



THE BOOK

This comprehensive text on **Pavement Engineering** is up-to-date with industry standards and best practices and offers an exhaustive coverage on design, construction and maintenance of pavements. The book has followed *AASHTO Guide for Design of Pavement Structures*, 1993, besides meeting latest code provisions and pavement design methods recommended by Indian Roads Congress (IRC) and Bureau of Indian Standards (BIS).

This book refers to the web uploaded volume 'User's Guide for Computer Applications' at web site **www.roadbridgedesign.com** to help readers learn various computer applications in pavement engineering.

KEY FEATURES

- ❑ The book has complete coverage on the subject.
- ❑ It covers all major facets of the whole pavement work under different sections.
- ❑ It describes various computer applications used nowadays in pavement engineering.

- ❑ The book is unique in its kind in terms of features.
- ❑ It provides up-to-date methodology and industry technical practice and guidelines of Pavement Engineering.

TARGET AUDIENCE

- ❑ BE/B.Tech, ME/MS/M.Tech (Civil Engineering and Transportation/Highway Engineering)
- ❑ Professionals of Highway/Road Construction Industry

THE AUTHOR

Sandipan Goswami, M.Tech (IIT Kharagpur) is Technical Advisor (Highway Design and Construction Project) Techsoft Engineering Services. He has been a consultant to the World Bank and different engineering companies such as Halcrow, SNC Lavalin, Egis, Lea Associates, CES (Jacobs), Deloitte Touche Tohmatsu, etc. His areas of research are, the reliability of the design for the construction of highway, tunnel and bridges of concrete and steel structures. He has participated in the lead roles in the design and construction of thousands of kilometers highways, hill roads, low cost rural roads, tunnels and bridges in India, Malaysia and Bangladesh.

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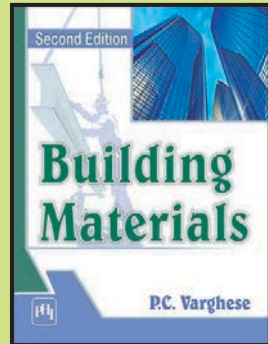
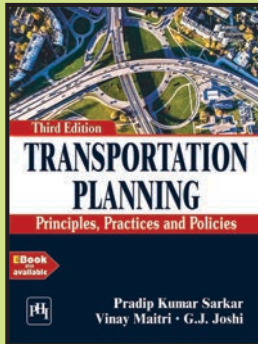
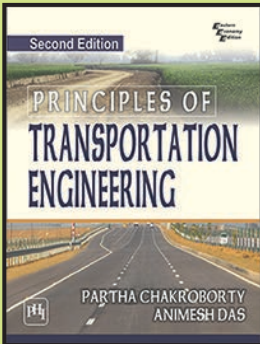
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1. Pavement Engineering and Highway Geometrics
2. Design of Flexible Pavement
3. Design of Rigid Pavement
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Pavement Engineering

(Design-Construction-Maintenance)

User's Guide for Computer Applications



Sandipan Goswami

Pavement Engineering – User's Guide for Computer Applications

Preface

The development of any country depends on its rate of industrial growth, which is highly dependent on the development transport network in the country. The road network in rural sectors provides facilities to MSMEs/SMEs for importing raw materials and transporting finished products to nearest sea/river/airports ports, and further to urban distribution warehouses. The developing countries have been facing challenges in constructing adequate road network to the rural sectors for easy access to ports and urban locations. There are various constraints in the development of adequate road network and primarily the hurdle is the cost. As traffic is not lucrative to attract private developers, therefore the road development mostly depends on the domestic funds. In addition to road development the Govt. also have to maintain funds for maintenance of roads, bridges, tunnels, railways, waterways etc. Currently, there is a race in developing road transport projects worldwide. The growth of the country depends on the development of the adequate road transport facilities for industries and tourism, which require new facilities in developing countries and maintenance of existing facilities in developing as well as developed countries. At the same time, there are projects that are running in parallel to upgrade the existing facilities in the developed countries.

The road development involves expenditures towards land acquisition for futuristic 'Right of Way', Cutting of Trees and subsequent environment restoration, inventory, condition survey, utilities relocation, engineering surveys, traffic surveys, topographical survey, site investigation, design and finally the construction. This book will be an overview of industrial project management, design, construction, and eventually providing a maintenance plan. Some times the cost are huge and can cost a billion dollars for one project, so the client, engineering firm, and contractor are in the same boat until they achieve project success through a strong planning and design system with desired level of technical competence.

In the construction of roads the pavement construction is very expensive and the design must make a proper balance between the cost and the sustainability. During the operation and maintenance period the costs for routine maintenance is to be kept as low as possible as there is less control towards cost control of the periodic maintenance. The reduction in cost for routine maintenance will relieve the Governments from unexpected expenditures.

In these types of projects, all the engineering surveys are done, but, unfortunately, the transport planners and design engineers usually don't receive the proper data from the field surveys and this leads to make various assumptions. If the Axle load survey, Origin-Destination survey, sensitivity analysis are not proper then data is most of the time on conservative side. This results the incorrect decision and expensive design, construction and maintenance. Therefore it is a challenge for the transport and road design engineers to acquire adequate knowledge about the region and transport demand during various times of the year and make a design with maximum possible accuracy for the road project.

This book 'Road Pavements' focuses on the pavement engineering various options and applications. The aim of this book is to provide up-to-date methodology and industry technical practice and guidelines to design, construct, and maintain the pavements in road projects. The essential processes of protection, repair, and strengthening of the pavement structures necessitated by deterioration or a change in the pavement structure are illustrated in this book. It is intended to make a guidebook for students of undergraduate and graduate courses as well as junior and senior engineers who work in

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design, construction, repair, and maintenance of flexible bituminous and rigid concrete pavements and to assist them through all of the stages of pavement design and construction projects.

The book is provided with provided with the appendices which are containing detail description of various laboratory tests, field test, soil treatment by stabilization, ground improvement, slope stability and slope protections measures to facilitate the construction and survival of the road pavement with minimum maintenance.

The 'Appendix – A' for describing the details about various laboratory tests and 'Appendix – B' for describing the details about various field tests relevant to pavement design and construction.

The 'Appendix – C' describes the details about treatment for unsuitable soil to make them suitable by stabilization techniques. Roads and highways are often constructed or widened along the sea coast. This sometimes extends towards the sea. In some cases the edge on the sea side is provided with gabion blocks or the land elevation is raised by dredging and depositing reclaimed soil to facilitate the road construction. The land thus made available by reclamation is often soft and needs to develop desired strength to support road construction. The use of 'Prefabricated Vertical Drains' (PVD) is described with real project case study for better understanding of various facets, instrumentation, observations, necessary measures for rectifications.

The 'Appendix – D' describes the 'Slope Stability Analysis' by Bishop's method, under various frequently observed ground conditions. The analysis is highly helpful for protecting the road pavement on high filled embankments.

The other challenge that faces structural engineers is that most of the undergraduate courses they studied in college focused mainly on real estate projects and housing. However, the characteristics of road development projects are different. This book provides a guide for the university faculties and construction managers to gain detail knowledge about various aspects of pavement design-construction-maintenance projects to successfully contribute towards knowledge transfer and/or leading road pavement project as per the authority's requirements.

On the other hand, from a technical point of view, this book describes the first principle of the codes and standards that are usually used in road projects and the most commonly applicable methods used in the design of the road pavement and the associated construction work. This book describes methods by drawing attention to current research and development in the design, construction, repair, and maintenance philosophy.

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An overview of offshore structure design and construction is very important and provides the tools to check the design and to control the project in all of its phases. Recently, there is a trend toward maintaining the reliability of the structure from both safety and economic points of view by developing the structural integrity management system, which will also be a part of this book.

The last chapter describes the soil investigation tests that are essential to the industrial projects and provides the main key to selecting the most reasonable type of test and also the main features for the pile foundation design.

This book provides a practical guide to designing the reinforced concrete and steel structures and foundations in industrial projects with the principle of repairing the concrete structures and the methodology to deliver a maintenance plan for the concrete and steel structures serving onshore and offshore facilities.

Sandipan Goswami

Kolkata, India

Email: sandipanmails@gmail.com

Author



Sandipan Goswami completed his graduate studies from the University of Calcutta and Post Graduate studies from Indian Institute of Technology in 1985. Over the span of 36 years he became well known for his technical contribution to the Academic Institutes and Project Authorities to make various preparatory work along with project planning, Design, Estimating, Funding and implementation successful for various State and Central Government projects.

He has remained member of various professional societies and institutions which includes, Member of the IRC, Member and Fellow of the Institution of the Engineers, Chartered Engineer, India, Professional Engineer (PE) as Foreign Engineer, Board of Engineers, Malaysia. He published the widely appreciated technical paper on "Project Development – Development of Rural Road Infrastructure, IRC" and many articles in ResearchGate (Germany) and Academia (USA).

The book “Computer Aided Highway Engineering” is published by CRC Press of Taylor and Francis Group, Florida, USA and book “Computer Aided Bridge Engineering” is under production by Nova Science Publishers, New York, USA.

Web page: <https://www.routledge.com/Computer-Aided-Highway-Engineering/Goswami-Sarkar/p/book/9780367493387>

In his 36 years of professional career he worked as Project Manager and Team Leader in various Detail Project Report (DPR), Feasibility Studies Report (FSR) and Construction Supervision projects in India, Malaysia, Bangladesh. As Senior Design Engineer he worked in various Highway, Bridge and Tunnel Projects including 256 kilometers long West Coast Expressway in Malaysia and Vehicular Tunnels in Kuala Lumpur City Centre, Malaysia. He was assigned to review of various DPRs, Design audit, Safety Audit for Highway, Bridges in various States highway projects assigned by ‘The World Bank’.

He also worked for Design and Build of various national and state highway projects, Bored Tunnel for Metro Rails projects, Airport Planning and Design projects etc. He also worked as Team Leader of Project Management Consultant for Urban Infrastructure Improvement, Institutional Strengthening, Capacity Building with e-Governance, Construction Management of state Govt. funded by ‘The World Bank’.

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C5.8.2 The data in the Input file will be, as shown below

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1

Pavement Engineering Overview for Computer Applications

Highway Pavement Development involves three stages

- Pavement Design
- Pavement Construction
- Pavement Maintenance .
-

Types of pavements

- Flexible Pavements
- Rigid Pavements

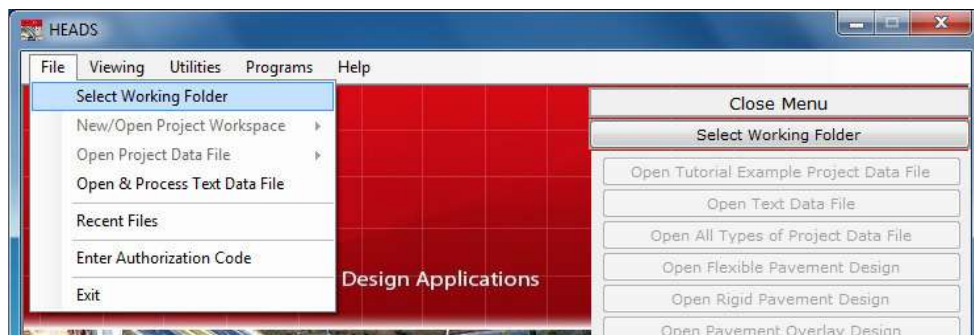
The pavements can be classified based for the structural performance of, flexible and rigid pavements. In flexible pavements, wheel loads are transferred by grain-to-grain contact of the aggregates through the granular structure. The flexible pavement, having less flexural strength, acts like a flexible sheet (e.g. bituminous road). On the contrary, in rigid pavements, wheel loads are transferred to sub-grade soil by flexural strength of the pavement and the pavement acts like a rigid plate (e.g. cement concrete roads). In addition to these, composite pavements are also available. A thin layer of flexible pavement over rigid pavement is an ideal pavement with most desirable characteristics. However, such pavements are rarely used in new construction because of high cost and complex analysis required.

Pavements form the basic supporting structure in highway transportation. Each layer of pavement has a multitude of functions to perform which has to be duly considered during the design process. Different types of pavements can be adopted depending upon the traffic requirements. Improper design of pavements leads to early failure of pavements affecting the riding quality also.

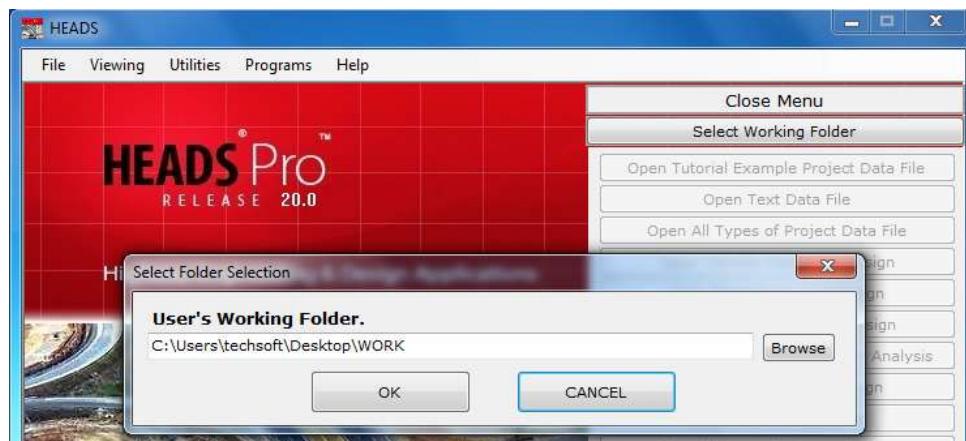
Computer Aided Design of Flexible Pavement **2**

2.1 Computer Aided Design of Flexible Pavement in AASHTO method

From main screen of HEADS Pro select 'File' menu item 'Select Working Folder',

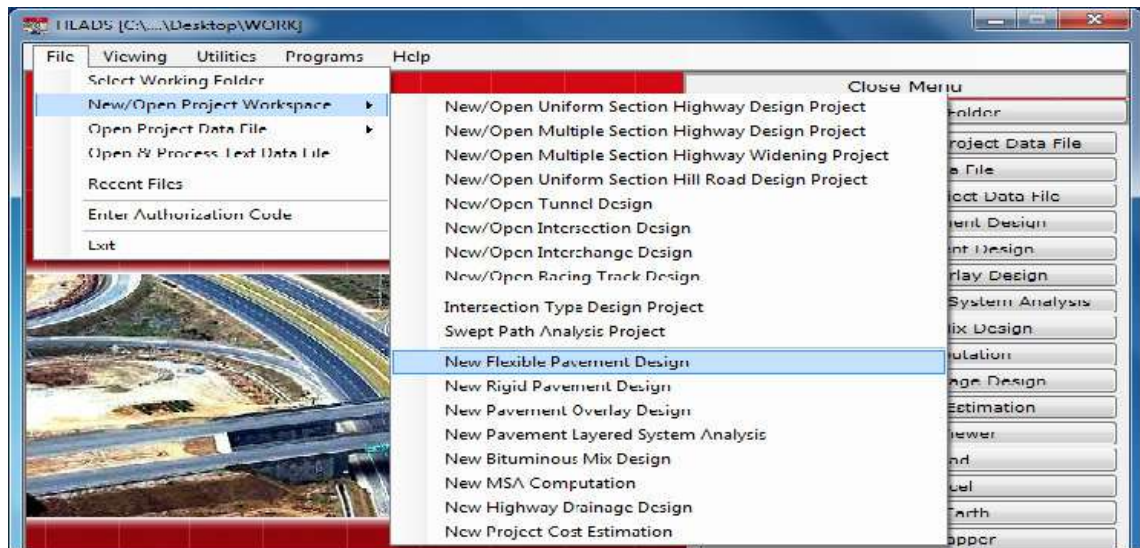


The working folder is selected as 'WORK' located on the Desktop,



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Next, from main screen of HEADS Pro select 'File' menu item 'New/Open Project Workspace', and then select item 'New Flexible Pavement Design',



The first Dialog Box comes for 'Design of Flexible Pavement by AASHTO Method', various default data are also given, which user may change as desired (in the licensed version only).

The 'Flexible Pavement Design' dialog box is shown. It contains the following sections:

- Project Name:** Design of Flexible Pavement by AASHTO Standard
- Design of Flexible Pavement by AASHTO Method:**
 - Analysis Period for Design Life [ANP]: 20 Years
 - Construction Stage(s) [STA]: 2
 - Overall Reliability [ORE]: 90
 - Standard Deviation [STD]: 0.45
 - Major Layer Types (Bound, Unbound and Granular) [LAY]: 3
 - Soil Resilient Modulus [ZR]: -1.645
 - Initial Serviceability [PS1]: 4.2
 - Serviceability Loss [DPE]: 0.0
 - CBR value of Subgrade in % [CBR]: 5.0 %
- Stage Design Table:**

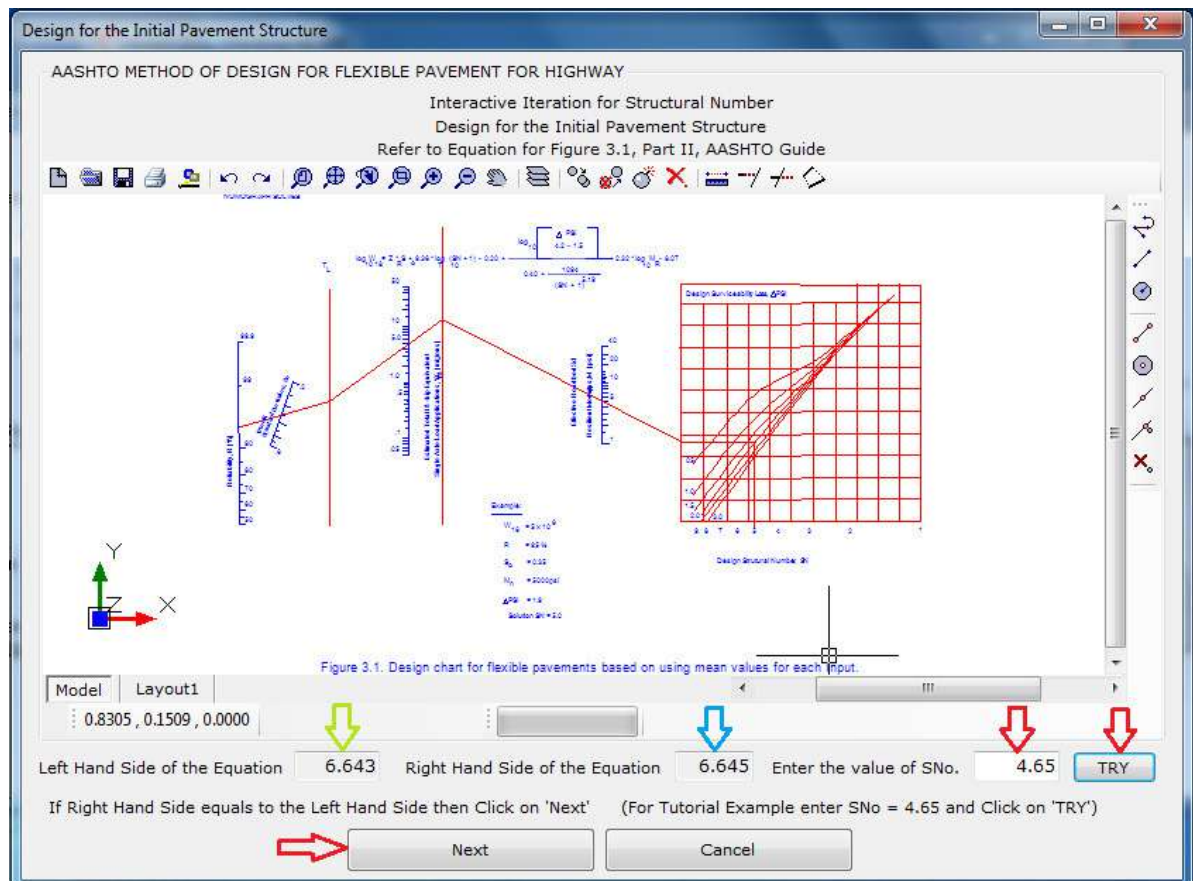
Construction Stages	Stage Design Life for each stage [PER] (years)	Equivalent Standard Axle value for each stage [ESA]	Terminal Serviceability at the end of each Stage [PS2]
1	10	4400000	2.5
2	10	11200000	2.0
- Major Layer Design Table:**

Elastic Modulus of Each Layer [E]	Layer Coefficient of Each Layer [a]	Drainage Coefficient of Each Layer [m]	Structural Number of Each Layer [SN]
400000	0.42	1.0	2.8
30000	0.14	1.2	4.2
11000	0.08	1.2	0.0
- Buttons:** Insert Row, Delete Row, Proceed (highlighted with a red arrow), Open Input Data File, Open Report File, Close.

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The next Dialog Box comes for the design of initial pavement structure for first 10 years stage of total 20 years of design life, Equation for Figure 3.1 of Part II, AASHTO Guide is to be solved by iteration, The left side of the equation has the value of 6.643, we enter value of Structural Number 'SNo' as 4.65 for the default data, we get right side of the equation has the value of 6.645, which is almost same as left side. So, the value of 'SNo' is obtained as 4.65.

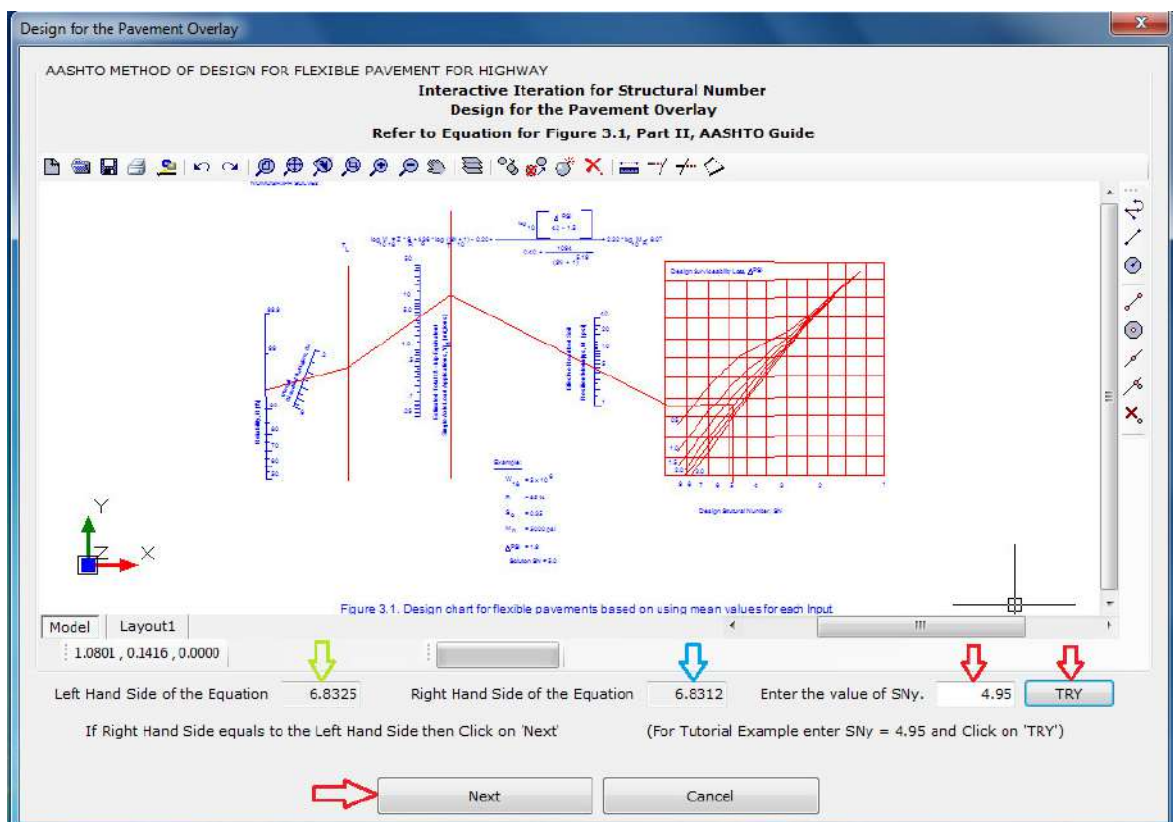
User may have to make several trials to solve the equation by iterative method.



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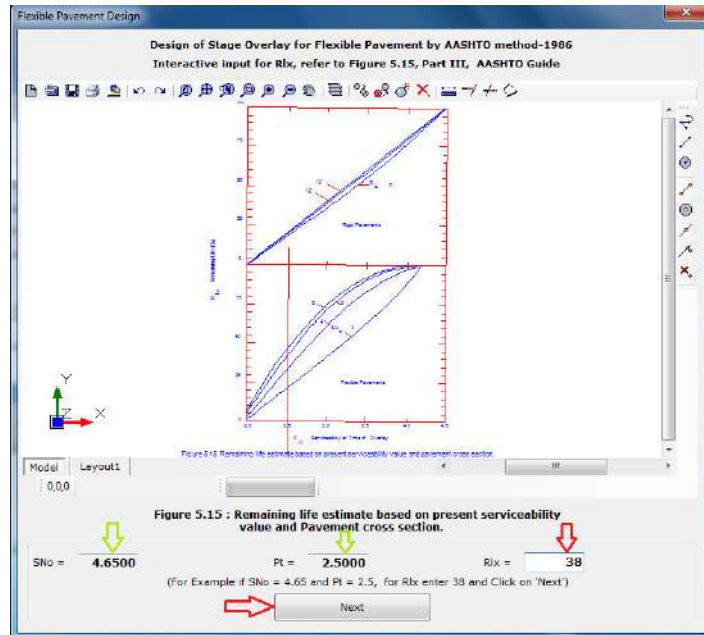
The next Dialog Box comes for the design of pavement overlay for next 10 years stage of total 20 years of design life, Equation for Figure 3.1 of Part II, of AASHTO Guide is to be solved by iteration, The left side of the equation has the value of 6.8325, we enter value of Structural Number 'SNy' as 4.95 for the default data, we get right side of the equation has the value of 6.8312, which is almost same as left side. So, the value of 'SNy' is obtained as 4.95.

User may have to make several trials to solve the equation by iterative method.

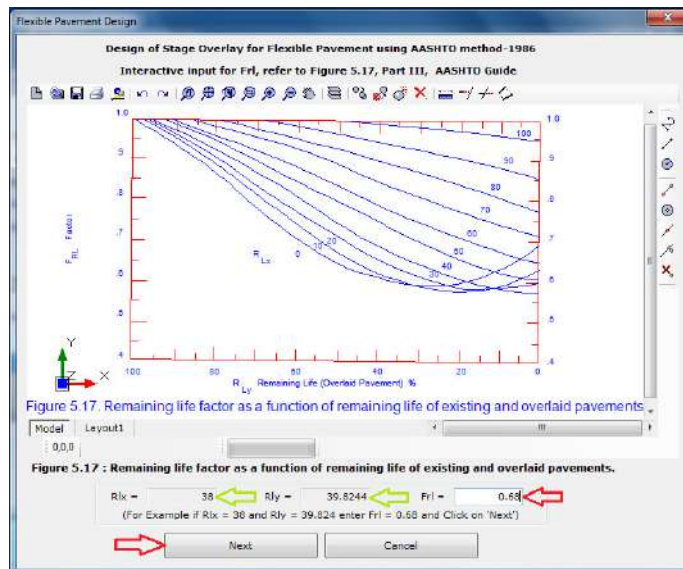


Pavement Engineering – User's Guide for Computer Applications

The next Dialog Box comes with the Charts by Figure 5.15, Part III, AASHTO Guide as displayed, from which using the values of 'SNo = 4.65' and 'Pt = 2.5', we get the value of Rlx=38 and is entered.

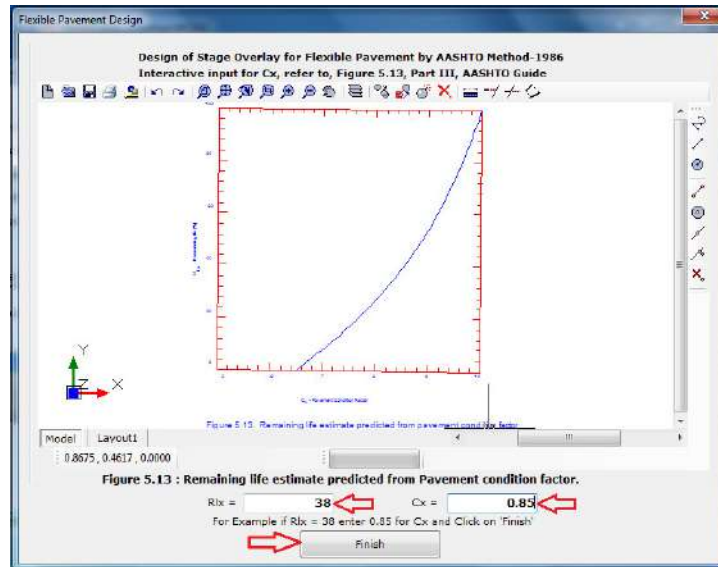


The next Dialog Box comes with the Charts by Figure 5.17, Part III, AASHTO Guide as displayed, from which using the values of 'Rlx = 38' and 'Rly = 39.8244', we get the value of Frl=0.68 and is entered.

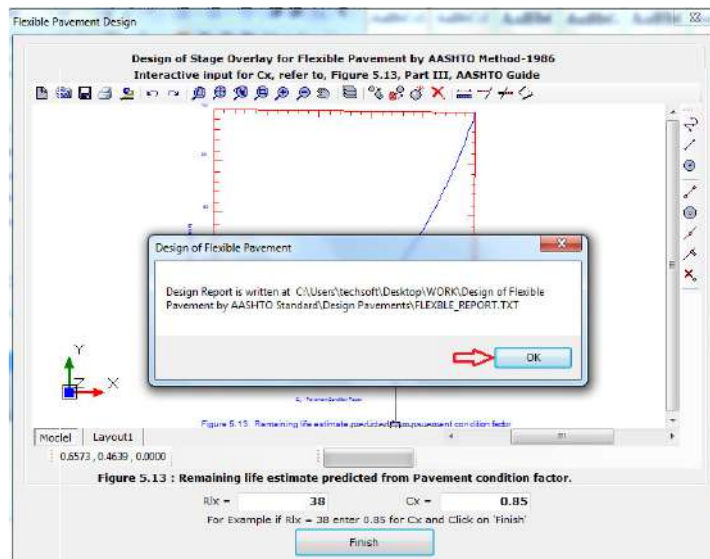


Pavement Engineering – User's Guide for Computer Applications

The next Dialog Box comes with the Charts by Figure 5.13, Part III, AASHTO Guide as displayed, from which using the value of ' $R_{1x} = 38$ ', we get the value of $C_x = 0.85$ and is entered.



Next, we click on button 'Finish' and the design process is over. The message comes informing about the Design Report, Click on button 'OK', next, the Design Report is opened for viewing and printing.



Pavement Engineering – User's Guide for Computer Applications

Report of Design of Flexible Pavement in AASHTO method

HEADS Pro Enterprise/Professional/Educational Version

Report on Flexible Pavement Design

Program was run on 8/4/2017 9:07:19 PM
TechSOFT Engineering Services.

Design of Flexible Pavement according to the Standard :
AASHTO Guide for Design of Pavement Structures - Parts I, II and III - 1986.

This version of AASHTO is based on detailed analytical methodology and was appreciated/accepted by the authorities of various countries worldwide.

INPUT DATA :

Strategy = 2 Stage (s)

Analysis Period = 20.00 years.

For Initial Structure :
Performance Period = 10.00 years.

For Overlay no 1 :
Performance Period = 10.00 years.

ESA for Stage 1 = 4400000.000
ESA for Stage 2 = 11200000.000

Individual Stage Reliability = 90.00
Overall Reliability = (0.90) ** 1/2
= 0.95

Standard Deviation = 0.450
Ref. Table 5.1, AASHTO Guide for Design of Pavement Structures- Part I- 1986

The value of Zr = -1.645
Initial Serviceability, Po = 4.20
Terminal Serviceability, Pt = 2.50
DEL-PSI = Po - Pt = 1.70

The Subgrade CBR = 5.00
The Soil Resilient Modulus Mr = 7500.00 psi
Mr = 1500 x CBR)

Pavement Engineering – User's Guide for Computer Applications

The Serviceability loss due to Environment,
required for the design of the overlay after the initial stage
=DEL-PSI-sw (here road-bed swelling only) = 0.000

THE ANALYSIS FOR THE INITIAL STRUCTURE (in Stage 1) :

Total number of Structural Layers = 3

For layer 1 :

Elastic Modulus = E1 = 400000.000 psi
Layer Coefficient = a1 = 0.420

From the Nomograph in Fig 3.1 AASHTO Design of Pavement Structures - 1986, with above values of Overall Reliability, Zr, So, W18 and using the values of Elastic Modulus E1 as Mr, we get :

Structural No. = SN0 = 2.800
Drainage Coefficient = m1 = 1.000

For layer 2 :

Elastic Modulus = E2 = 30000.000 psi
Layer Coefficient = a2 = 0.140

From the Nomograph in Fig 3.1 AASHTO Design of Pavement Structures - 1986, with above values of Overall Reliability, Zr, So, W18 and using the values of Elastic Modulus E2 as Mr, we get :

Structural No. = SN1 = 4.200
Drainage Coefficient = m2 = 1.200

For layer 3 :

Elastic Modulus = E3 = 11000.000 psi
Layer Coefficient = a3 = 0.080

From the Nomograph in Fig 3.1 AASHTO Design of Pavement Structures - 1986, with above values of Overall Reliability, Zr, So, W18 and using the values of Elastic Modulus E3 as Mr, we get :

Structural No. = SN2 = 0.000
Drainage Coefficient = m3 = 1.200

18-Kip ESAL available for Stage 1
ESA = 4400000.000

With the following values :

W18 = 4400000.000 ESA.
DEL-PSI = 1.700
Zr = -1.645
So = 0.450
Mr = 7500.000

Pavement Engineering – User's Guide for Computer Applications

The following Equation from Fig 3.1, Part II, AASHTO Guide - 1986
For Design of Pavement Structures, is solved by interactive
Trial and Error method :

$$\log_{10}(W_{18}) = Z_r \times S_o + 9.36 \times \log_{10}(S_{No} + 1) - 0.20$$

$$\log_{10} \left[\frac{DEL-PSI}{(4.2 - 1.5)} \right] + \frac{5.19}{0.40 + 1094 / (S_{No} + 1)} + 2.32 \times \log_{10}(M_r) - 8.07$$

Structural Number for Initial Pavement Structure has been determined as

$$S_{No} = 4.650$$

For layer 1 ,

Thickness :

$$\begin{aligned} \text{For } a_1 &= 0.420, \\ \text{and } SN_1 &= 2.800, \\ D_1^* &= SN_1 / a_1 \\ D_1^* &= 2.800 / 0.420 \\ D_1^* &= 7.000 \text{ inch.} \end{aligned}$$

Structural No :

$$\begin{aligned} SN_1^* &= D_1^* \times a_1 \\ SN_1^* &= 7.000 \times 0.420 \\ SN_1^* &= 2.940 \end{aligned}$$

For layer 2 ,

Thickness :

$$\begin{aligned} \text{For } a_2 &= 0.140, \\ m_2 &= 1.200, \\ \text{and } SN_2 &= 4.200, \\ D_2^* &= (SN_2 - [SN_1^* + SN_2^*]) / (a_2 \times m_2) \\ D_2^* &= (4.200 - 2.940) / (0.140 \times 1.200) \\ D_2^* &= 8.000 \text{ inch.} \end{aligned}$$

Structural No :

$$\begin{aligned} SN_2^* &= D_2^* \times a_2 \times m_2 \\ SN_2^* &= 8.000 \times 0.140 \times 1.200 \\ SN_2^* &= 1.344 \end{aligned}$$

For layer 3 ,

Thickness :

$$\begin{aligned} \text{For } a_3 &= 0.080, \\ m_3 &= 1.200, \\ \text{and } S_{No} &= 4.650, \\ D_3^* &= (S_{No} - [SN_1^* + SN_2^* + SN_3^*]) / (a_3 \times m_3) \\ D_3^* &= (4.650 - 4.284) / (0.080 \times 1.200) \\ D_3^* &= 4.000 \text{ inch.} \end{aligned}$$

Pavement Engineering – User's Guide for Computer Applications

Summary of Design for Initial Structure at stage 1 :

Total number of Structural Layers = 3

Thickness of Layer D3 = 7.000 inch. = 177.800 mm.

Thickness of Layer D3 = 8.000 inch. = 203.200 mm.

Thickness of Layer D3 = 4.000 inch. = 101.600 mm.

End of Design of Initial Structure in Stage 1.

THE ANALYSIS FOR THE DESIGN OF OVERLAY (in Stage 2) :

Layer(s) : Asphalt Concrete
Layer Coefficient a1 = 0.420

Serviceability loss due to traffic
DEL-PSI-tr = DEL-PSI - DEL-PSI-sw
= 1.700 - 0.000
= 1.700

18-Kip ESAL available for Stage 2
y = 11200000.000 - 4400000.000
= 6800000.000

With the following values :

W18 = 6800000.000 ESA.
DEL-PSI-tr = 1.700
Zr = -1.645
So = 0.450
Mr = 7500.000 psi

The Equation at Fig 3.1 AASHTO Pavement Design Guide - 1986,

$\log_{10}(W18) = Z_r \times S_o + 9.36 \times \log_{10}(SN_y + 1) - 0.20$

$$+ \frac{\log_{10} [\text{DEL-PSI} / (4.2 - 1.5)]}{0.40 + 1094 / (SN_y + 1)^{5.19}} + 2.32 \times \log_{10}(Mr) - 8.07$$

Structural Number for Initial Pavement and Overlay is determined by interactive Trial and Error method as :

SNy = 4.950

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Refer to the Fig. 5.15, from Part - III,
AASHTO Guide for Design of Pavement Structures - 1986.

For the values of $S_{No} = 4.650$
 $P_t = 2.500$
The value of $R_{lx} = 38.000$

Terminal Serviceability value for the Overlay is given as :
 $P_t' = 2.000$

$DEL-PSI' = P_o - P_t'$
 $= 4.200 - 2.000$
 $= 2.200$

With the following values :

$S_{Ny} = 4.950 \text{ ESA.}$
 $DEL-PSI' = 2.200$
 $Z_r = -1.645$
 $S_o = 0.450$
 $M_r = 7500.000$

From Equation at Fig 3.1 AASHTO Pavement Design Guide - 1986,

$\log_{10}(N_{fy}) = Z_r \times S_o + 9.36 \times \log_{10}(S_{Ny} + 1) - 0.20$

$$+ \frac{\log_{10}[DEL-PSI' / (4.2 - 1.5)]}{0.40 + 1094 / (S_{Ny} + 1)^{5.19}}$$
$$+ 2.32 \times \log_{10}(M_r) - 8.07$$

$= 7.053089$

Therefore 18-Kip ESAL,
 $N_{Fy} = \text{anti-logarithm of } [7.053089]$
 $= 11300267.080$

$R_{ly} = (N_{fy} - y) / N_{fy}$
 $= (11300267.080 - 6800000.000) / 11300267.080$
 $= 39.824$

Refer to the Fig. 5.17, from Part - III,
AASHTO Guide for Design of Pavement Structures - 1986.

For the values of $R_{lx} = 38.000$
 $R_{ly} = 39.824$
The value of $F_{rl} = 0.680$

Refer to the Fig. 5.13, from Part - III,
AASHTO Guide for Design of Pavement Structures - 1986.

For the values of $R_{lx} = 38.000$
The value of $C_x = 0.850$

The calculated value of SN_{xeff} :

Pavement Engineering – User's Guide for Computer Applications

$$\begin{aligned}\text{SN}_{\text{xeff}} &= C_x \times \text{SNo} \\ &= 0.850 \times 4.650 \\ &= 3.953\end{aligned}$$

The calculated value of SN_{ol} :

$$\begin{aligned}\text{SN}_{\text{ol}} &= \text{SN}_y - (\text{Frl} \times \text{SN}_{\text{xeff}}) \\ &= 4.950 - (0.680 \times 3.953) \\ &= 2.262\end{aligned}$$

The Designed Thickness of Asphalt Concrete Overlay :

$$\begin{aligned}\text{Dol} &= \text{SN}_{\text{ol}} / a_1 \\ &= 2.262 / 0.420 \\ &= 5.386 \text{ inch.} \\ &= 136.815 \text{ mm.} \\ &= 140.000 \text{ mm. (Say).}\end{aligned}$$

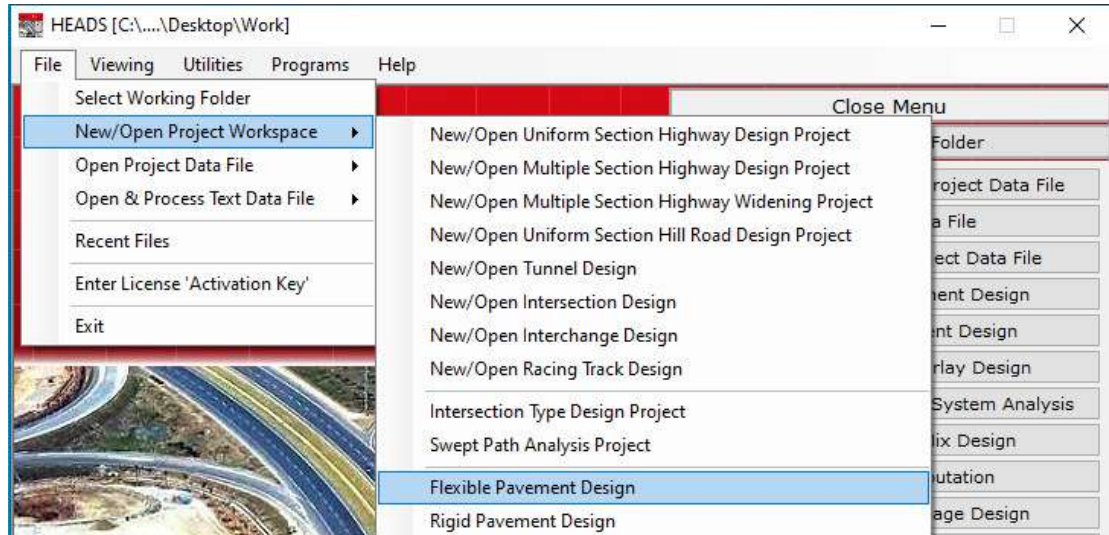
End of Design of Overlay for Stage 2.

END OF DESIGN

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2.2 Computer Aided Design of Flexible Pavement in IRC 37 method

The folder 'Work' on the desktop is already selected as the working folder, Select menu item 'File >> New/Open Project Workspace >> Flexible Pavement Design',



The design window opens, Select menu item 'Design of Flexible Pavement by IRC 37 Standard',

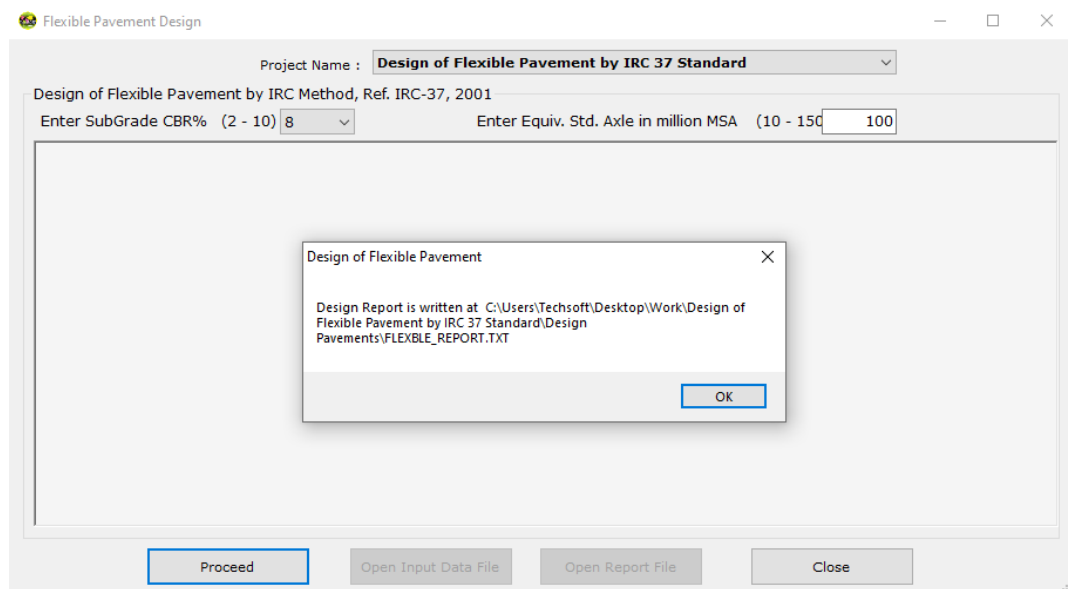
The screenshot shows the 'Flexible Pavement Design' window. The 'Project Name' dropdown is set to 'Design of Flexible Pavement by AASHTO Standard'. The 'Design of Flexible Pavement by AASHTO Method' section includes input fields for 'Analysis Period for Design Life [ANP]' (20 years), 'Construction Stage(s) [STA]' (2), 'Overall Reliability [ORE]' (90), 'Standard Deviation [STD]' (0.45), 'Soil Resilient Modulus [ZR]' (-1.645), 'Initial Serviceability [PS1]' (4.2), 'Serviceability Loss [DPE]' (0.0), and 'CBR value of Subgrade in % [CBR]' (5.0 %). The 'Stage Design' table shows two stages with construction stages 1 and 2, design life of 10 years, equivalent standard axle values of 4400000 and 11200000, and terminal serviceability values of 2.5 and 2.0. The 'Major Layer Design' table shows three layers with elastic modulus values of 400000, 30000, and 11000, layer coefficients of 0.42, 0.14, and 0.08, drainage coefficients of 1.0, 1.2, and 1.2, and structural numbers of 2.8, 4.2, and 0.0. Buttons for 'Insert Row', 'Delete Row', 'Proceed', 'Open Input Data File', 'Open Report File', and 'Close' are visible at the bottom.

Construction Stages	Stage Design Life for each stage [PER] (years)	Equivalent Standard Axle value for each stage [ESA]	Terminal Serviceability at the end of each Stage [PS2]
1	10	4400000	2.5
2	10	11200000	2.0

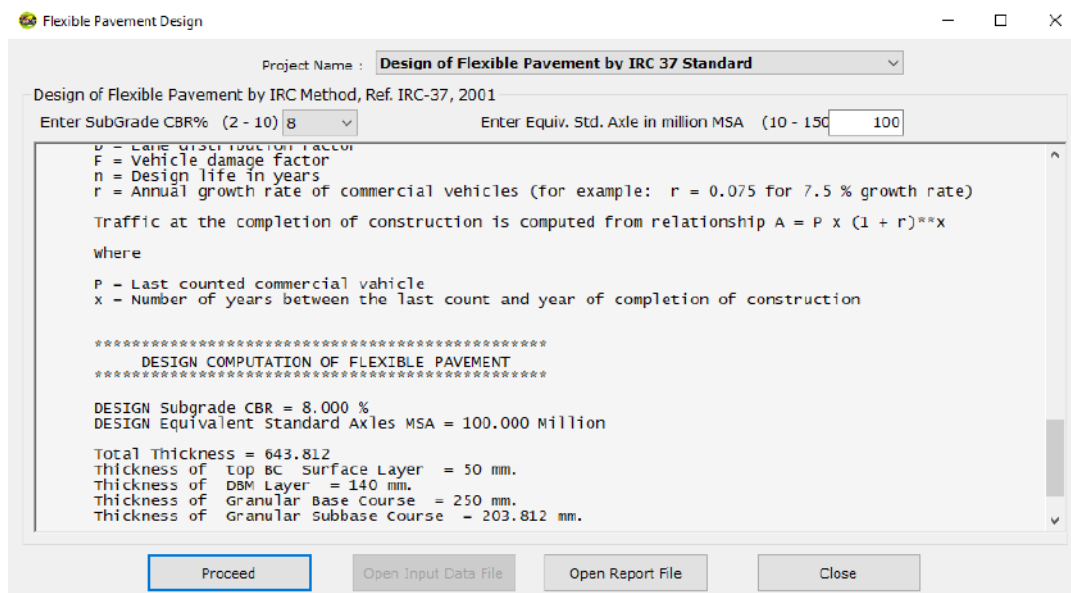
Elastic Modulus of Each Layer [E]	Layer Coefficient of Each Layer [a]	Drainage Coefficient of Each layer [m]	Structural Number of Each Layer [SN]
400000	0.42	1.0	2.8
30000	0.14	1.2	4.2
11000	0.08	1.2	0.0

Pavement Engineering – User's Guide for Computer Applications

The CBR is mentioned as 8% and ESAL in millions is mentioned as 100, click on button 'Proceed', the design is done and message comes informing about the design report.

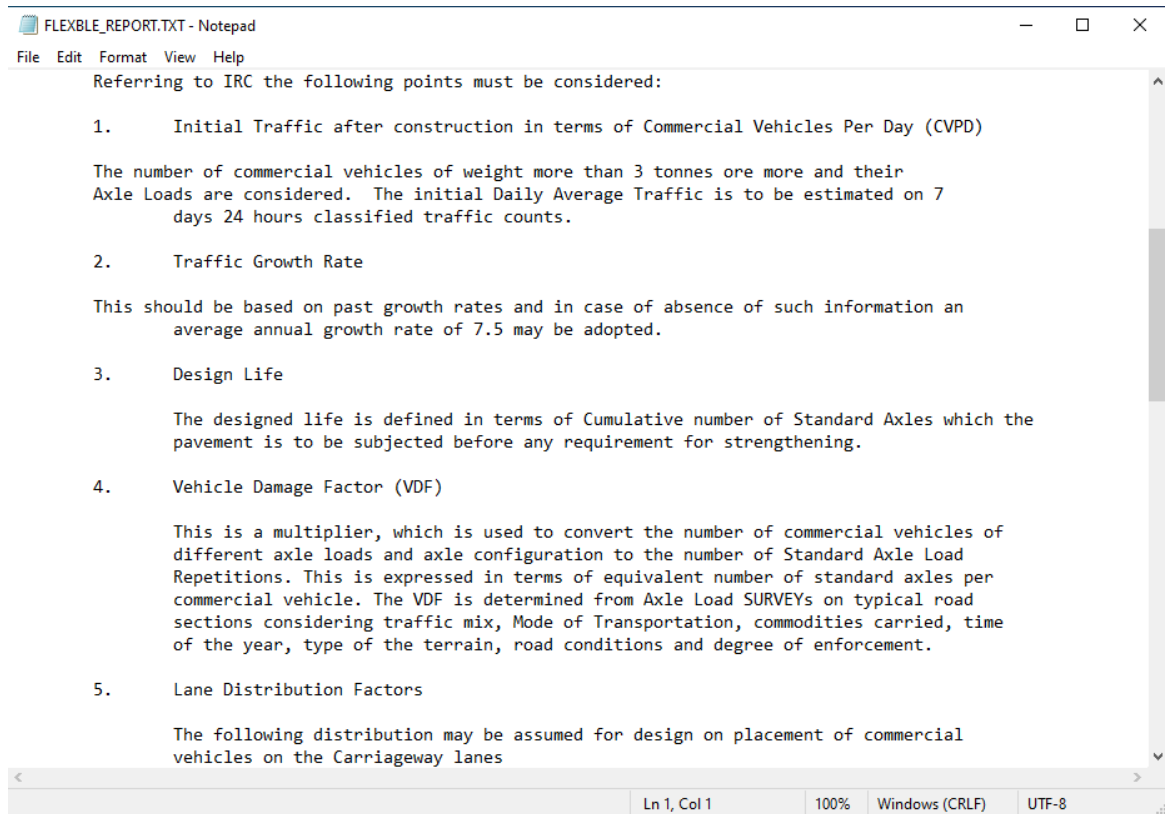


Click on 'OK', the design report is displayed in the window, click on button 'Open Report File',



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The design report is opened in Notepad, User may save it elsewhere or take printout.



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Report on Flexible Pavement Design in IRC 37

HEADS Pro Enterprise/Professional/Educational Version

Report on Flexible Pavement Design

Program was run on 19-04-2021 09:31:04

TechSOFT Engineering Services.

THE DESIGN OF FLEXIBLE PAVEMENT BY IRC 37 METHOD

Referring to IRC the following points must be considered:

1. Initial Traffic after construction in terms of Commercial Vehicles Per Day (CVPD)

The number of commercial vehicles of weight more than 3 tonnes or more and their Axle Loads are considered. The initial Daily Average Traffic is to be estimated on 7 days 24 hours classified traffic counts.

2. Traffic Growth Rate

This should be based on past growth rates and in case of absence of such information an average annual growth rate of 7.5 may be adopted.

3. Design Life

The designed life is defined in terms of Cumulative number of Standard Axles which the pavement is to be subjected before any requirement for strengthening.

4. Vehicle Damage Factor (VDF)

This is a multiplier, which is used to convert the number of commercial vehicles of different axle loads and axle configuration to the number of Standard Axle Load Repetitions. This is expressed in terms of equivalent number of standard axles per commercial vehicle. The VDF is determined from Axle Load SURVEYs on typical road sections considering traffic mix, Mode of Transportation, commodities carried, time of the year, type of the terrain, road conditions and degree of enforcement.

5. Lane Distribution Factors

The following distribution may be assumed for design on placement of commercial vehicles on the Carriageway lanes

Single lane roads: The design should be based on total number of commercial vehicles in both the directions because of maximum channelization resulting in concentration of wheel load repetition.

Two lane Single Carriageway roads: The design should be based on 75 % of the total number of commercial vehicles in both the directions

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Four lane Single Carriageway roads: The design should be based on 40 % of the total number of commercial vehicles in both the directions

Dual Carriageway roads: The design should be based on 75 %, 60 % & 45 % of the number of commercial vehicles in each direction for two, three & four lane Carriageway respectively.

The design should be based on the traffic in a lane where it is maximum, applying in the design for the entire width of the Carriageway.

The design traffic is computed in terms of cumulative number of Standard Axles for the entire design life using the formula:

$$N = (365 \times [(1 + r)^n - 1] \times A \times D \times F) / r$$

Where,

N = Cumulative number of Standard Axles in millions

A = Initial traffic after the completion of construction in number of commercial vehicles Per day

D = Lane distribution factor

F = Vehicle damage factor

n = Design life in years

r = Annual growth rate of commercial vehicles (for example: r = 0.075 for 7.5 % growth rate)

Traffic at the completion of construction is computed from relationship $A = P \times (1 + r)^x$

Where,

P = Last counted commercial vehicle

x = Number of years between the last count and year of completion of construction

```
*****
DESIGN COMPUTATION OF FLEXIBLE PAVEMENT
*****
```

DESIGN Subgrade CBR = 8.000 %

DESIGN Equivalent Standard Axles MSA = 100.000 Million

Total Thickness = 643.812

Thickness of top BC Surface Layer = 50 mm.

Thickness of DBM Layer = 140 mm.

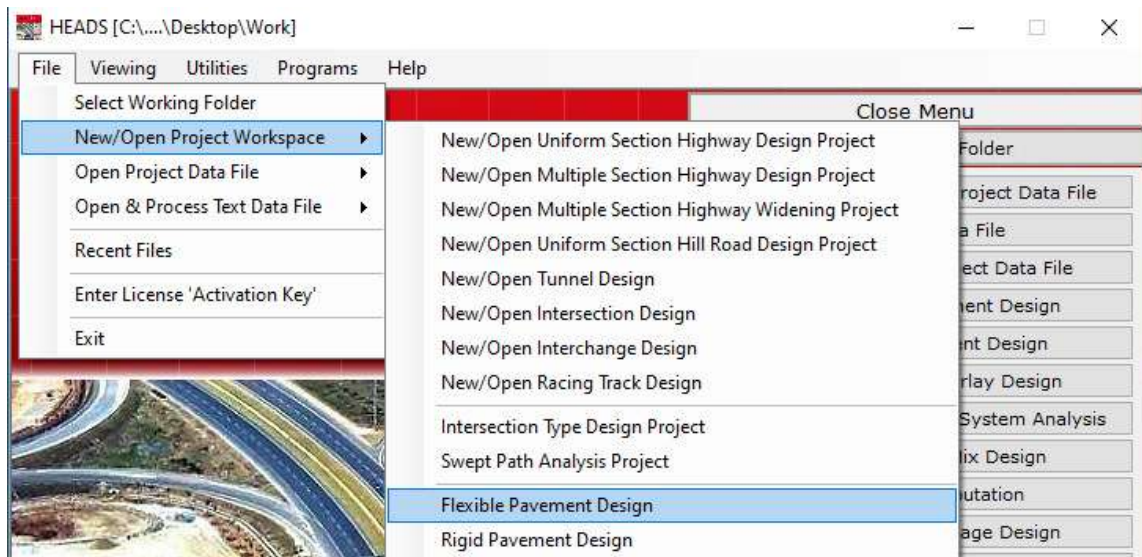
Thickness of Granular Base Course = 250 mm.

Thickness of Granular Subbase Course = 203.812 mm.

```
*****
END OF DESIGN OF FLEXIBLE PAVEMENT BY IRC 37 METHOD
*****
```

2.3 Computer Aided Design of Flexible Pavement in IRC SP 20 method

The folder 'Work' on the desktop is already selected as the working folder, Select menu item 'File >> New/Open Project Workspace >> Flexible Pavement Design',



The design window opens, Select menu item 'Design of Flexible Pavement by Low Cost Rural Road Standard',

The screenshot shows the 'Flexible Pavement Design' window. At the top, the 'Project Name' dropdown is set to 'Design of Flexible Pavement by AASHTO Standard'. Below it, the 'Design of Flexible Pavement by AASTHO Method' section contains input fields for 'Analysis Period for Design Life [ANP]' (20 years), 'Construction Stage(s) [STA]' (2), 'Overall Reliability [ORE]' (90), and 'Standard Deviation [STD]' (0.45). To the right, there are input fields for 'Soil Resilient Modulus [ZR]' (-1.645), 'Initial Serviceability [PS1]' (4.2), 'Serviceability Loss [DPE]' (0.0), and 'CBR value of Subgrade in % [CBR]' (5.0 %). The window is divided into two main sections: 'Stage Design' and 'Major Layer Design'. The 'Stage Design' table has columns for 'Construction Stages', 'Stage Design Life for each stage [PER] (years)', 'Equivalent Standard Axle value for each stage [ESA]', and 'Terminal Serviceability at the end of each Stage [PS2]'. The 'Major Layer Design' table has columns for 'Elastic Modulus of Each Layer [E]', 'Layer Coefficient of Each Layer [a]', 'Drainage Coefficient of Each layer [m]', and 'Structural Number of Each Layer [SN]'. At the bottom, there are buttons for 'Proceed', 'Open Input Data File', 'Open Report File', and 'Close'.

Construction Stages	Stage Design Life for each stage [PER] (years)	Equivalent Standard Axle value for each stage [ESA]	Terminal Serviceability at the end of each Stage [PS2]
1	10	4400000	2.5
2	10	11200000	2.0

Elastic Modulus of Each Layer [E]	Layer Coefficient of Each Layer [a]	Drainage Coefficient of Each layer [m]	Structural Number of Each Layer [SN]
400000	0.42	1.0	2.8
30000	0.14	1.2	4.2
11000	0.08	1.2	0.0

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Select design item 'Design of Low Cost Metal Surfaced Rural Road [IRC SP 20]', the sample default data is displayed, change the data as required, click on button 'Proceed',

The screenshot shows the 'Flexible Pavement Design' software window. The 'Project Name' is 'Design of Flexible Pavement by Low Cost Rural Road Standard'. The 'Design Options' dropdown is set to 'Design of Low Cost Metal Surfaced Rural Road [IRC SP 20]'. The design type is 'Design of Low Cost Metal Surfaced Rural Road [FLEXIBLE IRC SP 20]'. The input data is as follows:

Parameter	Value	Unit
Total Number of Animal Drawn Carts per day [ADC]	25	Nos.
Total Number of Bicycles per day [BC]	457	Nos.
Total Number of Motor Cycles per day [MC]	200	Nos.
Total Number of Passenger Cars and Jeeps per day [PC]	15	Nos.
Total Number of Heavy Commercial Vehicles per day [HCV]	9	Nos.
Total Number of Medium Commercial Vehicles per day [MCV]	55	Nos.
Construction Time [CT]	2	Years
(Time Duration in years between last count and roads opened for Traffic)		
CBR of Subgrade Soil [CBR]	5.0	%
Annual Growth Rate of Traffic in Percentage [GROWTH]	6.0	%
Design Life of Road in Years [LIFE]	10	Years
Lane Distribution Factor [LDF]	1.0	
Axle Load Survey	Done	
VDF for Heavy Commercial vehicle [HVD]	5.0	
VDF for Medium Commercial vehicle [MVD]	3.0	
Traffic Factor of Non-Harvesting Season and Peak Harvesting Season [FACT]	0.5	
Peak Harvesting period [PHT]	75	Days
Number of Harvesting Season in a year [NHS]	1	Nos.

Buttons at the bottom: Proceed, Open Input Data File, Open Report File, Close.

The design is done and message comes informing about the design report, click on 'OK',

The screenshot shows the 'Flexible Pavement Design' software window with the 'Design Options' dropdown set to 'Design of Low Cost Metal Surfaced Rural Road [IRC SP 20]'. The design type is 'Design of Low Cost Metal Surfaced Rural Road [FLEXIBLE IRC SP 20]'. The input data is as follows:

Parameter	Value	Unit
Total Number of Motorized Vehicles per day [MVPD]	70	Nos
Total Number of Commercial Vehicles per day [CVPD]	30	Nos
Annual Growth Rate of Traffic in Percentage		
Design Life of Road in Years [LIFE]		
Construction Time [CT]		
(Time Duration in years between last count and roads opened for Traffic)		
CBR of Subgrade Soil [CBR]	5.0	%
Annual Rainfall [RAINFALL]	1200.0	mm

A message box titled 'HEADS' is displayed in the foreground, stating: 'Design Report is written at C:\Users\Techsoft\Desktop\Work\Design of Flexible Pavement by Low Cost Rural Road Standard\Design Pavements\Flexible_Rural Road_SP20_REPORT.TXT'. The 'OK' button is highlighted.

Buttons at the bottom: Proceed, Open Input Data File, Open Report File, Close.

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The design report is displayed in the window, clicking on button 'Open Report File', the design report may be opened in Notepad, User may save it elsewhere or take printout.

The screenshot displays the 'Flexible Pavement Design' software window. The interface includes a title bar, a menu bar, and a main workspace divided into input fields and a report preview area.

Project Name: Design of Flexible Pavement by Low Cost Rural Road Standard

Design Options: Design of Low Cost Metal Surfaced Rural Road [IRC SP 20]

Design of Low Cost Metal Surfaced Rural Road [FLEXIBLE IRC SP 20]

Input Parameters:

- Total Number of Motorized Vehicles per day [MVPD]: 70 Nos
- Total Number of Commercial Vehicles per day [CVPD]: 30 Nos
- Annual Growth Rate of Traffic in Percentage [GROWTH]: 6.0 %
- Design Life of Road in Years [LIFE]: 10 Years
- Construction Time [CT]: 2 Years
- (Time Duration in years between last count and roads opened for Traffic)
- CBR of Subgrade Soil [CBR]: 5.0 %
- Annual Rainfall [RAINFALL]: 1200.0 mm

Design Report Preview:

```
*****
HEADS Pro Enterprise/Professional/E
Low Cost Rural Road Vers1
Design of FLEXIBLE Paveme
Following IRC SP 20
TechSOFT Engineering Services (I)
Program was run on 19-04-2021 00
*****

HEADS for Rural Roads
Design of Flexible Pavement for Rural Roads

////////////////////
Refer to IRC:SP:20 2002
////////////////////

=====
USER INPUT DATA for Design of Flexible Pavement for Rural R
Total Number of Motorised Vehicles per day = [MVPD] =
Total Number of Motorised Vehicles per day = [CVPD] =
Annual Growth Rate of Traffic in Percentage = [r] = 6.00
Design Life of Road in Years = [n] = 10.0
Construction Time = Time Duration in years between last cou
CBR of Subgrade Soil [CBR] = 5.0
=====
```

Buttons: Proceed, Open Input Data File, Open Report File, Close

Pavement Engineering – User's Guide for Computer Applications

Report on Design of Flexible Pavement Following IRC SP 20

HEADS Pro Enterprise/Professional/Educational Version
Low Cost Rural Road Version
Design of FLEXIBLE Pavement
Following IRC SP 20
TechSOFT Engineering Services (I) Pvt. Ltd.
Program was run on 19-04-2021 09:36:19

HEADS for Rural Roads
Design of Flexible Pavement for Rural Roads

=====
Refer to IRC:SP:20 2002
=====

=====
USER INPUT DATA for Design of Flexible Pavement for Rural Roads
Total Number of Motorised Vehicles per day = [MVPD] = 70
Total Number of Motorised Vehicles per day = [CVPD] = 30
Annual Growth Rate of Traffic in Percentage = [r] = 6.00
Design Life of Road in Years = [n] = 10.0
Construction Time = Time Duration in years between last count and roads opened for
Traffic [x] = 2.0
CBR of Subgrade Soil [CBR] = 5.0
Annual Rainfall = 1200.0 mm
END OF USER INPUT DATA for Design of Flexible Pavement
=====

COMPUTATION OF DESIGN TRAFFIC

=====
Refer to IRC:SP:20 2002
=====

From Traffic count data

Total Number of MVPD [motorised vehicles (Except 2 Wheelers) in both directions] =
70
Total Number of CVPD [Of Weight > 3 Tonnes laden Weight] = 30
Design Traffic (T) = $CVPD \times (1 + r)^{(n+x)}$
 $= 30 \times (1 + 6.00/100)^{(10.0 + 2.0)} = 60$

=====
Refer to Figure 5.1 of IRCSP20 2002
=====

Curve 'C' is used to obtain Total Crust Thickness in mm.
For CBR = 5.0 and CVPD = 60
From Fig. 5.1, IRC SP20, The Total Pavement thickness = D = 420.0 mm

=====
Refer to Figure 5.2 of IRCSP20 2002
=====

Designed Pavement crust thickness = D = 420.0 mm
Provide: Sub Base Course = 260 mm
Base Course = 150 mm

=====
Refer to Table 5.1 of IRCSP20 2002
=====

Projected Motorised Vehicles per day
= $MVPD + (1 + r)^{n+x}$
= $70 \times (1 + 6.00/100)^{10.0 + 2.0} = 140$

For Annual Rainfall 1200.0 mm and Projected Motorised Vehicles per day = 140
From Table 5.1 of IRCSP20, Provide Thin Bituminous Surfacing [2 Coat Surface Dressing]

Finally, The Proposed Pavement Composition:

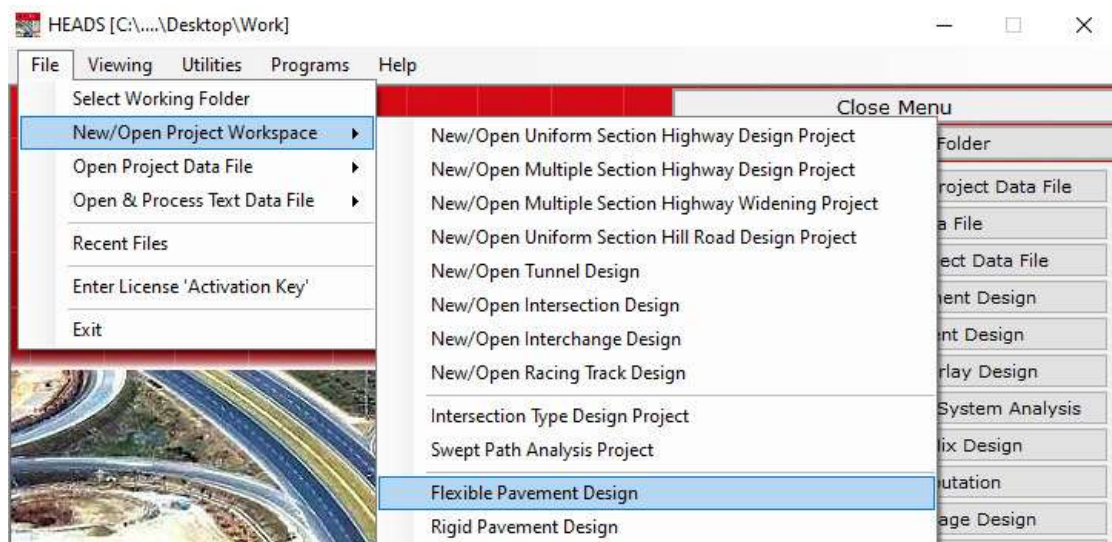
Surfacing = Thin Bituminous Surfacing, 2 Coat Surface Dressing
Base Course = 150 mm
Sub base Course = 260 mm

END OF DESIGN OF FLEXIBLE PAVEMENT BY IRC SP 20 METHOD

Pavement Engineering – User's Guide for Computer Applications

2.4 Computer Aided Design of Flexible Pavement in IRC SP 72 method

The folder 'Work' on the desktop is already selected as the working folder, Select menu item 'File >> New/Open Project Workspace >> Flexible Pavement Design',



(Image 2.21)

The design window opens, Select menu item 'Design of Flexible Pavement by Low Cost Rural Road Standard',

The 'Flexible Pavement Design' window is shown with the following input parameters and design results:

Project Name: Design of Flexible Pavement by AASHTO Standard

Design of Flexible Pavement by AASTHO Method

Analysis Period for Design Life [ANP]: 20 years

Construction Stage(s) [STA]: 2

Overall Reliability [ORE]: 90

Standard Deviation [STD]: 0.45

Soil Resilient Modulus [ZR]: -1.645

Initial Serviceability [PS1]: 4.2

Serviceability Loss [DPE]: 0.0

CBR value of Subgrade in % [CBR]: 5.0 %

Stage Design

Construction Stages	Stage Design Life for each stage [PER] (years)	Equivalent Standard Axle value for each stage [ESA]	Terminal Serviceability at the end of each Stage [PS2]
1	10	4400000	2.5
2	10	11200000	2.0

Major Layer Design

Elastic Modulus of Each Layer [E]	Layer Coefficient of Each Layer [a]	Drainage Coefficient of Each layer [m]	Structural Number of Each Layer [SN]
400000	0.42	1.0	2.8
30000	0.14	1.2	4.2
11000	0.08	1.2	0.0

Buttons: Proceed, Open Input Data File, Open Report File, Close

Pavement Engineering – User's Guide for Computer Applications

Select design item 'Design of Low Cost Gravel Surfaced Rural Road [IRC SP 72]', the sample default data is displayed, change the data as required, click on button 'Proceed',

The screenshot shows the 'Flexible Pavement Design' window. The 'Project Name' is 'Design of Flexible Pavement by Low Cost Rural Road Standard'. The 'Design Options' dropdown is set to 'Design of Low Cost Gravel Surfaced Rural Road [IRC SP 72]'. The input fields are as follows:

Parameter	Value	Unit	Parameter	Value	Unit
Total Number of Animal Drawn Carts per day [ADC]	25	Nos.	Lane Distribution Factor [LDF]	1.0	
Total Number of Bicycles per day [BC]	457	Nos.	Axle Load Survey	<input checked="" type="radio"/> Done <input type="radio"/> Not Done	
Total Number of Motor Cycles per day [MC]	200	Nos.	VDF for Heavy Commercial vehicle [HVD]	5.0	
Total Number of Passenger Cars and Jeeps per day [PC]	15	Nos.	VDF for Medium Commercial vehicle [MVD]	3.0	
Total Number of Heavy Commercial Vehicles per day [HCV]	9	Nos.	Traffic Factor of Non-Harvesting Season and Peak Harvesting Season [FACT]	0.5	
Total Number of Medium Commercial Vehicles per day [MCV]	55	Nos.	Peak Harvesting period [PHT]	75	Days
Construction Time [CT]	2	Years	Number of Harvesting Season in a year [NHS]	1	Nos.
(Time Duration in years between last count and roads opened for Traffic)					
CBR of Subgrade Soil [CBR]	5.0	%			
Annual Growth Rate of Traffic in Percentage [GROWTH]	6.0	%			
Design Life of Road in Years [LIFE]	10	Years			

Buttons at the bottom: Proceed, Open Input Data File, Open Report File, Close.

The design is done and message comes informing about the design report, click on 'OK'

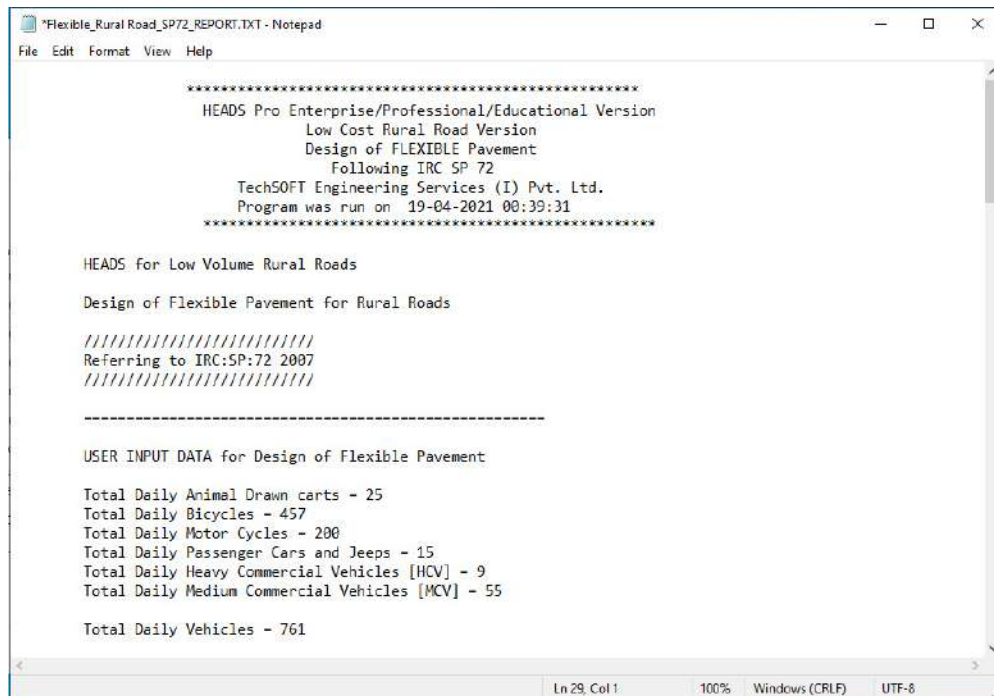
The screenshot shows the same 'Flexible Pavement Design' window as before, but with a message box overlaid. The message box contains the following text:

Design Report is written at
C:\Users\Techsoft\Desktop\Work\Design of Flexible Pavement
by Low Cost Rural Road Standard\Design
Pavements\Flexible_Rural Road_SP72_REPORT.TXT

The 'OK' button is highlighted in the message box.

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Report on Design of Gravel Surfaced Rural Road Pavement Following IRC SP 72, Opened in Notepad, for taking printout.



```
*Flexible_Rural Road_SP72_REPORT.TXT - Notepad
File Edit Format View Help

*****
HEADS Pro Enterprise/Professional/Educational Version
Low Cost Rural Road Version
Design of FLEXIBLE Pavement
Following IRC SP 72
TechSOFT Engineering Services (I) Pvt. Ltd.
Program was run on 19-04-2021 00:39:31
*****

HEADS for Low Volume Rural Roads

Design of Flexible Pavement for Rural Roads

////////////////////
Referring to IRC:SP:72 2007
////////////////////

-----
USER INPUT DATA for Design of Flexible Pavement

Total Daily Animal Drawn carts = 25
Total Daily Bicycles = 457
Total Daily Motor Cycles = 200
Total Daily Passenger Cars and Jeeps = 15
Total Daily Heavy Commercial Vehicles [HCV] = 9
Total Daily Medium Commercial Vehicles [MCV] = 55

Total Daily Vehicles = 761

Ln 29, Col 1 100% Windows (CRLF) UTF-8
```

Report on Design of Flexible Pavement Following IRC SP 72

```
*****
HEADS Pro Enterprise/Professional/Educational Version
Low Cost Rural Road Version
Design of FLEXIBLE Pavement
Following IRC SP 72
TechSOFT Engineering Services (I) Pvt. Ltd.
Program was run on 19-04-2021 00:39:31
*****
```

HEADS for Low Volume Rural Roads

Design of Flexible Pavement for Rural Roads

```
=====
Referring to IRC:SP:72 2007
=====
```

USER INPUT DATA for Design of Flexible Pavement

Total Daily Animal Drawn carts = 25
Total Daily Bicycles = 457
Total Daily Motor Cycles = 200
Total Daily Passenger Cars and Jeeps = 15
Total Daily Heavy Commercial Vehicles [HCV] = 9
Total Daily Medium Commercial Vehicles [MCV] = 55

Total Daily Vehicles = 761

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Construction Time = Time Duration in years between last count and roads opened for Traffic = 2

CBR of Subgrade Soil [CBR] = 5.0

Annual Growth Rate of Traffic in Percentage = [r] = 6.00

Design Life of Road in Years = [y] = 10

Lane Distribution Factor [LDF] = 1.00

VDF for Heavy Commercial vehicle [VDF_HCV] = 5.00

VDF for Medium Commercial vehicle [VDF_MCV] = 3.00

Traffic Factor of Non-Harvesting Season and Peak Harvesting Season [FACTOR] = 0.50

USER'S ASSUMPTIONS:

Peak Harvesting period (in days) = [t] = 75

Number of Harvesting Season in a year (n) = 1

END OF USER INPUT DATA for Design of Flexible Pavement

=====

COMPUTATION OF DESIGN TRAFFIC

Traffic in Peak Harvesting Season = 761

Traffic in Non-Harvesting Season = [T] = Total daily vehicles x FACTOR = 761 x 0.50 = 381

=====

Referring to IRC:SP:72 2007

=====

Annual Average Daily Traffic (AADT) = $T + (1.2 \times n \times T \times t) / 365$

$= 380.50 + (1.2 \times 1.0 \times 381 \times 75) / 365 = 474$

AADT for Road just before opening for Traffic = $[474 \times (1 + 6.00/100)]^{**2.0} = 533$

From Traffic count data

Percentage of HCV = $[9 \times 100] / 761 = 1.18$

Percentage of MCV = $[55 \times 100] / 761 = 7.23$

Number of HCV on AADT = $[1.18 \times 533] / 100 = 6$

Number of MCV on AADT = $[7.23 \times 533] / 100 = 38$

Total number of Commercial Vehicle per Day [CVPD] = HCV + MCV = 6 + 38 = 44

COMPUTATION OF CUMULATIVE ESAL APPLICATIONS

IN CASE : Axle Load Surveys for HCV and MCV are done and VDF data for HCV and MCV are available

ESAL_HCV per day = HCV at the opening year x VDF for HCV = 6 x 5 = 30

ESAL_MCV per day = MCV at the opening year x VDF for MCV = 38 x 3 = 114

ESAL_HCV per day + ESAL_MCV per day = [To] = 144.00

Cumulative ESAL during the Design Life = N

$= To \times ((1 + r)^{**y} - 1) \times LDF \times 365 / r$

$= 144.00 \times ((1 + 0.06)^{**10} - 1) \times 1.00 \times 365 / (1 + 0.06)$

$= 692782$

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DESIGN OF LOW COST PAVEMENT BASED ON SUBGRADE CBR and CUMULATIVE ESAL APPLICATIONS

=====

Referring to Figure 4, IRC:SP:72 2007

=====

For Cumulative ESAL Applications value = 692782

and Subgrade Strength CBR value = 5.00

Proposed Pavement Composition from Bottom to Top

Improved Subgrade 100mm + Granular Subbase 100mm + WBM 150mm + Bit. Surf Treated WBM 75mm

END OF DESIGN OF FLEXIBLE PAVEMENT FOR LOW VOLUME RURAL ROADS.

END OF DESIGN OF FLEXIBLE PAVEMENT BY IRC SP 72 METHOD

References:

1. AASHTO Guide for Design of Pavement Structures 1986, AASHTO
2. AASHTO Guide for Design of Pavement Structures 1993, AASHTO
3. Manual on Pavement Design, Arahan Teknik (Jalan) 5/85, JKR, Malaysia
4. Guidelines for the Design of Flexible Pavements, IRC 37-2001, IRC
5. Guidelines for the Design of Plain Jointed Rigid Pavements for Highways, IRC 58-2002, IRC
6. Tentative Guidelines for Strengthening of Flexible Pavements Using Benkelman Beam Deflection Technique, IRC 81-1997, IRC
7. Guidelines for the design of flexible pavements for low volume rural roads (IRC SP 72-2007)
8. Pavements and Surfacing for Highways and Airports, by Michel Sargious, Applied Science Publishers Ltd.
9. Thickness Design, Asphalt Pavement Structures for Highways and Streets, The Asphalt Institute.
10. Asphalt Pavements for Airports, The Asphalt Institute.
11. Highway Engineering by Paul H. Wright and Karen K. Dixon by John Wiley & Sons. Inc.
12. Principles of Highway Engineering and Traffic Analysis by Fred L. Mannering, Walter and Scott, by Wiley India
13. Software HEADS Pro, by Techsoft Engineering Services, www.techsoftglobal.com and www.roadbridgedesign.com

Computer Aided Design of Rigid Pavement 3

3.1 Computer Aided Design of Rigid Concrete Pavement in AASHTO method

Abstract: *The design procedures described in this chapter are based on the American Association of State Highway and Transportation Officials (AASHTO) Mechanistic Empirical Pavement Design Guide. The definitions and steps relate to AASHTO Mechanistic-Empirical Pavement Design Guide that is used for obtaining the rigid pavement thickness and is the basis for design tables included in this chapter.*

Note: For any query write to techsoftinfra@gmail.com

General

The design and construction of rigid concrete pavements is a fairly complex subject, and it seems desirable to list the major topics that will be covered in this chapter. Major subjects that will be discussed in subsequent sections include materials, proportioning of concrete mixtures, structural pavement design, and the construction of Portland cement concrete pavements.

Types of Rigid Concrete Pavements

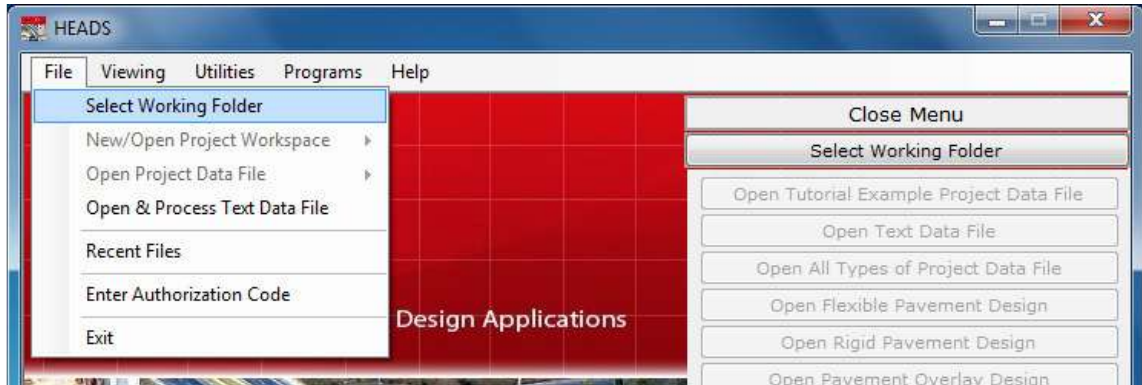
Rigid concrete pavements can be subdivided into three types:

- (i) Jointed reinforced concrete pavements (JRCP)
- (ii) Continuously reinforced concrete pavements (CRCP)
- (iii) Jointed plain concrete pavements (JPCP)
- (iv) Pre-stressed concrete pavement (PCP).

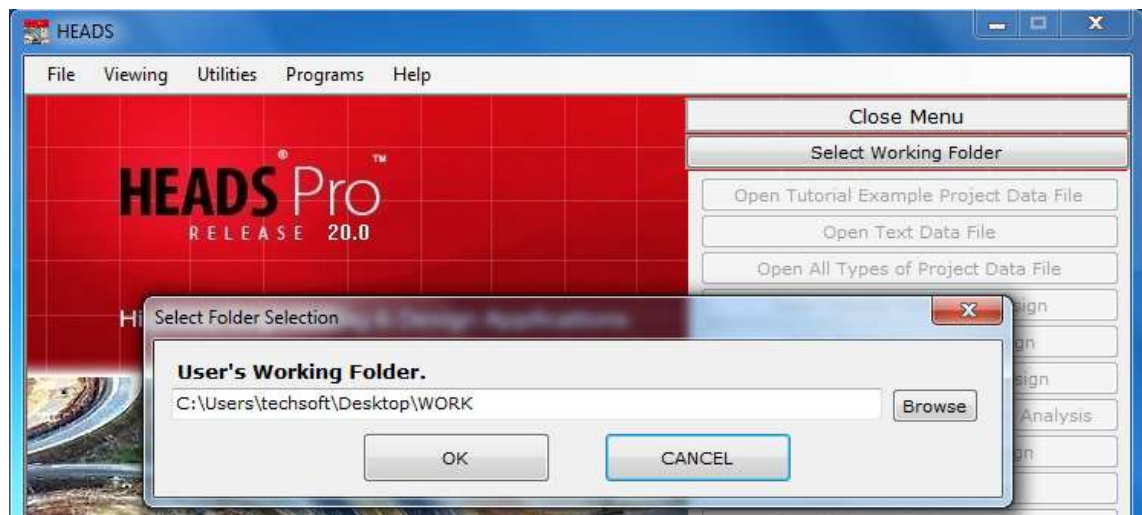
Pavement Engineering – User's Guide for Computer Applications

Computer Aided Design of Rigid Concrete Pavement in AASHTO Method

From main screen of HEADS Pro select 'File' menu item 'Select Working Folder',

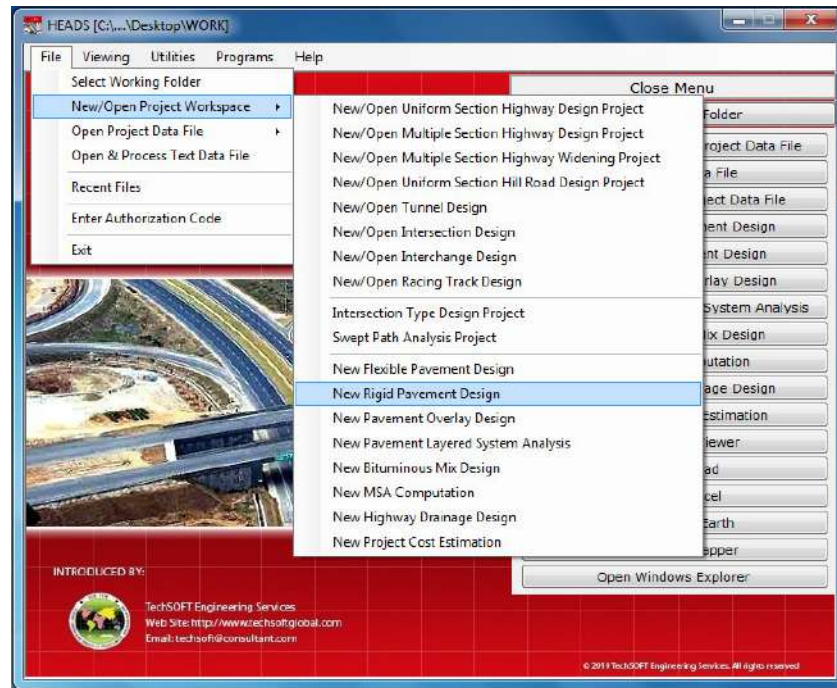


The working folder is selected as 'WORK' located on the Desktop,



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Next, from main screen of HEADS Pro select 'File' menu item 'New/Open Project Workspace', and then select item 'New Rigid Pavement Design'



The first Dialog Box comes for the design of Pavement Thickness in CRCP Rigid Pavement by AASHTO Method, various default data are also given, which user may change as desired (in the licensed version only).

Rigid Pavement Design

Project Name : Design of Rigid Pavement CRCP by AASHTO Standard

RIGID Pavement Type: CRCP (Continuously Reinforced Concrete Pavement)

PAVETHICK LONG BARS TRANS BARS TIEBARS DOWEL BARS

Data required to calculate Pavement Thickness PAVETHICK

Equivalent Standard Axle value [ESA]	17400000.0	psi
Elastic Modulus of Concrete [EC]	4200000.0	psi
Modulus of Rupture of concrete [SC]	650.0	psi
Drainage Coefficient [CD]	1.0	
Load Transfer Coefficient [J]	3.2	
Reliability in percent [REL]	90	%
Standard Deviation [SO]	0.35	
Soil Resilient Modulus [ZR]	0.8	
Initial Serviceability [PO]	4.5	
Terminal Serviceability [Pt]	2.5	
Effective Modulus of Sub-grade Reaction [K]	165	
Loss of Support [LS]	1.0	

Proceed Open Input Data File Open Report File Close

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The next Dialog Box comes for the design of Long Bars in CRCP Rigid Pavement by AASHTO Method, various default data are also given, which user may change as desired (in the licensed version only).

The screenshot shows the 'Rigid Pavement Design' dialog box with the 'LONG BARS' tab selected. The 'Project Name' is 'Design of Rigid Pavement CRCP by AASHTO Standard' and the 'RIGID Pavement Type' is 'CRCP (Continuously Reinforced Concrete Pavement)'. The 'LONG BARS' tab is highlighted with a red arrow. The dialog box contains a table of data required to calculate longitudinal steel reinforcements.

Data required to calculate Longitudinal Steel Reinforcements LONGBARS	
Diameter of the steel bars [PHI]	0.625 inch
Tensile strength of Concrete [FT]	550.0 psi
Concrete Shrinkage [Z]	0.0004 inch per inch
Wheel Load Stress [SIGW]	170.0 psi
Steel Thermal Coefficient [ALPS]	0.000005
Concrete Thermal Coefficient [ALPC]	0.0000038
Allowable Crack Width in the pavement slab [CW]	0.04
Yield Strength for Steel [SIGS]	60000.0 psi
Minimum Crack Spacing [XMIN]	3.5 Feet
Maximum Crack Spacing [XMAX]	8.0 Feet
Design Temperature Drop [DTD]	34.0 degrees Fahrenheit
Width of the Pavement Slab [WS]	16.0 Feet

Buttons at the bottom: Proceed, Open Input Data File, Open Report File, Close.

The next Dialog Box comes for the design of Transverse Bars in CRCP Rigid Pavement by AASHTO Method, various default data are also given, which user may change as desired (in the licensed version only).

The screenshot shows the 'Rigid Pavement Design' dialog box with the 'TRANS BARS' tab selected. The 'Project Name' is 'Design of Rigid Pavement CRCP by AASHTO Standard' and the 'RIGID Pavement Type' is 'CRCP (Continuously Reinforced Concrete Pavement)'. The 'TRANS BARS' tab is highlighted with a red arrow. The dialog box contains a table of data required to calculate transverse steel reinforcements.

Data required to calculate Transverse Steel Reinforcements TRANBARS	
Diameter of the steel bars required for the Transverse Reinforcement [PHI]	0.40 inch
Unit Weight of Concrete [GAMC]	0.0805 pci
Average Coefficient of Friction between the Sub-grade and the pavement Slab [FA]	1.5
Allowable Steel Stress [FS]	40000.0 psi

Buttons at the bottom: Proceed, Open Input Data File, Open Report File, Close.

Pavement Engineering – User's Guide for Computer Applications

The next Dialog Box comes for the design of Tie Bars in longitudinal joints in CRCP Rigid Pavement by AASHTO Method, various default data are also given, which user may change as desired (in the licensed version only).

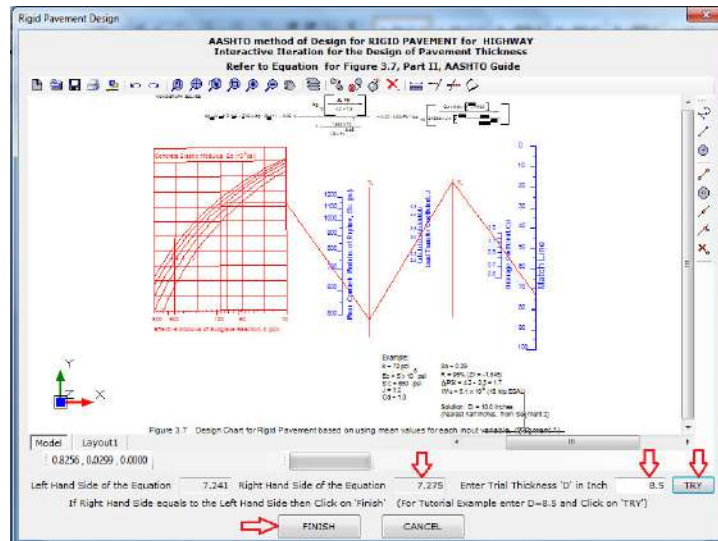
The screenshot shows the 'Rigid Pavement Design' dialog box. The 'Project Name' is 'Design of Rigid Pavement CRCP by AASHTO Standard'. The 'RIGID Pavement Type' is 'CRCP (Continuously Reinforced Concrete Pavement)'. The 'TIEBARS' tab is selected, indicated by a red arrow. Below the tabs, the text 'Data required to calculate Tie Bar Reinforcements TIEBARS' is displayed. A single input field is shown: 'Diameter of the Tie Bars [PHI]' with a value of '0.472 inch'. At the bottom, there are four buttons: 'Proceed', 'Open Input Data File', 'Open Report File', and 'Close'.

The next Dialog Box comes for the design of Dowel Bars in the transverse joints in CRCP Rigid Pavement by AASHTO Method, various default data are also given, which user may change as desired (in the licensed version only).

The screenshot shows the 'Rigid Pavement Design' dialog box. The 'Project Name' is 'Design of Rigid Pavement CRCP by AASHTO Standard'. The 'RIGID Pavement Type' is 'CRCP (Continuously Reinforced Concrete Pavement)'. The 'DOWEL BARS' tab is selected, indicated by a red arrow. Below the tabs, the text 'Data required to calculate Dowel Bars DOWELBARS' is displayed. Three input fields are shown: 'Diameter of the Dowel Bars [PHI]' with a value of '1.25 inch', 'Spacing of the Dowel Bars [YD]' with a value of '12.0 inch', and 'Length of the Dowel Bars [LD]' with a value of '18.0'. At the bottom, there are four buttons: 'Proceed', 'Open Input Data File', 'Open Report File', and 'Close'.

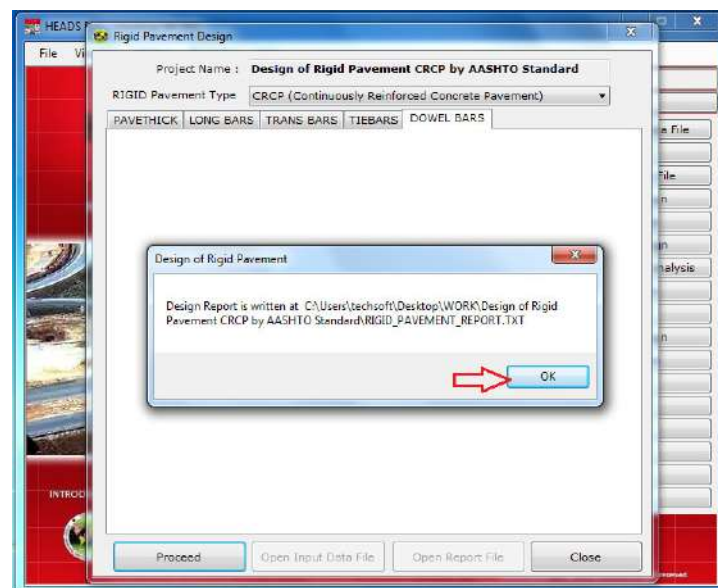
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The next Dialog Box comes for the design of CRCP Rigid Pavement by AASHTO Method. A trial value of pavement thickness, along with various previous data, is to be entered by the user to make the values for the left and right side of the equation equal.



Once the values for the left and right side of the equation equal, with some value of the pavement thickness, the job is complete, and click on button 'FINISH'.

The message comes mentioning about the design report for the design of CRCP Rigid Pavement by AASHTO Method. User may open and view the design report.



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HEADS Pro Enterprise/Professional/Educational Version

Report on Rigid Pavement Design

Program was run on 8/8/2017 2:32:19 PM
TechSOFT Engineering Services.

Design of Flexible Pavement according to the Standard :
AASHTO Guide for Design of Pavement Structures - Parts I, II and III.

Pavement Type : Continuously Reinforced Concrete Pavement (CRCP).

THE DESIGN OF THE PAVEMENT THICKNESS FOR THE SLAB :

Input Data :

W18	= 18-Kip ESAL	= 17400000.000
Sc	= Modulus of Rupture	= 650.000
J	= For CRCP, without tied Shoulder	= 3.200
Po	= Initial Serviceability	= 4.500
Pt	= Terminal Serviceability	= 2.500
DEL-PSI	= Po - Pt	= 2.000
Rel	= Reliability	= 90.000
So	= Standard Deviation	= 0.350
k	= Subgrade Modulus	= 165.000
Ls	= Potential Loss of Support	= 1.000

Design of the Pavement Thickness :

Now, from figure 3.7 of AASHTO Guide, Part-II,
in regards to the Nomograph the associated Equation
is as follows :

W18 = ESA = 17400000.00

$$\begin{aligned} \log_{10}(W18) = & Z_r \times S_o + 7.35 \times \log_{10}(D+1) - 0.06 \\ & + \frac{\log_{10}[\text{DEL-PSI} / (4.5 - 1.5)]}{1 + \frac{7}{1.624 \times 10^8 / (D+1)}} \\ & + (4.22 - 0.32 \times p_t) \times \log_{10} \frac{0.75 \times S_c \times C_d \times [D - 1.132]}{215.63 \times J \times [D - 18.42 / (E_c/k)]^{0.25}} \end{aligned}$$

Solving this equation by trial and error method by assuming various values of 'D' and comparing the values of the left and right hand sides of the equation, until both the sides are equal upto one decimal place, the final value of 'D' is obtained as follows :

With D = 8.50 inch. from the above equation,
Left hand side of the equation = 7.24055 and
Right hand side of the equation = 7.27464

Therefore, designed Pavement Thickness D = 8.50 inch.

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THE DESIGN OF THE LONGITUDINAL REINFORCEMENTS FOR THE SLAB :

Input Data :

D	=	Thickness of the Pavement Slab	=	8.500	inch.
phi	=	Diameter of Bars	=	0.625	inch.
ft	=	Concrete Tensile Strength	=	550.000	psi.
z	=	Concrete Shrinkage	=	0.00040	inch/inch.
sigma_w	=	Wheel Load Stress	=	170.00	psi.
alpha_s	=	Coeff. of Thermal Expan. of Steel	=	0.00000500	
alpha_c	=	Coeff. of Thermal Expan. of Concrete	=	0.00000380	
alpha_rat	=	alpha_s / alpha_c	=	1.316	
cw	=	Allowable Crack Width	=	0.04000	inch.
sigma_s	=	Allowable Steel Stress	=	60000.000	psi.
Xmin	=	Minimum Crack Spacing	=	3.500	ft.
Xmax	=	Maximum Crack Spacing	=	8.000	ft.
DTD	=	Design Temperature Drop	=	34.000	deg. F.
ws	=	width of Pavement Slab	=	16.000	Ft.

Design of the Longitudinal Reinforcement of the Pavement :

Percentage of minimum and maximum reinforcements will be determined from the three following criteria :

- (i) from crack spacing : Maximum and Minimum Long. Reinforcement.
- (ii) from crack width : Minimum Long. Reinforcement.
- (iii) from steel stress : Minimum Long. Reinforcement.

The Minimum reinforcement will be calculated from the minimum reinforcement percentage values of (i), (ii) & (iii)
(i) From the Crack Spacing Criterion :

Using the minimum Crack Spacing as Xmin = 3.50 ft.

Maximum Percentage of Longitudinal Reinforcements 'p1_max' is :

$$p1_max = \frac{1.062 \times (1 + ft/1000)^{1.457} \times (1 + \alpha_s/(2 \times \alpha_c))^{0.25} \times (1 + \phi)^{0.476}}{Xmin^{0.217} \times (1 + \sigma_w/1000)^{1.13} \times (1 + 1000 \times z)^{0.389}} - 1$$

$$= 0.61$$

Again,

Using the maximum Crack Spacing as xmax = 8.00 ft.

Minimum Percentage of Longitudinal Reinforcements 'p1_min' is :

$$p1_min = \frac{1.062 \times (1 + ft/1000)^{1.457} \times (1 + \alpha_s/(2 \times \alpha_c))^{0.25} \times (1 + \phi)^{0.476}}{Xmax^{0.217} \times (1 + \sigma_w/1000)^{1.13} \times (1 + 1000 \times z)^{0.389}} - 1$$

$$= 0.35$$

(ii) From the Crack Width Criterion :

Minimum Percentage of Longitudinal Reinforcements 'p2_min' is :

$$p2_min = \frac{1.435 \times 0.484}{0.358 \times (1 + ft/1000) \times (1 + \phi)} - 1 = 0.46$$

$$\frac{0.22}{cw} \times (1 + \sigma_w/1000)^{1.1079}$$

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(iii) From the Steel Stress Criterion :

Minimum Percentage of Longitudinal Reinforcements 'p3_min' is :

$$p3_min = \frac{0.155 \times 50.834 \times (1 + DTD/1000)^{1.493} \times (1 + ft/1000)}{0.365 \times \sigma_s \times (1 + \sigma_w/1000)^{1.146} \times (1 + z \times 1000)^{0.18}} - 1 = 0.45$$

The minimum percent of reinforcement is the maximum of the three minimum reinforcement percent values p1_min, p2_min & p3_min, which is equal to 0.46

The maximum percent of reinforcement is equal to 0.61

The Required number of Longitudinal Reinforcements :

$$Nmin = 0.01273 \times pmin \times (ws*12) \times D \times \phi = 25.00 \text{ nos.}$$

$$Nmax = 0.01273 \times pmax \times (ws*12) \times D \times \phi = 32.43 \text{ nos.}$$

Using 25 nos. of bars in 16.000 Ft. width, spacing = 7.680 inch c/c.

CHECKING OF VARIOUS PARAMETERS :

(i) Crack Spacing =

$$= \frac{1.32 \times (1 + ft/1000)^{6.70} \times (1 + \alpha_s/(2*\alpha_c))^{1.15} \times (1 + \phi)^{2.19}}{4.6 \times (1 + pmin)^{5.2} \times (1 + \sigma_w/1000)^{1.79} \times (1 + 1000*z)} - 1$$

$$= 5.55 \text{ Ft.}$$

Specified Minimum and Maximum Crack Spacing = 3.50 Ft. and 8.00 Ft.

(ii) Crack width =

$$= \frac{0.00932 \times (1 + ft/1000)^{0.53} \times (1 + \phi)^{2.20}}{(1 + pmin)^{4.55} \times (1 + \sigma_w/1000)^{4.91}} - 1$$

$$= 6 \text{ Ft.}$$

Allowable Crack width as specified = 0.040000 Ft.

(iii) Steel Stress =

$$= \frac{47300 \times (1 + DTD/1000)^{0.425} \times (1 + ft/1000)^{4.09}}{(1 + pmin)^{2.74} \times (1 + \sigma_w/1000)^{3.14} \times (1 + z \times 1000)^{0.494}} - 1$$

$$= 59473.98 \text{ psi.}$$

Allowable Steel Stress as specified = 60000.00 psi.

THE DESIGN OF THE TRANSVERSE REINFORCEMENTS FOR THE SLAB :

Input Data :

D	= Thickness of the Pavement Slab	= 8.500 inch.
gama_c	= Unit weight of Concrete	= 0.081 pci.
ws	= width of Pavement Slab	= 16.000 Ft.
fs	= Allowable Steel Stress	= 40000.000 psi.
fa	= Average Coeff. of Friction	= 1.500
phi_trans	= Diameter of Transverse Bars	= 0.400 inch.

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Design of the Transverse Reinforcement of the Pavement :

$$\begin{aligned}\text{Allowable Stress} &= 0.67 \times \text{Yield Stress} \\ &= 0.67 \times \sigma_s \\ &= 0.67 \times 60000.000 \\ &= 40000.000 \text{ psi.}\end{aligned}$$

$$\text{Area of Steel} = \frac{\text{gama_c} \times D \times (\text{Ws} \times 12) \times \text{fa}}{2.0 \times \text{fs}}$$

$$\begin{aligned}&= 0.002466 \text{ sq. inch. / inch.} \\ \text{Area of one bar of diameter 0.40 inch.} \\ &= 3.1416 \times \text{phi_trans} \times \text{phi_trans} / 4.0 = 0.125664 \text{ sq. inch.}\end{aligned}$$

Spacings of Transverse Reinforcements :

$$= 0.125664 / 0.002466 = 50.951 \text{ inch.}$$

THE DESIGN OF THE TIE BARS FOR THE SLAB :

Input Data :

D	= Thickness of the Pavement Slab	= 8.500 inch.
gama_c	= Unit Weight of Concrete	= 0.081 pci.
ws	= width of Pavement Slab	= 16.000 Ft.
fs	= Allowable Steel Stress	= 40000.000 psi.
fa	= Average Coeff. of Friction	= 1.500
phi_ti	= Diameter of Tie Bars	= 0.472 inch.

Design of the Tie Bars for the Pavement :

$$\begin{aligned}\text{Allowable Stress} &= 0.67 \times \text{Yield Stress} \\ &= 0.67 \times \sigma_s \\ &= 0.67 \times 60000.000 \\ &= 40000.000 \text{ psi.}\end{aligned}$$

$$\text{Area of Steel} = \frac{\text{gama_c} \times D \times (\text{Ws} \times 12) \times \text{fa}}{\text{fs}}$$

$$\begin{aligned}&= 0.004933 \text{ sq. inch. / inch.} \\ \text{Area of one bar of diameter 0.47 inch.} \\ &= 3.1416 \times \text{phi_tie} \times \text{phi_tie} / 4.0 = 0.174975 \text{ sq. inch.}\end{aligned}$$

Spacings of Tie Bars :

$$= 0.174975 / 0.004933 = 35.472 \text{ inch.}$$

DESIGN OF THE DOWEL BARS FOR THE PAVEMENT :

Input Data :

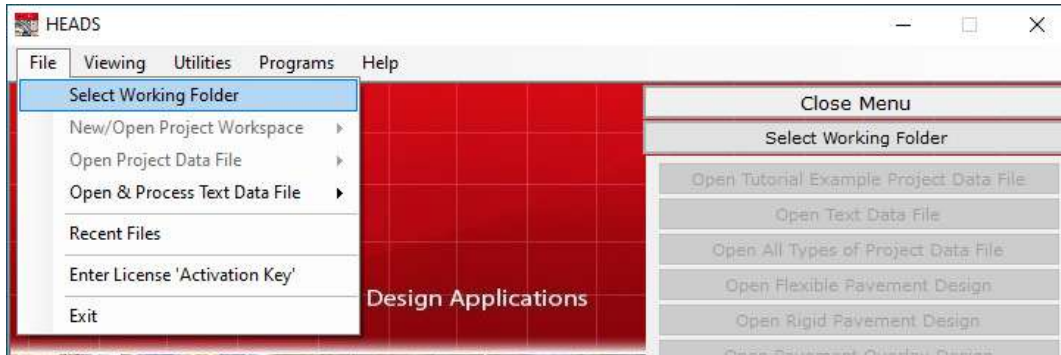
Referring to Table 4.4, Part II, AASHTO Guide

D	= Thickness of the Pavement Slab	= 8.500 inch.
Diameter of the Dowel Bars	= 1.25 inch.	
Spacings of the Dowel Bars	= 12.00 inch.	
Length of the Dowel Bars	= 18.00 inch.	

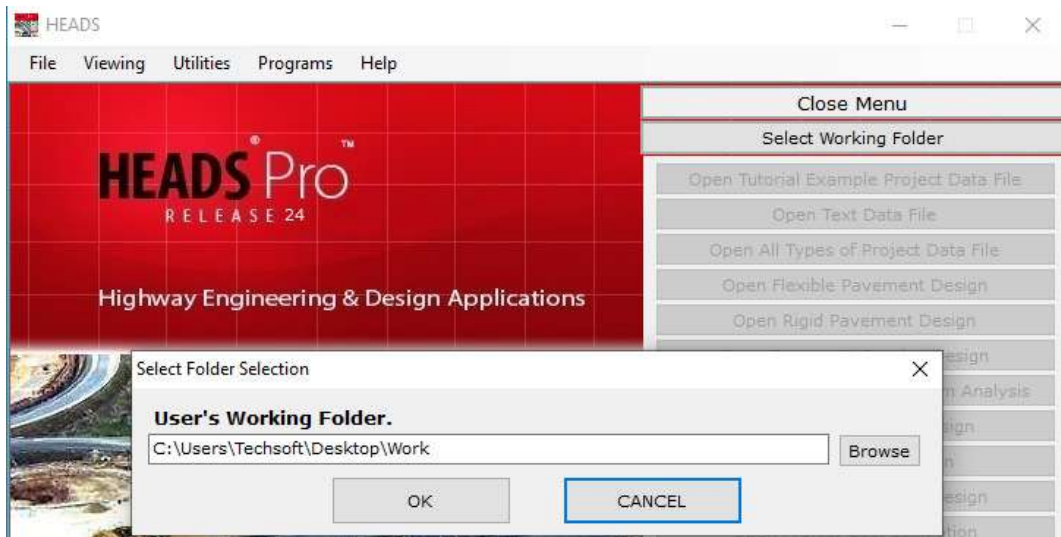
END OF DESIGN

3.2 Computer Aided Design of Rigid Concrete Pavement in IRC 58 Method

From main screen of HEADS Pro select 'File' menu item 'Select Working Folder',

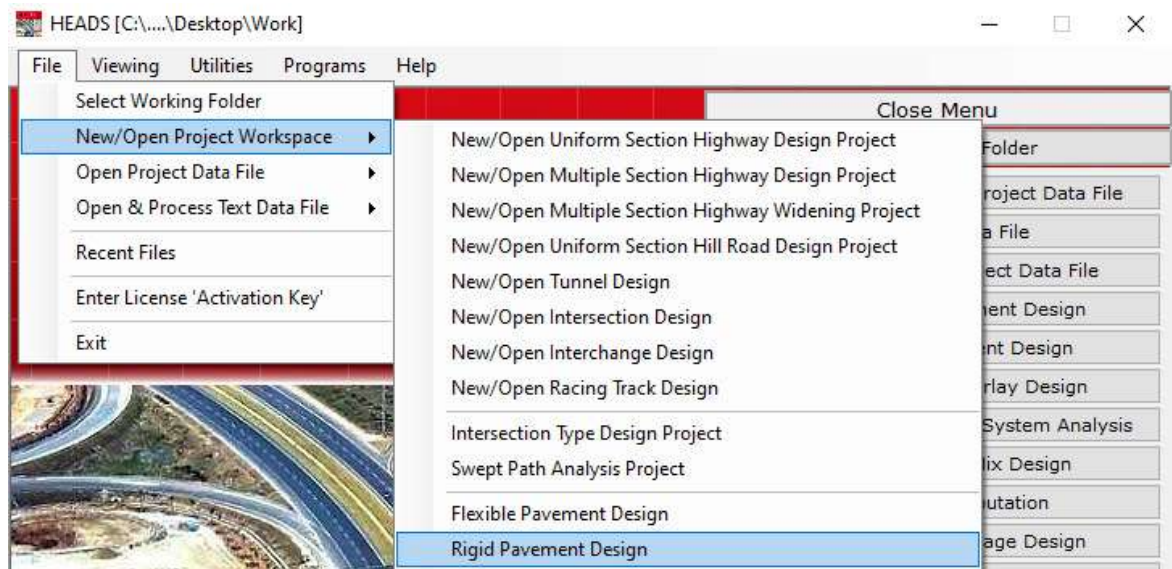


Select folder 'Work' on the Desktop as the working folder,

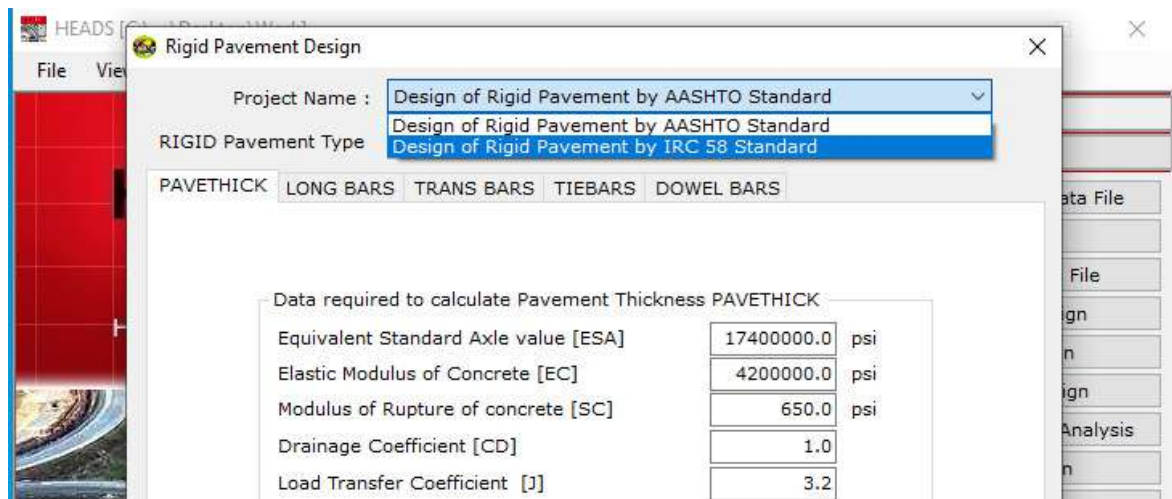


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Select menu item 'File', then 'New/Open Project Workspace', then 'Rigid Pavement Design',

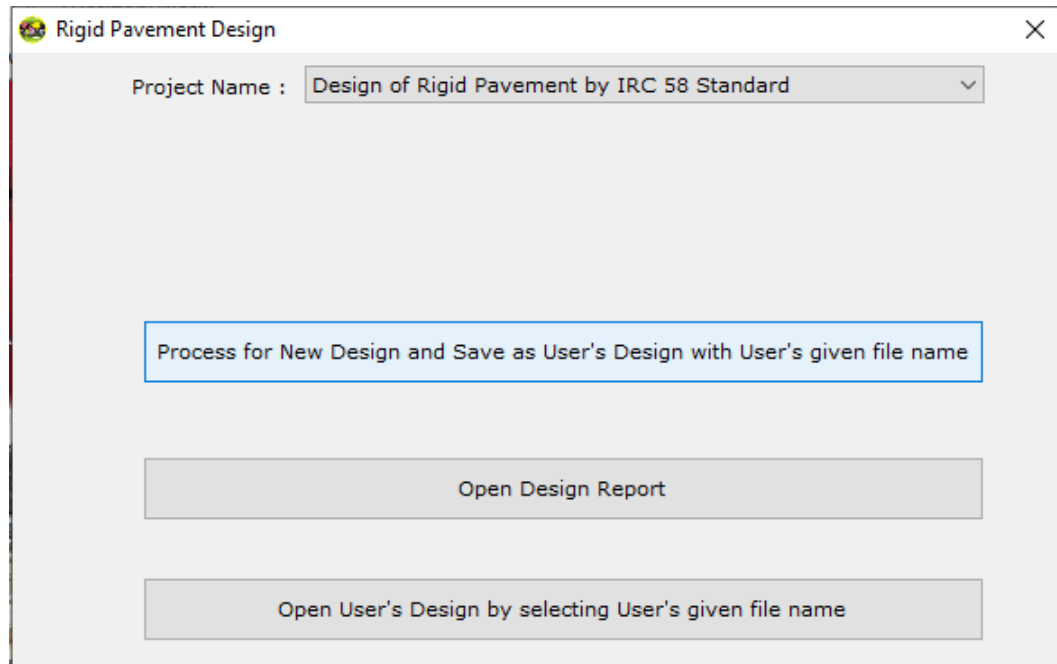


The window comes, further select option for 'Design of Rigid Pavement by IRC 58 Standard'.



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The next window opens for design of rigid pavement in IRC 58 Standard, Click on button 'Process for New Design....',

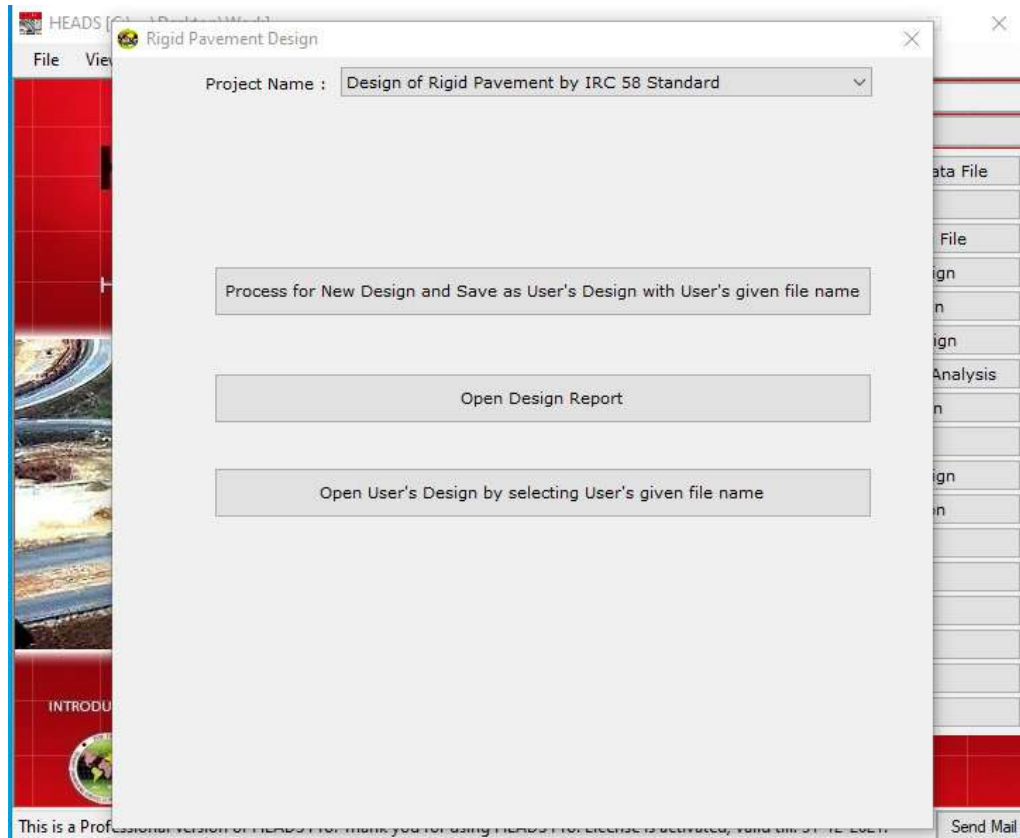


The Excel Worksheet opens with various default input data and embedded formula, User may change any input data as desired the design will change instantly, Save the file on the desktop by giving a desired file name. Close the Excel file. The file may be opened next time by using the button 'Open User's Design....',

PROJECT		DESIGNED	CHECKED	DATE	SHEET NO
Design of Rigid Pavement By IRC 58					
Design wheel load	P	5200 kg			As per IRC-6-2016 (11.2/2=5.1ton ,class A loading table 2)
Modulus of sub grade	K	21 kg/cm ²			AS per IRC58-2015 - (CBR -3 table 2).
Grade of concrete	Fck	40 N/mm ²			
Youngs Modulus	E	322352 Kg/Cm ²			
Allowable Flexural strength of concrete		46.12934479 Kg/Cm ²			
Design tyre Pressure	Pt	0.154643935 Kg/Cm ²			0.8 Mpa- As per IRC58-2015 - (clause- 5.2).
Radius of Contact Area (a)	a	22.72058116 Cm			
Poissons ratio of concrete		0.15			
Coefficient of Thermal expansion	α	0.000119			
Assumed maximum variation of temperature	Δt	18 °C			As per IRC58-2015 - (Table 1).
Coeff of friction between pavement and subgrade	f	1.5			
Length of contraction joint		4.5 m			
Assumed thickness of slab (h)	h	20 Cm			
Radius of relative stiffness(L)	L	56.88213886 Cm			
Radius of equivalent distribution of pressure	b	21.51370722 Cm			

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If the file is saved without giving a separate name it may be opened by using the button 'Open Design Report'. But next time if user uses the same working folder and wants a new design the last design will be overwritten, as it was not saved on desktop with a different name.



Design of Rigid Pavement By IRC 58

Design wheel load	P	5200	kg
Modulus of sub grade	K	21	kg/cm ³
Grade of concrete	Fck	40	N/mm ²
Youngs Modulus	E	322352	Kg/Cm ²
Allowable Flexural strength of concrete		45.12934479	Kg/Cm ²
Design tyre Pressue	P_t	8.154943935	Kg/Cm ²
Radius of Contact Area (a)	a	22.72058116	Cm
Poissons ratio of concrete		0.15	
Coefficient of Thermal expansion	α	0.000010	
Assumed maximum variation of temperature	Δt	19	°C
Coeff of friction between pavement and subgrade	f	1.5	
Length of contraction joint		4.5	m
Assumed thickness of slab (h)	h	20	Cm
Radius of relative stiffness(L)	L	56.88213886	Cm

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Radius of equivalent distribution of pressure	b	21.51370722	Cm
Slab length	L	2.5	m
Slab width	W	2.7	m

Stress Calculations

Stress at Corner	S_c	19.34786098	Kg/Cm ²
Stress at Interior	S_i	11.33003924	Kg/Cm ²
Temperature stress at edge region	S_{te}	19.4765358	Kg/Cm ²
Residual Concrete Strength		25.65280899	Kg/Cm ²
Load stress for edge region	S_{le}	19.45456194	Kg/Cm ²
factor of safety		1.159210693	
Traffic classification		D	
Required thickness adjustments		-2	Cm
Thickness of the pavement	d	18	Cm

Design of Dowels

permissible tensile strength in plain bars	F_t	1250	Kg/Cm ²
permissible tensile strength in deformed bars	F_t	2000	Kg/Cm ²
Assumed Dowel Diameter	d	3.2	Cm
Joint width(z)	z	2.5	Cm
Grade of steel	F_y	250	N/mm ²
Permissible bearing strength in concrete	F_b	297.9444083	Kg/Cm ²
Assumed spacing between dowel bars	s	30	Cm
Assumed length of dowel bars	L_d	50	Cm
No of dowells participating in load transfer	N	2	bars
Capacity factor		1.417781015	
Load carried by outer dowell bar		1467.08129	kg
Moment of inertia of dowel bar	I	5.144576	Cm ⁴
Relative stiffness of bar embedded in concrete	β	0.289799553	Cm
Bearing stress in dowell bar		99.75808741	Kg/Cm ²

Design of tie bars

Slab width		2.7	m
Assuming dia of tie bars		8	mm
Coeff of friction between pavement and subgrade		1.5	
Tensile stress in steel (deformed bars)		2000	Kg/Cm ²
Tensile stress in steel (plain bars)		1250	Kg/Cm ²
Maximum permissible bond stress			
1.Plain tie bars		17.5	Kg/Cm ²
2.Deformed bars		24	Kg/Cm ²

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Area of steel required per m width of joint	1.555	Cm ² /m
c/s Area of one bar	0.502	Cm ²
Perimeter of one bar	2.512	Cm
Spacing of tie bars (required)	32.30452675	Cm
spacing of tie bars (provided)	32	Cm
Length of tie bar (plain bars)	28.57142857	Cm
Length of tie bar(deformed bars)	33.33333333	Cm
Tolerance for bond loss due to painting	10	Cm
Tolerance in placement	5	Cm
Length of tie bar (plain bars) required	43.57142857	Cm
Length of tie bar(deformed bars) required	48.33333333	Cm

This is the end of the session for Computer Aided Design for Rigid Pavement.

Construction of Flexible Bituminous Pavement 4

Abstract: *This chapter describes the construction of Flexible Bituminous Pavements*

General

A highway pavement is a structure consisting of superimposed layers of processed materials above the natural soil sub-grade, whose primary function is to distribute the applied vehicle loads to the sub-grade. The pavement structure should be able to provide a surface of acceptable riding quality, adequate skid resistance, favorable light reflecting characteristics, and low noise pollution. The ultimate aim is to ensure that the transmitted stresses due to wheel load are sufficiently reduced, so that they will not exceed bearing capacity of the sub-grade. Two types of pavements are generally recognized as serving this purpose, namely flexible pavements and rigid pavements. This paper gives an overview of pavement layers, and their Construction



Figure 4.1 – Construction of Flexible Bituminous Pavement

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Structural components of Bituminous Flexible Pavements

Starting from bottom such roads consist of following components:

Typical layers of a flexible pavement

- Sub-grade,
- Sub-Base course,
- Base course, WBM (Low Traffic), WMM (High Traffic)
- Prime Coat-Dried-Tack Coat then Binder course, PMC (Low Traffic), DBM (High Traffic)
- Tack Coat then Surface course BC (High Traffic) on open graded PMC 20mm, Seal Coat 20mm.

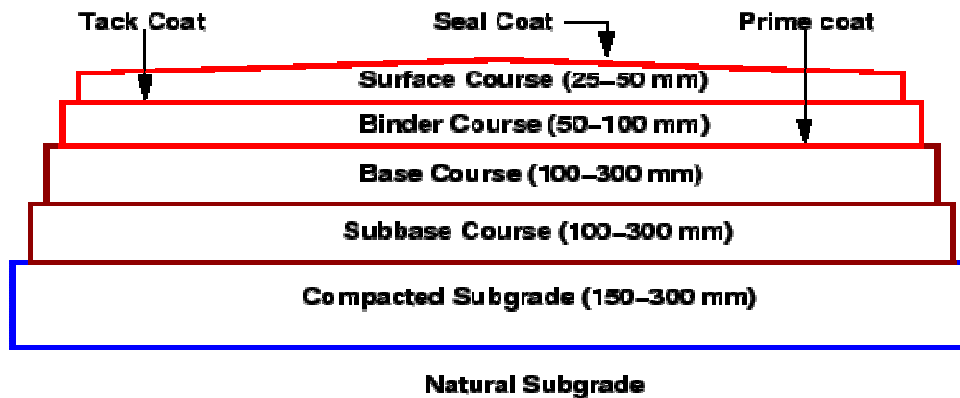


Figure 4.2 – Construction of Flexible Bituminous Pavement

4.1 Computer Applications for Processing Marshall Test Results and Design of Asphalt Concrete Mix using Excel Worksheet Reports

Download HEADS Pro from web site www.techsoftgloabl.com and install in the PC/Laptop. From main screen of HEADS Pro select menu item 'File >> Select Working Folder',

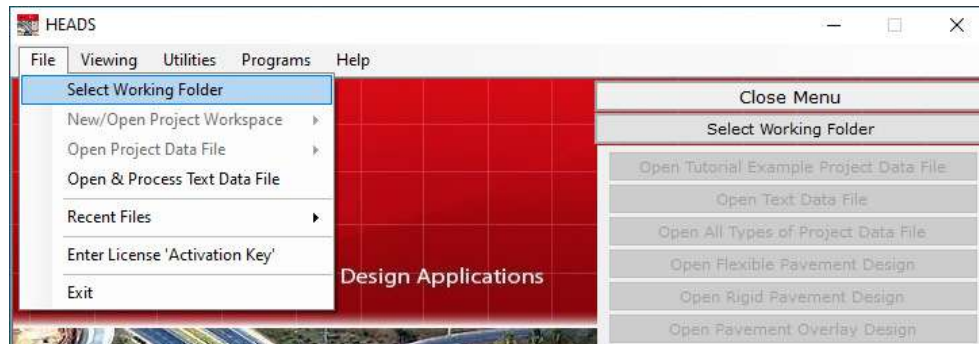


Image4.1

The folder 'Work' on the Desktop is selected as the 'Working Folder', click on 'OK',

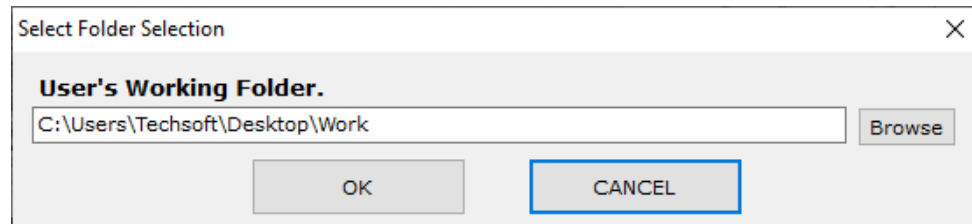


Image4.2

Next, select HEADS Pro menu item 'File>>New/Open Project Workspace>>Bituminous Mix Design,

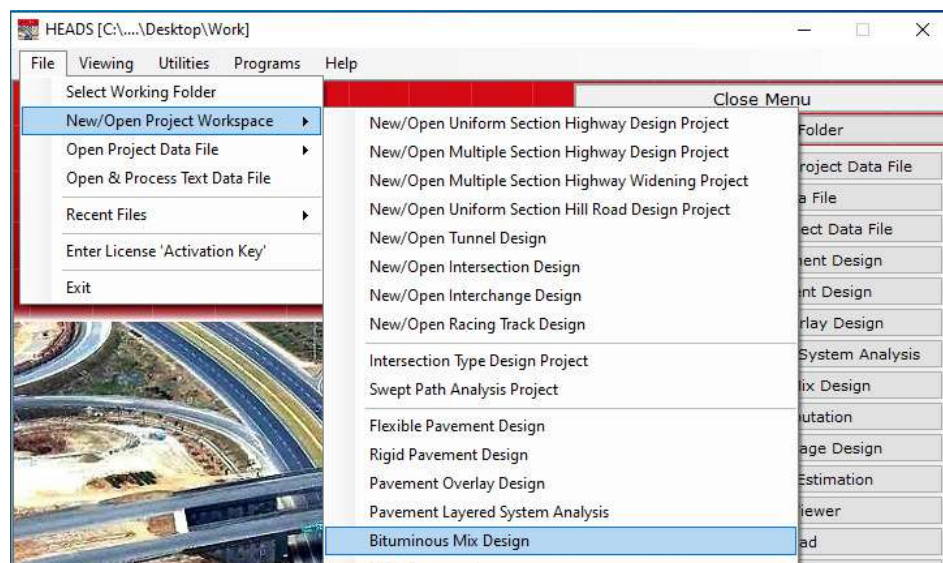


Image4.3

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The design workspace opens, the current Tab at the top is '[1]:Create Project', click on 'New Project' and use the Tabs in sequence to produce the design reports as a set of Excel files.

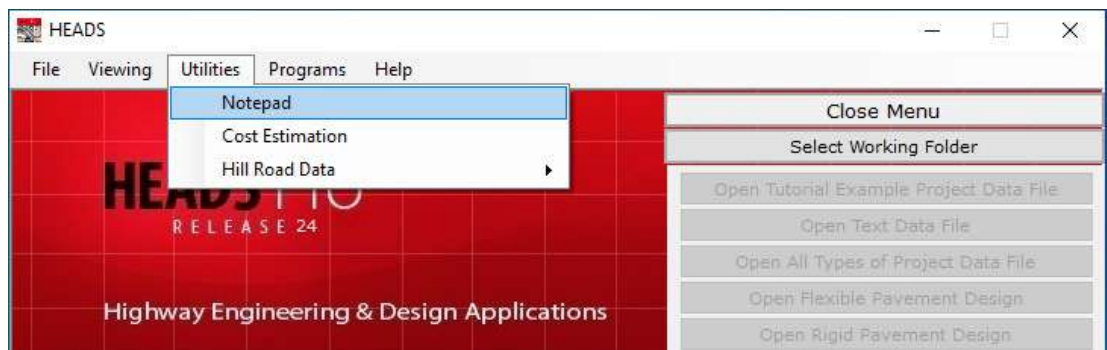
The screenshot shows the 'Bituminous MIX Design' application window. The top tab bar includes: [1]:Create Project, [2]:Bitumen Properties VG-40, [3]:Aggregate Grading, [4]:Individual Grading, [5]:Grading & Blending, [6]:Marsh, and a scroll button. The 'Project' section contains a 'JOB Name' field with 'DESIGN JOB #01' and 'New Project' and 'Open Project' buttons. Below this is a larger form with fields for 'Project Name' (containing 'Bituminous MIX Design Project'), 'Client' (containing 'Client's Name'), 'Independent Consultant' (containing 'Consultant's Name'), 'Concessionaire' (containing 'Concessionaire's Name'), and 'EPC Contractor' (containing 'EPC Contractor's Name'). A 'Note' section at the bottom states: 'User must process various steps [2] through [8] in proper sequence. Before Processing a particular step all the Excel files must be closed, otherwise Results will be INCORRECT. Values in Red color are taken from User's Input in the Dialog Box, the values in Blue color are taken from Previous Process. Values in Black color are calculated in the current page. All the Final Excel files are available under the tab "Report".'

Image4.4

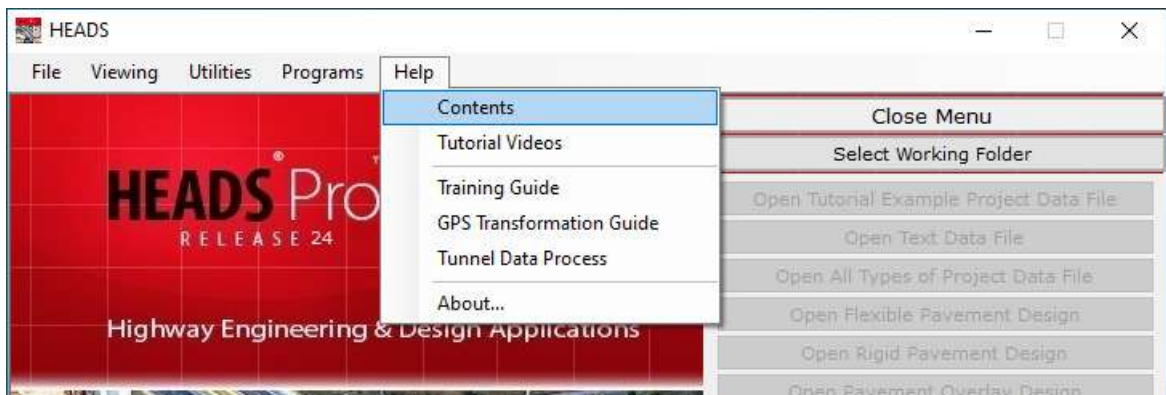
Alternatively,

4.2 Computer Applications for Processing Marshall Test Results and Design of Asphalt Concrete Mix using Text Data Files and Reports

In HEADS Pro main screen select menu item 'Utilities >> Notepad' and open blank Notepad,

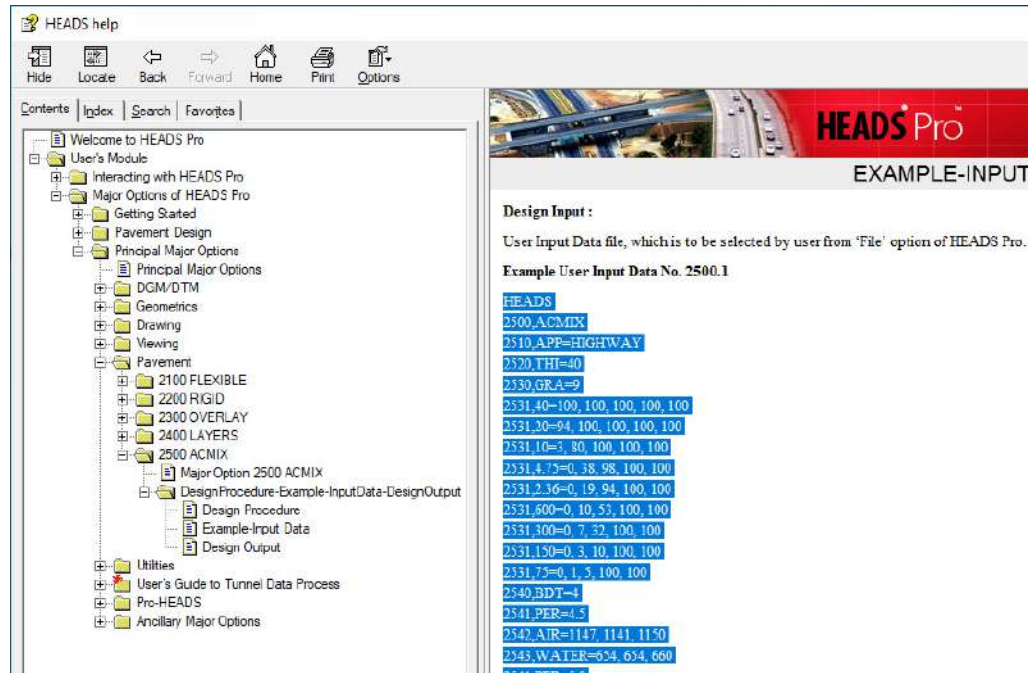


Next, select menu item 'Help >> Content',

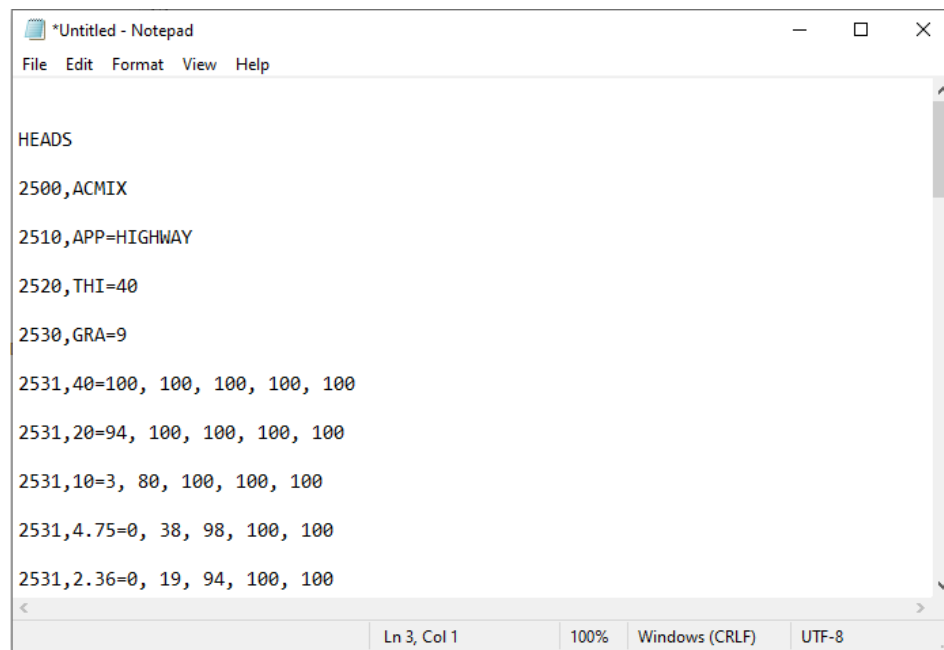


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As 'Help' window opens follow the path as shown below, the ACMIX design data is displayed on the right panel of 'Help' window. Each data starts with 'HEADS' and ends with 'FINISH', Select and copy the first data from HEADS to FINISH. The data is explained below the same right panel.

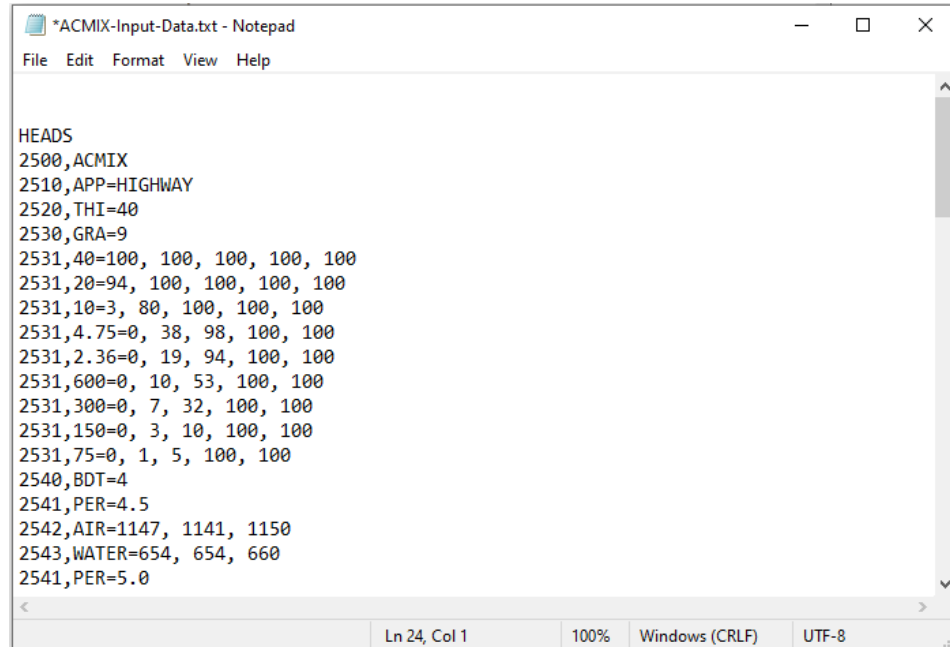


Paste the data in the Notepad, already opened and close 'HEADS-Help'. The data has gap between each two lines in the Notepad,



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Remove the gaps between the lines and save the data as text file ACMIX-Input-Data.txt in folder 'Work' on the Desktop.



```
*ACMIX-Input-Data.txt - Notepad
File Edit Format View Help

HEADS
2500,ACMIX
2510,APP=HIGHWAY
2520,THI=40
2530,GRA=9
2531,40=100, 100, 100, 100, 100
2531,20=94, 100, 100, 100, 100
2531,10=3, 80, 100, 100, 100
2531,4.75=0, 38, 98, 100, 100
2531,2.36=0, 19, 94, 100, 100
2531,600=0, 10, 53, 100, 100
2531,300=0, 7, 32, 100, 100
2531,150=0, 3, 10, 100, 100
2531,75=0, 1, 5, 100, 100
2540,BDT=4
2541,PER=4.5
2542,AIR=1147, 1141, 1150
2543,WATER=654, 654, 660
2541,PER=5.0

Ln 24, Col 1    100%    Windows (CRLF)    UTF-8
```

The input data in the file 'ACMIX-Input-Data.txt' is as below:

```
HEADS
2500,ACMIX
2510,APP=HIGHWAY
2520,THI=40
2530,GRA=9
2531,40=100, 100, 100, 100, 100
2531,20=94, 100, 100, 100, 100
2531,10=3, 80, 100, 100, 100
2531,4.75=0, 38, 98, 100, 100
2531,2.36=0, 19, 94, 100, 100
2531,600=0, 10, 53, 100, 100
2531,300=0, 7, 32, 100, 100
2531,150=0, 3, 10, 100, 100
2531,75=0, 1, 5, 100, 100
2540,BDT=4
2541,PER=4.5
2542,AIR=1147, 1141, 1150
2543,WATER=654, 654, 660
2541,PER=5.0
2542,AIR=1160, 1158, 1153
2543,WATER=668, 668, 667
2541,PER=5.5
2542,AIR=1160, 1160, 1157
2543,WATER=674, 672, 670
2541,PER=6.0
2542,AIR=1162, 1169, 1163
2543,WATER=670, 675, 671
2550,STT=4
2551,PER=4.5
2552,STA=789, 809, 786
```

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```
2553,VOL=493, 487, 490
2551,PER=5.0
2552,STA=992, 950, 962
2553,VOL=492, 490, 486
2551,PER=5.5
2552,STA=892, 923, 934
2553,VOL=492, 488, 487
2551,PER=6.0
2552,STA=828, 833, 834
2553,VOL=492, 494, 492
2560,FLT=4
2561,PER=4.5
2562,FLO=35, 35, 35
2561,PER=5.0
2562,FLO=37, 35, 35
2561,PER==5.5
2562,FLO=40, 35, 40
2561,PER=6.0
2562,FLO=40, 45, 40
FINISH
```

There are two more text files used in the design of Bituminous mix, these files are ACMIX.TXT and CORRFACT.TXT. These two files should exist in the application folder of 'HEADS Pro', as:

"C:\Program Files (x86)\HEADS Pro Release 24\Application",

The data in file "ACMIX.HDS", is opened in Notepad and is shown below. The data is about the various specifications for highway and airfield pavement in respect of "thickness, number of blows, minimum stability, flow, void in mix, void in aggregates filled with bitumen, specified blended combined grading and Specific gravities of aggregates, filler (cement or lime) and bitumen binder". User may modify the data in file ACMIX.HDS, by changing the numeric values but keeping the data syntax exactly the same. The file is to be saved in the same location as shown above.

```
th 40 40
blows 50 75
stability 341 818
flow 20 to 40 and 20 to 35
voids in mix 3 to 5 and 3 to 5
voids in aggs. 75 to 85 and 75 to 85
specified grading
40 mm 100 %
20 mm 100 %
10 mm 70 to 90 %
4.75 mm 50 to 70 %
2.36 mm 35 to 50 %
600 mic 18 to 29 %
300 mic 13 to 23 %
150 mic 8 to 16 %
75 mic 4 to 10 %
999
sp gr aggregates 2.65
sp gr filler 2.80
sp gr binder 1.00
999
FINISH
```

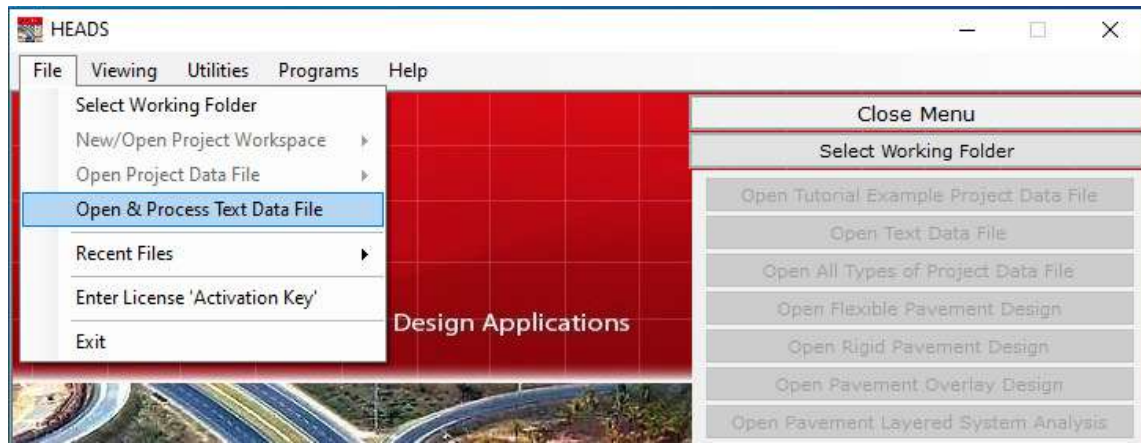
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Another file "CORRFAC.T.HDS", is opened in Notepad and is shown below. The data is about the correction factor for 'Stability' by the Marshall test.

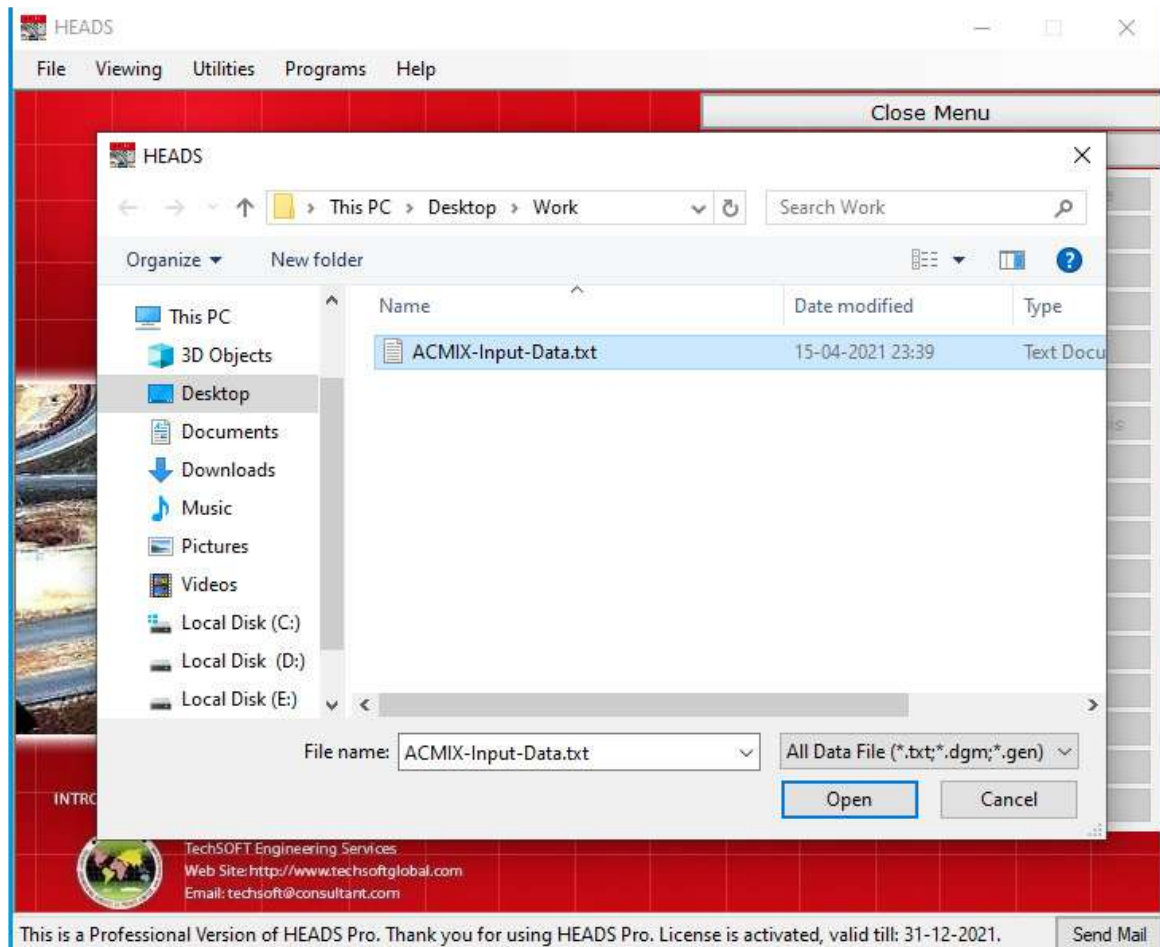
Volume in cc.	correction factor.
200 213	5.56
214 225	5.00
226 237	4.55
238 250	4.17
251 264	3.85
265 276	3.57
277 289	3.33
290 301	3.03
302 316	2.78
317 328	2.50
329 340	2.27
341 353	2.08
354 367	1.92
368 379	1.79
380 392	1.67
393 405	1.56
406 420	1.47
421 431	1.39
432 443	1.32
444 456	1.25
457 470	1.19
471 482	1.14
483 495	1.09
496 508	1.04
509 522	1.00
523 535	0.96
536 546	0.93
547 559	0.89
560 573	0.86
574 585	0.83
586 598	0.81
599 610	0.78
611 625	0.76

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In main screen of HEADS Pro, select the menu item 'File >> Open & Process Text Data File',

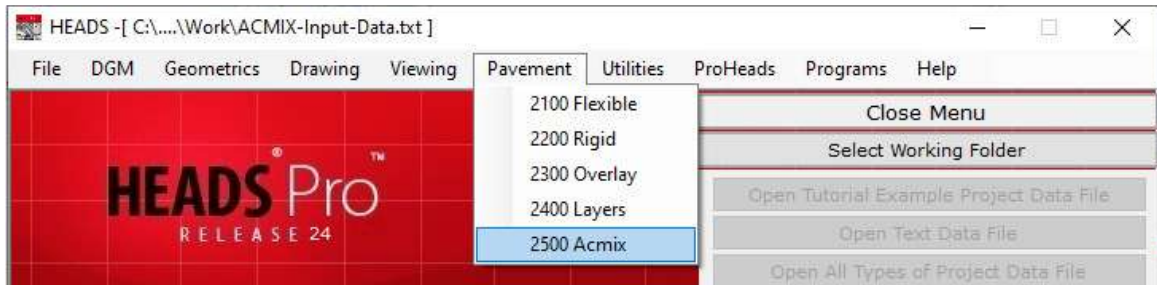


As the selection box comes, select the saved file "ACMIX-Input-Data.txt" and click on "Open", the data file is selected and once again opens in Notepad, close the Notepad.

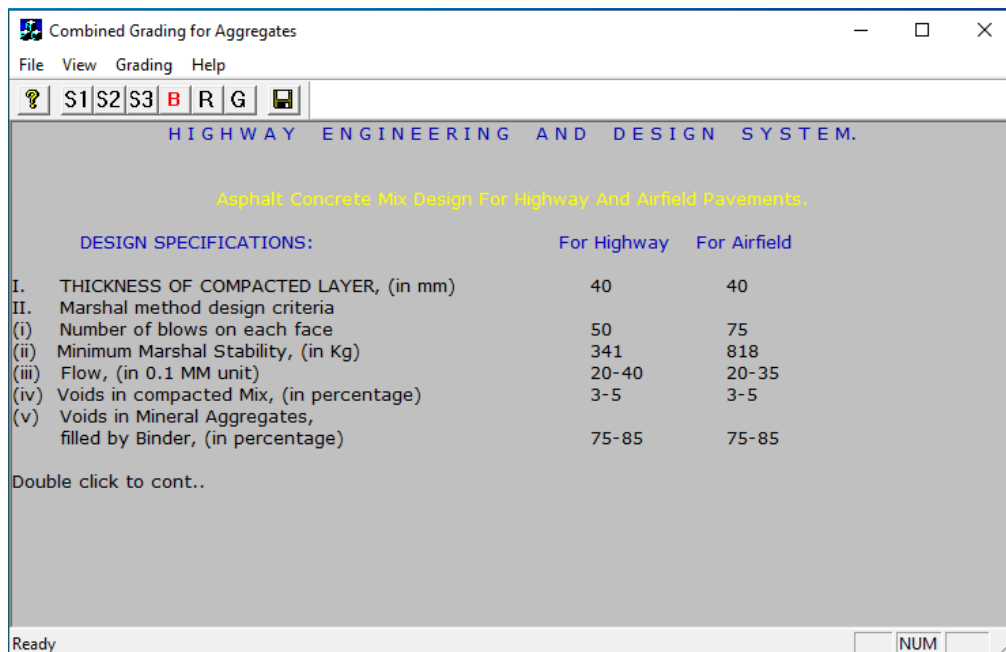


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Select menu item 'Pavement >> 2500 ACMIX',

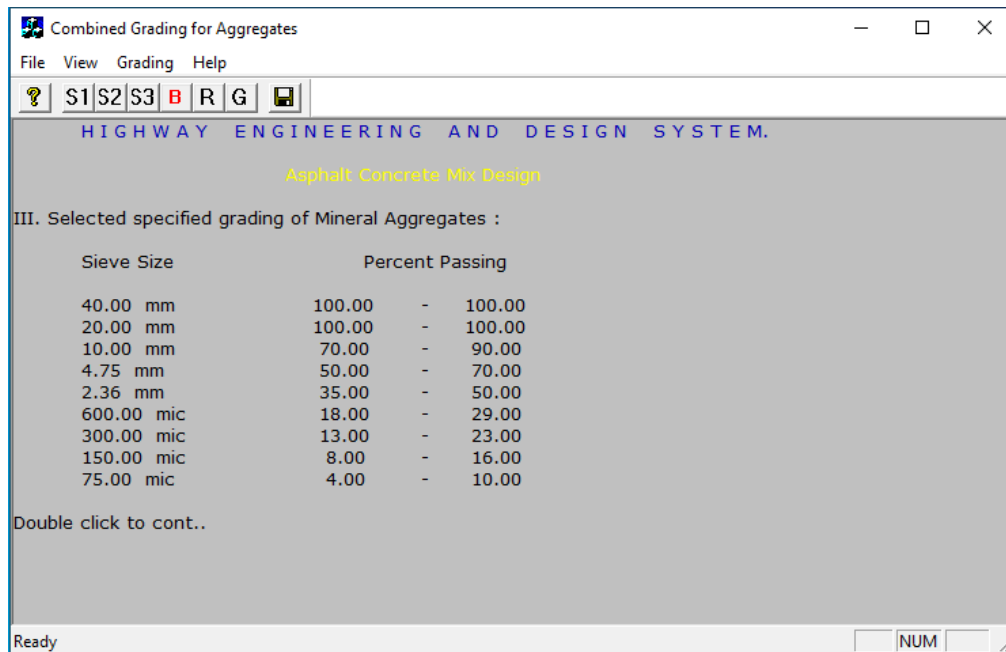


The design window comes with buttons S1, S2, S3, B, R, G for specific screens and a Save Icon, Currently the screen specific to button 'S1' is opened which is displaying specification data reading from file 'ACMIX.HDS',

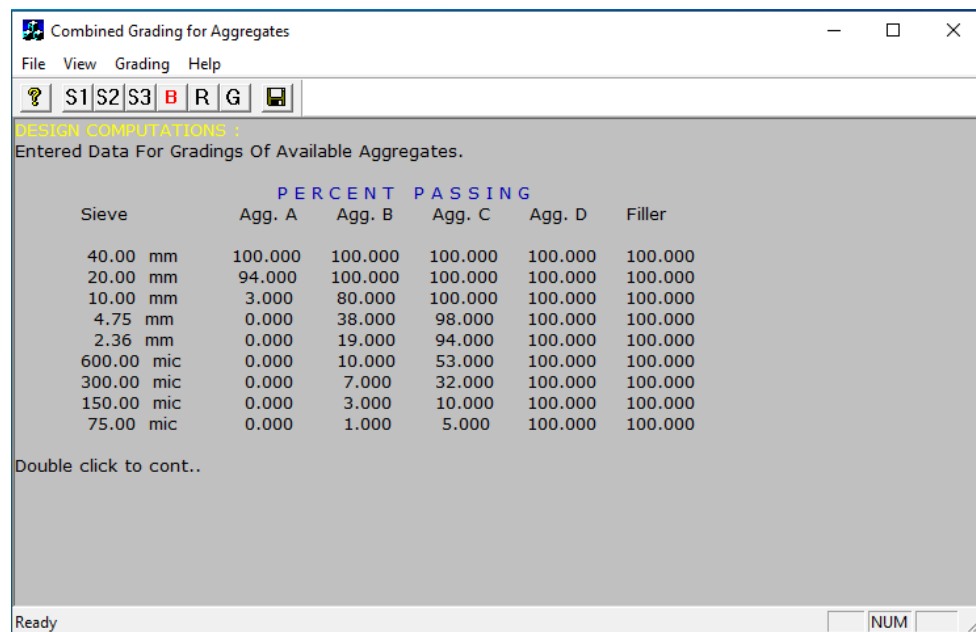


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Click on button 'S2', the screen changes showing specified aggregate grading of combined blend,

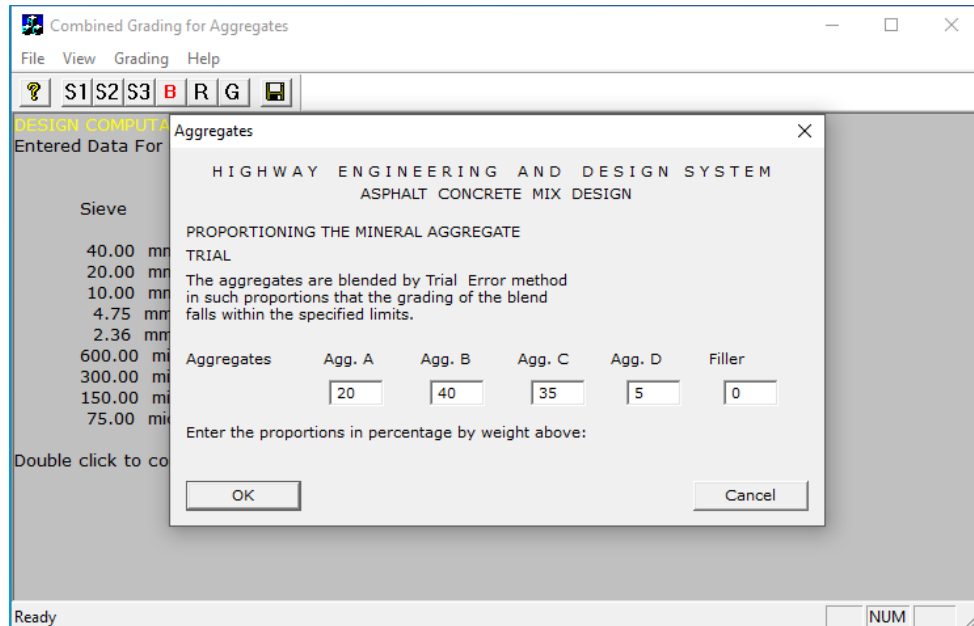


Click on button 'S3', the screen changes showing individual aggregate grading as procured at site,



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Click on button 'B', the screen comes with default blending percentages for individual aggregate grading as procured at site, user may change the percentage data as required, click on 'OK',



Click on button 'R', the screen comes with blended grading applying percentages on individual aggregate grading as procured at site, user may check the combined grading for specified grading range for passing designated sieve sizes,

Combined Grading for Aggregates

File View Grading Help

DESIGN COMPUTA Entered Data For

Sieve

40.00 mm

20.00 mm

10.00 mm

4.75 mm

2.36 mm

600.00 mic

300.00 mic

150.00 mic

75.00 mic

Double click to co

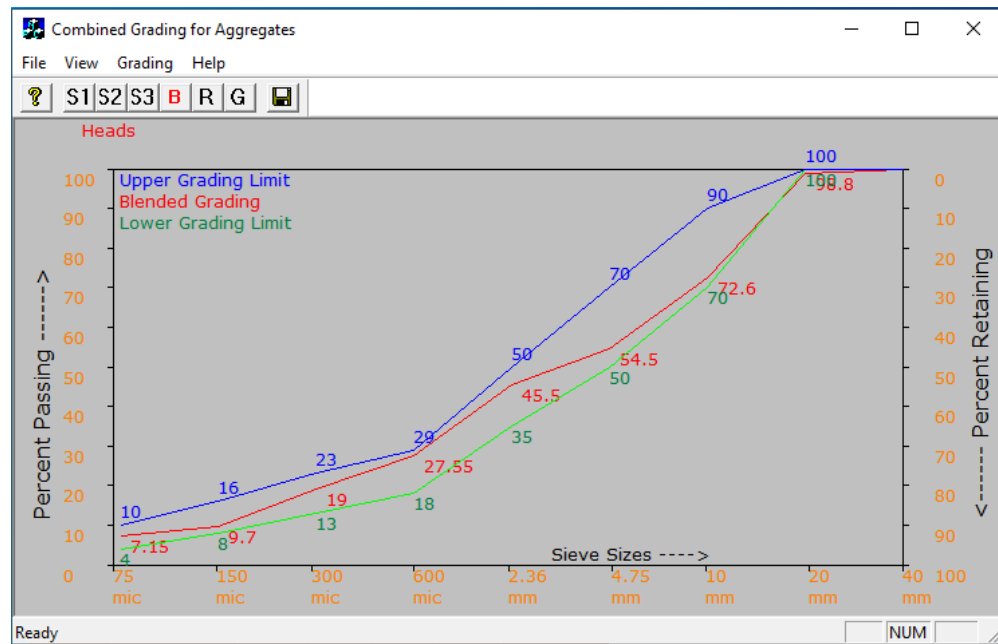
PROPORTIONED MINERAL AGGREGATE BLEND.

Sieve sizes	Agg-A 20.00,	Agg-B 40.00,	Agg-C 35.00,	Agg-D 5.00,	Filler 0.00	Blended Grading	Specified Grading
40.00 mm	20.0	40.0	35.0	5.0	0.0	100.00	100.00 - 100.00
20.00 mm	18.8	40.0	35.0	5.0	0.0	98.80	100.00 - 100.00
10.00 mm	0.6	32.0	35.0	5.0	0.0	72.60	70.00 - 90.00
4.75 mm	0.0	15.2	34.3	5.0	0.0	54.50	50.00 - 70.00
2.36 mm	0.0	7.6	32.9	5.0	0.0	45.50	35.00 - 50.00
600.00 mic	0.0	4.0	18.6	5.0	0.0	27.55	18.00 - 29.00
300.00 mic	0.0	2.8	11.2	5.0	0.0	19.00	13.00 - 23.00
150.00 mic	0.0	1.2	3.5	5.0	0.0	9.70	8.00 - 16.00
75.00 mic	0.0	0.4	1.8	5.0	0.0	7.15	4.00 - 10.00

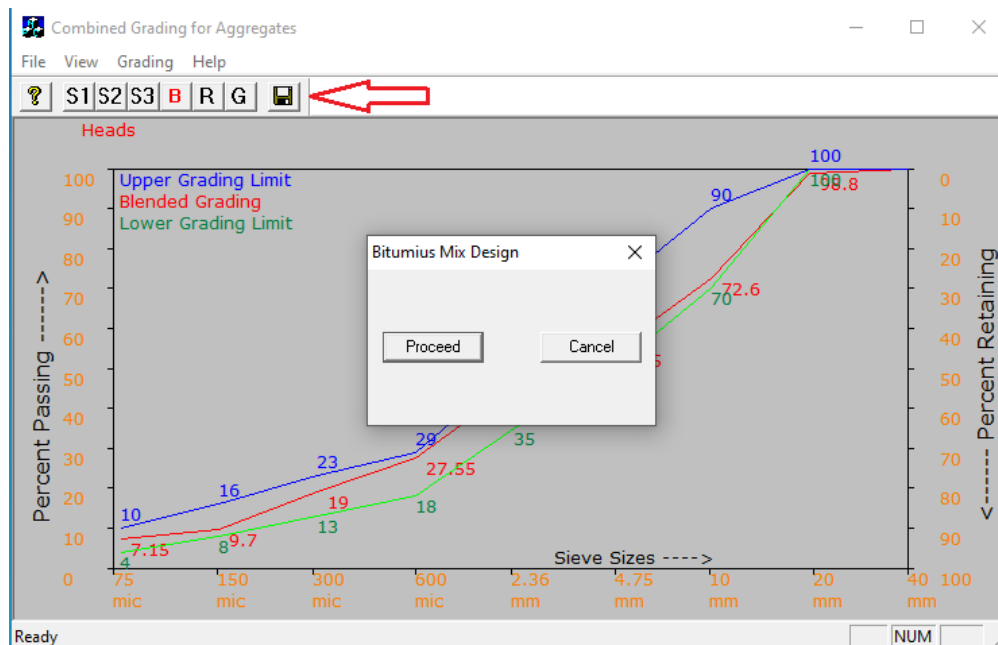
Ready NUM

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Click on button 'G', the screen comes with Graph, showing blended grading and specified ranges,

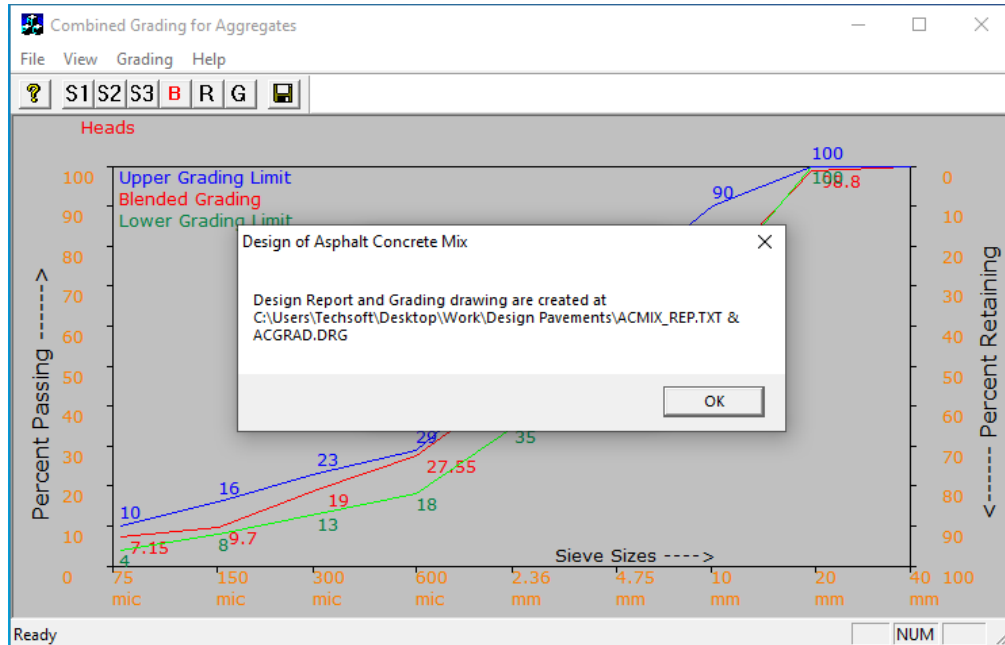


Finally click on the 'Save' icon, the dialog box comes with option for design process, click on 'Proceed',

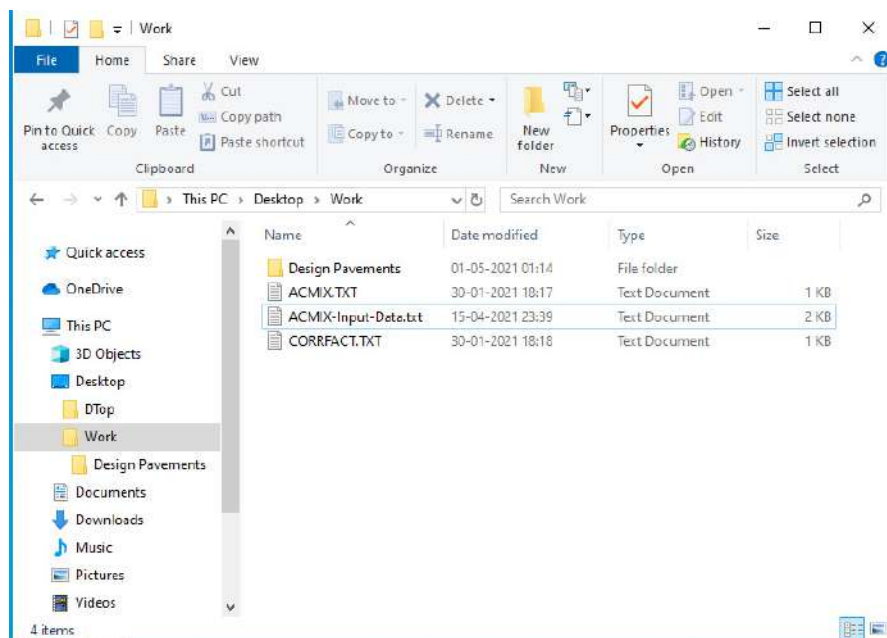


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The mix design is done using Input data file 'ACMIX-Input-Data.txt', specification file ACMIX.HDS and the file for Stability Correction Factors 'CORRFAC.T.HDS', the message comes, click on 'OK',

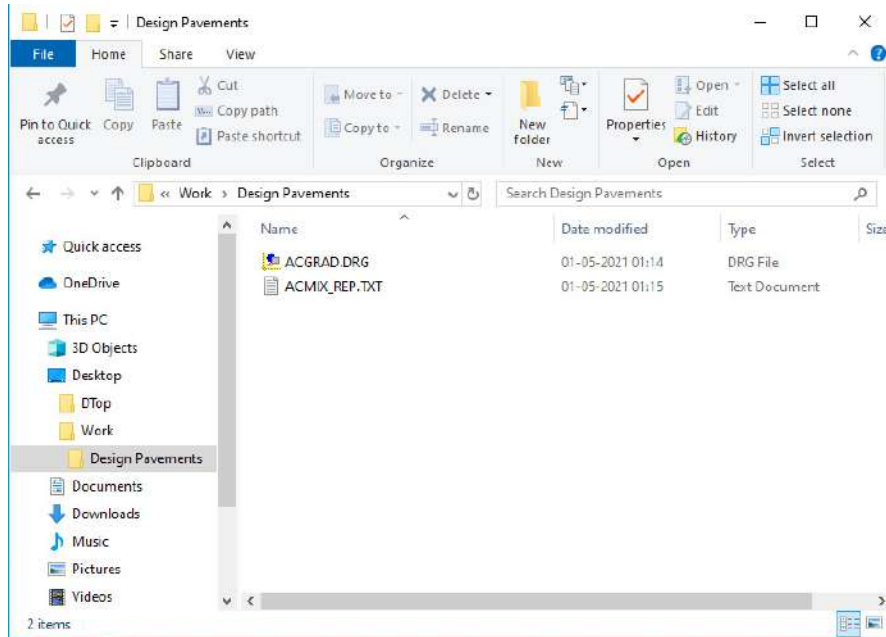


In the working folder 'Work' on desktop, the folder is created as 'Design Pavements',

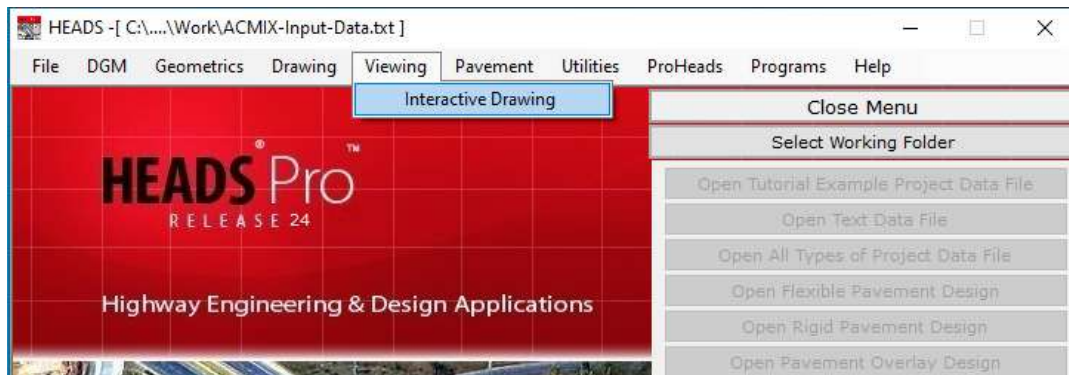


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Inside the folder 'Design Pavements' two files are created as ACGRAD.DRG and ACMIX-REP.TXT,

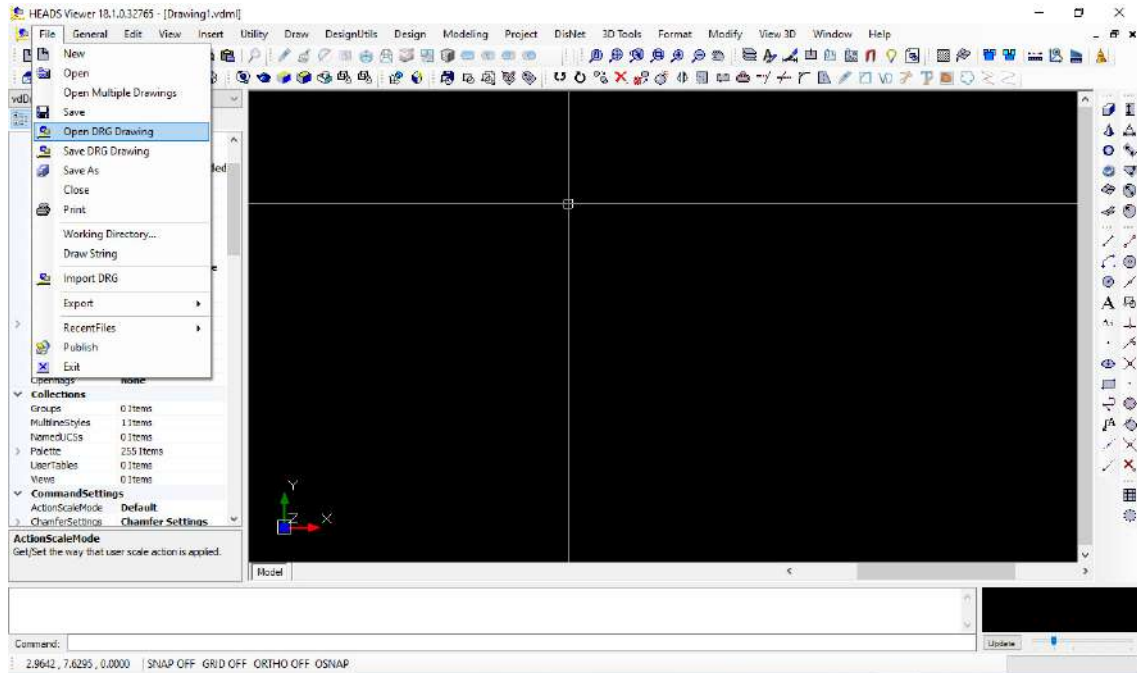


In HEADS Pro main screen select menu item 'Viewing >> Interactive Drawing',

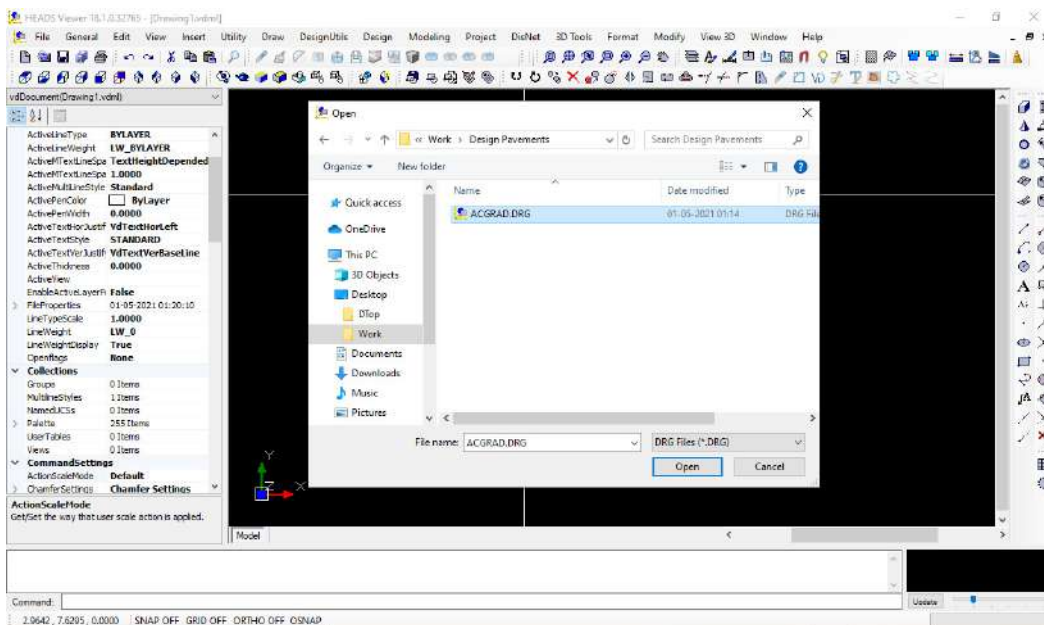


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As the Heads Viewer opens, select menu item 'File >> Open DRG Drawing',

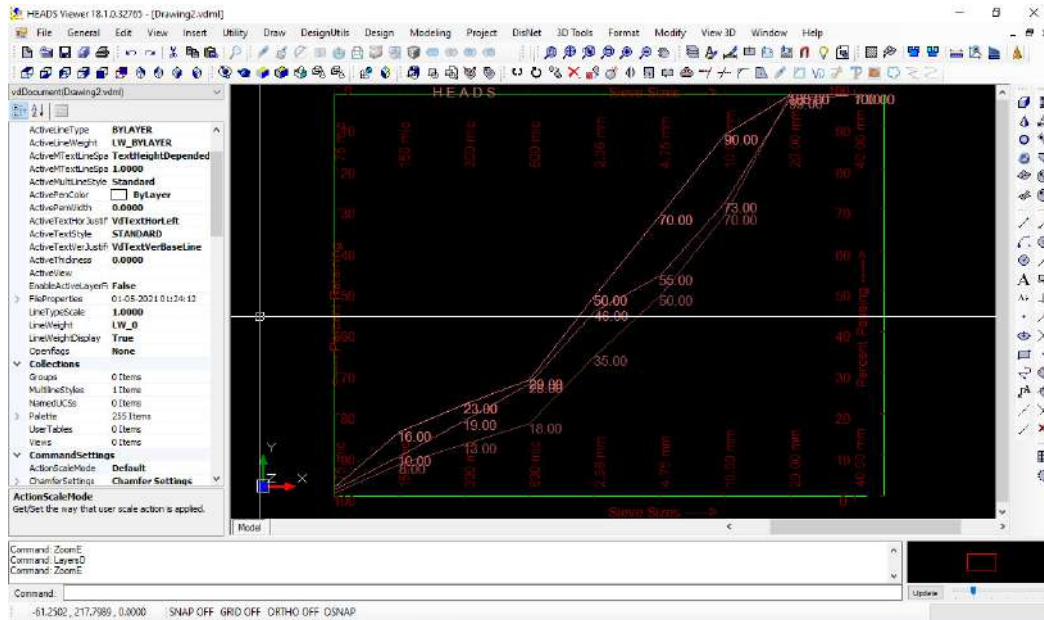


The selection box comes, select the file ACGRAD.DRG, just created, click on 'Open',



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The drawing for aggregate grading is opened for editing and printing, close the 'Heads Viewer',



The Mix design report file 'ACMIX_REP.TXT' may be opened in Notepad as contains the design details as below,

HEADS Pro/Site

Report on Mix Design for Asphalt Concrete/Semi Dense BC/DBM/BM

Program was run on Tue Apr 06 01:34:12 2021

TechSOFT Engineering Services.

DESIGN SPECIFICATIONS:		For Highway	For Airfield
I.	THICKNESS OF COMPACTED LAYER , (in mm)	40	40
II.	Marshal method design criteria		
(i)	Number of blows on each face	50	75
(ii)	Minimum Marshal Stability , (in Kg)	341	818
(iii)	Flow , (in 0.1 MM unit)	20 - 40	20 - 35
(iv)	Voids in compacted Mix , (in percentage)	3 - 5	3 - 5
(v)	Voids in Mineral Aggregates , filled by Binder , (in percentage)	75 - 85	75 - 85

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III. Selected specified grading of Mineral Aggregates :

Sieve Size	Percent Passing	
40.00 mm.	100.00 -	100.00
20.00 mm.	100.00 -	100.00
10.00 mm.	70.00 -	90.00
4.75 mm.	50.00 -	70.00
2.36 mm.	35.00 -	50.00
600.00 mic.	18.00 -	29.00
300.00 mic.	13.00 -	23.00
150.00 mic.	8.00 -	16.00
75.00 mic.	4.00 -	10.00

STEP 2.

PROPORTIONING THE MINERAL AGGREGATE BLEND.

TRIAL PROPORTIONING.

The aggregates are blended by Trial & Error method in such proportions that the grading of the blend falls within the specified limits.
Entered proportions in percentage by weight.

PROPORTIONED MINERAL AGGREGATE BLEND.

Sieve sizes.	Agg-A	Agg-B	Agg-C	Agg-D	Filler	Blended Grading	Specified Grading
20.00	40.00	35.00	5.00	0.00			
40.00 mm.	20.0	40.0	35.0	5.0	0.0	100.00	100.00 - 100.00
20.00 mm.	18.8	40.0	35.0	5.0	0.0	98.80	100.00 - 100.00
10.00 mm.	0.6	32.0	35.0	5.0	0.0	72.60	70.00 - 90.00
4.75 mm.	0.0	15.2	34.3	5.0	0.0	54.50	50.00 - 70.00
2.36 mm.	0.0	7.6	32.9	5.0	0.0	45.50	35.00 - 50.00
600.00 mic.	0.0	4.0	18.6	5.0	0.0	27.55	18.00 - 29.00
300.00 mic.	0.0	2.8	11.2	5.0	0.0	19.00	13.00 - 23.00
150.00 mic.	0.0	1.2	3.5	5.0	0.0	9.70	8.00 - 16.00
75.00 mic.	0.0	0.4	1.8	5.0	0.0	7.15	4.00 - 10.00

DESIGN OF THE BITUMINOUS PART OF THE MIX.

STEP 3.

Preparation of Marshal Test Specimens:

Mixes are prepared with the blended mineral aggregates with increasing percentages of the binder content, i.e., from 6.0 to 8.0 by weight of mineral aggregates with increment 4.5 to 6.0 by weight of mineral aggregates with increment of 0.5. For each binder content, 3 test specimens are to be prepared.

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STEP 4.

Testing for Marshal Test Specimen

BULK DENSITY :

Bulk Density is calculated as :

Bulk Density = $\text{Wt. in air} / (\text{Wt. in air} - \text{Wt. in water})$

Bulk Density Tests on 3 specimens at Percent Binder Content : 4.5				
Specimen Numbers	:	I	II	III
Weight in air (gms.)	:	1147.00	1141.00	1150.00
Weight in water (gms.)	:	654.00	654.00	660.00
Volume (cc.)	:	493.00	487.00	490.00
Bulk Density (gms/cc.)	:	2.327	2.343	2.347
AVERAGE BULK DENSITY	:	2.339 gms/cc.		

STEP 4.

Testing for Marshal Test Specimen

BULK DENSITY :

Bulk Density is calculated as :

Bulk Density = $\text{Wt. in air} / (\text{Wt. in air} - \text{Wt. in water})$

Bulk Density Tests on 3 specimens at Percent Binder Content : 5.0				
Specimen Numbers	:	I	II	III
Weight in air (gms.)	:	1160.00	1158.00	1153.00
Weight in water (gms.)	:	668.00	668.00	667.00
Volume (cc.)	:	492.00	490.00	486.00
Bulk Density (gms/cc.)	:	2.358	2.363	2.372
AVERAGE BULK DENSITY	:	2.364 gms/cc.		

STEP 4.

Testing for Marshal Test Specimen

BULK DENSITY :

Bulk Density is calculated as :

Bulk Density = $\text{Wt. in air} / (\text{Wt. in air} - \text{Wt. in water})$

Bulk Density Tests on 3 specimens at Percent Binder Content : 5.5				
Specimen Numbers	:	I	II	III
Weight in air (gms.)	:	1160.00	1160.00	1157.00
Weight in water (gms.)	:	674.00	672.00	670.00
Volume (cc.)	:	486.00	488.00	487.00
Bulk Density (gms/cc.)	:	2.387	2.377	2.376
AVERAGE BULK DENSITY	:	2.380 gms/cc.		

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STEP 4.

Testing for Marshal Test Specimen

BULK DENSITY :

Bulk Density is calculated as :

Bulk Density = Wt. in air / (Wt. in air - Wt. in water)

Bulk Density Tests on 3 specimens at Percent Binder Content : 6.0				
Specimen Numbers :	I	II	III	
weight in air (gms.) :	1162.00	1169.00	1163.00	
weight in water (gms.) :	670.00	675.00	671.00	
Volume (cc.) :	492.00	494.00	492.00	
Bulk Density (gms/cc.) :	2.362	2.366	2.364	
AVERAGE BULK DENSITY :	2.364	gms/cc.		
From all Average values the MAXIMUM BULK DENSITY : 2.380 gms/cc.				

STABILITY :

Stability Tests on 3 specimens at Percent Binder Content : 4.5				
Specimen Numbers :	I	II	III	
Marshal Stability(kg.) :	1162.00	1169.00	1163.00	
Volume (cc.) :	670.00	675.00	671.00	
Correction Factor :	1.09	1.09	1.09	
Corrected Stability(kg):	860	882	857	
AVERAGE STABILITY :	866	kg.		

Stability Tests on 3 specimens at Percent Binder Content : 5.0				
Specimen Numbers :	I	II	III	
Marshal Stability(kg.) :	1162.00	1169.00	1163.00	
Volume (cc.) :	670.00	675.00	671.00	
Correction Factor :	1.09	1.09	1.09	
Corrected Stability(kg):	1081	1036	1049	
AVERAGE STABILITY :	1055	kg.		

Stability Tests on 3 specimens at Percent Binder Content : 5.5				
Specimen Numbers :	I	II	III	
Marshal Stability(kg.) :	1162.00	1169.00	1163.00	
Volume (cc.) :	670.00	675.00	671.00	
Correction Factor :	1.09	1.09	1.09	
Corrected Stability(kg):	972	1006	1018	
AVERAGE STABILITY :	999	kg.		

Stability Tests on 3 specimens at Percent Binder Content : 6.0				
Specimen Numbers :	I	II	III	
Marshal Stability(kg.) :	1162.00	1169.00	1163.00	
Volume (cc.) :	670.00	675.00	671.00	
Correction Factor :	1.09	1.09	1.09	
Corrected Stability(kg):	903	908	909	
AVERAGE STABILITY :	907	kg.		

From all Average Values the MAXIMUM STABILITY : 1055 kg.

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FLOW :

=====			
Flow Tests on 3 specimens at Percent Binder Content : 4.5			

Specimen Numbers :	I	II	III
Weight in air (gms.) :	1162.00	1169.00	1163.00

AVERAGE FLOW , 0.1 mm unit : 35			
=====			
Flow Tests on 3 specimens at Percent Binder Content : 5.0			

Specimen Numbers :	I	II	III
Weight in air (gms.) :	1162.00	1169.00	1163.00

AVERAGE FLOW , 0.1 mm unit : 36			
=====			
Flow Tests on 3 specimens at Percent Binder Content : 5.5			

Specimen Numbers :	I	II	III
Weight in air (gms.) :	1162.00	1169.00	1163.00

AVERAGE FLOW , 0.1 mm unit : 38			
=====			
Flow Tests on 3 specimens at Percent Binder Content : 6.0			

Specimen Numbers :	I	II	III
Weight in air (gms.) :	1162.00	1169.00	1163.00

AVERAGE FLOW , 0.1 mm unit : 42			

From all Average Values the MAXIMUM FLOW : 42			

STEP 5.

Computation of percent air voids in compacted mix.

The following value is assumed, as not found in ACMIX.TXT file.

Specific gravity of aggregates (A,B & C) $ro1 = 2.65$

The following value is assumed, as not found in ACMIX.TXT file.

Specific gravity of filler (D) $ro2 = 2.80$

The following value is assumed, as not found in ACMIX.TXT file.

Specific gravity of binder $ro3 = 1.00$

Maximum theoretical density of compacted mix

$$\begin{aligned}
 &= \frac{100.0 + \% \text{ weight of binder}}{\frac{\% \text{ wt of aggs (A, B \& C)}}{\text{Sp.gr. of aggs}} + \frac{\% \text{ wt of filler}}{\text{Sp.gr. of filler}} + \frac{\% \text{ wt of binder}}{\text{Sp.gr. of binder}}} \\
 &= \frac{100.0 + 4.50}{\frac{95.00}{2.65} + \frac{5.00}{2.80} + \frac{4.50}{1.00}} \\
 &= \text{maxden}[1] \\
 &= 2.48
 \end{aligned}$$

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Percent air voids in compacted mix

$$\begin{aligned}
 &= \frac{(\text{maxden}[1] - \text{bdav}[1])}{\text{maxden}[1]} \times 100.0 \\
 &= \frac{(2.48 - 2.34)}{2.48} \times 100.0 \\
 &= \text{void}[1] \\
 &= 5.69
 \end{aligned}$$

Maximum theoretical density of compacted mix

$$\begin{aligned}
 &= \frac{100.0 + \% \text{ weight of binder}}{\frac{\% \text{ wt of aggs (A, B \& C)}}{\text{Sp.gr. of aggs}} + \frac{\% \text{ wt of filler}}{\text{Sp.gr. of filler}} + \frac{\% \text{ wt of binder}}{\text{Sp.gr. of binder}}} \\
 &= \frac{100.0 + 5.00}{\frac{95.00}{2.65} + \frac{5.00}{2.80} + \frac{5.00}{1.00}} \\
 &= \text{maxden}[2] \\
 &= 2.46
 \end{aligned}$$

Percent air voids in compacted mix

$$\begin{aligned}
 &= \frac{(\text{maxden}[2] - \text{bdav}[2])}{\text{maxden}[2]} \times 100.0 \\
 &= \frac{(2.46 - 2.36)}{2.46} \times 100.0 \\
 &= \text{void}[2] \\
 &= 4.02
 \end{aligned}$$

Maximum theoretical density of compacted mix

$$\begin{aligned}
 &= \frac{100.0 + \% \text{ weight of binder}}{\frac{\% \text{ wt of aggs (A, B \& C)}}{\text{Sp.gr. of aggs}} + \frac{\% \text{ wt of filler}}{\text{Sp.gr. of filler}} + \frac{\% \text{ wt of binder}}{\text{Sp.gr. of binder}}} \\
 &= \frac{100.0 + 5.50}{\frac{95.00}{2.65} + \frac{5.00}{2.80} + \frac{5.50}{1.00}} \\
 &= \text{maxden}[3] \\
 &= 2.45
 \end{aligned}$$

Percent air voids in compacted mix

$$\begin{aligned}
 &= \frac{(\text{maxden}[3] - \text{bdav}[3])}{\text{maxden}[3]} \times 100.0 \\
 &= \frac{(2.45 - 2.38)}{2.45} \times 100.0 \\
 &= \text{void}[3] \\
 &= 2.70
 \end{aligned}$$

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Maximum theoretical density of compacted mix

$$\begin{aligned}
 &= \frac{100.0 + \% \text{ weight of binder}}{\frac{\% \text{ wt of aggs (A, B \& C).}}{\text{Sp.gr. of aggs}} + \frac{\% \text{ wt of filler}}{\text{Sp.gr. of filler}} + \frac{\% \text{ wt of binder}}{\text{Sp.gr. of binder}}} \\
 &= \frac{100.0 + 6.00}{\frac{95.00}{2.65} + \frac{5.00}{2.80} + \frac{6.00}{1.00}} \\
 &= \text{maxden}[4] \\
 &= 2.43
 \end{aligned}$$

Percent air voids in compacted mix

$$\begin{aligned}
 &= \frac{(\text{maxden}[4] - \text{bdav}[4])}{\text{maxden}[4]} \times 100.0 \\
 &= \frac{(2.43 - 2.36)}{2.43} \times 100.0 \\
 &= \text{void}[4] \\
 &= 2.68
 \end{aligned}$$

TABLE :

VOIDS IN COMPACTED MIX.

Percent Binder Content :	4.50	5.00	5.50	6.00
Percent air voids in compacted mix :	5.69	4.02	2.70	2.68

The values of percent air voids in the compacted mix may be plotted against the values of percent binder contents.

STEP 6.

Computation of percent AIR VOIDS IN MINERAL AGGREGATES filled by the binder.

$$\begin{aligned}
 &\text{Percent volume occupied by the binder at binder percent 4.50} \\
 &= (\text{bulk density} \times \text{percent binder in total mix}) / \text{sp.gr. of binder} \\
 &= \text{bdav}[1] \times ((\text{p}[1] / (100.0 + \text{p}[1])) \times 100.0) / \text{ro3} \\
 &= \text{pbind}[1] = 10.07
 \end{aligned}$$

$$\text{Percent air voids at binder percent 4.50} = 5.69$$

$$\text{Total percent voids in mineral at binder percent 10.07} + 4.50 = 15.76$$

voids in Mineral Aggregates at Binder Content 10.07 :

$$\begin{aligned}
 &= \text{Percent of Binder} \times 100.0 / \text{Tot. percent Voids in Mineral Aggr.} \\
 &= 10.07 \times 100.0 / 15.76 \\
 &= 64.00
 \end{aligned}$$

$$\begin{aligned}
 &\text{Percent volume occupied by the binder at binder percent 5.00} \\
 &= (\text{bulk density} \times \text{percent binder in total mix}) / \text{sp.gr. of binder} \\
 &= \text{bdav}[2] \times ((\text{p}[2] / (100.0 + \text{p}[2])) \times 100.0) / \text{ro3} \\
 &= \text{pbind}[2] = 11.26
 \end{aligned}$$

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Percent air voids at binder percent 5.00 = 4.02

Total percent voids in mineral at binder percent 11.26 + 5.00 = 15.28

Voids in Mineral Aggregates at Binder Content 11.26 :

= Percent of Binder x 100.0 / Tot. percent Voids in Mineral Aggr.
 = 11.26 x 100.0 / 15.28
 = 74.00

Percent volume occupied by the binder at binder percent 5.50
 = (bulk density x percent binder in total mix)/sp.gr. of binder
 = bdav[3] x ((p[3]/(100.0 + p[3])) x 100.0)/ro3
 = pbind[3] = 12.41

Percent air voids at binder percent 5.50 = 2.70

Total percent voids in mineral at binder percent 12.41 + 5.50 = 15.11

Voids in Mineral Aggregates at Binder Content 12.41 :

= Percent of Binder x 100.0 / Tot. percent Voids in Mineral Aggr.
 = 12.41 x 100.0 / 15.11
 = 82.00

Percent volume occupied by the binder at binder percent 6.00
 = (bulk density x percent binder in total mix)/sp.gr. of binder
 = bdav[4] x ((p[4]/(100.0 + p[4])) x 100.0)/ro3
 = pbind[4] = 13.38

Percent air voids at binder percent 6.00 = 2.68

Total percent voids in mineral at binder percent 13.38 + 6.00 = 16.06

Voids in Mineral Aggregates at Binder Content 13.38 :

= Percent of Binder x 100.0 / Tot. percent Voids in Mineral Aggr.
 = 13.38 x 100.0 / 16.06
 = 83.00

TABLE : Voids in Mineral Aggregates.

Percent Binder Content :	4.50	5.00	5.50	6.00
Percent air voids in mineral agg filled by binder :	64	74	82	83

The values of percent voids in mineral aggregates may be plotted against the values of percent binder contents.

For Computation of Optimum Bitumen Content, the following two Binder Content in percent are to be determined

(a) Specified range of VOIDS IN COMPACTED MIX 3 to 5 percent
 Average value is 4 percent. Therefore, from the above table of VOIDS IN COMPACTED MIX, by interpolating the Binder Content percent corresponding to the Average Specified Void is calculated as :

Percent Binder Content = 5.0

(b) Specified range of voids in MINERAL AGGREGATES 75 to 80 percent
 Average value is 78 percent. Therefore, from the above table of VOIDS IN MINERAL AGGREGATES, by interpolating, the Binder Content percent corresponding to the Average Specified Void is calculated as :

Percent Binder Content = 5.2

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STEP 7.

Computation of OPTIMUM BINDER CONTENT (OBC) in percent.

Binder content percent for maximum bulk density = 5.500 (%)

Binder content percent for maximum stability = 5.000 (%)

Binder content percent for 77.5 % voids in compacted mix average of specified limits of 75.0 to 80.0 percent = 5.008 (%)

Binder content percent for 80 % voids in mineral aggregates filled by binder - average of specified limits of 75 to 80 percent = 5.219 (%)

Average i.e, Optimum Binder Content = 5.182 (%)

Now the values for Stability, Flow, Voids in Compacted Mix and Voids in Mineral Aggregates are to be computed at this Optimum Binder Content = 5.2 percent.

STEP 8.

Checking specification values at optimum binder content .
At 5.20 percent binder content ,the test values for various design criteria are given below alongwith the specified values.

Design Criteria	Computed Values	Specified Values.			
		Highways		Airfield	
Marshall Stability, kg.	1033.00	341 min.		818 min.	
Flow, 0.1 mm. unit	37.00	20 to	40	20 to	35
Voids p.c. in comp. mix.	3.50	3 to	5	3 to	5
Voids p.c. in mineral agg. filled by the binder.	77.00	75 to	85	75 to	85

STEP 9.

Computation of materials required for 40 mm. thick Asphalt Concrete layer for 100 sq.m. area.

Volume of layer = $100 \times 100 \times 100 \times (40 / 10)$
= 4000000 c.c.

Bulk Density at 5.18 per cent optimum binder content by weight of aggregate = 2.370 gms/cc.

Weight of materials = 4000000×2.370
= 9480000 gms.
= 9480 kg.

Final Mix Proportions with 5.18 binder content.

The weights are given in kg and volumes in cu.m..

Type of Agg.	by mix basis	Wt. in kg	Unit wt.	Quantity.cu.m.
Agg. A	20.00 x 100/(100+5.18) = 19.01	1803 kg.	1450	1.24
Agg. B	40.00 x 100/(100+5.18) = 38.03	3605 kg.	1500	2.40
Agg. C	35.00 x 100/(100+5.18) = 33.28	3155 kg.	1680	1.88
Agg. D	5.00 x 100/(100+5.18) = 4.75	451 kg.	-	-
Filler	0.00 x 100/(100+5.18) = 0.00	0 kg.	-	-
Binder	5.18 x 100/(100+5.18) = 4.93	467 kg.	-	-
Total		= 100 p.c. = 9480.00 kg.		
E N D O F D E S I G N .				

Construction of Rigid Concrete Pavement 5

General

Road having their wearing surface consisting of cement concrete slab are called as cement concrete road. Cement concrete roads are considered as most serviceable and rigid pavements.



Figure 5.1 – Construction of Rigid Concrete Pavement

There are a number of special considerations that need to be made when paving CRCP as opposed to JPCP. Most of these considerations are the result of the reinforcing steel. Since the steel is so important to the design and performance of CRCP, it is very important that it is placed at the proper level and that it stays where it is placed while the concrete is placed around it. Consolidation around the steel is extremely important to promote bond between the concrete and the steel to provide long term performance and keep the developing cracks tight. As with JPCP and all other concrete pavement types, curing is important to producing quality CRCP.

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The most common method for steel placement is installing the steel and placers by hand before paving. The steel is held in place at the proper vertical location by chairs. Transverse bars are also used to help hold the steel in place and make sure that any longitudinal cracks that develop are held tight. The system of steel reinforcement, chairs, and transverse steel bars must be secure enough to ensure that it does not move as the concrete is placed around the steel either by hand or by a slip-form paver. The system must also allow consolidation of the concrete to ensure proper performance. The reinforcing steel bars are lap-spliced together to build the length of the roadway. The laps are typically staggered or skewed to prevent any compaction issues. One key to ensuring good consolidation and good performance is the use of a proper concrete mixture produced at a steady production rate that is vibrated properly. Limiting the heat of hydration and keeping concrete temperatures between 50 and 90 degrees F has also shown good performance and a consistent cracking pattern. The HIPERPAV program can shed light on the effects of temperature changes on cracking patterns and what changes may need to occur.

One development in steel placement is the use of transverse bar assemblies or TBAs. These are used as supports for the reinforcing steel and can speed installation time significantly. TBAs provide transverse restraint to the steel reinforcement, while holding the reinforcement at the proper height and allowing it some longitudinal movement.

Structural components of concrete pavement

Starting from bottom such roads consist of following components :

- Sub-grade (Lowest Layer)
- Sub-base (DLC) (Intermediate Layer GSB or Dry Lean Concrete, M15)
- Concrete Slab (PQC) (Top Layer, Pavement Quality Concrete)

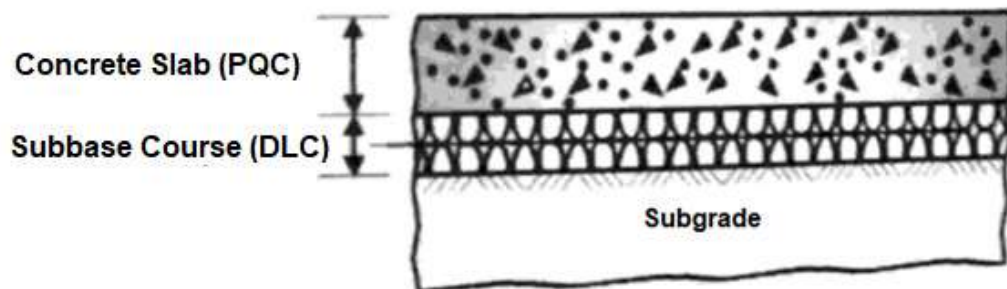


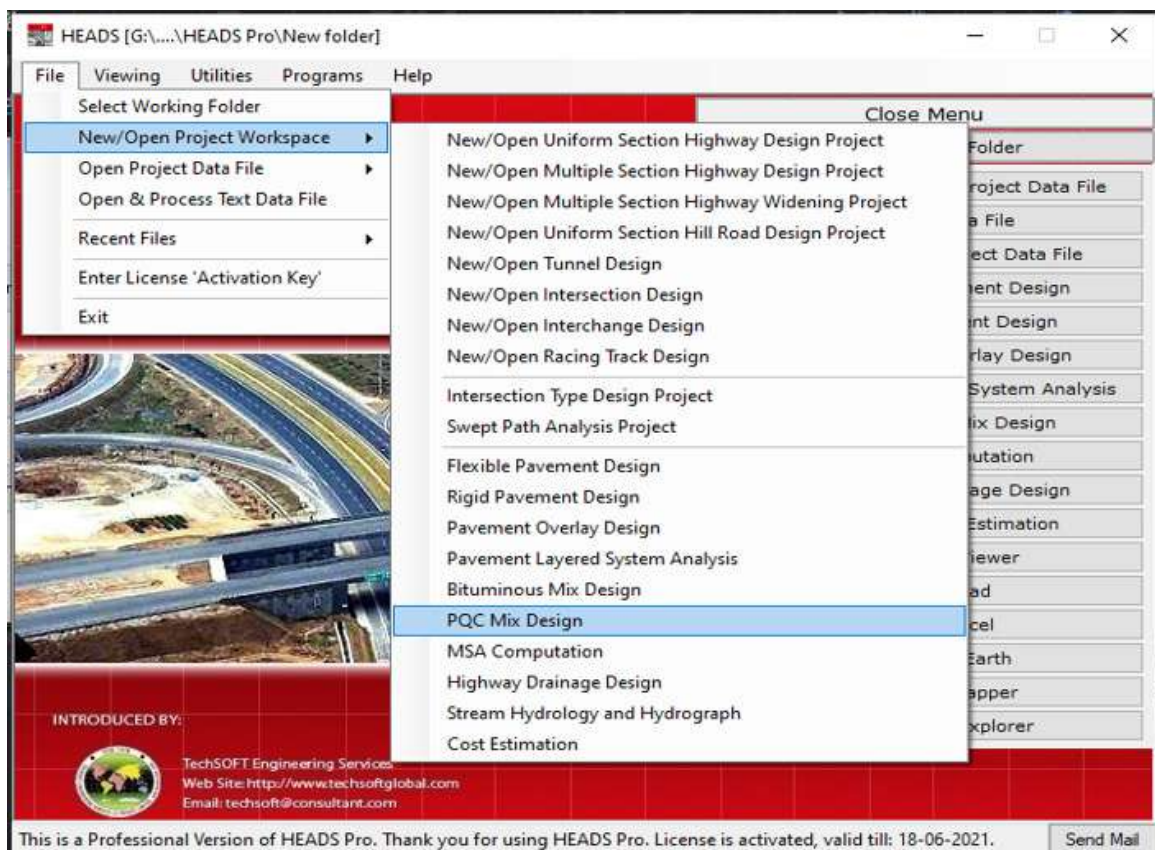
Figure 5.2 – Layers in Rigid Concrete Pavement

DLC (Dry Lean Concrete), which is popularly used for the construction of sub-base layer under **concrete** pavement in India, is a zero slump **concrete** with maximum aggregate–**cement** ratio of 15:1. The water content of **DLC** varies from 5.5% to 7% of total dry weight of the mixture.

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PQC (Pavement Quality Concrete), It's difficult to mention a Particular Grade that is used in all concrete pavements because the selection of Grade of Concrete, which depends on various factors and Traffic Load in MSA is one of them, commonly used concrete grade is M40, withly Slump 5-6mm.

5.1 Computer Applications for Mix Design of Pavement Quality Concrete (PQC)

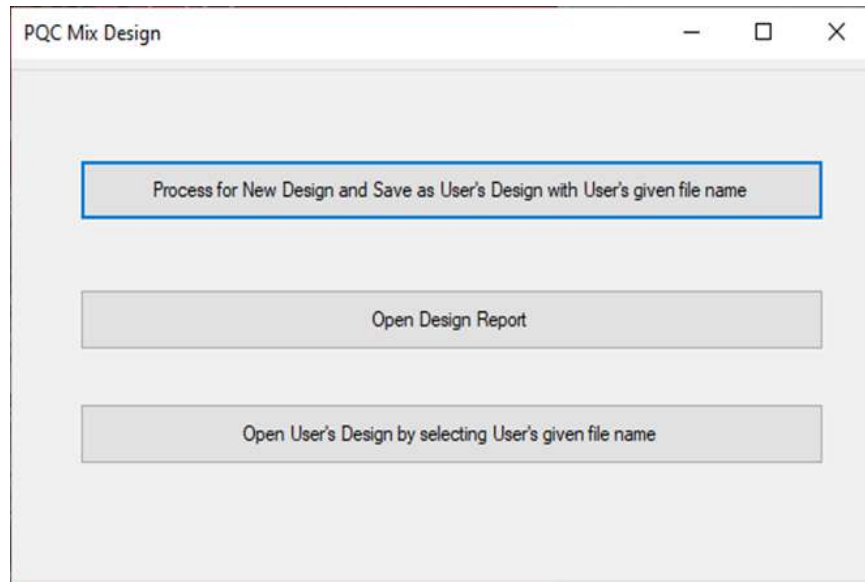


Pavement Quality Concrete (PQC) Mix design is carried out in the Project QC laboratory. The mix is to be used in rigid pavement as a Pavement Quality Concrete as per Technical specifications.

Representative materials of approved sources has been sampled and tested for suitability of each material for the design mix as per the requirements of the specifications.

The page opens with three options, click on button, 'Process for New Design....'. This opens the MS-Excel Worksheet for PQC Mix Design.

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User has to change the data as relevant and save the file on the Desktop, with a desired name. The file may be opened by using the button 'Open User's Design....' As shown in the image above.

FileHomeInsertPage LayoutFormulasDataReviewViewTell me what you want to do...BAN.SITEUSERShare

Paste

Clipboard

Font

Alignment

Number

Styles

Cells

Editing

Arial10A⁺A⁻

General

Conditional Formatting

Format as Table

Cell Styles

Insert

Delete

Format

D23

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The Source of Materials has as follows:

- 1) Cement : OPC 43 grade
- 2) Coarse aggregate: Coarse aggregate (20mm & 10mm aggregates) conforming to Specifications.
- 3) Fine aggregate: Natural sand from approved source conforming to specifications.
- 4) Water: Bore well point Conforming to the specification requirements.
- 5) Admixture: Super-plasticizing slump retaining admixture SUPAPLAST HP(S) from SWC Chemicals and conforming to specifications.
- 6) Physical Properties of Coarse Aggregate:

Table 5.5 – Physical Properties of Coarse Aggregate

S. No.	Name of Test	Result Achieved (%)	Specified Limit (%)
1.	Flakiness Index	12.35	Max 35
2.	Los Angeles Abrasion value	15.1	Max 35
3.	Aggregate Impact value	13.2	Max 30

Design Stipulations:

- | | | |
|---|---|---------------------------------------|
| 1) Concrete Grade | = | M40 |
| 2) Characteristic compressive strength at 28 days | = | 40 Mpa |
| Target Mean strength at 28 days (As per MORT&H Table 1700-5) = 52 Mpa | | |
| 3) Characteristic Flexural strength at 28 days | = | 4.5 Mpa |
| 4) Degree of quality Control | = | Good |
| 5) Standard deviation | = | 0.45 |
| 6) Target flexural strength at 28 days | = | 4.5+2.33*.45 |
| | = | 5.55 Mpa |
| 7) Water cement ratio | = | 0.5 Maximum. |
| (As per MORT&H Clause:602.3.3.1) | | |
| 8) Maximum size aggregate | = | 25 mm |
| (As per MORT&H Clause:602.2.4.2) | | |
| 9) Concrete workability (Slump) | = | 30 ± 15 mm |
| (As per MORT&H Clause:602.3.4.2) | | |
| 10) Cement Content | = | 410 Kg/cum |
| (As per MORT&H Clause:602.3.2) | | (Min.350 Kg./cum and Max .425 Kg/cum) |

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1 Test Data of Materials

a. Specific Gravity of Cement (gm/cc)	=	3.15
b. Specific Gravity of 20 mm aggregate (gm/cc)	=	2.867
c. Specific gravity of 10 mm aggregate (gm/cc)	=	2.858
d. Combined Specific Gravity of 20 & 10 mm aggregate (gm/cc)	=	2.863
e. Specific Gravity of Sand (gm/cc)	=	2.635
f. Water Absorption of 20 mm aggregate (%)	=	0.74
g. Water Absorption of 10 mm aggregate (%)	=	0.80
h. Combined Water Absorption of 20 & 10mm aggregate (%)	=	0.76
i. Water Absorption of Sand (%)	=	1.58

2 Admixture

Admixture Brand : SWC SUPAPLAST HP(S)	
Admixture used	= 0.70%
Specific Gravity (as per MTC) gm/cc	= 1.223

3 Selection of Water Cement Ratio

Considered Water/cement ratio	=	0.39(<0.5)
(As per MORT&H Clause:602.3.3.1)		
Cement Content	(As per MORT&H Clause:602.3.2)	= 410 kg
		(>350kg and <425 kg)

(W/C ratio and cement content has been considered after conducting trial mixes and observing several slump trials.

4 Selection of Water Content

From table – 2 of IS: 10262-2000, for 20mm nominal max. size aggregate,
maximum water content per cubic meter of concrete is = 186 Litre
(For 25mm to 50 mm slump range)

Based on trials with super-plasticizing admixture, water content reduction of 14 percent has been achieved.

Arrived water content = 186×0.86 = 159.96 say 160lit

5 Determination of Cement Content

Water cement ratio	=	0.39	
Water	=	say 160	lit
Cement	=	410	Kg

6 Proportion of Volume of Coarse Aggregate Content

From Table 3 of IS 10262 volume of coarse aggregate corresponding to 20mm size aggregate and fine aggregate (Zone II) for water-cement ratio of 0.50 = 0.62

In the present case water-cement ratio is 0.39. Therefore, volume of coarse aggregate is required to be increased to decrease the fine aggregate content. As the W/C ratio is lower by 0.11, the proportion of volume of coarse aggregate is increased by 0.02 (at the rate of ± 0.01 for every ± 0.05 change in water-cement ratio). Therefore, corrected proportion of volume of coarse aggregate for the water-cement ratio of 0.39 = 0.64.

But, based on trials the volume of coarse aggregate is fixed at 0.62
Therefore, volume of coarse aggregate = 0.62

Volume of fine aggregate content = $1 - 0.62 = 0.38$

7 Mix Calculations

The mix calculations per unit volume of concrete shall be as follows:

- | | | |
|--|---|--|
| a) Volume of concrete | = | 1 cum |
| b) Volume of cement | = | mass of cement/sp. gravity of cement $\times 1/1000$ |
| | = | $(410/3.15) \times (1/1000)$ |
| | = | 0.130 cum |
| c) Volume of water | = | Mass of water/Sp. gravity of water $\times 1/1000$ |
| | = | $(160/1) \times (1/1000)$ |
| | = | 0.160 cum. |
| d) Volume of chemical admixture @ 0.7 percent by mass of cement. | = | Mass of chemical admixture/sp.gravity of admixture $\times 1/1000$ |
| | = | $2.87/1.223 \times 1/1000$ |
| | = | 0.00235 cum |
| e) Volume of all in aggregate | = | $[a - (b + c + d)]$ |
| | = | $1 - (0.130 + 0.160 + 0.00235)$ |
| | = | 0.708 cum |
| f) Mass of coarse aggregate | = | e \times volume of coarse aggregate \times sp. gravity of coarse aggregate $\times 1000$ |
| | = | $0.708 \times 0.62 \times 2.863 \times 1000$ |
| | = | 1257 kg |
| g) Mass of fine aggregate | = | e \times volume of fine aggregate \times sp. gravity of fine aggregate $\times 1000$ |
| | = | $0.708 \times 0.38 \times 2.635 \times 1000$ |
| | = | 709 kg |

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8 The Mix Proportion for One Cubic meter of Concrete

Coarse Aggregates

20mm	754	kg/M ³ of Concrete
10mm	502.8	kg/M ³ of Concrete
Total	1257	kg/M ³ of Concrete

Table 5.6 – Mix Proportion for One Cubic Meter of Concrete

Water	Cement	Coarse Aggregate	Fine Aggregate	Admixture
160	410	1257	709	2.870
0.39	1	3.07	1.73	0.007

Note:

- 1- All the values for aggregate are for SSD condition.
- 2- To obtain slump in the range water reducing admixture brand SWC SUPAPLAST HP(S) with a dose of 0.7% by weight of cement shall be used.

Batch Weights Kg/cum (For one cu. m concrete)

Ingredients	SSD condition	DRY condition
W/C ratio	0.39	0.39
Cement	410	410
Water	160	181
Combined Agg.	1257	1247
20 mm(60%)	754	748
10 mm(40%)	503	499
Sand	709	698
Admixture	2.87	2.87

Weight of fresh concrete : 2538.9 kg

9 Concrete Trial Mix Design Results

a) Slump Observation:

Initial Slump	110 mm
After 30 minutes	65 mm
After 45 minutes	40 mm

b) Compressive Strength Test Results:

7 Days	42.0	N/mm ²
28 Days	54.9	N/mm ²

c) Flexural Strength Test Results:

7 Days Flexural Strength	4.9	N/mm ²
28 Days Flexural Strength	6.2	N/mm ²

d) Flexural Strength Ratio:

$$\begin{aligned} R &= \text{Avg.7 Days Flexural Strength} / \text{Avg.28 Days Flexural Strength} \\ &= 4.9 / 6.2 \\ &= 0.790 \end{aligned}$$

Maintenance of Flexible Bituminous Pavement **6**

Abstract: *This chapter describes the overview of various Pavements*

6.1 General

6.1.1 Pavement Damages / Distresses

- Potholes
- Pavement edge breaks
- Rutting
- Depressions
- Cracking in pavement
- Multiple cracks in the pavement
- Raveling

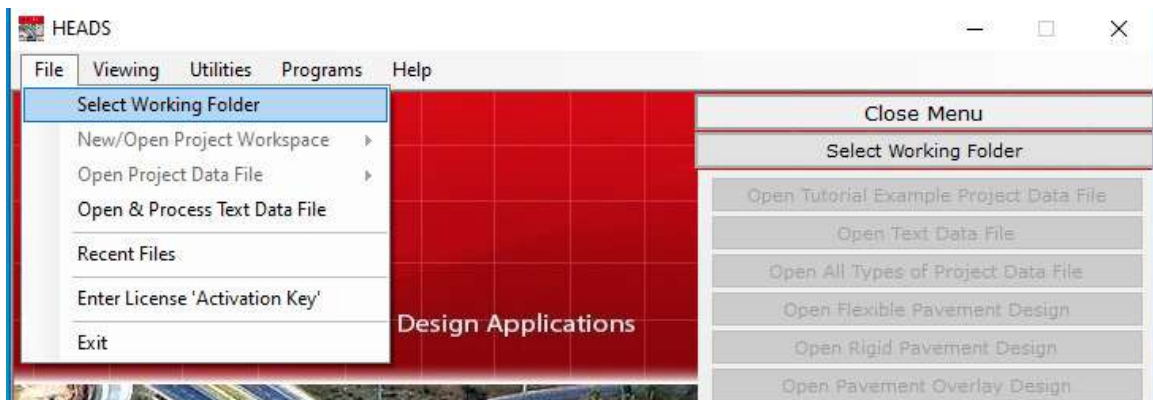
6.1.2 Repairing Measures

- Pothole Patching
- Crack Sealing Stress absorbing membrane and stress absorbing membrane interlayer for crack sealing and prevention of reflection cracking
- Utility Trench Patching
- Chip Seals
- Slurry Seals
- Micro-Surfacing
- Cleanliness of the pavement surface

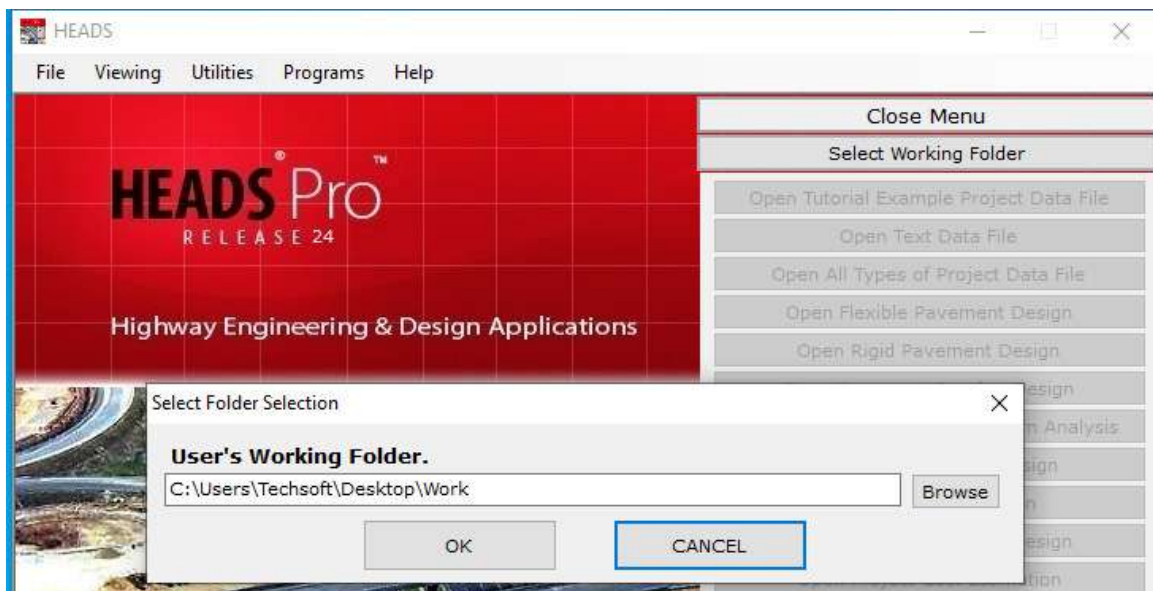
6.1 Computer Applications for Pavement Rehabilitation by Pavement Overlay using Benkelman Beam Deflection Survey Data

6.1.1 Production of Design Report in MS-Excel Worksheet

Open Main screen of HEADS Pro, Select menu item 'File >> Select Working Folder',

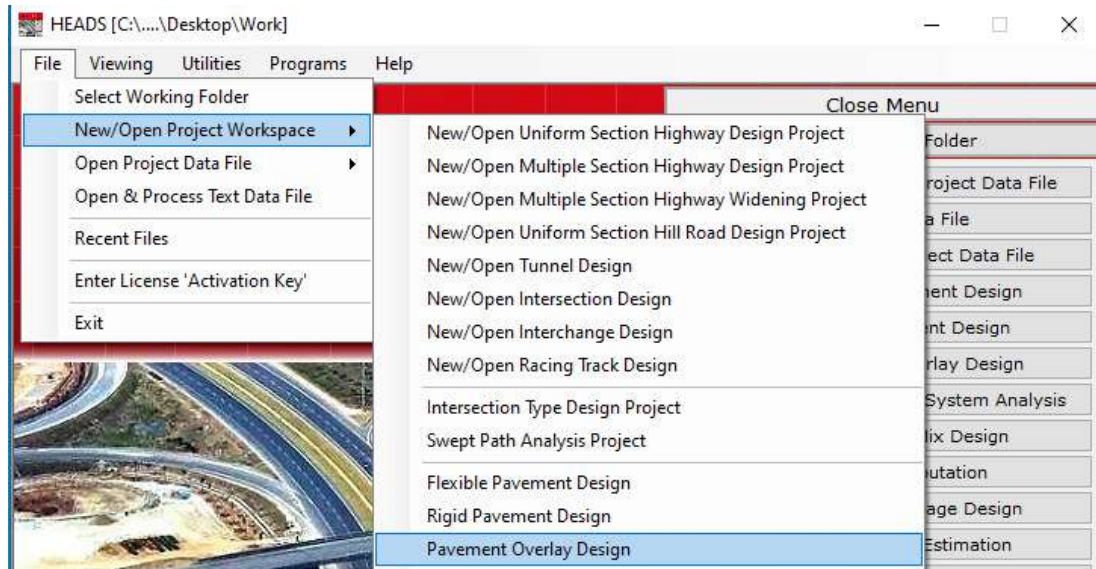


Select folder 'Work' on the Desktop as the working folder,

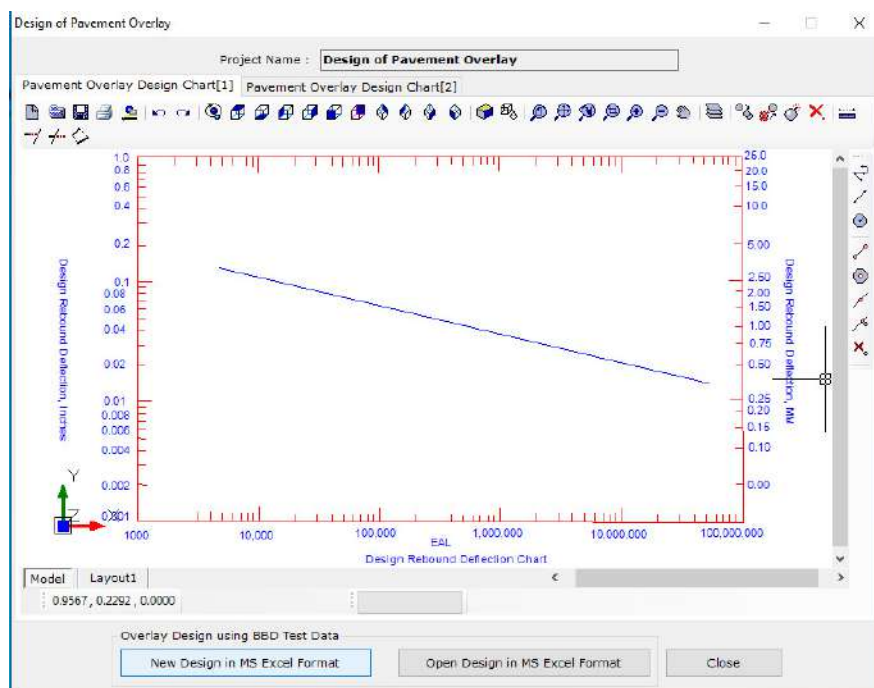


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Select menu item 'File >> New/Open Project Workspace >> Pavement Overlay Design'



The design window opens, on screen the 'Pavement Overlay Design Chart [1]' is displayed, Click on button 'New Design in MS-Excel Format',

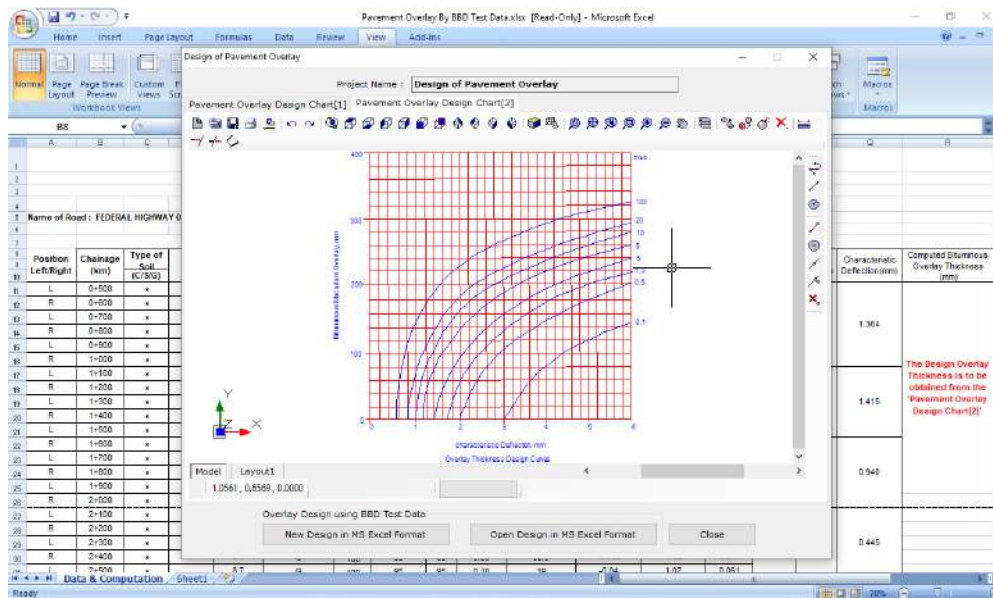


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The design MS-Excel Worksheet opens with sample data to help user understand the data. User may change the data as required and save the Excel Worksheet file on the Desktop by giving a different name, The Worksheet gives ‘Characteristic Deflection’ values.

Position	Chainage (km)	Type of Soil (C/S/S)	PI	Moisture Content %	Surface (Good/Fair/Poor)	Deflection (mm)	Pavement Temperature (Box Celsius)	Temp Correction	Seasonal Correction	Corrected Deflection (mm)	Mean Deflection (mm)	Std Deviation	Characteristic Deflection (mm)	Computed Minimum Overlay Thickness (mm)
L	0-500	s	8.7	6.7	G	100	72	69	0.79	37	-0.02	1.02	0.791	1.364
R	0-500	s	8.7	6.7	G	100	55	51	0.99	37	-0.02	1.02	0.900	
L	0-700	s	8.7	6.7	G	100	67	60	1.21	37	-0.02	1.02	1.212	
R	0-800	s	8.7	6.7	G	100	72	56	0.50	37	-0.02	1.02	0.551	
L	0-900	s	8.7	6.7	G	100	82	60	0.80	37	-0.02	1.02	0.790	
R	1-000	s	8.7	6.7	G	100	62	58	1.05	27	-0.02	1.02	1.054	0.890
L	1-100	s	8.7	6.7	G	100	86	70	0.82	27	-0.02	1.02	0.824	
R	1-200	s	8.7	6.7	G	100	81	73	1.01	27	-0.02	1.02	1.006	
L	1-300	s	8.7	6.7	G	100	80	62	1.11	30	-0.03	1.02	1.102	
R	1-400	s	8.7	6.7	G	100	74	69	0.91	30	-0.03	1.02	0.909	
L	1-500	s	8.7	6.7	G	100	61	61	0.38	38	-0.03	1.02	0.307	0.630
R	1-600	s	8.7	6.7	G	100	74	70	0.83	38	-0.03	1.02	0.820	
L	1-700	s	8.7	6.7	G	100	74	52	0.50	30	-0.03	1.02	0.500	
R	1-800	s	8.7	6.7	G	100	75	73	0.54	30	-0.04	1.02	0.516	
L	1-900	s	8.7	6.7	G	100	83	83	0.34	29	-0.04	1.02	0.308	
R	2-000	s	8.7	6.7	G	100	89	87	0.20	61	-0.08	1.02	0.204	0.406
L	2-100	s	8.7	6.7	G	100	95	95	0.10	61	-0.08	1.02	0.841	
R	2-200	s	8.7	6.7	G	100	89	89	0.22	41	-0.06	1.02	0.163	
L	2-300	s	8.7	6.7	G	100	82	81	0.38	38.5	-0.035	1.02	0.352	
R	2-400	s	8.7	6.7	G	100	86	85	0.30	38.5	-0.035	1.02	0.271	
L	2-400	s	8.7	6.7	G	100	95	95	0.10	50	-0.04	1.02	0.861	0.178

Next, we come back to the design window and open chart ‘Pavement Overlay Design Chart [2]’, For the value of ‘Characteristic Deflection’ and for the MSA value for traffic load as Equivalent Standard Axle Load in Millions, we obtain the thickness of pavement overlay.



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Design Example 6.1

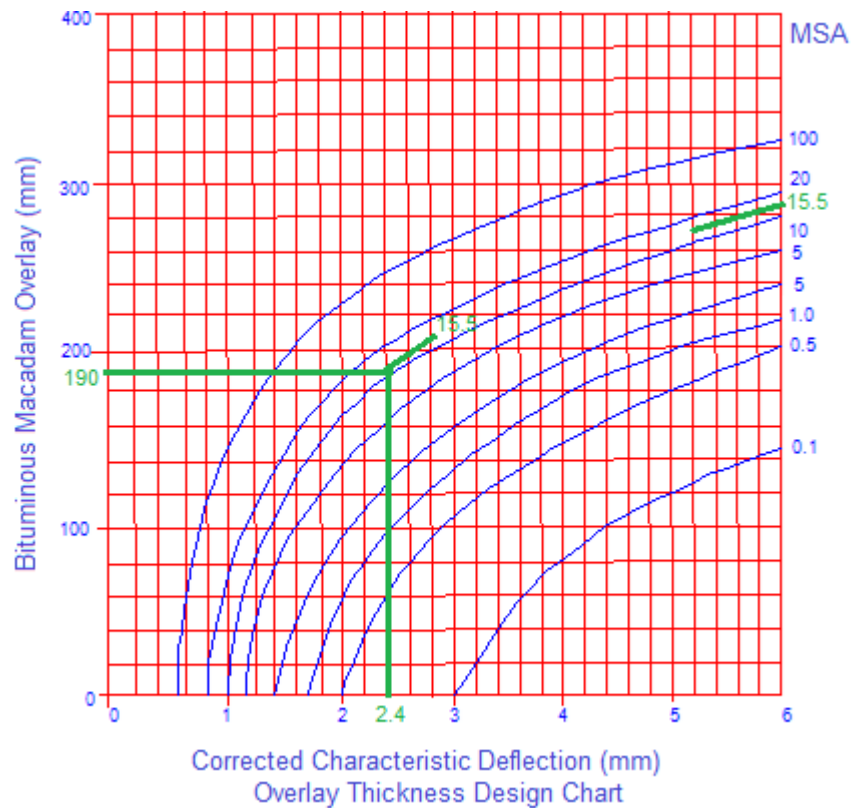
Equivalency Factor

- The thickness deduced from Fig is the overlay thickness in terms of bituminous macadam construction.
- In case other compositions are to be laid for strengthening, the equivalent overlay thickness to be provided may be determined using appropriate equivalency factors as suggested below:
 - 1 cm of Bituminous macadam = 1.5 cm of WBM/WMM/BUSG*
 - 1 cm of Bituminous macadam = 0.7 cm of DBM/AC/SDC
- From structural considerations, the recommended minimum bituminous overlay thickness is 50 mm Bituminous Macadam (BM) + an additional surfacing course of 50mm DBM or 40 mm Bituminous Concrete (BC).
 - BUSG: Built-up Spray Grout
 - WBM: Water Bound Macadam
 - WMM: Wet Mix Macadam
 - DBM: Dense Bituminous Macadam
 - AC: Asphalt Concrete
- Deflection values: 1.40, 1.32, 1.25, 1.35, 1.48, 1.60, 1.65, 1.55, 1.45, 1.40, 1.36, 1.46, 1.5, 1.52, 1.45 mm
- Pavement Temperature = 30°C
- Subgrade moisture content = 8%, clayey soil, PI>15
- Average annual rainfall = 1500 mm
- Design traffic = 15.5 msa
 - Two lane single carriageway
 - ADT of Trucks = 1000
 - Annual growth rate = 5%
 - VDF = 4.5

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- Mean deflection = 1.45 mm
- Standard Deviation = 0.107 mm
- Characteristic deflection = $1.45 + 2(0.107) = 1.664$ mm
- Correction for temperature = $0.01(35-30) = 0.05$ mm
- Characteristic deflection after temp. correction = $1.664 + 0.05 = 1.714$ mm
- Seasonal correction factor = 1.4
- Corrected characteristic deflection = $1.4 (1.714) = 2.3996$ mm = 2.4 mm

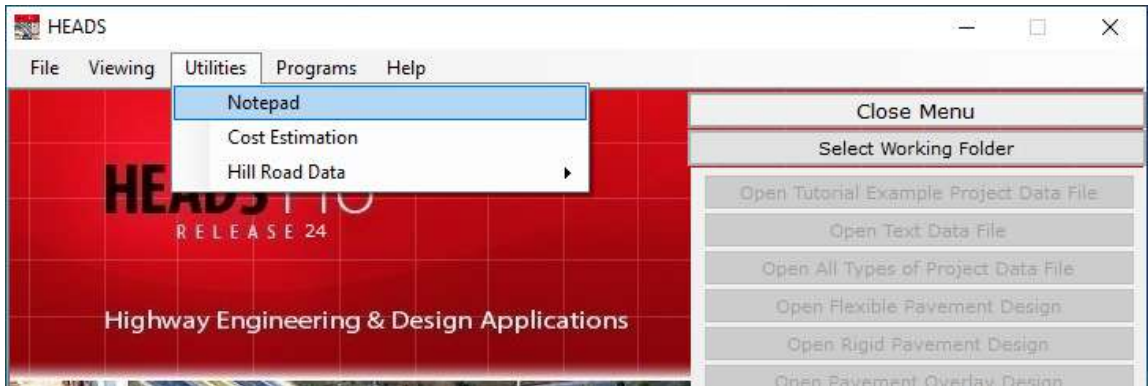
Hence for Design traffic = 15.5 msa and Corrected characteristic deflection = 2.4 mm From Chart below,



- Thickness of overlay in terms of BM from the chart = 190 mm
- Thickness in terms of DBM/AC = $190 \times 0.7 = 133$ mm
- Provide 40 mm AC and 95 mm DBM

6.1.2 Production of Design Report in Text File (Alternative Method)

Open the main screen of HEADS Pro, select menu item 'Utilities >> Notepad',

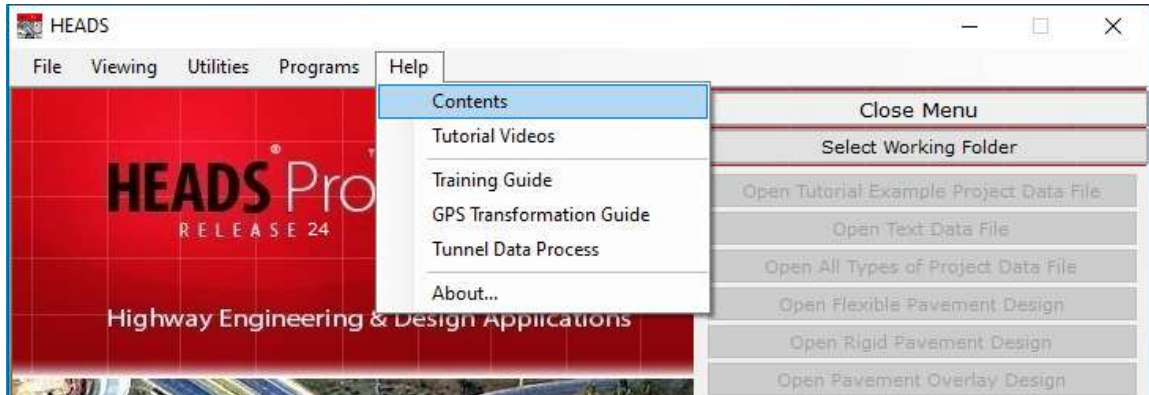


The Notepad opens with no data,

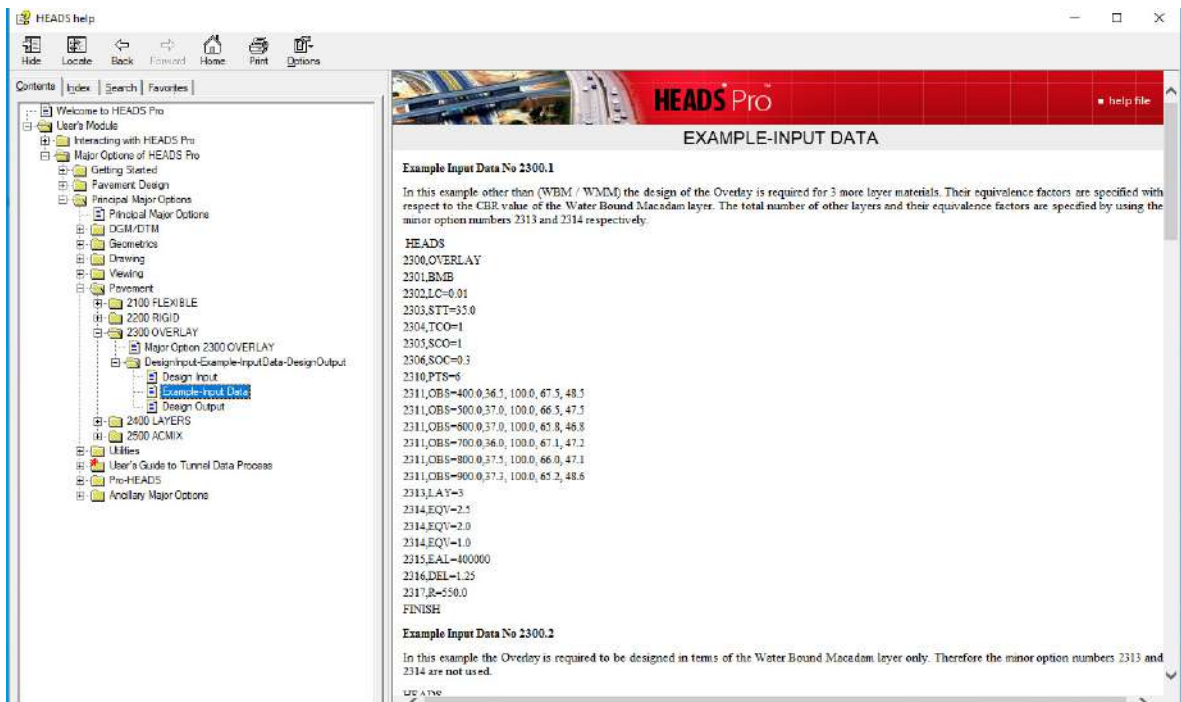


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Next, select menu item, 'Help >> Contents'

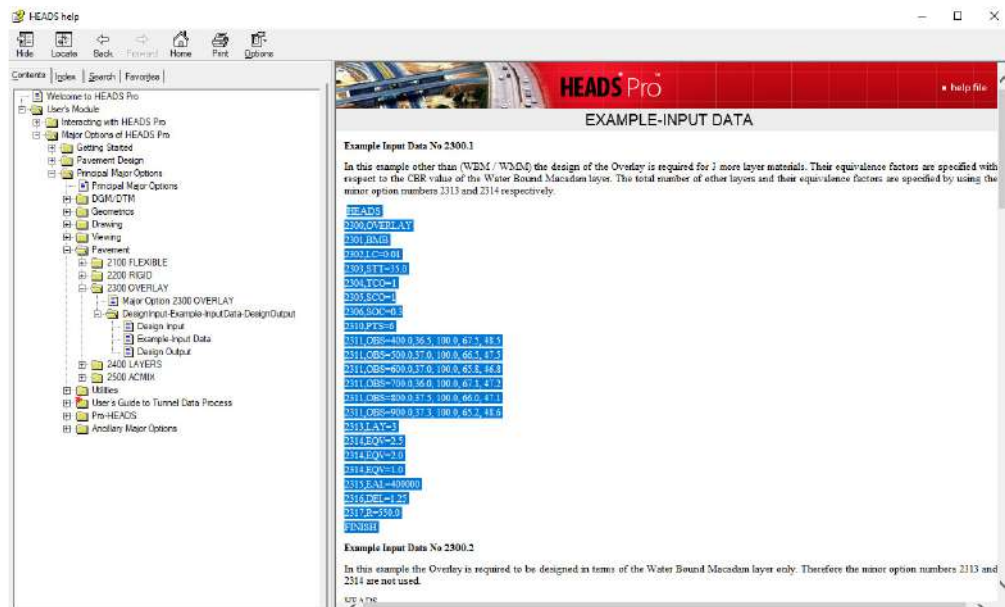


Go down the path on the left side panel to open '....Pavement >> 2300 Overlay >> Design >> Example Input Data', and the example data is displayed on the right side panel of 'Help' window.

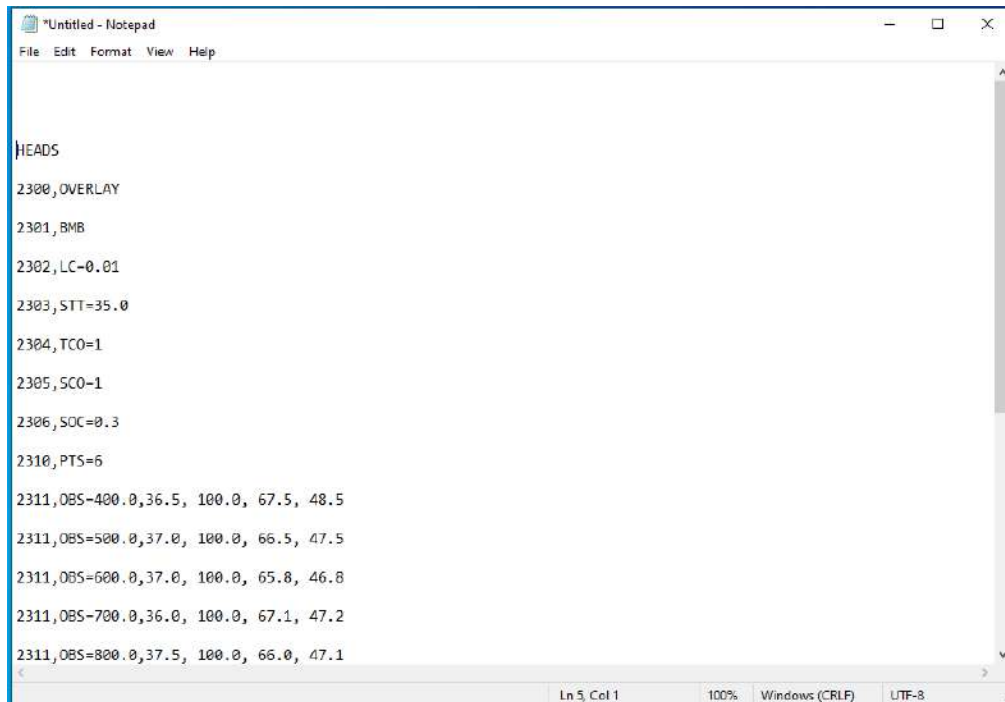


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The data is selected as highlighted, from the right side panel,

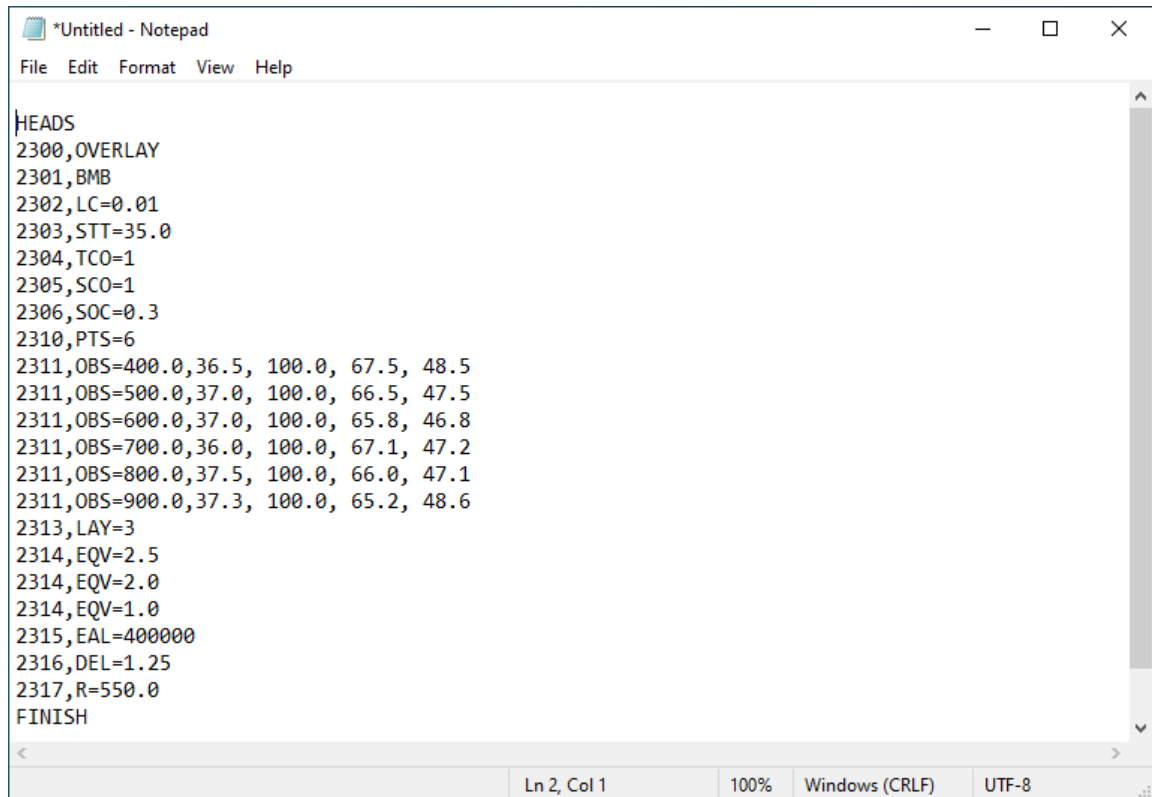


Paste the data in the 'Notepad' already opened, the data has blank lines between each two data lines,

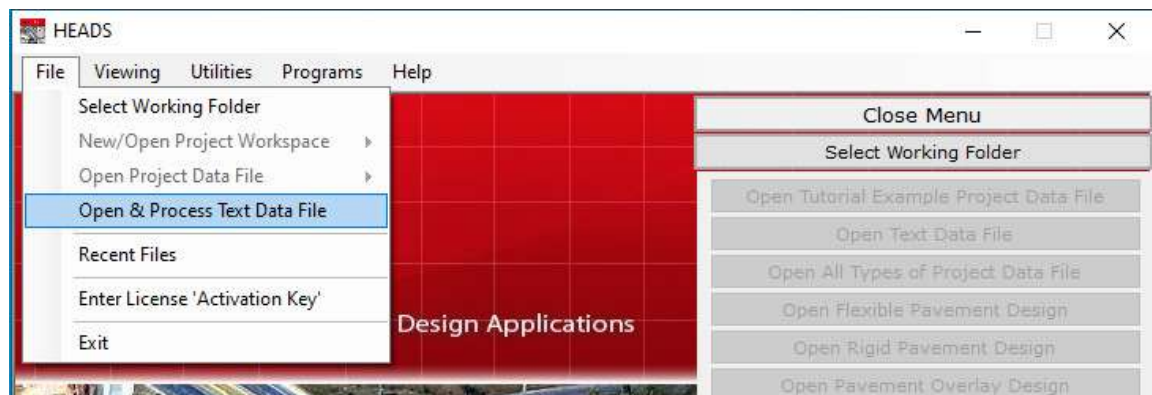


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The blank lines are removed from the data and the data is saved with name 'Overlay.txt' in working folder 'Work' on the Desktop,

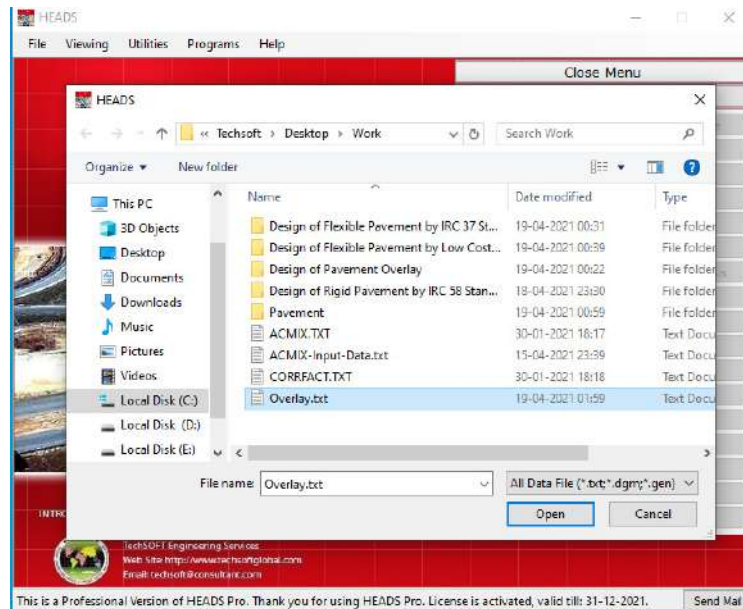


Next, select menu item 'File >> Open & Process Text data File',

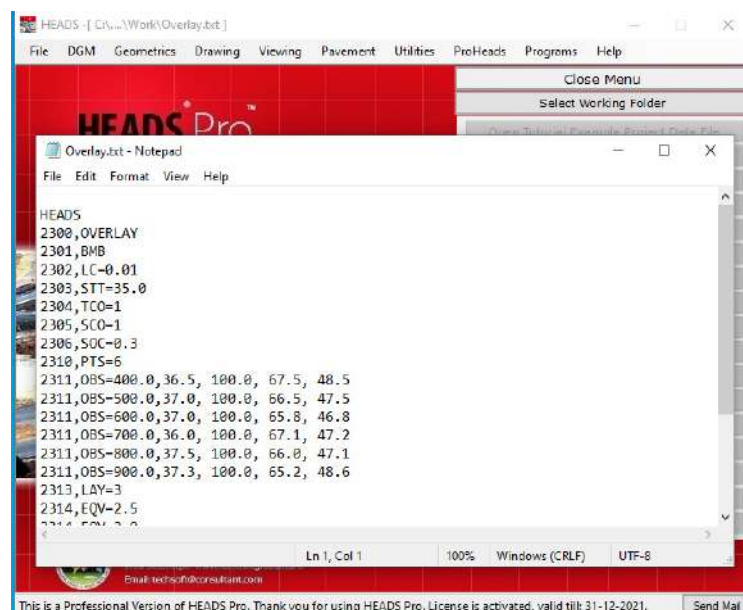


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Select the file 'Overlay.txt' just saved in folder 'Work' on the Desktop, click on button 'open',

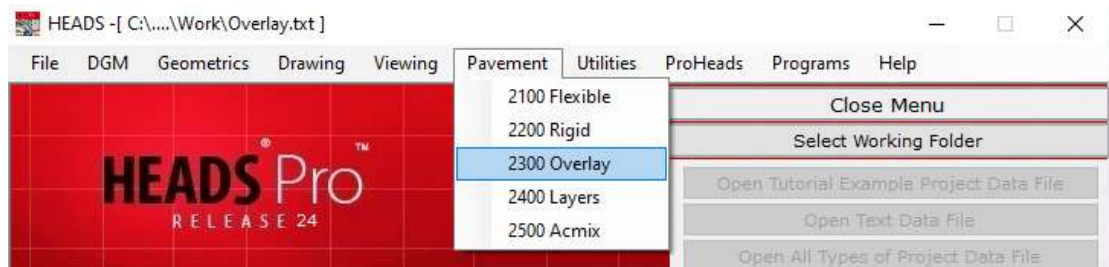


The data file 'Overlay.txt' is selected, and is also opened in Notepad, user may make changes as desired, save and close notepad.

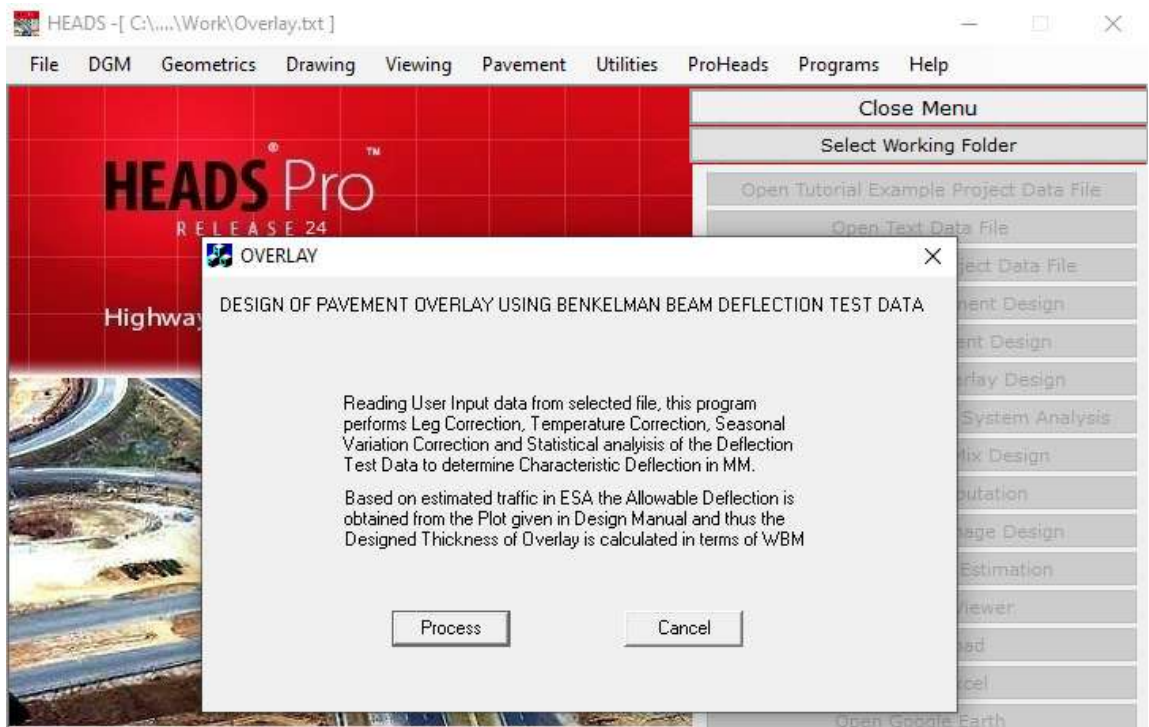


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Select menu item 'Pavement >> Overlay',

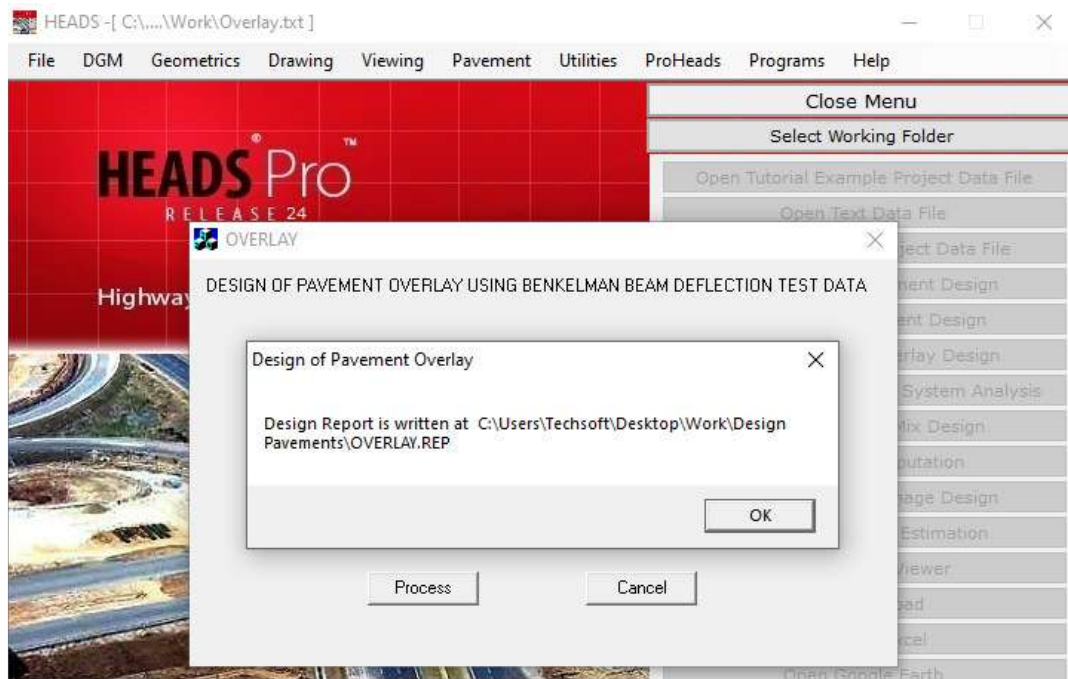


The message comes, click on button 'Process',

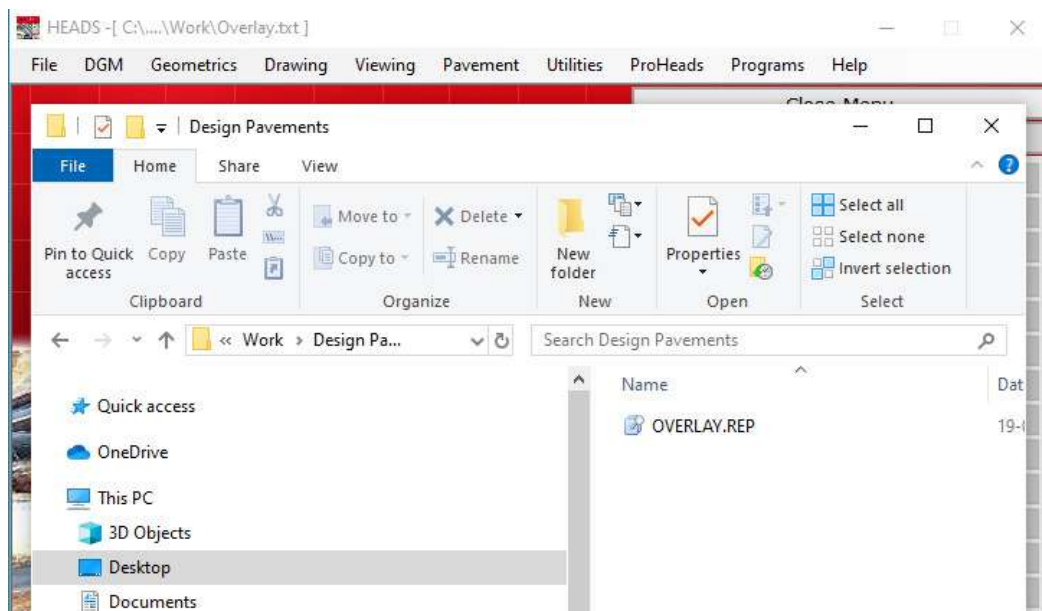


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As the design is done the message comes informing about the Design Report created, click on button 'OK',

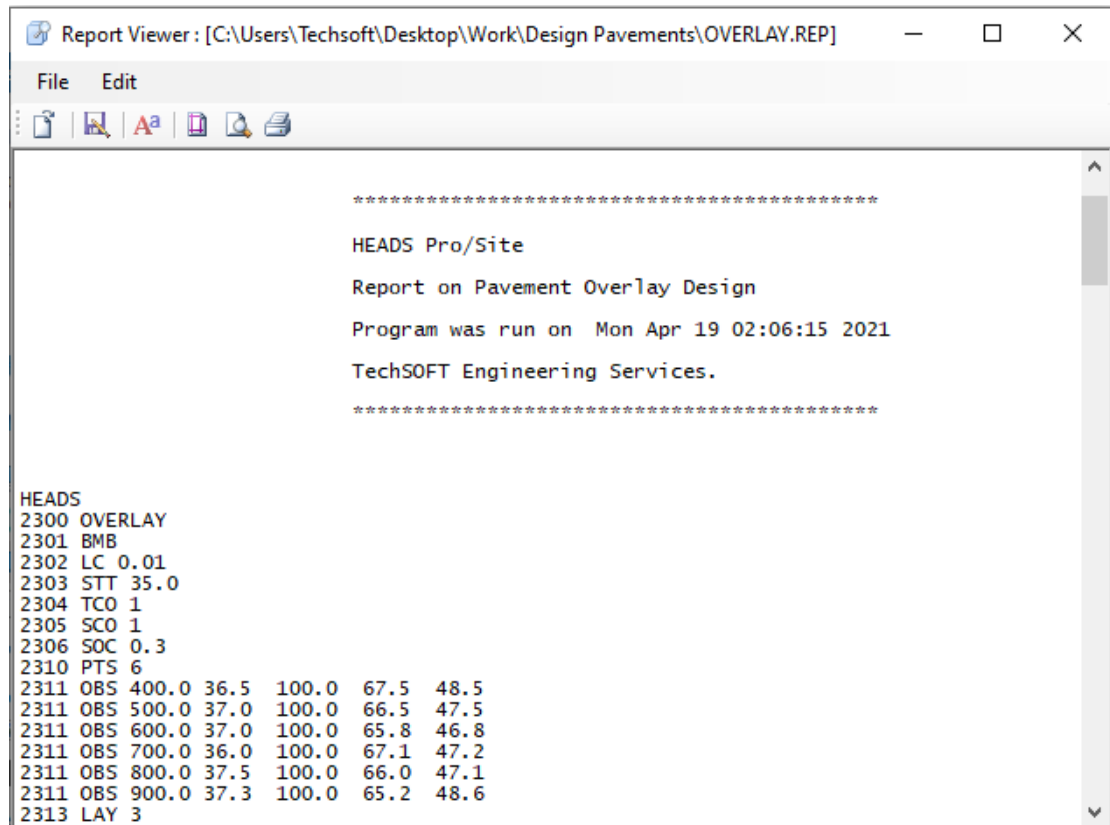


The design report file 'OVERLAY.REP' is created in the folder 'Design Pavements' inside the working folder 'Work' on the Desktop,



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The design report file 'OVERLAY.REP' is opened in Notepad as shown below,



```
Report Viewer : [C:\Users\Techsoft\Desktop\Work\Design Pavements\OVERLAY.REP]
File Edit

*****

HEADS Pro/Site
Report on Pavement Overlay Design
Program was run on Mon Apr 19 02:06:15 2021
TechSOFT Engineering Services.
*****

HEADS
2300 OVERLAY
2301 BMB
2302 LC 0.01
2303 STT 35.0
2304 TCO 1
2305 SCO 1
2306 SOC 0.3
2310 PTS 6
2311 OBS 400.0 36.5 100.0 67.5 48.5
2311 OBS 500.0 37.0 100.0 66.5 47.5
2311 OBS 600.0 37.0 100.0 65.8 46.8
2311 OBS 700.0 36.0 100.0 67.1 47.2
2311 OBS 800.0 37.5 100.0 66.0 47.1
2311 OBS 900.0 37.3 100.0 65.2 48.6
2313 LAY 3
```

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The design report for overlay design is as given below,

HEADS Pro/Site

Report on Pavement Overlay Design

Program was run on Mon Apr 19 02:06:15 2021

TechSOFT Engineering Services.

Design Input Data: (Refer to HEADS Pro – Help, to understand data and syntax)

HEADS
2300 OVERLAY
2301 BMB
2302 LC 0.01
2303 STT 35.0
2304 TCO 1
2305 SCO 1
2306 SOC 0.3
2310 PTS 6
2311 OBS 400.0 36.5 100.0 67.5 48.5
2311 OBS 500.0 37.0 100.0 66.5 47.5
2311 OBS 600.0 37.0 100.0 65.8 46.8
2311 OBS 700.0 36.0 100.0 67.1 47.2
2311 OBS 800.0 37.5 100.0 66.0 47.1
2311 OBS 900.0 37.3 100.0 65.2 48.6
2313 LAY 3
2314 EQV 2.5
2314 EQV 2.0
2314 EQV 1.0
2315 EAL 400000
2316 DEL 1.25
2317 R 550.0
FINISH

DESIGN OF PAVEMENT OVERLAY BASED ON DEFLECTION SURVEY DATA.

Processing of Benkelman Beam Test data.

Calculation for Static Load Deflection data.

LC = Least Count of Dial Gauge = 0.0100

DS = Initial Reading.

DI = Intermediate Reading.

DF = Final Reading.

If $\{ | DI \times LC - DF \times LC | \} \leq 2.5 \times LC \text{ mm.}$

Pavement Deflection = $2 \times \{ | DS \times LC - DF \times LC | \} \text{ mm.}$

Otherwise,

Pavement Deflection = $2 \times \{ | DS \times LC - DF \times LC | \} + \text{Leg Correction, mm.}$

where, Leg Correction = $2.91 \times 2 \times \{ | DI \times LC - DF \times LC | \} \text{ mm.}$

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PAVEMENT DEFLECTION DATA

TEST PT.	CHAIN -AGE.	TEST TEMP.	PAVEMENT INITIAL	DIAL GAUGE READINGS INTERMEDIATE	FINAL	LEG CORRECN.	PAVEMENT DEFLECTION
1	400.000	36.50	100.00	67.50	0.00	3.9285	5.9285 mm.
2	500.000	37.00	100.00	66.50	0.00	3.8703	5.8703 mm.
3	600.000	37.00	100.00	65.80	0.00	3.8296	5.8296 mm.
4	700.000	36.00	100.00	67.10	0.00	3.9052	5.9052 mm.
5	800.000	37.50	100.00	66.00	0.00	3.8412	5.8412 mm.
6	900.000	37.30	100.00	65.20	0.00	3.7946	5.7946 mm.

CORRECTION OF PAVEMENT DEFLECTION FOR TEMPERATURE :

TS = Standard Temperature = 35.000 Deg. Centigrade

T = Temperature During Observation in Deg. Centigrade

If $T < TS$, then,

Pavement Deflection = Pavement Deflection + Temp. Correkn. mm.

If $T > TS$, then,

Pavement Deflection = Pavement Deflection - Temp. Correkn. mm.

where, Temperature Correction = Positive value of $(TS - T) * 0.005$ mm.

TEST PT	CHAIN -AGE	STANDARD TEMP	TEST TEMP	TEMP CORRECN.	CORRECTED DEFLECTION
1	400.000	35.00	36.50	0.01	5.9210 mm.
2	500.000	35.00	37.00	0.01	5.8603 mm.
3	600.000	35.00	37.00	0.01	5.8196 mm.
4	700.000	35.00	36.00	0.01	5.9002 mm.
5	800.000	35.00	37.50	0.01	5.8287 mm.
6	900.000	35.00	37.30	0.01	5.7831 mm.

CORRECTION OF PAVEMENT DEFLECTION FOR SEASONAL VARIATION :

where, Correction Factor for Subgrade Soil = 0.300

Pavement Deflection = Pavement Deflection * $[1 + \text{Corrcn. Factor}]$ mm.

TEST PT	CHAINAGE	SEASONAL CORRECN.	CORRECTED DEFLECTION
1	400.000	0.30	7.6973 mm.
2	500.000	0.30	7.6184 mm.
3	600.000	0.30	7.5654 mm.
4	700.000	0.30	7.6703 mm.
5	800.000	0.30	7.5773 mm.
6	900.000	0.30	7.5181 mm.

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SUMMARY OF DEFLECTION CORRECTIONS

TEST PT.	CHAINAGES	LEG	CORRECTIONS TEMP	SEASON	CORRECTED DEFLECTION
1	400.000	3.93	0.01	0.30	7.6973 mm.
2	500.000	3.87	0.01	0.30	7.6184 mm.
3	600.000	3.83	0.01	0.30	7.5654 mm.
4	700.000	3.91	0.01	0.30	7.6703 mm.
5	800.000	3.84	0.01	0.30	7.5773 mm.
6	900.000	3.79	0.01	0.30	7.5181 mm.

STATISTICAL CALCULATIONS

STATISTICAL COMPUTATIONS FOR CHARACTERISTIC DEFLECTION

$x_1, x_2, x_3, \dots, x_n$ are the total n -numbers of observed deflection values after applying all the corrections.

X_M = Mean Deflection = $[x_1 + x_2 + x_3 + \dots + x_n] / n$

SD = Standard Deviation

$$= [\{ (x_1 - X_M)^2 + (x_2 - X_M)^2 + \dots + (x_n - X_M)^2 \} / (n-1)]^{1/2}$$

Δ_0 = Characteristic Deflection = $X_M + SD$.

MEAN DEFLECTION	STANDARD DEVIATION	CHARACTERISTIC DEFLECTION
7.6078	0.0675	7.6753 mm.

DETERMINATION OF OVERLAY THICKNESS :

h = Thickness of Granular Overlay in mm.

Δ_0 = Characteristic Deflection.

Δ = Allowable Deflection.

R = Constant =

Therefore, $h = R * \log_{10} * (\Delta_0 / \Delta)$

REQUIRED THICKNESSES FOR OVERLAY BY USING DIFFERENT LAYER MATERIALS DEFINED BY RELEVANT EQUIVALENCE FACTORS

Layer = Bituminous Macadam
Equivalence Factor = 1.000
Layer Thickness = 433.503 mm.

Layer # = 2
Equivalence Factor = 2.500
Layer Thickness = 173.401 mm.

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Layer # = 3
Equivalence Factor = 2.000
Layer Thickness = 216.751 mm.

Layer # = 4
Equivalence Factor = 1.000
Layer Thickness = 433.503 mm.

E N D O F D E S I G N.

6.2 Computer Based Analysis for Pavement Rehabilitation of Cracked Layered Flexible Pavement System by using Geo-Synthetics

The cost of Pavement structure is important for every road project and compromising with the structure will not allow the pavement to last long. So the cost reduction is to be done without lowering its strength. Now products are available to incorporate the pavement to increase the strength and longevity at a lesser cost. This is commonly done by laying Geosynthetics layer in between the layers of Asphalt concrete and Dense Bituminous Macadam and also by laying Jute Geotextiles at the top of the Subgrade layer.

Geosynthetics are not decomposable but Jute Geotextiles are decomposable and both increase the life of pavement significantly. Jute Geotextiles are very useful when the road is to be constructed in unstable ground. It is available in plenty in India and Bangladesh and is an inexpensive material. If the ground is stable but traffic load is high then the pavement structure becomes expensive. To reduce the cost either less expensive layers or lesser thickness are to be thought of. In such case if the strength is to be maintained same as the original structure then incorporating of Geosynthetics is an effective solution.

This requires correct analysis to determine the strength of the pavement structure by using various layers of different strength and thicknesses. Every pavement option may also be studied with and without incorporating the Geosynthetics.

Various field tests are available to test the strength of the pavement at site after construction but it is very rare to find a facility to estimate the results before construction. So the procedure described above is expected to be highly useful to project authorities and consultants to decide on the pavement structure and the type of reinforcement to be used. This will ensure increased design life of pavement at a lesser cost.

The strength of pavement is required by obtaining the Elastic modulus and Poisson's ratio. The various companies are producing Falling weight Deflectometer, one of them is Dynatest (www.dynatest.com) and for analysis software HEADS Pro may be used for various options. In HEADS Pro software option is available for Analysis for Layered pavement System. It may be used for 2 or 3 or 4 layer Pavement system, where the Layers may be combined to define as a Bound (Asphalt/Bituminous) Layer, Unbound layer (Granular Base), Sub Base and Sub Grade. The input is required for Thickness of each Layer (excluding Subgrade), Modulus of Elasticity of each Layer and Poisson's ration of each Layer (excluding Subgrade).

6.2.1 Process without applying Geosynthetics

Input Data

Let us consider 3 Layers as:

Thicknesses:

Asphalt Concrete: Thickness = 50 mm,

Wet Mix Macadam : Thickness = 200 mm,

Sub Grade : No Thickness,

Elastic Moduli:

Asphalt Concrete: Elastic Modulus = 3000 N/ Sq. mm.,

Wet Mix Macadam : Elastic Modulus = 600 N/ Sq. mm.,

Sub Grade: Elastic Modulus = 72 N/ Sq. mm.,

Poisson's Ratios:

Asphalt Concrete: Poisson's Ratio = 0.35,

Wet Mix Macadam : Poisson's Ratio = 0.35,

Sub Grade: Poisson's Ratio = 0.35,

Wheel Load = 87452 kN.

Tire Pressure = 0.892 N / Sq. mm.

HEADS Input Data:

HEADS

2400,LAYERS

2401,TOL=3

2402,TH=50,200

2403,EM=3000,600,72

2404,PR=0.35,0.35,0.35

2405,WL=87452

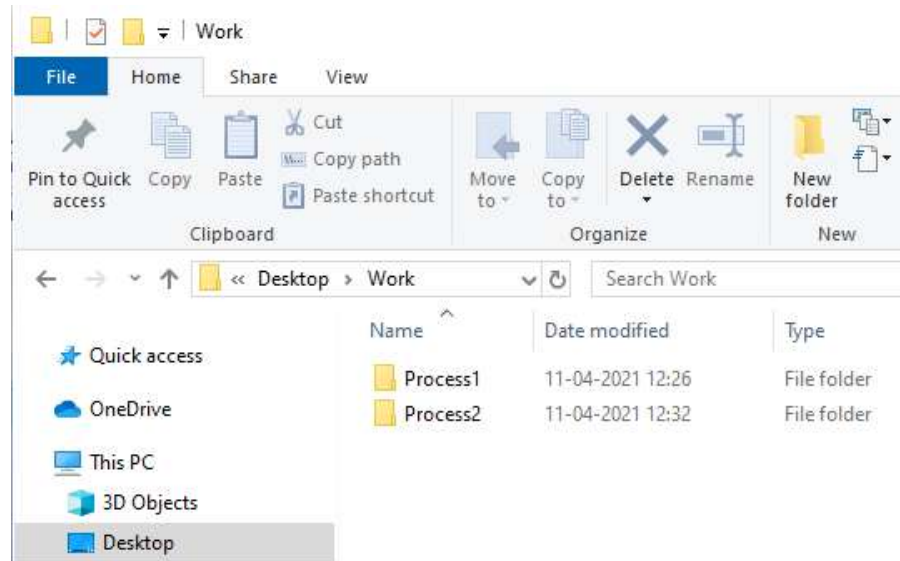
2406,TP=0.892

FINISH

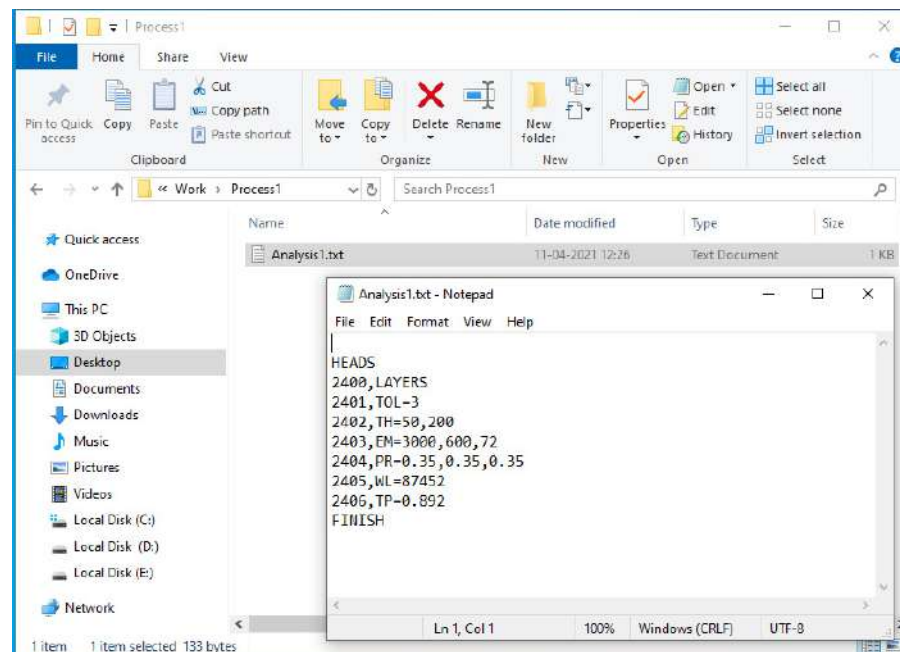
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PROCESS1:

A folder 'Work' is created on the desktop, inside folder 'Work' two more folders are created as 'Process1' and 'Process2'.

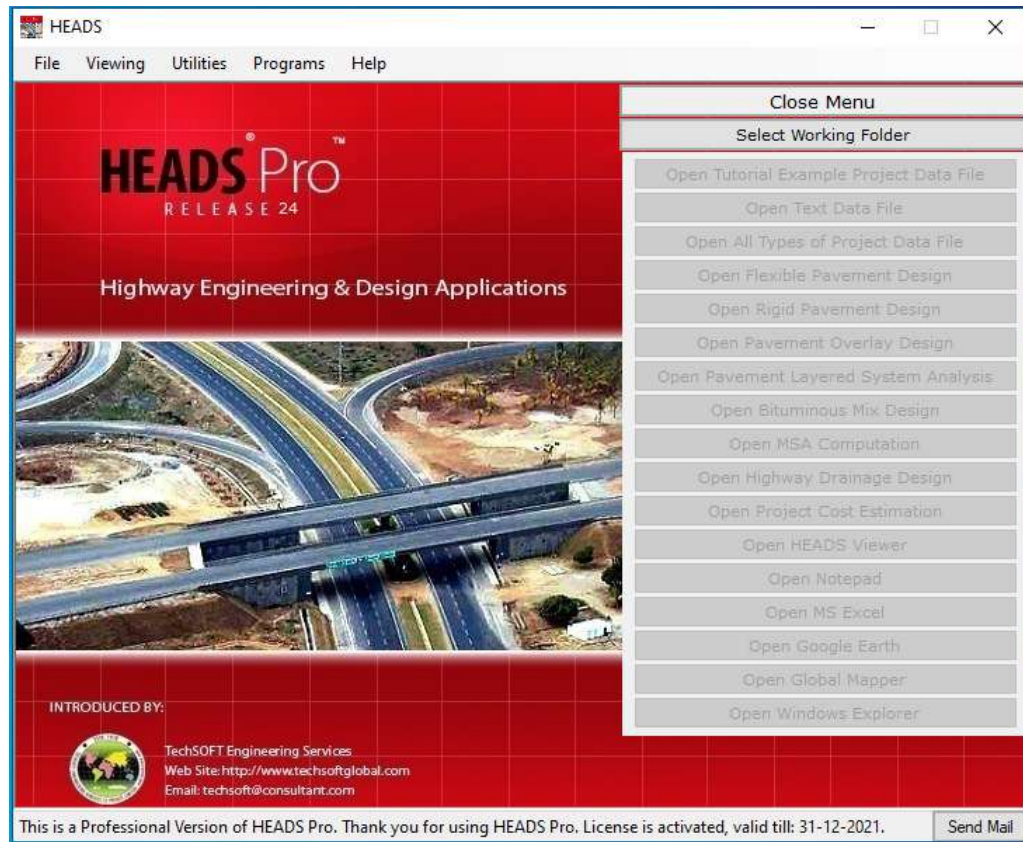


Inside folder1, there is a text file 'Analysis1.txt', the text file contains analysis data for three-layer pavement structure without any layer for 'Geo-synthetics',

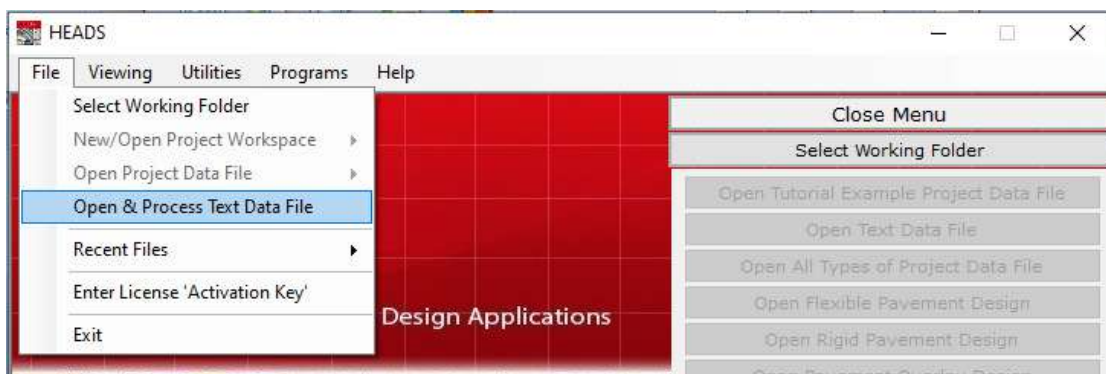


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Open main screen of HEADS Pro,

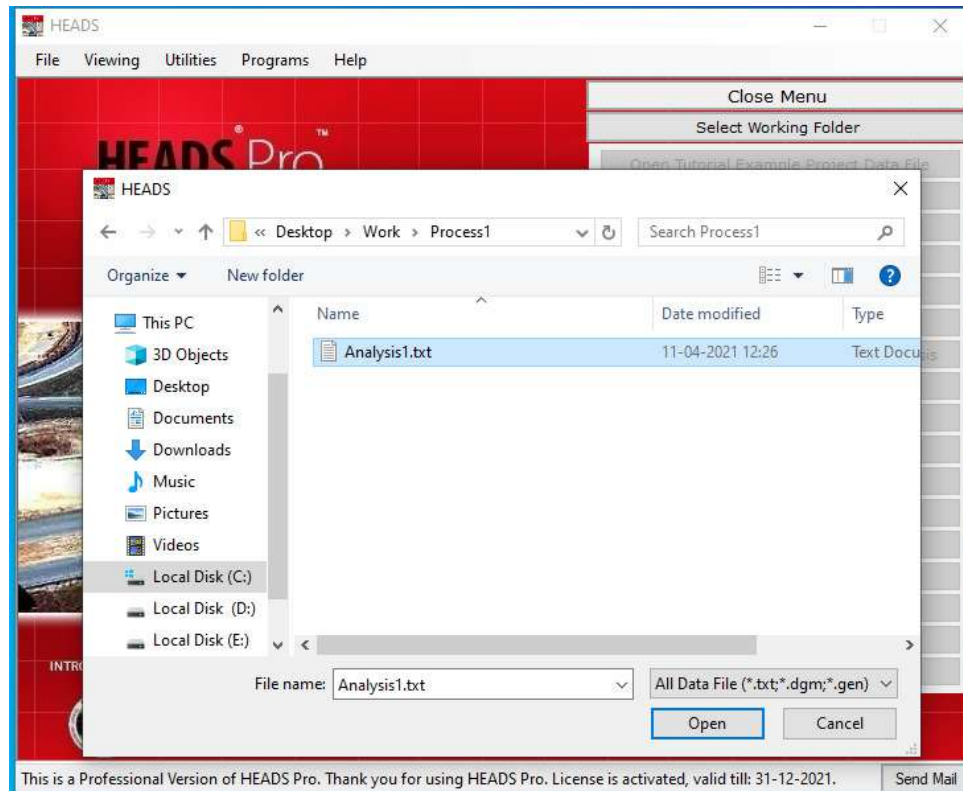


Select menu item 'File >> Open and Process Text Data File',

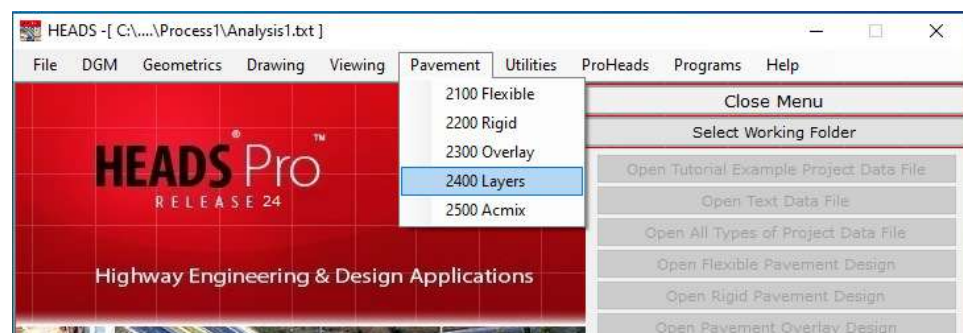


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On the desktop, inside folder 'Work', then, inside folder 'Process1', select the text data file 'Analysis1.txt', click on button 'Open', The data file is selected and opened in 'Notepad', close 'Notepad',

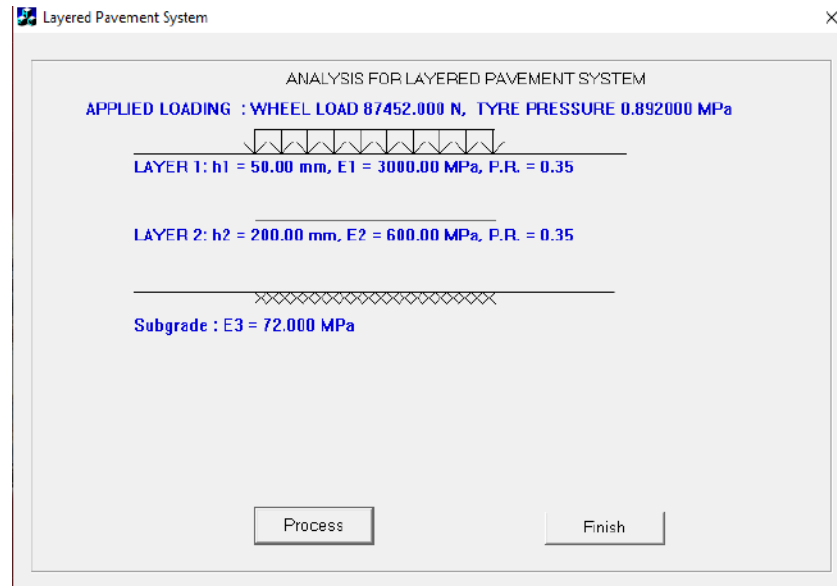


Select HEADS Pro menu item, 'Pavement >> 2400 Layers',

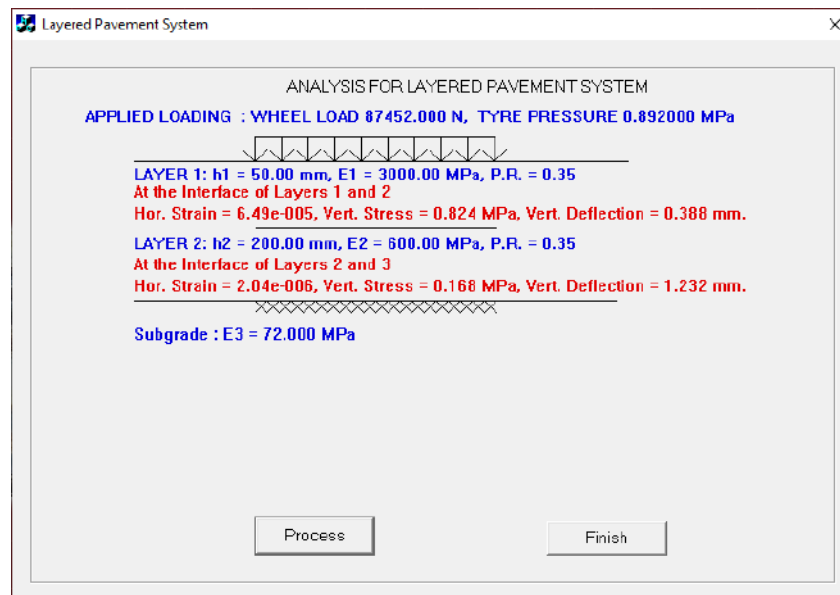


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The window opens for Layered System Analysis, Click on button 'Process',

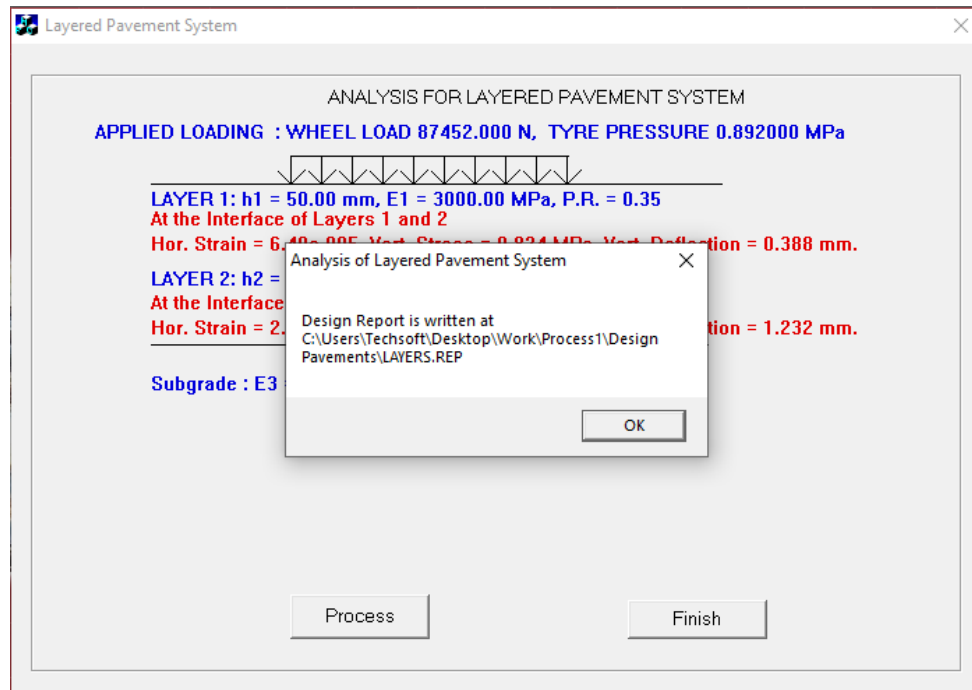


The analysis results as Horizontal Strain, Vertical Stress and Vertical Deflections are displayed in red color at the interface for Layer1 and Layer2 and at the interface of Layer2 and Subgrade, click on button 'Finish',

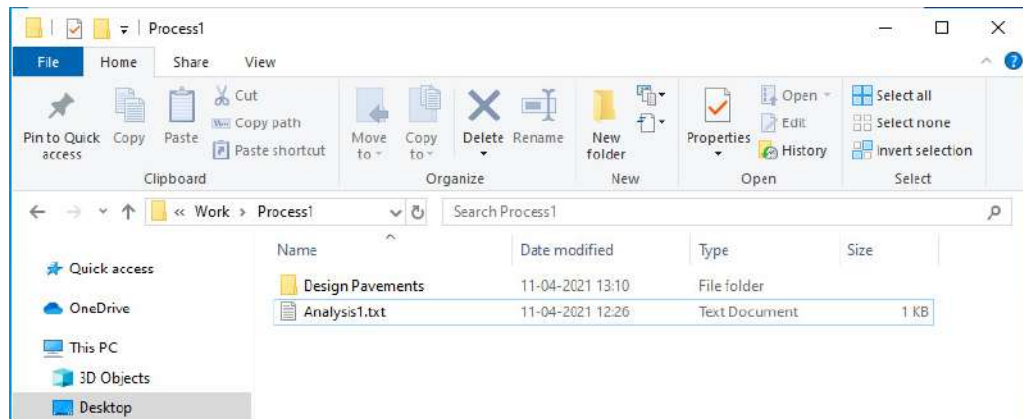


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The message comes informing about the detail analysis report created by HEADS Pro, inside the folder as described, click on button ‘OK’,

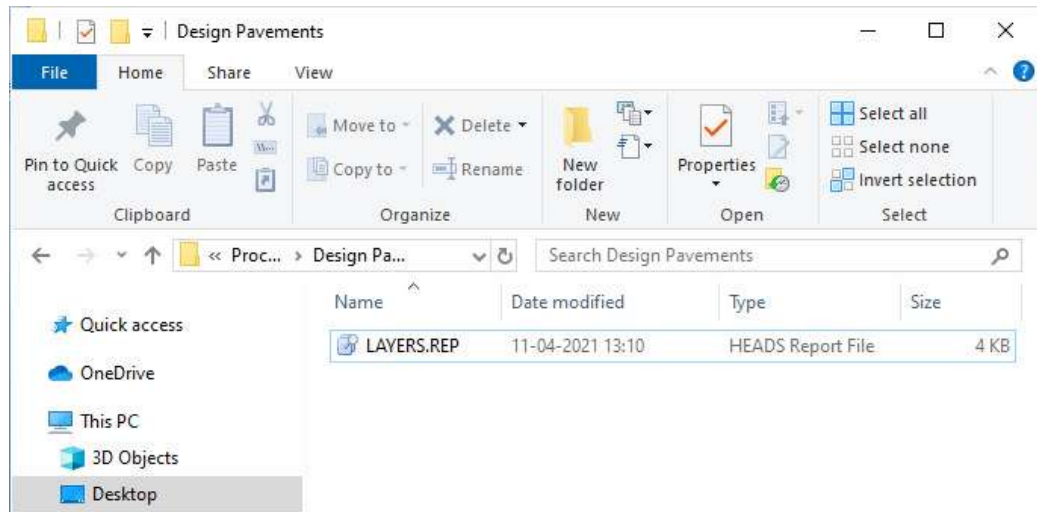


Inside the folder ‘Process1’ a new folder is created as ‘Design Pavements’,

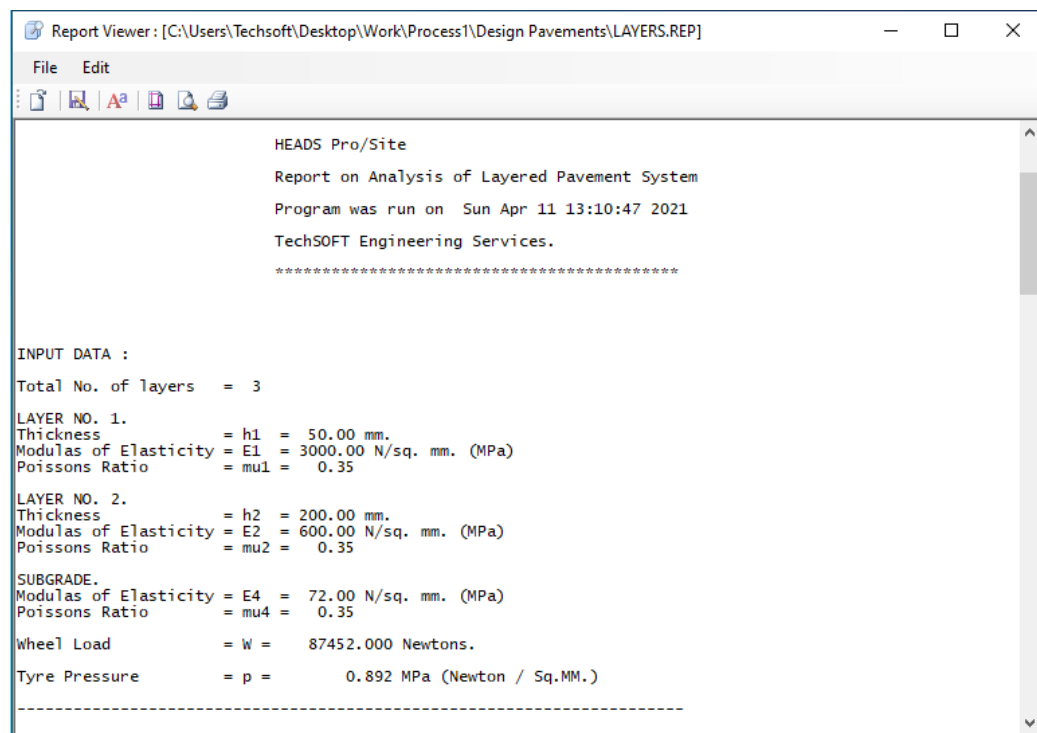


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Inside the folder ‘Design Pavements’ the analysis report is created as ‘LAYERS.REP’,



The report file is opened in ‘Notepad’ for viewing various details of the analysis, and/or for taking a printout, next, close the ‘Notepad’,



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Analysis Input Data File (Analysis1.txt) for Process without applying Geosynthetics

```
HEADS
2400,LAYERS
2401,TOL=3
2402,TH=50,200
2403,EM=3000,600,72
2404,PR=0.35,0.35,0.35
2405,WL=87452
2406,TP=0.892
FINISH
```

Analysis Report File (Layers.REP) for Process without applying Geosynthetics

```
*****
HEADS Pro/Site
Report on Analysis of Layered Pavement System
Program was run on Sun Apr 11 13:10:47 2021
TechSOFT Engineering Services.
*****
```

INPUT DATA :

Total No. of layers = 3

LAYER NO. 1.
Thickness = h1 = 50.00 mm.
Modulus of Elasticity = E1 = 3000.00 N/sq. mm. (MPa)
Poissons Ratio = mu1 = 0.35

LAYER NO. 2.
Thickness = h2 = 200.00 mm.
Modulus of Elasticity = E2 = 600.00 N/sq. mm. (MPa)
Poissons Ratio = mu2 = 0.35

SUBGRADE.
Modulus of Elasticity = E4 = 72.00 N/sq. mm. (MPa)
Poissons Ratio = mu4 = 0.35

wheel Load = W = 87452.000 Newtons.

Tyre Pressure = p = 0.892 MPa (Newton / Sq.MM.)

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DESIGN COMPUTATIONS :

Contact Area = wheel Load / Tyre Presssure = w / p sq.mm.
 Radius of Equiv Circ. Area = $a = \sqrt{\text{area}/3.1416}$ = 176.66 mm.

$\sigma_0 = p = 0.892$ N/sq.mm (MPa)

AT INTERFACE OF LAYERS I & II :

 $\text{check} = (h_1/a) * (E_1/E_2)$

Correction Factor for strain,

if check greater than 10,
 $fe = 1.13 - 0.0565 * [\log_e((h_1/a)**2)] / 2.303 * (E_1/E_2)$

if check less than or equals to 10,
 $fe = 0.96 + [0.73 * (a/he) * (E_2/E_1)]$

where, for the first layer, $he = h_1$,

Then, $fe = 1.476$

Correction Factor for stress,
 $fs = 0.99 - 0.07 * (h_1/a)$

$fs = 0.970$

Equivalent thickness of layer 1 with respect to E-value of layer 2,
 $h_{eq_e} = fe * h_1 * ((E_1/E_2)**0.33)$

$h_{eq_e} = 126.181$ mm.

$h_{eq_s} = (h_{eq_e}/fe)*fs$
 $= 82.949$ mm.

Assign, $z = h_{eq_s}$ mm.

$F1 = 1 + (1 + (3/(2*(1-\mu_1)))) * (z / a) * (z / a)$
 $= 1.729$

$F2 = (1 + (z / a)**2)**2.5$
 $= 1.646$

The Radius of Curvature,
 $R = E_2 * a / ((1 - \mu_1 * \mu_1) * \sigma_0) * F1 * F2$
 $= 385355.224$ mm.

Horizontal Strain,
 $\epsilon_{h_h} = h_1 / (2 * R)$
 $= 6.488e-005$ mm. / mm.

Vertical Stress,
 $\sigma_z = \sigma_0 * (1 - 1/(((1 + ((a/z)**2))**1.5)))$
 $= 0.824$ N/sq.mm. (MPa)

$F1 = ((1 + ((z/a)**2))**0.5) - (z/a)$
 $= 0.635$

$F2 = 1 / (((1 + ((z/a)**2))**0.5)) + (1 - 2*\mu_1)*F1$
 $= 1.096$

Vertical deflection,
 $dz = ((1 + \mu_1) * \sigma_0 * a * F2) / E_2$
 $= 0.388$ mm.

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AT INTERFACE OF LAYERS II & III (For a 3-layer system) :

Correction Factor for strain,

$$f_e = 0.96 + [0.73 * (a/h_e) * (E_3/E_2)]$$

where, for the second layer, $h_e = h_2 + h_{eq_e}$ (of last layer),

Then, $f_e = 1.007$

Correction Factor for stress,

$$f_s = 0.76 - 0.24 * (E_3/E_2)$$

$$f_s = 0.789$$

Equivalent thickness of layer 1 & 2 with respect to E-value of layer 3,

$$h_{eq_e} = f_e * [h_1 * ((E_1/E_3)**0.33) + h_2 * ((E_2/E_3)**0.33)]$$

$$h_{eq_e} = 583.128 \text{ mm.}$$

$$F_1 = 1 / (((1 + (1 + (3 / (2 * (1 - \mu_2)))))) * (z / a)**2) \\ = 23.095$$

$$F_2 = ((1 + (z / a)**2)**2.5) \\ = 163.449$$

The Radius of Curvature,

$$R = E_3 * a / ((1 - \mu_2 * \mu_2) * \sigma_0) * F_1 * F_2 \\ = 61340238.200 \text{ mm.}$$

Horizontal Strain,

$$\epsilon_{h_h} = (h_1 + h_2) / (2 * R) \\ = 2.038e-006 \text{ mm. / mm.}$$

$$h_{eq_s} = (h_{eq_e} / f_e) * f_s \\ = 456.573 \text{ mm.}$$

Assign, $z = h_{eq_s}$ mm.

Vertical Stress,

$$\sigma_z = \sigma_0 * (1 - 1 / (((1 + ((a/z)**2))**1.5))) \\ = 0.168 \text{ N/sq.mm. (MPa)}$$

$$F_1 = (1 + ((z/a)**2)**0.5) - (z/a) \\ = 0.187$$

$$F_2 = 1 / (\text{pow}((1 + ((z/a)**2))**0.5)) + (1 - 2 * \mu_2) * F_1 \\ = 0.417$$

Vertical deflection,

$$dz = ((1 + \mu_2) * \sigma_0 * a * F_2) / E_3 \\ = 1.232 \text{ mm.}$$

E N D O F A N A L Y S I S .

6.2.2 Process with Fiber Glass Geosynthetics

Input Data with Fiber Glass Geosynthetics

Thickness of Fiber Glass Geosynthetics = 3 mm.

Tensile Strength of Fiber Glass Geosynthetics = 200 kN/m.

= $(200 \times 1000) / (1000 \times 3) = 67 \text{ N/ Sq. mm.}$

Elastic Modulus = 67 N/ Sq. mm. Poisson's Ratio = 0.45

Let us consider 3 Layers as:

Thicknesses:

Asphalt Concrete: Thickness = 50 mm,

Fiber Glass Geo Synthetics: Thickness = 3 mm,

Wet Mix Macadam : Thickness = 200 mm,

Elastic Moduli:

Asphalt Concrete: Elastic Modulus = 3000 N/ Sq. mm.,

Fiber Glass Geo Synthetics: Elastic Modulus = 67 N/ Sq. mm.,

Wet Mix Macadam : Elastic Modulus = 200 N/ Sq. mm.,

Poisson's Ratios:

Asphalt Concrete: Poisson's Ratio = 0.35,

Fiber Glass Geo Synthetics: Poisson's Ratio = 0.45,

Wet Mix Macadam : Poisson's Ratio = 0.35,

Wheel Load = 87452 kN.

Tire Pressure = 0.892 N / Sq. mm.

HEADS Input Data:

HEADS

2400,LAYERS

2401,TOL=4

2402,TH=50,3,200

2403,EM=3000,67,600,72

2404,PR=0.35,0.45,0.35

2405,WL=87452

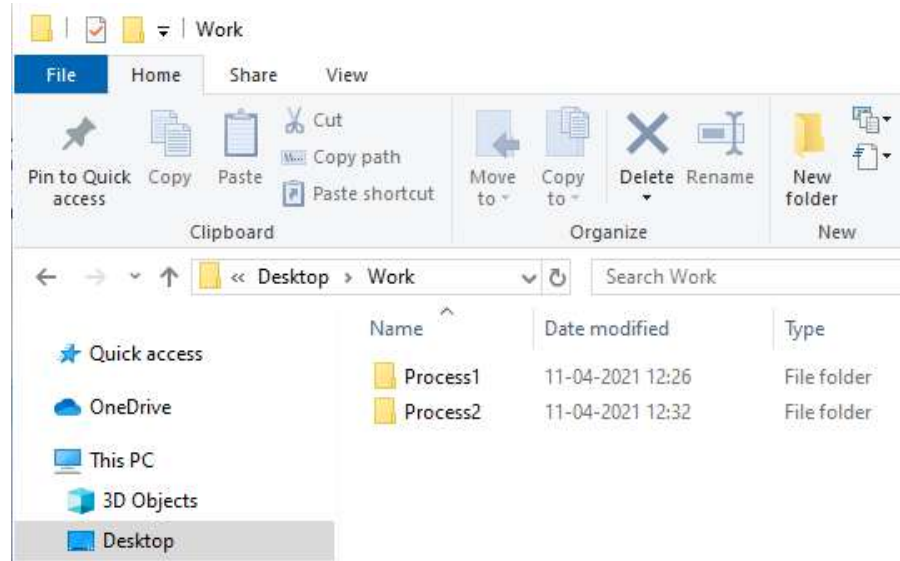
2406,TP=0.892

FINISH

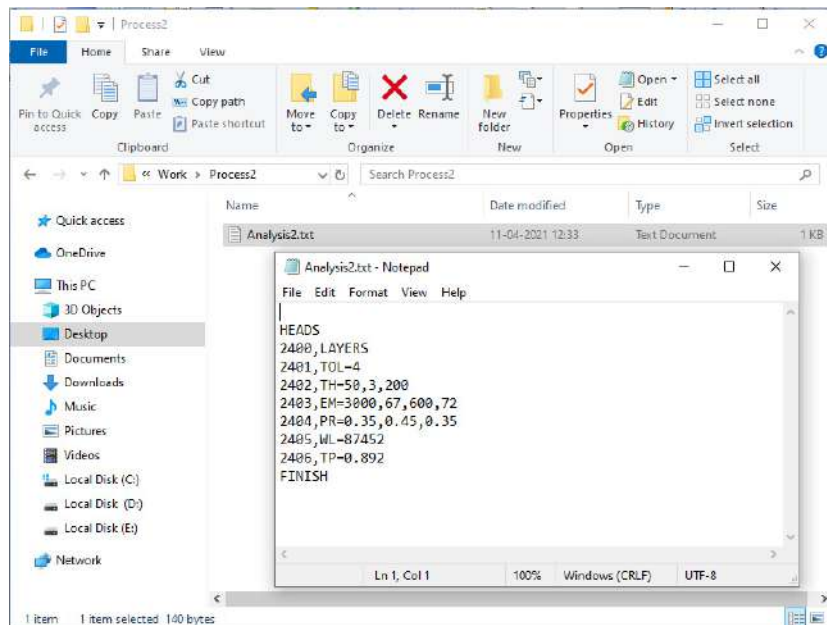
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PROCESS2:

We are come back to folder 'Work' on the desktop, inside folder 'Work' there are folders 'Process1' and 'Process2'. Process1 is already done, now we shall work for Process2,

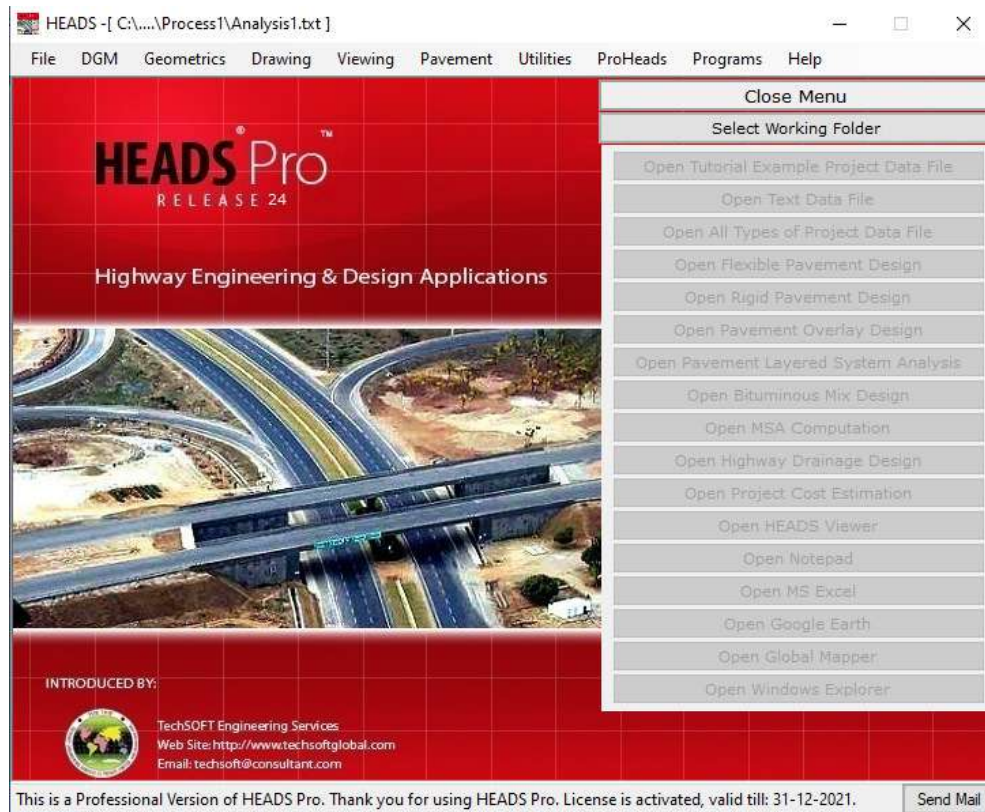


Inside folder2, there is a text file 'Analysis2.txt', the text file contains analysis data for four-layer pavement structure including the layer of 'Geo-synthetics' as a new layer,

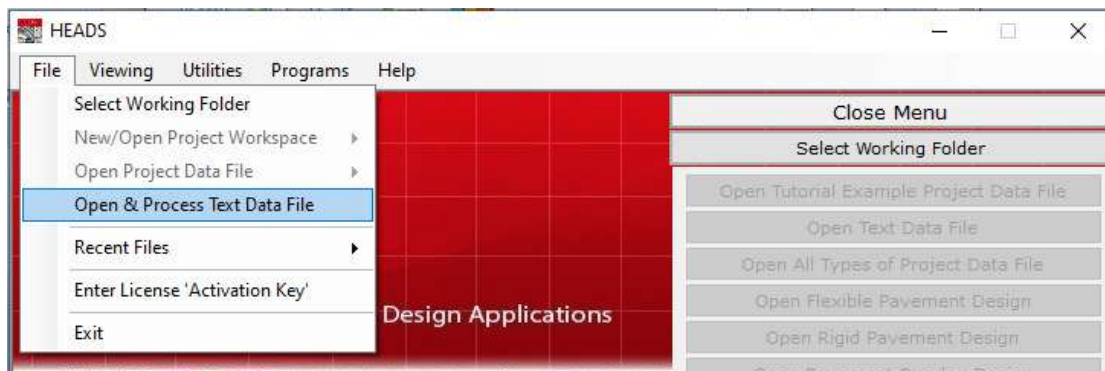


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The main screen of HEADS Pro is already opened,

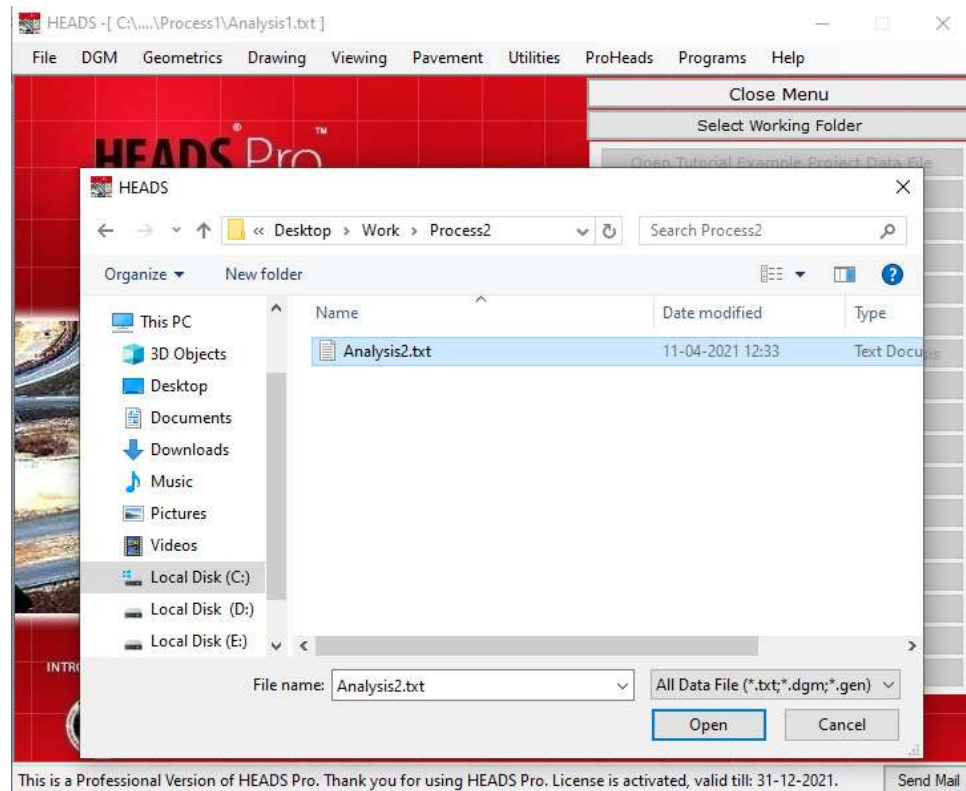


Once again select menu item 'File >> Open and Process Text Data File',

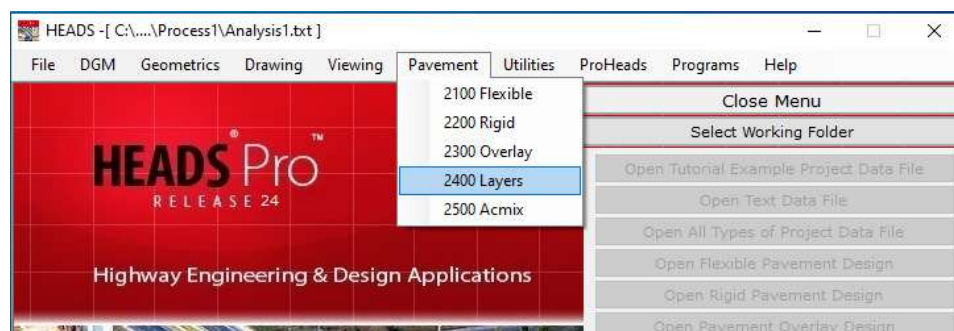


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On the desktop, inside folder 'Work', then, inside folder 'Process2', select the text data file 'Analysis2.txt', click on button 'Open', The data file is selected and opened in 'Notepad', close 'Notepad',

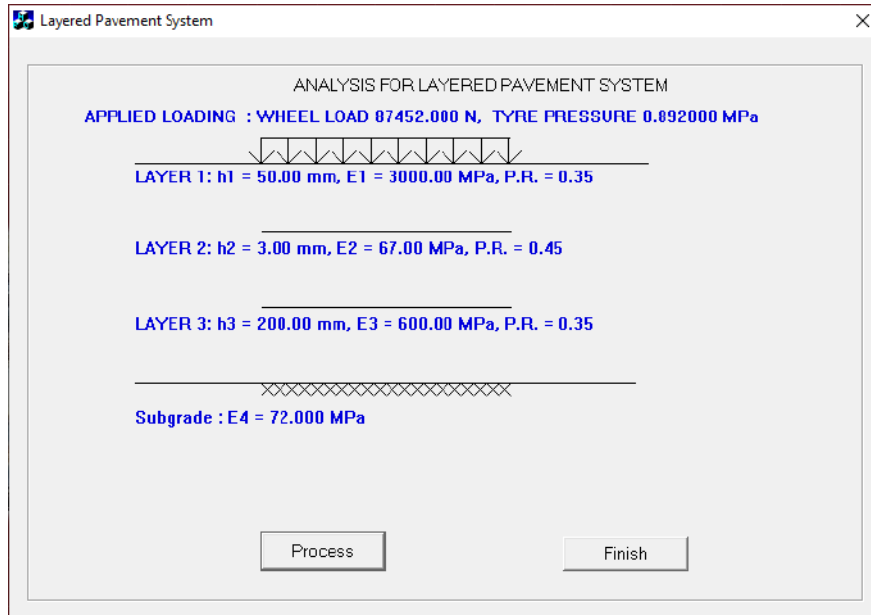


Once again select HEADS Pro menu item, 'Pavement >> 2400 Layers',

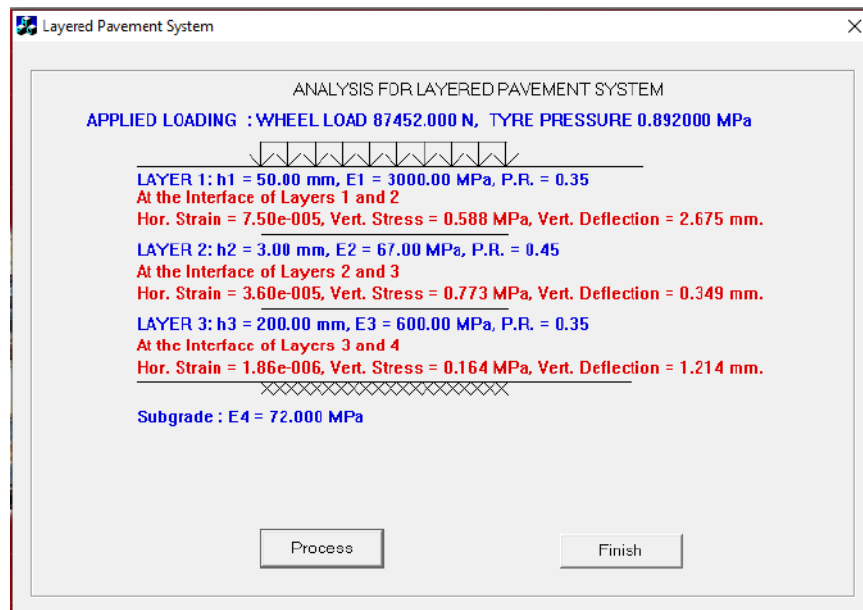


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The window opens for Layered System Analysis, Click on button ‘Process’,

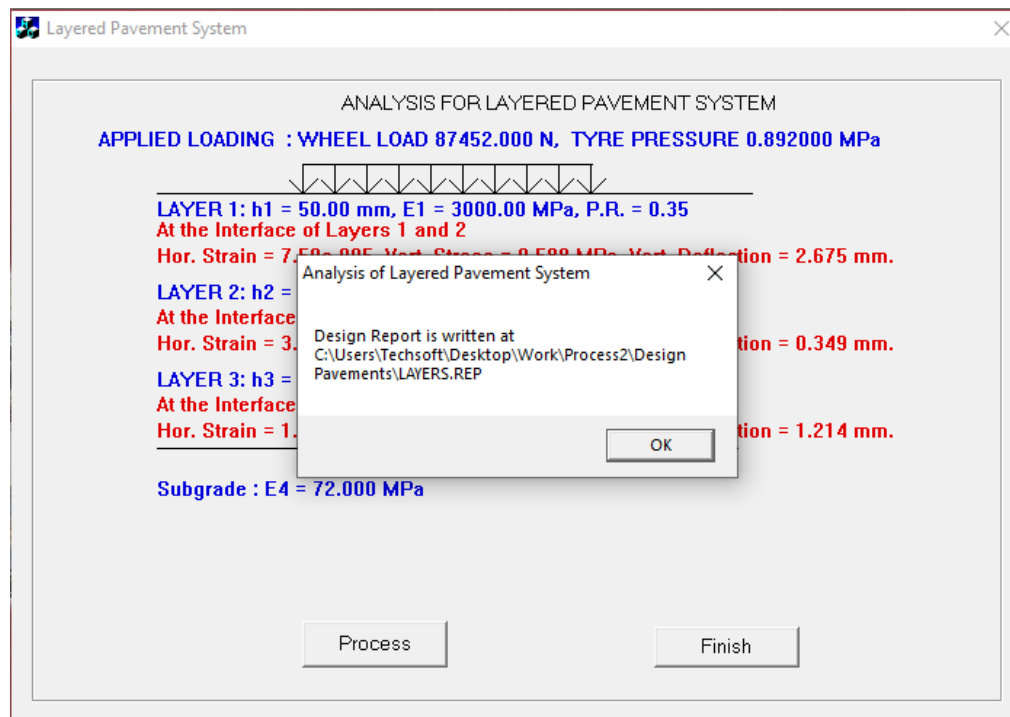


The analysis results as Horizontal Strain, Vertical Stress and Vertical Deflections are displayed in red color at the interface for Layer1 and Layer2, then at Layer2 and Layer3 and finally at Layer3 and Subgrade, click on button ‘Finish’,

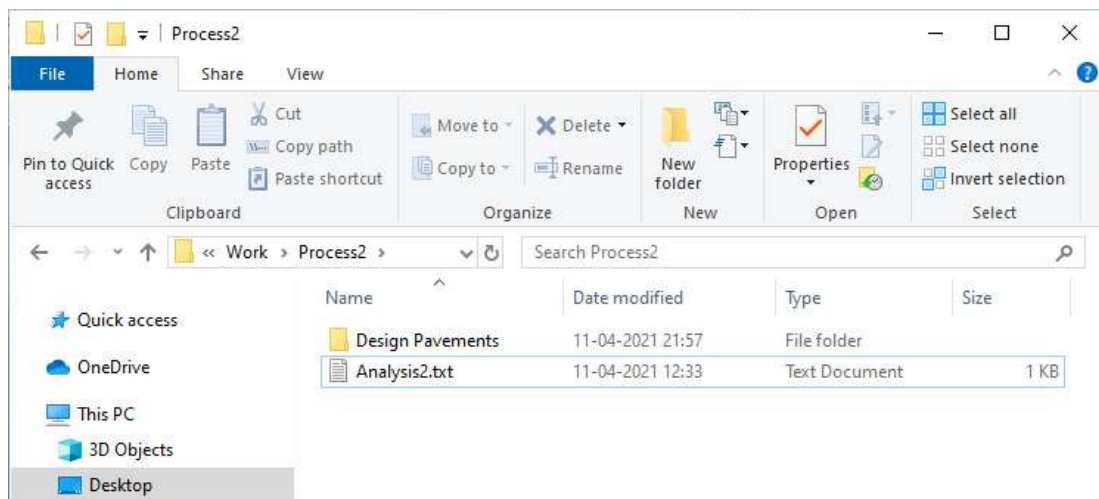


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The message comes informing about the detail analysis report created by HEADS Pro, inside the folder as described, click on button ‘OK’,

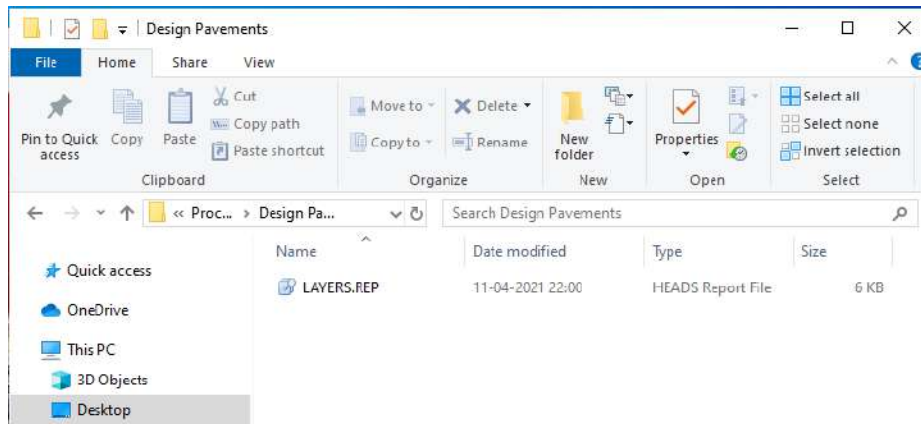


Inside the folder ‘Process1’ a new folder is created as ‘Design Pavements’,

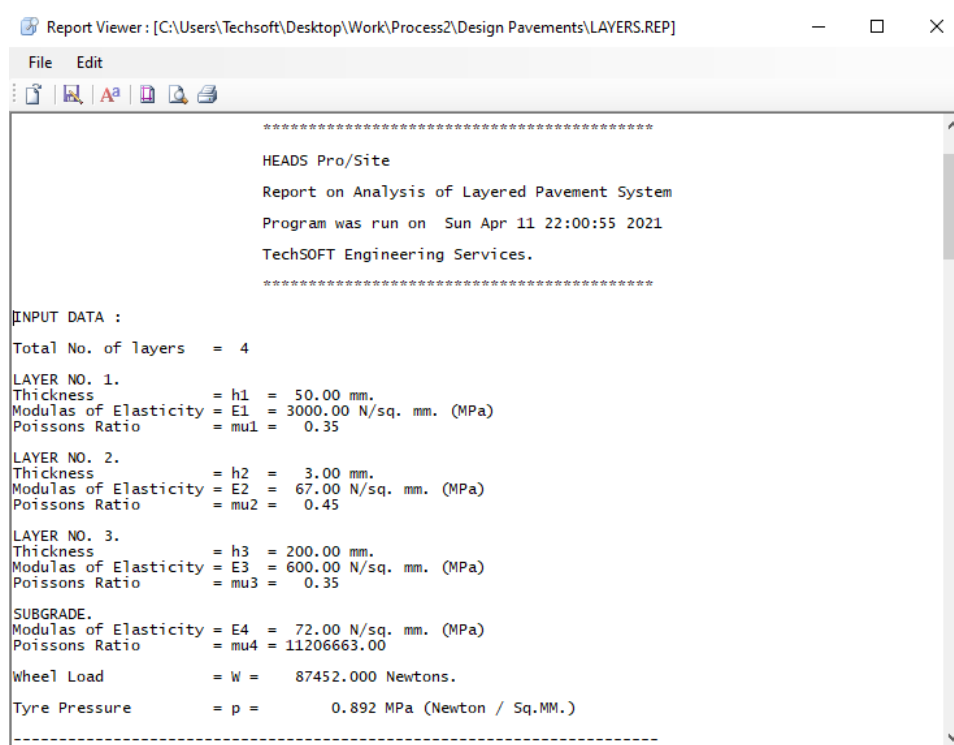


Inside the folder ‘Design Pavements’ the analysis report is created as ‘LAYERS.REP’,

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The report file is opened in 'Notepad' for viewing various details of the analysis, and/or for taking a printout, next, close the 'Notepad',



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Analysis Input Data File (Analysis2.txt) for Process by applying Geosynthetics

```
HEADS
2400,LAYERS
2401,TOL=4
2402,TH=50,3,200
2403,EM=3000,67,600,72
2404,PR=0.35,0.45,0.35
2405,WL=87452
2406,TP=0.892
FINISH
```

Analysis Report File (Layers.REP) for Process by applying Geosynthetics

```
*****
HEADS Pro/Site
Report on Analysis of Layered Pavement System
Program was run on Sun Apr 11 22:15:53 2021
TechSOFT Engineering Services.
*****
```

INPUT DATA :

```
Total No. of layers    = 4

LAYER NO. 1.
Thickness              = h1 = 50.00 mm.
Modulus of Elasticity = E1 = 3000.00 N/sq. mm. (MPa)
Poissons Ratio        = mu1 = 0.35

LAYER NO. 2.
Thickness              = h2 = 3.00 mm.
Modulus of Elasticity = E2 = 67.00 N/sq. mm. (MPa)
Poissons Ratio        = mu2 = 0.45

LAYER NO. 3.
Thickness              = h3 = 200.00 mm.
Modulus of Elasticity = E3 = 600.00 N/sq. mm. (MPa)
Poissons Ratio        = mu3 = 0.35

SUBGRADE.
Modulus of Elasticity = E4 = 72.00 N/sq. mm. (MPa)
Poissons Ratio        = mu4 = 12320775.00

Wheel Load              = W = 87452.000 Newtons.

Tyre Pressure           = p = 0.892 MPa (Newton / Sq.MM.)
```

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DESIGN COMPUTATIONS :

Contact Area = wheel Load / Tyre Pressure = w / p sq.mm.
 Radius of Equiv Circ. Area = $a = \sqrt{\text{area}/3.1416} = 176.66 \text{ mm.}$

$\sigma_0 = p = 0.892 \text{ N/sq.mm (MPa)}$

AT INTERFACE OF LAYERS I & II :

$\text{check} = (h_1/a) * (E_1/E_2)$

Correction Factor for strain,

if check greater than 10,
 $fe = 1.13 - 0.0565 * [\log_e((h_1/a)**2)] / 2.303 * (E_1/E_2)$

if check less than or equals to 10,
 $fe = 0.96 + [0.73 * (a/he) * (E_2/E_1)]$

where, for the first layer, $he = h_1$,

Then, $fe = 1.953$

Correction Factor for stress,
 $fs = 0.99 - 0.07 * (h_1/a)$

$fs = 0.970$

Equivalent thickness of layer 1 with respect to E-value of layer 2,
 $h_{eq_e} = fe * h_1 * ((E_1/E_2)**0.33)$

$h_{eq_e} = 346.774 \text{ mm.}$

$h_{eq_s} = (h_{eq_e}/fe) * fs$
 $= 172.254 \text{ mm.}$

Assign, $z = h_{eq_s} \text{ mm.}$

$F1 = 1 + (1 + (3/(2*(1-\mu_1)))) * (z / a) * (z / a)$
 $= 4.145$

$F2 = (1 + (z / a)**2)**2.5$
 $= 5.315$

The Radius of Curvature,
 $R = E_2 * a / ((1 - \mu_1 * \mu_1) * \sigma_0) * F1 * F2$
 $= 333143.225 \text{ mm.}$

Horizontal Strain,
 $\epsilon_{h_h} = h_1 / (2 * R)$
 $= 7.504e-005 \text{ mm. / mm.}$

Vertical Stress,
 $\sigma_z = \sigma_0 * (1 - 1/(((1 + ((a/z)**2))**1.5)))$
 $= 0.588 \text{ N/sq.mm. (MPa)}$

$F1 = ((1 + ((z/a)**2))**0.5) - (z/a)$
 $= 0.422$

$F2 = 1 / (((1 + ((z/a)**2))**0.5)) + (1 - 2*\mu_1) * F1$
 $= 0.842$

Vertical deflection,
 $dz = ((1 + \mu_1) * \sigma_0 * a * F2) / E_2$
 $= 2.675 \text{ mm.}$

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AT INTERFACE OF LAYERS II & III (For a 4-layer system) :

Correction Factor for strain,

$$f_e = 0.96 + [0.73 * (a/h_e) * (E_3/E_2)]$$

where, for the second layer, $h_e = h_2 + h_{eq_e}$ (of last layer),

Then, $f_e = 4.262$

Correction Factor for stress,

$$f_s = 1.04 - 0.176 * \ln(E_2/E_3)$$

$$f_s = 1.208$$

Equivalent thickness of layer 1 & 2 with respect to E-value of layer 3,

$$h_{eq_e} = f_e * [h_1 * ((E_1/E_3)**0.33) + h_2 * ((E_2/E_3)**0.33)]$$

$$h_{eq_e} = 370.527 \text{ mm.}$$

$$h_{eq_s} = (h_{eq_e}/f_e)*f_s$$

$$= 104.987 \text{ mm.}$$

Assign, $z = h_{eq_s}$ mm.

$$F_1 = (1 + (1 + (3 / (2 * (1 - \mu_2)))) * (z / a))^{**2}$$

$$= 2.316$$

$$F_2 = ((1 + (z / a))^{**2})^{**2.5}$$

$$= 2.130$$

The Radius of Curvature,

$$R = E_3 * a / ((1 - \mu_2 * \mu_2) * \sigma_0) * F_1 * F_2$$

$$= 735212.158 \text{ mm.}$$

Horizontal Strain,

$$\epsilon_{h_h} = (h_1 + h_2) / (2 * R)$$

$$= 3.604e-005 \text{ mm. / mm.}$$

Vertical Stress,

$$\sigma_z = \sigma_0 * (1 - 1 / (((1 + ((a / z)^{**2}))^{**1.5})))$$

$$= 0.773 \text{ N/sq.mm. (MPa)}$$

$$F_1 = ((1 + ((z/a)^{**2}))^{**0.5}) - (z/a)$$

$$= 0.569$$

$$F_2 = 1 / (((1 + ((z / a)^{**2}))^{**0.5})) + (1 - 2 * \mu_2) * F_1$$

$$= 0.917$$

Vertical deflection,

$$dz = ((1 + \mu_2) * \sigma_0 * a * F_2) / E_3$$

$$= 0.349 \text{ mm.}$$

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AT INTERFACE OF LAYERS III & IV :

Correction Factor for strain,

$$f_e = 0.96 + [0.73 * (a/h_e) * (E_4/E_3)]$$

where, for the third layer, $h_e = h_3 + h_{eq_e}$ (of last layer),

Then, $f_e = 0.987$

Correction Factor for stress,

$$f_s = 0.96 - 0.176 * \ln(E_3/E_4)$$

$$f_s = 0.798$$

Equivalent thickness of layer 1 & 2 with respect to E-value of layer 3,

$$h_{eq_e} = f_e * [h_1 * ((E_1/E_4)^{0.33}) + h_2 * ((E_2/E_4)^{0.33}) + h_3 * ((E_3/E_4)^{0.33})]$$

$$h_{eq_e} = 574.258 \text{ mm.}$$

$$h_{eq_s} = (h_{eq_e}/f_e) * f_s$$

$$= 464.215 \text{ mm.}$$

Assign, $z = h_{eq_s}$ mm.

$$F_1 = ((1 + (1 + (3 / (2 * (1 - \mu_3)))))) * (z / a)^2$$

$$= 23.841$$

$$F_2 = \text{pow}((1 + (z / a) * (z / a))^2.5)$$

$$= 175.712$$

The Radius of Curvature,

$$R = E_4 * a / ((1 - \mu_3 * \mu_3) * \sigma_0) * F_1 * F_2$$

$$= 68071901.821 \text{ mm.}$$

Horizontal Strain,

$$\epsilon_{h_h} = (h_1 + h_2 + h_3) / 2 / R$$

$$= 1.858e-006 \text{ mm. / mm.}$$

Vertical Stress,

$$\sigma_z = \sigma_0 * (1 - 1 / (((1 + ((a/z)^2))^{1.5})))$$

$$= 0.164 \text{ N/sq.mm. (MPa)}$$

$$F_1 = ((1 + ((z/a)^2))^{0.5}) - (z/a)$$

$$= 0.184$$

$$F_2 = 1 / (((1 + ((z/a)^2))^{0.5})) + (1 - 2 * \mu_3) * F_1$$

$$= 0.411$$

Vertical deflection,

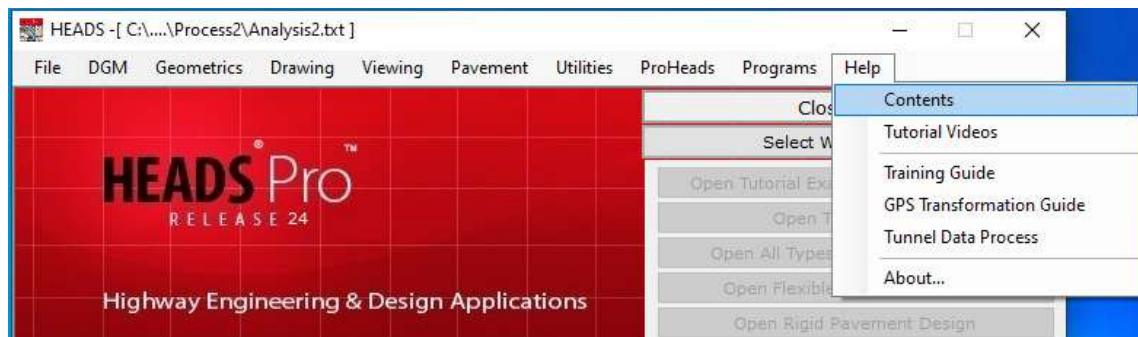
$$dz = ((1 + \mu_3) * \sigma_0 * a * F_2) / E_4$$

$$= 1.214 \text{ mm.}$$

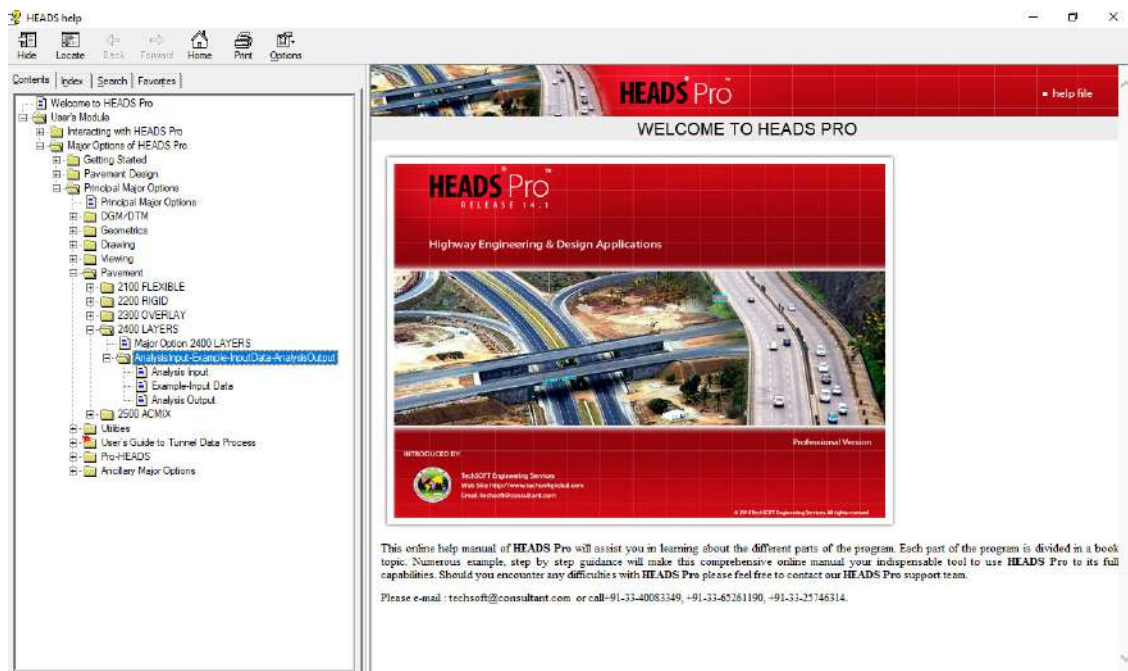
E N D O F A N A L Y S I S .

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6.2.3 For data format/syntax and data explanation on-screen 'Help' may be opened.



6.2.4 The path is shown below to access the details for '2400 LAYERS' to be used for analysis of Layered Pavement System.



This is the end of the analysis of layered pavement system for effect by introducing 'Geo-textiles', for rehabilitation of cracked and distressed pavements.

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The final result is produced as : **Horizontal Strain, Vertical Stress and Vertical Deflection at the interface of each two Layers.** The above results are improved by incorporating Geo-synthetics as Layer II. The Analysis Report is generated with details of all Formulae and Calculation Steps and is highly useful for the users.

**Table 6.2 - Comparative study of results
obtained from Analysis by Software HEADS PRO
(Results of Analysis for Layered Pavement System,
it shows effective for the repairing of Cracked Pavements).**

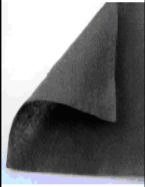

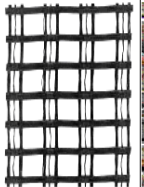
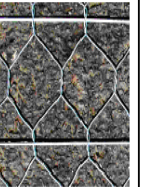
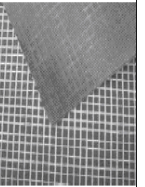
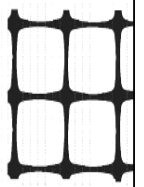
Output	Pavement without Geosynthetics At the interface of layers I and II	Pavement with Geosynthetics At the interface of layers II and III
Horizontal Strain	6.488e-005 mm./mm.	3.604e-005 mm./mm.
Vertical Stress	0.824 N/sq.mm. (MPa)	0.773 N/sq.mm. (MPa)
Vertical Deflection	0.388 mm.	0.349 mm.



Figure - Geogrids Laying is in progress over cracked surface before laying the asphalt wearing course and Geogrids Laid to strengthen the distressed wearing course, to be overlaid by new wearing course.

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Summary of Issues to consider when selecting Reinforcement Products

Issues to Consider	Paving Fabric	Paving Grids			Composite Paving Grids	
	a) Polyester or polypropylene ¹⁾	a) Glass fibre grids ²⁾	b) Polyester grids ³⁾	c) Steel mesh ⁴⁾	a) Stitched or Warp knitted ⁵⁾	b) Bonded ⁶⁾
Photos of Typical Products						
Overlay Stress Absorption	<ul style="list-style-type: none"> i. Act as stress absorbing interlays ii. Prevent ingress of water into pavement layers iii. Bridge shrinkage cracks iv. Provides increased overlay performance by 20 to 40% 	<ul style="list-style-type: none"> i. Modulus ratio of up to 20:1 over asphalt ii. High stiffness redirects crack energy iii. High stiffness resists deformation 	<ul style="list-style-type: none"> i. Increases tensile strength of asphalt layer ii. Reduces tensile peak stress iii. Assists with asphalt fatigue iv. Reduces formation of ruts 	<ul style="list-style-type: none"> i. Reduces peak tensile stress ii. Improves asphalt fatigue iii. Absorbs crack discontinuities iv. Good rut resistance 	<ul style="list-style-type: none"> i. High stiffness redirects crack energy ii. Reduces peak tensile stress iii. Improves asphalt fatigue 	<ul style="list-style-type: none"> i. Increase fatigue life of pavement with weak foundations ii. Used in above application, reduces rutting and controls reflective cracking iii. Susceptible to creep
Overlay Thickness	<ul style="list-style-type: none"> i. Generally 35mm but can be as little as 25mm 	<ul style="list-style-type: none"> i. Minimum overlay thickness of 40mm ii. 25mm overlay thickness achieved under controlled conditions 	<ul style="list-style-type: none"> i. 50mm with paver ii. 40mm manual installation 	<ul style="list-style-type: none"> i. 50mm minimum ii. 60mm unsupervised 	<ul style="list-style-type: none"> i. 40mm Minimum ii. 25mm used successfully in light trafficked areas with low loadings 	<ul style="list-style-type: none"> i. Stiff bi-axial grids used in 70mm overlays ii. Thinner composite polyester grids used in 50mm overlays

Selection of Geogrids

APPENDIX - A

Laboratory Tests and Quality Control (for Pavement Construction Work)

A1 Proctor Compaction Test

Standard Codes of Reference

ASTM D1557 and IS 2720 (Part 2)- 1980 Methods of test for soils: Part 7, Determination of water content- dry density relation using light compaction.

A1.1 General



Figure A1.1 - Proctor Test Moulds and Rammers
(Courtesy: Controls Group)

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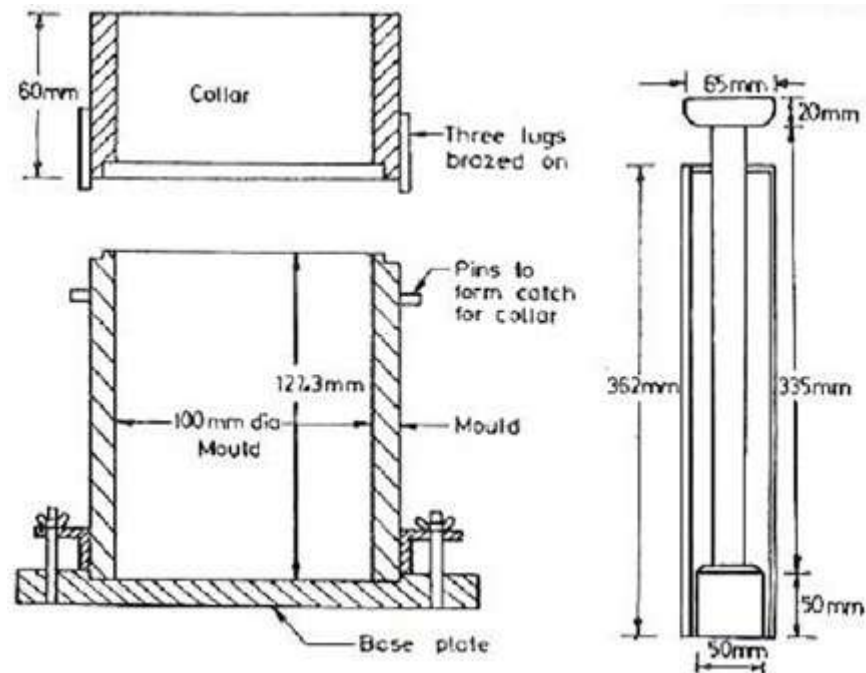


Figure A1.2 - Diagrammatic Representation of Standard Proctor Compaction Equipment.

(COURTESY: RESEARCHGATE)

A1.2 Apparatus Used

- Compaction Mould 1000 ml capacity, diameter 100 mm and height 127.3 mm.
- 6 kg rammer
- Detachable base plate
- Collar 60 mm high
- IS Sieve 4.75 mm
- Oven
- Moisture cans
- Desiccator
- Weighing balance with accuracy of 1g
- Large mixing pan
- Straight edge spatula
- Graduated jars
- Mixing tools, spoons, trowels.
- Steel ruler
- Vernier calipers
- Thermostat

A1.3 Soil Specimen

The soil specimen used must pass through 4.75 mm IS Sieve.

A1.4 Theory

Proctor's compaction test is based on the assessment of water content and dry density relationship of a soil for a specified compacting effort. The mechanical process of densification through reduction of air voids in the soil mass is called compaction. The amount of mechanical energy which is applied to the soil mass is the compacting effort. There are many methods to compact soil in the field, and some examples include tamping, kneading, vibration and static load compaction. This test will employ the tamping or impact compaction method using the type of equipment & methodology developed by R. R. Proctor in 1933, hence, the test is also known as the Proctor test.

Usually, two types of tests are performed:

1. The Standard Proctor test and
2. The Modified Proctor tests.

In the Standard Proctor Test, the soil is compacted by a 2.6 kg rammer falling at a distance of 310 mm into a soil filled mould. The mould is filled with three equal layers of soil and each layer is subjected to 25 blows of rammer. The Modified Proctor Test is identical to the Standard Proctor Test except it employs a 4.89 kg rammer falling at a distance of 450 mm & uses five equal layers of soil instead of three. The bulk density (γ_b) in gms/cc of each compacted specimen shall be calculated from the equation

$$\gamma_b = (m_2 - m_1) / V_m$$

where,

m_1 = mass in gms of mould and base;

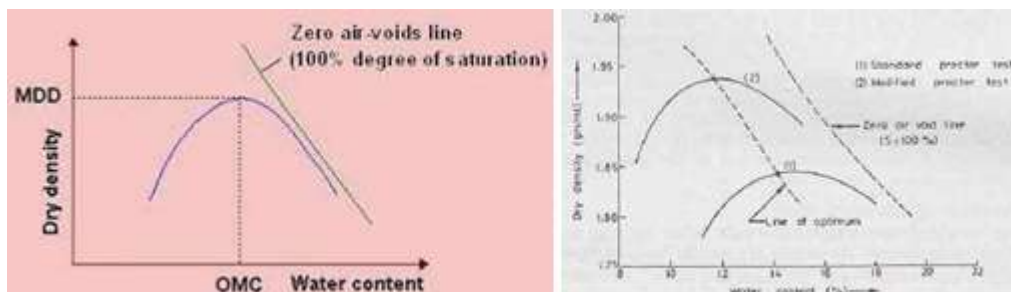
m_2 = mass in gms of mould, base with contained soil and,

V_m = Volume in cc (cm^3) of the mould

The dry density (γ_d) in gms/cc of each compacted specimen shall be calculated from the equation:

$$\gamma_d = (\gamma_b \times 100) / (100 + w)$$

where, w = moisture content of soil in percent.



A1.3 – Standard Plot for Dry Density and Moisture Content

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A1.5 Significance of the Experiment

- Mechanical compaction is one of the most common as well as effective means of stabilizing soils. The analyses of field control tests are done by the geotechnical engineers to ensure that the compacted fills are meeting the prescribed design specifications.
- The maximum density is measured in a laboratory test. The required field density of compacted soil at site must be a percentage of the maximum density, at the water content that are measured in the laboratory test.
- The maximum density for a specified compactive effort is obtained at an optimum water content. When the density is also measured at water contents higher than the optimum water content it results a relatively ductile, less pervious, softer, more susceptible to shrinking & less susceptible to swelling than soil compacted at optimum water content.
- The soil which is compacted leads to a flocculated soil structure that has the opposite characteristics of the soil that is compacted to the same density, at the optimum water content. This helps in construction to a great extent.
- If this property is not determined, the structure built on the soil may fail and later on the soil cannot be stabilized at a reasonable cost.

A1.6 Procedure

Step 1. The mould with base plate is cleaned and dried and weighed to measure to nearest 1 gm.

Step 2. Grease is applied inside the mould along and the base plate and collar.

Step 3. About 16 - 18 kg of air-dried pulverized soil is taken.

Step 4. If the soil is sandy, about 4% and if the soil is clayey, about 8% of water is added to the soil and mixed thoroughly. The soil is kept in an air tight container and allowed to mature for about an hour.

Step 5. About 3 kg of the above matured soil is taken and divided into approximately three equal portions.

Step 6. One portion of the soil is put into the mould and is compacted by applying 25 number of uniformly distributed blows.

Step 7. The top surface of the compacted soil layer is scratched by using a spatula before filling the mould with second layer of soil. The soil is compacted in the similar fashion as done in for the first layer and scratched it.

Step 8. The same procedure is repeated for third layer.

Step 9. The collar is removed and the soil sample is trimmed off the excess soil projecting above the mould by using a straight edge.

Step 10. The mould and also the base plate are cleaned from outside and weighed to the nearest gm.

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Step 11. The soil sample is taken in small aluminum containers from the top, middle and bottom of the sample in the mould, weighed separately and kept in the oven for a day. Next day, the containers are taken from the oven and weighed separately again and the average water content is determined.

Step 12. For a next test, about 3% water is added to a fresh portion of the matured soil and the steps from 5 to 12 are repeated. This step 12 is to be repeated 2/3 times to determine the intended result.

A1.7 Observations and Calculations

Table A1.1 – Sample of Observation and Calculation Table

S. No.	Observations and Calculations	Trial Numbers				
		1	2	3	4	5
1.	Diameter of mould (D) m					
2.	Height of mould (H) cm					
3.	Mass of empty mould and base (in gm)					
4.	Mass of mould, base plate and compacted soil (in gm)					
5.	Moisture content during compaction in %					
6.	Weight of soil (in gm)					
7.	Volume = $V = \pi \times D^2 \times H / 4$ cm ³					
8.	Bulk Density = $\gamma_b = M / V$ (gms/cc)					
9.	Dry Density = $\gamma_d = \gamma_b / (1 + w)$					
10.	Void Ratio = $e = [(G \times \gamma_b) / \gamma_d] - 1$					
11.	Degree of Saturation = $S = w \times G \times 100 / e$					

A1.8 Moisture Content

Table A1.2 – Sample of Moisture Content Observation Table

S. No.	Sample ID.	Weight of empty container in gms.	Weight of container + Wet Soil in gms.	Weight of container + Dry Soil in gms.	Weight of water in gms.	Weight of Dry Soil in gms.	Moisture content in percentage
1.	S-1						
2.	S-2						
3.	S-3						
4.	S-4						
5.	S-5						

Table A1.3 – Example Data from five Tests to be Plotted in Graph to obtain MDD & PMC

Trial	1	2	3	4	5
Dry Density (gms/cc)	1.529	1.590	1.630	1.595	1.533
Moisture (Water) Content (%)	8.07	12.00	17.14	20.95	23.72

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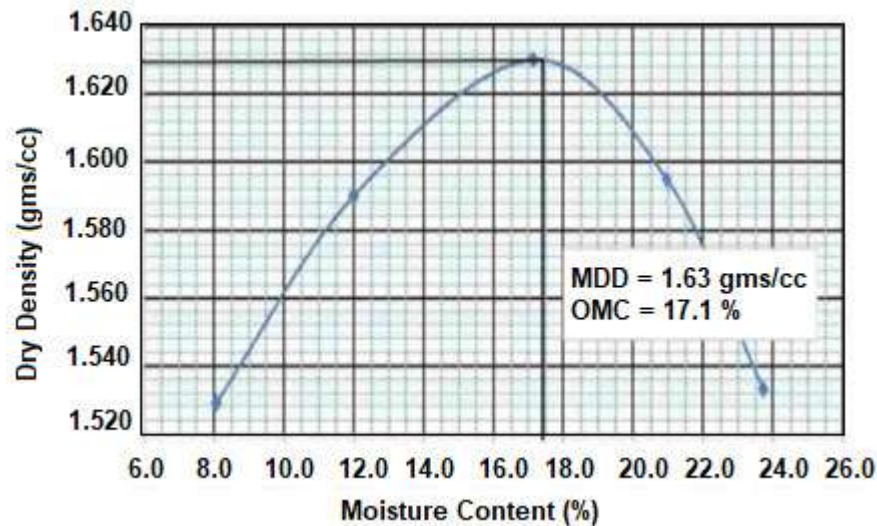


Figure A1.4 – Dry Density and Moisture Contents of five tests are Plotted and MDD and OMC are obtained.

A1.9 Discussion

The maximum dry density in gms/cm^3 is to be determined to the nearest 0.01 and the corresponding moisture content is to be determined to the nearest 0.5, as the optimum.

The dry densities obtained in a series of determinations are plotted against the corresponding moisture content in a graph sheet, and we get a smooth curve with convexity upward. At first the dry unit weight after compaction increases as the moisture content increases but after the optimum moisture content percentage is exceeded, any added water will result in a reduction in dry weight because the pore water pressure will be pushing the soil particles apart, decreasing the friction between them.

A1.10 Remarks

- The soil used in this test passes through 4.75 mm IS Sieve and through No. 4 U.S. Sieve. Aggregates of particles was broken down. After optimum moisture content no more dry density tends to increase.
- Brush should be used so that the particles on the mould do not get added to the weight taking process.
- Grease should be applied properly and completely so that particles don't stick.
- Adequate time should be allowed after mixing the water and before compacting into the mould.
- The blows must be uniformly distributed over the surface of each layer in order to have best results.
- For construction it is often necessary to compact the soil to improve its strength.

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References:

Proctor Compaction Test, Civil Engineering Portal

Web Link: <https://www.engineeringcivil.com/proctor-compaction-test.html>

A2 Field Density Test of Soil by Sand Replacement Method

Standard Codes of Reference

This test refers to: AASHTO T99-86, ASTM D 446-82, BS 1377 (Part 4), IS 2720-28 (1974).

A2.1 General

The sand replacement test method is used to determine in situ dry density of soil. The procedures, materials, equipment, and specifications of this test is based on the Indian Standard (IS 2720 part 28). This test is of significant importance and it has been widely used in various construction project sites.

The field density of natural soil is required for the estimation of soil bearing capacity for the purpose of evaluation of pressures on underlying strata for computation of settlement, and stability analysis of natural slope.

The sand replacement test method is also used to determine the in-place density of compacted soil in order to compare it with the designated compaction degree, hence it specifies how much the compaction of the soil is close to the designated compaction degree.

A2.2 Apparatus

- Sand – pouring cylinder
- Calibrating container, 100mm diameter and 150mm height
- Soil cutting and excavating tools, such as scrapper tool, bent spoon
- Plane surface: Glass or Perspex Plate or Other Plane Surface, 450mm square, 9mm thick or larger
- Metal container to collect excavated soil
- Metal tray, 300mm square and 40mm deep with a hole of 100mm in diameter at the center
- Weighing balance accurate to 1 gram
- Moisture content cans
- Oven
- Desiccator

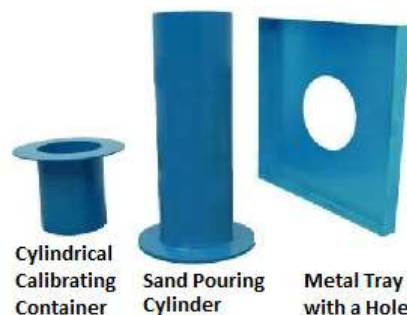


Figure A2.1 – Apparatus for Field Density Test by Sand Replacement Method

A2.3 Materials

Clean, uniform sand passing 1mm sieve and retained on 600 micron sieve in sufficient quantity. It is required to be free from organic substances. The sand should have been dried in an oven and kept in suitable storage for a period of time to allow its water content to reach equilibrium with atmospheric humidity.

A2.4 Calibrations

- Measure the internal dimensions of the calibrating container and then calculate its volume.
- Fill the sand-pouring cylinder with sand, within about 10mm of its top. Determine the weight of the filled cylinder (M_1).
- Place the sand-pouring cylinder vertically on the calibrating container. Open the shutter to allow the sand run out from the cylinder. When there is no further movement of the sand in the cylinder, close the shutter.
- Lift the pouring cylinder from the calibrating container and weigh it to the nearest gram (M_2).
- Place the sand pouring cylinder on the glass plate. Open the shutter and allow the sand to run out of the cylinder until no further movement of the sand is noticed (sand fills the cone of the cylinder), and then close the shutter and remove the sand pouring cylinder carefully.
- Take the sand on the glass plate and determine its weight (M_3)
- Repeat step 3 to step 6 two more times and record mean weight (mean M_2 and M_3)
- Determine the dry density of sand, as shown in Table A2.1.

Table A2.1 – Calibration for Dry Density of Sand

S. No.	Observations and Calculations	Test1	Test2	Test3
1	Volume of calibrating cone V_c			
2	Mass of pouring cylinder (M_1), filled with sand			
3	Mass of pouring cylinder after pouring sand into the calibrating container and cone (M_2)			
4	Mass of sand in the cone (M_3)			
5	Mass of sand in the calibrating container, $M_c = (1) - (2) - (3)$			
6	Dry density of sand, $\rho_s = M_c / V_c$			

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A2.5 Procedure

- Expose an area of about 450mm square on the surface of the soil mass. Trim the surface down to a level surface using a scraper tool.



- Place the metal tray on the leveled surface.



- Excavate the soil through the central hole of the tray, using the hole in the tray as a pattern. The depth of the excavated hole should be about 150mm.



- Collect the excavated soil in a metal container, determine weight of the soil (M).



- Remove the metal tray from the excavated hole. Fill the sand pouring cylinder within 10mm of its top. Determine its mass (M_1). Place the cylinder directly over the excavated hole. Allow the sand to run out the cylinder by opening the shutter. Close the shutter when the hole is completely filled and no further movement of sand is observed.

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- Remove the cylinder from the filled hole. Determine the mass of the cylinder (M_4).
- Take a representative sample of the excavated soil. Determine its water content.



- Determine the dry density of soil as shown in Table 2.2.

Table A2.2 – Observations and Calculations for Field Density Test of Soil by Sand Replacement Method

S. No.	Observations and Calculations	Test1	Test2	Test3
1	Mass of excavated soil (M)			
2	Mass of pouring cylinder (M_1), filled with sand			
3	Mass of pouring cylinder after pouring into the hole and cone (M_4)			
4	Mass of sand in the hole, $M_5 = M_1 - M_4 - M_3$			
5	Volume of sand in the hole $V = M_5 / P_s$			
6	Bulk density = M/V			
7	Water content			
8	Dry density using formula			

A2.6 Result

The dry density of the soil can be calculated using the following formula:

Dry density = Bulk density / (1 + water content)

References: In situ dry density sand replacement method, The Constructor

Web Link: <https://theconstructor.org/practical-guide/in-situ-dry-density-sand-replacement-method/2687/>

A3 Field Density Test of Soil by Core Cutter Method

Standard Codes of Reference

This test is suggested by AASHTO T 191-86, T 310, ASTM D 1556 – 82, D 2216-98, D 4643, D 1238-2010, BS 1377 (Part 4), IS: 2720 (Part XXIX) – 1975.

A3.1 General

Cylindrical core cutters of 130mm long and 100mm diameter are used for testing the in-situ compaction of cohesive and clay soils placed as fill. By using core cutter method, bulk density of soil can be quickly calculated and by determining the moisture content of the soil the dry density of the fill can be calculated and hence the voids percentage. A high percentage of voids indicates poor compaction of soil.

A cylindrical core cutter is a seamless steel tube. For determination of the dry density of the soil, the cutter is pressed into the soil mass so that it is filled with the soil without disturbing the core contents. The cutter filled with the soil is lifted up. The mass of the soil in the cutter is determined. The dry density is obtained as

Bulk density of Soil = $\gamma_b = (M/V)$

Dry density of the soil = $\gamma_d = \gamma_b / (1 + w) = (M/V) / (1 + w)$

Where,

M= mass of the wet soil in the cutter

V= internal volume of the cutter

w= water content.

A3.2 Equipment for Core Cutter Method

- Cylindrical core cutter, 100mm internal diameter and 130mm long
- Steel rammer, mass 9 kg, overall length with the foot and staff about 900mm.
- Steel dolly, 25mm high and 100 mm internal diameter
- Weighing balance, accuracy 1g.
- Palette knife
- Straight edge, steel rule etc.

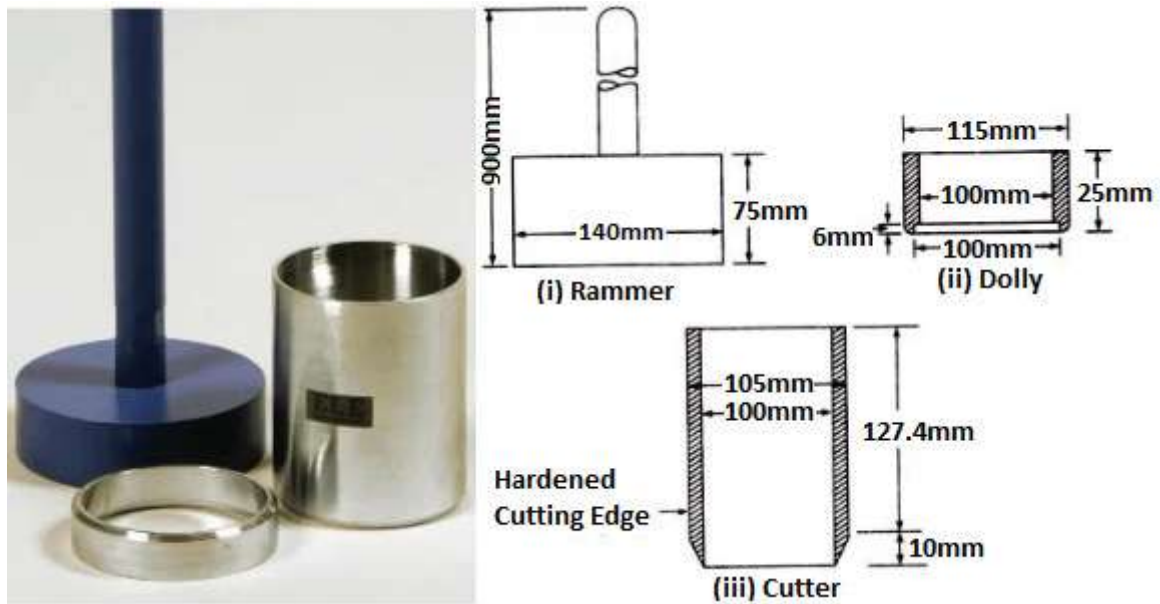


Figure A3.1 – Apparatus for Field Density Test by Core Cutter Method

A3.3 Procedure of Core Cutter Method

- Determine the internal diameter and height of the core cutter to the nearest 0.25mm
- Determine the mass (M1) of the cutter to the nearest gram.
- Expose a small area of the soil to be tested. Level the surface, about 300mm square in area.
- Place the dolly over the top of the core cutter and press the core cutter into the soil mass using the rammer. Stop the pressing when about 15mm of the dolly protrudes above the soil surface.
- Remove the soil surrounding the core cutter, and take out the core cutter. Soil would project from the lower end of the cutter.
- Remove the dolly. Trim the tip and bottom surface of the core cutter carefully using a straight edge.
- Weigh the core cutter filled with the soil to the nearest gram (M2).
- Remove the core of the soil from the cutter. Take a representative sample for the water content determination.
- Determine the water content.

A3.4 Observations and Calculations

Table A3.1 – Observations and Calculations for Field Density Test of Soil by Core Cutter Method

S. No.	Observations and Calculations	Test1	Test2	Test3
1	Core Cutter Nos.			
2	Inside Diameter			
3	Inside height			
4	Mass of Empty Core Cutter (M_1)			
5	Mass of Empty Core Cutter with Soil (M_2)			
6	$M = M_2 - M_1$			
7	Volume of Cutter (V)			
8	Water content			
9	Dry Density by using Formula			

A3.5 Results of Core Cutter Method

Bulk density of Soil = $\gamma_b = (M/V)$

Dry density of the soil = $\gamma_d = \gamma_b / (1 + w) = (M/V) / (1 + w)$

Dry density of the soil = gms/cc.

References: Dry density of soil by core cutter method, The Constructor

Web Link: <https://theconstructor.org/geotechnical/dry-density-of-soil-by-core-cutter-method/2680/>

A4 CBR Test

Standard Codes of Reference

This test refers to: ASTM D1883, EN 13286-47, BS 1377:4, AS 1289.6.1, IS: 2720 (Part IV)–1985.

A4.1 General

To determine the California bearing ratio by conducting a load penetration test in the laboratory.

The California bearing ratio test is a penetration test primarily meant for the evaluation of subgrade strength of roads and pavements. The results obtained by these tests are used with the empirical curves to determine the thickness of pavement and its component layers. This is the most widely used method for the design of flexible pavement.

This instruction sheet covers the laboratory method for the determination of C.B.R. of undisturbed and re-moulded /compacted soil specimens, both in soaked as well as un-soaked state.

A4.2 Definition of CBR

It is the ratio of force per unit area required to penetrate a soil mass with standard circular piston at the rate of 1.25 mm/min. to that required for the corresponding penetration of a standard material.

$$\text{C.B.R.} = (\text{Test load} / \text{Standard load}) \times 100$$

The table below gives the standard loads adopted for different penetrations for the standard material with a C.B.R. value of 100%. The test may be performed on undisturbed specimens and on re-moulded specimens which may be compacted either statically or dynamically.

Table A4.1 – Standard Loads Adopted for Different Penetrations for Standard Material with CBR value of 100%

Penetration of Plunger (mm)	Standard Load (kg)
2.5	1370
5.0	2055
7.5	2630
10.0	3180
12.5	3600

A4.3 Equipment and tool required

Equipment and tool required.

1. Cylindrical mould with inside diameter 150 mm and height 175 mm, provided with a detachable extension collar 50 mm height and a detachable perforated base plate 10 mm thick.

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2. Spacer disc 148 mm in dia and 47.7 mm in height along with handle.
3. Metal rammers. Weight 2.6 kg with a drop of 310 mm (or) weight 4.89 kg a drop 450 mm.
4. Weights. One annular metal weight and several slotted weights weighing 2.5 kg each, 147 mm in dia, with a central hole 53 mm in diameter.
5. Loading machine. With a capacity of atleast 5000 kg and equipped with a movable head or base that travels at an uniform rate of 1.25 mm/min. Complete with load indicating device.
6. Metal penetration piston 50 mm dia and minimum of 100 mm in length.
7. Two dial gauges reading to 0.01 mm.
8. Sieves. 4.75 mm and 20 mm I.S. Sieves.
9. Miscellaneous apparatus, such as a mixing bowl, straight edge, scales soaking tank or pan, drying oven, filter paper and containers.



Figure A4.1 – Apparatus for CBR Test.



Figure A4.2 – Samples are Soaked in Water for 4 Days for CBR Test.

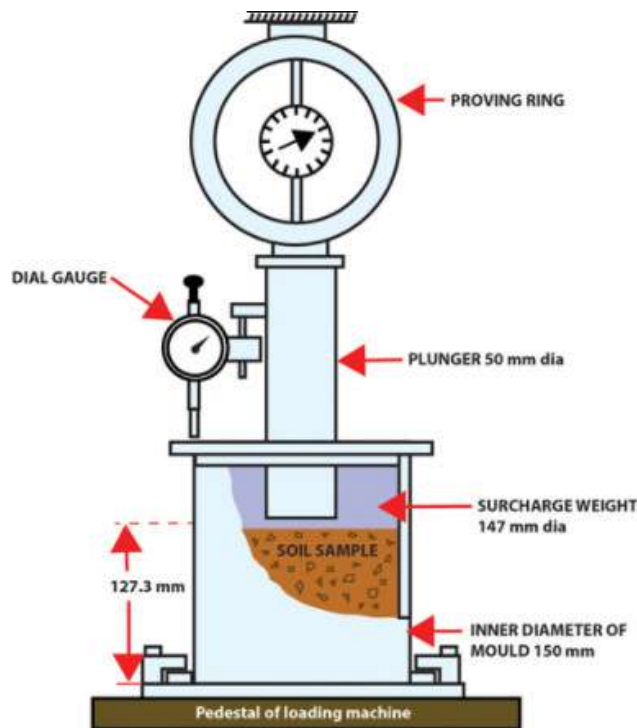


Figure A4.3 - Diagrammatic Representation of CBR Test Setup

A4.4 Preparation of Test Specimen

Undisturbed specimen

Attach the cutting edge to the mould and push it gently into the ground. Remove the soil from the outside of the mould which is pushed in. When the mould is full of soil, remove it from weighing the soil with the mould or by any field method near the spot.

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A4.5 Determine the density

Re-moulded specimen

Prepare the re-moulded specimen at Proctor's maximum dry density or any other density at which C.B.R. is required. Maintain the specimen at optimum moisture content or the field moisture as required. The material used should pass 20 mm **I.S.** sieve but it should be retained on 4.75 mm **I.S.** sieve. Prepare the specimen either by dynamic compaction or by static compaction.

Dynamic Compaction

Take about 4.5 to 5.5 kg of soil and mix thoroughly with the required water.

Fix the extension collar and the base plate to the mould. Insert the spacer disc over the base (**See Fig.38**). Place the filter paper on the top of the spacer disc.

Compact the mix soil in the mould using either light compaction or heavy compaction. For light compaction, compact the soil in 3 equal layers, each layer being given 55 blows by the 2.6 kg rammer. For heavy compaction compact the soil in 5 layers, 56 blows to each layer by the 4.89 kg rammer.

Remove the collar and trim off soil.

Turn the mould upside down and remove the base plate and the displacer disc.

Weigh the mould with compacted soil and determine the bulk density and dry density.

Put filter paper on the top of the compacted soil (collar side) and clamp the perforated base plate on to it.

Static compaction

Calculate the weight of the wet soil at the required water content to give the desired density when occupying the standard specimen volume in the mould from the expression.

$$W = \text{desired dry density} \times (1+w) \times V$$

Where

W = Weight of the wet soil

w = desired water content

V = volume of the specimen in the mould = 2250 cm³ (as per the mould available in laboratory)

Take the weight W (calculated as above) of the mix soil and place it in the mould.

Place a filter paper and the displacer disc on the top of soil.

Keep the mould assembly in static loading frame and compact by pressing the displacer disc till the level of disc reaches the top of the mould.

Keep the load for some time and then release the load. Remove the displacer disc.

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The test may be conducted for both soaked as well as unsoaked conditions.

If the sample is to be soaked, in both cases of compaction, put a filter paper on the top of the soil and place the adjustable stem and perforated plate on the top of filter paper.

Put annular weights to produce a surcharge equal to weight of base material and pavement expected in actual construction. Each 2.5 kg weight is equivalent to 7 cm construction. A minimum of two weights should be put.

Immerse the mould assembly and weights in a tank of water and soak it for 96 hours. Remove the mould from tank.

Note the consolidation of the specimen.

A4.6 Procedure for Penetration Test

Place the mould assembly with the surcharge weights on the penetration test machine. (Fig.39).

Seat the penetration piston at the center of the specimen with the smallest possible load, but in no case in excess of 4 kg so that full contact of the piston on the sample is established.

Set the stress and strain dial gauge to read zero. Apply the load on the piston so that the penetration rate is about 1.25 mm/min.

Record the load readings at penetrations of 0.5, 1.0, 1.5, 2.0, 2.5, 3.0, 4.0, 5.0, 7.5, 10 and 12.5 mm. Note the maximum load and corresponding penetration if it occurs for a penetration less than 12.5 mm.

Detach the mould from the loading equipment. Take about 20 to 50 g of soil from the top 3 cm layer and determine the moisture content.

A4.7 Observation and Recording

For Dynamic Compaction

Optimum water content (%)
Weight of mould + compacted specimen g
Weight of empty mould g
Weight of compacted specimen g
Volume of specimen cm³
Bulk density g/cc
Dry density g/cc

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For static compaction

Dry density g/cc
Moulding water content %
Wet weight of the compacted soil, (W)g
Period of soaking 96 hrs. (4days).

For penetration Test

Calibration factor of the proving ring	1 Div. = 1.176 kg
Surcharge weight used (kg)	2.0 kg per 6 cm construction
Water content after penetration test %	
Least count of penetration dial	1 Div. = 0.01 mm

If the initial portion of the curve is concave upwards, apply correction by drawing a tangent to the curve at the point of greatest slope and shift the origin (Fig. 40). Find and record the correct load reading corresponding to each penetration.

$$\text{C.B.R.} = (\text{PT} / \text{PS}) \times 100$$

Where,

PT = Corrected test load corresponding to the chosen penetration from the load penetration curve.

PS = Standard load for the same penetration taken from the Table A4.I.

Table A4.2 – Sample Observation Table for CBR Test

Penetration Dial	Load Dial	Corrected Load
Readings Penetration (mm)	Proving Ring Load Reading (Kg)	
0.5		
1.0		
1.5		
2.0		
2.5		
4		
5		
7.5		
10.0		
12.5		

A4.8 Interpretation and recording

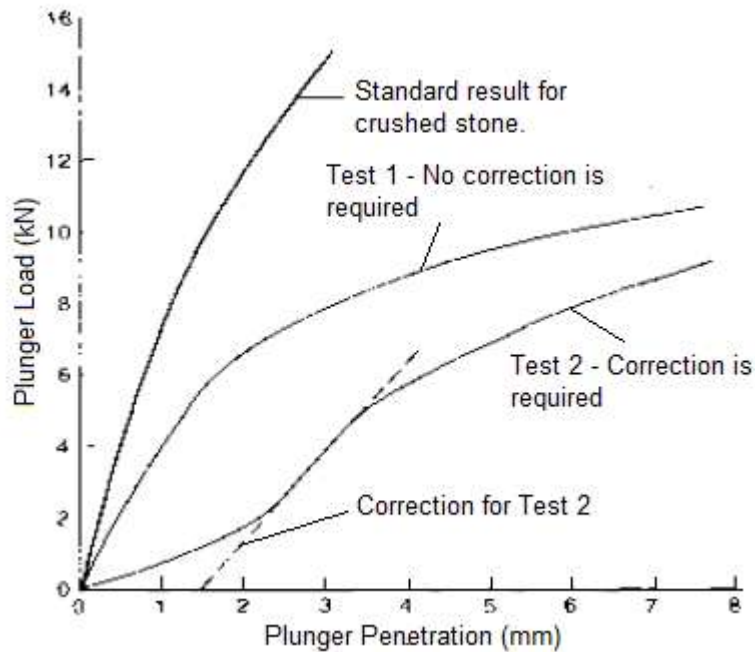


Figure A4.4 – Load versus Penetration Curve

C.B.R. of specimen at 2.5 mm penetration
C.B.R. of specimen at 5.0 mm penetration
C.B.R. of specimen at 2.5 mm penetration

The C.B.R. values are usually calculated for penetration of 2.5 mm and 5 mm. Generally, the C.B.R. value at 2.5 mm will be greater than that at 5 mm and in such a case the former shall be taken as C.B.R. for design purpose. If C.B.R. for 5 mm exceeds that for 2.5 mm, the test should be repeated. If identical results follow, the C.B.R. corresponding to 5 mm penetration should be taken for design.

References: California Bearing Ratio test, Civil Concept

Web Links: <https://www.civilconcept.com/california-bearing-ratio-test/>
<http://home.iitk.ac.in/~madhav/expt14.html>

A5 Atterberg Limits Test (for Liquid Limit, Plastic Limit and Plasticity Index)

Standard Codes of Reference

This test is suggested by AASHTO T 89, T 90, ASTM D 4318 (Part-I, II, II), D 2216, IS: 2720 (Part IV) – 1985.

A5.1 General

A5.1.1 Description of Test

This method describes the procedure for determining the plasticity index of fine-grained soils. This includes all soils having more than 50% of its particles passing a No. 400 mm Canadian Metric sieve.

A5.1.2 Units of Measure

The plasticity index of a soil is the numerical difference between its liquid and plastic limits. The lowest water content determined by the following procedure at which the soil remains plastic is the plastic limit. The water content determined by the following procedure at which the soil passes from a plastic state to a liquid state is the liquid limit.

A5.2 Apparatus and Materials

Pulverizing apparatus - a mortar and rubber covered pestle. Drying apparatus - oven or stove suitable for drying samples. Sieve - a 400 mm sieve with pan. Evaporating dish - a porcelain dish about 100 mm in diameter. Spatula - a spatula having a blade approximately 75 mm in length and 19 mm in width. Liquid limit device - a mechanical device consisting of a brass cup and carriage, constructed as shown in Figure 205-1. Grooving tool - a combination grooving tool and gauge as shown in Figure 205-1.

Surface for Rolling - a ground glass plate or piece of smooth unglazed paper used for rolling plastic limit determinations. Containers - suitable containers such as aluminum tares with covers to prevent the loss of moisture during the weighing of the sample. Balance - a balance sensitive to 0.02 g.

A5.3 Sample Preparation

Select a representative air dried soil sample to yield approximately 115 grams of material passing the No. 400 mm sieve. Separate the soil into two parts using a No. 400 mm sieve. Grind the retained fraction using the mortar and pestle to break up soil aggregations without fracturing the individual grains and re-sieve. Regrind the material retained until the grindings produce only a small quantity of material passing the sieve. Discard the material retained on the sieve. Recombine the portions passing the No. 400 mm sieve and mix thoroughly before determination of the liquid limit and plastic limit.

A5.4 Test Procedure

A5.4.1 Liquid Limit Determination

Adjust the liquid limit device by means of the adjustment plate on the device and the gauge on the handle of the grooving tool. Adjust so the centre of the wear point on bottom of cup is lifted exactly 1 cm above the base. Secure adjustment plate by tightening screws. Place approximately 100 grams of prepared soil in the evaporating dish and add 15 to 20 ml of water and mix water thoroughly with the soil. Then add sufficient water and thoroughly mix to form a uniform mass of stiff consistency. Place mixture in the cup over the spot where the cup rests on the base and spread into place with as few strokes as possible to a depth of 10 mm.

Take care to prevent the entrapment of air bubbles within the mass. Divide the soil in the cup with a firm stroke of the grooving tool along the diameter through the centre line of the cam follower. Sand soils may require several strokes of the grooving tool to make a satisfactory groove without tearing the soil. For these soils, increase the depth of the groove gradually with each stroke and only the last stroke should scrape the bottom of the cup. Turn the crank to raise and drop the cup twice per second until the two sides of the grooved sample come in contact at the bottom of the groove for a distance of 12.5 mm. Record the number of blows. If the number of blows is between 15 and 40, take a representative sample of the soil in the cup for moisture content. If the number of blows is outside the range of 15 to 40, transfer the soil from the cup to the evaporating dish. Adjust the moisture content by mixing, with or without the addition of water and repeat the test until the blows fall within the range of 15-40 blows. Clean the cup of the liquid limit device after each trial test. Determine the moisture content by placing the sample in a tared aluminum can and weighing. Repeat the test and obtain a second moisture sample. Oven dry the samples at 110o C to a constant weight and weigh after cooling.

A5.4.2 Plastic Limit Determination

Place approximately 20 grams of the prepared soil in evaporating dish and thoroughly mix with water until the mass becomes plastic enough to be shaped into a ball. Take approximately 8 to 12 grams of the moistened soil and form into a uniform mass roughly elliptical in shape.

Roll the ball of soil by hand on the rolling surface with just enough pressure to form an elongated thread as rolling proceeds. If the soil can be rolled to a thread 3 mm thick without crumbling, amass it and re-roll it. Repeat rolling and amassing until the soil crumbles under slight pressure required for rolling. Crumbling may occur when the soil has a diameter greater than 3 mm, however, this is considered a satisfactory end point, provided the soil has been previously rolled into a 3 mm thread. Gather portions of the crumbled soil and place in a tared aluminum can and weigh, then oven dry at 110o C to a constant weight and weigh again after cooling.

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A5.5 Results and Calculations

A5.5.1 Calculations for Liquid Limit

Record the numerical difference between the wet and dry weight as weight of moisture. The "weight of moisture" divided by the "dry weight of sample" and multiplied by one hundred is the percent moisture. The moisture content and corresponding number of blows for the two liquid limit determinations is used to calculate the liquid limit (w_L) at 25 blows. Adjust the moisture content of the sample by dividing by a denominator selected from the following chart:

Table A5.1 - Denominator of Formula for Liquid Limit

Nos. of Blows	Denominator of Formula	Nos. of Blows	Denominator of Formula
15	1.066	28	0.985
16	1.0592	29	0.980
17	1.050	30	0.976
18	1.043	31	0.972
19	1.036	32	0.968
20	1.029	33	0.964
21	1.023	34	0.960
22	1.017	35	0.956
23	1.011	36	0.952
24	1.005	37	0.948
25	1.000	38	0.945
26	0.995	39	0.942
27	0.990	40	0.939

Calculate the liquid limit for each test and average the results as shown in the following example.

Test	1	2
Blows	19	31
Moisture	33.79	31.50
W _L (25 blows)	33.79 / 1.037 = 32.6	31.50 / 0.972 = 32.4

The duplicate liquid limit results should be repeated if they do not fall within the limits shown below.

W _L 30	- repeatability within 1.0%
W _L 30-60	- repeatability within 2.0%
W _L 60	- repeatability within 3.0%

A5.5.2 Calculations for Plastic Limit

Record the difference between the wet and dry weights as the weight of moisture. Calculate the plastic limit W_p by dividing the "weight of moisture" by the "dry weight of sample" and multiply by 100.

$$\text{Plastic Limit (W}_p\text{)} = (\text{wt. of moist.} \times 100) / \text{wt. of oven dry soil}$$

A5.5.3 Calculations for Plasticity Index

Plasticity Index (I_p) of a soil is the numerical difference between its Liquid Limit and its Plastic Limit.

$$I_p = W_L - W_p$$

A5.5.4 Reporting Results

Report the liquid limit and plasticity index on Form MR-20 and MR-85.

Report as non-plastic if the liquid limit or plastic limit cannot be determined.

Report as non-plastic if the plastic limit is equal to or greater than the liquid limit.

A5.6 Added Information

References **ASTM D423 and D424**

It is important to take the correct amount of soil for the plastic limit test. The usual error is to take too large a sample which may give an incorrect answer.

For the liquid limit test, it is important to thoroughly mix soil and water. Avoid adding dry soil to a mixture which is too wet; dry it by stirring instead.

When making liquid limit tests for reference purposes, the following will apply:

Mixing of soil with water: 5 to 20 minutes, the longer period used for the more plastic soils.

Seasoning in the humidifier for 25 hours.

Add 1 ml of water and mix for 1 minute and remix until the soil appears to have such a consistency that the groove will close in 25 to 35 blows.

Place in the brass cup, testing and weighing within 3 minutes.

Washing cup and grooving tool, adding water and remixing within 3 minutes.

Repeat in triplicate and in no case shall dried soil be added to the seasonal soil being tested.

Do not record any trials requiring less than 15 drops or more than 35 drops of the cup.

For important research projects, the soil used for liquid and plastic limits should not be dried prior to preparation for testing; drying may alter soil by causing the particles to subdivide or agglomerate by driving off absorbed water which is not completely regained on rewetting. Also oven drying of organic soils may cause chemical changes which significantly lower the liquid limit values.

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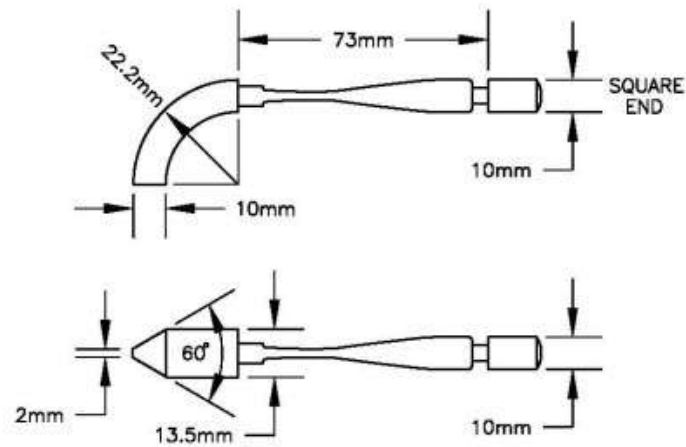


Figure A5.1 - A Combination Grooving Tool and Gauge

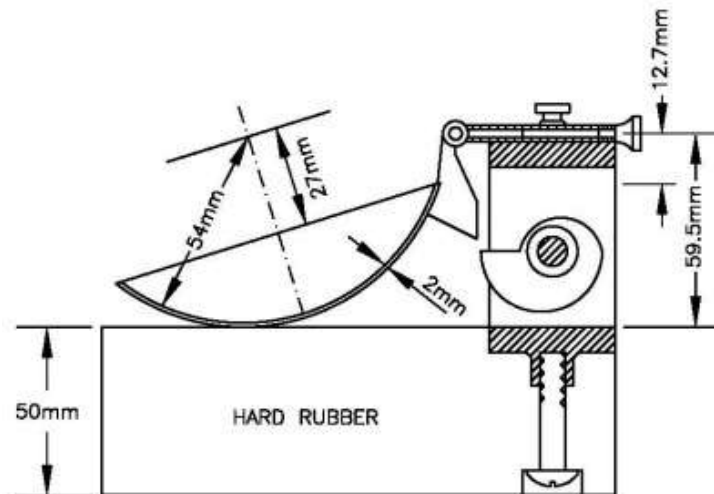


Figure A5.2 - Device Consisting of a Brass Cup and Carriage

References:

Atterberg Plasticity Index, Saskatchewan Highways and Transportation

Web Link: [http://highways.gov.sk.ca/205-](http://highways.gov.sk.ca/205-1/#:~:text=Record%20the%20difference%20between%20the.sample%22%20and%20multiply%20by%20100.&text=Plasticity%20Index%20(lp)%20of%20a.Limit%20and%20its%20Plastic%20Limit)

[1/#:~:text=Record%20the%20difference%20between%20the.sample%22%20and%20multiply%20by%20100.&text=Plasticity%20Index%20\(lp\)%20of%20a.Limit%20and%20its%20Plastic%20Limit](http://highways.gov.sk.ca/205-1/#:~:text=Record%20the%20difference%20between%20the.sample%22%20and%20multiply%20by%20100.&text=Plasticity%20Index%20(lp)%20of%20a.Limit%20and%20its%20Plastic%20Limit)

A6 Grain Size Analysis Test

Standard Codes of Reference

This test is suggested by AASHTO T 88, ASTM D 422, IS: 2720 (Part IV) – 1985.

A6.1 General

Grain size analysis is a typical laboratory test conducted in the soil mechanics field. The purpose of the analysis is to derive the particle size distribution of soils.

The analysis is conducted via two techniques. **Sieve Grain Size Analysis** is capable of determining the particles' size ranging from 0.075 mm to 100 mm. Any categorization of grains larger than 100mm will be conducted visually whereas particles smaller than 0.075 mm can be distributed using the **Hydrometer Method**.

A6.2 Grain Size Analysis by Sieves

The test is carried out with the utilization of a set of sieves with different mesh sizes. Each sieve has squared shaped openings of a certain size. The sieve separates larger from smaller particles, distributing the soil sample in 2 quantities. The grains with diameters larger than the size of the openings are retained by the sieve, while smaller diameter grains pass through the sieve. The test is conducted by placing a series of sieves with progressively smaller mesh sizes on top of each other and passing the soil sample through the stacked sieve "tower". Therefore, the soil particles are distributed as they are retained by the different sieves. A pan is also used to collect those particles that pass through the last sieve (No. 200).

The nomenclature of the sieves typically used for Grain Size Analysis of soils as well as the corresponding opening sizes are presented in **Table A6.1**. Based on the range of the particle sizes, and the **Unified Soil Classification System (USCS)**, soils can be classified in the generic categories presented in **Table A6.2**. Further categorizations are possible upon further analysis of the Grain Size Distribution results.

Table A6.1 - The Sieves Typically Utilized in the Grain Size Analysis Test

Sieve #	Opening Diameter (mm)
4	4.75
10	2.00
20	0.85
40	0.425
60	0.25
100	0.15
140	0.105
200	0.075

Table A6.2 - The Soil Classification Based on Particle Size Range (USCS)

Soil Type	Particle Size (mm)
-----------	--------------------

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Clay		< 0.002
Silt		0.002-0.075
Sand	Fine	0.075-0.42
	Medium	0.42-2.0
	Coarse	2.0-4.75
Gravel		4.75-75

A6.3 Test Set-up Components

A typical Sieve Analysis test set-up is composed of:

- Drying oven maintained at $110 \pm 5^\circ\text{C}$
- Standard sieves
- Sample splitter
- Mechanical sieve shaker
- Pans

A typical set-up of stacked sieves placed on a mechanical sieve shaker is shown in **Figure 1**.



Figure A6.1 – Stacked Sieves Placed on a Mechanical Sieve Shaker

A6.4 Step-by-Step Sieve Analysis Test Procedure

The typical testing procedure consists of the following steps:

1. Weigh a dry soil sample which should be at least 500gr.
2. Record the weight of the sieves and the pan that will be utilized during the analysis. Each sieve should be thoroughly cleaned up before the test.
3. Assemble the sieves in ascending order, placing those with the larger openings on top. Therefore, the No. 4 sieve should be on top and the No. 200 sieve on the bottom of the stack.
4. Place the soil sample into the top sieve and place a cap/lid over it.
5. Place the stack in a mechanical shaker and shake for 10 minutes
6. Remove the sieve stack from the shaker and measure the weight of each sieve and that of the pan placed at the bottom of the stack.

A6.5 Data Manipulation

The weight of the soil retained on each sieve is calculated by subtracting the weight of the empty sieve from the recorded weight of the sieve after the test. The total weights of particles retained are added and compared to the initial weight of the soil sample. A difference lower than 2% is required.

The percentage retained on each sieve is determined by dividing each weight retained by the initial weight of the soil sample. Subsequently, the total percentage passing from each sieve is calculated by subtracting the cumulative percentage retained in that particular sieve and the ones above it from totality.

A typical Grain Size Analysis data sheet is presented below (Table A6.3). Moreover, a typical grain size distribution curve of a medium sand is shown in Figure A6.2.

Table A6.3 – Typical Grain Size Analysis Data Sheet

Sieve #	Opening Diameter (mm)	Weight of Empty Sieve (g)	Weight of Sieve + Soil Retained (g)	Soil Retained (g)	Percent Retained (%)	Cumulative retained (%)	Percent Passing (%)
		A	B	C	D	E	F
				$C_i = B_i - A_i$	$D_i = \frac{C_i}{Total\ Weight} \times 100$	$E_i = D_i + E_{i-1}$	$F_i = 100 - E_i$
4	4,75						
10	2						
20	0,85						
40	0,425						
60	0,25						
100	0,15						
140	0,105						
200	0,075						
Pan							
			Total Weight =				

A6.6 The Uniformity Coefficient (Cu)

The uniformity coefficient (C_u) expresses the variety in particle sizes of soil and is defined as the ratio of D_{60} to D_{10} (**Figure A6.1**). The value D_{60} is the grain diameter at which 60% of soil particles are finer and 40% of soil particles are coarser, while D_{10} is the grain diameter at which 10% of particles are finer and 90% of the particles are coarser. Therefore, C_u is estimated as:

$$C_u = \frac{D_{60}}{D_{10}}$$

When C_u is greater than 4, the soil is classified as well graded, whereas when C_u is less than 4 the soil is classified as poorly graded/uniformly graded.

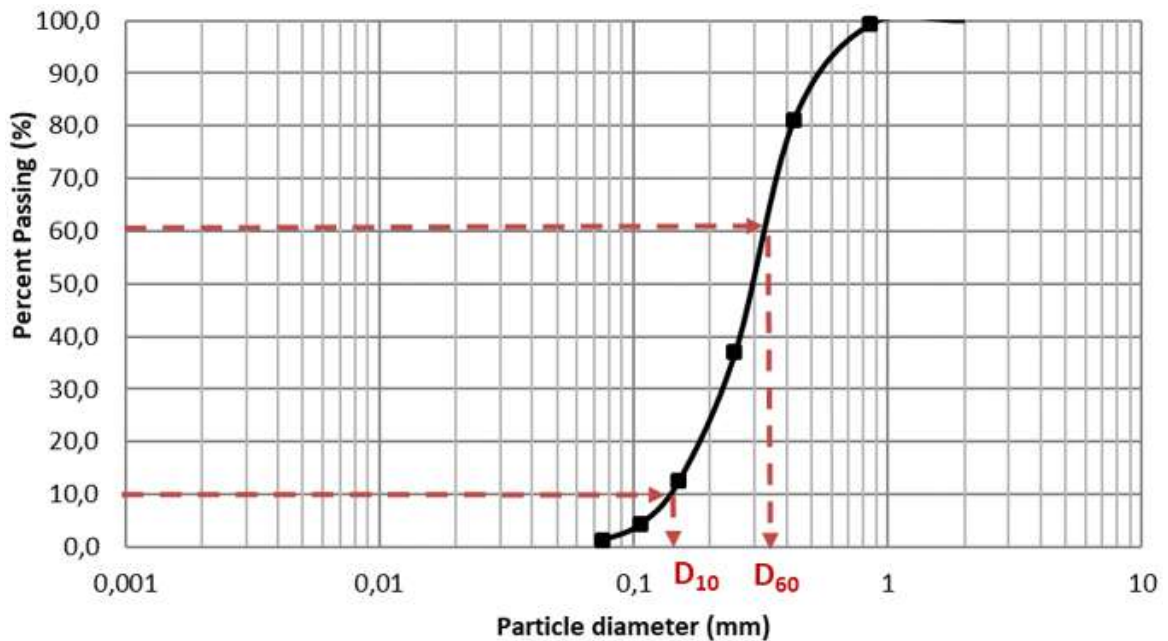


Figure A6.2 – Grain Size Distribution Curve of a Medium-Fine Sand

A6.7 Grain Size Analysis by Hydrometer

The hydrometer analysis is utilized for particle sizes finer than 75 μm . These particles pass through the last sieve (No. 200) of the Sieve Analysis.

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A hydrometer is a device designed to measure the relative density of a liquid which refers to the ratio of the actual density of the substance to the density of the water. The apparatus consists of a cylindrical stem and a bulb that contains a specific portion of mercury or lead at the bottom, calibrated to float upright in the liquid. The liquid is poured in a tall cylinder usually made out of glass and the hydrometer is placed inside until it is stabilized. The test is based on the principle that in a low-density liquid, the hydrometer will sink deeper until it balances.

The hydrometer contains a scale which is used to record the relative density of the liquid based on its submersion.

The hydrometer grain size analysis takes advantage of the change in the relative density of a soil-water mixture as the soil particles sink. The test relies on the fact that when the soil is poured in the liquid, the relative density of the soil-water mixture will rise. As the soil particles sink the density decreases until it reaches the initial density of the liquid. The heaviest particles (larger in diameter) will sink first.

A6.8 Test Set-up Components

A typical Hydrometer test set-up, shown in **Figure 3**, is composed of:

- Hydrometer device
- Drying oven maintained at $110 \pm 5^\circ\text{C}$
- Stirring apparatus
- 2 glass containers, each of 1000 ml volume
- Mercury thermometer ranging from $0\text{--}104^\circ\text{C}$
- Dispersing agent
- Desiccator
- Stopwatch
- Distilled water



Figure A6.3 – Hydrometer Test set-up

A6.9 Step-by-Step Hydrometer Test Procedure

The typical testing procedure consists of the following steps:

1. Sieve enough soil by hand through the #40 sieve
2. Dry soil at $110 \pm 5^\circ\text{C}$ overnight
3. Utilize a desiccator to place the sample and allow it to cool.

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4. Record the dry weigh of the soil (typically, 50 gr)
5. Place 500-600 ml of distilled water in a steel mixing cup.
6. Add 5gr of sodium-hexa-meta-phosphate solution and utilize a high-speed mixer to disperse it (~3 min.).
7. Add the soil to the mixture and mix for 5-6 minutes.
8. Clean the blade as no material should be lost.
9. Place the mixture in a 1-liter cylindrical container and fill it with distilled water.
10. Place a rubber cap on top of the cylinder and turn the container upside down multiple times.
11. Right after shaking, place the container on top of a table and start measuring time.
12. Slowly insert the hydrometer device into the container and take readings at 10, 20, 40, 60 and 120 seconds, respectively. The measurement should be taken at the top of the formed meniscus. Utilize a thermometer to measure the temperature.
13. Right after the 2 minutes reading, remove the hydrometer and place it into another container with distilled water.
14. Carefully insert the hydrometer and take subsequent measurements at 4, 6, 8, 15, 30, 60 and 90 minutes. Between readings, place the rubber cap on top of the container.

A6.10 Hydrometer Grain Size Analysis Calculations

If the temperature throughout the hydrometer test remains constant, the Stoke's Law can be utilized to derive the diameter of the particles.

The formula of Stoke's Law is presented below:

$$D = \left(\frac{18 * \mu}{\gamma_s - \gamma_f} \right)^{0.5} \cdot v^{0.5}$$

where:

D: The maximum diameter of soil particles corresponding to the percentages indicated by a single hydrometer test reading.

μ : The fluid's viscosity

v : the terminal velocity of settlement

γ_s : The unit weight of soil particles

γ_f : The unit weight of the fluid

Because the fluid's viscosity, the unit weight of soil particles and the unit weight of the fluid depend only on the temperature and the specific gravity of the soil particles, G_s (typical value $\sim G_s=2.70$), the first term of the equation is substituted by a constant known as *Sedimentation constant K*.

Therefore, Stoke's Law is simplified as following:

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$$D = K \cdot v^{0.5}$$

The terminal velocity of the particles (v , in cm/min) is calculated by dividing the sedimentation depth L (the distance from the surface of the suspension to the center of volume of the hydrometer, in cm) by time (t , in min).

Therefore, Stoke's Law is re-written as (D in mm):

$$D = K \cdot \left(\frac{L}{t}\right)^{0.5}$$

For a given hydrometer and cylindrical container, L values vary according to the hydrometer readings:

$$L = 16.3 - 0.1644 \cdot R$$

Where R is the hydrometer reading in grams/liter.

The Stoke's law calculates the larger possibly diameter of the particles that are in suspension.

To derive the particles' percentage passing for each reading stage the following equation is utilized:

$$PF(\%) = \frac{a \cdot (R - b + 1)}{W}$$

where:

- a : correction factor for particle density
- W : weight of the original dry soil (typically, 50 gr)
- b : correction factor associated with temperature and 1 is added to eliminate the meniscus effect.

Finally, the cumulative particle percentage passing is plotted versus the maximum Diameter of the soil particles on a semi-logarithmic scale.

A6.11 Stoke's Law Assumptions

The assumptions that are made using Stoke's Law in the hydrometer test are the following:

1. The particles are much larger than the molecules of water.
2. The particles are represented by smooth and rigid spheres with the same specific gravity.
3. The particles settle individually and they are not affected by collisions with other particles.
4. At time zero, the particles are at rest but instantly accelerate to their terminal settlement velocity

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Reference:

Grain Size Analysis, The International Information center for Geotechnical Engineer

Web Link: <https://www.geoengineer.org/education/laboratory-testing/step-by-step-guide-for-grain-size-analysis#:~:text=Grain%20size%20analysis%20is%20a,0.075%20mm%20to%20100%20m>.

A7 Los Angeles Abrasion Test

Standard Codes of Reference

This test is suggested by AASHTO T 96, ASTM C 131, IS: 2386 (Part IV) – 1963.

A7.1 General

Abrasion Test is the measure of aggregate toughness and abrasion resistance such as crushing, degradation and disintegration.

The aggregates are used for the surface course of the highway pavements and they are subjected to wearing due to movement of traffic.

When vehicles travel on the road, the soil particles present between the pneumatic tyres of vehicle and road surface creates the abrasion effect on aggregates. The steel plate wheels of animal-driven vehicles also cause significant abrasion of the road surface.

Therefore, the aggregates used in road construction must be hard enough to resist abrasion. The resistance offered by aggregate to abrasion is determined in the laboratory by the Los Angeles test machine.

The Working principle of the Los Angeles abrasion test is to produce abrasive action by use of standard steel balls, which when mixed with aggregates and rotated in a drum for some specified time for a specific number of revolutions also causes an impact on aggregates.

The percentage wear of the sample aggregates due to rubbing with steel balls is determined and is known as Los Angeles Abrasion Value.

The Los Angeles abrasion test on aggregates is done for the following purposes:

- To find out the Los Angeles abrasion value of aggregate.
- To find the suitability of aggregates for use in road construction projects.

A7.2 Apparatus used for Los Angeles Abrasion Test

The standardized apparatus consists of:

- Los Angeles abrasion machine
- **Abrasive charge balls: Cast iron or steel** balls, approximately 48mm in diameter and each weighing between 390 to 445 gm; six to twelve balls are required
- Sieve: 1.70mm , 2.36mm, 4.75mm, 6.3mm, 10mm, 12.5mm, 20mm, 25mm, 40mm, 50mm, 63mm, 80mm IS Sieves.
- The balance of capacity 5 kg or 10 kg is used.
- Oven Drying.
- Miscellaneous elements like a tray

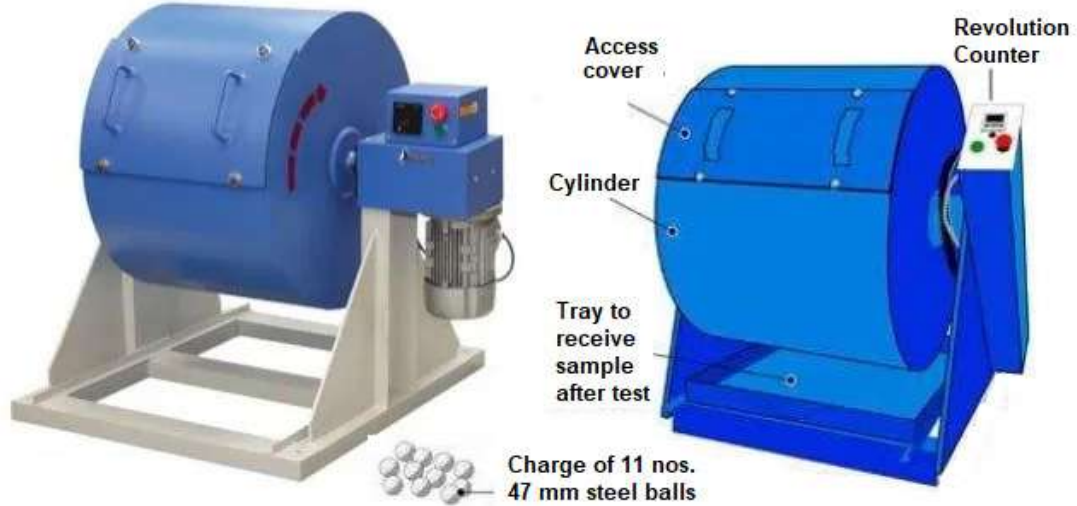


Figure A7.1 – Los Angeles Abrasion Testing Machine

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A7.3 Procedure for LA Abrasion Test

The aggregates sample consists of clean aggregates dried in an oven at 105° – 110°C. The aggregates sample should conform to any of the grading shown in the below table.

- Select the size of aggregate to be used in the test such that it conforms to the grading to be used in construction, to the maximum extent possible.
- Take exactly 5 kg of the sample for grades A, B, C & D, and 10 kg for grading E, F & G.
- Choose the abrasive charge balls as per Table 2 depending on the grading of aggregates.
- Place the aggregates and abrasive charge balls on the cylinder and fix the cover.
- After that Rotate the machine at a speed of 30 to 33 revolutions per minute. The number of revolutions should be 500 for grades A, B, C & D and 1000 for grading E, F & G.
- The machine is stopped after the specified number of revolutions and the aggregate sample is discharged to a tray.
- The entire stone dust made from a machine is sieved on 1.70 mm IS sieve.
- The material size more than 1.7 mm size is weighed correct to one gram.

Table A7.1 - Recommended Test Samples – *Tolerance of ± 12 percent permitted.

Sieve size (square hole)		Weight of test sample in gm						
Passing (mm)	Retained on (mm)	A	B	C	D	E	F	G
80	63					2500*		
63	50					2500*		
50	40					5000*	5000*	
40	25	1250					5000*	5000*
25	20	1250						5000*
20	12.5	1250	2500					
12.5	10	1250	2500					
10	6.3			2500				
6.3	4.75			2500				
4.75	2.36				5000			

Table A7.2 - Abrasive Balls Charge Selection

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Grading	Number of Steel Balls	Weight of Charge in gms.
A	12	5000 ± 25
B	11	4584 ± 25
C	8	3330 ± 20
D	6	2500 ± 15
E	12	5000 ± 25
F	12	5000 ± 25
G	12	5000 ± 25

A7.4 Observations

The original weight of aggregate sample	=	W1 g
The weight of the aggregate sample retained	=	W2 g
Weight sample passing 1.7mm IS sieve	=	W1 – W2 g
Abrasion Value	=	$(W1 - W2) \times 100 / W1$

A7.5 Sample Calculation

Let the weight of the aggregate be W1	=	5000 gram
Weight of the aggregate sample retained W2	=	2100 gram
Weight sample passing 1.7mm IS sieve	=	W1 – W2 g = 2900 gram
Abrasion Value	=	$(W1 - W2) \times 100 / W1$
	=	$(2900/50000) \times 100 = 58$

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A7.6 Results

Los Angeles Abrasion Value = 58

A Los Angeles test is performed to find the hardness of aggregates. On the basis of this value, the suitability of aggregates for different road constructions can be judged as given in the table below.

Table A7.3 - Recommended Los Angeles Abrasion Test Values for Pavements

S. No.	Pavement Layer	Maximum Permissible Abrasion Value
1.	Water Bound Macadam Subbase Course	60
2.	Water Bound Macadam Base Course with Bituminous Surface	50
3.	Bituminous Bound Macadam	50
4.	Water Bound Macadam Surface Course	40
5.	Bituminous Penetration Macadam	40
6.	Bituminous Surface Dressing, Cement Concrete Surface Dressing	35
7.	Bituminous Concrete Surface Course	30

A7.7 Conclusion

Following the LA Abrasion Test as mentioned above, we derived the abrasion value of the sample as 58.

From the table given above, the suitability of aggregates used for the pavements can be known.

For the sample calculation done here having abrasion value of 58, the aggregates can be employed only for WBM sub base course. Its maximum permissible abrasion value is 60.

Reference: Los Angeles Abrasion Test on aggregate, Civiconcepts
Web Link: https://civiconcepts.com/blog/los-angeles-abrasion-test-on-aggregate#Los_Angeles_Abrasion_Value_Test

A8 Aggregate Impact Value Test

Standard Codes of Reference

Pavement Engineering – User's Guide for Computer Applications

This test is suggested by **AASHTO T 96**, **ASTM C 131**, **IS: 2386 (Part IV) – 1963**: Resistance to Degradation of Small-Size Coarse Aggregate by Abrasion and Impact in Los Angeles Machine.

A8.1 General

Aggregate Impact Value is an ability or toughness of aggregates which resist sudden impact or shock load on it. Also, it can be defined as the resistance of aggregate to failure by impact load is known as the Impact Value of Aggregate.

The Characteristics of any material to resist sudden impact is known as toughness. When we use aggregate in roads and pavement on which vehicular movement is done and there is a possibility of impact load on aggregate. This impact load can break aggregate in smaller pieces and which results in the failure of roads and pavements.

Therefore, aggregate used for such propose should have sufficient toughness to resist their disintegration due to the impact. This important property of aggregate is determined by the aggregate impact value test.

This resistance of aggregate may differ from resistance to gradual load which we measure in the crushing value test.

Importance of Impact Value Test:

Generally, the majority of the aggregate test is carried out to know the response of aggregates to different kinds of loads that the aggregates will be subjected to during their service life. The need for impact value test is to find out the toughness of aggregates which is nothing but the ability of aggregates to resist the sudden loading or impact loading.

Find out the impact resistance of aggregate is important because, during service life, the aggregates will be actually subjected to this type of load. For example, aggregates will be subjected to impact after speed breaker, or when the vehicle is moving on the undulating road, etc. So, by impact test, we can find out the response of aggregates to sudden loading. If the result is satisfactory, we use those aggregates in pavement. If not, aggregates are rejected.

A8.2 Objective

This method of test used to measure the impact value of aggregate use in the construction of Highway pavement and concrete structure.

A8.3 Apparatus



Figure A8.1 – Aggregate Impact Value Testing Machine

1. Aggregate Impact testing machine:
2. The total weight of the machine used in the test is not more than 60kg or not less than 45 kg.
3. A steel cup shape as a cylinder usually internal dimension:
 - Diameter = 102mm
 - Depth = 50mm
 - Thickness = 6.3mm
4. 13.5 to 14 kg hammer with the end of cylindrical shape is used for the impact load:
 - Diameter = 100mm
 - Length = 50mm
 - Chamfer at lower edge = 2mm
 -
5. Sieves: The following sieves are used
 - 12.5mm
 - 10mm
 - 2.36mm

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6. Measure:

- The cylinder made with the steel used.
- The internal dimensions are as follows:

Diameter	=	75mm
Depth	=	50mm

7. Tamping rod:

- A straight tamping rod made with the metal of circular c/s is used to tamping the aggregate. Rounded at the end.
- The following are the dimension of the rod:

Diameter	=	10mm
Length	=	230mm

8. Balance:

- A balanced used for the weight measure is not less than 500 gm, or it readable or its accuracy is up to 0.1gm.

9. Oven:

- The oven used for drying the aggregates is well ventilated or it thermostatically controlled to maintain the temperature in the oven 100 to 110 °C.

A8.4 Preparation of the Test Sample

- The aggregate which uses for the sample is passed through the 12.5mm IS sieve and retain on the 10mm IS sieve.
- The aggregate which is retained on the 10mm sieve is dried in an oven for a time period of 4 hours at the oven temperature of 100 to 110° C.
- The cylindrical measure use for the collection sample filled about 1/3 full with the aggregates and it tamped 25 stokes with help of the rounded end of the tamping rod. Further similar 1/3 quantity of aggregates are filled in it and again 25 stokes are given. Than measure finally filled to overflowing than apply 25 stokes again and the extra aggregates are removing form the measure with the help of the straight portion of the tamping rod.

A8.5 Procedure

- The impact machine is rest on the horizontal flat surface like the level plate, block, or floor without any packing. So, the application of a hammer is straight and vertical.
- The cup is fixed in a proper position on the base of the impact machine.
- Transfer aggregate sample from cylindrical measure to cut fitted in an impact test machine and apply 25 strokes of the rod to compact it.
- Then the hammer is lifted 380mm and falls on the upper surface of aggregate which fills in the cup.
- A total of 15 blows are applied on the cup each delivered at an interval of not less than 1 second.

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- The crushed aggregates are removed from the cup and the crushed sample is sieve from the 2.36mm IS sieve.
- Weight the fraction of sample passing through 2.36mm IS sieve accuracy up to 0.1 gm. (W1)
- Weight the fraction of sample retains on the 2.36mm IS sieve. (weight W2)
- The total weight of the sample (W1 + W2) is less than the initial weight (weight W) by more than one gram than the result discarded.

A8.6 Calculation

Aggregate impact value is the ratio of the weight of the fraction passing through 2.36 mm (weight W2) by the total weight of the sample (weight W1 + W2).

$$\begin{aligned}\text{Aggregate Impact Value} &= (W2 / (W1+W2)) * 100 \\ &= 80 \text{ gm} / 300 \text{ gm} \\ &= 26.67 \text{ (Impact Value)}\end{aligned}$$

Where,

W1= Weight of the fraction passing through 2.36 mm IS sieve.(220 gm)

W2= Weight of sample retained on 2.36 mm IS sieves. (80 gm)

W1+W2 = Total Weight of Sample (220 + 80 = 300 gm)

A8.7 Result

The Aggregate Impact Value is _____

A8.8 Aggregate Impact Value and Classification

Table A8.1 – Range of Values for Various Aggregates

Aggregate Impact Value	Nature
<20%	Exceptionally Strong
10-20%	Strong
20-30%	Satisfy for Surface Course
>35%	Weak for Surface Course

A8.9 Maximum Allowable Impact Value of Aggregate for Different Types of Pavements

Table A8.2 – Impact Value of Aggregates for Different Types of Pavements

S. Nos.	Pavement Layer Type	Aggregate Impact Values (Max.)
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1.	Water Bound Macadam (WBM) Subbase Course	50
2.	Cement Concrete Base Course	45
3.	(i) WBM Base Course with Bitumen Surfacing (ii) Built-up Spray-grout Base Course	40
4.	Bituminous Macadam Base Course	35
5.	(i) WBM Surface Course (ii) Built-up Spray-grout Surface Course (iii) Bituminous Penetration Macadam (iv) Bituminous Macadam Binder Course (v) Bituminous Surface Dressing (vi) Bituminous Pre-mix Carpet (vii) Bituminous / Asphalt Concrete	30

Reference:

Aggregate Impact Value Test, Civiconcepts

Web Link: <https://civiconcepts.com/blog/aggregate-impact-value-test-procedure-result-standards>

A9 Aggregate Crushing Value Test

Standard Codes of Reference

This test refers to: AASHTO T 96 or ASTM C 131, BS 812 - 110 (Parts 1 and 2) and IS 2386-4 (1963).

A9.1 General

Aggregate Crushing Value Test is important and to be performed on aggregates. The strength of aggregate parent rock is determined by preparing cylindrical shape specimens of size 25 mm diameter and 25 mm height. This cylinder is subjected to compressive stress. Depending on the type of parent rock gives the different crushing value of aggregate as a compressive strength varying from a minimum of about 45 MPa to a maximum of 545 MPa.

It is a fact that parent rock compressive strength does not exactly indicate the strength of aggregate in concrete. For this reason, assessment of the strength of the aggregate is made by using a sample of bulk aggregate in a standardized manner. This testing method is known as an aggregate crushing value test.

The crushing value test of aggregate provides the resistance of an aggregate sample to crushing under gradually applied compressive load. Generally, the test is conducted on aggregate passing 12.5 mm and retained on a 10 mm sieve. The aggregate sample is filled in a cylindrical mold and a load of 40 tons is applied through a plunger in a compression testing machine.

The crushed aggregate sample which is finer than 2.36 mm is separated and expressed as a percentage of the original weight taken in the mould. The percentage of weight passed through the 2.36mm sieve is known as Aggregate crushing value.

Aggregate Crushing value is a relative resistance of aggregates to crushing under gradually applied compressive load.

In situations, when the aggregate crushing value is 30 or higher the result may be a mistake and in such cases, the "Ten percent fines value" should be determined and used instead.

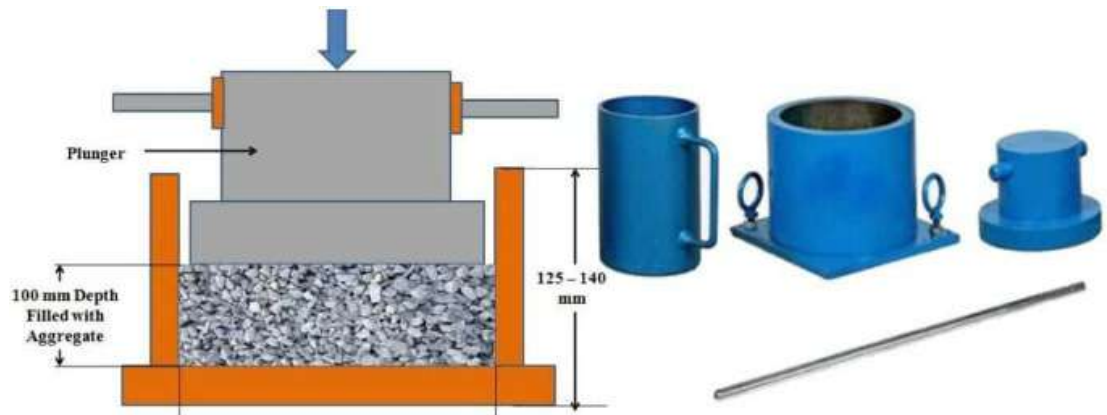


Figure A9.1 – Aggregate Crushing Value Testing Apparatus

A9.2 Apparatus

The apparatus consists of the following items:

1. A 15 cm dia. Steel cylinder with plunger and base plate.
2. A straight metal tamping rod 16mm diameter and 45 to 60cm long rounded at one end.
3. A Weigh balance of accuracy up to 1 gram.
4. Sieves of sizes 12.5mm, 10mm, and 2.36mm
5. A compression testing machine.
6. Cylindrical measure having a diameter of 11.5 cm and height of 18 cm.
7. A compression testing machine having a loading capacity of 40 tones and which can be operated to give a uniform rate of loading so that the maximum load is reached in 10 minutes.

A9.3 Sampling for Aggregate Crushing Value Test

For the Aggregate crushing value test, an aggregate sample passing through 12.5 mm IS sieve and retained on 10mm IS sieve is selected and dried to a temperature of 105 °C to 110 °C then cooled to room temperature.

To fill the cylindrical measure mould about 6.5 kg of a sample of aggregate is sufficient.

A9.4 Procedure for Aggregate Crushing Value Test

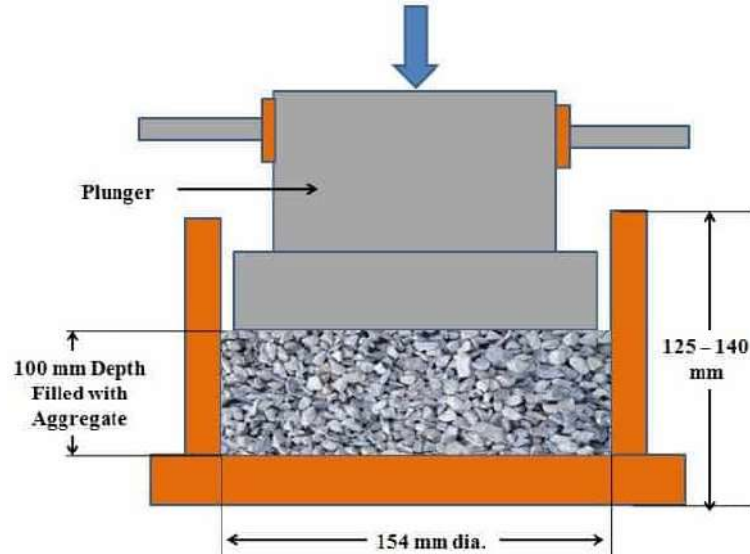


Figure A9.2 – Aggregate Crushing Value Test

Step 1: Take the Empty weight of cylindrical measure as **W1**.

Step 2: Fill the sample aggregates passing through 12.5 mm sieve and retained on 10 mm sieve in measuring cylinder in 3 equal layers by tamping each layer with 25 strokes using the tamping rod. Take the weight of aggregate with measuring cylinder as **W2**.

Step 3: The weight of aggregate sample is calculated as **W = W2 – W1**

Step 4: Now, fill the aggregate sample in 15 cm dia. and 13 cm height steel cylinder and level the surface of aggregates carefully and insert the plunger so that it rests on the surface of the aggregates.

Step 5: Place the steel cylinder with the plunger on the loading plate of the compression testing machine.

Step 6: Operate Compression machine such that 40 tonnes of the load is applied on aggregate in approximately 10 min.

Step 7: Release load and remove the steel cylinder from the machine.

Step 8: Take out the crushed aggregate sample and sieve on with 2.36mm IS sieve, care being taken to avoid loss of fines.

Step 9: Take off the weight of fraction passing through 2.36 mm sieve as (**W3**).

A9.5 Calculation of Crushing Value Test of Aggregate

Crushing strength of aggregate can be found by the following formula,

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Aggregate crushing value is determined by taking a percentage of weight of crushed aggregate samples passing through 2.36 mm Sieve to the weight of aggregate sample taken for test.

Aggregate crushing value formula is given below,

$$\text{Aggregate crushing value} = \frac{W3}{(W2 - W1)} = \frac{W3}{W}$$

W1 = Empty weight of cylindrical Measure.

W2 = Weight of Aggregate with Cylindrical Measure

W = W2 – W1 = Weight of Aggregate Sample

W3 = Weight of crushed aggregate sample passed through 2.36 mm Sieve.

For accurate result at least 3 times the test is conducted and an average of 3 values of percentages is taken.

A9.6 Standard Crushing Values for Pavement and Road Construction

The below table shows the minimum crushing value of aggregate or crushing value of aggregate limits for different types of roads

Figure A9.1 - Limits of Crushing Values of Aggregate for Different Types of Pavement Layers

Types of Pavement Layers	Limit of Aggregate Crushing Value
Flexible Pavements	
Soling	50
Water Bound Macadam	40
Bituminous Macadam	40
Bituminous Surface Dressing	30
Dense Mix Carpet	30
Rigid Pavements	
Other than Wearing Course	45
Surface and Wearing Course	30

Reference: Aggregate Crushing Value Test, Civiconcepts

Web Link: <https://civiconcepts.com/blog/aggregate-crushing-value-test-procedure-and-result>

A10 Flakiness and Elongation Index Test of Coarse Aggregate

Standard Codes of Reference

This test refers to: AASHTO T 248, AASHTO TP33, ASTM D 4791, ASTM 131, ASTM C 1252, IS: 2386 (Part I) – 1963.

A10.1 General

Flakiness and Elongation Index Test are very important tests to be performed on aggregate in the laboratory. This test gives the percentage of flaky and elongate aggregate present in the total aggregate sample.

Aggregate shape, size, and surface texture has significant effect on the properties of freshly mixed concrete more than the properties of hardened concrete. Angular, flaky, rough-textured, and elongated aggregate particles require more water to produce workable concrete than the smooth, rounded compact aggregate. Also, for such irregular-shaped aggregates the cement content must also be increased to maintain the required water-cement ratio.

Elongated and flaky aggregate, when used in the construction of pavement may result in failure of the pavement due to their random shapes under repeated loading and vibration.

It is important to keep the amount of flaky and elongated aggregate within permissible levels. Generally, flaky and elongated aggregate are avoided or are limited to about 15 % by weight of the total aggregate.

What is Flakiness Index?

The flakiness index of aggregates is the percentage of weight of the particles (aggregates) whose thickness is less than $\frac{3}{5}$ th (0.6 times) of their mean dimension.

What is Elongation Index?

The Elongation index of aggregates is the percentage of weight of the particles (aggregates) whose length is greater than 1 and $\frac{4}{5}$ th (1.8 times) of their mean dimension.

A10.2 Objectives

The tests measure the Flakiness Index and Elongation index of aggregates use in the construction of Highway pavement and concrete structure.

A10.3 Apparatus

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Figure A10.1 – Flakiness and Elongation Gauges

Balance:

The balance used to measure the aggregate has sufficient capacity. Its accuracy is 0.1% of the weight of the test sample.

Metal gauge:

The metal gauge used to measure the thickness/elongation is shown in the figure below:

The Set of Sieves:

63 mm, 50 mm, 40 mm, 31.5 mm, 25 mm, 20 mm, 16 mm, 12.5 mm, 10 mm, 6.3 mm.

Table A10.1 – Sizes of Aggregates

Passing Sieves	Retained on Sieves	Thickness Gauge	Length Gauge
1	2	3	4
63 MM	50 MM	33.90 MM	0
50 MM	40 MM	27.00 MM	81.0 MM
40 MM	31.5 MM	19.50 MM	58.5 MM
31.5 MM	25 MM	16.95 MM	0
25 MM	20 MM	13.50 MM	40.5 MM
20 MM	16 MM	10.80 MM	32.4 MM
16 MM	12.5 MM	08.55MM	25.6 MM
12.5 MM	10 MM	06.75 MM	20.2 MM
10 MM	6.3 MM	04.89 MM	14.7 MM

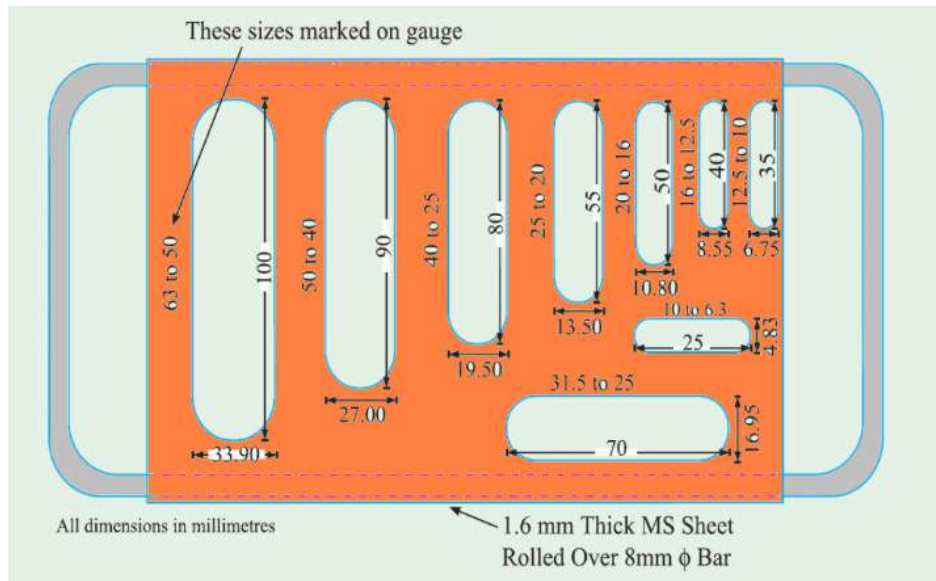


Figure A10.2 – Flakiness Index Gauge

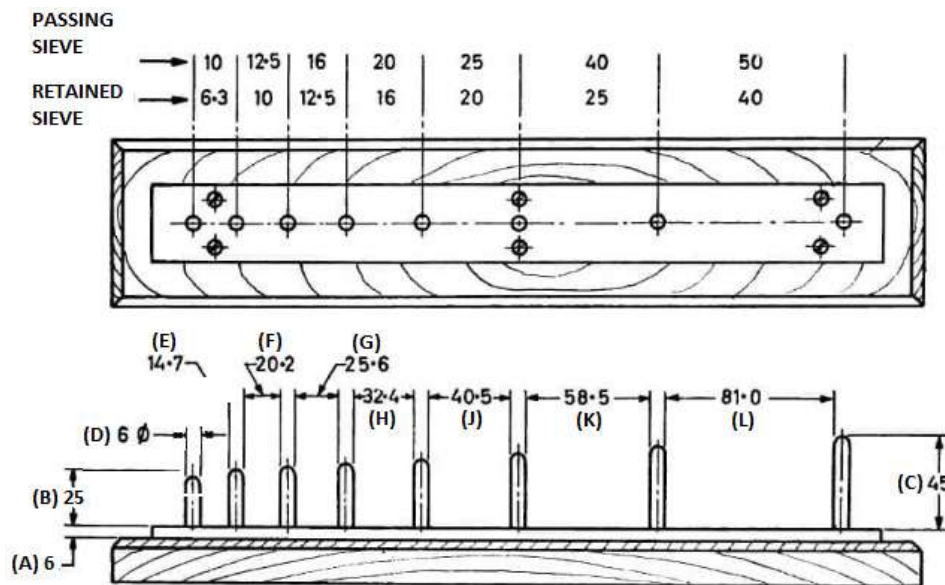


Figure A10.3 – Elongation Index Gauge

Elongation Index Gauge

- The Dimension shown in thickness gauge is equal to 0.6 times the mean sieve size.
- The Dimension shown in length gauge is equal to 1.8 times the mean sieve size

A10.4 Preparation of Sample For Flakiness and Elongation Index Test

- Prepare aggregate sample for the test by sieving between IS sieve 6.3 mm size at the bottom and 63 mm IS sieve at top.
- Separate sample retains on each sieve.
- The collected aggregate sample should have at least 200 numbers of individual aggregate for accurate test results from each range of IS sieve.
- We take a range of aggregate samples retained on 6.3 mm IS Sieve. (Separate test can be conducted on each range of Sieve).
- Take the weight of the total aggregate sample collected for tests as W1



Figure A10.4 – Sieves for Aggregate Grading

A10.5 Flakiness Test

- Try to pass the aggregate width-wise through the slot of 10mm to 6.3 mm on a flakiness gauge.
- Separate sample passed through this slot and weight it(W2)

Test may be conducted for each range of Flakiness gauge slot.



Figure A10.5 – Separation of Flaky Materials

Table A10.2 – Observation Chart for Flakiness test

Passing through Sieve (mm)	Retained on Sieve (mm)	Flakiness Gauge (mm)	Weight Passing on Flakiness Gauge (W1)
63	50	33.90	
50	40	27.00	
40	31.5	19.50	
31.5	25	16.95	
25	20	13.50	
20	16	10.80	
16	12.5	8.55	
12.5	10	6.75	
10	6.3	4.89	
Total			

Flakiness Index = Weight of aggregate passed through width gauge 10 mm to 6.3 mm (W2) / Total weight of aggregate sample taken (W1) = (W2/W1) x 100

For example, Weight of aggregate passed through width gauge of 10 mm to 6.3mm (W2) = 200 gm and Total weight of aggregate sample = 1000 gm

Flakiness Index = (200 gm / 1000 gm) x 100 = 20%

Result: Percentage of flaky aggregate in the range of 10 mm to 6.3 mm is 20%

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A10.6 Elongation Test

- Try to pass the aggregate length wise through slot of 10mm to 6.3 mm on length gauge.
- Separate sample retain on this slot and weight it(W2).

Test can be conducted for each range of elongation gauge slot.



Figure A10.6 – Separation of Elongated Materials

Table A10.3 – Observation Chart for Elongation Test

Passing through Sieve (mm)	Retained on Sieve (mm)	Elongation Gauge (mm)	Weight Retained on Length Gauge (W2)
63	50	0	
50	40	81.0	
40	31.5	58.5	
31.5	25	0	
25	20	40.5	
20	16	32.4	
16	12.5	25.6	
12.5	10	20.2	
10	6.3	14.7	
Total			

Elongation Index = Weight of aggregate retained on length gauge 10 mm to 6.3 mm (W2) / Total weight of aggregate sample taken (W1) = $(W2/W1) \times 100$

For example, Weight of aggregate retained on gauge of 10 mm to 6.3mm (W2) = 150 gm and Total weight of aggregate sample = 1000 gm

Elongation Index = $(150 \text{ gm} / 1000 \text{ gm}) \times 100 = 15\%$

Result: Percentage of Elongated aggregate in the range of 10 mm to 6.3 mm is 15%

Reference: Flakiness and Elongation Index Test of Coarse Aggregate, Civiconcepts

Web Link: <https://civiconcepts.com/blog/flakiness-and-elongation-index-test-of-coarse-aggregate>

A11 Test for Loss on Heating Bitumen

Standard Codes of Reference

This test refers to: AASHTO T 47- 93, ASTM D 6-90, IS:1203 - 1978, IS:1211 - 1978 for Loss on Heating Test of Bitumen.

A11.1 General

To determine the loss of volatiles, present in asphaltic bitumen on heating. Bitumen is commonly used as a binder in pavements. When it is exposed to the atmosphere, the volatiles present in it evaporates and bitumen hardens. The loss of volatiles like oil except water on heating the bituminous material to standard temperature and under predetermined conditions is termed as 'Loss on Heating'.

The volatile materials evaporate when they are subjected to high temperature. Bitumen is heated to about 163 °C so that the volatile materials are evaporated.

In the loss on heating test of bitumen, the bitumen specimen is subjected to an accelerated ageing process under the conditions specified by the standard.

By carrying out penetration test at the start and finish of the test, the amount by which bitumen hardens can be known.

A11.2 Apparatus

The apparatus used while conducting loss on heating test of bitumen are described below:

1. Hot Air Oven



Figure A11.1 - Hot Air Oven

Rectangular double-walled oven is taken. Its parts and their dimensions are mentioned below:

- Interior Height from top of the heating element to the top of chamber = 292-298 mm
- Chamber is provided in the oven on one side which has a tight-fitted door hinged to it. Chamber section $\geq 100 \text{ mm}^2$.

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- A glass window is provided at front for observation.
- Ventilation arrangement- One round vent hole at top or bottom. Alternatively, one round vent hole at top and bottom each are provided on sides. Diameter of Vent Hole = 12-16 mm
- Electricity is required for heating purpose

2. Perforated Metal Shelf



Figure A11.2 - Perforated Metal Shelf

It is made up of aluminum and placed in the centre of the oven. It is suspended by means of a vertical shaft which is rotated at the rate of 5-6 rev/min. Its diameter is about 250 mm.

3. Thermometer or Temperature Indicator and Controller



Figure A11.3 - Temperature Indicator and Controller

To measure the temperature of water in the ductility apparatus. It should be graduated in Celcius. The temperature range should be from 155-170 °C. it is supported on the shaft of the perforated shelf so that it can be kept in a vertical position. Distance between the top of contraction chamber and bottom of bulb of thermometer \geq 41 mm.

4. Weighing Balance

It should be accurate to 0.1 gram.

5. Other accessories required for conducting Loss on Bitumen Test are as follows

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- Container
- Asbestos gloves
- Penetrometer and water bath
- Container to mix bitumen of about 225 gram
- Hotplate

A11.3 Materials

- Bituminous material to be tested

A11.4 Precaution

Following precautions should be taken while performing the loss on heating test of bitumen to obtain accurate results:

- The loss on heating test of bitumen should be conducted in duplicate
- If there is presence of water in the sample, it should be tested in that composition only
- The temperature of the oven should be strictly maintained to 163 °C
- Hot air oven, analytical balance, penetrometer and water bath should be calibrated in a while
- Water content of the sample should be determined in accordance with IS 1211: 1978 as the method of determination of water content specified
- If the sample shows evidence of loss by foaming, that sample should not be considered but rejected

A11.5 Procedure of Test for Loss on Heating Bitumen

- Agitate the bituminous material by stirring as received so that a complete mixture is obtained. Heat the sample above its softening point, if necessary.
 - Heat the containers at 100-110 °C for about 30 minutes in the oven and then allow them to cool at room temperature and measure the weight of the empty containers.
 - Take a portion from the well-mixed material for testing in a container and allow it to cool. Prepare at least 2 such specimens.
1. Weigh the sample along with the container to the nearest 0.1 gram and subtract the weight of the container to obtain the weight of the sample- W_1 . The weight of the sample filled in the container is 50.0 ± 0.5 gram.
 2. If penetration values are to be compared too, then pour the sample in penetrometer cup also. Find out the penetration value as specified in IS: 1203- 1978.
 3. Set the temperature of the oven to 163 ± 1 °C.
 4. Place the container with the sample on the perforated shelf placed centrally and close the oven.

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5. Maintain the temperature to 163 ± 1 °C for 5 hours after the oven has achieved the specified temperature. The duration for which the sample is placed in the oven should not be more than 5 hours and 15 minutes.
6. Switch on the motor switch to allow the perforated plate to rotate at a rate of 5-6 rev/min.
7. Allow the sample in the containers to cool at room temperature.
8. Weight the sample to the nearest 0.1 gram- W2.
9. Determine the water content for the sample as done in the determination of water content test in accordance with IS: 1211- 1978.
10. Calculate weight loss of volatile materials because of heating from the above-observed values.
11. Arrange the containers on the perforated shelf turn on the oven for 5 minutes at the specified temperature.
12. If penetration test is also to be compared, then mix the sample of the containers and keep the mixture on a hotplate so that it remains in the liquid state. Pour the mixture into a penetrometer cup and determine the penetration value. The value will be less than the previous one.

A11.6 Observation

Following observation is made as the loss on heating test of bitumen is conducted:

The weight of the sample reduces on heating as the volatile materials are evaporated. Hence, the initial weight of the sample is more while the weight after testing is reduced. The difference in weight is the loss of volatile materials present in the bitumen.

Table A11.1 - A Sample Observation Format for Test of Loss on Heating Bitumen

Observations	Sample1	Sample2
Initial Weight of Sample in gm (W1)		
Final Weight of Sample in gm (W2)		
Loss on Heating (W1 – W2) in gm		
Penetration Value before testing P1 (in mm)		
Penetration Value after testing P2 (in mm)		
Reduction in Penetration Value P1 – P2 (in mm)		

Penetration values are to be noted only when hardening is measured along with loss on heating test of bitumen.

Loss in weight on heating = Average value of loss obtained for samples 1 and 2.

Apply corrections for water loss, if the water is present in the natural conditions of the sample. Water content is determined and is added to the final result.

A11.7 Result

The values obtained from the loss on heating test of bitumen are recorded as follows:

Loss in weight on heating (in gms)

= average of loss in weight on heating of the two samples. The result is reported as the percentage by mass of the original sample.

Thus, Loss in weight on heating (in %)

= (Average Weight Loss/ Original Average Weight of Sample) x 100

Loss in Weight (in gram)

= The average loss of weight of the two containers

Loss in weight after the test (in %)

= (Average Weight Loss/ Original Average Weight of Sample) x 100

in penetration value (in %)

= (Reduction in Penetration Value / Original Penetration Value of Sample) x 100

The loss in weight on heating is compared against the standard values.

- For road pavements, loss in weight should not exceed 1 % by mass.
- For a sample having penetration value of 150-200, loss in weight should not exceed 1 % by mass.
- If penetration values are also to be compared, then the penetration test is carried out before and after heating the sample in the oven. The difference in the Reduction in Penetration value of the residue after the test has been performed (in %) = (Reduction in Penetration Value / Original Penetration Value of Sample) x 100

A11.8 Conclusion

By performing the loss on heating test of bitumen, we can determine whether the given bitumen sample meets the specification requirements for loss on heating as given in the standards.

The value found out by the practical is compared to the standard value and thus It can be concluded whether the sample meets the criteria.

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Loss on heating test of bitumen gives a measure of the amount of volatile materials evaporated on heating. It serves as a measure as to how much bitumen will harden after the loss of these materials.

The hardness gained after the loss of volatile materials can be compared to the results of penetration tests.

Reference: Loss on Heating Test of Bitumen, Civiconcepts

Web Link: <https://civiconcepts.com/blog/loss-on-heating-test-of-bitumen>

A12 Softening Point Test of Bitumen (Ring and Ball Test)

Standard Codes of Reference

This test refers to: **AASHTO T 53** and **ASTM D 36**, **IS: 1205-1978** for the softening point test of bitumen.

A12.1 General

Softening Point Test of Bitumen sample is one of the important tests of bitumen to be conducted before road construction. It is also known as **Ring and ball test**. This is because ring and ball apparatus is a major equipment used for determining the softening point of bitumen.

The softening point helps in the determination of the temperature beyond which the bitumen is softened beyond a pre-specified softness.

Hence, the softening point test of bitumen helps in knowing the maximum temperature to which the given bitumen can be exposed.

This is utilized while using bitumen as a binder for road construction. Softening point value gives the temperature at which the bitumen should be heated before using it for pavement construction.

If the climatic temperature exceeds the softening point, the bitumen used as binder starts to melt and the pavement becomes sticky.

Softening Point: It is the degree of temperature at which a solid substance starts to convert into liquid state under certain specific conditions.

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Alternatively, the softening point test of bitumen determines the temperature at which the bitumen attains a particular degree of softening under specified conditions.

In case of bitumen, the softening is the temperature at which the steel ball passes through the bitumen specimen kept in a ball and travels the height of 2.5 centimetres when heated under water in the specified conditions.

A12.2 Apparatus

Ring and Ball apparatus (for Ring and Ball Test):

It comprises of the following:

2 Balls:

Each of the ball is of 9.5 mm diameter and its weight is 3.5 ± 0.5 gram.

2 Brass rings:

The rings are tapered from inside- the outside diameter of the ring is constant while the inner diameter decreases from top to bottom.

The rings have three pins which act as a centering guide and they form an imaginary diameter of about 7.5 mm. this arrangement enables the movement of steel ball during the test. Dimensions of each ring are mentioned below:

Depth of ring	=	6.4 mm
Inside diameter at Top	=	17.5 mm
Inside diameter at Bottom	=	15.9 mm
Outside diameter	=	20.6 mm

Support: A metallic frame is used to support the rings. Its requirements are specified below:

The topmost plate: It should have a hole in the topmost plate from which a thermometer can be suspended so that it can remain in level with the bottom of the rings. The thermometer should remain within 10 mm distance of the rings but it should not touch them.

The middle plate: It has three holes. Two outer holes support the rings in a horizontal position such that the upper surface of the ring is 50 mm below the water level. The thermometer passes from the central hole.

The bottom plate: It is a solid plate with no holes. The distance should be 25 mm exact between the bottom of the rings and the top of the bottom plate.

Beaker: It should be heat resistant and its capacity should be 600 ml.

Stirrer: It can be manual or mechanical. It is used to ensure a uniform heat distribution at all times when the sample is heated in the water bath. Stirrer should be so placed that when it is operated, the moulds are not disturbed.

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Thermometer: It is of mercury in glass type filled with nitrogen. Its back is enameled black and its top is provided with an expansion chamber and a glass ring. Its temperature range should be from -2 to 80 °C with a least count of 0.2 °C.

Water bath: A heat-resistant glass vessel is taken for the water bath having a minimum diameter of 85 mm and is 120 mm deep.

Other accessory apparatus include:

- Hot plate with energy regulator
- Vessel for heating bitumen
- Forceps
- Sharp-edged knife

A12.3 Materials for Softening Point Test of Bitumen

Following materials used in softening point test of bitumen-

- Bitumen to be tested
- Glycerine
- Dextrine
- Distilled water- for heating medium

A12.4 Precaution

Following precautions should be taken while performing the softening point test of bitumen to obtain accurate results:

- The water used in the practical should be freshly boiled distilled water. Else there is a possibility that air bubbles may form in the specimen which will affect the accuracy of the results
- To obtain accurate results, the rate of heating should be strictly followed as mentioned
- To prevent the material from sticking to the glass vessel, a conveniently weighed sheet of filter paper or thin amalgamated sheet can be placed on the bottom of the glass vessel. This will save time and would provide ease in cleaning

A12.5 Sample Preparation

- Take the bitumen sample in a vessel and heat it to a temperature of 75-100 °C-

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above its approximate softening point. Allow the bitumen to melt until it is converted completely into a liquid state. Stir the bitumen if necessary so that it is melted completely and is free from air bubbles and water. If the need arises, filter it through IS sieve 30.

- Prepare a mixture of glycerine and dextrine in equal proportions. Mix it well. Then coat the surface of the metal or glass plate. This prevents the bitumen from sticking to the plate.
- Heat the rings to approximately same temperature as that of the molten bitumen and place them on the glass or metal plate coated with the mixture of glycerine and dextrine.
- Pour the molten bitumen in the rings carefully till the rings are slightly filled above the top level.
- Allow the rings to cool at room temperature in air for 30 minutes.
- Cut the excess bitumen on the surface with the help of hot, straight-edged putty knife or spatula and level the top surface.

A12.6 Testing of the Specimen

- Fill the water bath with freshly boiled distilled water to such a height that the water level remains 50 mm above the upper surface of the rings. Maintain this water bath at 5 °C temperature.
- Fix the rings filled with bitumen to the ball guide. Assemble the rings in the support frame.
- Place the rings fixed in the support frame in the water bath at 5 °C for 15 minutes.
- Cool the steel balls to a temperature of 5 °C.
- Place a steel ball on the top of the ring guide of the rings attached to the support with the help of forceps.
- Place this assembly in a glass beaker filled with distilled water to such a height that the water level is at least 50 mm above the top surface of the ball.
- Put the beaker on the hotplate and adjust the stirrer. Insert the thermometer too.
- Allow the beaker to be heated at a uniform rate of 5 ± 0.5 °C/min. The rate can be adjusted with the help of the energy regulator.
- Stir the water continuously to ensure that the water is heated uniformly.
- Continue the heating till the bitumen filled in the ring softens and the ball placed on it starts to move down owing to its own weight.
- Note the temperature when each of the balls touches the bottom plate while sinking from the thermometer.

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A12.7 Observation

Following observation taken during softening point test of bitumen,

- As the water is heated continuously, the bitumen in the rings starts to melt.
- As a result, the steel balls placed over it starts to descend in the water due to its own weight.

The temperature at which the steel ball touches the bottom plate is observed and its value is recorded. This is done for both the balls.

Table A12.1 - Sample Observation Table for Softening Point Test or Ring and Ball Test

Observation	Ball No: 1	Ball No: 2
The temperature when the ball touches the bottom plate in °C		

A12.8 Result

The average of the softening point values of bitumen to the nearest 0.5 °C obtained for the two steel balls is the softening point of the given bitumen specimen.

The softening point value of the two readings should not exceed 1.0 °C for the softening point of bitumen range 40-60 °C and 1.5 °C for the softening point between 61-80 °C. Else the test is considered invalid and it is repeated.

Minimum softening point values standardized by Bureau of Indian Standard for Bitumen to be used for pavement for different grades are tabulated below:

Also softening points of various bitumen grades,

Table A12.2 - Softening Points of Various Bitumen Grades

Paving Bitumen Grades	Softening Points
VG 10	40° C
VG 20	45° C
VG 30	47° C
VG 40	50° C

A12.9 Conclusion

By performing the softening point test of bitumen, we can determine whether the given bitumen sample meets the specification requirements for softening point as given in the standards.

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The softening value found out by the practical is compared to the standard value and thus it can be concluded whether the sample meets the softening point criteria.

Softening point is one of the measures of the quality of bitumen along with other measures such as penetration value, ductility value, and so on.

Reference: the softening point test of bitumen, Civiconcepts

<https://civiconcepts.com/blog/softening-point-test-of-bitumen-ring-ball-test>

A13 Float Test of Bitumen

Standard Codes of Reference

This test refers to: **AASHTO T50**, **ASTM D139**, **IS: 1215-1978**, **IS : 1210-1978** which are for the Float test of Bitumen.

A13.1 General

A Float test of Bitumen is generally used to determine the consistency of a highly viscous bitumen sample. The consistency of bitumen is an important property of bitumen to be found out before using. The float test of bitumen gives us an idea about how the bitumen sample behaves in temperature changes and resistance to flow.

The float test of Bitumen measures the consistency of the bitumen when it is not possible to determine the consistency by penetration test or viscosity test.

Consistency of Highly viscous bitumen is tested by float test. The distillation residues and residues from emulsion are tested by float test.

Consistency of bitumen is an essential parameter to analyze the quality of bitumen.

Consistency measures the susceptibility of the bitumen to temperature change and resistance to flow.

The bituminous binder used in the pavement should be able to resist the deformations applied to it. and consistency affects the ability of bitumen to resist deformations. Hence, it is important to ascertain the consistency of bitumen before its use.

The consistency of bitumen measures the susceptibility of the bitumen to temperature change and its resistance to flow.

A13.2 Apparatus

The apparatus used in the float test of bitumen are described below:

Floating apparatus

Float is made up of aluminum alloy. The details and normal measurements of its various parts are as follows:

- Weight of Float = 37.9 ± 0.2 gram
- Total Height of Float = 35.0 ± 1.0 mm
- Height of Rim above the lower side of shoulder = 27 ± 0.5 mm
- The thickness of Shoulder = 1.4 ± 0.1 mm
- Diameter of Opening = 11.1 ± 0.1 mm

Collar

It is made up of brass. The details and normal measurements of its various parts are as follows:

- Weight of Collar = 9.8 ± 0.2 gm
- Overall Height of Collar = 22.5 ± 0.2 mm
- Inside Diameter at Bottom of Collar = 12.82 ± 0.10 mm
- Inside Diameter at Top of Collar = 9.70 ± 0.05 mm



Figure A13.1 – Float and Collar

Thermometer

To measure the temperature of water in the ductility apparatus. Its range should be from -2 to 80 °C.

Water Bath

Its dimensions are as follows:

- Minimum Internal Diameter– 185 mm
- Minimum Water Depth– 185 mm
- Height of Container above Water Level– 100 mm

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Figure A13.2 – Water Bath for Float Test of Bitumen

Calibration Assembly

The assembled float and collar in which collar is filled until bottom weighing 53.2 gram in total should float in the water such that its rim remains above the water level by 8.5 ± 1.5 mm.

Other accessories are:

- Knife or spatula
- Heating oven
- Beaker
- Stopwatch

A13.3 Materials for Float Test of Bitumen

- Bituminous material - cutback bitumen or asphalt or tar products
- Glycerin
- Dextrin

A13.4 Precaution for Test

Following precautions should be taken while performing the float test of bitumen to obtain accurate results:

- The collar should be tightly fitted to the float to ensure that no seepage of water takes place between them
- The melted bitumen should be stirred thoroughly before pouring in the collar to ensure its homogeneity
- No air bubbles should be entrapped in the specimen while pouring the melted bitumen in the collar

A13.5 Sample Preparation

- Take the bitumen sample in a beaker and heat it to the lowest possible temperature. Allow the bitumen to melt until it is converted completely into a liquid state sufficient for pouring.
- Stir the bitumen sample thoroughly to get a homogenous sample free from any air bubbles.
- Prepare a mixture of glycerin and dextrin in equal proportions. Mix it well.
- Take the brass plate and coat it with the mixture prepared in the above step. Then place the smaller end of the brass collar on the brass plate.
- Pour the air-free homogenous bitumen sample into the collar as per your convenience till it is filled slightly above the top.

A13.6 Asphalt and Asphalt Products

- Allow the specimen to cool down at room temperature for about 15 minutes.
- Place the specimen in the water bath for 5 minutes and maintain the temperature of 5 °C.
- Remove the sample from the water and trim the sample from the top of the collar with the help of a slightly-heated steel knife or a spatula.
- Place the collar and base plate with the sample again in the water bath for at least 15 minutes at 5°C. Care should be taken that the duration does not exceed 30 minutes.

A13.7 Tar Products

- Immerse the tar products in the water of the water bath immediately for 5 minutes and maintain the temperature of 5 °C.
- Remove the sample from the water and trim the sample from the top of the collar with the help of a slightly-heated steel knife or a spatula.
- Place the collar and base plate with the sample again in the water bath for at least 15 minutes at 5°C. Care should be taken that the duration does not exceed 30 minutes.

A13.8 Preparation of Warm Bath

- Heat the water bath to the temperature required for testing of the sample- 50 °C.
- Maintain this temperature throughout the test without stirring. Variations up to $\pm 5^{\circ}\text{C}$ from the specified temperature are allowed.

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- Ensure the temperature by immersing the thermometer in the water. The bulb of the thermometer should be immersed below the top of the water level to a depth of 40 ± 2 mm.
- Heat the water for a minimum of 15 minutes and not more than 30 minutes.

A13.9 Testing of the Specimen

- Remove the collar along with the sample from the base plate.
- Place the sample along with the collar to the aluminum float and fix it with a screw.
- Immerse this assembly in water for 1 minute at 5 °C.
- Take out the assembly from the water and remove the water from inside of the float, if present inside.
- Hover the assembly immediately into warm bath prepared previously of temperature about 50 °C ensuring that the collar is tightly attached to the float. No seepage of water should occur at the time of testing between the collar and float.
- Take a stopwatch and start the timer as soon as the assembly is introduced into the warm bath.
- Note the time in seconds at which the water breaks through the specimen with the help of a stopwatch. This time gives the float value.

A13.10 Testing Observation

As the specimen is kept in warm water, the water slowly breaks through the specimen in some time.

The time at which the thread of the bitumen breaks is to be noted down for both the specimen.

Following observations are to be recorded during the float test of bitumen

Table A13.1 - Sample Observation Table for Float Test of Bitumen

Observations	Specimen1	Specimen2
Time in Seconds		

Float Value = Average float value of 1 and 2.

A13.11 Result

The average of the float values to the nearest whole value obtained from the two samples is the float value of the given bitumen specimen.

A13.12 Conclusion

By performing the Float test of Bitumen, we can determine whether the given bitumen sample meets the specification requirements for penetration as given in the standards.

Higher the float value, stiffer is the bitumen specimen. Consistency in terms of penetration time is one of the measures of the quality of bitumen along with other measures such as ductility value and so on.

Reference: Float test of Bitumen, Civiconcepts

Web Link: <https://civiconcepts.com/blog/float-test-of-bitumen>.

A14 Penetration Test of Bitumen

Standard Codes of Reference

This test refers to: **AASHTO T50**, **ASTM D139**, **IS:1203-1978**, **IS: 1215-1978**, **IS: 1210-1978** which are for the **Penetration** test of Bitumen.

A14.1 General

The **Penetration Test of Bitumen** sample is one of the important tests of bitumen to be conducted before road construction.

The road pavement comprises of stone chippings attached together with the help of a binder like bitumen.

The more viscous the bitumen, the less distance needle is able to penetrate. Hence, the penetration value for viscous bitumen is less. Also, the softening point of viscous bitumen is more. Thus, a bitumen sample with a low penetration value has a high softening point.

Bitumen is available in the market in various grades. The suitable grade of bitumen is selected as per the surfacing type and climatic conditions as well.

The grading of the bitumen is done on the basis of the consistency of bitumen. The consistency, in turn, is measured by the penetration value obtained from the penetration test of the bitumen.

To determine the penetration value of asphaltic bitumen, fluxed native asphalt, and blown type bitumen.

In this practical, the consistency of the sample is determined by measuring the distance in one-tenth of a millimeter the distance by which the penetrating needle penetrates in the sample under certain loading (100 g) at 25 °C for 5 seconds.

Penetration is the distance travelled by a standard needle of 1 sq. mm cross-section and comprising specific size, weight, and shape under predetermined conditions of temperature as well as time in terms of one-tenth of a millimeter.

Design Standard for penetration test of bitumen is (IS:1203-1978)

A14.2 Apparatus

Penetrometer: It is the testing machine for penetration test of bitumen. It allows fixation of penetration needle, plunger, weight, and a dial. It allows the needle to penetrate into the sample placed in the cup without considerable friction. It should be accurately calibrated so that it gives the results in one-tenth of a millimeter.



Figure A14.1 – Penetrometer for Penetration Test of Bitumen

There are two types of penetrometer

1. Standard Penetrometer: The knob is to be pressed for five seconds manually so that the released needle can penetrate the sample.

2. Automatic penetrometer: The penetration needle is automatically released for five seconds with the help of a timer and then stops.

3. Standard Penetration needle: It is straight, highly polished, and cylindrical made up of stainless steel. It has a 3 mm diameter shank into which it is fixed such that its movement is restricted. The taper is pointed while the point is made blunt by grinding to a truncated cone.

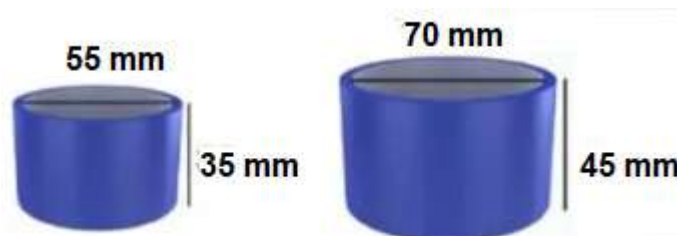
Total length of the needle = 50 mm

Diameter of needle = 1.0 mm

Diameter of the pointed end of needle = 0.14-0.16 mm

Weight = 50 gram

4. Container: The material is either metal or glass and it is cylindrical in shape. Its bottom is flattened. The dimensions of the container are mentioned below:



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Figure A14.2 – Container

For penetration value < 225

Diameter = 55 mm

Internal Height = 35 mm

For penetration value > 225

Diameter = 70 mm

Internal Height = 45 mm

5. Water bath: It should contain a minimum of 10 litres of water. Bitumen specimen should be immersed to at least 100 mm depth from the top. Moreover, the specimen should be supported over a perforated shelf which should be above 50 mm from the bottom of the bath. The water bath is maintained at 25 ± 0.1 °C temperature.



Figure A14.3 – Water Bath

6. **Transfer Dish:** It is a small dish or tray It helps in keeping the container fully immersed in the water when placed in the water bath. It has some means to ensure a firm bearing so that the container does not shake.

7. Thermometer or Temperature indicator– To measure the temperature of water in the water bath. Its range should be from 0 to 44 °C. Its least count is 0.2 °C.

8. Time device: If a standard penetrometer is used, a suitable time device like a stopwatch, electric timer, or another spring-actuated device may be used which is graduated 0.1 second or less. The timing device should have an accuracy of ± 0.1 seconds for a 60-second interval. Even an audible seconds counter where one beat corresponds to one second can also be used alternatively.

Other accessory apparatus include: **Cotton dab**

A14.3 Materials

Following materials required for penetration test of bitumen-

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- Bitumen of known grade
- Benzene

A14.4 Precaution

Following precautions should be taken while performing the penetration test of bitumen:

- For a sample having extraneous matter, the sample should be sieved through IS sieve 30
- Ensure no air bubbles or water remain entrapped after heating the sample
- An air oven can be used to avoid overheating of the bottom of the container. Or a sand bath can also be used.
- The needle is so adjusted that it just touches the bitumen sample- it should not penetrate on the bitumen surface beforehand
- If there is any movement of the container while the needle is penetrating, the test should be discarded.

A14.5 Sample Preparation

Below steps to be followed while performing penetration test of bitumen:

1. Take the bitumen sample in a beaker and heat it to a temperature above its approximate softening point. For tars and pitches, it is heated up to 60 °C while bitumen is heated up to 90 °C.
2. Allow the bitumen to melt until it achieves a pouring consistency. Keep stirring the sample so that the sample is homogenous and is free from air bubbles and water.
3. Pour the melted bitumen into the container of 35 mm depth until it is filled up to a depth of 10 mm more than the expected penetration.
4. Place the container on the transfer dish and allow it to cool in the atmosphere at temperature in-between 15-30 °C for 1.0-1.5 hours when a 35 mm deep container is used. The time period is increased to 1.5-2.0 hours when a 45 mm deep container is used.
5. Place the whole assembly with container and transfer dish in a water bath, which is maintained at 25 ± 0.1 °C for about 1.5-2.0 hours when 35 mm deep container is used. The time period is reduced to 1.0-1.5 hours when a 45 mm deep container is used.
6. When cutback bitumen or Digboi type bitumen is to be tested, the residue left after the distillation process is used for this test.

A14.6 Procedure

1. Remove the transfer dish from the water bath and fill the water from the water bath in the dish to such a depth that the container is covered completely.
2. Place the sample container in this transfer dish and place this assembly under the penetration needle on the penetrometer.
3. Clean the needle with cotton dabbed in benzene and allow it to dry. Load the needle to the specified weight.

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4. Adjust the needle so that its tip just touches the surface of the bitumen sample. Finely adjust the needle by raising the platform slightly with the help of adjusting the screw. Check the contact of the needle to the surface of bitumen by observing the image of the needle reflected by the surface of the material.

5. Clamp the needle in this position.

6. Load the needle holder with a weight such that the total moving weight is equal to 100 ± 0.25 grams.

Moving weight = weight of needle + weight of carrier + superimposed weights

7. Note down the initial reading of the dial of the penetrometer. Or bring the pointer to zero.

8. In a standard penetrometer, release the needle for exactly 5 seconds by pressing the knob. Measure the time with the help of a stopwatch.

9. In an automatic penetrometer, release the needle for exactly 5 seconds by pressing the start button.

10. Rotate the knob on the dial and note down the final reading from the dial as the penetration value in one-tenth of a millimeter.

11. Clean the needle with benzene after every test and allow it to dry.

12. Take at least 3 penetration values on the surface of the sample. (For a sample with penetration value > 225 mm, leave the needle in the sample and use another needle to take another reading so that the sample is not disturbed.)

Distance between two test points $\nless 10$ mm

Distance between the test point and side of the dish $\nless 10$ mm

13. Take out the average of the three values and note it as the penetration value of the sample.

14. If the penetration ratio is to be determined: Carry out the entire test procedure at 4°C . Tolerance of 0.1°C can be allowed.

In this test,

Take the total weight on the penetration needle = 200 ± 0.25 gram.

Increase the time of penetration to 60 seconds.

A14.7 Observation

In the penetration test of bitumen, as the needle of penetrometer is released, it penetrates the sample to some distance. The distance by which the needle penetrates in the bitumen sample is to be noted down for all the three samples.

Table A14.1 - A sample observation table for penetration test of bitumen

Test Temperature = oC

Penetrometer Dial Reading	Test1	Test2	Test3
Initial Reading (a)			
Final Reading (b)			
Penetration Value (b-a)			

Penetration Value of the Sample = Average Value of Penetration of Sample – 1, 2 & 3

Penetration Ratio:

Penetration at 4 °C – Wt. 200 gm – time 60s / Penetration at 25 °C – Wt. 100 gm – time 5s) x 100

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A14.8 Result

In the penetration test of bitumen, the average of the penetration values obtained from the three tests is the penetration value of the given bitumen specimen. The penetration values of the three readings should not differ from each other beyond certain limits. These limits are mentioned in the table below:

Table A14.2 –Limits of Differences in Penetration Value Readings

Penetration Values	Maximum Difference Between Readings
0 – 49	2
50 – 149	4
149 – 249	6
≥ 250	8

Minimum penetration values standardized by Bureau of Indian Standard for Paving Bitumen Type I for different grades are tabulated below:

Table A14.3 – Minimum Penetration (Ductility) Values for various grades of Paving Bitumen Type – I.

Grade of Bitumen Type - I	Ductility (in cm)
S – 35	30-40
S – 45	40-50
S – 55	50-60
S – 45	60-70
S – 90	80-100
S – 200	175-225

A14.9 Conclusion

By performing the above penetration test of bitumen, we can determine whether the given bitumen sample meets the specification requirements for penetration as given in the standards. Penetration value is one of the measures of the quality of bitumen along with other measures such as ductility value and so on. The penetration value found out by the penetration test of bitumen is compared to the standard value and thus it can be concluded whether the sample meets the penetration criteria.

Reference: Penetration Test of Bitumen, Civiconcepts

Web Link: <https://civiconcepts.com/blog/penetration-test-of-bitumen-procedure-result>

A15 Marshall Test of Bitumen

Standard Codes of Reference

This test refers to: **AASHTO T 245**, **ASTM D 6927 – 15** which are for the **Penetration** test of Bitumen.

A15.1 General

The mix design (wet mix) determines the optimum bitumen content. There are many methods available for mix design which vary in the size of the test specimen, compaction, and other test specifications. Marshall method of mix design is the most popular one and is discussed below, ref. ASTM D 6927.



Figure A15.1 – Marshall Test Setup

The Marshall stability and flow test provides the performance prediction measure for the Marshall mix design method. The stability portion of the test measures the maximum load supported by the test specimen at a loading rate of 50.8 **mm**/minute. Load is applied to the specimen till failure, and the maximum load is designated as stability. During the loading, an attached dial gauge measures the specimen's plastic flow (deformation) due to the loading. The flow value is recorded in 0.25 mm (0.01 inch) increments at the same time when the maximum load is recorded. The important steps involved in marshall mix design are summarized next.

A15.2 Specimen Preparation

Approximately 1200gm of aggregates and filler is heated to a temperature of 175 - 190°C. Bitumen is heated to a temperature of 121 - 125°C with the first trial percentage of bitumen (say 3.5 or 4% by weight of the mineral aggregates). The heated aggregates and bitumen are thoroughly mixed at a temperature of 154 - 160°C. The mix is placed in a preheated mould and compacted by a rammer with 50 blows on either side at temperature of 138°C to

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149°C. The weight of mixed aggregates taken for the preparation of the specimen may be suitably altered to obtain a compacted thickness of 63.5+/-3 mm. Vary the bitumen content in the next trial by +0.5% and repeat the above procedure. Number of trials are predetermined. The prepared mould is loaded in the Marshall test setup as shown in the figure A15.1.

A15.3 Properties of the Mix

The properties that are of interest include the theoretical specific gravity 'Gt', the bulk specific gravity of the mix 'Gm', percent air voids 'Vv', percent volume of bitumen 'Vb', percent void in mixed aggregate 'VMA' and percent voids filled with bitumen 'VFB'. These calculations are discussed next. To understand these calculation a phase diagram is given in Figure A15.2.

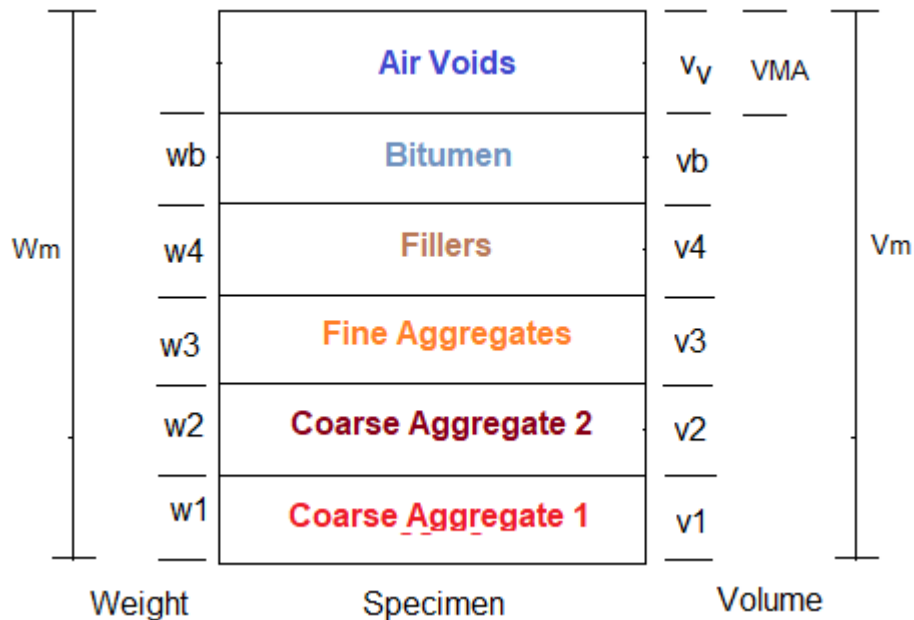


Figure A15.2 – Phase Diagram of a Bituminous Mix

A15.4 Theoretical Specific Gravity 'Gt' of the Mix

Theoretical specific gravity 'Gt' is the specific gravity without considering air voids, and is given by:

$$G_t = \frac{W_1 + W_2 + W_3 + W_b}{\frac{W_1}{G_1} + \frac{W_2}{G_2} + \frac{W_3}{G_3} + \frac{W_b}{G_b}} \text{-----}(1)$$

where, W1 is the weight of coarse aggregate in the total mix, W2 is the weight of fine aggregate in the total mix, W3 is the weight of filler in the total mix, 'Wb' is the weight of

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bitumen in the total mix, G_1 is the apparent specific gravity of coarse aggregate, G_2 is the apparent specific gravity of fine aggregate, G_3 is the apparent specific gravity of filler and G_b is the apparent specific gravity of bitumen,

A15.5 Bulk Specific Gravity ' G_m ' of Mix

The bulk specific gravity or the actual specific gravity of the mix G_m is the specific gravity considering air voids and is found out by:

$$G_m = \frac{W_m}{W_m - W_w} \text{-----}(2)$$

where, ' W_m ' is the weight of mix in air, ' W_w ' is the weight of mix in water, Note that ' $W_m - W_w$ ' gives the volume of the mix. Sometimes to get accurate bulk specific gravity, the specimen is coated with thin film of paraffin wax, when weight is taken in the water. This, however requires to consider the weight and volume of wax in the calculations.

A15.6 Air voids percent ' V_v '

Air voids ' V_v ' is the percent of air voids by volume in the specimen and is given by:

$$V_v = \frac{(G_t - G_m)100}{G_t} \text{-----}(3)$$

where ' G_t ' is the theoretical specific gravity of the mix, given by equation (1) and G_m is the bulk or actual specific gravity of the mix given by equation (2).

A15.7 Percent volume of bitumen ' V_b '

The volume of bitumen ' V_b ' is the percent of volume of bitumen to the total volume and given by:

$$V_b = \frac{\frac{W_b}{G_b}}{\frac{W_1 + W_2 + W_3 + W_b}{G_m}} \text{-----}(4)$$

where, W_1 is the weight of coarse aggregate in the total mix, W_2 is the weight of fine aggregate in the total mix, W_3 is the weight of filler in the total mix, W_b is the weight of bitumen in the total mix, G_b is the apparent specific gravity of bitumen, and G_m is the bulk specific gravity of mix given by equation (2).

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A15.8 Voids in Mineral Aggregate 'VMA'

Voids in mineral aggregate VMA is the volume of voids in the aggregates, and is the sum of air voids and volume of bitumen, and is calculated from:

$$VMA = V_v + V_b \text{ -----(5)}$$

where, 'Vv' is the percent air voids in the mix, given by equation (3). and 'Vb' is percent bitumen content in the mix, given by equation (4).

A15.9 Voids Filled with Bitumen 'VFB'

Voids filled with bitumen 'VFB' is the voids in the mineral aggregate frame work filled with the bitumen, and is calculated as:

$$VFB = \frac{V_b \times 100}{VMA} \text{ -----(6)}$$

where, 'Vb' is percent bitumen content in the mix, given by equation (4). and 'VMA' is the percent voids in the mineral aggregate, given by equation (5).

A15.10 Determine Marshall Stability and Flow

Marshall stability of a test specimen is the maximum load required to produce failure when the specimen is preheated to a prescribed temperature placed in a special test head and the load is applied at a constant strain (5 cm per minute). While the stability test is in progress dial gauge is used to measure the vertical deformation of the specimen. The deformation at the failure point expressed in units of 0.25 mm is called the Marshall flow value of the specimen.

A15.11 Apply Stability Correction

It is possible while making the specimen the thickness slightly vary from the standard specification of 63.5 mm. Therefore, measured stability values need to be corrected to those which would have been obtained if the specimens had been exactly 63.5 mm. This is done by multiplying each measured stability value by an appropriated correlation factors as given in Table below.

Table A15.1 – Factors for Volume Correction for Marshall Stability Values

Volume of Specimen (cm ³)	Thickness of Specimen (mm)	Correction Factor
457 - 470	57.1	1.19
471 - 482	68.7	1.14
483 - 495	60.3	1.09
496 - 508	61.9	1.04
509 - 522	63.5	1.00
523 - 535	65.1	0.96
536 - 546	66.7	0.93

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547 - 559	68.3	0.89
560 - 573	69.9	0.86

A15.12 Prepare graphical plots

The average value of the above properties are determined for each mix with different bitumen content and the following graphical plots are prepared:

Binder content versus corrected Marshall stability

Binder content versus Marshall flow

Binder content versus percentage of void (V_v) in the total mix

Binder content versus voids filled with bitumen (VFB)

Binder content versus unit weight or bulk specific gravity (G_m)

A15.13 Determine optimum bitumen content

Determine the optimum binder content for the mix design by taking average value of the following three bitumen contents found from the graphs obtained in the previous step.

1. Binder content corresponding to maximum stability
2. Binder content corresponding to maximum bulk specific gravity (G_m)
3. Binder content corresponding to the median of designed limits of percent air voids (V_v) in the total mix (i.e. 4%)

The stability value, flow value, and VFB are checked with Marshall mix design specification chart given in Table below. Mixes with very high stability value and low flow value are not desirable as the pavements constructed with such mixes are likely to develop cracks due to heavy moving loads.

Table A15.2 – Marshall Mix Design Specification

Test Property	Specified Value
Marshall stability,	340 kg (minimum)

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Flow value, 0.25 mm units	8 - 17
Percent air voids in the mix $V_v\%$	3 - 5
Voids filled with bitumen $VFB\%$	75 - 85

Reference:

Marshall mix design, Prof. Tom V. Mathew, Transportation Systems Engineering, IIT Bombay

Web Link: https://www.civil.iitb.ac.in/tvm/nptel/407_InTse/web/web.html

A16 Superpave

Standard Codes of Reference

The research described herein was supported by the Strategic Highway Research Program (SHRP). SHRP is a unit of the National Research Council. Ref. for Coarse aggregate angularity is ASTM - D 5821 and for fine aggregate is AASHTO - T304, for flat, elongated particles is ASTM - D 4971.

A16.1 General

Superpave procedure is considered as an improved method for producing a good performing asphaltic paving mix. It is a short name for Superior Performing Asphalt Pavement. It was developed in 1987 by the Strategic Highway Research Program (SHRP) researchers. It is an improved system for specifying asphalt binders and mineral aggregates, designing asphalt mixtures, and predicting pavement performance. In this method of mix design, the specifications are considered as a performance based rather than empirical as in Marshall method. The tests and analyses followed in this method have direct relationships to field performance. The binder tests measure the physical properties at temperatures and aging conditions that more realistically represents those encountered by in-service pavements.

The Superpave mixture design and analyses system uses three rigorous degrees of testing and analyses to produce a well performing mixture for a given pavement project, which include:

- Careful material selection and volumetric proportioning
- Intermediate mix testing and analyses in order to optimize the asphalt mixes based on estimate of pavement performance. (for critical, high traffic volume projects).
- Advanced (complete) mix testing and analyses in order to get better optimization of the asphalt mixes through sophisticated estimate of pavement performance (for more critical, higher traffic volume projects).

This handout describes the material selection and volumetric proportioning part of the Superpave mix design (volumetric design).

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A16.2 Basics and terms

• Objective of a mix design: The objective mix design is to select and proportion the mix materials to get an economical blend of a paving mix, having certain desired properties over the service life of the pavement: These desired properties include:

- Sufficient strength to resist traffic loading without permanent deformation.
- Sufficient flexibility to resist fatigue cracking.
- Good workability to enable proper lay down and compaction.
- Water resistive. Did not disintegrate or strip due to adverse effect of water.
- Durable - to have the original good properties over the service life without unacceptable aging or water induced damages.
- Enough surface friction properties to resist skid.

To attain these desired properties, the asphalt mix composition should have:

- Sufficient quantity of asphalt binder to resist the adverse effect of water, but not high enough that produce a weak strength.
- Sufficient voids in mineral aggregate (VMA) to have enough spaces (room) for asphalt to enable good aggregate coating and film thickness. This is to get good durability.
- Suitable air voids (AV) content. Enough air void content to allow for further compaction (densification) due to traffic but not so high that can result in permanent deformation.
- Asphalt Mixtures: Asphalt mixtures sometimes referred as hot mix asphalt (HMA). It is a paving material that consists of asphalt binder and mineral aggregate. The asphalt binder acts as a binding agent to glue aggregate particles into a dense mass and to waterproof the mixture.
- Important binder characteristics: Three binder characteristics are important in asphalt mixture performance: viscoelasticity, temperature susceptibility, and aging behavior.
- Bank-run aggregate: a natural aggregate simply mined from river deposits with or without further processing.
- Processed aggregate: A Quarried crushed aggregate separated into size fractions.
- Main Pavement distress types: The primary pavement distress types that Superpave consider include: Permanent deformation, fatigue cracking, and low temperature cracking.
- Nominal maximum size of aggregate: In Superpave it is defined as one sieve size larger than the first sieve that retain more than 10%.
- Maximum size of aggregate: One sieve size larger than the nominal maximum size.

A16.3 Levels of Superpave Mix Design

The design and analyses procedures to be used depend on the traffic level of the pavement project:

- Level 1 - Volumetric mix design , for low traffic pavements
- Level 2 - Volumetric mix design and intermediate performance prediction tests.
- Level3 - volumetric mix design and enhanced performance prediction tests.

Table A16.1 - Recommended Design Levels

Design level	ESAL
Level 1 - Volumetric	$\leq 10^6$
Level 2 – Volumetric and Intermediate Analysis	$\leq 10^7$
Level 3 – Volumetric and Complete Analysis	$> 10^7$

A16.4 Volumetric Mix Design Steps

The volumetric design consists of following main steps:

- Material selection
 - Asphalt Binder selection
 - Design aggregates Selection
- Selection of the design asphalt binder content
- Evaluation of the moisture sensitivity of the mix

A16.5 Material Selection

Superpave mix design process starts with asphalt binder and aggregate selection.

A16.5.1 Asphalt Binder selection

The binder is selected based on the climate and traffic in which they are intended to serve. The selected binder(PG) should achieve a certain required physical properties at a certain temperatures corresponding to the climate of the pavement site.

The Performance grade (PG) specifies the condition where the binder can be used. For example if the required binder is classified as a PG 64 – 22, it means that the binder must meet high temperature physical property requirements up to a temperature 64 °C and low temperature property requirements down to -22 °C. These temperatures are the high and low pavement temperatures (average maximum 7- day and minimum 1- day pavement temperatures respectively). Asphalt binder specifications consider permanent deformation, fatigue cracking, and low temperature cracking.

A16.5.2 Selection of Design Aggregates

Aggregate properties play a central role in overcoming permanent deformation. It have a slight effect on fatigue cracking and low temperature cracking. Aggregate properties were classified into two categories ; consensus properties and source properties.

Consensus Aggregate Properties

Certain aggregate characteristics were critical and needed to be achieved in all cases to a arrive at well performing HMA. Those properties are:

- Coarse aggregate angularity.
- Fine aggregate angularity.
- Flat, elongated particles.
- Clay content

Aggregate Angularity

The required minimum values for coarse and fine aggregate angularity are based on traffic level and position within the pavement. The test procedure for measuring coarse aggregate angularity is ASTM-D5821 and for fine aggregate is AASHTO-T304.

Coarse Aggregate Angularity

Coarse aggregate angularity is defined as the percent by weight of the aggregate particles larger than 4.75 mm with one or more fractured faces. A fractured face is defined as an angular, rough, or broken surface of an aggregate particle created by crushing, by other artificial means, or by nature. A face is considered fractured only if it has a projected area at least as large as one quarter of the maximum projected area (maximum cross sectional area) of the particle and also has sharp and well-defined edges.

Coarse aggregate angularity is measured on the coarse particles of the blended aggregate (specifically those retained on the 4.75 mm sieve) by Pennsylvania [Department of Transportation] Test Method No. 621, Determining the Percentage of Crushed Fragments in Gravel z. The results of this test are used in the laboratory during the Superpave mix design process, as well as in field control to monitor aggregate production.

Coarse aggregate angularity criteria for increasing levels of total traffic in equivalent single axle loads (ESALs) over the planned or estimated service life of the pavement are presented in table 2.6. These are the requirements for level 1, 2, and 3 mix designs.

Fine Aggregate Angularity

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Fine aggregate angularity is defined as the percent of air voids present in loosely compacted aggregate that passes the 2.36 mm sieve. Fine aggregate angularity is measured on the fine aggregate portion of the blended aggregate by AASHTO Standard Method of Test TP 33 (ASTM Standard Method of Test C1252), un-compacted Void Content of Fine Aggregate (as Influenced by Particle Shape, Surface Texture, and Grading). The results of this test are used in the laboratory during the Superpave mix design process, but not as a field control tool to monitor aggregate production. Fine aggregate angularity criteria for increasing levels of total traffic in ESALs over the planned or estimated service life of the pavement are presented in the table 2.7. These are the requirements for level 1, 2, and 3 mix designs.

Table A16.2 - Superpave Coarse and Fine Aggregate Requirements

Traffic ESAL in Millions	Angularity, Minimum %				Flat Elongated Particle Maximum%	Sand Equivalent Minimum%
	Coarse Aggregate		Fine Aggregate			
	Depth from Surface		Depth from Surface			
	< 100 mm	> 100 mm	< 100 mm	> 100 mm		
< 0.3	55/-	-/-	--	--	--	40
< 1	65/-	-/-	40	--	--	40
< 3	75/-	50/-	40	40	10	40
< 10	85/80	60/-	45	40	10	45
< 30	95/90	80/75	45	40	10	45
< 100	100/100	95/90	45	45	10	50
>= 100	100/100	100/100	45	45	10	50

Note ; 85/80 means 85% aggregate has one fractured face and 80% has two fractured faces. Criteria for fine aggregate are as percent air voids in loosely compacted fine aggregate.

Superpave mix design system contains specific performance-related characteristics (sections 2.1.1 through 2.1.9) to assist in selecting acceptable aggregate materials and developing a satisfactory aggregate blend for a given situation. Aggregate characteristics are specified as either agency standards or consensus standards. Except as noted in section 2.1.1, the aggregate criteria in this section apply to materials intended for 25 dense graded paving mixes as well as special purpose paving mixes such as stone matrix asphalt (SMA).

The Superpave mix design system accommodates coarse aggregate up to a maximum size (see definition in section 2.1.1) of 50.0 mm. Table 2.1 presents recommended maximum aggregate sizes for the design of paving mixes for base, binder, and surface courses. This table provides guidelines for situations where the maximum size is not already dictated by existing agency policy.

Table A16.3 - Suggested Maximum Aggregate Sizes

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The system	Pavement Layer	Nominal maximum Aggregate Size	SUPERPAVE" provides three levels of mix design. This permits the development of designs that will meet the demands of progressively higher levels of traffic, environmental severity and reliability. Materials selection criteria are recommended as either guidelines or requirements, depending upon the level of mix design being employed. A guideline is defined as a specified value which is provided as design guidance, but to which conformance is not mandatory. A requirement is a specified value which the material must meet.
	Surface	9.5 – 12.5 mm	
	Binder	25.0 – 37.5 mm	
	Base	25 – 37.5 mm	

Any sound, clean aggregate of local origin that meets these criteria may be chosen as a candidate material in the Superpave mix design system. There are no requirements for geologic type or composition other than the need to meet the optional asphalt binder compatibility guideline in section 2.2.3 and the moisture sensitivity requirements for mixtures presented in chapters 3, 4, and 5. If aggregates that meet the project requirements are unavailable locally, the cost of importing suitable materials must be carefully weighed against the penalties in long-term performance that will arise from the use of substandard materials.

Techniques for gradation analysis and for the proportioning of two or more aggregates to meet specific blended gradation requirements are beyond the scope of this manual. Appendix A, "Gradation Analysis of Aggregates", of Asphalt Institute Manual MS-2, Mix Design Methods for Asphalt Concrete, 1994 edition, gives complete, detailed instructions on these topics.

The following rules apply generally to aggregate gradation analysis and the development of aggregate blends in the Superpave mix design system:

- Washed sieve analysis of all fractions, including filler, must be conducted in order to ensure maximum accuracy in proportioning.
- The gradation of an aggregate or aggregate blend is specified on the basis of the total aggregate gradation, that is, on the total percent by weight passing the designated sieve sizes.
- Following the conventions stated in Asphalt Institute Manual MS-2, the individual fractions of the total aggregate gradation are designated as shown in table 2

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Table A16.4 - Definition of Aggregate Fractions

Coarse aggregate	Retained on the 2.36 mm sieve
Fine aggregate	Passing the 2.36 mm sieve
Mineral filler	Passing the 75 lam sieve

When aggregate materials are designated by the terms rock, sand, and filler, these terms are generally defined as shown in Table A16.5:

Table A16.5 - Definition of Aggregate Types

Rock	Predominately coarse aggregate (retained on the 2.36 mm sieve)
Sand	Predominately fine aggregate (passing the 2.36 mm sieve)
Filler	Predominately mineral dust (passing the 75 μ m sieve)

Gradation Control

The Superpave mix design system guides selection of an acceptable aggregate gradation for a dense graded paving mix by means of control points and a restricted zone. The control points and restricted zone are graphed on the Federal Highway Administration grading chart on which the percentage of aggregate passing a sieve size is plotted against the sieve opening size raised to the 0.45 power. The ASTM sieves specified for the SUPERPAVE system are presented in table 2.4.

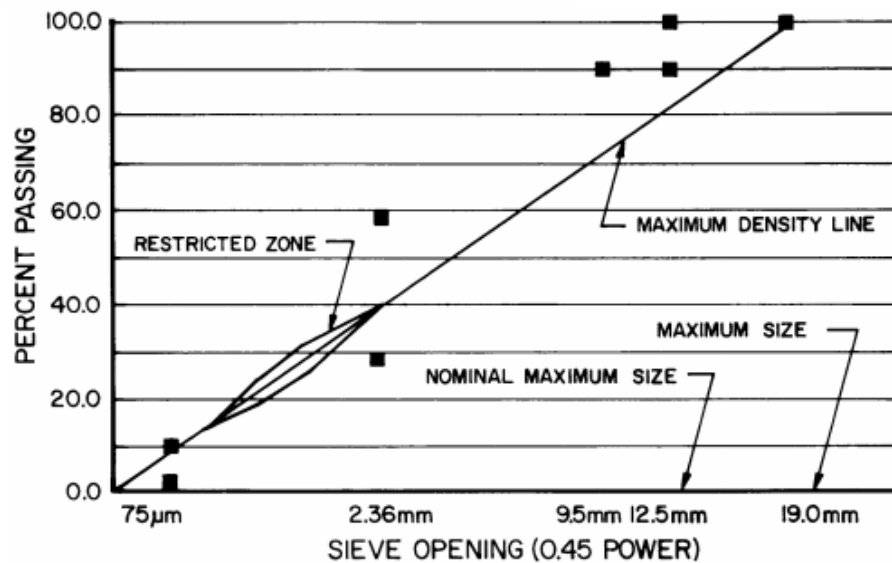


Figure A16.1 - **Superpave Gradation Control Points** and **Restricted Zone** for a 12.5 mm Nominal Maximum Size Aggregate Gradation.

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A typical example of gradation control is shown in figure 2.2 for a 12.5 mm nominal maximum aggregate size. This figure illustrates the following definitions:

- Nominal maximum size: one sieve size larger than the first sieve to retain more than 10 percent of the aggregate.
- Maximum size: one sieve size larger than the nominal maximum size.
- Maximum density line: a line drawn from the origin of the 0.45 power chart to the point at which the maximum sieve size intersects the 100 percent passing line.
- Restricted zone: a zone lying on the maximum density line and extending from the 300 μ m sieve to the 2.36 mm sieve through which it is usually undesirable for the gradation to pass. For 25 mm and 37.5 mm nominal maximum size gradations, the restricted zone extends to the 4.75 mm sieve.
- Control points: maximum and minimum limits established for each set of gradation controls. Table 2.4 illustrates the limits to which the blended aggregate gradation must conform for a 12.5 mm nominal maximum size mixture.

Table 2.4 illustrates the limits to which the blended aggregate gradation must conform for a 12.5 mm nominal maximum size mixture.

Table A16.6 - Aggregate Gradation Control Points

Seive Size	Control Point (Percent Passing)	
	Minimum	Maximum
75 μ m	2	10
2.36 mm	28	58
9.5mm	-	90
Nominal maximum (12.5 90 100 mm)	90	100
Maximum (19.0 mm)	100	-

Tables A16.7.1 through A16.7.6 in appendix A present the control points and restricted zones for gradations with nominal maximum sizes of 37.5 mm, 25.4 mm, 19.0 mm, 12.5 mm, and 9.5 mm.

Superpave Aggregate Gradation Control Points and Restricted Zones

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Table A16.7.1 - 37.5 mm Nominal Maximum Size

Seive Size	Control Point (Percent Passing)	
	Minimum	Maximum
75 µm	0	6
2.36 mm	15	41
25.0 mm	-	90
Nominal maximum (37.5 mm)	90	100
Maximum (50.0 mm)	100	-

Table A16.7.2 - 25.0 mm Nominal Maximum Size

Seive Size	Control Point (Percent Passing)	
	Minimum	Maximum
75 µm	1	7
2.36 mm	19	45
19.0 mm	-	90
Nominal maximum (25.0 mm)	90	100
Maximum (37.5 mm)	100	-

Table A16.7.3 - 19.0 mm Nominal Maximum Size

Seive Size	Control Point (Percent Passing)	
	Minimum	Maximum
75 µm	2	8
2.36 mm	23	49
12.5 mm	-	90
Nominal maximum (19.0 mm)	90	100
Maximum (25.0 mm)	100	-

Table A16.7.4 - 12.5 mm Nominal Maximum Size

Seive Size	Control Point (Percent Passing)	
	Minimum	Maximum
75 µm	2	10
2.36 mm	28	58
9.5 mm	-	90
Nominal maximum (12.5 mm)	90	100
Maximum (19.0 mm)	100	-

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Table A16.7.5 - 9.5 mm Nominal Maximum Size

Seive Size	Control Point (Percent Passing)	
	Minimum	Maximum
75 µm	2	10
2.36 mm	32	67
4.75 mm	-	90
Nominal maximum (9.5 mm)	90	100
Maximum (12.5 mm)	100	-

Table A16.7.6 – Boundaries of Aggregate Restricted Zone

Sieve Sizes Within Restricted Zones	Minimum and Maximum Boundaries of Sieve Size for Nominal Maximum Aggregate Size (Min/Max Percent Passing)				
	37.5 mm	25.0 mm	19.0 mm	12.5 mm	9.5 mm
4.75 mm	34.7/34.7	39.5/39.5	--	--	--
2.36 mm	23.3/27.3	26.8/30.8	34.6/34.6	39.1/39.1	47.2/47.2
1.18 mm	15.5/21.5	18.1/24.1	22.3/28.3	25.6/31.6	31.6/37.6
600 µ	11.7/15.7	13.6/17.6	16.7/20.7	19.1/23.1	23.5/27.5
300 µ	10.0/10.0	11.4/11.4	13.7/13.7	15.5/15.5	18.7/18.7

Table A16.8 - Common Sieves Sizes Used by State Highway Agencies

AASHTO Sieve Size and Designation	Approximate English Equivalent
63.0 mm	2½ in.
50.0 mm	2 in.
37.5 mm	1½ in.
25.0 mm	1 in.
19.0 mm	¾ in.
12.5 mm	½ in.
9.5 mm	¾ in.
4.75 mm	#4
2.36 mm	#8
1.18 mm	#16
600 µm	#30
300 µm	#50
150 µm	#100
75 µm	#200

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In general, it is recommended (but not required) that as the traffic level increases, the aggregate gradation move toward the minimum control points, below the restricted zone. Gradations that pass above or below the restricted zone, but within the relevant control points, should produce acceptable mixtures in the Superpave mix design system. No guideline or requirement is given for selecting the appropriate nominal maximum size. Specifying agencies may select or specify a nominal maximum size for surface, binder, and base course mixtures according to established policy, their past experience, or by reference to the suggested nominal maximum sizes.

Gradation control requirements in this section do not apply to special-purpose paving mixes such as SMA or porous asphalt. Past experience and engineering judgment should be used to develop specific gradation controls for the design and field control of special purpose paving mixes.

A16.6 Flat, Elongated Particles

The required maximum value for flat, elongated particles in coarse aggregate is a function of traffic level. Elongated particles are undesirable because they have the tendency to break during construction and under traffic. The test procedure for flat, elongated particles is ASTM-D4971. See last table.

A16.7 Clay Content

Clay content is the percentage of clay material contained in the aggregate fraction that is finer than sieve 4.75 mm. It is measured by AASHTO-T176 by use of the sand equivalent test. See last table.

A16.8 Source Aggregate Properties

Other aggregate characteristics were critical. However, critical values of these properties could not be reached by consensus because needed values were source specific. Specified values are established by local agencies. Those properties are:

- Toughness: Measured by AASHTO-T97 by use of Los Angeles machine.
- Soundness: Measured by AASHTO-T104 using sodium sulphate or magnesium sulphate.
- Deleterious Materials: Clay lumps and friable particles in aggregate measured by AASHTO-T112.

A16.9 Standard Test Method for Super Pave Mix Design

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PURPOSE: The purpose of the Superpave mix design procedure is to determine the design asphalt content for the asphalt mix that meets the criteria of Superpave.

A 16.9.1 Test Equipment

The primary device used in Superpave mix design is the Superpave Gyratory Compactor (SGC). The SGC is used to produce specimens for volumetric analysis and records data of specimen density throughout the test.

A 16.9.2 Additional Test Equipment

Ancillary test equipment required in the preparation of Superpave asphalt mixtures includes:

1. Ovens, thermostatically-controlled, for heating the aggregates, asphalt, and equipment.
2. Mechanical Mixer: commercial bread dough mixer 10 liter (10 qt.) capacity or larger, equipped with two metal mixing bowls and two wire whips.
3. Flat-bottom metal pans for heating aggregates.
4. Round metal pans, approximately 10 liter (10-qt.) capacity, for mixing asphalt and aggregate.
5. Scoop for batching aggregates.
6. Containers: Gill-type tins, beakers, or pouring pots, for heating asphalt.
7. Thermometers: armored, glass, or dial-type with metal stem, 10°C to 235°C, for determining temperature of aggregates, asphalt and asphalt mixtures.
8. Balances: 10-kg capacity, sensitive to 1 g, for weighing aggregates and asphalt; 10-kg capacity, sensitive to 0.1 g, for weighing compacted specimens.
9. Large Mixing Spoon or small trowel.
10. Large spatula.
11. Welders gloves for handling hot equipment.
12. Paint, markers, or crayons, for identifying test specimens.
13. Paper disks, 150 mm, for compaction.
14. Fans for cooling compacted specimens.
15. Computer or printer for data collection and recording

A16.9.3 Specimen Preparation

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- Determine the mixing and compaction temperature using a plot of viscosity versus temperature.
- Select mixing and compaction temperature corresponding to binder viscosity of 0.17 ± 0.02 Pa.S and 0.28 ± 0.03 respectively.
- Specimen used for volumetric determination only, that requires approximately 4500 grams of mixture, to arrive at a specimen 115 ± 5 mm height.
- Specimen used for performance testing, that requires approximately 5500 grams of mixture to fabricate a specimen that as 135 mm in height approximately.
- At least one loose sample should remain un-compacted to obtain a maximum theoretical specific gravity using AASHTO-T209.
- For performing moisture sensitivity test (AASHTO - T283). Test specimen are fabricated to a height of 95 mm which requires approximately 3500 grams of mixture.
- Place the pan containing the aggregate in an oven set approximately 15 °C higher than the mixing temperature. Two to four hours are required for the aggregates to reach the mixing temperature, heat all mixing implements.
- Heat the asphalt binder to the desired mixing temperature.

A16.9.4 Preparation of Mixtures

- Place the hot mix bowl on a scale and zero the scale.
- Charge the mixing bowl with the heater aggregate.
- From a crater in the blended aggregate and weight the required asphalt in to the mixture to achieve the desired batch weight.
- Mix the asphalt and aggregate using a mechanical mixer until the aggregate is thoroughly coated.
- Place the mix in a flat shallow pan at an even thickness of 21-22 Kg/m² and place the pan in the forced draft oven at 135 °C. Short term aggregate the specimen for 4 hours.

A16.9.5 Compaction of Volumetric Specimen

- The compactor is initiated by turning on the main power. The vertical pressure should be set at 600 kpa (± 18 Kpa). The gyration counter should be zeroed and set to stop when the desired number of gyrations is achieved. Three gyration levels are of interest:

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- **Design number of gyration** (Ndes)
- **Initial number of gyration** (Nini)
- **Maximum number of gyration** (Nmax)

Table A16.9 - The range of values for Nini, Ndes, Nmax are shown

Superpave Design Gyrotory Compactive Efforts												
Design ESAL (Millions)	Average Design High Air Temperature											
	< 39 °C			39-40 °C			41-42 °C			43-44 °C		
	Nini	Ndes	Nmax	Nini	Ndes	Nmax	Nini	Ndes	Nmax	Nini	Ndes	Nmax
< 0.3	7	68	104	7	74	114	7	78	121	7	82	127
0.3 - 1	7	76	117	7	83	129	7	88	138	8	93	146
1-3	7	96	134	8	95	150	8	100	158	8	105	167
3-10	8	96	152	8	106	169	8	113	181	9	119	192
10-30	8	109	174	9	121	195	9	128	208	9	135	220
30-100	9	126	204	9	139	228	9	146	240	10	153	253
> 100	9	142	233	10	158	262	10	165	275	10	172	288

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- Prepare the compaction while mixture specimen are short term aging. This includes verifying that the compaction pressure. The compaction angle and number of gyration are set to their proper values and setting the desired number of gyrations N_{max} , ensure that the data acquisition device is functioning.
- Approximately 45 - 60 minutes before compaction of the first specimen place the compaction mold and base/top plates in oven set at the compaction temperature.
- After four hours of short term aging remove the mixture from the oven. If the compaction temperature is greater than 135 °C, place the mixture in another oven for no longer than 30 minutes until it reaches the desired compaction temperature. If the compaction temperature is less than 135 °C place the mixture at room temperature until the compaction temperature is reached.
- Remove the mold and base plate from the oven. Place the base plates in the mold and place a paper disk on top of the base plate.
- Charge the mold with the aged asphalt mixture in a single lift. The top of the uncompacted specimen should be slightly rounded.
- Place a paper disk on top of the mixture.
- The mold is placed in the compactor and centered under the room, then start the gyration system.
- When N_{max} has been reached the compactor automatically slopes, angle and pressure are released.
- Remove the mold containing compacted specimen.
- Cool the mold for a suitable period for 5 to 10 minutes will facilitate specimen removal without undue distortion.
- Extrude the specimen from mold.
- Remove the paper disk from top and bottom of the specimen and allow the specimen to cool undisturbed.
- Identify specimen with a suitable marker.

A16.9.6 Data Analysis and Presentation

Superpave gyratory compaction data is analyzed by computing the estimated bulk specific gravity, corrected bulk specific gravity, and corrected percentage of maximum theoretical specific gravity for each desired gyration. During compaction, the height is measured and recorded after each gyration.

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G_{mb} of the compacted specimen and G_{mm} of the loose mixture are measured. An estimate of G_{mb} at any value of gyration is made by dividing the mass of the mixture by the volume of the compaction mold:

$$G_{mb}(\text{estimated}) = \frac{W_m / V_{mx}}{\gamma_w}$$

where,

G_{mb} (estimated) = estimated bulk specific gravity of specimen during compaction

W_m = mass of specimen, grams

γ_w = density of water = 1 g/cm³

V_{mx} = volume of compaction mold (cm³) calculated using the equation:

$$V_{mx} = \frac{\pi d^2 h_x}{4} * 0.001 \text{ cm}^3 / \text{mm}^3 = 17.6715 h_x$$

where, d = diameter of mold (150 mm), and h_x = height of specimen in mold during compaction (mm), π = 3.1416

This calculation assumes that the specimen is a smooth-sided cylinder, which it is not. Surface irregularities cause the volume of the specimen to be slightly less than the volume of a smooth-sided cylinder. Therefore, the final estimated G_{mb} at N_{max} is different than the measured G_{mb} at N_{max}. Therefore, the estimated G_{mb} is corrected by a ratio of the measured to estimated bulk specific gravity:

$$C = \frac{G_{mb}(\text{measured})}{G_{mb}(\text{estimated})}$$

where,

C = correction factor,

G_{mb}(measured) = measured bulk specific gravity after N_{max},

G_{mb}(estimated) -estimated bulk specific gravity at N_{max}.

The estimated G_{mb} at any other gyration level is then determined using:

G_{mb} (corrected) = C x G_{mb}(estimated)

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where,

Gmb(corrected) = corrected bulk specific gravity of the specimen at any gyration.

C = correction factor, and

Gmb(estimated) = estimated bulk specific gravity at any gyration.

Percent Gmm at any gyration level is then calculated as the ratio of Gmb (corrected) to Gmm (measured), and the average percent Gmm values for the two companion specimens are calculated.

Using the Nmax, Ndes, and Nini gyrations levels previously determined from design traffic level and the design high air temperature for the particular project (Table 5.1), Superpave volumetric mix design criteria (VMA, VFA, and dust ratio) are established on a four percent air void content at Ndes. Superpave mix design also specifies criteria for the mixture density at Nini, Ndes, and Nmax.

The percentage of air voids at Ndes is determined from the equation:

$$V_a = 100 - \% \text{ Gmm @ } N_{des}$$

Va = air voids @ Ndes, percent of total volume

% Gmm @ Ndes = maximum theoretical specific gravity @ Ndes percent.

The percent voids in the mineral aggregate is calculated using:

$$\%VMA = 100 - \left(\frac{\%G_{mm} @ N_{des} * G_{mm} * P_s}{G_{sb}} \right)$$

where,

VMA = voids in mineral aggregate, percent of bulk volume

%Gmm@ Ndes = maximum theoretical specific gravity @ Ndes, percent

Gmm = maximum theoretical specific gravity

Gsb = bulk specific gravity of total aggregate

Ps = aggregate content, cm³ /cm³ , by total mass of mixture

If the percentage of air voids is equal to four percent, then this data is compared to the volumetric criteria and an analysis of this blend completed. However, if the air void content at Ndes varies from four percent (and this will typically be the case), an estimated design asphalt content to achieve 4 percent air voids at Ndes is determined, and the estimated design properties at this estimated design asphalt content are calculated. The estimated asphalt content at Ndes = 4% air voids is calculated using this equation:

$$P_{b, \text{ estimated}} = P_{bi} - (0.4 \times (4 - V_a))$$

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Where,

Pb,estimated = estimated asphalt content, percent by mass of mixture

Fbi = initial (trial) asphalt content, percent by mass of mixture

Va = percent air voids at Ndes (trial)

The volumetrics (VMA and VFA) at Ndes and mixture density at Nini and Nmax are then estimated at this asphalt binder content using the equations that follow.

For VMA:

$$\% \text{VMA}_{\text{estimated}} = \% \text{VMA}_{\text{initial}} + C \times (4 - V_a)$$

where:

$\% \text{VMA}_{\text{initial}}$ = % VMA from trial asphalt binder content

C = constant = 0.1 if V_a is less than 4.0 percent

= 0.2 if V_a is greater than 4.0 percent

Specified minimum values for VMA at the design air void content of four percent are a function of nominal maximum aggregate size and are given in Table A16.10.

Table A16.10 - VMA Criteria

Nominal Maximum Aggregate Size	Minimum VMA, percent
9.5 mm	15.0
12.5 mm	14.0
19.0 mm	13.0
25.0 mm	12.0
25.0 mm	11.0

For VFA:

$$\% \text{VFA}_{\text{estimated}} = 100 * \frac{(\% \text{VMA}_{\text{estimated}} - 4.0)}{\% \text{VMA}_{\text{estimated}}}$$

The acceptable range of design VFA at four percent air voids as a function of traffic level is shown in Table A16.11.

Table A16.11 - VFA Criteria

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Traffic in Millions of ESAL	Design VFA, percent
< 0.3	70-80
< 1	65-78
< 3	65-78
< 10	65-75
< 30	65-75
< 100	65-75
>= 100	65-75

For %Gmm at Nini:

%Gmm estimated @ Nini = %Gmm trial @ Nini -(4.0 – Va)

The maximum allowable mixture density at Nini is 89 percent.

For %Gmm at Nmax:

%Gmm estimated @ Nmax = %Gmm trial @ Nmax -(4.0 – Va)

The maximum allowable mixture density at Nmax is 98 percent.

Finally, there is a requirement for the dust proportion. It is calculated as the percent by mass of the material passing the 0.075 mm sieve (by wet sieve analysis) divided by the effective asphalt binder content (expressed as percent by mass of mix). The effective asphalt binder content is calculated using:

$$P_{be} = - (P_s * G_b) * \left(\frac{G_{sc} - G_{sb}}{G_{sc} * G_{sb}} \right) + P_{b, estimated}$$

Where,

Pbe = effective asphalt content, percent by total mass of mixture

Ps = aggregate content, percent by total mass of mixture

Gb = specific gravity of asphalt.

Gse = effective specific gravity of aggregate

Gsb = bulk specific gravity of aggregate

Pb = asphalt content, percent by total mass of mixture

Dust Proportion is calculated using:

$$DP = \frac{P_{0.075}}{P_{be}}$$

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Where,

P_{0.075} = aggregate content passing the 0.075 mm sieve, percent by mass of aggregate

P_{be} = effective asphalt content, percent by total mass of mixture

An acceptable dust proportion ranges from 0.6 to 1.2 for all mixtures.

After establishing all the estimated mixture properties, the designer can look at the trial blends and decide if one or more are acceptable, or if further trial blends need evaluation.

A16.9.7 Selection of the Design Asphalt Binder Content

After the selecting the binder grade from the environmental and weather station data:

- Evaluate an initial design asphalt trial content for each gradation.
- The trial asphalt binder content is determined for each trial blend by estimating the effective specific gravity of the blend and using the calculation.
- The effective specific gravity (G_{se}) of the blend is estimated by:

$$G_{se} = G_{sb} + 0.8 * (G_{sa} - G_{sb})$$

G_{se} = effective specific gravity of blend (combined aggregate)

G_{sb} = bulk specific gravity of blend (combined aggregate).

- The volume of asphalt binder (V_{ba}) absorbed in to aggregate is estimated using this equation:

$$V_{ba} = \frac{P_s * (1 - V_a)}{\left(\frac{P_b}{G_b} + \frac{P_s}{G_{se}} \right)} * \left(\frac{1}{G_{sb}} + \frac{1}{G_{se}} \right)$$

V_{ba} = volume of absorbed binder, cm³ /cm³ of mix

P_b = percent of binder (assumed 0.05)

P_s = percent of aggregate (assumed 0.05)

G_b = specific gravity of binder (assumed 1.02)

- The volume of the effective binder (V_{be}) can be determined from this equation:

$$V_{be} = 0.081 - 0.02931 * [\ln (S_n)] = \text{cm}^3 / \text{cm}^3 \text{ of mix.}$$

where S_n = the nominal maximum sieve size of the aggregate blend (in inches).

- Finally, the initial trial asphalt binder (P_{bi}) is calculated from this equation:

$$P_{bi} = \frac{G_b * (V_{be} + V_{ba})}{[G_b * (V_{be} + V_{ba})] + W_s}$$

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Where,

Pbi = percent (by weight of mix) of binder.

Ws = weight of aggregate, gms.

Reference:

Superpave Mix Design, Pavement Interactive, Strategic Highway Research Program (SHRP), Asphalt Institute. *Superpave Mix Design*. Superpave Series No. 2 (SP-02). Asphalt Institute. Lexington, KY

Web Link:

<https://pavementinteractive.org/reference-desk/design/mix-design/superpave-mix-design/>

A17 Polymer Modified Bitumen (PMB)

Standard Codes of Reference

This test refers to: AASHTO M 316, AASHTO M 208, AASHTO T 240, ASTM D 2397, ASTM D 4402, ASTM D 977-98, IS 15462 : 2019, for the Polymer Modified Bitumen.

A17.1 General The interest in the modification of bitumen using polymers, whether virgin, scrap or polymer blends, is intense. The last two decades, in particular, have seen an increase in the number of academic groups studying polymer-modified bitumen and correspondingly the peer-reviewed literature in the field has increased. Initially, studies on **polymer modified bitumen** focused more on engineering and empirical measurements, e.g. ageing and softening point. However, in recent years a plethora of techniques have been employed in the study of the effect of the addition of polymers on a range of bitumen properties, polymer—bitumen morphology and polymer—bitumen interactions.

Polymer Modified Bitumen (PMB) is one of the specially designed and engineered bitumen grades that are used in making pavement, roads for heavy duty traffic and home roofing solutions to withstand extreme weather conditions. PMB is a normal bitumen with the added polymer, which gives it extra strength, high cohesiveness and resistance to fatigue, stripping and deformations, making it a favorable material for infrastructure.

Pavements designed and constructed for heavy-duty traffic and extreme weather conditions require specially designed engineered Bitumen Grades. By changing the characteristics of normal bitumen with the addition of a polymer, either they are of elastomeric nature or elastomeric, we succeed to obtain bitumen that allows the mixture to be more cohesive, with much more strength and significant higher resistance to parameters like fatigue and permanent deformations for road pavements.

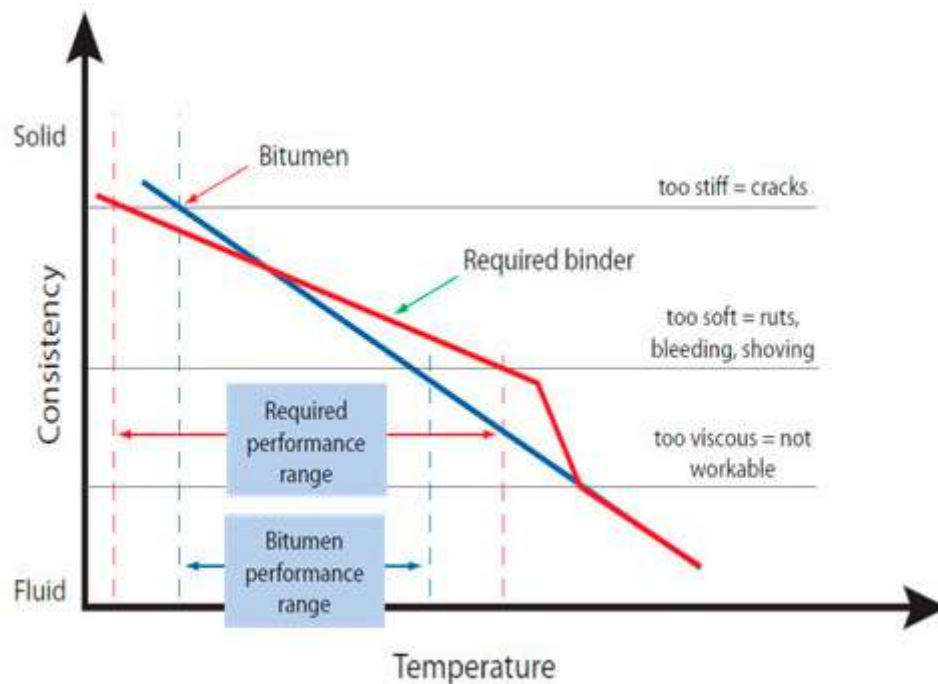


Figure A17.1 – Modified Consistency in Extended Temperature Range

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When a polymer is added to regular bitumen, it becomes more elastomeric, which provides it with additional elasticity. The polymer that is added is styrene butadiene styrene (SBS), which acts as a binder modification agent. The primary objective of SBS polymer modified bitumen is to provide extra life to pavement, roads and construction designs. Some of the qualities exhibited by PMB are:

- Higher rigidity
- Increased resistance to deformation
- Increased resistance to cracks and stripping
- Better water resistance properties
- High durability

A17.2 Advantage of using polymer modified bitumen

- stronger road with increased Marshall Stability value and greater Rigidity.
- better resistant towards rainwater and water stagnation.
- no stripping and no potholes.
- Better resistance to permanent deformation
- reduction in pores in aggregate and hence less rutting and raveling.
- Much higher durability

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Table 17.1 - General Purpose of Use

Type	General Purpose of Use	Generic Examples
Filler	Fill voids and therefore reduce optimum asphalt content Meet aggregate gradation specifications Increase stability Improve the asphalt cement-aggregate bond.	Mineral filler crusher fines lime Portland cement fly ash Carbon black.
Extender	Substituted for a portion of asphalt cement (typically between 20–35 % by weight of total asphalt binder) to decrease the amount of asphalt cement required.	Sulfur Lignin
Rubber	Increase HMA stiffness at high service temperatures increase HMA elasticity at medium service temperatures to resist fatigue cracking Decrease HMA stiffness at low temperatures to resist thermal cracking (see Figure).	Natural Latex Synthetic latex (e.g., Polychloroprene latex) Block copolymer (e.g., Styrene-butadiene-styrene (SBS)) Reclaimed rubber (e.g., crumb rubber from old tires)
Plastic		Polyethylene/polypropylene Ethylene acrylate copolymer Ethyl-vinyl-acetate (EVA) Polyvinyl chloride (PVC) Ethylene propylene or EPDM Polyolefin
Rubber-Plastic Combinations		Blends of rubber and plastic
Fiber	Improving tensile strength of HMA Mixtures Improving cohesion of HMA Mixtures Permit higher asphalt content without the significant increase in the drain down.	Natural: Asbestos Rock wool Manufactured: Polypropylene Polyester Fiberglass Mineral Cellulose.
Oxidant	Increase HMA stiffness after the HMA is placed	Manganese salts
Antioxidant	Increase the durability of HMA mixtures by retarding their oxidation	Lead compounds Carbon, Calcium salts
Hydrocarbon	Restore aged asphalt cement to current specifications increase HMA stiffness in general	Recycling and rejuvenating oils Hard and natural asphalts
Antistripping Agents	Minimize stripping of asphalt cement from aggregates	Amines, Lime
Waste Materials	Replace aggregate or asphalt volume with a cheaper waste product	Roofing shingles, Recycled Tires, Glass

A17.3 Use as Needed

While the benefits of using modified asphalts are widely acknowledged, not all asphalt mixes or treatments need to be modified. Each application should be evaluated to determine if the traffic loading, anticipated service life, environmental conditions and desired performance justify the use of modifiers. Modified asphalts can be a good investment.

- The rheological properties of conventional binders may be modified by the introduction of:
 - Elastomers;
 - Plastomers;
 - Crumb rubber;
- The modification is costly and is normally justified when bituminous surfacing are subjected to severe conditions such as:
 - Steep gradients;
 - Very high road surface temperature;
 - High traffic loading; or
 - Heavily trafficked intersections.
- Modification may also be advantageous for surfacing on highly flexible and cracked pavements, where an improvement in the rheological properties of the bitumen is required.
- Use in such applications should be guided by expert opinion.
- In addition to the primary aims above, the range of properties improved include
 - Durability;
 - Aggregate retention;
 - Resistance to permanent deformation;
 - Resistance to fatigue cracking;
 - Cohesion (internal strength);
 - Elasticity;
 - Viscosity less susceptible to temperature changes.
 - Modification agents
- The primary aim of the modification of bitumen for use in structural layers is to increase the resistance of these layers to permanent deformation at high road temperatures without compromising the properties of these layers over the rest of the prevailing temperature range.
- The use of polymer modified bitumen to obtain improved performance is rising as a result of increases in tire pressures, axle loads, and higher traffic volumes.
- Improved performance can be achieved in two ways, both of which are aimed at reducing the permanent strain:
 - An increase in the elastic component with an associated reduction in the viscous component; and

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- Stiffening of the bitumen to reduce the total viscoelastic response of the layer.

The modification is achieved by the introduction of polymers (including crumb rubber), aliphatic synthetic wax or naturally occurring hydrocarbons.

Polymers can be broadly categorized as “elastomers” (sometimes referred to as thermoplastic elastomers) for improving the strength and elastic properties of a binder, and “plastomers” (sometimes referred to as thermoplastic polymers) for increasing the viscosity of the bitumen.

Reference:

Polymer Modified Bitumen, Raha Bitumen

Web Link: <http://rahabitumen.com/polymer-modified-bitumen/>

A18 Viscosity Grade Bitumen

Standard Codes of Reference

This test refers to: **AASHTO M226-80 (2008)**, **ASTM Standard D3381-09**, **IS 73**, which are for the Viscosity grade of Bitumen.

A18.1 General

Viscosity grade bitumen in four different types including VG10, VG20, VG30 and VG40 is the most widely used bitumen in India, which has been replaced by bitumen penetration grade according to the Standard (IS:73). This kind of bitumen is more reliable than others due to its advantages and properties. In the following, we will tell you more about the applications, descriptions and specification of each grades, and you will find the best VG bitumen that you need.

A18.2 Difference between Penetration grade and Viscosity grade bitumen

The difference between penetration grade bitumen and viscosity grade bitumen is that the penetration grade classification is based on the Penetration value in 25 °C, while VG system is based on Viscosity value in both 60 °C and 135 °C that shows it is suitable for both low and high temperature so the bitumen behaviour in different thermal conditions can be predicted well.

A18.3 Classification of Viscosity Grade Bitumen

Viscosity test on bitumen results the classification of bitumen in a range of grades, including VG10 Bitumen, VG20 Bitumen, VG30 Bitumen, and VG40 Bitumen. The higher the number, the harder the bitumen in the viscosity grading system. To know more about VG's specification and comparison of the properties of grades, see the table at the end of the article.

A18.4 Viscosity grade Standards

Viscosity Grade bitumen is determined by IS 73, ASTM and AASHTO. According to the IS, grading of Bitumen is changed from penetration grade to viscosity grade in 2006.

Read the article to understand the viscosity grade bitumen definition, applications and specifications.

A18.5 Application of viscosity grade bitumen

Generally, each of these viscosity grade bitumen has different application according to the two important factors such as climate and traffic conditions. The important use of VG bitumen is due to the existence of two important properties of this material which are water impermeability and high adhesion. Today, more than 250 different applications for VG bitumen have been considered that it seems to be impossible to replace them with other materials. In the table you can see different applications and properties of VG bitumen.

Table A17.1 - Specifications of various Viscosity Grades of Bitumen

Characteristics	VG-10	VG-20	VG-30	VG-40
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Viscosity Range at 60 C, Poise, (Absolute Viscosity)	800-1200	1600-2400	2400-3600	3200-4000
Kinematic Viscosity, 135 C, CST, Minimum	250	300	350	400
Flash Point, C, Minimum	220	220	220	220
Solubility in trichloroethylene, %, minimum	99	99	99	99
Penetration Value of vg40, at 25° C	80-100	60	50-70	40-60
Softening point, °C, min	40	45	47	25
Tests on residue from Infinity Galaxy thin film over test (RTFOT)				
Viscosity ratio at 60 °C, Max	4	4	4	4
Ductility at 25° C, Cm, Min, after thin film over test	75	50	40	25
Specific Gravity at 25 °C			1.0131	

A18.6 Typical Applications

VG – 10: Recommended for road construction in cold regions. Ideal for spraying, used in production of bitumen emulsion

VG – 20: Used in cold regions for road construction.

VG – 30: Maximizes asphalt performance in hot regions. A more reliable choice than bitumen penetration grade 60/70.

VG – 40: Works in areas with high traffic loads. Well suited for road construction in hot weather.

Reference:

Infinity Galaxy, Viscosity Grade Bitumen

Web Link: <https://infinitygalaxy.org/viscosity-bitumen/#viscosityvspenetration>

A19 Viscosity Test of Bitumen

Standard Codes of Reference

This test refers to: AASHTO M 316, AASHTO M 208, AASHTO T 240, ASTM D 2397, ASTM D 4402, ASTM D 977-98, IS 15462: 2019, for the Viscosity test of Bitumen.

A19.1 General

Viscosity test of Bitumen is carried out to determine the viscosity of bitumen specimen indirectly with the help of different viscometers available.

The procedure of determining the viscosity of bitumen along with apparatus, materials, and precautions is described below. The significance and conclusion of the viscosity test of bitumen are also described below. Different types of viscometers are used to determine different kinds of viscosity as discussed below. The types viscosity measured are:

- Industrial viscosity
- Dynamic/Absolute viscosity
- Kinematic viscosity

Viscosity test of the bitumen sample is one of the important **tests on bitumen** to be conducted before road construction. Viscosity measures the degree of fluidity of the bitumen sample. It ensures the quality of the bitumen used as a binder by giving a measure of fluidity at a particular temperature. If the bituminous binder used in the pavement has much lower viscosity value, then it will act as a lubricant only. It will not help in the binding of the particles which is its primary purpose.

A highly viscous binder will restrict the flow, thus it restricts the ability of binder to spread and fill up the voids between the aggregates during pavement construction. Also, it will require more efforts comparatively, and there is also a possibility of it forming a heterogeneous mix. Workability will also be affected of a highly viscous bitumen specimen. Therefore, it is essential to select a binder with appropriate viscosity so that it can form a uniform coat and fill up the voids between aggregates effectively.

A19.2 Industrial Viscosity Test of Bitumen

It is the property of a bitumen, road tar, and cutback bitumen by virtue of which it is able to resist the flow caused by internal friction. It is measured by determining the time taken by 50 cm³ volume of material to flow from a cup through a specified orifice in standard conditions of temperature and a specified temperature.

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A19.2.1 Apparatus

1. **Tar viscometer** is used for determining industrial viscosity of bitumen whose parts are discussed below:



Figure A19.1 – Tar Viscometer

2. **Cup:** It is also known as a 10-mm cup and has a specified orifice and valve. It is made up of hard brass tube. It has an external brass collar at the upper open end of the cylindrical cup. The collar helps in supporting the cup in the sleeve of the water bath. While a phosphor-bronze plate which is circular in shape is screwed into the cylinder and made conical. It has centrally-located extensions made up of same material and cylindrical in shape. The plate helps in the drainage of tar after use.



Figure A19.2 – Tar Viscometer Cup

The extension is drilled and polished to form an orifice of 10 mm diameter. If the upper rim of the orifice is not perfectly circular, then the valve cannot be properly seated.

Internal Diameter of cup = 40.0 ± 0.5 mm, External Diameter of cup = 42.0 ± 0.5 mm, Diameter of orifice = 10.000 ± 0.025 mm, Length of jet = 5.000 ± 0.025 mm

For **Cutback Bitumen**, 4-mm cup is used instead of 10-mm. and the diameter of the orifice is reduced to 4.000 ± 0.025 mm.

3. **Water bath:** It is cylindrical in shape and is made of a copper sheet. Heating of the bath is done electrically. Ensure that local heating is avoided. The water bath is placed on three equidistant legs riveted to the wall of the bath. The legs are of such length that a 100-mm cylinder can be placed below the cup orifice.

Diameter of water bath = 160 mm

Depth of water bath = 105 mm

4. **Valve:** It is in the shape of a sphere and is made up of phosphor-bronze. It is attached to a metal rod provided with a levelling peg at upper part and hemisphere by which the valve is held. It helps in closing the orifice attached to the 10-mm cup.

5. **Sleeve:** It is in the form of a stout brass tube, which is bronzed into a central hole cut in the bottom of the water bath. It helps in receiving the cup so that the cup remains in position. It has an easing sliding fit.

Internal Diameter of sleeve = 45 mm

Height of sleeve = 105 mm

6. **Stirrer:** It has 4 vertical vanes. The upper and lower portions of the stirrer can be turned in opposite directions. The stirrer is mounted on a cylinder slipping on the sleeve through an easy sliding fit.

The cylinder is cut between the vanes so that heat can be transferred from water in the bath to the tar in the cup. To prevent the water from the water bath from entering the tar cup on raising the stirring system, vertical grooves are provided on the inner surface of the cylinder.

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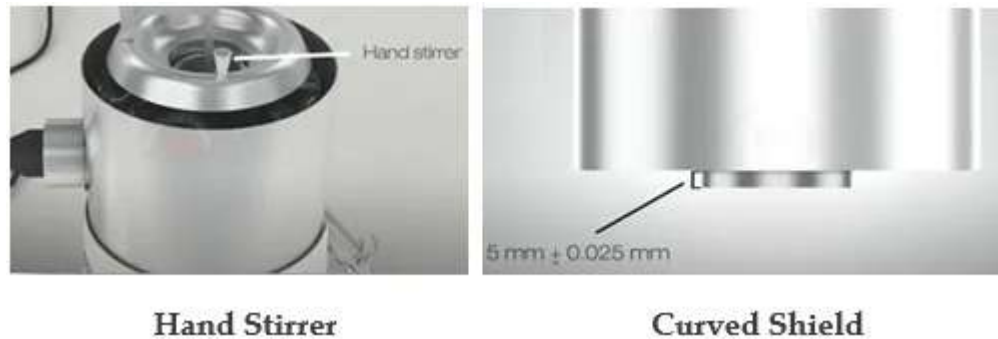


Figure A19.3 – Tar Viscometer

7. **Curved shield:** It is fixed at the upper edge of the cylinder. It is extended to a distance within 5 mm of the walls of the water bath. An insulating handle to facilitate rotation of stirrer, a swivelled support for the valve, and support for thermometer are present on this curved shield.

8. **Receiver:** It is a measuring cylinder with graduations at 20, 25, and 75 ml capacities. Its total capacity is 100 ml. Internal Diameter \geq 29 mm

9. **2-Thermometer**– Two standard thermometers are required- one is placed in the water bath and another in the cup. The range of both the thermometers should be from 0 to 44 °C. The least count is 0.2 °C.

10. Other accessory apparatus for viscosity of bitumen test include:
Timing device like stopwatch or stop clock- It should be able to measure up to 0.5 seconds.

A19.2.2 Materials

- Bitumen sample to be tested
- Non-corroding Solvent, e.g. Phenol-free light tar oil

Following precautions should be taken while performing the viscosity of bitumen test to obtain accurate results:

- Rotate the stirrer when the sample is heated in tar cup
- The temperature should be strictly adhered to during the entire test
- Test temperature should not be lower than 20 °C and it should be in the multiples of 5 °C
- Care should be taken while using the tar cup during its cleaning- it should be cleaned gently
- Non-corroding solvents should be used such as light tar oils free from phenols
- Do not use duster for cleaning as it may lead to abrasion of the metal
- The orifice at the top of the tar cup should be checked for its diameter frequently with a gauge
- Calibration of thermometer should be done periodically

A19.2.3 Test Procedure for Industrial Viscosity of Bitumen

- Adjust the tar viscometer in such a way that the top of the tar cup is levelled.
- Heat the water bath to the specified temperature of the test and maintain it at the same temperature throughout the test duration. Tolerance of ± 0.1 °C is allowed. The specified test temperatures are 35°C, 45°C, 55°C, and 65°C.

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- Rotate the stirrer gently at frequent intervals. Continuous rotation is preferred.
- Clean the orifice of the tar cup of viscometer with a suitable solvent and allow it to dry completely.
- Heat the bitumen sample to a temperature of about 20°C above the specified test temperature and allow it to cool. Meanwhile, rotate the stirrer continuously.
- Allow the sample to cool up to a temperature slightly above than the specified test temperature.
- Pour the tar into the tar cup then after till the levelling peg on the valve is just immersed. During this, the valve rod should be kept vertical.
- Take 20 ml of mineral oil or 1 % by weight solution of soft soap and pour it into the graduated receiver.
- Place the graduated receiver under the orifice of tar cup.
- Place the second thermometer in the tar.
- Continue stirring the tar until the temperature falls within ± 0.1 °C of the specified test temperature.
- Then, suspend the thermometer co-axially to the cup with its bulb at the geometric centre of the tar approximately.
- Allow the assembly of the apparatus to stand for 5 minutes. During this time, the thermometer reading should remain within 0.5 °C of the specified test temperature.
- Remove the thermometer.
- Also, remove the excess of tar, if any quickly. This ensures that the final level is on the centre line of the levelling peg when the valve is vertical.
- Suspend the valve on the valve support by lifting it.
- Start the stopwatch or stop-clock or any other time recording device when the reading on the cylinder is 25 ml.
- Note the time in seconds when the reading on the cylinder is 75 ml by stopping the stopwatch.
- Repeat the test for a few times with the same material and note down the results.

A19.2.4 Observation

The time at which the reading of the cylinder reaches 75 ml from 25 ml is to be noted down for all the three samples along with the specified test temperature.

Whether the sample is tested as received or dried before testing is also reported. If drying is done, then the method of drying is also noted. If the sample is tested as received, then water is present in it. And the presence of water has a notable effect on viscosity.

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Table A19.1 - Industrial viscosity test of bitumen

Specified Temperature = __ °C	Test 1	Test 2	Test 3
Times in Seconds			

Industrial Viscosity of the sample = Average value of Times from Tests 1, 2 and 3.

A19.2.5 Result

The time taken by 50 ml of tar to flow out is recorded to the nearest whole number as the viscosity of the tar sample. The time taken by the three tar samples should lie within $\pm 4\%$ of the mean value of the three readings.

While using **Cutback Bitumen**, the tolerance in the difference in readings for industrial viscosity test of bitumen is tabulated below:

Table A19.2 – Tolerances for Industrial viscosity test of bitumen

Viscosity	Repeatability
< 20 Sec	2 Sec
20 – 40 Sec	2 Sec
40 Sec	5% of the mean

A19.3 Determination of **Absolute Viscosity Test of Bitumen**

Absolute Viscosity is also known as Dynamic Viscosity of Bitumen.

To determine the absolute viscosity of bitumen and cutbacks. In this practical, the absolute viscosity is determined by **vacuum capillary viscometers** at any stated temperature. This method is valid for materials having a viscosity range of 42-200000 poises.

Absolute Viscosity: If a tangential force equal to 1 dyne is acting on planes of unit area separated by a unit distance of liquid produces tangential velocity of one unit, then the internal friction acting at that time is called absolute or dynamic viscosity. CGS unit of dynamic viscosity is Poise. (1 dyne = 0.00001 N)

Newtonian liquid: It is a liquid in which shear stress is directly proportional to the rate of shear strain.

Alternatively, it is the liquid in which the coefficient of viscosity is constant.

Coefficient of viscosity: It is a ratio of shear stress to the rate of shear strain.

A19.3.1 Apparatus

Either of the 3 viscometers discussed below shall be used for absolute viscosity test of bitumen:

- **Viscometer- Capillary type:** It is made up of borosilicate glass. There are three types of capillary type viscometers as discussed below:
 - **Cannon Manning Vacuums Viscometer**- The range of the viscosity that this viscometer is able to measure depends on the size of the viscometer. It has two measuring bulbs- B and C.
 - **Asphalt Institute Vacuum Viscometer**- The range of the viscosity that this viscometer is able to measure depends on the size of the viscometer. It has three measuring bulbs- B, C, and D of length 2 cm. The three bulbs are situated on viscometer arm M. This arm is a precision bore glass capillary.
 - **Modified Koppers Vacuum Viscometer**- The range of the viscosity that this viscometer is able to measure depends on the radii of the viscometer. It has a separate filling tube joined to a precision bore glass capillary vacuum tube with a borosilicate ground glass joint that has 24/40 standard taper. The glass capillary has 2 cm long 3-bulbs separated by timing marks.

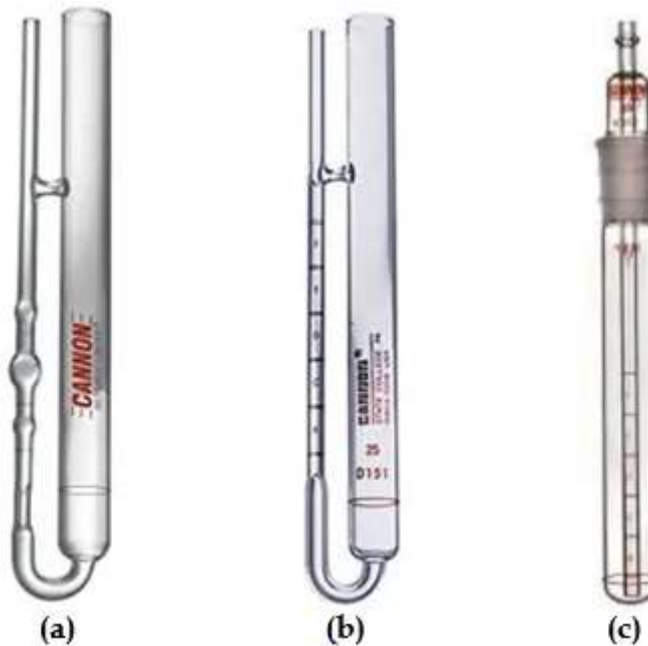


Figure A19.4 – (a) Cannon Manning Vacuums Viscometer, (b) Asphalt Institute Vacuum Viscometer and (c) Modified Koppers Vacuum Viscometer

- **Bath:** It is used for the immersion of viscometer. The viscometer is so immersed in the bath that the liquid reservoir or the capillary top- whichever is the uppermost- is at least 20 mm below the top level of the bath. It has a provision that allows the viscometer and thermometer to be visible. Firm support is also present on the bath to hold the viscometer.

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- The accuracy of viscometer should be 0.1°C over the length of viscometer or from two different viscometer positions in the bath.
- **Thermometer or Temperature indicator**– Its range should be from 0 to 44°C . Its least count should be 0.2°C .
- **Vacuum System**– A vacuum or aspirator pump can be used as the source of vacuum in the vacuum system. All the glass joints of the vacuum system should be airtight so that no vacuum is lost in the duration of the experiment. It should be capable of maintaining the vacuum within $\pm 0.05\text{ cm}$.

Other accessory apparatus for the **absolute viscosity test of bitumen** include:

- Timing device like stopwatch or stop clock- It should be able to measure up to 0.5 seconds.
- Stirrer
- Oven

A19.3.2 Materials

- Bitumen or cutback to be tested
- Solvent
- Distilled water
- Acetone
- Chromic acid

A19.3.3 Precautions

Following precautions should be taken while performing the viscosity of bitumen test to obtain accurate results:

- Rotate the stirrer when the sample is heated in tar cup
- The temperature should be strictly adhered to during the entire test
- Local heating should be prevented by stirring the sample
- Viscometer should be cleaned periodically with chromic acid so that organic deposits are removed
- Calibration of thermometer should be done periodically

A19.3.4 Steps for Sample Preparation

- Take the bitumen sample in a beaker and heat it to a temperature, not more than 60°C - for tars and pitches while the temperature should not exceed 90°C - for bitumens. Allow the bitumen to melt at slightly above the softening points of the material until it achieves a pouring consistency.
- Pour 20 ml of melted bitumen in a suitable container and maintain it to a temperature of 135°C with $\pm 5.5^{\circ}\text{C}$ tolerance.
- Stir the bituminous material occasionally so that local heating can be avoided. Also, the entrapped air can also escape in this while.
- Charge the viscometer. For charging, pour the prepared sample into the cup within $\pm 2\text{ mm}$ of the fill line E.
- Place the viscometer into bath or oven after charging and maintain it to $135 \pm 5.5^{\circ}\text{C}$. maintain this temperature for $10 \pm 2\text{ min}$ to facilitate the escape of air bubbles.

A19.3.5 Steps for Testing of the Specimen

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- Maintain the temperature of the bath at 135 °C as done previously, however, tolerance is reduced from 5.5 °C to 1.0 °C.
- Place the charged viscometer in the water bath vertically with the help of a holder. Adjust the depth of thermometer such that the uppermost timing mark remains a minimum of 2 cm below the liquid surface.
- Connect the vacuum system to the viscometer after establishing a vacuum of 30 ± 0.05 cm of mercury. Keep the valve of the vacuum system closed.
- Allow the assembly to stand in the water bath for 30 ± 5 min.
- Open the valve and allow the asphalt to flow into the viscometer.
- Measure the time required for the leading edge of the meniscus to pass between the two consecutive pairs of the timing marks in seconds. Accuracy of 0.5 seconds is required.

A19.3.6 Cleaning of Viscometer After Testing

- Remove the viscometer from the bath.
- Place the viscometer in the oven at 135 ± 5.5 °C in an inverted position. Allow the viscometer to stand in the oven till all the asphalt has drained off thoroughly.
- Rinse the viscometer several times with an appropriate solvent.
- Pass a slow stream of dried air through the capillary for about 2 minutes so that the tube is completely dry.
- Once in a while, clean the viscometer with chromic acid. It helps in the removal of organic deposits.
- Rinse the instrument thoroughly with distilled water and acetone.
- Dry the instrument with clean air.

A19.3.7 Observation

As the valve of the viscometer is opened, the asphalt starts flowing into the viscometer.

The time taken by the leading edge of the meniscus to pass between two successive pairs of timing marks is to be noted down in seconds.

Table A19.3 - Observation Table for absolute viscosity test of bitumen

Test Temperature		
Constant 'K'		
Descriptions	Test1	Test2
Efflux time (t) in sec		
Viscosity (K * t) in poise		

Absolute Viscosity of the sample = Average value of 1 and 2.

A19.3.8 Calculation

Absolute viscosity of the sample is calculated and reported to the three significant figures. Following equation is used for calculating absolute viscosity:

Absolute Viscosity = K.t

K = selected calibration factor; in poise/second

t = time of flow; in seconds

absolute viscosity is obtained in Poise

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A19.3.9 Result

The value of absolute viscosity is recorded to the three significant figures.
The result is reported along with the test temperature as follows:
Absolute viscosity at 60 °C temperature = 'X' poise Summary

A19.4 Kinematic Viscosity Test of Bitumen

A19.4.1 Kinematic viscosity of Newtonian liquid

Kinematic Viscosity: It is the quotient of the dynamic viscosity divided by the density of the liquid under consideration, both taken at the same temperature.

CGS unit of kinematic viscosity is Stoke. (1 stoke = $1 \text{ cm}^2/\text{s}$)

Newtonian liquid: It is a liquid in which shear stress is directly proportional to the rate of shear strain.

Alternatively, it is the liquid in which the coefficient of viscosity of a liquid is constant.

Dynamic Viscosity: If a tangential force equal to 1 dyne is acting on planes of unit area separated by a unit distance of liquid produces tangential velocity of one unit, then the internal friction acting at that time is called absolute or dynamic viscosity.

CGS unit of dynamic viscosity is Poise. (1 dyne = 0.00001 N)

Coefficient of viscosity: It is a ratio of shear stress to the rate of shear strain.

Density: It is the ratio of mass per unit volume.

CGS unit of density is g/cm^3 .

A19.4.2 Apparatus

Either of the 2 viscometers discussed below shall be used for determining viscosity.

- **Viscometer- Capillary type:** It is made up of borosilicate glass. There are three types of capillary type viscometers as discussed below: **Cannon-Fenske Viscometer (for Opaque liquids)**- This is a reverse flow annealed viscometer.

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**Figure A19.5 – (a) Cannon-Fenske
Viscometer**

**(b) BS U-Tube Modified Reverse-flow
Viscometer**

BS U-Tube Modified Reverse-flow Viscometer - It is made up of borosilicate or other heat-resisting glass that does not have any visible defects. All the glass tubes of a single viscometer should be similar in composition. The instrument should be thoroughly annealed after finishing.

- **Bath:** It is used for the immersion of viscometer. The viscometer is so immersed in the bath that the liquid reservoir or the capillary top- whichever is the uppermost- is at least 20 mm below the top level of the bath. It has a provision that allows the viscometer and thermometer to be visible.

The stirring efficiency and balance between heat losses and its input should be such that the temperature of viscometer is maintained within ± 0.1 °C over the length of viscometer.

- Thermometer or Temperature indicator – Its range should be from 0 to 44 °C. Its least count should be 0.2 °C.
- Other accessory apparatus for kinematic viscosity test of bitumen include:
 - Timing device like stopwatch or stop clock- It should be able to measure up to 0.5 seconds.
 - Stirrer
 - Oven

A19.4.3 Materials

- Bitumen of known grade
- Non-corroding Solvent, e.g. Phenol-free light tar oil

A19.4.4 Precaution

Following precautions should be taken while performing the viscosity of bitumen test to obtain accurate results:

- Rotate the stirrer when the sample is heated in tar cup
- The temperature should be strictly adhered to during the entire test
- Test temperature should not be lower than 20 °C and it should be in the multiples of 5 °C
- Care should be taken while using the tar cup during its cleaning- it should be cleaned gently
- Calibration of thermometer should be done periodically
- Non-corroding solvents should be used such as light tar oils free from phenols
- Do not use duster for cleaning as it may lead to abrasion of the metal
- The orifice at the top of the tar cup should be checked for its diameter frequently with a gauge

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A19.4.5 Sample Preparation

- For **Cutback Bitumen and Oil Distillates**
 - Open the sample container. Thoroughly mix the sample by stirring it for 30 sec. ensure that the entrapped air is avoided.
 - If the sample is too viscous: Take the bitumen sample in a sealed container and heat it in a bath or oven at about 60°C temperature.
 - Pour 20 ml of melted sample in a clean and dry container of 30 ml capacity and seal the container immediately.
- For **Bitumen**
 1. Heat the sample to a temperature, not more than 60 °C- for tars and pitches while the temperature should not exceed 90°C- for bitumens. Allow the bitumen to melt at slightly above the softening points of the material until it achieves a pouring consistency.
 2. Pour 20 ml of melted bitumen in a 30 ml container.
 3. Avoid local heating and the entrapped air in the sample.

A19.4.6 Testing of the Specimen

- For **Cannon-Fenske Viscometer**
 1. Charge the viscometer by inverting it and applying the suction to the tube CL. Immerse the tube N in the liquid sample.
 2. Draw liquid through the N tube so that the bulb D is filled to the fill mark- G.
 3. Wipe off the excess sample from the tube N.
 4. Invert the viscometer so that it is in a normal position.
 5. Align the viscometer in the water bath so that it is vertical. The verticality can be checked by visual inspection which is sufficient, but for accuracy, a plumb bob can be suspended in the tube L.
 6. Maintain a constant temperature of the water bath in which viscometer is placed to such a time that the sample reaches an equilibrium in terms of temperature.

For 38 °C, heating is to be done for 20 min, for 100 °C 25 min is required and for 135 °C 30 min is required to attain a temperature equilibrium.
 7. Remove the stopper in the tubes N and L consecutively after the test temperature is obtained.
 8. Allow the sample to flow by gravity.
 9. Measure the time required for the leading edge of the meniscus to pass between the two consecutive pairs of the timing marks- E to F in seconds. Accuracy of 0.1 seconds is required.
 10. If this efflux time comes less than 60 sec, select a smaller capillary diameter viscometer and repeat the above procedure.
- For **BS U-Tube Modified Reverse-flow Viscometer**
 1. Adjust the BS U-tube viscometer in the bath maintained at a constant temperature such that the L tube is kept vertical.

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2. Pour the sample through the tube N until it reaches up to the filling mark G.
3. Allow the sample to flow freely through capillary R ensuring that the liquid remains continuous or unbroken till it reaches the lower filling mark H.
4. When it reaches the filling mark, close the timing tube with a cork/rubber-stopper in tube L so that the flow is arrested.
5. Add more liquid, if needed, so that the level reaches slightly above the mark G.
6. Allow the sample to attain the bath temperature. Also, allow the air bubbles to rise to the surface if any. Usually, this takes 20-30 min.
7. Loosen the stopper gently and allow the sample to flow till it reaches the lower filling mark H. At this point, arrest the flow by pressing back the stopper.
8. Remove the excess of the sample above the filling mark G. The sample can be removed with the help of a special pipette. Insert it till its cork rests on the top of tube N.
9. Apply suction gently so that air is completely drawn and the upper meniscus shall coincide with mark G.
10. Allow the viscometer to stand in the bath maintained at a constant temperature for a such that the sample reaches temperature equilibrium.

For 38 °C, heating is to be done for 20 min, for 100 °C 25 min is required and for 135 °C 30 min is required to attain a temperature equilibrium.

11. Remove the stopper in the tubes N and L consecutively after the test temperature is obtained.
12. Allow the sample to flow by gravity.
13. Measure the time required for the leading edge of the meniscus to pass between the two consecutive pairs of the timing marks- E to F in seconds. Accuracy of 0.1 seconds is required.
14. If this efflux time comes less than 60 sec, select a smaller capillary diameter viscometer and repeat the above procedure.

A19.4.7 Observation

As the valve of the viscometer is opened, the asphalt starts flowing into the viscometer. The time taken by the leading edge of the meniscus to pass between two successive pairs of timing marks is to be noted down in seconds.

Table A19.4 - Observation Table for Kinematic Viscosity Test

Test Temperature		
Constant 'K'		
Descriptions	Test1	Test2
Efflux time (t) in sec		
Viscosity (K * t) in poise		

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Absolute Viscosity of the sample = Average value of 1 and 2.

A19.4.8 Calculation

Kinematic viscosity of the sample is calculated and reported to the three significant figures.

Following equation is used for calculating kinematic viscosity:

Viscosity = cSt

Viscosity = Ct

C = calibration constant of viscometer; in centistokes/second

t = efflux time; in seconds

absolute viscosity is obtained in centistokes

A19.4.9 Result

The value of kinematic viscosity is recorded to the three significant figures.

The result is reported along with the test temperature as follows:

Kinematic viscosity at 60 °C temperature = 'X' cSt

Reference:

Viscosity test of Bitumen, Civiconcepts,

Web Link: <https://civiconcepts.com/blog/viscosity-test-of-bitumen>

A20 Flash and Fire Test of Bitumen

Standard Codes of Reference

This test refers to: AASHTO T 48, ASTM D 92, IS 1209: 1978, for Flash and Fire Point test of Bitumen.

A20.1 General

Flash and Fire Point Test of Bitumen is used to determine the safe temperature up to which bitumen sample can be exposed.

The test is conducted to determine the flash point and fire point of asphaltic bitumen, fluxed native asphalt, blown type bitumen, or cutback bitumen.

Here, the flash and fire point determination by Pensky-Martin's Closed Cup tester is described. Sometimes, the Cleveland Open Cup Apparatus is also used for conducting the flash and fire point test of a bituminous specimen.

Flash and fire point test of bitumen sample is one of the important tests of bitumen to be conducted before road construction. This test measures the temperature at which the material is at risk of catching fire. Temperature at which the vapour of the bituminous material catches an instant fire or the material burns for some seconds is different for different types and grades of bitumen binders. Bituminous materials are primarily hydrocarbons and hence at high temperatures, they release various volatile materials. These liberated volatile compounds catch fire with a flash. And this can prove hazardous. Bitumen is heated for its application as bitumen binder for road pavements. While dealing with hot bitumen during the processes like heating, mixing, or application, the temperature should be kept well below the critical temperatures determined by flash and fire point.

Flash point: It is the lowest temperature whose application causes the vapour of the bituminous material to catch an instant fire in a flash form in specified conditions.

Fire point: It is the lowest temperature whose application causes the binder material to ignite for at least 5 seconds in specified conditions.

A20.2 Apparatus

The apparatus required for performing the flash and fire point test of bitumen are described below:

A20.2.1 Pensky Martens Tester: It is the testing machine for flash and fire point test of bitumen. The various parts of this apparatus are described below:



Figure A20.1 - Pensky Martens Tester for Flash and Fire Point test of Bitumen

A20.2.2 Closed Cup Tester: It is made up of brass and its inside diameter is slightly larger above the filling mark. The outside surface of the cup is tapered above the flange. The flange also has devices for receiving the lid of the cup and the cup too when kept in the stove. A handle is also fixed to the cup flange permanently.

The wall thickness at the upper edge is not less than 1 mm.

Flange width = 12 mm approx.

Flange thickness = 3mm

A20.2.3 Open Cup Tester: In this tester, the cover of the cup is replaced by a clip that encircles the upper rim of the cup. It also has provision for test flame and thermometer.

A lid includes the following

1. Stirring device: It is in the form of vertical shaft mounted centrally on the cup. It has brass propellers-2 in number. Propeller blades are mounted on a collar. The stirrer is operated mechanically in a close cup test. For testing cutback bitumen in a close cup, a hand-operated stirrer is needed.

The dimension details of various parts of the stirring device are as follows-

Diameter of Vertical steel shaft = 2.5-3 mm

Width of the propeller blade = 8 mm at approximately 45 ° angle

Overall diameter of smaller propeller = 20 mm

Overall diameter of larger propeller = 32-44 mm

Thickness of propeller blades = 1.2-2 mm

Horizontal and vertical dimensions of collar $\nless 10$ mm

Distance between the plane of the centre of propeller and rim of the cup = 10 mm

2. Cover proper: It is made up of brass. Its rim projects almost to the flange of the cup and closely fits the outside of the cup. A proper locating device is present on the cover that engages with the corresponding locating device on the flange of the cup. Thickness of cover measured from just inside the rim is in-between 0.8-2 mm.

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In open cup test, instead of cover, a clip that encircles the upper rim of the cup and has a provision for carrying thermometer and test flame is used.

There are four openings in the cover as follows:

Opening A: It is the area formed by the arcs of two concentric circles and intersected lengths of the two radii.

Radius of the outer circle = 22.4-24.6 mm

Radius of inner circle = 13.5-14.2 mm

Chord of the arc of the outer circle = 12.7-13.7 mm

Openings B and C: Both have equal areas. Their general form is similar to the opening A but these openings have approximately half the angular width.

Chord of outer arc = 4.8-5.6 mm

Sum of areas of these openings = 75-100 % of the area of Opening A

Radii drawn through their centre is at an angle = 135-140°

Opening D: It is in the form of a split tube that helps to grip the thermometer. Its height should be such that when a standard thermometer is kept in position the bottom of the bulb lies below the rim of the cup by 35.6-40.6 mm.

Distance between the centre of the opening and the centre of lid = 20 mm

Angle of the opening to the perpendicular = 10-15°

3. Shutter: It is made up of brass and it is attached to the lid. It operates on the plane of the upper surface of the lid. Its shape and mounting are so decided that it rotates on the horizontal axis of the centre of the lid. When it is placed in an extreme position, openings A, B, and C of the lid are closed completely. When it is placed in the other extreme position, openings A, B, and C of the lid are opened completely.

4. Flame exposure device: It has a tip with 0.7-8 mm diameter opening. It also has an operating mechanism which depresses the tip when the shutter is open. A pilot frame is also provided that helps in relighting of the exposure flame automatically. A 4 mm diameter bead is also placed on the lid to regulate the size of the test flame. The shutter operating mechanism is of spring-type and constructed in such a way that openings A, B, and C are closed when it is at rest.

5. Heating device with temperature regulator:

Thermometer: Its range should be from -7 – +110 °C with a least count of 0.5 °C.

6. Other accessory apparatus used in flash and fire point test include:

- Ignition source device
- Air bath

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A20.3 Materials

- Bituminous material
- Solvent for cleaning

A20.4 Precaution

Following precautions should be taken while performing the flash and fire point test of bitumen to obtain accurate results:

- Size of the test flame and the frequency of the application of test flame should be adhered to, else the surface layer may be superheated.
- The bluish flame surrounding the test flame should not be confused with the true flame.
- The traces of solvent used for cleaning the apparatus should be removed completely
- The locating devices of the cup and the lid should be engaged properly

A20.5 Procedure for Flash and Fire Point Test of Bitumen

There are two types of tester for determining the flash point of bitumen- closed cup tester and open cup tester. However, fire point of bitumen is determined by an open cup tester only.

A20.5.1 Determination of Flash Point by Closed Cup Tester

The procedure for finding out the flash point in closed cup tester varies for cutback bitumen. Both the procedure are described below

For Bituminous material EXCEPT for Cutback Bitumen

- Clean all the parts of the cup and its accessories thoroughly and allow them to dry.
- Take the bitumen sample in a beaker and heat it to a temperature of 75-100°C- above its approximate softening point. Allow the bitumen to melt until it is converted completely into a liquid state.
- Fill the melted bitumen in the cup up to the filling mark indicated on the cup.
- Close the cup with the help of the lid. Ensure that the locating devices of the cup and lid are properly engaged.
- Place the cup on the stove.
- Insert a thermometer and adjust the test flame to a size of a bead of about 4 mm diameter. Control the rate of the application of heat such that the temperature increases by 5-6 °C per minute as recorded from the thermometer.
- Turn the stirrer at the rate of 60 rev/min.
- Apply the first test flame when the temperature reaches approximately 17 °C before the actual flash point. When test flame is applied, discontinue the stirring.
- Apply test flame at every reading of the temperature up to 104 °C in the multiples of 1 °C. When the temperature exceeds 104 °C, carry out the test at an interval of 2 °C.

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- Operate the device to apply the test flame by controlling shutter and test flame burner in such a way that the flame is lowered in 0.5 seconds for. It remains in the lowered position for a second and is raised quickly to the higher position.
- Note down the temperature at which a distinct flash is observed in the interior of the cup.

For **Cutback Bitumen**

- Heat the material to a temperature 17 lower than the expected flash point. Bring the tester to the same temperature too.
- Fill the space between the cup and the interior of the air bath with the same temperature to remove the air.
- The same procedure as above is followed then after. Except for the rate of the application of heat. Control the rate of the application of heat such that the temperature increases by 1-1.5 °C per minute as recorded from the thermometer.
- Turn the stirrer at the rate of 70-80 rev/min.
- Then the test flame is applied and the other steps are followed as described previously except for the interval at which test flame is applied. Instead of applying test flame at intervals of 1 °C, the test flame is applied at every 0.5 °C rise in temperature.

A20.5.2 Determination of Flash Point and Fire Point by Open Cup Tester in Flash and Fire Point Test

- Clean all the parts of the cup and its accessories thoroughly and allow them to dry. Ensure all the traces of solvent used for cleaning are removed.
- Take the bitumen sample in a beaker and heat it to a temperature of 75-100°C- above its approximate softening point. Allow the bitumen to melt until it is converted completely into a liquid state.
- Fill the melted bitumen in the cup up to the filling mark indicated on the cup.
- Fix the clip in position on the cup. Ensure that the locating devices of the cup and lid are properly engaged.
- Insert a thermometer and adjust the test flame to a size of a bead of about 4 mm diameter. Control the rate of the application of heat such that the temperature increases by 5-6 °C per minute as recorded from the thermometer.
- Observe the surface of the material carefully during heating.
- Then after the same steps are followed as mentioned in closed cup tester except that the stirrer is rotated **manually** instead of mechanically.
- Note down the temperature when the first distinct flash appears at any point on the surface of the material. Record this temperature as the flash point.
- Continue the heating beyond the flash point and applying the test flame at intervals of 1 °C.
- Note down the temperature when the material ignites and continues to burn for at least 5 seconds. Record this temperature as fire point.

The entire practical is repeated for 2 more times. The corresponding average value of the three tests is the flash point and fire point of the given bitumen specimen.

A20.6 Observation

As the test flame is applied, at a certain temperature, a flash can be observed. This temperature is noted down as flash point.

Again, the test flame is continued to be applied. At a certain temperature, the fire occurs for at least 5 seconds. This temperature is noted down as fire point.

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Observation Table:

Table A20.1 - Observation Table for Flash and Fire Point Test

	Test Trial 1	Test Trial 2	Test Trial 3
Flash Point in °C			
Fire Point in °C			

Flash point of the sample = Average value of flash point of 1, 2, 3.

Fire point of the sample = Average value of fire point of 1, 2, 3.

A20.7 Result

The average of the flash point and fire point values to the obtained from the three tests are the flash point and fire point values of the given bitumen specimen respectively.

For close cup tester, the value of flash point carried out for three times should not differ by more than ± 2 °C for temperature below or equal to 104 °C. And for temperature beyond 104 °C, the tolerance value is increased to ± 5.5 °C.

For open cup tester, the values of both the flash and fire point carried out for three times should not differ by more than ± 8 °C.

Flash and fire point of the bitumen specimen is dependent on the grade of the bitumen.

A20.8 Conclusion

By performing the flash and fire point test of bitumen, we can determine the flash point and fire point of the bitumen sample.

Flash point and fire point gives a measure of the critical temperatures beyond which the bitumen should not be exposed to. Thus, these critical temperatures help in preventing a hazardous situation.

Reference:

Flash and Fire Point Test of Bitumen, Civiconcepts,

Web Link: <https://civiconcepts.com/blog/flash-and-fire-point-test-bitumen>

A21 Stripping Value Test

Standard Codes of Reference

This test refers to: AASHTO T 182, ASTM D 1664 (5), BS 812 - Part 110 1990, IS: 6241-1971, for Stripping Value Test of Aggregates.

A21.1 General - Application of Stripping Value Test

The Bitumen and Tar are used as a binding agent. It is mixed with the aggregates in construction of roads.

They are easily blendable when the aggregates are in dry condition. In case of water presence in Bitumen mixture, it will affect the bitumen bonding with the aggregates and shall disintegrate the road pavement.

So, many different tests on bitumen have been conducted to find the quality, strength and durability.

The primary purpose of stripping value test is to determine the presence of water molecules in adhesion between the bitumen and the aggregates. The stripping value test results are based on visual observation.

The adhesion is good between the aggregates and bitumen when the aggregates are free from wet and dust. If the water is present in the aggregates then, bitumen or tar will become weak.

So, the stripping value test is conducted to evaluate the bitumen binder, whether it is suitable or not for road construction. The stripping value of aggregates is determined by static method (Ref. IS 6241). The stripping value is calculated in percentage by the ratio of uncovered area of aggregates which is not coated by bitumen to the total aggregates area.

A21.2 Apparatus

Following apparatus are required for stripping value test,

- Oven
- Water Bath
- Beaker of 500 ml capacity
- Sieves 20mm and 12.5mm
- Batch mixer to mix the bitumen and aggregates

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A21.3 Test Procedure

The steps for the test are described below in proper sequence,

- Take 200 gms of dried aggregates and place in 20mm sieve, shake the sieve, and collect the aggregates retained on 12.5mm sieve.
- Take bitumen binder as 5% of the weight of collected aggregates.
- Heat the collected aggregates at 150°C and heat the bitumen binder at 160°C.
- The aggregates and bitumen are to be mixed well until the bitumen coat completely covers the aggregates.
- Take the mixture into a glass beaker and allow it to cool at room temperature for 2 hours.
- Add water into the beaker up to complete immersion of the aggregate-bitumen mix,
- Place the beaker, in the water bath at 40°C for 24 hours.
- Remove the beaker and allow it to cool at room temperature.

A21.4 Results

The uncoated stripping area of aggregates can be estimated visually,

Stripping value of aggregates = (uncovered area of aggregates x 100) / Total area of aggregates

The stripping value of aggregates is expressed in percentage.

At least three sample aggregates should be tested, and the average value is the stripping value of the aggregates. The stripping value should not be more than 5% of the total area of aggregates.

The result may be recorded as shown in the table below.

Table A21.1 – Stripping Value Test Results

S. No.	Stripping Percentage	Average %
1.		
2.		
3.		

A21.5 Precautions

For stripping value test following precautions may be taken.

- The aggregates should be free from dust and moisture
- Distilled water should be used for the test
- Ensure proper mixing of aggregates and bitumen.

Reference:

Stripping Value of aggregate, Civil Planets,

Web Link: <https://civilplanets.com/stripping-value-of-aggregate/>

A22 Core / Bitumen Extraction Test

Standard Codes of Reference

This test refers to: AASHTO T 164 (2014), AASHTO T 308, ASTM D 2172 (M), IS: 6241-1971, for Bitumen extraction test from cores.

A22.1 General

Bitumen extraction test is conducted to find the quantity of bitumen used in asphaltic concrete pavement in percentage by cold dissolver method as per ASTM D 2172.

A particular piece of bitumen sample shall be collected after completion of pavement work and tested to find out the quality of the bitumen binder.

The quality, durability and strength of the asphaltic pavement depend upon the percentage of bitumen used. So it is necessary to determine the value to check the quality of the pavement.

A22.2 Apparatus



Figure A22.1 – Centrifuge Extractor

The various apparatus used for the bitumen extraction test are the following:

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- Centrifuge Extractor
- Beaker
- Weighing Machine
- Oven
- 500 gms Bitumen Samples
- Filter Paper

A22.3 Test Procedure

- Weigh 500 gms bitumen sample (W1) and place the sample in the cup of the centrifuge.
- Weigh the filter paper and mark it as (F1)
- Fill the Benzene (dissolver) into the cup up to the sample top (as thoroughly soaked)
- The benzene will separate the aggregates and bitumen.
- Place the filter paper and cover the sample in the centrifuge apparatus.
- Place the glass beaker at the outlet of the centrifuge to collect the extracted sample.
- Allow up to one hour for the solvent to dissolve the bitumen and other ingredients of asphaltic pavement.
- Start the refuge machine and increase the speed gradually to 3600 revolutions per minute.
- The bitumen and benzene will completely drain out in the glass beaker. Again add some more benzene into the cup and once again repeat the same process.
- Run the centrifuge again and again until the bitumen extraction is completed. The extracted material seems to light straw in color.
- Weigh the extracted material as W2.
- Remove the filter paper and dry it in the oven at a temperature range from 1050°C to 1100°C and allow it to cool to room temperature.
- Finally, weigh the dried filter paper as F2.

The test should be conducted on at least three samples, and the average value is the percentage of bitumen used in the pavement.

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A22.4 Test Result

Table A22.1 – Test Observations

S. No.	Description	Sample1	Sample2	Sample3
1.	Weight of mixture sample (W1)			
2.	Weight of Filter Paper before extraction (F1)			
3.	Weight of mixture after extraction (W2)			
4.	Weight of Filter Paper after extraction (F2)			
5.	Weight of Filler collected in Filter Paper = $F1 - F2 = W3$			

A22.5 Calculations

The percentage of Bitumen = $(W1 - (W2 + W3)) \times 100 / W1$

Reference:

Bitumen extraction test, Civilplanets,

Web Link: <https://civilplanets.com/bitumen-extraction-test-procedure/>

A23 Compressive Strength of Concrete

Standard Codes of Reference

This test refers to: AASHTO T 106/T 106M, ASTM C 349, IS: 516-1959, IS: 456-2000, for Compressive strength of concrete cube test.

A23.1 General

The **Compressive strength of concrete** cube test is very important to test that provides an idea about all the characteristics of concrete. By performing this test, we can predict about manufactured Concrete is done properly or not. The Concrete strength for normal construction work varies from 15 MPa (2200 psi) to 30 MPa (4400 psi) and more in commercial and industrial structures.

Strength of concrete depends on following factors such as water-cement ratio, strength of cement use, quality of concrete materials, quality control during production of concrete etc.

Test for compressive strength is can be done by either on cube or cylinder. There is various standard codes recommends concrete cylinder or concrete cube as the standard specimen for the test. The American Society for Testing Materials of construction ASTM C39/C39M provides Standard Test Method for Compressive Strength of cube and Cylindrical Concrete Specimens.



Figure 23.1 – Compression Test Machine and Concrete Cube Mould

Compressive strength can be defined as the ability of material or structure to carry the loads on it without any crack or deflection. A material under compressive load tends to reduce the size, while in tension, size elongates.

Check Compressive Strength of Concrete of any element can be calculated by dividing load applied at the point of failure to the cross-section area of the face on which load was applied.

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Compressive Strength = Load at failure / Cross-sectional Area of element



Figure 23.2 – Concrete Cube, Cylinder Beam Specimens with Moulds

A23.2 Apparatus and Specimens

For cube test of concrete two types of specimens either cubes of 15cm X 15cm X 15cm or 10cm X 10cm x 10cm depending upon the size of aggregate are used for concrete making. For most of the concrete works cubical Moulds of size 15cm x 15cm x 15cm are commonly used.

A23.3 Procedure

This concrete is filled in the mould and tampered properly so as to minimize any air voids available in concrete and then After 24 hours these Moulds are opened and test specimens are soaked in water for curing. The upper surface of these specimen should be made even and smooth. This is done by spreading cement paste on whole area of specimen.

These cubes are tested by compression testing machine after 7 days curing or 28 days curing. The Load on cube should be applied gradually at the rate of 140 kg/cm² per minute till the Specimens fails. Load at the failure of cube divided by area of specimen gives the compressive strength of concrete.

Following is the method for testing Compressive strength of Concrete Cubes

Minimum 9 cubes of 15 cm size Max. M15 or above

Mix the concrete can be done by hand or in a laboratory batch mixer

A23.3.1 Hand Mixing

Mix the coarse aggregate, cement and fine aggregate on a water platform until the mixture is thoroughly blended and is of uniform color. After that add water and mix it until the concrete appears to be homogeneous and of the desired consistency.

A23.3.2 A Sampling of Cubes for Test

Clean the moulds of concrete and apply oil. Fill the concrete in the moulds in 3 layers. Compact each layer of mix with not less than 35 strokes per layer using a tamping rod (Steel rod 16mm diameter and 60cm long, bullet pointed at lower end) The top level of concrete cube and smoothen it with a trowel.



Figure 23.3 – Concrete Cube are being Cured in Water

A23.3.3 Curing of Concrete Cube

The test specimens are stored in a moist air atmosphere for 24 hours and after this period the specimens are given marking and removed from the moulds and kept submerged in freshwater until taken out for testing.

A23.3.4 Precautions for Tests

The water used for curing should be tested every 7 days and the temperature of water must be at 27 ± 20 C.

A23.3.5 Procedure for Concrete Cube Test

- Remove the concrete cube from water after specified curing time and clear out excess water from the surface.

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- Measure the dimension of the specimen to the nearest 2mm.
- Clean the test surface of the testing machine.
- Place the cube sample in the machine in such a manner that the load shall be applied to the opposite sides of the cube cast.
- Place the specimen centrally on the base plate of the machine.
- Rotate the machine's movable portion gently by hand so that it touches the top surface of the specimen.
- Apply the load gradually to the cube without shock and continuously at the rate of 140 kg/cm²/minute till the specimen fails.
- Record the breaking load and note any unusual features in the type of failure.

A23.4 Calculations of Compressive Strength

Check Compressive Strength of Concrete

Size of the concrete cube	= 15 cm x 15 cm x 15 cm
Area of the cube sample	= 225 cm ²
Characteristic compressive strength (f _{ck}) at 7 days	=..... (20 N/mm ²)
Expected maximum load	= f _{ck} x area x f _s
Range to be selected is	
Same calculation should be done for 28 day compressive strength	
Maximum load applied	=.....KN(400KN)
Compressive strength	= (Load in N/ Area in mm ²)
	=..... N/mm ² (400/22.5)
	=..... N/mm ² (17.77 N/mm ²)

A23.5 Reports of Cube Test

Identification mark: M-20, Date of Cast:

Date of test:

Age of specimen: 3, 7, 28 days

Results of Concrete Cube Test:

Avg. compressive strength of the concrete cube = N/mm² (at 7 days)

Avg. compressive strength of the concrete cube =..... N/mm² (at 28 days)

Compressive Strength of Concrete days:

The strength of concrete is increases with age. Following table shows the strength of concrete at different ages in comparison with the strength at 28 days after casting.

A23.6 Percent of **Strength Increases in Days**

1 day	– 16 %
3 day	– 30 %
7 days	– 65 %
14 days	– 90 %
28 days	– 99 %

Reference:

Compressive Strength of Concrete, Civiconcepts,

Web Link: <https://civiconcepts.com/blog/method-of-testing-compressive-strength-of-concrete>

A24 Slump Test of Concrete

Standard Codes of Reference

This test refers to: AASHTO T 119, ASTM C143 / C143M - 20, IS: 1199 -1959, IS: 7320-1974, for Slump test of concrete.

A24.1 General

The slump test is conducted to measure the consistency of concrete in that specific batch. Here, consistency of concrete refers to workability or fluidity of freshly made concrete, and therefore it is a measure of the ease with which concrete flows.

Therefore, the slump test is used on-site to measure the workability of concrete or degree of fluidity in the concrete mix. Generally, loose wet mixes are more workable than drier mixes, but concrete of the same consistency may vary in workability.

The test also shows the change of fluidity of mix between two batch mixes. The slump test is widely used in construction to a few apparatus and simple procedure. The slump is also used to ascertain the effects of plasticizers on their introduction in the concrete mix.

A24.2 Principle

The Slump test concrete is carried out with the help of a conical cone open at both ends. The slump cone filled with concrete is lifted up and the resulting height of concrete spread over surface shows the degree of fluidity, workability, and consistency of the concrete mix. It also indicates the amount of water added is correct or not.

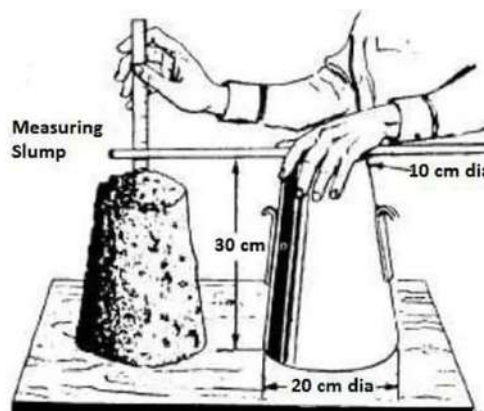


Figure 24.1 – Concrete Slump Test

A24.3 Design Standard

IS: 1199 – 1959 Methods Of Sampling And Analysis Of Concrete
IS: 7320 -1974 Specification For Concrete Slump Test Apparatus

A22.4 Procedure

The test is carried out using a conical shape metal body known as slump cone or Abrams cone that is open at both ends and has an attached handle.

The slump cone has internal dia. 100 mm at top and 200 mm at the bottom and total height of the cone is about 300 mm.

- First, the slump cone is placed on a hard non – water-absorbing surface. (metal or concrete)
- The Slump cone is filled with freshly mixed concrete in three layers.
- Each layer of the concrete mix must be tapped with a 2 ft (600 mm)-long bullet-nosed metal rod measuring 5 8 in (16 mm) in diameter.
- After filling the last layer extra concrete must be removed from the top of the cone.
- Slump cone then quickly lifted upward with a twisting effect with both hands. The concrete is then free to fall.
- After free fall the remaining height is known as slump and is measured to the nearest 5 mm if the slump is 100 mm.

A24.5 Types of Slump

According to free fall shape of concrete, the slump is classified into three types termed as true slump, shear slump or collapse slump.

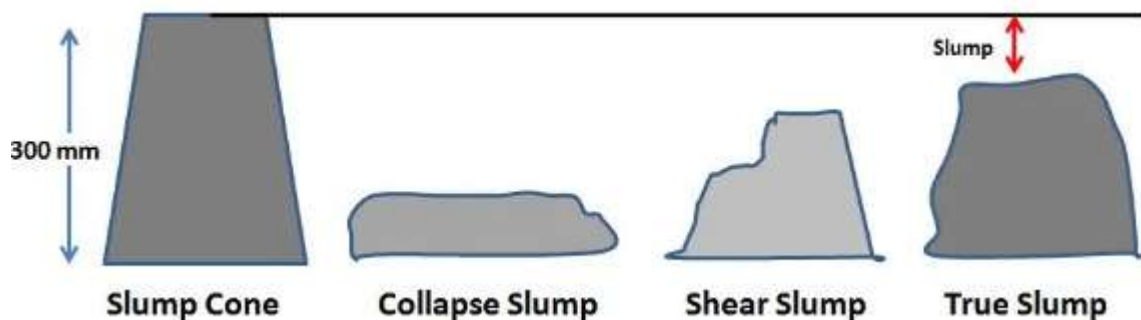


Figure 24.2 – Collapse Slump, Shear Slump and True Slump of Concrete

Table 24.1 – Collapse Slump, Shear Slump and True Slump of Concrete

Collapse Slump	Shear Slump	True Slump
In a collapse slump the concrete collapses completely	In a shear slump the top portion of the concrete shears off and slips sideways	In a true slump, the concrete simply subsides, keeping more or less to shape.

A24.6 True Slump

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- If a shear or collapse slump is achieved, a fresh sample should be taken and the test repeated.
- Only a true slump is of any use in the test.

A24.7 Limitations

- Slump test not applicable when aggregate is more than 40 mm.
- This test does not give an accurate result for high dry and stiff concrete.

A24.8 Advantages

- It has a simple apparatus can be performed in the laboratory as well as on-site.
- It requires fewer apparatus compares to other tests.
- Testing time is too short and gives result quickly;

A24.9 Applications of Desired Slumps

Table 24.2 – Slump Values Recommended for Intended Applications

S. No.	Slump Values	Recommended Applications
1	0-25 mm	Road Pavement Construction
2	10-40 mm	Foundations with light reinforcement
3	50-90 mm	Normal reinforced concrete placed with vibration
4	➤ 100 mm	High workability concrete

Reference:

Slump Test of Concrete, Civiconcepts,

Web Link: <https://civiconcepts.com/blog/slump-test-type-of-slump-test-method>

APPENDIX - B

Pavement Condition Survey and Field Investigations

B1 Benkelman Beam Rebound Deflection Survey

B1.1 General

Benkelman beam is a simple apparatus to conduct structural evaluation of pavement.

- It does so by measuring the deflection of pavement under standard loading conditions
- Following points on the beam are to be understood: Probe point (or Arm Tip), Pivot and dial gauge point.
- From Probe Point to Pivot it is 2.44m and from hinge to dial gauge point it is 1.22m. So the total usable length is 3.66m

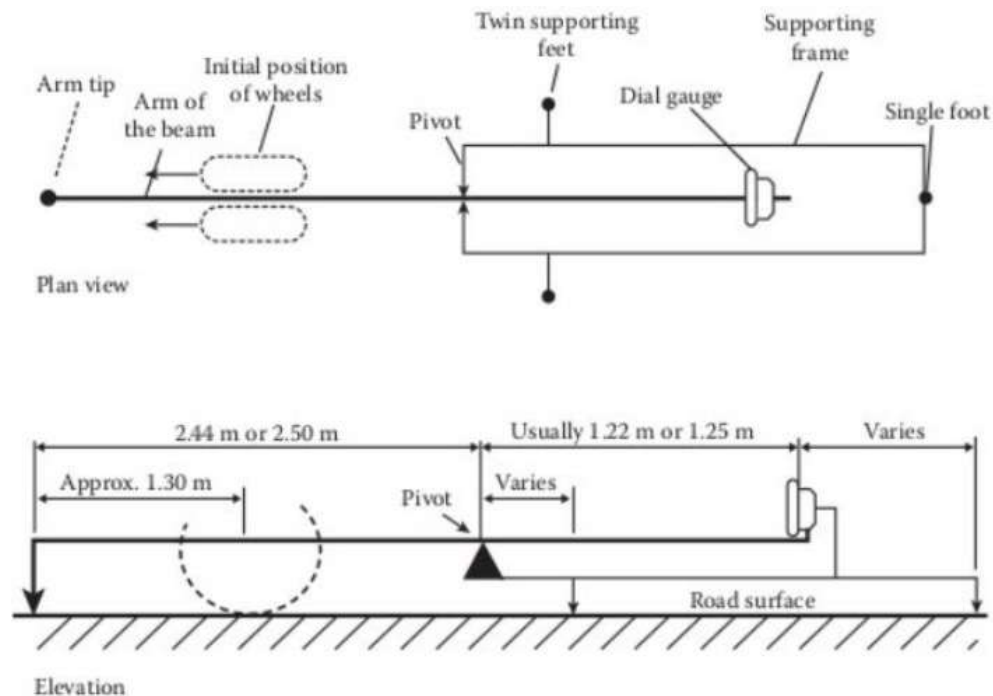


Figure B1.1 – Benkelman Beam Dimensional Diagram



Figure B1.2 – Benkelman Beam Setup Diagram

B1.2 Equipment

The primary device used in the static loading method is the Benkelman beam. The Benkelman beam device is based on level arm principles, where the tip of the device is placed between the dual tires of a single axle loaded to 80 kN (18,000 lbf), and the tires are inflated to 480 to 550 kPa (70 to 80 lbf/inch²) (see figure 2). The operator records the dial measurement as the pavement rebounds from the weight of the axle as the truck is moved forward. Limitations of the Benkelman beam include its inability to measure a deflection basin and only the maximum surface deflection, its relatively labor-intensive requirements for use, and its slow rate of testing that requires traffic control for stopped conditions. Perhaps the primary benefit of the Benkelman beam is that it is relatively inexpensive.

- In Benkelman beam test, the rebound deflection of pavement is measured under static load of the rear axle of a standard truck (Axle load: 8170kg).

- The equipment shall include:

- Benkelman Beam

- (a) Length of probe arm from pivot to probe point: 2.44 m

- (b) Length of measurement arm from pivot to dial gauge: 1.22 m

- A 5 tonne truck is recommended as the reaction. The vehicle shall have 8170 kg rear axle load equally distributed over the two wheels, equipped with dual tyres.

- Spacing between the tyre walls should be 30-40 mm. The tyres shall be 10 x 20, 12 ply inflated to a pressure of 5.60 kg/cm. The use of tyres with tubes and rib treads is recommended.

- Tyre pressure measuring gauge

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- Thermometer (0-100°C) with 1° division
- A mandrel for making 4.5 cm deep hole in the pavement for temperature measurement. The diameter of the hole at the surface shall be 1.25 cm and at bottom 1 cm.



Figure B1.3 – Benkelman Beam Test Procedure

B1.3 Procedure

The point on the pavement to be tested is selected and marked.

– For highways, the point should be located 60 cm from the pavement edge if the lane width is less than 3.5 m and 90 cm from the pavement edge for wider lanes.

– For divided four lane highway, the measurement points should be 1.5 m from the pavement edge.

- The dual wheels of the truck are centred above the selected point.
- The probe of the Benkelman beam is inserted between the duals tyres (as shown in Figure) to place on the selected point.
- The locking pin is removed from the beam and the legs are adjusted so that the plunger of the beam is in contact with the stem of the dial gauge. The beam pivot arms are checked for free movement.

- The dial gauge is set at approximately 1 cm. The initial reading is recorded*.
- The truck is slowly driven by 2.70m and an intermediate reading is recorded*.
- The truck is further driven by 9 m (from 2.7m point) to record the final reading*.

*All readings are to be recorded when the rate of recovery of pavement is equal to or less than 0.025 mm per minute

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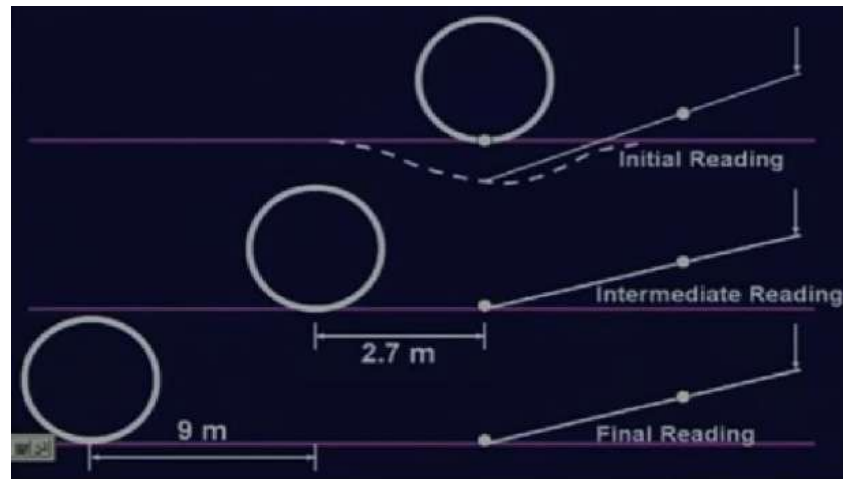


Figure B1.4 – Benkelman Beam Test - Deflection Reading

- Pavement temperature is recorded at least once every hour inserting thermometer in the standard hole and filling up the hole with glycerol.
- The tyre pressure is checked at two or three hour intervals during the day and adjusted to the standard, if necessary.

B1.4 Deflection Measurement

- Initial deflection was noted when rear axle of the truck was at A. (Recovery rate less than or equal to 0.025 mm/min.)
- Note next deflection reading after slowly moving the truck to B (2.7 m away from A) called intermediate deflection.
- After moving the truck to C (9 m away from B) next deflection reading called final deflection should be noted.
- Record pavement temp. every hour at a small hole made with mandrel & filled with 'Glycerol'.
- Determine twice the difference between final & initial (Apparent Pavement deflection) as well as between intermediate & final readings (Y).
- If the second difference is less than 0.025 mm, then pavement deflection is twice of final reading.
Otherwise, True Pavement deflection
= Apparent Pavement deflection + 2.91 times 'Y'

B1.4.1 Sample Calculation for Deflection

- Initial deflection 0.0 mm
- Intermediate deflection 0.46 mm

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- Final deflection 0.49 mm
- True Pavement deflection
= Apparent Pavement deflection + 2.91 times Y
= $2 \times (0.49 - 0) + 2.91 \times 2 \times (0.49 - 0.46)$
= $0.74 + 2.91 \times 0.06 = 1.155$

B1.4.2 Correction for Temperature Variations

- The stiffness of bituminous layers changes with temperature of the binder & consequently the surface deflection of a given pavement will vary as per binder temperature. It is necessary that the measured deflections be corrected to a common standard temperature.
- For areas in the country having tropical climate the standard temperature recommended is 35°C. Correction not applicable for roads with thin toppings.
- Correction should be 0.01 mm per degree centigrade change.

The Correction:

Positive for temp. lower than 35°C.
Negative for temp. higher than 35°C.

- For altitudes > 1000 m, deflection measurements should be done only when ambient temperature is 20°C only. No correction.

B1.4.3 Correction for Seasonal Variations

- For different types of soil, different types of plasticity & different rainfall intensities, different moisture corrections are needed.
- During different times in a year, soil moisture content goes on changing.
- Actual soil moisture content should be measured. Appropriate moisture corrections corresponding to different data should be calculated using proper graph. (6 graphs)

B1.4.4 Determination of Characteristic Deflection

From series of True deflection values, its mean & Standard deviation values should be determined.

- For National Highways & State Highways - Characteristic deflection = Mean + 2*SD
- For other roads - Characteristic deflection = Mean + SD

B1.5 Computation of Rebound Deflection

The rebound deflection D is computed by one of the two following conditions:

1. If $(D_i - D_f) \leq 2.5$ divisions, of the dial gauge or 0.025mm:

(i). $D = 2(D_o - D_f)$ divisions or

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(ii). $D = 0.02(D_o - D_f)$ mm

2. If $(D_i - D_f) \geq 2.5$ divisions:

(i). $D = 2 D_o - D_f + 2K (D_i - D_f)$ divisions

The value of K for Benkelman beams available in Indian is calculated to be 2.91,
Hence, $D = 0.02 D_o - D_f + 0.0582(D_i - D_f)$ mm

B1.6 Equivalency Factor

- The thickness obtained from Figure B1.5 is the overlay thickness in terms of bituminous macadam (BM) construction.
- In case other compositions are to be laid for strengthening, the equivalent overlay thickness to be provided may be determined using appropriate equivalency factors as suggested below:
 - 1 cm of Bituminous macadam = 1.5 cm of WBM/WMM/BUSG*
 - 1 cm of Bituminous macadam = 0.7 cm of DBM/AC/SDC
- From structural considerations, the recommended minimum bituminous overlay thickness is 50 mm Bituminous Macadam(BM) + an additional surfacing course of 50mm DBM or 40 mm Bituminous Concrete (BC).

BUSG: Built-up Spray Grout

WBM: Water Bound Macadam

WMM: Wet Mix Macadam

DBM: Dense Bituminous Macadam

AC: Asphalt Concrete / Bituminous Concrete (BC)

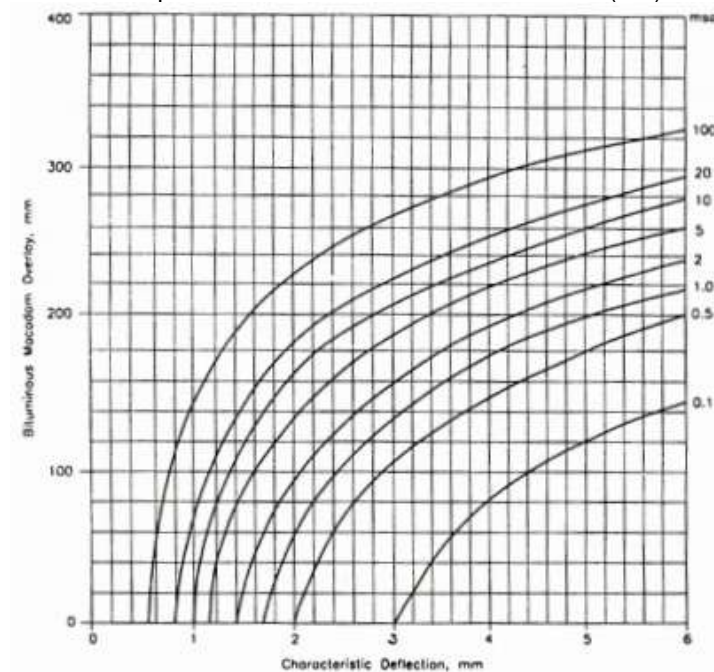


Figure B1.5 – Overlay Thickness Design Chart

Example B1.1

Deflection values: 1.40,1.32,1.25,1.35,1.48,1.60,1.65,1.55,1.45,1.40,1.36, 1.46,1.5,1.52,1.45 mm

- Pavement Temperature = 30 °C
- Subgrade moisture content = 8%, clayey soil, PI>15
- Average annual rainfall = 1500 mm
- Design traffic = 15.5 msa

- Two lane single carriageway
- ADT of Trucks = 1000
- Annual growth rate = 5%
- VDF = 4.5

- Mean deflection = 1.45 mm
- Standard Deviation = 0.107 mm
- Characteristic deflection = $1.45 + 2(0.107) = 1.664$ mm
- Correction for temperature = $0.01(35-30) = 0.05$ mm
- Characteristic deflection after temp. correction = $1.664 + 0.05 = 1.714$ mm
- Seasonal correction factor = 1.4
- Corrected characteristic deflection = $1.4 (1.714) = 2.3996$ mm
- Thickness of overlay in terms of BM from the chart = 190 mm
- Thickness in terms of DBM/AC = $190 \times 0.7 = 133$ mm

Provide 40 mm AC and 95 mm DBM

Example B1.2

Design the overlay required for the pavement which is on a National Highway using the following data:

- Subgrade: Sandy Soil
- Moisture Content: 8%
- Pavement Temperature=35 °
- Annual Rainfall: < 1300mm
- CVPD: 5000
- Design Period: 10years
- Traffic Growth Rate: 7%
- VDF: 4.5

S. No.	D0	Di	Df	Di - Df	D0 - Df	D
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B1.1

1	100	52	50	2	50	1.00
2	100	55	53	2	47	0.94
3	100	53	52	1	48	0.96
4	100	52	52	0	48	0.96
5	100	53	51	2	49	0.98
6	100	55	53	2	47	0.94
7	100	55	53	2	47	0.94
8	100	57	56	1	44	0.88
9	100	59	58	1	42	0.84
10	100	49	47	2	53	1.06
11	100	54	52	2	48	0.90
Mean	SD	De			Moisture Correction	Temperature Correction
0.95	0.056	1.06			1.03	0

Table
-

Calculations for Example B1.2

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$$N = 365 \times [(1 + 0.07)^{10} - 1] \times 5000 \times 0.75 \times 4.5 / 0.07 = 85.1 \text{ msa}$$

- Characteristic Deflection, $D_a = 0.95\text{mm}$

- Standard Deviation, $\sigma = 0.056\text{mm}$

- Characteristic Deflection $D_c = 0.95 + 2 \times 0.056 = 1.06\text{mm}$

- Correction Factor for Temperature = $0.01\text{mm}/^\circ\text{C}$ rise

- +ve for temp higher than 35

- -ve for temp. lower than 35

- Since test time temperature, Temp= 35°C , so no correction.

- Referring to Chart for Sandy Soil, Low Rainfall, correction factor is: 1.03

- Corrected Deflection = $1.06 \times 1.03 = 1.09\text{mm}$

- So for 85.1 msa and Characteristic Deflection of 1.09, the overlay thickness from the Chart is:
160mm BM

Hence, We can provide: 100mm BM + 45mm BC

B2 Falling Weight Deflectometer

B2.1 General

The need to accurately characterize the structural condition of existing pavements has increased with the ongoing development of mechanistic-empirical thickness design procedures and particularly with the release of the *Mechanistic-Empirical Pavement Design Guide* (MEPDG) prepared under National Cooperative Highway Research Program (NCHRP) Project 1-37A.^(1,2) In the MEPDG, the performance of the pavement being designed is projected by simulating the expected accumulated damage on a monthly or semimonthly basis over the selected design period. The amount of incremental damage occurring during each computation interval (either monthly or semimonthly) varies as the effects of prevailing environmental conditions, changes in material properties, and effects of traffic loading are directly considered. Ultimately, the incremental damage accumulated during each computation interval is converted into the development of physical pavement distresses and projected roughness levels using calibrated performance models.

An integral part of this process is the accurate characterization of material parameters of each layer in the pavement structure. Deflection data collected by the falling weight Deflectometer (FWD) can be used to characterize the parameters of the paving layers through back-calculation, in which the engineering material parameters of the paving layers (elastic modulus, E , or dynamic modulus, E^*) and underlying soil (resilient modulus, M_R , or modulus of subgrade reaction, k) are estimated based on the measured surface deflections, the magnitude of the load, and information on the pavement layer thicknesses. In essence, the set of characteristics for the paving layers and subgrade material is determined such that it produces a pavement response that best matches the measured deflections under the known loading conditions.

Taken as a whole, pavement deflection data can be used in a number of ways, including the following:

- Evaluate maximum deflections normalized to a standard load.
- Back-calculate material parameters of pavement layers.
- Evaluate subgrade support conditions.
- Assess potential areas of localized weakness.
- Assess the degree of deterioration in the pavement structure.
- Determine structural enhancement requirements (overlay thickness).
- Determine structural remaining life for projected traffic loadings.
- Evaluate the structural capacity of the pavement structure.
- Develop load limits and/or seasonal load restrictions.
- Determine the presence of built-in upward curling (for rigid pavements).

Over the years, researchers and practitioners have developed numerous approaches to back-calculate pavement layer and subgrade moduli, as well as numerous programs to perform the calculations, in ongoing efforts to better characterize the material properties of the existing pavement structure.

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This report provides best practice guidelines for deflection testing of existing pavement structures, as well as recommended back-calculation techniques and data interpretation procedures to analyze those results as part of a mechanistic-empirical pavement evaluation and rehabilitation process. The focus of the report is the use of the FWD because it is the most commonly used deflection testing device.

Pavement deflection testing is a quick and easy way to assess the structural condition of an in-service pavement in a nondestructive manner. Over the years, a variety of deflection testing equipment has been used for this purpose, from simple beam-like devices affixed with mechanical dial gauges to more sophisticated equipment using laser-based technology. Nevertheless, all pavement deflection testing equipment basically operates in the same manner—a known load is applied to the pavement, and the resulting maximum surface deflection or an array of surface deflections located at fixed distances from the load, known as a deflection basin, are measured. Figure 1 is a schematic of a deflection basin.

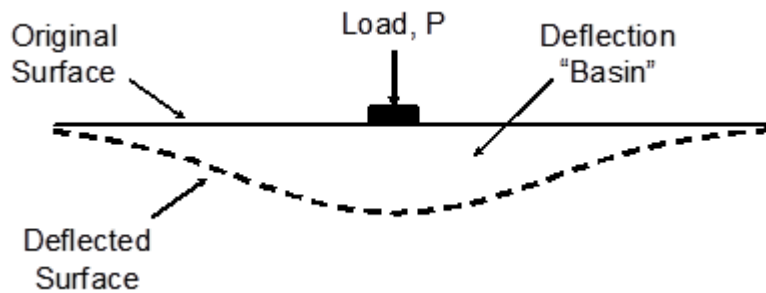


Figure B2.1 - Typical Pavement Deflection Basin

B2.2 Impulse Loading

Impulse loading is conducted by dropping weights at various drop heights to apply an impulse load, ranging from 6.7 to 120 kN (1,500 to 27,000 lbf), to the pavement surface. Deflections are measured using seismometers, velocity transducers, or accelerometers. Devices of this type — known as FWDs and are available through various manufacturers—are capable of measuring a deflection basin and more closely simulate truck traffic loading (see figure 4). As with steady-state testing devices, traffic control is required with FWDs.



Figure B2.2 - Impulse loading or FWD device

Because the majority of State transportation departments use the FWD for deflection testing, that device is the focus of this report.

B2.3 Purpose of Deflection Testing

The primary purpose of deflection testing is to determine the structural adequacy of an existing pavement and to assess its capability of handling future traffic loadings. As observed in the early work by 'Hveem', there is a strong correlation between pavement deflections (an indicator of the structural adequacy of the pavement) and the ability of the pavement to carry traffic loadings at a prescribed minimum level of service. This early work attempted to identify maximum deflection limits below which pavements were expected to perform well; these limits were based on experience and observations of performance of similar pavements. This concept quickly lent itself to overlay design, in which the required overlay thicknesses could be determined based on trying to reduce maximum pavement deflections to tolerable levels.

When complete deflection basins are available, deflection testing can provide key parameters for the existing pavement structure through back-calculation of the measured pavement responses. Specifically, for hot-mix asphalt (HMA) pavements, the elastic modulus (E) of the individual paving layers can be determined, along with the resilient modulus (MR) of the subgrade. For portland cement concrete (PCC) pavements, the elastic modulus (E) of the PCC slab and the modulus of subgrade reaction (k) can be determined. In addition, deflection testing conducted on PCC pavements can be used to estimate the load transfer efficiency (LTE) across joints or cracks as well as for the identification of loss of support at slab corners.

These parameters of the pavement layers and of the subgrade are used in pavement design procedures or in performance prediction models to estimate the remaining life or load-carrying capacity of the pavement. They can also be used in elastic layer or finite element programs to compute stresses and strains in the pavement structure and are also direct inputs in many overlay

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design procedures to determine the required overlay thickness needed for the current pavement condition and the anticipated future traffic loadings.

Deflection data can also be used in other ways to help characterize the condition of the existing pavement. For example, plots of deflection data along a pavement project can be examined for non-uniformity, which may suggest areas that require further investigation using other means, including destructive sampling and testing (see the section later in this report titled Computed Indices from Deflection Data). In addition, daily or seasonal deflection data can provide insight into a pavement's response to environmental forces, including the effects of thermal curling, frozen support conditions, and asphalt stiffening. Some agencies also use deflection criteria to establish seasonal load restrictions for certain low-volume roads. Deflection testing has also seen some limited use as a means of monitoring the quality of a pavement during construction. Finally, a few agencies conduct deflection testing at the network level to provide a general indication of the structural capacity of the pavement structure.

B2.4 Back-calculation of Deflection Data

As described previously, pavement deflection data can be analyzed in a number of ways to help provide detailed information about a specific pavement. Perhaps the most common use of deflection data is in the back-calculation process through which the fundamental engineering properties of the pavement structure, such as the modulus values of the paving layers and the subgrade, are determined. An underlying assumption in the back-calculation process is that a set of layer modulus values exists that produces the measured deflections under the applied load. It is important to note, however, that the solution may not be unique. To obtain good results, engineering judgment must be used to ensure that the modulus value selected for each layer is within a reasonable range for the material type. Back-calculation results can be highly variable owing to variability in pavement condition, subsurface condition, material properties, and pavement structure along the project.

Different back-calculation methodologies are employed for flexible and rigid pavements, but even for a specific pavement type, a number of different approaches can be used. Common procedures include iterative methods, closed-form solutions (currently available for two-layer pavement systems), and simultaneous equations (using nonlinear regression equations). However, varying results can be obtained from these approaches because of differences in the way the pavement structure is modeled. Chapter 3 provides more detailed information on recommended back-calculation procedures and approaches for both flexible and rigid pavements.

B2.5 Computed Indices From Deflection Data

A number of deflection-based indices are often computed as a means of characterizing some aspect of the existing pavement structure. A few of the more common indices are described in the following subsections.

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B2.6 AREA Method

Hoffman and Thompson first introduced the AREA method to characterize the deflection basin for a simple two-parameter back-calculation procedure for flexible pavements, but its use has been expanded to rigid pavements as well. The AREA method represents the normalized area of a vertical slice through a deflection basin between the center of the test load and at varying radial distances from the test load. For a four-sensor configuration, the AREA method equation is shown in Equation B2.1.

$$AREA_{36} = 6 \left(1 + 2 \frac{d_{12}}{d_0} + 2 \frac{d_{24}}{d_0} + \frac{d_{36}}{d_0} \right)$$

AREA method equation for a four-sensor configuration,.....(B2.1)

Where:

- d_0 = Surface deflection at center of test load (inches).
- d_{12} = Surface deflection at a distance of 300 mm (12 inches) from load.
- d_{24} = Surface deflection at a distance of 600 mm (24 inches) from load.
- d_{36} = Surface deflection at a distance of 900 mm (36 inches) from load.

The AREA method equation for a seven-sensor configuration is shown in Equation B2.2.

$$AREA_{60} = 4 + 6 \cdot \frac{d_8}{d_0} + 5 \cdot \frac{d_{12}}{d_0} + 6 \cdot \frac{d_{18}}{d_0} + 9 \cdot \frac{d_{24}}{d_0} + 18 \cdot \frac{d_{36}}{d_0} + 12 \cdot \frac{d_{60}}{d_0}$$

AREA method equation for a seven-sensor configuration,.....(B2.2)

Where:

- d_8 = Surface deflection at a distance of 203 mm (8 inches) from load.
- d_{18} = Surface deflection at a distance of 457 mm (18 inches) from load.
- d_{60} = Surface deflection at a distance of 1,219 mm (48 inches) from load.

Typical AREA values (four-sensor configuration) and D_0 , the surface deflection at the center of test load (in mm (inches)) are shown in Table B2.1, while typical trends are shown in Table B2.2.

Table B2.1 - Typical AREA values (four-sensor configuration) and D_0

Pavement Type	AREA Value	AREA Value	D_0	D_0
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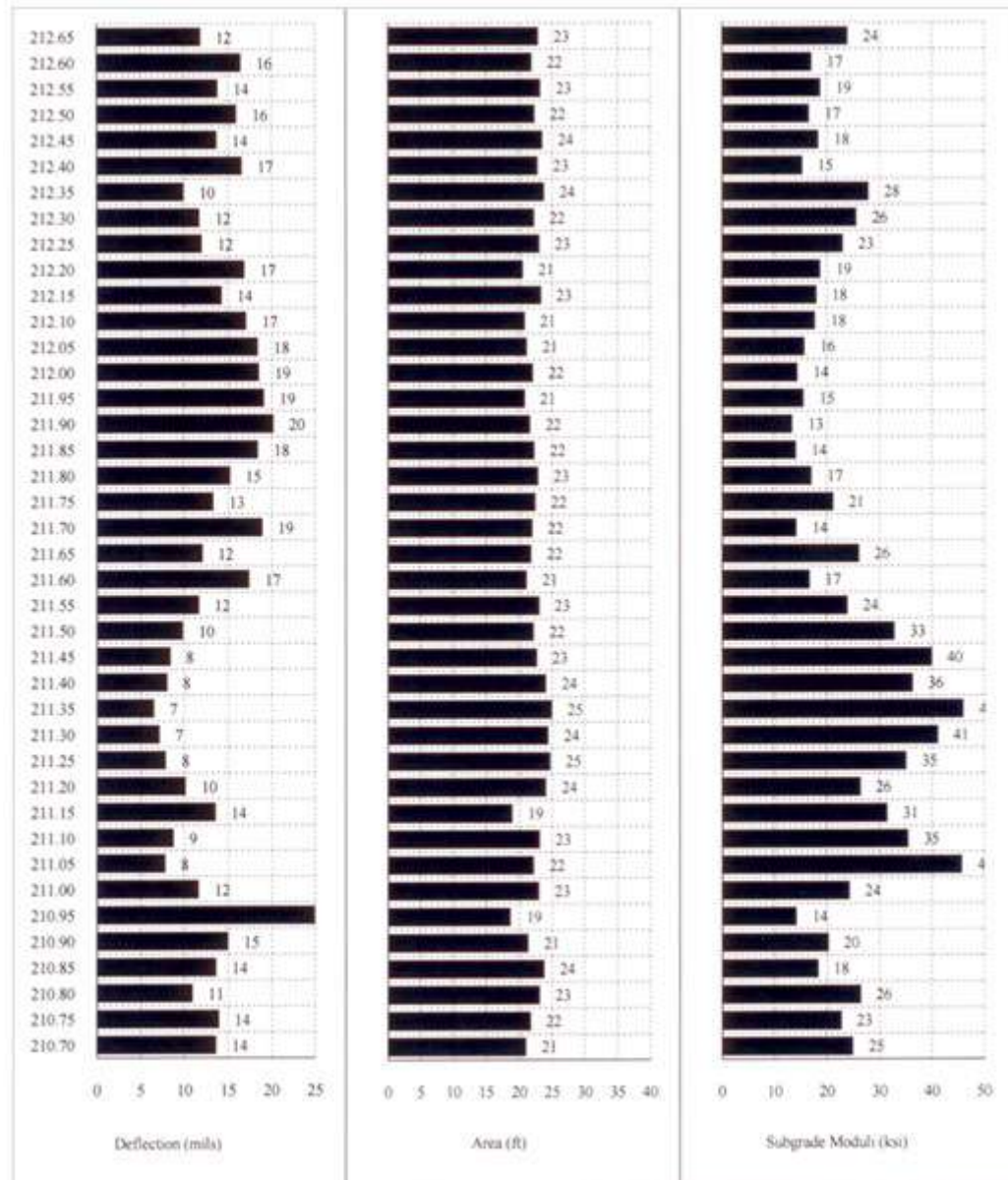
	(mm)	(inches)	(μ m)	(mil)
PCC	740-810	29-32	250-500	10-20
Thick HMA, \geq 200 mm (4 inches)	530-760	21-30	500-1,000	20-40
Thin HMA, \leq 200 mm (4 inches)	410-530	16-21	760-1,200	30-50
Chip seal	380-430	15-17	760-1,200	30-50
Weak chip seal	300-380	12-15	1,000-1,500	40-60

Table B2.2 - Trends of D0 and AREA Values

AREA Value	Maximum Surface Deflection (D_0)	Generalized Conclusions
Low	Low	Weak structure, strong subgrade
Low	High	Weak structure, weak subgrade
High	Low	Strong structure, strong subgrade
High	High	Strong structure, weak subgrade

As demonstrated in figure 7, plotting maximum deflection, *AREA* value, and subgrade can be used further for identifying areas needing further investigation, coring, or additional testing and analysis. In figure 7, the HMA layer thickness for the pavement section considered is greater than 150 mm (4 inches), which indicates a lower than expected *AREA* value has been determined for this thickness of pavement (refer to table 1 and table 2). Looking at the maximum center deflection, a higher deflection occurs over the first half of the project length and corresponds to lower subgrade modulus; conversely, lower maximum deflections are noted from milepost (MP) 211.50 to MP 211.05, with corresponding higher subgrade moduli. Coordinating the type of the plot shown in figure 7 with a pavement conditions survey can also be beneficial and assist in determining locations for any needed coring, boring, and additional material sampling.

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1 mil = 0.0254 mm.

1 ft = 0.305 m.

1 ksi = 6,895 MPa.

Figure B2.3 - Chart. Maximum deflection, AREA, and Subgrade Modulus.

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F - 1 Shape Factor

The F - 1 shape factor represents the amount of deflection basin curvature and is inversely proportional to the ratio of the pavement stiffness to the subgrade stiffness. The F - 1 shape factor is defined by the relation in Equation (B2.3),

$$F - 1 = \frac{(D_0 - D_2)}{D_1} ,.....(B2.3)$$

Where:

D_1 = Surface deflection at a distance of 300 mm (12 inches) from load (mm (inches)).
 D_2 = Surface deflection at a distance of 600 mm (24 inches) from load (mm (inches)).

Base Layer Index

The Base Layer Index (BLI), sometimes referred to as the Surface Curvature Index (SCI), gives an indication of the structural condition of the base layer. Equation B2.4 shows the relation for BLI.

$$BLI = D_0 - D_{300} ,.....(B2.4)$$

Where:

D_{300} = Surface deflection at a distance of 300 mm (12 inches) from load (mm (inches)).

Middle Layer Index

The Middle Layer Index (MLI), also referred to as the Base Curvature Index (BCI), provides an indication of the subbase structural condition. Equation B2.5 shows the relation for MLI.

$$MLI = D_{300} - D_{600} ,.....(B2.5)$$

Where:

D_{600} = Surface deflection at a distance of 600 mm (24 inches) from load (mm (inches)).

Lower Layer Index

The Lower Layer Index (LLI), also referred to as the Base Damage Index (BDI), provides an indication of the structural condition of the subgrade layers. Equation B2.6 shows the relation for LLI.

$$LLI = D_{600} - D_{900} ,.....(B2.6)$$

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Where:

D_{900} = Surface deflection at a distance of 900 mm (36 inches) from load (mm (inches)).

Radius of Curvature

The radius of curvature (RoC) was developed in South Africa and provides an indication of the structural condition of the surface and base course. Equation B2.7 shows the relation for RoC.

$$RoC = \frac{L^2}{2D_0 \left(1 - D_{200}/D_0 \right)}, \dots\dots\dots(B2.7)$$

Where:

L = 200 mm (8 inches).

D_{200} = Surface deflection at a distance of 200 mm (8 inches).

For BLI, MLI, LLI, and RoC, Horak and Emery determined benchmark classification for various flexible pavement sections (Ref. Table B2.3)

Table B2.3 - Benchmark Values for Deflection Bowl Parameters BLI, MLI, LLI, and RoC

Pavement Section	Structural Condition Rating	Do (µm)	Do (µm)	Do (µm)	Do (µm)	Do (µm)
Granular base	Sound	< 500	> 100	< 200	< 100	< 50
	Warning	500-750	50-100	200-400	100-200	50-100
	Severe	> 750	< 50	> 400	> 200	>100
Cementitious base	Sound	< 200	> 150	< 100	< 50	< 40
	Warning	200-400	80-150	100-300	50-100	40-80
	Severe	> 400	< 80	> 300	> 100	> 80
Bituminous base	Sound	< 400	> 250	< 200	< 100	< 50
	Warning	400-600	100-250	200-400	100-150	50-80
	Severe	> 600	< 100	> 400	> 150	>80

Surface Modulus

The plot of the surface modulus (E_0) can be used to provide an indication of the layer stiffness at different equivalent depths E_0 , at an equivalent depth (r), approximates a combined modulus of the underlying layers. For values of r that are greater than the total pavement equivalent thickness, E_0 is approximately equal to the subgrade modulus. The equations B2.8 for E_0 are shown below.

$$\begin{aligned} E_0 &= \left[\frac{2 \times (1 - \nu^2) \times \sigma_0 \times a}{d_r} \right] \\ E_r &= \left[\frac{(1 - \nu^2) \times \sigma_0 \times a^2}{r \times d_r} \right] \end{aligned} \quad , \dots \dots \dots (B2.8)$$

Where:

E_0 = Surface modulus at the center of the loading plate (MPa (lbf/inch²)).

E_r = Surface modulus at a distance r (MPa (lbf/inch²)).

ν = Poisson's ratio.

σ_0 = Contact pressure under the loading plate (MPa (lbf/inch²)).

a = Radius of loading plate (mm (inches)).

r = Distance from sensor to loading center (mm (inches)).

d_r = Deflection at distance r (mm (inches)).

The equation B2.9 for equivalent depth is shown below:

$$h_{e,n} = f_i \left[h_1 \sqrt[3]{\frac{E_1}{E_2}} + h_2 \sqrt[3]{\frac{E_2}{E_3}} + \dots h_{n-1} \sqrt[3]{\frac{E_{n-1}}{E_n}} \right] \quad , \dots \dots \dots (B2.9)$$

Where:

$h_{e,n}$ = Equivalent depth (mm (inches)).

f_i = Factor (0.8-1.0, depending on the modular ratio, thickness, and number of layers).

h_i = Thickness of layer i (mm (inches)).

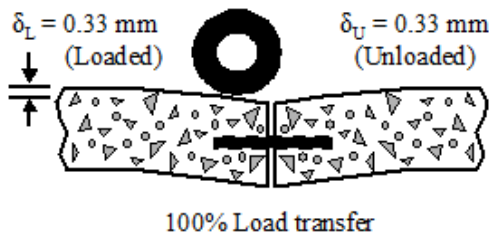
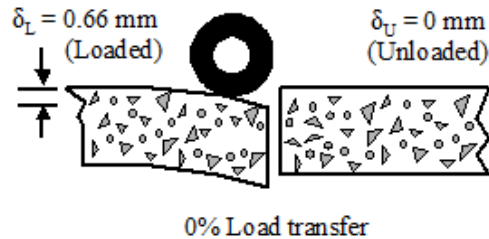
E_i = Stiffness modulus of layer i (MPa (lbf/inch²)).

E_n = Stiffness modulus of layer n (MPa (lbf/inch²)).

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LTE

LTE is a parameter that can be computed from deflection testing to characterize the ability of joints and cracks in rigid pavements to effectively transmit load from one side of the joint or crack to the next (see Figure B2.4). This can be done in the field with an FWD by applying a load on one side of the joint or crack and measuring the deflections on the loaded and unloaded slabs under that loading.



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1 mm = 39.3 mil.

δ_L = Deflection at loaded slab edge.

δ_U = Deflection at unloaded slab edge.

Figure B2.4 - Load Transfer concept

The equation B2.10 as shown below is used to express deflection-based LTE.

$$LTE = \beta \frac{d_u}{d_l} \text{ ,.....(B2.10)}$$

Where:

LTE = Load transfer efficiency (percent).

β = $d_{0center}/d_{12center}$, slab bending correction factor.

d_u = Deflection on the unloaded slab (mm (inches)).

d_l = Deflection on the loaded slab (mm (inches)).

In theory, the slab bending correction factor (β) is necessary because the deflections d_0 and d_{12} , measured 305 mm (12 inches) apart, would not be equal even if measured in the interior of the slab. However, this correction factor is somewhat controversial and is not always used.

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The LTE definition given above is based on deflections, but LTE is sometimes defined in terms of stress as shown by Equation B2.11 below.

$$LTE_{\sigma} = \frac{\sigma_u}{\sigma_l} 100, \dots \dots \dots (B2.11)$$

Where:

LTE_{σ} = Stress LTE (percent).

σ_u = Corresponding stress at the joint of the unloaded slab (MPa (lbf/inch²)).

σ_l = Maximum stress at the joint of the loaded slab (MPa (lbf/inch²)).

Because deflections can be easily measured in the field, and because stress-based LTE is much more affected by geometry of the applied load than deflection LTE, the deflection-based LTE is the more commonly used expression for LTE.

The theoretical deflection-based LTE ranges from 0 percent (no deflection on the unloaded slab) to 100 percent (equal deflections on the loaded and unloaded slabs). Generally speaking, the following guidelines can be used to define different levels of deflection LTE:

- **Excellent:** 90 to 100 percent.
- **Good:** 75 to 89 percent.
- **Fair:** 50 to 74 percent.
- **Poor:** 25 to 49 percent.
- **Very Poor:** 0 to 24 percent.

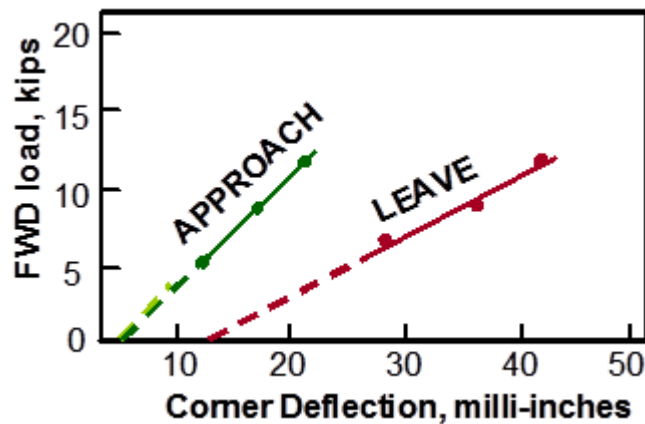
Void Detection

Pumping of underlying foundation materials (i.e., base, subbase, and subgrade) from beneath a concrete slab can lead to loss of support or voids at slab corners. Although small (typically 0.25 mm (0.01 inches) or smaller), these voids can lead to significant pavement deterioration, such as faulting and corner breaks.

One method of detecting voids beneath concrete slabs is based on the analysis of corner deflections under variable loads.⁽¹³⁾ In this method, corner deflections are measured at three load levels, and the results are plotted to establish a load-deflection relationship at each corner, as shown in figure 18, which is adapted from figure III-5 in *Joint Repair Methods for Portland Cement Concrete Pavements*.⁽¹³⁾

The figure illustrates an example in which, for the approach joint, the load-deflection line crosses the x-axis close to 0 at 0.051 mm (0.002 inches). For the leave joint, the load-deflection line crosses the deflection axis at a much greater distance away from the origin, indicating greater deflections under the same load. A line crossing the deflection axis at a point greater than 0.076 mm (0.003 inches or 3 mil) suggests the potential for a void under the slab.

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1 kip = 453.6 kg.

1 milli-inch (mil) = 0.0254 mm.

Figure B2.5 - Example of void detection plot using deflection data

To ensure that built-in curling of the concrete slab is not presenting a false indication of voids, deflection testing should not be conducted in the early morning when pavement slabs are typically exposed to negative temperature gradients. Higher midday temperatures should also be avoided during deflection testing to minimize the possibility of joint lockup and slab curl.

B2.7 Factors Affecting Deflections

A number of factors affect the magnitude of measured pavement deflections, which can make the interpretation of deflection results difficult. To the extent possible, direct consideration of these factors should be an integral part of the deflection testing process so that the resultant deflection data are meaningful and representative of actual conditions. Recognizing and accounting for these factors before the establishment of a field testing program helps ensure the collection of useful deflection data that can be used in subsequent back-calculation analyses. The major factors that affect pavement deflections include pavement structure (type and thickness), pavement loading (load magnitude and type of loading), and climate (temperature and seasonal effects). Each of these is discussed briefly in the following subsections.

B2.7.1 Pavement Structure

The deflection of a pavement represents an overall system response of the surface, base, and subbase layers, as well as the subgrade itself. Thus, the parameters of the surface layer (thickness and stiffness) and of the supporting layers (thickness and stiffness) all affect the magnitude of the measured deflections. Generally speaking, weaker systems deflect more than stronger systems under the same load, with the exact shape of the deflection basin related to the stiffness of the individual paving layer. Other pavement-related factors that can also affect deflections include the following:

- Testing near joints, edges, or cracks or in areas containing structural distress (such as alligator cracking), can produce higher deflections than testing at interior portions of the pavement.
- Random variations in pavement layer thickness can create variability in deflection.
- Variations in subgrade parameters and the presence of underlying rigid layers (such as bedrock/stiff layer or a high water table level) can provide significant variability in deflections.

B2.7.2 Pavement Loading

One of the most obvious factors that affects pavement deflections is the magnitude of the applied load. Most modern deflection equipment can impose load levels from as little as 13 kN (3,000 lbf) to more than 245 kN (55,000 lbf), and it is important to target appropriate load levels for each application. The type of loading can also affect pavement deflection—a slow, static loading condition produces a different response than a rapid, dynamic loading condition. In general, the more rapid the loading, the shorter the load pulse, and the smaller the deflections.

B2.7.3 Climate

Temperature is a very important factor that must be considered as part of any pavement deflection testing program. In HMA pavements, the stiffness of the asphalt layer decreases as the temperature increases, resulting in larger deflections. Therefore, correction of the measured deflections to a standard temperature (commonly 21 °C (70 °F)) is required to perform meaningful interpretations of the data. Deflections on PCC pavements are also affected by temperature because differences in temperature between the top and bottom of the slab cause the slab to curl either upward (i.e., when the slab surface is cooler than the slab bottom) or downward (i.e., when the slab surface is warmer than the slab bottom). If basin testing is conducted when the slab is curled down or if the corner testing is conducted when the slab is curled up, the slab could be unsupported and greater deflections may result. Temperature also affects joint and crack behavior in PCC pavements. Warmer temperatures cause the slabs to expand and, coupled with slab curling effects, may lock up the joints. Deflection testing conducted at joints when they are locked up results in lower joint deflections and higher load transfer efficiencies, which are misleading regarding the overall load transfer capabilities of the joint.

B2.7.4 Testing Season

Seasonal variations in temperature and moisture conditions also affect pavement deflection response. Generally speaking, deflections are greatest in the spring because of saturated conditions and reduced pavement support and are lowest in the winter when the underlying layers and subgrade are frozen. PCC pavements are less affected by seasonal variations in support conditions.

B2.8 FWD Testing Guidelines

The guidelines discussed in the following subsections are related to the physical testing equipment configuration (such as sensor locations and load levels), as well as the type and location of deflection data that are obtained during FWD testing. The discussion of equipment configuration is as generic as possible but may reflect specific capabilities found in the Dynatest® FWD equipment because this equipment is used in the Long-Term Pavement Performance (LTPP) Program.

B2.9 Sensor Configuration

The LTPP Program's nine-sensor configuration is recommended for most routine roadway testing, but other configurations are also acceptable as long as the sensor configuration is known when analyzing the deflection data. The advantage to nine sensors is the ability to perform PCC joint or crack LTE testing without relocating a sensor from the HMA testing configuration. Table 4 presents commonly used seven- and nine-sensor LTPP configurations.

Table B2.4 - Summary of LTPP Deflection Sensor Locations, Sensor Offset

Deflection Sensor	Nine Sensors mm (inches)	Seven Sensors (HMA) mm (inches)	Seven Sensors (PCC) mm (inches)
D1	0	0	0
D2	203 (8)	203 (8)	-305 (-12)

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D3	305 (12)	305 (12)	305 (12)
D4	457 (18)	457 (18)	457 (18)
D5	610 (24)	610 (24)	610 (24)
D6	914 (36)	914 (36)	914 (36)
D7	1219 (48)	1219 (48)	1219 (48)
D8	1524 (60)	N/A	N/A
D9	-305 (-12)	N/A	N/A

N/A = Not applicable.

B2.10 Number of Drops and Load Levels

The LTPP Program recommends multiple drops at different load levels for both HMA and PCC pavements. The different load levels vary the mass of the weight package or release it from different heights. The designated drop heights, target load, acceptable load range, and drop sequence for each pavement type are summarized in table 5.

Table B2.5 - Summary of LTPP load levels and testing (drop) sequence

Height Designation	Target Load kN (lbf)	Acceptable Range kN (lbf)	No. of HMA Drops ^a	No. of PCC Drops ^a
Seating ^b	N/A	N/A	3	3
1 ^c	26.7 (6000)	24.0-29.4(5,400-6,600)	4	N/A
2	40.0 (9000)	36.0-44.0(8,100-9,900)	4	4
3	53.4 (12000)	48.1-58.7(10,800-13,200)	4	4
4	71.2 (16000)	64.1-78.3(14,400-17,600)	4	4

^aThe last drop of each recorded set contains full load history data.

^bSeating drop data are not recorded in project data; drop is performed at height 3.

^cHeight 1 is not used for testing PCC pavements.

N/A = Not applicable.

The multiple drops per load level allows checking of uniformity (or variation) of the applied load and deflections. Multiple load levels also allow evaluation of nonlinear material behavior and, for PCC pavements, can be used to evaluate the potential for voids beneath slab corners. The LTPP Program testing protocol also requires seating drops (data are not collected) and a complete time history of the drop is required for the fourth drop in the testing sequence.

In deflection testing outside of the LTPP data collection program, multiple drops at each load level are often not performed so that testing productivity is increased and lane closure times are reduced. ASTM D4694, "Standard Test Method for Deflections with a Falling-Weight-Type Impulse Load Device," recommends that at least two drops be performed, whereas the American Association of State Highway and Transportation Officials (AASHTO) T 256, *Standard Method of Test for Pavement Deflection Measurements* and ASTM D4695, "Standard Guide for General Pavement Deflection Measurements" suggests one or more drops at any load level. AASHTO T 256 and ASTM D4695 also indicate that seating drops should be recorded for the analysis of pavement conditioning and further suggests that multiple load levels be used to evaluate nonlinear behavior.

Based on review of the various testing protocols and studies, a test sequence of four drops at varying load magnitudes is recommended. The first drop should be a seating drop, and the next three drops should be recorded data at 27, 40, and 53-kN (6,000, 9,000, and 12,000 - lbf) target loadings. This test sequence reduces the time at each test location, allows assessment of nonlinear material behavior, and can be used for evaluating the potential for voids under PCC pavements, but

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it does not allow for repeatability analysis. Moreover, the use of the 27- to 53-kN (6,000-12,000-lbf) load range is recommended because heavier loadings often result in lower back-calculated moduli for granular and subgrade materials.

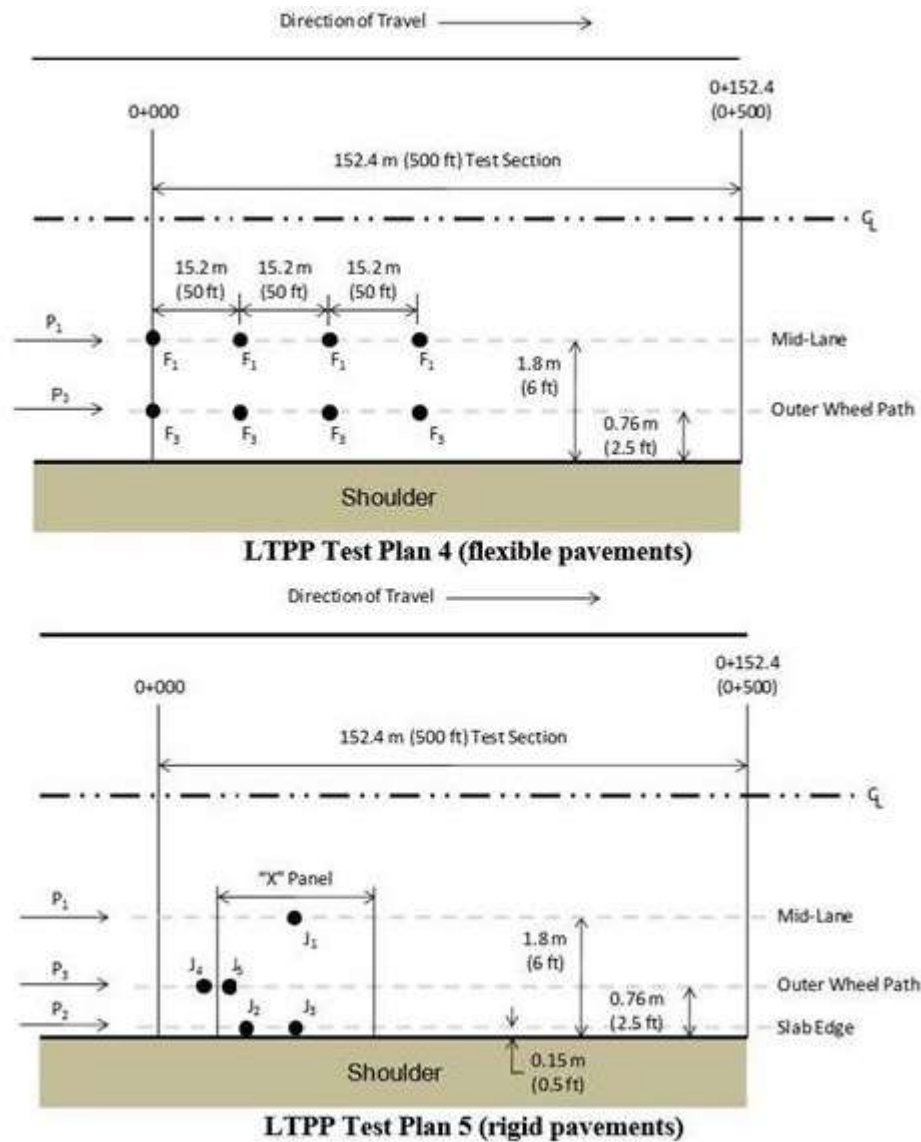
The LTPP Program drop sequence presented in Table B2.5 should be considered for some test locations to provide repeatability analysis, such as at the beginning of testing, end of testing, and every 100 test locations (or a minimum of 3 repeatability test locations per project). Recording the time history for at least the last drop is recommended.

B2.11 Testing Location

FWD testing locations generally consist of basin tests for flexible and rigid pavements and tests at joints (either midpanel along the joint or at the slab corner) or cracks for rigid pavements. Basin tests are used for back-calculating pavement layer parameters and are generally taken at non-distressed areas for flexible pavement and at midpanel (non-distressed) locations for rigid pavements. However, the MEPDG recommends that FWD testing be conducted at distressed HMA areas as well to determine the “damaged” modulus.

FWD testing is generally performed in the outermost lane (adjacent to the shoulder) for roadways with multiple lanes in one direction. The LTPP Program developed 11 test plans based on the experiment type (general or specific) and pavement type. Testing layouts similar to test plans 4 and 5 of the LTPP data collection guidelines (see Figure B2.6) are recommended. Note that flexible pavement testing includes two lanes of basin tests, one midlane, and one in the outer wheelpath, while rigid pavement testing is also conducted at midlane and in the outer wheelpath but also includes load transfer and corner testing in addition to basin tests. For two-lane roadways, consideration can be given to staggering the test points by directional lane, assuming that the traffic levels are directionally similar. This can provide efficient testing coverage of the pavement project but does require additional traffic control planning and setup.

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P_1 , P_2 , P_3 = Pass through mid-lane, pavement edge, and outer wheel path, respectively.

F_1 , F_3 = Measurement location along P_1 and P_3 , respectively.

J_1 , J_2 , J_3 , J_4 , J_5 = Measurement location along P_1 —mid-panel, along P_2 —corner, along P_2 —mid-panel, along P_3 —joint approach, and along P_3 —joint leave, respectively.

CL = Center line.

Figure B2.6 - Illustration of Flexible and Rigid Pavement Test Plans

AASHTO T 256 and ASTM D4695 indicate testing can be conducted either at midlane or in the outer wheelpath or both, and for rigid pavements suggests that a minimum of 25 percent of the joints associated with the basin tests should be tested. Furthermore, for a detailed project analysis of a rigid pavement, AASHTO T 256 and ASTM D4695 recommend a closer basin testing interval and that all joints corresponding to basin tests should be tested.

B2.12 Testing Increments

Testing increments are typically different for network-level and project-level evaluation. Network-level testing is commonly performed to obtain a general indication of the load-carrying capacity of the pavement structure as a means of identifying and prioritizing projects for maintenance and rehabilitation. Studies by several highway agencies suggest that testing intervals of between two and three points per 1.6 km (per mi) are adequate for network-level analyses.

For project-level testing, much closer testing intervals are required to better characterize the pavement structure. The 11 testing plans developed for the LTPP Program testing show testing intervals of 7.6, 15.2, or 30.5 m (25, 50, or 100 ft) for flexible pavements and intervals of every 10 or 20 slabs for rigid pavements. (Note that if a crack is present near midpanel, the slab is considered two effective slabs.) However, these intervals are for relatively short pavement test sections (generally 150 m (500 ft) or less). More universal guidance is offered by the MEPDG, AASHTO T 256, and ASTM D4695, which suggests basin test spacing of 30 to 150 m (100 to 500 ft) for project-level investigations. For joint testing, the MEPDG recommends that testing should be performed across joints (or cracks) every 30 to 150 m (100 to 500 ft) and also suggests that depending on the length of the project and the availability of resources, the increment can be increased to every 305 m (1,000 ft). In addition, it is also recommended that a minimum of 12 to 15 tests be conducted per uniform test section.

B2.13 Temperature Measurements

Temperature measurements should be collected during FWD testing. Because HMA is a temperature-dependent material, the modulus obtained during back-calculation represents the material's temperature at the time of testing. Having accurate temperature data helps determine the correction factor to apply to the back-calculated HMA modulus to obtain a value at a standard temperature (typically 21 °C (70 °F)) for use in design.

FWD testing on PCC pavements must consider the temperature at the time the testing is conducted. Ideally, testing should be performed at a time (typically night or early morning hours) when the slab is in a neutral or flat condition (that is, the edges or center are not potentially lifted off the base). However, this may be impractical for an agency that must test many kilometers (miles) of pavement every day. In general, deflection testing on PCC pavements should be conducted when the ambient temperature is below 27 °C (80 °F). Although the back-calculation procedures for PCC pavements do not currently incorporate temperature corrections, temperature measurements are also useful in evaluating back-calculation results for PCC pavements, particularly in terms of whether the slabs are exhibiting any curling that may be affecting the results. In addition, knowledge of the temperature conditions at the time of testing assists in evaluating LTE data.

Air and Surface Temperature

Air and pavement surface temperatures should be recorded at each test location, and most FWD equipment has temperature sensors and operating software that record the data automatically. Air and surface temperatures can be used in procedures to estimate the mean temperature of the pavement but direct measurement is generally preferred. The daily average temperatures for the 5 days preceding testing should also be obtained, particularly if the air and surface temperatures will be used to predict the mean pavement temperature.

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Temperature Gradients

The LTPP Program testing includes measuring the temperature gradient within the pavement surface layer. This is accomplished by drilling holes to varying depths and measuring the temperatures with thermometers. The LTPP Program uses up to five holes drilled to depths summarized in Table B2.6. (Note that hole depths that extend into the unbound layer may be eliminated, and the deepest hole should be drilled 25.4 mm (1 inch) above the bottom of the bound layer.)

Table B2.6 - LTPP Program Temperature Measurement Hole Depths

Hole Number	Hole Depth, mm (inches)
1	25.0 (1.0)
2	50.0 (2.0)
3	100.0 (4.0)
4	200.0 (8.0)
5	300.0 (12.0)

Holes are generally drilled at one end of the project section in the outer wheelpath, and temperature readings are obtained at the beginning and end of the testing, as well as at selected intervals. Although the LTPP Program recommends retrieving temperatures every 30 min, this may not be practical given the time restraints of many project site closures; temperature readings every 1 h are recommended as a more practical interval. A minimum of three temperature readings, roughly correlated with the beginning, middle, and end of testing, should be obtained for smaller projects with shorter testing times.

When direct measurement of the temperature gradient is performed, the air and surface temperatures should also be taken at the temperature holes. This allows correlation of the air and surface temperatures at each test location to the measured mean pavement temperature.

B2.14 Joint/Crack Opening

The LTPP Program recommends collecting joint (and crack) width measurements at a minimum of 25 percent of the joint (or crack) deflection testing locations; however, if time allows, measurement at all testing locations is preferred. For joints, the saw cut width is measured, and for cracks, the width of the full-depth crack (not necessarily the surface width) is measured. Joint/crack measurements can be reviewed during the analysis of LTE. In general terms, a tight joint/crack should have higher LTEs.

B2.15 Safety Guidelines

Safety during FWD testing applies to the operation of the equipment and working in (or adjacent to) moving traffic. The FWD includes high-pressure hydraulics, electronics, and heavy moving parts that create many potential work hazards. Equipment manufacturers provide extensive documentation on the operation and maintenance of the testing equipment, and it is strongly recommended that operators become familiar with the documented materials and are well trained with the equipment.

Working around moving traffic can be a hazardous situation regardless of the work activity. Traffic control measures and work zone requirements must adhere to the guidelines of the governing agency.

B2.16 Summary of FWD Testing Recommendations

Table B2.7 provides an overall summary of the FWD testing recommendations described in the previous subsections. Recommendations include sensor configuration, load levels and drops, testing locations, testing increments, and temperature measurements.

Table B2.7 - Summary of Deflection Testing Recommendations

Testing Component	HMA Pavements Recommendation	PCC Pavements Recommendation
Sensor Configuration	0, 203, 305, 457, 610, 914, 1,219, 1,524, -305 (0, 8, 12, 18, 24, 36, 48, 60, -12)	0, 203, 305, 457, 610, 914, 1,219, 1,524, -305 (0, 8, 12, 18, 24, 36, 48, 60, -12)
Load Level kN (lbf)	Seating, 26.7, 40.0, and 53.4 (6,000, 9,000, and 12,000)	Seating, 40.0 and 53.4 (9,000 and 12,000)
Number of Drops	One for each load level	One for seating, 9,000- and 12,000-lbf (40- to 15.2-kN) load levels
Testing Locations	<ul style="list-style-type: none"> Testing in outer traffic lane on multiple lane facilities Possible directionally staggered testing on two-lane facilities Midlane and outer wheelpath 	<ul style="list-style-type: none"> Testing in outer traffic lane on multiple lane facilities Possible directionally staggered testing on two-lane facilities Midlane, outer wheelpath, and transverse joint
Testing Increments, General	12 to 15 tests per uniform pavement section, at 30.5- to 152.4-m (100- to 500-ft) intervals	12 to 15 tests per uniform pavement section, at 30.5- to 152.4-m (100- to 500-ft) intervals
Testing Increments, Project Level	25- to 50-ft (7.62- to 15.24-m) intervals	25- to 50-ft (7.62- to 15.24-m) intervals
Temperature Measurements, air and surface	Measure at each test location	Measure at each test location
Temperature measurements, in pavement	Measure at 1-h intervals at depths of 25.0, 50.0, 100.0, 200.0 and 300.0 mm (1, 2, 4, 8, and 12 inches)	Measure at 1-h intervals at depths of 25.0, 50.0, 100.0, 200.0 and 300.0 mm (1, 2, 4, 8, and 12 inches)

B2.17 Data Checks

Types of Errors

The following data checks should be enabled in the FWD data collection software to flag certain conditions suggestive of errors or problems:

- Roll-off (i.e., deflection sensor does not return to near 0 within 60 ms of the trigger activation):** The roll-off error can occur when there is poor contact between the pavement surface and the deflection sensor or when the magnitude of the deflection approaches the resolution of the geophone.
- Non-decreasing deflections (i.e., deflection measurements do not decrease with increasing distance from the load plate):** This error may occur if a transverse crack or other discontinuity exists between two adjacent geophones or on very stiff PCC pavements if the deflection difference is less than the random error inherent to the deflection sensor. If neither condition exists, this error can typically occur owing to poor seating between the sensor and the pavement surface.

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- **Overflow (i.e., measured deflections exceed the range of the deflection sensor):** In general, this error is only expected with extremely weak pavements or when testing at extremely high load levels. (See next section for a discussion on dealing with deflection errors.)
- **Load variation (the peak load for multiple drops at the same height varies by more than $\pm (0.18 \text{ kN} + 0.02 \text{ kN} \times \text{applied load})$ ($\pm (40.5\text{-lbf} + 0.02 \times \text{applied load})$):** This condition may occur on an extremely weak pavement if the FWD testing has damaged the pavement structure, when testing during a spring thaw or if the load plate is not properly seated on the pavement surface. (See next section for a discussion on dealing with load errors.)
- **Deflection variation (the load-normalized peak deflections from repeat drops varies by more than $\pm (2 \text{ }\mu\text{m} + 0.01\mu\text{m} \times \text{deflection})$ ($\pm (0.079 \text{ mil} + 0.01 \times \text{measured deflection})$):** This error can occur for pavement structures that are unaffected by FWD testing (e.g., extremely weak pavements or the unbound layers of the pavement are saturated). In addition, it can be caused by poor seating of either the load plate or the deflection sensors, or possibly by vibration caused by heavy equipment (e.g., trucks traveling in an adjacent lane) in the vicinity of the FWD test. (See next subsection for discussion on dealing with deflection errors.)

Addressing Deflection Errors

When deflection errors are encountered during FWD testing, the following steps are recommended to resolve the issue:

1. Verify the condition of FWD by ensuring the deflection sensor(s) is seated securely to the sensor holder(s), all screws holding the sensor magnet and sensor holder are tight, and the holder springs and foam bushing are in good shape. If multiple sensors have errors, check all analog connections.
2. Verify the pavement condition by ensuring the sensor holder is not resting on a loose stone or crack.
3. Reject the original data and repeat the test without moving the FWD.
4. If the error persists and the FWD can be repositioned, move forward 0.6 m (2 ft) and retest. If the error still persists, accept the data and note that the error could not be resolved.
5. If the error persists and the FWD cannot be repositioned (e.g., load transfer test), accept the test and note that the error could not be resolved.

Addressing Load Errors

If load errors are experienced, the following steps are recommended to resolve the issue:

6. Reject the data and retest without repositioning the FWD.
7. If the error persists, check all analog connections to ensure the weight/height targets are tight, raise the load plate and ensure the swivel moves easily, and ensure the rubber sheet and pavement surface beneath the load plate are clear of debris.
8. Reject the data and repeat the test without repositioning the FWD.
9. If the error persists and the FWD can be repositioned, move forward 0.6 m (2 ft) and retest. If the error still persists, accept the data and note that the error could not be resolved.

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10. If error persists and the FWD cannot be repositioned (e.g., load transfer test), accept the test and note that the error could not be resolved.

B2.18 Additional Sources for Deflection Testing Guidelines

In addition to the LTPP Program (Federal Highway Administration (FHWA)), AASHTO, and ASTM International documents that are cited as primary source documents in this chapter, a number of additional sources provide guidance on FWD testing and data collection. These include documents prepared by NCHRP, the Department of Defense, the Federal Aviation Administration, and the European Commission Directorate General Transport. (See references 4, 21, 22, and 11.) In addition, many highway agencies have developed their own custom FWD testing procedures and protocols.

B2.19 FWD Calibration

Routine FWD calibration is a vital component to ensure accurate loading and deflection measurements. As outlined in AASHTO R32-09, FWD calibration should include the following:

- **Annual calibration of the load cell and deflection sensors using an independently calibrated reference device (referred to as reference calibration):** Deflection sensors are also compared with each other (referred to as relative calibration). Annual calibration should also be conducted as soon as possible after load cell or deflection sensor replacement. Annual calibration is performed by a certified technician.
- **Monthly relative calibration of the deflection sensors:** Monthly deflection sensor calibration is conducted using a relative calibration stand supplied by the FWD manufacturer and is different than the relative calibration conducted during annual calibration. Relative calibration should also be conducted immediately after replacement of a deflection sensor. Relative calibration does not require a certified technician.

B2.20 Summary

This chapter presents an overview of deflection testing. Pavement deflection testing is recognized as a reliable, quick, and inexpensive method for determining the structural condition of existing pavements. Specifically, deflection measurements can be used for back-calculating the elastic moduli of the pavement structural layers and for estimating the load-carrying capacity of both HMA and PCC pavements. In addition, in PCC pavements, loss of support at slab corners can be identified and evaluation of the joint or crack load transfer can be performed using deflection testing.

Pavement deflections represent an overall system response of the pavement structure and subgrade soil to an applied load. The major factors that affect pavement deflections can be grouped into categories of pavement structure (type and thickness), pavement loading (load magnitude and type of loading), and climate (temperature and seasonal effects). Consideration of these factors should be an integral part of the deflection testing process so that the resultant deflection data are meaningful and representative of actual conditions.

Overall recommendations for setting up a FWD testing program are presented, including sensor configuration, loading levels and drop sequencing, testing locations and intervals, and temperature measurements. In addition, the types of errors commonly encountered during FWD testing are briefly described, along with ways of addressing these items during the testing program.

B3 Road Roughness Survey

B3.1 General

Pavement roughness is generally defined as an expression of irregularities in the pavement surface that adversely affect the ride quality of a vehicle (and thus the user). Roughness is an important pavement characteristic because it affects not only ride quality but also vehicle delay costs, fuel consumption and maintenance costs. The World Bank found road roughness to be a primary factor in the analyses and trade-offs involving road quality vs. user cost (UMTRI, 1998 [1]). Roughness is also referred to as “smoothness” although both terms refer to the same pavement qualities.



Figure B3.1 - Road Roughness Survey

For detailed information, the University of Michigan Transportation Research Institute (UMTRI) maintains an excellent Web page on roughness and all related aspects at <http://www.umtri.umich.edu/divisionPage.php?pageID=62>.

B3.2 Measurement

Today, roughness is typically quantified using some form of either present serviceability rating (PSR), international roughness index (IRI) or other index with IRI being most prevalent.

B3.3 Present Serviceability Rating (PSR)

The AASHO Road Test (Highway Research Board, 1972[2]) developed a definition of pavement serviceability, the present serviceability rating (PSR), that is based on individual observation. PSR is defined as “The judgment of an observer as to the current ability of a pavement to serve the traffic it is meant to serve” (Highway Research Board, 1972[2]). To generate the original AASHO Road Test PSR scores, observers rode around the test tracks and rated their ride using the quantitative scale shown in Figure B3.1. This subjective scale ranges from 5 (excellent) to 0 (essentially impassable). Since PSR is based on passenger interpretations of ride quality, it generally reflects road roughness because roughness largely determines ride quality.

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Acceptable?		5		Very Good
		4		Good
		3		Fair
		2		Poor
		1		Very Poor
		0		
Yes	<input type="checkbox"/>			
No	<input type="checkbox"/>			
Undecided	<input type="checkbox"/>			
Section Identification _____ Rating _____				
Rater _____ Date _____ Time _____ Vehicle _____				

Figure B3.2 - Reproduction of an Individual Present Serviceability Rating

B3.4 International Roughness Index (IRI)

The international roughness index (IRI) was developed by the World Bank in the 1980s (UMTRI, 1998 [1]). IRI is used to define a characteristic of the longitudinal profile of a travelled wheel-track and constitutes a standardized roughness measurement. The commonly recommended units are meters per kilometre (m/km) or millimetres per meter (mm/m). The IRI is based on the average rectified slope (ARS), which is a filtered ratio of a standard vehicle's accumulated suspension motion (in mm, inches, etc.) divided by the distance travelled by the vehicle during the measurement (km, mi, etc.). IRI is then equal to ARS multiplied by 1,000. The open-ended IRI scale is shown in Figure B3.2.

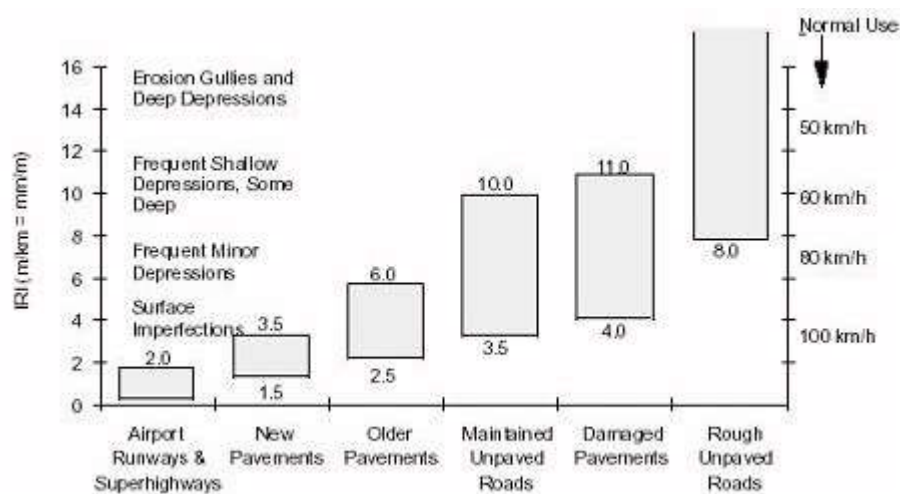


Figure B3.3 - IRI Roughness Scale (Replotted from Sayers et al., 1986)

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B3.5 Correlations Between PSR and IRI

Various correlations have been developed between PSR and IRI. Two are presented here. One was reported in 1986 by Paterson:

$$PSR = 5e^{-0.18 (IRI)}$$

Another correlation was reported in a 1992 Illinois funded study performed by Al-Omari and Darter (1992[3]):

$$PSR = 5e^{-0.26 (IRI)}$$

This study used data from the states of Indiana, Louisiana, Michigan, New Mexico, and Ohio for both flexible and rigid pavements. The associated regression statistics are $R^2 = 0.73$, $SEE = 0.39$, and $n = 332$ sections. Correlations are highly dependent upon the data that are used.

B3.6 Measurement Techniques

The equipment for roughness survey data collection can be categorized into the four broad categories shown in Table B3.1.

Table B3.1 - Roughness Measuring Equipment

Equipment / Technique	Complexity
Rod and level survey	Most Simple
Dipstick profiler	Simple
Profilographs	Simple
Response type road roughness meters (RTRRMs)	Complex
Profiling devices	More Complex

The following discussion with a few modifications was taken directly from the "Pavement Condition Data Collection Equipment" article in the FHWA Pavement Notebook (1989[4]).

B3.7 Survey

A survey (performed by a survey crew) can provide an accurate measurement of the pavement profile. The use of surveys for large projects, however, is impractical and cost prohibitive.



Figure B3.4 - Dipstick 2000.



Figure B3.5 - Dipstick Operation

B3.8 Dipstick Profiler

The dipstick profiler can be used to collect a relatively small quantity of pavement profile measurements. The Dipstick Profiler (see Figures B3.3 and B3.4) consists of an inclinometer enclosed in a case supported by two legs separated by 305 mm (12 in.). Two digital displays are provided, one at each end of the instrument. Each display reads the elevation of the leg at its end relative to the elevation of the other leg. The operator then “walks” the dipstick down a pre-marked pavement section by alternately pivoting the instrument about each leg. Readings are recorded sequentially as the operator traverses the section. The device records 10 to 15 readings per minute. Software analysis provides a profile accurate to ± 0.127 mm (± 0.005 in.). A strip can be surveyed by a single operator in about one-half the time of a traditional survey crew. The dipstick is commonly used to measure a profile for calibration of more complex instruments.

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B3.9 Profilographs

Profilographs have been available for many years and exist in a variety of different forms, configurations, and brands. Due to their design they are not practical for network condition surveys. Their most common use today is for rigid pavement construction inspection, quality control, and acceptance. The major differences among the various profilographs involve the configuration of the wheels and the operation and measurement procedures of the various devices.

Profilographs have a sensing wheel, mounted to provide for free vertical movement at the center of the frame (Figure B3.5). The deviation against a reference plane, established from the profilograph frame, is recorded (automatically on some models) on graph paper from the motion of the sensing wheel (Figure B3.6). Profilographs can detect very slight surface deviations or undulations up to about 6 m (20 ft) in length.



Figure B3.6 - Profilograph



Figure B3.7 - Profilograph Output

B3.10 Response Type Road Roughness Meters (RTRRMs)

The third category of roughness data collection equipment is the response type road roughness meters (RTRRMs), often called “road meters”. RTRRM systems are adequate for routine monitoring of a pavement network and providing an overall picture of the condition of the network. The output can provide managers with a general indication of the overall network condition and maintenance needs.

RTRRMs measure the vertical movements of the rear axle of an automobile or the axle of a trailer relative to the vehicle frame. The meters are installed in vehicles with a displacement transducer on the body located between the middle of the axle and the body of a passenger car or trailer. The transducer detects small increments of axle movement relative to the vehicle body. The output data consists of a strip chart plot of the actual axle body movement versus the time of travel.

The disadvantage of a RTRRM is that its measured axle body movement vs. time depends on the dynamics of the particular measurement vehicle, which results in two unwanted effects (UMTRI, 1998[1]):

- **Roughness measuring methods have not been stable with time.** Measures made today with road meters cannot be compared with confidence to those made several years ago.
- **Roughness measurements have not been transportable.** Road meter measures made by one system are seldom reproducible by another.

Because of these two effects, profiling devices are becoming more popular.

B3.11 Profiling Devices

Profiling devices are used to provide accurate, scaled, and complete reproductions of the pavement profile within a certain range. They are available in several forms, and can be used for calibration of RTRRMs. The equipment can become fairly expensive and complex. Three generic types of profiling systems are in use today:

- **Straight edge.** The simplest profiling system is a straight edge. Modifications to the straight edge, such as mounting it on a wheel, result in a profilograph.
- **Low speed systems.** Low speed systems such as the CHLOE profilometer are moving reference planes. The CHLOE is a long trailer that is towed at low speeds of 3 to 8 kph (2 to 5 mph). The slow speed is necessary to prevent any dynamic response measurement during the readings. A few agencies still use the CHLOE to calibrate their RTRRMs.
- **Inertial reference systems.** Most sophisticated road profiling equipment uses the inertial reference system. The profiling device measures and computes longitudinal profile through the creation of an inertial reference by using accelerometers placed on the body of the measuring vehicle to measure the vehicle body motion. The relative displacement between the accelerometer and the pavement profile is measured with either a “contact” or a “non-contact” sensor system.

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The earliest profiling devices used a measurement system in direct contact with the pavement to measure profile. Several contact systems have been used, and are still in use today. The French Road Research Laboratory developed the Longitudinal Profile Analyzer (APL) in 1968.

Systems used today in the United States are frequently installed in vans (Figure 7) which contain microcomputers and other data handling and processing instrumentation. Older profiling devices are usually contact systems, while the more recently manufactured devices use non-contact sensors. The non-contact systems use probes, either acoustic or light, to measure differences in the pavement surface.

For instance, the South Dakota road profiler simultaneously collects three ultrasonic profiles, one for each wheelpath and one for the lane center. These profiles are used to calculate (by computer) a mathematical measure of roughness and an estimate of rutting at specified intervals along the roadway. A hybridized South Dakota road profiler combines the three ultrasonic sensors with two laser sensors, one for each wheelpath, for simultaneous measurement of the same roadway by two different sensor types under identical conditions (Virginia Transportation Research Council, 1996 [5]). Integrated analysis units can continuously collect a wide variety of data at highway speeds such as:

- Transverse profile/rutting
- Grade, cross-slope
- Pavement texture
- Pavement condition or distress
- GPS coordinates
- Panoramic right-of-way video
- Pavement video
- Feature location



Figure B3.8 - South Dakota Road Profiler (van-mounted)

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Figure B3.9 - Imaging Survey Van is used by the State Materials Office of the Florida DOT. The profiler is the grey/silver box attached on the front bumper

B3.12 Summary of Measurement Devices

A summary of the most commonly used roughness data collection devices, their measurement principles, relative costs, relative degrees of accuracy, and current and projected future use is contained in Table B3.2.

Table B3.2 - Roughness Data Collection Equipment (from FHWA, 1989[4])

Roughness Data Collection Device	Principle of Measurement	Relative Initial Cost	Relative Data Collection Cost (Network)	Relative Degree of Accuracy	Approx. Time of Development	Extent of Current Use	Projected Extent of Use
Dipstick	Direct Differential measurement	Low	impractical	Very High	1980s	Limited, Used for Calibration	Same as Current Use
Profilographs	Direct Profile Recording	Low	impractical	Medium	1960s	Extensive for Const. Acceptance	Same as Current Use
BPR Roughometer	Device Response	Low	Low	Medium	1940s	Limited	None
Mays Meter	Vehicle Response	Low	Low	Medium	1960s	Extensive	Decreasing Continuously
South dacota Road Profiler	Direct Profile Recording	Low	Low	High	1980s	Growing	Rapidly Increasing
Contact Profiling Dev.	Direct Profile Recording	High	Medium	Very High	1970s	Limited	Decreasing
Non-Contact Profiling Dev.	Direct Profile Recording	High	Medium	Very High	1980s	Medium	Increasing Continuously

B4 Axle Load Traffic Survey

B4.1 General

The axle loads and gross vehicle weights (GVW) of commercial vehicles can be obtained either by static or dynamic method. Many equipment are available in different parts of the world for measuring axle loads but few equipment like Transport and Road Research Laboratory (TRRL) weigh bridge, Wheel Weighing Pad etc. are commonly used.

B4.1.1 Static Method

In this method vehicles are weighed in halt position. TRRL portable weigh bridge and wheel weighing pad are the commonly used equipment in this method.

A portable weigh bridge used to measure the wheel load was designed by overseas unit of the Transport and Road Research Laboratory (Refer to Figures 2.1 and 2.2). It consists of a portable weighing platform of dimensions 70cm x 50cm x 9cm connected to a dial readout unit. The unit is operated by a standard 12 volt battery and has a load range of 0 – 10,000 kg.

The loading platform is located in the road shoulder, adjacent to the pavement. Special concrete pits are constructed for better positioning. Normally, a level area for a minimum length of 2 metres on either side of the platform is provided to ensure that all the wheels of the vehicle weighed were on a plane. The weighing unit permits only one wheel (single or twin tired assembly) to be weighed at a time. Each vehicle to be weighed is aligned so that all wheels on one side of the vehicle are in line with the weighing unit. The vehicle is then driven slowly to the weighing platform and stopped with the wheel to be weighed on the centre of the platform. The vehicle is stopped just long enough to show a stable trace on the chart recorder. The procedure is the repeated with the next wheel. It is assumed that load on each axle is evenly distributed and the axle load is taken to be twice the wheel load. Total vehicle weights are obtained by summing weight on the individual axles.



Figure B4.1 – Static Method of Axle Load Survey - Wheel Weighing Pad

The wheel weighing pad consists of two high quality cover plates of aluminium alloy having high stiffness and anti-corrosive properties. Two load bars are sandwiched between these cover plates. The weighing pad is connected to the indicator unit which gets power through a 12V potable maintenance free rechargeable battery. The indicator gives instant display of the applied load. It also indicates the case of overloading or not functioning of the equipment, which enables sorting out the same during the operation or calibration.

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The wheel weighing pad needs to be calibrated before use in the field to determine correction factor for each unit. The weighing procedure is to place the wheel weighing pad on the road near the edge of the pavement. Each vehicle is allowed slowly to the weighing pad with the wheel(s) to be weighed positioned on the pad. All wheels on one side of the vehicle are weighed and the axle loads are assumed to be twice the individual wheel loads as in case of TRRL portable weigh bridge.

The modified wheel weighing pad, in which the indicator was in-built within the weighing pad itself are also in the use. There is no need for calibration of these weighing pads while using them. P1 shows the modified wheel weighing pad.

B4.1.2 Dynamic Method

In this method, axle loads of vehicles are obtained without stopping or diverting the vehicle from other traffic. The axle loads are recorded automatically while vehicles are moving at permanent weighing stations. This technology is said to be the 'Weight-in-Motion (WIM)' technology. The WIM technology has many advantages, the chief amongst them being fewer surveying facilities, improved safety to through traffic and surveying staff, automatic processing of truck weight data, full coverage and easy transferability of data to computer system. In addition there would be no visible arrangement for stopping or diverting traffic. The chances of drivers trying to avoid measurements are also reduced.

Several types of dynamic weighing equipment are now available throughout the world. In its simplest form, the system consists of one or more floating platforms of concrete or steel supported on strain-gauge load cells. Each cell is incorporated into a wheat-stone bridge electrical resistance network such that the electrical potential difference of the output terminals of the network is proportional to the compression force. These systems, however, are massive and due to their inertia, cannot respond to rapid changes. Difficulty in levelling, lateral translation of the platform, moisture effects on the load cells and massive construction required for installation are some of the drawbacks of this system.



Figure B4.2 – Dynamic Method of Axle Load Survey – Weigh in Motion

To overcome these difficulties, portable light-weight weighing platform on strain-gauge load cells have been developed now. The German design uses strain-gauges bonded to the underside of the steel plate 16mm thick x 1.2m x 0.5m and requires only 50mm deep excavation for installation. Another WIM system developed in Germany works on hydraulic displacement concept, in which 2-3mm space between steel diaphragm works on hydraulic displacement concept, in which 2-3mm space between steel diaphragm plates is filled with a fluid and fluid displaced under load is used as a measure of load imposed. The French WIM system uses three piezometric quartz crystals to

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support a weighing platform and uses the electrical signal generated under load to interact the weight. Another French system consists of a co-axial cable filled with pressure sensitive piezo-electric ceramic material. The cable is placed in a groove, cut across the lane of travel and covered with sealing material. The load is transmitted through the road surface to the cable, causing it deform and the change in pressure generates an electric charge on the conductive surface of the cable. The accumulated charge is used to produce a voltage difference that is proportional to the applied load.

Table B4.1 – Equivalence Factors (Damaging Power) of Different Single Axle Loads

Axle Load		Equivalence Factor
kg	lbs	
910	2000	0.0002
1810	4000	0.0025
2720	6000	0.01
3630	8000	0.03
4540	10000	0.09
5440	12000	0.19
6350	14000	0.35
7260	16000	0.61
8160	18000	1.00
9070	20000	1.50
9980	22000	2.30
10890	24000	3.20
11790	26000	4.40
12700	28000	5.80
13610	30000	7.60
14520	32000	9.70
15420	34000	12.10
16320	36000	15.00
17230	38000	18.60
18140	40000	22.80

B4.2 The Relative Damaging Effects of Axles

Over past 60 years various attempts have been made to quantify the damaging effect of axles in terms of their loading. The most comprehensive data has been obtained from the AASHO Road Test completed in the USA in 1962. This showed that the damaging effect of the axles on both concrete and flexible pavements was approximately proportional to the fourth power of the axle load.

The major object of the AASHO Road Test was to establish, using normal road construction and normal road vehicles, the relative damaging effects of a wide range of axle loads. The number of passages of each axle load required to cause the same damage as one passage of a standard axle load, which is 18000 lbs or 8160 kg for single axles and 33500 lbs or 15200 kgs for tandem axles, was expressed as an equivalence factor. These equivalence factors given in Table B4.1 and Table B4.2 represent average of more detailed factors derived by Liddle from the results of the AASHO Road Test.

Table B4.2 – Equivalence Factors (Damaging Power) of Different Tandem Axle Loads

Axle Load		Equivalence Factor
kg	lbs	
4540	10000	0.01
5440	12000	0.02
6350	14000	0.04
7260	16000	0.07
8160	18000	0.11
9070	20000	0.16
9980	22000	0.23
10890	24000	0.31
11790	26000	0.42
12700	28000	0.55
13610	30000	0.70
14520	32000	0.89
15420	34000	1.11
16320	36000	1.38
17230	38000	1.69
18140	40000	2.06
19050	42000	2.49
19960	44000	2.99
20870	46000	3.58
21770	48000	4.25

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Figures B4.3 and B4.4 show relationship between design loads and other axles applications at terminal PSI (Present Serviceability Index) values of 2.5 and 1.5 respectively. Equivalence factors can be obtained by relating the damage caused by applications of one axle load to the corresponding damage caused by applications of a “standard axle”. The standard axle load normally adopted was 18000 lbs (8160 kg) for single axles and 33500 lbs (15200 kg) for tandem axles. The generally accepted interpretation of results yield the well known fourth power relation between the magnitude of axle load and its structural damaging power. The equivalence factor of 0.0002 for the single axle load of 910 kg (2000 lbs) means that 5000 passes of such an axle do the same amount of damage as one pass of the standard axle. Equivalence factors were found not to vary much either with the thickness or type of the pavement (flexible bituminous or rigid concrete). The damaging effect of axles thus increases steeply with the axle load, the relationship being represented by approximately a fourth power law.

The equivalence factors can be used to classify vehicles of known axle loading in terms of an equivalent number of standard axles to which they will subject, the road during one passage. Figure B4.5 refers to four types of vehicles loaded in various ways to give the maximum overall weights permitted by construction and use regulations and without exceeding the 10 ton axle load. Vehicle 2 of 24 tons gross weight do substantially more damage than vehicles 6, 7 and 8 of 32 tons and marginally more damage than vehicles 3 and 4 of 30 tons. However, by rearranging the loading of 4-axle vehicle as in case of 5, the position reversed. It follows that the size and overall weight of a vehicle is not necessarily a guide to the damage which it may do but it is the axle load which is important in assessing damage of the pavement.

B4.3 Vehicle Damage Factor (VDF)

The traffic volume, primarily commercial vehicles and their loading are the most important factors affecting the deterioration of a pavement structure along with the environmental conditions in which road is located. The effect of traffic has been expressed in terms of total number of Equivalent Standard Axles (ESA) of 8.16 tonnes for single axles and 15.2 tonnes for tandem axles.

In order to convert axle load data into number of equivalent standard axles, the axle loads were grouped in the intervals of (i) 1000 kg from 1000 kg to 20,000 kg for single axles and (ii) 2000 kg from 6000 kg to 24,000 kg for tandem axles and the frequency of each group was found out. Equivalence factors were derived for each group from the ‘Fourth Power Rule’. The product of frequency of axles for each group and corresponding equivalency factor gave the total number of Equivalent Standard Axle Loading (ESAL) for vehicles. In other words,

Total ESAL = Number of axles in each weight class x Load equivalency factor of the weight class

Equivalence value and Vehicle Damage Factor (VDF) of the road are obtained from total ESAL. Equivalence value of a road is a multiplying factor and is the equivalent number of standard axle load repetitions. In other words, VDF is the number of standard axles for each commercial vehicle plying on a road. These are given below,

Equivalence value = Total number of ESAL / Total number of ‘**axles**’ weighed.....B4.1

VDF = Total number of ESAL / Total number of ‘**vehicles**’ weighedB4.2

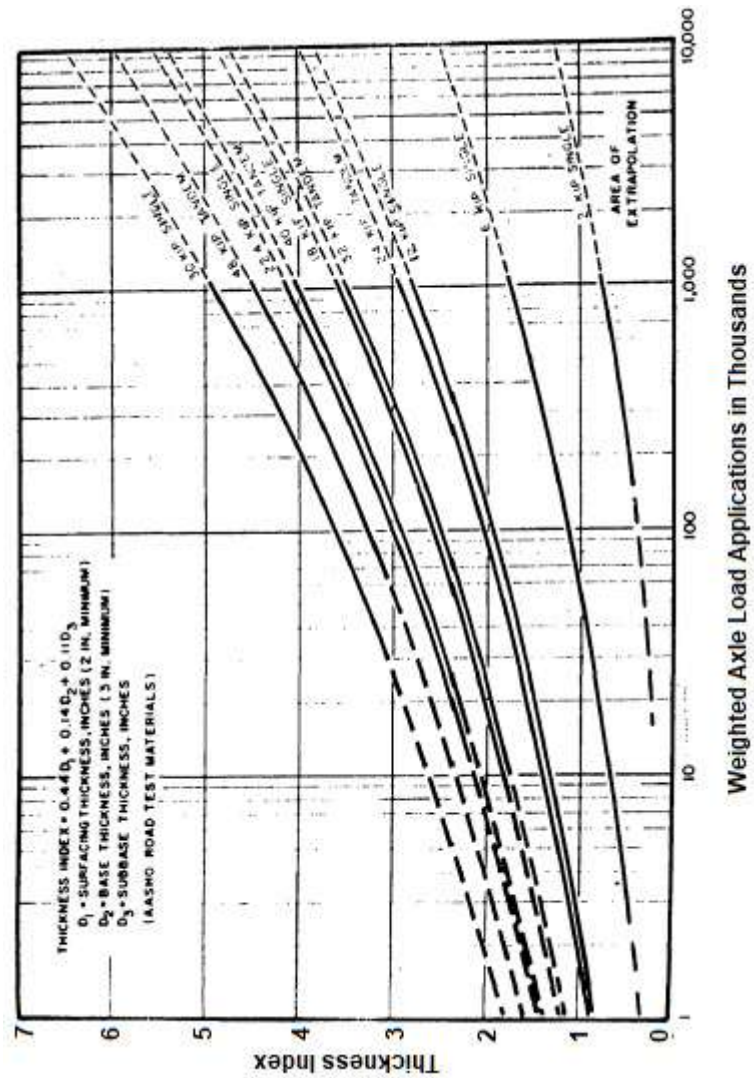


Figure B4.3 - AASHO Road Test Relationship Between Design and Axle Load Application at terminal PSI of 2.5

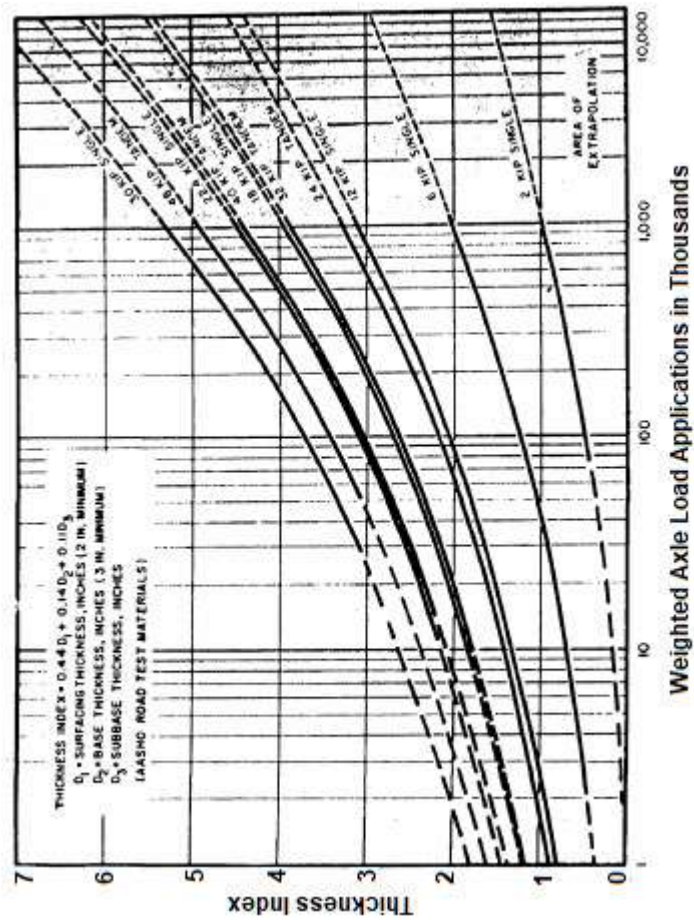


Figure B4.4 - AASHO Road Test Relationship Between Design and Axle Load Application at terminal PSI of 1.5

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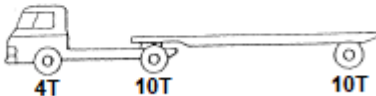


					Gross Vehicle Weight	Number of Standard Axles for Vehicle Axle Load Distribution
Vehicle 1		6T	10T		16T	$0.3 + 2.4 = 2.7$
Vehicle 2		4T	10T	10T	24T	$0.056 + 2.4 + 2.4 = 4.9$
Vehicles		4T	8T	8T	10T	$0.056 + 1.0 + 1.0 + 2.4 = 4.5$
1		4T	8T	8T	10T	30T
2		4T	8T	9T	9T	30T
3		4T	6T	10T	10T	30T
						$0.056 + 0.3 + 2.4 + 2.4 = 5.2$
Vehicles		4T	1T	2T	3T	2T
6		4T	5T	6T	9T	8T
7		4T	4T	6T	10T	8T
8		4T	4T	6T	10T	8T
						$0.056 + 0.6 + 0.6 + 0.6 + 0.6 = 2.5$
						$0.056 + 0.14 + 0.3 + 1.6 + 1.0 = 3.1$
						$0.056 + 0.056 + 0.3 + 2.4 + 1.0 = 3.8$

Figure B4.5 – Damaging Effect of Loaded Vehicles

B4.4 Axle Load Survey Procedure

The axle load survey is conducted at various strategic locations in the project highway for continuous 48 to 72 hours depending on traffic variation during a day. In case of weather problem or any other reasons for traffic disruptions the duration may be made for 7 days. All commercial vehicles of weight 3 tonnes and above were weighed during the survey.

The weighing units used in the surveys are portable static electronic weighing pads. The weighing pads consists of an aluminium alloy platform of size 50cm x 70cm x 9cm. The platform is connected to an electronic indicator unit operating at 12 volts, using a set of DC batteries. If the platform has built-in indicator then no calibration is required. Otherwise the weighing pads need to be calibrated by loading the platform in steps of 1000 kg. A graph for observed and known (obtained from proving ring) values are plotted. Any observed discrepancy may be corrected by tightening the screws securing lower and upper plates/or by adjusting the indicator calibration value.

B4.5 Observations

The classification of commercial vehicles adopted during the survey are shown in Figure B4.6

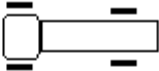
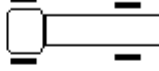
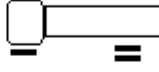
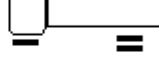
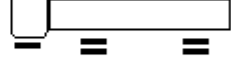
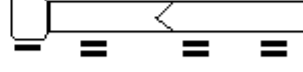
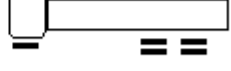
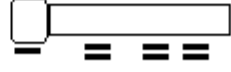

Vehicle Code	Vehicle Description	Vehicle Configuration
1	LCV GOODS	
2	LCV PASSENGER	
3	2-AXLE TRUCK	
4	BUS	
5	SEMI-TRUCK-TRAILER (Single Rear Axle)	
6	TRUCK-TRAILER (Single Rear Axle)	
7	3-AXLE-TRUCK	
8	SEMI-TRUCK-TRAILER (Tandem Rear Axle)	
9	TRUCK-TRAILER (Tandem Rear Axle)	

Figure B4.6 – Axle Configuration of Commercial Vehicles

Table B4.3 – Summary of Axle Load Spectrum

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Vehicle Code	Vehicle Type	Nos. of Vehicles Weighed	Percentage of Each Category of Vehicle
1	LCV (Goods)	842	9.58%
2	LCV (Passenger)	13	0.15%
3	Rigid Chassis Truck	7242	82.44%
4	Bus	597	6.80%
5	Semi Trailer	33	0.38%
6	Truck Trailer	0	0.00%
7	Rigid Chassis Rear Tandem	18	0.20%
8	Semi Trailer Rear Tandem	39	0.44%
9	Truck Trailer Rear Tandem	1	0.01%
10	Truck Trailer	0	0.00%
Total =		8785	100
(LCV = Light Commercial Vehicle)			

Table B4.4 - Summary of Axle Load Spectrum

S. No.	Location	Date	Numbers of Vehicles Weighed				VDF	ESAL
			Busses	Trucks	Multi Axle Trucks	Total		
1.	SH-90, Chainage km 15	04 Sep 14	65	854	16	935	7.83	3.90
2.	SH-90, Chainage km 22	08 Sep 14	37	917	14	968	8.59	4.28
3.	SH-90, Chainage km 31	12 Sep 14	55	825	15	895	5.52	2.75
4.	SH-90, Chainage km 44	17 Sep 14	82	896	16	994	6.67	3.32
5.	SH-90, Chainage km 56	21 Sep 14	66	825	9	900	5.74	2.86
6.	SH-90, Chainage km 68	24 Sep 14	88	829	11	928	7.13	3.55
7.	SH-90, Chainage km 78	28 Sep 14	48	810	9	867	7.31	3.64

B5 Plate Load Test

<https://www.civilknowledges.com/plate-load-test/>
<https://civiltoday.com/geotechnical-engineering/site-investigation/192-plate-load-test>
<https://theconstructor.org/practical-guide/determine-modulus-subgrade-reaction/139447/>
<https://www.fhwa.dot.gov/pavement/concrete/pubs/hif16005.pdf>

The modulus of subgrade reaction is determined by the field plate load test. In this test, compressive stress is applied to the soil layer through rigid plates, and the deflections are measured for different values of stress.

The k-value is determined by means of a plate load test in accordance with AASHTO T 122 and ASTM D 1996

B5.1 General

A Plate Load Test is a field test to determine the ultimate bearing capacity of the soil and the probable settlement under a given loading.

The test essentially consists of loading a rigid plate at the foundation level and determining the settlement corresponding to each load increment.

The ultimate bearing capacity then takes as the load at which the plate starts sinking at a rapid rate.

The bearing plate is squared, of minimum size, 30 cm² and the maximum size is 75 cm². The thickness of the steel plate should not be less than 25 mm.

B5.2 Method of Plate Load Test

There are two methods

- Reaction truss method
- Gravity loading platform method

B5.3 Reaction Truss Method

Here the reaction of the jack is borne by a reaction truss. The truss is held to the ground through ground anchors. These anchors are firmly driven in the soil with the help of hammers.

The reaction truss is usually made of mild steel sections. Guy ropes are used for the lateral stability of the truss.

The use of the reaction truss is more popular nowadays since this simple, quick and less clumsy.

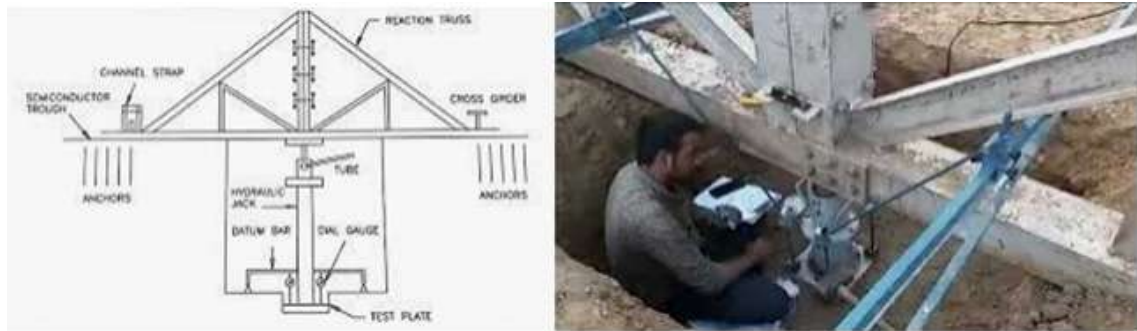


Figure B5.1 - Test Setup for Reaction Truss Method

B5.4 Gravity Loading Platform Method

In this case, a platform is constructed over a vertical column resting on the test plate, and the loading is done with the help of sandbags, stones, or concrete blocks.

When loading is applied to the test plate, it sinks or settles. The settlement of the plate is measured with the help of sensitive dial gauges. For the square plate, two dial gauges are used.

The dial gauges are mounted on an independently supported datum bar. As the plate settles, the ram of the dial gauges moves down and settlement is recommended.

The load is indicated on the load gauges of the hydraulic jack.

B5.5 Plate Load Test Apparatus/Equipment

The following plate load test apparatus is necessary for performing the test.

1. Test plate
2. Hydraulic jack & pump
3. Reaction beam or reaction truss
4. Dial gauges
5. Pressure gauge
6. Loading columns
7. Necessary equipment for the loading platform.
8. Tripod, Plumb bob, spirit level, etc.

B5.6 Plate Load Test Procedure

The necessary steps to perform a plate load test is written below:

1. Excavate test pit up to the desired depth. The pit size should be at least 5 times the size of the test plate (B_p).
2. At the center of the pit, a small hole or depression is created. The size of the hole is the same as the size of the steel plate. The bottom level of the hole should correspond to the level of the actual foundation. The depth of the hole is created such that the ratio of the depth to width of the hole is equal to the ratio of the actual depth to the actual width of the foundation.
3. A mild steel plate is used as a load-bearing plate whose thickness should be at least 25 mm thickness and size may vary from 300 mm to 750 mm. The plate can be square or circular. Generally, a square plate is used for square footing and a circular plate is used for circular footing.
4. A column is placed at the center of the plate. The load is transferred to the plate through the centrally placed column.
5. The load can be transferred to the column either by gravity loading method or by truss method.

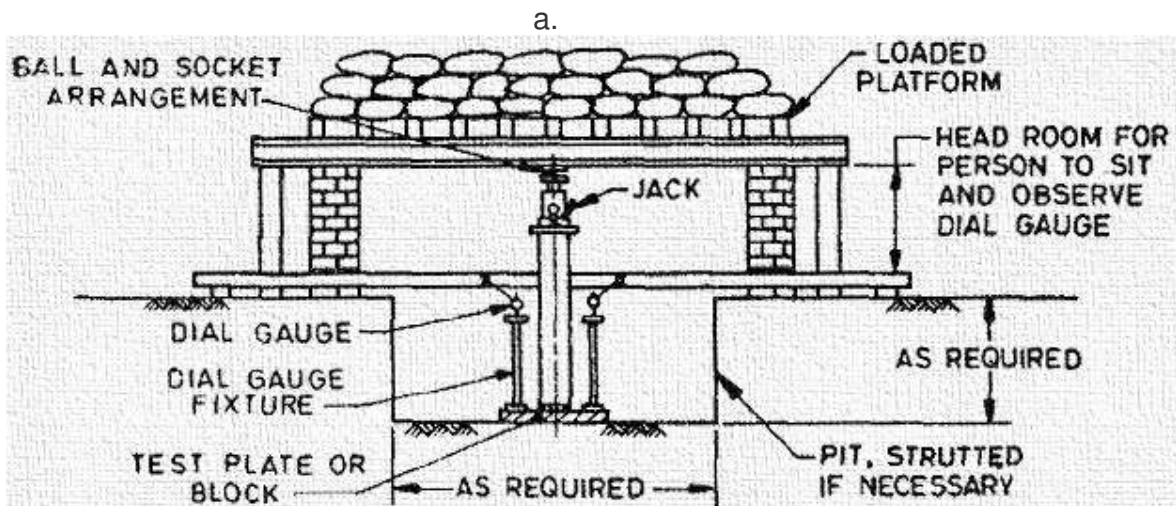


Figure B5.2 - Test Setup for Gravity Loading Platform Method

6. For gravity loading method a platform is constructed over the column and load is applied to the platform by means of sandbags or any other dead loads. The hydraulic jack is placed in between column and loading platform for the application of gradual loading. This type of loading is called reaction loading.
7. At least two dial gauges should be placed at diagonal corners of the plate to record the settlement. The gauges are placed on a platform so that it does not settle with the plate.
8. Apply seating load of $.7 \text{ T/m}^2$ and release before the actual loading starts.
9. The initial readings are noted.

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10. The load is then applied through the hydraulic jack and increased gradually. The increment is generally one-fifth of the expected safe bearing capacity or one-tenth of the ultimate bearing capacity or any other smaller value. The applied load is noted from the pressure gauge.

11. The settlement is observed for each increment and from dial gauge. After increasing the load-settlement should be observed after 1, 4, 10, 20, 40, and 60 minutes and then at hourly intervals until the rate of settlement is less than .02 mm per hour. The readings are noted in tabular form.

12. After completing the collection of data for a particular loading, the next load increment is applied and readings are noted under new load. This increment and data collection is repeated until the maximum load is applied. The maximum load is generally 1.5 times the expected ultimate load or 3 times of the expected allowable bearing pressure.

B5.7 Calculation of Bearing Capacity from Plate Load Test

After the collection of field data, the load-settlement curve is drawn. It is a logarithmic graph where the load applied is plotted on X-axis and settlement on Y-axis. From the graph, the ultimate load for the plate is obtained which is the corresponding load for settlement of one-fifth of the plate width.

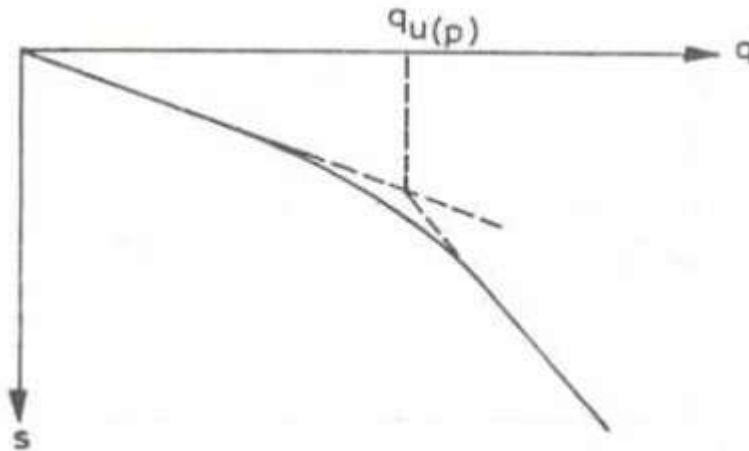


Figure B5.3 - Load - Settlement Graph

When the points are plotted on the graph, the curve is broken at one point. The corresponding load to that breakpoint is considered to be the ultimate load on the plate. The ultimate bearing capacity can be calculated from the ultimate load from the plate. The ultimate bearing capacity is then divided by a suitable factor of safety to determine the safe bearing capacity of soil from the foundation.

B5.8 General Equations for Calculation of Bearing Capacity of Soil

Soil Bearing Capacity Calculation for Clayey Soil

Following is the equation to determine soil bearing capacity for clay from the plate load test.

$$\textbf{\textit{Ultimate Bearing Capacity = Ultimate Load for the Plate.}}$$

Soil Bearing Capacity Calculation for Sandy Soil

The followings are the equation to determine soil bearing capacity for sand from the plate load test.

$$\textbf{\textit{Ultimate bearing capacity = Ultimate load for plate} \times \textit{Width of Pit} / \textit{Size of Plate}}$$

$$\textbf{\textit{Safe bearing capacity = Ultimate Bearing Capacity} / \textit{Factor of Safety}}$$

Typically, the range for the factor of safety varies from 2 to 3.

B5.9 Equations for Foundation Settlement Calculation from Plate Load Test

The following equations can be used for foundation settlement calculation.

Foundation Settlement Calculation for Clayey Soil

Following is the equation to determine foundation settlement for clay from the plate load test.

$$\textbf{\textit{Settlement of the foundation = Settlement of plate} \times \textit{Width of Pit} / \textit{Size of Plate}}$$

Foundation Settlement Calculation for Sandy Soil

Following is the equation to determine foundation settlement for sand from the plate load test.

$$\textbf{\textit{Settlement of the foundation = Settlement of plate} \times (\textit{Width of Pit} \times (\textit{Size of Plate} + 0.3) / \textit{Size of Plate} \times (\textit{Width of Pit} + 0.3))^2}$$

B5.10 Modulus of Subgrade Reaction

Modulus of Subgrade reaction or coefficient of Subgrade reaction is the reaction pressure sustained by the soil sample under a rigid plate of standard diameter per unit settlement measured at a specified pressure or settlement.

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It is a stiffness parameter that is commonly used to define the support conditions of footings and mat foundations. This parameter explains a linear elastic response. Hence during the design, the pressure generated by the modulus of Subgrade reaction is limited to the allowable bearing pressure of the soil.

The coefficient of sub-grade reaction (k) is the primary parameter used in the design of pavements, foundation, and soil-structure interaction studies. Modulus of Subgrade reaction is defined as the pressure per unit deformation of the Subgrade at specific pressure or deformation. It is expressed as:

$$k = p/s$$

Here, ' k ' is the modulus or coefficient of Subgrade reaction, ' p ' is the pressure, and ' s ' is the deformation of soil settlement. The coefficient of Subgrade reaction is measured and expressed as load intensity per unit of displacement. In the SI unit system, it is expressed as $\text{kN/m}^2/\text{m}$.

The test setup consists of a loading frame that has a hydraulic jack, a reaction beam, and a proving ring. The settlement of the plate is measured using a dial gauge. The dial gauges are attached to a separate datum frame as shown in Figure-1. The standard size of the plate is 750 mm in diameter.

The plate is properly placed on the test site. The test site must be levelled and prepared before placing the plate. As shown in the figure, the stiffening plates are placed on the bottom plate in the order of decreasing diameter. The test procedure is commenced by applying a seating load of 7 kN/m^2 . This load is released after a few seconds. The load is then applied to cause a settlement of $s = 0.25 \text{ mm}$.

When the settlement does not increase, or it is less than 0.025 mm per minute, then the dial gauge reading is taken, and the average settlement is determined. This procedure is repeated till the settlement goes to 0.175 cm . As shown in Figure FB5.4, a graph is plotted between the bearing pressure (p) and the corresponding settlement caused (s) or Δ .

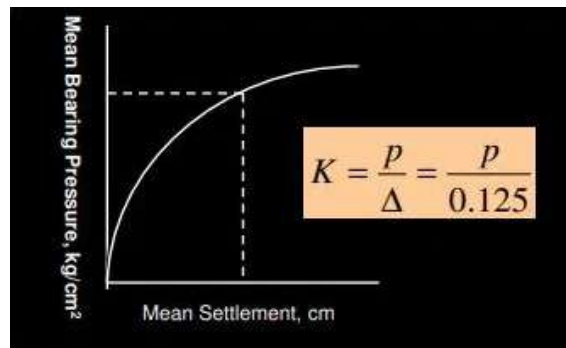


Figure B5.4 - Mean Bearing Pressure versus Mean Settlement Curve

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The pressure corresponding to the settlement 0.125 cm is read from the plot, and the value of 'k' is determined as:

$$k = p / 0.125$$

The soil, after construction, is likely to soften due to external vibrations, water penetration, and other imposed loads. This effect is considered by providing certain corrections to the value of 'k'. Here, 'k' is corrected for full saturation. For this, two specimens from the site are subjected to consolidation test in soaked and 'Un-soaked' condition. The consolidation test is conducted, and the pressure corresponding to 0.125 cm is determined for both the specimens. Then the corrected modulus of rupture is given by:

$$k' = k \times [p/p_s]$$

Where p and p_s are the pressures for 'Un-soaked' and soaked specimen as shown in Figure-3.

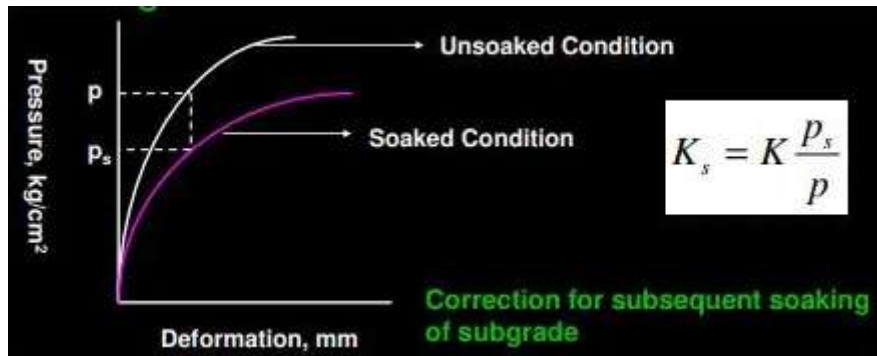


Figure B5.5 - Subsequent Corrections for Modulus of Subgrade Reaction under Soaking Condition

In practice, it is used by geotechnical and structural engineers for the analysis of structure. The analysis provides insights into the settlement of the structure and settlement-induced stresses within the structure.

Modulus of Subgrade reaction is defined as the pressure per unit deformation of the Subgrade at specific pressure or deformation. It is expressed as $k=p/s$.

Here, k = modulus or coefficient of Subgrade reaction; p = applied pressure and s = deformation of soil settlement. The coefficient of Subgrade reaction is expressed in load intensity per unit of displacement.

B6 Dynamic Cone Penetration Test

B6.1 General

The Dynamic Cone Penetrometer (DCP) is used to measure the rate of penetration through the various layers of a pavement. In South Africa, the measurement of the in-situ shear strength of the pavement layers using a DCP, has also led to the development of pavement evaluation and rehabilitation design methods. The DCP component (layer by layer) analysis method is empirically derived based on material shear strength and can only be accurate if used for the evaluation and analysis of pavements similar to those from which they were derived.

It follows that the DCP test provides meaningful, but incomplete information about the expected behaviour of a pavement. Although in-situ material shear strength data are relatively complete with the DCP, if found applicable, it can give extremely good results and information about the future behaviour of a pavement. It follows that the key to the successful application of empirically derived methods based on CBR and/or DCP measurements lies in the applicability of the method to a specific pavement. It is, however, advised that the method be used in a multi-analysis approach together with other methods.

The figure below shows a schematic of the DCP.

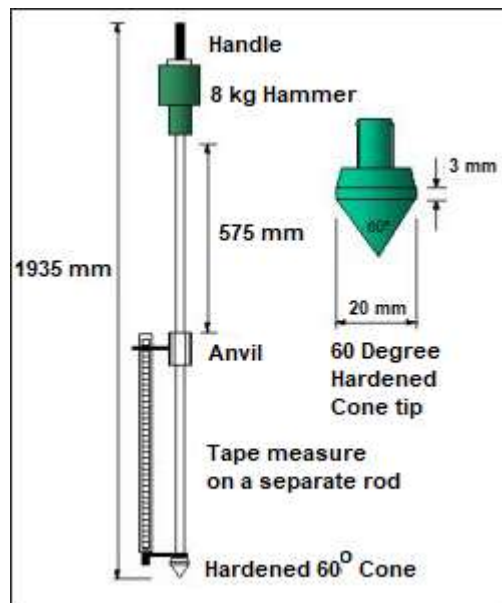


Figure B6.1 – Basic Schematic of Dynamic Cone Penetrometer (DCP)

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B6.2 Conducting the Test and Collecting Data

The Dynamic Cone Penetration Test provides a measure of a material's in-situ resistance to penetration. The test is performed by driving a metal cone into the ground by repeated striking it with a 17.6 lb (8 Kg) weight dropped from a distance of 2.26 feet (575 mm). The penetration of the cone is measured after each blow and is recorded to provide a continuous measure of shearing resistance up to 5 feet below the ground surface. Test results can be correlated to California Bearing Ratios, in-situ density, resilient modulus, and bearing capacity.

The cost of purchasing or manufacturing a DCP test apparatus is relatively low. Currently, prices vary from \$1,600 to \$2,200 USD for portable hand-operated equipment. More expensive, automated DCP rigs can be purchased that are towable. Depending on the DCP apparatus the test can be conducted efficiently with as little as 1 to 2 people. Again, dependent upon the test apparatus, depth of testing, and the condition of the subgrade, the test requires 5 to 10 minutes to complete.

B6.3 Procedure and Test Apparatus

The DCP apparatus consists of a 5/8"-inch diameter steel rod with a 60 degree conical tip. The rod is topped with an anvil that is connected to a second steel rod. This rod is used as a guide to allow an 8kg hammer to be repeatedly raised and dropped from a height of 575mm. The connection between the two rods consists of anvil to allow for quick connections between the rods and for efficient energy transfer from the falling weight to the penetrating rod.

Once the test apparatus is assembled the DCP is placed at the test location and the initial penetration of the rod is recorded to provide a zeroing scale. While holding the rod vertically, the weight is raised to the top of the rod 575 mm above the anvil and dropped. The penetration of the rod is measured after each drop. In cohesive soils the penetration rod may be rotated slightly to prevent binding of the soil and the rod influencing test results. The test shall be terminated if the desired depth is reached or if the rod penetrates less than 1/8-inch in 10 drops.

B6.4 Correlations to Engineering Parameters

Several correlations exist between common engineering parameters and the dynamic penetration index.

$$\text{CBR} = 292 / (\text{DPI}^{1.12})$$

$$\log(\text{DPI}) = -1.05 + 2.03 \times \log(\text{SPT})$$

Where:

CBR = California Bearing Ratio

DPI = DCP Penetration Index

SPT = Standard Penetration Test (ASTM D 1586-64)

APPENDIX - C

Stabilization of Granular Soil, Reclaimed Soft Soil and Slope Stability Analysis

C1 Stabilization of Granular Soil for Road Building

C1.1 General

Stabilization has been in use for many years and has made vast progress in improving the quality of pavements, also there is rapid progress in stabilization. The age long technique is no longer limited to subgrade or embankment but has entered to the pavement layers like sub-base and base and in some special cases even in wearing course.

This section deals with stabilization of soil and granular material using cement, lime & fly ash, which are traditionally being used as stabilizers to improve the strength and durability characteristics of various types of soils and granular materials in pavement structure and are termed as conventional stabilizers (CS) in this section.

A number of companies have been promoting different types of Commercial Chemical Stabilizers (CCS) in the market. The companies indicate that such stabilizers are special chemical compounds, which have been evolved after a long research and should be mixed with cement to enhance the strength and durability characteristics of soil cement mix. The dosage of such CCS to be mixed varies from 0.5 per cent to 5.0 per cent of cement content. These chemical stabilizers are available either in powder form or in liquid form. The categories of different CCS available in the country are as follows:

- Natural Inorganic Powder Binders
- Water Repelling Nano Chemicals
- Waste Oil
- Petroleum Based Products
- Liquid Stabilized Products
- Synthetic Polymers
- Sulphonate Lignin etc.

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Some companies mix these chemical compounds in cement itself at the manufacturing plant and sell such products (cement mixed with admixtures), with a commercial name. Such products are ready to use and therefore can be directly mixed with soils or granular materials for site specific requirements in the desired quantity as determined by detailed laboratory/field tests. However, some companies provide the CCS separately, which is required to be mixed at site with cement in a manner as suggested by the company before being used with soil or granular materials. It is claimed that the materials stabilized with CCS not only yield better strength but result in improved elastic and thermal properties of the mix and therefore less prone to cracking and shrinkage cracks. Since long term performance of roads constructed with such special products is not available, it becomes difficult to accept such products for large scale application.

C1.2 Applications in Pavement Constructions

Requirement of improvement for Soil Modification/Subgrade CCS/CS can be used for modification or improvement of subgrade soil or for construction of cementitious base/sub-base layers meeting the requirements of the specifications. It is recommended from economic consideration that mix-in-place methods of construction be used for subgrade improvement. The main requirement for CCS/CS modification or stabilization of subgrade soils shall be as described below.

The pavement performance of a stabilized road will be largely governed by the gradation and the type of soil/granular material used for the purpose of stabilization. The quality of material to be stabilized should meet the minimum standard set out in specifications. Stabilized layers constructed from such material is likely to perform satisfactorily even if, it is affected by carbonation during its life time. Materials which do not comply with the requirements given in the specifications can be stabilized but more additive will be required and the risk from cracking and carbonation will increase. The strength of stabilized materials can be evaluated in many ways, of which the most popular are the Unconfined Compressive Strength (UCS) test and the California Bearing Ratio (CBR) test.

C1.3 Stabilization with Cement

Requirement for Soil Modification/Subgrade Improvement

Cement stabilized materials can be used for soil modification or improvement of subgrade soil. It is recommended from economic consideration that mix in-place methods of construction be used for subgrade improvement and only granular materials and silty cohesive materials be used. (The assumption being that more clayey materials would be more effectively stabilized with lime). The main requirements for cement modification or stabilization of subgrade soil are summarised in the table C1.1.



Figure C1.1 – Soil Stabilization with Cement

Table C1.1 – Soil Characteristics for Cement Modified Soil / Improved Subgrade

S. No.	Properties	Specific Values
1.	Liquid Limit (%)	< 45
2.	Plasticity Index	< 20
3.	Organic Content (%)	< 2
4.	Total SO ₄ Content	0.2% Max
5.	Minimum Laboratory CBR at Specified Density (%)	15
6.	Minimum Cement Content (%)	2
7.	Degree of Pulverization (%)	➤ 60
8.	Temperature for Mixing	More than 10 °C
9.	Time for Completing Compaction	2 Hours Max.

In case of deployment of highly effective mechanical equipment for spreading of cement, breaking clods and blending are used, the minimum percentage of cement for stabilization could be reduced to 0.5%. Extensive laboratory testing of soil samples mixed at site may indicate the CBR achieved with the cement content in the mix.

C1.4 Stabilization with Lime

Requirement for Soil Modification/Subgrade Improvement

The properties of soil-lime mixes is usually assessed on the basis of strength tests made on the materials after the stabilizer has been allowed sufficient time to harden. The strength of stabilized soils can be evaluated in many ways, of which the most popular is California Bearing Ratio (CBR) test for lime stabilized soils. Lime stabilization is generally recommended to improve the subgrade soils which are cohesive in nature. Lime is recommended for such soils because of its beneficial effects on plasticity, workability and strength gain. The main requirements for lime stabilized improved subgrade are summarized in the Table C1.2.

Table C1.2 – Requirements for Improvements in Lime Stabilized Subgrade Soils

S. No.	Properties	Specific Values
1.	Passing 75mm Sieve	100 %
2.	Passing 26.5mm Sieve	95-100 %
3.	Passing 75 micron Sieve	15-100 %
4.	Plasticity Index	➤ 10
5.	Organic Content (%)	< 2 %
6.	Total SO ₄ Content	< 0.2 %
7.	Minimum Lime Content (%)	2.5 %
8.	Degree of Pulverization (%)	>60 %
9.	Minimum Temperature for Mixing at Field	10 °C
10.	Unconfined Compressive Strength (UCS) MPa	As per contract specifications.

In case of deployment of highly effective mechanical equipment for spreading of lime, breaking clods and blending are used, the minimum percentage of lime for stabilization could be reduced to 0.5%. Extensive laboratory testing of soil samples mixed at site may indicate the CBR achieved with the cement content in the mix.

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In case the subgrade soil is highly plastic, it can be modified with lime and/ or fly ash before being mixed with CCS/CS. The technical requirements for lime and fly ash modification for subgrade improvement shall be as discussed below.

The properties of soil-lime mixes should be based on strength tests made on the materials after allowing the required time to harden the mix. The strength of stabilized soil may be determined by CBR test of lime-stabilized soils. Lime stabilization is generally recommended for cohesive soils that are intended to be used in the subgrade. Lime has beneficial effects on plasticity, workability and gaining strength.

C1.5 Stabilization by Using Fly Ash

The laying and compacting of cement treated fly ash to construct the sub-base layer on prepared subgrade or base course layer on prepared sub-base in accordance with the specifications. This technique is also applicable for strengthening a weak subgrade layer. However, the proportions of fly ash and cement in the mix should fulfil the following requirements:

- Provide adequate strength and durability
- Be easily placed and compacted
- Be economical

Cement less than 2 percent is not generally suitable to proper mixing and hence not recommended. Once the proportions of cement and fly ash are decided for the trial mix the proctor density for dry density and moisture content is to be determined.

The test for 'Unconfined Compressive Strength' is to be done on samples compacted at maximum dry density and optimum moisture content. The mix proportion should be designed to obtain the minimum unconfined compressive strength value 17.5 kg/cm^2 after 7 days moisture curing in a humidity chamber of samples with a length to diameter ratio 2:1. For curing the recommended temperature range is 30°C to 38°C . The design mix should indicate proportions of cement and fly ash, the quantity of water to be mixed and a specified compacted density that is required to achieve the specified strength.

Cement conforming to specification should be used. When fly ash is used as ingredients, Portland Pozzolona Cement should not be used for stabilization

Water should be clean and free from injurious amounts of oil, salt and acid etc. It should meet the specification. Potable water is generally considered as acceptable for stabilization work. The permissible limits for solids in water are given in the table C1.3:

Table C1.3 – Permissible limits for solids in water for soil stabilization

Solids	Permissible Limit (Maximum)
Organic	200 mg/litre
Inorganic	3000 mg/litre
Sulphates (SO ₄)	400 mg/litre
Chlorides (CL)	2000 mg/litre
Suspended Matters	2000 mg/litre

C1.6 Test Requirements

C1.6.1 Unconfined compressive strength test

This test is carried out on cylindrical or cubical specimens prepared by mixing the soil at a pre-determined moisture content and stabilizer content and compacting the mixed material into a mould at either a pre-determined density or at a given compactive effort. The choice of specimen size and shape depends on the grading of the soil; it is clearly desirable to keep as small as possible the ratio of the maximum particles size to the smallest dimension of the mould. The following sizes of specimen for different group of material are recommended (Table C1.4).

Table C1.4 - Suggested Size of Moulds for Casting Materials Samples

Fine grained material	Cylindrical specimens 100 mm high and 50 mm diameter, or 150 mm cubic specimens
Medium grained material	Cylindrical specimens 100 mm high and 50 mm diameter, or 150 mm cubic specimens
Coarse-grained material	150 mm cubic specimens

Compressive strength results on identical materials from strength tests on cubical specimens would be higher than those obtained from cylindrical specimens; and cylindrical specimens with a height/diameter ratio of 2:1; have lower strength than cylindrical specimens with a height/diameter ratio of 1:1. Allowance therefore has to be made for this when comparing results obtained with specimens of different shapes. For the relatively low strengths encountered in cement-stabilized soils the results on different sized test specimens may be multiplied by the correction factors given in **Table C1.5** to calculate the approximate equivalent strength of a 150 mm cube. However, there is no unique relation between the strengths of specimens of the two shapes as the ratio depends primarily on the level of strength of the material.

Table C1.5 - Correction Factors for Various Sizes and Shapes of Test Specimens

S. No.	Specimen Size	Correction Factor
1.	150 mm cube	1.00
2.	100 mm cube	0.96
3.	200 mm x 100 mm diameter cylinder	1.25
4.	115.5 mm x 105 mm diameter cylinder	1.04
5.	127 mm x 152 mm diameter cylinder	0.96

C1.6.2 Durability of Stabilized Materials

In order to check the durability of the stabilized mix for sub-base/base, the following two methods are recommended. Method 1 is recommended for moderate temperature and climate conditions, whereas method 2 is recommended for those regions where there is large variation in temperature and climatic conditions. The decision regarding the adoption of a particular method should be as directed by the Engineer-in-Charge.

Method 1: Prepare two identical sets (containing 3 specimens each) of UCS specimen which are cured in a normal manner at constant moisture content for 7 days. At the end of 7 days period one set is immersed in the water while the other set is continued to cure at constant moisture content. When both sets are 14 days old they are tested for UCS. The strength of the set immersed in water as a percentage of the strength of set cured at constant moisture content is calculated. This index is a measure of the resistance to the effect of water on strength. If the value is lower than 80 percent it is considered that the stabilizer content is low and its value should be increased.

Method 2: This test is done as per ASTM standard No. ASTM D 559. It is generally known as Wetting and Drying test for determining durability of stabilized soil mixes, which determines the weight losses, moisture changes and volume (swell and shrinkage) produced by repeated wetting and drying of hardened stabilized soil specimens. The other is a freezing and thawing test which follows a similar procedure except that wetting and drying is replaced by cycles of freezing and thawing.

In the wetting and drying test, the test specimens are subjected to 12 cycles of wetting and drying, consisting of immersion in water for 5 hours followed by drying at 71°C for 42 hours. After each cycle the specimens are brushed in a standardized manner with a wire scratch brush (18-20 strokes on the sides and 4 strokes at each end). The loss in weight of the brushed specimens, after each cycle are determined. In a parallel test the volume and moisture changes of the specimens are recorded after each cycle.

The freezing and thawing test is similar to the wetting and drying test but the test cycles consist of subjecting the specimens to freezing conditions at -23°C for 24 hours followed by thawing at 21°C for 23/24 hours. Similar to the wetting and drying test, after each thawing cycle the specimens are brushed. For climatic conditions prevailing in tropical countries, the durability under wetting and drying would have to be taken into consideration, where the durability under freeze/thaw condition does not generally apply.

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The principal criterion may be that, depending on soil type, the loss in weight of the specimens after 12 cycles of both freezing & thawing and wetting and drying should not exceed certain limits. Granular soils of low plasticity are permitted to lose up to 14 percent of their original mass and cohesive clay soils are permitted to lose only 7 percent of their original mass. The reason of the difference is that granular materials abrade more readily than cohesive soils and the wire brushing removes some material in addition to that loosened by the alternate cycles of freezing and thawing and wetting and drying. However, as per some other studies, the above requirements were found to be too stringent and following values have been recommended.

- Base: Less than 20 percent
- Sub-base: Less than 30 percent
- Shoulder: Less than 30 percent

C1.7 Procedure of Stabilization

The construction of stabilized road pavement layers follows the same basic procedures whether the stabilizing agent is cement, lime or other hydraulic binder. The procedure can be divided into two main groups:

- Mix-in-place stabilization
- Plant-mix stabilization

C1.7.1 Mix-in-place stabilization

The main advantage of the mix-in-place procedure is its relative simplicity and hence it is particularly suitable for work in remote areas where plant mixing could be logistically difficult. Its disadvantages are not obtaining efficient mixing i.e. good distribution of the stabilizer, constructing thicknesses of more than 200 mm and of poor levels.

In this process the material is stabilized in-situ which requires the stabilizing agent to be spread before or during the pulverization and mixing of the soil and stabilizer. This is generally carried out with a purpose made machine although for small scale work in remote areas agricultural machinery can be adapted for use. In-situ stabilization generally involves the following operations:

Initial Preparation

This involves excavating down to the in-situ material to be stabilized or placing imported material on the formation. The material to be stabilized then has to be graded to approximately the required levels. After which it is usually necessary to plough to loosen the material, one or two passes is normally sufficient.

Spreading the Stabilizer

Spreading the stabilizing agent at the required dosage rate can be carried out manually or by machine. When manual methods are used bags of stabilizer are spotted at a set spacing, they are then broken open and the stabilizer raked across the surface as uniformly as possible. Where quicklime is being used, necessary precaution need to be taken to protect the operators. This is especially true when the stabilizer is being spread by manual method.

Lime has a much lower bulk density than cement and it is possible, therefore, to achieve a more uniform distribution with lime when stabilizers are spread manually. The uniformity of the layer of

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stabilizer spread over the surface, before the mixing operation, determines the uniformity of the mixed material produced.

Mechanical spreaders automatically monitor the required amount of stabilizer to be spread on the surface of the soil. Their use results in a much more uniform spread of stabilizer over the surface than can be achieved by hand spreading. The equipment need to be calibrated before use to ensure that the correct rate of spread is achieved and subsequently checked at regular intervals to ensure that the rate of spread remains within specified tolerances.

Addition of Water

If it is necessary to add water to bring the moisture content to the required value this can either be done as part of the mixing operation or after the material has been prepared prior to the addition of the stabilizer. To ensure a through distribution of the added water, it is preferable to add water as part of the mixing operation. Water added during the mixing process should be through a spray system such that it is added in a uniform manner over the required area and mixed uniformly to the required depth. Where the mixing plant does not enable water to be added or where it is not possible to add enough water during mixing it should be added to the prepared material using a spray system that enables the amount to be controlled over the whole area. The material to be stabilized should then be mixed prior to the addition of the stabilizer to ensure the distribution of the water throughout the layer.

Mixing Soil, Water and Stabilizer

Robust mixing equipment of suitable power for the layer being processed is required to pulverize the soil and blend it with the stabilizer and water. The most efficient of the machines available carry out the operation in one pass, enabling the layer to be compacted quickly and minimizing the loss of density and strength caused by any delay in compaction. Multi pass machines are satisfactory, provided the length of pavement being processed is not excessive and each section of pavement can be processed within an acceptable time.

The plasticity of the material is overriding factor in the ability of mixing plant to mix the soil with stabilizer. A review of work showed that all plastic soil could be satisfactorily mixed with cement using the plant. For cohesive soils a factor of the plasticity index of the soil multiplied by the percentage of the fraction of the soil which was finer than 425 micron in particle diameter may be used to suggest the values for the different types of mixing plant available, which are given in Table C1.6.

Table C1.6 Soil Plasticity Limits for Stabilization Using Different Types of Plant

Type of Plant*	Plasticity Index x Percentage of fraction finer than 425 micron	Normal maximum depth (mm) capable of being processed in one layer
Agricultural Disc harrows, Disc ploughs, rotavators	Less than 100	120-150
Light duty rotavators (> 100 hp)	Less than 2000	150
Heavy duty rotavators (> 100 hp)	Less than 3500	200-300 (depending on soil type and horsepower of mixer)

* Selection of the appropriate plant should be left to the decision of the Engineer-in-Charge.

Graders have been used to mix stabilized material but they are inefficient for pulverizing cohesive soil and even with granular materials a large number of passes are needed before the quality of

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mixing is acceptable. For these reasons, the use of grader for mixing is not suggested, rather agriculture tractors fitted with long teeth scrappers at the back followed by mixing with rotavator is a better choice.

Compaction

Compaction is carried out in two stages:

- a) An initial rolling and trimming which may be carried out followed by a final mixing pass of the rotavator.
- b) In case of cement stabilized material, final compaction and leveling must be completed within two hours of mixing. Delay in the stabilization is less critical for soil modification. There may even be benefits in completing the final mixing, levelling and compaction between one and seven days after the initial mixing. This time gap allows for the reactions between the lime and clay to take place and thus provide a more workable soil. However, for lime stabilization as distinct from modification, the aim should be to complete compaction within three hours after mixing lime with soil. This is particularly true in hot climates where problems of evaporation and carbonation are more likely to occur. In case of cement stabilization, this time period should be reduced to two hours.

Curing

Proper curing is very important for three reasons:

- a) It ensures that sufficient water is retained in the layer so that the hydration reactions between the stabilizer, water and the soil can continue.
- b) It reduces shrinkage, and
- c) It reduces the risk of carbonation from the top layer.

In temperate climate curing presents few problems. It is usually carried out by sealing the compacted surface to prevent escape of water during the curing period (usually seven days) during which time all construction traffic must be kept off the stabilized material. Before spraying is started the surface should be swept free of loose material and any damp areas should be free of standing water. The following methods of curing are suggested:

- a) Covering with an impermeable sheeting with joints overlapping at least 300mm and set to prevent ingress of water.
- b) Final compaction and levelling in the case of cement stabilized material, must be completed within two hours of mixing. Delay in lime stabilization are less critical and for soil modification there may even be benefits in completing the final mixing, leveling and compaction between one and seven days after initial mixing. This time gap allows for the reactions between the lime and clay to take place and thus provide a more workable soil. However, for lime stabilization as distinct from modification, the aim should be complete compaction within three hours after mixing lime with soil. This is particularly true in hot climates where problems of evaporation and carbonation are more likely to occur. In case of cement stabilization, this time period should be reduced to two hours.
- c) Spraying with a resin based aluminous curing compound similar to those used for concrete. This has particular application where it is desirable to reduce the increase in temperature immediately under the surface which would result from the use of a black (bituminous) seal.

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In a hot dry climate, the need for good curing is most important but the prevention of moisture loss is very difficult. If the surface is constantly sprayed and kept damp day and night the moisture content in the main portion of the layer will remain stable but the operation is likely to leach stabilizer from the top portion of the layer. If the spraying operation is intermittent and the surface dries from time to time, which is common in this method, the curing will be completely ineffective.

Curing through spraying water can be much more efficient if a layer of sand 30 mm to 40 mm thick is first spread on top of the layer. In this case, the number of spraying cycles per day can be very much less and there is a considerable saving in the amount of water used.

When the stabilized layer is to be covered by other pavement layers the construction of the upper sections will provide a very good curing seal but care has to be taken to ensure that this work does not damage the top of the stabilized layer. During the period of time prior to the construction of the next layer some system of curing is required because, this is the most critical period in terms of shrinkage in the layer.

Primer can also serve as a curing membrane but, results have shown that a prime coat breaks down when it penetrates into the surface and completely loses any ability to seal it. A portion of any curing membrane must sit on the surface to achieve an effective seal if the top of the stabilized layer is sprayed lightly with water followed by an application of a viscous cutback bitumen, the loss of moisture is effectively reduced to zero. Similarly the top of the stabilized layer can be sprayed with an emulsion to achieve the same result. It is essential, however, that all traffic is kept off the curing membrane for several days at which time excess bitumen can be absorbed by the surface.

C1.7.2 Plant-Mix Stabilization

In this process, the materials are separately batched and mixed at a mixing plant. They are then transported to the site where they are laid by a bituminous paver and compacted. The advantages of the process are the good control on proportioning of the materials, multi-layer work can be executed and good compacted levels are readily achievable. The disadvantages are that output is lower than in the 'mix in place' process, cohesive materials cannot usually be mixed and in the case of cement stabilization, the mixing plant has to be relatively close to the site so that mixing, laying and compaction can all be completed within the stipulated two-hour time limit. The process is not, therefore, applicable to small-scale projects unless there is a mixing plant near at hand.

To ensure complete distribution of the relatively small quantities of stabilizer, mixing should be carried out in a forced action mixer and except for non-cohesive granular materials, free fall mixes of the type used for mixing concrete should not be used. If it is proposed to use a mixer other than one with a forced action preliminary trials should be made to ensure that satisfactory mixing is achieved.

Vehicles transporting the mixed material should be of sufficient number and capacity to meet both the output of the mixer and spreading and compaction operations. International standards and specifications, for plant mixed cement stabilized material require it to be spread by a bituminous paver and spreading by grader is not permitted. If graders are used for spreading, much of the advantage of plant-mix stabilization is lost as it is difficult to control levels and thicknesses of construction.

Compaction

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Whatever method is used for mixing the soil with water and stabilizer material, the methods used for compaction are the same. In the case of cement stabilized materials, once the cement has begun to harden, it is important that the matrix is not disturbed; hence the requirement that compaction must be completed within two hours of mixing. The compacted density of the stabilized layer is a measure of the effectiveness of compaction and hence of its strength. The degree of compaction to be achieved in the field can be specified in two ways. In an end product specification, the density of the layer in the field is determined and compared with a specified target density. Provided that the measured field density is greater than or equal to this limit the compaction in the field is deemed to be satisfactory. The main disadvantages of an end product specification are that a large amount of site testing is required and many of the methods in use are time consuming. This means that the result of the tests may not be available in time to remedy any deficiencies in compaction.

C2 Land Reclamation and Improvement of Soft Soil for Road Building



Figure C2.1 – Land Reclamation by Dredging

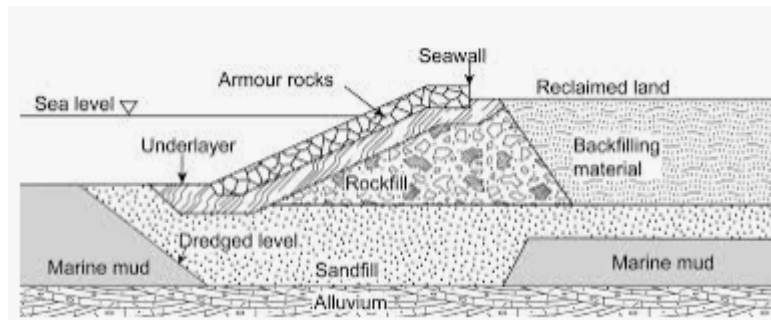


Figure C2.2 – Land Reclamation by Design

C2.1 General

Land reclamation, usually known as reclamation, and also known as land fill (not to be confused with a waste landfill), is the process of creating new land from oceans, seas, riverbeds or lake beds. The land reclaimed is known as reclamation ground or land fill.

<https://www.iadc-dredging.com/subject/concept-contract-comp-of-letion/borrow-area-selection/>

C2.2 Selection of Borrow Area

Borrow area selection affects the feasibility of a project as it defines the amount of available, suitable fill at a reasonable distance from the site.

Importance of the Borrow Area

In order to proceed with a dredging project, the presence of sufficient suitable fill is a necessity. To select a borrow area for a reclamation project or beach replenishment, the following should be considered:

- the haulage distance from the borrow area to the project location;
- the quantities of fill available; and
- the quality of the fill including fill mass behaviour, in-situ characteristics

Data Collection and Soil Investigations

Data collection and soil investigations are required to evaluate if the quality is suitable and the quantities are available. In some case, a borrow area for a reclamation project may be able to be retrieved from a nearby capital or maintenance dredging project where a harbour is being deepened.

Boundary Conditions

In addition to the quality and quantity of suitable fill, certain so-called boundary conditions must be considered. These include:

- The exposure of the borrow area to waves, swell and wind that could hamper dredging activities;
- If the distance between the borrow area and reclamation site is too long it could affect the economics of the works;
- Presence of large volumes of unsuitable overburden may require removal prior to dredging the suitable materials;
- Bathymetry at and near the borrow area may have an impact on the type of equipment being used;
- Geological balance and Environmental impact are to be considered;
- Government regulations need to be considered when selecting the borrow area;
- If royalty payments for the borrow area are high, an alternative might be needed;
- Existing infrastructure like pipelines, cables, or other obstacles may need to be avoided or removed.

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Special investigations and surveys will be needed to evaluate the above mention boundary conditions could have adverse effects on the feasibility of the borrow site.

C2.3 Planning

Planning is the visualization of the anticipated progress of the project in order to organize and connect activities in a logical way.

Reasons for Planning

The aim of planning varies from one situation and work phase to the other. The most important reasons for planning are:

- progress control in order to compare the actual progress of a project with the planned progress to make adjustments where necessary;
- marking milestones to show that (parts of) the project will finish on time;
- managing the use of resources;
- to find the optimum working method;
- establish communications especially with the client;
- as a contractual tool for identifying and managing changes;
- to assist in the control of project costs.

Planning Phases

The planning to realize a project falls into three phases:

- preparation;
- construction;
- maintenance.

Preparation

Preparation begins after the award of the contract, and good preparation is the key to a successful execution of the project. It will guide the way for all aspects of construction and maintenance.

Construction

To move into the construction phase, detailed specifications and contract document must be prepared and used to solicit tenders for the works.

This step may be supported by an expert consulting firm to assist in creating the tender documents and evaluating the submitted bids. The owner of the project must ensure that there is a structure for the administration of the contract and an inspection system is in place.

Maintenance

The maintenance phase occurs after the completion of construction. However, a major dredging project should also plan for maintenance dredging as based on experience and collected data predicting plausible siltation and other depreciation of a project is possible.

C2.4 Selection of Type of Dredger

Choosing the appropriate dredger is determined by soil conditions, transport options, dredging location, water depths and placement, and economics.

Soil Conditions

The type of soil being dredged is a determining factor for selecting a dredger. For instance, rock that has not been pre-treated generally limits the types of dredgers to mechanical or cutterhead dredgers designed specifically for rock dredging. Where the material is sand and can be dredged effectively by several dredger types, then a more detailed consideration of operating parameters is required, such as transport options.

Transport Options

Transport options affect the decision-making process and must consider the longer distances over water which might favour hoppers and barges, the longer distances over land favouring pipelines or pumping from a nearshore position, and shorter distances over water or over land.

Dredging Location, Water Depth, Placement

Trailing suction hopper dredgers are able to work efficiently in entrance channels where sea and traffic conditions make stationary plant less desirable and effective. The location of the dredged material placement areas and access to them may also play an important role in the decision on the most suitable and effective dredger type.

Economics

The final choice may need to be made on the basis of economics, ecological effects or both. All these factors require both technical and economic analysis in the decision process. For instance, the most effective dredger may not be available close to the work site and then mobilisation time and cost must be factored into the decision. To give an approximate sense of the employment of various types of dredgers, the worldwide distribution of dredgers by type are:

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- Trailing hopper dredgers,
- Backhoes/dipper dredgers,
- Cutter-head dredgers,
- Bucket dredgers,
- Plain suction dredgers,
- Grab/clamshell dredgers,
- Dustpan dredgers,
- Grab hopper dredgers,
- Bucket-wheel dredgers,
- Water injection/agitation dredgers,
- Barge unloaders, and
- Auger dredger.

In summary, there are a variety of tools or dredgers capable of being used on a project. All of the factors mentioned above will play a part in selecting the most suitable equipment for a particular job. Ultimately, however, if the work is to be accomplished by contract, the competitive bidding process will often serve as the final decision-making mechanism.

C2.5 Preliminary Planning and Design

Preliminary planning and design is rightfully a broad term in order to cover the range of activities to be considered during the planning and design phase.

Socio-Economic Questions for the Planning and Design Phase

Some of the basic elements of this phase are illustrated by the following questions:

- Is the project consistent with national policies and objectives?
- On the basis of total infrastructure needs, does it make good investment sense?
- Will the project further the national, regional and local socio-economic interests?
- Is the planning horizon sufficiently long both in hindsight data as well as future use projections?
- Does it incorporate the latest state-of-the-art in terms of equipment, procedures and throughput taking into account the ability to compete for scarce capital resources?
- Does it consider alternatives and include an analysis of trade-offs and cost and benefits for each of the alternatives?
- Does it adequately address the short, medium and long-term environmental consequences?
- Are data available from previous dredging works, whether capital, deepening and widening of channels and/or maintenance dredging?

Dredging and Engineering Aspects

Focusing on the dredging and engineering aspects, consideration should be given to the following:

Do adequate data exist relating to bathymetry, morphology, hydrodynamics, geology, meteorology and geography of the potential development sites? If not, these data sets are essential to evaluate the following: potential design scenarios; proper specifications for the works; good construction estimates; maximum use of natural forces both for construction and maintenance, such as scouring currents; impacts of salt water intrusion; alternative uses of dredged material; and finally, the subsequent maintenance requirements and costs.

- Does there exist what is often called, a List of Requirements? Such a list is a quantified assessment of all functions and parts of the project.
- Does there exist an adequate engineering staff to collect and analyze existing data, or adequately direct the collection and analysis of such engineering data; or, should consultants be employed? If consultants are to be employed, on which elements should they be used? Often it is advantageous to select a consultant at the beginning who has the capabilities and expertise to carry on through the various phases. This avoids the costs of the repetitive learning curve to bring new consultants on board for each phase.
- Does the plan consider the use or relocation of dredged material during construction and subsequent maintenance? Were sufficient options considered to minimise transportation costs for the dredged material and to consider the potential impacts of weather, sea state, and other elements on the construction and maintenance processes?
- In many cases today, a risk analysis or probabilistic analysis is conducted on the project. Using a technique such as a factor tree, the analysis will study which elements or facts result in the highest risk for the project. When the high-risk elements have been determined, the greatest study effort is directed to these factors or elements.

Importance of Data Collection

These are rather basic factors, but often the site investigation and data needs, between the planning or feasibility process and the design and construction process, are misunderstood. In many cases, two separate data sets are required. The latter and more detailed is that in the design phase, where such data are essential, to adequately specify the works in such a manner as to avoid unnecessary risk to the owner and to the contractors. There are many instances where preliminary site investigation data were inappropriately extrapolated into designs and specifications with costly and unsatisfactory results.

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Once the project feasibility has been determined from the planning phase, a preliminary design is often the next step. It further focuses the development of the project on the most effective engineering, environmentally responsible and economic alternative, and avoids some of the pain of overlooking any critical engineering elements.

C2.6 Design Phases of a Dredging Project

The design of a dredging project will go through several phases – a preliminary design, a more detailed design, a final construction design, and maintenance.

Design Phases

Dredging projects are undertaken for a variety of reasons, such as land reclamation, port development, flood control and environmental remediation. Once the decision has been made to proceed with a project, a rational design must integrate functional and performance requirements. In addition, boundary conditions, equipment and working methods need to be taken into account.

A good design will ensure the desired outcome of a project in the most cost-effective way without undesirable environmental impacts.

Preliminary Design

For all projects, whether capital, maintenance or remedial, a detailed design must be prepared. The design phase presumes that all feasible alternatives have been considered, and a single project has been identified which meets the economic, financial and environmental criteria established.

Detailed Design

The preliminary design is then developed into a detailed engineering design package. This detailed design package is the primary vehicle for developing a set of specifications suitable for soliciting international tenders from dredging contractors.

Furthermore, at the end of the detailed design, the client should have a reliable cost estimate upon which to base an evaluation of submitted tenders. There are a number of very important elements in the detailed design process.

Construction Phase

Following the design phase, the project moves into construction phase. This will require:

- Creating detailed specifications for the contract documents;
- Soliciting and awarding tenders;
- Establishing an administration structure and inspection scheme; and
- Establishing monitoring parameters for during construction.

Operation and Maintenance Phase

Upon the completion of the construction phase a plan should be in place to make sure that the operation and maintenance of the dredging project continues in a way to ensure the project functions properly. Since maintenance costs can be significant and are recurring, provisions should be made from the start and included in the overall design and planning.

C2.7 Project Management

Management of a dredging project requires a systematic, well thought out plan from the initiation of the project through construction and maintenance.

Types of Dredging Projects

Dredging projects are usually categorized into three groups – capital, maintenance and remedial – and each of these dredging processes occurs in four phases – excavation, lifting, transportation and placement. Yet each of these varies considerably from each other. Each dredging project will present its own unique set of issues and risks and each will require careful management.

Life Cycle Approach

To meet the demands, an integrated or life cycle project management approach and structure should be used. This should be collaborative and involve all parties and disciplines from the outset. These parties include:

- planners;
- design, cost and construction engineers;
- dredging experts;
- environmental scientists;
- economists;
- financial planners; and
- transportation specialists.

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The integrated and collaborative project management process ensures that all these players are involved throughout the process and not only in what is perceived as their “special” phases of the project.

Although the project evolves through the various disciplines, certainly each discipline and phase closely impacts on the others. Only through integration of the disciplines will the project avoid oversights, risks and costly errors.

C2.8 Design of a Reclamation Area

The design of a reclamation area is directly related to the performance requirements for the intended use of the area, within various boundary conditions.

Iterative Design Philosophy

The design philosophy should be based on a calculated process of matching the anticipated loading response of the fill mass to the requirements imposed by the future use of the reclaimed land. This takes place within the technical boundary conditions of the project. Clearly, the function of a beach nourishment differs from that of a reclamation area intended to accommodate an airport or a container terminal. Consequently, the corresponding technical specifications resulting from the design process will also differ. This philosophy must ensure that reclamation is not over-designed and that geotechnical specifications are reasonable, measurable and feasible.

An essential element of the design philosophy is an iterative design procedure: this means that during the development process the structural design of the superstructures may have to be re-considered.

Boundary Conditions

Once the functional requirements of a fill are defined, an adequate inventory of all boundary conditions has to be made. Boundary conditions may be external, often uncontrollable conditions that could affect the performance of the hydraulic fill including the quality and quantity of the fill material available, the nature of soils underlying the future fill area, earthquakes and other natural hazards, tides and waves.

Construction Methods

Usually the contractor selects or proposes the most cost-effective and suitable construction method to realize the fill mass properties according to the performance requirements and

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within the boundary conditions. It is essential, however, that the designer of a hydraulic fill also has a thorough understanding of the possibilities and limitations of the various construction methods including dredging and reclamation operations and soil improvement techniques.

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Design Cycle

The design of the fill mass is one part of the design cycle of a total project. The design of the total project follows these steps:

- Define functional requirements of fill mass;
- Define performance requirements of fill mass;
- Determine boundary conditions;
- Choose construction method;
- Based on the above, create technical design;
- Evaluate feasibility of design including timing, risks and quality control;
- Establish the technical specifications.

If it not be possible to identify a construction method (and – if required – a soil improvement technique) that results in a fill mass that meets the specified performance requirements without excessive costs, the engineer may have to reconsider the starting points of the fill design.

This will generally imply adjustments of the design of the superstructures resulting in adapted performance requirements.

Design Changes

Design changes of the structures may include modifications to foundations, slope angles, drainage facilities and/or revetments, alterations of final grade levels, the introduction of soil retaining structures, and other aspects.

Whether to apply a soil improvement technique or to adjust the design of the superstructures may often be decided on the basis of costs, but technical feasibility and low failure probability may be important criteria as well.

Soil improvement methods include techniques such as surcharging (in combination with vertical drains), compaction, deep soil mixing and vibro replacement.

Economics and technical feasibility of the reclamation and soil improvement techniques may affect the geometry of a reclamation area as well. Long pumping distances, for instance, may result in the need for a booster pump station. Adjustment of the geometry of the proposed reclamation area could reduce the pumping distance and, hence, lower the costs.

C2.9 Systems Engineering

Systems Engineering is a worldwide accepted tool for the design of complex projects. The essence of Systems Engineering is that it is interdisciplinary, focusses on defining customer needs and required functionality early in the development cycle and is subject to a structured evaluation and feedback system as this cycle rolls out.

Client's Needs

The client's needs and functionality requirements are input to the design and feasibility analysis of the project. Steps in the development cycle are evaluated at certain pre-defined moments and if needed adjusted. Such adjustments may include the design itself, but also the needs and functional requirements formulated for the project.

Structured evaluation and feedback throughout the development cycle is aimed to lead to the most effective and economical solution of the client's needs. As the development cycle proceeds the outline of the system becomes increasingly detailed.

Development Cycle

To make maximum use of the appropriate expertise, the integrated approach may enlist advisors and contractor specialists at an early stage of the development cycle. This will ensure that the needs of the client and stakeholders are satisfied in a high quality, trustworthy, cost-efficient and schedule-compliant manner throughout a systems' entire life cycle. This systems engineering process is usually comprised of the following seven tasks:

- State the problem;
- Investigate alternatives;
- Model the system;
- Integrate;
- Launch the system;
- Assess performance, and;
- Re-evaluate.

These functions (State, Investigate, Model, Integrate, Launch, Assess and Re-evaluate) are summarized with the acronym 'SIMILAR'. It is important to note that the Systems Engineering Process is not sequential and functions are performed in a parallel and iterative manner.

Boundary Conditions

The transformation of the functional requirements of a project into a technical design is directed by the boundary conditions and the construction method. These boundary conditions are established by project data such as soil and environmental conditions.

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Given the project's boundary conditions and the construction methods, the feasibility of the design must be judged by the criteria established for its ultimate use. If these criteria are not met, then the project may have to be re-considered.

C2.10 Financing

Financing of a project is often based on the owner providing a clear plan and preliminary engineering design, including adequate environmental provisions.

Lending Institutions

Financing for reclamation and development projects is critical, especially when they are multi-year projects that require large investments upfront that will not reap economic benefits until after completion.

Worldwide there are many lending institutions, ranging from the World Bank, to more than a half dozen regional development banks. Most of them attach great importance to environmental aspects of development projects.

For reclamation projects as well as port and harbour developments and expansions, the owners and governing authorities, consulting firms, and engineers and other staff need to offer effective and thorough environmental input into the project concept, preparation, detailed engineering, construction and operation.

Environmental Aspects

In addition to traditional lending institution requirements, financing without adequate environmental assessments and mitigation measures where necessary, as well as attention to public health and safety, are prerequisites to secure loans.

Equator Principles for Emerging Markets

Another important development in financing is The Equator Principles, a risk management framework, adopted by financial institutions, for determining, assessing and managing environmental and social risk in projects. It is primarily intended to provide a minimum standard for due diligence to support responsible risk decision-making.

The Equator Principles apply globally, to all industry sectors and to four financial products:

- Project Finance Advisory Services
- Project Finance
- Project-Related Corporate Loans and
- Bridge Loans.

Since its establishment in 2003, almost 90 international banks have adopted the Equator Principles, including the majority of the world's leading project lenders. They are known as Equator Principles Financial Institutions (EPFIs) and are present in 37 countries, covering more than 70 percent of international project finance debt in emerging markets. The Equator Principles apply to all new project financings globally with total project capital costs of US\$10 million or more, and across all industry sectors.

Public-Private Partnerships

Major dredging operations like reclamation and port infrastructure projects are generally very costly, and can strain the financial capacity of state and local agencies, including quasi-public port authorities with independent bonding and taxing powers.

Public-Private Partnerships (PPPs) are a means to attract the private sector to the development, planning, financing, construction and operation of infrastructure projects. The financing aspect is especially of great interest, because in a Public-Private Partnership the private sector is expected to provide (part of) the capital for an investment. For the duration of the contract, the contractor takes over the construction and/or operating risks in exchange for earnings from the project. The private partner will also assist in maintaining and/or operating the infrastructure in exchange for providing some or all of the financing.

Negotiating this type of financing agreement requires the public and private partners to identify their roles and respective risks. Risks for the public partner include higher port fees necessitated by private financing. Those for the private partner include contractual limits on the adjustments it can make to address cost overruns and revenue shortfalls.

C2.11 Types of Dredging Projects

Dredging projects are often classified into three categories: capital, that is, for new infrastructure, maintenance, or remedial (environmental) projects.

Capital Dredging

Dredging which is carried out in a new location and in material that has never been dredged before, is referred to as a capital project. For instance, the land reclamation for airport platforms and artificial islands, for new ports development, deepening and widening access channels, berthing places, docks, and turning basins are a permanent necessity in order to meet the needs of growing trade and economy. Capital dredging is conducted with attention to sustainability and with respect for nature.

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Maintenance Dredging

Maintenance dredging is recurrent dredging to maintain or improve existing waterways, even if a number of years pass between consecutive dredging campaigns. Once a dredging operation has been completed, maintaining the site or port is of great importance to ensure that the initial investment remains intact and functions as planned.

Remedial Dredging

Remedial dredging is the term used to describe projects where the removal of the material is being carried out purely for environmental reasons, to improve the quality of the site in some way. This often entails the removal of contaminated soils. It often occurs at industrial sites where historically less control of pollution was in place and where the regulations today demand an environmental solution.

C2.12 Identification of Stakeholders

Identifying all stakeholders and bringing them into the project development process as soon as possible is a crucial step in ensuring a successful outcome.

Stakeholders are Shareholders

All stakeholders in a major maritime project are shareholders because they share a common interest in the success of the project. Stakeholders may be individuals or organizations, public or private, but each has specific concerns and information. They also have unique knowledge of the project from many different sides of the issues.

Consensus

One reason to identify all stakeholders is that this will give them an opportunity to point out problematic issues that the client and contractor may not be aware of. It gives them the chance to confront issues as challenges and help in finding suitable solutions. Identification of all stakeholders encourages these parties to “buy in” to a project and that may eventually make the difference between having a group of partners supporting the development, rather than having a group of organizations in opposition to development. Whilst this process takes more time upfront, finding a consensus amongst stakeholders will allow a project to run more smoothly and will save time in the long run.

Types of Stakeholders

Stakeholders for any given project may include:

- Politicians on local, national or international levels;
- Financiers like bankers, also on local, national or international institutions;
- Activists and pressure groups;
- Contractors who are working on the project;
- Consultants hired as advisers with specific pools of knowledge;
- Administrators and standards institutes that regulate projects – owners who are responsible for the project; and
- Polluters, who may have to contribute to remediation efforts and costs.

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Systematic Approach

To fully utilise the information that stakeholders can provide, a system should be in place that allows time for the analysis of potential challenges and time for all parties to react. Such a system would include:

- identifying the challenges and raising awareness;
- searching for solutions which will require input from all interested parties, including risk analysis and probability studies;
- implementation of the solution, which will lean heavily on the expertise of the contractor; and finally,
- management of the works following guidelines and procedures that have been previously established.

By identifying stakeholders during the planning of the project and inviting them into the decision-making process, many obstacles can be overcome and a dredging project can be implemented in a smooth and timely fashion.

C2.13 Work Plan for Reclamation Projects

Before starting the filling operations at a reclamation site, a work plan should be made of the dredging, transport and reclamation activities.

Work Plan Elements

Such a work plan for reclamation could include:

An inventory of all relevant contractual and technical specifications including the lay-out of the reclamation area, the required fill (mass) properties, testing and monitoring requirements, possible milestones, etc.

- An inventory of all relevant boundary conditions such as:
 - Bathymetry (of borrow area, the sailing route of the dredging vessels, and of the reclamation area and its surroundings)
 - Nature of the existing subsoil at borrow area and reclamation site
 - environmental regulations
 - Navigational regulations
 - Required permits;
- availability of plant;
- a detailed project planning;
- a dredging plan of the borrow area (including dredging equipment);

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- a fill transport plan (pipeline, booster pump stations, trailing suction hoppers, barges, or other equipment);
- the results of geotechnical analyses (settlement and stability of bunds and fill);
- a construction method of the bunds;
- a filling plan (including the plant and equipment required, the number and thickness of lifts, consolidation periods, testing schedules;
- a soil/fill ground improvement plan (vertical drains, compaction), if necessary;
- a monitoring and quality control plan;
- a risk assessment including possible mitigating measures.

Work Underwater or Above Water

Fill may be placed underwater or above water with land-based equipment or with waterborne equipment. Working methods and equipment required for these operations differ considerably and resulting densities, natural slopes, segregation, and so forth, will be different as well.

When considering various construction methods it is good to remember that the most appropriate methods usually depend heavily on the local conditions, the availability of plant, contractual requirements, and so on, and can only be selected after a careful inventory of all relevant aspects has been made.

C2.14 Post Construction Maintenance

Maintenance after completion of a dredging project will ensure the investment in the project is maximized by lengthening the lifetime of the project.

Maintenance Follow-Up

Once a dredging project is completed, the need to care for the works that have been constructed should not be underestimated. Construction and extension of a port and access channels is not the end of the process.

Following the deposition of sediments, access channels and harbour basins may become silted up quite rapidly. It is of vital importance to ports that the depth of waterways is at all times maintained so that they always remain accessible to shipping traffic. Therefore, large-scale maintenance contracts are often put out to tender, and this sometimes will result in long-term contracts.

As sedimentation and siltation are natural phenomena, they are always a threat to navigation and accessibility of ports, even when the ports have been dredged to a navigable depth during port expansion.

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Cost of Maintenance

For that reason, maintenance should not be overlooked, especially as the costs for maintenance can be high and the need is not always obvious. A plan for maintenance dredging should be part of a management plan and the costs of recurring operations should be budgeted in the total project plan.

Amongst issues that may arise that require maintenance are:

- siltation of channels that need to be deepened;
- breakwaters and groynes needing repair; and
- the addition of navigational aids.

Maintenance dredging will require clear knowledge about the dynamics of an estuary, and even more experience on how to deal with tides, currents, flood channels, bars, bends, and the various nature of suspended material. A contractor who has executed the original capital dredging will possess detailed knowledge of these elements and so creating a plan for maintenance of the works is a logical extension of a capital dredging project.

Water Quality

Maintenance dredging projects may also aim at improving the water quality, where contaminated sediments are removed and decontaminated or disposed in a safe manner.

As with all dredging work, maintenance dredging must be carried out with respect for the environment, reducing overflow and limiting turbidity. Maintenance dredging when executed properly and in a timely fashion will support the natural processes and uses the river dynamics for keeping the gateways open.

C3 Prefabricated Vertical Drains (PVD)

<https://www.geoengineer.org/education/web-class-projects/cee-542-soil-site-improve-winter-2014/assignments/prefabricated-vertical-drains>

C3.1 General



Figure C3.1 – Site of Prefabricated Vertical Drains installed by Rigs

Prefabricated Vertical Drains (PVDs) or 'Wick Drains' are composed of a plastic core encased by a geo-textile for the purpose of expediting consolidation of slow draining soils. They are typically coupled with surcharging to expedite preconstruction soil consolidation. Surcharging means to pre-load soft soils by applying a temporary load to the ground that exerts stress of usually equivalent or greater magnitude than the anticipated design stresses. The surcharge will increase pore water pressures initially, but with time the water will drain away and the soil voids will compress. These prefabricated wick drains are used to shorten pore water travel distance, reducing the preloading time. The intent is to accelerate primary settlement. Pore water will flow laterally to the nearest drain, as opposed to vertical flow to an underlying or overlying drainage layer. The drain flow is a result from the pressures generated in the pore water. Figure 2 below demonstrate vertical water flow without the use of prefabricated wick drains, and horizontal water flow with the use of wick drains.

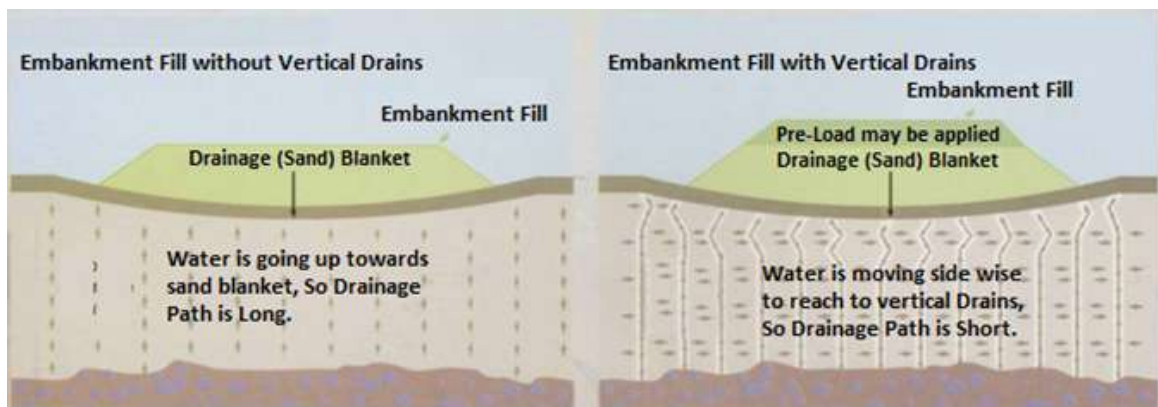


Figure C3.2 – Cross Section with and without Vertical Drains

Initial settlements are seldom of much practical concern, except for loads on thick plastic or organic soils having marginal stability wherein large shear deformation may continue due to un-drained creep.

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Prefabricated wick drains have several other purposes. They may be applied to reduce potential down drag on piles, or increase storage capacity for future landfills and waste containment sites. A developing application for PVD is the collection and extraction of contaminated groundwater, which may be coupled with cutoff walls to ensure full withdrawal.

C3.2 Drain Features

PVDs have a channeled or studded plastic core wrapped with a geotextile. The plastic core functions as support for the filter fabric, and provides longitudinal flow paths along the drain length. It also provides resistance to longitudinal stretching as well as buckling of the drain. The drain jacket acts as a filter to limit the passage of fine grained soil into the core area. It also functions to prevent closure of the internal water flow paths under lateral soil pressure.



Figure C3.3 – Layfield and Geosupply Wick Drains.

Light-weight drains have a width to thickness ratio of 30-35. It is desirable for the surface area, which will permit seepage into the drain, to be $0.2\text{-}0.3\text{ in}^2$ ($150\text{-}200\text{ mm}^2$) per 0.4 in (1 mm) length. The wick drains should be installed with a center to center spacing of 3 to 8 ft. This will be discussed more in the design parameters section.

C3.3 Equivalent Diameter

Wick drains have an oblong shape, but the available theories of radial consolidation have been derived for drains having a circular shape, often taken as equal to the nominal diameter of the sand drain. Fellenius (1977) suggested that the equivalent cylindrical diameter of a sand drain is the nominal diameter of the sand drain multiplied by the porosity of the sand in the drain [2]. The porosity of loose, free-draining sand can range from 0.4 to 0.5. This would result an equivalent cylindrical diameter of a sand drain of approximately half of the nominal diameter. In order to apply these theories to the design of field installations of wick drains, the equivalent diameter of an oblong shape is needed. Many equations have been suggested for computing the equivalent diameter, but each computation yields different results because they were derived under different assumptions.

The equations predicting the progress of consolidation around vertical drains assume a circular cross section for both the drain itself and the area the drain influences (Barron 1948).

Common wick drains have a plastic core surrounded by a geotextile:

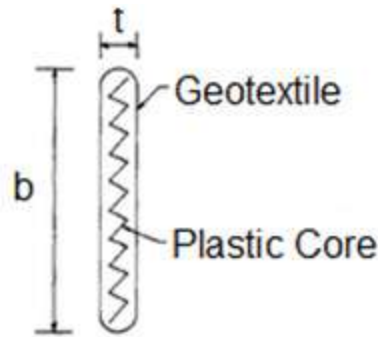


Figure C3.4 – Cross Section of PVD

By using a finite-element analysis Hansbo (1979) reported that the equivalent diameter of a drain is given as:

$$dw = 2 \times ((b + t) / \pi) \quad (1)$$

Where:

dw = equivalent diameter of the drain

b = width of the prefabricated drain

t = drain thickness

Rixner (1986) suggested the following few equations:

$$dw = (b + t) / 2 \quad (2)$$

$$dw = [(4 \times (b \times t)) / \pi] \times 0.5 \quad (3)$$

$$dw = 2 \times (b' + t') / \pi \quad (4)$$

$$dw = [(4 \times (b' \times t')) / \pi] \times 0.5 \quad (5)$$

Where:

b' and t' are reduced dimensions to account for clogging of the drain surface

$2 \times (b' + t')$ is used for free or open drain circumference

$b' \times t'$ is used for free or open drain cross section

In an independent evaluation by Richard P. Long and Alvaro Covo (1994), an electrical analog was used to define an equivalent diameter through the evaluation of varying results by a set of previously listed equations'. A 'Sunshine Analog Field Plotter' and electrically conducting paper were used. This technique uses the analogies of electrical potential to hydraulic head and electrical current to flow of water.

The procedure is represented as steady-state water flow for a model of $1/4$ of the drain cross section. A quarter of the cross section is used to eliminate repetition since the drain has two axes of symmetry. The model, seen below, is not applicable in the immediate vicinity of the drain at short times after consolidation begins. When water flows through a circular drain, rather than the supposed oblong profile, a shape factor for the flow net should be used.

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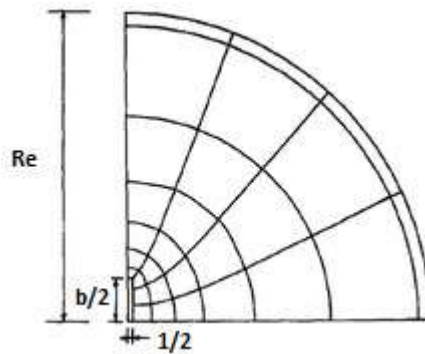


Figure C3.5 – Flow Net for Flow to Oblong Drain from Circular Source

$$\text{Shape Factor} = N_f / N_d = 2 \times \pi \times \ln(R_e / R_w) \quad (6)$$

R_e = outside radius of the area

R_w = radius of central well

N_f = number of flow paths

N_d = number of head drops

R_e , b , and t were varied throughout the Foott & Ladd (1981) study, and for each combination of values, a flow net was constructed using an analog field plotter. Using the shape factor and a summarization of the field plotter results, an equivalent diameter was determined.

$$d_w = 2 \times R_w$$

The results of the analog modeling are plotted below. The ratio of the computed equivalent diameter of the drain, d_w , to the width, b , is plotted as the vertical axis and the ratio of the thickness of the drain, t , to width of drain, b , is plotted as the horizontal axis. The ratio of the thickness to width, t/b , in the analog model varied from 0 to 1.0, which includes all possible combinations of drain thickness and width. The results are represented by using an x and a straight line of least squares fit is shown.

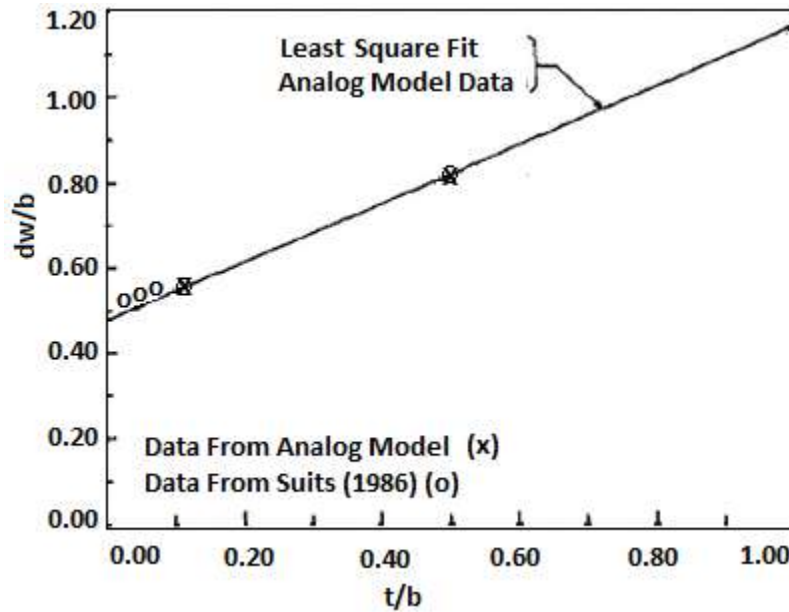


Figure C3.6 – Plot of Equivalent-Drain-Diameter to Width-of-Drain Ratio, d_w/b versus Thickness-of-Drain to Width-of-Drain Ratio, t/b

In Fig. C3.4 circles are representing Suits et al. (1986) results from an experimental evaluation of equivalent diameters of drains.

Suits et al. method requires determining the coefficient of consolidation for remolded soil samples by standard laboratory tests, then consolidating the same soils using sand wick drains. The soils used for this experiment include, but are not limited to clay, organic silty clay, and peat. The reported values are approximately equal to the prediction from the linear least squares model.

Assuming negligible resistance to water flow, a unique value for d_w , based on the model data for drain thickness and width, may be calculated. Equation (7) indicates that for $t = b$, $d_w = 1.2b$, approximate diameter of a circle with area b^2 . Equation (7) yields an appropriate value of the equivalent diameter of the drain for all t/b .

$$d_w = 0.5 \times b + 0.7 \times t \quad (7)$$

Values calculated from equations (1), (2), (3), and (7), and are combined with average experimental values from Suits et al. (1986) are shown in Table 1. The product brands VINYLEX and AMER-DRAIN were tested with clay only, but all over drain types were tested with each soil. Equations (2) and (7) report values similar to the experimental values. Whereas equation (1) values are consistently larger, and equation (3) values are smaller. Equations (2) and (7) seem to yield regularly agreeable values, however when $b = t$, the two methods yield results that vary by 20%.

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Table C3.1 – Equivalent Drain Diameters by Various methods

Drain Type (1)	Drain Diameter						Suits et al. (1986) (8)
	b (mm) (2)	t (mm) (3)	Eq. (1) (4)	Eq. (2) (5)	Eq. (3) (6)	Eq. (7) (7)	
ALIDRAIN	103	7	70	55	30	56	43-55
VINYLEX	95	5	64	50	25	51	48
AMER-DRAIN	98	4	66	51	22	52	40
COLBOND CX1000	98	5	66	51	25	52	55-57
BANDO	94	4	63	48	22	50	46-51
CASTLEBOARD	94	4	63	48	22	50	42-45

In summary, the equivalent diameter is essential for proper interpretation of field data to assess field conditions. The equivalent diameter of the shape of prefabricated drains was determined by using an electrical analog field plotter. Values determined by this method agree with experimental values of Suits et al. (1986). Equivalent diameters of presently available prefabricated drains can be computed with reasonable accuracy either from equation (2) or from equation (7), but (7) yields a more accurate value for all t/b.

C3.4 Advantages and Disadvantages of Wick Drains

C3.4.1 Advantages

- Decrease overall time required for completion of primary consolidation due to preloading
- Decrease the amount of surcharge required to achieve the desired amount of Pre-compression in the given time
- Increase the rate of strength gain due to consolidation of soft soils when stability is of Concern
- Comparison to sand drains:

Economic competitiveness, less disturbance to the soil mass compared to displacement sand drains, and the speed and simplicity of installation. Also feasible to be installed in a non-vertical orientation.

Rixner, Kraemer, and Smith (1986) have also outlined some technical advantages of PV drains compared to sand drains. They separate these advantages by sand drain type. There is displacement drains, which displace the soil as the drain is installed, and non-displacement, which does not displace the soil during drain installation. They found the following advantages of PVDs:

Displacement

- considerably less disturbance of cohesive soils during installation due to: smaller physical displacement by mandrel and top, and typically static push rather than driving
- installation equipment usually lighter, more maneuverable on site
- do not require abundant source of water for jetting

Non-Displacement

- do not require control, processing and disposal of jetted spoil materials; fewer environmental control problems
- field control and inspection not as critical
- definite potential for cost economy
- eliminate cost of sand backfill of drains, quality control problems and related truck traffic
- job control and inspection requirements are reduced due to simplification of installation procedures

General

- there is greater assurance of a permanent, continuous vertical drainage path; no discontinuities due to installation problems
- PVDs can withstand considerable lateral displacement or buckling under vertical or horizontal soil movement
- faster rate of installation possible
- where very rapid consolidation is required, it is practical to install PVDs at close spacing
- PVDs can be installed underwater and in a non-vertical orientation more conveniently

C3.4.2 Disadvantages

- If the compression layer is overlain by dense fills or sands, very stiff clay or other obstructions, drain installation could require predrilling, jetting, and/or use of a vibratory hammer, or may not even be feasible
 - Under such conditions, general pre-excavation can be performed if appropriate
- Where sensitive soils are present or where stability is of concern, disturbance of the soil due to drain installation may not be tolerable
 - In such cases, sand drains installed by non-displacement methods or an alternate soil improvement technique may be more practical
- Winter Considerations
 - During cold winter conditions, the ground is subject to frost. A frost line, the depth at which the ground is subject to freezing, can typically be measured at 3 ft in the mid-west United States, for example. The frost can reduce or prevent the drain discharge at the groundwater table or into the drainage blanket at the ground surface, building up a back pressure. The build-up of back pressure will temporarily retard the time development of the consolidation settlement; perceived as a flattening time-settlement curve. The flattening may lead to a false premise that primary consolidation has reached an end. However, once winter conditions pass, and the ground within the frost line thaws, settlement has potential to reinstate.

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C3.5 Suitable Soils

It is commonly assumed that wick drains should be used in all soils that compress very slowly under natural drainage condition due to low soil permeability and relatively great distance between natural drainage boundaries. However, PVD are not versatile for all soil types and geological conditions. The drains can be implemented in soils that are moderately to highly compressible under static loading. Applicable soils include:

- Inorganic silts and clays of low to moderate sensitivity
- Organic layers
- Decomposed peat
- Clayey and silty sands
- Dredge spoils
- Varved (sedimentary stratification) cohesive deposits

C3.6 Installation

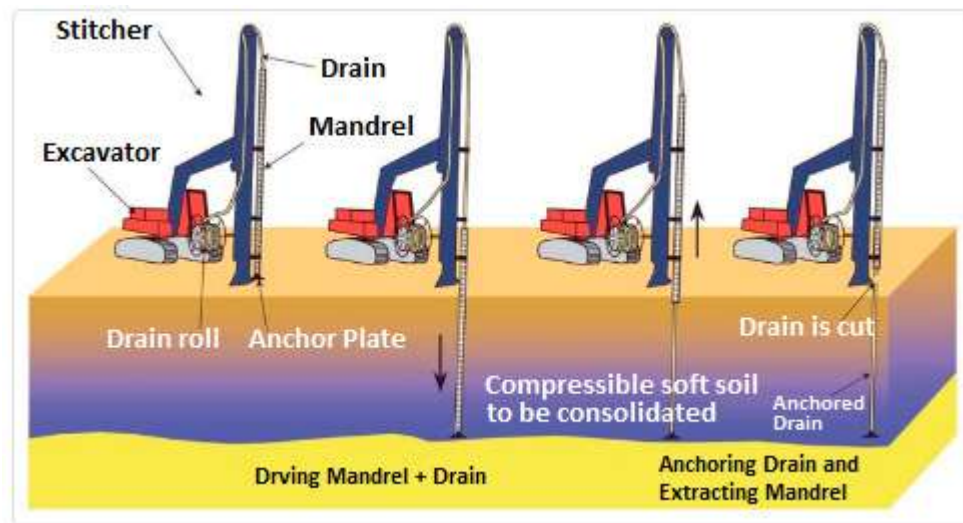


Figure C3.7 – PVD Installation

PVDs are installed by a hollow steel mandrel encasing the wick drain material. The mandrel is driven into the ground by a stitcher attached to an excavator carrier, as seen in figure C3.5. This is a vibrating force, but static options are also available for areas near underground utilities. At the base of the mandrel, the wick is looped through a steel anchor to secure the drain in place. Once the desired depth is reached, the drain is anchored and the mandrel is extracted. The mandrel is withdrawn 15 to 20 cm above the surface for the wick drain to be cut. If the soil, in which the mandrel is driving into is exceedingly stiff, and the mandrel cannot be vibrated or hammered into the ground, predrilling may become necessary.

C3.6.1 Depth of Installation

Drains are not likely to accelerate consolidation if induced effective stress is not greater than the pre-consolidation stress. The optimum depth of the wick drains lies within the pre-consolidation stress

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margin as the stress from the surcharge diminishes with depth. However, if there is a pervious soil layer below the pre-consolidation margin, the wick drain should be extended into that soil layer. This will aid in assuring the discharge of the water.

C3.6.2 Width of Installation

Soil strata are not defined as entirely uniform layers, therefore there may not be equal volumes of water to be drained. If some portions of a layer have a greater amount of drainage, the soil will settle to fill those voids. This leads to differential settlements and could prolong the consolidation time. To help avoid this issue, wick drains should be distributed across the entire footprint of an embankment and a small distance beyond. It is advised to place the outermost rows of drains between one third and one half of the proposed embankment's height beyond the embankment. However, when designing the wick drain's layout, homogeneous soil can be assumed for simplicity.

C3.7 Design Parameters

The design of PVDs requires an evaluation of design parameters including Soil Characteristics and Drainage Properties as well as the effects of PVD installation. The size of the project can influence the amount of effort needed for evaluation.

This section refers to the following project descriptions as defined by Rixner et al. (1986):

Table C3.2 - Project Categories and descriptions by Rixner et al. (1986)

Project Category	Description
A	Basically uniform soil (No varving, low to moderate sensitivity) Simple construction (no staged loading) PVDs few in number
B	Increased complexity, Intermediate
C	Unusual soils (Varved, or high sensitivity) Staged loading, complicated construction PVDs numerous in number (length > 18m (60ft))

C3.8 Soil Characteristics

All soil property values used in design of wick drains should be evaluated at the maximum vertical effective stress that shall be applied to the compressible soil in the field.

Coefficient of Consolidation for Horizontal Drainage (C_h) and Coefficient of Permeability for Horizontal Seepage (k_h)

Category 'A' projects may use C_h conservatively estimated as approximately C_v ($k_h/k_v = 1$) measured in lab from one-dimensional consolidation tests (ASTM D2435). There are estimates for k_h , but field and lab measurements should be taken to compare and adjust the value.

$$C_h = (k_h / k_v) \times C_v \quad (8)$$

Category 'C' projects should have more accurately estimated values for C_h using any variation of methods. In-situ piezometer probes and analysis of pore pressure dissipation curves may be of reference. In-situ determination of k_h by small-scale pumping tests in piezometers or self-boring permea-meters can be used with laboratory m_v values to form a relationship for C_h [9].

$$C_h = k_h / (m_v \times \gamma_w) \quad (9)$$

Where:

γ_w = unit weight of water

M_v = coefficient of volume change

It is generally recommended to employ conventional consolidation tests to measure C_v combined with field and laboratory investigations to estimate k_h/k_v and then evaluate C_h via the Category A equation.

Coefficient of Permeability in the Horizontal Direction in the Disturbed Soil (k_s)

The Ratio of k_h/k_s range 1-5, and is expected to vary with soil sensitivity and the presence or absence of soil macro-fabric.

C3.9 Drainage Properties

Drain Influence Zone

Table C3.3 - Drain Influence Zone (D)

Pattern	D as a function of S
Square	$D = 1.13 \times S$
Triangular	$D = 1.05 \times S$

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The drain influence zone (D) is a function of drain spacing (S) only. Vertical drains are commonly installed in square or triangular patterns. Square pattern layouts have greater ease and control in the field. However, the triangular patterns are preferred to provide more uniform consolidation between drains. Shown below are diagrams of the square and triangular patterns.

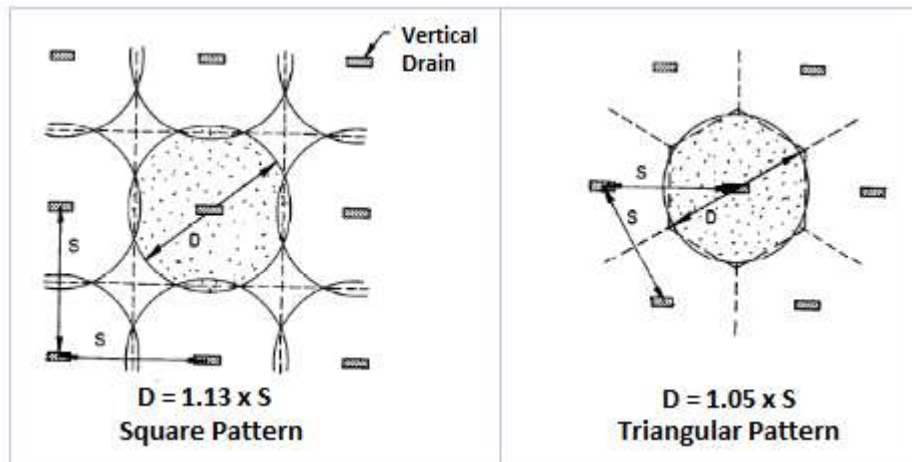


Figure C3.8 – Installation Patterns of Vertical Drains

C3.10 Effectiveness of Prefabricated Vertical Drains (PVDs)

In order for effective expedition of the consolidation process, the PVDs must accept water from the surrounding soil, and discharge it. The initial design conditions and changes in these factors throughout the length of the project will affect the consolidation rate/success of the project.

C3.10.1 Water Flow into Drain

Hydraulic Conductivity

The hydraulic conductivity of the soil surrounding the drain will control the rate at which water can enter the drain and relieve pore water pressures in the soil.

Smear Zone

The installation of PVDs require the use of a steel mandrel, which firmly clamps the drain during its insertion into the ground. Accompanying the mandrel, is an anchor plate fixed to the bottom of the drain, which serves to prevent soil from entering the drain and to keep the drain in place upon removing the mandrel. Below is a simple diagram of the anchoring system [11].

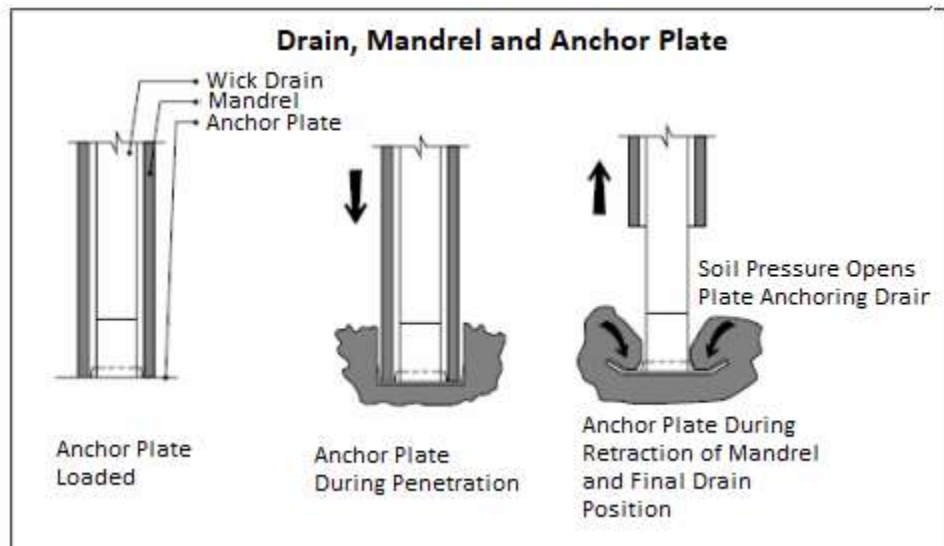


Figure C3.9 – Wick Drain Anchoring System and Steps

As the mandrel is pushed downward and upon its removal once the drain is in place, it disturbs the soil surrounding the drain producing shear strains and displacement that decrease its hydraulic conductivity.

Minimizing the mandrel cross sectional area will reduce potential for soil displacement and disturbance. It may be suitable to taper the mandrel tip, as long as the stiffness is not sacrificed. For soil profiles with many different layers, large k_h/k_v ratios, the laying may enhance horizontal permeability. It is possible to retard the lateral seepage of pore water into the drains by smearing of pervious and less pervious layers. Static pushing is preferred to drive or vibrating the mandrel in sensitive soil, but may cause buckling or wobbling of the mandrel. An idealization of the mandrel disturbance area is shown in Figure C3.8.

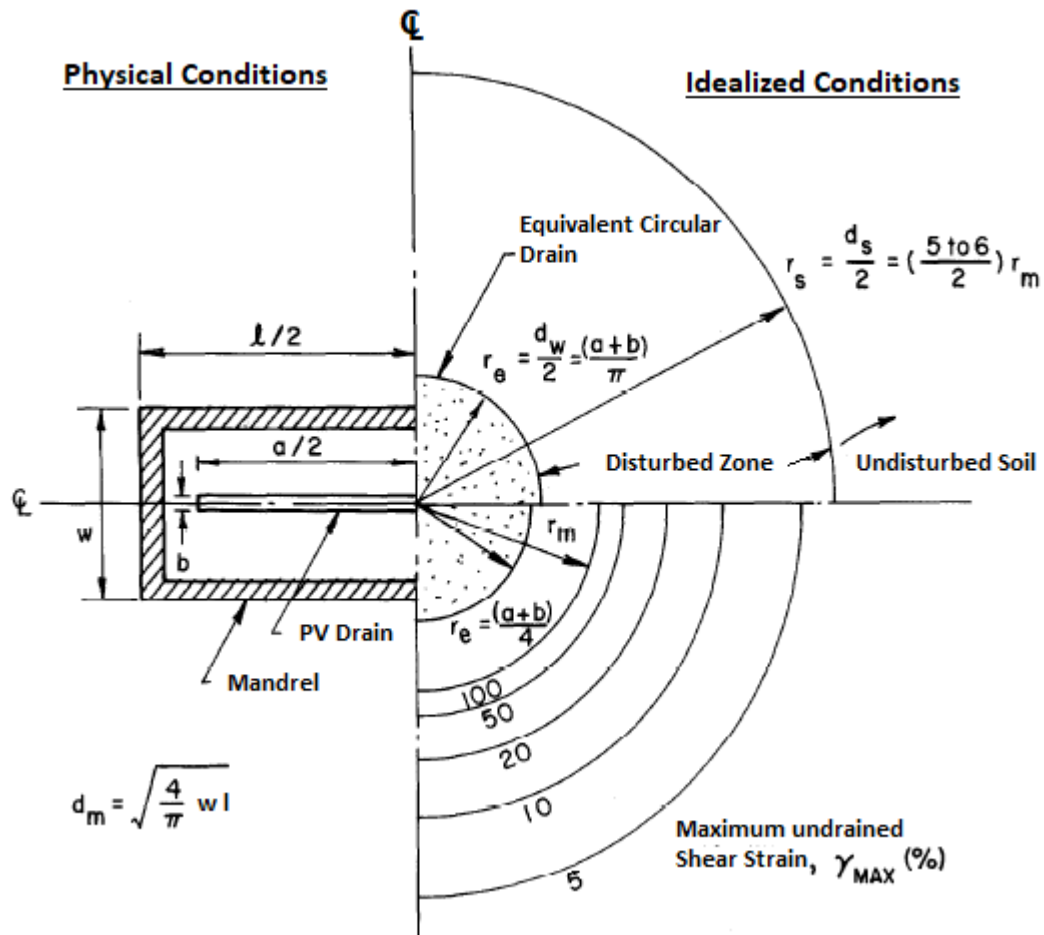


Figure C3.10 – Approximated Disturbed Zone Around the Mandrel

Although there have been numerous studies conducted to assess properties of the smear zone including range, shape, and effect on hydraulic conductivity, there is no precise consensus among researchers. Nevertheless, some generalities are listed below:

- The larger the mandrel, the larger the smear zone.
- The shape of the mandrel affects the shape of the smear zone
- Square/Circular Mandrel – Square/Circular Smear Zone
- Rectangular Mandrel – Ellipsoidal Smear Zone
- The outer boundary of the Smear Zone has been found to range from 4-18 times the equivalent mandrel radius.
- The overlap of smear zones from adjacent drains complicates smear zone calculations further

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C3.10.2 Discharge Capacity

Design

The design discharge capacity of the drain is a function of its cross-sectional area (core available for flow). This capacity can and will likely be reduced immediately upon installation and over the lifetime of the project.

Installation

The installation of the drains is a rough process and is not only destructive of the installation equipment itself (mandrels often need be replaced over the course of a project, and installation equipment repaired), but is also presents the critical case for the mechanical properties of the drain. Grab and puncture tests as per ASTM standard of the geosynthetic material are important in this regard.

Reduction with Time

Over the lifetime of the project a number of factors can reduce the discharge capacity of the drain and slow down the consolidation process.

Clogging

Clogging of the PVD can significantly reduce the discharge capacity of the drain. A filter with appropriate properties (such as AOS – Apparent Opening Size, per ASTM D 4751) will pass water, while retaining clay particles and limit clogging. The larger the drainage channel (cross-sectional area of the drain), the less the discharge capacity will be affected by clogging, all else held equal.

Bending/Kinking

As the soil consolidates, reducing the thickness of the clay layer, the drain undergoes bending and/or kinking deformation. There are various forms of possible bending/kinking mechanisms.

Whether the drain kinks or bends depends that depends on the flexibility of the drain and modulus of the surrounding clay. A more flexible drain core will lead to greater reduction in discharge capacity. Furthermore, sharp kinks will reduce flow through the drain more than gradual bends.

Lateral Earth Pressures

Lateral earth pressures can cause the filter of the drain to pass into the core in a punching manner, and thus reducing the flow.

Biological Degradation

Biological and chemical activity can reduce discharge capacity as well.

C3.11 Recent Development and Future of PVDs

Quality control of PVDs have improved in recent years with the incorporation of electronics to the installation equipment. Depth, installation force, GPS coordinates, and date/time information has been adopted in certain cases. This level of quality control is often not needed, as projects have consistently been successful without such measures.

As geo-synthetics continue to improve on the basis of the targeted function, installation monitoring techniques become more sophisticated, and researchers continue to better understand factors that affect the discharge capacity of drains and the smear zone effect on hydraulic conductivity of the surrounding soil, surcharge loading with PVDs will only improve as a method of expediting consolidation of cohesive soils for settlement sensitive projects.

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Figure and Table List

[A] Prefabricated Vertical Drains

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[D] Geosupply Wick Drain Example

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[G] Plot of Equivalent-Drain-Diameter to Width-of-Drain Ratio, d/b versus Thickness-of-Drain to Width-of-Drain Ratio, t/v

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C4 Case Study: Mega Changi East Land Reclamation Project

Quality management of prefabricated vertical drain materials in mega land reclamation projects: A case study Myint Win Bo^a, Arul Arulrajah^b, Suksun Horpibulsuk^{b,c,*}, Melvyn Leong^d

^aDST Consulting Engineers Inc., Thunder Bay, Canada

^bSwinburne University of Technology, Melbourne, Australia

^cSchool of Civil Engineering, Suranaree University of Technology, 111 University Avenue, Muang District, Nakhon-Ratchasima 30000, Thailand

^dGeofrontiers Group Pty Ltd., Melbourne, Australia

The Japanese Geotechnical Society (www.sciencedirect.com) Journal homepage: www.elsevier.com/locate/sandf Soils and Foundations

C4.1 General

A case study of quality management of PVD materials in the Mega Changi East Land Reclamation Project in the Republic of Singapore is presented in this technical report. PVDs are increasingly being used as the preferred ground improvement option in mega land reclamation projects on soft soil or ultra-soft soil deposits, particularly in Asia. This is because they are economical, simple to install and readily availability. PVDs are particularly suitable for projects that have long construction durations, since this ensures sufficient time for the required degree of consolidation to be attained. PVDs are often used in locations with thick deposits of soft soil in marine or estuarial environments. PVDs are implemented with a combination of surcharge preloading or vacuum preloading to induce the occurrence of consolidation settlement during the construction period and to minimize post construction settlement.

The Changi East land reclamation project in the Republic of Singapore, undertaken from 1992 to 2005, was a mega land reclamation and ground improvement project. Approximately 140 million linear meters of PVD were installed in this project in an area totalling approximately 1000ha. Due to the large quantity of drains which needed to be installed, it was not practical to solely rely on one specific type of PVD supplied from a single source and installed by a single contractor.

As such, various types of PVD and several installation contractors with different types of installation rigs were deployed in the project. A rigorous quality management system was adopted in the project to ensure the performance verification of PVDs in the field. The quality management system implemented included controlling the quality of PVDs at every stage of the project inclusive of commencement, selection of PVD materials and ending with the installation of PVDs. The stages of the adopted quality control process in PVD works in this project were as follows: (i) selection of material against specification; (ii) planning and deployment of suitable type of vertical drain rig, including special type of rigs required for difficult ground conditions; (iii) selection and quality control of accessories such as mandrels and anchors; (iv) quality control of PVD material; (v) quality control of PVD installation. This process of quality management for all these stages of PVD implementation in Changi East land reclamation project is described in this section. For the quality management of PVD materials, three types of laboratory testing organizations were adopted: an on-site laboratory, a research laboratory and a third party laboratory. The on-site laboratory was used to carry out two types of test such as discharge capacity and tensile strength tests since these are essential parameters for vertical drain performance and the strength required to withstand penetration force. Discharge capacity is a critical parameter for vertical drain performance and the tests to determine this can be undertaken in the laboratory or carried out by in-situ testing or instrumentation in the field.

C4.2 Selection of Material by Following the Specification

The popularity of PVD in ground improvement projects in the last two decades has given rise to a phenomenal increase in the variety of PVDs produced by various geosynthetic manufacturers. PVDs now come in various shapes and sizes. The PVD core and filter design, including the type of material used varies from one type and one manufacturer to another. The performance of PVDs depends primarily on the site Conditions and surrounding soil properties. The quality of PVDs deteriorates with time and it is thus necessary to have a reliable specification, which accounts for all these factors as well as any specific project design requirements. The proper-ties of PVD are important and might change after installation and during the consolidation process. As such, specification must be adopted for specific conditions, which will reflect the actual in-situ condition.

Due to the differences in site conditions, various combinations of specifications were specified. Due to very deep installation of PVD greater than 40 m, several requirements of PVD were specified. These requirements included higher tensile strength in combination with smaller apparent opening size and higher discharge capacity under confining pressures greater than 350 kPa.

C4.3 Planning and Deployment of Vertical Drain Rigs

The selection of PVD installation rigs is essential for implementation in PVD projects and is based predominantly on the following factors:

(i) Bearing capacity of PVD installation platform (ii) Depth of installation (iii) Type of soil (iv) Production capacity of rig.

The size and weight of the rig should be suitably matched with the bearing capacity of the PVD installation platform. In this project, the installation platform was a hydraulically fill granular sand with a minimum thickness of 6 m. The mobilization of an overweight rig would have led to the instability of the equipment. On the other hand, a low capacity rig would not have provided sufficient penetration capacity or reaction against penetration resistance. The depth of installation and soil types are important factors. Determining the most suitable height of rigs depends on the depth of installation.

The capacity and type of rig also needs to be selected based on the type of in-situ soil. The productivity of the rig is similarly important. For this project, installation rigs with a minimum capacity of 8000 m/day were required to cope with the project schedule. Table C4.2 shows types and details of rigs mobilized in the project. Table C4.3 shows the specification for high power installation rigs mobilized for difficult conditions.

C4.3.1 Managing Difficulties in Installation

During the course of the installation of PVDs, installation difficulties are sometimes encountered. Difficulties are often encountered in hard ground and very soft ground conditions. The types of difficulty and suitable trouble shooting techniques adopted in the Changi East Land Reclamation project are described in this section. Various types of PVD installation equipment were used for penetrating difficult ground. Table C4.4 shows the various specialized rigs mobilized for trouble shooting at site.

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Table C4.1 - Specification for Prefabricated Band-Shaped Plastic Vertical Drain (Requirement Vs. Supplied Materials)

Property	Material	Core	Unit	Specified requirements	CX 1000 Continues plastic drain core wrapped in non-woven geotextile material	Colbond Holland Polyester	MD 7007 Korea Corrugated	MD 7007 Malaysia Corrugated	MD 7007 FD 767 Corrugated	Flexi Corrugated
		Filter				Filament Polyester	38 groves PP	40 groves PP	37 groves PP	39 groves PP
						Non-woven	non-woven	non-woven	non-woven	non-woven
Dimension of drain	Width	mm	100 ± 2			100	103	103		100 ± 2
	Thickness	mm	3 to 4			5	3.1	3.4		3.5 ± 0.2
Darcy Permeability		m/s	$> 5 \times 10^{-4}$			15×10^{-4}	1×10^{-4}	1×10^{-4}	1.02×10^{-4}	1×10^{-4}
Discharge capacity of drain		m ³ /s	$> 25 \times 10^{-6}$	At 350 kN/m soil pressure after 4 weeks		90	95	100	69*	52
Discharge capacity of drain under deformation		m ³ /s	$> 10 \times 10^{-6}$	At 25 % relative compression		80	71	77	45*	45
Soil retention capacity		microns	AOS 095 < 75			< 75	73	73	40*	75
Tensile strength of entire drain	Dry	kg/10 cm width	> 100	At elongation minimum 2% maximum 10%		210	220	250	147*	294*
	Wet	kg/10 cm width	> 100	At elongation minimum 2% maximum 10%		232*	212*	275*	167*	310*
Tensile strength of filter	Dry	kg/cm	> 3	Test at 1% strain/min after saturated in H ₂ O at 10 °C for 48 h		11	40.8	40.8	89*	80*
	Wet	kg/cm	> 3			11*	49.7*	68.4	97*	95*
Elongation of entire drain	Dry	%	< 10	At 100 kg/10 cm width		6*	3% at 50 kg/cm	2*	4*	
	Wet	%	< 10	At 3 kg/cm		2*	4*	4*	1.4*	2*

*Tested at third party laboratory.

Table C4.2 - Types of Vertical Drain Installation Rigs used at the Changi East Land Reclamation Project

Description	Type of base machine	Weight of base machine: (ton)	Penetration power (ton)	Height of rig (m)	Maximum penetration depth (m)	Mechanism of penetration	Maximum production/day (m/14 h)
Cofra	O & Kexcavator RH30, RH40	70–110	20–30	36–55.5	50.5	Hydraulic motor, Multi pulley system	33 400
Econ	O & KExcavator RH30, RH40 (01)Hitachi Excavator EX1100	70–120	20–30	36–56.1	51.5	Hydraulic motor, Multi pulley system	27 500
Yuyang	Samsung CX800 crane, Daewoosolar 450IIIExcavator, Zeppelin crane, P & H Crane, Komaso excavator IHI crane	45–100	25–30	43–55.8	53	Hydraulic motor, Multi pulley system, Driven chain and cable system	15 300
Chosuk	Daewoo solar 450 Excavator	45	25	56	51	Hydraulic cylinder, multi pulley system	17 900
Daeyang	Daewoosolar 450III Excavator	33–55	20–34	42–56	52	Hydraulic motor Push in roller and clamp system	15 200
B + B	Excavator	–	–	31–47	29–45	Hydraulic sprocket and chain	19 200
B + B	Excavator	–	–	43–50	41–48	Vibro push-in	8 600

Table C4.3 - Specification of High Power Installation Rig

Displacement	min	3140	cc/revolution
Operating pressure	max	350	bar
Operating speed	max	280	rpm
Flow	max	700	l/min
Torque		35,000	N/m
Power		275	kW
Weight		200	kg

C4.3.2 Top Hard Crust at Seabed

Sometimes recent alluvial deposits such as sand were found overlying a marine clay deposit. The density of such a deposit varies and is dependent upon the depositional environment at the old estuary. In such cases, difficulty in penetrating through dense sand deposit just below the sand fill was encountered. This condition was overcome by loosening the dense granular material using a penetration rig with a high capacity water jetting system. There were also cases where very stiff to hard clay was encountered at the seabed due to desiccation of the upper part of the marine deposit during the previous fluctuations of the sea level. This situation was overcome by using pre-punching equipment or auguring equipment to penetrate the hard crust encountered at shallow depths.

C4.3.3 Intermediate Hard Crust at Intermediate Depth

During the geological history of deposition, some nonconformity or change in type of deposition could have occurred due to a change in the environment. For example, the desiccation of a young deposit due to the receding sea level before the next layer of deposition of younger marine deposits at Changi. Another example is the deposition of an alluvial deposit during the paused period of marine formation deposition. In such cases, an intermediate hard layer of clay or dense layer of sand might be encountered. The degree of hardness of the desiccated clay and the extent of desiccation are dependent upon the duration and thickness of the cohesive deposits exposed to the atmosphere. The problem of penetration in an intermediate hard layer was solved by using the same type of equipment described earlier or alternatively by utilizing a vertical drain rig with a vibratory system.

C4.3.4 Hard or Dense Layer at Deep Seated Formation

On rare occasions, a thin layer of hard clay or dense sand was encountered at a deep-seated depth above soft soil deposits of significant thicknesses, which would thus require PVD treatment. In such situations, using the equipment described earlier would have been impossible and would have consumed a lot of time and cost. In this project, high-powered low speed PVD installation rigs were fabricated to be punched through this deep seated hard layer and to treat the underlying soft marine clay. The equipment used was able to penetrate a hard layer, having a Standard Penetration Test blow count of 30 at a depth of 30– 40 m from the installation platform level. The specification of this special equipment is shown in Table C4.3.

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Figure C4.1 - Various types of PVD installation equipment for penetrating difficult ground

Table C4.4 - Specialized Rigs Mobilized for Trouble Shooting

Type of rigs	Purpose
Vibratory rig	To penetrate stiff layer at depth
Rig with water jetting	To penetrate through dense granular soil at intermediate depth
Prepunching rig	To punch through dense or dessicated stiff layer at seabed.
Auger rig	To auger through dense and stiff layer at shallow depth
Rig with water balancing system	To prevent soil ingress into the mandrel during installation in very soft soil
High power slow speed Rig	To penetrate through dense layer at deeper depth

C4.3.5 Installation Through Ultra-Soft Clay

Another major problem encountered in installation of PVDs was the necessity for installation through ultra-soft clay at various locations. Problems encountered in this situation were extrusion of mud along the annulus of the penetration hole, which could contaminate the drainage layer or the intrusion of mud into the mandrel leading to unsuccessful anchoring of the vertical drain. Both problems were reduced by introducing a water balancing system to counter balance the excess pore pressure encountered in the formation. In order to counter balance the excess pore pressure encountered during installation, a water head equivalent to excess pore pressure was applied within the mandrel. A smaller dimension mandrel with a smaller anchor was also suitable in this situation.

C4.4 Selection and Quality Control of Mandrels and Anchors

There are various types of mandrels: rectangular, square, rhombic and circular. However, circular and square mandrels are now rarely used. In order to minimize the disturbance of soil to be improved, smaller size mandrels are normally preferred. The rhombic type mandrel has reduced dimensions, leading to lesser disturbance. This was suitable for shallow depth penetration in soft to very soft soil because smaller dimension mandrels had inadequate penetrability in deep depth penetration, leading to non-verticality during installation. Apart from the selection of mandrel, regular checking and maintenance of mandrels were also required. Due to the installation through the highly abrasive granular fill, mandrels were frequently worn out. A tilted tip of mandrel leads to deviation in verticality. As such, the condition and dimension of the mandrel were regularly checked. A maintenance schedule for the replacement of the tip of the mandrel as well as the entire length of mandrel based on the total penetration length of PVD was necessary.

In addition to the mandrel, another important accessory is the anchor. Anchors should be strong enough to anchor the drain and to prevent the ingress of soil into the mandrel. The two main types of anchors used in this project were steel bars and flexible metal plates. There are pros and cons in using steel bars and the flexible metal plates. The steel bar minimizes disturbance due to its smaller dimension and is suitable for smaller mandrels with tapered shoe in which a steel bar can be fitted. If the steel bar is used with the larger mandrels, a water balancing system will be required. Although the flexible metal plate creates more disturbances, it is suited for large dimension mandrels and may not require a water balancing system. The formation of smear zone varies depending upon the size and shape of selected mandrels and anchors. The brief discussion on the smear effect is described in subsequent paragraph. Figure C4.5 shows the types of mandrels and anchors used in this project.

C4.4.1 Smear effect

Several researchers have reported that the field performance of PVD is far slower than theoretical prediction especially due to the smear column which forms around the vertical drain due to penetration disturbance. Indraratna and Redana (1997, 1998), Bo et al. (1997) and Hansbo (1981) reported that the consolidation rate in the field is much slower than that calculated due to the smear effect. Bo (2004) has reported that excess pore pressure during penetration was detected even at a distance of 1.3 m from the installation point of PVDs (Figure C4.6), but was not detected at a distance of 2.85 m. This implies that the disturbance covered an area of radius greater than 1.3 m but not more than 2.85 m from the mandrel. The extent of the smear zone and ratio of permeability reduction are largely dependent upon the type and sensitivity of the clay, size and shape of the mandrel, size and shape of the anchor and equipment used for the installation works.

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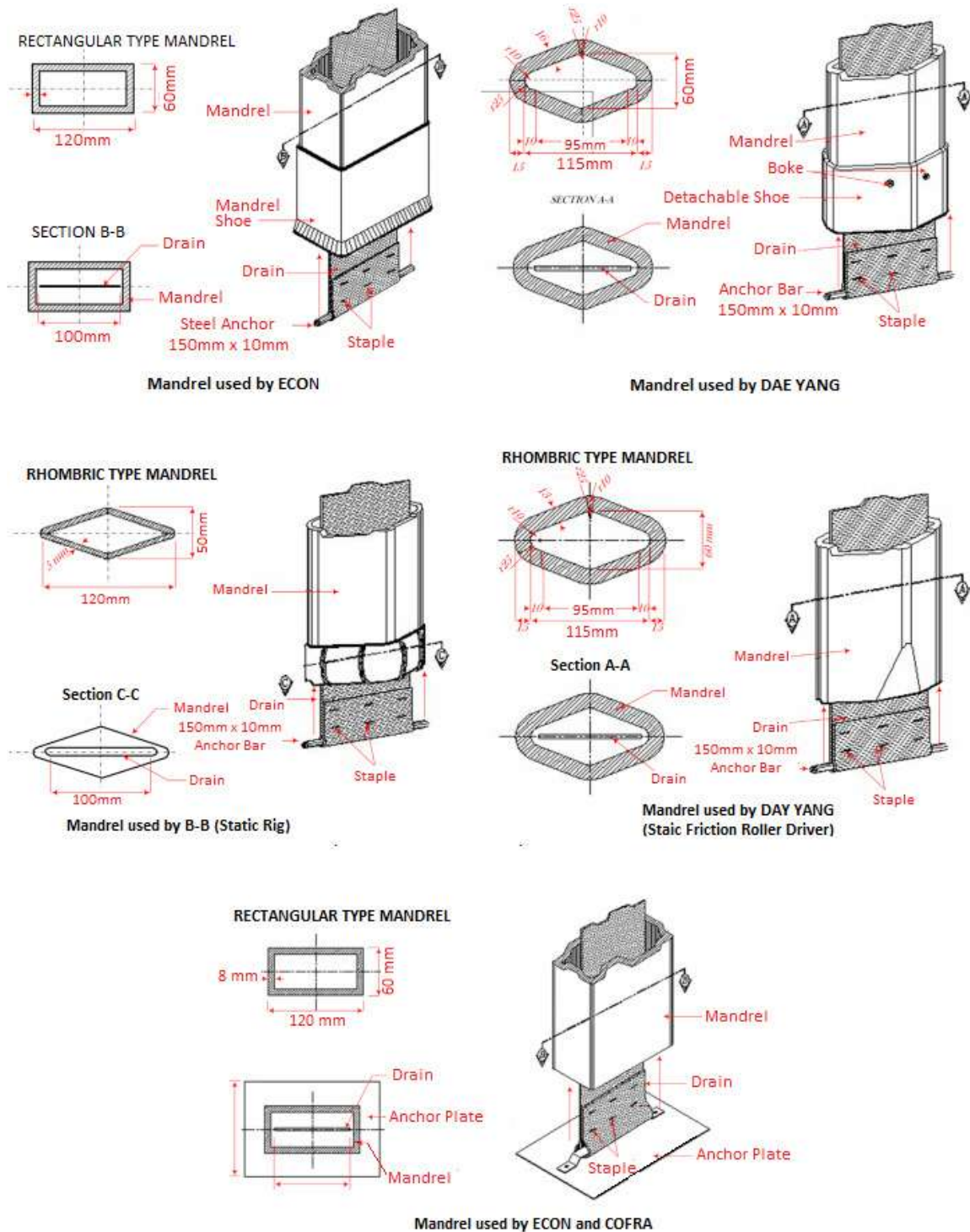


Figure C4.2 - Mandrels and Anchors of Various Shapes used in the Reclamation at Changi East project – Phase 1B

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Depending upon the type of clay and anticipated disturbance, a suitable and conservative smear effect parameter should be applied in the design. Since there were several unknown factors in the in-situ condition such as coefficient of consolidation due to horizontal flow (Ch), smear zone ratio and permeability reduction ratio. The permeability reduction ratio is defined as the ratio between reduced permeability due to smearing of soil during mandrel installation and original undisturbed permeability. Several pilot test areas with different PVD designs were constructed in the project site prior to the selection and installation of PVDs to determine the optimum PVD design consideration.

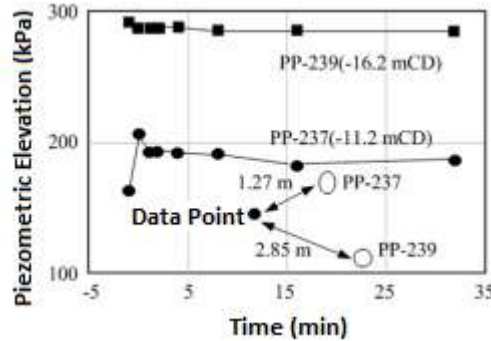


Figure C4.3 – Pore Pressure Measurements during Mandrel Penetration

C4.5 Quality control on Prefabricated Vertical Drain Material

Generally, PVDs were delivered to the site in rolls of 200 – 300 m in length. A single delivery to the site could consist of up to one million meter lengths of PVDs delivered in ten containers. Therefore, the specification required full scale testing of PVD materials for every one-million-meter length of PVD delivered to the site by a third party accredited laboratory-1. An in-situ specialized laboratory-2 was also set up at the project site to verify the discharge capacity of every 100,000 m length of PVD to be used for installation as well as for carrying out other tests such as Apparent Opening Size (AOS), permeability and specialist tests under various conditions.

In addition, another on-site laboratory-3 was set up by the PVD installation contractor to carry out routine discharge capacity and tensile strength tests on every 20,000 linear meter lengths of vertical drains. Consistency in the quality of PVD was monitored and controlled based on the results from these three laboratories.

C4.6 Control parameters and test results

C4.6.1 Dimensions of PVD

The PVD supplied by various manufacturers had the same dimensions: the standard dimensions were 100 mm width and 4 mm thick, and some larger dimension of PVDs were produced for special purposes. The specifications of the project required the PVD dimensions to be 10072 mm width and 3 to 4 mm thickness and most PVDs complied with the specified dimensions although some were found to be slightly less than the specified margin.

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The effect of PVD dimensions on the consolidation time is shown in Fig. 4. It is evident from Fig. 4 that variation of time required for 90% consolidation with specified spacing due to maximum dimension variations of 6 mm in width and 2 mm in thickness in certain types of soil is found to be only 12 days for approximately one year consolidation duration. The maximum percentage difference for one (1) year duration design was only 3%. Since the thickness controls the consolidation time, the thickness and width of the PVD under pressure were frequently checked. At the site laboratory, the width was measured by a vernier scale. Changes in the thickness under pressures were measured at the site laboratory using direct simple shear equipment, which could measure accurately vertical displacements under different loads. Fig. 5 shows variation in the thickness of PVDs under various normal pressures. It is evident in the figure that the variation of thickness of Colbond type PVD was greater than that of the Mebra type PVD; however, both types met the specifications requirements.

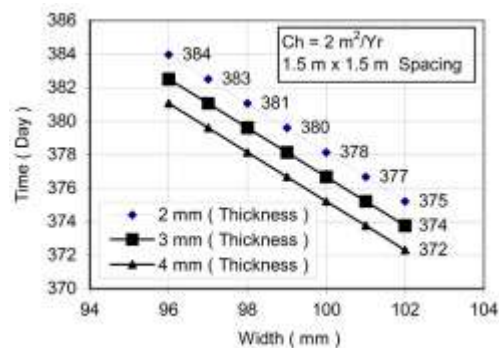


Figure C4.4 – Time Required to Complete 90% degree of Consolidation with various Dimensions of PVDs.

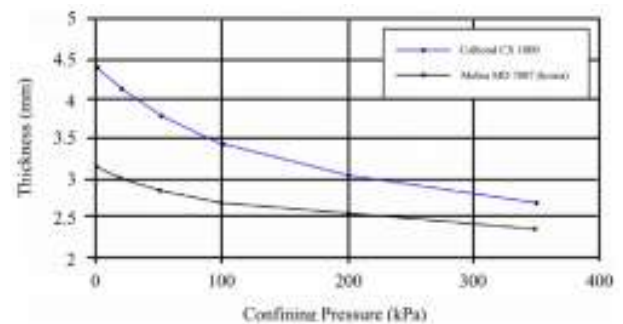


Figure C4.5 – Variation of Thicknesses of PVDs under various Normal Pressures.

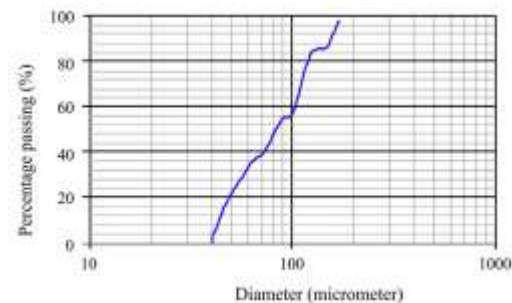


Figure C4.6 – Grain Size Distribution of Marine Clay at Singapore

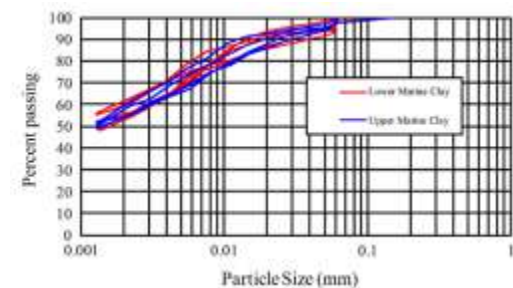


Figure C4.7 – Cumulative Particle Size Distribution Chart for standard Glass Beads.

C4.6.2 Apparent Opening Size (AOS)

An Apparent Opening Size (AOS) requirement was specified for the PVD filter material. Since the function of the filter is to prevent fine-grained soil from entering into the core and create a clogging potential (Basu and Madhav, 2000) and at the same time to provide sufficient permeability, an AOS of O_{95} (opening size for 95% retained by weight) less than or equal to $75\ \mu\text{m}$ was specified. For a woven or non-woven geotextile, if AOS is smaller than D_{85} (grain diameter for 85% passing by weight) of the surrounding soil, piping will not occur. The D_{85} of natural soft clay at this site was generally greater than $75\ \mu\text{m}$. Therefore, specifying O_{95} of $75\ \mu\text{m}$ was sufficient to retain the surrounding soil. On the other hand, the AOS of O_{95} or O_{15} (opening size for 15% retained by weight) should also be large enough to prevent clogging. The following criteria were implemented in the project to prevent clogging as recommended by Bo and Choa (2004)

$$\text{AOS} = O_{95} \geq 3D_{15} \quad (1)$$

or

$$O_{15} = 2-3D_{15} \quad (2)$$

The typical grain size distribution of Singapore Marine Clay at Changi is shown in Figure C4.8. The AOS of O_{95} of $75\ \mu\text{m}$ is much greater than D_{15} of natural clay at Changi. Therefore, the filter would not become clogged in Singapore Marine Clay. The mineralogy and geotechnical properties of Singapore

Marine Clay at Changi have been reported recently by Bo et al. (2015). The AOS tests were carried out using standard glass beads with grain size of diameter, ranging from 40 to $170\ \mu\text{m}$. The AOS is obtained from the grain size distribution curve provided by a glass beads manufacturer, as shown in Figure C4.9. The test method complies with ASTM D4751 (1987).

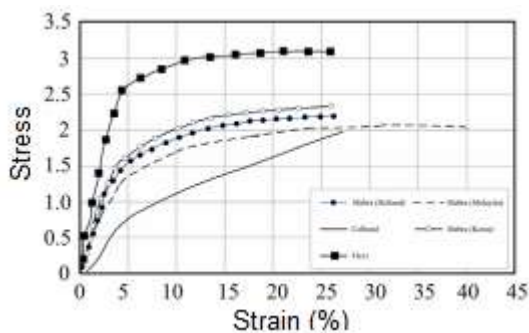


Figure C4.8 – Tensile Strength Test Results On various PVDs under wet condition.

Table C4.5 – Permeability of various PVDs

Types of drain under laminar	Permeability flow at 20° (1×10^{-4} m/s)
Colbond CX 1000	15
Mebra (Holland)	1.6
Mebra (Korea)	1.58
Mebra (Malaysia)	1.02
Flexi	4.25

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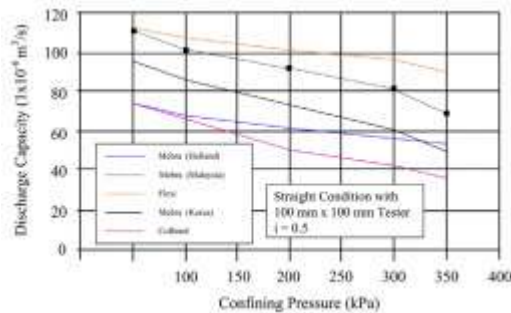


Figure C4.9 – Discharge capacity of PVDs Under straight condition tested with 100mm x 100mm tester.

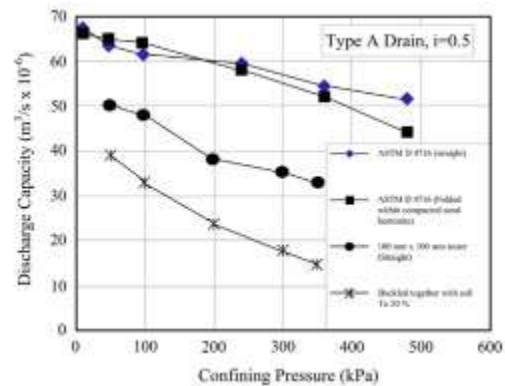


Figure C4.10 – Variation of discharge capacity of PVDs measured by different Types of apparatus.

C4.6.3 Tensile Strength

The PVD should be able to withstand the tensile stresses caused by PVD installation works. Elongation of the PVD may also occur during installation. Therefore, the PVD should have the required tensile strength, at an allowable elongation, and be capable of more or less maintaining the dimensions of the drain without major deformation.

For the Changi East Land Reclamation project, the tensile strength of PVDs was specified as 100 N/cm or 1 kN/10 cm at 10% elongation for both the dry and wet conditions. However, the actual elongation test carried out with vertical drain installation rig showed that elongation of Mobra MD 7007 is as low as 1%. It indicates that the stresses occurring due to the friction between the roll of vertical drain during penetration was insignificant. Typical tensile strength test results of Mobra MD 7007, Colbond CX 1000 and Flexi FD 767 under wet conditions are shown in Figure C4.8. It is evident from Figure C4.8 that the strain in PVD at the specified tensile strength was normally lower than the specified strain of 10% for most PVDs. The tensile strength tests at the site laboratory were carried out with a modified triaxial compression machine where the PVD was gripped across the whole section at the two ends. The size of the jaw face was 140 mm in width and 50 mm in height.

Vertical displacement was measured during extension with linear vertical displacement transducer and the stress incurred was measured using an extension proving ring. The tensile strength tests were carried out under a strain rate of 7% strain/ min on PVD samples with a gauge length of 200 mm. The method of testing complied with ASTM D459 (1986). The effect of test strain rate on strength of geotextile was found to be insignificant in the tests.

C4.6.4 Permeability

To meet the permeability requirement for the PVDs, Holtz et al. (1991) suggested that the permeability of the geotextile should be at least 10 times more than that of the surrounding soil. For the Changi East Land Reclamation project, the specified permeability of filter is greater than 5×10^{-6} m/s while the permeability of the soil is in an order of 10^{-9} m/s (Bo et al., 1998). Permeability tests were carried out with a simple constant head permeability apparatus. The apparatus used is in accordance with ASTM D4491-85. The device consists of an upper and lower unit, which were fastened together. The sample was positioned between the two units. There are manometers connected to the upper and lower units for water supply and head measurement. Permeability tests were carried out under various head differences. The permeability measured for various PVDs, by Darcy's method, is shown in Table C4.5.

C4.6.5 Discharge Capacity

The discharge capacity of drain is affected by factors such as lateral stress, buckling and siltation. As such, discharge capacity of drains was measured under straight and buckled conditions. The specification requires the discharge capacity to be 25×10^{-6} m³/s under straight condition and 10×10^{-6} m³/s under buckled condition with an axial strain of 25% for the Changi East Land Reclamation project. Discharge capacity tests were performed under various lateral stresses and under various hydraulic gradients for straight and buckled conditions. Some of the results of the discharge capacity tests are shown in Figure C4.9 and C4.10. The discharge capacity under straight condition was carried out with a 100 mm x 100 mm tester, in which the sample was surrounded by soil and tested under various pressures, and various hydraulic gradients. The tester was connected to the inflow and outflow water pipes, which provide the discharge water and measured the outflow discharge rate of water. Manometers were also connected to the tester to measure hydraulic gradient. The discharge capacity test was carried out in a cylinder, under buckled condition, in which the PVD was installed and compressed with surrounding soil. Table C4.6 shows the discharge capacity of type D PVD under various deformation configurations. It is evident from the results that all the tested types of PVDs met the specified requirement both in straight and buckled conditions. PVDs with corrugated core were found to have comparatively high discharge capacities.

Table C4.6 – Discharge Capacity of Type D PVD under various configurations of deformation (after Bergado et al., 1996)

Type of deformation	Discharge capacity (m ³ /s $\times 10^{-6}$)
Non-deform	62
15% free bend	36
20% free bend	32
90° twisted	31
180° twisted	30
20% sharp folding	16

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C4.7 Field Measurements

The field performance of ground improvement work using PVD was monitored and verified by the use of geotechnical instrumentation such as settlement plates, deep settlement gauges installed at various levels and piezometers installed at various elevations within the compressible soil mass both at pilot areas and actual ground improvement areas. Both an acceleration in the settlement and the dissipation of pore pressures were clearly shown in all cases. An example of field performance measurements is shown in Fig. 11. Details of field performance measurements using geotechnical instrumentation were discussed in Bo (2004) and details of assessment of degree of consolidation and performance evaluation were discussed in details in Bo (2004) and Bo (2004).

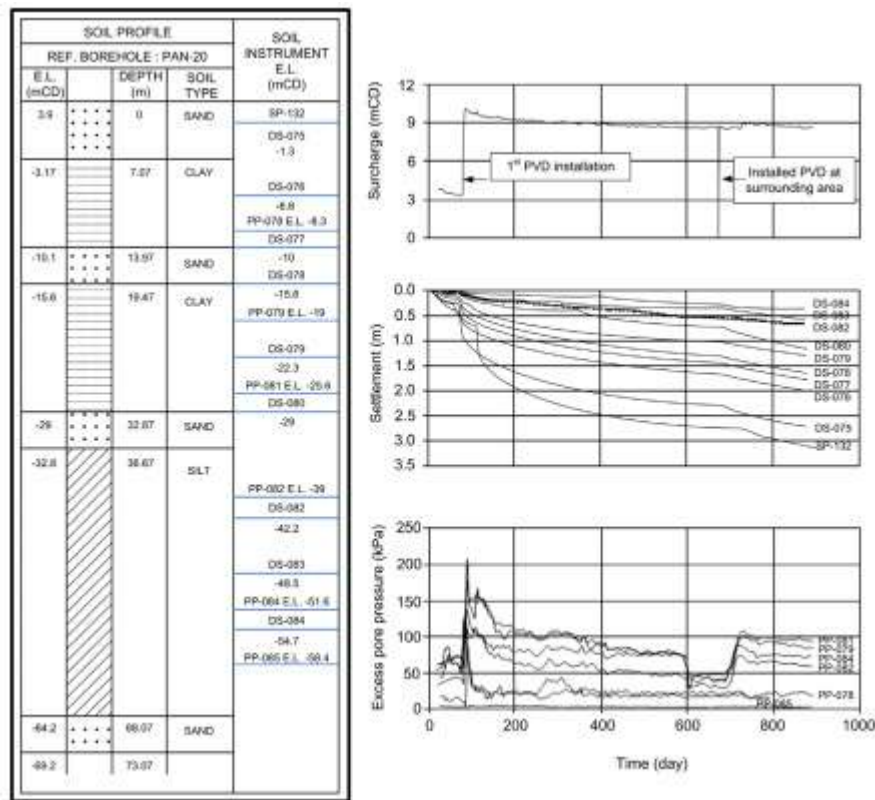


Figure C4.11 – Settlement and pore pressure measurement at Area A (North) Pilot Area using 1.5 m x 1.5 m drain

C4.8 Conclusions

This technical report presents the quality management of PVD materials in the mega Changi East land reclamation project in the Republic of Singapore. The following key conclusions are useful and can be applied to other similar mega projects. The quality management of PVD materials involves several detailed processes.

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Firstly, comprehensive specifications must be adopted for the selection of material. The adopted specification should suit the ground conditions and geotechnical characteristics of the in-situ compressible soils. Before the commencement of PVD works, the correct planning and deployment of suitable types of PVD installation rigs and accessories such as mandrels and anchors are essential. The selected installation rig should be suitable for the prepared installation platform. The chosen mandrels and anchors should be suited for the in-situ soil during installation. It may also be necessary to mobilize special type of rigs if difficulties are encountered in the installation, such as hard or dense layers at any depth or very soft soil conditions. The construction of pilot trial embankments with several types of PVD spacing and configurations are required in order to verify the design spacing with the selected material, installation equipment and accessories. Establishing a quality control system, consisting of on-site laboratory and third party laboratory, is absolutely necessary to control the quality and consistency of the supplied PVD materials. Having an in-situ laboratory specialist on site should also be considered in order to assess the measured parameters and variation of parameters with the various test and equipment conditions. Based on the quality control tests carried out on the PVD materials used against specified parameters shown in Table 1, the supplied PVDs generally met the required specification and performed well in the field.

Acknowledgments

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C5 Slope Stability Analysis (SSA)

Abstract: This section describes the overview of Proctor Compaction Test. This test is performed to determine the relationship between the moisture content and dry density of a soil for a specified compactive effort.

C5.1 General

Laying of roads for highways often involve cutting and filling of earth and construction of culverts, bridges and flyovers. Construction of embankments is an important aspect of highways be it in filling or for approach for bridges. Embankments may have varying heights and may be constructed on soft ground. They may serve as water retaining structures besides being a highway and may be located in earthquake prone regions.

Design of an embankment essentially requires evaluation of its stability against failure. Slip circle method based on limit equilibrium is the common method adopted for conducting stability analysis of embankments. Due to its improve accuracy simplified Bishop's method is increasingly adopted for the analysis. A computer application may be used based on Bishop's method containing improved and additional features.

The analysis includes (i) earthquake force (ii) pore water pressure as obtained from pore pressure ratio, 'ru' or flow/water table and (iii) reinforcement force at the embankment-foundation interface in the existing program. The graphics in the program helps in presentation of the results. Procedure to evaluate material parameters is discussed next.

C5.2 Method of Analysis

In this method, overall stability of the sliding mass on an assumed circular surface is evaluated using limiting equilibrium method. Factor of safety with respect to incipient failure on the rupture/failure surface is determined using moment equilibrium conditions. Method of slices is adopted and inter-slice are considered.

Factor of safety F is defined as 'F' = Available shear strength of soil / Required shear strength to maintain equilibrium

The mobilized shear strength, 'S' is defined as

$$S = \frac{1}{F} [c' + (\sigma_n - \mu) \tan \phi']$$

c'	=	effective cohesion intercept
ϕ'	=	effective angle of shearing resistance
σ_n	=	total normal stress
μ	=	pore pressure

Considering the equilibrium of potential sliding mass of unit thickness bound by a circular arc of radius R with centre O. Figure C5.1, factor of safety by simplified Bishop's method (1955) is derived with the following notations.

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E_n, E_{n+1}	=	The horizontal forces on the sections n and n+1
X_n, X_{n+1}	=	The total vertical shear forces
W	=	The total weight of the slice of soil
P	=	The total normal force acting on its base
h	=	The height of the slice
b	=	The breadth of the element
l	=	The length BC
α	=	The angle between BC and the horizontal
Total normal stress $\sigma_n = P/l$		

$$F = \frac{1}{\sum W \sin \alpha} \sum [\{ c' b + \tan \phi' ((W - bu) + (X_n - X_{n-1})) \}] \frac{1}{\cos \alpha \frac{\tan \phi' \sin \alpha}{F}}$$

with earthquake forces in the vertical and horizontal forces $\alpha_v W$ and $\alpha_h W$ and reinforcement force T located as shown in Figure C5.2.

$$F = \frac{\sum c' b + \tan \phi' (W(1 + \alpha_v) - ub + (X_n - X_{n-1})) \frac{1}{m_n} + \frac{TV_1}{R}}{\sum [W(1 + \alpha_v) \sin \alpha + W \alpha_h \frac{Y}{R}]}$$

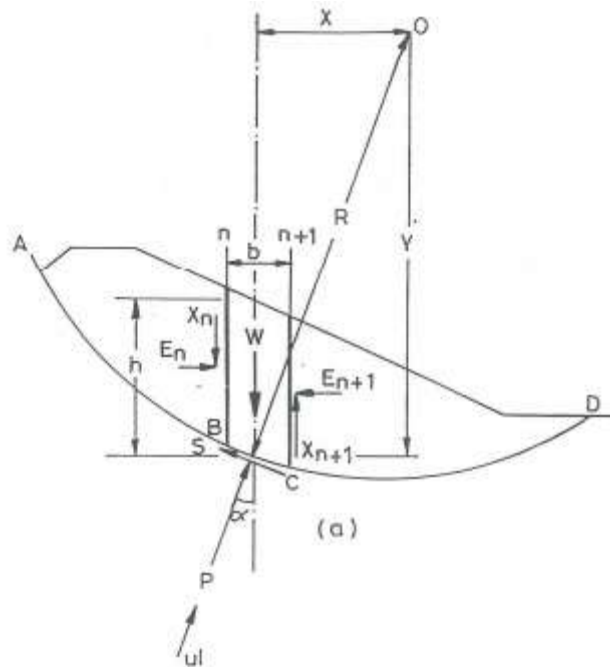


Figure C5.1 - Forces in the Slices (Bishop)

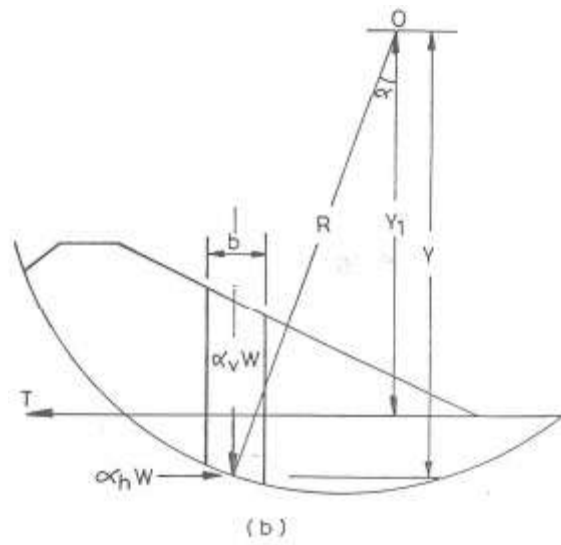


Figure C5.2 – Earthquake and Reinforcement Forces

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where,

$$m_a = \cos \alpha + \frac{\tan \phi \sin \alpha}{F}$$

In simplified Bishop's method, $\sum (X_n - X_{n-1}) = 0$, then

$$F = \frac{\sum c' b + \tan \phi' \{W(1 + \alpha_v) - ub\} \frac{1}{m_a} + \frac{TY_1}{R}}{\sum [W(1 + \alpha_v) \sin \alpha + W \alpha_h \frac{Y}{R}]}$$

Defining pore pressure ratio, $r_u = \frac{u}{rh}$

$$F = \frac{\sum c' b + \tan \phi' \{W(1 + \alpha_v) - r_u\} \frac{1}{m_a} + \frac{TY_1}{R}}{\sum [W(1 + \alpha_v) \sin \alpha + W \alpha_h \frac{Y}{R}]}$$

C5.3 Soil Parameters

The stability analysis of highway embankments is conducted using Simplified Bishop's method. In the analysis effective stresses are used. The material parameters that are used are cohesion intercept c' and angle of shearing resistance ϕ' in addition to total unit weight of soil, γ_t .

The embankment and foundation material may consist of coarse grained soil such as sand and silty sand or fine grained soil such as clay and silty clay.

In the case of coarse grained soil, direct shear test or consolidated drained test or consolidated drained triaxial test may be conducted. For fine grained soils either consolidated undrained tri-axial test with pore water pressure measurement or consolidated drained triaxial test with pore water pressure measurement or consolidated drained triaxial test with volume change measurement may be performed.

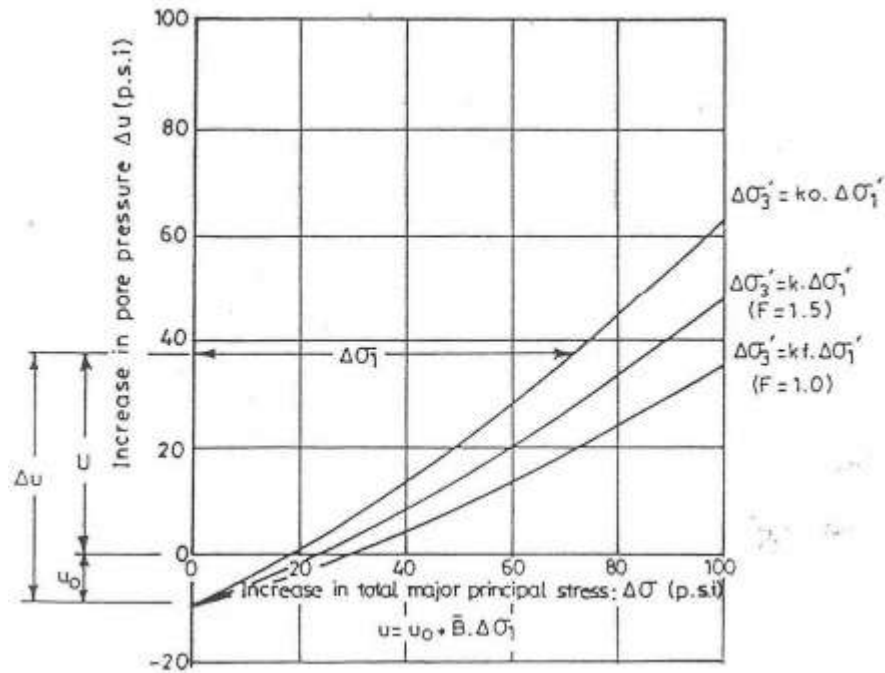


Figure C5.3 - Diagrammatic Variation of Pore Pressure Parameter 'B'
with Principal Stress Ratio and Major Principal Stress
[Bishop & Morgenstern 1960]

The samples may be prepared to a density that is anticipated in the field. The normal stress for direct shear test and the confining pressure for triaxial tests to be used may be in the average range of stress in the embankment foundation system. The details of the tests are found in various books and codes (for example Bishop and Henkel, 1962, Lambe, 1951, IS code IS:2720 Part 12 (1981)).

The unit weight of the soil may be determined using standard procedure outlined in various BS Codes.

C5.4 Pore Pressure Ratio, r_u

The procedure for evaluation of r_u is presented in detail by Bishop and Morgenstern (1960). For earth fill placed dry of optimum, the value of r_u is taken as zero. In the case of earthfill placed wet of optimum, the value of r_u is assumed to be equal to the pore pressure parameter \bar{B} which is equal to $\Delta u / \Delta \sigma$. The value of \bar{B} is essentially a function of stress ratio as shown in Fig. 2. The relationship shown in Fig. 2 is obtained by conducting a consolidated undrained triaxial test with pore pressure measurement.

C5.5 Reinforcement Force

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Reinforcement is used at the interface between embankment and foundation to steepen the slope for the given height of embankment or to increase the height of embankment for the given slope. Reinforcement in the form of geosynthetic materials is often used when the embankment is constructed on soft foundation. The embankment in such cases is designed taking into consideration various modes of failure viz, sliding failure, squeezing failure, bearing capacity failure and rotational failure. The reinforcement force is determined for reinforcement stiffness with an allowable axial strain. The details on these are found in various books such as Soil Reinforcement with Geotextiles by Jewell (1996).

C5.5 Seismic Coefficient

Based on occurrence of earthquake, India has been divided into five zones as per the BS code on criteria for Earthquake Resistant Design of Structures. The horizontal earthquake coefficients to be used for various zones are as follows:

Zone I	-	0.01
Zone II	-	0.02
Zone III	-	0.04
Zone IV	-	0.05
Zone V	-	0.08

Depending on the location of the embankment, suitable earthquake coefficient as above may be chosen for the analysis.

C5.6 Terms Used in the Program

TITLE	:	Title of the problem
NINT	:	No. of top external soil lines defining the outer embankment geometry
NLINE	:	No. of total lines (including external and internal soil lines)
NSL1	:	No. of Slices
NRT	:	Indicator for Reinforcement (0=No, 1=Yes)
NPROB	:	Indicator for pore water pressure (1=Using r_u , 2=Using Water table line)
NMAT	:	No. of materials in the problem
SUR	:	Surcharge over the embankment in mass/area ² (assumed within XTOP & XTOP1)
XTOP	:	X- Coordinate for the highest point towards left edge of the embankment
XTOP1	:	X-Coordinate for the highest point towards right edge of the embankment
X1(I)	:	Starting X-Coordinate for the i-th line
Y1(I)	:	Starting Y-Coordinate for the i-th line
X2(I)	:	Ending X-Coordinate for the i-th line
Y2(I)	:	Ending Y-Coordinate for the i-th line
WT(I)	:	Density of the soil below the i-th line
CHS(I)	:	Cohesion
FRC(I)	:	ϕ

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UPP(I)	:	r_u for the soil below the i-th line if NPROB = 1
		Y_w for the soil below the i-th line if NPROB = 2
H	:	X coordinate for starting center of circle
G	:	Y coordinate for starting center of circle
HS	:	Incremental shift for X coordinate of center of circle
RHS	:	Incremental shift for Y coordinate of center of circle
ALPHAN	:	Horizontal seismic coefficient
ALPHAV	:	Vertical seismic coefficient
TFR	:	Reinforcement force
YT	:	Y coordinate of reinforcement level
CTCHI	:	Y coordinate for the starting elevation to which all circles are tangent
DCTCII	:	Incremental shift for CTCH for finding absolute minimum factor of safety

C5.7 Description of Computer Program

The default folder path for program application SSA is 'C:\program files\SSA\....', in which SSA setup installs the application.

On running the Program needs the input and output file names. If the input data is saved in a file, give the file name, otherwise give the input through user interactive forms. After all the input data is read, the program searches for least value of factor of safety, for each value of CTCH begins. Initial value of Factor of safety is assumed as one (1.0) and RHS is calculated and compared with LHS. Newton Raphson method is followed to achieve an early convergence. This procedure starts for the center of circle as specified in the input file. This center is shifted in both the directions by incremental distances specified (first in X and then in the Y direction). The calculation stops if the tendency of an increase in factor of safety is observed all around a point, and this point corresponds to the center for the critical circle.

The above procedure for finding critical circle for a given CTCH (the level to which all circles are tangent) is repeated changing the values of CTCH. The least factor of safety is thus located, and the corresponding circle of failure is identified.

While using the program, a consistent system of units is to be followed, i.e. all the input values should have the same unit for Mass, Length and Time.

It has been noticed that sometimes while executing the program, the computer hangs up. To avoid this, rerun the program with changed values of HS, RHS and CTCHI.

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Program can include surcharge of the embankment and negative values of 'X' and coordinates can also be specified.

Approaches to bridges are often subjected to high flood level. Due to this, the soil in the embankment may get saturated. The pore water pressure so generated can be calculated as ' $u = \gamma_w \times h$ ', where 'h' is the average depth of the slice base from the HFL. Pore water pressure so calculated can be reduced to a single value of ' r_u ' by averaging out over the whole area of the embankment cross section. The pore water pressure above the HFL being generated due to capillary action (predominant in fine grained soils) can be ignored as these are negative pore water pressure adding to the stability of the slope.

After the execution of the program the output can be seen in the graphics form. If the results are not satisfactory, modify the input data and re-run the program. The output file consists of all the input data for verification and safety factor tables for each value of CTCH with minimum factor of safety written below it. Safety factor table consists of center of circle coordinates, radius of circle, intersection points with the outer soil lines (XC1 and XC2) and the corresponding safety factor. The output file can be edited if required before taking the printout.

C5.8 Data and Input

The input data for this program can be given in two ways

- By using 'input data file'
- By using interactive Dialog Boxes

C5.8.1 Input data file

Data for a sample problem with figure is given in the following for each input data set.

Input Format for data file is as follows, Sample Data are made in bold:

1. Title of the problem

TITLE: Title of the problem (Maximum of 80 Characters)

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SAMPLE PROBLEM

2. Control Data – I

NINT : No. of top external soil lines defining the outer embankment geometry
NLINE : No. of total lines (including external and internal soil lines) (maximum 50 lines)
NSLI : No. of slices (Maximum 200)
NRT : Indicator for reinforcement (0=No, 1=Yes)
NPROB: Indicator for pore water pressure (1=Using r_u , 2=Using water table line)
NMAT : No. of materials in the problem

5 7 100 1 2 3

3. Control Data-II

SUR : Surcharge over the embankment in mass/area²(to be within XTOP & XTOP)
XTOP : X- Coordinate for the highest point towards left edge of the embankment
XTOP1 : X-Coordinate for the highest point towards right edge of the embankment

0 100.0 108.0

4. Line and soil data

X1(I) : Starting X-Coordinate for the i-th line
Y1(I) : Starting Y-Coordinate for the i-th line
X2(I) : Starting X-Coordinate for the i-th line
Y2(I) : Starting Y-Coordinate for the i-th line
WT(I) : Density of the soil below the i-th line
CHS(I) : Cohesion
FRC(I) : Angle of internal friction (ϕ)
UPP(I) : r_u for the soil below the i-th line if NPROB = 1
 y_w for the soil below the i-th line if NPROB = 2

Note:

1. The above line and soil data has to be repeated NLINE (Total no. of lines) times.
2. For NPROB= 2 some of the lines and lines below UPP(I) = y_w is given and for the rest of the lines UPP(I) = 0.

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0		200	86.5	200	15	17	0	9.81
86.5	200	100	206	20	0	32	0	
100		206	108	206	20	0	32	0
108		206	121.5	200	20	0	32	0
121.5	200	164	200	15	17	0	9.81	
86.5	200	121.4	200	15	17	0	9.81	
0		196	164	196	15	10000	45	0

5. Coordinate Data -1

H : X coordinate for starting center of circle

G : Y coordinate for starting center of circle

HS : Incremental shift for X coordinate of center of circle

RHS : Incremental shift for Y coordinate of center of circle

ALPHAN: Horizontal seismic coefficient (+ away from embankment)

ALPHAV: Vertical seismic coefficient (+ in the direction of gravity) (Ref. Figure C5.2)

92 208 0.5 0.5 0 0

6. Reinforcement data (if NRT > 0)

TFR : Reinforcement force

YT : Y coordinate of reinforcement level

263.7 200

7. Data for tangency point and minimum factor of safety

CTCHI : Y coordinate for the starting elevation to which all circles are tangent

DCTCH: Incremental shift for CTCH for finding absolute minimum factor of safety

198 0.5

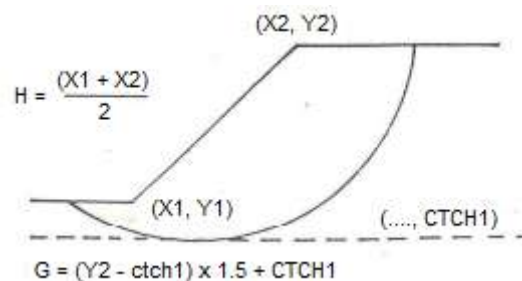


Figure C5.4 - Problem Diagram

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8. Data for Labelling (Used to write the labels in graphics)

Name of the material : Name of the material to be written in graphics

Density, Cohesion, Angle of Internal Friction and r_u : Properties of the above material

Label X coordinate : X coordinate of the label (name of the material)

Label Y coordinate : Y coordinate of the label

Note: The above labels data has to be repeated NMAT times

Soil 1

15 17 0 9.81 105 199

Soil 2

20 0 32 0 105 204

Soil 3

15 1000 45 0 105 195

Notes:

- (1) The guidelines for finding H & G are as shown in Figure C5.3
- (2) Recommended value for the incremental shift is 0.2m

C5.8.2 The data in the Input file will be, as shown below:

Sample problem

5	7	100	1	2	3		
0	100.0	108.0					
0	200	86.5	200	15	17	0	9.81
86.5	200	100	206	20	0	32	0
100	206	108	206	20	0	32	0
108	206	121.5	200	20	0	32	0
121.5	200	164	200	15	17	0	9.81
86.5	200	121.5	200	15	17	0	9.81
0	196	164	196	15	10000	45	0
92	208	0.5	0.5	0	0		
263.7	200						
198	0.5						
Soil 1							
15	17	0	9.81	105	199		

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Soil 2

20	0	32	0	105	204
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Soil 3

15	10000	45	0	105	195
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C5.8.3 Computer Application

The analysis process is an iterative method. The intention is to find out the lowest value of the 'Factor Of Safety' for the slope, say 2H : 1V, this done by changing the centre and the radius of the of the slip circle which may determine the factor of safety as per the Slope-failure or Toe-failure or Base-failure. If the value of 'Factor of Safety' is less than the desired value, then either the soil is to be modified with material having better C-Ø values or by changing the slope or by introducing geotextiles or by stabilizing the fill construction.

From HEADS Pro main screen select menu item 'Programs >> Slope Stability Analysis', and this will open the Slope Stability Program, in case it was included in HEADS Pro setup.

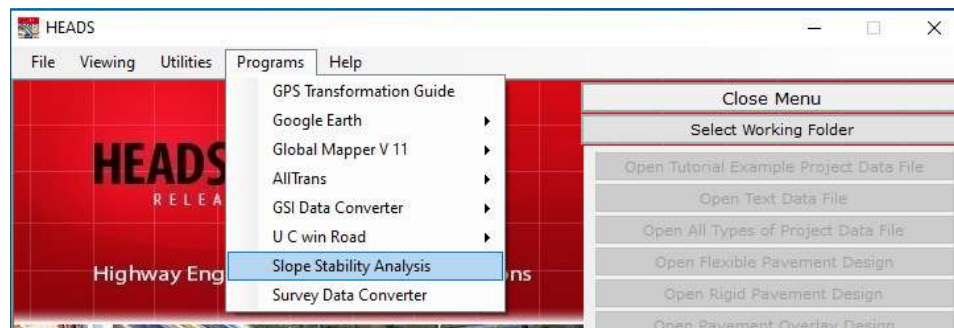


Image C5.1

Another popular program is 'SSA' is discussed here, which users need to procure and install separately as a third party program. If SSA is installed in the computer then it may be called from HEADS Pro by selecting the same menu item as shown above, in absence of its own program.

In this session we shall see the procedure to process Slope Stability Analysis, by using program S-S-A for Slope Stability Analysis. The main screen of Program SSA is opened,

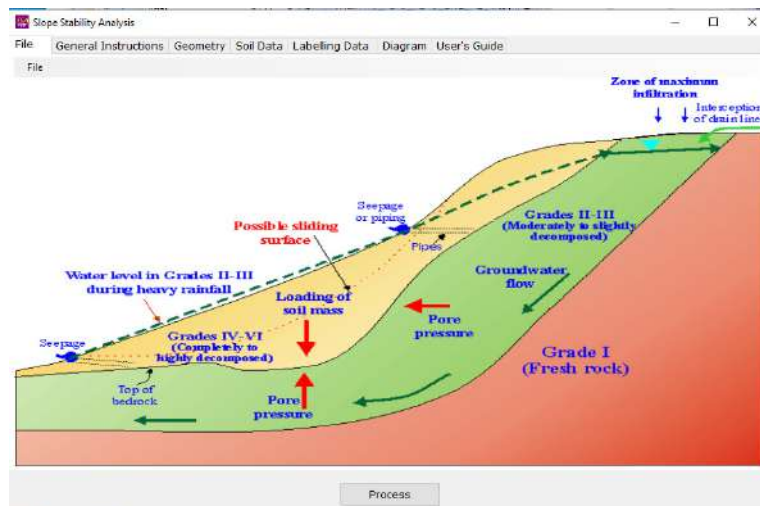


Image C5.2

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Select menu item 'General Instructions', Click on button 'Browse', Select drive "C", Folder "Program Files (x86)" and the folder "SSA", (located at path: C:\Program Files (x86)\SSA), finally click on OK.

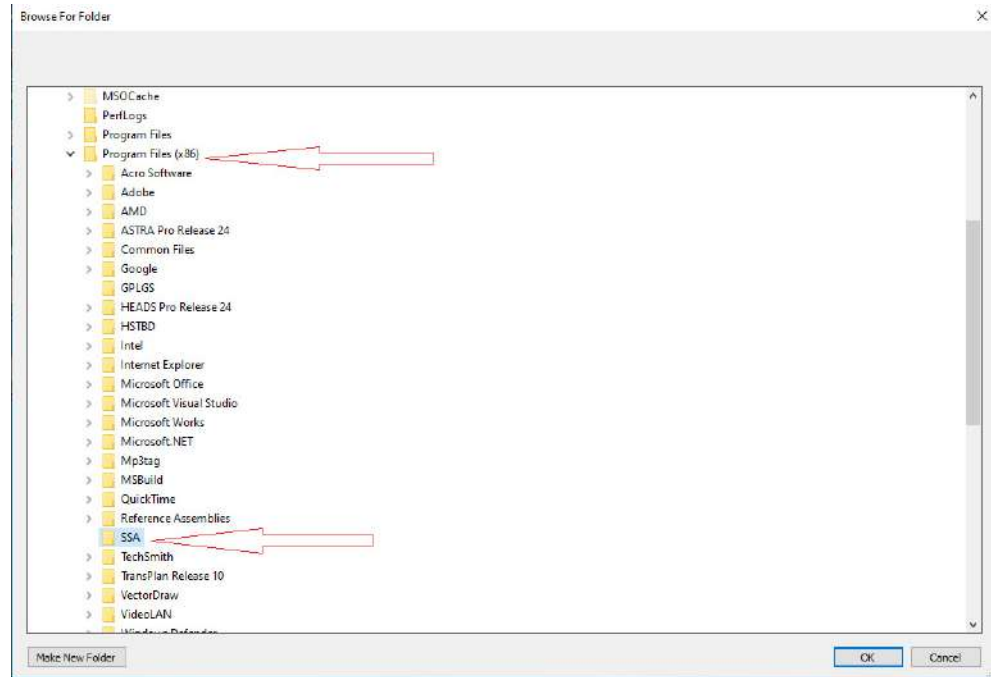


Image C5.3

Next, select "Example1" in the box below and load the data.

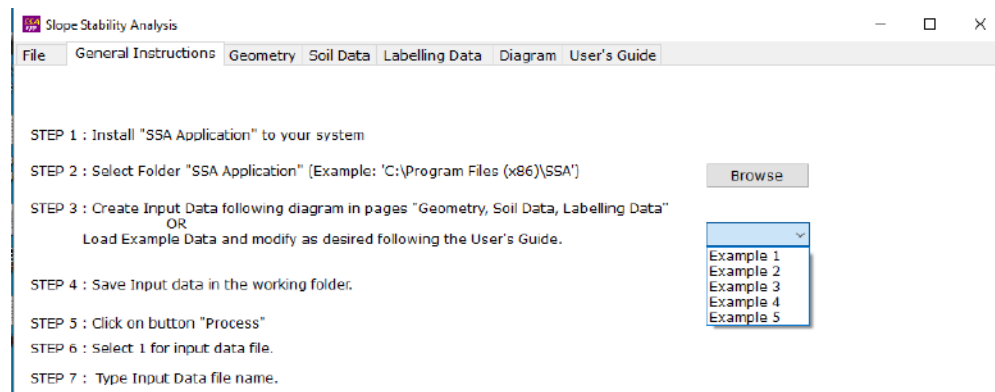


Image C5.4

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The message comes asking about loading an example data, click on button 'Yes',

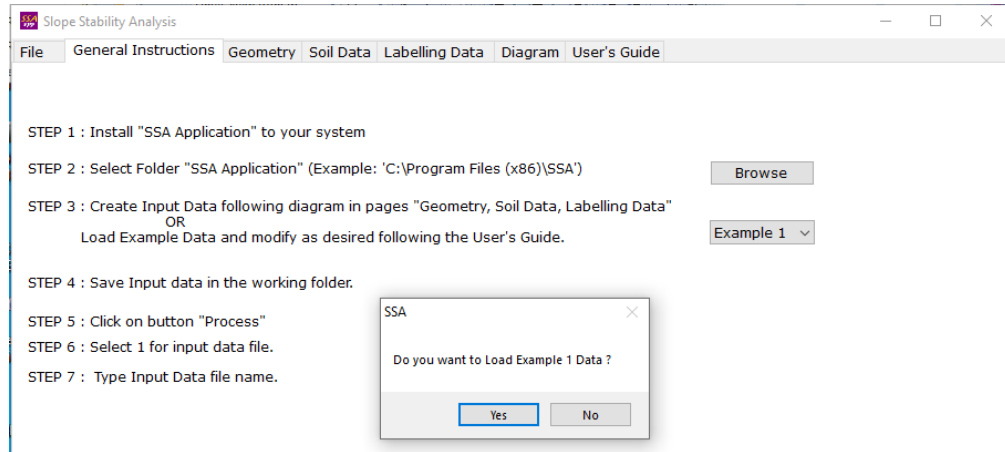


Image C5.5

The data may be seen and modified in tab pages "Geometry", "Soil Data", "Labelling Data" and "Diagram". Below the page for 'Geometry' is opened, user may change any data as relevant,

The screenshot shows the 'Slope Stability Analysis' software window with the 'Geometry' tab selected. The interface contains several input fields for defining a slope stability problem. The 'Title of the problem' is 'Sample Problem 1'. Under 'Control Data - I', fields include 'No.0 of top external soil lines' (5), 'No.0 of slices' (100), 'Indicator for reinforcement' (Yes), and 'Indicator for pore water pressure' (Water). Under 'Control Data - II', fields include 'Surcharge over the embankment' (0), 'X-Coordinate for the highest point towards left edge' (100.0), and 'X-Coordinate for the highest point towards right edge' (108.0). Under 'Coordinate Data - I', fields include 'X coordinate for starting centre of circle' (92), 'Y coordinate for starting centre of circle' (208), 'Incremental shift of X coordinate' (0.5), 'Incremental shift of Y coordinate' (0.5), 'Horizontal seismic coefficient' (0), and 'Vertical seismic coefficient' (0). Under 'Reinforcement Data', fields include 'Reinforcement Force' (263.7) and 'Y coordinate of Reinforcement Level' (200). Under 'Data for tangency point and minimum factor of safety', fields include 'Y coordinate for the starting elevation' (198) and 'Incremental shift for CTCH' (0.5). A 'Process' button is at the bottom.

Image C5.6

Below the page for 'Soil Data' is opened, user may change any data as relevant, or may enter data in blank boxes on the left side and click on button 'ADD' to include the data in the analysis input data.

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Slope Stability Analysis

File General Instructions Geometry Soil Data **Labelling Data** Diagram User's Guide

ADD Data

Starting X-Coordinate [X1]

Starting Y-Coordinate [Y1]

Ending X-Coordinate [X2]

Ending Y-Coordinate [Y2]

Density of the soil [WT]

Cohesion [CHS]

Angle of internal friction [FRC]

Ru for the soil [UPP]

ADD INSERT DELETE

	X1	Y1	X2	Y2	WT	CHS	FRC	UPP
▶	0	200	86.5	200	15	17	0	9.81
	86.5	200	100	206	20	0	32	0
	100	206	108	206	20	0	32	0
	108	206	121.5	200	20	0	32	0
	121.5	200	164	200	15	17	0	9.81
	86.5	200	121.5	200	15	17	0	9.81
	0	196	164	196	15	10000	45	0
*								

Process

Image C5.7

Below the page for 'Labelling Data' is opened, user may change any data as relevant, or may enter data in blank boxes on the left side and click on button 'ADD' to include the data in the analysis input data.

Slope Stability Analysis

File General Instructions Geometry Soil Data **Labelling Data** Diagram User's Guide

ADD Data

Material Density

Cohesion

Angle of Internal Friction

Soil pore Pressure Coefficient [ru]

X coordinate of the label

Y coordinate of the label

ADD INSERT DELETE

	Density	Cohesion	Angle of Internal Friction	ru	Label X coordinate	Label Y coordinate
▶	15	17	0	9.81	105	199
	20	0	32	0	105	204
	15	10000	45	0	105	195
*						

Process

Image C5.8

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Below the page for 'Diagram' is opened, the problem model is shown as a diagram with the analysis input data.

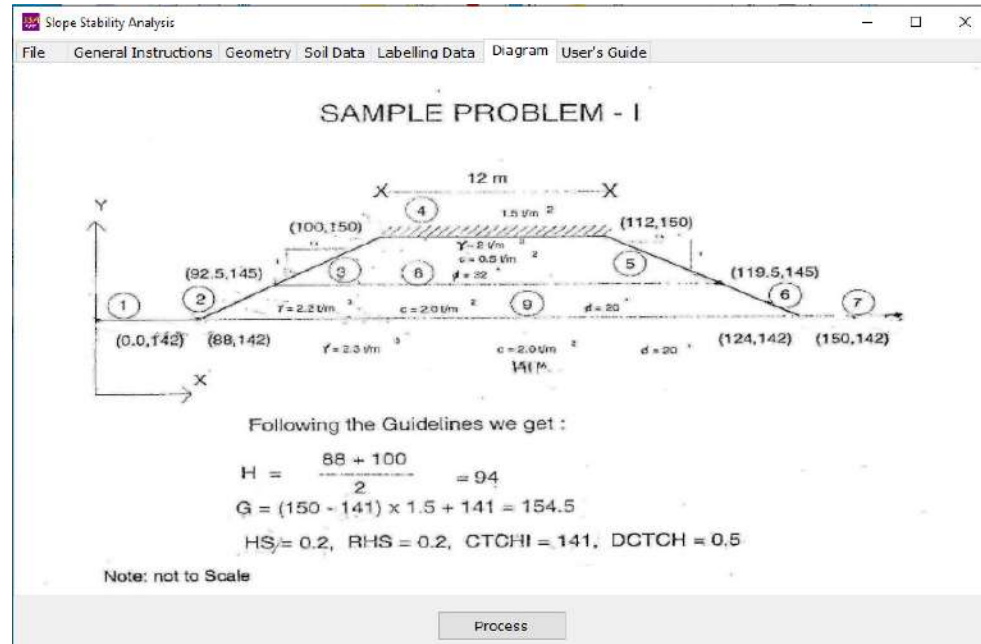


Image C5.9

Next, select menu 'File' and once again 'File', and then select menu item "Save As",

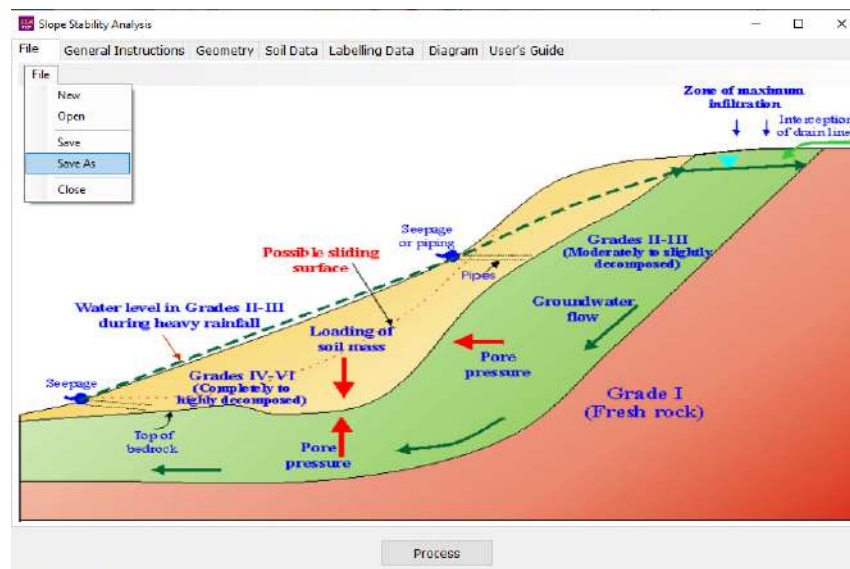


Image C5.10

Save the entered data by typing the file name as "SSA TEST.txt" as shown below, or any other name.

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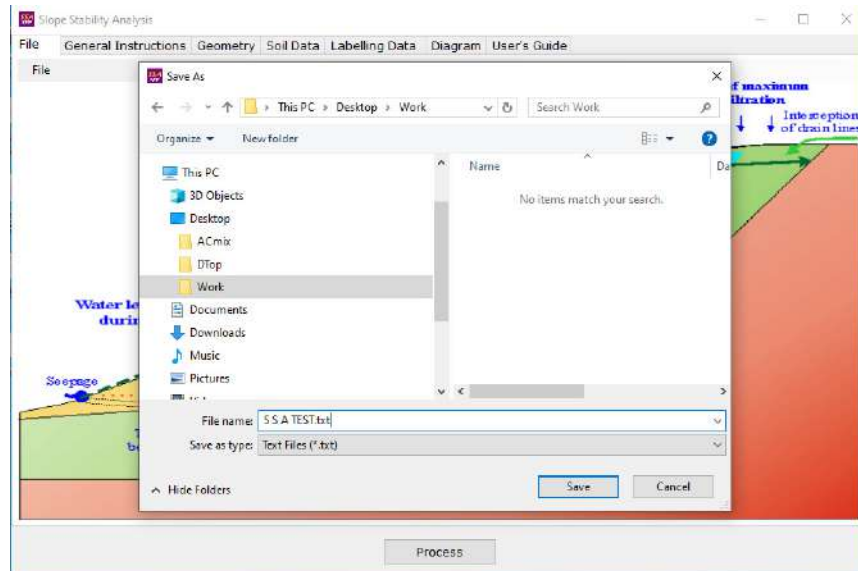


Image C5.11

The message comes confirming that the data is saved in the file.

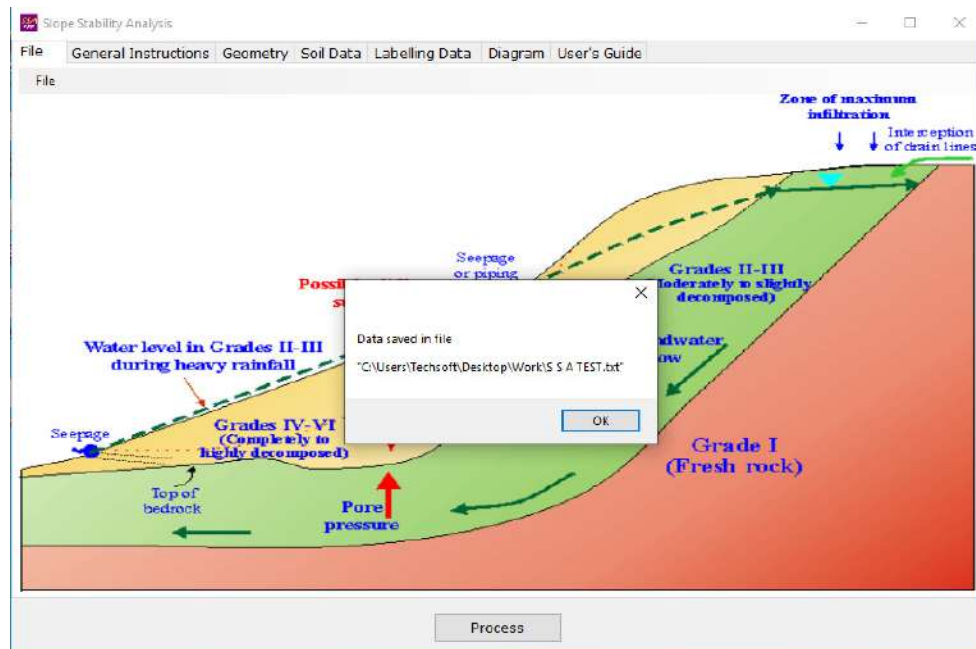


Image C5.12

Next, click on button 'Process' at the bottom, the message comes, click on "OK", As the S S A

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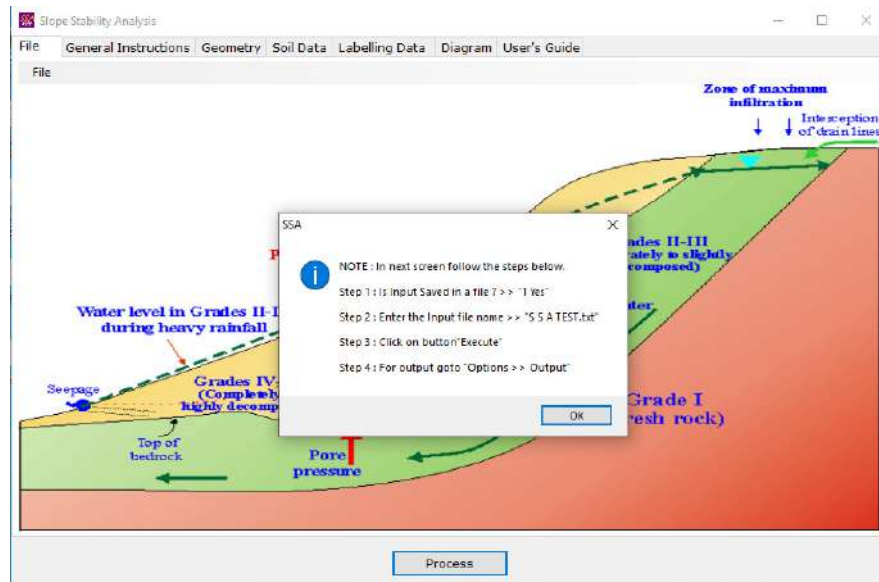


Image C5.13

Window comes, select "1 Yes",

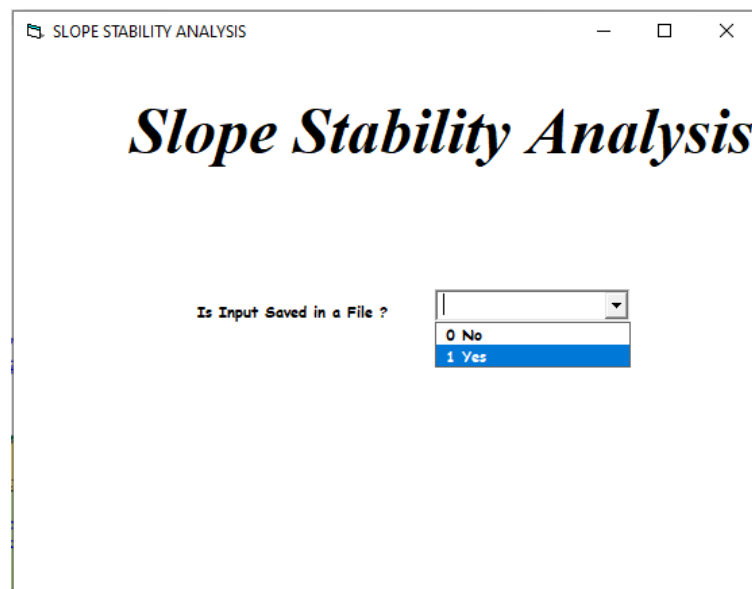


Image C5.14

Type the input file name as "S S A TEST.txt", Click on button "Execute".

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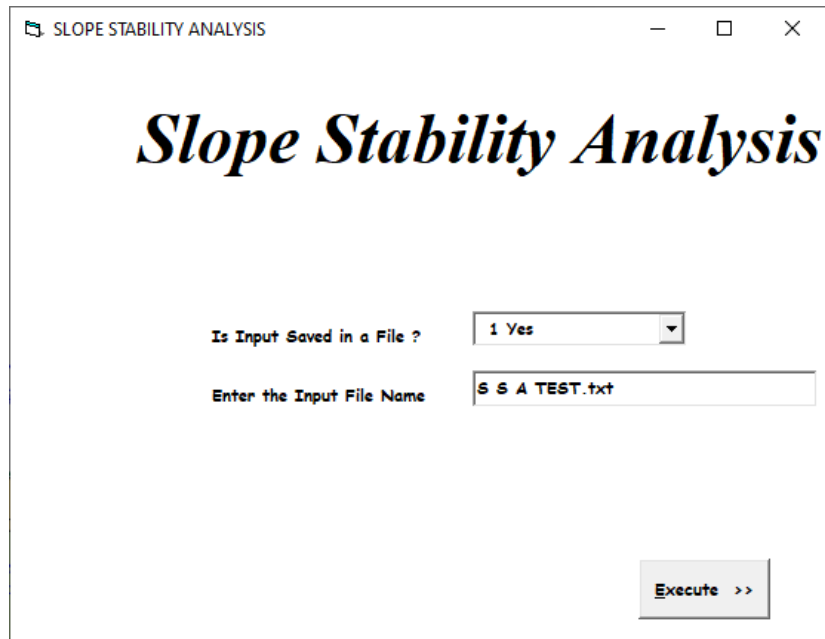


Image C5.15

The Output Result window opens, select a desired option to have a view or take a print.

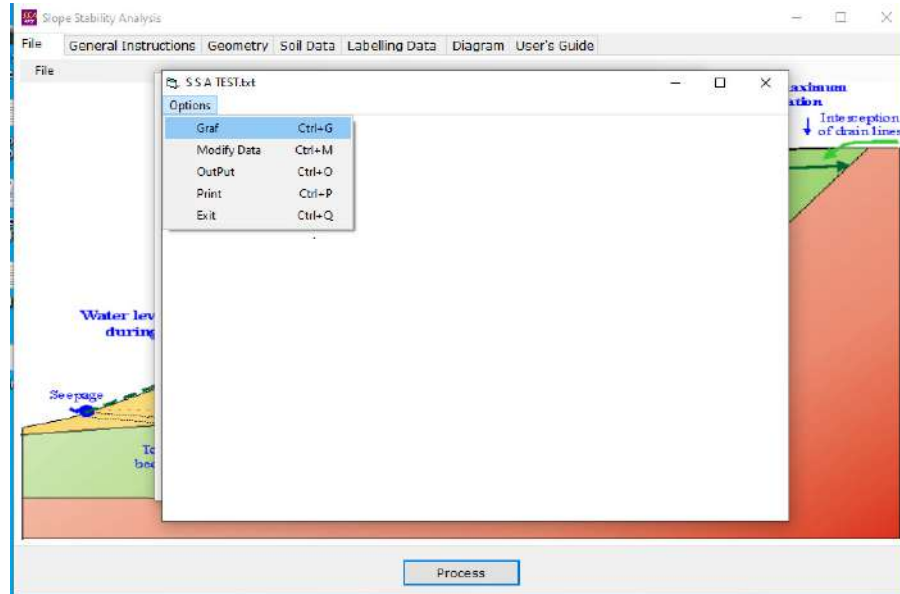


Image C5.16

On selecting menu item 'Graf', in last step, the window opens with the 'Diagram' and the related results.

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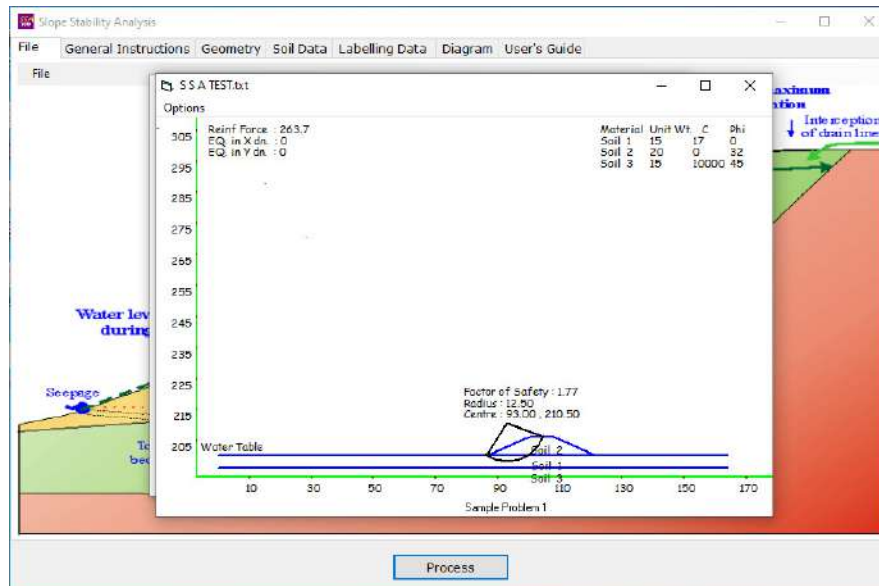


Image C5.17

Next, select the menu item 'Options >> Output', as shown below,

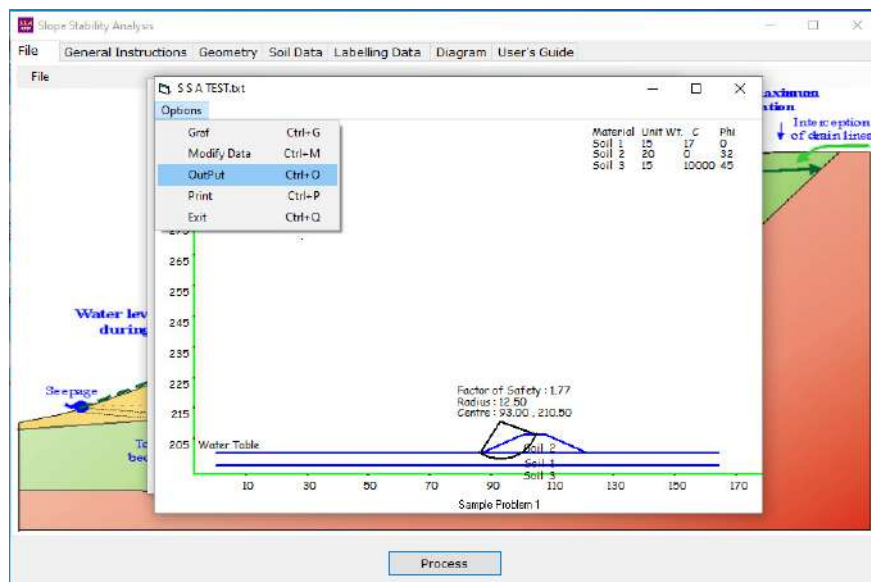


Image C5.18

A new window opens displaying the detail analysis output,

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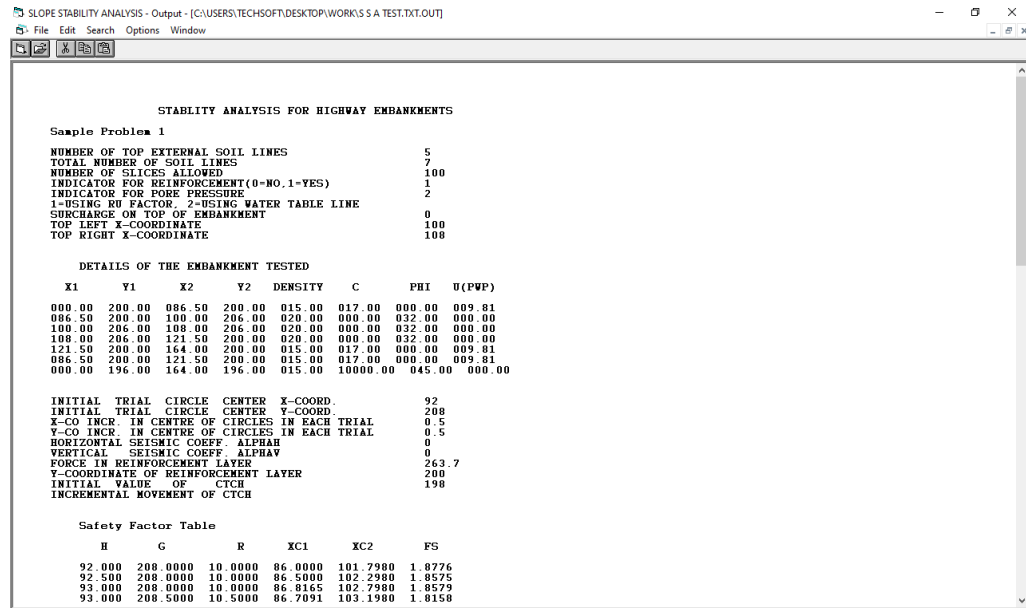


Image C5.19

After getting all the above steps done, user has to close the program SSA.

C5.8.4 Analysis Output:

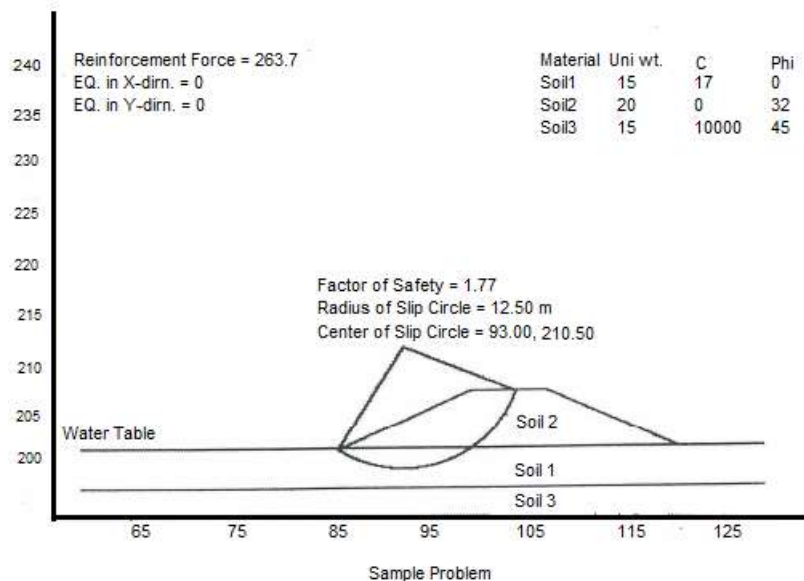


Figure C5.4 - Output Diagram

C5.8.5 Output File

STABILITY ANALYSIS FOR HIGHWAY EMBANKMENTS

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Sample Problem

NUMBER OF TOP EXTERNAL SOIL LINES	5
TOTAL NUMBER OF SOIL LINES	7
NUMBER OF SLICES ALLOWED	100
INDICATOR FOR REINFORCEMENT (0=No, 1=Yes)	1
INDICATOR FOR PORE PRESSURE	2
1=USING RU FACTOR, 2=USING WATER TABLE LINE	
SURCHARGE ON TOP OF EMBANKMENT	0
TOP LEFT X-COORDINATE	100
TOP RIGHT X-COORDINATE	108

DETAILS OF THE EMBANKMENT TESTED

X1	Y1	X2	Y2	DENSITY	C	PHI	U(PWP)
000.0	200.00	086.50	200.00	015.00	017.00	000.00	009.81
086.50	200.00	100.00	206.00	020.00	000.00	032.00	000.00
100.00	206.00	108.00	206.00	020.00	000.00	032.00	000.00
108.00	206.00	121.50	200.00	020.00	000.00	032.00	000.00
121.50	200.00	164.00	200.00	015.00	017.00	000.00	009.81
086.50	200.00	121.50	200.00	015.00	017.00	000.00	009.81
000.00	196.00	164.00	196.00	015.00	10000.00	045.00	000.00

INITIAL TRIAL CIRCLE CENTER X-COORED	92
INITIAL TRIAL CIRCLE CENTER X-COORED	208
X-CO INCR. IN CENTRE OF CIRCLES IN EACH TRIAL	0.5
X-CO INCR. IN CENTRE OF CIRCLES IN EACH TRIAL	0.5
HORIZONTAL SEISMIC COEFF. ALPHAH	0
VERTICAL SEISMIC COEFF. ALPHAV	0
FORCE IN REINFORCEMENT LAYER	263.7
Y-COORDINATE OF REINFORCEMENT LAYER	200
INITIAL VALUE OF CTCH	198
INCREMENTAL MOVEMENT OF CTCH	

Safety Factor Table (1)

H	G	R	XC1	XC2	FS
92.000	208.0000	10.0000	86.0000	101.7980	1.8776
92.500	208.0000	10.0000	86.5000	102.2980	1.8575
93.000	208.0000	10.0000	86.8165	102.7980	1.8579
93.000	208.5000	10.5000	86.7091	103.1980	1.8158
93.000	208.5000	10.5000	87.0254	103.6980	1.8258
92.500	208.0000	10.5000	86.3356	102.6980	1.8450
92.500	209.0000	11.0000	86.1754	103.0830	1.8158
93.000	209.0000	11.0000	86.6078	103.5830	1.8149
93.500	209.0000	11.0000	86.9186	104.0830	1.8241
93.500	209.5000	11.5000	86.8174	104.4545	1.7928
94.000	209.5000	11.5000	87.1281	104.9545	1.8111
93.000	209.5000	11.5000	86.5116	103.9545	1.8097
93.000	210.0000	12.0000	86.3668	104.3137	1.7886
93.500	210.0000	12.0000	86.7213	104.8137	1.7931
92.500	210.0000	12.0000	85.8668	103.8137	1.7966
92.500	210.5000	12.5000	85.7177	104.1619	1.7806

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93.000	210.5000	12.5000	86.2177	104.6619	1.7715
93.500	210.5000	12.5000	86.6291	105.1619	1.7798
93.500	211.0000	13.0000	86.5420	105.5000	1.7872
94.000	211.0000	13.0000	86.8384	106.0000	1.7783
94.500	211.0000	13.0000	87.1396	106.5000	1.8058

Minimum Factor of Safety = 1.7715

All Circles Touch at Elevation : 198.000

Safety Factor Table (2)

H	G	R	XC1	XC2	FS
92.000	208.0000	10.0000	86.0000	101.7980	1.8776
92.500	208.0000	10.0000	86.5000	102.2980	1.8575
93.000	208.0000	10.0000	86.8165	102.7980	1.8579
93.000	208.5000	10.5000	86.7091	103.1980	1.8158
93.500	208.5000	10.5000	87.0254	103.6980	1.8258
92.500	208.5000	10.5000	86.3356	102.6980	1.8450
92.500	209.0000	11.0000	86.1754	103.0830	1.8158
93.000	209.0000	11.0000	86.6078	103.5830	1.8149
93.500	209.0000	11.0000	86.9186	104.0830	1.8241
93.500	209.5000	11.5000	86.8174	104.4545	1.7928
94.000	209.5000	11.5000	87.1281	104.9545	1.8111
93.000	209.5000	11.5000	86.5116	103.9545	1.8097
93.000	210.0000	12.0000	86.3668	104.3137	1.7886
93.500	210.0000	12.0000	86.7213	104.8137	1.7931
92.500	210.0000	12.0000	85.8668	103.8137	1.7966
92.500	210.5000	12.5000	85.7177	104.1619	1.7806
93.000	210.5000	12.5000	86.2177	104.6619	1.7715
93.500	210.5000	12.5000	86.6291	105.1619	1.7798
93.500	211.0000	13.0000	86.5420	105.5000	1.7872
94.000	211.0000	13.0000	86.8384	106.0000	1.7783
94.500	211.0000	13.0000	87.1396	106.5000	1.8058

Minimum Factor of Safety = 1.7715

All Circles Touch at Elevation : 198.000

Safety Factor Table (3)

H	G	R	XC1	XC2	FS
92.000	208.0000	10.0000	86.0000	101.7980	1.8776
92.500	208.0000	10.0000	86.5000	102.2980	1.8575
93.000	208.0000	10.0000	86.8165	102.7980	1.8579
93.000	208.5000	10.5000	86.7091	103.1980	1.8158
93.500	208.5000	10.5000	87.0254	103.6980	1.8258
92.500	208.5000	10.5000	86.3356	102.6980	1.8450
92.500	209.0000	11.0000	86.1754	103.0830	1.8158
93.000	209.0000	11.0000	86.6078	103.5830	1.8149

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93.500	209.0000	11.0000	86.9186	104.0830	1.8241
93.500	209.5000	11.5000	86.8174	104.4545	1.7928
94.000	209.5000	11.5000	87.1281	104.9545	1.8111
93.000	209.5000	11.5000	86.5116	103.9545	1.8097
93.000	210.0000	12.0000	86.3668	104.3137	1.7886
93.500	210.0000	12.0000	86.7213	104.8137	1.7931
92.500	210.0000	12.0000	85.8668	103.8137	1.7966
92.500	210.5000	12.5000	85.7177	104.1619	1.7806
93.000	210.5000	12.5000	86.2177	104.6619	1.7715
93.500	210.5000	12.5000	86.6291	105.1619	1.7798
93.500	211.0000	13.0000	86.5420	105.5000	1.7872
94.000	211.0000	13.0000	86.8384	106.0000	1.7783
94.500	211.0000	13.0000	87.1396	106.5000	1.8058

Minimum Factor of Safety = 1.7715

All Circles Touch at Elevation: 198.000

Centre Coordinates: X = 93.0000 Y = 210.5000

Radius(R)= 12.5000

XC1 = 86.2177 YC1=200.0000

XC2 = 104.6619 YC2=206.0000

Slip Circle touches elevation = 198.0000

The Factor of Safety is = 1.7715

Edit the output file as per your requirements and from the File menu print the file

- Click the mouse on **File** menu
- Click the mouse on **Exit**, to exit from the output file
- From the **Options** menu, Click the mouse on **Exit** to exit from the program

11.2 Input through user interactive windows

- Click the mouse on **Start**
- From the programs options select 'SSA' (Click the mouse on SSA)
- A window will appear titled '**Slope Stability Analysis**' asking '**Is Input saved in a file?**'
- Pull down the combo box
- Click the mouse on **0 NO**

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The various tutorial data are available in the two primary folders 'Book Tutorials' and 'Software Tutorials', which exist in the website of this book, as mentioned below:

1. Website: www.roadbridgedesign.com, for downloading as follows:

There is menu item 'Book Tutorials', under this menu item, select 'Road Pavements', which contains 'User's Guide for Computer Applications in Pavement Engineering', and all other items of this book. Each computer application may be processed by following the step-by-step guide in the 'User's Guide for Computer Applications in Pavement Engineering', for respective applications.

2. Website: www.techsoftglobal.com, for downloading as follows:

There is menu item 'Software Tutorials', under this menu item, select 'HEADS Pro', which contains various items used in this book, and are to be downloaded. Software 'HEADS Pro R24 Installation Setup' is to be run to install HEADS Pro in the computer. Chapter wise various data may be processed by following the step-by-step guide in the 'User's Guide for Computer Applications in Pavement Engineering', for respective chapters of this book.

APPENDIX - D

Road Condition Survey – Pavement Structure and Underground Utilities

D1 Ground Penetrating Radar Survey (GPRS)

D1.1 General

Ground Penetrating Radar (GPR) uses high-frequency pulsed electromagnetic waves to map subsurface information. GPR uses transmitting and receiving antennae, which are dragged along the ground surface. The transmitting antenna radiates short pulses of high-frequency radio waves into the ground. The wave spreads out and travels downward. If it hits a buried object or a boundary with different electrical properties, the receiving antenna records variations in the reflected return signal. The principles involved are similar to reflection seismology, except that electromagnetic energy is used instead of acoustic energy, and the resulting image is relatively easy to interpret. Integration of GPR data with other surface geophysical methods reduces uncertainty in site characterization. GPR provides the highest lateral and vertical resolution of any surface geophysical method.

Best penetration is achieved in dry sandy soils or massive dry materials such as granite, limestone, and concrete. GPR provides the greatest resolution of currently available surface geophysical methods. It is the only reliable method for detecting buried plastic containers.

D1.2 Limitations and Concerns

Depth of penetration (typically 1 to 15 meters) is less than direct current (DC) resistivity and electromagnetic (EM) methods, and is further reduced in moist and/or clayey soils and soils with high electrical conductivity. Penetration in clays and in materials having high moisture is sometimes less than 1 meter.

The GPR method is sensitive to noise—i.e., interference caused by various geologic and cultural factors. For example, boulders, animal burrows, tree roots, and other phenomena can cause unwanted reflections or scattering. Cultural sources of noise can include reflections from nearby vehicles, buildings, fences, power lines, and trees. Electromagnetic transmissions from cellular telephones, two-way radios, television, and radio and microwave transmitters may also cause noise on GPR records. Shielded antennae are used to limit these types of reflections.

The bulkiness of equipment can limit use in rough terrain.

Unprocessed images require processing, as they provide only approximate shapes and depths (continuous microwave methods are still in developmental stages).

D1.3 Applicability

A GPR system is made up of three main components:

- Control unit

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- Antenna
- Power Supply

GSSI GPR equipment can be run with a variety of power supplies ranging from small rechargeable batteries to vehicle batteries and normal 110/220-volt. Connectors and adapters are available for each power source type. The unit in the photo above can run from a small internal rechargeable battery or external power.



Figure D1.1 - Ground Penetrating Radar and Control Unit, GSSI GPR Equipment

Ground penetrating radar is a widely accepted field screening technology for characterizing and imaging subsurface conditions. Ground penetration radar (GPR) is most commonly used for locating buried objects such as tanks, pipes, and drums, as well as shallow unexploded ordnance (UXO). It can also be used to map the depth of a shallow water table, identify soil horizons and the bedrock subsurface, and demarcate trench boundaries. It is also used to delineate the physical integrity of manmade structures, such as slurry walls and permeable reactive barrier walls. GPR can also detect plastic, glass, concrete, or wood.

D1.4 GPR Control Unit and Antenna

The control unit contains the electronics which trigger the pulse of radar energy that the antenna sends into the ground. It also has a built-in computer and hard disk/solid state memory to store data for examination after fieldwork. Some systems, such as the GSSI SIR 30, are controlled by an attached Windows laptop computer with pre-loaded control software. This system allows data processing and interpretation without having to download radar files into another computer.

The antenna receives the electrical pulse produced by the control unit, amplifies it and transmits it into the ground or other medium at a particular frequency. Antenna frequency is one major factor in depth penetration. The higher the frequency of the antenna, the shallower into the ground it will

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penetrate. A higher frequency antenna will also 'see' smaller targets. Antenna choice is one of the most important factors in survey design. The following table shows antenna frequency, approximate depth penetration and appropriate application.

Table D1.1 - Antenna Frequency, Approximate Depth Penetration and Appropriate Applications

Appropriate Application	Primary Antenna Choice	Secondary Antenna Choice	Approximate Depth Range
Structural Concrete, Roadways, Bridge Decks	2600 MHz	1600 MHz	0–0.3 m (0-1.0 ft)
Structural Concrete, Roadways, Bridge Decks	1600 MHz	1000 MHz	0-0.45 m (0-1.5 ft)
Structural Concrete, Roadways, Bridge Decks	1000 MHz	900 MHz	0-0.6 m (0-2.0 ft)
Concrete, Shallow Soils, Archaeology	900 MHz	400 MHz	0-1 m (0-3 ft)
Shallow Geology, Utilities, USTs, Archaeology	400 MHz	270 MHz	0-4 m (0-12 ft)
Geology, Environmental, Utility, Archaeology	270 MHz	200 MHz	0-5.5 m (0-18 ft)
Geology, Environmental, Utility, Archaeology	200 MHz	100 MHz	0-9 m (0-30 ft)
Geologic Profiling	100 MHz	MLF (16-80 MHz)	0-30 m (0-90 ft)
Geologic Profiling	MLF (16-80 MHz)	None	Greater than 30 m (90 ft)

D1.5 GPR Method

GPR works by sending a tiny pulse of energy into a material and recording the strength and the time required for the return of any reflected signal. A series of pulses over a single area make up what is called a scan. Reflections are produced whenever the energy pulse enters into a material with different electrical conduction properties or dielectric permittivity from the material it left. The strength, or amplitude, of the reflection is determined by the contrast in the dielectric constants and conductivities of the two materials. This means that a pulse which moves from dry sand (dielectric of 5) to wet sand (dielectric of 30) will produce a very strong reflection, while moving from dry sand (5) to limestone (7) will produce a relatively weak reflection.

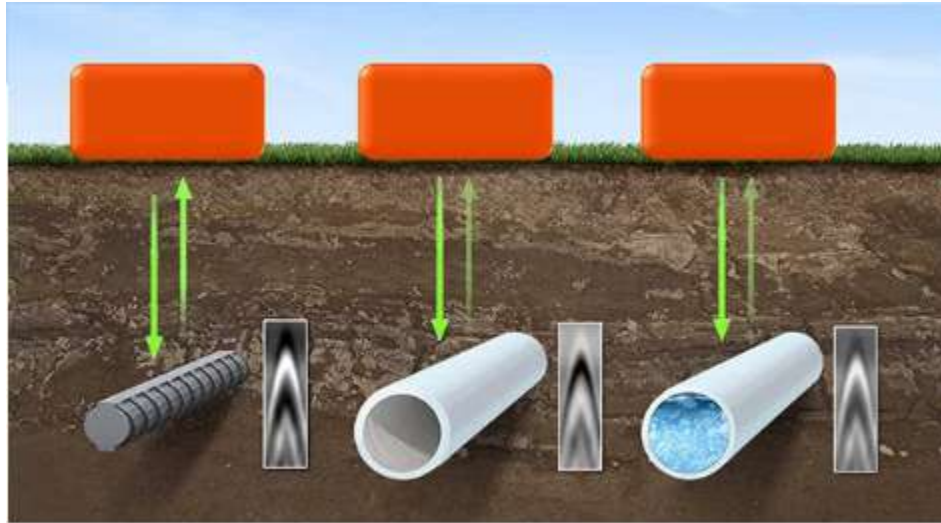


Figure D1.2 – Pulse of Energy Sent and Reflected Signal Received by Different Subsurface Objects

While some of the GPR energy pulse is reflected back to the antenna, energy also keeps traveling through the material until it either dissipates (attenuates) or the GPR control unit has closed its time window. The rate of signal attenuation varies widely and is dependent on the properties of the material through which the pulse is passing.

Materials with a high dielectric will slow the radar wave and it will not be able to penetrate as far. Materials with high conductivity will attenuate the signal rapidly. Water saturation dramatically raises the dielectric of a material, so a survey area should be carefully inspected for signs of water penetration.

Metals are considered to be a complete reflector and do not allow any amount of signal to pass through. Materials beneath a metal sheet, fine metal mesh, or pan decking will not be visible.

Radar energy is not emitted from the antenna in a straight line. It is emitted in a cone shape (picture on left). The two-way travel time for energy at the leading edge of the cone is longer than for energy directly beneath the antenna. This is because that leading edge of the cone represents the hypotenuse of a right triangle.

Since it takes longer for that energy to be received, it is recorded farther down in the profile. As the antenna is moved over a target, the distance between the two decreases until the antenna is over the target and increases as the antenna is moved away. It is for this reason that a single target will appear in the data as a hyperbola, or inverted “U.” The target is actually at the peak amplitude of the positive wavelet.

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Figure D1.3 – Pulse of Energy Sent GSSI GPR Equipment

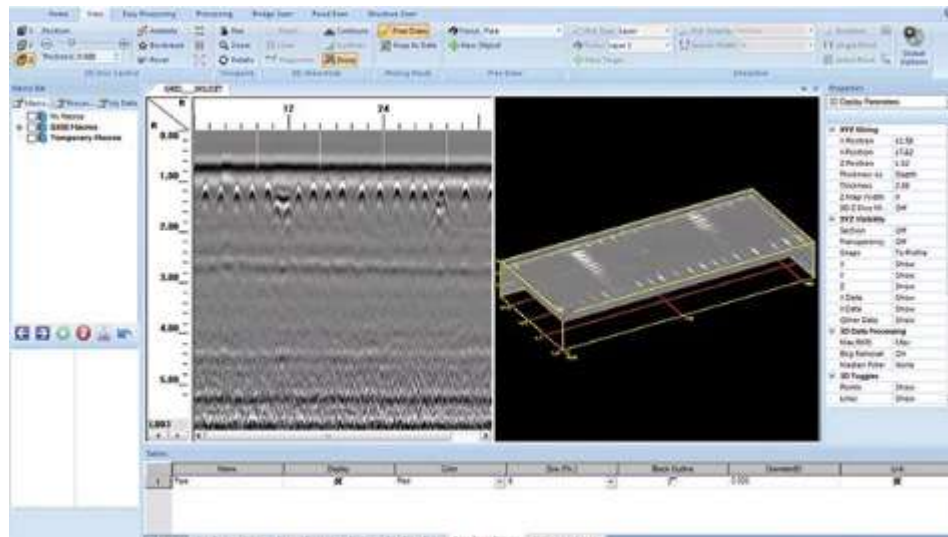


Figure D1.4 – The Computer Produces a Horizontal Surface at a Particular Depth (Depth Slice) to Interpret a Plan View of the Survey Area with the Record of Different Subsurface Objects

D1.6 Data Processing

Data is collected in parallel transects and then placed together in the appropriate locations for computer processing in a specialized software program such as GSSI's RADAN. The computer then

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produces a horizontal surface at a particular depth in the record. This is referred to as a depth slice, which allows operators to interpret a plan view of the survey area.

In many situations, a GPR operator will simply note the location of a target so that it can be avoided. For these clients, it may only be necessary to use a simple line-scan format in order to mark the approximate area of the target on the survey surface. Other clients may require detailed subsurface maps and depth to features.

These situations will require the operator to use GSSI GPR processing software, which applies mathematical functions to the data in order to remove background interference, migrate hyperbolas, calculate accurate depth and much more.

D1.7 Pavement inspections with GPR

Single channel GPR pavement thickness mapping was one of the first civil engineering applications. Early work was done by Maser and Scullion (1992) showing that a single horn antenna could accurately map multiple layers of pavement with the added benefits of being operated at highway speed without the need for road closures. Saarenketo and Scullion (2000) illustrated the wide variety of research done in the field of civil engineering pertaining to layer detection. Hugenschmidt (2004) shows an example of GPR being used to detect the thickness of a bridge deck using layer detection and the layer of reinforcement. Loizos and Plati (2007) show layer detection of multiple pavements with GPR systems at different frequencies, and delve into the accuracy of the systems by checking them against the ground truth with good results. The dielectric constant of pavement can be calculated by evaluating the wave velocity within the top pavement layer or the reflection factor of the pavement surface (Saarenketo, 2006). In multi-layer scenarios, reflecting waves may interfere with each other, requiring special data processing approaches to extract the layer information (Lahouar and Al-Qadi, 2008; Zhou *et al.*, 2010).

There are various service provider in underground utility mapping. Their underground utility survey team uses Ground Penetrating Radar technology to detect all types of underground utilities. By using GPR scanner device the radio waves are used to detect buried objects which includes metal and plastic pipes, subsurface drainage system and channels, electricity, telecom and fiber optic cables. Once detected a physical markup is created on the ground for easy identification for site personnel. Also, the final survey results are presented in the form of a CAD drawing with all underground information accurately recorded on to the ground model drawing and in topographical survey report.

Such services include handling projects of all volumes and dimensions. By using a good combination of survey equipment and software required for maximizing ground penetration for capturing accurate details of all buried objects. The advantages are:

- Avoid clashes with underground objects and prevent service disruption.
- Locates metallic as well as non-metallic utilities.
- Rapid data acquisition and versatility.
- It's a safe, non-destructive method for detecting hidden elements.
- Avoid destroying the structure.
- Instant feedback on sub-surface structures, quick control of system setup.
- Quickly and easily view and note anomalies in

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