Engineering News

A QUARTERLY JOURNAL OF THE PAKISTAN ENGINEERING CONGRESS

Feb. 1999

PAKISTAN ENGINEERING CONGRESS
THE EXECUTIVE COUNCIL FOR THE 68TH SESSION

COVER PHOTO

A VIEW OF JINNAH FLY - OVER (GURUMANGAT) LAHORE

41st YEAR OF PUBLICATION

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EDITORIAL

ROAD SAFETY

Pakistan ranks eleventh in fatality rate out of twenty eight countries significantly known for road accidents. This was disclosed in a seminar on road safety held in December, 1998 at Lahore under the auspices of NHA, Punjab C&W Department, NTRC and consultants Finnroad Oy and Adcon; The other startling facts brought about in the seminar were: About 7000 people die in Pakistan every year in road accidents. Total annual cost of accidents is estimated to be about a billion in US Dollars. Even material costs exceed 200 million US dollars per year. Those who die and suffer injuries belong to the most active and educated group of citizens. The number of deaths due to road accidents is climbing rapidly and is higher than the number of deaths due to many of the biological causes.

Studies have shown that there are three basic factors in road accidents viz: the road and its condition, the vehicle and its condition and the road user. As long as these factors function smoothly, the accidents can be minimized. The accidents occur only when one or more of these factors fail to come into play. According to the findings of the researchers, 5% accidents at the most can be attributed to the road condition, which appears to be quite an insignificant figure compared with other causes. Statistics also show that only in 3.5% of the accidents, cause could be the faulty function of the vehicle. It thus leads to the conclusion that major role in accidents is of the road user. The road user involved in the accident may be the driver of one or more vehicles, the passengers, the pedestrians or the stray animals on the roads. The greatest scope for minimizing the road accidents, therefore, lies in making the road user to realize the responsibility for his personal safety and the safety of others.

While the engineering measures relating to design of roads and the maintenance of vehicles are also necessary for improving the road safety, enforcement of traffic rules and education of road users is the most vital to make the travelling safer. For this purpose, a coordinated effort by all the concerned parties is required. A National Road Safety Council, therefore, need to be established which would regulate their activities, promote public discussion and development.

NEWS

WELCOME TO NEW MEMBERS

The Executive Council of the Pakistan Engineering Congress approved membership of the following new members into the Congress fold. The Engineering News congratulates all of them and welcomes to PEC.

Members admitted on 30-11-1998

1. Lt.Col.(R) Sarwat Ahmad,
   435-Street No. 50, G-9/1,
   Islamabad.
2. Mr. Sajid Tanvir Ahmad Khan,
   SDO, Education Building Division,
   Gujranwala.
3. Mr. Muhammad Tahir Jamil,
   18-Akhwan St. Wafaqi Colony,
   Iqbal Town, Lahore.
4. Mr. Mujahid Aziz,
   R.O.Govt.Engg. Academy Lahore.
5. Syed Waheed ul Hasan,
   136-A, Faisal Town, Lahore.
6. Mr. Nazir Ahmad Raja,
   DD, I&P, Govt. of the Punjab,
   Lahore.
7. Dr.Muhammad Anwar,
   S.O.(Opt) I&P Sectt., Lahore.
8. Mr. Muhammad Navid Iqbal,
   Retd. Lt Col.
   32/4, Sarwar Road, Lahore Cantt.
9. Mr. Faiz-ul-Hasan,
   Pr. Engineer, ACE, Ltd.,
   M.M. Alam Road, Gulberg-III,
   Lahore.
10. Dr.Muhammad Ashiq,
    Asstt. Professor, CEWRE,UET,
    Lahore.
11. Mr. Naveed Alam,
    249- Jinnah Colony, Faisalabad.
12. Mr. Muhammad Sohaib Khan,
    54-Q,.Block, D.G. Khan.
13. Mr. Muhammad Munir,
    House No.45, Allama Iqbal,
    St. No.139, Ichhra, Lahore
14. Mr. Kashif Butt,
    House No. BV-56/3
    Eidgah, Jhulum.
15. Mr javed Iqbal,
    Khokhar Manzil,
    dilshad St.No.18. Shailmar Town,
    Lahore.
16. Mr. Amjad Hussain Fraz,
    117- Ali Block, New Garden.
    Town, Lahore.
17. Mr. Fayyaz Qayyum Shahzad,
    E-480/5-A, Block-C, link.
    Road No.2, Nishat Colony,
    Lahore Cantt.
18. Mr. Asad- ullah Qazi,
    101-Saleem Park Sargodha.
19. Mr. Muhammad Yaqoob,
    141-A, Faisal Town,Lahore.
20. Mr. Ahmed Mufeez Sadiq,
    145-N, Model Town Extension,
    Lahore.
21. Mr. Awais Jamil Chaudry,
    456-B, Canal View Housing,
    Society, Lahore.
22. Mr. Abbas Ali Qaisar
    House No.3, St.No.3,
    Siddiqia Colony, Badami Bagh,
    Lahore.
23. Malik Zahid Bashir,
    193-A, Ahmad Block,
    New Garden Town, Lahore.
24. Mr. Shaukat Ali Ayub,
    189-E/I, Phase-I,
    Hayatabad, Peshawar.
25. Syed Muhammad Ali
    194-E/I, St. No. 8,
    Phase-I, Hayatabad.
    Peshawar.
26. Mian Muhammad Ali,
    194-E/I St.No.8, Phase-I,
    Hayatabad, Peshawar.

NEWS IN PICTURES

Engr. Ch. Abdus Salam delivering a lecture on "Private Finòncing of Roads and Bridges _ Boot opportunities in Pakistan?

Engr. Barkat Ali Luna delivering a lecture on "Effects of Upstream Storages on the Present Eco-System in Sindh Area Downstream Kotri Barrage.

OBITUARIES

May their souls rest in peace

1. Engr. Muhammad Rafiq Shad, Retd; Chief Engineer PHED and former Secretary, Pakistan Engineering Congress.
2. Engr. S.M. Safdar, Retd. Chief Engineer, I&P Department.
3. Engr. Mian Saleem Hassan, Retd. General Manager, WAPDA.
4. Engr. Abdur Rashid Sheikh, Retd. Superintending Engineer, I&P Department.
5. Engr. Nazar Muhammad Malik, Executive Engineer, Irrigation and Power Department.
6. Engr. Ch. Mushtaq Ahmad Khan, Brother of Engr. Ch. Aftab Ahmad Khan. Retd. Chief Engineer, WAPDA.
7. Wife of Engr. Abdul Hameed Arif, Retd, Chief Engineer, Irrigation and Power Department.

Engr. Rafiq Shad (Late)

Former Secretary of Pakistan Engineering Congress Engineer Muhammad Rafiq Shad, breathed his last on Nov. 6 1998 in Lahore.

The President and Members of Pakistan Engineering Congress have expressed deep sorrow and grief on the sad demise of its former Secretary.

Mr. Rafiq Shad, who was Chief Engineer (Retd.) of Public Health Engineering, as an engineer of exceptional qualities and a great organiser.

He played a pivotal role in strengthening the PEC. He is admired and remembered among his colleagues for his qualities of leadership. He served the engineering community with dedication and devotion throughout his life.

The PEC President, Engr. S.N.H. Mashhadi placed a floral wreath on the grave of deceased Rafiq Shad on behalf of all members of Pakistan Engineering Congress.

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Vol. LII No. 339 Lahore, Sha'aban 23, 1419
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Pakistan
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uild water storage dams
Engr. Barkat Ali Luna said that due to siltation, enough storage water will not be available for maturing of Rabi and sowing of Kharif crops. As such Sindh farmers will be
recently carried
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WEDNESDAY, OCTOBER 28, 199
Concern over unemploy
LAHORE (PR)–Pakistan Engineering Congress (PEC) has expressed great concern on the rising unemployment among the young engineers and has urged upon the Government to promulgate an ordinance requiring all private industrial establishments and firms of engineering consultants and
contractors to emp
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PEC urges govt to a engineers as project
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INTERNATIONAL THE NEWS
Saturday December 12, 1998
NDC chief stresses need to build new dams on Indus
Chairman National Development Consultants (NDC), Engr Lana said that new water storage dams are urgently required to on the Indus river, due to rapid depletion of reservoirs at Mangla, and Chashma projects.
ressing a lecture meeting organized by Pakistan Engineers Congress Friday, he said that due to siltation, enough storage water will
PEC for appointment of native engineers as PDs
LAHORE: Pakistan Engineering Congress (PEC) Wednesday urged the Government to appoint engineers as Project Directors for the execution of development schemes in the country to enable them fulfil
status in the society.
The PEC members pressed surprise at the ne ternational tenders are b for the privatization o will be catastrophic if ment of water supply goes into the hands tractors and an insi the local enginee
Daily
واسا کو غیر ملکی کمپنیوں کے حوالے کرنا تباہی کے مترادف ہو گا

LETTER

Subject: SOIL MAPPING OF FAISALABAD

After having read the article titled "Soil Mapping of Faisalabad Metropolitan City" By Engr. M. Naeem Chughtai which appeared in July- Sept. 1998 issue of Engg. News, I could not resist the temptation of expressing my candid and frank opinion about the substance and the standard of the paper (or abstract) published.

I must admit that it is novel idea to map or even attempt to map soil properties for a metropolitan city spread over an area of about 30,000 acres. And it is naive to hope that Atterberg limits, moisture content, and bulk density of few dozen sites can be used for the preparation of a soil map of the whole city. Instead of providing information about classification and moisture content at various depths which is more pertinent and more useful to indicate the soil behaviour under load, the writer has collected irrelevant data of Atterberg limits in the false hope that it will be useful to predict the bearing capcaity. This is over simplification of science of soil mechanics to the extent of being ridiculous.

The table 'A' supposed to give index properties only includes Atterberg limits (liquid limit, plastic limit and plasticity index) moisture content and bulk density which properties only partially indicate plastic behaviour of soils. How these properties have been categorized as index properties could surprise by engineer? The statement of the author that these properties rank amongst the most important factors in determine the bearing pressure of soils is utterly wrong. If it was that simple, why would any soil scientist determine the particle size, compressive strength of confined & unconfined sample, angle of internal friction and resistance to penetration etc. etc. to estimate the bearing capacity? As if this was not enough, the author has condensed the data in table 'A' without any reference to the depth although everyone knows that these properties vary a great deal with the depth of sample.

I do not understand the terminology of common range in table 'A' either. Data compilation and presentation always requires statistical approach where frequency of ranges of some properties is tabulated for further statistical analysis. But then one should ask oneself a loud question; what use is it to the engineers if they know the variation in Atterberg limits or the moisture content in soils of an area?

I have been shocked to see from table 'A' that moisture content variation has been reported as 1% to 29%. The figure of 1% moisture content is absolutely wrong as it is close to ovendry condition which can never be found in practice in the field. All engineers know that Atterberg limits depend on soil classification and if one knows the soil constituents of a particular spot and variation in moisture content with the depth, one can make an intelligent guess of its bearing capacity but even then soil exploration of a particular site cannot always be dispensed with.

I am equally surprised to see table 'C' containing recommendation for dimensions and type of foundation to be used. It shows complete ignorance and lack of experience. Firstly, lightly loaded residential buildings is an ambiguous term in engineering termino-logy.

One should have categorized the buildings in term of load per ft of walls or load per column. Secondly, how can any engineer, leave alone a research engineer, recommend the adoption of strip footing for lightly loaded walls? The term strip footing is also a new innovation because building foundations either comprise of brick footings or strip foundation or raft foundation excluding isolated foundation of columns. The recommended width & depth of 3ft for the so called lightly loaded single storey buildings carrying say 1.25 tons load per foot should not have base concrete wider than 2" ft and depth exceeding 2ft. even where bearing capacity is 0.5 ton per sft. But then 0.5 ton bearing capacity is indicative of very poor soil including land filled sites. Even in waterlogged area, instead of digging foundations, one should attempt to build up a compacted earthen platform for the entire covered area of a building. The brick footings can then be placed on this platform and protected by additional bank extending beyond the face of walls. This would necessarily require higher plinth and additional filling plus a dwarf wall and apron to serve as plinth protection. The cost per sft will be much less than strip or raft foundation being recommended these days by the BRS. Foundation of this type have been tried in India successfully; the BRS, Lahore also used such a foundations for a 4 room workshop (13"' X 40') at 2 Lake Road in 1963. The basic idea is to keep building foundation much above the water table because digging deep would require wider foundation & perhaps a RC strip or a Raft which is very expensive.

By innovating new terms like strip footings, the research laboratory is trying to confound the site engineers and attempting to take refuge of the term to include strip foundation in the event of any failure. The recommendations for other foundations in table 'C' are equally prepostrous and without any basis. Engineers must decide on the type of foundation depending on its size, shape, load, soil, properties, water table and behaviour of adjoining structures. Research has to provide us maximum possible data about soil properties while the choice of foundation to be adopted has been left to field engineers. There is no research method or tool which can be applied by a laboratory to indicate to a research scientists the type of foundation to be adopted at a given situation. Therefore, the BRS is advised to desist from recommending any particular foundation for a given site.

The writer has not indicated the number of sites tested but knowing the limitation of the Building Research Station, I doubt if more than 2 dozen sites would have tested in the city of Faisalabad during two years from 1996-97. This much data would be considered to be insufficient even for a single site of about 10 acres. I hope the expected publication of BRS depicting the type of soil deposits in Faisalabad is not based on investigation of a few scores of sites. The only way to map a city is to divide the city in a convenient grid say 100mx100m and exploring soil at grid junctions and drawing the sections. The exploration should give soil classification, moisture content, Atterberg limits, unconfined & confined compression strength and angle of internal friction of every 1' depth upto water table. But this is a Herculian task and not even worth attempting. The Research Station should seriously consider whether such an exercise will be worth- while, I personally think that it will be of no practical or heoretical value and will be sheer waste of human and material resources.

I hope that a technocrat administrator in the C&W Department will stop this wastages and direct the research to more serious & productive problems because the research has been neglected for too long in the engineering field.

ASHFAQ HASAN
ENGINEER EMERITUS
HON.MEMBER (TECHNICAL), P&D
FORMER PRESIDENT PAK ENGG. CONGRESS.

REPORT

BUSINESS MANAGER'S REPORT ON VISIT TO JINNAH (GURUMANGAT) FLY - OVER

It was a fine bright, sunny and pleasant morning of the first day of December, 1998 when delegation of about 30 Engineers from Pakistan Engineering Congress, visited the under construction Gurumangat Fly-over between the Gulberg area and the Cavalry Extension Area of Lahore. This Fly-over had been demanded by the inhabitants of Cantonment and Defence areas since long as they had to face holdups quite frequently and to wait for long periods to cross the Railway line due to heavy train traffic. The worthy Chief Minister of the Punjab listened to their voice and realizing their problems, ordered to construct this Fly-over. Accordingly the scheme was prepared by TEPA and the foundation stone was laid by the Chief Minister in March 1998 with a completion period of one year.

This visit was arranged by the Pakistan Engineering Congress in connection with Mid-term Visit to Engineering Projects under construction. Originally the mid-term visit was planned for Ghazi Bharotha Hydel Power Project but due to unavoidable circumstances, the concerned authorities requested to postpone the visit till the situation improves.

The delegation comprised of both in-serivce and retired Engineers from different Government Departments (Communication & Works, Irrigation & Power WAPDA, Railways etc.) as well as from Private sector (Consultants & Industry etc.) It included retired Secretaries to the Government, General Managers, Chief Engineers down to the level of junior Engineers from various disciplines , which showed their interest in the project.

All the participants were requested to assemble at PEC Headquarters by 10.30 A.M. on that day. The response from the participants was not only very encouraging but they also reached the Head Quarters by 10.30 A.M. The Business Manager led them to the site of works where the commands of the delegation was taken over by the Secretary PEC in the absence of the President of Pakistan Engineering Congress who had to leave for Islamabad the same morning to participate in some important meeting.

The delegation was accorded a very warm welcome jointly by Brig (Retd.) Muhammad Suleman (Chief Engineer, TEPA), MR. Ziarat Gull (Project Manager Gammons Pakistan Limited), Mr. Ahmed Jamil Hassan (General Manager, Structures, NESPAK), Ch. Muhammad Akram (Resident Engineer, NESPAK) and their team at the site. It will not be out of place to mention here that this visit was made possible mainly due to the very kind cooperation and keen interest taken by the Chief Engineer, TEPA, who despite his extremely hectic schedule, agreed to accommodate the PEC delegation for their proposed visit.

The delegates were welcomed at the bridge deck where the briefing had been arranged. Mr. Ahmed Jamil Hassan of NESPAK apprised the participants of the salient features and other aspects of the project. The briefing was followed by question-answer session which was rather more informative particularly for the younger participants/ visitors. During the discussion, Mr. Hassan informed that the project was undertaken by Gammons Pakistan Limited on turn-key basis, which included both phases i.e. the design and the construction of the Fly-over. The design part was, however, sublet to M/S. NESPAK who have designed a very economical structure. He further informed that it was a two lane dual carriageway fly-over with a total width of 19.98 meters. In the two lanes of the roadway, as a matter of practice, usually 4-girders are required on each side supporting the deck slab whereas M/S. NESPAK have achieved a substantial saving by providing 3-girders on each side. The girders are precast and prestressed with a longitudinal span of 30 meters. The transverse system supporting these girders has been designed on the concept of balanced cantilever, whereas the central supporting column is resting on the pile cap over a group of piles. He added that the project was being completed solely through indigenous expertise. The most interesting and challenging aspect of the project was that it was awarded to M/S. Gammons with a time limit of 12 months (March 1998 to Feb. 1999) but the worthy Chief Minister, Punjab, in order to ensure an earlier completion, announced a 10% bonus if the project was completed within 9 Months (by 25th December, 1998). Keeping in view this target, the whole of the team was working day and night to achieve this uphill and onerous task.

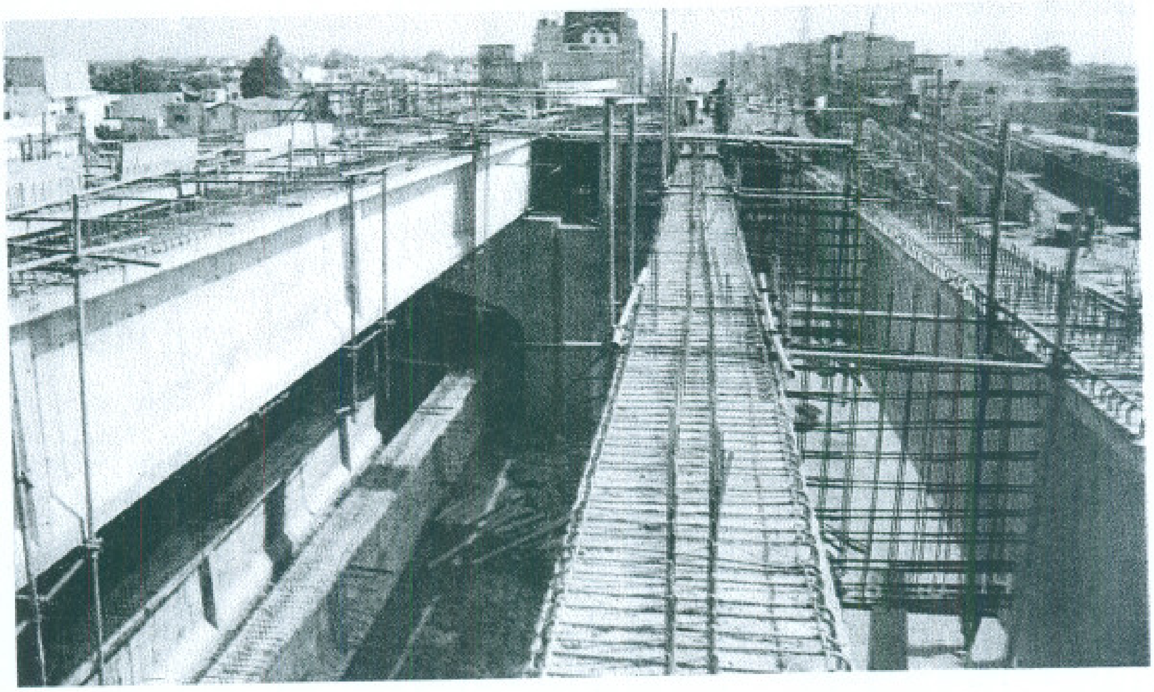
After the question-answer session was over, the participants were served with snacks and hot tea arranged by M/S. Gammons Pakistan Limited. Later on the participants were invited to have a round of the project. Once again, the hosts alongwith their team accompanied the delegates and showed them various parts of the project and attended the queries at the spot, thus bringing the visit to a logical and beneficial culmination.

At the end, Engineer Syed Mansoob Ali Zaidi, Secretary Pakistan Engineering Congress, while thanking the organizers, especially Brig. (Retd) Muhammad Suleman, and Mr. Ziarat Gull, for making excellent arrangements during the visit, also complemented the designers who has won the appreciation from a galaxy of Engineers for their good job.

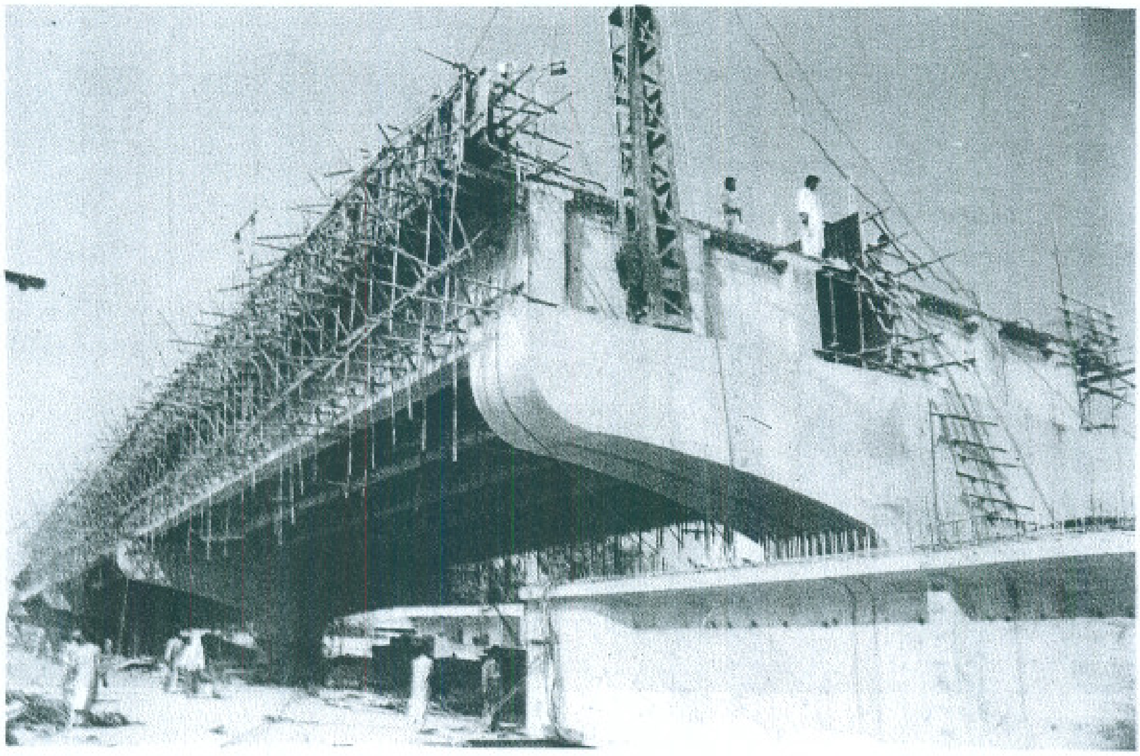
The visit ended at 14.30 hours and the participants dispersed to follow their own programmes. A set of photographs depicting different activities of the project is exhibited in appendix-1.

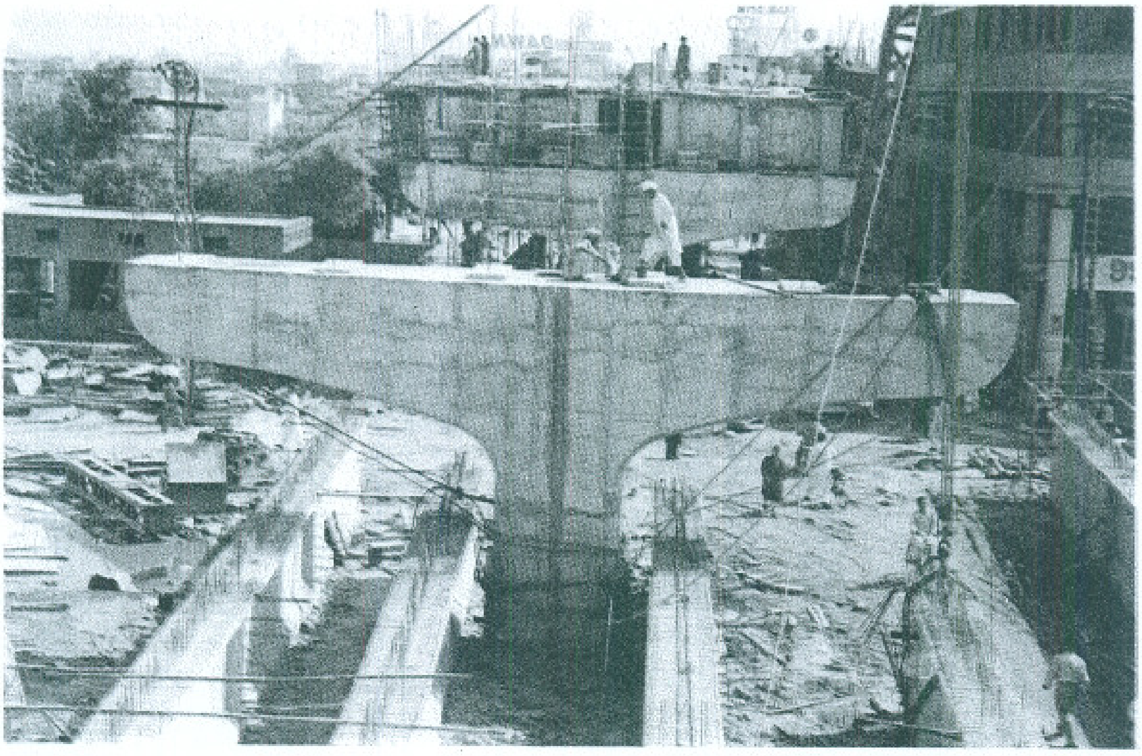
The length of the fly- over is 979 meters with a gradient of 3.5%.

Jinnah Fly Over under Construction



Jinnah Fly Over under Construction





LECTURE

PRIVATE FINANCING OF ROADS AND BRIDGES--BOOT OPPORTUNITIES IN PAKISTAN

*ENGR. CH. ABDUS SALAM**

There is growing demand for improved infrastructure all over the world. The additional funds required for this purpose have outstripped the financial resources of various countries (barring few exceptions like Switzerland, Brunei, and some oil rich states in the Middle East). In the developed countries of the world (per capita GDP in excess of U.S dollars 2000) there is increasing trend to spend more on hospitals, schools, housing and other welfare projects leaving inadequate funds to improve or augment the infrastructure which is under strain; in U.K. during 1970 - 1991, passenger cars drove 116 % more miles while only an extra nine percent of motorways and main roads were built. In U.S.A, vehicle miles almost doubled, while length of urban roads increased by just four percent.

In the newly industrialized countries (Malaysia, Thailand, Hongkong, South Africa, South Korea, Taiwan, Singapore, Mexico,Etc) the enormous migration of people into cities has led to urban explosion and greater demand on urban infrastructure. In Bangkok, Population has grown from six million in 1987 to nine million in 1991 and is likely to hit 12 millions by 1998. Economic growth means more people can afford cars. In Bangkok, every day 500 new vehicles take to the roads. countries in Indo Pak sub continent, African and some South American countries (commonly categorized as Third World countries), infrastructure is limited. These countries are badly indebted and much of their national budget is consumed for debt servicing leaving fewer budgetary resources of their own to finance the improvement or augmentation of their infrastructure.

In the 1970's, it became clear that practically all governments globally has major fiscal inadequacies in funding their infrastructure projects (which includes power stations, water supply and distribution projects, sewerage projects, roads, bridges and airports etc). There is a limit to financing through levying heavy taxes which is not an acceptable solution and would in turn smother the general development of the country.

Private financing of road projects does provide an answer and this lecture would refer to privately financed road and bridge projects in Pakistan. This concept requires a willing government, a viable project and entrepreneurs willing to take the risk of embarking on such projects. The technique commonly referred as **BOOT** (build own operate transfer) starts with the grant of concession (generally by the government of the country) which empowers the sponsor or concessionaire (commonly a financial group or a consortium of contractors) to build operate and profit from the road or bridge created by the concession. On the expiry of the concession (generally 10 to 30 years terms), the completed road/bridge is transferred at no cost to the government. The community benefits by

*Retd Secretary, C&W Department, Punjab.

having the use of facility without additional taxation to build the new facility. The following benefits accrue to the public/government.

(a) Relief of financial burden on government.

(b) Relief of administrative load and reduction in the size of bureaucracy.

(c) Better service to the public and increased impetus to development programme.

(d) Government can divert greater attention to other welfare sectors like health, education, housing etc.

(e) It is recognized that private sector enterprises are more efficient than public ones. The community thus benefits from the expertise and better handling by private organizations.

For the success of **BOOT** projects, it needs to be stressed that viable projects must be identified; every project is not amenable to **BOOT** technique. The favourable points are listed as under:

(a) A willing government to help push the project.

(b) Reliable/realistic traffic projections with impressive traffic growth rate.

(c) An expertly designed project with modest cost estimate employing experienced contractors consultants.

(d) A fixed price contract to avoid cost over run.

(e) Shorter construction period so that toll revenue is realized early.

(f) A champion of the project providing drive and push to the realization of the project Ferdinand de Lesseps for Suez canal, Lord Kadoorrie for Hong Kong cross harbour tunnel project etc.

(g) Low interest rates.

As against above, the factors which lead to poor results for **BOOT** projects are long protracted construction period, unreliable traffic projections leading to low toll receipts (due to availability of alternative untolled routes, insufficient connecting links) no champion to control the project, high interest rates, cost over runs due to poor engineering. A notable example in this context is 6000 km long toll road project in Mexico. Based on hasty traffic and revenue projections prepared by a Mexican Government department, the project was pushed without proper screening by the nationalized banks providing loans. A short concession period was proposed and to get back the return in this period very high toll rates were proposed forcing road users to follow existing old untolled roads despite longer travelling time. The expected traffic thus failed to materialize; this coupled with cost over runs of more than 50 percent forced Mexican government to intervene. The concession period was extended from 15 years to 30 years and heavy vehicles were banned from using old road network.

This lecture illustrated with slides would show a list of known **BOOT** projects giving an overview in respect of various countries in the world. Some of these projects, as would be seen being proposed/negotiated. This list would illustrate the acceptability of **BOOT** mode of handling infrastructure projects from inception to completion and operating the facility till the close of concession period. Some case studies of major **BOOT** projects would also be discussed in detail like Hong Kong cross harbour tunnel, Malaysian North South Highway and Dartford bridge U.K. highlighting the main features and the way unqualified success was achieved in case of these projects recently completed. After illustrating the organizational set up for

BOOT projects financing of such projects through loans from commercial banks, subsidy from client government and raising equity from contractor and public shares would be discussed.

For successful handling of **BOOT** projects, a considerable spade work needs to be done. This includes financial appraisal referring to life cycle costing and financial modelling to cover 'what if' situations. Financial modelling on predicted traffic is done, this modelling is also done if the traffic actually using the roads or bridge facility is 10 percent more or 10 percent less than the estimated predicted traffic. Modelling is also done with varying interest rates of lenders during the completion of the **BOOT** project.

Like all human endeavours the **BOOT** projects can face a daunting array of risks, which can be mitigated through suitable advance planning. They can be briefly listed as below:

(a) Fluctuations in foreign exchange and interest rate.

This is particularly relevant to developing countries where equipment and some materials have to be imported.

(b)Changes in market rates for constructional materials.

(c)Revenue receipts less than predicted.

For road and bridge projects, this can be due to over optimistic traffic forecasts, wrong assumptions, availability of alternative untolled road/ bridges or insufficient connecting road network.

(d)Cost Over run

This happens when construction cost exceeds original estimate, either due to inflation or too many design changes.

(e)Political risk

These are of two types viz. sovereign risk and instability risks. The former can be due to a change in government composition, a military coup, or change of legislation. In developing countries volatility of promises given under strong personal influence. Instability risk can range from labour unrest to political decision to stop work (due to war or hostility between countries or major political uprising.)

(f) Technical risks

Construction difficulties due to unforeseen soil conditions and equipment break down. Completion delays due to poor coordination or late design changes. Operation risks due to toll collection facilities hindered by equipment break down or discovery of defects in the completed project.

The **BOOT** technique to create new roads and bridges is a 'cost less' scenario for financially constricted government like Pakistan where traditional sources of funding road and bridge projects are drying up. In our country, in numerous cases need for byepasses for towns dualization of heavily trafficked roads elevated carriage ways in urban areas and above all major river bridges at selected locations cannot be over emphasized. In order to meet the urgent needs of new roads and bridges, we need to find new means. Recourse to **BOOT** technique is answer to the present difficult situation and like many other countries of the world, we should exploit this mode of financing. This would incidentally, be a great step towards self reliance and reduce dependency on foreign loans.

EFFECTS OF UPSTREAM STORAGES ON THE PRESENT ECO-SYSTEM IN SINDH AREA DOWNSTREAM KOTRI BARRAGE

Engr. Barkat Ali Luna*

1.0 INTRODUCTION

The ecological disciplines which can possibly be affected in Sindh area downstream of Kotri Barrage by construction of upstream storages on the Indus River are the following:

- Sea Water Intrusion
- Mangrove Forests
- Fisheries
- Riverine Forests
- Riverine Irrigation
- Domestic Water Supply
- Irrigation Supplies to the Sindh Area

National Development Consultants (NDC) have recently carried out a detailed study to evaluate possible impact on the above disciplines. This study is extremely important in view of the apprehensions expressed by a section of the people in Sindh who oppose the construction of Kalabagh Dam on the Indus River. The study results have revealed that their apprehensions are the result of lack of knowledge and unrealistic assumptions. It has highlighted the urgent need for informing the people-at-large about the facts and non-facts as well as real issues and non-issues about Kalabagh Dam or any other dam on the Upper Indus River.

2.0 STUDY AREA

The study area comprises mainly the Riverine area below Kotri, and the Indus Active Delta which is an integral part of the Indus Tidal Delta.

2.1 Indus Tidal Delta

The Southern region of Sindh that forms the Tidal Delta comprises all the lands that are inundated by high tides in the Arabian Sea and are exposed at low tide. This area, of about 1,525,930 acres, stretches from Karachi in the West to the Rann of Kutch along the Indian Border. Its width varies between five and twenty-five miles. It contains mangroves swamps and thickets of tamarisk, which lie along the coast and tidal estuaries.

2.2 Indus Active Delta

From the point where the Indus bifurcates to the point where it joins the sea the area is known as Active Delta. It is a part of Indus Riverine area, and covers of about 294,244 acres. This Active Delta is now the only part of the Tidal Delta that is still receiving regular supplies of fresh water and silt from the Indus River through Kotri Barrage. The Active Delta has the soil of almost recent formation due to accretion in the River bed. The area is low lying and,

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well drained and is of better composition.

2.3 Indus Riverine Area (Active Flood Plain)

An active flood plain is found on both sides of the Indus River from Kotri upto the Active Delta. Bunds have been constructed to confine flood flows within the active flood plain. It covers about 336,540 acres and may be described as the summer bed of the Indus River. The river keeps on changing its course within this landform. Erosion and deposition takes place over vast area and the surface becomes stable only after the flood water has receded.

3.0 SEA WATER INTRUSION

3.1 Possible Effect on Groundwater

The Sea tides rise and fall into the Indus estuary twice daily. This phenomenon can possibly affect the tidal delta area in two ways. (i) the sea water seeping from the estuary may render the ground water saline and make it unfit for irrigation and drinking purpose and (ii) it may cause salinity in the soil above the ground water and render it unfit for growing any crops. As far as no (i) is concerned, the ground water in the entire reach from Kotri to Sea(174 miles) is already saline and hazardous for irrigation and drinking purposes. Proof of this fact is available on record in the PC-1 Proforma of Kotri Barrage Project. It was saline in the prehistoric days; it is saline at present and will remain saline in future. Certainly it will not be affected of 6.1 MAF is stored by the Kalabagh Dam Project or any other dam on the Indus River.

3.2 Possible Effect on Soil above the Ground-water

Regarding the soil above the groundwater there is no flow down stream Kotri during the Winter season. During this period the tidal waves rise and fall unchecked in the Indus Active area and the soil above the ground water is already saline to the maximum possible extent and therefore unfit for sowing any Rabi crops. During the Summer no crops are grown in the riverine area due to the possibility of flood inundation at any time. It is obvious that under such circumstances Sea water intrusion does not carry any meaning. Therefore, any amount of water released downstream of Kotri during the flood months to check Sea water intrusion will be a sheer waste. Wisdom demands that surplus water should be gainfully utilized for growing food and fibre for the human population instead of wasting it into the Sea.

Adverse effects due to Sea-water intrusion is a non-fact. In fact that the idea of Sea-water intrusion does not carry any meaning.

4.0 MANGROVES FORESTS

Mangrove forests cover 321,510 acres in the Tidal Delta area from Karachi to the Indus Delta to the Rann of Kach. Occasional fresh water supplies (about 17-20%) are needed by mangrove forests to flourish. The Indus River since 1910 has been contained with in a system of "bunds" on both the banks below Kotri Barrage. As such the only area now receiving Indus water (about 40 MAF annually) directly, is the Active Delta where mangroves cover only 7400 acres which is just 2.3% of the total area of mangrove forests. For the Tidal Delta areas, outside the bunds Indus water is completely cut off since a very long period of time and in these areas mangroves species "Avicennia Marina" which is highly salt tolerant thrives. These areas are under the control of the Sindh Forest Department which exercises management control over growth as well as cutting and grazing of the plants; while the area of the Active Delta was placed under the control of the Revenue Federal Department for growing red-rice and is completely neglected as far as forest management measures are concerned. The highly salt tolerant variety "Avicennia Marina" can also be grown in the Active Delta area which can be brought at par with other areas provided this area is also included in the protected forest areas of the Sindh Forests Department. Thus there will be no reduction of mangrove in the Active Delta due to Kalabagh Dam or any other upstream storages on the Indus River.

It is obvious that the assumption about extinction of mangroves in the Indus Active Delta due to construction of a storage dam is un-realistic. As a matter of fact we need to adopt forest management measures to increase the area of mangrove forests.

5.0 FISHERIES

5.1 Palla Fish

Palla is a migratory Seawater fish which breeds in the sweet waters of Indus and grows to the adult age in the Sea water. Before the barrage was built at Sukkur in 1932, Palla fish were reported to migrate as far north as Multan. Later in 1955 the Kotri Barrage constructed near Hyderabad (about 174 miles from the sea) resulted in further reduction in the Palla run. The fish ladders provided at this barrage proved a failure. This fish now migrate only upto Kotri Barrage where they congregate and are consequently caught in large numbers.

For up- stream migration which starts in June with the commencement of fresh flows in the Indus, Palla needs a channel depth of about five feet which occurs at a flow of about 30,000 cusec. This depth of flow is generally available in the Indus through out the flood season. The Indus delta mangroves are important as nursery ground for fish especially for the commercial Shrimp fishery. Mangroves provide shelter and protection to many fish species during their early stages of life.

5.2 Shrimp Fishery

The shrimp fishery in Sindh forms the most important sector in fisheries because of its foreign exchange earning potential and the creation of jobs in the industry. There is no evidence available to establish that due to any reduction in the flows d/s Kotri fish population has declined. Statistics confirm that there has been a gradual increase in the production of fish since 1947.

It is clear that the construction of Kalabagh Dam will have no effect on the Palla fish or shrimp fisheries in the Active Delta of the Indus River. We need to increase mangrove forests in the Active Delta area by adopting forest management measures and remodel the fish ladders at Kotri Barrage to protect the fisheries.

6.0 RIVERINE FORESTS

Riverine forests below Kotri cover an area of 104,405 areas, which rely on annual flooding by Indus for their survival. The study has revealed that with discharge of 500,000 cusecs only 45.7% of the forest area receives water in 5 out of 11 years. Almost 16% of the high level forests receive flood water only once in 12 years when the peak discharge is over 675,000 cusecs. The productivity of the reverine forests is on decline because of the reduced extent and duration of annual flooding.

As riverine forests meet the important social & ecological needs of lower Sindh area these forests can not be allowed to degenerate. Irrigated forestry is therefore essential for the maintenance of the forest ecosystem. It is possible to provide irrigation water to these forests through pumps from the canals running paralled to the bunds along both sides of the River. Those canals can be remodelled and supplied with additional water from the northern storage to meet the requirements of the present forested area which has been estimated to be 0.643 MAF.

It would be realized that the riverine forests which are presently de-generating can only be sustained by providing pumped supplies for which additional water can come only from an Upper Indus storage Dam.

7.0 RIVERINE IRRIGATION

At present 31,479 acres out of the total area of 95,038 acres under cultivation are under lift pump irrigation. About 250 pumps provide irrigation to 24,951 acres using water from canals outside the riverine area. About 6,528 acres get water from the Indus River through 83 pumps.

The spills of River Indus during summer provide irrigation to an area of 63,559 acres. Flood irrigation carries a high element of risk. Crops are grown on a single irrigation with the result that their yields are erratic and much lower than their potential. With varying levels and discharges in the Indus, the only way to attain an assured water supply for crops is to install more lift pumps on the existing canals running outside, along the two bunds. This arrangement which is feasible after the construction of a storage dam, would provide more dependable irrigation supplies to the riverine area for achieving much higher yields from the crops.

It would be seen that additional supplies from the Kalabagh Dam will help riverine irrigation. The existing canals out side the bunds can be remodelled to carry additional discharge which will be provided by a storage dam on the Indus River.

8.0 DOMESTIC WATER SUPPLY

8.1 Riverine Area

The population of the riverine area below Kotri is estimated to be about 120,000 out of which that of the active delta area is over 10,000. There are about 200 villages in the Riverine area, out of which about 135 villages get their water supply directly from the canals of Kotri Barrage running parallel to the bunds. Thirty four villages collect water from the Indus. The remaining 31 villages depend partly on the canals and partly on the Indus River.

8.2 Active Delta

The only source of drinking water for about 67 villages, in Active Delta located from Karo Chan to Sea, is the creek channels. When during floods the water level rises in the creeks, they store water in ponds for drinking purposes. If water stored proves insufficient, they are forced to bring it by boats from other creeks, places or buy it from water sellers. The salinity of water being used for drinking in this area is much higher than the Pakistan standard.

Domestic Water Supply siltation can be improved if pumped irrigation is adopted in the riverine areas and this is possible only with the availability of additional stored supplies.

9.0 IRRIGATION SUPPLIES TO THE SINDH AREA

Average seasonal canal withdrawals of three barrages in the province for different periods are given below:-

Seasonal Canal Withdrawals in Sindh After the Construction of Mangla and Tarbela Dams

Average Annual Withdrawals(MAF)

Years	Stage	Kharif	Rabi	Total	Increase in Rabi Season
1960-67	Pre-Mangla	24.91	10.65	35.56	
1967-77	Pre Tarbela	28.34	12.34	40.68	16%
1977-96	Post Tarbela	29.14	15.11	44.25	42%

It will be noted that during the pre-storage era of 1960-67, average annual canal withdrawals of Singh were 35.56 MAF. After Mangla & Tarbela the corresponding figure rose to 40.68 MAF and 44.25 MAF respectively. It shows that valuable supplies in Rabi season increased by 42%. With Kalabagh dam (or any other new storage), Rabi supplies will further increase.

Water Apportionment Accord of 1991 had envisaged further development of water resources and hence allocated 117.35 MAF of water for canal withdrawals by the four provinces. During 1992 to 1997 period the canal withdrawals have remained at an average of 105.35 MAF. It has not been possible to make up the short-fall of 12.00 MAF, because Winter supplies (to be used during late Rabi, early Kharif and late Kharif) are not available, although enough supplies are available in Summer for the Kharif crops. This shortfall can be met with inter-seasonal transfer of water by building new storage dams. Out of 12.00 MAF Sindh will get 5.18 MAF provided 1.55 MAF is available from the new storage. The balance 3.63 MAF is already available from the river flows in Summer. Details are shown in Table-4.

Kalabagh dam (or any other Upper Indus storage) would store surplus water during flood season and make it available to the four provinces during winter months of the year in accordance with provincial shares stated in the Water Accord as under:-

Province	Percentage	Supply (MAF)
Punjab	37	2.257
Sindh	37	2.257
MWFP	14	0.854
Baluchistan	12	0,732
Total:-	100	6.100

From the above table it should be clear that after construction of Kalabagh Dam, the canal withdrawals in Sindh would further increase by about 5.18 MAF under WAA allocations. The additional water availability to the farmers will result in more crops and a pleasant effect on the eco-system in the Sindh area.

In the absence of new storage our Sindhi brothers are going to be the worst sufferers after 2010 when after depletion of online reservoirs at Mangla, Chashma and Tarbela, enough storage water (about 5.3 MAF) would not be available due to siltation for maturing of Rabi and sowing of Kharif crops. Sindhi farmers will be in a most disadvantageous position as compared to NWFP or even Punjab which are better placed due to more intensive rains as well as fresh ground water which can be utilized for augmentation of the canal irrigation supplies.

TABLE-4

WATER APPORTIONMENT ACCORD (1991)

PRESENT AND FUTURE DISTRIBUTION OF SUPPLIES AMONG THE FOUR PROVINCES

PART-A

WATER APPROPRIATION SPECIFIED IN INDUS WATER ACCORD (1991)				Present Canal Withdrawals (Actual) (MAF)	Additional Supplies Allocated Under WAA (MAF)
Province	Kharif (MAF)	Rabi (MAF)	Total (MAF)		
Punjab	37.07	18.87	55.94	54.36	1.58
Sindh	33.94	14.82	48.76	43.58	5.18
NWFP (a) Irrigation Canals	3.48	2.30	5.78	5.43	3.35
(b) Civil Canals	1.80	1.20	3.00		
Balochistan	2.85	1.02	3.87	1.98	1.89
Total:-	**79.14**	**38.21**	**117.35**	**105.35**	**12.00**

PART-B

Seasonal Distribution of Additional Supplies Available for Sindh with the New Storage.

Crop Period		Additional Supplies for 4 Provinces (MAF)	Additional Supply for Sindh (MAF)	Remarks
i)	Late Rabi	0.60	0.25	To be made available by new storage
ii)	Early Kharif	2.20	0.90	-do-
iii)	Late Kharif	0.80	0.35	To be made available by new storage
	Sub-total (i)+(ii):- +(iii)	3.60	1.55	
iv)	Central Kharif Period	8.40	3.63	Already available during the Summer
	Total:-	12.00	5.18	

ASSESSMENT OF THE RISK OF SCOUR AT BRIDGES

Engr. Ghulam Qadir*

SYNOPSIS

The paper presents advice and guidelines which will enable the Engineers/Scientists to assess hydraulic aspects of bridges erected over water. Possible causes of failure are discussed and guidelines are presented to assess individual structures with respect to scour. The guidelines are based on bridges geometry and river characteristics. Features of the bridge and river are discussed and quantified where feasible. The guidelines have been assigned to allow a bridge to be inspected and assessed in a short time and without special equipment, by using score assigned to each hydraulic parameter. The deserving scores of the structures have been placed in calculation sheets to arrive at the preliminary priority rating/risk of scour.

1. INTRODUCTION

All bridges and structures associated with waterways are potentially at risk of failure from hydraulic causes. Govt. is responsible for a large number of such structures, many of which were built in the nineteenth century. It is therefore a need to prepare guidelines to assess the level of potential risk of individual structures. The guidelines enable the authority to assess the frequency and level of inspection that is appropriate for each individual structure. In addition, the guidelines identify the high priority structures requiring prompt, more detailed inspection.

An approach has been adopted in this work to categorize structures based on priority, but this should not encourage complacency and the belief that particular structures are 'safe'. In reality there will always be a non-zero probability of failure. Nor should the risk associated with a particular structure be regarded as something fixed in time. Changes may take place, particularly with regard to the river, which may significantly change the factors influencing the hydraulics of the structure. Thus, for example, changes in the alignment of a river channel may radically affect the risk of failure due to scour.

The major cause of bridge failure is

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undermining of peir and abutment foundations following scouring or erosion of the channel bed.

Scour at bridges is highly complex process and cannot be precisely predicted. In this work, the main and most easily measurable parameters which could affect the risk of scour have been highlighted, and have, with the help of published research findings, established how scour is affected by each parameter. The combined effect of all the parameters will be an indication of the severity of scour, but it is difficult to accurately predict the maximum scour depth at the bridge. In assessing the risk of failure, the depth and structural condition of the bridge foundations are as important as the depth of scour, but may not be known with certainty. A bridge with potential for deep scour relative to its foundations will be at higher risk than one which causes little scour and has good deep foundations. In this work, the two quantities of scour depth and foundation depth are assessed and compared. This comparison provides the basis for categorisation of peirs and abutments within the main river channel with respect to their risk of failure due to scour.

The guidelines have been prepared with the aim of requiring data which can be obtained from observation, simple site measurements and a small amount of desk work. In some causes, this work advises the use of additional data where this is available, in order to provide a more accurate assessment.

2. TYPES OF FAILURE DUE TO HYDRAULIC CAUSES

Several possible causes of failure are discussed in this section. Failure is most likely to occur at high flows, when the river is in flood. Scour is the most frequent cause of failure and is the chief subject of this attempt.

2.1 Failure Due to Scour

Most rivers have beds and banks of more or less mobile material. During a flood, the bed level may fall as bed material is transported by the moving water. A bridge across the river can result in additional lowering of the bed level at the bridge. Two possible causes of this extra erosion, or scour, are a general increase in flow velocity due to a construction of the channel and a local disturbance of the flow due to a bridge pier or abutment. These two types of scour are called general and local scour. General scour may affect the whole width of the river, while local scour occurs adjacent to piers or abutments. Where both types of scour occur, the total depth of scour is the sum of general and local scour. In addition, scour may be increased on navigable waterways, by the action of vessels causing rapid displacement of water and high local flow rates. Local scour at a bridge pier is normally deepest near the upstream nose of the pier, due to the local geometry and flow or the nature of the sediment. However, there may be exceptions where the local scour is deeper in other areas adjoining the pier.

Many features of the bridge and river affect depth of scour and the complex nature of the problem means that accurate prediction of scour is not possible except in very simple cases. We can, however, identify the most important features and predict trends of how scour depends on each feature, and assess the expected severity of scour for a given bridge by combining the effects of the significant features. This forms the basis of assessment methods in this work.

The depth of the foundations is important in determining the risk to a bridge from a given degree of scour. Deep foundations subjected to severe scour may be safer than a shallow spread footing in only moderate scour.

A bridge constructed on spread foundations will be at risk from scour when the adjacent scour reaches the level of the base of footing. However, if the substructure member is subjected to lateral loads which are partially or wholly resisted by passive pressure then the foundation may be at risk before scour reaches the footing level. These lateral forces may be increased by hydrodynamic effects.

Scour adjacent to piled foundations may result in a loss of skin friction and reduction in load bearing capacity of the piles, even if they have not been undermined.

Structural analysis of the bridge foundations may be required to accurately assess the critical bed level below which the foundations become unsafe.

2.2 Failure Due to Bank Erosion

Most natural rivers tend to change their course with time. A mechanism by which this occurs is bank erosion. A structure such as a pier or abutment located on a flood plain may be placed at risk if the main channel moves sufficiently close to the structure to cause loss of support or undermining. Bank erosion may occur very slowly in time, or may be very rapid, particularly during times of flood. The rate of bank erosion depends partly on the character of the river, a river with a steep gradient and high flow velocities will in general be more active and prone to bank erosion than a river with a fairly flat slope and lower velocities.

2.3 Failure of Approach Embankments

Overtopping of the approach bank and or turbulent flow adjacent to the approach embankments can lead to erosion and scour to the side slopes and toes of the embankment. This may lead to instability of the approach embankments and possible loss of the structure. Loss of fill material around and behind the wing walls can lead to instability and failure of the wing walls.

2.4 Failure Due to Hydraulic Forces on Piers

Water flowing past a bridge pier exerts a force on the pier. This force can be resolved into two components one along the pier, which is referred to as the drag force and one normal to the pier, which is referred to as the lift force.

The applied forces depend upon the depth of flow and the length of the pier, with a marked dependence upon the flow velocity. If the flow is aligned with the pier, the lift force is zero but as the angle of attack increases, the lift force increases rapidly. The ability of a pier to withstand drag forces will depend upon the structure of the bridge and the foundation details and may be reduced during a flood if significant scour around the base of a pier takes place. A method for evaluating these forces is given in Faraday and Charlton (1983).

Debris which is caught on piers can result in increased hydraulic forces by increasing the effective pier width, while floating debris which collides with piers can cause dynamic loading. Both types of loading will probably be most severe when the river is in flood.

2.5 Failure Due to Hydraulic Forces on the Bridge Deck

If the water level reaches the soffit level of a bridge, or the springing in the case of an arch, the flowing water will exert a force on the bridge deck. The drag on the deck may be calculated in a similar way to drag on a pier, and is again very dependent on the flow velocity. A force applied to the deck of the bridge is potentially more dangerous due to the large overturning moment about the pier foundations. If it is known that historic flood levels have approached the bridge deck, it may be appropriate to carry out a site-specific study to assess future flood levels, flow velocities and hydraulic forces and the resistance of the bridge to these forces. Estimates should also be made of possible dynamic loads imposed by collision of floating debris. These types of study are outside the scope of this work. For further information consult Faraday and Charlton (1983)

2.6 Failure Due to Debris

Build up of trash and debris against bridge components can significantly affect the hydraulic performance of bridges. Difficulties are normally associated with small, single-span bridges which tend to be more easily blocked than large multi-span structures. For such single span bridges, the blockage can be extensive, reaching up to 90% of the bridge opening. This may result in large increases in water level and flooding upstream. Debris may restrict the flow leading to significant scour around piers or abutments threatening the safety of the structure.

Debris may also result in additional *"drag"* and *"lift"* forces on piers, and impact forces may result from debris colliding with piers.

3. MODIFICATIONS TO THE RIVER OR CATCHMENT

3.1 Modification in the Neighbourhood of the Bridge.

Structure Constructed Later than the Bridge

Another bridge built upstream can affect the hydraulic conditions at the downstream existing bridge. Increased turbulence and non-uniform approach velocity can both result in deeper scour depths at the existing bridge. The upstream structure may alter the flow direction and cause erosion of abutments and deeper scour at the bridge piers. This effect is illustrated in Fig.1.

- Bank Protection Measures in the Neighbourhood of the Bridge

Work may have been carried out to protect the banks and stabilise the course of the river. The presence of bank protection measures which post date the bridge may indicate that the river has in the past actively eroded its banks and may have changed its course and its cross sectional geometry. Both types of change can reduce the safety of an adjacent bridge.

3.2 Modifications to the River

If water and/or sediment flows in a river are altered, the equilibrium state of the river may change. The plan geometry, cross section and discharges at the bridge site may be affected. The bridge may be subjected to worse erosion and scour and higher water levels.

It is not possible to give guidelines on whether changes due to river works are likely to improve or worsen the bridge's safety, as the river geometry depends on complex interaction between discharge and sediment movement. In general, an unstable river will be more liable to change than a stable river.

Changes in flow can be caused by a change in the river upstream of a bridge. River works can cause these flow changes, if they have been carried out since the construction of the bridge. Some river works which could cause these effects are identified below:-

* Construction of flood embankments .

* Construction of flood detention basins.

* Construction or removal of bridges.

* Construction or removal of bank potection or river training works.

* Changes in water abstraction patterns.

* Schemes for water transfer between river basins.

Approach velocities may be increased by the construction of flood banks, which eliminate flood plain storage and increase flows; the construction or removal of another bridge, which may alter flow velocities and directions, and by the construction or removal of bank protection or river training works. The above effects are illustrated in Figure.1.

Other river works can cause changes in bed level:

* Channel improvement schemes such as dredging or gravel abstraction, weed clearance, realignment.

* Reservoir impoundment.

* Construction or removal of weirs.

The above effects are illustrated in Figure-2.

3.3 Modifications to the Catchment

Changes to catchment characteristics can affect rates/changes in discharge, water levels and channel geometry. Changes to a catchment which have occurred since the bridge was constructed can therefore affect the safety of the bridge.

Some examples of catchment changes which may be significant are given below:

- Urbanisation.
- De-forestations.
- Change in land drainage.
- Change in groundwater regime.

Recent modifications either to the structure or the river should be recorded and their potential impact on the hydraulics of the bridge considered. When modifications are proposed their potential impact should be assessed.

4. ASSESSMENT PROCEDURE OF RISK

4.1 Introduction

The bridge piers or abutments which lie within or protrude into the main river channel. They are assessed by combining the effects of features of the bridge and river which could be significant in determining the risk of failure of the bridge due to scour.

Each feature is given a score, and the scores are combined to give a *''risk number''* which reflects the potential risk to the bridge. The brief theory behind this procedure is described in the following sections.

In certain circumstances, e.g. if the bridge is founded on bed-rock, it may not be necessary to carry out the full assessment described in this section.

This assessment is designed to provide a method for surveying the hydraulics of bridges. It has been impossible to include all aspects of the hydraulics of bridge structures. For example, if a bridge is immediately downstream of a bend then the geometry of the river may lead to the formation of large eddies in the neighbourhood of the bridge. This can have a dramatic effect on both the magnitude and direction of the velocities of flow. If it is suspected that any bridge is peculiar in nature because of the presence or absence or any feature then this should be noted and further advice sought.

The history of scour at a bridge may give further indication of the susceptibility to future scour problems.

There may be significant difference between the various elements of the bridge. Thus if a bridge has a number of piers then the size and shape of the piers or the depth of foundations may differ significantly. Thus a number of markings may be obtained for one structure. The appropriate action to be taken should then be based on the worst scour obtained.

A new bridge would normally be designed to with-stand a flood of specified magnitude. The magnitude is normally expressed in terms of return period. If a given discharge is exceeded, on average, once every T years then that discharge is said to have a T year return period. Calculation of design discharge in rivers is a specialist topic which is not being included in this works. If the design discharge is known, then methods are availabe for calculating water levels and flow velocities.

In order to compare risk at a number of bridges, it would, strictly, be necessary to calculate flow conditions at all the bridges for the same return period flows. This would be a major task. By making some simplifying assumptions, here, the procedure avoids the need to calculate flow conditions for particular return periods.

Where the procedure refers to *''flood conditions''* this should be taken to mean conditions during an historically high flood. This could, for example, be a flood with a selected high return period (eg 50 or 100 years), or alternatively this could be the highest recorded flood.

4.2 Measurement of Main Dimensions

4.2.1 Channel width *Wu* and Depth *Yu*

Wu is the bank to bank channel width, measured upstream of the bridge. *Yu* is the mean channel depth upstream of the bridge. On smaller rivers, the typical bank to bank channel width upstream of the bridge can be measured directly. Where this is not practicable, estimate the width as accurately as possible. Large scale maps may also be used in some cases.

If the width varies significantly then take the average width in the reach upto a distance of 10 channel widths upstream of the bridge.

The mean depth of the channel from the bed to the bank tops. *Yu*, should also be determined. *Yu* is defined more precisely as the cross sectional area of flow in the main channel divided by the channel width *Wu*. In most cases it will not be possible to measure this depth directly without taking soundings from a boat, though in some cases information may be available from previous survey work.

If no estimate can be made of water depth, which will often be the case for larger rivers, then the following formula may be used.

$$Yu = 0.185\ Wu^{0.7} \qquad (1)$$

This formula may give an indication of the approximate bankful depth, but applies only to alluvial rivers whose dimensions are not controlled by features such as rock, bank protection or highly cohesive banks or beds. Other factors such as bank vegetation may also affect the width/depth relationship. The above formula should be used when no other method for estimating channel depth can be used.

4.2.2 Channel Width W_b and depth Y_b at the Bridge

W_b is the width of the main channel under the bridge, defined as the distance from bank to bank, minus the width of bridge piers.

Y_b is the mean channel depth at the bridge. This is defined as the cross sectional area of flow in the main channel divided by the channel width W_b.

Note that the same reference level (i.e. either bankful level or, if known, flood water level) should be used for assessing both Y_u and Y_b.

4.2.3 Flood Plain Width *Wo* and Flood Plain Depth *Yo*

The amount of flow which approaches the bridge on the flood plain will influence scour conditions at the bridge. The most accurate estimate will be obtained if the width of flow on the flood plain, and the average depth of flow on the flood plain, can be estimated.

Yo is the average depth of flow over the plain, at a typical cross section approximately 10 river widths upstream of the bridge. This is equivalent to the cross-sectional area of flood plain flow divided by the total water surface width of the flood plain *Wo*. Guidelines for estimating *Yo* and *Wo* are given below.

In order to estimate the flood plain flow depth *Yo*, the following methods may be used.

i. Hydraulic analysis of the river under design flood conditions enables *Yo* to be calculated. This method should give a reasonably accurate estimate of the water level and hence average flow depth over the flood plain.

ii. Water levels observed during high flood events may be estimated to obtain mean flood plain flow depth. It is useful to sketch the cross section upstream of the bridge, including flood plain ground levels, in order to estimate average flow depth.

iii. The extent of flooding, together with information of ground levels, may be used to estimate flood plain flow depths. The extent of flooding from high floods may have been recorded or observed.

iv Or flood records, such as *''flood levels reach underside of bridge''* or *''flood levels reaching track level or approach embankment''* should be used, if available to estimate flood water levels.

Note that high values of *Yo* will tend to result in higher estimates of scour.

If no estimate of *Yo* can be made, then it is recommended that the ratio *Yo/Yu* is set to 0.3 if it is known that flooding does not occur, then *Yo/Yu* should be set to 0.

The flood plain width *Wo* is the combined width of the right and left flood plains, measured perpendicular to the main flow direction.

In order to estimate *Wo,* the following methods should be used. It should be noted that *Wo* is the water surface width of the flooded cross section, minus the channel width *Wu,* measured at a typical location within approximately 10 channel widths upstream of the bridge.

i. The geometry of the river valley may indicate the extent of the flood plains. For example, relatively level, flood prone areas may be bounded by steeper lopes or flood embankments, giving a clear indication of flood water.

ii. Hydraulic analysis of the river under design flood enables *Wo* to be calculated but the more detailed analysis will lead to more accurate results.

If no estimate of *Wo* can be made, it is recommended that the ratio *Wo/Wu* is set to 5. If it is known that flooding does not occur, then *Wo* should be recorded as zero.

In view of the uncertainties in establishing flood plain flow depth and width sensitivity tests can be carried out to assess the effect on the final *''priority rating''* of a range of flood plain depths and widths.

4.2.4 Pier Width *Wp* and Length *Lp*

The effective width of each pier should be measured and estimated. Use the following notes for guidance.

- For the simplest case of a single, uniform pier extending to below the general scoured bed level, the pier width *Wp* is defined as the width of the pier, measured perpendicular to the long axis of the pier if the pier is elongated.

- If the pier has an enlarged footing or base, part of which lies above the general scoured bed level, then the effective pier width W_p is taken to be the width of the enlarged footing or base.

- If an abutment projects into the main channel by a width W_a then its effective width is W_p. W_P is calculated from the width of projection of the abutment W_a:

$$W_p = 2W_a \qquad (2)$$

W_a is the width of projection of the abutment into the main river channel. If the abutment and river bank are both vertical, then W_a is simply the distance from the line of the river bank to the face of the abutment. If the face of the abutment or river bank are not vertical, then W_a is the average projection of the abutment from the river bank.

- If the abutment has an enlarged footing or base, part of which lies above the general scoured bed level, then the abutment width W_a should be assumed to be equal to the projection of the footing from the river bank, W_a. If the river bank or face of the footing are not vertical, then W_p is the average projection of the footing from the river bank.

- If the pier consists of a group of two or more circular columns, with centre-centre spacing of less than 5 W_p, then measure or estimate separation of the columns. If the columns are aligned approximately with the flow direction, then measure or estimate the centre-centre separation C_p. If the columns are aligned approximately perpendicular to the flow direction, then measure or estimate the centre-centre separation C_2. The pier with W_p is the width of an individual column within the group.

- If a group of columns is founded on a single base or footing which is partly above the general scoured bed level, the effective pier width W_p is equal to the width of the base or footing.

The length of the pier, L_p is measured along the long axis of the pier. For a circular pier, $L_p = W_p$. For the case of two adjacent elongated piers, with one positioned downstream of the other, the measurement L_p depends on the gap between the piers. If the gap is greater than 3 W_p., then L_p is the length of an individual pier. If the gap is less than 3 W_P, then L_p

is the sum of the length of the individual piers.

4.3 Assessment of Features Significant in Scour

4.3.1 Constriction of Channel and Flood Plain

A constriction in the width of a channel , for example by bridge abutments or piers, tends to result in a decrease in bed level within the constriction. This reduction in bed level is known as general scour due to channel constriction, and depends on the ratio of the channel width at the constriction. W_b to the channel width upstream W_u. These dimensions are estimated in Sections 4.2.1 and 4.2.2.

Approach embankments which cross a flood plain will force flood water through the bridge opening, resulting in higher flows in the main channel under the bridge and a general lowering of the river bed level at the bridge site. This is known as general scour due to flood plain constriction, and depends on the extent to which embankments obstruct or block the flood plain flow.

General scour due to flood plain constriction depends on the depth of flow over the flood plain Y_o. the width of flow over the flood plain W_o, and the depth and width of the main river channel upstream of the bridge. Y_u and W_u. These dimensions were estimated in sections 4.2.1 and 4.2.3.

Calculate the ratios W_u/W_b, Y_o/Y_u and W_o/W_u. Enter values on calculation sheet.

Based on the values of these ratios, use figures 3 to 6 to obtain a value of d_{g1}/Y_u. Interpolate between lines if necessary, and interpolate between figures if necessary.

Alternatively, the following equation may be used.

Enter the value of D_{g1}/Y_u in the space provided on the calculation sheet.

4.3.2 Additional Scour Due to River Bends

In this section, the risk that a pier or abutment is exposed to deeper than average depth of scour due to a bend in the river is assessed. The assessment should be based upon whether there are any significant bends and their severity. The relevant reach of the river includes the bridge site and extends approximately 5 channel widths upstream of the bridge. If this stretch of river is straight it should be marked accordingly. If this section of river contains a bend then its severity should be assessed and marked.

If the channel is straight within this reach, the location of the deepest scour may shift across the width of the channel unpredictably. A curved channel will tend to adopt a triangular cross sectional shape, with the deepest point towards the outside of the bend.

For each abutment which lies in the river channel, obtain a scour from the following table:

LOCATION OF ABUTMENT

Bend Sharpness	Inside of Bend	Outside of Bend
Straight	4	4
Slight	3	5
Moderate	2	6
Severe	1	7

For each pier which is in the river channel, obtain score from the following table:

LOCATION OF PIER WITHIN CHANNEL

Sharpness of Bend	Inner 1/3 of Bend	Central 1/3	Outer 1/3 of Bend
Straight	4	4	4
Slight	3	4	5
Moderate	2	5	6
Severe	1	6	7

Enter the highest of the scores for the pier/abutment on data sheet.

4.3.3. Relative Flow Depth

Local scour at a pier is reduced if the flow depth is shallow compared with the pier width. This is most likely for wide piers or where flows will normally be relatively shallow.

The maximum flow depth at the pier should be calculated. If the pier or abutment is in the river channel, the maximum depth is the general scoured depth, which is calculated below in table in Section 4.5.1. As the assessment of relative flow depth requires information derived in Section 4.5.1, its assessment is delayed until then.

4.3.4. Angle of Attack and Pier Thickness

A pier which is not well aligned with the oncoming flow can result in greatly increased scour depths.For a given angle of attack, a pier which is slender will be more severely affected than one which is square or circular in plan.

Estimate the angle of attack, which is the angle between the long axis of the pier and the approach current, using the following information.

- Observe the direction of current at the surface, from floating debris etc. just before it reaches the pier. Estimate the maximum angle between this approach current and the pier.
- Estimate the angle between the main channel direction immediately upstream of the bridge, and the bridge pier.

The angle of attack at high flows may be different from that at low flows. If the river floods, the flood plain flow may be deflected by approach embankments, increasing the angle of attack. Take account also of any abutments or works upstream of the bridge, which may deflect the current at high flows to increase the angle of attack.

Estimate the thinness of the pier, defined as

Thinnes $\frac{\text{Pier length}}{\text{Pier width}} = \frac{L_p}{W_p}$

L_p and W_p were estimated in Section 4.2.4 Find a score for angle of attack and pier thinness from the following table.

INTERPOLATE IF NECESSARY:

Angle of Attack (degree)	Thinness 1	2	4	6	8	12	16
0	1.0	1.0	1.0	1.0	1.0	1.0	1.0
5	1.0	1.1	1.2	1.3	1.4	1.5	1.6
10	1.0	1.2	1.4	1.6	1.7	2.0	2.5
20	1.0	1.4	1.8	2.0	2.2	.2.7	3.5
30	1.0	1.5	2.0	2.3	2.7	3.4	4.2
45	1.0	1.5	2.3	2.8	3.2	4.1	5.1
60	1.0	1.6	2.5	3.1	3.7	4.8	5.8

Enter for angle of attack and pier thinnes on data sheet.

4.3.5 Group of Columns

If a pier consists of a group of circular columns, with a centre-centre spacing of less than $5W_p$ then the effects of the columns may interact to increase the depth of scour.

If a pier does not consist of columns, then the following calculations should be omitted, and a scour of 'I' should be entered for *"group of columns"* on the sheet.

Use the following measurements depending on the arrangement of the columns.

C_1: Centre-centre distance of columns which are arranged approxi-mately parallel to the flow direction.

C_2: Centre-centre distance of columns which are arranged approxi-mately perpendicular to the flow direction.

W_p: Width of a column within the group.

Note that if four or more columns are arranged in a rectangular or square layout, then values for C_1 and C_2 will be obtained.

(If column widths are not all equal, W_p should be the width of the widest colunm).

α Angle of attack of flow

(See Fig. 7 to obtain scores using the values of C_1/W_p and α and /or C_2/W_p and (90 α) as appropriate.

If more than one score is obtained, choose the maximum score.

Enter score for group of columns on the calculation sheet:

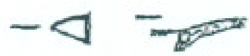
4.3.6 Pier Nose Shape

The plan shape of the upstream end of the pier has a small influence on scour. If the pier is not well aligned, however, the shape of the pier ceases to significantly affect the scour.

Match the upstream end of the pier or abutment, or both ends if tidal conditions prevail, to one of the following drawings, and obtain a score for pier nose or abutment shape.

Nose shape in plain Pier/Abutment **Score**

Angle of attack >5° (See 4.3.5) 4

 1

 2

 4

 7

Enter the highest score or pier nose of abutment shape on data sheet

4.3.7 Assessment of Bend Sediment Size and Grading

Probe the bed of the river, and where possible take samples of the bed material at several locations upstream and downstream of the bridge and beneath the bridge. A layer of fine material (silt) may have been deposited on the bed if the flow velocity is low at the time of inspection. If gravel or sand is found beneath a layer of silt, ignore the silt and sample the underlying gravel or sand.

Examine the sampled bed material, and put ticks in the following table corresponding to material which is present in significant proportions.

Table for assessing bed sediment size and grading

Description	Clay	Silit	Sand	Gravel	
				Fine Medium	Coarse
Particle size (mm)	0.002	0.002-0.06	0.06-2	2.20	20
Tick if present					

Assess the grading of the bed material using the following table and obtain a score for bed material grading.

Number of ticks in above table	Bed material grading Narrow	Score
1	Narrow	7
2		5
3		3
4		2
5	Wide	1

Enter score for bed material grading on data sheet.

4.3.8 Blockage Due to Trapped Debris

Scour may increase significantly if debris such as vegetation becomes trapped on a pier, blocking part of the waterway opening.

It the bridge has one or more piers in the river channel or has a single span of less than 10 m. obtain a score for the effect of trapped debris from the following table.

Catchment Vegetation					
		Heavily forested	Wooded	Fertile bank vegetation	Few trees and bushes
Catchment topography	Steep	7	5	4	2
	Hilly	6	4	3	2
	Moderate	5	3	3	1
	Flat	4	2	2	1

If debris is present which is causing a significant blockage to the flow through the bridge then score 7. If the bridge has a history of debris blockage, then score 7.

Enter score for trapped debris on data sheet:

4.4 Foundations

4.4.1 Introduction

This section presents guidance for assessing the foundation depth. The depth of foundations is important as it partly determines the ability of the bridge to withstand scour.

Spread foundations are most susceptible to scour. Safety is increased if the foundation is on bearing piles or surrounded by a sheet pile curtain which extends below the base of the foundations.

Pile foundations are inherently safer than spread foundations, but excessive scouring can result in loss of skin friction or pile stability. Old piles may have lost strength due to deterioration.

The type of bed material on which the pier or abutment is founded is important in determining the integrity of the foundation if it is subjected to scouring forces.

The critical foundation level is the level to which the bed level can fall without endangering the bridge. If the bed level falls below the critical foundations, level, then it is assumed that the foundations is in danger of failing. For the case of spread foundations, the critical level may be the level of the underside of the base. For abutments, the critical level may be higher than the underside of the base, if bed material acts to resist lateral forces on the abutment due to pressure from the embankment.

The critical level for pile foundations depends on the type of pile (skin friction, end bearing or both) and its stability. The critical level will often be higher than the base of the piles, but more detailed analysis would be required to determine this accurately.

In many cases piers are founded on an enlarged base which is underpinned by columns. Here again, more detailed analysis would be needed to accurately determine the critical level.

The "foundation depth" is measured from the river bed to the critical foundation level.

4.4.2 Measurement of Foundation Depth

The foundation depth is taken to be the vertical distance between the bed of the river and the critical foundation level. The measurement of level of the bed of the river should be taken in the region surrounding the pier up to a distance of $2W_p$ away from the centre of the pier. The minimum bed level within this region should be taken.

Case (a) in Figure 8 shows a depression in the bed around a pier and d_1 is measured at point close to the pier.

Case (b) in Figure 8 shows that there has been significant scour away from the pier and in this case d_1 is measured from the minimum bed level within an area of radius $2W_p$ from the centre of the pier. In this case (b) the measurement position is at a distance $2W_p$ from centre-line of the pier and the depth of foundation d_1 is negative if the bed level at measurement position is lower than the lowest part of the foundation.

Based on the estimates of critical foundation level and river bed level, obtain best estimate for the foundation depth.

Enter foundation depth d_1(m) on data sheet.

4.5 Calculation of scour Depth d_1

4.5.1 Calculation of Depth of General Scour

In this section, a value for general depth is calculated using the value of d_{g1} / Y_u calculated in section 4.3.1.

The following calculation determines d_g/Y_u which is dependent upon d_{g1}/Y_u and factors which related scour due to bends and bed material grading. Enter the scores from appropriate sections into Table-1.

Table-1

Section	Description	Score S	Calculation	Result	Factor
4.3.2	Scour due to bends		0.25S+0.25		B
4.3.7	Bed material grading		0.05S+0.65		BMG

To determine d_g/Y_u, find d_{g1}/Y_u from section 4.3.1.

to $\frac{d_{g1}}{Y_u}$

1. Add1
2. Multiply by B and BMG
3. Subtract BMG

the result gives a value for d_g/Y_u

The value of Y_u is known (section 4.2.1)

Multiply D_g/y_u by Y_u to obtain value for d_g

Enter d_g on data sheet:

4.5.2 Calculation of Depth of local Scour

Calculate the depth of flow at the bridge including the effect of general scour. Y_m

$$Y_m = Y_u + dg \quad (4)$$

Obtain a score for relative flow depth from the following table.

Y_m/W_n		Score
<0.2		1
>0.2	≤ 0.5	2
>0.5	≤ 0.8	3
>0.8	≤ 1.2	4
>0.2	≤ 1.6	5
>1.6	≤ 2.3	6
>2.3		7

Enter score for relative flow depth on data sheet.

Enter score from appropriate sections into table-2.

Table-2

Section	**Description**	**Score S**	**Calculation**	**Result**
4.3.3	Relative flow depth		0.11 x S+0.23 =	
4.3.4	Angle of attack and pier thickness		1.0 x S =	
4.3.5	Group or columns		1.0 x S =	
4.3.6	Pier nose shape		0.1 x S + 0.6 =	
4.3.7	Bed material grading		0.05 x S + 0.65 =	
4.3.8	Trapped debris Product FL =		0.08 x S + 0.92 =	
				Product FL

Calculate local score : = 1.5 x Fl x W_p

4.5.3 **Calculation of Total Depth of Scour**

Obtain an initial estimate of total scour d_1 from

$$d_1 = d_1 + d_g \quad (5)$$

The following paragraphs describe a correction which may be applied to the calculation of scour d_1. The correction accounts for the fact that in some cases, scour which has occurred during a flood may not have completely *"filled in"* during low flow periods. The following correction accounts for this. Note that this is optional but should be used if sufficient bed level data is available.

Figure-9 shows the measurement of d_1 being made from a point of minimum bed level within a radius of $2W_p$ from the centre of the pier. But the calculation of total scour d_1 is, strictly, the difference between the bed level at the bridge during flood and the bed level in a typical channel section upstream of the bridge. Therefore in order to make an accurate comparison of scour depth and foundation depth, a correction may be applied to the calculated scour depth.

At some bridges, general and local scour may already exist at the time of inspections, as shown in Figure 9. During a flood, the general and local scour will increase, lowering the bed level and then possibly fill in again during lower flow conditions.

The total calculated scour depth, d_1, is greater than that in the actual case due to the existing scour of depth AF. Therefore d_1 should be adjusted by subtracting the adjustment factor AF.

This will ensure that a valid comparison of d_1 and depth foundation d_1 can now be made.

The value AF is the difference between the minimum bed level at a typical cross section upstream of the bridge, (preferably beyond any channel constriction associated with the bridge) and the bed level from which the foundation depth was measured. Note that this correction can only be applied if sufficient knowledge of bed levels is available e.g. from rivers, reports or echo sounding surveys. If insufficient information is available, then AF should be assumed to be zero.

It is possible that the bed level in the upstream channel will be lower than the bed level under the bridge if dredging etc has taken place in which case AF should be assumed to be zero.

Enter adjustment factor AF (m) on data sheet.

Calculate revised scour depth

$$d_1 = d_1 - AF \quad (6)$$

4.5.4 Foundation Depth

The foundation depth d_1(m) is calculated in section 4.4.2.

4.6 Assessment of Risk Due to Scour

The scour depth d_1 (m) and foundation depth d_1(m) are now combined to give a preliminary priority rating for the bridge. The priority rating is then modified to account for other factors such as the river type, and load bearing material.

Use the graph on Figure 10 to obtain the preliminary priority rating, based on values of d_1 and d_1 obtained above. Interpolate between curves where necessary. Alternatively, the following formula may be used .

Preliminary priority rating

$$= 15 + \ln\left[(d_1 - d_1)/d_1 + 1\right]$$

Note that this is valid only when the foundation depth d_1 is greater than O.

6. RECOMMENDATIONS

1. A number of bridges should be assessed in a number of regions, in collaboration with IRI, using the guidelines contained in this paper. This will enable calibration of the method.

2. Once the guidelines are judged to be satisfactory, bridges should be examined with the aid of the guidelines to estimate degrees of risk and to assess levels and frequencies of future inspections.

3. Details of any existing or planned modifications to the bridges or river which are relevant to the hydraulic safety of the bridge should be obtained. The effect of modifications on the safety of the bridge should be assessed.

4 A database should be set up containing all available information relevant to hydraulic conditions at each bridge site.

5. Further research is needed regarding scour at bridges over rivers. A programme of research which includes field measurements of scour at a number of bridge sites during floods would enable improved methods of scour prediction to be developed.

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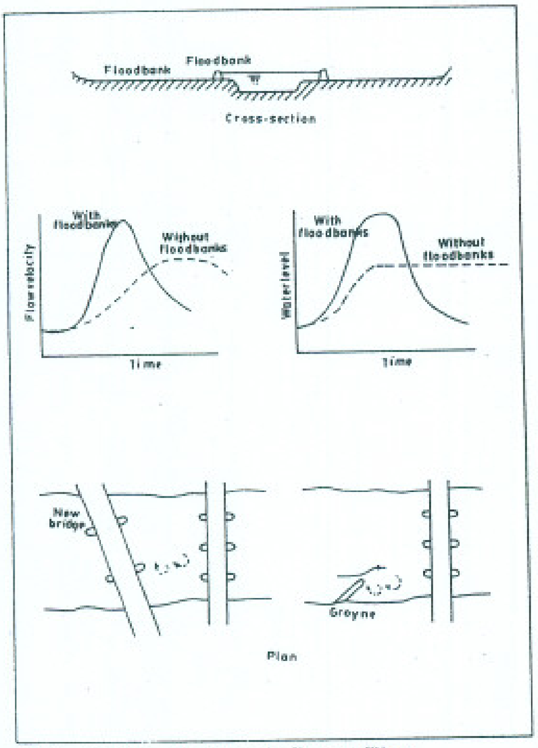

Fig: 1 Examples of changes in flow conditions

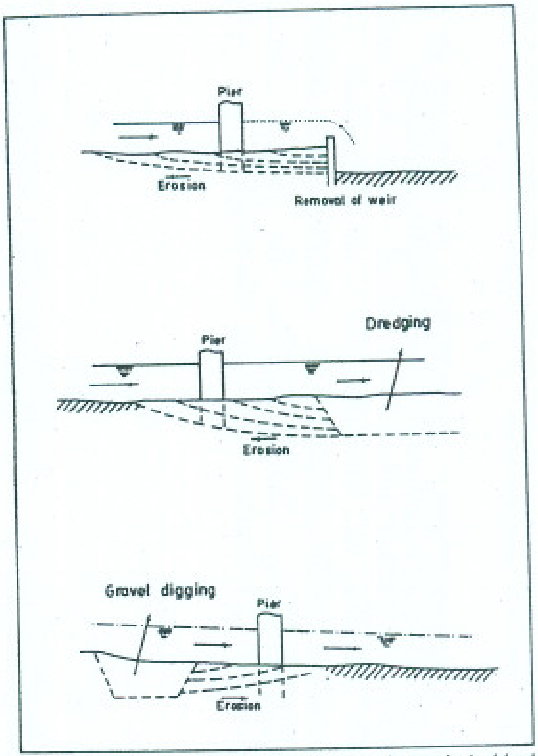

Fig: 2 Examples of modifications which cause changes in bed level

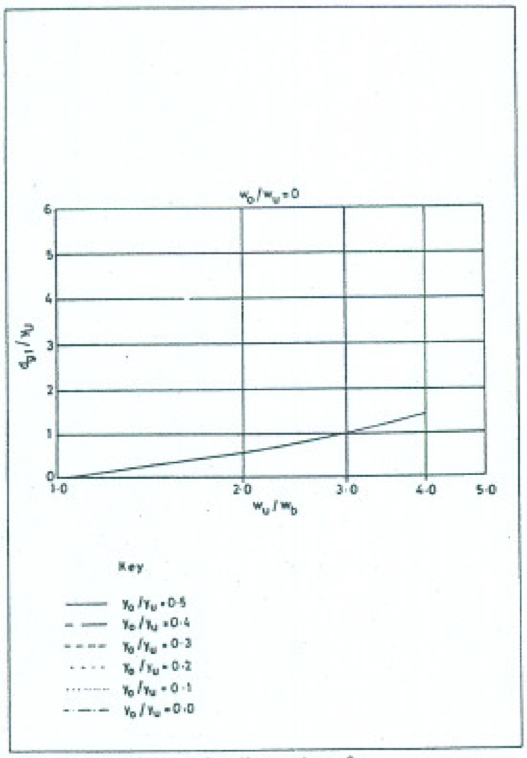

Fig: 3 General scour function $w_o/w_u = 0$

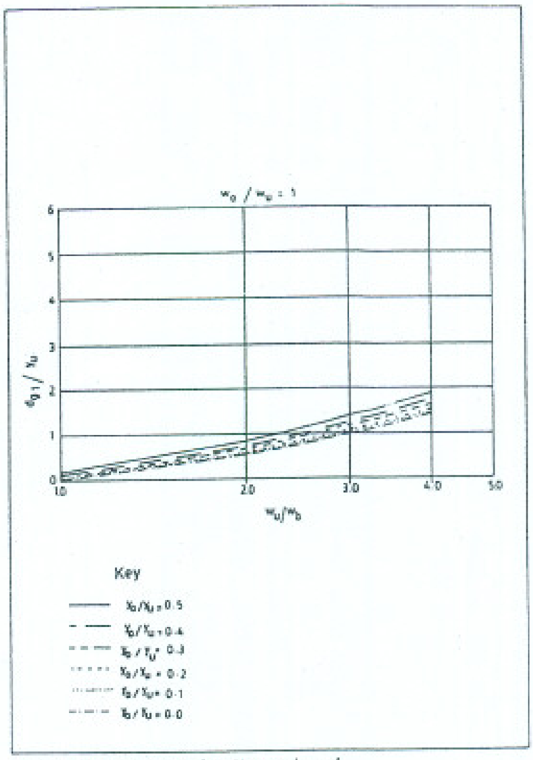

Fig. 4 General scour function, $w_o/w_u = 1$

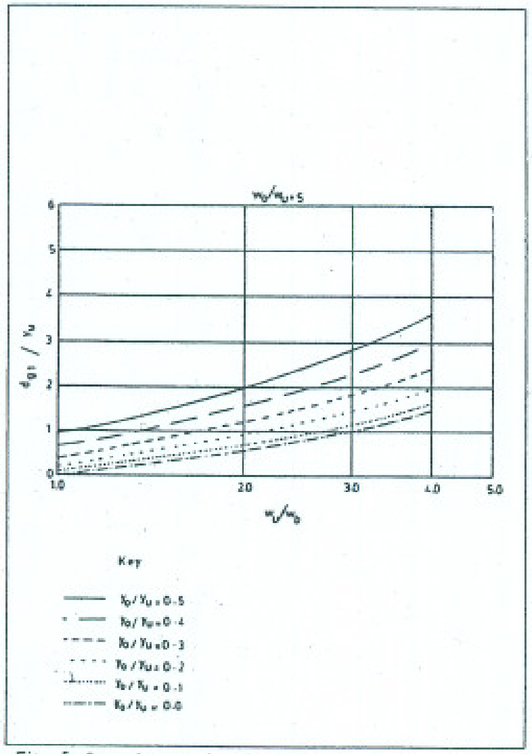

Fig: 5 General scour function, $w_o/w_u = 5$

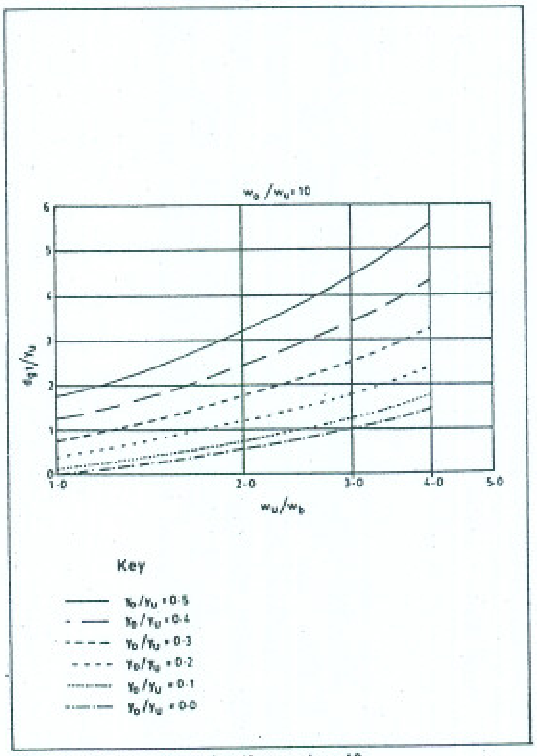

Fig: 6 General scour function, $w_o/w_u = 10$

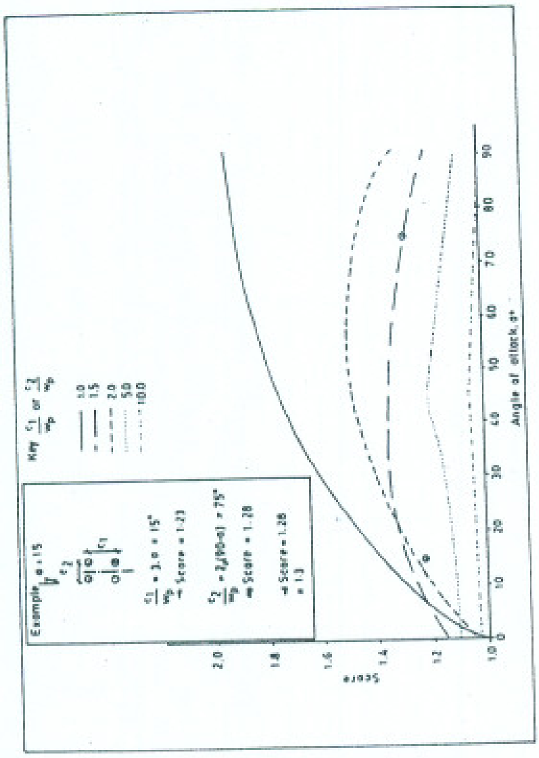

Figure : 7 Scour at piers consisting of group of columns

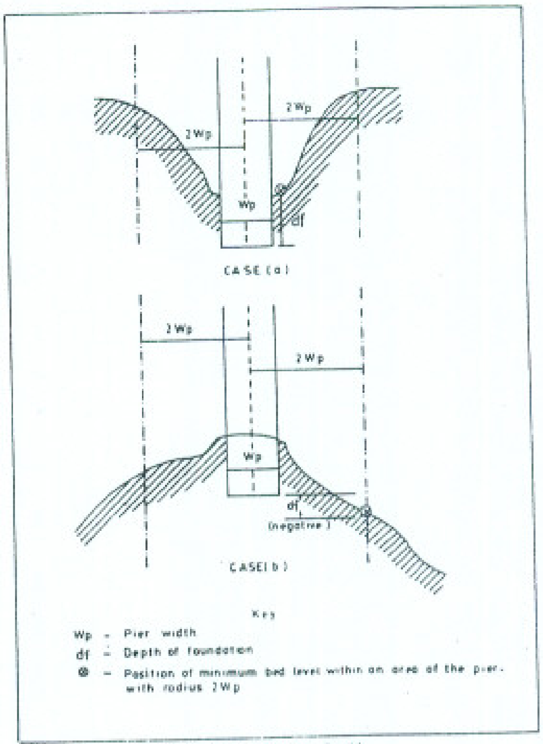

Fig. 8 Measurement of foundation depth

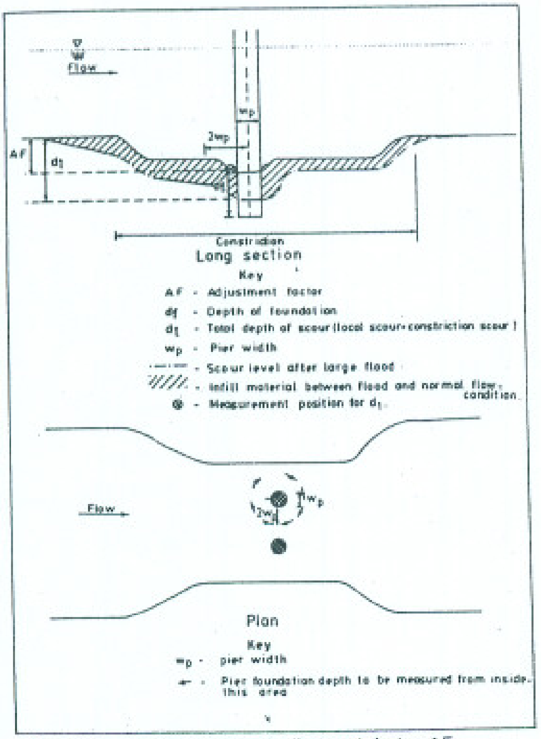

Fig: 9 Foundation depth and adjustment factor AF

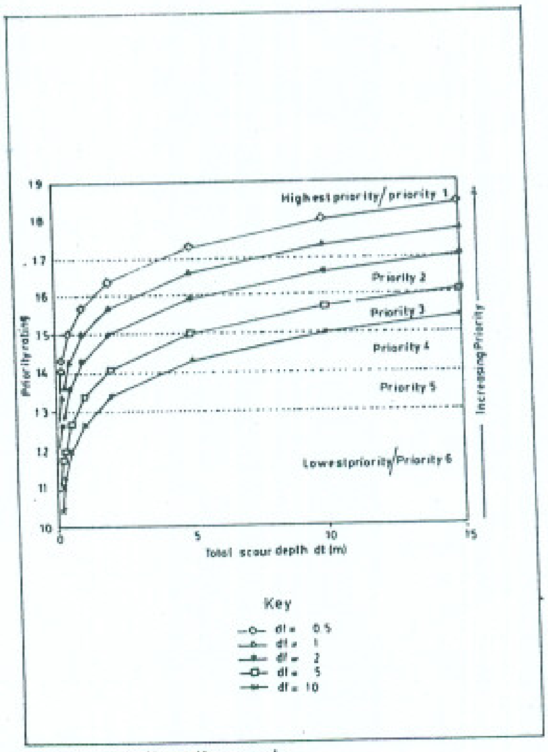

Fig. 10 Priority rating graph