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EDITORIAL NOTE

It is a pleasure to inform our readers and researchers that "Technical Journal" will be published a semiannual. This issue is the first issue (January-June) for Volume 18, 2013. We have also succeeded in getting our journal indexed and abstract in Pro Quest services which include the following database.

- ProQuest Engineering Journal
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Besides this the journal has been added for abstracting in "Agris" for papers related to water resources, agriculture etc.

For the current issue, we received 15 papers and all of them were checked for similarities through the Turnitin software. As per the university policy, papers that have a similarity index above 20%, are not accepted for publication. Thus out of the 15 papers received, only 7 were selected for publication. This was due to a high similarity index, reviewers comments, and other discrepancies. The selected papers were then forwarded to be evaluated by three independant reviewers out of which one is selected from international universities/agencies, and based on their assessment; the papers were revised by the authors again.

In addition to the research papers that were received from various universities all over Pakistan, we also received two international submissions from Egypt (Cases of Slope failure of Irrigation and Drainage Channels in Egypt and Their Rehabilitation) and Iran (Estimation of Traffic Accident Costs: A simple equation adapted for Islamic Countries). These two research papers shows interest of Regional and International participation. The publication encompasses a wide range of topics, including research papers on Water Resources, Road Safety, Computer and Mechanical Engineering.

Nuzhat Yasmin
Editor

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• CONTENTS

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Cases of Slope Failure of Irrigation And Drainage Channels In Egypt and Their Rehabilitation

Sherif M. Elkholy[1]

Abstract

Side slope failures of Irrigation and drainage water channels may cause major disasters with devastating losses of both human life and property, especially in the case where such water channels are higher in level than urban areas along with a complete failure of the embankment. Additional live loads, seepage forces, erosion, and/or gravitational forces may instigate complete or partial slope failure of irrigation and drainage channels. The present article looks at some case studies of slope failure trying to identify the causes of failure and consequently recommending the suitable technique of rehabilitation in the case of partially collapsed slope or reconstruction in the case of a complete failure. The reasons behind the slope failures of the studied channels were mainly piping (migration of soil particle or internal erosion) in salt affected or dispersive soil, seepage forces in sandy or clayey soil, poor quality of the embankment soil mass as in the case of soft clay or loose sand, overtopping and consequently erosion caused by excessive use of irrigation water or using surface irrigation at areas close to the embankment, high velocity of the flow in the channel, and/or mechanical dredging which distorts the channel cross section. There is no single or general way of rehabilitation for slope failures because of the uniqueness of the characteristics and circumstances of each site. Therefore, each site has to be studied separately and in details to find out the best way to deal with it.

Keywords : Piping, Rehabilitation, Seepage, Slope Failures.

Introduction

Stable earth slopes, both natural and man-made, are of great importance as demonstrated by the consequences of slope failures given in the Cedergren (1977). Slope failures may be attributed to many reasons such as, seepage, piping, and/or excessive settlement (Burgi and Karaki, 1971; Rhee and Bezuijen, 1992; Govindaraju, 1998, and Budhu and Gobin, 1996). Studies on the failure mechanisms of the banks of Ohio River (Hagerty and Spoor, 1989) and the Illinois water way (Spoor and Hagerty,1989) identified piping and seepage as causes of widespread failures in their banks. Other investigators emphasized on the importance of the emerging seepage in failure of alluvial Monogahela River (Hamel, 1988) and the Mississippi River (Nakato and Anderson, 1998). Hagerty and Parola (2001) concluded in their study that the riprap revetments even those protected by filters might fail as a result of excessive seepage. Furthermore, leaching of salts in soil with high salt concentration leads to internal erosion and successive failure of side slope and embankment (El-Ashaal et al., 2003 and Heza et al., 2003). Embankments constructed on soft clay foundation typically have a potential failure mode in the form of an approximately circular slip surface that extends into the soft foundation. Failure, if it occurs, will follow the path along which the factor of safety is the minimum (Low, 1989).

El-Ashaal, Hikal, and Abdel-Motaleb (2000) demonstrated the rehabilitation and upgrading of a cracked zoned fill dike. They indicated that the rehabilitation works were divided into two stages. The first one was to carry out urgent protection measure by providing a stabilizing mass of graded sandstone to support the dike back slope and the second one was to execute a permanent protection measure by providing a stabilizing prism of loamy sand as an extension for the berm at the upstream dike slope. Stability of the embankment slopes may be achieved using riprap revetment and such revetment can be protected against seepage using filters (Hagerty and Parola, 2001). Biotechnical stabilization of slopes was also used as a measure for slope protection (Gry and Sotir, 1992). Using drains is considered as one of the most effective methods for improving the stability of slopes when an unfavorable

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ground-water condition is the main cause of instability (Cedergren, 1977, Zaruba and Mencl, 1982, and Resnick and Znidarcic, 1990)

Using piles to stabilize slopes has been used successfully for many years. DeBeer and Wallays (1970) reported the use of bored piles reinforced with steel beams in the stabilization of a slope. Poulous and Davis (1980) pointed out that large diameter piles (1.0 to 1.5m) were used to stabilize active landslide areas in the United States. In Japan, steel-pipe piles with 30-cm diameter were also used to stabilize landslides. Chow (1996) and Hassiotis et al. (1997) presented the design of slopes reinforced with a single row of piles. El-Ashaal, Abdel-Motaleb, and Haggag (2000) used reinforced concrete piles to stabilize the embankment of a major canal made of uncompacted marine deposits and founded over soft clay. Despite the frequent use of piles to stabilize slopes, there has not been a standard method for the design of such piles.

Stark et al. (2008) presented a case study where PVC geomembrane was successfully used to mitigate a failure in water ponds.

In the resent study, five case studies of slope failures are presented. For each case, the geotechnical investigation, causes of failure, and the suitable technique of rehabilitation are demonstrated. Those case studies were chosen to cover a wide range of soil nature, seepage flow conditions, boundary conditions, and site restrictions enforcing certain limitations on the rehabilitation technique. It will be shown that there is no straightforward solution for the rehabilitation of the failed slope. The main lessons inferred from the case studies and the conclusions are mentioned.

Area of study

The current study presents five case studies as examples of the irrigation and drainage channels in Egypt. They are located at the northern, eastern, and southern parts of Egypt. The selected cases include several open channels that are part of the irrigation and drainage channels which are part of the Egyptian irrigation/drainage system. Some of these channels lie in the old agriculture lands in the southern part of the irrigation system in western part of the River Nile while the others lie in the eastern part of Egypt which transfers the water to the new reclaimed areas in Sinai. The studied channels include agricultural drains and canals which are used for both irrigation and drinking purposes.

Materials and methods

The studied cases cover a wide variety of soil characteristics and groundwater and flow conditions. An extensive site investigation program for each location was carried out. It included drilling boreholes, 15 20 m in depth, to extract soil samples. The investigations included conducting some field tests, Standard Penetration Tests (SPT) and Seismic Cone Penetrometer Test (SCPT). While an extensive laboratory testing program was also conducted to determine the natural, mechanical, and chemical properties of the different soil deposits. Samples of surface as well as ground water were also collected in order to determine their nature, properties, and their effect on the stability of channels side slopes. The determined properties of soil and water are used in the investigation of the stability of the studied cases. The field and lab testing program varies from one case study to the other and their details will be discussed in the following sections for each case separately. The conducted studies and testing programs ran during the period of 2001-2003.

The stability of the channels side slopes are investigated using the computer program package called “GEO-SLOPE Office” was used for the seepage and the slope stability analysis of the problem. This package is based on the finite element method. The program package consists of several programs. Each one of them is meant to study a different type of problem. For example, the program SEEP/W is used to perform seepage analysis, SLOPE/W to perform slope stability analysis, and SIGMA/W to perform stress and deformation analyses. The pore water pressure inside these slopes and the effect of seepage forces were taken into consideration by using SEEP/W.

A. Background and Soil Investigation of Case Study 1

In this case, a reach of the left embankment of a major canal in the eastern part of Egypt is studied. The height of the embankment is 5.50 m and the water depth is about 2.50 m. The problem started in 1997 when a main road was being constructed on the left embankment. Part of the construction process was to reshape the embankment side slope to its original slope 2 (horizontal):1 (vertical) and then put riprap to protect it. Then, a 100.0 m reach of the side slope failed with the riprap. The local authority made a replacement for a shallow depth of the soil and replaced the riprap. Unfortunately, the road suffered a large settlement and a deeper replacement was made again. But the road settled again causing lateral displacement for the riprap cracks along the road. At this time, the local authorities looked for technical assistance. Consequently, a complete geotechnical study was conducted including three boreholes; two of them with a depth of 15.0 m and the third with a depth of 19.5 m. Disturbed and undisturbed samples were extracted and SPT were conducted. Three holes on the left embankment with a depth of 10.0 m and six holes on the slope and inside the water with a depth ranging between 3.8 and 5.0 m were executed to perform SCPT. Since the natural soil of the left embankment was replaced for a significant depth, the original strength of the soil was investigated by conducting SPT and SCPT along the right embankment. Three holes on the right embankment were executed to conduct SPT and four holes on the right side slope and inside the water to conduct SCPT.

The boreholes on the left embankment showed that the soil deposit consisted of two layers. The first layer is a loose gravelly sand soil with clay traces with a thickness ranging between 3.00 m and 4.65 m. This layer was mixed with the gravelly sand replacement soil. The SPT count in this layer ranges from 2 to 7 blows, which means that the replacement soil was very loose and not compacted. The second layer is highly plastic silty clay with strength medium to stiff. The silty clay layer extended to the end of the boreholes. On the right embankment, the SPT count in the surface loose layer ranges from 1 to 15 blows, which means that the natural soil is also very loose to medium dense. Talking to the people living close to the studied area, they mentioned that a bomb hit this reach of the canal during 1973 war, which might explain the very loose nature of the surface soil.

B. Background and Soil Investigation of Case Study 2

This case study represents an irrigation channel having a bed width of 10.0 m, a maximum water depth of 2.75 m, and a side slope of 3:1. The channel cross-section was lined by gabions of gravel. As reported by the owner, the channel side slopes were stable until a parallel drain was constructed. The drain bed width is 5.0 m and the maximum water is 0.25 m. Once the drain was constructed the instability problem was clear. Slope failures of the drain, collapse of the embankment surface, and collapse of the gabions on the canal side slopes were observed.

In order to investigate the cause of the problem, it was necessary to identify the stratification of the soil profile in the site and determine the soil physical and engineering properties. Therefore, an extensive soil investigation was carried out including drilling 54 boreholes along both sides of the canal path to extract soil samples at different levels. One open pit was excavated to collect undisturbed soil samples to determine their natural, chemical, and engineering properties. Samples of ground water as well as surface water in the site (from the canal, the drain, and the open pit) were also collected to determine their nature and find their effect on the soil. Furthermore, comprehensive laboratory test program was conducted. Using the results of the field and laboratory tests, the soil profile along the canal path consisted mainly of layers of medium dense sand, clayey silt, and soft clay with shells as shown in Figure 1. Chemical analysis of the different soil types, ground water, and surface water of the canal and the drain showed high salt concentration.

C. Background and Soil Investigation of Case Study 3

In this case, a drain in northern part of Egypt with a bed width that ranges from 2.0 m to 5.0 m. The drain bed level is (-4.55), berm level is (0.00), and the embankment level is (2.00) m. The ground level is (1.20) m. All levels are referenced to mean sea level. The problem in this case was noticed by the increasing amount of sand collected in front of the suction basin of the pump station. Therefore, two sand traps were constructed to collect the sand but this solution did not solve the problem. Then, consecutive failures of the drain embankment were obvious and the flowing water in the drain washed out each collapsed mass.

To identify the problem, an extensive site investigation program was carried out including 60 boreholes with a depth of 15.0 m and SPT tests along both sides of the drain path. The results of the site investigation and laboratory tests

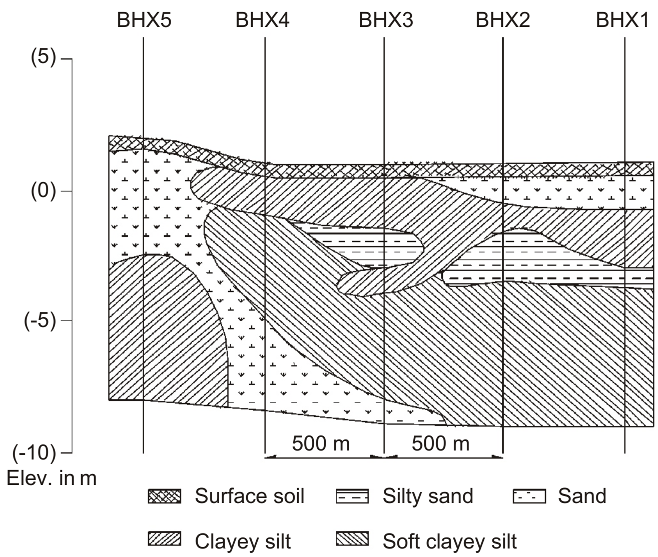

Figure 1 : Soil profile of case 2

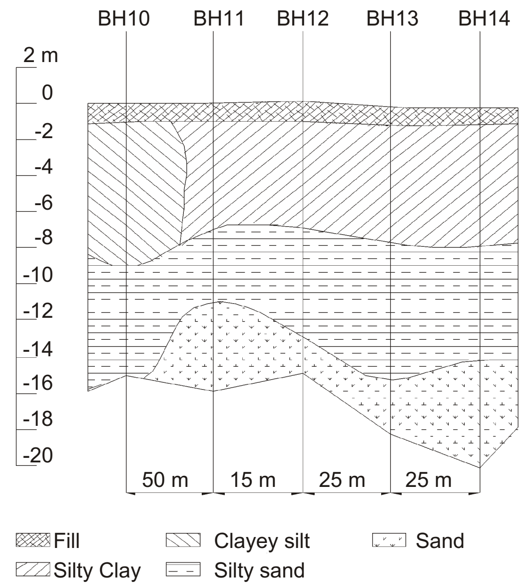

Figure 2 : Soil profile of case 4

Showed that the soil profile consists mainly of layers of poorly graded fine to medium sand intermitted by layers of clayey silty sand. There were also lenses of silty clay or clayey silt. The SPT results varied significantly along the drain path with corrected values from 3 to 50.

D. Background and Soil Investigation of Case Study 4

The canal considered in this case is a major element of the irrigation system of southern Egypt. It has a bed width of 22.0 m and side slopes of 3:2. The designed cross section of the canal has no berm and the embankment height is about 6.0 m. After the winter closure period (a period during which the discharge from the high dam is at its lowest level because of the lowest water demands), several failures along the canal path of lengths between 120.0 m and 380.0 m were observed at the internal side slope of the canal which resulted in reducing the canal cross section at some areas and endangering the heavy traffic on the canal embankment. During the field investigation, Deep cracks, parallel to the canal, were noticed at the left embankment and a drain parallel to the canal was observed at a distance not more than 50.0 m.

A complete geotechnical study of the site was carried out including drilling 20 boreholes with a depth between 15.0 m to 20.0 m and disturbed and undisturbed samples were collected. Furthermore, field and laboratory tests in order to determine the physical and engineering properties of the soil deposits in the site were carried out. The results of the field and laboratory tests showed that the soil stratification in the site consists of a surface soil layer with a thickness of 1.0 m followed by about 7.0 m of medium stiff silty clay with traces of lime. Then, a layer of silty fine sand with a thickness of about 7.0 m exists, followed by a fine sand layer with a thickness of about 5.0 m. Figure 2 shows the soil profile of case 4.

E. Background and Soil Investigation of Case Study 5

A major drain in the northeastern part of Egypt has suffered a slope failure through a reach of about 400.0 m. The height of the embankment is 6.0 m over the bed of the drain with a berm at 4.90 m above the bed and side slopes 2:1. To explore the nature of the soil deposits at the site, four boreholes with a depth of 15.00 m to extract disturbed samples only since the soil deposits at the site were very soft to extract undisturbed samples. Two holes for field vane shear tests were drilled to determine the strength of the soft soil layers.

The boreholes showed that the soil deposit consisted mainly of two layers. The first one is highly plastic soft silty clay layer that extends to a depth ranging from 6.5 m to 8.0 m. The second one is poorly graded sand that extends to the end of the borehole. A lens of peat with a thickness ranging from 0.35 m to 0.60 m existed at a depth 3.00 m. Figure 3 shows the soil profile of this case. The value of the cohesion of the soft silty clay layer resulting from the field vane shear test ranged from 0.11 kg/cm^2 to 0.29 kg/cm^2 and its liquid limit ranged from 50% to 107%.

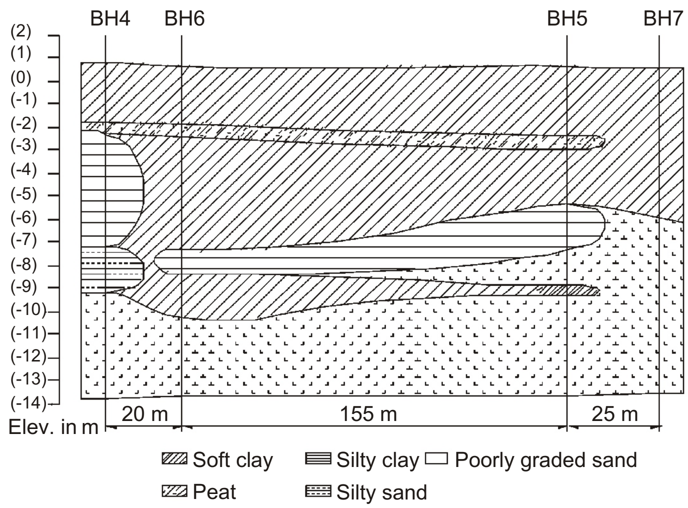

Figure 3 : Soil profile of case 5

Investigating possible causes of slope failure

After completing the site investigation, the natural, physical, mechanical, and/or chemical properties of the soil deposits in each site were identified. Then, the site conditions were examined and a slope stability study was conducted trying to identify the causes of the failures in each case. The Geo-Slope software package was utilized to investigate the stability of the side slopes of the channels. The effect of the seepage forces was taken into consideration using the SEEP/W module and the resulted output were integrated into the SLOPE/W module to investigate the probable causes of failure and the proposed method of mitigation. The critical factor of safety of the failed slopes at different locations was calculated with and without the effect of seepage forces to determine their effect on the slope stability.

A. Causes of Slope Failure of Case Study 1

In this case, when examining the site history and condition, it was found that a main road was constructed on the left embankment of the canal to allow heavy traffic loads to pass along it. The construction process included completing the embankment width and reshaping the side slope to its original slope 2:1 and then put riprap to protect it. It was also known that a bomb hit the site during 1973 war. Consequently, the site investigation was directed to investigate the presence of a camouflets which is an underground cavity formed by the explosion of a bomb without the formation of a crater (British Standards Institution, 1957). Not finding any cavity, a slope stability study of the left embankment was conducted. An equivalent load of a 60-ton lorry and the case of sudden draw down of the water level in the canal were considered. The critical factor of safety was almost 1.0 and the sliding circle lied inside the loose surface soil layer. This value of the factor of safety is low and explains the repeated failure of the left embankment and the riprap on the side slopes. When using a slope of 2.5:1 and the properties of dense soil in the stability analysis, the critical factor of safety became 1.48, which is accepted according to the Egyptian code. Hence, it was concluded that the explosion at the site caused the top sandy soil to loosen along the failing 100.0 m reach. Then, after constructing the main road, the loose nature of the top soil layer could not stand the new heavy traffic loads causing the repeated slope failures. Figure 4 shows the slope failure of the embankment.

Figure 4 : Slope failure of case 1

Figure 5 : Salt leaching and slope failure of case 2

B. Causes of Slope Failure of Case Study 2

Stability analysis of the slopes of the channel and drain was carried out to identify the main cause of the slope failure. This study was carried out under different boundary conditions such as with and without seepage effect, with and without sudden draw down of the water level in the channel, and with and without live load on the embankment. In all cases, the factor of safety achieved the requirements of the Egyptian code of practice. Therefore, the existing slope failure cannot be attributed to any of the above-mentioned conditions. Looking at the laboratory test results, it was noted that the chemical analysis of the soil samples showed the presence of a high percentage of dissolved salts.

Hence, the attention was drawn to the results of the chemical analysis of the soil and the water and it was concluded that the failure occurred because of the leaching effect of the high salt concentration within the soil particles and consequently initiating the piping phenomenon (internal erosion). Figure 5 shows salt traces due to leaching at the drain side slopes. The developed seepage forces started the fine particles washing (migration) and that in turn reduced the stability of the drain side slopes and consequently causing the slope failure of the drain. Furthermore, the continuing process of internal erosion created cavities within the soil mass between the canal and drain causing collapse of parts of the embankment surface and parts of the gabions on the canal side slopes. Field observations supported the above-mentioned conclusion of the cause of failure.

C. Causes of Slope Failure of Case Study 3

Examining the drain cross-sections, field observations, the site investigation data, and the laboratory test results, two major characteristics of this site were noticed. The first one is that the soil is poorly graded fine to medium sand as revealed from the grain size distribution of the soil samples. The poorly graded sand is hard to be compacted efficiently and as well known, the maximum stable slope angle for dry sand, under no external loads, is its angle of internal friction (Budhu and Gobin, 1996). If seepage or surface erosion is permitted through the sand mass of such drain, it will collapse ending up with a flatter slope. The second major characteristic of the site is that the water table in the neighboring agricultural land is quite higher than the water level in the drain with a difference that ranges from 1.40 m to 2.40 m. Such a difference caused high seepage forces on the drain side slopes. This in turn caused the sliding of the slope mass into the drain at different locations along the drain path as indicated in Figure 6. The failures were worse in the locations closer to the pump station because of the severity of the bed erosion and the higher values of the hydraulic exit gradient on the side slopes. Then, the failing mass was washed out by the flowing water causing the sedimentation of the soil particles inside the suction basin of the pump station.

A slope stability analysis was carried out using a computer program. In the beginning, the stability of the slopes on the dry condition was examined and the factor of safety was 1.1, which shows that the slope does not satisfy the requirements of the Egyptian code of practice. Then, additional runs were carried out with seepage forces and live loads and the factor of safety was 0.96. The failure of such case can be attributed to two main causes, the first one is the instability of the slope due to the gravitational effect and the seepage forces, which cause the fine particles to migrate causing internal erosion. The second one is due to the high velocity of the flowing water that causes surface erosion either form the side slopes and/or the bed of the drain especially near the suction basin of the pump station.

Figure 6 : Slope failure of case 3

Figure 7 : Slope failure of case 4

D. Causes of Slope Failure of Case Study 4

Figure 7 shows the failure of the canal side slopes. A slope stability analysis was carried out to determine the probable causes of failure. Noting that the failures occurred after the winter closure, a sudden draw down of the water level was considered in the analysis. Also, the case where the seepage line is from the existing drain to the canal was studied. In both cases, equivalent loads of 60-ton lorry have been taken into consideration since heavy traffic loads were allowed on the road on the canal embankment. The factor of safety of the normal working conditions (without heavy traffic loads and sudden draw down) is 1.50, which is safe according to the Egyptian code of practice. While the minimum factor of safety was 0.70 in the case of sudden draw down and heavy traffic loads. Therefore, it was concluded that the sudden draw down combined with the heavier traffic loads caused the slope failure of the canal.

E. Causes of Slope Failure of Case Study 5

Examining the results of the site investigation especially the field vane shear tests and the laboratory tests, it was clear that the cause of failure was the very weak nature of the soil deposits at the site especially the peat lenses in addition to the seepage from the neighboring agricultural land to the drain. Slope stability analysis of the drain cross section was conducted. The seepage from the neighboring agricultural lands was considered. The slip surface was forced to pass through the peat lens since it is much weaker than the surrounding soil deposits. The critical factor of safety was 0.69, which is quite lower than the requirements of the Egyptian code of practice.

Rehabilitation of the failed slope

Rehabilitation of a failed slope is a complex process because of the restrictions enforced on the type of solution. These restrictions may be due to scarcity of suitable replacement soil, the necessity to keep the channel and/or the road on its embankment functioning, and/or the impossibility of changing the dimensions of the cross section. Therefore, there is no fixed solution for the failures of water channels. In other words, there no fixed criteria for a good solution. A good solution is the one that surely satisfies the safety of the embankment and also takes the imposed restrictions into account.

A. Rehabilitation of Case Study 1

In this case, the slope stability study showed that the side slope 2:1 is not suitable and that changing it to 2.5:1 and compacting the soil mass of the slope would satisfy the requirements of the Egyptian code of practice. However, the restriction imposed in this case is the impossibility of changing the side slopes in this reach. Therefore, it was suggested to use a supporting system for the embankment as an alternative solution. This supporting system consisted of reinforced concrete piles with a diameter of 60 cm and a center-to-center space of 1.50 m. The factor of safety of the stabilized embankment was 1.96 and the sliding circle was a base circle. It was recommended to use partial displacement piles, to improve the density of the loose layer, with a depth of 15.0 m and the length of the stabilized reach extended for 20.0 m from both sides of the failing reach of the embankment. A reinforced concrete beam with a width of 70 cm and a depth of 60 cm was used to connect the piles together. Then, the slopes should be dressed to the original slope 2:1 and the riprap replaced with a thickness of 40.0 cm. The details of the rehabilitation technique are shown in Figure 8.

B. Rehabilitation of Case Study 2

Since the problem in this case is mainly due to the seepage that causes salt leaching and consequently cavities, the rehabilitation aimed mainly at controlling the seepage line from the canal to the drain to reduce the values of the exit gradient and protect the soil mass against piping. This goal can be achieved using cutoff wall technique. Several schemes were attempted by changing the number of cutoffs, their positions, and their depths to reach the optimum cutoff solution. The amount of water seeping from the canal to the drain, as well as the values of the critical exit gradient were obtained for each case and the results were compared with the case of no cutoff. The case of inverse seepage from the drain to the canal was also studied to account for the canal maintenance at which the water level drops to almost zero. Based on the results, a system of two cutoff walls was suggested. The first cutoff is at the canal left berm with a depth of 6.0 m and the second one is at the drain right embankment with a depth of 14.0 m. Figure 9 shows the general layout of the problem area and the suggested solution. Figure 10 shows the details of the protection filter to safeguard the soil mass against fine particle migration to minimize the internal erosion.

C. Rehabilitation of Case Study 3

It was indicated when talking about the causes of this problem that the basic problems in this drain are the loose nature of the original soil and the big difference between the level of the water table in the agricultural lands and the water level in the drain. Therefore, the rehabilitation of this drain depends mainly on satisfying two requirements. The first one is to lay the original soil on a flat slope that will be stable under the working conditions of the drain i.e. with water flowing through the embankment under 2.40 m of hydraulic head. The second one is to put a filter on the original soil to stop the washing out of the fine particles of the original soil.

Hence, to satisfy the first requirement the original soil was laid on a side slope 4:1. The factor of safety for this case is 1.47, which satisfies the requirement of the Egyptian code of practice. Then, a soil filter made of three transition layers was laid to prevent the fine particles from being washed out and to reshape the cross section to its original side slope 2:1. The factor of stability of this case is 1.29, which is less than the requirements of the Egyptian code of practice. Therefore, the side slopes were altered to 2.5:1 with an intermediate berm at level (-1.50). The factor of safety for this cross section is 1.74, which is acceptable according to the Egyptian code. Figure 11 shows the details and dimensions of the suggested cross section.

D. Rehabilitation of Case Study 4

It was stated in the discussion of the causes of failure of this channel that the sudden draw down in the water level combined with the heavier traffic loads caused the slope failure of the canal. Sudden draw down have two negative effects on the stability of the side slope. The first is the destabilizing seepage forces associated with it and the possibility of washing out the finer particles from the original soil. Because of that, the solution depended on flattening the slope of the original soil and then using a filter on it to protect the fine particles. In this case, the slope of the original soil was made 4:1 and then a two-layer filter was laid. Then a gravelly sand soil was used to reshape the canal side slopes to the original one i.e. 3:2 and finally riprap was used with a thickness of 50 cm to protect it. The factor of safety for rehabilitated cross section is 1.52. The details of the suggested cross section are shown in Figure 12. It was recommended that the solution would be executed on alternate sections; each one is 20.0 m long.

E. Rehabilitation of Case Study 5

The drain cross section was stabilized using earth reinforcement technique. The soil was reinforced using geogrid layers at a spacing of 0.60 m and a length of 9.0 m starting from the bed until the berm level. The peak strength of the geogrid is 30.0 KN/m and the yield point elongation is 10%. A single layer of geotextile was laid 0.30 m below the bed level starting from the drain centerline and extending under the embankment width. Silty sand soil was used to replace the natural soil of the site and was compacted accordingly on layers with a thickness of 0.30 m. Figure 13 shows the details of the solution.

Discussion

Irrigation and drainage channels are the backbone of the development in Egypt. Therefore, keeping them efficiently functioning for the longest possible period should be a target for the designer. It is easier and economical to achieve this target during the first construction than during a later rehabilitation. Therefore, this target should be considered during the design, construction, and operation stage of the channel. With the rapid expansion in Egypt, the government was forced to expand beyond the Nile delta. The soil in the new expansion areas is much different than the delta. Therefore, extensive geotechnical studies for those areas including careful site investigation and stability analysis must be carried out.

Many failures have been observed when slope 2:1 is used with fine loose sand. The angle of the channel slope in such case is 26.6º while the friction angle of such soil is about 30º. Therefore, the factor of safety of the slope in the dry condition and without any live loads is 1.15, which is less than the requirements of the Egyptian code of practice. To achieve those requirements, dense sand with an angle of internal friction not less than 37º is needed. However, when having live loads and seepage conditions, 37º is not enough. Hence, when the available local construction material is fine to medium loose sand and importing suitable construction materials is very expensive, slopes ranging from 4:1 to 3:1 should be considered according to the working conditions of the channel.

The designer should look ahead beyond the working conditions of the first operation of the channel. The possibility of having a ground water table higher than the water level in the channel and hence causing seepage forces should be checked carefully especially when designing a drain. In this case, a flatter slope may be needed along with a filter to prevent washing out of the fines of the embankment soil. Furthermore, the future possibility of constructing a road with heavy traffic loads should be examined.

Modern techniques and technology should be considered and implemented; e.g. earth reinforcement, gabions, geotextile, slurry cutoff, flexible, etc... These techniques can deal with exceptional conditions; e.g. limited available area for needed flatter slopes, high cost of importing soil for needed graded filter, etc…

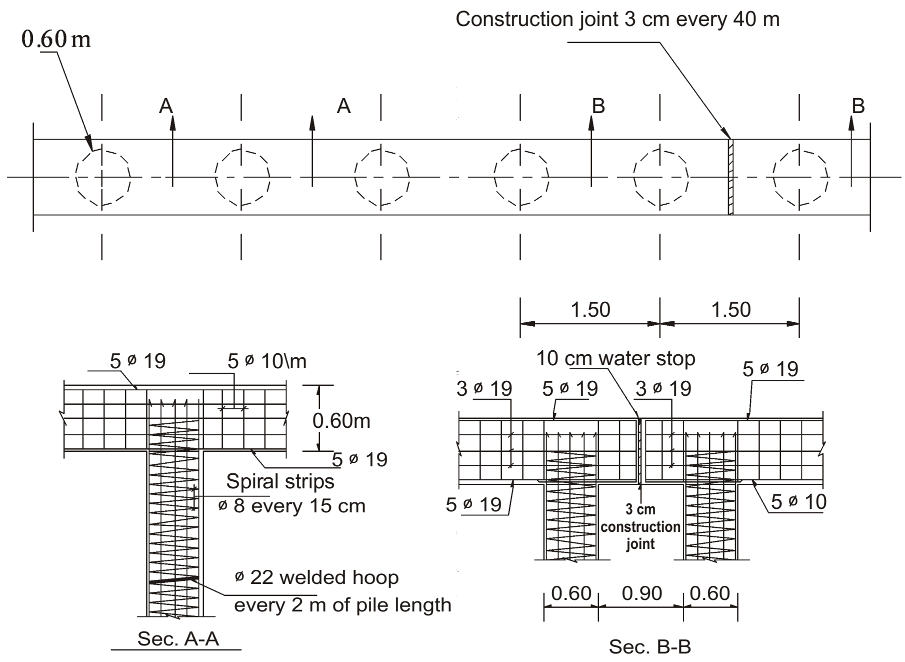

Figure 8 : Rehabilitation technique of case 1.

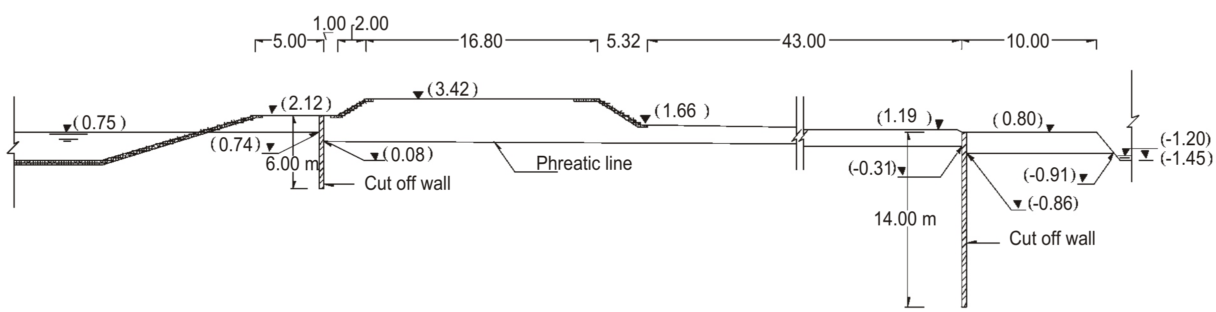

Figure 9 : The general layout of the problem area and the rehabilitation of case 2.

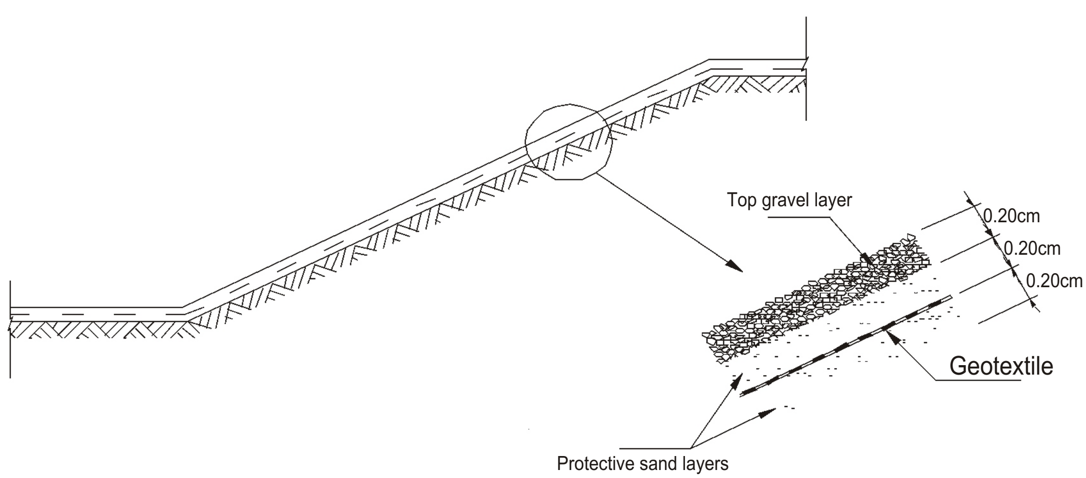

Figure 10 : Protective filter against piping in case 2.

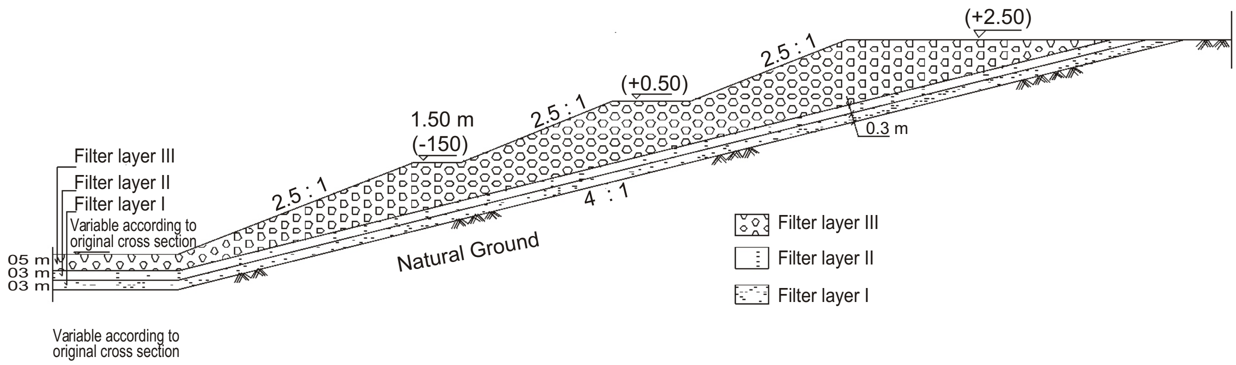

Figure 11 : The details and dimensions of the rehabilitated cross section of case 3

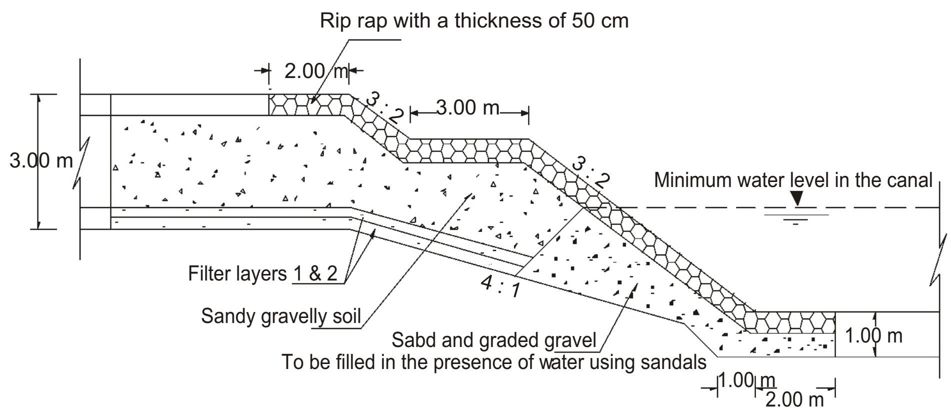

Figure 12 : The details of the rehabilitated cross section of case 4

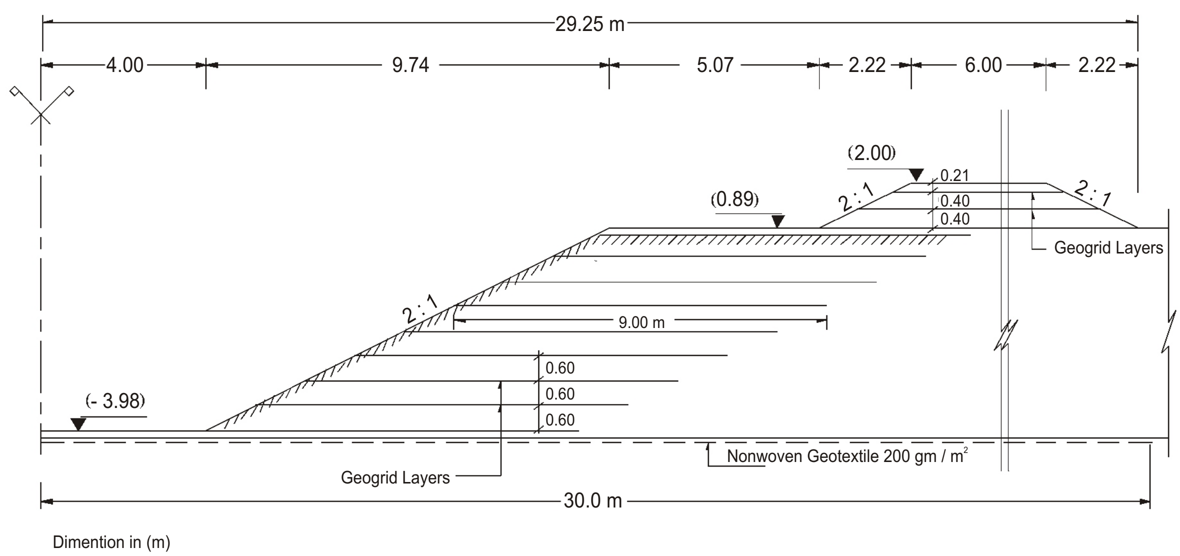

Figure 13 : The details of the rehabilitated cross-section of case 5

Conclusions

Based on the field investigation, laboratory test results, seepage analysis, and stability analysis of five slope failure cases in Egypt, the following conclusions can be drawn:

1- Common causes of failure of water channels in Egypt include:
   - Careless geotechnical investigation of the construction site especially when dealing with the newly developed areas outside the Nile delta.
   - Paying no attention to the various possible working conditions especially those causing seepage either from a channel to another or from the neighboring cultivated land to the channel.
   - Overlooking the future expected conditions especially the possibility of having a road on the embankment with heavy traffic.
   - Expecting the soil to behave as a man made material with standard properties wherever it is found; e.g. expansive soil is mainly clayey soil, however it behaves completely different from the clayey soil of the Nile delta.

2- Using side slope 2:1 in cohesionless soil, i.e. sandy soil, in Egypt is disastrous and is a common cause of numerous slope failures.

3- No single or general way of rehabilitation of channels suffering from slope failures because of the uniqueness of the working conditions and characteristics of each site. Therefore, each site has to be studied carefully to find out the best way to deal with it.

4- Rapid draw down of the water level in the channels during the winter closure must be considered when studying the stability of slopes since it was proved to stand behind some dangerous failures.

5- The presence of a high percent of salt may lead to internal erosion due to leaching and thus causing failure.

6- Adopting the modern technology is a must for more efficient water channels.

Acknowledgments

The author would like to acknowledge the Construction Research Institute staff for their assistance in the laboratory and field investigations and their general support.

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Indus Basin Transboundary Water Issues in Past and Present Perspective

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Abstract

Indus basin river system comprises the main Indus and its five tributaries. Irrigation infrastructure in the basin makes it the largest contiguous block of irrigated agriculture in the world. At the time of partition of the sub-continent in 1947, boundaries between Pakistan and India were demarcated without due consideration of headwaters and their command areas. Resultantly, the rivers source waters and headworks of canals commanding huge area in Pakistan went under Indian control, giving rise to severe water sharing dispute. Inter-riparian negotiations failed but with the mediation of World Bank, Indus Water Treaty was finally signed in 1960. The treaty is comprehensive document whereby the three western rivers were allocated to Pakistan and the three eastern to India. The other provisions of the Treaty were constitution of Indus Water Commission, data exchange and dispute resolution mechanism. Over the years, the Treaty worked well for resolving transboundary water issues between the riparians. However, population growth and surging food and energy demands coupled with water scarcity gave rise to differences and disputes during the last couple of decades. This paper overviews water disputes between the two countries and sharing transboundary waters in the past and the present perspective. It highlights lacunae of the Treaty in the context of current era of climate change, environmental degradation and technological developments. It further suggests framing and implementing universal laws based upon equitable and fair sharing of transboundary waters all over the world for avoiding water wars.

Keywords : IndusBasin, Transboundaries, Water Resources, Water Issues

Introduction:

The Indus Valley is one of the most advanced ancient civilizations. It is also called cradle of civilizations owing to its substantial fertile land resources, waters of mighty Indus river system and favorable agro-climatic potential. The valley is segmental homogeneous with plain extending from foot hills of Himalayas to the coast of Arabian Sea, bounded on the west by Kirthar and Suleiman mountains ranges, on the east by Punjab plains up to Ambala and Kalka in India whereas low lying ridges divide it from the plains of Yamuna River. Length of Indus plains from Himalayan piedmont to Arabian Sea is more than 1500 km whereas its width in Punjab is up to 325 km. Expanse of the Indus Basin is shown in Figure 1. Almost 53 percent of the basin area lies in Pakistan. Indus basin is geographically divided into doabas, the land between two adjacent rivers, and each doaba derives its name from the bounding rivers. The initial agriculture in the plains commenced in about 3000 BC (Fahlbusch *et al.*, 2004). Inundation canals were traditional practices for managing irrigated agriculture. Modernization and expansion of irrigation system however commenced in the British era in the middle of the last century. It is now one of the world's marvelous and the largest contiguous irrigation system.

Figure 1 : Expanse of the Indus basin
(Source: http://www.rifah.org/site/the-indus-accord-1960)

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Climate

In general the Indus basin is divided into three climatic regions namely, Himalaya zone, Sub-Himalaya zone and plains. The overall climate varies from sub-tropical, semi-arid and arid in plains to sub-humid and alpine in mountainous highlands of the north. Precipitation has both temporal and spatial variation occurring in summer as well as winter. High precipitation occurs in southern slopes at altitudes between 900 to 1530 m whereas it is about 1250 mm in a strip of around 125 km wide east of main Jhelum River and goes up to 1800 mm in Himachal Pradesh which falls in Bari Doab. Precipitation in most of the upper Indus catchment is less than 800 mm and falls as low as 75 mm in Leh, where it mostly occurs in the form of snowfall at an altitude above 5000 m. Climate of Ladakh range areas is also very dry where rainfall is scanty but snowfall is frequent and at times heavy. Sub-Himalayan regions have 800 mm annual rainfall in the east and 375 mm in the west which decreases generally from northeast to southwest from about 750 mm to less than 125 mm in Sind (Fahlbusch et al., 2004). India is the second wettest country of the world where average annual rainfall is 1170 mm against the world average of 870 mm (Kumar, 2006). Contrary to that, climate of Pakistan is predominantly arid and semi-arid where more than half of the country receives less than 205 mm rainfall (Kahlown and Majeed, 2004)

Indus River System

Indus River System comprises the mighty Indus River and its 27 tributaries, of which 13 are in the hilly areas and 14 are in the plains. Total area of the **Indus Basin** is about 114 million hectares of which Pakistan covers major part of about 60 million hectares as detailed in Table 1, Table 2. It is fed by the world's three highest mountain ranges namely the Himalaya, the Karakoram and the Hindukush. The major tributaries are Jhelum, Chenab, Ravi, Beas and Sutlej which join the main Indus from the east, whereas the River Kabul combined with River Swat join from the west as detailed in Table 2. The average annual discharge of Indus River system is 207 BCM and stands to be the 21st largest river in terms of flow and the 6th largest in terms of length.

Table 1 : Distribution of the catchment area of Indus River Basin

Country	Drainage Area (million ha)	Percent area of country in the basin (%)	Comments
Pakistan	59.80	52.48	-
India	38.32	34.35	1600 km^2 Indian control, claimed by China
China	8.58	6.83	9600 km^2 Chinese control, claimed by India
Afghanistan	7.21	6.33	-
Total	**113.91**	**100**	-

Source: Estimated from data obtained from Government of Pakistan, Water and Power Development Authority, Lahore. Cited in, Akhtar 2010 "Emerging Challenges to Indus Water Treaties"

Table 2 : Salient features of the Indus River and its tributaries

River	Origin	Path	Length/ /annual flow	Tributaries	Glaciated Area (%)
Indus (Main)	Mansarowar lake, Tibet	Kashmir, Skardu, Tarbella, Multan and Arabian Sea	2880 km 100 BCM	Hunza and Gilgit at Raikot, Kabul at Attock and Chenab at Mithankot	12
Jhelum	Verinag spring, Indian Kashmir	Wular Lake, Muzzaffarabad, Mangla and Trimmun	820 km 28 BCM	Neelum and Kunhar at Muzzaffarabad and Chenab at Trimmu	1
Chenab	Himachal Pradesh (India)	Kishtwar, Marala Panjnad, Mithankot	1361 km 28 BCM	Jhelum at Trimmu, Ravi at Ahmadpur Sial, Sutlej at Panjnad	13
Ravi	Rohtang Pass in Kangra (India)	Chamba, Madhopur, Lahore, Ahmadpur Sial	894 km 7.8 BCM	Chenab at Ahmadpur Sial	3
Beas	Rohtang Pass in Kulu (India)	Kangra, Singbol, Hoshiarpur, Talwara	467 km 15.6 BCM	Sutlej at Harike	5
Sutlej	Lake Rakshastel Western Tibet	Ludhiana, Ferozpur, Bahawalpur	1542 km 16.64 BCM	Chenab at Panjnad	11
Kabul	Kabul, Afghanistan	Chitral, Kabul, Warsak, Nowshehra	480 km 21.4 BCM	Indus at Attock	-

(Source: Fahlbusch et. al, 2004)

The main Indus River emerges from Mansarowar Lake in Tibet at an altitude of 5494 m and after traveling some 3000 km through Himalayas, adjoining ranges and plains, it embraces Arabian Sea in the east of Karachi. Mountainous catchment of the Indus is about 44 million ha and has the largest frozen water reserves outside the polar region, which is major contributor of snowmelt and rainfall runoff. Total ice reserves of the basin have been reported about 2,738 BCM, which is 16 times the average annual Indus river flows (Ahmad and Joya, 2003). Snowmelt zone lies between altitude of 2500 to 5500 m, which accounts for most of the summer runoff that increases gradually from March to June and continues until September. Winter snow cover of the Himalaya goes up by 30-40%. All the major tributaries drain into the upper reach of 2150 km and the Indus with its tributaries make up one of the most important river systems in the world. The basin was converted into an extensively cultivated area during the British colonial era, with millions of hectares irrigated by large canals.

Origin of Indus Basin Water Dispute

The Pak-India water dispute originated from partition of sub-continent when water origins and diversion structures were not given any consideration while demarcating political boundaries between Pakistan and India. The agreed upon bases of division was that the Muslim majority areas would be included in Pakistan whereas right of accession was given to the princely states to join either of the two countries. The agreed upon division formula was however not Adhered to and a Muslim majority district of Gurdaspur, a gateway to Kashmir, was included in India that provided it with easy access to Kashmir, which is origin or pathway of the waters of the Indus River System. Accession of Kashmir to India declared by Hindu Raja was immediately acceded to, whereas similar accession desired by the ruler of the princely state of Junagarh was not honored. In pursuit, India immediately moved to occupy Kashmir, but Kashmiries resisted and took control of a part of it, resulting in break out of war in 1948 between the newly born states. India took the issue to UN where stance of Pakistan was acceded to and it was decided to resolve the issue by holding a free and fair plebiscite under UN for determining right of accession of Kashmiries on majority vote. The then army positions were declared cease fire line under UN observers, later declared as Line of Control (LoC) in 1972. Unfortunately, the UN resolution has not so far been implemented. Resultantly, most of the headwaters of Indus and its tributaries went under Indian control including Modhupur Headworks on the River Ravi and Ferozepur Headworks on River Sutlej, falling at the periphery of Pak-India border. Those head works were feeding Upper Bari Doab Canal and Deepalpur Canal, which had 90% command area in Pakistan. India blocked supply of these canals thereby depriving irrigation water to the substantial tract of its fertile command area in Pakistan. Pakistan's irrigated areas suffered badly and agricultural production perilously threatened. An interim standstill agreement was signed in December 1947, whereby partial supplies were ensured uptill 31st March, 1948. However, the supplies were again cut off on 1st April, 1948, resultantly the important Kharif crop on the command area was damaged. New inter-dominion agreement was signed on 4th May, 1948 whereby the supplies were resumed for giving Pakistan time to develop alternate sources. India claimed all the waters of eastern rivers and demanded payment from Pakistan for the received water. Whereas, Pakistan contended to pay only for operation and maintenance and not for water that belonged to Pakistan prior to partition. After protracted negotiation mediated by the World Bank, the Indus Water Treaty (IWT) was agreed upon in 1960.

Indus Water Treaty

At the time of partition in 1947, altogether 64 percent of irrigated area and 83 percent of the cultivated areas of the Indus basin came under Pakistan (Fahlbusch, 2004). The treaty was result of stoppage of canals supplies and apprehensions of Pakistan that India owing to upper riparian of Indus Basin could potentially create havoc by blocking water supplies thereby inducing droughts and famine like situations in Pakistan. At that time about 8.5 million ha in Pakistan (Ahmad, 2010) were irrigated with Indus waters whereas boundary between the two countries was drawn disregarding prevailing irrigation network. Though it was expressly agreed by the stakeholders before the Arbitral Tribunal that the authorized zones in the common water supply would continue to be respected, but immediately after winding up the Arbitral Tribunal in 1948, India blocked waters in all the irrigation canals originating across the border thereby affecting 0.65 million ha of irrigated land in Pakistan. That necessitated immediate resolution of the dispute for future use and distribution of combined waters. India claimed proprietary rights of waters of the rivers in Indian Punjab and refused to accept Pakistan's demand of providing due share of those waters based upon the time honored formula that the prevailing uses were sacrosanct and excess waters should be divided proportionate to area and population of the riparians. The stance of Pakistan was in accordance with the accepted principle adopted in several treaties between states and provinces in the subcontinent. Although through an interim agreement of 1948, reduced supplies remained available, but direct negotiations between the two countries failed and the issue became too serious to sustain peace in the region. The negotiations recommenced in 1952 under the offices of World Bank and after prolonged bumpy proceedings, an agreement what is called Indus Water Treaty was finally signed in September 1960 that took effect retrospectively from 1st April 1960.

Salient Features of Indus Water Treaty

Distribution of the Eastern and the Western Rivers:

I. All the waters of the eastern rivers (Ravi, Sutlej and Beas, Figure 2) shall be available for unrestricted use of India. Pakistan shall be under an obligation to let flow and shall not permit any interference with water of the Sutlej Main and the Ravi Main in the reaches where these rivers flow in Pakistan and have not yet finally crossed into Pakistan. Pakistan however can only use it for non-consumptive and domestic purposes.

ii. All the waters, while flowing in Pakistan, of any tributary which in its natural course joins the Sutlej main or the Ravi Main after these rivers have finally crossed into Pakistan shall be available for the unrestricted use of Pakistan

iii. Pakistan shall receive unrestricted use of all the waters of the western rivers (Indus Jhelum and Chenab, Figure 2). India shall be under an obligation to let flow all the waters of the Western rivers, and shall not permit any interference with these waters (World Bank, 1960).

Furthermore, the aggregate storage capacity on western rivers was provided to India as per details given in Table 3.

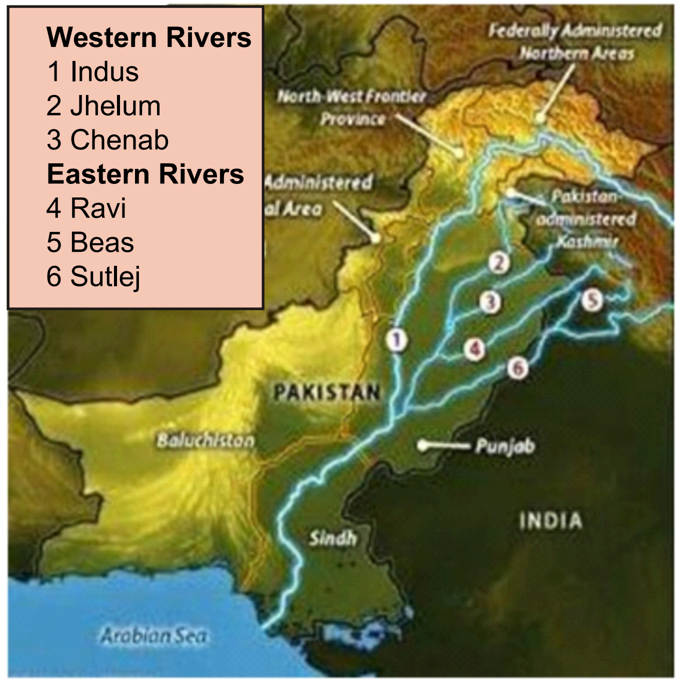

Figure 2: The Indus River system showing western and eastern rivers

Table 3: Storage provision on western rivers to India

Sr. No	River	General Storage Capacity (BCM)	Power Storage Capacity (BCM)	Flood Storage Capacity (BCM)
1	The Indus	0.30	0.19	-
2	Jhelum (excluding the Jhelum Main)	0.62	0.30	0.93
3	Jhelum Main	-	-	Conditional to a duly prescribed flood level
4	The Chenab (excluding the Chenab Main)	0.62	0.74	-
5	The Chenab Main	-	0.74	-
Total		**1.54**	**1.97**	**0.93**
Grand Total		**4.44 BCM**		

(Source: World Bank, 1960, Indus Waters Treaty Annexure-E)

General Provisions:

i. The use of the natural channels of the rivers for the discharge of flood or other excess waters shall be free and not subject to limitation by neither party, nor any party shall have claim against the other in respect of any damage caused by such use.

ii. Each party declared its intention to prevent, as far as practical, undue pollution of the waters and agreed to ensure that, before any sewage or industrial waste is allowed to flow into the rivers, it would be treated, where necessary, in such manners as not to materially affect those uses.

iii. India was given entitlement not only to continue irrigate areas of 0.26 million ha from western rivers which were so irrigated as on the effective date, but was also entitled to irrigate another area of 0.28 million ha from the western rivers thereby making total provision of 0.54 million ha (68% of which was from the River Jhelum and 23% from the Chenab).

iv. The agreement precluded building of storages by India on the rivers allocated to Pakistan. However, if India wants to generate hydroelectric power it can only build run-of-the-river hydroelectric projects (unlike a dam or a reservoir), which do not create storages beyond that detailed in Table 3. Paragraph 2 (c & d) in Article III of the Treaty allows and Annexure D and E explain that how India can use the waters of western rivers for hydroelectric projects.

V. Pakistan agreed to make arrangements through replacement works for transfer of water from western rivers to eastern rivers for irrigation of command areas, headwaters of which were allocated to India (World Bank, 1960).

Indus Basin Replacement Works

As a result of the Treaty, Pakistan embarked on a gigantic project what is called Indus Basin Replacement Works (IBRW). The project was accomplished through Indus Basin Development Fund generated by the World Bank and other donor countries at an unprecedented cost of Pak Rs. 124 billion. It was the biggest irrigation project of the world at that time, which was completed in only ten years, and by which 17 BCM/year of water was managed to transfer from western to eastern rivers in addition to modernization and upgradation of the existing irrigation infrastructure. Resultantly, Pakistan side of the basin now has 3 large dams, 85 small dams, 19 barrages, 12 link canals, 45 canal commands, 110,000 watercourses and about 1 million tubewells. Total length of main canal system is estimated about 60,000 km and that of watercourses and field channels is about 1.6 million km. The present worth of the entire Indus basin irrigation infrastructure is more than US $ 300 billion which irrigates more than 19 million hectares of land (GoP, 2009). On the Indian side, the basin has 4 dams, 5 barrages, 2 link and 10 irrigation canals commanding an area of more than 5 Mha.

Indus Water Commission

In accordance with Article VIII of the Treaty, both India and Pakistan each created one post of Indus Water Commissioner, Figure 3 which together constitutes Permanent Indus Water Commission. The Commissioner of each country acts on behalf of his government for all the matters of the Treaty and servers as regular channel of communication unless either of the government decides to take any particular matter directly with the other government. The Commission meets regularly at least once in a year, or when requested by either of the Commissioners, alternately in each of the countries and can inspect, assisted by up to two advisors, works on Indus basin of both the countries. The major functions of Commission are:

- To establish and maintain cooperative arrangements for implementation of the Treaty, to promote cooperation between the parties in the development of the waters of the rivers particularly on specified aspects.
- Furnish yearly or other desired reports to both the governments by fixed date.
- To share, on a regular basis, extensive and comprehensive data on the water flows in each river, and water withdrawn from each reservoir, etc.
- To inform each other with relevant data of 'engineering projects'if either of the two countries plans to construct any of such projects on any of the rivers which causes interference in the water flow.

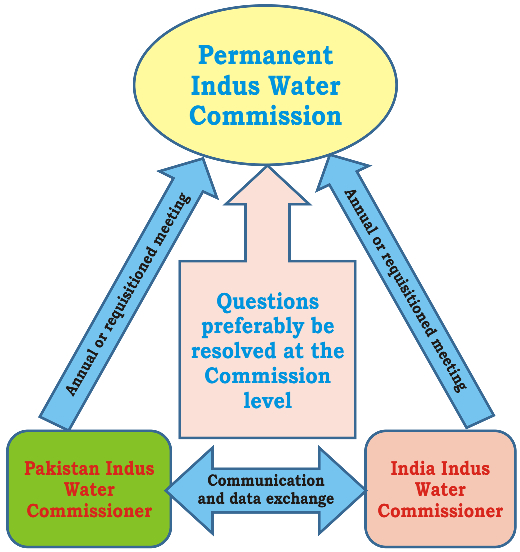

Figure 3: Schematic diagram of Permanent Indus Water

- To undertake once in every five years, a general tour of inspection of the rivers for ascertaining the facts connected with the various developments and works on the rivers.
- To undertake promptly, at the request of the either Commissioner, a tour of inspection of such works or sites on the river as may be considered necessary by him for ascertaining the facts connected with those works or sites.
- Make efforts to settle the differences between the parties promptly in accordance with the relevant provisions of the treaty (World Bank, 1960).

Dispute Resolution Mechanism

The treaty envisages a comprehensive tier mechanism Figure 4 for addressing any question which arises between the parties concerning interpretation or application of the Treaty or existence of any fact, which if established, might constitute a breach of the treaty. The Commission would first examine the question and endeavor to resolve the question by mutual agreement. If the Commission fails to reach an agreement, the question on the request of the either Commissioner would be dealt by Neutral Expert, who would decide whether or not the entire question or a part thereof falls in his purview as per contents of the Treaty and if it falls so shall render decision on merit accordingly. In case the question or a part thereof does not fall within the provisions of the contents of the Treaty, the neutral expert shall inform the Commission that in his opinion the question or a part thereof may be treated as a dispute, then the dispute would be deemed to have arisen, which would be referred by the Commissioners along with their views to their governments. Under such a situation, either government shall invite the other to resolve the dispute by agreement along with nomination of the negotiators together with enlisting the mediators for assistance of the negotiators if so agreed by both the governments. Alternatively, if the

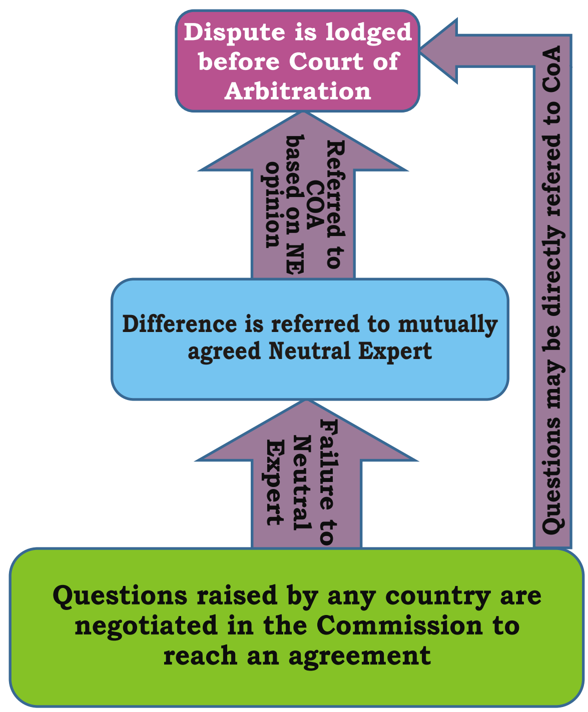

Figure 4: Dispute resolving mechanism of IWT

dispute is not resolved or not likely to be resolved through mutual negotiation or mediation, a Court of Arbitration (CoA) would be established for resolving the dispute. The Court would comprise altogether seven members with two members nominated by each of the country and the remaining three mutually nominated if so agreed, or by the World Bank if the two countries fail to reach an agreement for the three mutually agreed nominations. Although the treaty worked well for the first three decades but thereafter disputes started emerging, some of which are discussed in what follows (World Bank, 1960).

Baglihar Dispute

India started in 1999 the construction of run-of-the-river hydroelectric power generation dam (900 MW, 144 m high, 52 million m^3 pondage) at Baglihar over the River Chenab and commissioned in April 2009. After finalizing plans of the dam, India shared the project design with Pakistan who raised objections on the grounds that the design was in violation of the provisions of the IWT, especially the technical norms with regard to pondage level and gated spillway which would obstruct the flow of the Chenab River. Pakistan after formally exhausting all the available options as per provisions of the Treaty finally moved to the World Bank in January 2005 for appointment of Neutral Expert. The Neutral Expert gave a final decision in February 2007, and upheld objections of Pakistan by recommending reduction of pondage capacity by 13.5%, reduction of freeboard from 4.5 m to 3.0 m and power intake tunnel raising by 3 meters. Pakistan thinks that some design parameters were still too lax than were needed for feasible power generation thereby giving India a strategic leverage in times of political tension or war to maneuver or block flow of the river owing to its gated spillway. Under the Treaty, India could not reduce the Chenab River's flow to Pakistan below 1560 m^3/sec (55,000 cusecs) between June 21 and August 31 whereas it remained as low as 567 m^3/sec (20,000 cusecs) over the said period in 2008 (Akhtar, 2010).

Kishanganga Dispute

India is developing a 330 MW hydropower project in Kashmir over the Kishanganga River Figure 5, which originates in Indian held Kashmir and runs through Pakistan's part of Kashmir where it is called the River Neelum until it joins with the Jhelum River at Muzzaffarrabad. The Kishanganga hydropower project involves construction of a 37 m high dam across the river in Gurais and an underground powerhouse in Bandipora, both connected by a 24 km long head race tunnel. Water from the Kishanganga River would be diverted through the tunnel to the powerhouse. The diverted water after power generation would be drained into the Bonar nallah, which flows into the Wular Lake on the River Jhelum while flowing through Indian held Kashmir on its way to Pakistan. The line plan of the scheme is given Figure 5. Pakistan thinks that the project would have multi-faceted impact on its downstream agriculture, ecology and hydal projects.

NeelumJhelum Hydropower Project

Pakistan has already commenced in 2008 the construction of 969 MW Neelum-Jhelum Hydel project through a Chinese Consortium who was awarded the contract in 2007. The Neelum-Jhelum Project is a part of run-of-the-river power scheme designed to divert water from the Neelum-River to a power station on the River Jhelum. The power station is located in Azad Kashmir some 22 km south of Muzzaffarabad. More than 50% of the work has by now been completed and the project has been planned to accomplish in 2016. India's upstream Kishanganga Project has put at stake the very objective of Neelum-Jhelum Project as the Kishanganga Project will get diversion at upstream of the same river on which in Pakistan's part of Kashmir is the Neelum-Jhelum Project, thereby severely hampering its power generation. Indian point of view is that the project will divert only 10 percent of the river's flow while other estimates stand as high as 27 percent which would also affect irrigated area of 133,000 ha in Neelum Valley of Pakistan (Akhtar, 2010). Although, the impact on water flow below the NeelumJhelum Dam seems to be minimal as both projects would be diverting water to the Jhelum River, but Kishanganag project will have adverse impacts on the ecology of Neelum valley. Moreover, reservoir cushion provided by Wular Lake also deserve due consideration where Wular barrage is already a disputed project. Therefore in 2010, Pakistan lodged a case before the Court of Arbitration, complaining that

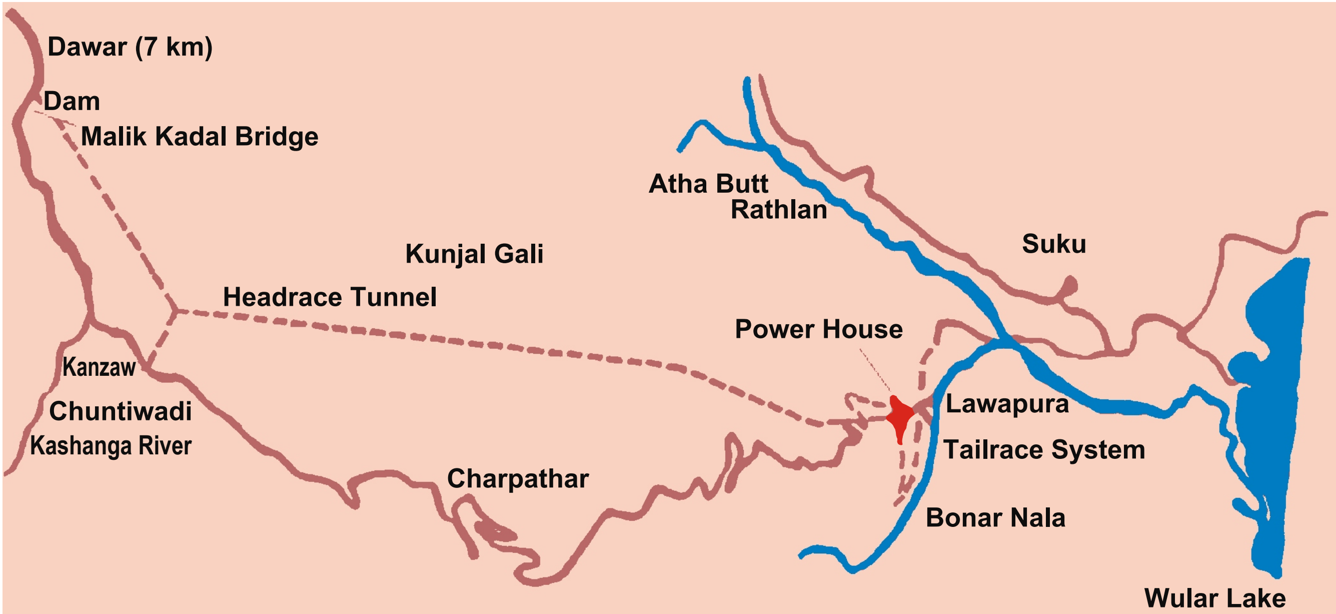

Figure 5: Line plan of Kishenganga project
(Source: www.tunneltalk.com/India-Mar10-Kishanganga-hydro-TBM-design.php)

the Kishanganga Project is violation of Indus Water Treaty as it would deprive Pakistan of its water rights due to aforesaid reasons. In 2011, the CoA after visiting both the projects asked India to stop construction of any permanent works that would inhibit restoration of the river. Although India cannot construct the dam, it has continued work on the tunnel and power plant with the intention to boost up the progress as soon as it gets some legal space. It is pertinent to mention that India has reduced dam height from the originally planned 107 m to 87 m and ultimately to 37 m, that exhibits the Indian design was against the provisions of the treaty.

Wular Barrage Dispute

Wular Lake is India's largest freshwater lake in Bandipura district of Indian held Kashmir. The lake is fed by River Jhelum and varies in size from 30 to 260 km^2, depending on the season. India in 1984, started construction of a barrage (also called Tulbul Project) having storage capacity of 0.37 BCM at the mouth of Wular Lake to regulate the release of water from the storage in the lake to maintain a minimum draught of 1.4 m in the river up to Baramula during the low flow periods for navigational facilitation. There has been an ongoing dispute between the two countries over the Wular Barrage Project since 1987, when Pakistan declared it violation of the Treaty. India stopped work on the project, but has since been pressing to restart construction with the contention that the Jhelum River through the Kashmir valley below Wular Lake provides an important navigational route for which an optimum depth of water is needed to sustain navigation throughout the year. India contends that this makes development of the Wular Barrage Project permissible. Pakistan maintains that the project is a clear violation of the treaty as will not only change catchment and ecology of the River Neelum but will also have adverse impact on the Neelum-Jhelum Project of Pakistan due to natural storage facility of Wular Lake. In fact, Kishanganaga and Wular Barrage are closely interlinked and India intends to develop the River Jhelum for navigation either through Wular Barrage or through Kishanganga Project with power generation as additional benefit.

Indian Projects on Western Rivers

India has either completed, carried out investigation or planned to construct a large number of dams on western rivers, of which the major ones include 13 on the River Chenab, 5 on the River Jhelum and 11 on the River Indus as detailed in Table 4 (Abbasi, 2012 and Akhtar, 2010 etc). The aggregate power generation capacity of those dams is several thousand megawatts, which gives an index of capacity of their operating pools and impact on downstream flow regime. In addition, investigation on many other small dams has also been reported, the exact details of which are not yet available. Amazingly, India gives very limited data of its projects, completed or planned, on western rivers. Pakistan apprehends that completion of all those projects would enable India to substantially maneuver waters of the western rivers in order to aggravate water shortage problems of Pakistan.

Table 4: Indian major hydropower projects and dams on western rivers

The River	Sr. No.	Project name	Location	Specifications	Status
Chenab	1	Baglihar I-II	Doda district, Kashmir	900 MW, 144 m high	Completed 2008
	2	Dulhasti I-II	District Kishtwar, Kashmir	780 MW, 70 m high	Completed 2007
	3	Salal I-II	District Reasi Kashmir	790 MW, 118 m high	Completed 1998
	4	Kirthai	Athuli, Kashmir	990 MW	Completed 2011
	5	Pakal-dul	Doda, Kashmir	1000 MW, 167 m high	In Indian Court
	6	Bursar	Doda, Kashmir	1200 MW, 253 m high 2.5 BCM	Under investigation
	7	Sawalkot I-II	Doda, Kashmir	1200 MW, 197 m high	Under investigation
	8	Sali	Chenab, Kashmir	715 MW	Under investigation
	9	Raltle I-II	Chenab, Kashmir	560 MW	Under investigation
	10	Karwar	Kishtwar, Kashmir	520 MW	Under investigation
	11	Gypsa I-II	Bagha tributary	395 MW	Under investigation
	12	Naunath	Chenab River	400 MW	Under investigation
	13	Shamnot	Bhutnala	370 MW	Under investigation
	14	Barinium	Chenab River	240 MW	Under investigation
	15	Ans	Ans tributary	200 MW	Under investigation
	16	Raoli	Chenab River	150 MW	Under investigation
	17	Bichari	Mohu mangat nala	104 MW	Under investigation
Jhelum	1	Uri I & II	Baramula, Kashmir, close to control line,	730 MW	Uri-I completed 1997, Uri-II awaited
	2	Kishenganga	Bandipur, Kashmir	330 MW, 37 m high	Construction halted by CoA.
	3	Ujh dam	Indian Held Kashmir	280 MW	Planning stage
	4	Gangabal	Indian Held Kashmir	100 MW	Planning stage
	5	Sonamark	Indian Held Kashmir	165 MW	Planning stage
Indus	1	Chutak	District Kargil Susu Tributary of Indus	44 MW, 15 m high	Commissioned 2011
	2	Nimu Bazgo	Alhai, on Indus River, Leh, Kashmir	45 MW, 57 m high	Under construction
	3	Dumkhar I-II	Leh Khalsi Batalik Road	45 MW, 42 m high, 70 MW, 20 m high	Planning stage
	4	Ulitopp	Indian Held Kashmir	85 MW, 40.25 m high	Planning stage
	5	Khaltsi	Indian Held Kashmir	99 MW, 20 m high	Planning stage
	6	Achinathang-	Indian Held Kashmir	220 MW, 40 m high	Planning stage
	7	Sunit	Indian Held Kashmir	295 MW, 20 m high	Planning stage
	8	Parkachik-	Indian Held Kashmir	100 MW, 61 m high	Planning stage
	9	Kirkit	Indian Held Kashmir	100 MW, 30 m high	Planning stage
	10	Drass-Suru I-II	Indian Held Kashmir	95 MW, 25 m high	Planning stage

(Source: Akhtar, 2010 and Abbasi, 2012)

India has already commissioned Baglihar, Dulhasti, Salal, Kirthai and Chutak dams and is set to complete Nimo-Bazgo hydropower project. Amongst the planned projects, Bursar dam alone on the River Chenab would have 2.5 BCM storage against cumulative multi-purpose storage provision of 4.44 BCM on all the western rivers (Kiani, 2012). Moreover, it has also been reported (PILDAT, 2010) that India is planning to implement a mega project for linking the water surplus basins with water deficit ones through a network of 30 link canals by interlinking 37 rivers, which may include western rivers as well through tunneling. India has also aggrieved its other transboundary water stakeholders such as Bangladesh, Bhutan and Nepal through multiple self serving hydroelectric and irrigation projects (Tariq, 2010) and is now bent upon to get control of western rivers of Pakistan as well.

Critical review of the treaty

It has been stated elsewhere that bombs and shells cannot make as much damage to the lands of Pakistan as can potentially be done through blockage of water by India. Therefore one of the biggest advantages of the treaty was that it prevented an imminent war between the two basin states. Researchers at Oregon State University have found that the world's 263 transboundary rivers generate more cooperation than conflict. Over the past half century, 400 treaties have been signed on the use of rivers. Of the thirty seven incidents that involved violence, 30 occurred in dry and bitterly contested region formed by Israel and its neighbors (Economist May 1, 2008).The treaty is therefore said to provide a good foundation for resolving water dispute between the two riparians subject to the provisions of the treaty are adhered to in true letter and spirit. As a result of the treaty, each country became independent of using, planning and developing waters of the rivers allocated to it as per its own wish, will, demand, supply and interests without interference of either country, which reduced chances of disputes and tensions. Resultantly substantial storage Reservoirs, inter-river link canals and barrages based diversion infrastructure was developed owing to which canal diversions in Pakistan increased from 83 BCM to 129 BCM. That facilitated to make the irrigation system more demand oriented while it was earlier based on run of the river diversions contrary to the hydrological features of the basin having almost 80% of waters available only during monsoon months of July to September. Nevertheless, disadvantages of the treaty cannot be overlooked. The treaty in fact resulted in distribution of rivers rather than distribution of their waters. On overall all basis, Pakistan lost its historic legitimate share of waters of Indus basin (Waseem, 2007). India secured full rights for use of the waters of the three eastern rivers allocated to it and compelled downstream users to abandon traditional sailaba (flood) irrigation due to disappearance of seasonal flood waters which used to permit cultivation on considerable part of the area. It also severely damaged ecology of the eastern rivers some of which present a scene of either wastewater stream or river bridges constructed on sand dunes. The infrastructure developed also required additional heavy burden of financial resources for repair and maintenance and supplementing the silted up reservoirs. Social, environmental and economic implications of storage reservoirs developed inter-provincial controversies which not only compelled to abandon, or defer, several development projects but also threatened to rip the very fabric of the country. Kalabagh dam is still the issue of inter provincial hot debate and commencement of construction of Bhasha dam is yet to be realized, whereas existing dams are silting up at an alarming rate of 0.25 BCM per annum.

The Indus Water Treaty was signed in an era when groundwater development hardly started in the subcontinent. The Treaty therefore does not envisage any article or clause regarding usage or development of groundwater resources. Now when groundwater development has boosted up and become a substantial supplemental source of irrigation water, India is geographically in much better position for harnessing groundwater as well. The eastern rivers, which India has fully secured, were primary source of groundwater recharge in the peripheries of doabas. In the absence of the recharge source and owing to rapid growth of tube wells, groundwater mining is occurring at an alarming rate in Pakistan in the lower reaches of eastern rivers. Natural slope of eastern river doabas is also from Indian side towards Pakistan's territory. India has promoted rapid development of tubewells and their operation at highly subsidized electric tariff. That is certainly affecting the yield potential of downstream highly transmissive aquifer on western Punjab in Pakistan.

The treaty also does not take into account the climate change implications and the ecological changes that would occur half a century later, which may reduce runoff from its mountainous glaciers. As such the Treaty requires reconsidering the regimes of excess and scarcity of water (Dawn February 20, 2010). The Treaty gives very flimsy touch to the today's hot issue of pollution, the direct victim of which are always downstream inhabitants. The text of the relevant clause (Article 5 Clause 10) endeavors to conserve quality, without appropriate monitoring and surveillance provisions, required for the intended uses. As the surface water based water supply schemes are developing, downstream riparian would have serious health consequences because of deteriorated water quality at the upstream due to agricultural, industrial and population growth. Mr. Abbassi, a Pakistani Water Expert, is of the view that a satellite-based, real-time telemetry system in Indian Kashmir, installed at a minimum of 100 locations for monitoring water quality and quantity would help remove mistrust on data exchange (Abbasi, 2012).

Furthermore, Kashmir is an earthquake prone area where dam safety is of immense importance. Whereas India's dam success rate is not so good and 7 out of 67 have collapsed so far. Dam collapse always has catastrophic impacts on downstream settlements, but Indus Water Treaty does not envisage any clause for compensations to the victims. Another lacuna of the Treaty is that no country is bound to exchange any data when any project is at the planning stage and data sharing is carried out only six months before the commencement of construction. By the time the aggrieved country, after completing all the prerequisites of the Treaty, refers to the Neutral Expert and/or Court of Arbitration, the construction work continue and reach the advanced stage before the final verdict is declared, and ultimately gives leverage to the country having already completed substantial part the project. Making self serving explanations of the treaty, India has either completed or near to completion of several projects on western rivers, a summary of some of those is given in the Table 4. Those dams, inter alia, would substantially enhance evaporation and seepage losses as well. Economists are therefore of the view that the construction of large number of dams on the western rivers would enable India to maneuver or block water supplies especially in low flow periods. Resultantly, the delaying or missing of irrigation would have severe harmful impacts on our agriculture which constitutes more than 21% of GDP and is key contributor to our food security, international trade and for providing raw material to textile and sugar industries. Consequently Pakistan's food security and economy will both be at stake (Khan, 2008).

Sharing Water with Afghanistan

Pakistan and Afghanistan share nine rivers with an average annual flow of about 22.58 BCM, of which the River Kabul alone contributes 20.36 BCM. However, the River Chitral, originating from Pakistan, contributes about 10.5 BCM (47 percent of the shared resources) of the entire flow (Dawn, 20^{th} April, 2011). The River Kabul originate from east of Kabul where it is small stream and its flow swells after being joined by a number of small tributaries and the River Chitral in Afghanistan. The River Kabul enters Pakistan at Warsak. On Afghanistan side, there are three dams on the River Kabul namely Naghlu (100 MW, 110 m high and 0.55 BCM), Darunta (installed 40 MW, present 11.5 MW), and Surobi (22 MW), which are located east of Kabul (Wikipedia, 2012). In Pakistan, a dam was built in 1956 on the River Kabul at Warsak near Afghan Border. The dam silted up very soon and is now serving only as hydropower plant providing 40 MW. The other major shared rivers are the River Kurram and the River Gomal, all of which join the main River Indus on its western side. The River Kurram and the River Gomal is almost non perennial with little flows having catchment area of 0.68 Mha, mostly feeding civil canals in KPK (Fahlbusch, 2004). So far there is no serious dispute on waters shared with Afghanistan due to insignificant storages. However, several projects are reportedly under planning stage. Therefore both the countries are endeavoring to reach an agreement regarding the use of shared waters.

Conclusions

The Indus Water Treaty is a comprehensive document that gives major to minor details of sharing transboundary waters and tier mechanism for resolving differences and disputes. It seemingly served to prevent wars on water disputes between India and Pakistan. But the wars so far fought between the two countries were in fact indirectly water wars, as each remained endeavoring to control head waters of Indus Basin and that too in an era when both the countries were water surplus. With rapid population growth on both sides of border, Pakistan has entered into the category of water scarce countries owing to its inherent arid and semi climate, whereas India despite being the second wettest country of the world has also become water stressed. Being amongst the thickly populated countries of the world, food security is their major concern together with cheap energy to sustain industrial growth. Climate change implications have further added to these concerns like accelerated glaciers melting, may benefit only India. Sharing of waters under droughts also remains unaddressed. Under these circumstances, India being the upper riparian is trying to get more and more out of waters flowing though its territory taking advantages of the subjective clauses and/or self serving interpretations of the Treaty. The quantum of work completed, initiated or planned on the head waters of Pakistan has added to its apprehensions that India will use the structures on western rivers as strategic, political and economic tools. Because in sharing transboundary waters, not only quantity but timing of flows is also of extreme importance. Violations of the timing for filling of Buglihar dam, duly mentioned in the Treaty, have strengthened these apprehensions. Climate change, environmental concerns, and using state of the art data sharing mechanisms are the issues least tackled in the treaty. That may give rise to multiple dimensions to differences and disputes. The Court of Arbitration is, in fact, not a Court of Justice but facilitator for arbitration. Trust deficit is already there and both the countries are nuclear powers. The international community is therefore required to frame and implement universal laws based upon equitable and fair sharing of transboundary waters all over the world for avoiding water wars.

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Effectiveness of Pedestrian Bridges in Islamabad/Rawalpindi

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Abstract

Pedestrian safety is a serious issue in Pakistan and needs urgent attention. Yearly over 7000 road accidents occur in Pakistan, as a result, a large number of people are being killed and several thousands are injured/crippled.

A pedestrian safety survey was carried out to evaluate the effectiveness of eight pedestrian bridges located in urban areas of Rawalpindi/Islamabad. Traffic around these bridges in terms of physical and operational parameters,location of bridges, road furniture and level of enforcement were observed and analyzed.

A number of observations have been made in terms of poor infrastructure provided for safe pedestrian crossing. It is recommended to provideproper road signs, road marking, provision of rumble barsand proper fence around these areas to ensure that the pedestrians use the provided facility.

Keywords : Safety,Accident, Traffic, Pedestrian crossing.

Introduction

The economic cost of road crashes and injuries is estimated to be over 100 billion rupees for Pakistan. However, the loss is more than just numbers, as road traffic injuries push many families more deeply into poverty by the loss of their breadwinners and inflict a tremendous continuous burden on the disabled victims and their families; and on national health care system (Ahmed, 2007).

An estimate of the total national cost of road accidents will help governments to realize the heavy economic losses being incurred annually as described in the "gross output" method of accident costing and socio-economic aspects of road accidents in developing countries. Governments must try to reduce these losses by providing road safety improvements and should see expenditure on road safety as an investment and not as a cost.Road Crash Problem (2013).

Road safety studies have revealed that human elements contribute to 95% of all accidents, road factors to 28% and vehicle factors of 8.5%. The road user is the sole contributor in 65% of accidents; in contrast, road and vehicle factors are usually lined with a road user factor.The 'Zebra' Pedestrian Crossings Regulations (1971) and Country road safety officer association (1986).

1. Perpetual errors e.g. driver or pedestrian looks but fails to see, distraction or lack of attention, misjudgment of speed or distance.
2. Lack of skill e.g. inexperience, lack of judgment, wrong action or decision.
3. Impairment e.g. fatigue illness, emotional stress, drugs and alcohol.

   Manner or execution e.g. deficiency in actions (fast, improper overtaking, failure to look, following too closely, taking wrong path), deficiency in behavior (irresponsible or reckless, frustrated, aggressive).

Observations at Different Pedestrian Foot Bridges

a) Near Shifa Hospital on Jhelum/G.T. Road.

1. Pedestrians cross the dual carriageway in haphazard manner. Most of them are school children and elderly who are more vulnerable to fatal accidents.
2. There is no facility of zebra crossing. Pedestrians are noticed standing on the narrow median.
3. No warning signs on the approach road of the hospital.

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4. Commercial vehicles and private cars do not observe speed limits.
5. There is no facility of bus stop on north-side of the carriageway for embarking/disembarking the passengers.
6. Public Service Vehicles (PSV) are being carelessly parked in double / triple parking pattern on northbound and underneath the bridge creating obstruction to through traffic.
7. Street traders have encroached on the main road.
8. There is no road marking near the bridge.
9. Due to the lengthy ramp of the bridge, it becomes tiring and very inconvenient for the pedestrian to cross the bridge. Moreover due to absence of fence,people cross the road through the median.

b) Near Mall Plaza Chowk on Mall Road

1. There is an army mess and telephone exchange on the north-bound, Askari Plaza on south bound. The pedestrian bridge is seldom used by the pedestrian in-spite of the fact that the bridge is located in the critical area having maximum pedestrian and vehicular movement.
2. Manhole and open drains have also been observed on one side of the footpath thus creating safety hazards.
3. Location of the bridge is inappropriate.
4. Open fences have been observed at two locations beneath the bridge thus encouraging the pedestrian to walk through the barrier. Pedestrian flow behavior is quite erratic. Reasons for not using the bridge are also attributed to sharp risers and substantial difference in elevation of the bridge.
5. Level of enforcement is poor in the vicinity of the bridge especially at the intersection where maximum pedestrian and vehicle movement occurs.
6. Zebra crossing strips are not conspicuous and have faded away.
7. PSV and motorcar drivers generally drive at a high speed even on turnings. The road users also make parallel lanes while turning.
8. Pedestrians cross the road much beyond the zebra crossing at their own convenience.

c) Near Old Saint Mary's School on Murree Road

1. There are two schools on the west and one on the east side of Murree road.
2. About 3000 school children use the pedestrian bridge during morning and evening. In contract the use of bridge by general public is minimal.
3. No safety fence on either side of the road.
4. It has been observed that most of the pedestrians and students climb over the fence and become exposed to high-speed traffic on the main road.
5. Absence of car parking facility results in illegal double-parking. Due to the absence of school parking facility traffic congestion occurs when the school children, are picked and dropped.
6. Pedestrian footbridge is being poorly maintained.

d) Near Faizabad Interchange on Murree Road

Observations on West Side

1. Pedestrian footbridge is not maintained, but is being used by the pedestrians.
2. Fence beneath the bridge had been removed at two locations.
3. In-spite of the bus bay facility, PSV and wagons are being parked underneath the pedestrian footbridge.
4. Pedestrians normally climb over the barrier to cross the road. School children climb the fence and cross the road rather than using the bridge and become fully exposed to the traffic.
5. Triple parking of large PSVs, and wagon in irregular way towards west side of road has been

Observed which not only block the road causing traffic congestion but also creates serious threat to the road users.

6. PCO billboard have been built at two locations, Newspaper stand, which is illegally placed on the footpath, obstructs the pedestrians flow and distracts the attention of road user.
7. Fruit sellers have encroached underneath the bridge that has reduced the effective width of the footpath. Due to encroachments, the pedestrian flow is also obstructedand pedestrians are forced to use the road and become exposed to accident. Billboards placed on the footpath also reduce the effective width of the footpath.
8. Taxis are being illegally parked on the ramp along the footpath which ultimately reduce the effective road width
9. Due to the presence or grocery stalls, the traffic approaching towards the ramp and entering the interchange is being obstructed and the effective width of theramp is reduced.
10. Advertisements pasted on the bridge distract attention of road users.
11. Obstacles such as charity boxes and large boulders have been placed on the curb edge.

Observations on East Side

1. There is no facility of footpaths and parking for coaches.
2. Bus stop is utilized occasionally.
3. Coaches embark / disembark the passengers beyond the bus stop.

e) Near Faizabad Interchange on Islamabad Expressway

1. Bridge is not being used by the pedestrians.
2. Fences have been removed from the barrier and pedestrians cross the bridge in jaywalking manner.
3. PSVs drivers stop the vehicles prior to the bus stop.
4. Double, triple halt of cars have been observed on the main road, which reduces the effective width of the road and creates potential accident hazards besides obstructing the flow of traffic.
5. Temporary U-turn close to the bridge has caused great potential hazards for pedestrians and bicyclists.
6. PSVs are overloaded and over clustered with passengers who sit on the rooftop of the vehicles.
7. Government transports do not bother to wait for passengers.
8. Enforcement is ineffective and traffic signs are non-existent.
9. Passengers are observed standing on the road for being embarked / disembarked in the absence of adequate bus stop facility and are fully exposed to potential road accidents.

f) Near Kurri Road on Islamabad Express Way

1. Animal drawn carts have been observed moving along the edge of the pavement.
2. Pedestrian footbridge is not being cleaned and maintained properly.
3. Pedestrian bridge has a sharp gradient thus creating difficulty for senior citizens.
4. Bus stop is not being used.
5. PSVs erratically stop the vehicles in the middle of the road resulting in double parking that reduces effective width of the pavement.
6. Passengers wait on the edge of the road pavement and become more vulnerable to potential road accidents.

g) Near Zia Masjid on Islamabad Express Way

1. Pedestrians use this bridge.
2. Fences have been removed-from the barrier.
3. No bus stop facility.
4. Pedestrians stand on the edge of pavement and become vulnerable to potential road accidents.
5. PSVs and car drivers usually drive at high speed.
6. PSV drivers generally stop in the middle of the road.
7. The approach road is not adequately paved and is encroached upon, by the fruit sellers. Further, there is no parking facility for the taxis and private cars on the approach road.
8. Shoulders are poorly constructed on one side of the highway.

Identified Discrepancies
Physical Parameters

a) Structural Conformity

Almost all the bridges have steep risers, and sharp gradient which create immense problems for pedestrians, children, old and disabled people.

b) Effectiveness of the Bridges.

1. **Near Shifa Eye Hospital On Jhelum / G. T road.** The ramp of the bridge is too long, it is tiring and hectic for pedestrians / patients to use the bridge.
2. **Near Mall plaza Chowk on Mall Road.**The location of bridge is inappropriate and this facility is not being utilized.
3. **Near old Saint Mary's School on Murree Road**.Only school students use the bridge during opening and closing hours of the school and other pedestrians rarely utilize this facility.
4. **Near Rawalpindi General Hospital on Murree**. About 1500 to 2000 patients visit the hospital every day. The visitors / patients have to cross the bridge ten to fifteen times a day to purchase the medicines from the chemists shops across the hospital, they get irritated and normally cross through road.

c) Encroachment

Street hawkers and street sellers have encroached upon the footpaths. Effective width of the footpath is substantially reduced.

d) Fence opening beneath the bridge

Pedestrians and school children enter through fence openings. They also climb over the fence barrier to cross the road and become most vulnerable to potential road hazard.

e) Road Furniture

The footpaths are poorly designed, extensively broken and uneven along various sections of Murree Road. Electric poles are located in the middle of the footpath, which not only obstructs pedestrian traffic, but also reduces effective width of the footpath.

f) Physical Obstructions

Bill boards stalls of street sellers, inappropriate located electric poles, construction materials and road furniture obstructs pedestrians and vehicular traffic flow.

g) Traffic Channelization

No channelization and lane restriction to separate the pedestrians/Cyclists from motor traffic on Mall Road and Murree road.

h) Road Signs and Markings

Road signs, marking and advance warning signals near Faizabad and old Saint Marry's school and Shifa eye hospital located on Jehlum road are nonexistent. Parking arrangement for public transport, private vehicles and taxis is inadequate along the various segments of Jehlum road, Muree road and

Islamabad expressway. Public service vehicles and wagons rarely use bus stops. Bus stopsare located very close to entrance gate of hospitals where maximum vehicular movement occurs.

i) Illuminations Arrangements

The lamp poles on all the bridges are insufficient and lamp lights are not operational during night. Further, absence of roof covers on all the bridges expose the pedestrians to rain and sunlight. These bridges are also Poorly maintained.

j) Environment

Vehicular smoke emission and loud horn aggravates the environment for the pedestrians.

K) Level of Enforcement

Driver's behavior at traffic signals, pedestrian crossingand priority intersections is generally very poor and they do not followregulations and disrespect traffic law. Furthermore, traffic constables are inefficient and are often found absent during peak hours.

Conclusions

1. An average of 36.5(%) use pedestrian bridges and almost 63.5 % cross the road through openings in the fences under the bridges or close to the bridges.
2. Most of the pedestrian's bridges have sharp gradients and steep risers. The pedestrians are reluctant to use the facility.
3. The locations of bridges are inadequate due to which pedestrians do not use foot bridges.
4. Absence of restriction for various vehicular traffic and loading and unloading of goods creates incompatible mix that further contributes to traffic congestion.
5. Advance warning signs and signals are required to be placed at appropriate locations.
6. Speed reduction devices should be introduced on Islamabad Expressway where maximum pedestrian's activities occur.
7. Shortage of public transport results in over crowdedness of passengers who cling to the rooftops of public service vehicles.
8. Level of traffic rules enforcement and road user behavior is very poor.

Recommendations

1. In view of the ineffectiveness of the bridges in terms of the utilization and safety of pedestrian, it is recommended to provide raised Zebra crossings or pelican crossings for easy and safe movement of pedestrians.
2. Modified fences should also be provided to force the pedestrians to use the bridge.
3. Extra lane for taxis may be provided near Faizabad interchange.
4. Vehicle speeds may be reduced on Islamabad expressway and Jhelum road by providing rumble bars.
5. Guardrails should be constructed along the Islamabad expressway.
6. Safety fences should be fixed along the right of way of Islamabad expressway to stop trespassing of wild animals, which usually causes road accidents.
7. Road safety education campaign should be vigorously launched from grass root level.
8. Appropriate level of enforcement should be implemented and strict punitive measure should be undertaken against those who violate the traffic rules.
9. Pedestrians should also be penalized if they violate traffic rules.
10. Barriers should be extended in both the directions.
11. Further to this, a detail study should be carried out to evaluate the optimum utilization of the bridges prior to their installation.

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Effect of Jatropha Methyl Ester on the Performance of an Off-Road CI Engine

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Abstract

In this paper the prospects of Jatropha methyl ester (JME), as an alternative fuel for compression ignition engines has been studied. JME was developed using acid and base catalysts and 95.8 % conversion efficiency was obtained by applying two step transesterification method. The chemical, physical, and thermodynamic properties of JME were determined, which were analogous to those of mineral diesel. The fuel, in blended form, was used in a four stroke, three cylinders, water cooled naturally aspirated stationary diesel engine. The performance of the engine was evaluated by using blends of JME with diesel in various ratios at loads ranging from 8 to 89%. The experimental results showed that the blend of 25% JME and 75% diesel (J25) exhibited the best results for optimum load of 82%, which were: about 6% increase in brake specific fuel consumption, 29% decrease in CO, 3% decrease in HC, and about 2.5 % increase in NO_x, as compared to mineral diesel.

Keywords : Jatropha oil; Biodiesel; CI Engines; Exhaust Emission

Introduction

The importance of compression ignition (CI) engine has been established for many years but its significance is increasing rapidly now days. It is being used in power plants, construction industry, and agriculture, industrial, and transportation sectors. Generally the CI engines are used to develop large power and are preferred because of their higher thermal efficiency and longer life.

In general, the engines are fueled with mineral diesel, the reserves of which are depleting rapidly. It is anticipated that the reserves of mineral diesel will be exhausted in next 40 years (Global Reports and Publications 2007).

The other problem associated with diesel fuel is deterioration of the environment. Combustion of too much fuel creates global warming, green house effect, and damaging the ozone layer. The exhaust gasses produced as a result of combustion of mineral fuels are also health hazardous i.e. damaging nervous, circulatory, and respiratory systems and are also causing skin diseases.

Serious efforts are needed to be done to save the future of our next generation. The scientists and researchers are reviewing the prospects of use of biodiesel as fuel for CI engines, which is obtained by modifying the chemical structure of algae, vegetable oils and animal fats. The fuel is recyclable and degradable, available in every season and every where in the world at economical rates. Lower amount of CO_2, CO and un-burnt HC is found in the tail pipe of the engines, whereas the exhaust emissions are absolutely free from oxides of sulfur (Yuan, et. al. 2008, Phan and Phan, 2008). Unfortunately a bit higher amount of NO_x is observed in the exhaust emissions (Yage, et. al. 2008).

More than 300 vegetable oils have been identified which can be used to fuel the CI engines, which includes edible, non-edible and used cooking oils. Jatropha oil can be one of the best possible feedstock, which can be used to develop biodiesel (shahid and jamal 2011). It is non-edible oil, which can be grown in barren areas; less water and care is needed to grow this plant. But the problem associated with the use of jatropha oil to fuel the CI engines is its higher viscosity and specific gravity, and lower calorific value. The oil is chemically modified so as its physical properties become comparable with those of mineral diesel.

The modified oil is called biodiesel which can be used to fuel CI engines, in pure and blended form without any modification to be made to the engine. It has been reported that there is a reduction of about 68% in CO, 19% in CO_2, 8 % in soot, and 12% in un-burnt HC. However nearly 6 % increase in the amount of NO_x emission has been reported (Yuan, et. al. 2008, Rao et. al. 2008).

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In this paper, the prospects of jatropha oil as a substitute fuel for compression ignition engine has been studied.

Jatropha Plants

Jatropha plants have a wide variety with beautiful flowers, and have many species, but the most commonly grown in many parts of the world is Jatropha curcas. Pakistan has also started the plantation of Jatropha curcas. The average life of the plant is 50 years. Generally the plant height is 3 to 5 meters but can be up to 8 meters in favorable conditions. It can be grown in tropical areas and areas of heavy rains. Each plant yields 3 to 9 kg of fruit which contains 37% oil. The nuts are crushed to extract the oil.

Transesterification of Jatropha oil

The kinametic viscosity of jatropha oil is about 10 times higher as compared to mineral diesel, which is due to its complicated structure. The specific gravity of jatropha oil is 8-10% higher than that of diesel (Haldara et al. 2009). Various methods are applied to reduce the kinametic viscosity and specific gravity which includes pyrolysis, cracking, blending, micro-emulsification and transesterification.

Transesterification is most commonly used to modify the chemical structure of jatropha oil so as the kinametic viscosity is decreased from 38.1 to 3.8 cSt and specific gravity from 0.92 to 0.88 (Enweremadu and Mbarawa, 2009). Mainly the jatropha curcas oil consists of triglycerides with a little content of diglyceride and monoglyceride. The triglycerides and diglyceride needs to be converted into monoglycerides and the byproduct glycerin has to be removed.

In this study, jatropha oil was treated with methanol in the presence of acid catalyst, with which its acid number was reduced from 3.6 to 1.1 mg KOH/g.

It was then reacted with methanol in the presence of NaOH catalyst for about two hours. The reaction was carried out at the temperature of 65±2°C. Pale yellow oil was floating above the thick dark brown glycerol. The glycerol or glycerine settled in the bottom of tank was removed and the pale yellow oil, named as jatropha methyl ester (JME) or biodiesel was separated. Although stiochiometric value of methanol to oil is 13 yet due to reversibility of the reaction higher amount of methanol (about 20%) is used to shift the balance of reaction toward the product side (Ban-Weiss et. al. 2007). The excess amount of methanol remained present in JME which was recovered with the help of distalisation method.

The PH value of JME was found to be 10, which was due to the presence of NaOH. The JME was washed with normal hot water repeatedly, to remove the NaOH till its PH value was reduced to 7.05, showing the neutrality of JME.

The physical, chemical, and thermodynamic properties of JME were tested and compared with those of mineral diesel as shown in Table 1.

Experimental Setup

The fuel was tested in a four strokes, three cylinder, direct injection, water cooled, naturally aspirated engine. The engine was connected to a three phase A.C electric generator. The required instrumentation was installed to measure the fuel flow rate, speed. The engine exhaust was measured with a five gas exhaust gas analyzer (IMR 3000), as shown in Figure 1.

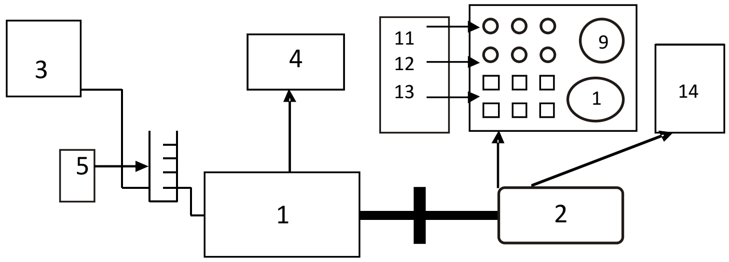

Figure 1: Schematic Diagram of Experimental Setup

1.	Diesel Engine	2.	Electric Generator	3.	Fuel Tank
4.	Graduated Glass Cylinder	5.	Tachometer	6.	Speed Controller
7.	Volt meters	8.	Ammeters	9.	Load Control Switches
10.	Load Bank	11.	Exhaust Gas Analyzer		

Results and Discussion

Blends of JME with diesel in ratios of 5, 10, 25, 50 and 75%, named as J5, J10, J25, J50 and J75 correspondingly, were prepared. Engine performance and exhaust emissions were evaluated using the blended fuel for the loads varying from 8 to 89% at a constant speed of 1500 rpm.

1. Effect of JME on Brake Specific Fuel Consumption

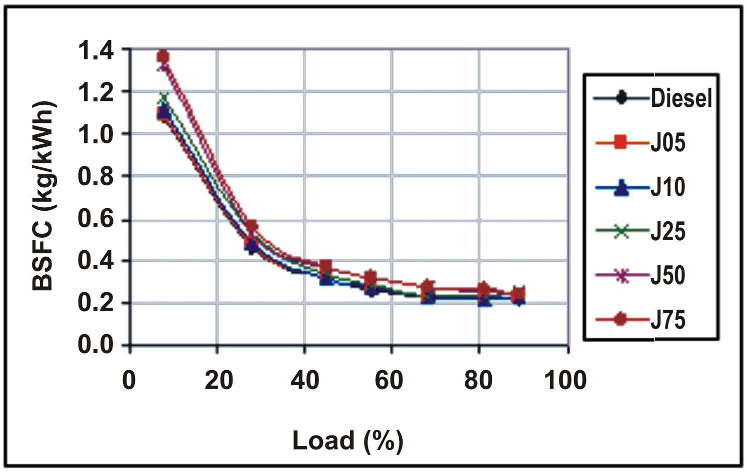

Figure 2: Effect of load on BSFC for various ratios of JME

It can be observed from Figure 2 that the amount of fuel consumed per unit brake power, known as brake specific fuel consumption (BSFC), decreases as load increases, which is due to the improvement in combustion phenomenon for higher loads. The decline is very sharp from 8 to 28% loads, and then it decreases gradually up to 65% load and becomes almost constant afterword.

The value of BSFC increases with the increase of ratio of JME in the blends for all loads. The reason is that the calorific value of JME is lower as compared to that of diesel, as shown in Table 1. However, the difference is not notable for J5, J10, and J25, particularly for optimum load of 82%.

2. Effect of JME on Carbon Monoxide Emission

Carbon monoxide (CO) is a colorless and odorless gas which is the most dangerous pollutant emission that destroys the nervous, circulatory, and respiratory systems. It causes sickness, headache, nausea, fatigue, dizziness, and even fatal (http://www.epa.gov/iaq/co.html 2012).

Carbon monoxide is produced when insufficient air/oxygen is available during combustion of fuel or insufficient time is available for combustion. The presence of CO in the exhaust emissions of engine indicates that the thermal efficiency has been reduced.

It can be observed from Figure 3 that the amount of CO increases with the increase of load using diesel and blends of diesel and JME. For instance, in case of use of diesel, CO increases from 80 to 180 ppm as load increases from 8 to 55% and then increases sharply till 68% load. Finally the curve becomes smooth till peak value of 310 ppm for 89% load.

The amount of CO decreases as the ratio of JME increases in the blends and the lowest amount of CO is found in the

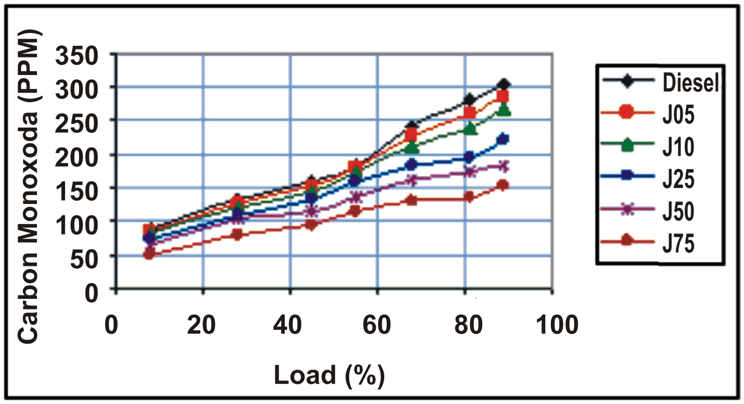

Figure 3: Effect of load on CO Emission for various ratios on JME

exhaust emissions when the engine is fueled with J75. For example its amount is reduced from 80 to 51 ppm and 310 to 150 ppm for extreme loads of 8 and 89% respectively.

The reason of decrease in CO in the exhaust emissions when CI engine is fueled with blends of JME is the presence of oxygen in biodiesel.

Combustion quality of JME fuel improves, as compared to diesel due to its higher cetane number which reduces the ignition delay and increases the combustion duration due to which amount of CO emission is reduced. Moreover the presence of oxygen in JME fuel improves the combustion. Huang et al. 2008 and Jindal et al. 2010 also reported the similar type of results.

3. Effect of JME on Total Hydrocarbon Emission

Although the presence of THC is not as harmful as CO yet a few of its compounds affect the nervous system. Breathing in presence of toluene (a compound of THC), with concentration of higher than 100 ppm, for several hours may cause fatigue, headache, nausea, and drowsiness (Draggan 2010).

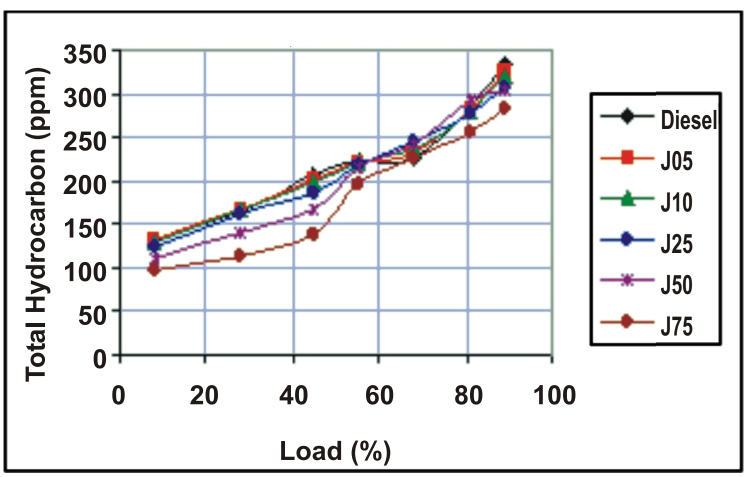

Figure 4: Effect of load on THC emission for various ratios of JME

It can be observed from Figure 4 that the amount of THC emission increases with the increase of load, in case of use of diesel as well as all blends of diesel and JME. For instance the amount of THC emission is 132 ppm, for 8% load, which increases to 333 ppm for 89% loads.

Fortunately, the amount of THC emission decreases with the increase of ratio of JME in the blends. For 8% load, 2 to

28% decrease in the amount of THC emission can be observed, as the ratio of JME in the blends is increased from 5 to 75%, respectively. Almost similar type of pattern can be observed for other loads. Some researchers also showed similar type of trends (Reddy and Ramesh 2006, Puhan et. al. 2005).

The reason of reduction of THC emission is the same, as has already been discussed, that the combustion quality is improved when JME is added in the fuel.

The combustion phenomenon can further be improved by modifying engine operating parameters like injection timing and injection pressure and making certain modifications in the injection system.

4 Effect of JME on Oxides of Nitrogen Emission

Oxides of nitrogen (NO_x) are very serious precarious pollutant emission. These are produced when the temperature inside the cylinder becomes higher than 1500ºC and its formation rate increases exponentionally when temperature becomes higher than 1800ºC. It reacts with water and forms nitric acid which makes corrosion on the tips of piston and cylinder. The higher concentration of NO_x in the environment may be one of the causes of acid rains.

The amount of NO_x increases for higher loads which are due to combustion of higher amount of fuel. Generally higher amount of NO_x are produced in the exhaust emissions of the engine when it is fueled with biodiesel(Nabi et. al. 2009).

It can be observed from Figure 5 that the amount of NO_x emission increases from 22 to 210 ppm as load increases from 8 to 82 %, because the rate of NO_x formation increases for higher loads due to rapid increase in temperature.

Unfortunately, the amount of NO_x emission increases when diesel fuel is replaced with biodiesel. Its value increases with the increase of ratio of JME in the blends. For instance the amount of NO_x emission is found to be 240.5, 241.5, 244.0, 256.0, and 270.7 ppm when engine is fueled with J5, J10, J25, J50, and J75, respectively, as compared to 239.5 ppm in case of use of diesel, for 89 % load.

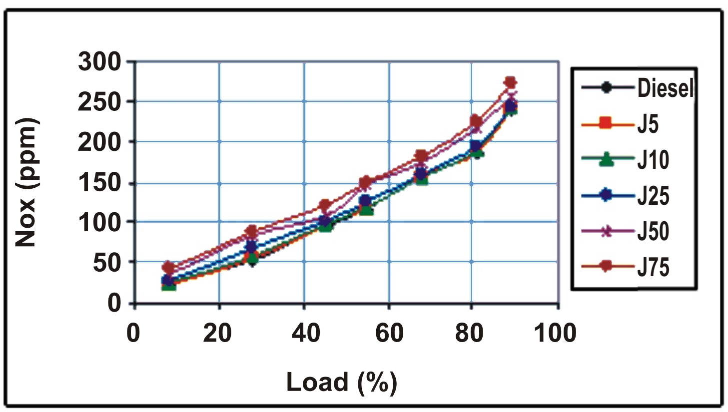

Figure 5 : Effect of load on No_x Emission for various ratios of JME

Almost similar type of trend is found for others loads. The increase of NO_x emission is due to the improvement of combustion quality, which has already been discussed. Less amount of soot is produced in case of use biodiesel, as compared to diesel (Ban-Weiss et. al. 2007). The soot is a source of dissipating heat, the temperature inside cylinder rises when less soot is formed, due to which higher amount of NO_x is produced.

It can be concluded that the difference in NO_x production is not big up to J25. Some other researchers also reported similar type of results (Yage, et. al. 2008, Kegl 2008). Hence it is suggested that the use of blend of diesel and JME with a ratio higher than 25% should be avoided. However, higher ratio of JME in blends can be used by modifying the fuel injection system and/or using after treatment, catalytic conversion, and exhaust gas re-circulation methods.

Table 1 : Properties of Diesel and JME.

Property	Unit	Diesel	JME	ASTMD6751-02
Density at 20 °C	Kg/m^3	850	890	870-890
Kinematic	mm^2/s	3.6	3.8	1.9 to 6.0
Viscosity at 40°C				
Flash Point	°C	68	195	> 120
Pour Point	°C	-6	-8	-15 to 10
Calorific Value	MJ/kg	42	38.5	--

Table 2 : Engine Specifications

Make/Type	Perkins/AD 3.152
Volumetric efficiency	85%
Bore	91.4 mm
Stroke	127.0 mm
Injection Timing	17° BTDC
Injection Pressure	190 bars
No. of cylinders/ Nozzles	3
No. of orifices in each Nozzle	4
Brake mean effective pressure	7.157 bars
Maximum engine power @ 1500 rpm	37 kW

Conclusions:

Jatropha curcas oil has sufficient potential to be used as feedstock for biodiesel.

The oil can be used after modifying the chemical structure via transesterification method.

JME can be successfully used to run a compression ignition engine.

The brake specific fuel consumption increases by 6% when J25 is used as fuel.

JME is an environmental friendly fuel. There is a decrease of about 29% in CO and 3% in THC, at optimum load of 82%, in case of use of J25 fuel.

There is about 2.5% increase in NOx when J25 is used as fuel at optimum load of 82%.

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Evaluation of Moisture Sensitivity of Various Asphalt Mix

Shiraz Ahmed[1], Faizan Ali[2] and Kamran Muzaffar Khan[3]

Abstract

Moisture in the asphalt mix can damage it severely, as it reduces the adhesion between the binder-aggregate interface and cohesive failure within the binder. Moisture damage in asphalt mixture causes reduction in strength, stiffness and life, the damage most commonly known as Stripping. Various tests are available to the resistance of asphaltic mixtures against moisture damage, and one of them is AASHTO T283. In this paper, moisture susceptibility of two different asphalt mixes, i.e. HMA 60/70 and Polymer Modified Bitumen 60/70 is determined and compared using the method describe in AASHTO T 283 (2003 Modified) by forming Marshall specimen. The gradation and various properties of the mix are detailed in the paper. The indirect tensile strength of Marshall Specimens of HMA and PMB was found out using compression Testing Machine. The result shows that PMB specimens have more IDT(Indirect Tensile) strength as compared to conventional Hot Mix asphalt specimens so PMB is more resistance to moisture damage.

Key Words : Adhesion, Stripping, Moisture Susceptibility, Marshall, Indirect tensile strength.

Introduction

Pavement fails prematurely due to presence of water. The failure then shows out in the form of isolated distresses and in the form of early cracking and rutting. Moisture sensitivity has been an important consideration in the design of flexible pavements for a long time. (Hveem,1940) emphasized that the moisture sensitivity is as important as the other parameters that are essential during the selection of quality asphalts for flexible pavement construction. A recent survey carried out (Aschenbrener,2002) by 55 states and federal highway agencies shows that 87% of their moisture sensitivity tests, 82% of highways require some maintenance and treatment to defend against moisture damage. Stripping has a severe effect on the performance of Pavement and unexpected increase in maintenance costs is often experienced. Temperature and seasonal variations and moisture can have an intense effect on the strength and functional performance of flexible pavements. When serious climatic conditions are combined with heavy loading conditions and poor construction materials, early failure may occur due to decrease of adhesion between asphalt binder and aggregate particles.

Problem Statement

The damage caused by moisture on asphalt concrete pavement is known as stripping. Stripping has severed effects on the pavement's structural integrity. Many tests has been developed in the past to for the prediction of moisture sensitivity. In many of the methods developed, thaw freeze cycle is performed on the cores taken from the field and on the Marshall samples prepared in the laboratory. In most of the cases single freeze thaw cycle was applied on the cores. The effects of a no. of freeze thaw cycles on strength of HMA are not well known.

Objectives

The primary objectives of the research are to:

- Explore moisture sensitivity of different asphalt mixes
- Assess the effect of various number of freeze thaw cycles on the tension strength of Marshall Specimens of the mix.
- Compare the moisture sensitivity of asphalt mixes with different binders.

For achieving the above mentioned objectives conditions set by AASHTO T 283 has been used.

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Literature Review

The moisture effect on physical properties and mechanical behavior of asphalt mixtures has been known for many years but even though it has proven to be very difficult to confidently predict this type of distress in the laboratory because of numerous factors involved.

Moisture can effect in different form. Adhesive failure between the bonding material and aggregate results in de-bonding which, in an advanced state, is identified as “stripping” as shown in figure 2. As a result of stripping, high strength HMA pavement layer reduces to weaker untreated asphalt section. When stripping occurs in different isolated regions in pavements, it results in the development of potholes and if it occurs in large area, rutting ad fatigue cracking may develop due to decrease in structural support of the pavement.

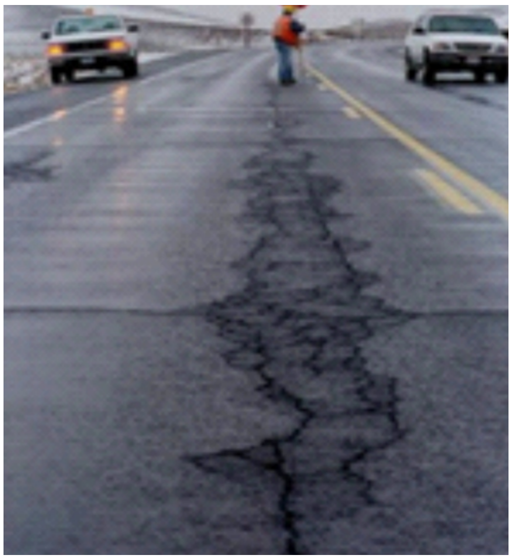

Figure 1 : Fatigue cracking due to stripping in asphalt pavement

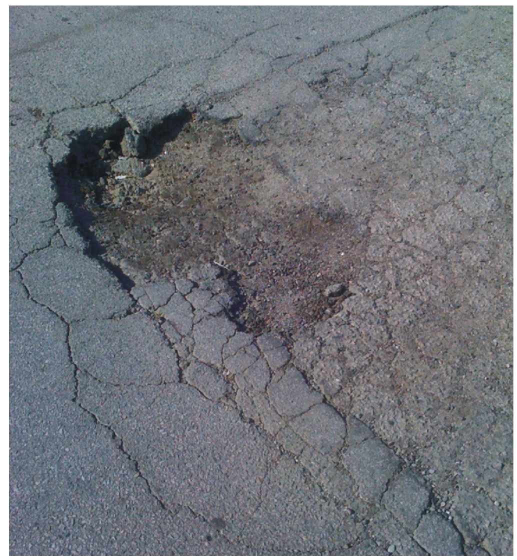

Figure 2 : Stripping in asphalt pavement due to lack of bond b/w asphalt and aggregate

(Source:http://www.pavementinteractive.org/article/moisture-susceptibility)

Historical Development

The immersion compression test was the first moisture damage test under ASTM standards introduced in 1950 on the compacted specimens. Thelen conducted work on the surface energy of asphalt and aggregate bonding relationship (Thelen,1958). Andersland and Goetz developed the sonic test to assess the resistance against stripping in HMA samples (Andersland and Goetz,1956). In 1978 the Lottman's laboratory test was a breakthrough in industry for predicting stripping in asphalt concrete. This test was later modified and standardized as AASHTO T 283 1978 the Lottman's. Root and Tunnicliff developed some advancement in Lottman test after an extensive evaluation of anti-stripping additives(Root and Tunnicliff ,1980).

In the 1980s, Kennedy et. al. at University of Texas presented two new test methods to the industry: Freeze-Thaw Pedestal Test (Kennedy et. al.1982) and Boiling Test (Kennedy et. al.1984) **.**The freeze-thaw pedestal test was a modification of the method introduced earlier in 1980 by Plancher et al and the boiling test was similar to the test used by Saville and Axon in 1937. Due to sponsored research by SHRP for assessing better the effect of moisture on various mixes, Al-Swailmi and Terrel develop Environmental Conditioning System (ECS) (Al-Swailmi and Terrel,1992) and Aschenbrener and Currier introduced Hamburg wheel-tracking device in United States (Aschenbrener and Currie,1993).There is an extensive quantity of literature regarding detecting moisture susceptibility of asphalt concrete. However, the research conducted so far is of empirical nature. The general agreement among the designers/engineers is that the tests developed so far could not properly simulate the field conditions and we could not exactly judge the field performance in the laboratory (Roberts et. al. 1996)**.**

Despite of some short comings in AASHTO T 283, it is still the best available procedure to investigate the moisture sensitivity of HMA mixes. NCHRP launched a research program to evaluate the test constraints of AASHTO T 283 with a purpose of improving the consistency of the testing procedure (Epps, et. al. 2000). Anderson and Dukatz in1982 analyzed various anti-stripping additives that are available commercially and their effect on asphalt properties. They came up with the result that anti-stripping additives have a tendency to soften the asphalt, decrease temperature susceptibility, and refine the aging characteristic of asphalt (Anderson and Dukatz, 1982).

Aschenbrener, et. al. (1995) compared the performance of HMA mix of known antistripping potential in field with 04 moisture susceptibility procedures: 1. AASHTO T-283, 2. ASTM D-3625, 3. Environmental Conditioning System, and 4. Hamburg wheel-tracking device. And they concluded that, AASHTO T-283 gave better results by simulating the field conditions in the lab (Aschenbrener 1995). Pan et al in 1999 evaluated seven different mixes using AASHTO T 283 and PUR Wheel tracking device. They concluded from AASHTO T 283 test that stripping resistance of the mixes has influenced by moisture conditioning and from PUR Wheel test that temperature and moisture conditions has greatly affected the severity of stripping (Pan and White, 1999).

(Aiery et al in 2007) introduced a new test known as the Saturation Ageing Tensile Stiffness (SATS) test. The test is a laboratory test in which combined effect of aging and moisture damage is checked. The test consists of initial saturation of the compacted cylindrical specimens before placing them into moist atmosphere, high temperature & pressure for a longer duration. The stiffness modulus and the saturation measured both before and after the test, are used as indication of the sensitivity against moisture damage of the mixes. They concluded that, compared to the AASHTO T283, the SATS test was more aggressive conditioning procedure (Airey et.al. 2008).

Erol Islander et. al. in 2007 compared the laboratory samples with the field sample by taking cores from wearing course of identical sizes, like Marshall Samples, for dense graded asphalt mixtures. Both the types were subjected to three different moisture conditions and indirect tensile strength was carried out to compare the performance of both types of samples. Laboratory mixtures gave 1.22 and 1.30 times more values as compared with field samples at 10 C and 20 C respectively in the indirect tensile strength test. Marshall Samples gave higher resilient modulus for all control and conditioned mixtures (Erol and Atakan, 2012).

Methodolgy

As from the literature studied above the AASHTO T 283 is the best available procedure so far for detecting moisture sensitivity of compacted HMA specimens, so this method is used. The test is performed by compacting specimens at an air void level of six to eight percent. Two samples were tested as a control samples and tested as dry, and four samples are selected to be conditioned by saturation with water undergoes several cycles of freeze-thaw different from both type of mixtures i.e. -HMA 60/70 and PMB 60/70. The samples were then tested for indirect tensile test. The tensile strength of the conditioned sample was compared with the specimen unconditioned / control ratio to determine the tensile strength. (TSR)

1. Flow Sheet of Sampling And Testing

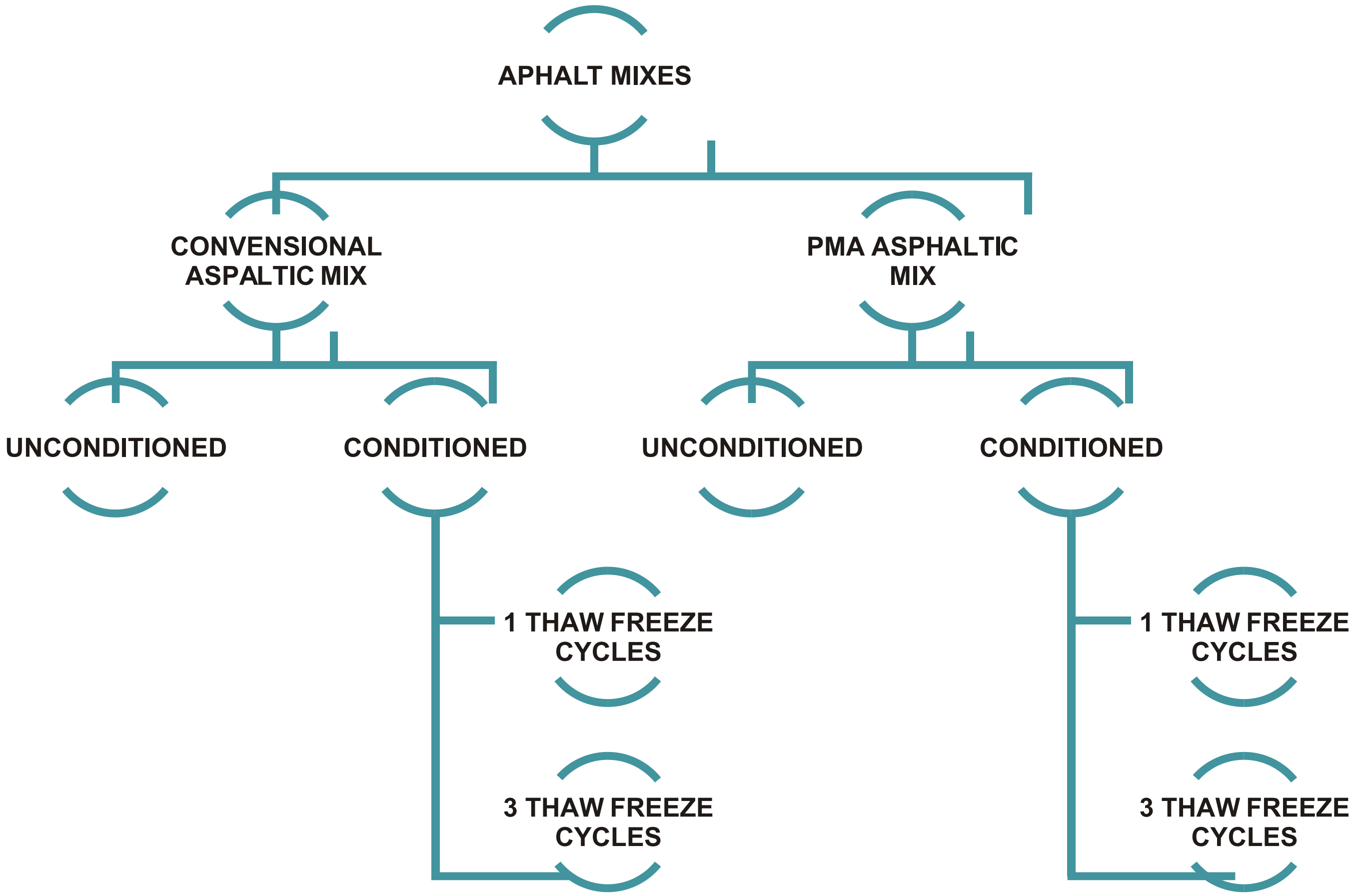

Figure 3 : Flow Chart of condition of mixes for testing

2. Testing Matrix

Table 1 : Testing, Standards and Apparatuses used in Test

Description	Standards	Apparatus
Gradation of Aggregates	NHA class A	Sieves analysis Balance
Optimum Asphaltic Content	Marshall method	Marshall stability & flow tester
Sample Preparation	Job Mix Formula	Marshall compactor
Gmb of Samples	AASHTO T 166	Gmb apparatus
Gmm of Samples	AASHTO T 209	Vacuum container
Grouping of samples	AASHTO T 269	
Testing of conditioned samples(S2)	ASSHTO T 283	Compression testing Machine
Testing of unconditioned samples(S1)	ASSHTO T 283	Compression testing Machine
Determination of tensile strength ratio	ASSHTO T 283	TSR= S2/S1

3. Testing Procedure.

Six Samples of HMA and PMB each are prepared. Marshal samples of standard sizes are made. After mixing, pour it in the marshal moulds. Compact the mix to the 7 percent air voids level, using Marshall Hammer. Then allow the specimens to remain at room temperature for 24 hours. Calculate the theoretical maximum specific gravity (Gmm), bulk specific gravity (Gmb), height, volume and air void content (Va) of each sample. Divide the six samples of each

type, into three subsets of two. The average air void content (Va) for each subset should be similar. One subset will be "unconditioned" (tested in a dry state) and the other will be "conditioned" (tested in a saturated state).
The inputs for the research regarding Marshall Samples Gradation, Physical properties of Aggregate, Properties of asphalt binders, Properties of Mix are given in tables below.asphalt binders, Properties of Mix are given in tables below.

Table 2 : Adopted Gradation

SLEVE SIZE		Combined gradation, asphalt wearing course class A	
Inch	Mm	Adopted gradation	NHA specifications class A
1	25	100	100
3/1	19	90	90-100
1/2	12.50	-	-
3/8	9.50	56	56-70
#4	4.75	35	35-50
#8	2.36	23	23-35
#50	0.300	5	5-12
#200	0.075	2	2-8

Table 3 : Physical properties of Aggregates

Text description	**Specification reference**	**Result**	**Test description**	**Specification reference**	**Result**
Aggregate crushing value (ACV)	BS 812, PART 1	22.5	Aggregate impact value (AIV)	BS 812 PART 3	13.5
Toughness index (TI)	BS 812, PART 1	74	Los-Angeles abrasion value (LAA)	ASTM C131	23%
10% fine value (TFV)	BS 812, PART 3	0.70	Elongation index (EI)	BS 812, PART 1	11%
Sod. Sulphate soundness value	AASHTO T104	3.32%	Flakiness index (FI)	BS 812, PART 1	4.75%

Table 4 : Properties of different type of Asphalt Binder

Sr. No.	**Description**	**PMA (1.6+Elvaloy4170)**	**Asphalt penetration grade "60/70"**
1	Type	Modified	Neat
2	Ring & ball softening pt	58	49
3	Penetration	46	65
4	Ductility@25^{O}C	45	100
5	Specific gravity	1.023	1.03

Table 5 : Hot Mix Asphalt and Polymer Modified Asphalt (optimum asphalt content) design properties.

MIX TYPES	OAC (%)	Gsb	Gmm	Gmb	VA (%)	VMA (%)	VFA (%)	Stability (kg)	Loss of Stability (%)	Flow (0.25mm)	Stiffness Index Stab./flow
1a PMA	3.83	2.65	2.52	2.37	5.9	13.90	58	1378	11	10.58	128
1b 60/70	3.87	2.65	2.51	2.37	5.7	13.99	59	1305	14.30	11.00	119

4. Results

The results of the testing are shown below in tables and graphs.

Table 6 : Sample standard size

Dia. inch	thick inch	Vol. inch^3
4	2.5	31.4
Dia. cm	**Thick cm**	**Vol. Cm^3**
10.16	6.35	514.55

Table 7 : Calculation of Bulk Specific Gravity Gmb of each Sample

S. No.	Wt. of Dry sample A grams	Wt. of SSD B grams	Wt. of submerged C grams	Gmb Bulk Sp. Gravity
1	1179	1185	648	2.195
2	1209	1219	662	2.17
3	1196	1207.6	653	2.16
4	1211	1222	660	2.154
5	1174	1185	636	2.14
6	1120	1127	625	2.23
7	1185	1196	639	2.13
8	1203	1211	655	2.16
9	1177	1185	633	2.13
10	1191	1202	645	2.14
11	1219	1225.5	659	2.15
12	1207	1217	659	2.16

Table 8 : Calculation of Max. Specific Gravity Gmm of each Sample

S.No.	Wt. of Dry Sample A grams D grams	Wt. of Con. + water E grams	Wt. of Con. + Water +Sample	Gmm Max. Sp. Gravity
1	1179	18886	19568	2.37
2	1209	18886	19582	2.356
3	1196	18886	19573	2.35
4	1211	18886	19578	2.33
5	1174	18886	19564	2.367
6	1120	18886	19553	2.47
7	1185	18886	19570	2.36
8	1203	18886	19574	2.225
9	1177	18886	19563	2.35
10	1191	18886	19567	2.33
11	1219	18886	19588	2.36
12	1207	18886	19571	2.31

Table 9 : Calculation of Vol. of absorbed water for Conditioned Samples

Sample	wt. of Dry sample A grams	wt. of SSD B grams	Vol of ab. Water J cm^3
F2	1196	1227	31
F6	1209	1239	30
E1	1174	1211	37
F3	1120	1158	38
S5	1177	1212	35
S4	1191	1221	30
S3	1219	1253	34
S6	1207	1241	34

Table 10 : Indirect tensile strength of Samples

S. No.	Sample	Cond-ition	Gmb.	Gmm.	Va %	Va cm^3	J=B-A cm^3	S=100 J/Va %	load KN	Load lb	Indirect tensile Strength psi	
					Gmm-Gmb/ Gmm *100	Va% E/100	Vol. of Ab. water	Deg. of Satur-ation				
1	F4	DRY	2.195	2.37	7.38	37.97	--	--	10.5	2360.50	150.35	HMA 60/70
2	F5	DRY	2.17	2.356	7.66	39.41	--	--	10	2248.1	143.19	
3	F2	CON 1	2.16	2.35	8.1	41.7	31	74.34	5.5	1236.4	78.75	
4	F6	CON 1	2.154	2.33	7.55	38.85	30	77.2	6	1348.8	85.91	
5	F1	CON 2	2.14	2.367	9.6	49.4	37	74.9	4.5	1011.6	64.43	
6	F3	CON 2	2.23	2.47	9.7	49.9	38	76.15	4	899.2	57.27	
7	S1	DRY	2.13	2.36	9.7	49.9	--	--	12.2	2742.7	174.69	PMB
8	S2	DRY	2.16	2.225	7.5	38.6	--	--	12	2697.7	171.83	
9	S5	CON 1	2.13	2.35	9.3	47.85	35	73.1	8	1798.4	114.55	
10	S4	CON 1	2.14	2.33	8.1	41.67	30	72	7.5	1686.1	107.39	
11	S3	CON 2	2.15	2.36	8.9	45.8	34	74.23	6	1348.8	85.91	
12	S6	CON 2	2.16	2.31	6.5	33.44	34	80.7	6	1348.8	85.91	

Note:DRY: Unconditioned Samples, CON 1: Specimens subjected to one freeze-thaw cycles, CON 2: Specimen subjected to three freeze- thaw cycles.

Form the above results following comparison tables are made to compare the performance of conventional Mix and Polymer modified bitumen.

Table 11 : The comparison of indirect tensile Strength of HAM and PMB mix.

S. No.	Sample	HMA 60/70	Sample	PMB 60/70	Cond.
1	F4	150.35	S1	174.69	DRY
2	F5	143.19	S2	171.83	DRY
3	F2	78.75	S5	114.55	CON 1
4	F6	85.91	S4	107.39	CON 1
5	F1	64.43	S3	85.91	CON 2
6	F3	57.27	S6	85.91	CON 2

Table 12 : Comparison of Avg. Tensile strength in psi of HMA and PMB.

S. no	Condition	average Strength psi	
		HMA 60/70	PMB
1	Dry	146.77	173.26
2	CON 1	82.33	110.97
3	CON 2	60.85	85.91

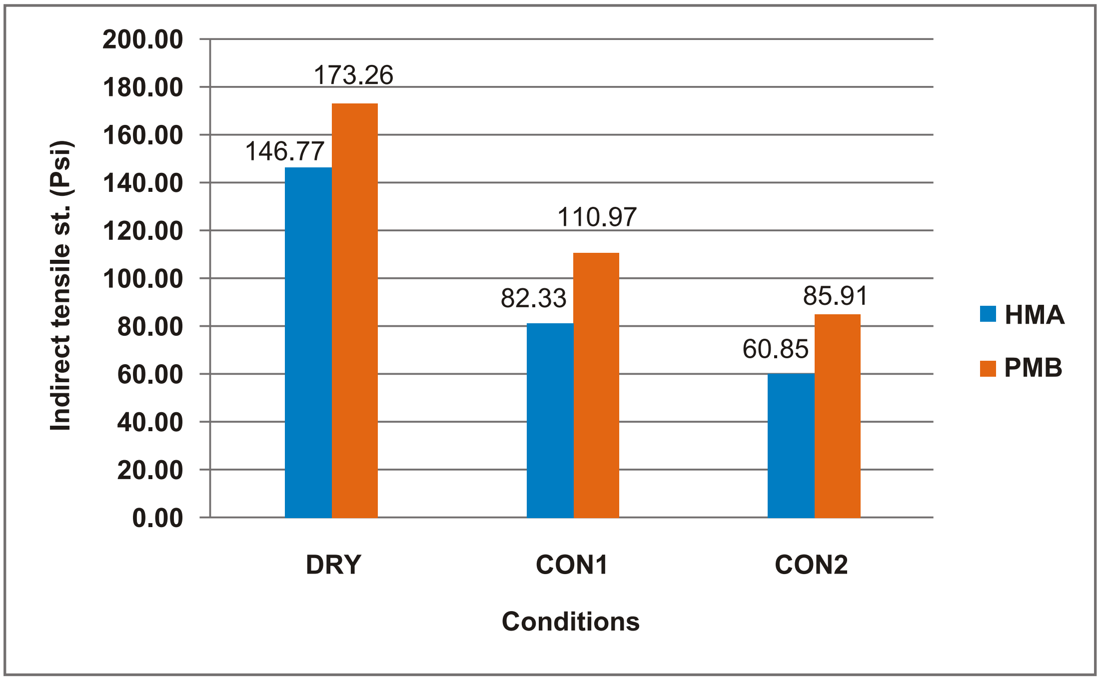

Figure 4 : Comparison of Avg. tensile strength in psi of HMA and PMB.

Table 13 : the tensile Strength Ratios of HMA and PMB

MIX	Condition	Ratio
HMA	Con 1	0.6
	Con 2	0.4
PMB	Con 1	0.65
	Con 2	0.5

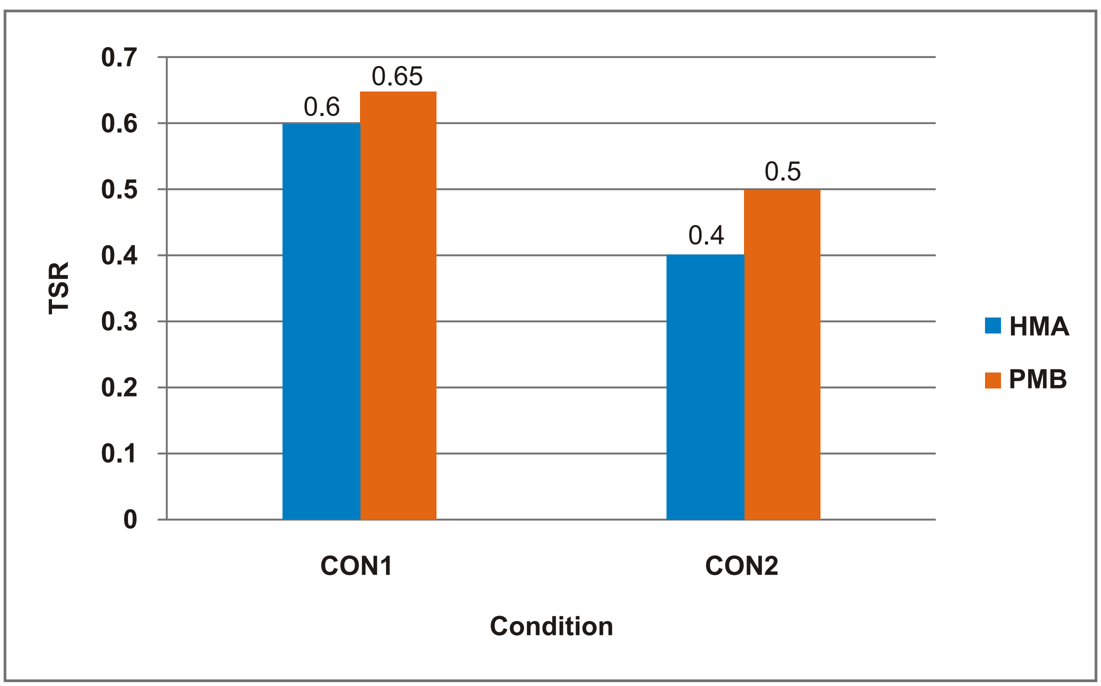

Figure 5 : Comparison of TSR of HMA and PMB

The tensile strength ratio (TSR) was calculated by dividing the average tensile strength of the conditioned samples by the average tensile strength of the unconditioned control samples, 80 percent TSR is normally required for the mixes to be resistance against moisture damage.

Conclusion

After experimentation it has been concluded that:

1. Effect of thaw-freeze cycle on all type of binder mixes has been significant as the Tensile strength reduces very much after first cycle.
2. Average tensile strength of both Binders mixes decreases with number of freeze-thaw cycles, and this decrease.
3. The avg. tensile strength of PMB mixes in all conditions is more than HMA 60/70 mixes' strengths, which shows that PMB mixes have more resistance against moisture damage.
4. The TSR of HMA and PMB are both less than the required, but the PMB has more TSR as compared to their HMA counterpart.
5. The required TSR are 0.7-0.8, so this means that the NHA Class A gradation (coarser Side) is less efficient against Moisture induced Damage.

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Performance of Pavements under Extreme Loading and Temperature Conditions using Polymer Modified Bitumen

Kamran Muzaffar Khan[1] and Ali Asghar[2]

Abstract

Roads and pavements in Pakistan are prone to varying temperature and extreme loading conditions throughout year which results in disintegration, rutting, development of stresses in road. Accumulated strain, Resilient strain, Creep stiffness and Modulus of resilience of asphalt concrete mixes have been studied by changing temperatures at 500KPa loading to understand the pavement behaviour under high temperature. Tests were performed using Universal Testing Machine (UTM 5P) on the mixes prepared in the laboratory. Polymer Modified Bitumen (Pmb) is used in 50% mixes to compare the behaviour of in situ Asphalt Concrete mixes with and without Pmb under same temperature & loading conditions. Binder 60-70 was used in all mixes. Percentage of Pmb used was 3.0%, 3.5%, 4.0%and 4.5% for mixes. The results of laboratory tests on asphalt mixes under the influence of load, environmental and mix variables are studied and presented in this search. Main properties have been tabulated and analysis has been made, which shows certain important results. Tests have been performed at 40°C and 55°C for 60-70 binder and Pmb separately. UTM-5P was used to study the accumulated strain, resilient strain, creep stiffness and modulus of resilience of each sample. Results obtained from the tests depicts that Pmb shows better performance as compared to the ordinary mixes.

Key Words : Polymer modified bitumen, Accumulated strain, Resilient strain, Creep stiffness, Modulus of resilience.

Introduction

There have been different problems associated with the performance of asphalt in the high temperature areas of country. Asphalt having dual nature of ductility and flow, behaves as liquid at extreme temperature and shows ductile nature at low temperature. The main cause of failure of pavements in Pakistan is rutting due to high variations in temperatures, uncontrolled heavy axle loads and limitations of pavement design procedures to meet local conditions. Several ways of failure that an asphalt pavement may experience are rutting, fatigue cracking and low-temperature cracking. Repeated, heavy traffic loads can permanently deform an asphalt pavement causing rutting. Such occurs during the warm climates due to a decrease in asphalt viscosity. Low-temperature cracking occurs at sub freezing temperatures when the viscosity of asphalt is high and is caused by the tensile stress that develops as a result of shrinkage. Whereas, fatigue cracking is caused due to cyclic loading under repeated stress. For a pavement to resist rutting, fatigue cracking and low-temperature cracking, it must perform well under a wide range of environmental conditions. Use of Polymer Modified Bitumen (Pmb) has shown better results in the field of pavement construction as it better addresses the relevant premature failures (Fatigue Cracking, Rutting, and Low Temperature Cracking), hence asphalt can sustain more.

Bitumen Composition and Structure

Bitumen is a complex combination of predominantly hydrocarbon nature. The composition depends on the origin of crude oil and the processes used during bitumen manufacture. The chemical composition of the bitumen is highly complex and its analysis is very difficult. Generally bitumen can be divided into two main chemical groups:-

a) Asphaltenes

b) Malatenes

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Asphalt Modifiers

Elvaloy is "an ethylene glycidyl acrylate (EGA) terpolymer that chemically reacts with asphalt. Roads using Elvaloy have been in use since1991".Witczak et. al, (1995) investigated the behavior of Elvaloy modified asphalt concrete mixes. They used two asphalt binders of different grades and each of them was modified with 0%, 1.5% and 2%, Elvaloy by weight of asphalt binder.

Selection of source of material

Following sources for materials were selected:-

Aggregate from Margalla Hills, Bitumen (60-70) from Attock Oil Refinery and (Pmb) from Attock Oil Refinery and Following test were carried out for aggregates.

a) Abrasion Test b) Soundness test c) ductility of bitumen d) Penetration Test e) Softening point

Bitumen Content Variation

Table 1: Percentage of Binders used

Binder %Used	MOULD COMPACTED	
	60 - 70	Pmb
3.0	9	9
3.5	9	9
4.0	9	9
4.5	9	9

Molding of Sample

Hot mix sample prepared at 155°C was placed in the mould and compacted with 10 lb hammer with 75 blows each side. The sample was ejected after 24 hrs and 2 samples of each binder variation were tested for its stability and flow.

Polymer Modified Bitumen (Pmb)

Polymer modified bitumen, obtained from Attock Refinery Limited, Rawalpindi; having the base AC of penetration grade 60-70 modified with 1.3% Elvaloy has been used. Specifications of Pmb are shown in Table 2.

Table 2: Specifications of Pmb

Sr. No	Test Description	Pmb	Max/Min
1	Flash Point, °C	232	Min
2	Softening Point, °C	60 ± 2	Min
3	Torsional Recovery	12	Min
4	Viscosity @ 165°C, Pa*S	0.75	Min
5	Loss on Heating, wt %	0.6	Max

Gradation of Aggregates

Aggregates have been collected from Margala; near Taxila as per specifications of NHA Class-A Asphaltic Base Course. The aggregates were weighed accordingly in the Highway Lab of UET Taxila on an electric balance, hence forming a total of 7000 in each Mix Sample. NHA Class A aggregate details qualifying the 0.45 Power Curve Criteria are as below in table 3.

Table 3: Aggregate Gradations (NHA Class-A)

S. No.	Sieve Size		Master Band		Trial Blend , Weight = 7000 gms			
	Mm	Inch	Min(% Passing)	Max(% Passing)	Trial Blend, Material Passing (%)	Material Retained (%)		Weight Retained (gms)
1	0	0	-----	-----	----	5	(In Pan)	350
2	0.075	(No.200)	2	8	5	6.5	----------	455
3	0.3	(No.50)	5	12	11.5	18.5	----------	1295
4	2.36	(No.8)	23	35	30	16	----------	1120
5	4.75	(No.4)	35	50	46	19.5	----------	1365
6	9.5	3/8 inch	56	70	65.5	24.5	----------	1715
7	19	3/4 inch	90	100	90	10	----------	700
8	25	1 inch	100	100	100	0	----------	0
Total						**100%**		**7000gms**

The aggregates were thoroughly mixed at a temperature range of about 150°C to 155°C and then Bitumen (Penetration Grade 60-70) confirming to Performance Grades at pavement temperatures up to 64°C at 4.3 % of the total aggregate sample was accordingly mixed in order to prepare a trial blend A. The Rheological Tests on binder were conducted on DSR.

Procedure

1 Uni-Axial Load Strain test

The Uni-Axial Load Strain Test Results may include the following data relevant to the mix's performance under loads, temperatures and frequencies.

Accumulated Strain

Resilient Strain

Creep Stiffness

Resilient Modulus

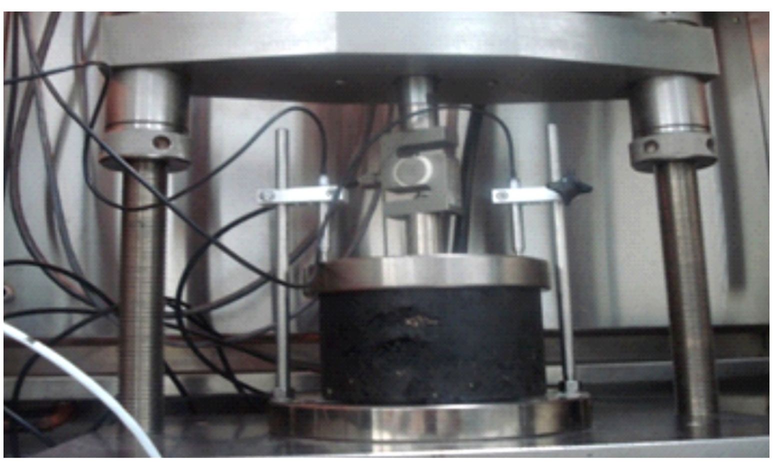

Figure 1: UTM-5P Testing Apparatus

These properties of the mix determine the stability of the mix against temperature, loading and time of loads acting on the pavement.

Test of samples in lab on UTM 5 for repeated Uniaxial Load Test

The samples were first of all measured. Then the temperature of UTM was adjusted and samples were placed inside UTM and settings for the sample were made. After placing and setting the sample, the parameters for sample testing were amended on software as under:

a. Load Applied = 500Kpa
b. Pulse Period = 2000
c. Pulse width = 500
D. Terminal Pulse Count = 1200

Table 4: Based upon above parameters following test results for various Binder Contents of bitumen 60 70 and Pmb were achieved.

Binder Content	Value of	A/U	Temp 40ºC		Temp 55ºC	
			60-70	Pmb	60-70	Pmb
4.5	Accumulated Strain	%	0.2274	0.1331	0.6268	0.2005
	Resilient Strain	%	0.1338	0.1061	0.1690	0.1022
	Creep Stiffness	Mpa	24.6310	48.9870	47.55	68.83
	Resilient Modulus	Mpa	135.7740	465.53	306.02	476.23
4.0	Accumulated Strain	%	0.2013	0.1468	0.5134	0.1892
	Resilient Strain	%	0.1685	0.0621	0.1769	0.1406
	Creep Stiffness	Mpa	49.6700	169.88	51.65	144.62
	Resilient Modulus	Mpa	295.15	806.32	283.34	813.35
3.5	Accumulated Strain	%	0.2464	0.2040	0.6396	0.2046
	Resilient Strain	%	0.1141	0.0452	0.1124	0.1673
	Creep Stiffness	Mpa	39.9620	57.560	49.54	70.96
	Resilient Modulus	Mpa	435.8570	842.32	312.27	496.08
3.0	Accumulated Strain	%	0.1831	0.0872	0.6663	0.1527
	Resilient Strain	%	0.1405	0.1098	0.1745	0.1171
	Creep Stiffness	Mpa	40.21	53.71	47.35	67.74
	Resilient Modulus	Mpa	430.2090	653.904	325.29	467.2700

Graphical representation

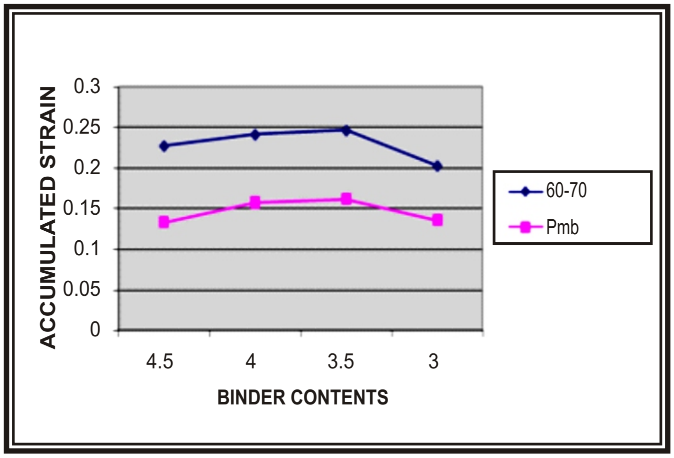

Figure 2(a): (Relationship b/w Accumulated strain & Binder content at 40°C)

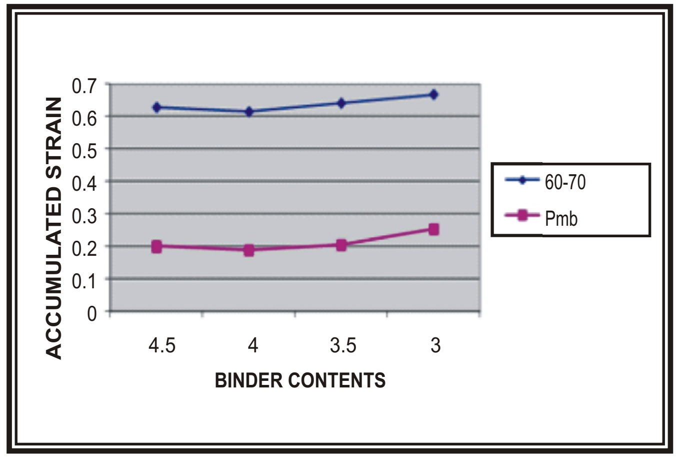

Figure 2(b): (Relationship b/w Accumulated strain & Binder content at 55°C)

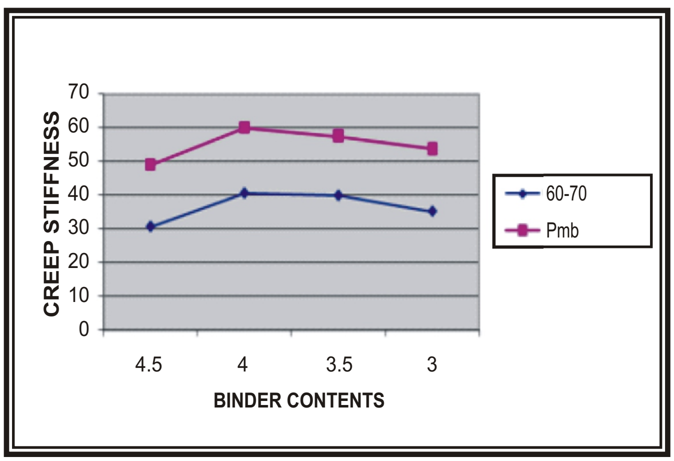

Figure 3(a): (Relationship b/w Creep stiffness & Binder content Binder 40°C)

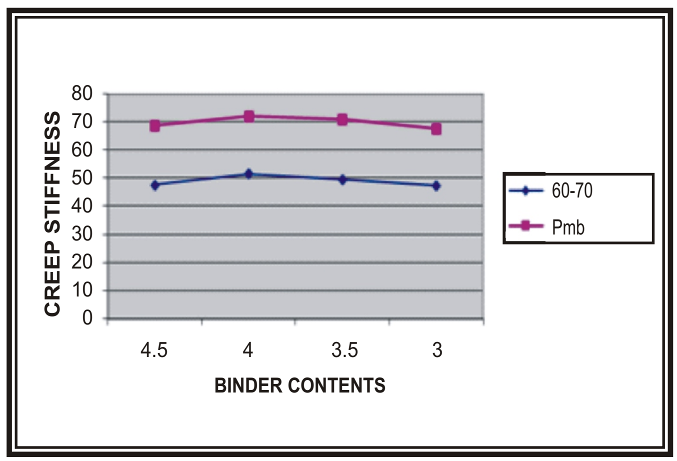

Figure 3b): (Relationship b/w Creep stiffness & Binder content at 55°C)

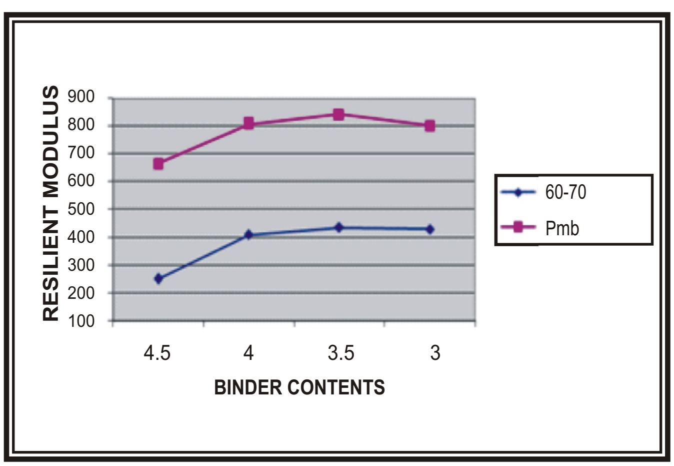

Figure 4(a): (Relationship b/w Modulus of Resilience & Binder content at 40°C)

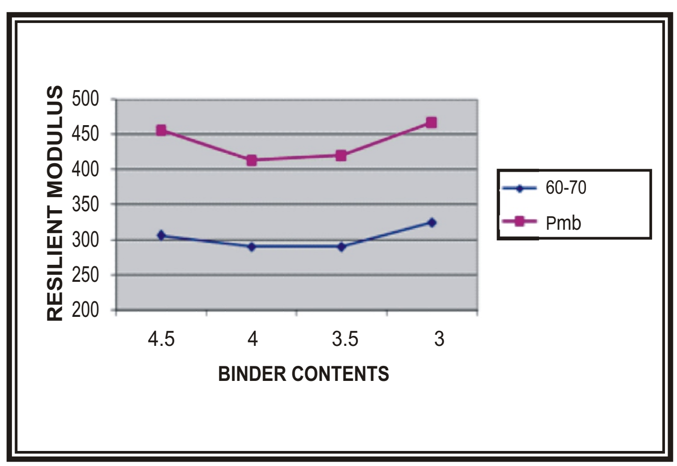

Figure 4(b): (Relationship b/w Modulus of Resilience & Binder content at 55°C)

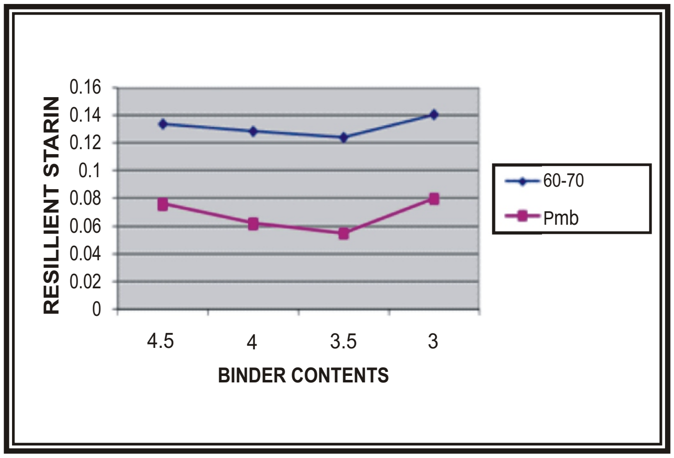

Figure 5(a): (Relationship b/w Resilient strain & Binder content at 40°C)

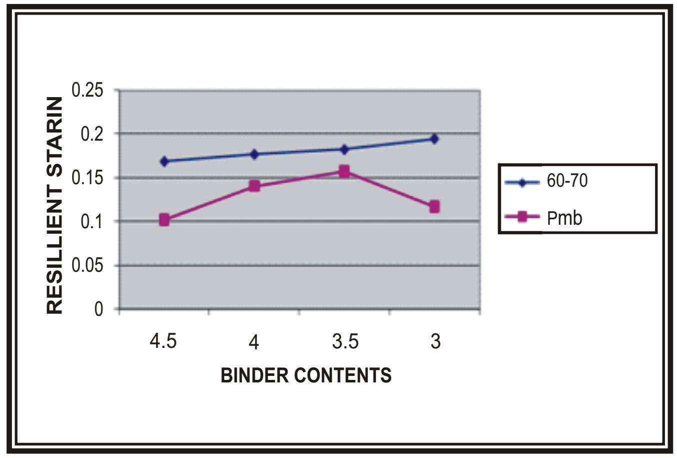

Figure 5(b): (Relationship b/w Resilient strain & Binder content at 55°C)

Results and Analysis

Results obtained from the graphical representation have been given below:-

Accumulated strain at 40°C has been represented in the Figure 2(a). From this graph it is observed that the level of accumulated strain was found to be more in 60-70 binder as compared to the Pmb binder. This accumulated strain is more in both cases at binder contents 3.5 and it decreases as contents amount is increased or decreased from binder content 3.5. This indicates that at constant temperature for a particular binder content 3.5 accumulated strain is maximum for both 60-70 and Pmb samples.

Accumulated strain at 55°Cfor same binder content and 60-70 samples has been represented in Figure 2(b). It is observed that value of accumulated strain has been observed to be less even at 55°C in the Pmb sample. Although rate of change has been approximately same in both samples yet level of accumulated strain is less which is important as a matter of fact about 30% of accumulated strain is reduced by using polymer modified bitumen.

It is observed that there has been change in accumulated strain for both Pmb and 60-70 samples at 40°C and also there is smooth change in the accumulated strain for 55°C. This shows that the behavior of accumulated strain is temperature dependent as temperature increases its rate of change becomes smooth. It was also observed that there has been more change in accumulated strain at both 40°C and 50°C for binder content 3 to 3.5 where as for both 40°C and 50°C there has been smooth change in accumulated strain. This also depicts that there has been certain specific value of binder content which gives designed accumulated strain. On comparison of both graphs it is observed that there has been inverse relation of temperature at 40°C and 55°C. Unquestionably the value of accumulated is less for Pmb.

Creep Stiffness has been presented in the Figure 3(a) at 40°C. The value of creep stiffness is more for Pmb samples as compared to the 60-70 samples. It has been observed that for 40°C the creep stiffness shows rapid rate of change with increase in binder contents from 3 to 3.5 and 3.5 to 4, but from 4 to 4.5 there is relatively smooth change in the value of creep stiffness.

Figure 3(b) shows even smoother rate of change in the creep stiffness at 55°C and there has been smooth change as compared to 40°C. There has been increase in value of creep stiffness with the rise of temperature and same for 60-70 samples. The stiffness was found to be more satisfactory in Pmb even at higher temperature of 55°C as compared to the 60-70 binder at 55°C.

Overall performance of Pmb specimen has been observed to be better both at 40°C and 55°C and in both cases value of Creep stiffness is specific at binder content 4.0.

The behavior of resilient modulus has been presented in the Figure 4(a) and 4(b). At 40°C value of resilient modulus show a continuous changing behavior. At binder content 3 to 3.5 for both Pmb and 60-70 samples value increases and for 3.5 to 4 this value approximately remain uniform. However from 4 to 4.5 again there is change in the resilient modulus. Comparing rate of change in both Pmb and 60-70 at 40°C, it has been observed that for both cases this change has been observed uniform in both.

At 55°C there has been changing behavior of resilient modulus in the Pmb samples from 3 to 4.5, where as an approximately uniform behavior in change has been observed in the 60-70 samples. Comparing values of resilient modulus at 40°C and 55°C, it is observed that value of resilient modulus has decreased with rise in temperature. Hence we conclude that resilient modulus is temperature dependent. Result is that resilient modulus of Pmb is found much higher than 60-70.

Figure 5(a) shows the temperature effects on resilient strain. Resilient strain is much lesser in the Pmb samples than 60-70 at 40°C. Rate of change of resilient strain is more in Pmb samples as compared to 60-70. An abrupt change has been observed in resilient strain from 3 to 4 for 40°C.

At 55°C Resilient Strain Behavior has been shown in Figure 5(b). Value of resilient strain has been recorded to be low for Pmb samples as compared to 60-70 samples. However there is gradual decrease in the value of 60-70 samples for resilient strain with the increase in the binder contents from 3 to 4.5. For Pmb resilient strain value also decreases but from 3 to 3.5 binder content it increases and then from 3.5 to 4.5 decreases. There has been more decrease in the Pmb than 60-70. Resilient strain was also found to be more in Pmb and was recorded to be less in the 60-70.

Conclusions

Accumulated Strain is less in case of Pmb as compared to 60-70 at 40°C and 55°C.

Creep Stiffness is more in Pmb as compared to 60-70 at 40C and 55°C.

Resilient Modulus is higher in Pmb as compared to 60-70 at 40°C and 55°C.

Creep Stiffness is less in Pmb as compared to 60-70 at 40°C and 55°C.

The performance of Pmb was found to be far better than 60-70 at high temperature ranges.

Overall performance parameters are also better in Pmb than 60-70.

As the performance of Pmb was found to be consistent even at a higher temperature, it is concluded that use of Pmb gives better performance as compared to 60-70 samples, so it is recommended to use Pmb even in the temperature prone pavements.

Recommendations

The use of Pmb in high temperature zone gives better performance than 60-70.

The use of Pmb increases the efficiency of pavements by 30%.

In Pakistan environment use of Pmb can result in the better performance of roads and flexible pavements.

As Pmb gives better performance so Usage of Pmb ensures increase in the age of pavement.

Use of Pmb in the southern Punjab will ensure better performance of pavements and roads.

Use of Pmb in the colder areas of country will ensure better performance of pavements and roads.

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Chatter Measurement on Turning Cutting Tool Using Piezoelectric Ceramic Material

Ambreen Tajammal Jarral[1] and Riffat Asim Pasha[2]

Abstract

Machining is a complex process in which many factors affect results. Chatter produced is undesired in all kinds of machining process as it directly affects surface finish and tool life. Tool condition monitoring has gained substantial importance in the industries in past few years, as it significantly influences the economy process and the quality of machining surface finish. Advancement in the technology has led to the develop of various methods to improve surface quality by reducing vibration. For this various kinds of sensors and transducers are introduced in the machining to sustain tool performance These sensors are characterized by their measuring capability and accuracy. Among them, cutting tool material, feed rate, cutting depth, spindle speed and cutting angle are involved. All these deleteriously affect result by producing vibration. Vibration is the most adverse phenomenon which influences surface finish quality, precision of the components machined and life of the cutting tool. Small patch of piezoelectric ceramic material is successfully incorporated into turning cutting and examine to to reduce vibrations in machining processes.

Keywords : Chatter Measurement, Condition Monitoring, Piezoelectric Ceramic

Introduction

Tool life importance cannot be denied, it is influenced by vibration produced during machining. During the machining process both the cutting tool and work piece confront severe vibrations which are known as chatter. It highly causes harm to tool life and desired surface finish. Intense chatter, in the working environment of the tool is usually because of relative movement of the cutting tool and the work piece. In all cutting operations including turning, vibrations are produced due to the deformation of cutting tool edge and work piece. Chatter produced as a result of cutting operation on machines in tools results to decrease tool life. It also results economical loss as well as environmental. During this phenomenon unbearable noise is also produced. Chatter vibrations critically affect tool life, surface finish and production rate in machining processes, regeneration and mode coupling were numbered as the major cause for tool chatter (Davies, et. al. 2000). Chatter and rate of production are directly related to each other, to reduce chatter from metal removing process, small depth of cut is given. Machine-tool vibrations are not in regular in pattern they depend on tool material, cutting position and design and geometry of both the work piece and cutting tool (Merritt, 1965). The piezoelectric sensor and actuator approach provides numerous benefits which includes material, cost, design flexibility and better surface finish (Turner, et al. 1994). Improvement in design flexibility and better production rate can be achieved by eliminating tool vibration error in all kinds of cutting machines. It can also reduce industrial waste which will ultimately save money. By incorporating smart structures into the cutting tool and work piece in various machining process we can easily control chatter (Rashid, 2005) various scientific groups are working on it as it can result to achieve satisfactory results.

Experimentation

The experimental setup is shown in the Figure 1 & 2 vibration meter is attached with cutting tool. Turning cutting tool is designed with high strength steel 4340. Vibration meter is used to analyze the displacement amplitude of vibration produced during turning operation. First of all, tool without piezoelectric patch is used to study. After that piezoelectric patch is attached with turning cutting tool and readings are taken turning tool is fixed; the readings of vibration

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Figure 1 : Experimental setup on BI115

Figure 2 : Experimentation and readings with vibration meter

are calculated from vibration meter. Values are collected and tabulated in Table 1 & 2 for displacement at various feed rate and spindle speed.

Table 1 : Feed rate and displacement values for simple cutting tool

Sr. No.	Feed rate	Displacement
1	0.08	0.11
2	0.1	0.15
3	0.12	0.185
4	0.14	0.21
5	0.16	0.25
6	0.18	0.27
7	0.2	0.87
8	0.22	1.46
9	0.24	2.07
10	0.26	2.15

Table 2 : Spindle speed and displacement values for simple cutting tool

Sr. No.	Spindle speed	Displacement
1	50	0.048
2	70	0.065
3	100	0.073
4	200	0.08
5	300	0.12
6	400	0.154
7	500	0.17
8	700	0.225
9	1000	0.25
10	1200	0.39

Figure 3 & 4 clearly shows that the displacement in vibration amplitude increase as the feed rate and spindle speed is Increased.

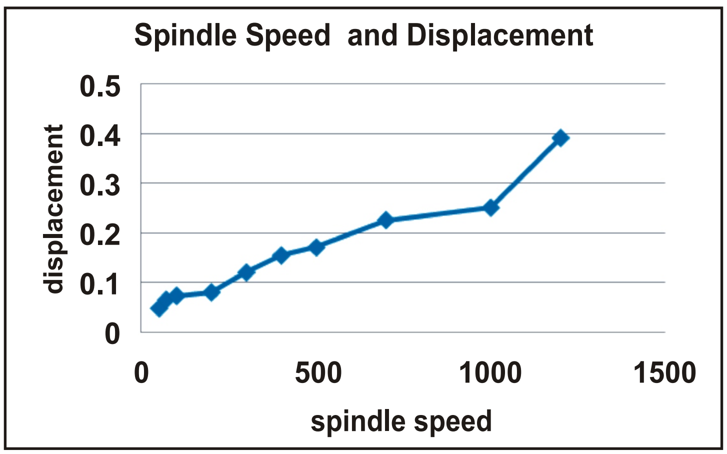

Figure 3 : Displacement and spindle speed of simple cutting tool

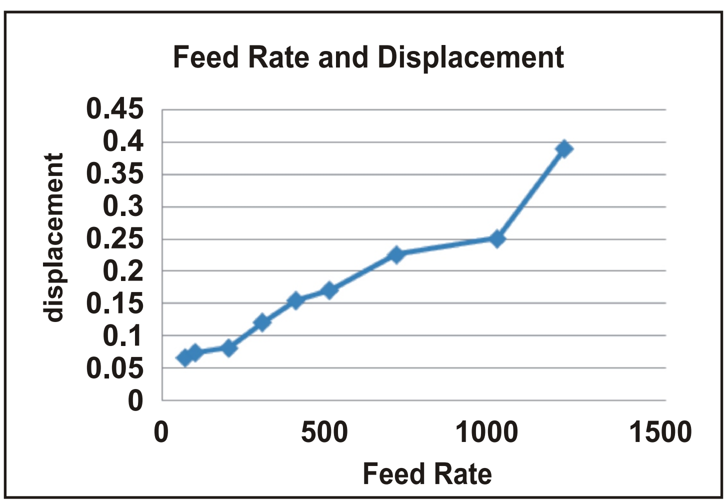

Figure 4 : Feed rate and displacement of simple cutting tool

The experiment is repeated on same feed rate and same spindle speed using pizeoelectric patch. Pizeoelectric patch is connected with cutting tool but insulated through mica sheets steel strips are used to take voltage from voltmeter .This assembly is than fixed on tool post and values collected are indicated in Table 3 & 4.

Table 3 : Feed rate and displacement values for PZT designed cutting tool

Sr. No.	Feed rate	Displacement
1	0.08	0.092
2	0.1	0.13
3	0.12	0.165
4	0.14	0.183
5	0.16	0.197
6	0.18	0.21
7	0.2	0.45
8	0.22	0.73
9	0.24	1.04
10	0.26	1.53

Table 4 : Spindle speed and displacement values for PZT designed cutting tool

Sr. No.	Spindle speed	Displacement
1	50	0.024
2	70	0.037
3	100	0.045
4	200	0.0557
5	300	0.062
6	400	0.069
7	500	0.072
8	700	0.074
9	1000	0.08
10	1200	0.1

The trend shown in Figure 5 & 6 clearly showing that a decrease in displacement occurs if we use piezoelectric material 5A ring of 1 inch, displacement increases as feed rate and spindle speed increases.

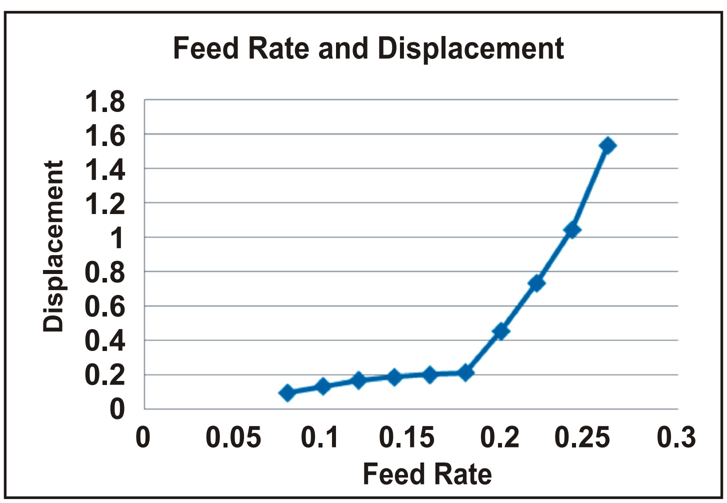

Figure 5 : Displacement and feed rate with pzt

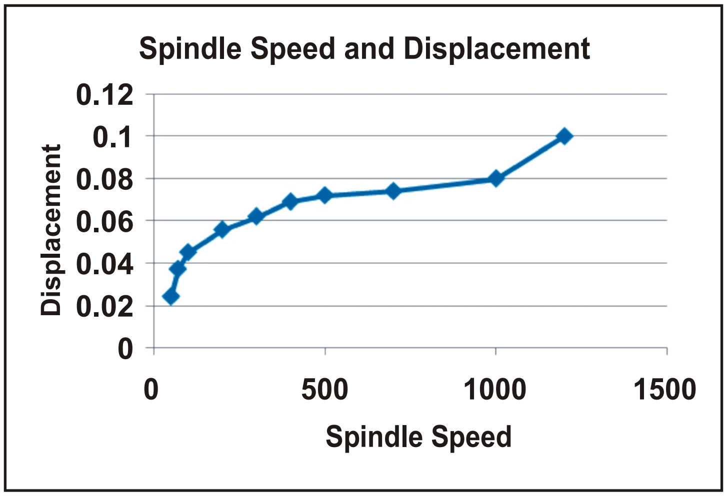

Figure 6 : Displacement and spindle speed with pzt

The comparison between both turning cutting tool vibrations at feed rate and spindle speed without and with piezo material are indicated in Figure 7 and comparative less displacement has been found in case of piezo attachment.

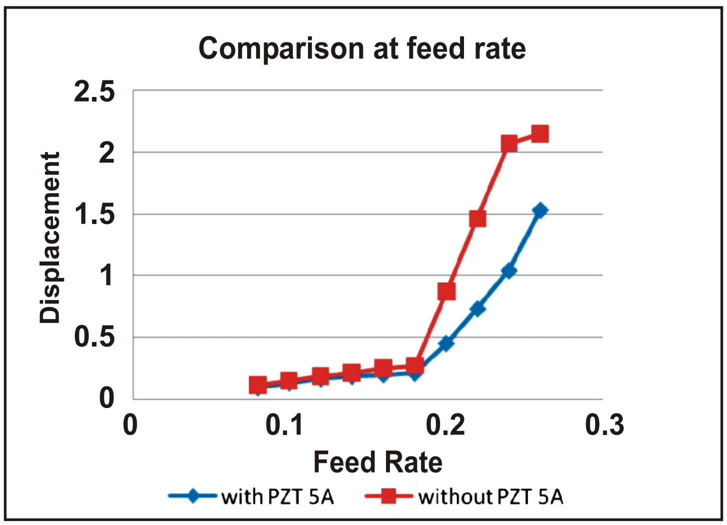

Figure 7 : Comparison between with and without pzt cutting tool at feed rate

The Figure 8 clearly depicts that displacement amplitude is lower in case of pzt material. The decrease in displacement case of spindle speed is remarkable.

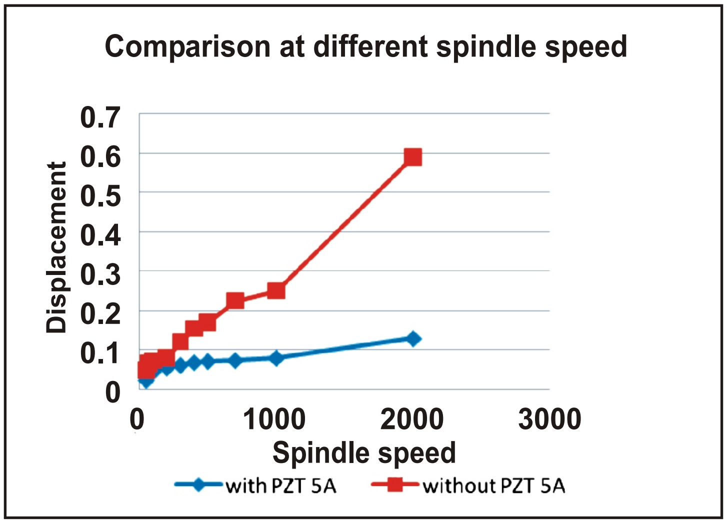

Figure 8 : Comparison between with and without pzt cutting tool at various spindle speed

Conclusion

The feed rate and spindle speed are directly proportional to the amplitude of vibration. Both factors play an important role in producing vibration as their values are increased but vibration produced in the turning tool without piezoelectric patch is slightly higher as compare to the tool with which piezoelectric patch is attached. Amplitude of vibration is lower in case where PZT material is used. It is therefore concluded that by using the piezo patches the displacement and amplitude can be controlled in cutting tools. Moreover the voltage generated by piezo materials can further be utilized.

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