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Vattamalaipalayam, N.G.G.O. Colony Post, Coimbatore – 641 022

PARTICIPATIVE LEARNING

PROFESSIONAL SUMMARY REPORT

PROFESSIONAL SUMMARY REPORT

WATER LEVEL INDICATOR USING LEVEL SWITCHES

Submitted by

KARTHIKA.K (1906013)

MADUMITHAA.K (1906019)

SHOBIKA.V (1906030)

in partial fulfillment of the course

16EI214 – INDUSTRIAL INSTRUMENTATION II

ELECTRONICS AND INSTRUMENTATION ENGINEERING

SRI RAMAKRISHNA ENGINEERING COLLEGE

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VATTAMALAIPALAYAM, N.G.G.O. COLONY POST, COIMBATORE – 641 022.

JUNE: 2022

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COIMBATORE – 641 022

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Certified that this Professional Summary Report is prepared and submitted by **"Karthika K.(1906013), Madumithaa K.(1906019), Shobika V.(1906030)"** during the period from **March 2022 to June 2022**.

COURSE CODE	**16EI214**
COURSE TITLE	**INDUSTRIAL INSTRUMENTATION II**
PRESENTATION TITLE	**WATER LEVEL INDICATOR USING LEVEL SWITCH**

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Submitted for the Oral Presentation held on 4. 6. 2022

4/6/22

INTERNAL EXAMINER

PROFESSIONAL SUMMARY REPORT

HELIX TYPE FLOWMETER

Submitted by

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16EI214 – INDUSTRIAL INSTRUMENTATION - II

ELECTRONICS AND INSTRUMENTATION ENGINEERING

SRI RAMAKRISHNA ENGINEERING COLLEGE
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COURSE TITLE	**INDUSTRIAL INSTRUMENTATION - II**
PRESENTATION TITLE	**HELIX TYPE FLOWMETER**

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INTERNAL EXAMINER

PROFESSIONAL SUMMARY REPORT

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Submitted by

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16EI214 – INDUSTRIAL INSTRUMENTATION II

ELECTRONICS AND INSTRUMENTATION ENGINEERING

SRI RAMAKRISHNA ENGINEERING COLLEGE
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COURSE CODE	**16EI214**
COURSE TITLE	**INDUSTRIAL INSTRUMENTATION - II**
PRESENTATION TITLE	**ELECTROMAGNETIC TYPE FLOWMETER**

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Submitted for the Oral Presentation held on 4.06.2022

4/6/22

INTERNAL EXAMINER

DESIGN OF FLEXIBLE PAVEMENTS

PROFESSIONAL SUMMARY REPORT

Submitted by

NAVEENA K G (1909025)

in partial fulfillment for the award of the degree

of

BACHELOR OF ENGINEERING

IN

CIVIL ENGINEERING

SRI RAMAKRISHNA ENGINEERING COLLEGE

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DECEMBER 2021

SRI RAMAKRISHNA ENGINEERING COLLEGE

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16CE216
HIGHWAY ENGINEERING
Design of flexible pavements

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INTERNAL EXAMINER

EXECUTIVE SUMMARY

Flexible pavements are so named because the total pavement structure deflects, or flexes, under loading. A flexible pavement structure is typically composed of several layers of materials. Each layer receives loads from the above layer, spreads them out, and passes on these loads to the next layer below. Thus the stresses will be reduced, which are maximum at the top layer and minimum on the top of subgrade. In order to take maximum advantage of this property, layers are usually arranged in the order of descending load bearing capacity with the highest load bearing capacity material (and most expensive) on the top and the lowest load bearing capacity material (and least expensive) on the bottom.

Indian roads congress has specified the design procedures for flexible pavements based on CBR values. The Pavement designs given in the previous edition IRC:37-1984 were applicable to design traffic upto only 30 million standard axles (msa). The earlier code is empirical in nature which has limitations regarding applicability and extrapolation. This guidelines follows analytical designs and developed new set of designs up to 150 msa in IRC:37-2001.

INTRODUCTION

Since the beginning of the 20th century, as the automobile and truck have offered ever higher levels of mobility, vehicle ownership per head of population has increased. Road needs have been strongly influenced by this popularity and also by the mass movement of people to cities and thence to suburban fringes—a trend that has led to increasing travel needs and road congestion and to low-density cities, which are difficult to service by public transport. Often the building of new roads to alleviate such problems has encouraged further urban sprawl and yet more road travel. Long-term solutions require the provision of alternatives to car and truck transport, controls over land use, and the proper pricing of road travel. To this end, road managers must be concerned not merely with lines on maps but also with the number, type, speed, and loading of individual vehicles, the safety, comfort, and convenience of the traveling public, and the health and welfare of bystanders and adjoining property owners.

Ideally, the development of a major road system is an orderly, continuous process. The process follows several steps: assessing road needs and transport options; planning a system to meet those needs; designing an economically, socially, and environmentally acceptable set of roads; obtaining the required approval and financing; building, operating, and maintaining the system; and providing for future extensions and reconstruction.

Pavement

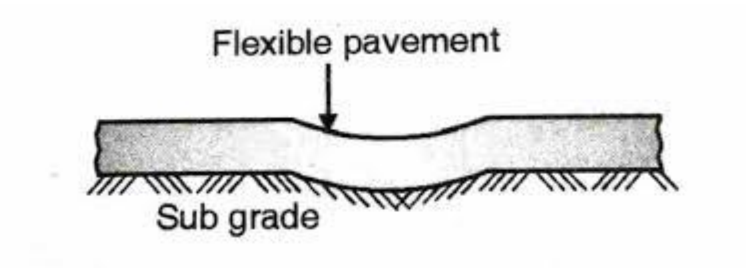
Road traffic is carried by the pavement, which in engineering terms is a horizontal structure supported by in situ natural material. In order to design this structure, existing records must be examined and subsurface explorations conducted. The engineering properties of the local rock and soil are established, particularly with respect to strength, stiffness, durability, susceptibility to moisture, and propensity to shrink and swell over time. The relevant properties are determined either by field tests (typically by measuring deflection under a loaded plate or the penetration of a rod), by empirical estimates based on the soil type, or by laboratory measurements. The material is tested in its weakest expected condition, usually at its highest probable moisture content. Probable performance under traffic is then determined. Soils unsuitable for the final pavement are identified for removal, suitable replacement materials are earmarked, the maximum slopes of embankments and cuttings are established, the degree of compaction to be achieved during construction is determined, and drainage needs are specified.

In a typical rural pavement (as shown in the figure), the top layer of the pavement is the wearing course. Made of compacted stone, asphalt, or concrete, the wearing course directly supports the vehicle, provides a surface of sufficient smoothness and traction, and protects the base course and natural formation from excessive amounts of water. The base course provides the required supplement to the strength, stiffness, and durability of the natural formation. Its thickness ranges from 4 inches (10 centimetres) for very light traffic and a good natural formation to more than 40 inches (100 centimetres) for heavy traffic and a poor natural formation. The subbase is a protective layer and temporary working platform sometimes placed between the base course and the natural formation.

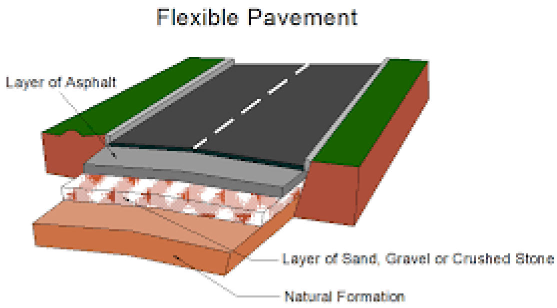
Pavements are called either flexible or rigid, according to their relative flexural stiffness.

Flexible pavement

Flexible pavements (see figure, left) have base courses of broken stone pieces either compacted into place in the style of McAdam or glued together with bitumen to form asphalt. In order to maintain workability, the stones are usually less than 1.5 inches in size and often less than 1 inch. Initially the bitumen must be heated to temperatures of 300°–400° F (150°–200° C) in order to make it fluid enough to mix with the stone. At the road site a paving machine places the hot mix in layers about twice the thickness of the stone size. The layers are then thoroughly rolled before the mix cools and solidifies. In order to avoid the expense of heating, increasing use has been made of bitumen emulsions or cutbacks, in which the bitumen binder is either treated with an emulsifier or thinned with a lighter petroleum fraction that evaporates after rolling. These treatments allow asphalts to be mixed and placed at ambient temperatures.



The surface course of a flexible pavement protects the underlying base course from traffic and water while also providing adequate tire friction, generating minimal noise in urban areas, and giving suitable light reflectance for night-time driving. Such surfaces are provided either by a bituminous film coated with stone (called a spray-and-chip seal) or by a thin asphalt layer. The spray-and-chip seal is used over McAdam-style base courses for light to moderate traffic volumes or to rehabilitate existing asphalt surfaces. It is relatively cheap, effective, and impermeable and lasts about 10 years. Its main disadvantage is its high noise generation. Maintenance usually involves further spray coating with a surface dressing of bitumen. Asphalt surfacing is used with higher traffic volumes or in urban areas. Surfacing asphalt commonly contains smaller and more wear-resistant stones than the base course and employs relatively more bitumen. It is better able to resist horizontal forces and produces less noise than a spray-and-chip seal.



IRC method

Indian roads congress has specified the design procedures for flexible pavements based on CBR values. The Pavement designs given in the previous edition IRC:37-1984 were applicable to design traffic upto only 30 million standard axles (msa). These guidelines will apply to design of flexible pavements for Expressway, National Highways, State Highways, Major District Roads, and other categories of roads. Flexible pavements are considered to include the pavements which have bituminous surfacing and granular base and sub-base courses conforming to IRC/ MOST standards. These guidelines apply to new pavements.

LITERATURE REVIEW

1. **Design of Flexible Pavement: A Case Study**
   A. V. Hankare, P. B. Bhujbal , A. B. Shinde, R. G. Wagh

Although several studies observed pavement responses after flooding, no detailed quantification has been done to date. This paper has estimated "Design of flexible pavement's" with the help of "IRC-37, 2012" This was shown in the traffic design v/s thickness of pavement, the study of Traffic volume, Traffic count, and calculates traffic design with the help of IRC-37, 2012. (Use IS code formulae), CBR of strong pavement built to a high standard is the most flood – resilient, which may be adopted as a pre-flood strategy. Results obtained using CBR in Year 1 over the probability of flooding, and the loss of road strength and service life, as well as flood consequences, provided similar results. Road authorities should consider changing their roads to flood-resilient pavements in the future.

2. **A Detailed Study of Cbr Method for Flexible Pavement Design**
   Er. Devendra Kumar Choudhary1 , Dr. Y. P Joshi 2
   As per IRC recommendation, California Bearing Ratio (CBR) value of subgrade is used for design of flexible pavements. California Bearing Ratio (CBR) value is an important soil parameter for design of flexible pavements and runway of air fields. It can also be used for determination of sub grade reaction of soil by using correlation. It is one of the most important engineering properties of soil for design of sub grade of roads. CBR value of soil may depends on many factors like maximum dry density (MDD), optimum moisture content (OMC), liquid limit (LL), plastic limit (PL), plasticity index (PI), type of soil, permeability of soil etc. Besides, soaked or unsoaked condition of soil also affects the value. These tests can easily be performed in the laboratory. the estimation of the CBR could be done on the basis of these tests which are quick to perform, less time consuming and cheap, then it will be easy to get the information about the strength of subgrade over the length of roads, By considering this aspect, a number of investigators in the past made their investigations in this field and designed different pavements by determining the CBR value on the basis of results of low cost, less time consuming and easy to perform tests. In this study, attempts have been made to seek the values of CBR of

different soil samples and correlate their CBR values for the design purpose of flexible pavement as per guidelines of IRC: SP: 37-2001.

3. **Design of Rigid and Flexible Pavements by Various Methods & Their Cost Analysis of Each Method**
   Saurabh Jain1 , Dr. Y. P. Joshi2 , S. S. Goliya3
   Highway and pavement design plays an important role in the DPR projects. The satisfactory performance of the pavement will result in higher savings in terms of vehicle operating costs and travel time, which has a bearing on the overall economic feasibility of the project. This paper discusses about the design methods that are traditionally being followed and examines the "Design of rigid and flexible pavements by various methods & their cost analysis by each method". Flexible pavement are preferred over cement concrete roads as they have a great advantage that these can be strengthened and improved in stages with the growth of traffic and also their surfaces can be milled and recycled for rehabilitation. The flexible pavements are less expensive also with regard to initial investment and maintenance. Although Rigid pavement is expensive but have less maintenance and having good design period. The economic part are carried out for the design pavement of a section by using the result obtain by design method and their corresponding component layer thickness. It can be done by drawing comparisons with the standard way and practical way. This total work includes collection of data analysis various flexible and rigid pavement designs and their estimation procedure are very much useful to engineer who deals with highways.

4. **A Comparative Study on Rigid and Flexible Pavement: A Review**
   Milind V. Mohod 1*, Dr. K.N.Kadam2
   : The last century has seen an intensive process of urbanization in rural as well as metro cities. This has led for a need of rapid construction of roads and transportation infrastructure. The demand for better roads and services required researchers, designers and builders to explore innovative and cost effective engineered products to satisfy increasing demand that would economize the construction as well as increase durability. Pavements are essential features of the urban communication system and provide an efficient means of transportation. Flexible pavements are preferred over cement concrete roads because of their certain advantages like they can be

strengthened and improved in stages with the growth of traffic. The flexible pavements are less expensive in regards to initial cost and maintenance. The concrete pavements are now a day's becoming more popular in India because of steep rise in the cost of bituminous pavement. The largest advantage of using rigid pavement is its durability and ability to hold a shape against traffic and difficult environmental conditions. Although concrete pavement is less expensive but has less maintenance and good design life. The main objective of this study is to present a comparative review on suitability of pavement depending on various parameters such as material, loading, longer life, cost effectiveness etc

5. **A DETAILED STUDY OF C.B.R. METHOD FOR FLEXIBLE PAVEMENT DESIGN**

Er. Jitendra Khatti[1], Er. Amit Kumar Jangid[2], Dr. K. S. Grover[3]

According to IRC recommendation, the California bearing ratio (CBR) value of subgrade is used for design of flexible pavements. The design of pavement may affect by the material which is used as pavement material. Black Cotton soil is expansive soil which expand when it contacts with water and this is the major reason of failure of black cotton soil strata. The engineering properties of black cotton soil may be used by fibre, ash, lime and sludge etc. CBR value depends on the liquid limit (Wl), Plastic limit (Wp), plasticity index (Ip), maximum dry density, optimum moisture content, shrinkage, swelling pressure, degree of expansiveness and permeability of soil or mix specimen. These tests are performed in laboratory of University Teaching Department, Rajasthan Technical University, Kota. This research paper deals with design of flexible pavement by using black cotton soil with different percentage of Kota stone slurry. In this research, the Kota stone slurry is mixed from 5% to 30% in black cotton soil. The engineering parameters are also determined by performed tests. For studying the behaviour of black cotton soil with different percentage of Kota stone slurry, the Atterberg's limits (Liquid Limit, Plastic Limit, Plasticity Index), Sieve analysis, standard proctor test, California Bearing Ratio are performed.

DESIGN PROCEDURE OF IRC FOR FLEXIBLE PAVEMENT

Based on the performance of existing designs and using analytical approach, simple design charts and a catalogue of pavement designs are added in the code. The pavement designs are given for subgrade CBR values ranging from 2% to 10% and design traffic ranging from 1 msa to 150 msa for an average annual pavement temperature of 35 C. The later thicknesses obtained from the analysis have been slightly modified to adapt the designs to stage construction. Using the following simple input parameters, appropriate designs could be chosen for the given traffic and soil strength:

Design traffic in terms of cumulative number of standard axles; and CBR value of subgrade.

Design criteria

The flexible pavements has been modeled as a three layer structure and stresses and strains at critical locations have been computed using the linear elastic model. To give proper consideration to the aspects of performance, the following three types of pavement distress resulting from repeated (cyclic) application of traffic loads are considered: 1. vertical compressive strain at the top of the sub-grade which can cause sub-grade deformation resulting in permanent deformation at the pavement surface. 2. horizontal tensile strain or stress at the bottom of the bituminous layer which can cause fracture of the bituminous layer. 3. pavement deformation within the bituminous layer. While the permanent deformation within the bituminous layer can be controlled by meeting the mix design requirements, thickness of granular and bituminous layers are selected using the analytical design approach so that strains at the critical points are within the allowable limits. For calculating tensile strains at the bottom of the bituminous layer, the stiffness of dense bituminous macadam (DBM) layer with 60/70 bitumen has been used in the analysis.

Failure criteria

Fig. 7.2.2 shows the flexible pavement failure in wearing course or seal coat.

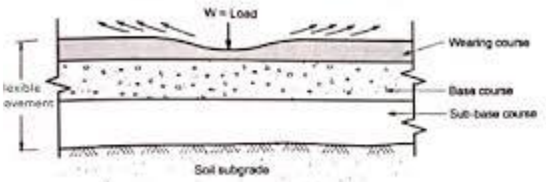


Fig. 7.2.2 : Wearing course or seal coat failure

Fig. 7.2.3 Shows the flexible pavement failure in base course

A and B are the critical locations for tensile strains . Maximum value of the strain is adopted for design. C is the critical location for the vertical subgrade strain since the maximum value of the occurs mostly at C.

Fatigue Criteria: Bituminous surfacings of pavements display flexural fatigue cracking if the tensile strain at the bottom of the bituminous layer is beyond certain limit. The relation between the fatigue life of the pavement and the tensile strain in the bottom of the bituminous layer was obtained as

$$N_f = 2.21 \times 10^{-4} \times \left(\frac{1}{\epsilon_t}\right)^{3.89} \times \left(\frac{1}{E}\right)^{0.854} \qquad (1)$$

in which, is the allowable number of load repetitions to control fatigue cracking and is the Elastic modulus of bituminous layer. The use of equation would result in fatigue cracking of 20% of the total area.

Rutting Criteria The allowable number of load repetitions to control permanent deformation can be expressed as

$$N_f = 2.21 \times 10^{-4} \times \left(\frac{1}{\epsilon_t}\right)^{3.89} \times \left(\frac{1}{E}\right)^{0.854} \qquad (2)$$

is the number of cumulative standard axles to produce rutting of 20 mm.

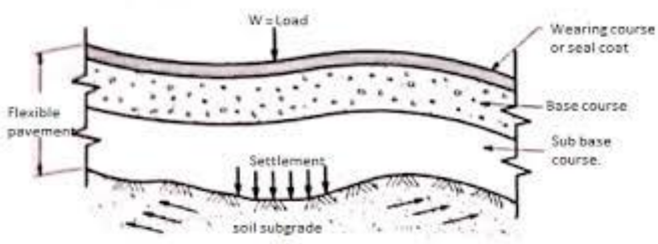


Fig. 7.2.1 : Subgrade failure of flexible pavement.

DESIGN PROCEDURE

Based on the performance of existing designs and using analytical approach, simple design charts and a catalogue of pavement designs are added in the code. The pavement designs are given for subgrade CBR values ranging from 2% to 10% and design traffic ranging from 1 msa to 150 msa for an average annual pavement temperature of 35 C. The later thicknesses obtained from the analysis have been slightly modified to adapt the designs to stage construction. Using the following simple input parameters, appropriate designs could be chosen for the given traffic and soil strength:

- Design traffic in terms of cumulative number of standard axles; and
- CBR value of subgrade.

DESIGN TRAFFIC

The method considers traffic in terms of the cumulative number of standard axles (8160 kg) to be carried by the pavement during the design life. This requires the following information:

1. Initial traffic in terms of CVPD
2. Traffic growth rate during the design life

3. Design life in number of years
4. Vehicle damage factor (VDF)
5. Distribution of commercial traffic over the carriage way.

Initial traffic

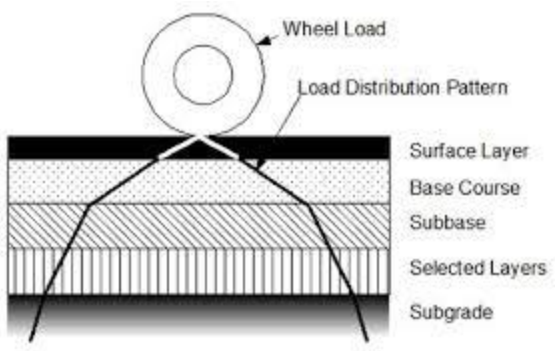
Initial traffic is determined in terms of commercial vehicles per day (CVPD). For the structural design of the pavement only commercial vehicles are considered assuming laden weight of three tonnes or more and their axle loading will be considered. Estimate of the initial daily average traffic flow for any road should normally be based on 7-day 24-hour classified traffic counts (ADT). In case of new roads, traffic estimates can be made on the basis of potential land use and traffic on existing routes in the area.

Traffic growth rate

Traffic growth rates can be estimated (i) by studying the past trends of traffic growth, and (ii) by establishing econometric models. If adequate data is not available, it is recommended that an average annual growth rate of 7.5 percent may be adopted.

Design life

For the purpose of the pavement design, the design life is defined in terms of the cumulative number of standard axles that can be carried before strengthening of the pavement is necessary. It is recommended that pavements for arterial roads like NH, SH should be designed for a life of 15 years, EH and urban roads for 20 years and other categories of roads for 10 to 15 years.



Vehicle Damage Factor

The vehicle damage factor (VDF) is a multiplier for converting the number of commercial vehicles of different axle loads and axle configurations to the number of standard axle-load repetitions. It is defined as equivalent number of standard axles per commercial vehicle. The VDF varies with the axle configuration, axle loading, terrain, type of road, and from region to region. The axle load equivalency factors are used to convert different axle load repetitions into equivalent standard axle load repetitions. For these equivalency factors refer IRC:37 2001. The exact VDF values are arrived after extensive field surveys.

Vehicle distribution

A realistic assessment of distribution of commercial traffic by direction and by lane is necessary as it directly affects the total equivalent standard axle load application used in the design. Until reliable data is available, the following distribution may be assumed.

- **Single lane roads:** Traffic tends to be more channelized on single roads than two lane roads and to allow for this concentration of wheel load repetitions, the design should be based on total number of commercial vehicles in both directions.
- **Two-lane single carriageway roads:** The design should be based on 75 % of the commercial vehicles in both directions.
- **Four-lane single carriageway roads:** The design should be based on 40 % of the total number of commercial vehicles in both directions.
- **Dual carriageway roads:** For the design of dual two-lane carriageway roads should be based on 75 % of the number of commercial vehicles in each direction. For dual three-lane carriageway and dual four-lane carriageway the distribution factor will be 60 % and 45 % respectively.

Design traffic

The design traffic is considered in terms of the cumulative number of standard axles in the lane carrying maximum traffic during the design life of the road. This can be computed using the following equation:

$$N = \frac{365 \times [(1+r)^n - 1]}{r} \times A \times D \times F \tag{3}$$

where is the cumulative number of standard axles to be catered for the design in terms of million standards axle (msa), is the initial traffic in the year of completion of construction in terms of the number of commercial vehicles per day, is the lane distribution factors, is the vehicle damage factor, is the design life in years, and is the annual growth rate of commercial vehicles (=- 0.075 if growth rate is 7.5 percent per annum). The traffic in the year of completion is estimated using the following formula:

$$N = \frac{365 \times [(1+r)^n - 1]}{r} \times A \times D \times F \tag{4}$$

where is the number of commercial vehicles as per last count, and x is the number of ears between the last count and the year of completion between the last count and the year of completion of the project.

Pavement thickness design charts

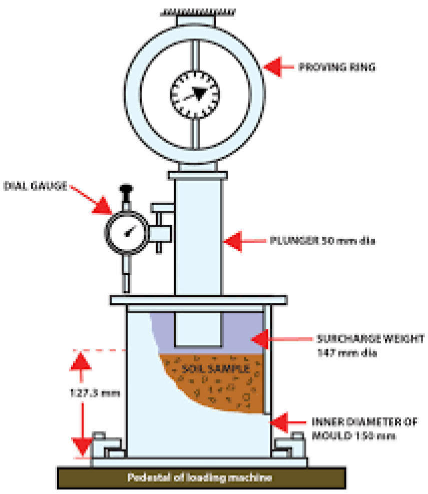
For the design of pavements to carry traffic in the range of 1 to 10 msa, use chart 1 and for traffic in the range 10 to 150 msa, use chart 2 of IRC:37 2001. The design curves relate pavement thickness to the cumulative number of standard axles to be carried over the design life for different sub-grade CBR values ranging from 2 % to 10 %. The design charts will give the total thickness of the pavement for the above inputs. The total thickness consists of granular sub-base, granular base and bituminous surfacing.

CBR Method (IRC: 37-1970 & 1984)

The California bearing ratio (CBR) value of the subgrade soil was the basis for the method of design of flexible pavements, developed originally by the California State Highway Department, and adopted by The Road Research Laboratory, London, for developing their own design procedure and design charts.

The advantage of the CBR method is that it can be used to find the total thickness of the pavement and that of the individual courses in addition to the thickness of the subgrade soil (provided the CBR-values of the materials of the courses are also known).

The thickness required could be got for known CBR-values from design curves for different wheel loads. The Indian Roads Congress has adopted this general procedure and developed design charts for the depth of construction versus the CBR-value for different traffic classifications based on the number of commercial vehicles per day.



The CBR-method is known to suffer from the following disadvantages:

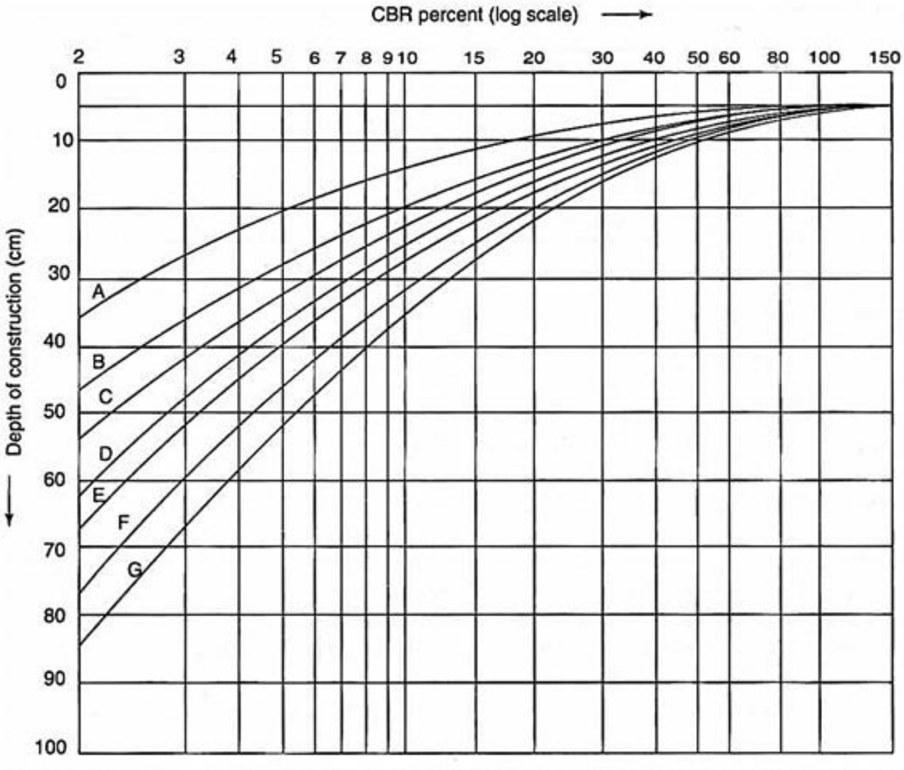
(i) It is an arbitrary test which does not reflect the soil strength directly.

(ii) The moisture and soaking conditions are also arbitrary, and in arid zones, over-design.

(iii) The curves are applicable only under the conditions for which they were method is not reliable for high values of CBR.

IRC Guidelines for Design of Flexible Pavements:

The thickness of construction is given by a set of 7 curves

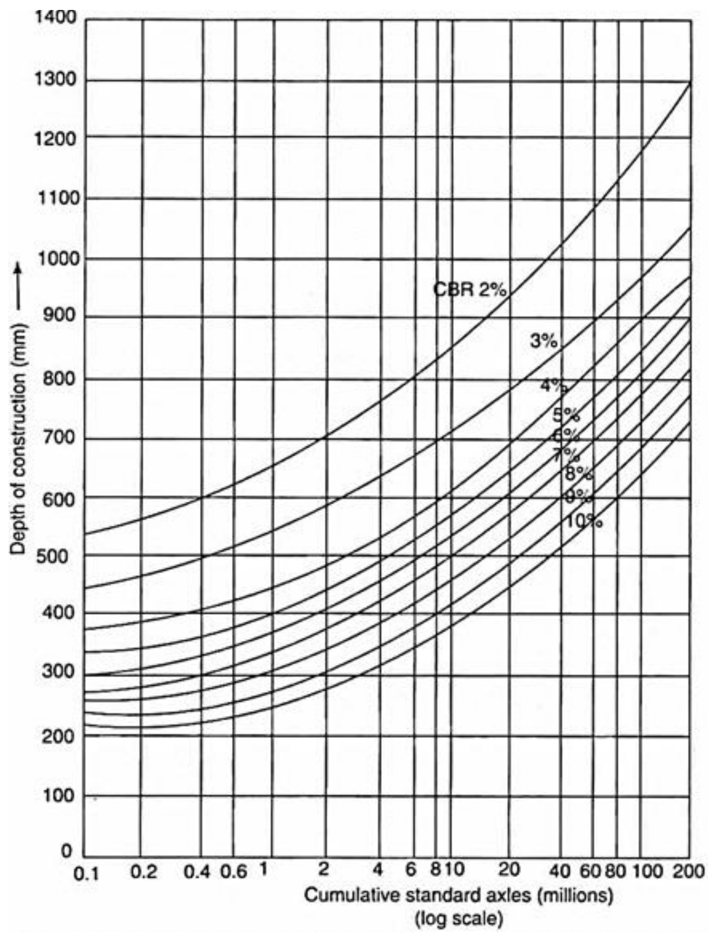


Traffic classification

Curve	No. of commercial vehicles per day (exceeding 3 tonnes weight)
A	0–15
B	15–45
C	45–150
D	150–450
E	450–1500
F	1500–4500
G	Above 4500

Traffic' denotes the total number of vehicles in both directions (irrespective of whether the design is for a two-lane or a dual carriageway.) For estimation of future traffic, the growth rate is assumed as 7.5 per cent.
For single-lane roads, the traffic intensity is taken to be twice that for two-lane roads (because of the concentration of traffic on one lane only).

IRC Guidelines-revised in 1984 [IRC: 37-1970-Revised (1984)]:
The salient features of IRC guidelines revised in 1984 are:
1. New set of design curves relating the cumulative standard axles and the CBR value to the total pavement thickness.
2. Recommendations on the types of pavement materials suitable for various courses.
The method of computing the cumulative standard axles; the number of standard axles per commercial vehicle is designated as 'vehicle damage factor (VDF)'.
The pavement design chart extrapolated up to 200 million standard axles



The total thickness obtained from the chart is distributed into surface course of thickness x, base of thickness y, and sub-base of thickness z

Table 7.3 Composition of pavement (IRC: 37–1984)

Cumulative standard axles (million)	Minimum thickness of course (mm)		
	surfacing (*x*)	base (*y*)	sub-base(z)
0·5	20 mm PC or 2 coats S.D	150	Total–50 Minimum thickness 100 mm on subgrade of CBR < 20%
0·5–2	20 mm PC or MS	225	T–225 Min. thickness 150 mm on subgrade of CBR <20%
2–5	20 mm PC/MS/SDC over 50 mm /75 mm BM	250	T–330/325 Min. thickness 750 mm on subgrade of CBR <30%
5–10	20 mm SDC/BC over 60–80 mm DBM	250	T–300 to 355 – do –
10–15	40 mm BC over 65–80 mm DBM	250	T–355 to 370 – do –
15–20	40 mm BC over 80–100 mm DBM	250	T–370 to 390 – do –
20–30	40 mm BC over 110–115 mm DBM	250	T–390 to 405 – do –

SD-Surface dressing
MS-Mix seal
BC-Bituminous concrete
DBM-Dense bituminous macadam
PC-Premix carpet
SDC-Semi-dense carpet
BM-Bituminous macadam
SDBC-Semi-dense Bituminous carpet

PAVEMENT COMPOSITION

Sub-base

Sub-base materials comprise natural sand, gravel, laterite, brick metal, crushed stone or combinations thereof meeting the prescribed grading and physical requirements. The sub-base material should have a minimum CBR of 20 % and 30 % for traffic upto 2 msa and traffic exceeding 2 msa respectively. Sub-base usually consist of granular or WBM and the thickness should not be less than 150 mm for design traffic less than 10 msa and 200 mm for design traffic of 1:0 msa and above.

Base

The recommended designs are for unbounded granular bases which comprise conventional water bound macadam (WBM) or wet mix macadam (WMM) or equivalent confirming to MOST specifications. The materials should be of good quality with minimum thickness of 225 mm for traffic up to 2 msa an 150 mm for traffic exceeding 2 msa.

Bituminous surfacing

The surfacing consists of a wearing course or a binder course plus wearing course. The most commonly used wearing courses are surface dressing, open graded premix carpet, mix seal surfacing, semi-dense bituminous concrete and bituminous concrete. For binder course, MOST specifies, it is desirable to use bituminous macadam (BM) for traffic upto o 5 msa and dense bituminous macadam (DBM) for traffic more than 5 msa.

SUMMARY

The design procedure given by IRC makes use of the CBR value, million standard axle concept, and vehicle damage factor. Traffic distribution along the lanes are taken into account. The design is meant for design traffic which is arrived at using a growth rate.

Flexible pavements are preferred over cement concrete roads because of their certain advantages like they can be strengthened and improved in stages with the growth of traffic. The flexible pavements are less expensive in regards to initial cost and maintenance. Advantages of flexible pavement are that it is easily renewable with a new top layer of asphalt, temperature variations don't stress flexible pavement as much as rigid pavement, and its flexibility means it can withstand a certain amount of wear and tear.

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GEOMETRIC DESIGN OF RAILWAY TRACKS

PROFESSIONAL SUMMARY REPORT

Submitted by

MOORTHY.P

REG NO:1909022

in partial fulfillment for the award of the degree

of

BACHELOR OF ENGINEERING

IN

CIVIL ENGINEERING

SRI RAMAKRISHNA ENGINEERING COLLEGE

[Educational Service: SNR Sons Charitable Trust]
[Autonomous Institution, Accredited by NAAC with 'A' Grade]
[Approved by AICTE and Permanently Affiliated to Anna University, Chennai]
[ISO 9001-2015 Certified and all eligible Programmes Accredited by NBA]
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MAY 2021

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BONAFIDE CERTIFICATE

Certified that this Professional Summary Report is prepared and submitted by **"MOORTHY.P"** during the period from **JANUARY 2021 to MAY 2021.**

16CE208
HDROLOGY AND HYDRAULICS ENGINEERING
Hydralic Jump

SIGNATURE

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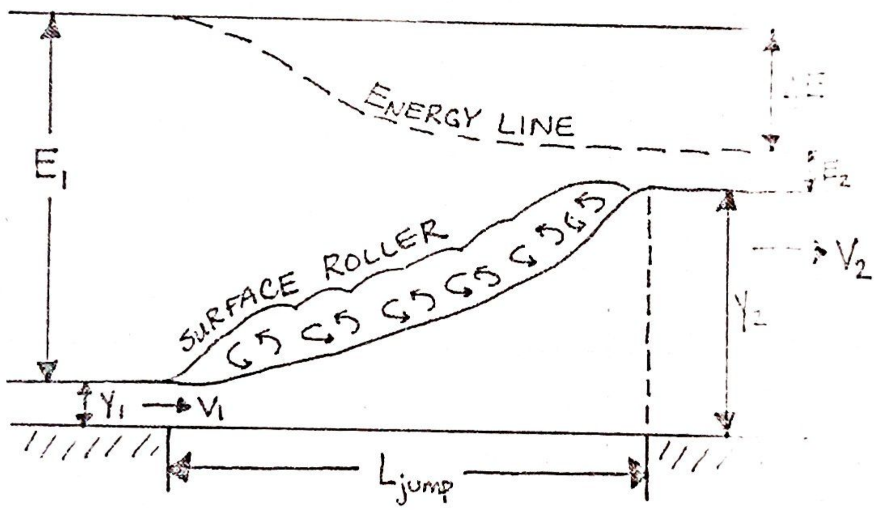
EXECUTIVE SUMMARY

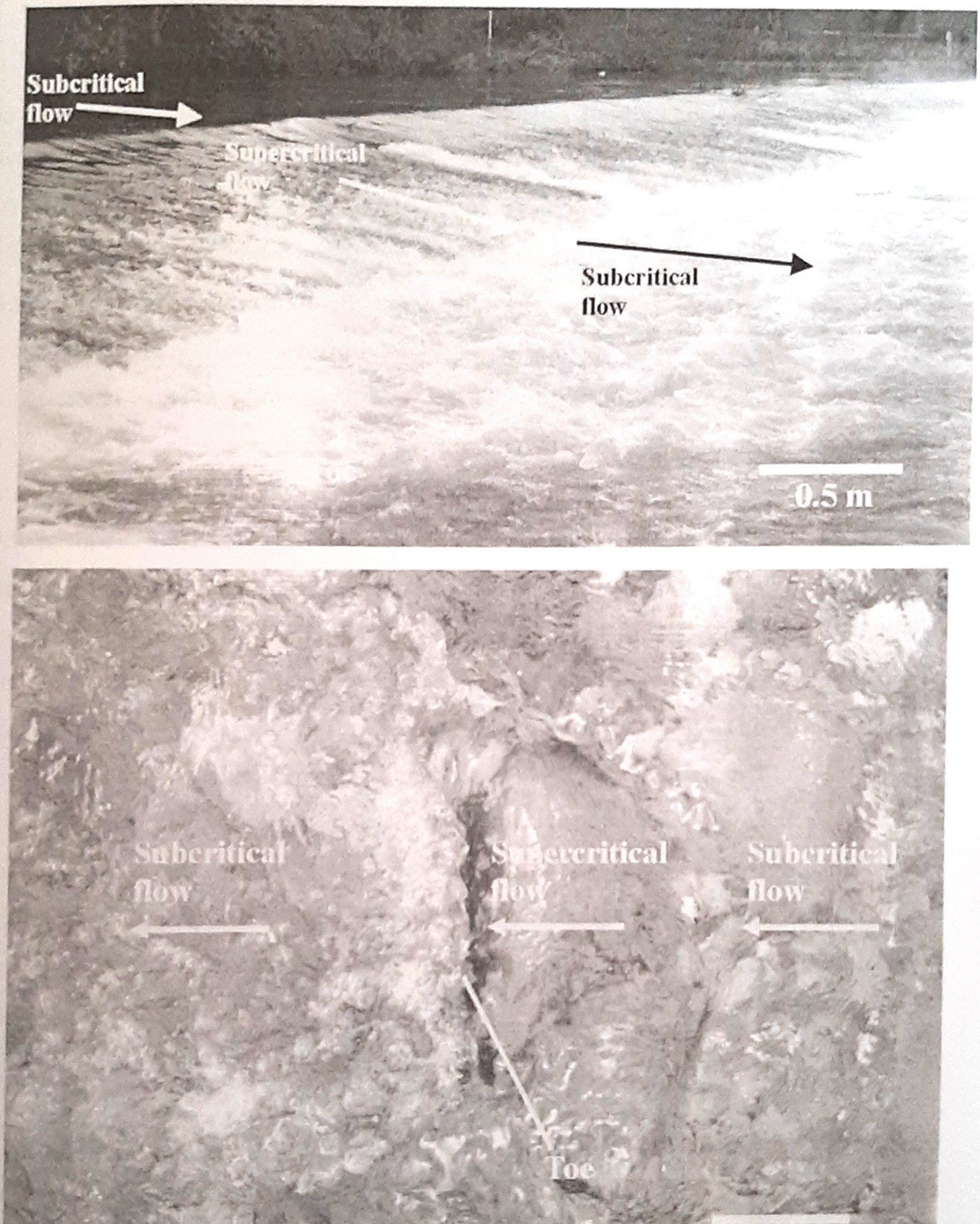
Hydraulic jump is a phenomenon which has received significant attention in recent years and it is still studied because of its capacity to dissipate a considerable amount of the flow energy. Nevertheless, the importance of the topic still requires significant efforts from the scientific community. Namely, the prediction of the main lengths of the hydraulic jump are still an open question, as the actual knowledge on the topic does not cover all the possible configurations and boundary conditions which can usually be found in practical applications. In particular, the effects of bed roughness, bed slope, channel geometry, and air concentration on the conjugate depths ratio are still not fully understood. The present paper aims to furnish a synthetic picture of the state of art regarding the hydraulic jump properties in a wide range of both boundary conditions and geometric configurations. In particular, the analysis will be focused on the effect of both relative roughness and bed slope on the conjugate depth ratio, including the effect of air entrainment on the estimation of the effective depth. Furthermore, some predicting relationships proposed by different authors will be compared and discussed. Hydraulic jumps were formed downstream of a bed-attached hemisphere, acting as a boulder on a stream bed. Flow velocities and water surface shapes were recorded. Vorticity patterns were inferred from observations and theory. Sediment transport patterns were observed in runs with selected pumping rates, each with water surface patterns involving discrete forms of hydraulic jump. Small amounts of sediment were used, and the starved bed load features that formed are compared to published bed load features with a hemisphere but no hydraulic jump

INTRODUCTION

A hydraulic jump is a wave with a stationary mean position and an increase in flow depth from upstream to downstream . As flow passes through a hydraulic jump a lot of kinetic energy is transformed to gravitational potential energy. heat and sound You will be able to hear most hydraulic jumps in rivers when walking past them along the riverbank. Hydraulic jumps are also eye catching in rivers because they entrain air into the flow via a complex system of breaking at their surface. causing the flow to appear white. They add to the excitement of white water rafting and canoeing but can be dangerous to recreational users of rivers. People have been dragged under hydraulic jumps and drowned.



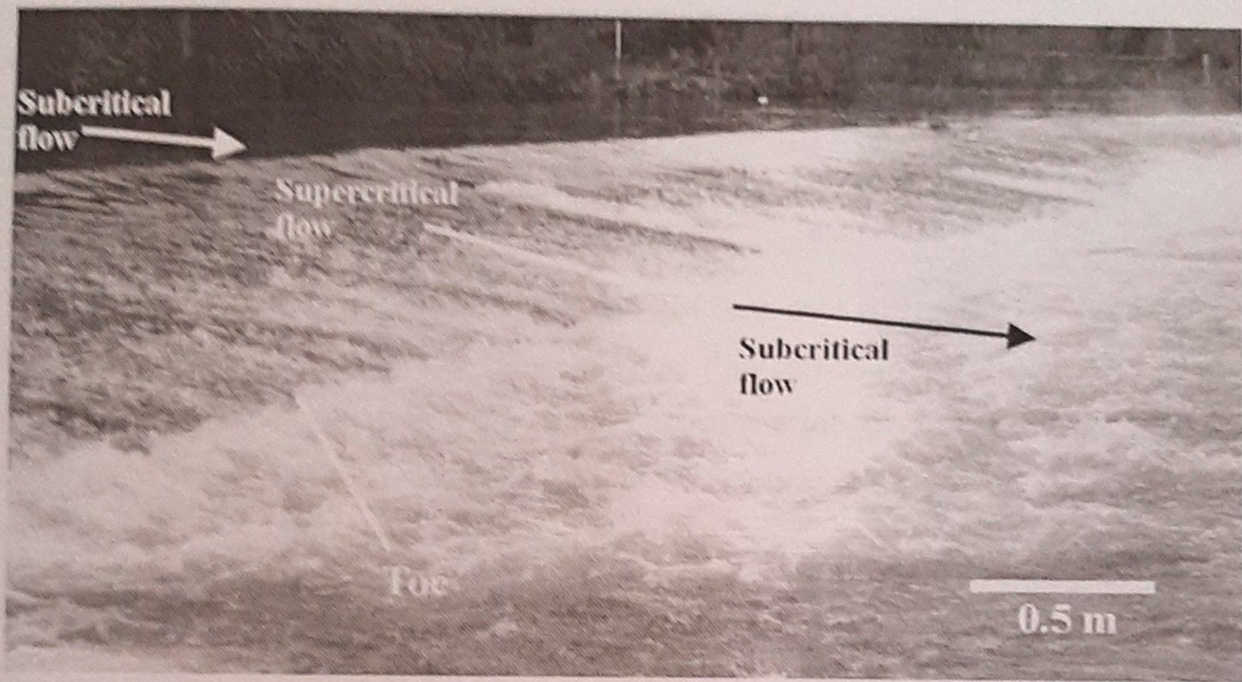
As water flows from upstream to downstream of any hydraulic jump, some of its kinetic energy (including turbulent kinetic energy) is transformed to gravitational potential energy, sound and heat. Where air or sediment is present within the flow, some of the kinetic energy of the water may be transformed to kinetic or gravitational potential energy of the air or sediment. Some of the kinetic energy of the sediment load may be transformed to kinetic energy of the water.

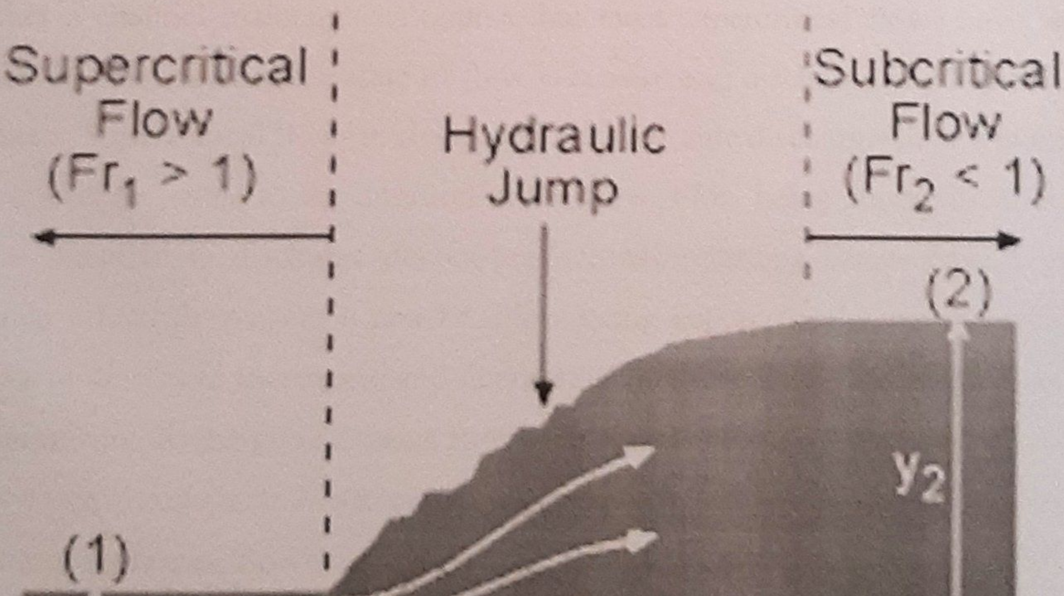

Subcritical flow
Supercritical flow
Subcritical flow
0.5 m
Subcritical flow
Supercritical flow
Subcritical flow
Toe
0.1 m

In most respects supercritical flows in open channels are alike the supercritical flows over smooth floors in theory . Regardless of the characteristics of the floor of a channel, all supercritical flows I observed in British rivers and read about in other settings had a hydraulic jump associated with them. Most supercritical flows were thinner than the water downstream of the hydraulic jump that was associated with it. The value of h in most supercritical flows generally decreased from up slope to down slope. All supercritical flows that I observed were less than 0.3 m thick and most were less than 0.2 m thick. In all supercritical flows I observed, coherent flow structures were too small to see. Bursts and sweeps must have occurred . but at too small a scale to see. Where supercritical flow passed over a smooth undulation on a bedrock floor the water surface height rose over it. Some characteristics of supercritical flow are not predicted by theory. I observed the hydraulic jumps in the modern British rivers and read all the literature to assess the spatial prevalence of supercritical flows and hydraulic jumps in modern natural open channel flows. Assess how long individual stretches of supercritical flow persist for in modern open channel flows. These two assessments allowed me to optimise open channel 16 environments or paleo environments in which to look for hydraulic jump deposits . Assess the range of shapes of supercritical flows in modern natural open channels. This allowed me to design experimental supercritical flows with shape characteristics that supercritical flows in ancient rivers might have had.

supercritical open channel flows:

Parts of river flows can become supercritical because the flow accelerates under gravity, when passing over smooth sloping planar floors and when passing over the downstream face of large roughness elements. Flows can become supercritical because they are made narrower by passing through a constriction. Flow can be constricted from many sides when passing into a slot. Flow can be constricted from above where undershot sluice gates are present in channal, from below by levees and from the sides by emergent obstacles or downstream-narrowing valley walls. In the flume I used for the experiments I chose a high pumping rate to discharge water supercritically through a restricted pipe so that flow entered the open test channel supercritically





Spatial prevalence of supercritical flows in rivers:

Supercritical flow can be absent from large stretches of most rivers because friction, drag and other retarding forces are equal to or greater than gravitational acceleration of the flow. However supercritical flow can be common in space. Individual stretches of supercritical flow can be at least tens of meters long and wide (e.g. immediately downstream of abrupt lateral expansions. Grant et al. (1990) assessed that supercritical flows and hydraulic jumps constitute up to 10% of flow over riffles and 15 - 50% of flow in rapids (as typical ranges of values). Where flow travels over chutes and pools and cyclic steps, supercritical flows, hydraulic jumps and subcritical flows occur in a repeating stream wise sequence. Cyclic steps on steep hill slopes frequently repeat on the order of tens of centimeters to tens of meters.

Time persistence of supercritical flows in rivers:

Supercritical flows in rivers have a maximum possible lifetime equal to the length of time that a channel maintains its course, but most supercritical flows have shorter lifetimes because the Froude-state of flow is sensitive to discharge and local channel conditions. Supercritical flows in rivers with very variable discharge have a maximum possible lifetime equal to the duration of the flow. Flow being supercritical down a slope is sensitive to discharge decreasing because retarding forces have a greater combined effect, decreasing u and Fr. Flow being supercritical over an obstacle is sensitive to discharge increasing and decreasing. A decreasing flow stage associated with decreasing discharge can cause flow to cease travelling over an obstacle (where thin flow over the obstacle was supercritical) but continue to travel around the obstacle. If discharge increases, flow directly over the obstacle generally increases in thickness; decreasing Fr. Discharge varies constantly in all natural rivers. It is more likely that more hydraulic jumps will have shorter lifetimes the more variable discharge is

Because changing discharge, sediment supply and vegetation can change channel boundary conditions over time, they have a long term effect on the potential for flow to become supercritical. Large flows can change the pattern of channel floor obstacles by eroding and depositing sediment bed topography. Because the sediment transport pattern over chute and pools and cyclic steps varies with time it can influence the character of supercritical flow and even cause it to switch on or shut down. In my flume experiments I chose to set initial conditions (at time, $t \leq 0$) involving constant discharge and constant channel floor characteristics (involving no sediment transport) so that the hydraulic jump would persist in character before sediment was supplied to it. Having a steady system at the start of a run involving sediment made the effect of the supercritical flow and hydraulic jump on the sediment transport patterns easy to isolate.

The shape of supercritical flows in river:

Having designed the mechanisms by which supercritical flows would form in the experiments and designed that the supercritical flows would persist for a long time before sediment was allowed to influence the flow, I wanted to design the supercritical flows to have shapes similar to those that occur in modern rivers. I present a range of shape characteristics of supercritical flows that I have seen, or were presented by other authors, in bold italic type. My choices for further study are discussed in plain type.

Supercritical flows in rivers do not generally span the full width of the channel:

. In general the characteristics of river channel floors are not uniform across stream and some parts of channel floors are more prone to flow travelling supercritically over them than others are, for given ranges of discharge. I designed one experiment such that supercritical flows did not span the width of a flume in most runs ; a different setup to all experimental supercritical flows in flumes (or pipes) that I have come across in literature. The full width of natural narrow channels or subchannels can be supercritical (e.g. in slots or tunnels and at knickpoints in resistant bedrock) and I selected for supercritical flows to span the width of a flume in one experiment.

Supercritical flows above smooth channel floor obstacles are themselves smooth:

Supercritical flow generally changes in thickness along smooth slopes. As predicted by the physical reason, supercritical flow generally becomes thinner down smooth slopes. This was designed into the experiments. Where two smooth slopes make an abrupt contact with each other, supercritical flow can be thicker at the contact and thinner immediately upstream of the contact. Because an abrupt contact occurred between the smooth slopes of the obstacle and the flume floor in the experiment , this characteristic was permitted in that experiment. It was not permitted in the initial condition of the experiment .because there was only one slope (slope of the flume floor). In general the more irregular the boundary conditions, the more complicated the water surface shape of a supercritical flow. Supercritical flows over smooth channel floors tend to have smooth upper surfaces and supercritical flows over flat smooth floors tend to be cylindrical . Surface breaking of supercritical flow can occur during passage over bed roughness elements that are large compared to the depth of flow. Flow over densely packed large roughness elements invariably causes the water surface to have an intricate shape. Water surface breaking causes dissipation of the energy of supercritical flows and cause to break down. I decided to integrate a non-breaking surface into the supercritical flow in the experiments so that energy transformations might occur predictably. Supercritical flow passing through a constriction can be thicker than subcritical flow downstream of the constriction. Where a channel has a lateral constriction, flow accelerates through the constriction (it must do to conserve the volume flux of water). If flow is subcritical upstream of the constriction, it can be transformed to supercritical through the constriction. Supercritical flow may be thinner, the same thickness or thicker than the subcritical flow upstream of it depending on the values of the Froude number upstream of the constriction. FrA 2 and the ratio of channel width upstream of and through the constriction. B1/B2.

DOCUMENTATION OF HYDRAULIC JUMPS:

I documented hydraulic jumps in more than thirty British rivers and read reports of observations of hydraulic jumps and data recorded from hydraulic jumps, by other investigators of open channel flows. I assessed the ranges of characteristics of flow through hydraulic jumps and the general prevalence of each characteristic. With the range of documented hydraulic jumps in open channels to hand I selected characteristics to reproduce in experiments. Some characteristics of hydraulic jumps are alike in open channels and in theory. Most hydraulic jumps have one supercritical flow associated with them. Most do not occur periodically. Most hydraulic jumps generally thickened between their upstream limit and their downstream limit. I observed flows at the downstream limit of hydraulic jumps that were several centimetres thick, up to more than a metre thick. In general. hydraulic jumps associated with cylindrical supercritical flows were themselves cylindrical . The hydraulic jumps and the flow immediately downstream of them contained coherent turbulent structures, some of which were large compared to the flow depth and larger than the depth of the associated supercritical flow. Long et al. (1991) recorded video footage of hydraulic jumps in a flume with Fr1 = 4 – 9 and h2 = 0.3 m. Vortices formed approximately 0.2 m in diameter and had a cross stream axis of rotation. The height of the water surface of both hydraulic jump and tailwater was locally increased by the presence of vortices close to the water surface. As was the case for supercritical flows, the spatial prevalence, time persistence and range of shapes of hydraulic jumps are not predicted by theory. I observed the hydraulic jumps in the modern British rivers and read literature on hydraulic jumps to assess these three attributes of hydraulic jumps in modern natural open channel flows. These assessments allowed me to design experimental hydraulic jumps with shape characteristics that hydraulic jumps in ancient open channel flows might have had. The assessments also helped me to optimise open channel environments or paleo environments in which to look for hydraulic jump deposits .

Position of formation of hydraulic jumps:

All hydraulic jumps formed at positions that were associated with one or more topographic controls and the position varied with the Froude number of the supercritical flow (hence varied with discharge). All hydraulic jumps were seen in regions of significant flow deceleration (e.g. at a downstream transition from smooth to rough channel floor, at a break in slope gradient, at a position where flow abruptly widened from upstream to downstream or where an end wall of a flume, or an overshot weir slowed the flow). The mean position of a lot of hydraulic jumps that I watched did not change by more than the excursion of the oscillation of the toe because discharge did not vary significantly. In general though hydraulic jumps do move downstream with significant increases in discharge. For instance Kieffer (1985), Valle and Pasternack (2002) and Wyrick and Pasternack (2008) reported hydraulic jumps moving downstream by as much as several metres when flow increased above the mean annual flows of the Colorado river and the American river, California, USA.

Spatial prevalence of hydraulic jumps in rivers:

Hydraulic jumps can be absent from large stretches of most rivers because flow does not become supercritical. Hydraulic jumps are common in space where supercritical flows are common in space. Hydraulic jumps can also be common in space at the downstream limit of a single stretch of supercritical flow that passes downstream into an abrupt channel expansion. The downstream limit of the supercritical flow, in the wider part of the channel, often has an arc shape in plan view with the apex of the arc pointing downstream and many hydraulic jumps, each with toes that are slightly curved in plan view, form around the arc. The hydraulic jumps are distinct from each other because their fronts are slightly offset upstream or downstream from each other. In general hydraulic jumps that I observed had a smaller plan area than the supercritical flows I observed. Downstream of the abrupt channel expansions where one supercritical flow produced many hydraulic jumps, the hydraulic jumps had a smaller combined plan area than the supercritical flow. I saw hydraulic jumps that were several centimetres long, up to several metres long. In my experiments the hydraulic jumps spanned a smaller plan area than the supercritical flow

. Experimental open channel flume:

Laboratory experiments are carried out on a rectangular open channel flume 5 m long (Armfield Model No. C4-MKII-5.0-11) and a basic hydraulic bench (Armfield Model No. F1-10-A) as shown in figure1. The open channel width is 10 cm, whereas the maximum water depth can reach about 30 cm. The transparent side walls of channel are made of acrylic glass. whereas the bottom is made of stainless steel. The flume is provided with a digital flow meter for discharge measurement. Also, a point gauge is implemented for measuring water depth. An over-shot weir is installed at the end of the channel to control water depth while a sluice gate is installed in the open channel to form the hydraulic jump. The basic hydraulic bench includes a submersible pump that supplies the flume with the required discharge controlled by a valve. The water flows through a connected pipe beneath the flume where flow returns back to the hydraulic bench in a closed cycle flow. Several experimental runs are performed by varying the discharge and measuring the corresponding conjugate depths. Accordingly. Froude number can be indicated for both sub-critical and super-critical flows. Also, the length of jump can be measured by a calibrated scale on the top of channel. Moreover, the location of hydraulic jump can be determined by varying the gate openings. The former analytical models are simulated and compared with the experimental results in order to overcome the improper defects in such models and develop a new mathematical model that maintains more accurate outcomes. Table 1 shows the former equations regarding the relation between conjugate depths and flow discharge, energy dissipated in the hydraulic jump and the influence of upstream conjugate depth on Froude number for different velocities.

Super-critical flow and Froude number simulation:

Froude number is associated with the change of water depth and discharge in open channels. Figure 2 shows the simulation results of downstream conjugate depth (Y2) by varying upstream conjugate depth (Y1) and flowing discharge (Q) in the conjugate depth equation mentioned in table 1. Figure 2a, illustrates that the upstream conjugate depth decreases as the downstream conjugate depth increases at constant discharge, whereas an increment in the downstream depth is obtained as discharge increases at constant upstream depth. Figure 2a also indicates that Y2 reaches about 0.1m at Y1 of 0.01m and specific discharge (q) of 0.01m3/sec/m. Accordingly, figure 2b indicates that the Froude number at the upstream depth boosts as upstream depth declines that can be attributed to the increase in flow velocity. On the other hand, an enhancement in the upstream Froude number is achieved by increasing the flow discharge at constant upstream depth as shown in figure 2b. Moreover, a significant increase in Froude number is recognized as a result of a minor decrement in the upstream depth compared to the change in flow discharge that can lead to bed scouring.

A hydraulic jump experiment was accomplished in a rectangular open channel flume under the influence of various flow structures. Several experimental runs were accomplished to obtain Y2 by measuring Y1 and Q using different flow structures and various gate openings. The measured values of Y2 disagree with the simulated results of former equations. Also, the measured Y2 varies by varying the flow structure for the same Y1 and Q that reveals that the former equations of hydraulic jump should be modified to show the impact of shear force due to friction between hydraulic jump and bed of water canals. The change in downstream depth is relevant to the existence of shear force resistance obtained by the flow structure in water canals that can lead to bed scouring. Regarding open channel flume. the impact of shear force may owe to the friction between hydraulic jump flow and wall sides. Also, the experimental runs revealed that upstream Froude number increases from 1.4 to 2.6 by increasing downstream depth from 5 cm to 6 cm at constant discharge and gate opening where the jump returns back to the gate owing to the decline in specific force vena contraction.

Hydraulic jump properties on both smooth and rough beds in sloping and adverse channels:

HYDRAULIC JUMP ON HORIZONTAL BEDS:

The theoretical analysis of the hydraulic jump was conducted by Bélanger, who furni-shed the well-known Eq. (1) to estimate the conjugate depth ratio Y = y2/y1, where F1 is the Froude number at the upstream section of the hydraulic jump and λ = 8: 2105. The previous equation is valid under the following hypotheses: horizontal rectangu-lar channel; uniform velocity distribution; hydrostatic pressure distribution; and negli-gence of boundary flow resistance. sketch of a hydraulic jump on a smooth horizontal bed. It is evident that such conditions are very unusual in practical applications. Furthermore, successive studies showed that some corrections to the Bélanger`s equations were necessary to predict Y, especially for high F1, for which Eq. (1) systematically over-estimates the sequent depth ratio. In particular, Govinda Rao and Ramaprasad [1966] proved that λ

Successive studies involved bed roughness effect. The de□nition of the effec-tive bed level (ET), from which the water depths are measured, is of relevant importance. According to Hughes and Flack [1984], ET can be set at 0.2d65 below the average level of the physical tops of the bed material (PT). in the case of spatial uniformity of bed roughness. Rajaratnam [1966] and Ead and Rajaratnam [2002] showed that the bed roughness represents one of the most influencing parameters, deeply modifying the dissi-pative mechanism and the characteristic lengths of the hydraulic jump. In other words, the presence of corrugated/rough bed increases shear stresses resulting in a significant modi-fication of velocity profiles and in a reduction of the variable Y. Therefore, Rajaratnam [1966] analyzed the hydraulic jump occurring on horizontal rough beds, applying the general expression of the momentum equation.

$$P1 + M1 + W\sin\alpha = P2 + M2 + F\tau$$

where: P1 and P2 – the hydrostatic forces, M1 and M2 – momentum fluxes, $F\tau$ – the integrated shear stress per unit width, in which the subscripts 1 and 2 repre-sent the upstream and downstream hydraulic jump sections, respectively, W – the weight of the control volume, α – the angle of the bed slope respect to horizontal (i.e., $\alpha = 0$ for horizontal beds, $\alpha < 0$ for adverse-sloped beds and $\alpha > 0$ for sloping beds). He proposed that the integrated shear stress $F\tau$ can be assumed equal to $\varepsilon P1$, where ε is the non-dimensional shear force coefficient. Following a similar approach, Ead and Rajaratnam [2002] showed that $F\tau$ can be also expressed as function of M1, resulting in $F\tau = \varepsilon 1M1$. In both cases, the effect of shear stresses only depends on the flow conditions at the upstream section of the hydraulic jump. This assumption allowed derivation of a simple analytical expression for the sequent depth Y, as follows.

Where, for both smooth and rough beds, µF F=–+016080 1121.. (3b)and ε = 2ε1F12. It is worth noting that Ead and Rajaratnam [2002] estimated the integrated bed shear stress for both smooth and rough beds, showing that, in the tested range of parameters, the local friction coeficient cf was either equal to 0.011 or 0.069, respectively. The large difference of cf values coupled with the experimental validation of Eq. (3a) for both smooth and rough beds states that much larger Reynolds shear stresses should occur in the presence of rough beds, involving a significant reduction of the sequent depth ratio for a constant inflow Froude number (F1). The same results were also obtained by other researchers, who followed different approaches, including the estimation of the average boundary shear stress over the wetted perimeter of the jump.

where δ depends on the vertical velocity distribution, the longitudinal flux of turbulent momentum, pressure distribution and the channel bed boundary conditions. Nevertheless, Eq. (4), as theoretically derived by Leutheusser and Kartha [1972], presents some difficulties in practical applications, as Hughes and Flack [1984] noted that a quantitative assessment of δ is not so easy to be done in most of the cases. Therefore, Hughes and Flack [1984] and Pagliara et al. [2008a] proposed a simplification of the mentioned approach. In particular, Pagliara et al. [2008a] analysed the conjugate depth ratio reduction on rough beds made of both uniform and non-uniform materials. They proposed the following equation to estimate the correction coefficient δ:

THE FLUMES USED IN THE EXPERIMENTS:

The small flume used for trial experiments A small flume that was used for trial experiments is called the sediment transport demonstration flume by the manufacturers and is referred to in this thesis as the demonstration flume . It recirculates water but not sediment. The test channel is straight, is 0.077 m wide and 1.5 m long. It has a flat stainless steel floor and colourless vertical Perspex sidewalls that are 0.11 m high. At the downstream end of the test channel, water passes over a fixed sharp-crested overshot weir . The weir is 0.060 m wide and 0.050 m high. Passing over the weir, water falls into a holding tank. Water in the holding tank is drawn by a pump, through a recirculation pipe, back into the test channel. A filter on the entry to the recirculation pipe prevents the recirculation of sediment. Discharge in the recirculation pipe is adjusted by a tap valve. Pumping rate is user-selected 39 from three discrete factory default pumping rates. The volume of water in the flume can also be varied. The slope of the test channel is adjustable between 0 – 5.7° using a jacking screw. Sediment can be put directly into the test channel and removed from the test channel or holding tank by scooping. Squares of 0.005 m are drawn on the outside of the right hand sidewall between x = 0.525 – 1.025 m, between the bottom and the top of the sidewall. The set of squares provides a scale for video recordings and photographs. It was beneficial to use the demonstration flume to trial experiments because it can be cleaned of sediment and re-set up in hours whereas it took approximately three days to clean out sediment put into the flume used for the main experiments and set it up for a new experimental run.

Coordinate systems and dimensions in the flumes:

When I describe the left or right-hand side of a channel it is when looking down-channel. The coordinate system used in both flumes is Cartesian with x, y and z denoting the streamwise, cross-stream and floor-perpendicular dimensions. The origin of the coordinate system (x = y = z = 0) is the position of the upstream limit of the test channel, at the left hand sidewall, on the flume floor (Fig. 2.2A). The direction x is positive downstream, y is positive towards the right hand sidewall and z is positive upwards perpendicular to the flume floor. The line y = 0.5 is the channel centreline in plan view. In 3d view y = 0.5 is the centre plane but in both views I term it the centreline in the thesis. The streamwise, cross-stream and vertical components of velocity are u, v and w, where u is positive to downstream, v is positive towards the right and w is positive upwards perpendicular to the flume floor. Flow thickness is measured perpendicular to the flume floor and given the nomenclature h. The water surface height, H = h except over an obstacle or a deposit where H > h. Time, t has the value 0 at the start of an experimental run. Downstream-dipping slopes are positive whilst upstream-dipping slopes are negative.

Reynolds number is constant in the test channels:

Treating a test channel as being of unit width (unit of 1 m in the flume). Equation 1.7 becomes μ ρQ Re = (2.1) within the test channel. Discharge. Q is as constant as the pumping rate in a recirculating system. Taking density and viscosity to be constants, the value of Re is equal when comparing any cross-section of the flume test channel. Since Re is a measure of the turbulent character of a flow (§1.4.4) this fact made the turbulent character of the experimental flows easy to control. If the rest of the loop does not have unit width, Equation 2.1 will not be valid there and the value of Re will vary around the loop.

Types of Hydraulic Jumps – Based on Froude's Number:

Basically a hydraulic jump occurs in many types depending on topographical features and bed surface roughness and many other natural interface relations. This hydraulic jump types can be probably expressed based on Froude's number.

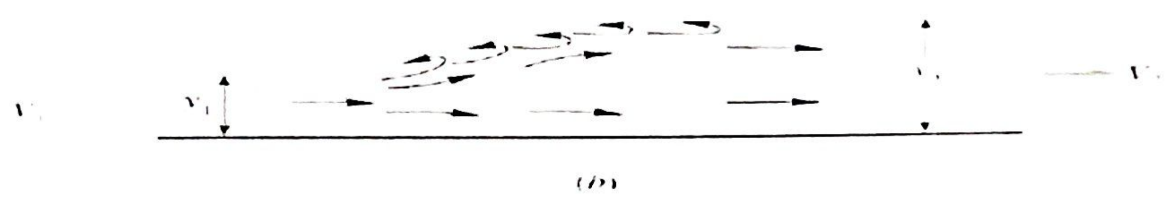
1. Undular Hydraulic Jump – Froude Number (1 to 3):

Undular Jump is irregular, not properly formed and there are certain turbulences in water particles.

(a)

2. Weak Jump – Froude Number (3 to 6) :

Weak jump takes place when the velocity in water is very less and the water particles cannot be stable and flows in various ways.



3. Oscillating Hydraulic Jump – Froude Number (6-20) :

Oscillating jump forms when an oscillating jet enter into super critical state and there the number of particles starts oscillating in clockwise. or either anticlockwise direction, forming slighter tides or waves to the top surface. Also the flow is dependent on heavy blow of air in one direction.

4. Steady Hydraulic Jump – Froude Number (20 to 80):

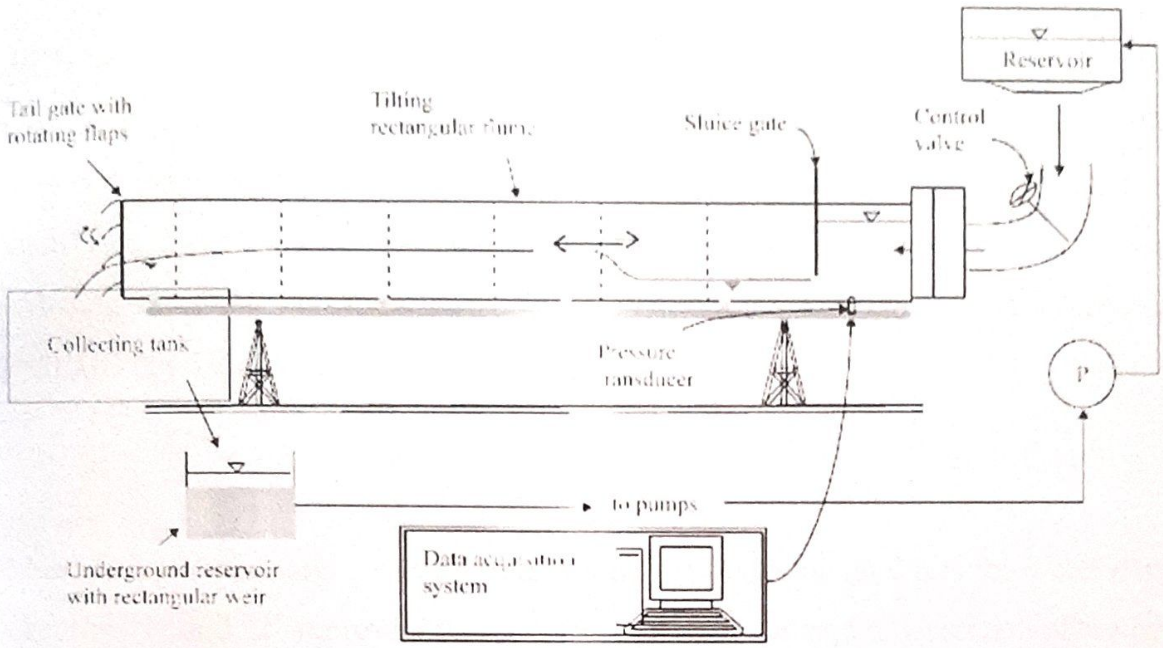
In steady jump, the bed surface is quite rough so the particles start to tend in one direction with heavy velocity and turbulence, frictional losses are more in this type of jump

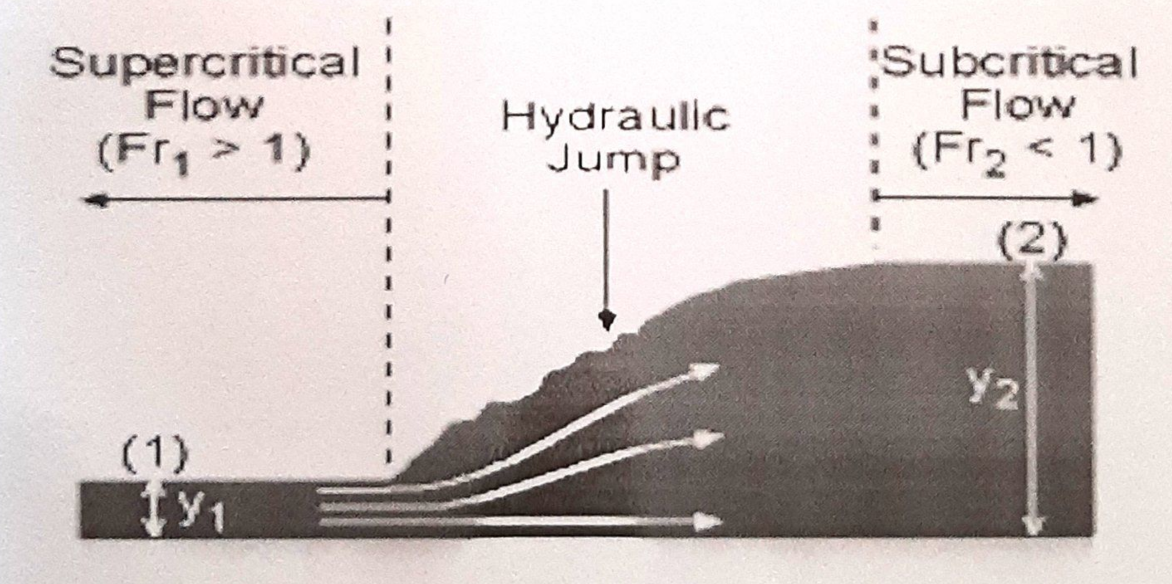
5. Strong Hydraulic Jump – Froude Number (greater than 80):

Strong jump is a perfect jump formed when frictional losses are more, air pressure division is equal and velocity is very high that losses take place. The water changes its state from super critical to subcritical in very shorter length when compared to all other types of hydraulic jumps, so this jump is highly preferred in dam structures.

Methodology:

In the laboratory flume, the flow is regulated from the upstream end by a sluice gate so that a shallow and rapid "supercritical flow" develops. At the downstream end, an adjustable weir can be placed to form a barrier which forces the flow in front of the weir to pile up and becomes "subcritical". A hydraulic jump then forms at the transition from the upstream supercritical flow to the downstream subcritical flow.


Reservoir
Tail gate with rotating flaps
Tilting
rectangular
Sluice gate
Control valve
Collecting
Pressure
P
to pumps
Underground reservoir with rectangular weir
Data
system


Supercritical Flow ($Fr_1 > 1$)
Hydraulic Jump
Subcritical Flow ($Fr_2 < 1$)
(1)
y_1
(2)
y_2

The dynamics of hydraulic jump is governed by the flow continuity and the momentum equation. As we shall see, one of the major characteristic of a hydraulic jump is its large energy dissipation. Therefore, energy equation cannot be used at this point because the head loss is unknown (and not negligible). Using a control volume enclosing the jump as shown in Figure 1, the continuity equation is expressed as $Q=bV1h1=bV2h2$

Where Q is the discharge, V represents the averaged velocity and h is the water depth. The subscript "1" and "2" represent flow information upstream and downstream of the hydraulic jump, respectively.

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