deposits based on observed data from American reservoirs, Equation 2.21. It takes into account the particle size and reservoir operation.

$$\gamma_t = \gamma_0 + K \log t \; --- (2.30)$$

Where:-

γ is the specific weight of a deposit aged t years,

γ_0 is initial specific weight of the deposit, generally considered one year after the beginning of reservoir operation and

K is the compaction factor.

Both K and γ_0 are function of the sediment size and method of operation the reservoir.

The values of γ_0 and K were supplied by those authors but later work by Lara and Pemberton(1963) based on data from 1300 reservoirs in United States, present new values of γ_0 and K which are given in table (2.1). The US Rec lamation Bureau recommends the use of those values because they are more up to date.

The Lane and Koelzer equation gives the bulk density of sediment deposit after t years of operation. The average value to the bulk density of the sediments deposited during t years is given by Equation 2.3 presented by Miller (1953).

Table No.(2.2):Values of K and γ_0 (kg/m^3)

after Lara and Pemberton (1963)

Reservoir Operation	Sand		Silt		Clay	
	γ_0	K	γ_0	K	γ_0	K
Sediment continuously submerged	1554	0	1121	91	416	256
Reservoir with periodical drawdown	1554	0	1137	29	561	135
Normally empty reservoir	1554	0	1153	0	641	0
Riverbed sediment	1554	0	1169	0	961	0

$$\overline{\gamma_t} = \gamma_0 + 0.4343 * K\left[\frac{t}{t+1}\ln(t-1)\right] --- (2.31)$$

Equation (2.31) is the result of the integration of equation 2.30 from 1 to t years divided by (t -1) years. The symbols in it are as in Lane and Koelzer's equation.

The expression of miller is related to deposits which are composed of sand or silt or clay. To determine the specific weight of deposit formed by fraction of those materials, the Equation 2.31 can be used to find a value of k if f_i, the percentage of fractions, is known

$$K = \frac{1}{100}\sum f_i K_i ---(2.32)$$

Where:-

K_i is the compaction factor for the i-th fraction.

It should be pointed out that the Lane and Koelzer method has been used for almost half a century without any improvement. More research on this subject is necessary considering the theory of consolidation of submerged sediments. A particular problem to the study of this phenomenon is that river sediments are composed of cohesive and no cohesive, fine and coarse sediments making it difficult to determine expectative values for the particles.

2.7.4. Reservoir Trap Efficiency:-

The reservoir trap efficiency T_e is defined as the amount of the inflowing sediment load which is retained by the reservoir, often expressed in percentage. That is,

$$T_e = (\frac{V_1 - V_0}{V_1}) * 100 ---(2.33)$$

Where:-

V_1 the inflowing sediment load and

V_0 is the outflow sediment load

The T_e is dependent of some variable related with the reservoir and with the inflowing sediments, these main variables are:

1. The inflowing sediment size distribution.

2. The reservoir operation.
3. The reservoir detention time T_d.
4. The reservoir capacity-inflow ratio C/I

Commenting on each one of the variables cited above, the sediment size distribution has influence on the T_e because a sediment load composed of coarser particles will be trapped in bigger amount compared with another composed of finer sediment. This happen since the fall velocity of bigger particles is higher. Then the consequence is the deposition of the coarser particles in shorter time avoiding their release.

A study with data from 17 American reservoirs carried out by Dendy (1974) , concluded that in 10 reservoir a maximum of 4% of the affluent sand was released and 7 remaining, no sand was released. The study suggests that coarser particles are almost totally retained by reservoirs. Particles of small dimensions can stay longer in suspension and are more likely to be freed.

The adequate management of a reservoir for sediment removal is perhaps the most important step to be taken after it is put into operation. A reservoir correctly operated can have the end of its useful life postponed. If it is equipped with sluice gates, these must be maintained open during the floods to permit the sediments to be flushed. This simple rule is cited by several authors as very efficient action to clean sediments from reservoirs. Brune (1953) observed that if the reservoir is flushing water at the same time as occurrence of density currents, the flushing of sediments can be multiplied by three or four,as shown in figure (2.14). Nordin (1990) cited Stevens who mentioned the old high Aswan dam in Egypt, which is equipped with sluice gates rather than spillways. In that dam the sluice gates remain open during all the Nile floods resulting in a reservoir clean of sediments. Suggest the elimination of the use of dead storage volume in the projects of the future reservoirs and the adoption of sluice gates.

Another aspect related to T_e, is the reservoir detention time which is the average time that water remain in the reservoir

$$T_d = \frac{C}{Q} --- (2.34)$$

Where:-

T_d is the detention time,

C is the reservoir capacity and

Q is the reservoir annual average affluent flow A long T_d means that the inflowing water remains

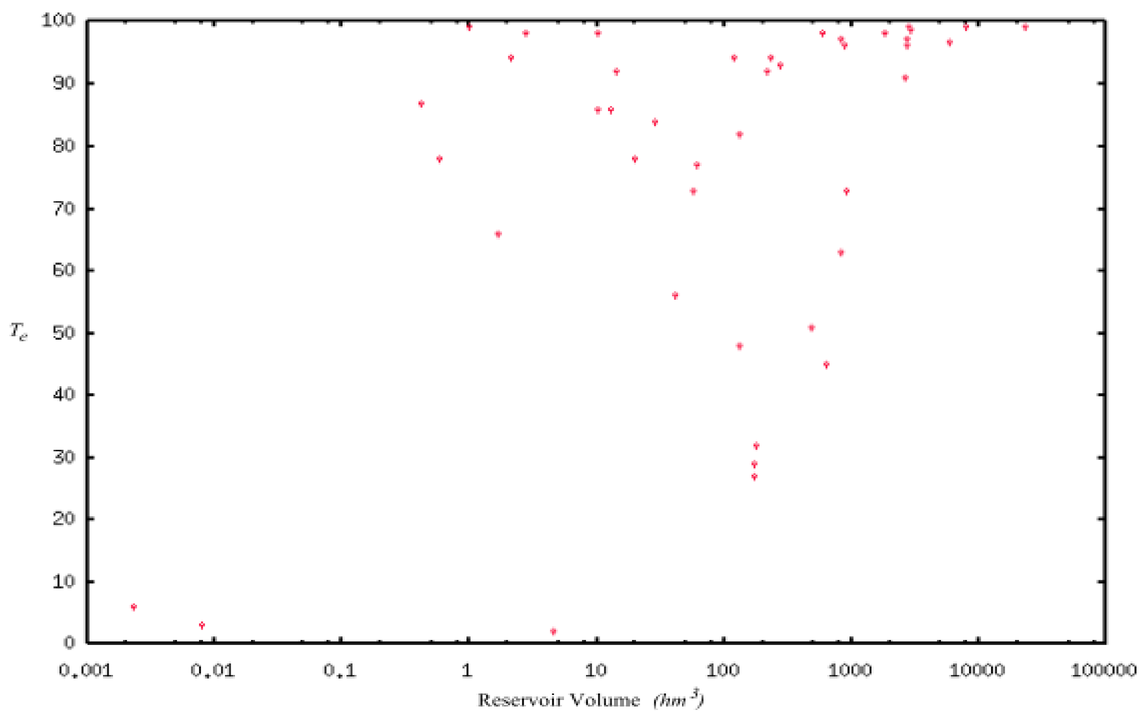


Fig.No.(2.14): The Relation Between T_e and Reservoir Capacity in Brune's Work (After Lopes, 1993)

Longer within the reservoirs making possible the deposition of bigger part of the suspended sediment load.

The quotient C/I where the C is the reservoir capacity and I is the characteristic annual inflow,

In several methods of predicting T_e, this variable is the used. It seems that C/I give the best relationship between T_e and reservoir characteristics. Apparently, the reservoir capacity alone is not good variable for relating the trap efficiency and reservoir characteristics. Figure (2.14) built by Lopes (1993) with Brune's data, shows the relation of T_e in 44 American reservoirs versus their capacity. Apparently, there is not a clear relationship between T_e and reservoir capacities

2.7.5 Reservoir Trap Efficiency Prediction:-

There are many methods to estimate the trap efficiency of reservoir from several authors. They tried to define a relation between the T_e of the reservoirs and the parameters involved, generally though curves but there are also equations. Excepting the Einstein method, all other methods are based on data of T_e from existing reservoir. According to Gill (1979), the first T_e estimating method to be published was the pioneer work by. This author plotted a graph of trap efficiency versus C/A, the ratio of the reservoir capacity (in acre-feet) and the catchment area (in square miles).The method comes with general equation for T_e (originally called E)

$$T_e = 1 - \frac{1}{1 + kC/A} \quad --- (2.35)$$

Where:-

k is the coefficient which varies from 0.046 to 1.0 for the data used by Brown.

After the above mentioned work, the second method for T_e prediction was that presented by Churchill (1948). This author established relation between the sedimentation indexes, SI and sediment load flushed by the reservoir (100- T_e), in percentage. The sedimentation index is defined as follows.

$$SI = \frac{T}{V} --- (2.36)$$

Where:-

$$V = \frac{QL}{C} \quad \text{and} \quad T = \frac{C}{Q}$$

Where

T is the detention time (in s)

V is the mean velocity of the flow through the reservoir (ms^{-1}),

C is the capacity of the reservoir in mean operation pool elevation (m^3),

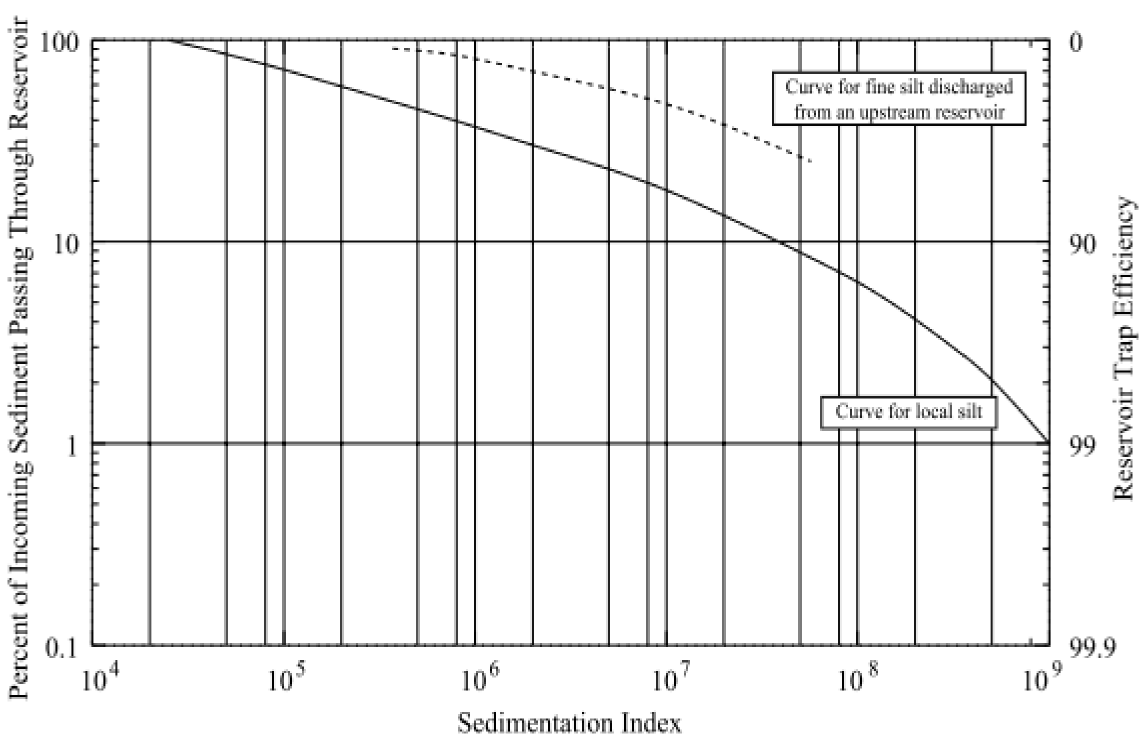
Q is the average reservoir inflow $(m^3 s^{-1})$ and

L is the reservoir extension in the mean operation pool elevation (m).

Figure (2.15) contains the curves of Churchill (1948). Note that in this method SI is not dimensionless variable. Churchill defined two curves to the released sediments. One is for fine silt discharged from upstream reservoir and other is for local silt. That is, for sediments originated in the catchment. Several authors such as Borland (1971) and Rice (2008) stated that the method of

Churchill is the most accurate to predict T_e. Despite this, it is not the most used possibly because of the number of variables involved in the calculations and the dimensional nature of the SI.

Following Churchill, the next method for T_e prediction to be proposed was one by Brune (1953). This method is probably the most used. It is based on data from 44 normal ponded reservoirs in United States. The author plotted the T_e against the reservoir C/I. the Brune digram is composed of three curves, one is median and two envelope ones. Figure (2.16) is the result of Brune's study.



<u>Fig.No.(2.15) Reservoir Trap Efficiency by Churchill</u>

According to Borland (1971), Einstein proposed a T_e model in 1965 based on laboratory data. It is probably the only experimental method for T_e

prediction and apart from Borland, no other reference to Einstein's work was found in the reviewed literature. Thus, it is not possible to state if it was used or tested with real reservoir data.

Einstein defined T as the half-life time to the sediment concentration to be reduced one half (in s)

$$T = 0.657 \frac{d}{w_0 * \eta} --- (2.37)$$

Where:-

η is the ratio between the volume of water above the river bed and the volume of system of flumes used in Einstein experiments,

d is the water depth (in ft) and

w_0 is the characteristic Particle fall velocity (in fts^{-1}).

The value of η can be considered as unitary in rivers and reservoirs.

The fraction of the inflowing material deposited over the reservoir settling basin is given by equation

$$p = 1 - e^{-0.693 l / L} --- (2.38)$$

Where:-

l is the extension of settling basin (in miles) and

L is the extension of the laboratory flume within which one half of the particles are deposited (in miles).

The value of L is

$$L = \frac{VT}{5280} - - - (2.39)$$

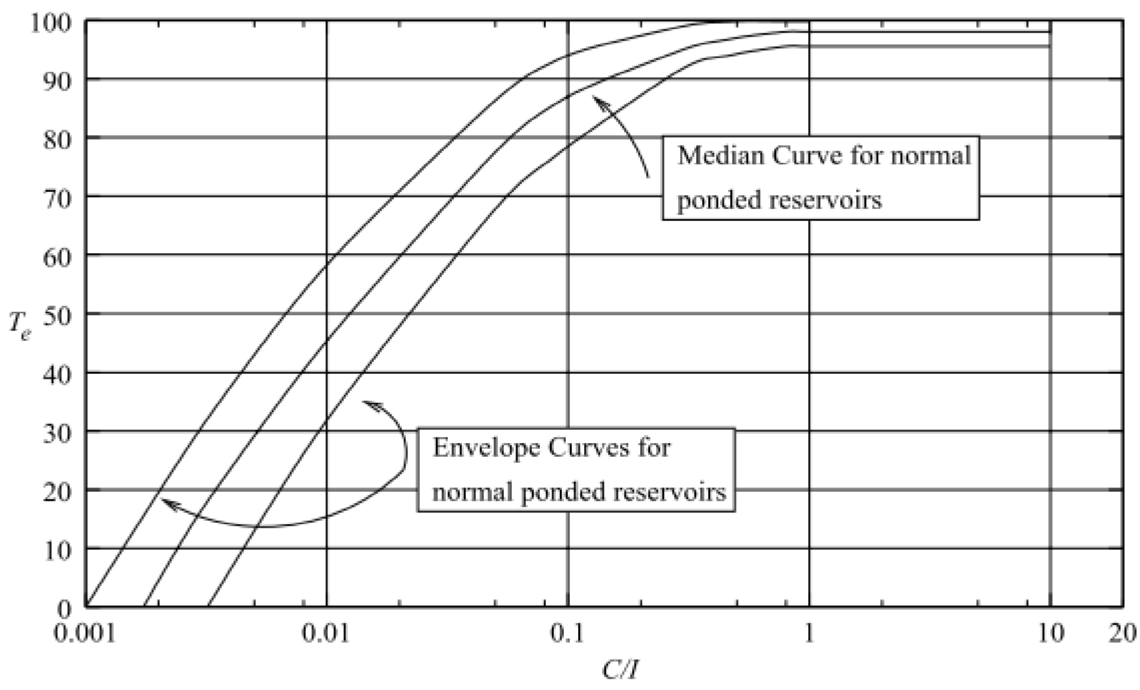
Where:-

V (in fts^{-1}) is the average flow velocity and

T is the defined above.

Combining Equation 2.37and Equation 2.39 and isolating L to be substituted in Equation2.38, an Equation for p is found, which is the trap efficiency in percent if multiplied by 100.

$$p = 100[1 - e^{-\frac{5569.31 l w_0}{vd}}] - - - (2.40)$$



<u>Fig.No.(2.16) Reservoir Trap Efficiency by Brune</u>

Karaushev's equation (1966) is a different method from the others because it takes into account a sediment property, the fall velocity. None of the other methods described above consider any sediment characteristics. The Karaushev equation for T_e in percent is

$$T_e = 100\left[(1 - C/I) * e^{\left(\frac{\phi C/I}{1-C/I}\right)}\right] --- (2.41)$$

Where:-

$$\phi = \frac{w_0 T}{H} --- (2.42)$$

Where:-

e is the Euler number,

T is the reservoir overflow time,

H is the reservoir average depth,

w_0 is sediment fall velocity and

C/I is the capacity quotient as defined previously.

According to the author, the value of ϕ =30 gives the best fit of his equation to the average Brune curve. However, the exact of ϕ is impossible to be calculated because Brune did not supply all the necessary variables.

When the volume of sediment accumulated in the reservoir reach a reasonable quantity, it is necessary to recalculate the reservoir T_e at certain time periods, usually after each 10 years of operation. Theoretically. T_e decrease as the reservoir becomes older.

2.7.6. Reservoir flood-control:-

A reservoir is a depository for the storage of water, to its maximum level. This is defined as the maximum static full pool, which coincides with the level attained when water crests and spills over without a release device. This is the gate-top level for controlled reservoirs, and the spilling crest for uncontrolled reservoirs. Since spilling water implies passage through a critical hydraulic section, a dynamic storage volume can be filled up only during spills. Defined as the discharge pool, its upper boundary is the maximum design water surface. Normally, it is taken as the level at which water spills over the dam. The operational pool is the volume between the minimum level at which controlled releases can be made and the maximum static full pool. It is the water volume under control. For multi-purpose dams, the operational pool is conceptually divided into conservation and flood control pools. This conceptual division stems from the way a reservoir must act to fulfill its objective. The maximum possible empty space is desirable for flood control, while water storage is required for the remaining objectives of water supply, irrigation, hydropower, etc. The upper level of the conservation pool may or may not coincide with the spillway crest. The water level can only exceed this level during and shortly after a flood event. Since flood risk differs according to the season, the flood control pool typically varies according to the time of the year. For countries where dam property and objectives are clearly defined, the flood control pool is contractually established and the conservation managers are constrained from exceeding that level (Simeons, C. (1980)). If a single agency is responsible for flood control and all remaining objectives, the upper level of the conservation pool is more closely considered. In the latter case, added flexibility commonly results in better economic performance but with a substantial risk increase (U.S. Army Corps of Engineers. (1991)

2.7.7 Re-regulation of Hourly Hydropower Reservoir Operations:-

Hourly operations of hydropower reservoirs often involve sudden changes in releases associated with the hourly fluctuations in energy prices. This release pattern, known as hydro peaking, affects stream ecosystems by changing flow conditions on short time scales. Within the Federal Energy Regulatory Commission (FERC) licensing process in the United States, operations are often restricted by limiting rates of change of reservoir releases and by setting minimum releases to the stream. Although more sophisticated approaches in stream flow regime alteration have been recently developed. yet the relationship between flow alteration features and river health still lacks a quantitative characterization (Vehviläinen, I., Pyykkönen, T. (2005). Consequently, simpler approaches are still the most widely used for regulatory purposes.

These operational restrictions limit the ability of the system to follow the pattern of energy prices and potentially reduce the economic value of daily generation. This effect can be alleviated if a water storage facility downstream of the power house re-regulates the release pattern. Such an alternative typically exists in cascade hydropower systems, where the most downstream reservoir can be used for this purpose.

It is important to distinguish between re-regulation facilities whose sole purpose is to mitigate hydro peaking operations by an upstream reservoir, and those used for the double purpose of re-regulation and power generation. In the first case all releases from the re-regulation facility are discharged into the stream. In this case, the re-regulation facility has to mitigate the hydro peaking operations and provide lower and upper bounds to in stream flows.

In the second case, water releases from the re-regulation storage facility can be allocated either for power generation in a downstream plant or for in stream flow in the downstream reach. Here, besides buffering ramping effects, the downstream facility ensures water is allocated to the stream at all times so the MIF requirement is met.

2.7.8. Importance of Reservoirs and Dams:-

A reservoir is a man-made lake which is created when a dam is built on a river or a stream. The river or stream water back up behind the dam creating a reservoir which is used for various purposes. The reservoir is equipped with an outlet structure which is constructed with concrete or pipe. They are mostly constructed for the following purposes

• To hold water for domestic use, agricultural purposes (irrigation) and for industrial use. When water is retained in reservoirs they are made to go through a period of self purification since sedimentation is allowed to take place. The water is then clear to some extent to be used for domestic purposes.

• To hold water to prevent flooding when there is intense rainfall that can cause flooding to a community. Reservoirs commonly used for this purposes are called attenuation reservoirs and are used to prevent flooding of low lying areas. They store water during periods with abnormally high rainfall and gradually release the water during periods of low rainfall.

• To hold water for the purposes of electricity generation and for powering wind mills. The reservoirs for this purpose are equipped with turbines which generate the electricity. Reservoirs can be constructed for secondary purposes as recreation such as sailing, fishing and water skiting.

CHAPTER THREE

3. METHODOLOGY MATERIALS AND EQUPMENTS USED

3.1. Road Map:-

The road map is the methodology including materials and equipments used to analyze the problems. The prevailing problems are mainly intakes reduced hydropower. Sediment transportation, and decreasing cultivated areas, as well as complexity in operation and river bank erosion are other main problems.

Analysis of the problems clarified and paved the road to the objectives fulfillment. The objectives are mainly assessment of the impact of sediment on irrigation water and optimization of use and consumption of water for irrigation. It indicated the interaction among discharge sediment and power. It also determined impact of the Reservoir operation on the schemes of the Gezira, Suki, Rahad, and North West Sennar Sugar Factory areas. The objective was extended to indicate the erosion occurring during flood season, and suggest some recommendations to measures that can be taken to reduce the amount of deposited sediment. The road map necessarily included both desk and filed studies including laboratory works and investigations.

3.2. Intake Blockage and Hydropower:-

In Alexandria Engineering Journal Volume 52 issue 4 December 2013 Sayed Mahgoub indicated that enhanced sediment distribution at the vicinity of power plant intakes using double rows of vanes and groins as a case study, new Tebbin power plant was very effective. Many studies were carried out to mitigate the entrance of sediment into intakes. Among these are: Abdel-Fattah (2004) studied the river morphological changes. He used two dimensional (2-D) numerical model and investigated sediment distribution at El-Kurimat thermal power plant intake.He used both dredging and adding groins upstream the intake using different scenarios. He revealed that using groins and dredging upstream intake increased the flow ratio in front of intake and diverts sediment away off it. Surveying site after implementing, he found that the intake still faces sedimentation problems. Abdel-Fattah, (2004)

AbdelHaleem (2008) using a single row of submerged vanes executed an experimental study to minimize the sediment that enters the intake channel. He defined the optimum heights, angles, and positions of the submerged vanes in front of the intake channel AbdelHaleem (2009) executed an investigation using double and triple rows of vanes perpendicular to the flow direction. The main objective of his work was the determination of the optimum vane characteristics. He indicated that the optimum vane characteristics should be having an attack angle of 30° and vane height 0.3 m. water depth, which diverted sediment by 50–90% in case of triple rows and by 50–85% in case of double Hassanpour and Ayoubzadeh (2008) used experiments to investigate the hydraulic performance of submerged vanes within high Froude numbers. They reveald that in supercritical flow, a sudden increase in flow depth occurred downstream close to intake compared to the condition with no vanes. They indicated that the application of 25° angle submerged vanes caused an increase in intake ratio. They observed a reduction in intake ratio in the case of 15° angle vanes and no change occurred to the intake ratio when the angle was 20. Hossain et al. (2004) also used experimental studied for scour around and downstream the bottom vanes. They developed empirical formulae for predicting the maximum scour depth to serve the determination of a safe depth for bottom vanes. The developed formulas related the flow depth and the projected area of the vane. They did not consider other parameters due to the fact that the equilibrium live-bed scour do not vary by increasing the velocity or the grain size Odgaard (2005) described the Iowa vanes, as structures placed in an eroding streambed that cause the flow to be redirected. This resulted in the deposition of sediment on the eroding bank. He indicated that vanes stabilize the stream without affecting the sediment load and velocity of other parts of the stream. Sadjedi Sabegh (2004) conducted experimental investigation on sediment control in intakes using submerged vanes in the intake of Bishe-Zard River in Iran. His conclusion was that, in the three vanes orientation, the best result was obtained when the distance of inner vanes from the channel wall was 3 h (h is the vane height) and the distance between the other two vanes, in each row, was 2 h. For vanes with zigzag form, when the inner distance was 1 h and the distance across between vanes was 3 h, the best solution was obtained. He indicated that, in case of using three vanes in each row, the sedimentation in the intake and delivery channel was decreased by 55%. However; in case of applying vanes in zigzag orientation, the depth and shape of groove became more suitable and the sediment deposition decreased by 75%. Tan Soon-Keat et al. (2005) investigated the three dimensional flow around the submerged vane in nature. They mentioned that the flow may be divided into four different zones according to the different locations around the vane. They described the flow structures in these flow zones (i.e., the left and right head zones in the direction of the flow, the immediate frontal zone, and the lee zone) Sayed(2013)

In a recent investigation, the bed changes in a section of the river were computed using a three dimensional (3-D) model. The results were in accordance with the regular bed level surveys before and after the flow discharge at maximum, minimum, and dominant flow conditions.

Utilizing the reviewed literature, extended study was initiated. In Alexandria Engineering Journal Volume 52 issue of 4th December 2013 Sayed Mahgoub focused on investigating the efficiency of installing double rows of vanes in front of the intake in the flow direction with the addition of groins to the left side of the flow to redistribute the sediment downstream of the intake channel. Abdel-Fattah (2004)

He used a movable bed model, with a scale of 1:50 and relative density of 2650 kg/m3, constructed at Hydraulics Research Institute (HRI), the National Water Research Center (NWRC). The used particles have a mean diameter D50 of 0.17 mm. A comprehensive model test program was designed to cover the different river flow conditions and operation modes of the power plant. Sixteen (16) experiments were run at different flow conditions. Double rows of submerged vanes were mounted vertically at an angle of 60° to the main flow direction. These rows were set to generate a secondary circulation in the main flow in order to modify the near bed flow pattern thus re-distributing the flow and the sediment transport within the channel cross-section. For comparison purposes, a case was tested in the absence of vanes. Groins were also added at the left bank (i.e., downstream of the intake structure along the flow direction) in order to minimize the sediment deposition downstream of the intake structure. The study results showed that, in case of absence of vanes, sediments with rates 1–2 m3/week were stuck within the sediment trap under the winter conditions. The results indicated that the submerged vanes play an important role in preventing the sediment intrusion. It was clear that using groins might lead to enhancing the sediment distribution at the intake vicinity.

Sediment deposition at the entrance of river intake structures is a serious problem; it reduces the withdrawn capacity, causes damages to the pumping system, and partial or full blockage of the intake, resulting in plant stoppage.

Submerged vanes controlled the channel cross- section erosion, its maintenance, adjusting the stream, and created new bed morphology. The vanes produced a scour trench in front of the intake, allowing the vanes to minimize the bed sediment intrusion into the diversions of the alluvial channels.

New Tebbin Power Plant (NTPP) was investigated as a case study, in the Hydraulics Research Institute (HRI) experimental hall, the National Water Research Center (NWRC). The main objective of the study was to investigate the morphological conditions at the vicinity of power plants under different operation modes in order to mitigate the sedimentation problems at the intake structure vicinity. Figure (3.1) and figure (3.2) present the location of velocity cross-sections measurements and the bed level deformations measurements, respectively. Figure (3.3) present a comparison between the flow velocity distribution in the model and prototype, and figure (3.4) shows the groin location at the left bank of the channel.

It was concluded that: when the intake structure was tested with normal flow conditions without submerged vanes, some of the bed sediment material entered the intake and was stuck within the sediment trap. This sedimentation process occurs only in the winter time with a rate of approximately 1–2 m3/week. The bed sediment of river at the project vicinity has a mean diameter, D50 that varies from 290 to 450 μm.

The model only tested the sediment bed load, while suspended sediment load is minor in the Nile River in Egypt. The suspended sediment always has D50 less than 300 μm.This is different than that in the upstream.

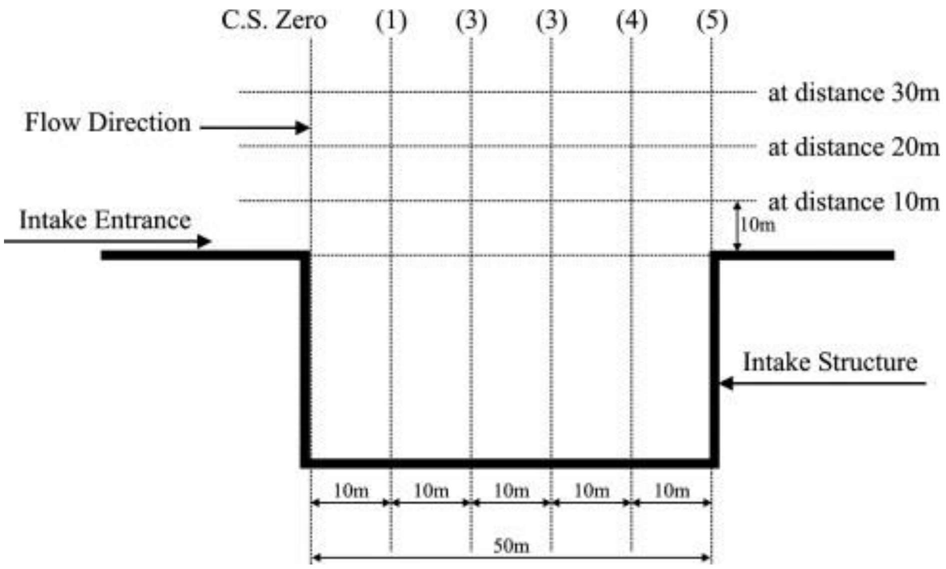


Fig.No.(3.1): Location of the Bed Level Deformations Measurements

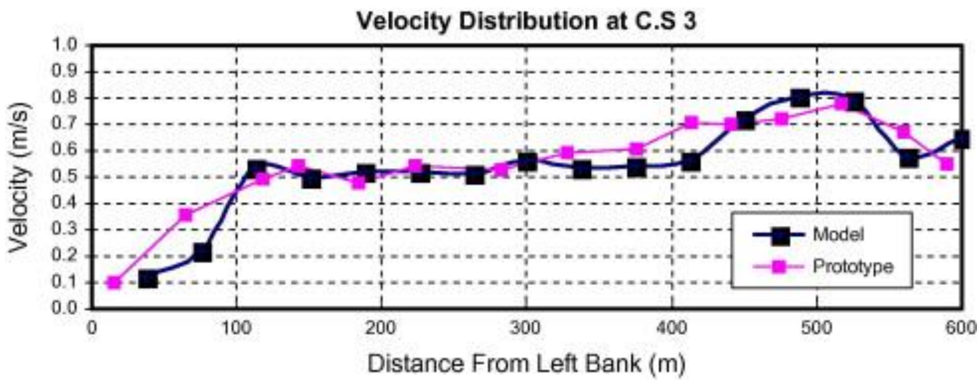
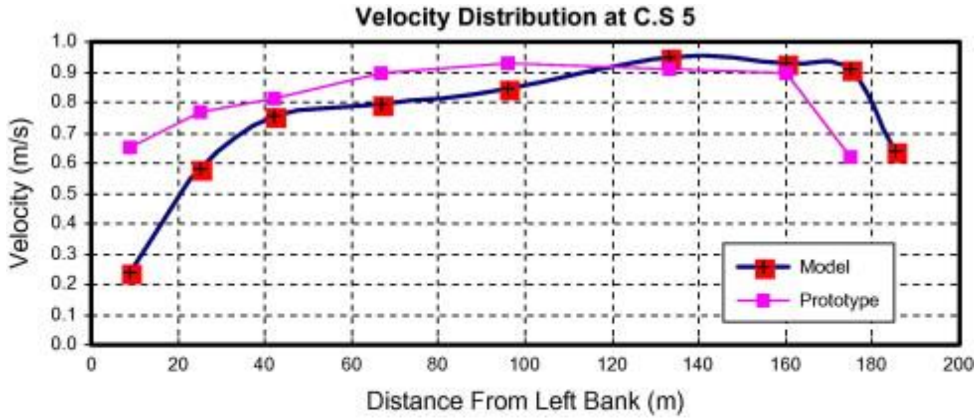
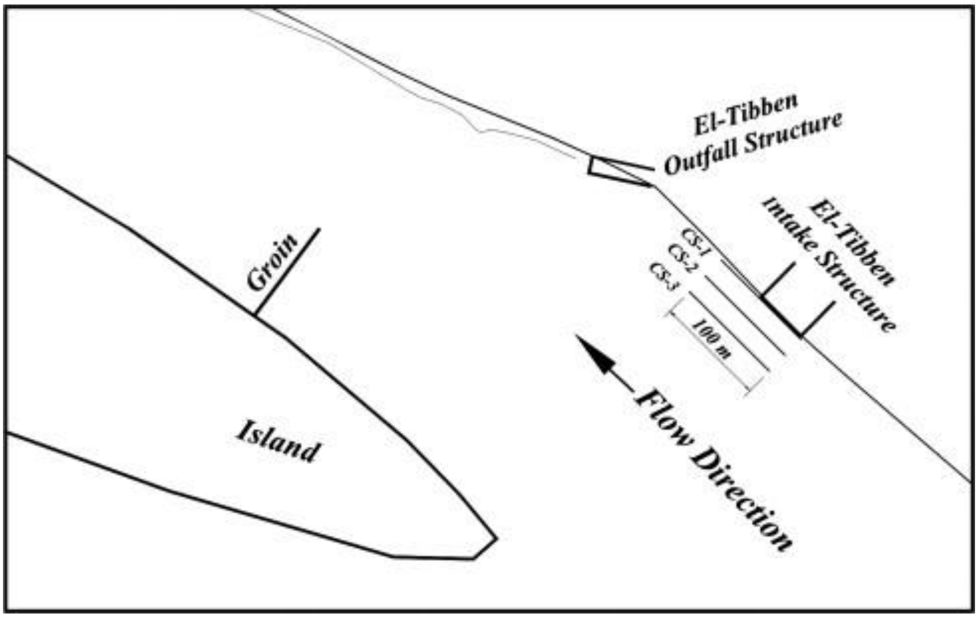


Fig.No.(3.2): Measured and Computed Flow Velocity at a C.S.



<u>Fig.No.(3.3): Measured and Computed Flow Velocity</u>



<u>Fig.No.(3.4): Groin Location at The Left Bank Of The Channel.</u>

Although the submerged vanes prevent any sediment from entering the intake under all normal flow conditions even in winter, there is still a possibility that some bed sediment load could enter the intake during emergency cases such as, dredging or construction of any new structure within the plant vicinity. These emergency cases are the main reasons for dredging within the intake.

Introducing groins in the area of sediment accumulation were necessary to redistribute the sediment downstream of the intake structure. The cross-flow did not exceed the critical value inside the navigation path in the winter flow conditions; thus, cross-flow does not present any problem for the navigation issue. Hence based on Sayed In strategic perspective and options assessment of Blue Nile multipurpose developmet eastern Nile water resource modeling using Mike Basin,Asegdew Gashaw ,researcher in AAU/Intern, ENTROindicated that there are a number of dams planed in the upper Blue Nile (Abay) river basin. He considered the reservoirs:- Karadobi , Bekoabo,Mendia, Gerd, Tekeze Ethiopia; Rosaries, Sennar, Merewe, Jebel Aluia , Khashim El Gibra Sudan;and HAD Egypt.

The objectives of his research was to provide quantitative analysis of water resources management for the eastern Nile region by considering the current water use situation and proposed reservoirs The scenarios were evaluated based on economical benefits for the whole eastern Nile and benefits on the project site. He used a model its input was Rivers (catchment),reservoirs, loss and gain, water use demand, hydropower, and STREAM flow model out put: used water generated hydro power deficit water demand deficit reservoir level, storage, net flow to nodes. He represented mean monthly energy production of Rosaries (GWH) in figure (3.5) He conducted his schematization**,** and his different scenarios, to get the best combination of the reservoirs operating simultaneously. Three discharge locations of the whole Eastern Nile Basin (Eldiem, Khartoum and Dongola) have been selected depending on the availability of data. He found that the coefficient of determination for both Eldiem and Khartoum is 0.94 and that of Dongolo is 0.84 which shows very good performance. His result outflow at Roseries Sennar reservoirs are presented in figures (3.6 and 3.7). Mean monthly energy production of Rosaries (GWH) shown in figure (3.8) and table (3.1) present the power production in the three countries.

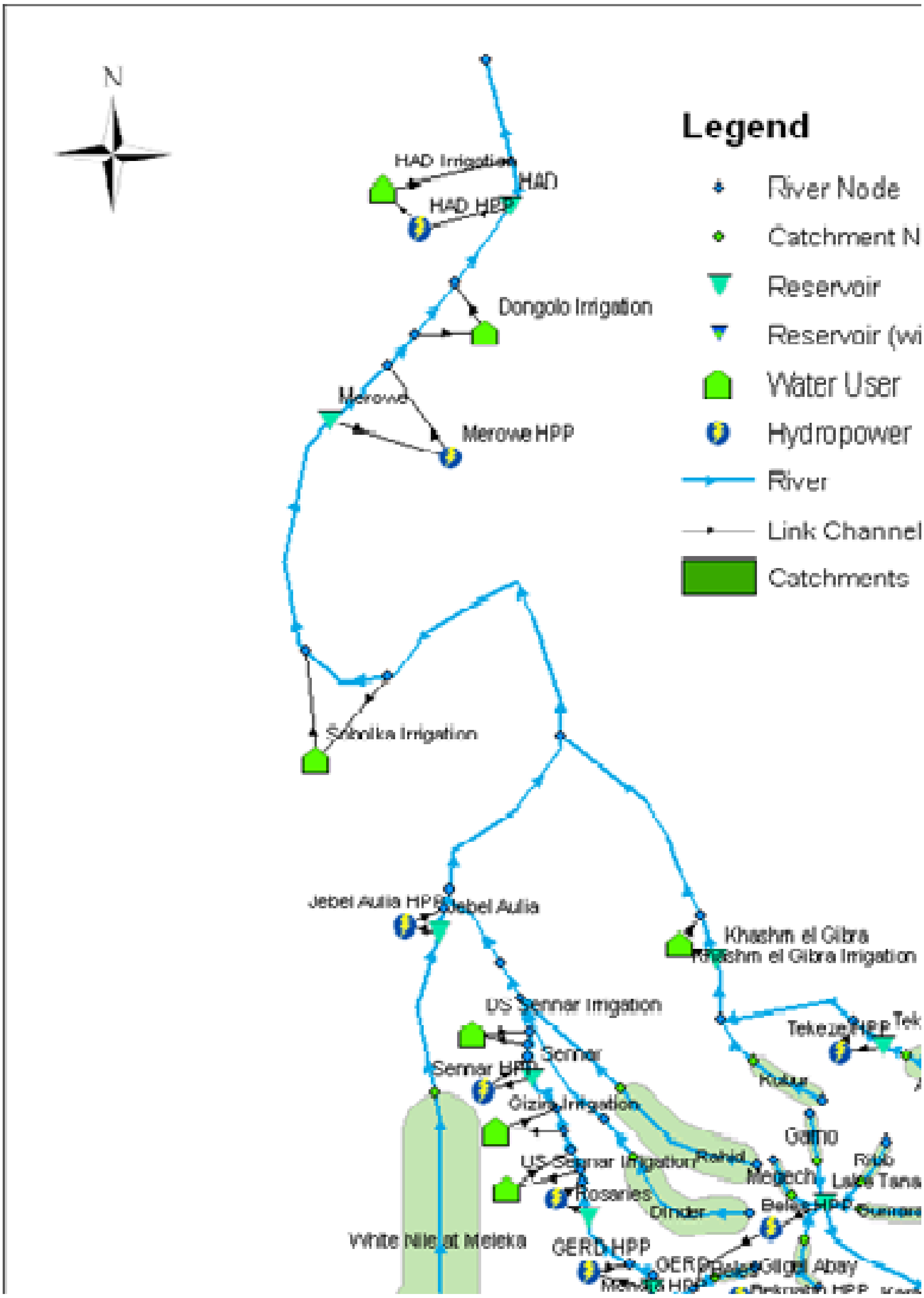


Fig.No.(3.5):Mode Schematization

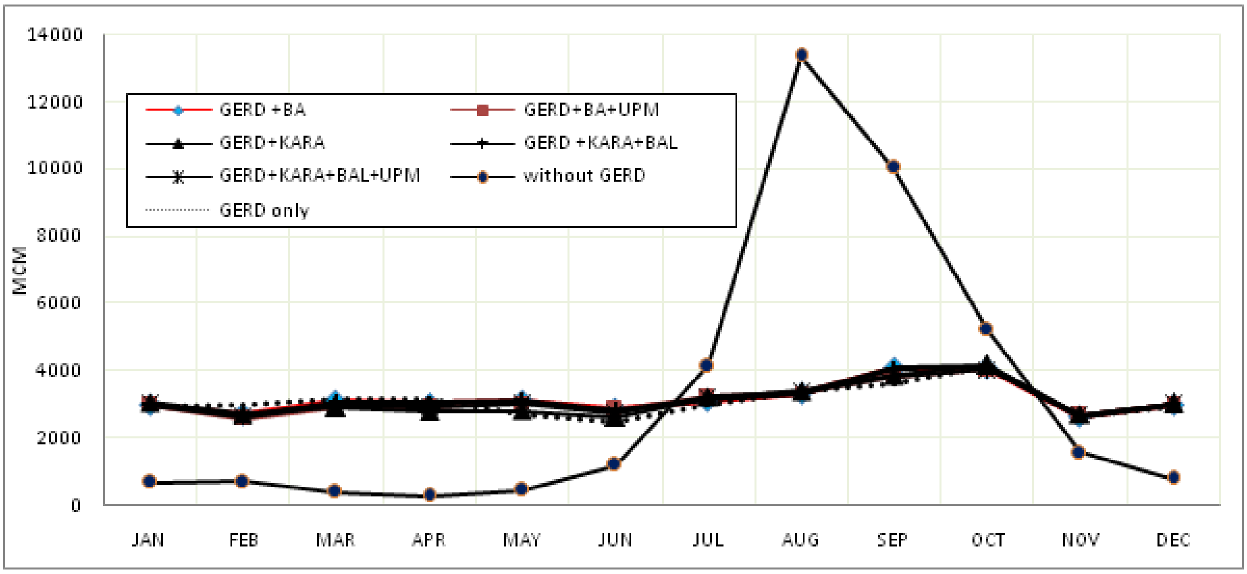


Fig.No.(3.6):Rosaries Inflow

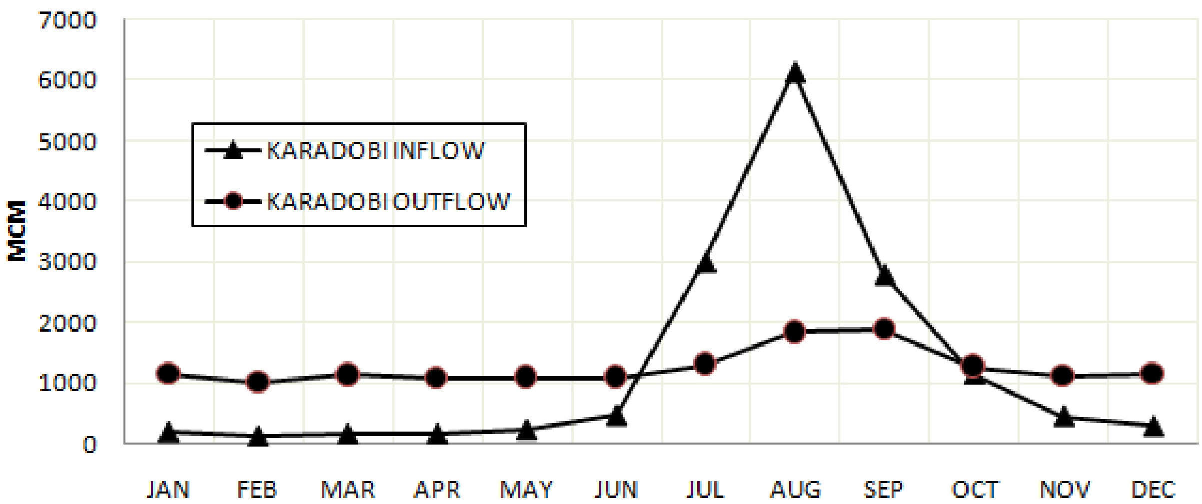


Fig.No.(3.7):Sennar Inflow

Table No.(3.1): Power Production in the Three Countries

Power (GWH)	Ethiopia	Sudan	Egypt
Max	39154	11598	8007
Min	3990	8651	7718

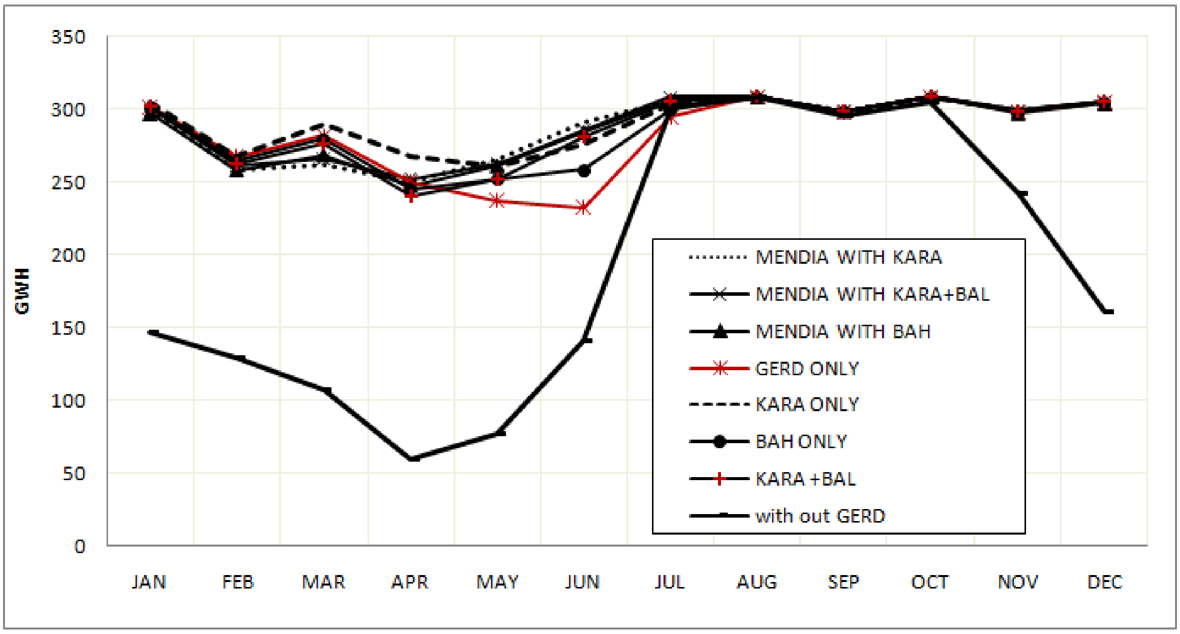


Fig.No.(3.8):Mean monthly Energy production of Rosaries (GWH)

Yasir (14) conducted a study of the effects of erosion control practices in the upper Blue Nile River on the downstream sedimentation rates. His objectives were to assess and understand the sedimentation processes; erosion, transport and deposition in the Blue Nile River system. It also included assess of the locations in which the effects of erosion control practices would be mostly felt. To quantify these effects; and estimate sedimentation rates, sediment origin and timing of sediment transfer; Yasir (2014) of the Hydraulic Research Station Sudan, using a morph dynamic model in Roseires dam 550 km south of Khartoum and 110 km from the border, with a capacity 3.3 billion m3 at 481 m.a.s.l in 1966 revealed that the lost quantity was 42.3 % by 2007.

Yasir methodology used bathymetric surveys of 1985, 1992, 2005, 2007 and 2009.He conducted analysis of discharge and water level, suspended sediment measurements and bed load data. He Developed 2D-depth averaged model reservoir and the river reach upstream based on Delft 3D . He conducted calibration of the model for year 2009 and validation for year 2010 hydrodynamic ally. Then he calibrated the morphdynamic model for 1986-1992 and validated for 1992-2007.He presented his results in cross sections, a typical one is as shown in figure (3.9).

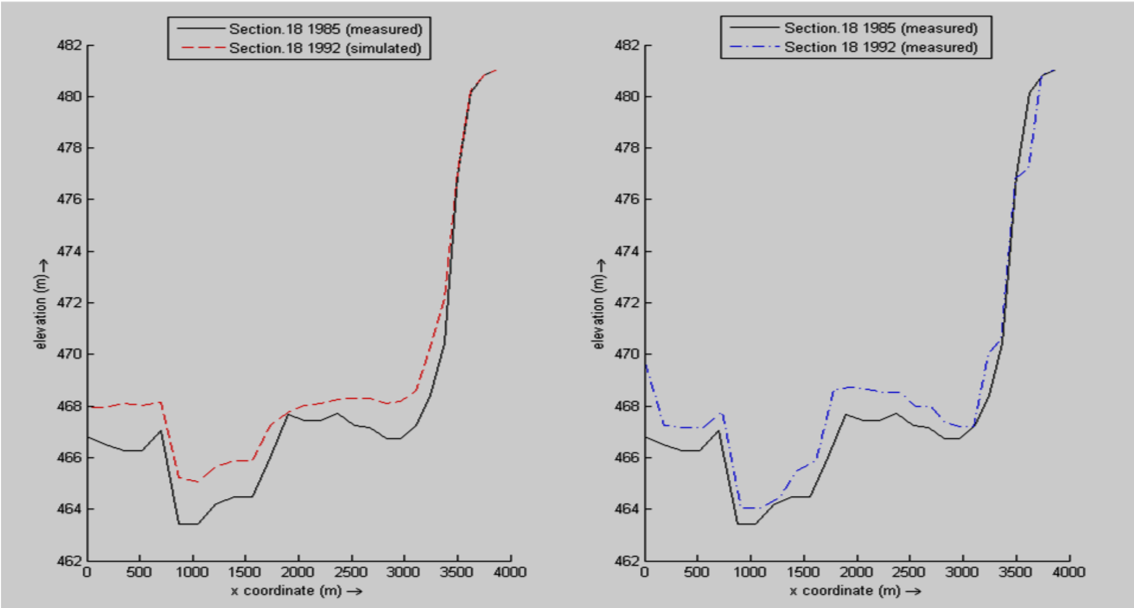


Fig.No.(3.9):The Cross Section 18 Simulated and Measured Section in 1992 With Respect To 1985

Yasir then quantified the water uses along the Blue Nile River network using 1D hydrodynamic model. He indicated the results were, basin area about 330,000 km2, around Lake Tana ;2900 , and m.a.s.l join White Nile at Khartoum; 490 m.a.s.l Yasir extended his objectives to study the water distribution along the entire Blue Nile River system, develop Hydrodynamic model using Sobek River, and used the calibrated model with Delft 3D Delwaq to study the sediment transport. He made schematization shown in figure (3.10). Yasir used three reaches to calibrate the water levels, the first one was at El Deim station upstream Roseires.

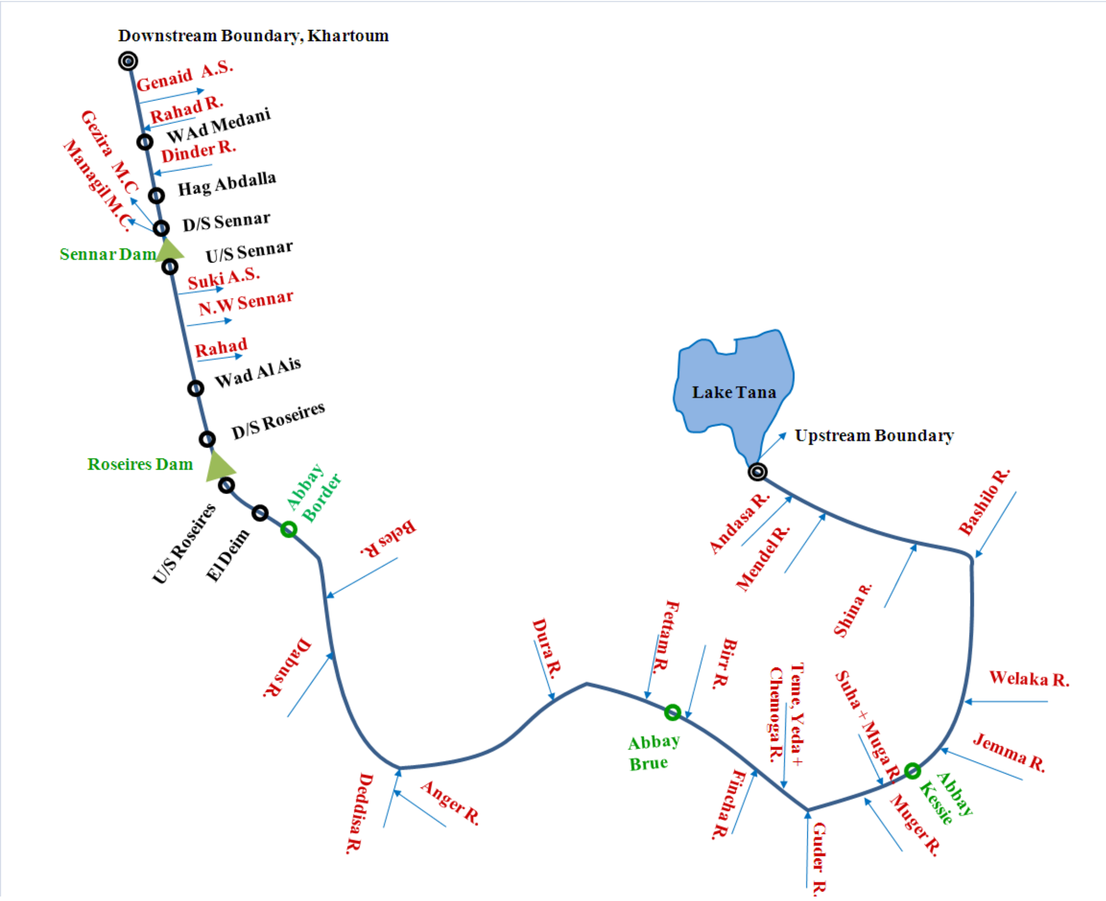
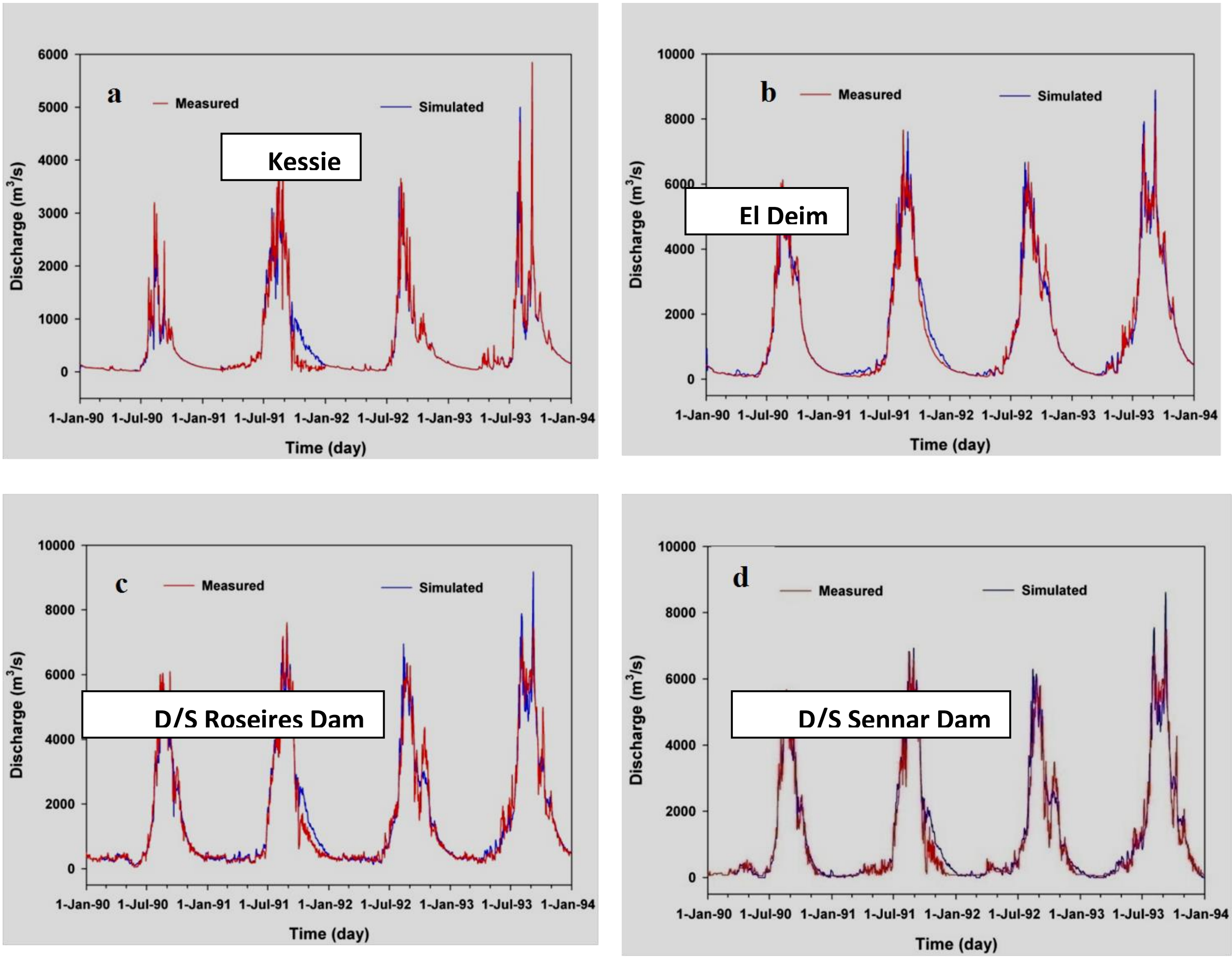


FIG.NO.(3.10):Yasir Mode Schematization

Station, the second was Wad Al Ais station upstream Sennar station, and third one was Hag Abdalla station Wad Medani station. The result is as shown in figure (3.11).



<u>Fig.No.(3.11):Calibration (Flow)</u>

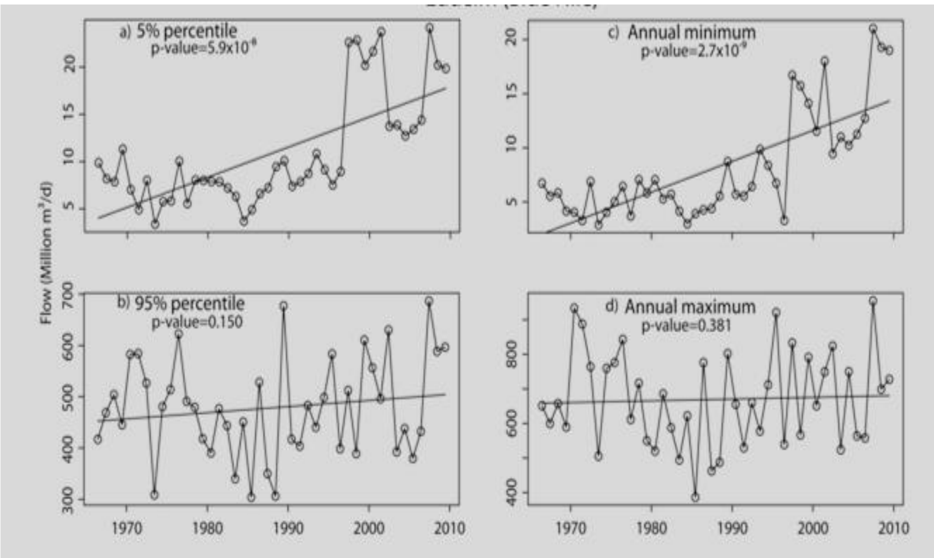
In a statistical analysis of Nile flows study Ageel I. Bushara of The hydraulics Research Center (HRC) Ministry of Water resources and Electricity, Sudan conducted his research on Blue Nile flow analysis at El Deim Khartoum and other stations. He introduced the map to show the locations of the stations in figure (3.12).He proposed his research question as what are the long-term trends of the Nile flow: annual, low and high?

Fig. No.(3.12):The Locations of the Two Stations

He used Spell STAT software; and obtained the result as in table (3.2), and figure (3.13) at El Deim station.

Table No.(3.2)Summary of Statistical Tests of Flow

Station	Mean – Billion m^3/year	Max flow-Billion m^3/year	Min flow-Billion m^3/year	Flow range-Billion m^3/year	Variance	Coefficient of variation	Skewness	Kurtosis
Eddeim	47.2416	67.2586	29.902	37.3566	88.5371	0.199	0.2858	-0.4872
Khartoum	43.5299	76.3574	14.807	61.5504	170.3567	0.3	0.4619	0.1967
Malakal	31.4369	39.3775	24.7009	14.6766	11.6253	0.108	0.0705	-0.2414
Dongola	71.999	109.3324	39.9832	69.3492	248.8045	0.219	0.2963	-0.2661



<u>Fig.No.(3.13): Annual Flows at El Deim Station</u>

He determined the trends of low and high flows as shown in table (3.3), revealed that 1925 Sennar Dam has Storage capacity:1 BCM ,with Irrigation:200 000 ha.

<u>Table No.(3.3):Trends of Low and High Flows</u>

Station	Annual flows- (Flow in Billion m^3)	5% percentile of annual daily flow (Million m^3/d)	95% percentile of annual daily flow (Million m^3/d)	Annual minimum daily flow (Million m^3/d)	Annual maximum daily flow (Million m^3/d)
Eddeim	$=41.8330+0.2404x$	$y=3.6991+0.3193x$	$y=451.2151+1.1973x$	$y=1.7398+0.2855x$	$y=658.0922+0.5023x$
Khartoum	$=37.4386+0.2707x$	$y=17.6114-0.1427x$	$y=456.3250+3.0184x$	$y=15.1672-0.1799x$	$y=623.5508+1.4887x$
Malakal	$=32.1355-0.0310x$	$y=55.8886-0.0591x$	$y=121.3837-0.3282x$	$y=54.0477-0.0516x$	$y=121.7539-0.2901x$
Dongola	$=71.8685+0.0058x$	$y=70.8103-0.4060x$	$y=601.2114+2.3741x$	$y=65.7669-0.4399x$	$y=731.9241+1.6762x$

3.3.Upstream Downstream River Aggradation Degradation and Scour:-

In 2009, a report by researchers Black, Richard (2009-09-21)from the University of Colorado at Boulder in the journal Nature Geoscience said that reduced aggradation was contributing to an increased risk of flooding in many river deltas.Aggradation (or alleviation) is the term used in geology for the increase in land elevation, typically in a river system, due to the deposition of sediment. Aggradations occur in areas in which the supply of sediment is greater than the amount of material that the system is able to transport. The mass balance between sediment being transported and sediment in the bed can be estimated by change in elevation.

Typical aggradational environments include lowland alluvial rivers, river deltas, and alluvial fans. Aggradational environments are often undergoing slow subsidence which balances the increase in land surface elevation due to aggradation. After millions of years, an aggradational environment will become a sedimentary basin, which contains the deposited sediment, including paleo channels and ancient floodplains.

Aggradation can be caused by changes in climate, land use, and geologic activity, such as volcanic eruption, earthquakes, and faulting. For example, volcanic eruptions may lead to rivers carrying more sediment than the flow that can be transported This leads to the burial of the old channel and its floodplain. In another example, the quantity of sediment entering a river channel may increase when climate becomes drier. The increase in sediment is caused by a decrease in soil binding those results from plant growth being suppressed. The drier conditions cause river flow to decrease at the same time as sediment is being supplied in greater quantities, resulting in the river becoming choked with sediment.

In the net Engineering 112 (6): *497.* doi:10.1061/(ASCE)0733-9429(1986)112:6(497),revealed that in geology, degradation refers to the lowering of a fluvial surface, such as a stream bed or floodplain, through emotional processes. It is the opposite of aggradations. Degradation is characteristic of channel networks in which either bedrock erosion is taking place, or in systems that are sediment-starved and are therefore entraining more material than is being deposited. When a stream degrades, it leaves behind a fluvial terrace. This can be further classified as a strath terrace—a bedrock terrace that may have a thin mantle of alluvium—if the river is incising through bedrock. These terraces may often be dated with methods such as cosmogony radionuclide dating, and pale magnetic dating to find when a river was at a particular level and how quickly it is down cutting.

Hydraulic Engineering Circular No. 18 Manual (HEC-18) was published by the Federal Highway Administration (FHWA). This manual includes several techniques of estimating scour depth. The manual gave empirical scour equations for live bed scour, clear water scour, and local scour at piers and abutments. The total scour depth was determined by adding three scour components which included the long-term aggradations and degradation of the river bed, general scour at the bridge and local scour at the piers or abutments. Richardson, E. V., & Davis, S (2001) However, research had shown that the standard equations in HEC-18 over-predict scour depth for a number of hydraulic and geologic conditions. Most of the HEC-18 relationships are based on laboratory flume studies conducted with sand-sized sediments increased with factors of safety that are not easily recognizable or adjustable. Chase, K. J., Holnbeck, S. R(2004) Sand and fine gravel were the most easily eroded bed materials, but streams frequently contained much more scour resistant materials such as compact silt, stiff clay, and shale. The consequences of using design methods based on a single soil type are especially significant for many major physiographic provinces with distinctly different geologic conditions and foundation materials. This lead to overly conservative design values for scour in low risk or non-critical hydrologic conditions. Thus, equation improvements were continued to be made in an effort to minimize the underestimation and overestimation of scour.

3.4.Assessment of Sediment Impact and Optimized Consumption of Irrigation Water:-

Islam Al Zayed1et.al.in an analysis of irrigation efficiency using comparative performance indicators conducted a case study of Gezira Scheme, Sudan. Indicated that previous researchers revealed that Gezira Scheme is wasting irrigation water, there is always poor distribution and inadequate irrigation management .With the objective to assess the irrigation performance for Gezira Scheme, they relied on their argument that irrigation indicators improve irrigation management. Figure (3.14) represent the area of study of the Gezira Scheme boundaries.

They used metrological,irrigation,crop yield and cultivated areas (1970 -2007),for cotton, groundnut, sorghum and, wheat. Figure (3.15) shows the cotton production.

Hence their main conclusions were maximum demand for cotton is (1430 mm),the water availability is higher than the demand, inadequate supply of moisture from rainfall, rainfall should be utilized and harvested to reduce the need of supplemental irrigation, Gezira Scheme has low productivity values.

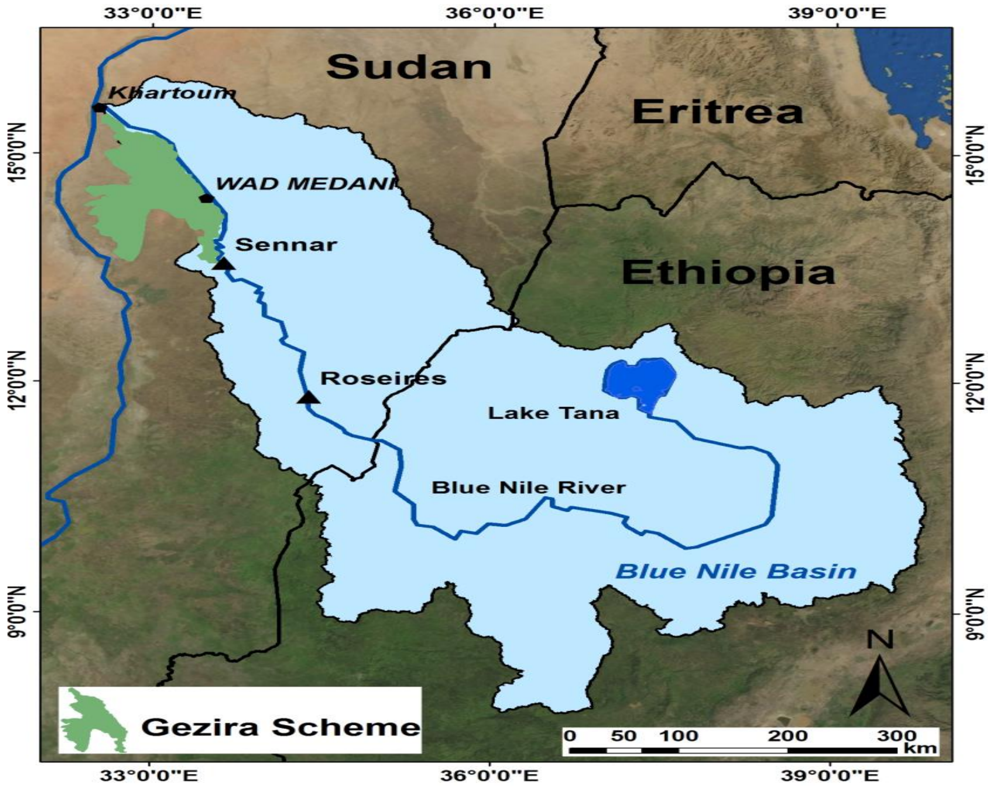


Fig.No.(3.14):Area Of The Gezira Scheme Boundaries

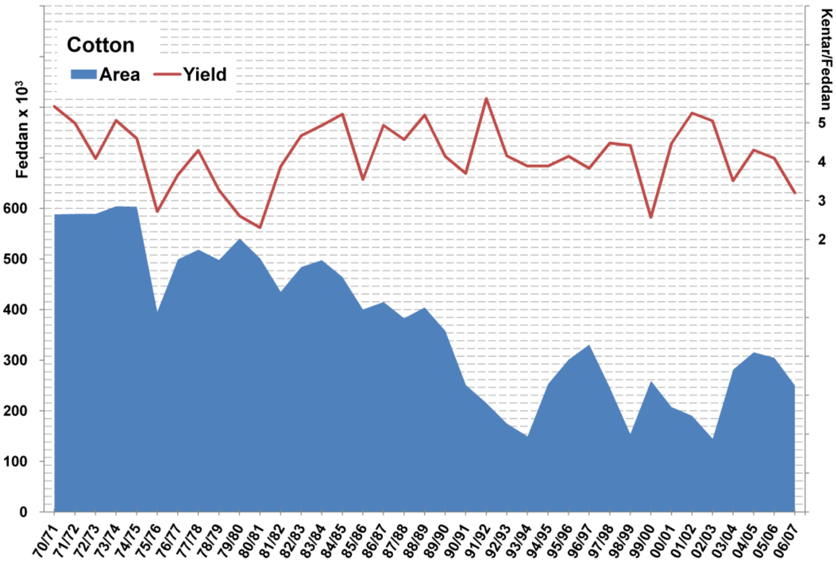


Fig.No.(3.15): Cotton Production

Quantification of water uses along the Blue Nile River network using 1D hydrodynamic model conducted by Yasir revealed that Basin area is about 330,000 km2. Originate a round Lake Tana; 2900 m.a.s.l,and Join White Nile at Khartoum; 490 m.a.s.l. This required study the water distribution along the entire Blue Nile River system. Develop Hydrodynamic model using Sobek River Use the calibrated model with Delft 3D Delwaq to study the sediment transport. The methodology adopted consisted of a certain Mode schematization as presented in figure (3.16).

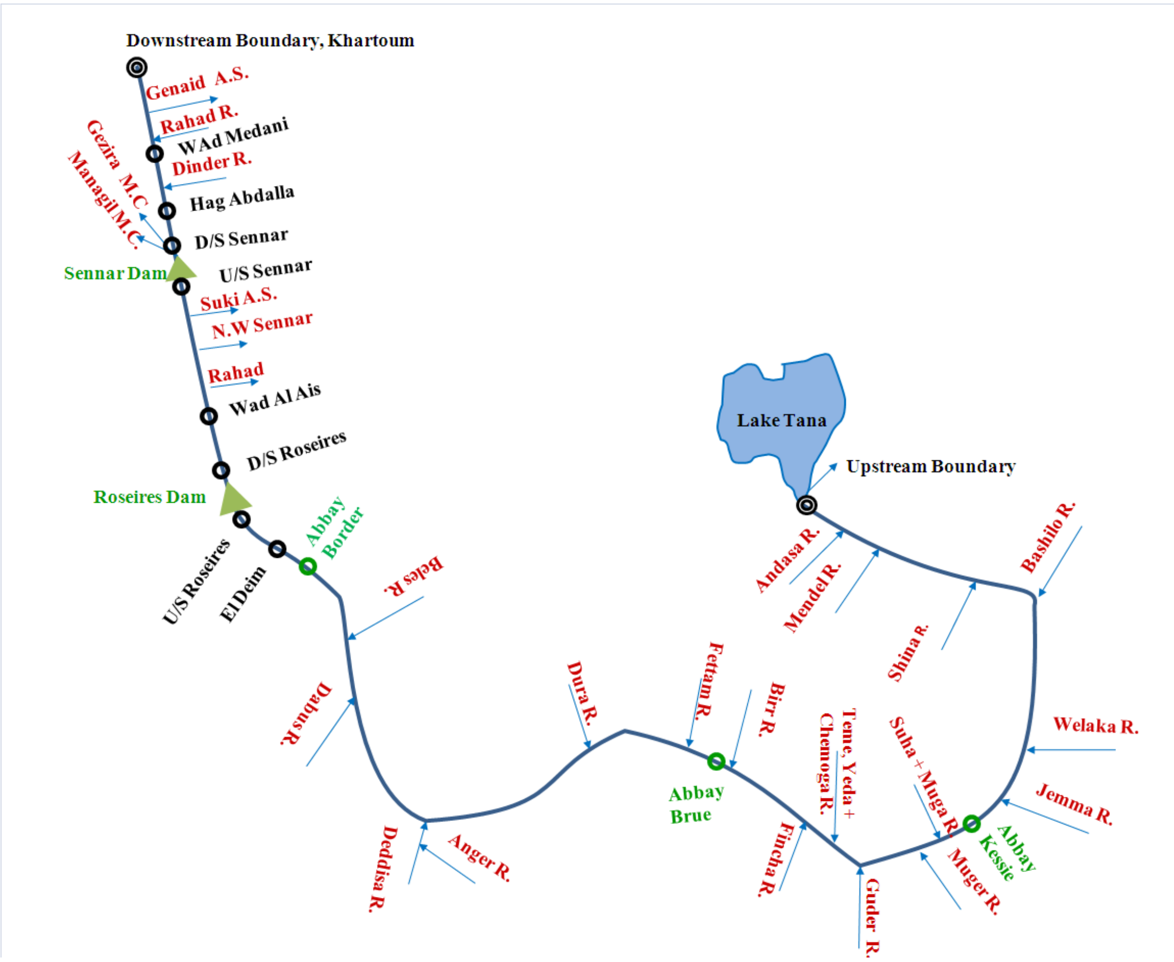


Fig. No.(3.16):Mode Schematization

He calibrated the flow in three reaches as shown in figures (3.17) to (3.22).

First Reach Water level calibration.

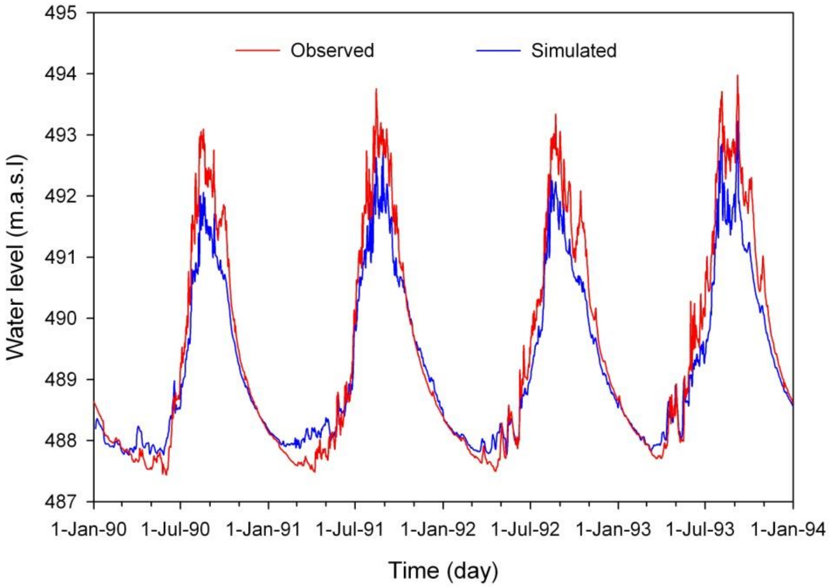


Fig.No.(3.17):El Deim station

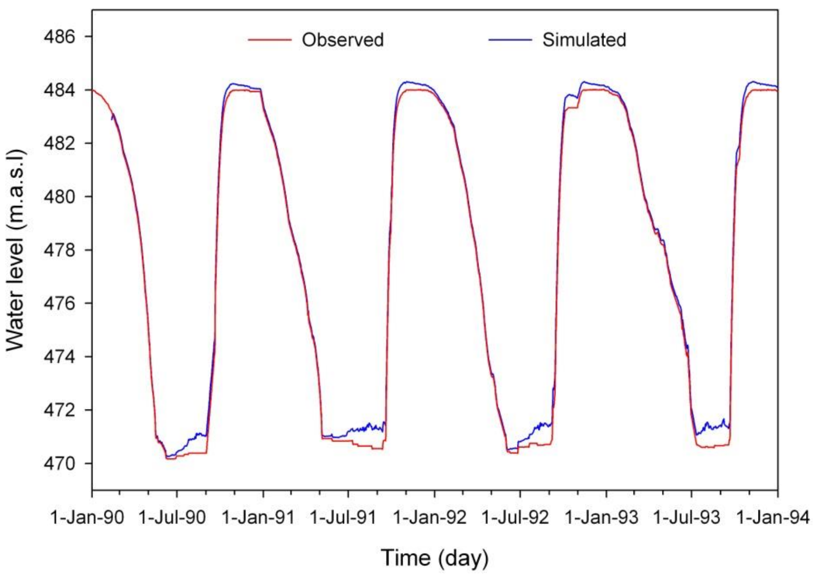


Fig. No.(3.18):Upstream Roseires station

- The model was unable to catch the peak water level
- The model slightly overestimated the water level during the low flows
- The model slightly overestimated the water level during the high flows (low water levels)
- Second Reach Water level calibration

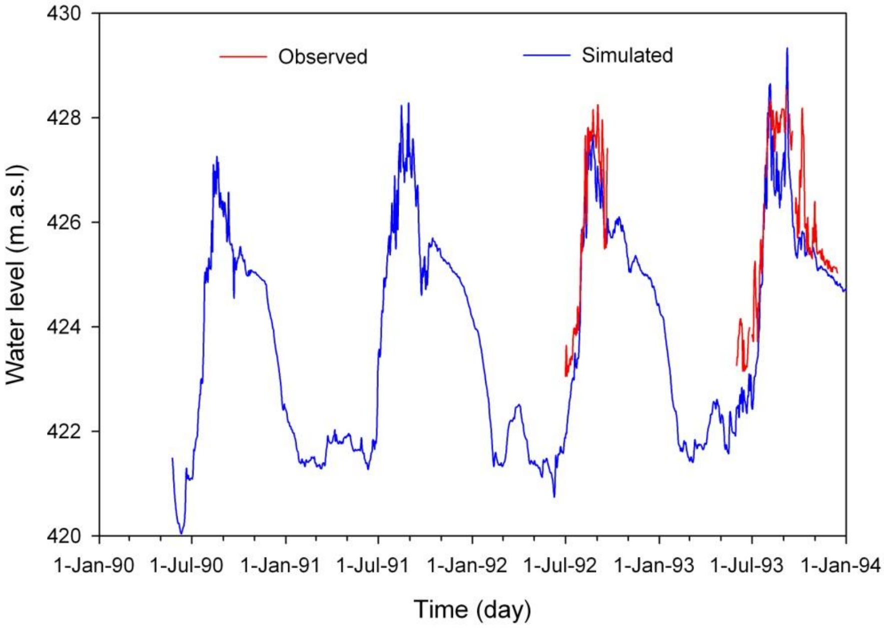


Fig.No.(3.19):Wad Al Ais station

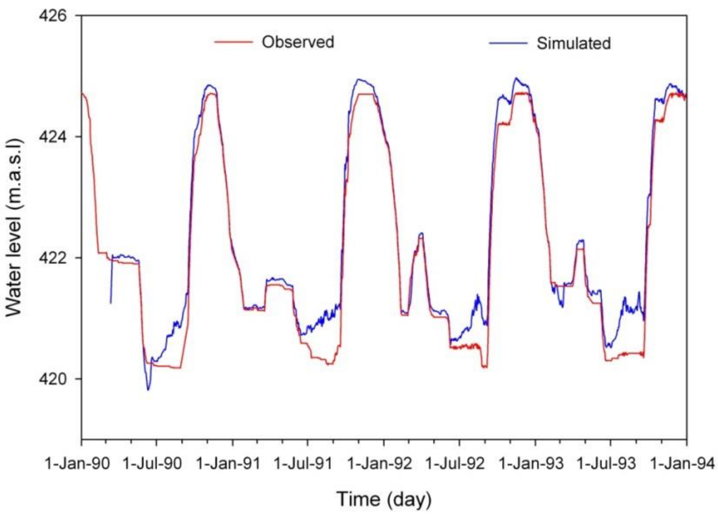
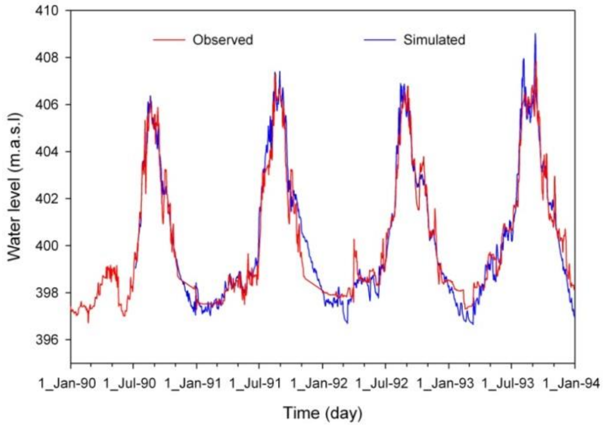
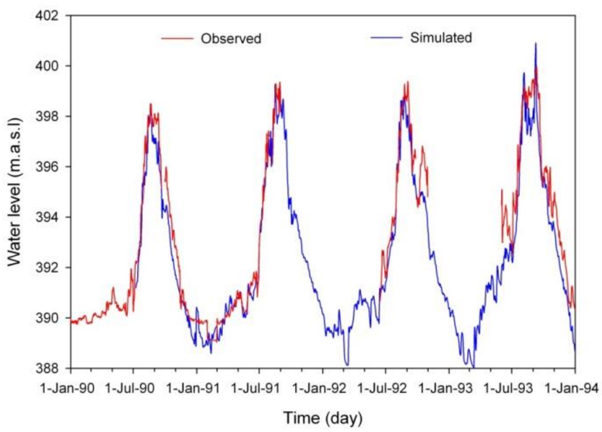


Fig.No.(3.20):Upstream Sennar station

- The model was able to catch the peak water level.
- The model slightly overestimated the water level during the high flows (low water levels)
- No measurement during low flows
- Third Reach Water level calibration.



<u>Fig.No.(3.21):Hag Abdalla station</u>

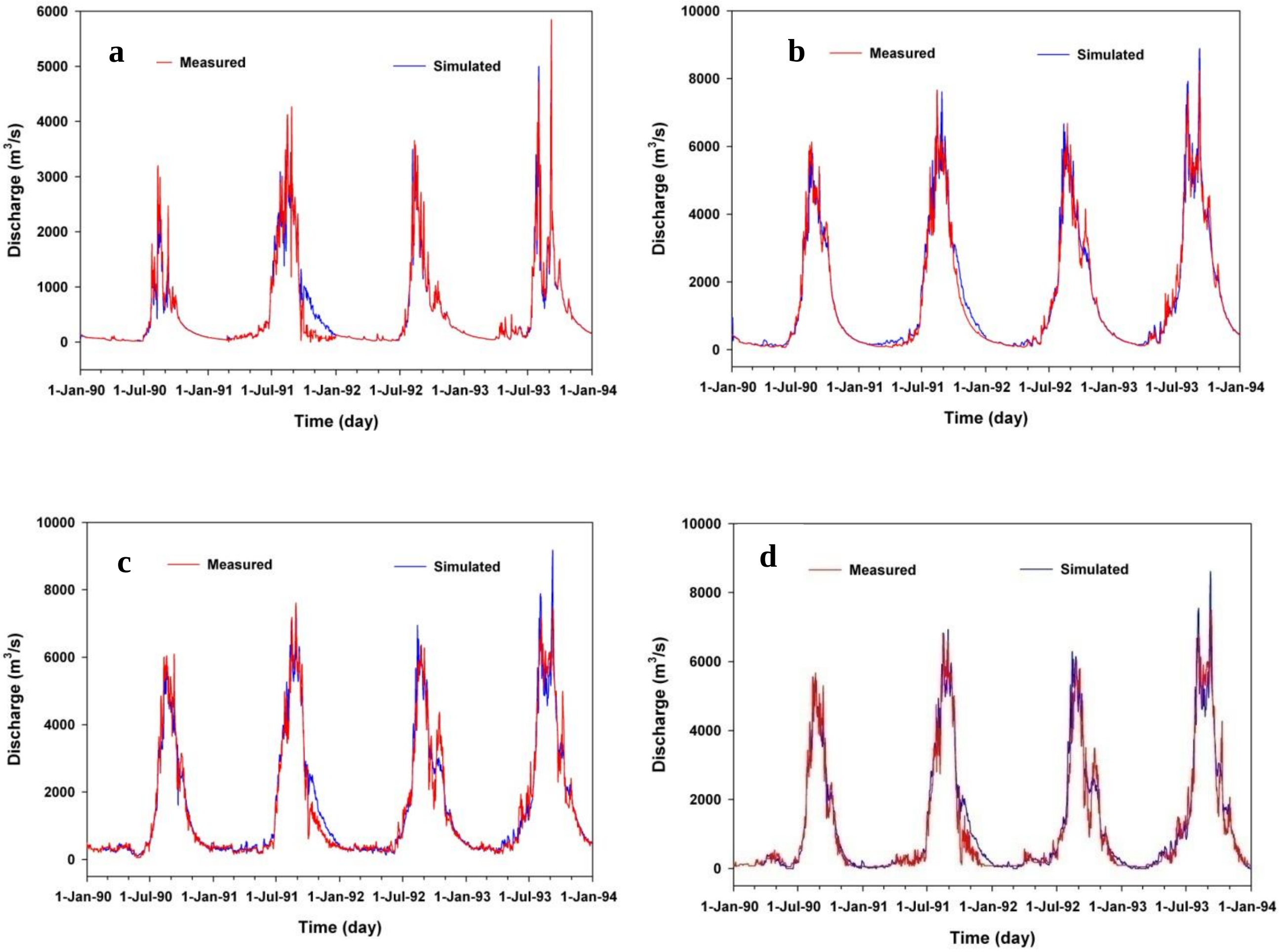


<u>Fig.No.(3.22):Wad Medani Station</u>

- ❑ The model was able to catch the water level during the high flows.
- ❑ The model was able to catch the peak water level.
- ❑ The model slightly underestimated the water level during the low flows

Then he conducted a Calibration (flow) at the four locations with the results shown in figures (3.23).

Kessie Bridge **El Deim station**



D/S Sennar Dam **D/S Roseires Dam**

<u>Fig. No.(3.23):Calibration (Flow) Results at The Four Locations</u>

3.5.Rouseires Dam Operation And Maintenance Difficulties:-

The Rosaries dam, which falls on the Blue Nile about 630 km upstream of Khartoum, is a 1000 m long and 68 m high concrete dam with the crest at 482.2 m. The dam was completed in 1966 with an initial capacity of 3.3 milliardsm3 at level 480 m to be used for irrigation water supply as first priority, and hydropower generation comes secondly. The dam contains five deep sluices and a gated spillway, consisting of seven units, with a maximum discharge capacity of 16.500 m3/s at level 480 m, the hydro-electrical potential abounds to 212 MW, figure (3.24) shows the location of the reservoir together with Eddiem station and figure (3.25) shows the longitudinal profile along the Blue Nile.

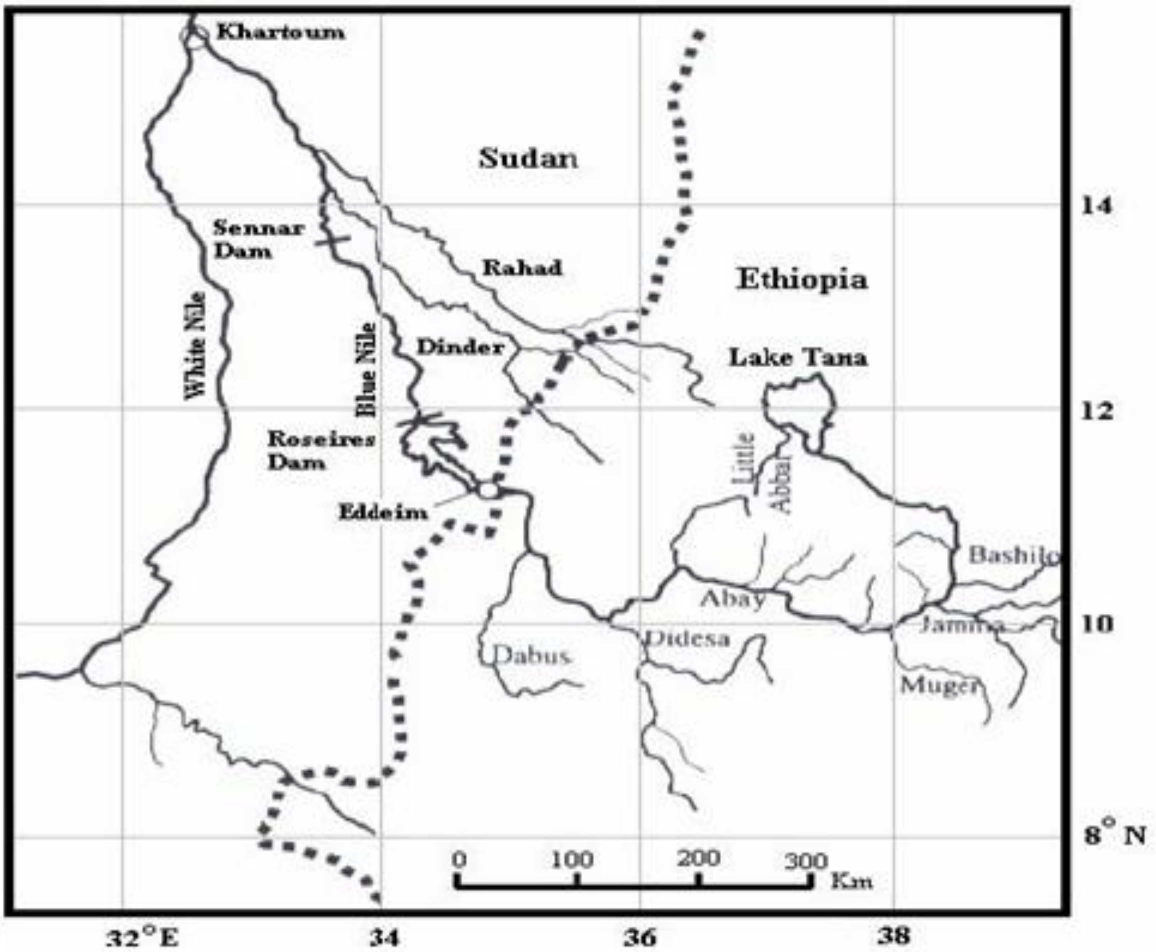


Fig. No. (3.24): Location of the Rosaries Dam and Eddiem Station Within Blue Nile River

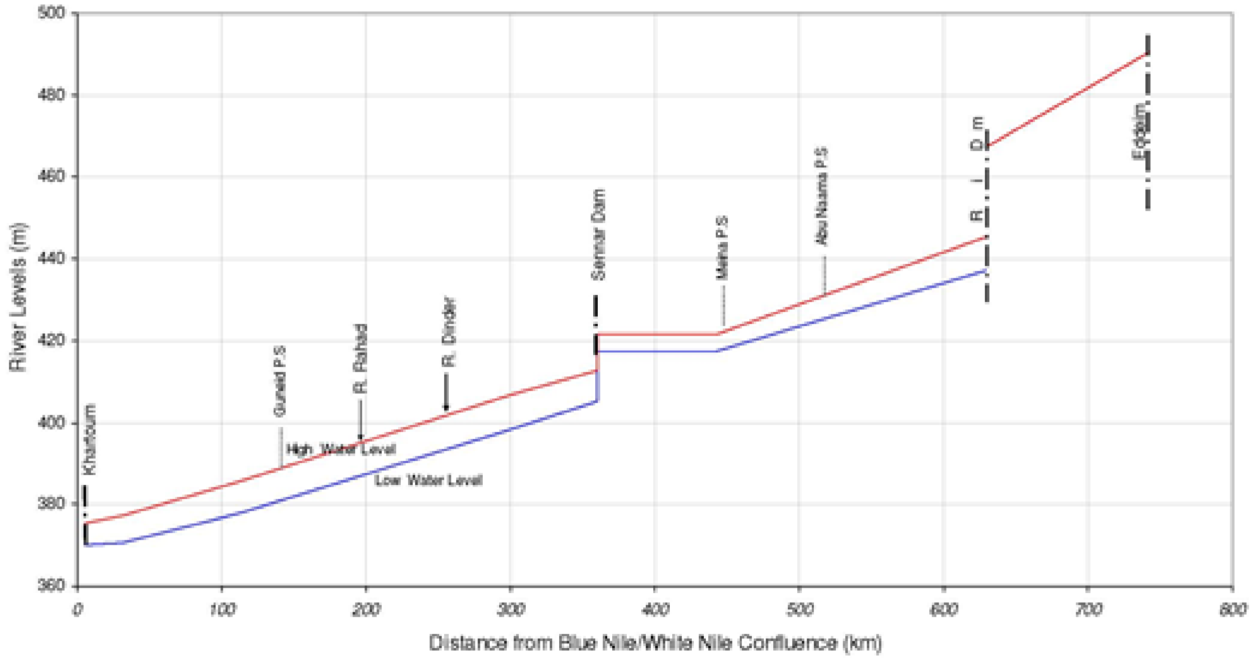


Fig. No. (3.25):Longitudinal Profile of the Blue Nile River From Rosaries Dam to Khartoum City

The design of the present dam made provision for a subsequent increase in height of ten meters. The limitation on the extent of the heightening was dictated by the necessity to avoid creating adverse effects in Ethiopia. The foundations and first few meters of buttress deepening required for the ultimate height were provided for in the initial construction and the sections of the concrete dam adjoining the earth dam were built to the ultimate profile. Dam heightening is accompanied with extension in earth embankment bringing the total length of the dam to about 25km.

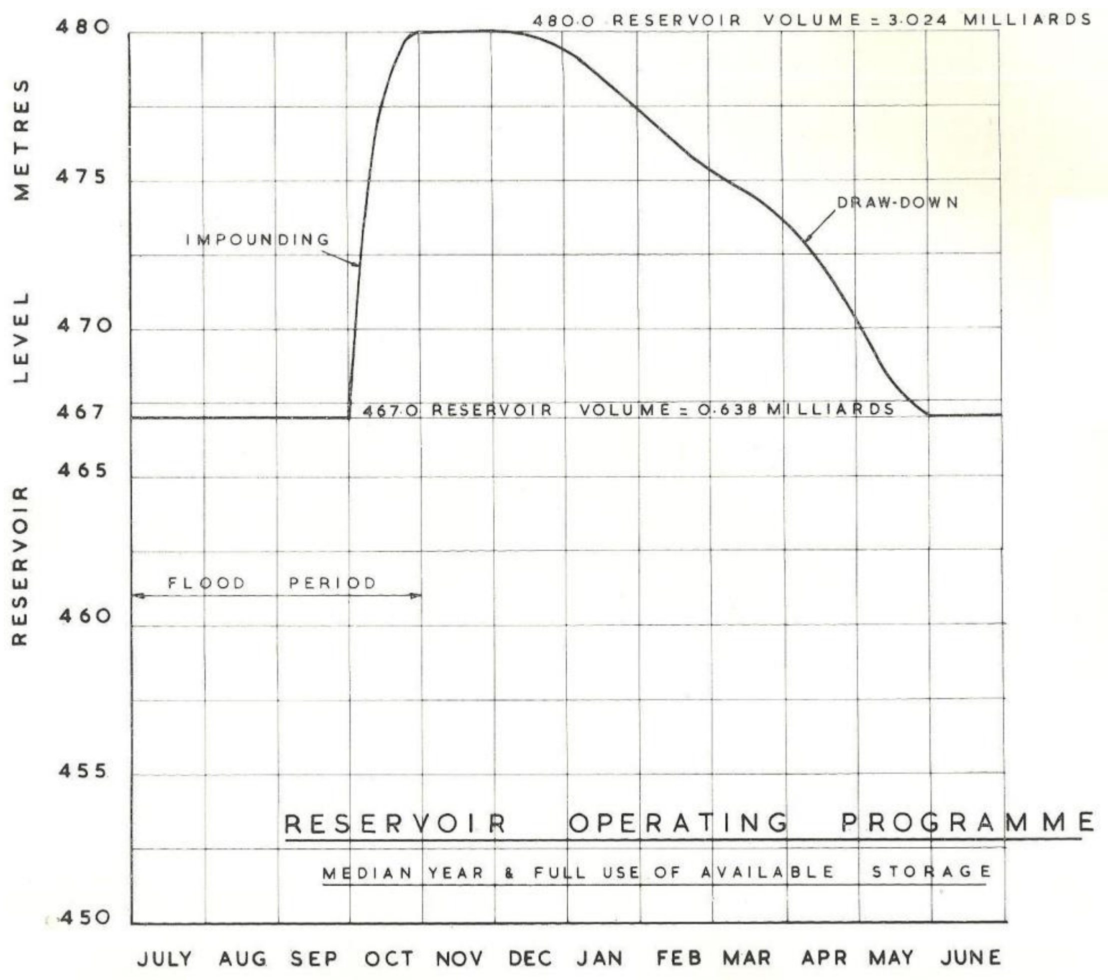
The volume of the reservoir was originally 3.0 milliard cubic meters at a level of 480 m with a surface area of about 290,000 km2 extending over a length of 75 km. The storage capacity has considerably been affected by siltation and is now about 30 percent less. The recent capacity is estimated to be about 2.1 milliard cubic meters. A special operation strategy, maintaining low reservoir level and high flow velocities during the passage of the flood, is applied to reduce the siltation (UNESCO Chair in Water Resources, 2011).

The main inflow to the reservoir is monitored at Eddiem Gauging Station 102 kms south-east of Rosaries dam on the Ethiopian Sudanese border. This allows for an intervening catchment of 14,578 km2 which is totally un-gauged. The rainfall over the reservoir lake is monitored at Damazin Gauging Stations. The significance of the contribution of the intervening catchment and the direct rainfall over the reservoir is alleged to be negligible.

Operation of reservoirs depends on rules set for that purpose, which is mainly based on water balance of the system among other factors. Such rules are rarely revised during the life time of the reservoirs and Rosaries is not an exception (Ahmad, Q.H.2003).During its lifetime, the reservoir suffered from serious sedimentation to the limit that, its present capacity is less than 2.0 km3. Operation of the dam is maintained closely with Sennar Dam, according to the operation policy. Discharge in Rosaries Dam through gates (deep sluices and spillways) is computed by the dam operation engineer using flow charts prepared from a physical model before dam construction in 1965.

The deep sluices with sill levels of 435.5 m alms are used to pass the main volume of the flood, and to flush the sediment as much as possible. The deep sluices are always closed in low flow seasons to reduce the possibility of cavitations 'damage on the downstream apron at low tail water levels. The spillways with sills at 463.7 m amsl are used to pass the peak of major floods and also, if necessary, to augment the downstream flows in the low flow season to provide the releases required for irrigation and other downstream water utilizations. At the extremities of the concrete section on each bank provision has been made for gravity supplies for future irrigation canals. On the west bank the head works would supply a future scheme in the Kenana area and on the east bank; supplies could be made for a future irrigation scheme in the Rahad, Dinder or Rosaries areas.

The maximum design retention level of the reservoir is 480.0 m amsl.; however a study in 1973 (Sir Alexander Gibb & Partners, 1973.) came to the conclusion that the reservoir could probably be filled to 481m each year for a limited number of years without incurring an unacceptable degree of risk. Nevertheless the operating range of the reservoir is kept between 467 m and 480 m. The live storage volume between these two levels is about 2386 million cubic meters, which is released for use downstream between November and June as shown in figure (3.26) which shows the monthly variation of Roseires reservoir operating level. The reservoir is held at the lower level through the flood season to minimize sediment deposition and is filled to 480 m on the falling flood when the sediment load is much lower. A recent bathymetric survey has indicated that since the dam was built some 1.408 milliard cubic meter of the sediment has been deposited in the reservoir, this definitely affected the storage characteristics of the reservoir.



<u>Fig. No. (3.26):Monthly Variation of Roseires Reservoir Operating Level</u>

3.5.1Sennar Dam and Reservoir:-

Sennar Dam is a dam on the Blue Nile near the town of Sennar, Sudan as shown in figure (3.27). It was built in 1925 by the British engineer, desert explorer and adventurer, Stephen "Roy" Sherlock, under the direction of Weetman Pearson. The dam is 3025 meters long, with a maximum height of 40 meters. It provides water for crop irrigation in the Al Gezira region. Once completed, the Dam was constructed on the Blue Nile River at Sennar Town some 300 kilometers south Khartoum the capital of Sudan. Construction works began in 1920 and completed on 1925/1926.Its main object is to provide irrigation water for the giant Gezira agricultural scheme to help irrigates cotton and other crops. It provides irrigation water for about 60 percent of the country`s agricultural production. In addition it helps in providing the hydro-electric power. The dam extended over about 3.10m, i.e. over 3 km (3,025 meters) in length, and of a maximum height of 40-45m (Dam Implementation Unit, 2012).

Fig. No. (3.27):Location of Sennar Dam Within Blue Nile River

3.5.2General Principles of Regulation:-

The working of the two reservoirs in accordance with those rules must at all times be closely coordinated together. With this object in view, the working of them both will be controlled and supervised by the assistant under-secretary (Dams), unless otherwise arranged by the under-secretary, ministry of irrigation. In other respects, however the resident engineers at Roseires and at Sennar will each remain responsible in the usual way for the working and the administration of his division.

Water is stored in Sennar reservoir for the following purposes:

1. For subsequent use in supplementing the natural flows of the river:

a) To meet the requirements of irrigation from the reaches of the river upstream or downstream of Sennar Dam;
b) To maintain minimum flow in the river downstream of Gunned, of 5 million m3 per day when possible, and at least 3.5 million m3/day;
c) To maintain through Sennar dam power station so far as may be possible the flows of water requisite for the generation of power. For this purposes, the flow passed downstream from Sennar Dam in the irrigation season should normally be not less than 8 million m3 per day.

2. To ensure command of the Gezira and Managil canals by gravity flow, at least throughout the season of irrigation , and at other times of the year so far as may be possible.
3. To maintain suitable heads on Sennar Dam for the generation of power, so far as may be found possible.

Water is stored in Roseires reservoir for the following purposes; for subsequent use in contributing to purpose (1) to (3) above.

4. For subsequent use in supplementing the natural flows of the river:
5. Meet the requirements of irrigation from the reach of the river upstream of Roseires Dam. To maintain through Roseires Dam power station so far as may be found possible the rate of flow requisite for the generation of power.
6. To ensure command of the east bank main canal by gravity flow, at least through out the season of irrigation, and at other times of the year so far as may be possible.
7. To maintain suitable heads on Roseires Dam for the generation of power, so far as may be possible.

In order to restrict the deposit of sediment in the reservoirs during the period of high flood in July and August, at this time no more water is kept stored in Sennar reservoir than is necessary for purposes (2) and (3), and no more Roseires reservoirs than is necessary for purpose (6) and (7). The subsequent fillings of both reservoirs, as the river flows diminish after the flood, should be begun at dates no earlier than are necessary to ensure, in the actual condition of each year, that sufficient water surplus to current requirements will filling before the period of shortage begins. The programs of filling are designed accordingly on stored water during the period of shortage; the object should be to use water from either reservoir as may be most advantageous to meet the needs of the particular season, as foreseen beforehand. The rate of lowering of either reservoir should at no time exceed certain limits, defined in their respective “operation manuals”, intended to avoid the risk of slips in the embankment sections of the dams (MOIWRS, 1968).

3.5.3Data Requirement and Data Collection:

Analysis of the system requires upper boundary condition that specifies the inflow to the system which is considered as lateral inflow hydrograph. So for this study Eddiem observed flow is used as upper boundary. The downstream boundary condition at Khartoum as the downstream end of the study area is set as normal depth. The effects of Rahad and Dindir were accounted for by lateral inflows at their respective confluences with the Blue Nile River. The required available data for each of these stations were collected for the Ministry of water resources and electricity.

Accoding to the study in 1973 (Sir Alexander Gibb & Partners, 1973.) which came to the conclusion that Roseiris reservoir could probably be filled to 481m each year for a limited number of years without incurring an unacceptable degree of risk.The collected data were processed for use in a model calibration. An integral part of the data is shown below in table (3.4).

a. Water level:

In the reservoir there are two observation stations; one is at the upstream of the reservoir (Eddiem station) and other at the dam. The water level, discharge and sediment concentration are measured daily. The water level data used in this study are from the dam station. From the 1990 to 2014, was shown in figure (3.28).

Table (3.4): Field Data Collected in This Study

Covered period	Sampling frequency	Sample site	Parameter
From 1995 to 2014	Daily	Roseires Dam	Water levels
From 2003 to 2013	Daily	Inflow from Eddiem station Outflow from dam station	River Discharges
From 1992 to 2007	Daily	Eddiem station	Sediment sample
From 1992 to 2007	various	Roseires reservoir area	Bathymetric surveys

b. River Discharges:

Daily discharges levels are recorded by Ministry of Irrigation & Water resources at the Blue Nile River at one a day. The discharge of Eldeim station and dam station from 2003 to 2013 are used in the present research .The discharge is shown in figure (3.29) and figure (3.30).

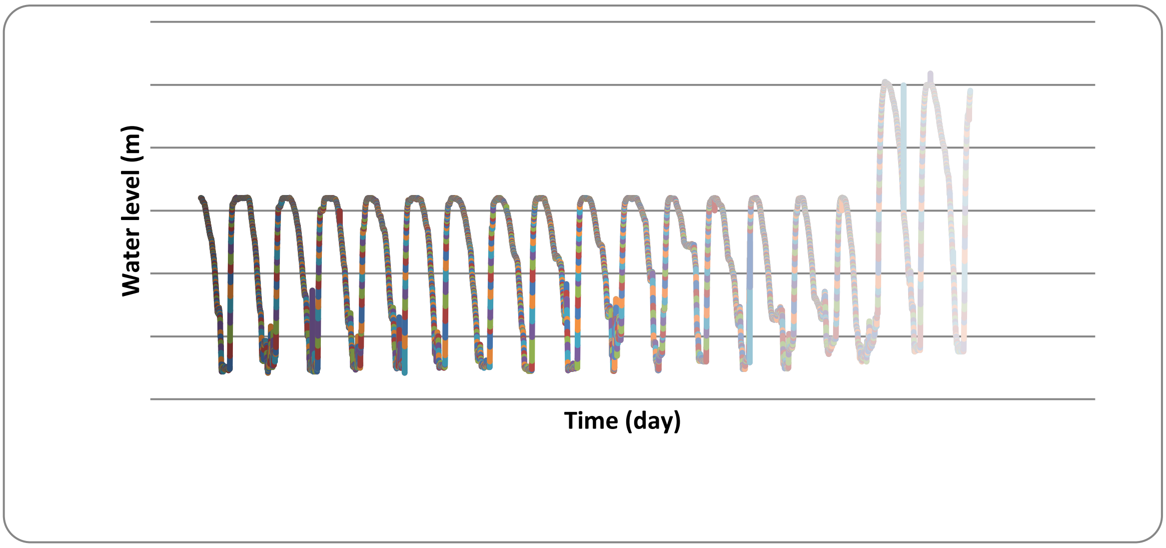


Fig. No. (3.28): Water Level at Roseires Dam

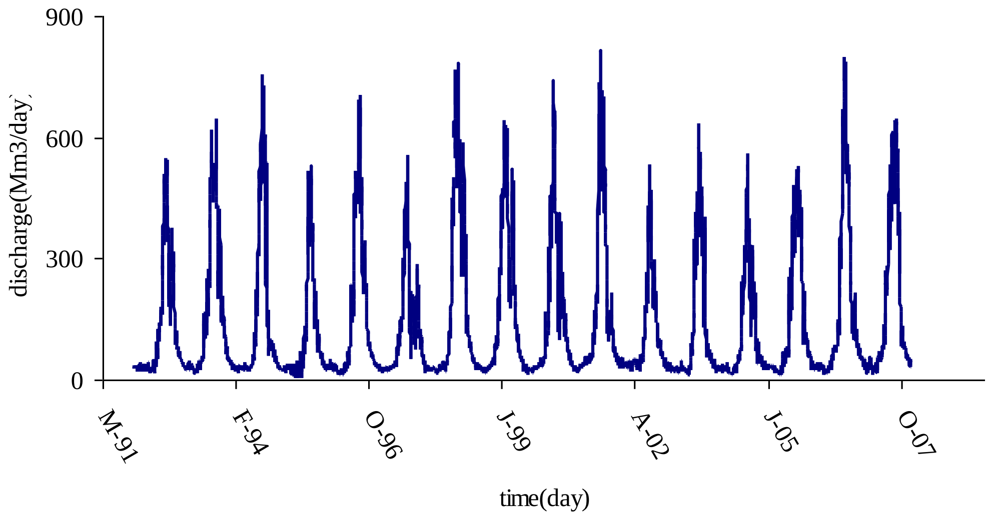


Fig. No. (3.29):Outflow at Roseires Dam

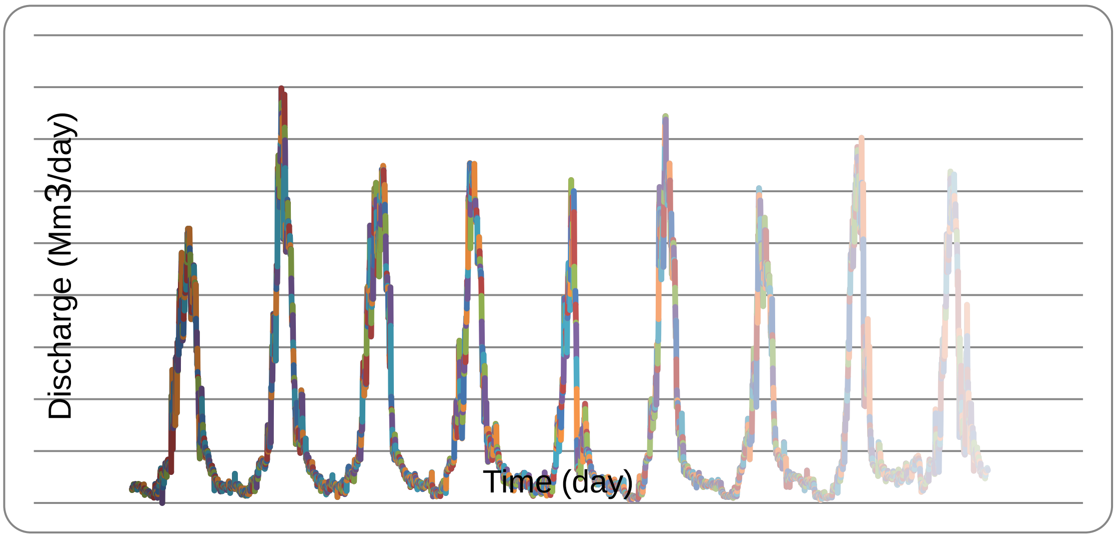


Fig. No. (3.30): Inflow at Eddiem Station

c. Sediment Data:-

Daily sediment concentration is recorded by the Ministry of Irrigation and Water resources at the different stations at the Blue Nile River once a day. In this study the sediment concentration and sediment discharge are measured from the Eddiem station at the upstream.

d. Bathymetric surveys:-

Several data sets are taken in to account to represent as currently as possible the topographic and bathymetric features around the Roseires Reservoir and its vicinities. The bathymetric data collected by the Ministry of Irrigation and Water resources consisted of 35 cross-sections covering most of the reservoir. Several surveys on the bathymetry are available for present research as shown in figure (3.31).The modelled bathymetry is obtained by digitizing the available maps which covers most part of the whole reservoir.

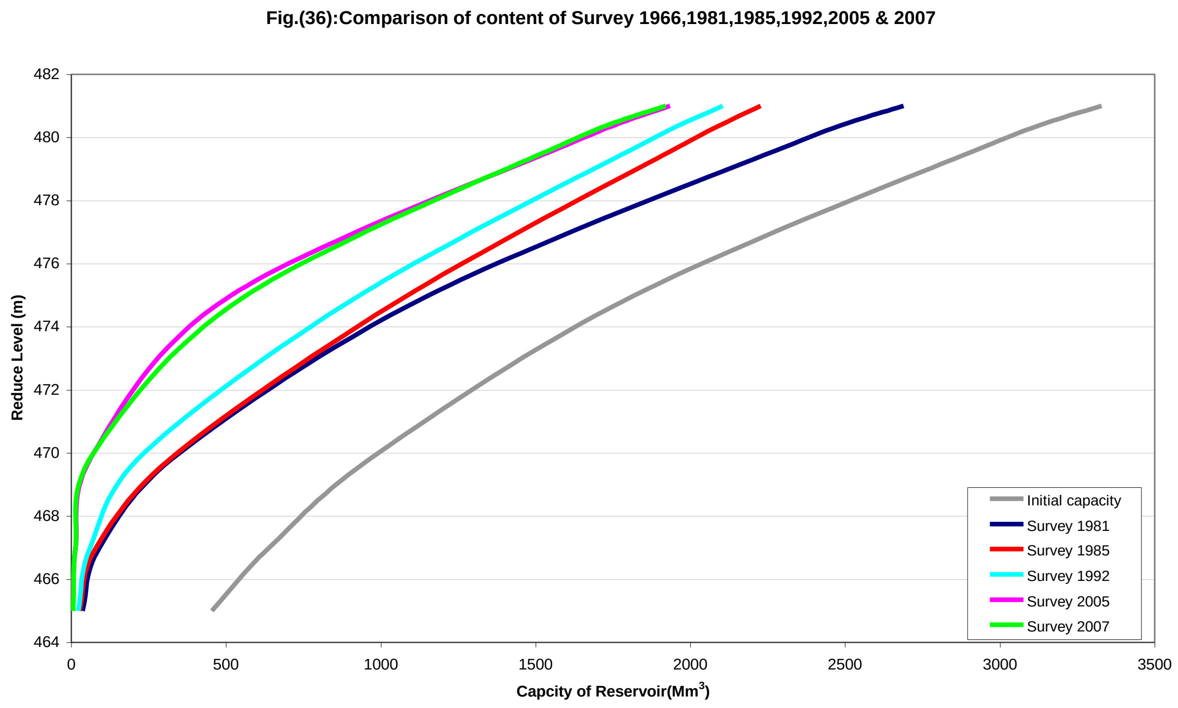


Fig. No. (3.31): Reservoir Capacity From Different Surveys at Roseires Reservoir

The data mentioned above was used to simulate Roseires Reservoir to investigate morphological process. It was enough to make the simulation.

CHAPTER FOUR

4. DATA COLLECTION AND ANALYSIS

4.1. Hydraulic Theory:-

River engineering constructions are very expensive. At the first stage of design resort must be made to theoretical approaches. If the whole design or part of it can not be predicted by theory, it is accordingly advisable to study the performance of the whole or part of the prototype by means of a hydraulic model. A hydraulic model is a precisian device for experimental investigation of a hydro mechanical phenomemon.It has contributed significantly in the design of hydraulic structures, training of rivers and basic hydraulic research. It is common practice to construct hydraulic model research to verify or modify the design of such works, particularly in study of complex flow phenomenon, for which theory is either not available or not complete. In this case the model research becomes more an art than a science.

Generally hydraulic models are of two types. Those designed to solve a special hydraulic problem as for example a definite reach of a known river, and those designed for research for establishing hydraulic laws applicable to special problems within the field of river engineering. The first type produces qualitative results only applicable to known prototype river, while the second type produces quantitative results applicable to any prototype involving the same special problem with the same hydraulic laws. Unfortunately, the former can not be applied, because a large hydraulic laboratory is needed which is not available. Similarly the latter can not be applied because of lack of sophisticated equipments usually needed in such case. However simple conceptual mathematical models using the standing computers strong SPSS techniques can be applied.

❖ 4.2. SPSS Theory and Application:-

Several mathematical models have been developed to predict the effects of hydrological and morphological events. Since field data for such events are very difficult to obtain, there is a vital need for data to verify these mathematical models.

As defined by Wikipedia Encyclopedia SPSS Statistics is a software package used for statistical analysis. The current versions are officially named IBM SPSS Statistics. The software name originally stood for Statistical Package for the Social Sciences (SPSS). SPSS is a widely used program for statistical analysis in social science.

The many features of SPSS Statistics are accessible via pull-down menus or can be programmed with a proprietary 4GL command syntax language. Command syntax programming has the benefits of reproducibility, simplifying repetitive tasks, and handling complex data manipulations and analyses.

SPSS Statistics places constraints on internal file structure, data types, data processing, and matching files, which together considerably simplify programming. SPSS datasets have a two-dimensional table structure, where the rows typically represent cases (such as individuals or households) and the columns represent measurements (such as age, sex, or household income). Only two data types are defined: numeric and text (or "string"). All data processing occurs sequentially case-by-case through the file (dataset).

Files can be matched one-to-one and one-to-many, but not many-to-many. In addition to that cases-by-variables structure and processing, there is a separate Matrix session where one can process data as matrices using matrix and linear algebra operations.

The graphical user interface has two views which can be togged by clicking on one of the two tabs in the bottom left of the SPSS Statistics window. The

'Data View' shows a spreadsheet view of the cases (rows) and variables (columns).

In this chapter, using this technique the details of SPSS Models facilities and analysis procedures are applied. A complete set of data on hydrological and morphological aspects events results are analyzed and presented in the form of graphs and tables.

4.2.1Discharge and Power Analysis:-

As shown in appendix (A) in figures (A.F.1), to (A.F.12), there is always linear relationship between discharge and power with a high correlation coefficient during the whole year. Its highest value is in April of 0.965, and lowest value in November of 0.244.The maximum and minimum discharges and the maximum and minimum generated power during the whole year from January to December are clearly shown.

4.2.2Discharge and Sediment Analysis:-

The correlation of discharge and sediment from June to October gave as in appendix (B), in figures (B.F.1) to (B.F.5) gave

R^2 ranging from 0. 80016 to 0. 694 which is a fair fit.

4.2.3Discharge and Water Requirement Analysis:-

The correlation of discharge and water requirements analysis from January to December gave as in appendix (C), in figures (C.F.1) to (C.F.12) gave:-

R^2 ranging from 0. 606 to 0. 323 which is a poor and unreliasble fit.

4.3. Use of Dimensional Analysis:-

Physical laws are expressed in terms of certain characteristic parameters governing the behavior of the phenomenon completely. These parameters will produce quantitative properties of the phenomenon. Using all parameters including those which are not dependent or ineffective will lead to wrong theorizing and erroneous results. However it is quite safe at the first stage of the formulation to include all parameters related to the phenomenon and then discard those which prove to be not effective. Hence assuming the remaining reduced number of the parameters to be associated with the phenomenon concerned, the method of dimensional analysis can be used to describe the phenomenon by one single equation.

A property (A), of any phenomenon can be expressed in terms of all or some of the (n), characteristic parameters of the phenomenon, in a functional relation of the form:-

$$A = \int_A (x_1, x_2, x_3 \ldots\ldots x_n) \text{-------} (4.1)$$

By definition various properties $A_1, A_2, A_3, A_4, \ldots\ldots\ldots A_n$ of the same phenomenon, are various functions of the (n) characteristic parameters of the phenomenon. It is not necessary that all the (n) characteristic parameters of the phenomenon are functions of every property of the phenomenon. These characteristic parameters are each expressible in terms of the basic dimensions Mass M ,Length L , Time T ……etc.

It is well known that correct expression of a natural law satisfies dimensional homogeneity. The dimensional parameters are used to form the so called "Complete set of dimensionless products." According to Backingham π

theorem, which is based on dimensional homogeneity, the (n) dimensional parameters will have a general equation expressed as a function of $(n - m)$,dimensionless π terms, where (m) being the basic dimensions in terms of which the (n) parameters are given. Each dimensionless π term will have

$(m+1)$ Parameters of which only one need be changed .The dimensionless version of equation (4.1) is:-

$$\pi_A = \int_A \left(\pi_1, \pi_2, \pi_3 ..... \pi_{(n-m)}\right) --- (4.2)$$

The repeated variables must be independent of each other. If non of the selected three repeated parameters can be formed by power product of the other two, and power product formed by combination of the three parameters does not form a dimensionless product the three basic parameters are said to be independent. In other words the arrangements of these three repeated variables must contain a non zero determinant of order $m = r = 3$.These $m = r$ repeated parameters are combined with the other remaining $(N = n - r = n - m)$ characteristic parameters of the phenomenon under study to form (N)dimensionless groups. The (N)dimensionless groups so obtained are independent because each of them contains one independent characteristic parameter that does not appear in the remaining $(N-1)$ dimensionless groups. The arrangements are such that the influence of the repeated three basic parameters $(m)$is not taken into account by any of the $(N = n - m)$ independent dimensionless groups because they appear in every dimensionless group. This technique is desirable because when one of the dimensionless groups is varied the others remain constant.

The selection of effective dimensionless groups depends on understanding the mechanism of the phenomenon under study (Asegdew Gashaw, 2013).Howevever there are some cases were the existing stage of knowledge is inadequate to indicate the significant dimensionless group. In this case even a crude theory of the mechanism of the phenomenon under study may reveal the action of the important parameters. Constant parameters such as the acceleration of gravity, density, dynamic viscosity …etc, are essential when they combine with others to form influential and significant dimensionless groups. Indeed there is frequent use of variables that influence motion. They are:-

- ❖ Geometrical ones such as length, width, diameter, and depth.
- ❖ Kynametic and dynamic such as velocity and pressure gradient.
- ❖ Physical properties such as density, and specific weight.

Interpretation of these variables reveals that the well known standard dimensionless products of Reynolds No. , Froude No., etc, can readily be formulated. Furthermore the ratio of any two variables of the same kind associated with a phenomenon such as length, and width or length and depth form dimensionless groups.

Considering flow in the vicinity of a dam the main flow separates at the converge entrance of the dam gates and diverges downstream. As shown in the simple figure (4.1).It defines the shape and geometry of flow in the vicinity of the dam.

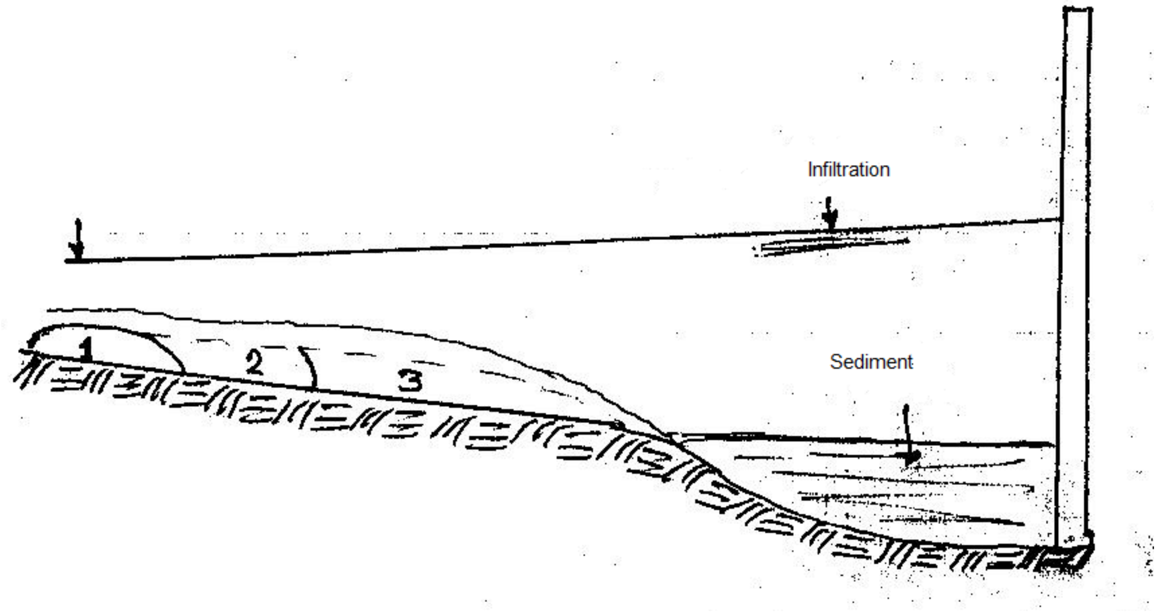


Fig. No.(4.1):Shape and Geometry of Flow in The Vicinity of a Dam

The parameters involved can be classified into three main categories.

a) Geometrical Parameters;-

I. Width of the channel upstream the dam B

II. Agricultural area downstream A

III. Channel bed slope i

b) Flow Parameters:-

I. Upstream approaching velocity V

II. Channel Water depth upstream the dam D

III. Sour depth downstream d_S

IV. Acceleration of gravity g

V. Density of water ρ

VI. Dynamic viscosity of water μ

VII. Discharge Q

VIII. Power Generated Pow

c) Sediment Parameters:-

I. Medium grain of sediment d_{50}

II. Standard deviation σ

III. Sub wt sediment γ_{Sub}

σ , and γ_{Sub} can be defined as follows:-

$$\sigma = \frac{1}{2}\left(\frac{d_{84}}{d_{60}} + \frac{d_{50}}{d_{16}}\right) --- (4.3)$$

$$\gamma_{Sub} = g\left(\rho_s - \rho_\omega\right) --- (4.4)$$

Where:-

d_{84} = Grain size of which 84 % is finer.

d_{60} = Grain size of which 60 % is finer

d_{50} = Grain size of which 50 % is finer.

d_{16} = Grain size of which 16 % is finer.

ρ_s = Density of sediment material.

ρ_ω = Density of water.

Furthermore, in any study involving flow around any obstruction, the effects of both shear stress and fall velocity are not to be ignored. These two parameters are usually used to compute dimensions of physical models. The shear stress can be expressed as:-

$$\tau = \int(\rho g D S) --- (4.5)$$

Similarly the fall velocity can be expressed as:-

$$W_s = \int(d_{50}, g, \mu, \gamma_{sub}, C_D) --- (4.6)$$

Also expressed as :-

$$W_s = \sqrt{\frac{4(s-1)gd}{3C_D}}$$

Where:-

C_D = Drag coefficient.

The slope of the channel i, the standard deviation σ,are dimensionless. To obtain dimensionless groups from the remaining parameters, the dimension of each parameter (M, L, T) are displayed in matrix form .Each column consists of the exponent in the dimensional expression for the corresponding parameter as given in the matrix form table (4.1).

Table No.(4.1): Matrix Form of Dimensional Parameters

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
	B	A	V	D	d_s	g	ρ	μ	Q	P	d_{50}	Q_s	γ_s	τ	W_s
M	0	0	0	0	0	0	1	1	0	1	0	1	1	1	0
L	1	2	1	1	1	1	-3	-1	3	2	1	1	-2	-1	1
T	0	0	-1	0	0	-2	0	-1	-1	-3	0	-3	-2	-2	-1

The above matrix is (3×15) matrix of rank (3).The number of the dimensionless groups is the number of the parameters (n),minus the rank of the matrix $(r=3)$.

The number of the dimensionless π terms is –

$$(15-3=12)$$

It is necessary to obey the matrix rule to insure that the above (3×15) matrix has rank (3).The matrix has a determinant of (3), columns and (3) rows. If the determent has an extension value greater than zero, the rule of the matrix to be (3),will be satisfied. Choozing $\gamma_s, \tau, and\ W_s$ as the selected deteminent its value is calculated as follows as given in table (4.2),of the determinant taken from the matrix form of table (4.1).

<u>Table No.(4.2): Determinant Taken From The Matrix Table (4.1).</u>

1	1	0
-2	-1	1
-2	-2	-1

$$=\Delta=1\times\left[(-1\times-1)-(-2\times1)\right]=1\times\left[(1)-(-2)\right]=3$$

From the matrix showing the parameters, the homogenous linear equations, whose coefficiets,are number of the rows of the matrix can

$$k_1+2k_2+k_3+k_4+k_5+k_6-3k_7-k_8+3k_9+2k_{10}+k_{11}+k_{12}-2k_{13}-k_{14}+k_{15}=0---(4.8)---[L]---(b)$$

$$k_7+k_8+k_{10}+k_{12}+k_{13}+k_{14}=0---(4.7)---[M]---(a) \quad -k_3-2k_6-k_8-k_9$$

Again

choosing $\gamma_s, \tau, and\ W_s$ as the repeating variables, and. solving for their coefficients $(k_{13}, k_{14}, and\ k_{15})$ in terms of the other $ks(k_1\ to\ \mathrm{k}_{12})$:-

Gave the solution:-

$$K_{13} = K_1 + 2K_2 + K_4 + K_5 - K_6 + K_8 + 2K_9 + 2K_{10} + K_{11} + K_{12}$$

$$k_{14} = -k_1 - 2k_2 - k_4 - k_5 + k_6 - k_7 - 2k_8 - 2k_9 - 3k_{10} - k_{11} - 2k_{12}$$

$$k_{15} = -k_3 - 2k_6 + 2k_7 + k_8 - k_9 - k_{10} - k_{12}$$

GhsSubstituting these values in the matrix give the solution in table (4.3). Hence as shown in table (4.3), the twelve (12), dimensionless groups are calculated as given below:-

$$\pi_1 = \frac{B\,\gamma_s}{\tau} \qquad \pi_2 = \frac{A\,\gamma_s^2}{\tau^2} \qquad \pi_3 = \frac{V}{W_s} \qquad \pi_4 = \frac{D\,\gamma_s}{\tau}$$

$$\pi_5 = \frac{\mathrm{d_s}\gamma_s}{\tau} \qquad \pi_6 = \frac{\mathrm{g}\tau}{\gamma_\mathrm{s}\mathrm{W_s^2}} \qquad \pi_7 = \frac{\rho W_s^2}{\tau} \qquad \pi_8 = \frac{\mu\gamma_\mathrm{s}}{\tau^2}\mathrm{W_s}$$

$$\pi_9 = \frac{\mathrm{Q}\,\gamma_s^2}{\tau^2 W_s} \qquad \pi_{10} = \frac{\mathrm{P}\,\gamma_\mathrm{s}^2}{\tau^3\mathrm{W_s}} \qquad \pi_{11} = \frac{d_{50}\gamma_\mathrm{s}}{\tau} \qquad \pi_{12} = \frac{Q_s\gamma_\mathrm{s}}{\tau^2\mathrm{W_s}}$$

Furthermore adding the slope of the channel i ,and the standard deviation σ , which are dimensionless. The total number of the dimensionless groups will be fourteen (14).

$$\pi_{13} = i \qquad \pi_{14} = \sigma$$

These equations can be put in the form of equation (4.2):-

$$\pi_0 = \int_A \left(\frac{B\gamma_s}{\tau}, \frac{A\,\gamma_s^2}{\tau^2}, \frac{V}{W_s}, \frac{D\gamma_s}{\tau}, \frac{d_s\gamma_s}{\tau}, \frac{g\tau}{\gamma_s W_s^2}, \frac{\rho W_s^2}{\tau}, \frac{\mu\gamma_s W_s}{\tau^2}, \frac{Q\gamma_s^2}{\tau^2 W_s}, \frac{P\gamma_s^2}{\tau^3 W_s}, \frac{d_{50}\gamma_s}{\tau}, \frac{Q_s\gamma_s}{\tau^2 W_s}, i, \sigma \right) --- (4.7)$$

The above is the method of calculating a complete set of dimensionless groups of any given set of parameters. Use of standard products can be used instead of calculations. For π to be dimensionless the exponents of M,L,and T must be equal to zero as in the equations of the ks.These ks equations possess an infinite number of solutions. Any values can be assigned to $k_1, k_2, k_3 ..... k_{12}$ and the equations solved for the remaining unknowns. This produces an arbitrary trivial solution, however, it can be anticipated that a complete set of dimensionless groups is always obtained. Such solution is known as linear combination of solutions of ks. To obtain solutions which are linearly independent on each other, the fundamental system of solutions has to be obtained. The 13th, 14th and 15th columns in the matrix of solutions are the coefficients in the equations of $k_{13}, k_{14}, and, k_{15}$. The first twelve (12) columns of the matrix of solutions, consist of zero values except the ones in the principle diagonal. Alternatively the solution can be written by inspection of the ks equations. Examination of the determinant on the right hand side of the matrix indicates that its rank is 3.This constitute a fundamental system of $(n-r)$ solutions. Each raw is a set of dimensionless group. The first variable B occurs only in π_1,the second variable A occurs only in π_2,and so on. The resulting equation is the equation developed by the researcher in order to be able to solve the problems of the study and fulfill the objectives as well.

Table No.(4.3): Dimensionless π Parameters

	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
	B	A	V	D	d_S	g	ρ	μ	Q	P	d_{50}	Q_s	γ_s	τ	W_s
π_1	1	0	0	0	0	0	0	0	0	0	0	0	1	-1	0
π_2	0	1	0	0	0	0	0	0	0	0	0	0	2	-2	0
π_3	0	0	1	0	0	0	0	0	0	0	0	0	0	0	-1
π_4	0	0	0	1	0	0	0	0	0	0	0	0	1	-1	0
π_5	0	0	0	0	1	0	0	0	0	0	0	0	1	-1	0
π_6	0	0	0	0	0	1	0	0	0	0	0	0	-1	1	-2
π_7	0	0	0	0	0	0	1	0	0	0	0	0	0	-1	2
π_8	0	0	0	0	0	0	0	1	0	0	0	0	1	-2	1
π_9	0	0	0	0	0	0	0	0	1	0	0	0	2	-2	-1
π_{10}	0	0	0	0	0	0	0	0	0	1	0	0	2	-3	-1
π_{11}	0	0	0	0	0	0	0	0	0	0	1	0	1	-1	0
π_{12}	0	0	0	0	0	0	0	0	0	0	0	1	1	-2	-1

4.4. Significance of Dimensionless Groups;-

In practice some dimensionless groups are more useful than others. Transformation may bring about an equation:-

$\pi = \int(\pi_1, \pi_2, \pi_3 ..... \pi_n)$; into a more tactable form .This transformation should as much as possible satisfy the researcher desire in obtaining dimensionless groups that are possible to be controlled, while keeping others constant. This can be achieved if in the dimensional matrix, the dependent variable is set first, followed by that which can be regulated easily, followed by the next easiest to regulate and so on. This is difficult because often a few of the variables can not be regulated. When a variable has a negligible influence of the phenomenon under study the dimensionless group containing the variable can be discarded. However if that variable is repeated in more than one dimensionless group it is necessary to change to another complete set of dimensionless groups. The new dimensionless groups must be independent on each others, and equal in number to the original set of groups. From a given set of dimensionless groups as equation (4.7); it is possible to form various complete sets of dimensionless groups. Accordingly equation

(4.7); is transformed to the form:-

$$\frac{Q\gamma_s^2}{\tau^2 W_s}, = \int_A \left(\frac{P\gamma_s^2}{\tau^3 W_s}, \frac{Q_s\gamma_s}{\tau^2 W_s}, \frac{A\,\gamma_s^2}{\tau^2}, \frac{B\gamma_s}{\tau}, \frac{V}{W_s}, \frac{D\gamma_s}{\tau}, \frac{d_s\gamma_s}{\tau}, \frac{g\tau}{\gamma_s W_s^2}, \frac{\rho W_s^2}{\tau}, \frac{\mu\gamma_s W_s}{\tau^2}, \frac{d_{50}\gamma_s}{\tau}, i, \sigma \right) ---(4.8)$$

The dimensionless groups $\frac{B}{D}, i\,\frac{A\tau^4 W^4}{\gamma_s^6}$ define the geometry of the system. The groups $\frac{V}{W_s}, \frac{d_s\gamma_s}{\tau}, \frac{Q\gamma_s^2}{\tau^2 W_s}$;define flow characteristic and pattern within the system. The group $\frac{d_{50}\gamma_s}{\tau}$ $\frac{Q_s\gamma_s}{\tau^2 W_s}$ define sediment motion, while the group $\frac{P\gamma_s^2}{\tau^3 W_s}$ defines the power generation.

Consequently equation (4.8) can be written in any form similar to that of equation (4.2).Hence for examples putting, $\frac{A\tau^4 W^4}{\gamma_s^6}$, or, $\frac{Q_s\gamma_s}{\tau^2 W_s}$,or, $\frac{P\gamma_s^2}{\tau^3 W_s}$,as the subject equation (4.8) becomes;-

$$\frac{A\,\gamma_s^2}{\tau^2}=\int_A\left(\frac{P\gamma_s^2}{\tau^3W_s},\frac{Q_s\gamma_s}{\tau^2W_s},\frac{Q\gamma_s^2}{\tau^2W_s},\frac{B\gamma_s}{\tau},\frac{V}{W_s},\frac{D\gamma_s}{\tau},\frac{d_s\gamma_s}{\tau},\frac{g\tau}{\gamma_sW_s^2},\frac{\rho W_s^2}{\tau},\frac{\mu\gamma_sW_s}{\tau^2},\frac{d_{50}\gamma_s}{\tau},i,\sigma\right)---(4.9)$$

$$\frac{Q_s\gamma_s}{\tau^2W_s}=\int_A\left(\frac{P\gamma_s^2}{\tau^3W_s},\frac{Q\gamma_s^2}{\tau^2W_s},\frac{A\,\gamma_s^2}{\tau^2},\frac{B\gamma_s}{\tau},\frac{V}{W_s},\frac{D\gamma_s}{\tau},\frac{d_s\gamma_s}{\tau},\frac{g\tau}{\gamma_sW_s^2},\frac{\rho W_s^2}{\tau},\frac{\mu\gamma_sW_s}{\tau^2},\frac{d_{50}\gamma_s}{\tau},i,\sigma\right)---(4.10)$$

$$\frac{P\gamma_s^2}{\tau^3W_s}=\int_A\left(\frac{Q\gamma_s^2}{\tau^2W_s},\frac{Q_s\gamma_s}{\tau^2W_s},\frac{A\,\gamma_s^2}{\tau^2},\frac{B\gamma_s}{\tau},\frac{V}{W_s},\frac{D\gamma_s}{\tau},\frac{d_s\gamma_s}{\tau},\frac{g\tau}{\gamma_sW_s^2},\frac{\rho W_s^2}{\tau},\frac{\mu\gamma_sW_s}{\tau^2},\frac{d_{50}\gamma_s}{\tau},i,\sigma\right)---(4.11)$$

Also equations (4.8), it as well as equations (4.9) to (4.11) can be reduced discarding the ineffective groups. This gives the following equations:-

$$\frac{Q\gamma_s^2}{\tau^2W_s},=\int\left(\frac{P\gamma_s^2}{\tau^3W_s},\frac{Q_s\gamma_s}{\tau^2W_s},\frac{A\,\gamma_s^2}{\tau^2}\right)---(4.12)$$

$$\frac{A\,\gamma_s^2}{\tau^2}=\int\left(\frac{P\gamma_s^2}{\tau^3W_s},\frac{Q_s\gamma_s}{\tau^2W_s},\frac{Q\gamma_s^2}{\tau^2W_s}\right)---(4.13)$$

$$\frac{P\gamma_s^2}{\tau^3W_s}=\int\left(,\frac{Q_s\gamma_s}{\tau^2W_s},\frac{A\,\gamma_s^2}{\tau^2},\frac{Q\gamma_s^2}{\tau^2W_s}\right)---(4.14)$$

$$\frac{Q_s\gamma_s}{\tau^2W_s}=\int\left(\frac{P\gamma_s^2}{\tau^3W_s},\frac{Q\gamma_s^2}{\tau^2W_s},\frac{A\,\gamma_s^2}{\tau^2}\right)---(4.15)$$

4.5. Application Of Statistical Theorems To Dimensionless Groups:-

It is always desirable to reveal how closely two sets of dimensionless groups are associated. This can be tackled by ranking one of the dimensionless groups in increasing or decreasing order of magnitude and note the corresponding order of the other, which was previously conducted and presented in the appendices (A to C). The aim is to find the order of association to calculate the so called correlation coefficient (r_{xy}).This can be expressed symbolically as:-

$$r_{xy}=\left(\frac{Cov(x,y)}{\sqrt{Var(x)Var(y)}}\right)=\frac{S_{xy}}{\sqrt{S_{xx}}\sqrt{S_{yy}}}=\frac{\sum_1^n(x-\bar{x})(y-\bar{y})}{\sqrt{\sum(x-\bar{x})^2\sum(y-\bar{y})^2}}---(4.16)$$

Where:-

$\mathrm{var}(x), \mathrm{var}(y) =$ The mean variance of x and y.

$\{\mathrm{var}(x)\}^{\frac{1}{2}} \{\mathrm{var}(y)\}^{\frac{1}{2}} =$ The standard deviations of x and y.

$\mathrm{cov}(x, y) = Coveriance$ of $(\mathrm{x,y}) =$ Measure of extend to which x and y coincidentally depart from their average values.

The numerical value of the linear correlation coefficient must lie in the range ± 1.The nearer this value to ± 1 the better the correlation and the closer x,y set of pairs plot a straight line. On the other hand the closer this value to zero, the more random the plot of x, y pairs.

Linear least square regression equation or model is a technique which can be used to give the relationship between a dependent variable (y) with one or more independent variables (x).It has many useful properties and most widely used. It assumes that independent variables are measured without error, while errors are in the equation and dependent variable. These errors are linear and independent on one another.The equation is expressed symbolically in the form:-

$$y_i = a + b(x_1 - x) + e_i \; --- (4.17)$$

Where:-

$i =$ 1,2, ...n (n= Pair of $\mathrm{x_i}, y_i measurements$)

x = Mean of the value of x_i.

e_i = Error of equation and y_i.

a, b = Coefficients determined from the data.

This procedure gives the best fit straight line .The constant (a), being the intercept at $y = a$, $and\ x =$ zero. The coefficient (b) related to the correlation coefficient $(r_{xy})^{\frac{1}{2}}$,and independent on (a). This solution indicates that the so called total sum of squares of $\left(y\right)$ values (TSS),may be separated into the so called explained model equation squares (MSS),and the unexplained or residual squares (RSS).The larger the value of (MSS),the better is the fit of the model equation.

In statistics a sample of a data is usually considered as a single realization of many that could have been made from a population. This difficult and important concept underpins the significant tastings in statistical theory. To test the goodness of fit of equation (4.17), it is assumed that $(e_i),$ is a realization of random variable with mean equal to zero. Thus the estimate is as likely too large as is to be too small,(best linear unbiased estimation).This indicates that $(a),$ is normally distributed and evaluation of the regression equation is constructed by use of standard statistical tables, such as those of Student $\left(t\right)$ Test ,and Fisher Diagrams. This is conducted by finding the ratio of $\left[\frac{MMSS}{MRSS}\right]$,compared with that of Fisher Diagram. If this ratio exceeds 95 %, the equation is a good fit. $\left(MMSS\right)$,and $\left(MRSS\right)$, are the model sum of squares, and model residual sum of squares.

The regression out put gives the intercept $\left(x = \text{zero}; y = a;\ \text{the slope}\ \ b\ \right)$,together with standard error of estimate of $\left(y, and, b\right)$.The values of $\left(b\right),$ its standard error, together with standard values of TSS,MSS,and RSS,and the standard error of $(a),$ are given by the following equations:-

$$b = \frac{S_{xy}}{S_{xx}} --- (4.18)$$

$$St.er.of\ \mathrm{b} = \frac{\mathrm{S}}{\sqrt{\mathrm{S_{xx}}}} --- (4.19)$$

$$TSS\ =\ \mathrm{S_{yy}} --- (4.20)$$

$$MSS =\ \mathrm{b}^2 S_{xx} =\ \mathrm{S}^2 --- (4.21)$$

$$RSS =\ \mathrm{S_{yy}} -\ \mathrm{b}^2 S_{xx} --- (4.22)$$

$$St.er.of\ \ a\ = \mathrm{S}\ [\frac{\mathrm{S_{xx}} + n\bar{x}^2}{nS_{xx}}]^{\frac{1}{2}} --- (4.23)$$

$$St.er.of\ \ \mathrm{y}\ = \mathrm{S}\ [1+\frac{1}{\mathrm{n}}\frac{(x-\bar{x})^2}{S_{xx}}]^{\frac{1}{2}} --- (4.24)$$

The standard error of an estimate gives an indication of the accuracy of the estimate. If a coefficient is estimated to be close to zero with a large standard error, the coefficient will be taken as zero. Applying the (t) Test reveals whether or not the coefficient is significantly different from zero. The test is conducted by dividing the coefficient by its standard error. The value thus obtained is compared with that of the (t) value, with degrees of freedom $(n-2)$,and appropriate RSS.The coefficient is significantly different from zero, if the computed value is greater than that obtained from the (t) table.

In the accepted tested model equation, the likely error is indicated by the regression error $(MRSS)^{\frac{1}{2}}$ .The regression error $(MRSS)^{\frac{1}{2}}$ is the average amount of variation which was not explained by the model. Using the values of the constant (a), and the coefficient (b), the standard error of (y) is given by equation (4.24).

Usually the dependent dimensionless group in a set of dimensionless groups is predicted by several groups. The technique of regression equation of a single variable is extended in a similar manner. The procedure is exactly the same but more calculations of the increased number of coefficients are involved.

Very often in practice the relationship between a set of pairs of variables is nonlinear. To aid in determining an equation connecting the variables, the set of pairs are plotted in a rectangular coordinate system. The resulting set of points is known as the scatter diagram. From the scatter diagram it is possible to draw a curve approximating the data. The nonlinear relationship between the variables can be formulated by a certain equation. If the form of the equation is known the constants in the equation can be obtained by choosing as many points in the curve as there are constants in the equation. This has the disadvantage of being laborious when the number of constants is increased.However, some nonlinear relationships can sometimes be reduced to linear relationships by transformation of variables. For example if a curve of log y versus log x shows a linear relationship, its equation can be expressed in the form:-

$$y = \mathrm{a\,x^{b}} \quad \text{or} \rightarrow \ \mathrm{Log\,y = Log\,a\ + b\,Log\,x} \ \text{---}(4.25)$$

Although the method of linear least square regression has many useful properties, and most widely used, yet it has its short comings. A linear model is usually assumed for convenience, while it is rare to have a physical reason why a linear equation is appropriate. Furthermore errors are assumed to be normally distributed, though this is not true, but it is an important assumption for (t), and Fisher Tables performance. However, the concept of regression model equations is frequently used now days at large because of its strong predictive power consigned to computers. But before the application of regression the required data must be prepared for the application.

4.6. Guide Lines Arrangements:-

The general equation of the dimensionless groups was given by equation (4.8).Those defining flow by the equation (4.12)….etc.Those equations do not only show the relationships among dimensionless groups of the dependent property under consideration, and the independent dimensionless groups, but also offer guide lines to arrangements of data that reveals the inner mechanism of the phenomenon. This procedure will make it possible to single out, control and vary one dimensionless group at a time, while keeping the other constant. Considering equation:-

$$\pi_A = \int_A\left(\pi_1, \pi_2, \pi_3.....\pi_{(n-m)}\right) --- (4.2)$$

Equation (4.2) can also be written in the form:-

$$\pi_1 = \int_1\left(\pi_2, \pi_3, \pi_4.....\pi_{(n-1)}\right) --- (4.26)$$

A family of curves can be formed for π_1 versus π_2 ,for different values of $\left(\pi_3, \pi_4.......\pi_{n-3}\right)$. similarly, another family of curves can be formed for π_1 versus π_3 ,for different values of $(\pi_2, \pi_4.......\pi_{n-3})$. This process can be repeated for all the remaining π terms. The collection of families of cures thus formed gives the complete functional relationship among the dimensionless groups of π - terms referred to as particular property, and the dimensionless groups of the phenomenon. If it was found that the dimensionless groups of the particular property does not depend on one or more of the dimensionless groups, such dimensionless groups are discarded as adopted above.

Proceeding with this technique and considering equation (4.8)

$$\frac{Q\gamma_s^2}{\tau^2 W_s},=\int_A\left(\frac{P\gamma_s^2}{\tau^3 W_s},\frac{Q_s\gamma_s}{\tau^2 W_s},\frac{A\,\gamma_s^2}{\tau^2},\frac{B\gamma_s}{\tau},\frac{V}{W_s},\frac{D\gamma_s}{\tau},\frac{d_s\gamma_s}{\tau},\frac{g\tau}{\gamma_s W_s^2},\frac{\rho W_s^2}{\tau},\frac{\mu\gamma_s W_s}{\tau^2},\frac{d_{50}\gamma_s}{\tau},i,\sigma\right)---(4.8)$$

According to the objectives and study limitations; some of the dimensionless groups are constant such as

$$\left(\frac{D\gamma_s}{\tau},\frac{d_s\gamma_s}{\tau},\frac{g\tau}{\gamma_s W_s^2},\frac{\rho W_s^2}{\tau},\frac{\mu\gamma_s W_s}{\tau^2},\frac{d_{50}\gamma_s}{\tau},i,\sigma\right).$$

Hence equation (4.8), was reduced to the following form

$$\frac{Q\gamma_s^2}{\tau^2 W_s}=\int\left(\frac{P\gamma_s^2}{\tau^3 W_s},\frac{Q_s\gamma_s}{\tau^2 W_s},\frac{A\,\gamma_s^2}{\tau^2},\frac{B\gamma_s}{\tau},\frac{V}{W_s}\right)---(4.27)$$

Equation (4.27), is the basic equation for the required studies. It reveals the relations among the discharge ratio group $\frac{Q\gamma_s^2}{\tau^2 W_s}$,power generation ratio $\frac{P\gamma_s^2}{\tau^3 W_s}$ group, cultivated areas ratio group $\frac{A\,\gamma_s^2}{\tau^2}$,and sediment ratio group $\frac{Q_s\gamma_s}{\tau^2 W_s}$ .Hence,accordingly,from equation (4.27), equations,(4.12) ,(4.13),(4.14) and,, (4.15),could also be obtained.

4.7. Application Of The Equations:-

The main objective is to assess the impact of sediment on irrigation water and optimized use and consumption of water for irrigation. These equations will reveal these aspects using the available collected data. The other extended parts of the objectives are the determination of the power relations as well as the cultivated areas, which will also be revealed by the application of these equations.

Some of the involved dimensional parameters or variable are to be calculated. The shear stress can be obtained using the velocity profile method;-

$$\frac{V}{V_*}=6.25+5.75\ \mathrm{Log}\ \frac{\mathrm{R}}{\mathrm{d}_{50}}---(4.28)$$

Where:-

$V =$ Velocity of flow $m/\sec$

$V_* =$ Shear velocity $m/\sec$

R= Hyraulic radius m

$d_{50} =$ Particle size which 50 % finer.

And the equation of :-

$$V_*^2 = \frac{\tau}{\rho} \quad ---(4.29)$$

Where:-

$\rho =$ Equals to one gram/cm^3, *equal* to one Ton/m^3.

$$\tau = V_*^2 \quad ---(4.30)$$

Where:-

$\tau =$ In Ton/m^2, and $V_* =$ In m/sec

4.8. Tabulation of Measured and Computed Data:-

The measured quantities are the cultivated area. It is measured on wheat area basis. The unit area of each crop is taken proportional to the crop water consumption from sowing to harvest. Total year Cultivated Area in all Gezira,Managil,Rahad,Suki,Sugar (Sennar + Guneid).Wheat 1 feddan unit (2528), Sorghum 1.12 feddan unit (2820), ground nut 1.44 feddan unit,(3632),Cotton 2.27 feddan units,(5728) Sugar 3.61 feddan units.(9126) (Feddan unit = 4200 m^2). The areas for the five crops wheat ground nut sorghum cotton and sugar are as in table (4.4) in all the schemes downstream Rosaries dam.

Table no.(4.4): The Area For The Five Crops In All The Schemes Downstream Rosaries Dam.

Year	Wheat $\times 10^6 m^2$	Cotton $\times 10^6 m^2$	G.nut $\times 10^6 m^2$	Sorghum $\times 10^6 m^2$	Sugar $\times 10^6 m^2$	Total $\times 10^6 m^2$
2005	882	3432	1149	3175	622	9260
2006	979	3385	1198	3629	622	9813
2007	949	3642	1282	3730	678	10281
2008	2012	1592	1173	4133	678	9939
2009	1470	1621	1718	1210	694	6713
2010	1428	1916	1282	957	678	6761
2011	769	1392	1869	1310	678	6018
2012	655	877	1814	1058	694	5098
2013	508	1144	1300	756	694	4402
2014	722	1573	1663	6552	622	11132
2015	643	1754	1724	806	678	5605

The discharge (Q) is also measured in $\mathrm{m}^3/\sec$ $(\mathrm{m}^3/\mathrm{year})$, together with the water depth and width $(D \text{ and } \mathrm{B})$ upstream the dam in m (average year) , as well as the scour depth downstream the dam below the water surface d_s in m (average day or month or year,which is not available) .The other measured parameters are d_{50} , in m (average day or month or year) , the fall velocity W_s,in $m/\sec$,the power generation in GWH ($\left(\frac{kgr.m^2}{\sec^3}\right)$),and the sediment in $\frac{kgr\ \mathrm{m}}{\sec^3}$ .The computed parameters are the velocity in $m/\sec$,the wetted parameter P in m ,the hydraulic radius R in m ,shear velocity V_* in $m/\sec$, $\tau = \text{shear stress in } \frac{kgr}{\mathrm{m}.\sec^2}$ .Taking all averages in day or month or year.

The collected data of discharge, power, sediment, and the areas of the five crops wheat ground nut sorghum cotton and sugar of table (4.4) are shown in table (4.5).

Table No.(4.5): Discharge Power Sediment And Areas Data

Year	$Discharge\ Q \times 10^9 m^3 (milliard)$	$Power\ P_{watt} \times 10^6$	$Sediment\ ton \times 10^3$	$Area \times 10^5 m^2$
2005	49349.10	1077.633	180.72	1567.76
2006	61263.36	1176.168	156.68	1345.90
2007	62640.15	1272.210	262.24	1601.44
2008	58932.45	1312.166	321.56	1702.00
2009	39941.88	1096.722	194.95	1086.52
2010	56211.78	1040.351	815.93	985.33
2011	47691.09	1094.961	630.52	940.32
2012	51897.71	1053.157	636.41	801.50
2013	57454.57	1503.732	639.67	648.70
2014	63958.48	1670.644	523.16	783.27
2015	42519.33	1496.295	464.45	843.65

It is important to note here that the values of the parameters $\gamma_s, \tau,$ and W_s Where not easily obtainable. A visit to the responsible personnel in the old Ministry of Irrigation, among whom were the past resident engineers of Roseires Dam, gave the following values.

$$\gamma_s = \frac{1500\ \text{kgr}}{\text{m}^2\,\text{sec}^2}; \quad \tau = \frac{0.52\ \text{kgr}}{\text{m sec}^2}, and \quad W_s = 0.22\ \text{m/sec}$$

Being the only available data for the time being the researcher used them in the present study, which may attribute to some errors. However, being very important parameters the researcher recommended their availability in future studies, including their variation in consequecative years. Furthermore the sediment data consisted of suspended sediment only. As it is well known that the bed load as a proportion of the suspended is in the range from 5 % to 25 %,it was increased by an amount of 20%,in table (4.5),which will also be a source of error.

The values of the important dimensionless quantities of equation (4.12) to (4.15) are shown in table (4.6) as total of the year.

Table No.(4.6):Measured And Computed Data

Year	$\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}$	$\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}$	$\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}$	$\frac{A\,\gamma_s^2}{\tau^2}\times 10^{15}$
2005	19.50	86.1	477.00	77.04
2006	24.30	89.7	413.40	81.62
2007	24.84	96.5	693.24	85.33
2008	23.38	99.6	852.28	82.70
2009	15.85	92.9	515.16	55.83
2010	22.31	79.1	2162.40	56.24
2011	18.92	82.9	1669.56	50.09
2012	20.60	79.8	1685.40	42.43
2013	22.80	114.0	1695.00	36.61
2014	25.38	126.9	1386.48	92.60
2015	16.87	114.0	1230.72	46.68

4.9. Multiple Regression Analysis Of Dimensionless Groups:-

To reveal the relationship among $\frac{Q\gamma_s^2}{\tau^2 W_s}$,and each of the dimensionless groups on the right hand side of equation (4.12), simple regression analysis could be conducted. The operation can be generated by taking $\frac{Q\gamma_s^2}{\tau^2 W_s}$,as the dependent variable with one group of the right hand side of equation (4.12),as the independent variable in each operation. Similarly the determination of the relation among $\frac{Q_s}{\gamma_s W_s}$ and the other dimensionless groups on the right hand side of equation (4.13) could be obtained. Furthermore to reveal the dependence of $\frac{A\,\gamma_s^2}{\tau^2}$,on the other groups of equation (4.13),simple regression operations could be conducted.

As it is usual to predict a dependent group by a set of independent groups, it is not necessary to conduct these operations. Prediction of a dependent group by a set of independent groups is done by multiple regressions.

The procedure of multiple regression analysis is similar to that of simple regression analysis. In literature of theory on estimation of discharge it is generally accepted that the discharge is exponential. It is therefore assumed that the relationship between the dependent dimensionless group $\frac{Q\gamma_s^2}{\tau^2 W_s}$ and the others independent dimensionless groups of equation (4.12) are non – linear. In order to obtain linear relationship transformation to log relations are assumed.

The multiple regression analysis is conducted using a computer program Statistical Package for Social Sciences (SPSS) Model. The relevant dependent and independent dimensionless groups are arranged and fed to the computer. The transformed linear relationship among the dependent dimensionless group $\frac{Q\gamma_s^2}{\tau^2 W_s}$ and independent dimensionless groups $\left(\frac{P\gamma_s^2}{\tau^3 W_s}, \frac{Q_s\gamma_s}{\tau^2 W_s}, \frac{A\,\gamma_s^2}{\tau^2}\right)$, are revealed in the form of transformed regression equations. Each transformed model equation is expressed in the form of equation:-

$$Log\ \mathrm{y} = \mathrm{Log\ a_0} + a_1 Log\ \mathrm{x_1} + a_2 Log\ \mathrm{x_2} + a_3 Log\ \mathrm{x_3} --- (4.31)$$

Where:-

$y =$ Dependent variable taken as $\frac{Q\gamma_s^2}{\tau^2 W_s}$

$\mathrm{a_0} =$ Constant coefficient

$\mathrm{x_1 x_2 x_3} =$ Independent variables taken as $\left(\frac{P\gamma_s^2}{\tau^3 W_s}, \frac{Q_s\gamma_s}{\tau^2 W_s}, \frac{A\ \gamma_s^2}{\tau^2}\right)$

$\mathrm{a_1 a_2 a_3} =$ Exponent coefficients of $\mathrm{x_1 x_2 x_3}$ respectively.

The output of the transformed linear regression gives the correlation (r),the constant $\mathrm{a_0}$ and the exponential coefficients $\mathrm{a_1 a_2 a_3}$ with their standard error .Statistical test results namely Student $(t), Test$ to the coefficients and excellence of fit F – value are also given by the computer. The model regression equations accepted are those which produce 95 % confidence level having a correlation coefficient close to (± 1),with F – Value and Student (t) values greater than the tables values.

It is also very important before the application of multiple regressions on this equation to verify that the researcher developed equations are dimensionless. The verification is carried out by the substitution of the dimensional terms units to each supposed or obtained dimensionless groups as follows:-

$$\frac{Q\gamma_s^2}{\tau^2 W_s} = \frac{\mathrm{L^3}.M^2.L^2.T^4.T}{\mathrm{T.L^4}.T^4.M^2.T.L} = \frac{\mathrm{M^2}.L^5.T^5}{\mathrm{M^2}.L^5.T^5} \rightarrow\rightarrow \therefore \rightarrow O.K.$$

$$\frac{P\gamma_s^2}{\tau^3 W_s} = \frac{\mathrm{M.L^2}.M^2.L^3 T^6.T}{T^3.L^4.T^4.M^3.L} = \frac{\mathrm{M^3}.L^5.T^7}{\mathrm{M^3}.L^5.T^7} \rightarrow \rightarrow \therefore \rightarrow \mathrm{O.K.}$$

$$\frac{Q_s\gamma_s}{\tau^2 W_s} = \frac{\mathrm{M.L.M.L^2}.T^4.T}{\mathrm{T^3}.L^2.T^2.M^2.L} = \frac{\mathrm{M^2}.L^3.T^5}{\mathrm{M^2}.L^3.T^5} \rightarrow \rightarrow \therefore \rightarrow \mathrm{O.K.}$$

$$\frac{A\ \gamma_s^2}{\tau^2} = \frac{\mathrm{L^2}.M^2.L^2.T^4}{L^4.T^4.M^2} = \frac{\mathrm{M^2}.L^4.T^4}{\mathrm{M^2}.L^4.T^4} \rightarrow \rightarrow \therefore \rightarrow \mathrm{O.K.}$$

Thus the four groups are dimensionless. The substitution of the values of the quantities is shown in table (4.6).

Referring to tables (4.5), of the areas of the crops, and (4.6) of the dimensionless groups verified above and table (4.1) of all the other dimensional Parameters the results obtained are presented in the graphs in figures (4.2) to (4.5).

- **Discharge Regression**

Variables Entered/Removed

Model	Variables Entered	Variables Removed	Method
1	logD, logx, logZ	.	Enter

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.679	.461	.230	.06036

ANOVA

Model		Sum of Squares	Df	Mean Square	F	Sig.
1	Regression	.022	3	.007	1.997	.203
	Residual	.026	7	.004		
	Total	.047	10			

Coefficients

Model		Unstandardized Coefficients		Standardized Coefficients	T	Sig.
		B	Std. Error	Beta		
1	(Constant)	.060	.684		.088	.932
	logx	.063	.285	.063	.222	.830
	logZ	.144	.092	.537	1.568	.161
	logD	.395	.170	.803	2.329	.053

Residuals Statistics					
	Minimum	Maximum	Mean	Std. Deviation	N
Predicted Value	1.2655	1.4229	1.3244	.04672	11
Residual	-.06780-	.08470	.00000	.05050	11
Std. Predicted Value	-1.260-	2.107	.000	1.000	11
Std. Residual	-1.123-	1.403	.000	.837	11

$$\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20} = 1.14815\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right)^{0.063}\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right)^{0.144}\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)^{0.395}$$

$$Log\,\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20} = 0.06 + 0.063\log\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right) + 0.144\log\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right) + 0.395\log\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)$$

$$Log\,\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20} = 0.06 + 0.063(1.94) + 0.144(2.68) + 0.395(1.89) = 1.31389$$

$$\therefore \frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20} = 10^{1.31389} = 20.60$$

Chart

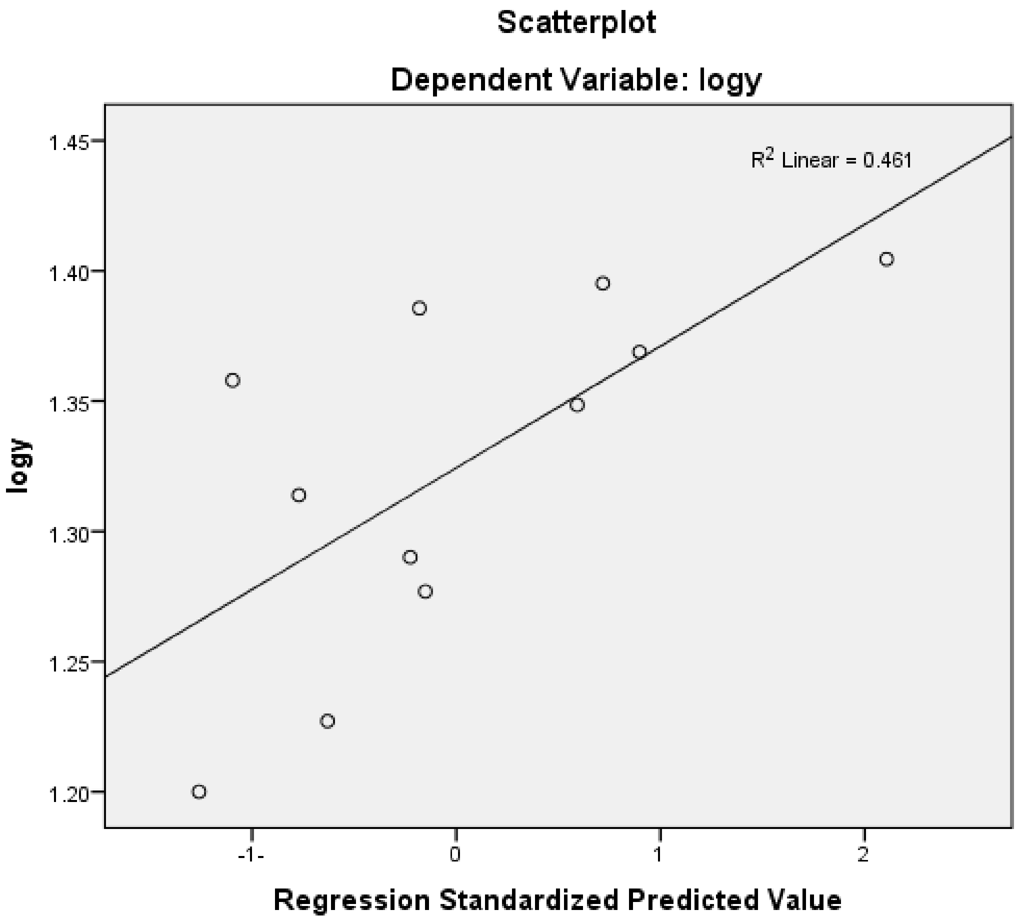


Fig.No.(4.2):Relationship Among Discharge power Sediment and Cultivated Areas

❖ **Power Generation Regression**

Variables Entered/Removed			
Model	Variables Entered	Variables Removed	Method
1	logD, logy, logZ	.	Enter

Model Summary				
Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.220	.048	-.360-	.07984

ANOVA						
Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.002	3	.001	.118	.947
	Residual	.045	7	.006		
	Total	.047	10			

Coefficients						
Model		Unstandardized Coefficients		Standardized Coefficients	T	Sig.
		B	Std. Error	Beta		
1	(Constant)	1.599	.673		2.374	.049
	logy	.111	.498	.111	.222	.830
	logZ	.035	.141	.130	.248	.811
	logD	.072	.298	.148	.243	.815

Residuals Statistics					
	Minimum	Maximum	Mean	Std. Deviation	N
Predicted Value	1.9526	2.0062	1.9795	.01503	11
Residual	-.09283-	.09730	.00000	.06680	11
Std. Predicted Value	-1.789-	1.773	.000	1.000	11
Std. Residual	-1.163-	1.219	.000	.837	11

$$\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15} = 39.7192\left(\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}\right)^{0.111}\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right)^{0.035}\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)^{0.072}$$

$$Log\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right) = 1.599 + 0.111og\left(\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}\right) + 0.035\log\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right) + 0.072\log\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)$$

$$Log\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right) = 1.599 + 0.111og(19.50) + 0.035\log(477.00) + 0.072\log(77.04) = 1.971785557$$

$$Log\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right) = 10^{1.971785557} = 93.71$$

Charts

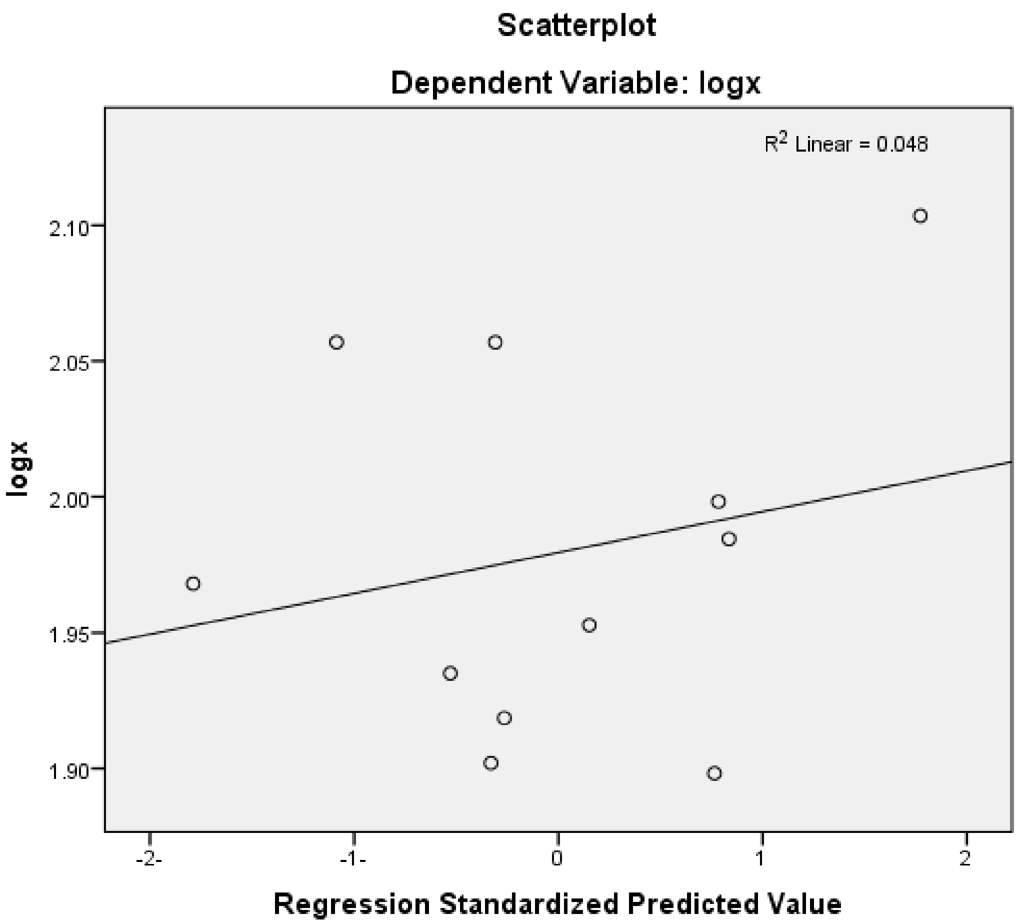


Fig. No.(4.3):Relationship Among Power Discharge Sediment and Cultivated Areas

❖ **Sediment Regression**

Variables Entered/Removed

Model	Variables Entered	Variables Removed	Method
1	logD, logx, logy	.	Enter

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.717	.514	.305	.21370

ANOVA						
	Model	Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.338	3	.113	2.466	.147
	Residual	.320	7	.046		
	Total	.658	10			

Coefficients						
	Model	Unstandardized Coefficients		Standardized Coefficients	T	Sig.
		B	Std. Error	Beta		
1	(Constant)	2.821	2.174		1.297	.236
	logy	1.805	1.151	.484	1.568	.161
	logx	.250	1.007	.067	.248	.811
	logD	-1.512-	.560	-.824-	-2.698-	.031

Residuals Statistics					
	Minimum	Maximum	Mean	Std. Deviation	N
Predicted Value	2.7802	3.4216	3.0022	.18380	11
Residual	-.30285-	.25183	.00000	.17879	11
Std. Predicted Value	-1.208-	2.282	.000	1.000	11
Std. Residual	-1.417-	1.178	.000	.837	11

$$\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right) = 662.265\left(\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}\right)^{1.805}\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right)^{0.250}\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)^{-1.512}$$

$$Log\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right) = 2.821 + 1.805\log\left(\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}\right) + 0.250\log\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right) - 1.512\log\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)$$

$$Log\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right) = 2.821 + 1.805\log(19.50) + 0.250\log(86.1) - 1.512\log(77.04) = 2.780548255$$

$$\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right) = 10^{2.780548255} = 603.32$$

Charts

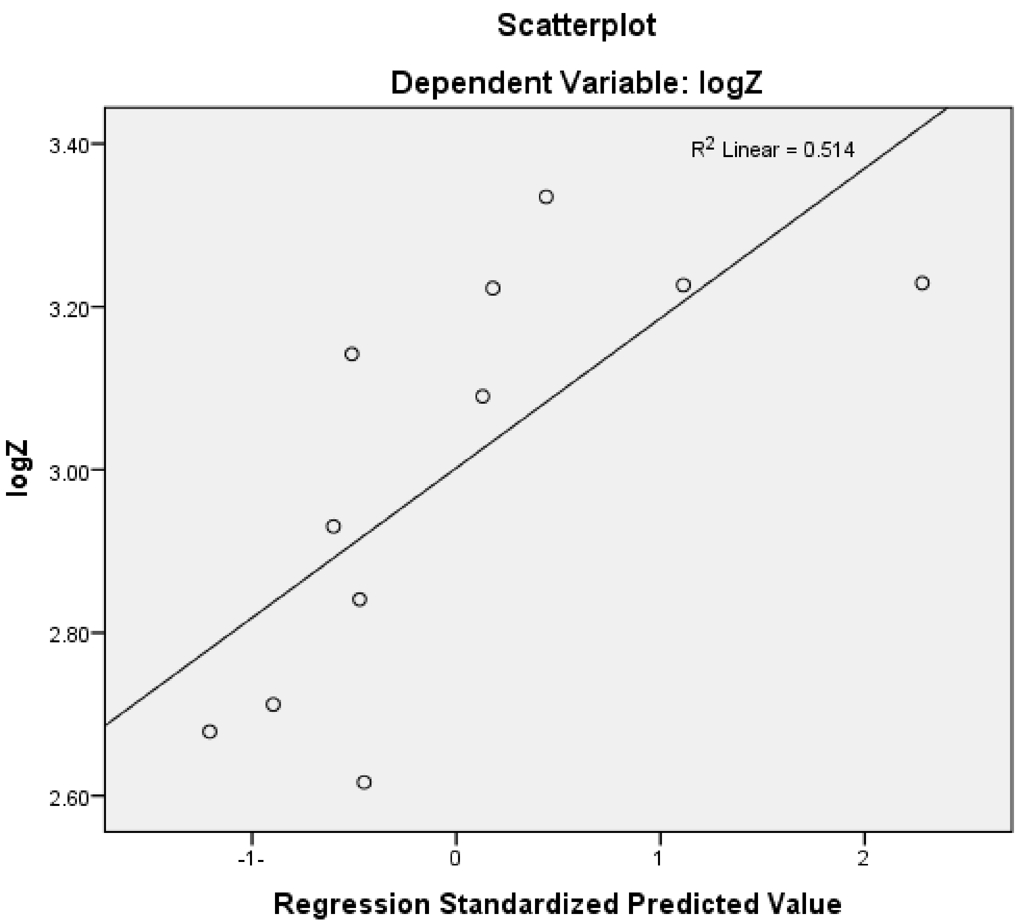


Fig.No.(4.4):Relationship Among Sediment Discharge Power and Culivated Areas

❖ Agricultural Areas Regression

Variables Entered/Removed

Model	Variables Entered	Variables Removed	Method
1	logw, logx, logy	.	Enter

Model Summary

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate
1	.797	.635	.478	.10093

ANOVA						
Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	.124	3	.041	4.058	.058
	Residual	.071	7	.010		
	Total	.195	10			

Coefficients						
Model		Unstandardized Coefficients		Standardized Coefficients	T	Sig.
		B	Std. Error	Beta		
1	(Constant)	1.108	1.064		1.041	.332
	logy	1.105	.474	.544	2.329	.053
	logx	.116	.476	.057	.243	.815
	logw	-.337-	.125	-.619-	-2.698-	.031

Residuals Statistics					
	Minimum	Maximum	Mean	Std. Deviation	N
Predicted Value	1.6544	1.9829	1.7883	.11136	11
Residual	-.19410-	.12266	.00000	.08445	11
Std. Predicted Value	-1.202-	1.748	.000	1.000	11
Std. Residual	-1.923-	1.215	.000	.837	11

$$\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)=12.0781\left(\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}\right)^{1.105}\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right)^{0.116}\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right)^{-0.337}$$

$$Log\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)=1.108+1.105\log\left(\frac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}\right)+0.116\log\left(\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}\right)-0.337\log\left(\frac{Q_s\gamma_s}{\tau^2 W_s}\times 10^{10}\right)$$

$$Log\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right)=1.108+1.105\log(19.50)+0.116\log(86.10)-0.337\log(477.00)$$

$$\left(\frac{A\gamma_s^2}{\tau^2}\times 10^{15}\right) = 10^{1.8552879}=71.66$$

Charts

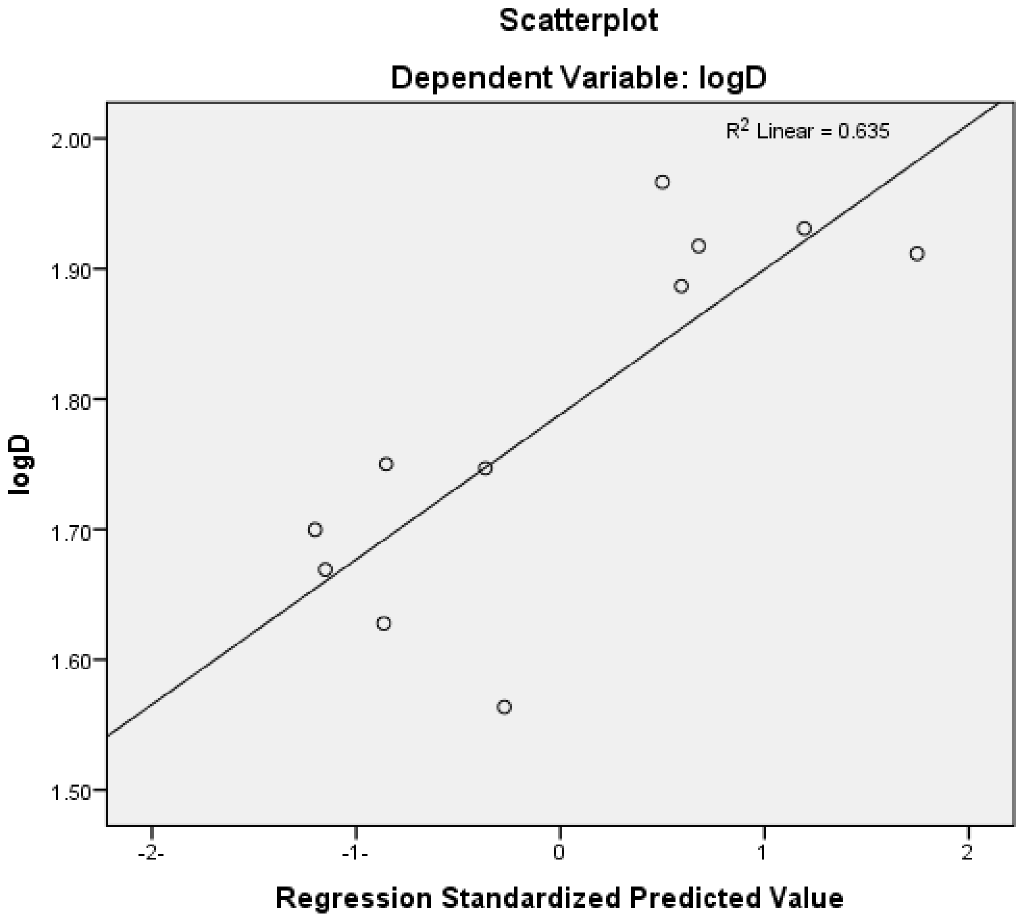


Fig.No.(4.5): Relationship Among Cultivated Areas Discharge Power and Sediment

CHAPTER FIVE

5. RESULTS AND DISCUSSIONS

5.1.Practical Application:-

The relevant data similar to that in table (4.6) is not available because no previous investigators have conducted similar investigations. Most of the previous investigators have conducted experimental works about discharge sediment and power generation. No investigator has conducted work about cultivated areas of the different crops in the different schemes. Those who have conducted studies about discharge sediment and power generation, obtained quantitative and qualitative results of certain and specific areas that can not be applied in this study.

Although the previous investigators have covered important studies yet it was selective and not covering the parts studied by the researcher. However it was all covered in the literature review so that the study would not be incomplete. The procedures adopted by the researcher mainly rely on basic and advanced knowledge about dimensional analysis and theory of models backed with SPSS supported by the advancing knowledge of the computer analysis. Consequently it is very difficult if not impossible to apply the developed empirical equations (4.32) to (4.35) to any of the previous investigators. Equations (4.32) to (4.35) are, therefore applied to the data taken from Roseires Dam.

5.2. Discharge Aspects Relations:-

Islam Al Zayed1 et.al.in an analysis of irrigation efficiency using comparative performance indicators, conducted a case study of Gezira Scheme, Sudan. They used metrological, irrigation, crop yield and cultivated areas (1970

-2007), for cotton, groundnut, and sorghum and, wheat. They concluded that the maximum demand for cotton was (1430 mm), and the water available was higher than the demand, due to inadequate rainfall moisture utilization and harvesting to reduce the need of supplemental irrigation, indicating that Gezira Scheme has low productivity.

In the present study the discharge relation with the other aspects was considered. The results to the four dimensionless groups developed by the researcher are shown in tables (5.1), (5.2), (5.3), and (5.4), respectively.

Tables No.(5.1): Discharge Aspects Relations $\dfrac{Q\gamma_s^2}{\tau^2 W_s}\times 10^{20}$

Year	Predicted	Actual	Error
2005	20.58046	19.5	1.08046
2006	20.67918	24.3	-3.62082
2007	22.77655	24.84	-2.06345
2008	23.22182	23.38	-0.15818
2009	18.41228	15.85	2.562285
2010	22.47368	22.31	0.163676
2011	20.7451	18.92	1.825103
2012	19.40853	20.6	-1.19147
2013	18.74121	22.8	-4.05879
2014	26.44571	25.38	1.065714
2015	19.69994	16.87	2.829941

5.3. Power Aspects Relations:-

This is to be conducted considering the power relation with the other aspects. The relevant equation is (4.33).

Tables No.(5.2): Power Aspects Relations $\frac{P\gamma_s^2}{\tau^3 W_s}\times 10^{15}$

Year	Predicted	Actual	Error
2005	93.75933	86.1	7.659328
2006	95.99609	89.7	6.29609
2007	98.3017	96.5	1.801695
2008	98.12976	99.6	-1.47024
2009	89.76863	92.9	-3.13137
2010	98.09309	79.1	18.99309
2011	94.65429	82.9	11.75429
2012	94.44854	79.8	14.64854
2013	94.52786	114	-19.4721
2014	101.5525	126.9	-25.3475
2015	91.99658	114	-22.0034

5.4.Sediment Aspects Relations:-

Quantification of water uses along the Blue Nile River network using hydrodynamic model conducted by Yasir revealed that **Basin** area is about 330,000 km2.It required study of the water distribution along the entire Blue

Nile River system. He Develop Hydrodynamic model using Sobek River to study the sediment transport. The methodology adopted consisted of a certain Mode schematization. He found that the model slightly overestimated the water level during the high flows. The model slightly overestimated the water level during the high flows. The model slightly underestimated the water level during the low flows.

In this study the power relation with the other aspects was conducted The relevant equation is (4.34)

Table No.(5.3):Sediment Aspects Relations $\frac{Q_s \gamma_s}{\tau^2 W_s} \times 10^{10}$

Year	Predicted	Actual	Error
2005	502.3195	397.5	104.8195
2006	691.8788	344.5	347.3788
2007	685.4882	577.7	107.7882
2008	649.3667	710.2	-60.8333
2009	572.9939	429.3	143.6939
2010	1009.09	1802	-792.91
2011	903.2958	1391.3	-488.004
2012	1340.726	1404.5	-63.7741
2013	2199.936	1412.5	787.4358
2014	674.3261	1155.4	-481.074
2015	884.5655	1025.6	-141.034

5.5. Cultivated Area Aspects Relations:-

This is to be conducted considering the power relation with the other aspects. The relevant equation is (4.35).

Table No.(5.4):Cultivated Area Aspects Relations $\frac{A\,\gamma_s^2}{\tau^2}\times 10^{15}$

Year	Predicted	Actual	Error
2005	71.49407	77.04	-5.54593
2006	96.13466	81.62	14.51466
2007	83.4411	85.33	-1.8889
2008	73.05669	82.7	-9.64331
2009	55.89462	55.83	0.064619
2010	49.34464	56.24	-6.89536
2011	45.12321	50.09	-4.96679
2012	49.19489	42.43	6.764891
2013	57.24005	36.61	20.63005
2014	69.8152	92.6	-22.7848
2015	45.71303	46.68	-0.96697

5.6. Blockage and Power:-

It can be stated that using groins and dredging as suggested and conducted by Abdel-Fattah (2004) is a partial solution for reduction of sedimentation problems at intake structures.

It can also be stated that using vanes which are similar to groins conducted by AbdelHaleem (2009) is a partial solution for reduction of sedimentation problems at intake structures. However it is better than that of Abdel-Fattah (2004), because it solved more than 50% of the sediment problem.

Using experimental results suggested by Hassanpour and Ayoubzadeh (2008), they revealed that implementing submerged vanes at 20° to 25°to the direction of the intake improved the flow into the intake reducing the sediment entering the intake. However no full solution was obtained, and when the intake angle was made 15°the inflow into the intake was reduced with increased sediment, resulting in an incomplete solution.

Similar results of field observations and laboratories experiments were obtained by Hossain et al. (2004), Odgaard (2005), Sadjedi Sabegh (2004) and Tan Soon-Keat et al. (2005).

Sayed Mahgoub 2013 work using movable bed model, conducted a comprehensive model test to cover the different river flow conditions and operation modes of the power plant in sixteen (16) experiments. Introducing double rows of submerged vanes mounted vertically at an angle of 60° to the main flow direction, his work resulted in preventing sediment intrusion. Mahgoub 2013 research work is recommended to be applied in Roseires Dam.

5.7. River Aggradation Degradation and Scour:-

Black, Richard (2009-09-21) indicated that reduced aggradation contributes to increased flooding in rivers deltas. It is used in geology for the increase in land elevation, typically in a river system, due to the deposition of sediment. It occurs in areas in which the supply of sediment is greater than the amount of material that the system is able to transport, estimated as change in elevation.

In the net Engineering *112 (6): 497.* doi*:*10.1061/(ASCE)0733-9429(1986)112:6(497),revealed that in geology, degradation refers to the lowering of a fluvial surface, such as a stream bed or floodplain, through processes Degradation is characteristic of channel networks in which either bedrock erosion is taking place, or in systems that are sediment-starved and are therefore entraining more material than is being deposited.

Hydraulic Engineering Circular No. 18 Manual (HEC-18) included several techniques of estimating scour depth. Empirical scour equations for live bed scour, clear water scour, and local scour at piers and abutments. The total scour depth was determined by adding three scour components which included the long-term aggradation and degradation of the river bed, general scour at the bridge and local scour at the piers or abutments. The equations in HEC-18 over-predict scour depth for a number of hydraulic and geologic conditions. Most of the HEC-18 relationships are based on laboratory flume studies conducted with sand-sized sediments streams frequently contained much more scour resistant materials such as compact silt, stiff clay, and shale. In this research the data obtained was downstream Rosaries Dam. The only data found was water depth above the downstream blanket of the dam. Table (5.5) shows the recorded depths during the years 2005 to 2015.

<u>Table No.(5.5) Recorded Depths Downstream Roseires Dam</u>

year	Water Depth (m.)	year	Water Depth (m.)
2005	15.88	2010	16.98
2006	17.38	2011	16.58
2007	16.88	2012	16.18
2008	15.78	2013	16.38
2009	16.48	2014	16.98
		2015	14.58

Table (5.5) indicates that there is fluctuation in the water levels with fluctuation of scour and deposition downstream the dam.

5.8.Rouseris Dam Operation And Maintenance Difficulties:-

The Roseires Dam, completed in 1966 with capacity of 3.024 km3 for irrigation as first priority, and hydropower generation comes secondly. The recent dam heightening was accompanied with extension in earth embankment bringing the total length of the dam to about 25 km. The storage capacity has considerably been affected by siltation and is now about 30 percent less. A special operation strategy, maintaining low reservoir level and high flow velocities during the passage of the flood, is applied to reduce the siltation (UNESCO Chair in Water Resources, 2011).

Operation of reservoirs depends on rules set for mainly water balance of the system among other factors, which are rarely revised during the life time of the reservoirs and Roseires is not an exception.

The deep sluices with sill levels of 435.5 m a l m s are used to pass the main volume of the flood, and to flush the sediment as much as possible.

The reservoir capacity was 2.8 billion m^3 when completed in 1966. The future design included raising additional 10 m, implemented 2008 - 2012, with additional storage 3.9 billion m^3, for the development of Power Station" Irrigation Projects Power before and after are respectively 1,188 GWh and 1,643 GWh (with a MOL of 470.00 m AD).

Filling of the Roseires and Sennar Reservoirs usually starts in September and completed in October, following a clearly defined filling program.

Annual flow releases of Roseires Dam average 45,960 Mm^3.

Two dams are located on this section of the Blue Nile: Sennar Dam, about 320 km south-east of Khartoum, in operation since 1926,Roseires Dam, about 270 km upstream of Sennar Dam, in operation since 1966 Currently four main irrigation schemes are supplied with water from the Blue Nile between Roseires Dam and Sennar Dam: These are:Es Suki Irrigation Scheme,Gezira and Managil Irrigation Schemes,NW Sennar Irrigation Scheme and,Meina Irrigation Pump Station. Between Sennar Dam and Khartoum, located nearby to the confluence with the White Nile, the following irrigation schemes are supplied with Blue Nile water: El Waha Irrigation Scheme Guneid Irrigation Scheme Irrigation Pumps Schemes.Two further rivers flow into the Blue Nile in this section: The Dinder River , The Rahad River .

The filling procedure was changed after completion of the Roseires Dam heightening in order to correspond to the new conditions.

Four distinct seasonal demand profiles were identified for the existing schemes, which are fed from upstream of Sennar Dam including Gezira and Managil, Es Suki, NW Sennar and Irrigation Meina Pump. The raising of the minimum operating level to 472 m became necessary to accommodate annual dredging work. Moreover, the dredging performance needs to be improved significantly. The Gauging Station at El Deim: implementation of a further upstream located Gauging Station. Rising of the MOL to 472 m to improve the dredging performance for a safe passing of the flood, flood routing identified the release structures capacity of the Sennar Dam as critical.

CHAPTER SIX

6. CONCLUSIONS AND RECOMMENDATIONS

6.1. Conclusions:-

6.1.1. Depletion has been reported world-wide in drought prone areas. In the Sudan yearly losses attained the range from 0.3% to 1.67%.

6.1.2. Although Sudan irrigated agriculture produces about 50 % of the total crop production, yet it is associated with painstaking of removing sediments from the irrigation network system and reservoirs.

6.1.3. Based on the results obtained in this research, it could be admitted that Roseires Reservoir lost a great part of its capacity due to the sedimentation problems.

6.1.4. Data from 2005 to 20015 was used to calibrate the hydrodynamic and morph dynamic model of the Roseires Reservoir, and the calibration results showed good agreements to observed data.

6.2. Recommendations:-

6.2.1. Complexity in reservoir operation and maintenance coupled with downstream the dam river bank erosion, sediment deposition, insufficient irrigation water for the agricultural schemes, with problems in power generation; require urgent mitigation.

6.2.2. The assessment of the impact of sediment on irrigation water and optimization of use and consumption of water for irrigation suggested in this research are recommended.

6.2.3. Further research is required to evaluate the extend of direct and indirect impact of sedimentation on existing reservoirs where real data are available. This will bring about the understanding, through case studies.

6.2.4. Further research is required using modern sophisticated model to investigate the Reservoir sedimentation problems

6.2.5. Dams and reservoirs data about soil, shear, and water depth are essential tools used in research. It is therefore highly recommended to establish a data base recoding all relevant research parameters.

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Discharge and Power Analysis:-

Appendix A

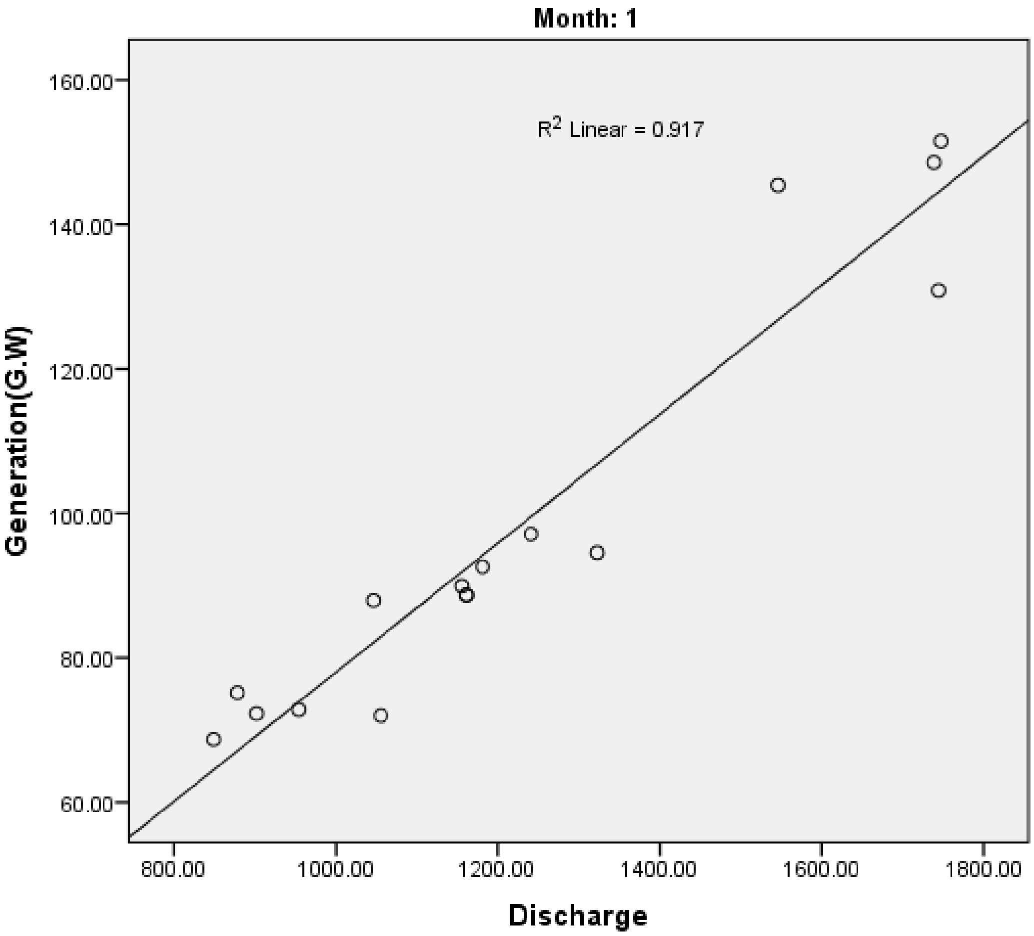


Fig. No.(A.F.1): January Discharge Generation Regression

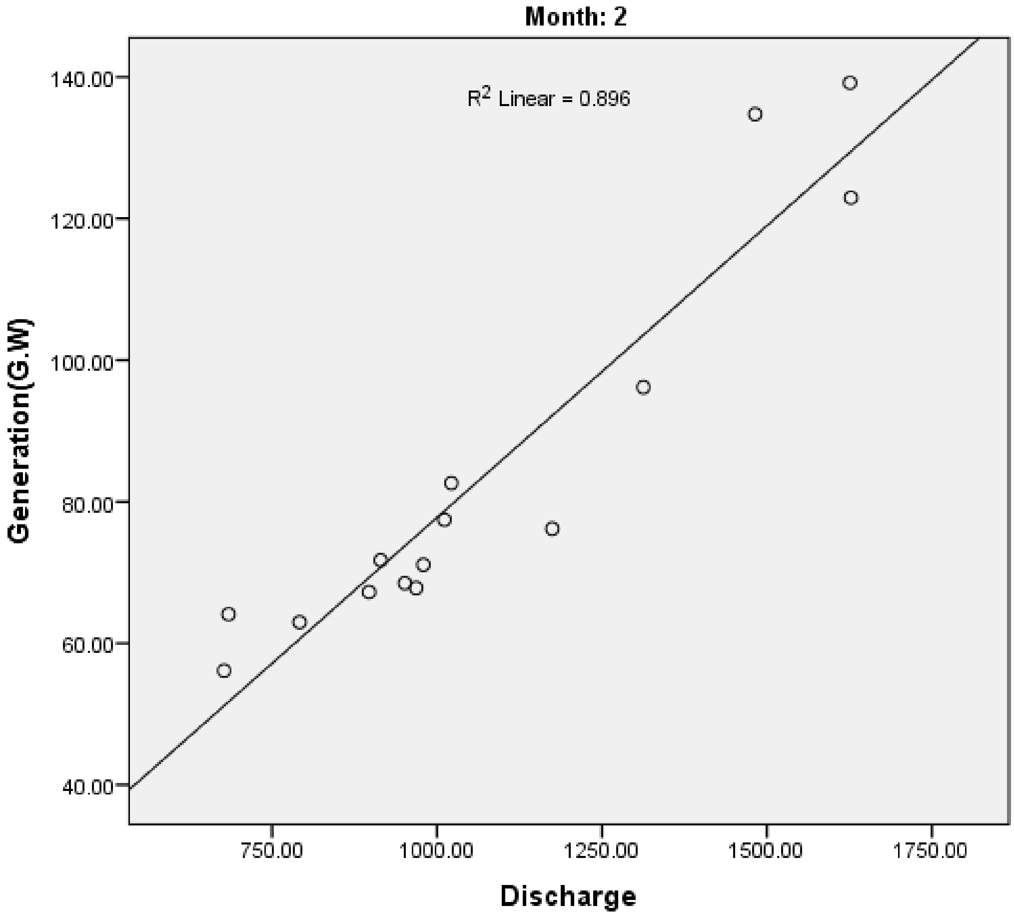


Fig. No.(A.F.2): February Discharge Generation Regression

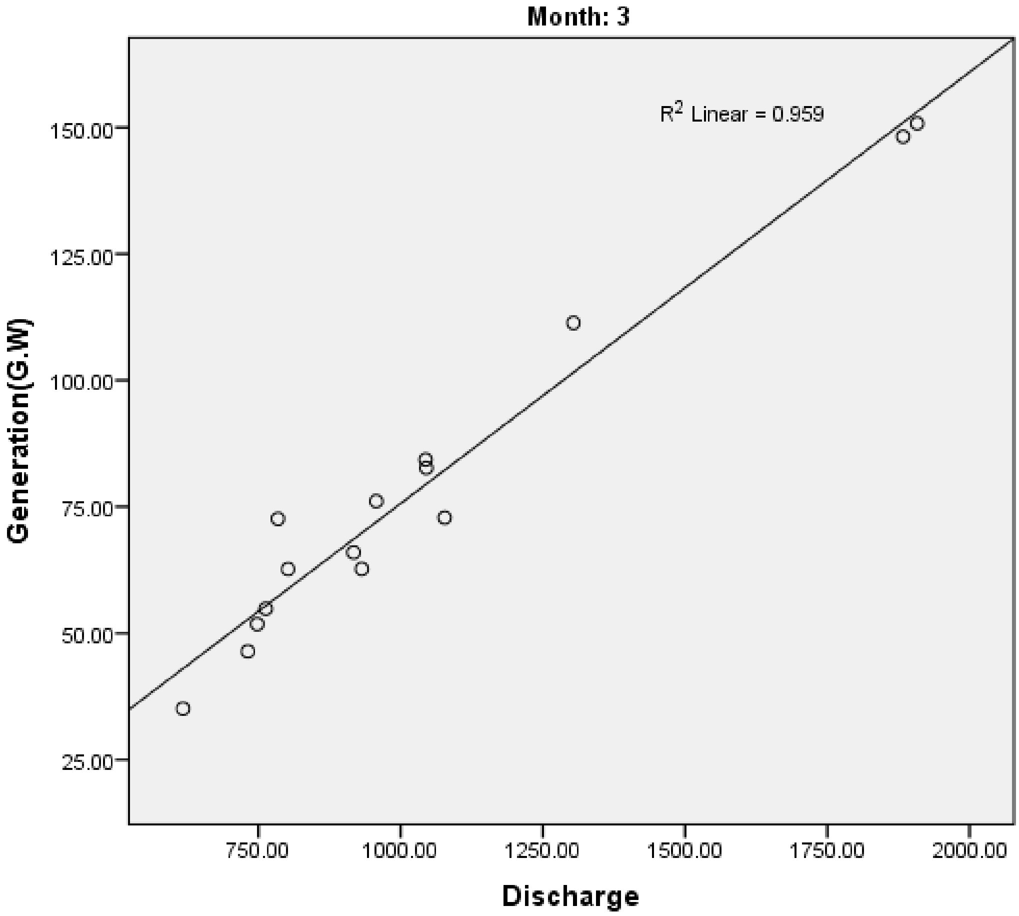


Fig. No.(A.F.3): March Discharge Generation Regression

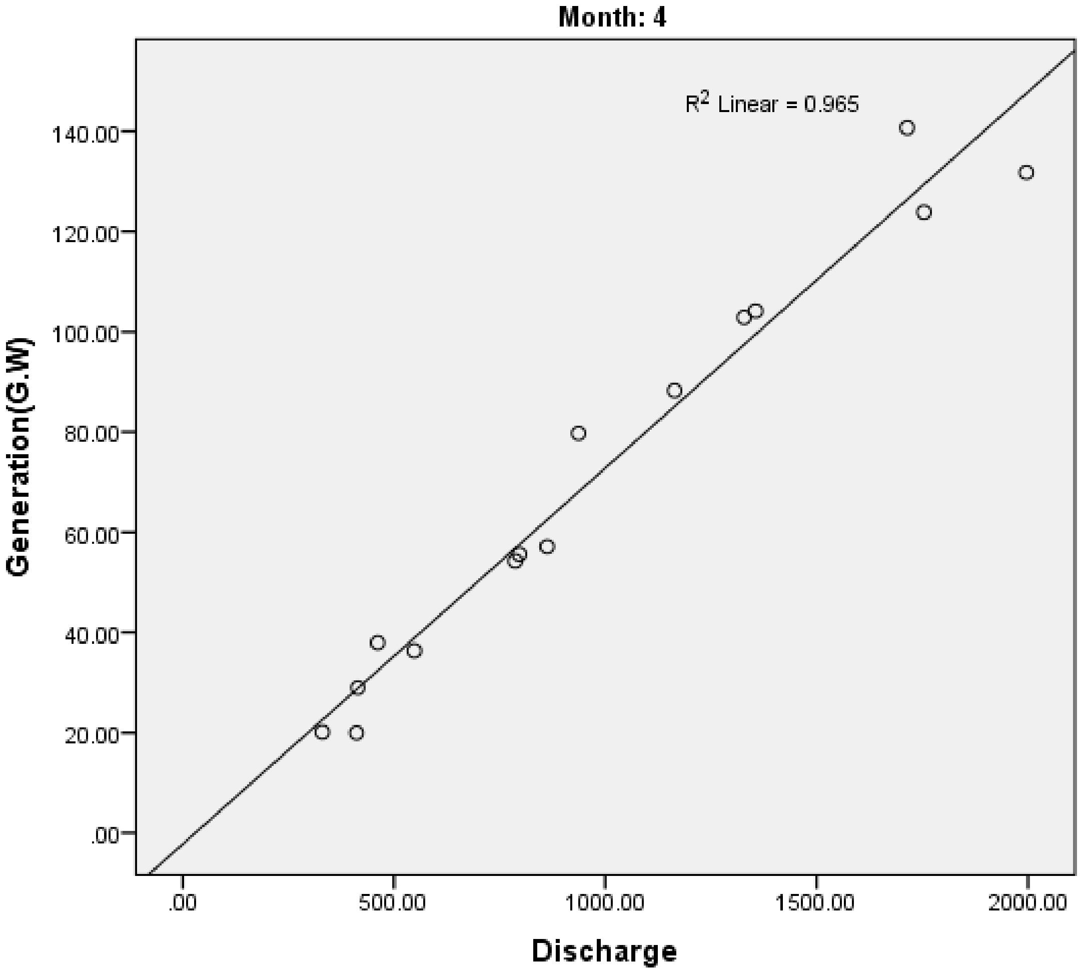
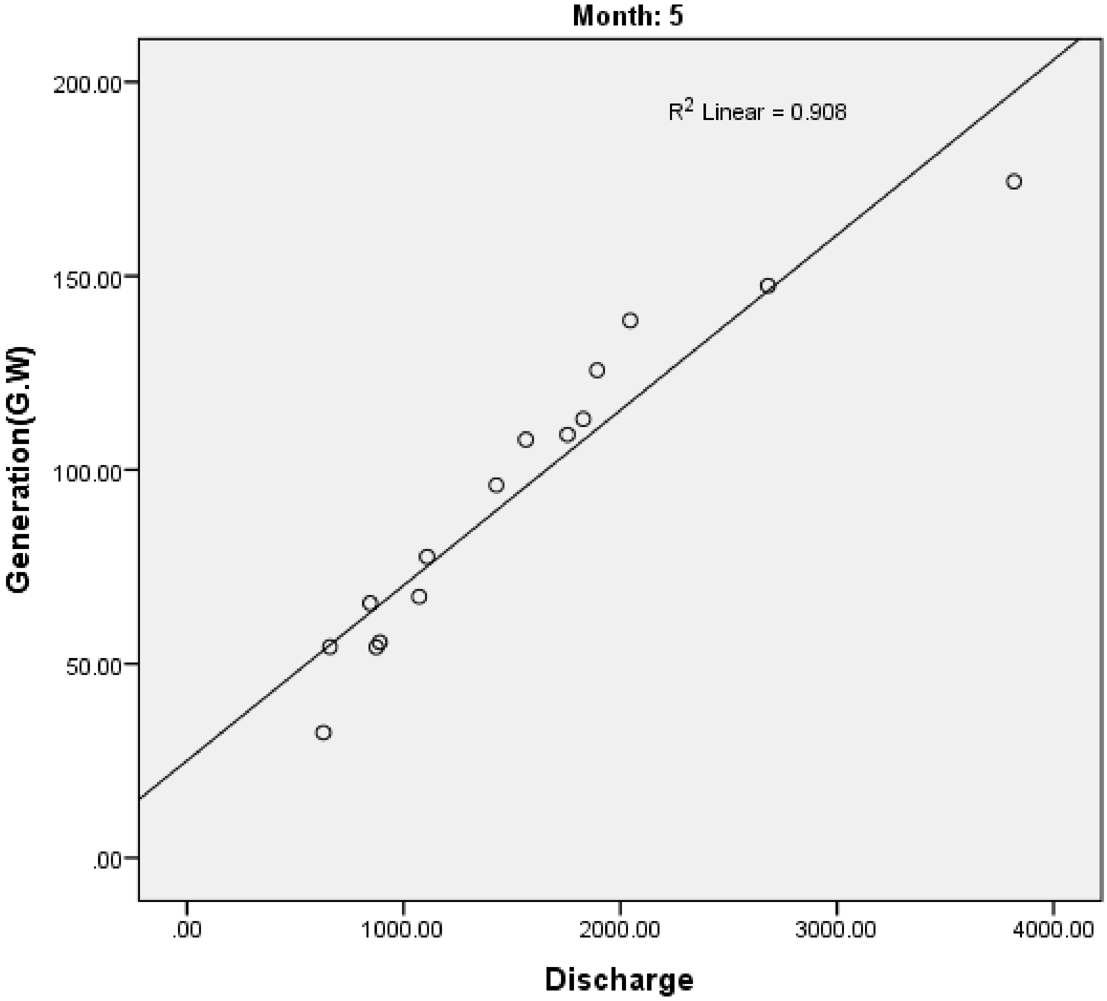


Fig. No.(A.F.4): April Discharge Generation Regression



<u>Fig. No.(A.F.5): May Discharge Generation Regression</u>

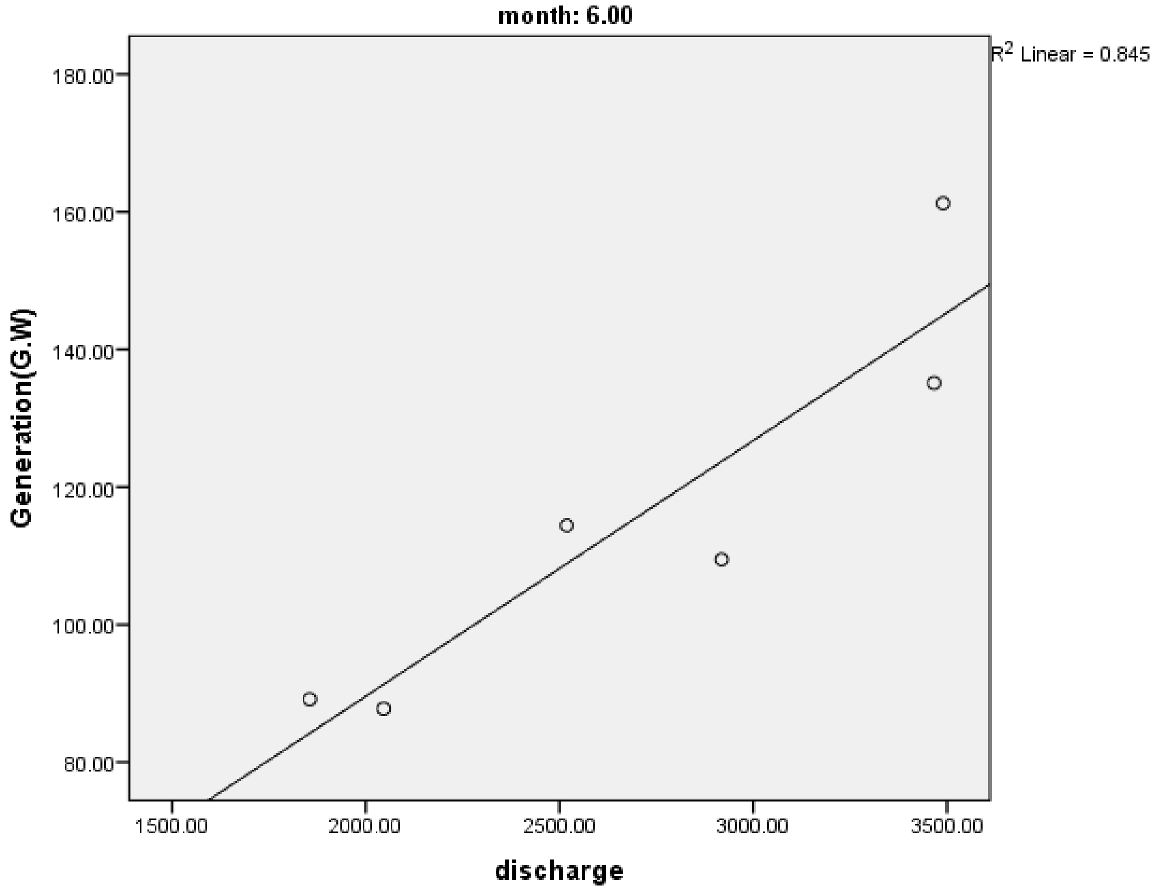


Fig.No.(A.F.6): June Discharge Generation Regression

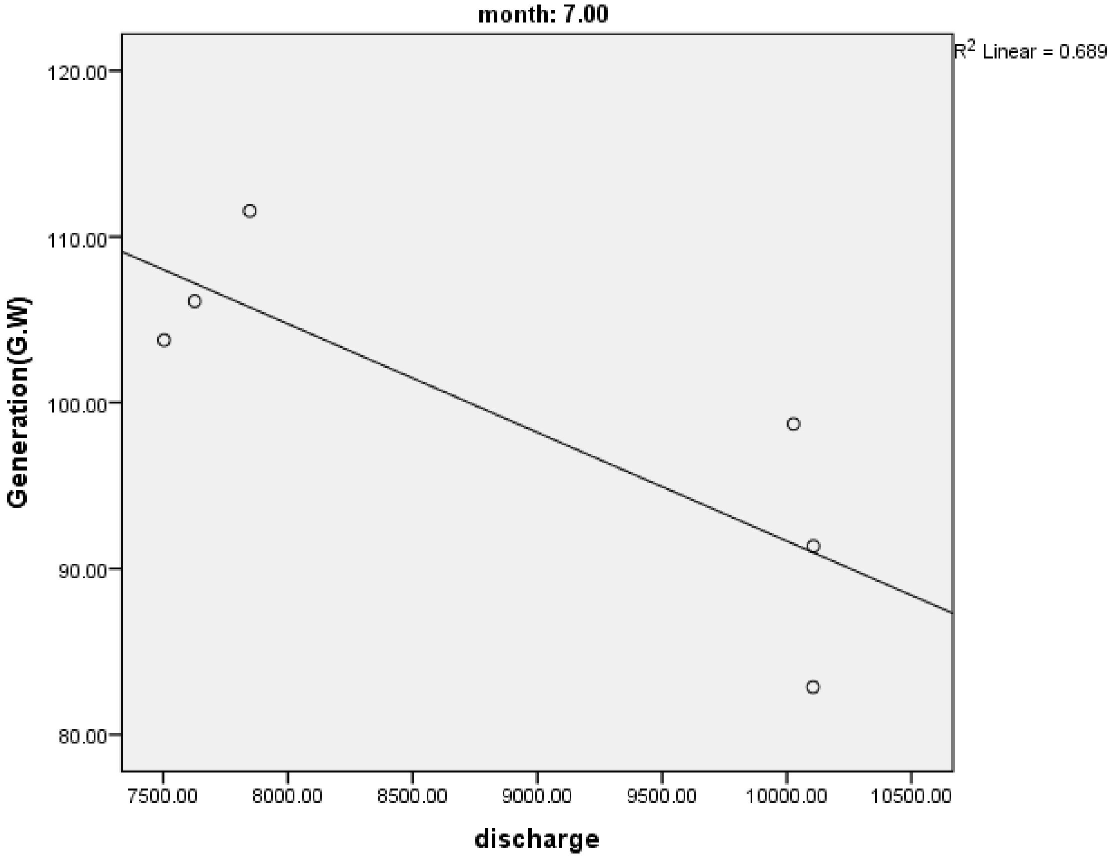


Fig. No.(A.F.7): July Discharge Generation Regression

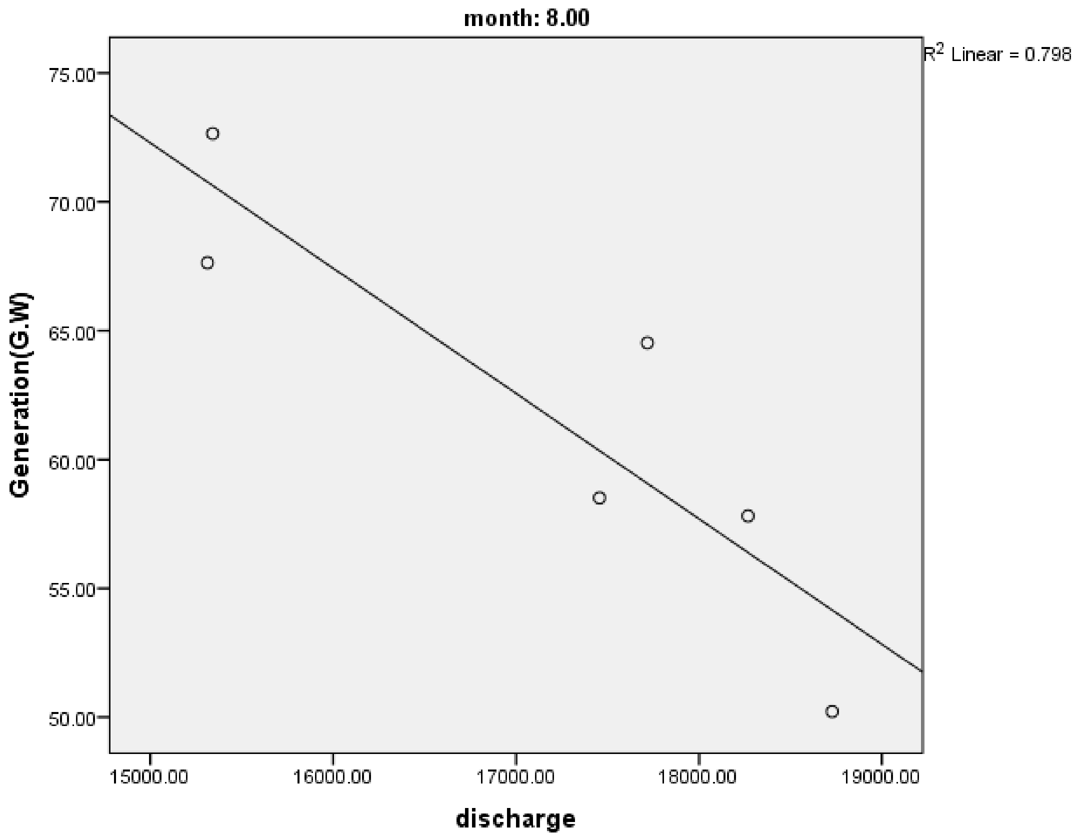


Fig.No.(A.F.8): August Discharge Generation Regression

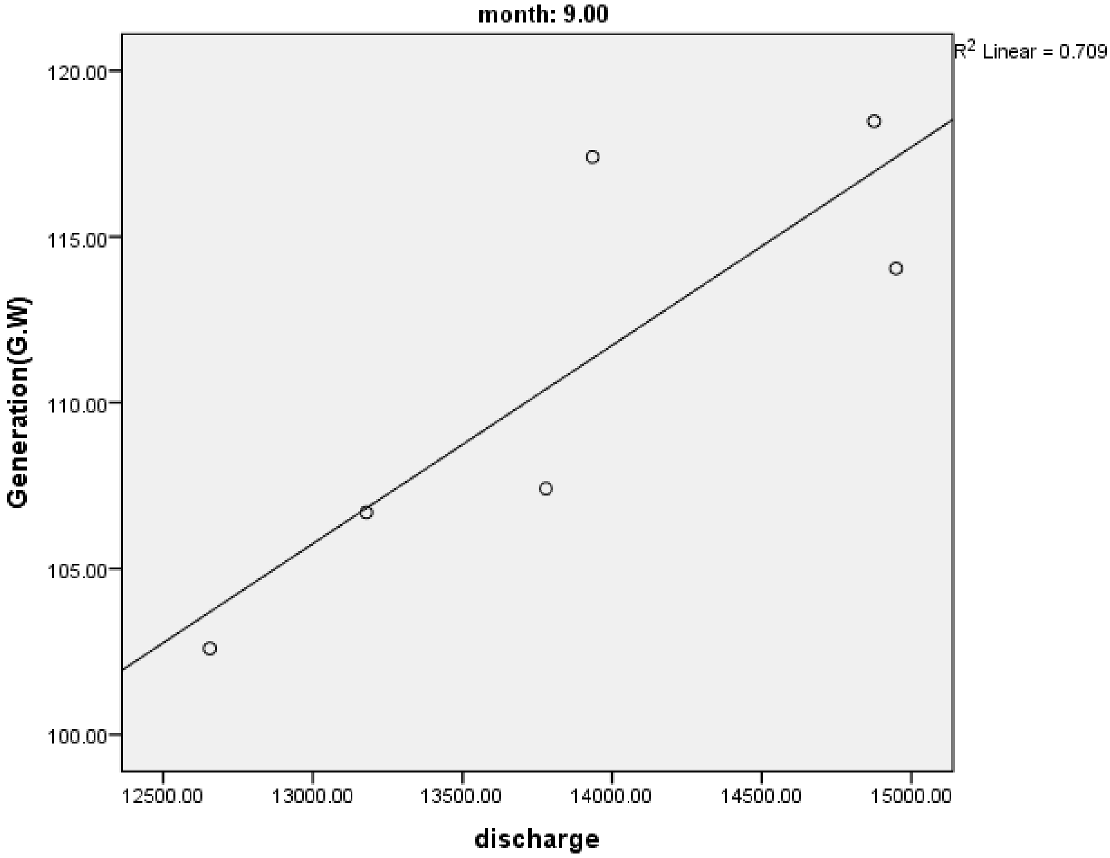


Fig. No.(A.F.9): September Discharge Generation Regression

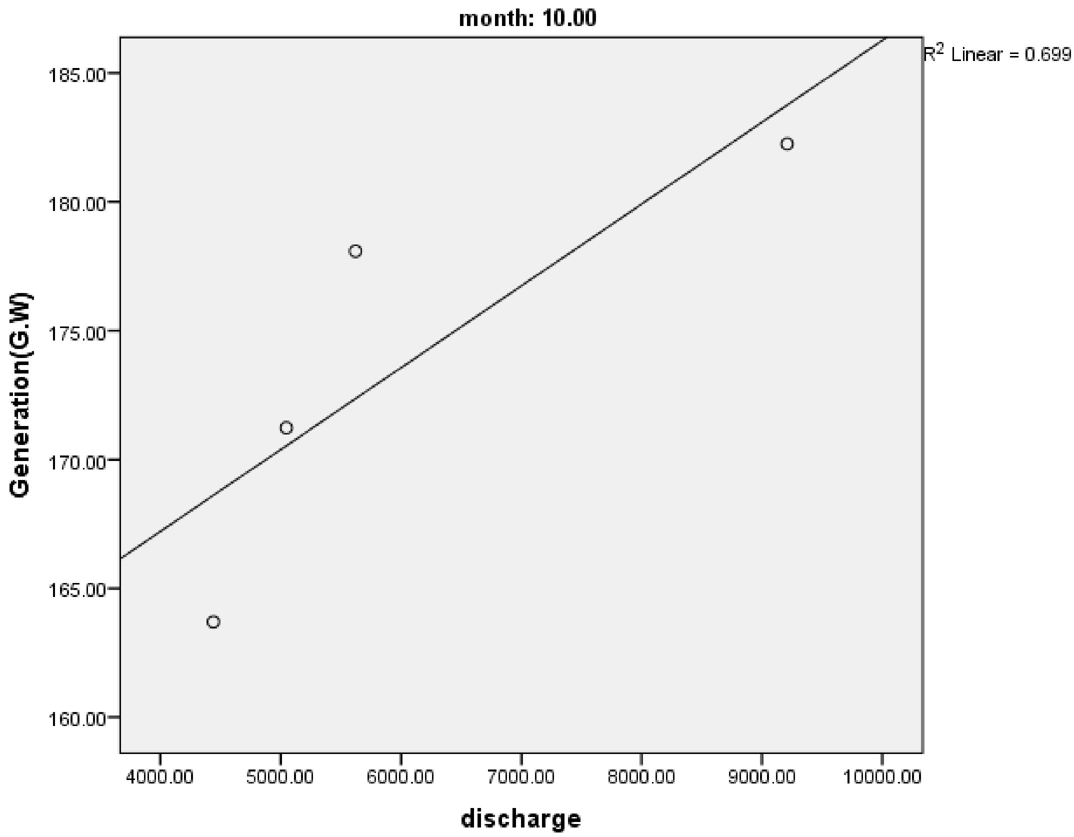


Fig.No.(A.F.10): October Discharge Generation Regression

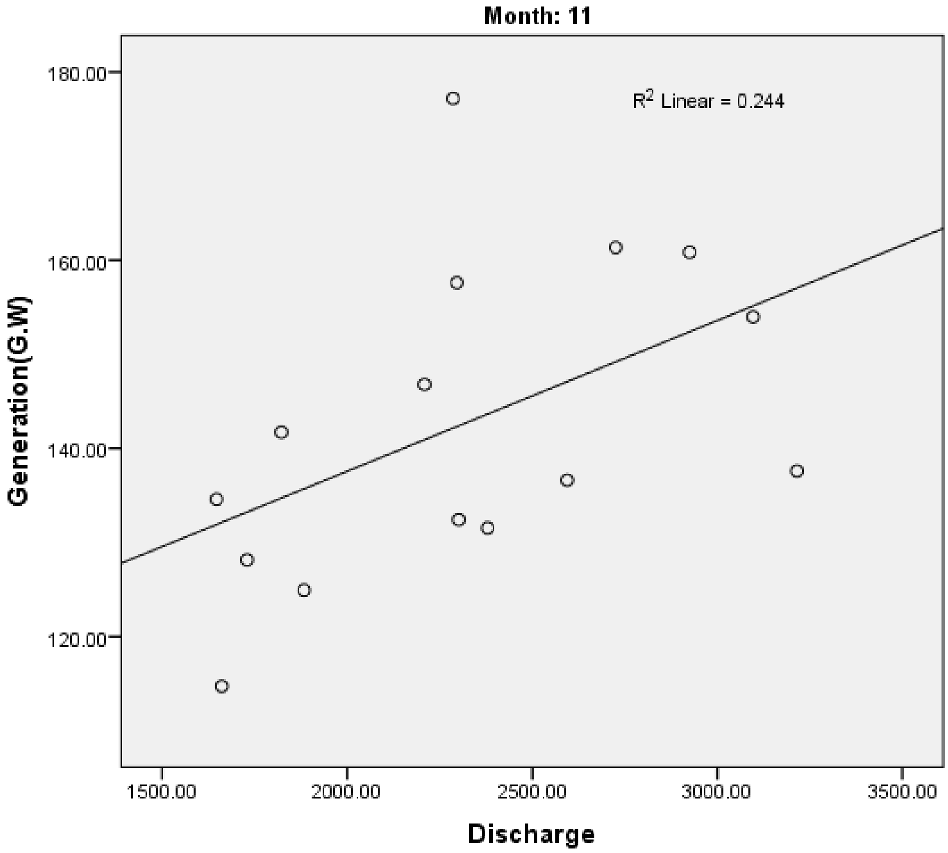
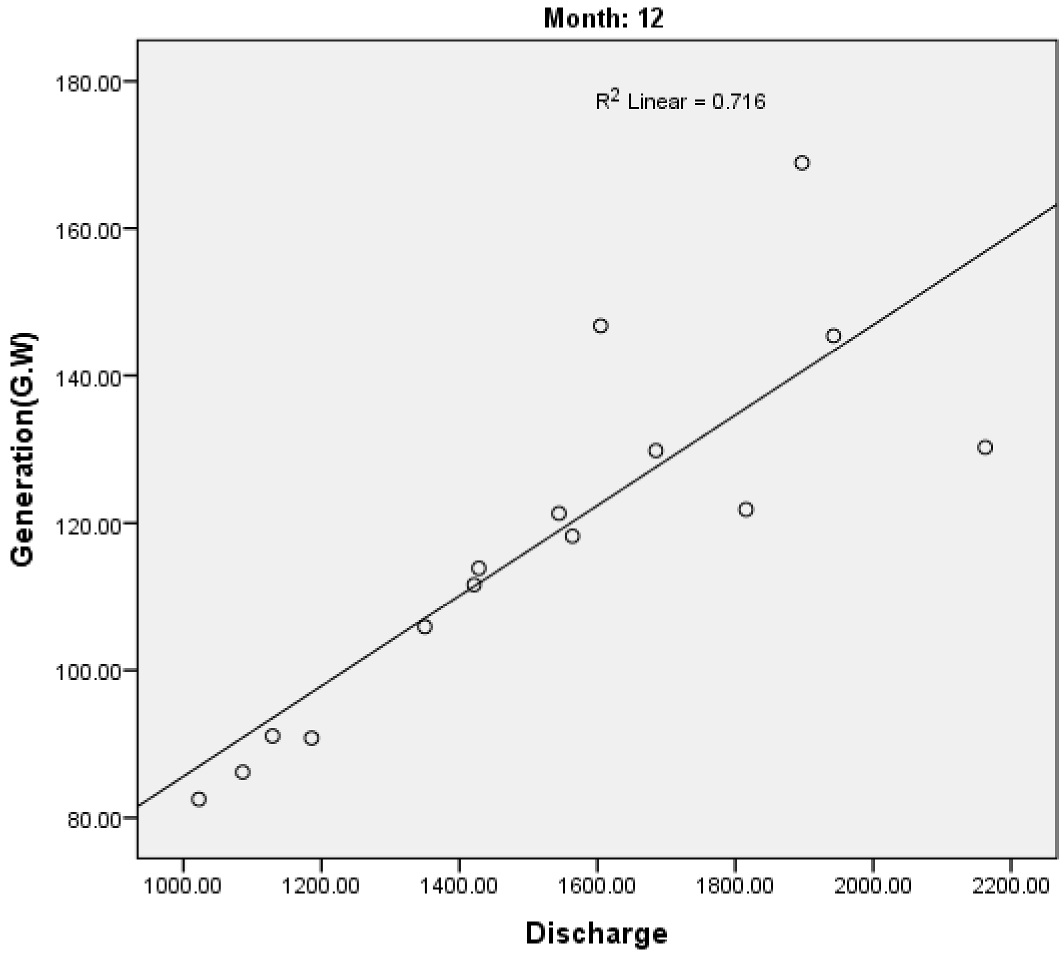


Fig.No.(A.F.11): November Discharge Generation Regression



<u>Fig.No.(A.F.12): December Discharge Generation Regression</u>

Discharge and Power Data

Time	Discharge	Generation	Time	Discharge	Generation	Time	Discharge	Generation
Jan 01	1046.39	87.945	Feb 02	1021.75	82.647	May 16	2904.11	147.151
Jan 02	1160.92	88.717	Feb 03	684.39	64.111	June 01	2896.29	113.928
Jan 03	878.14	75.164	Feb 04	791.77	63.005	June 02	1684.49	93.925
Jan 04	848.98	68.699	Feb 05	896.91	67.225	June 03	1453.15	79.298
Jan 05	902.14	72.302	Feb 06	951.15	68.492	June 04	1716.97	101.784
Jan 06	954.42	72.856	Feb 07	914.54	71.724	June 05	3043.9	126.225
Jan 07	1181.2	92.601	Feb 08	1011.37	77.461	June 06	2664.3	132.87
Jan 08	1241.08	97.126	Feb 09	1312.74	96.164	June 07	4589.33	138.826
Jan 09	1161.1	88.693	Feb 10	968.26	67.824	June 08	5229.51	141.154
Jan 10	1055.48	72.003	Feb 11	979.43	71.104	June 09	1631.98	89.147
Jan 11	1155.47	89.939	Feb 12	1174.52	76.189	June 10	3383.91	135.116
Jan 12	1322.71	94.56	Feb 13	1626.05	139.167	June 11	2890.45	109.465
Jan 13	1546.06	145.436	Feb 14	1482.06	134.761	June 12	1966.1	87.779
Jan 14	1747.22	151.543	Feb 15	1627.44	122.934	June 13	2852.71	114.401
Jan 15	1744.32	130.87	Feb 16	1384.46	119.831	June 14	3995.35	161.233
Jan 16	1738.63	148.589	Mar 01	1043.69	84.33	June 15	3656.91	101.335
Oct 01	4364.36	140.534	Mar 02	1044.75	82.667	June 16	4054.63	131.893
Oct 02	3222.03	146.147	Mar 03	784.24	72.634	Jul 01	9284.23	89.446
Oct 03	3964.9	166.067	Mar 04	802.09	62.667	Jul 02	6183.73	102.445
Oct 04	5319.32	126.375	Mar 05	747.57	51.822	Jul 03	7797.66	68.635
Oct 05	4460.88	151.765	Mar 06	762.87	54.846	Jul 04	6878.01	69.347
Oct 06	4863.39	148.559	Mar 07	956.79	76.082	Jul 05	9099.92	80.675
Oct 07	4903.66	146.763	Mar 08	917.27	65.977	Jul 06	8917.35	103.88
Oct 08	3717.54	153.217	Mar 09	1077.29	72.859	Jul 07	11538.61	87.42
Oct 09	3870.68	127.702	Mar 10	731.21	46.409	Jul 08	9792.44	108.683
Oct 10	3876.94	171.233	Mar 11	931.31	62.704	Jul 09	6130.9	90.769
Oct 11	3902.6	165.89	Mar 12	617.2	35.092	Jul 10	9557.98	91.353
Oct 12	3390.18	182.094	Mar 13	1907.55	150.814	Jul 11	5688.78	111.545
Oct 13	5577.23	173.488	Mar 14	1303.65	111.397	Jul 12	9340.06	79.855
Oct 14	10131.46	182.246	Mar 15	1882.92	148.22	Jul 13	10297.73	64.71
Oct 15	3475.79	139.295	Mar 16	1631.02	134.436	Jul 14	7973.6	106.105
Nov 01	2594.78	136.617	Apr 01	1355.65	104.134	Jul 15	6281.76	103.853
Nov 02	1729.32	128.15	Apr 02	1328.37	102.847	Aug 01	18512.46	55.134
Nov 03	1646.89	134.59	Apr 03	936.78	79.721	Aug 02	12727.17	83.274
Nov 04	1883.24	124.92	Apr 04	461.29	37.934	Aug 03	14006.48	58.541
Nov 05	2301.41	132.427	Apr 05	548.19	36.345	Aug 04	12493.29	61.153
Nov 06	3215.48	137.601	Apr 06	786.81	54.265	Aug 05	13185.57	77.588
Nov 07	2208.99	146.787	Apr 07	1164	88.263	Aug 06	20460.49	75.698
Nov 08	3097.04	153.976	Apr 08	796.71	55.588	Aug 07	16446.25	81.002
Nov 09	1660.91	114.719	Apr 09	862.42	57.115	Aug 08	18477.45	80.932

Nov 10	1822.01	141.719	Apr 10	330.42	20.125	Aug 09	13108.5	82.646
Nov 11	2296.29	157.612	Apr 11	414.28	28.959	Aug 10	18086.77	50.223
Nov 12	2285.6	177.182	Apr 12	411.36	19.97	Aug 11	14465.03	48.505
Nov 13	2725.65	161.353	Apr 13	1754.1	123.826	Aug 12	17938.6	27.811
Nov 14	2925.19	160.84	Apr 14	1714.37	140.69	Aug 13	17391.52	44.525
Nov 15	2378.92	131.539	Apr 15	1996.45	131.77	Aug 14	15722.18	57.641
Dec 01	1544.3	121.309	Apr 16	1482.21	113.136	Aug 15	9826.93	90.26
Dec 02	1085.77	86.195	May 01	1564.12	107.8	Sep 01	11148.76	93.197
Dec 03	1022.35	82.528	May 02	1108.5	77.6	Sep 02	6124.26	133.325
Dec 04	1129.04	91.095	May 03	659.39	54.365	Sep 03	9233.4	103.586
Dec 05	1185.66	90.809	May 04	845.37	65.596	Sep 04	7209	111.555
Dec 06	1815.6	121.844	May 05	1428.37	96.047	Sep 05	11548.58	96.351
Dec 07	1428.12	113.889	May 06	1756.44	109.073	Sep 06	14115.06	96.184
Dec 08	1684.9	129.814	May 07	1893.7	125.694	Sep 07	15414.96	103.159
Dec 09	1349.8	105.896	May 08	2046.32	138.482	Sep 08	10920.82	109.756
Dec 10	1421.16	111.64	May 09	1072.03	67.319	Sep 09	6703.53	103.693
Dec 11	1563.72	118.229	May 10	874.37	54.292	Sep 10	14103.27	78.414
Dec 12	1604.6	146.792	May 11	890.55	55.538	Sep 11	12513.18	75.471
Dec 13	1896.76	168.874	May 12	629.73	32.242	Sep 12	11217.05	93.591
Dec 14	1942.61	145.43	May 13	1828.78	113.1	Sep 13	8050.43	104.038
Dec 15	2162.58	130.249	May 14	3817.84	174.354	Sep 14	11202.95	144.404
Feb 01	677.39	56.153	May 15	2681.77	147.491	Sep 15	4803.54	118.479

Discharge and Sediment Analysis:-

Appendix B

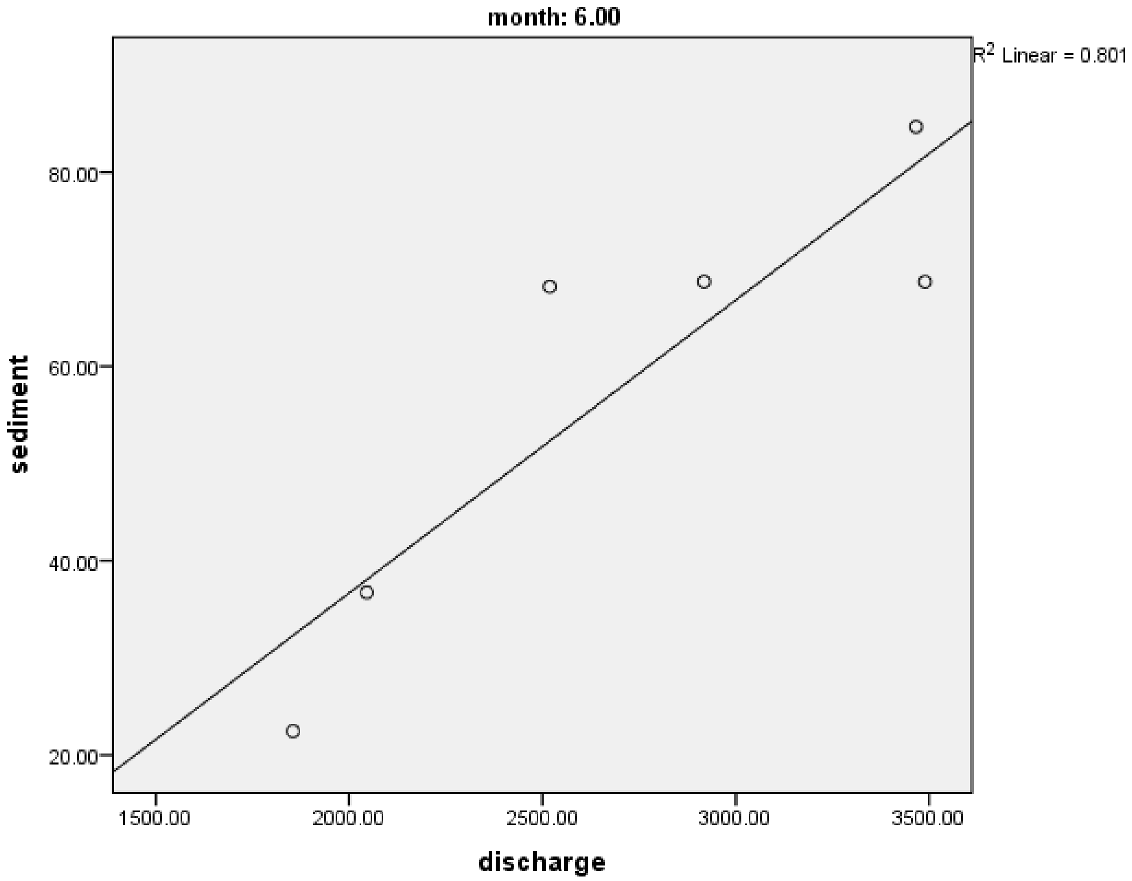


Fig. No.(B.F.1): June Discharge Sediment Regression

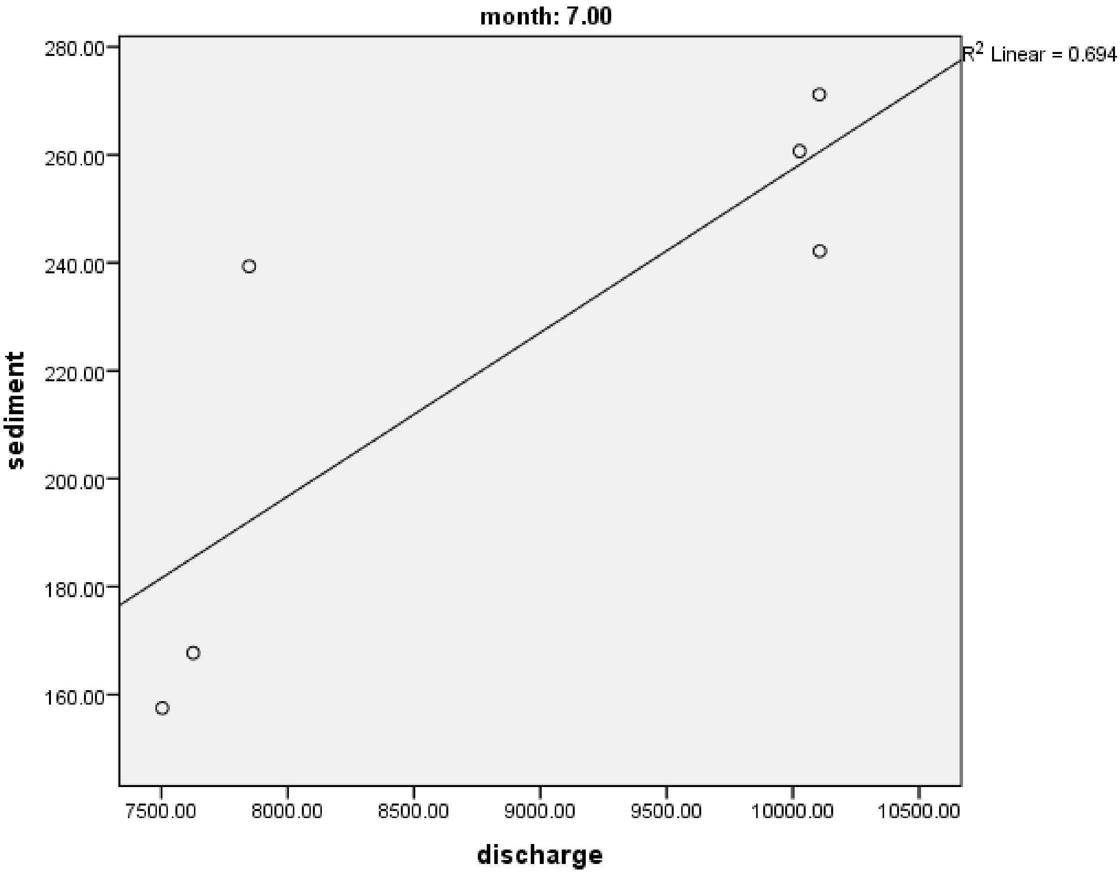


Fig. No.(B.F.2): July Discharge Sediment Regression

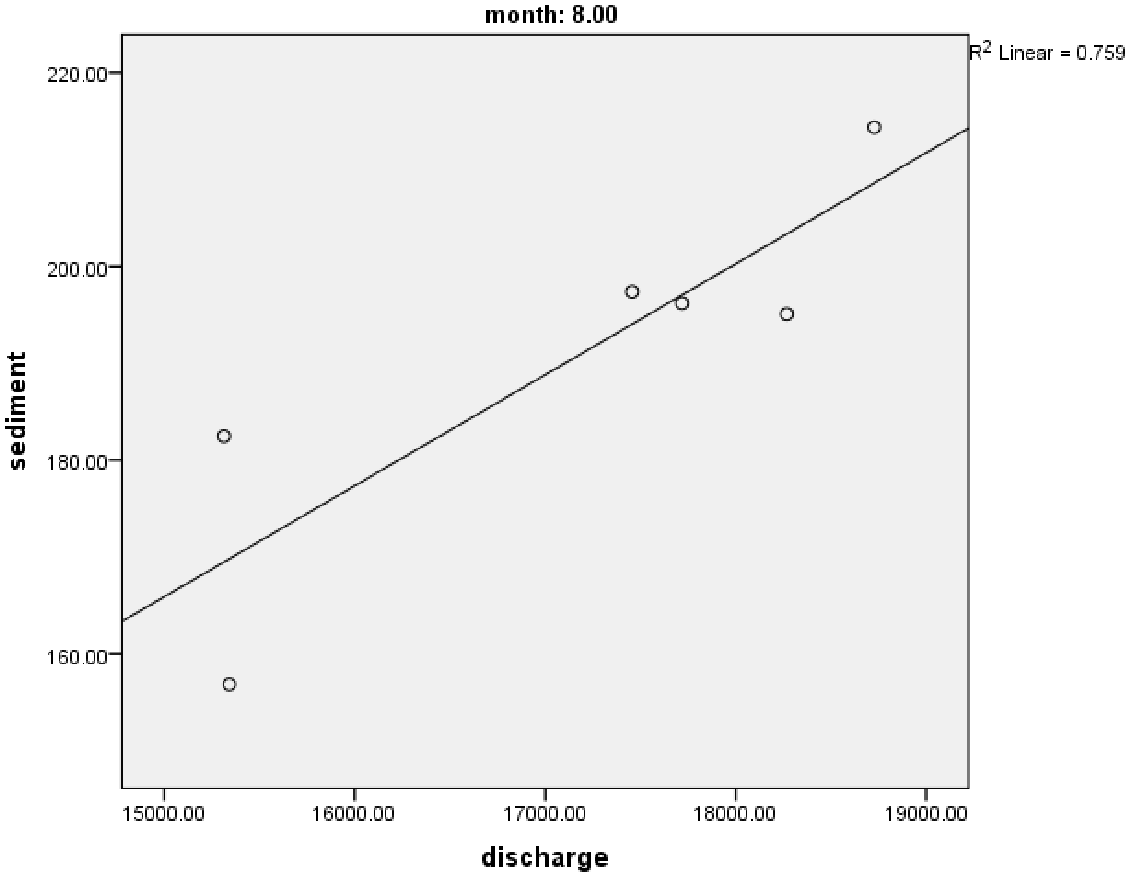
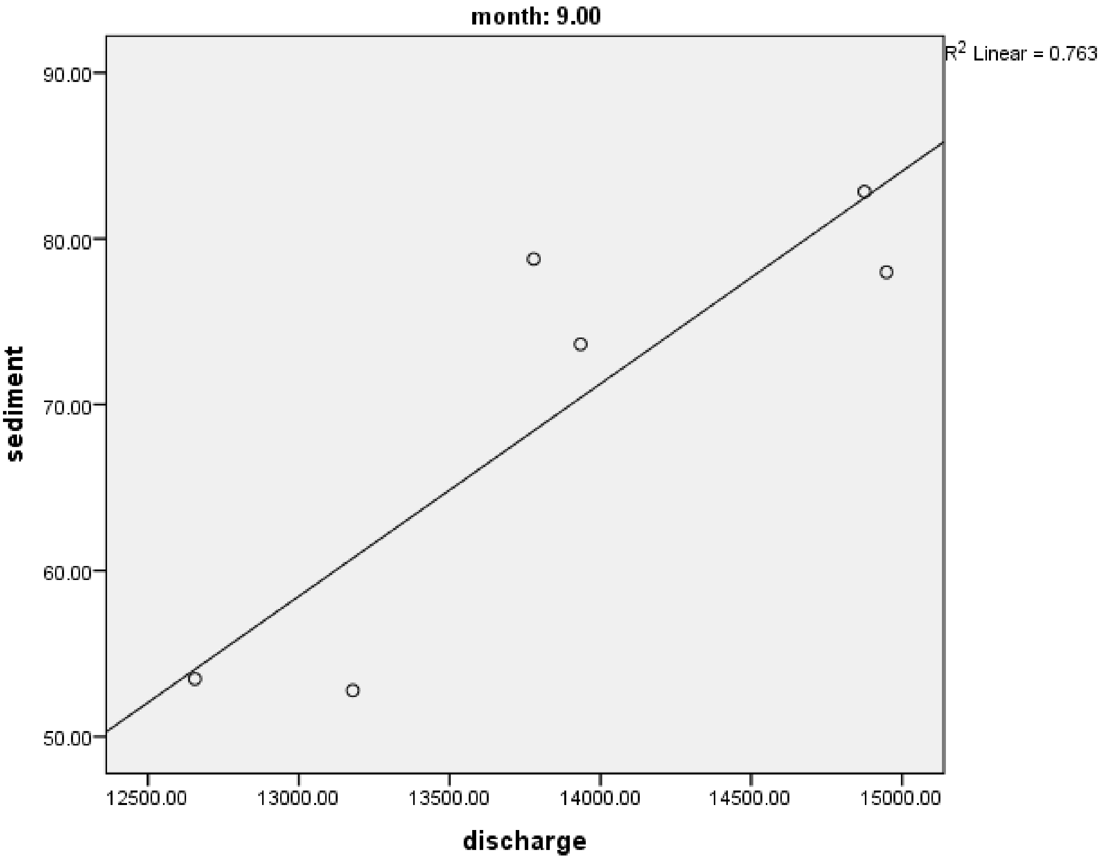


Fig.No.(B.F.3): August Discharge Sediment Regression



<u>Fig.No.(B.F.4): September Discharge Sediment Regression</u>

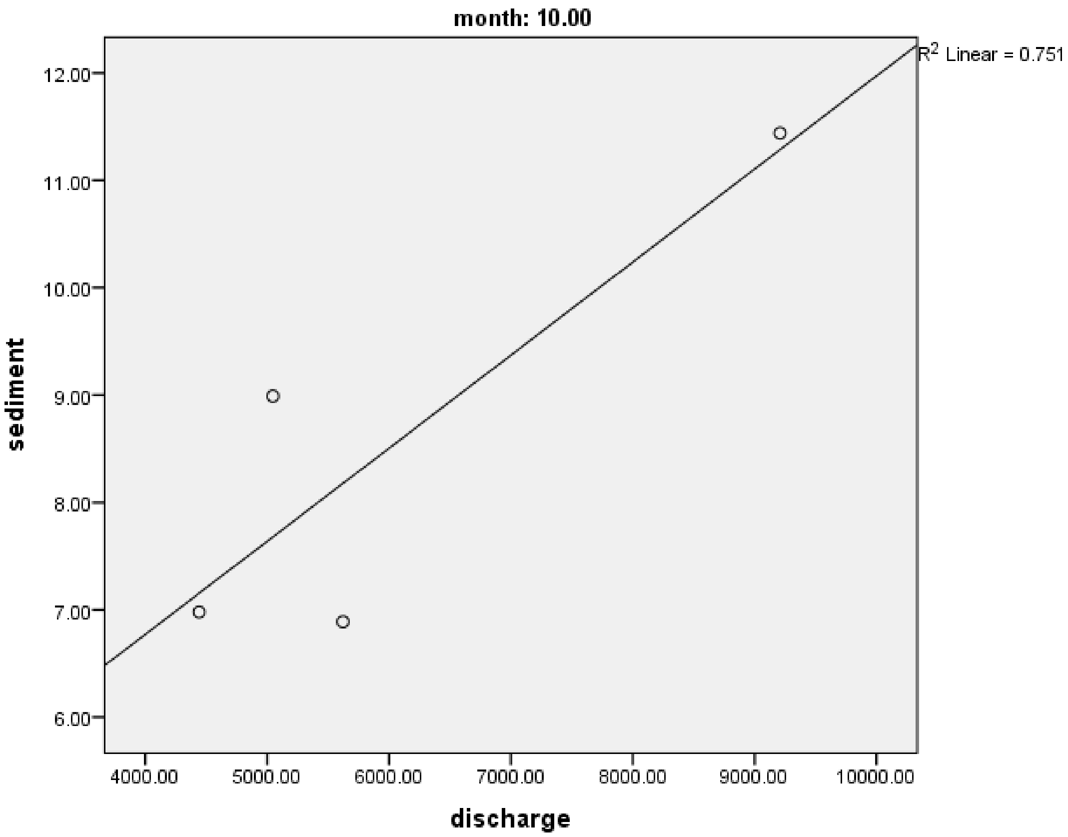


Fig.No.(B.F.5): October Discharge Sediment Regression

Discharge and Sediment Data

Time	Discharge	Sediment	Time	Discharge	Sediment
Jun 09	1854.7	22.45	Aug 11	17456	197.37
Jun 10	3466.4	84.67	Aug 12	18267.5	195.07
Jun 11	2917.8	68.73	Aug 13	17718	196.18
Jun 12	2045.7	36.73	Aug 14	15312.83	182.46
Jun 13	2518.8	68.22	Sep 09	13179	52.78
Jun 14	3489.57	68.72	Sep 10	13778.8	78.77
Jul 09	7503.3	157.49	Sep 11	12655.6	53.47
Jul 10	10106.6	242.16	Sep 12	14948.3	77.98
Jul 11	7847.1	239.33	Sep 13	13934	73.64
Jul 12	10105.3	271.17	Sep 14	14875.19	82.84
Jul 13	10026.8	260.68	Oct 09	4442.2	6.98
Jul 14	7625.51	167.71	Oct 10	5046.8	8.99
Aug 09	15341.3	156.83	Oct 11	5622.7	6.89
Aug 10	18727.3	214.35	Oct 12	9208.65	11.44

Discharge and Water Requirement Analysis:-

Appendix C

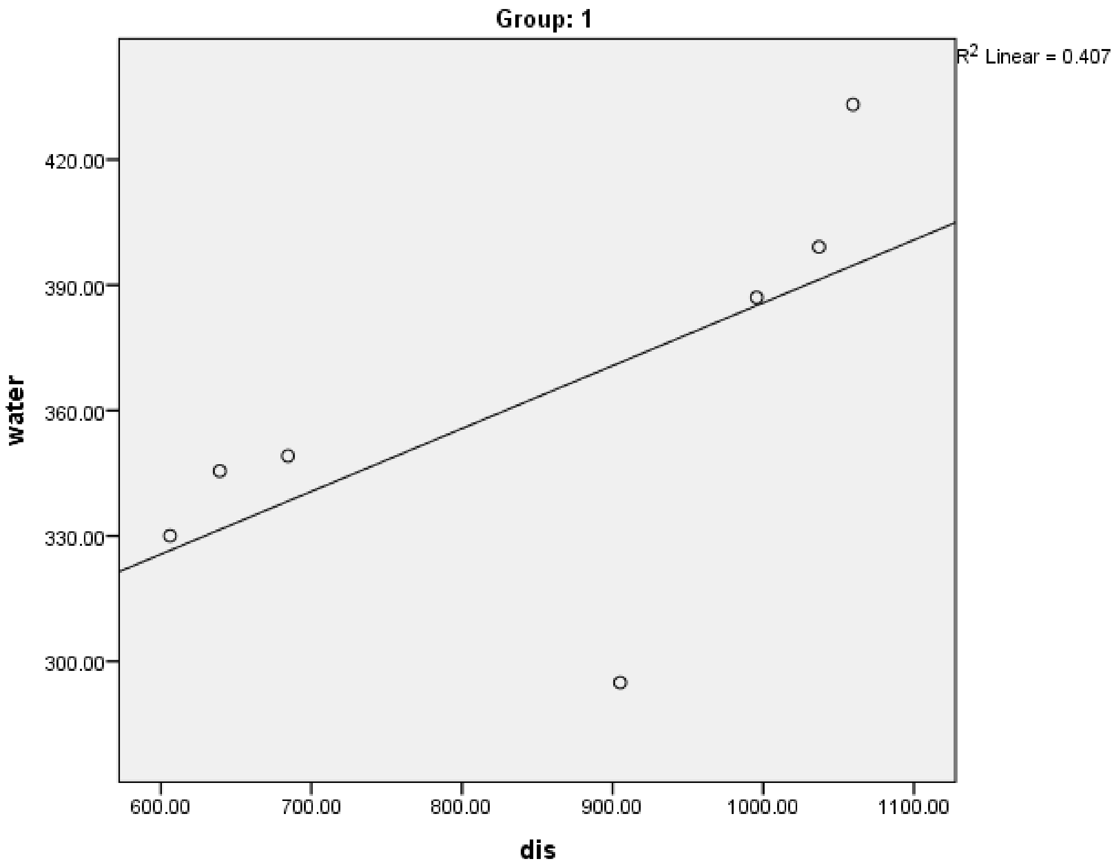
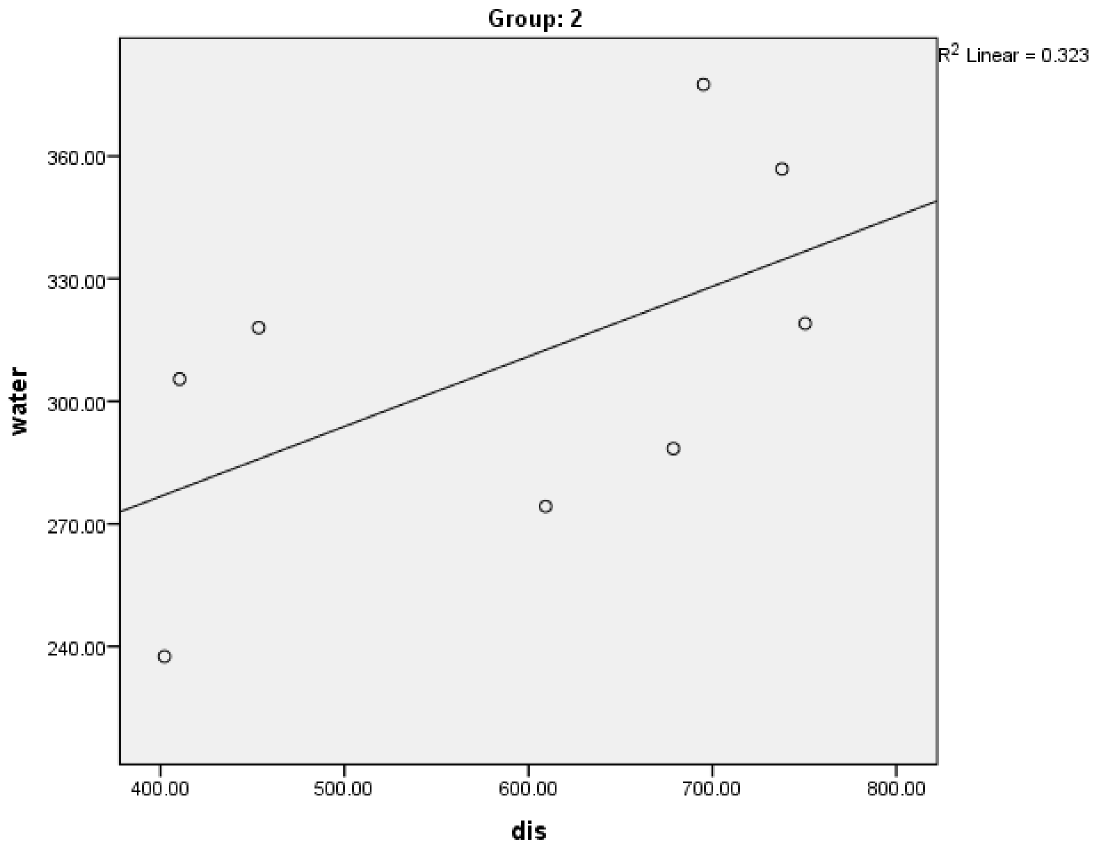


Fig. No.(C.F.1): January Discharge Water Requirment Regression



<u>Fig.No.(C.F.2): February Discharge Water Requirment Regression</u>

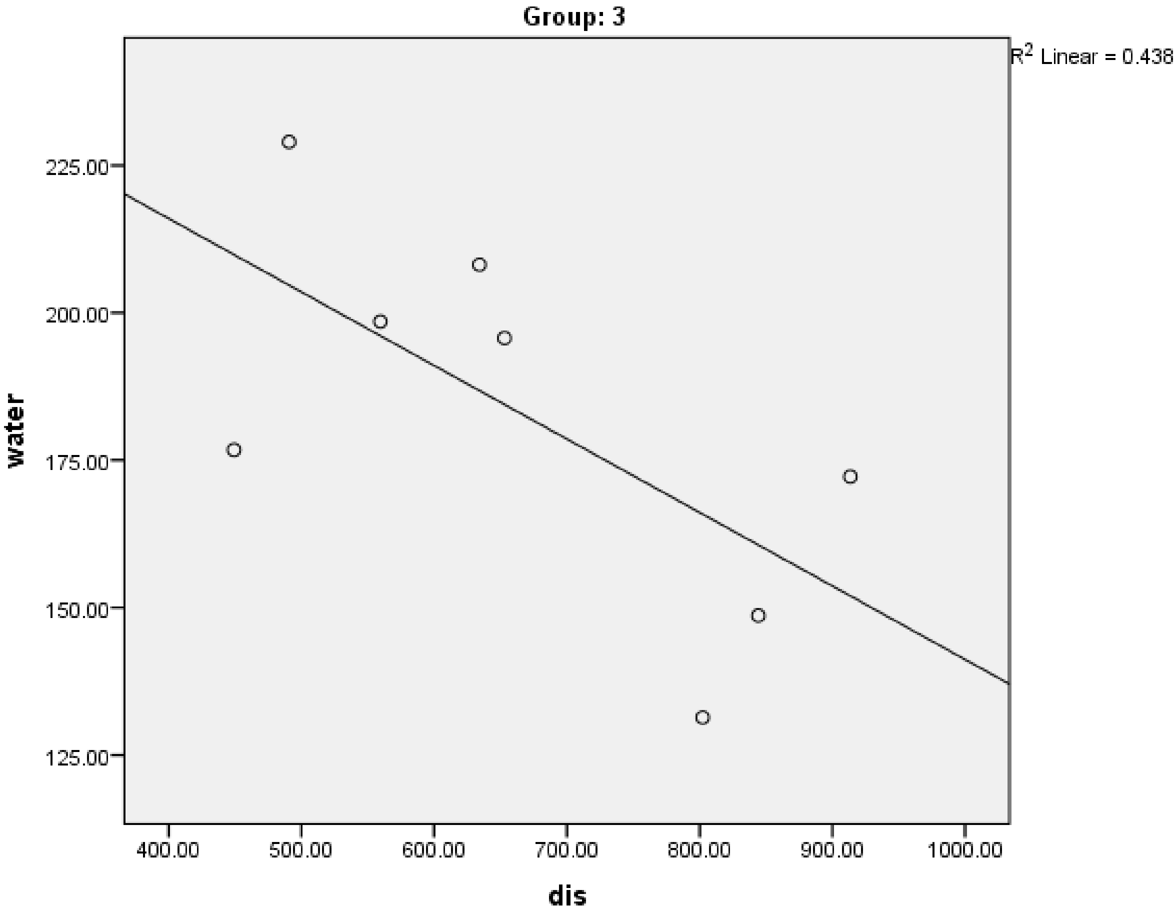


Fig.No.(C.F.3): March Discharge Water Requirment Regression

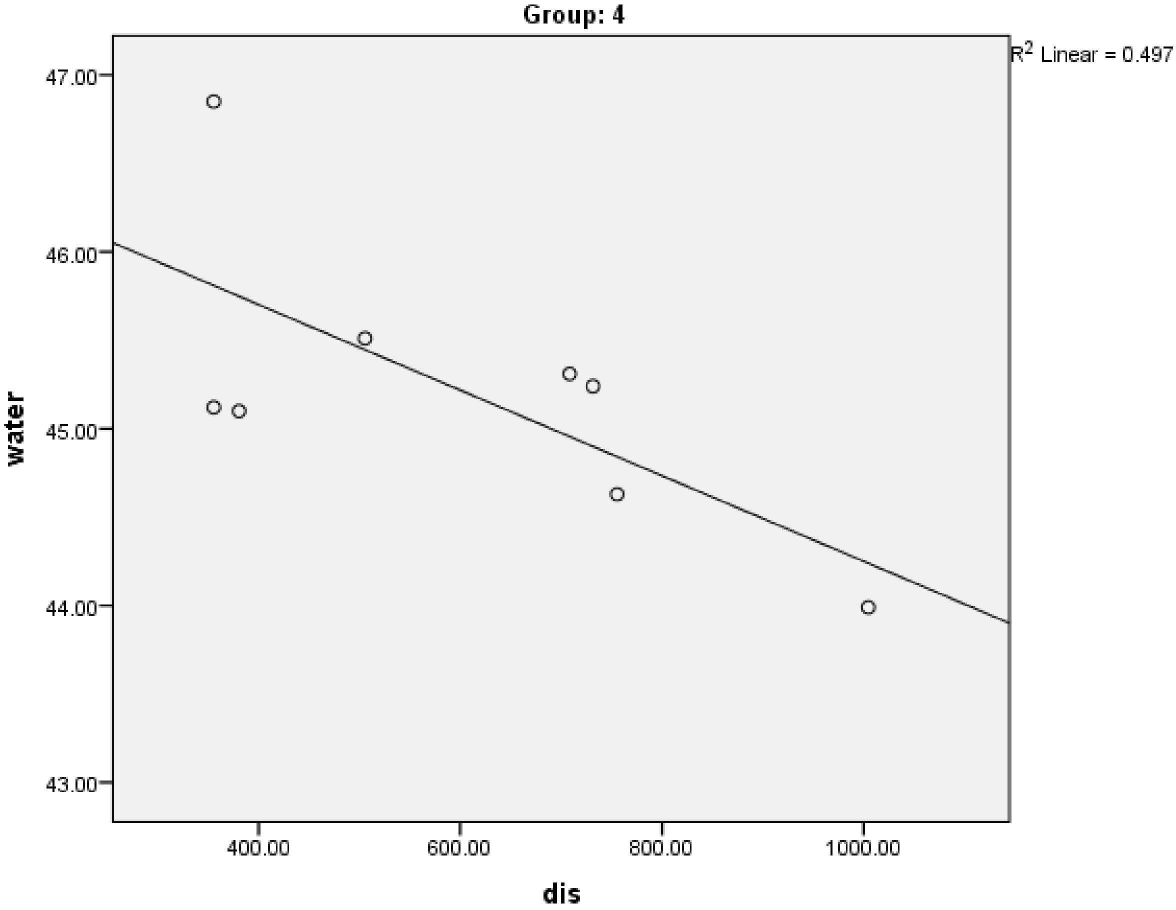


Fig. No.(C.F.4): April Discharge Water Requirement Regression

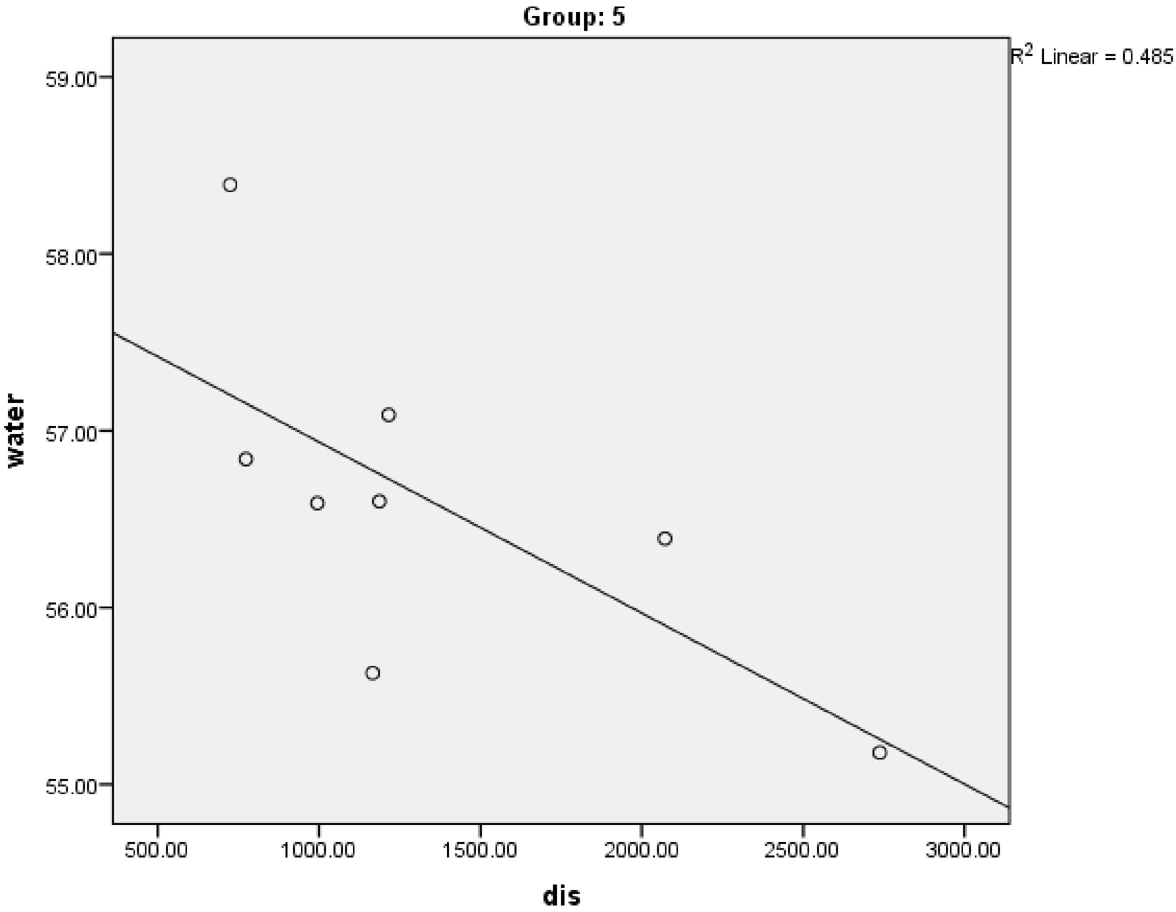
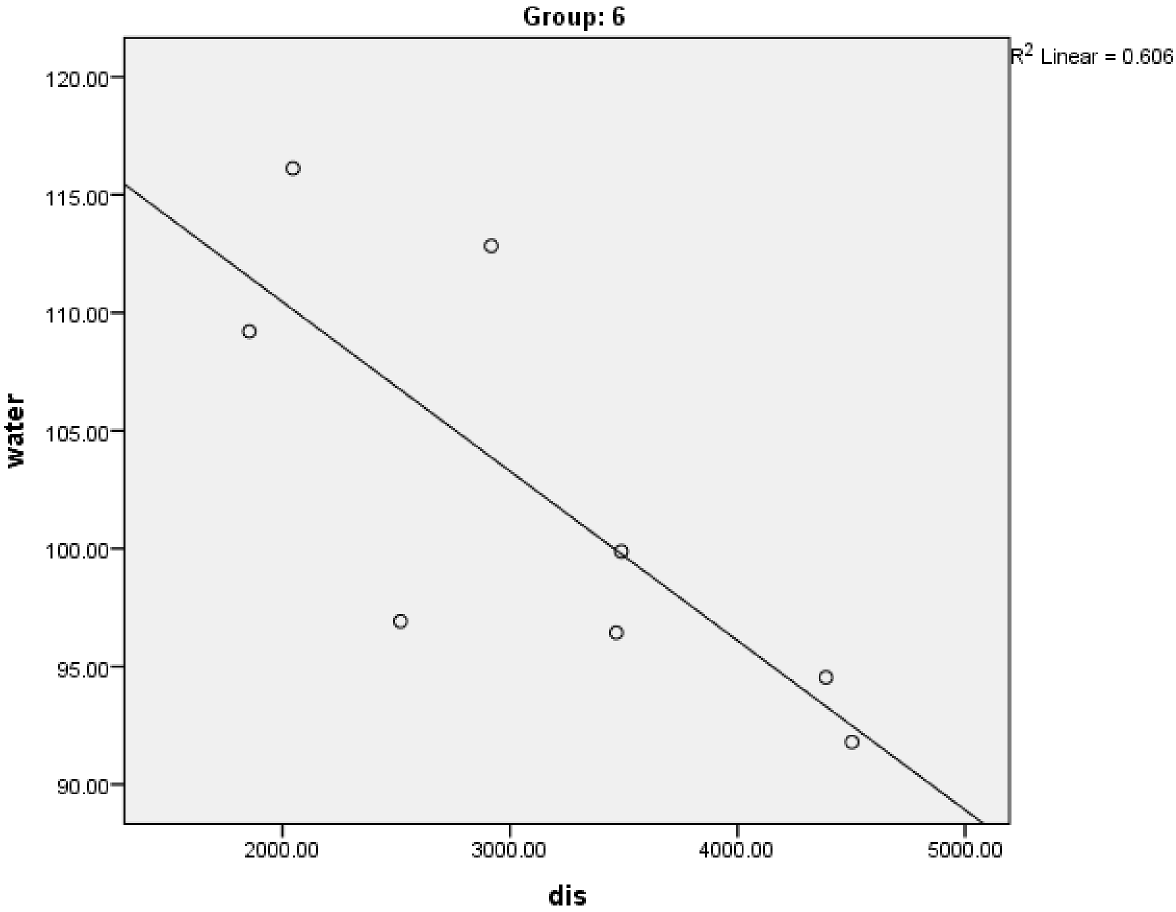


Fig.No.(C.F.5): May Discharge Water Requirement Regression



<u>Fig.No.(C.F.6): June Discharge Water Requirement Regression</u>

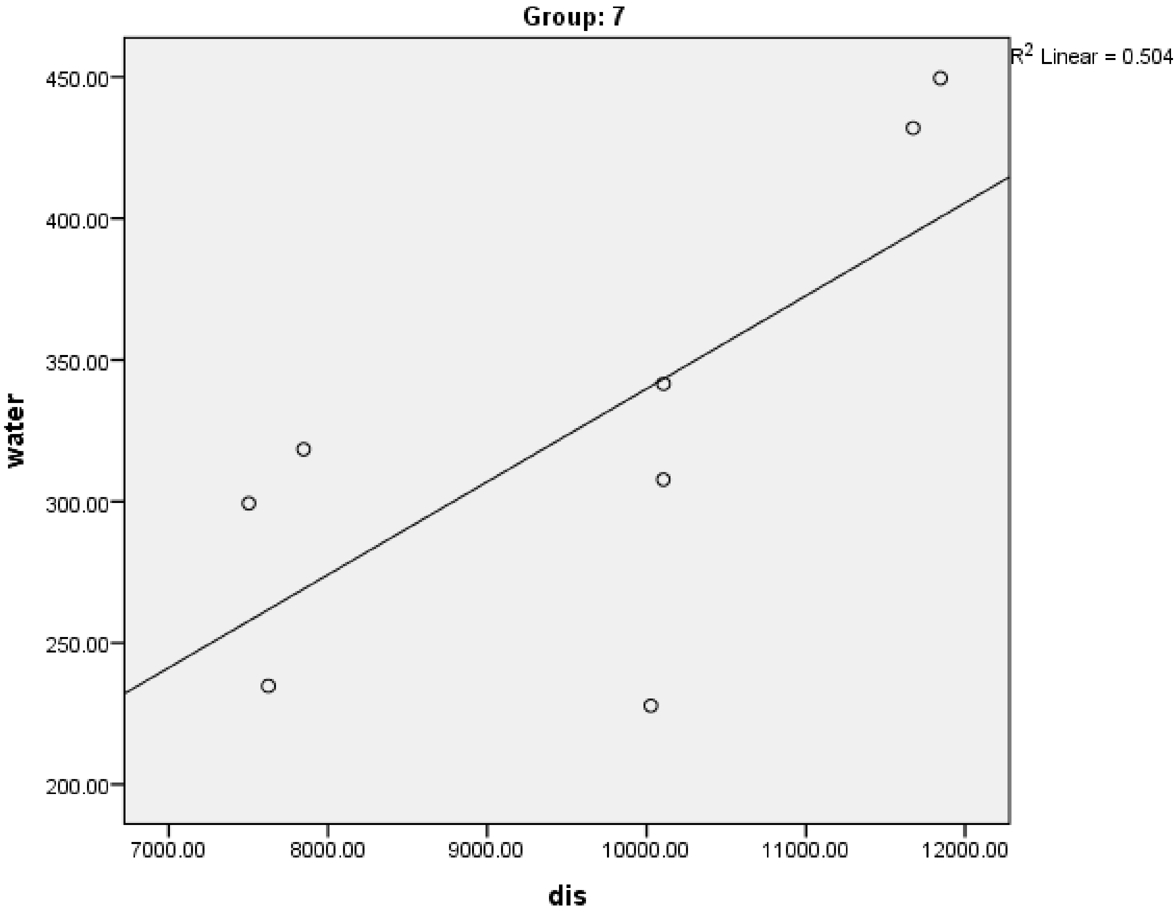
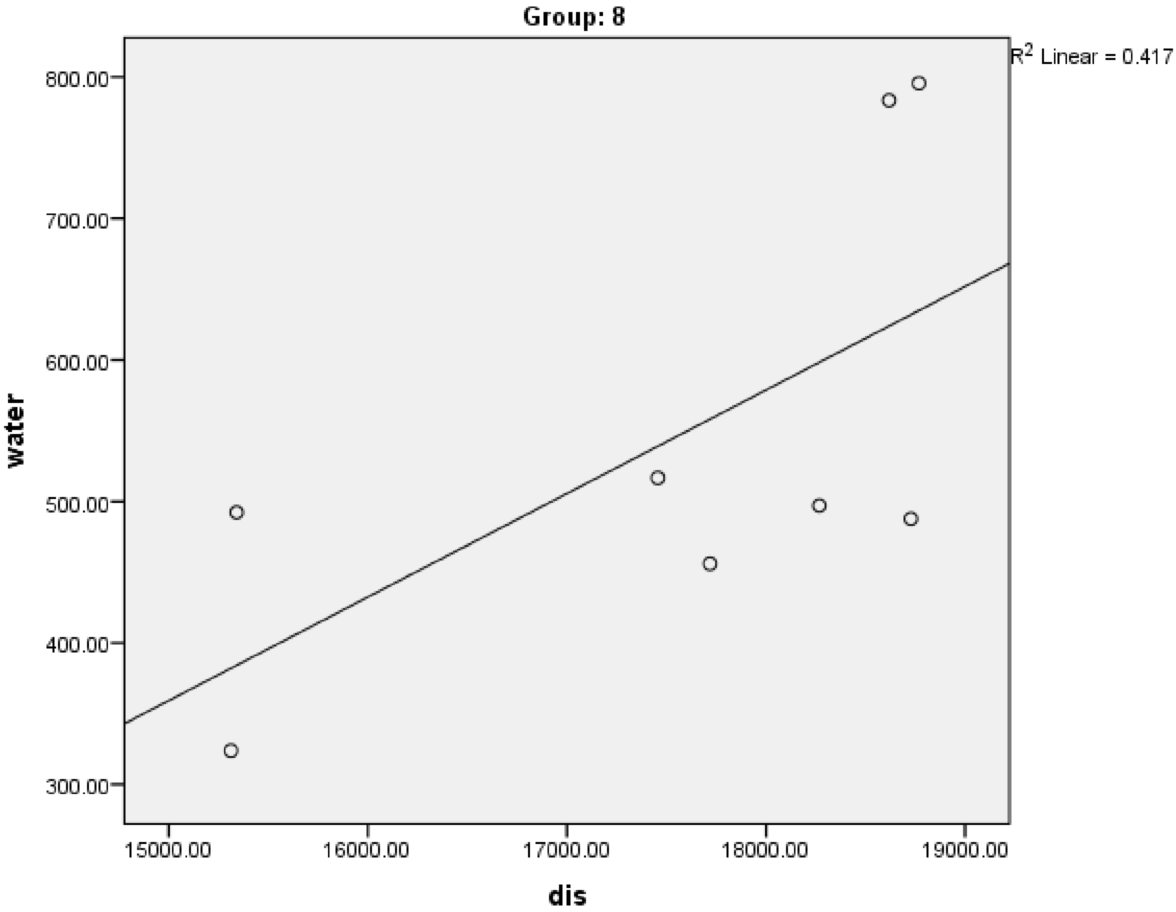


Fig.No.(C.F.7): July Discharge Water Requirment Regression



<u>Fig.No.(C.F.8): August Discharge Water Requirement Regression</u>

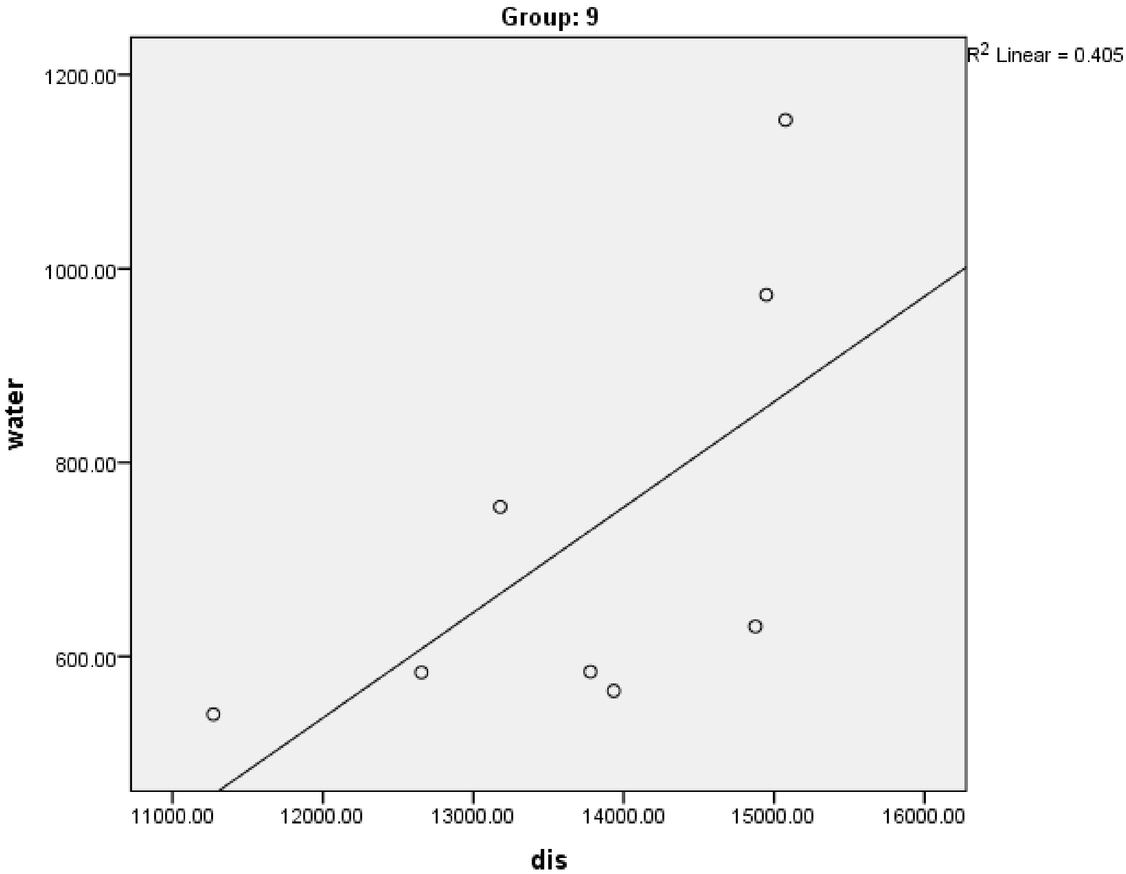


Fig.No.(C.F.9): September Discharge Water Requirement Regression

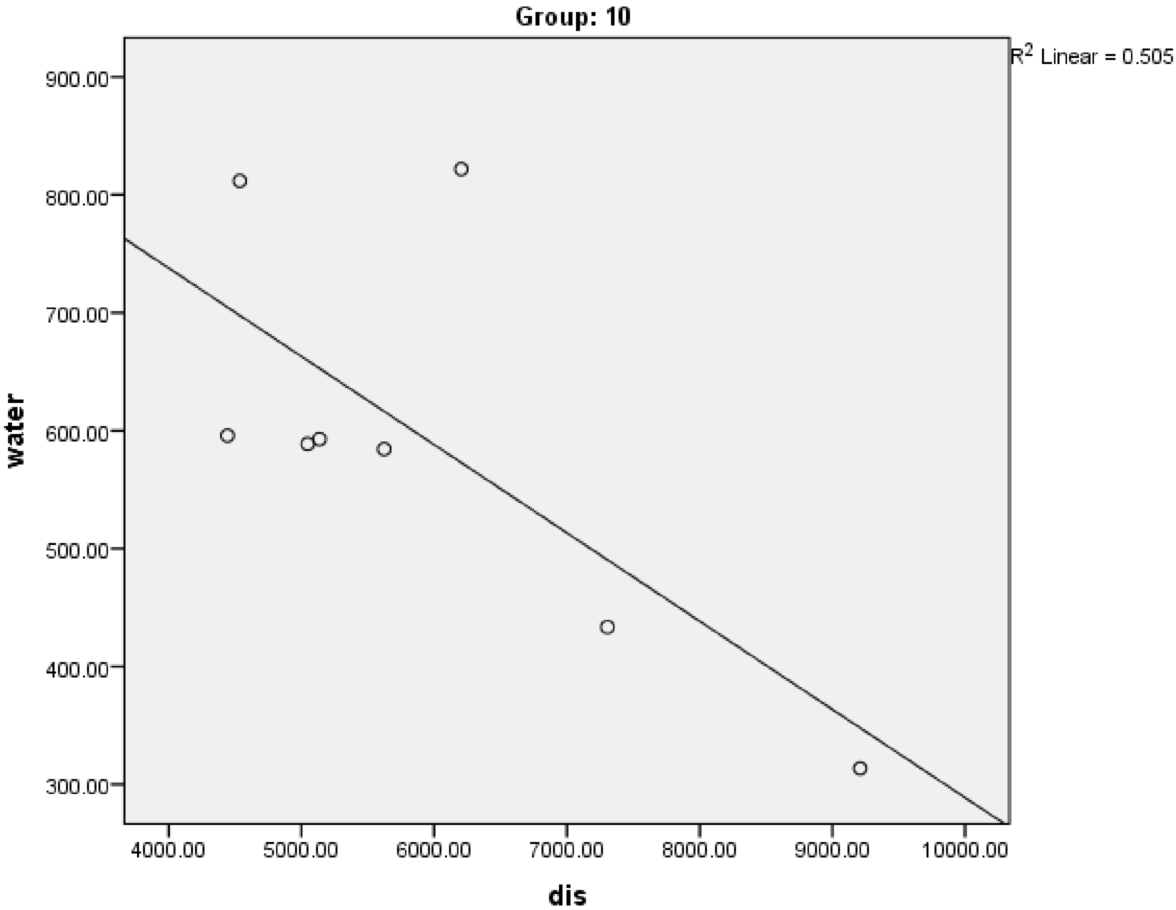
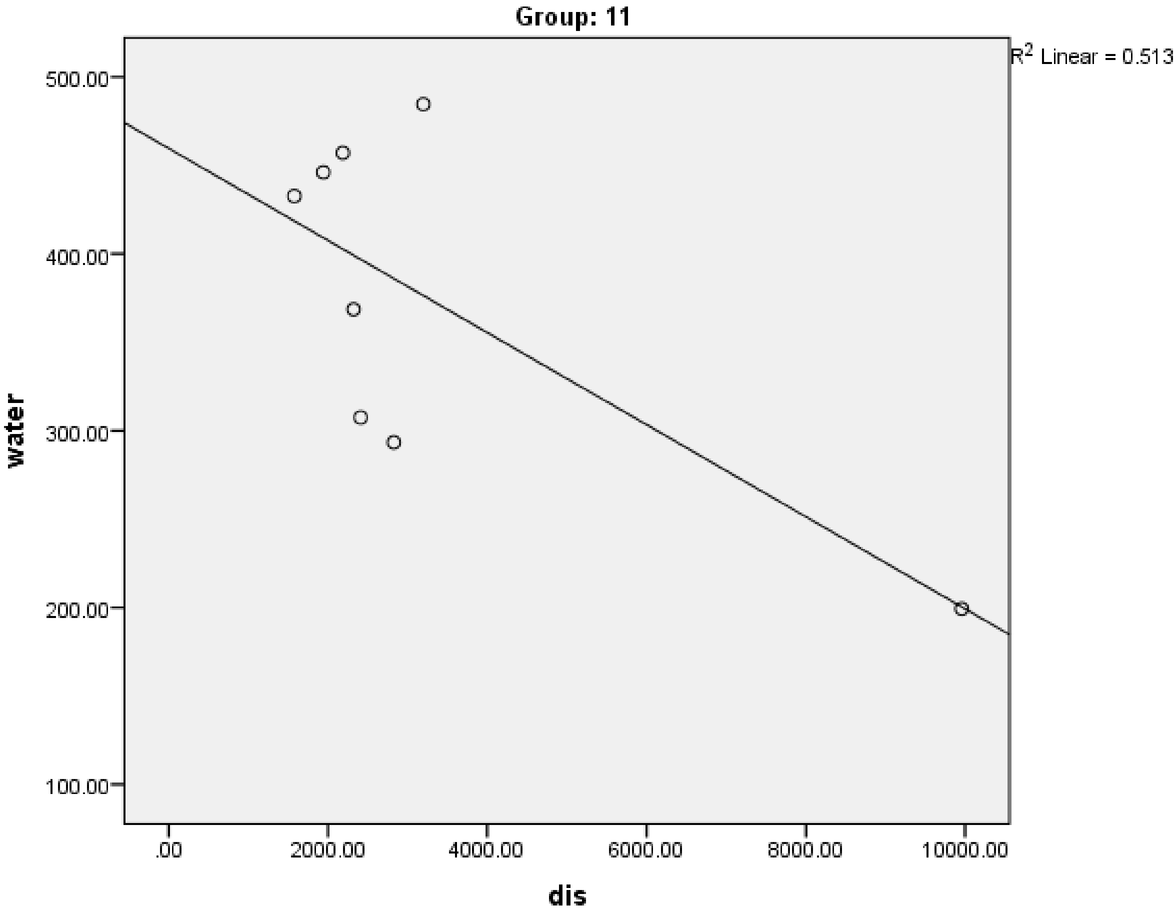


Fig.No.(C.F.10): October Discharge Water Requirement Regression



<u>Fig.No.(C.F.11): November Discharge Water Requirement Regression</u>

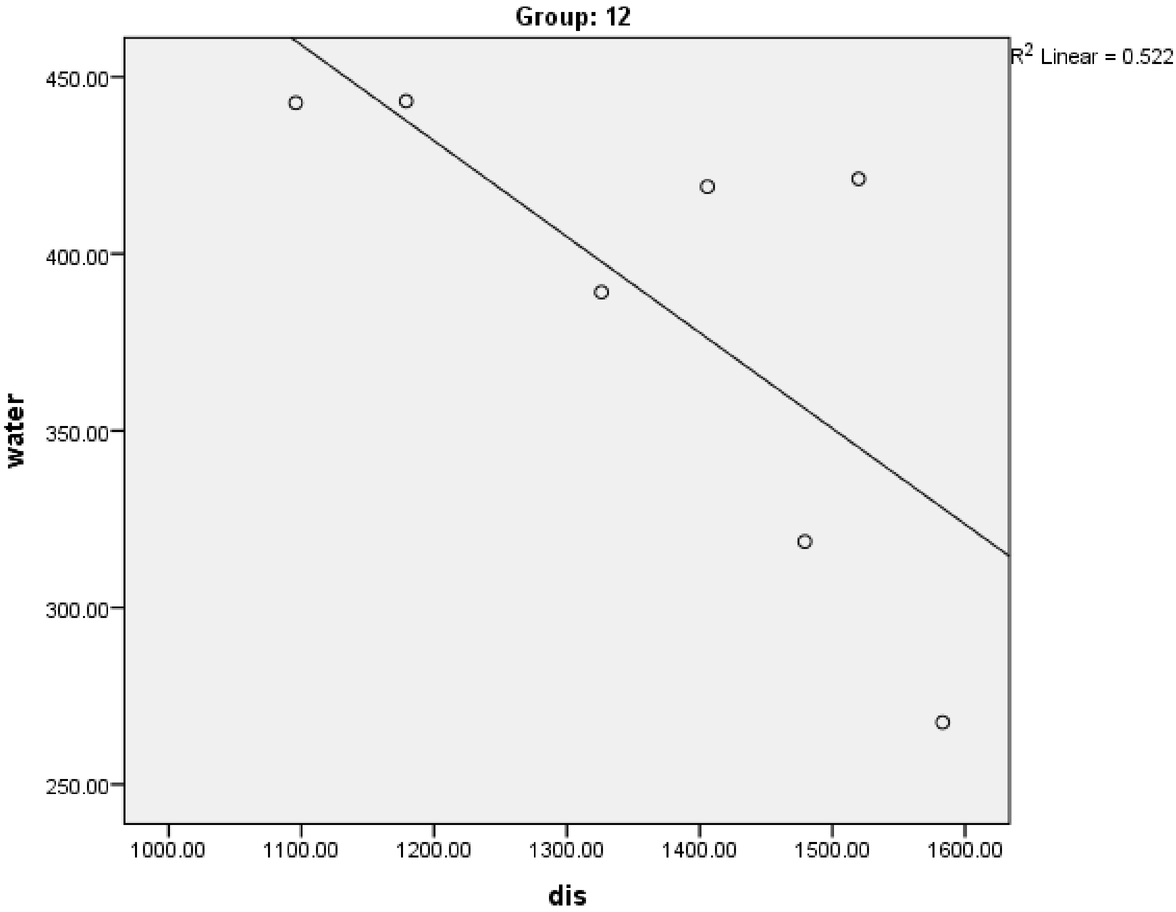


Fig. No.(C.F.12): December Discharge Water Requirement Regression

Discharge and Water Requirement Data

Date	Qw M3/Month	Agriculture Areas feddan	Date	Qw M3/Month	Agriculture Areas feddan
Jan 08	995.6	387.04	Jul 07	11672.6	431.92
Jan 09	1059.6	433.17	Jul 08	11844.2	449.57
Jan 10	606.1	330	Jul 09	7503.3	299.33
Jan 11	684.4	349.12	Jul 10	10106.6	341.58
Jan 12	639.1	345.51	Jul 11	7847.1	318.47
Jan 13	905	294.91	Jul 12	10105.3	307.72
Jan 14	1037	399.09	Jul 13	10026.8	227.88
Feb 07	750.4	319.03	Jul 14	7625.51	234.78
Feb 08	695.1	377.51	Aug 07	18767.6	795.53
Feb 09	737.8	356.84	Aug 08	18617.5	783.5
Feb 10	410.3	305.42	Aug 09	15341.3	492.24
Feb 11	453.3	318	Aug 10	18727.3	487.75
Feb 12	402.1	237.53	Aug 11	17456	516.61
Feb 13	609.3	274.29	Aug 12	18267.5	497.08
Feb 14	678.74	288.41	Aug 13	17718	456.05
Mar 07	802.3	131.37	Aug 14	15312.83	323.96
Mar 08	634.2	208.14	Sep 07	15075.7	1153.43
Mar 09	653	195.71	Sep 08	11271.2	540.36
Mar 10	559.5	198.52	Sep 09	13179	754.23
Mar 11	490.7	228.95	Sep 10	13778.8	584.21
Mar 12	449.3	176.76	Sep 11	12655.6	583.5
Mar 13	913.4	172.21	Sep 12	14948.3	972.9
Mar 14	844.11	148.64	Sep 13	13934	564.33
Apr 07	755.4	44.63	Sep 14	14875.19	630.88
Apr 08	731.5	45.24	Oct 07	6203.2	821.87
Apr 09	708.5	45.31	Oct 08	4535.1	811.86
Apr 10	355.5	45.12	Oct 09	4442.2	595.91
Apr 11	380.6	45.1	Oct 10	5046.8	588.75
Apr 12	355.5	46.85	Oct 11	5135.7	592.89
Apr 13	505.6	45.51	Oct 12	5622.7	584.47
Apr 14	1004.58	43.99	Oct 13	7305.3	433.54
May 07	1165.9	55.63	Oct 14	9208.65	313.66
May 08	2071.7	56.39	Nov 07	2185.9	457.05
May 09	773.5	56.84	Nov 08	3195.9	484.57
May 10	1187	56.6	Nov 09	1577	432.63

May 11	994.4	56.59	Nov 10	1942.2	446.1
May 12	724.5	58.39	Nov 11	2321.9	368.66
May 13	1215.8	57.09	Nov 12	2410.4	307.48
May 14	2737.48	55.18	Nov 13	2826	293.37
Jun 07	4388.3	94.54	Nov 14	9957.62	199.33
Jun 08	4502	91.8	Dec 07	1405.8	418.97
Jun 09	1854.7	109.21	Dec 08	1519.7	421.19
Jun 10	3466.4	96.44	Dec 09	1178.8	443.17
Jun 11	2917.8	112.83	Dec 10	1095.8	442.69
Jun 12	2045.7	116.12	Dec 11	1326	389.11
Jun 13	2518.8	96.92	Dec 12	1479.2	318.68
Jun 14	3489.57	99.88	Dec 13	1583	267.55