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About Conference

Technical Research Organisation India (TROI) is pleased to organize the 2nd International Conference on Biotechnology, Civil and Chemical Engineering (ICBCCE 2015).

ICBCCE is a comprehensive conference covering the various topics of Biotechnology, Civil and Chemical Engineering. The aim of the conference is to gather scholars from all over the world to present advances in the aforementioned fields and to foster an environment conducive to exchanging ideas and information. This conference will also provide a golden opportunity to develop new collaborations and meet experts on the fundamentals, applications, and products of Biotechnology, Civil and Chemical engineering. We believe inclusive and wide-ranging conferences such as ICBCCE can have significant impacts by bringing together experts from the different and often separated fields of Biotechnology, Civil and Chemical Engineering. It creating unique opportunities for collaborations and shaping new ideas for experts and researchers. This conference provide an opportunity for delegates to exchange new ideas and application experiences, we also publish their research achievements. ICBCCE shall provide a plat form to present the strong methodological approach and application focus on Biotechnology, civil & chemical engineering that will concentrate on various techniques and applications. The conference cover all new theoretical and experimental findings in the fields of Civil, Chemical and Biotechnology engineering or any closely related fields.

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Editor-in-Chief

Prof. Sanjith J

Editorial

The conference is designed to stimulate the young minds including Research Scholars, Academicians, and Practitioners to contribute their ideas, thoughts and nobility in these two integrated disciplines. Even a fraction of active participation deeply influences the magnanimity of this international event. I must acknowledge your response to this conference. I ought to convey that this conference is only a little step towards knowledge, network and relationship.

The conference is first of its kind and gets granted with lot of blessings. I wish all success to the paper presenters.

I congratulate the participants for getting selected at this conference. I extend heart full thanks to members of faculty from different institutions, research scholars, delegates, TROI Family members, members of the technical and organizing committee. Above all I note the salutation towards the almighty.

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METHODOLOGY FOR THE ENVIRONMENTAL RISK ASSESSMENT OF FLY ASH USE IN HIGHWAY EMBANKMENTS

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Abstract: Fly ash can be used in highway embankments for larger and convenient consumption. Techno-economic considerations of fly ash use enforce the need to evaluate various models. The selected models also need to be evaluated from environmental considerations. So a flexible and suitable risk assessment methodology has been suggested in this paper by adopting different models. The environmental impact is based upon many factors and their relative weight-ages. Based upon subjective or specific judgment the values can be assigned to these factors. The quantitative analysis of these values can give sufficient indication in regard to the suitability of use of fly ash in embankment. The positive impact assessment values denote larger adverse environmental impact and the negative values denote the favourable environmental impact.
Key words: fly ash, environmental impact, risk assessment, models, highway embankment

Introduction

Fly ash is causing environmental pollution, creating health hazards and requires large areas of precious land for disposal. Due to increasing concern for environmental protection and growing awareness of the ill effects of pollution, disposal of ash generated at thermal power plants has become an urgent and challenging task. Also in developed urban and industrial areas, natural burrow sources are scarce, expensive or inaccessible. The environmental degradation caused due to the use of top soil for embankment construction may be un-measurable. Moreover, many power plants are located in urban areas, and therefore, fly ash can provide an environmentally better alternative to natural burrow soil.
The major environmental concerns with respect to the potential impact of fly ash usage in roadwork including embankment, are wind erosion, surface water erosion and leaching of toxic heavy metals into water bodies including under ground aquifers. During the construction of fly ash embankments, keeping the ash moist during compaction can minimize wind erosion.

Major portions of inorganic compounds in fly ash are present as alumino-silicate glass. Most of the other elements are present in very small quantities and are largely encapsulated in the glassy material. Typically less than 2% of the fly ash is water-soluble; calcium and silicate constitute the majority of soluble fraction (Lindon K.A.Sear, 2003). There are smaller amounts of sodium, potassium and magnesium. The pH is mainly determined by water-soluble calcium and sulphate producing an alkaline environment. When used in structural fill applications fly ash has very low permeability, which means that there is very little passage of water through it and very little potential leachate. Most trace elements are held in alumino silicate matrix and are not available to leach. A deposit of fly ash in embankment is further aids retention of metals. The highway embankment constructed using fly ash being a narrow stretch on land use has close to nil impact on water quality of the area. Fly ash embankment contains almost no biodegradable organic material and produces no gas as a product of such degradation. The radioactivity of fly ash embankment is similar to that of conventional construction materials. (Lindon K.A.Sear, 2003).

Despite the large volumes of ash produced, the total quantity of heavy metals is relatively small, and an even smaller amount of these elements can be released to the environment. However, it is important to note that despite these relatively low concentrations, if improperly managed, any waste can have a negative impact on the environment.

Adverse health effects from skin contact with coal ash appear to be extremely unlikely in highway embankment construction.

The physical location of the power plant often has a great impact on disposal and use of fly ash in highway embankments. Plants located in urban areas may have no space for on-site disposal necessitating transport to other locations for disposal. However, as these locations become completely filled, new land must be found for disposal. New sites may require environmental reviews and regulatory hurdles.

In some situations, the fly ash is not mixed with water, but instead loaded directly into covered trucks or pneumatic tank trucks for transport. If this material is disposed, then handling at the disposal site is normally more challenging due to potential dusting issues. The fly ash used in embankment construction must be kept suitably moistened as otherwise wind blowing may cause fly ash inhalation to workers and also its spread to adjoining areas.

The fly ash can be more easily spread and compacted than conventional soil, which may require lumps to be broken. Less compactive effort may save equipment-operating time thus having reduced impact on environment.

alkaline which

In some situations fly ash with or without additives may improve the engineering properties of the subgrade thus necessitating reduced pavement thickness requirement in the case of flexible pavements. This may considerably save on cost, quarry operations, transportation and other factors, which adversely affect the environment.

Risk Assessment

All wastes or by-product materials should be evaluated prior to use to fully assess the inherent hazard potential of the material, if used in the proposed application. Simply because a waste legally may not be subject to hazardous waste regulation is not necessarily an indicator that it is not potentially chemically hazardous or contains constituents that could pose threats to human health or the environment

Historical Perspective (Gupta 2004)

Prior to 1970, there was little if any environmental regulatory oversight regarding the use of waste and by-product materials in pavement construction applications. In general, those materials that exhibited acceptable engineering properties and were both cost-effective and not considered to be "harmful" to workers or the environment were often used. During that period, however, there were no specific procedures or criteria available to quantify potential environmental concerns or "harmful" impacts.

Hazardous wastes may be identified as of two types. They may be referred to as listed wastes or characteristic wastes. A listed waste is a waste that is classified as hazardous due to its source and the way it is produced. A characteristic waste is a waste that must be tested to determine if it exhibits one of four properties: (1) ignitability, (2) corrosivity, (3) reactivity, or (4) toxicity.

General Requirements (Gupta 2004)

Due primarily to the increased pressure to recover and use waste and by-product materials, in recent years, most state environmental regulatory agencies (especially those in industrial areas) have begun the process of formalizing their regulatory procedure for approving the use of waste and by-product materials. At the present time, however, there are no universally accepted environmental approval and permitting procedures.

Regulatory requirements in general can take one or more of the following forms:

No approval is required — material is considered acceptable due to previous history of use.

Approval is required — material must not exhibit the characteristics of a hazardous waste.

Environmental or risk assessment is required — a field and/or desktop evaluation must be provided to demonstrate that the material will have no adverse impact on human health and the environment.

Although the first two requirements are rather straightforward, the latter requirement can necessitate a series of evaluations that could include the preparation of an environmental assessment, a human health risk assessment, or an ecosystem risk assessment.

Environmental assessments generally require a quantification of emissions or discharges from a proposed activity (e.g., construction of a pavement using a waste or by-product material) and a projection of the impact of this emission or discharge on the ambient environment. The magnitude of the impact is usually assessed by comparing the source discharge or the projected ambient impact to some source discharge standard (e.g., groundwater or surface water discharge limits) or some ambient air, water or soil quality standard (e.g., ambient air or water quality criteria). Projections of impacts to the ambient environment are normally estimated using environmental models (e.g., air and water quality models).

Human health assessments provide for a linking of discharges and emissions from specific sources to vulnerable human receptors in an attempt to quantify risks (using reference doses for carcinogenic and no carcinogenic effects) associated with a specific activity. They attempt to account for all potential contaminants and exposure routes (e.g., ingestion, inhalation, and dermal absorption) that might affect the identified receptor.

Ecosystem risk assessments are evaluations that focus on potential impacts to flora and fauna, usually in the immediate environment of the action. Like human health risk assessments, they tend to focus on specific transfer routes to identifiable flora and fauna and the impact on these organisms. They sometimes address long-term cumulative impacts that may result from the proposed action, such as bioaccumulation and potential food chain effects.

For environmental suitability of using waste and by-product materials in embankment construction applications, there are common elements to all environmental assessments that form the basis for determining the potential impacts associated with a proposed application. These common elements include the following:

Identification of potential hazards posed by the use of the material.

Identification of persons or media (air, water, soil) likely to be impacted by the identified hazard.

Identification of the magnitude of the potential impact.

Identification of Potential Hazards

Some waste and by-product materials may contain concentrations of trace metals or trace organics that are higher in concentration and/or more environmentally mobile than those found in conventional materials. Others may contain highly alkaline materials (e.g., free lime), high concentrations of soluble salts, very fine particles that may be susceptible to dusting and may also be respirable. Still others may contain volatile organic or inorganic material that could be released in high-temperature environments.

Impacted Persons or Media

The highway embankment construction process comprises numerous operations including material storage, handling, production, placement, excavation, and disposal or recycling operations. These operations are all part of the embankment construction, service life, and post service life activities. Potential dust or volatile emissions or liquid discharges from these operations could have an impact on ambient air, surface or groundwater, soils, or the worker environment.

The identification of each of these operations is important when identifying impacted persons or media.

Magnitude of Impact

Techniques for determining the magnitude of the impact will depend in great part on the type of evaluation that is required (i.e., traditional environmental assessment, human health risk assessment, or ecosystem risk assessment). In all cases the use of source emission, ambient air, surface water, and groundwater models will probably be required. An attempt has been made to have subjective environmental risk assessment by assuming different models (Fig 1 to 6) as given in the table-1 (Gupta 2004).

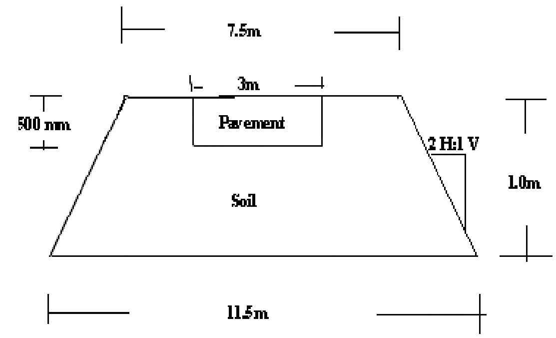
The Table 1 illustrates the environmental risk assessment model that can be used as a guide for evaluating the use of fly ash in highway embankments. The environmental impact is based upon many factors and their relative weightage. Based upon subjective or specific judgement the values can be assigned to these factors. The quantitative analysis of these values can give sufficient indication in regard to the suitability of use of fly ash in embankment. The below mentioned table has variation of cumulative weightage values between 3 and 78. Thus from 3 to 78 the value range can be divided into four broad categories if desired. **From 3 to 21 least impact, 22 to 40 moderate impact, 41 to 59 significant impact, and 60 to 78 large impact.** Thus for any specific site, the cumulative weighted environment impact value can be estimated and desirable inference drawn. The factors and their relative weightage can be reviewed from time to time as per past experience and best judgement (Gupta 2004).

As an example, six models (1 to 6) are subjectively assessed to have weighted environmental risk assessment value of 15,36,25,14,24 and 30 that are as per the earlier assumed classification. The model-4 denotes least impact to environment.

The various models (Gupta 2004) taken for study of highway embankment construction are:

- **Model-1:** Embankment constructed with the locally available soil of 2.3% CBR. Pavement crust thickness required is 500mm (sub base 350 mm + base 150 mm).

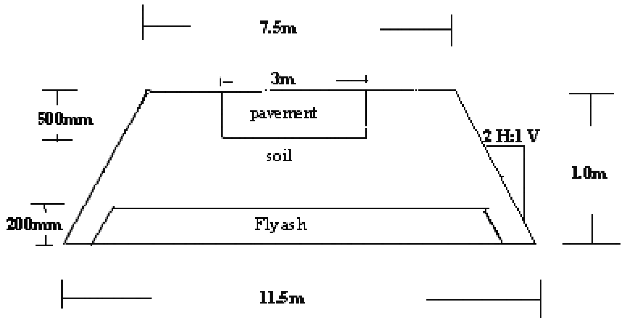
Fig. 1



Typical cross section of highway embankment with pavement (Model-1)

- **Model-2:** Embankment constructed with the fly ash and with 1m-earth cover on sides and top 0.3m of earth with 2.3% CBR to form sub grade. Pavement crust thickness required is 500mm (sub base 350mm + base 150 mm).

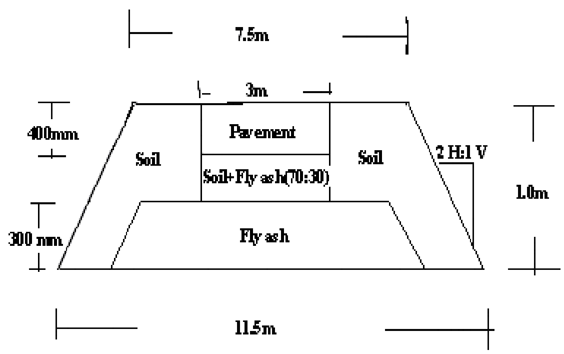
Fig. 2



Typical cross section of highway embankment with pavement(Model-2)

- **Model-3:** Embankment constructed with earth and with 1m-earth cover on sides and sub grade constructed with soil-fly ash mixture (70:30) of 3.3% CBR. Pavement crust thickness required is 400mm (sub base 250mm + base 150mm).

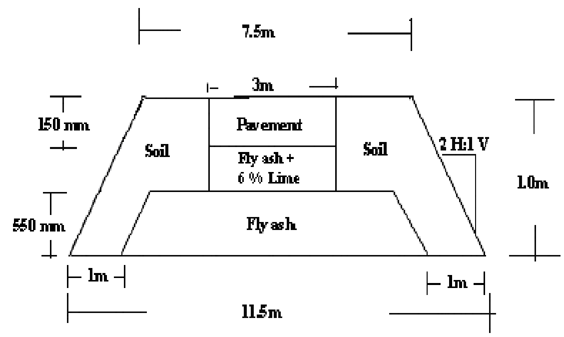
Fig.3



Typical cross section of embankment with pavement(Model-3)

- **Model-4:** Embankment constructed with the fly ash using suitable earth cover and sub grade constructed using stabilised layer (fly ash + 6% lime) of 18% CBR. Pavement crust thickness required is 150mm (base 150mm).

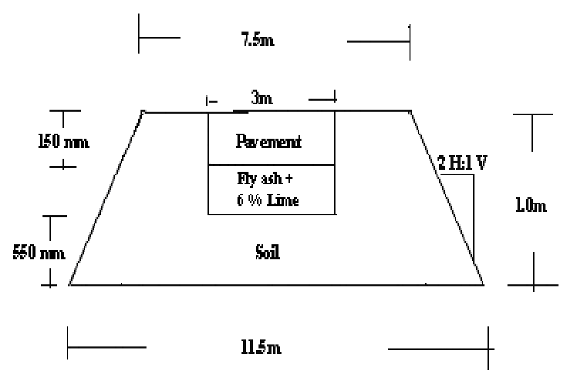
Fig 4



Typical cross section of embankment with pavement (Model-4)

- **Model-5:** Embankment constructed with earth and sub grade constructed using stabilised layer (fly ash + 6% lime) of 18% CBR. Pavement crust thickness required is 150mm (base 150mm).

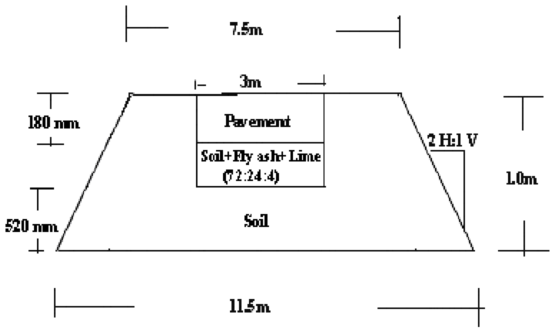
Fig. 5



Typical cross section of embankment with pavement (Model-5)

- **Model-6:** Embankment constructed with earth and sub grade constructed with soil-fly ash-lime mixture (72:24:4) of 15% CBR (sub base 30mm + base 150 mm).

Fig 6



Typical cross section of embankment with pavement (Model-6)

Only those cases have been considered where maximum use of fly ash could be achieved in comparison to the conventional soil. For comparison purpose the silty soil has been considered. These are only sample models and other models could also be prepared by using fly ash in stabilised sub base layer and in fly ash stabilised base layer.

The positive impact assessment values denote larger adverse environmental impact and the negative values denote the favourable environmental impact.

Conclusions

- Handling, transportation and storage of fly ash for fly ash based highway embankment does involve environmental impact. This impact may depend upon factors like carbon dioxide emissions, wind erosion, land degradation, health hazards, quantum utilisation, fly ash disposal procedure, technology of use, etc. All these factors have different qualitative and quantitative impact on environment.
- Environmental degradation being a broader term may not be assessable from all angles. However, for estimating the environmental impact of fly ash use in highway embankment and for getting a closer picture of impact, suitable weighted impact value may be assigned subjectively to each considered factor. A risk assessment model so prepared can give an appropriate overview of environmental impact of each project model so as to facilitate the decision making in regard to use of fly ash in embankment.
- The selected models may have global applications and the environmental risk assessment model so prepared can be applied, with or without modification, for monitoring the related impact of fly ash use in highway embankments.

References:

• Gupta P.K. 2004. Fly ash utilisation in highway embankment (Design and developmental aspects considering Indian conditions). Ph.D. thesis, Panjab University Chandigarh, Chandigarh, India.

• Lindon, K.A. Sear 2003. Paper No. 20. International Ash utilization symposium, Centre for Applied Energy Research, University of Kentucky,

Table 1. Environmental Risk Assessment Model for Fly ash use in highway Gupta 2004)

Factor	Class	weightage	Class Weightage	Final Weightage	Max. Value	Min. Value	Model No.					
							1	2	3	4	5	6
Quantum of fly ash used in Embankment (% By weight)	(70-100) (30-70) (<30)	1	1 2 3	1 2 3	3	1	3	3	2	1	3	3
Green house gases	Savings in energy Extra energy required	1	-1 1	-1 1	1	-1	-1	1	1	1	1	1
Haul distance	<5 Km 5-50 Km 50-100 Km >100 Km	8	1 2 3 4	8 16 24 32	32	8	0	16	16	16	16	16
Mode of transportation	By trucks By train By pneumatic trailer By open trailer	2	2 3 1 4	4 6 2 8	8	2	0	4	4	4	4	4
Health	Manual Handling Mechanised Handling	3	2 1	6 3	6	3	0	3	3	3	3	3
Land contamination Through non use of fly ash in embankment	(70%-100%) (30%-70%) (<30%)	4	1 2 3	4 8 12	12	4	12	12	8	4	12	12
Water contamination	Disposed in pond Used in embankment	1 0	1 0	1 0	1	0	1	0	0	0	0	0
Change in sub grade CBR	Significant decrease Small decrease Significant increase Small increase	6	2 1 -2 -1	12 6 -12 -6	12	-12	0	0	-6	-12	-12	-6
Psychological barriers	Full motivation and awareness Less motivation and awareness	3	-1 1	-3 3	3	-3	0	-3	-3	-3	-3	-3
Total	-	-	-		78	3	15	36	25	14	24	30

TECHNO-ECONOMIC EVALUATION OF FLY ASH BASED SUBGRADE

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ABSTRACT

The top 30 cm to 50 cm layer of the embankment acts as sub grade for pavement. In rural roads the top 30 cm of the cutting or embankment at the formation level shall be considered as sub grade. The embankment may be constructed with conventional locally available suitable soil, or with fly ash in the core with appropriate soil cover over top and sides, or with soil and fly ash blend material, or with one of the above and with modified sub grade so as to improve its CBR value.

Sub grade with Increased CBR value will require thinner flexible pavement for the similar other design parameters. It is possible that the cost of sub grade modification is lesser than the cost of extra pavement thickness required for unmodified sub grade.

Introduction

The ultimate purpose of all highway embankments is to support the pavements for carrying traffic. The top 30 cm to 50 cm layer of the embankment acts as sub grade for pavement. In rural roads the top 30 cm of the cutting or embankment at the formation level shall be considered as sub grade (IRC: SP:20-2002). For Expressways, National Highways and for Major District Roads, the top 50 cm portion of the roadway is considered as sub grade (IRC: 37-2001). The embankment may be constructed with conventional locally available suitable soil, or with fly ash in the core with appropriate soil cover over top and sides, or with soil and fly ash blend material, or with one of the above and with modified sub grade so as to improve its CBR value.

Sub grade with Increased CBR value will require thinner flexible pavement for the similar other design parameters. It is possible that the cost of sub grade modification is lesser than the cost of extra pavement thickness required for unmodified sub grade. It will also save the scarce natural materials for other construction purposes. If the CBR value of the unmodified sub grade
is low then the sub grade improvement has larger impact on pavement thickness.

As an example if the CBR value of the soil can be improved from 2 % to 6 %, the resulting saving in rural road pavement thickness for CBR curve C will be (595-325)=270 mm. For rural road considering pavement width of 3.75m, the total saving in paving material per kilometer will be more than (3.75 x 1000 x 0.27) = 1013 m^3. For Punjab if the sub base material is within 2 km, its cost per m^3 may be around Rs 80 and if the lead is 200 km, its cost may be around Rs 400 per m^3. Thus saving in cost of the pavement per km may be between Rs 81000 to Rs 405000. Net saving will depend upon the cost of soil sub grade improvement cost.

Depending upon the quality of soil and that of fly ash, the improvement can be in the form of soil + fly ash, soil + lime, fly ash + lime or fly ash + lime + soil. All these combinations will have different cost and proportions for the same amount of improvement. And at the same cost, all are likely to have different strength. Out of these the soil is generally available within 15m to 5 Km in rural area, and in urban area it may available from outside the city limits. It is generally available free and has only handling and transportation costs. The lime is required in considerably less quantity compared to the other components of the sub grade and its purchase price is its main cost.

Thus for the use of fly ash in embankment construction and sub grade improvement, the viability and its mode of use will be site specific, and its usefulness will depend upon many factors like lead of sub grade materials, royalty, characteristics of each material to be used, their mix proportions, environmental regulations, lead of sub base materials, awareness of potential use of fly ash, and the willingness of the user agencies to adopt new and innovative technologies.

Laboratory Tests For Pond Ash use in Highway Embankments

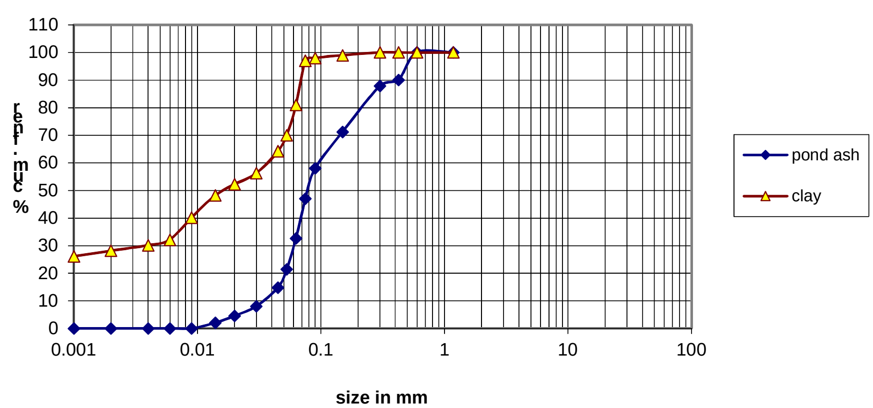
The highway embankments may consist of conventional material like soil, coal ash (fly ash, bottom ash, pond ash) in the core with be sufficient soil cover on sides and on top. Similarly the sub grade material in the embankment may consist of only soil, only coal ash, soil + lime, coal ash + lime, soil + coal ash or soil + coal ash + lime.

Pond ash (a mixture of around 80% fly ash and 20% bottom ash) and which is the most easily available type of coal ash from thermal power plants, has been taken from the Guru Govind Singh Thermal Power Plant, gate No-9, Ropar and locally available soil samples have been used for testing. The properties such as grain size, OMC (optimum moisture content), maximum dry density, Atterberg's limits, CBR (California bearing ratio) values, shear strength parameters and permeability have been evaluated as follows:

Grain Size Analysis

Figure 1 shows the grain size analysis curve of the materials used. The tests have been carried out as per IS: 2720 (Part 4)-1985.

Fig.1. grain size analysis curve for different soils



Liquid Limit and Plastic limit

The liquid limit of clay was 32.5% and plastic limit was 21.92%. The tested sample of clay falls under CL group of BIS (Bureau of Indian Standards) soil classification system. The Table 1 shows liquid limit and plastic limit test results of the tested materials.

Table: 1

Liquid limit and plastic limit test results (IS: 2720 Part-5)

S. No.	Type of soil	Liquid limit (LL) %	Plastic Limit (PL) %	Plasticity Index (PI) %
1.	Pond ash	NP	NP	-
2.	Clay	32.5	21.92	10.58

Optimum Moisture Content

These tests have been carried out as per heavy compaction method (IS: 2720 Part- (8)-1983), in which the soil sample is compacted, using the same mould as used for light compaction test, in five layers each layer being given 25 blows.

Figure 2 shows the moisture density relationship curve for each type of embankment material. Table 2 shows the compaction test results for clayey soil and for pond ash. Table 3 shows the compaction test results for clayey soil mixed with different proportion of pond ash. Table 4 shows the compaction test results of each type of soil mixed with fixed quantity of lime and with varying proportion of pondash

Table: 2

Modified Proctor Compaction Test Results

Pond Ash					
Moisture content (w) %	15.38	17.86	23.08	27.14	31.08
Dry Density ((Y_d) gm/cc	1.14	1.18	1.216	1.17	1.12
Clay					
Moisture content (w) %	13.89	16.67	17.2	18.45	22.12
Dry Density ((Y_d) gm/cc	1.833	1.862	1.88	1.857	1.70

Fig 2 Compaction Curve for different materials

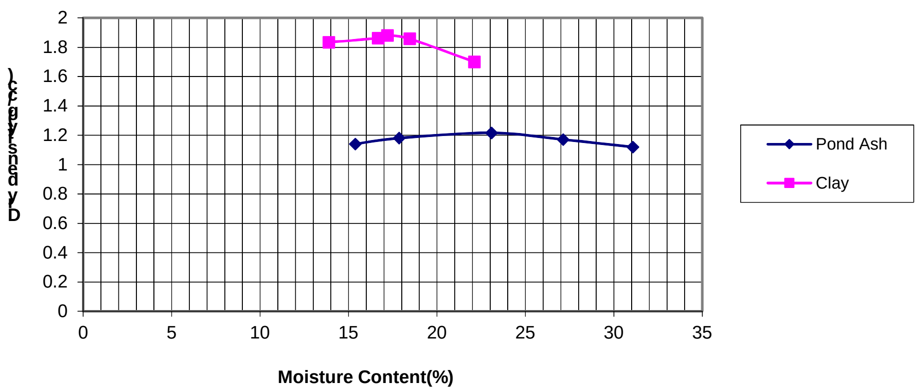


Table: 3

Compaction test results for different proportion of clay + Pond ash

S.No.	Type of soil	(Soil +pond ash) %	γ_d^{max} g/cc	OMC %
1.	Clay	80+20	1.74	18.37
2.		70+30	1.68	18.96
3.		60+40	1.61	19.55
4.		50+50	1.55	20.14

Table: 4

Compaction test results for different proportions of Lime+pond ash+clay

S.No.	Type of soil	(Soil+ pond ash +lime)%	γ_d gm/cc	OMC %
1.	Clay	80+15+5	1.68	17.22
2.		70+25+5	1.62	17.81
3.		60+35+5	1.55	18.39
4.		50+45+5	1.49	18.98

California Bearing Ratio Test (CBR)

In this study, the various embankment materials like clayey soil, fly ash (pond ash) have been tested for CBR, individually as well as with their different mixture proportions, with or without addition of lime .The CBR test was conducted as per IS: 2720-(Part-16). Static compaction method was employed to achieve the density as per heavy compaction method for the specimen sample remoulded at corresponding OMC. In case of different soils and pond ash mixed with varying percentage of lime, damp sand was placed in the space between the slotted weights, moulds were covered with wet gunny bags and samples were allowed to cure for 3 days (IRC: 51-1992). A small quantity of water was sprinkled everyday to keep the sand moist. After three days curing, samples were placed in a water tank and allowed to remain fully immersed in water for 4 days. Table 5 shows the consolidated CBR values of clayey soil with or without different admixture proportion of lime and pond ash.

Interpretation Of Various Test Results

a) From Table 3, it is clear that increasing addition of fly ash to clayey soil decreases the standard maximum dry density and increases the OMC. This may be due to lesser density of pond ash and due to greater surface area of pond ash.

As per Figure3, the addition of 2% lime has increased the CBR value of clay from 1.9% to 12%. The increase in clay may be mainly due to better compaction achieved due to decreased plasticity.

b) As per Figure 3, with the addition of 6% or more lime to clay the CBR value obtained was more than 20%. As per rural road manual of IRC (Indian Roads Congress) on such improved sub grade, there is no need to provide sub base especially in low volume roads. The reduction in pavement thickness required may be up to 425 mm depending upon the original soil CBR and traffic conditions. The approximate quantity of lime required per kilometre for improving 300 mm thick sub grade for 3m wide pavement section would be around 97 tones (=0.3 x 1000 x 3 x 1.8 x 6/100) costing around Rs 2,91,000 (=Rs 97 x 3000). This may translate into around 3 lakhs including lime-soil blending cost. The cost of compacted sub base material in Punjab may vary from Rs 270/cu.m. to Rs 540/cu.m depending upon its haulage distance. Thus the total reduction in sub base cost may vary up to, from Rs 3,44,250 (=270 x 3 x 1000 x 0.425) to Rs 6,88,500 (=540 x 3 x 1000 x 0.425). Thus

c) wherever fly ash is being used as fill
d) from thermal power stations, the top 30 cm of fly ash embankment below the pavement can be stabilised with suitable proportion (say 6%) of lime to offset the fly ash haulage cost through reduction in required pavement cost.
e) Figure 4 show that optimum percentage addition of pond ash, based upon CBR value improvement, to clay was 30%. Thus in fly ash embankment, the blending of local clay soil with optimum quantum of fly ash alone in sub grade layer only, may improve its strength. This may provide reduction in required pavement thickness without any significant increase in cost.
f) Figure 2 show a peculiar fall in plunger resistance of CBR test and its subsequent regain in clay-pond ash blend mixed with fixed, near optimum, percentage of lime. This may be due to material especially at longer distances breaking of cemented bonds in the said mixture. However the regain in strength is less than earlier strain resistance.

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Appendix-A

Table: 5

Consolidated CBR test results for each specimen type (Unless specified the test results are under Relevant IRC standard soaked conditions)

S.No.	Type of test material	CBR (%)
7	C (Un)	9.2
8	C (S)	1.9
9	C + 2% L	12.0
10	C + 4%L	15.5
11	C + 6%L	22.4
12	C + 8%L	23.2
25	C (.8)+Pa (.2)	3.1
26	C (.7)+Pa (.3)	3.8
27	C (.6)+Pa. (.4)	1.4
28	C (.5)+Pa. (.5)	1
37	C (.8)+Pa(.15)+L(.05)	14
38	C (.7)+Pa(.25)+L(.05)	16
39	C (.6)+Pa(.35)+L (.05)	17
40	C (.5)+Pa(.45)+L (.05)	17

AN EXPERIMENTAL STUDY ON USE OF HYPO SLUDGE IN CEMENT CONCRETE

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ABSTRACT

For sustainable development, among others, protection of environment is one of the major challenges. In order to reduce the consumption of natural resources, which are limited, use of industrial wastes to replace fresh materials is one of the viable options to control degradation of environment.

Millions of tonnes of solid industrial wastes are produced in India every year. A large quantity of this is contributed by paper industry where three kinds of wastes are generated, i.e. , fibrous sludge called reject which is bio-degradable, Hypo sludge, solid wastes generated during calcium hypo chlorite generation and Lime sludge, generated during causticisation of green liquor. The hypo sludge and lime sludge are purely chemical wastes and require large spaces of landfill for their disposal. Limited land fill sites augment the disposal problems of these wastes.

Use of these wastes in cement concrete can not only solve the problem of their disposal but economize the concrete by partially replacing cement. Response to various loads and durability of such concretes might be prime concern of construction engineers, structural designers and owner of the structure. Therefore, a scientific experimental study of such concretes is inevitable.

This paper presents the test results of chemical and physical analysis investigating the utilization of hypo sludge, lime sludge and fly ash in cement concrete. Chemical analyses have been conducted, to evaluate optimum proportions of these materials to be used in concrete following direct enumeration method. Study shows some important parameters such as workability, cube strength, stress-Strain characteristics (cylinder), modulus of elasticity and failure patterns for M20 (1:1.5:3) mix with 10%, 20%, 30%, 40% cement replacement with hypo sludge and comparison with that of conventional cement concrete.

Keywords: Sustainable Development, Hypo sludge, Lime sludge, Fly ash.

I. INTRODUCTION:

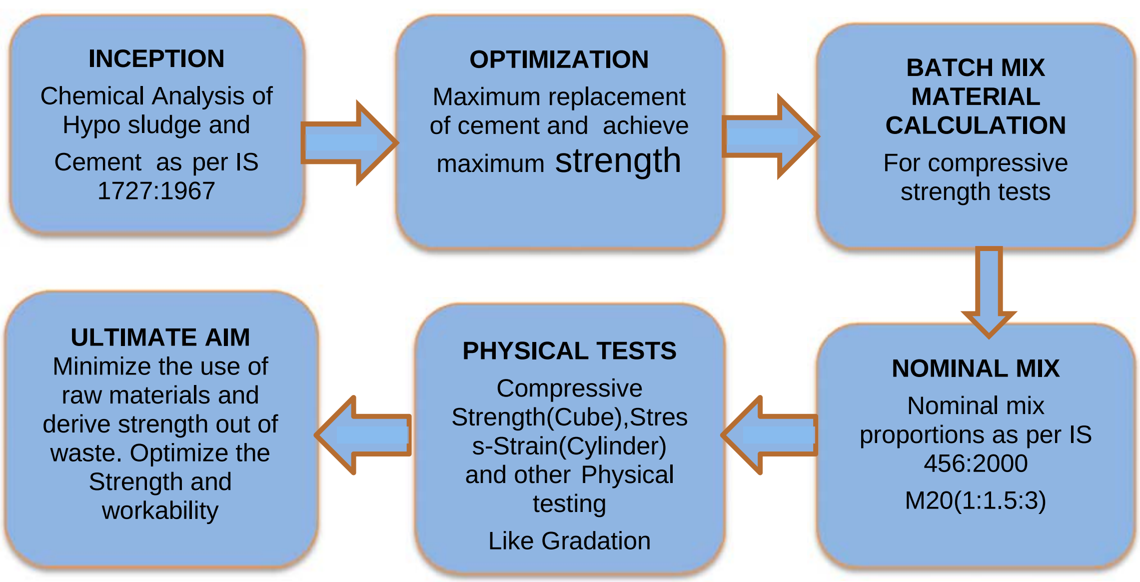
Natural resources are not unlimited therefore, they must be optimally consumed. This shall help not only to control degradation of environment but conserve them also for the use of future generation. This can be achieved by the process of recycling and, making use of industrial wastes, disposal of which otherwise is a serious problem. Hypo sludge is such an industrial waste produced in plenty by paper mills. Construction industry is found to be apprehensively reluctant to use wastes for making concrete mixes. This paper presents the physical and chemical analysis of hypo sludge and its use in cement concrete as a partial substitute of cement which economizes the cost of concrete. Objective of this experimental study was to find out the important parameters such as compressive strength, modulus of elasticity, strain at maximum load and ultimate strain of cement concrete in which hypo sludge replaced the cement by 10, 20, 30 and 40 percent for certain slump workability and to find out the optimum percentage of hypo sludge in M20 nominal mix concrete. 100 mm slump has been considered for different replacement levels of cement. Two decision variables that are strength and workability had been considered to optimize the replacement percentage.

II. SOURCE OF HYPOSLUDGE:

The process of formation of paper from pulp includes the following processes during which the Hypo sludge is formed as waste by-product is purely a chemical wastes and do not contain any bio-degradable element**.** Most of the mills are using only woody raw material (bamboo, eucalyptus, casuarinas, poplar and other hardwood species), but some other mills are using bagasse in substantial quantity as raw material.

Most of the paper mills in India prepare bleach liquor (calcium hypochlorite) using lime and elemental chlorine. Six mills among eight mills are using ClO_2 as bleaching agent either as partial substitution of elemental chlorine or in final stage of bleaching to attain desired brightness level. These mills are producing ClO_2 with environmental friendly process. Three mills among eight mills are still using calcium hypo chlorite in final stage for bleaching. Solid wastes generated during calcium hypo chlorite generation are called hypo sludge.

III. METHODOLOGY



IV. CHEMICAL ANALYSIS

IS-1727:1967 “Test Methods for Pozzolanic Materials” had been followed for chemical analysis of the hypo-sludge and cement. Table 1 gives the results of chemical analyses.

Table 1: Results of Chemical Analysis

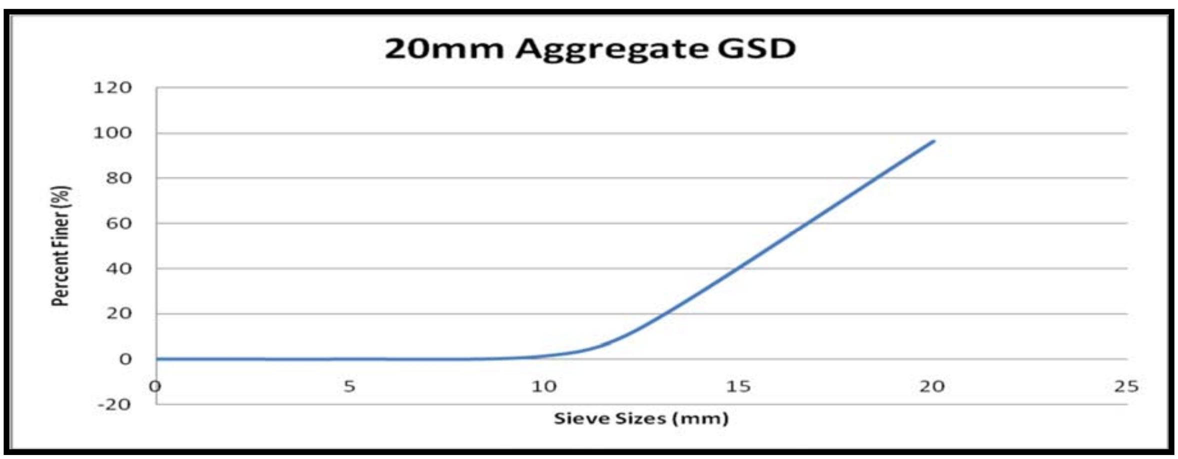
S. No.	Constituent	Hypo sludge (%)	Cement (%)
1.	**Silica**	3.0175	18.175
2.	**R_2O_3**	9.64	7.16
3.	**Calcium oxide**	43.17	63.26
4.	**Magnesia**	2.85	3.07
5.	**Loss of ignition**	1.78	0.57

➢ Inference From Chemical Analysis:

Following inference can be drawn from the results of chemical analyses of cement and hypo-sludge.

- The Hypo sludge procured seems to be a comparable material to cement on the basis of above tabled parameters which are responsible for the development of strength in cement.
- The silica, calcium oxide, alumina, ferric oxide and magnesia content in hypo sludge are very desirable.
- Hypo sludge is the most suitable material as a replacement of cement.

V. GRADATION



(GSD= GRAIN SIZE DISTRIBUTION)

Table 2A: Sieve Analysis of 20 mm aggregate (Weight of sample taken **= 2 Kg)**

Sieve Size (mm)	**Weight Retained (Kg)**	**Percent Weight Retained (%)**	**Cumulative Percent Weight Retained(%)**	**Percent Passing or Finer (%)**	**Fineness Modulus (FM)**
20	0.074	3.7	3.7	96.3	
12.5	1.65	82.5	86.2	13.8	
10	0.249	12.45	98.65	1.35	**FM = Σ Column 4 / 100**
4.75	0.027	1.35	100	0	
2.36	0	0	100	0	
1.18	0	0	100	0	
0.6	0	0	100	0	
0	0	0	100	0	= 6.89

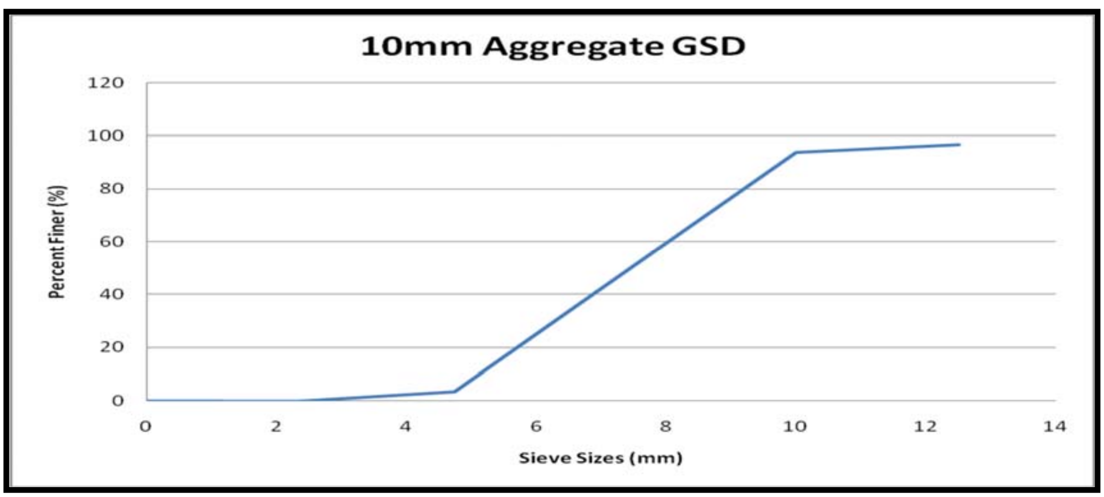


Table 2B: Sieve Analysis of 10 mm aggregate (Weight of sample taken **= 2 Kg)**

Sieve Size (mm)	Weight Retained (Kg)	Percent Weight Retained (%)	Cumulative Percent Weight Retained(%)	Percent Passing or Finer (%)	Fineness Modulus (FM)
12.5	0.062	3.1	3.1	96.9	
10	0.056	2.8	5.9	94.1	
4.75	1.81	90.5	96.4	3.6	**FM = Σ Column 4 / 100**
2.36	0.072	3.6	100	0	
1.18	0	0	100	0	
0.6	0	0	100	0	
0.3	0	0	100	0	
0	0	0	100	0	= 6.05

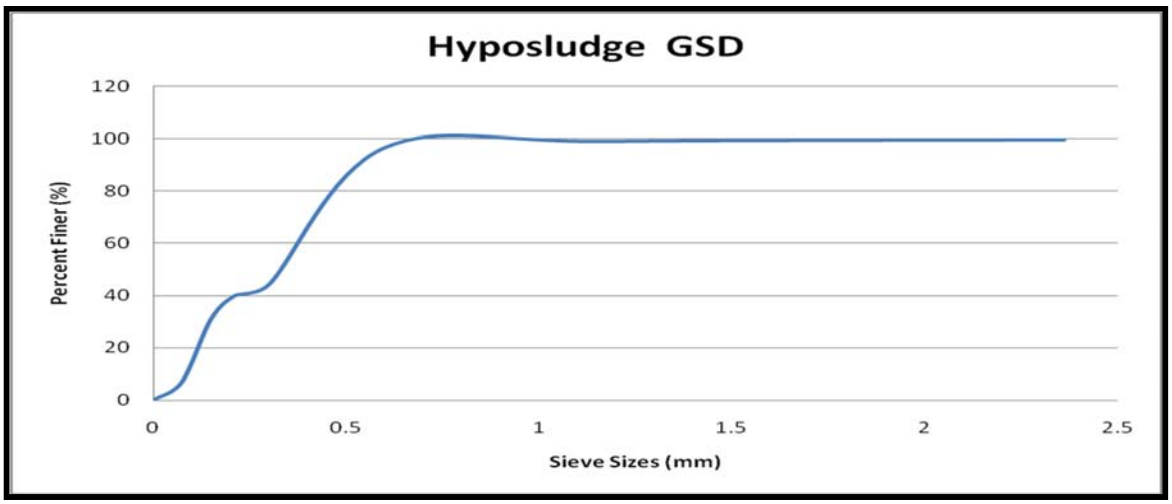


Table 2C: Sieve Analysis of Hypo sludge (Weight of sample taken **= 1Kg)**

Sieve Size (mm)	Weight Retained (Kg)	Percent Weight Retained (%)	Cumulative Percent Weight Retained(%)	Percent Passing or Finer (%)	Fineness Modulus (FM)
2.36	0.002	0.2	0.2	99.8	
1.18	0.006	0.6	0.8	99.2	**FM = Σ Column 4**
0.6	0.024	2.4	3.2	96.8	**100**

0.3	0.526	52.6	55.8	44.2	
0.212	0.042	4.2	60	40	
0.15	0.09	9	69	31	
0.075	0.24	24	93	7	
0	0.07	7	100	0	= 3.82

VI. BATCH MIX MATERIAL QUANTITY CALCULATION - M20 (1:1.5:3)

Table 3: Quantity Calculation for Batch

Material	Weight (Kg)	Batch	Water-Cement ratio (w/c)	Plasticizer percentage by weight of cement (%)	Mix Proportions	Slump (mm)
Cement	26.498	**0%**	**0.55**	**0**	**1 : 1.5 : 3**	**100**
Water	14.5739					
Sand	39.747					
20mm	39.747					
10mm	39.747					
Hyposludge	0					
Cement	23.85	**10%**	**0.55**	**0.8**	**1 : 1.5 : 3**	**100**
Water	14.5739					
Sand	39.747					
20mm	39.747					
10mm	39.747					
Hyposludge	2.651					
Cement	21.196	**20%**	**0.55**	**0.9**	**1 : 1.5 : 3**	**98**
Water	14.5739					
Sand	39.747					

20mm	39.747					
10mm	39.747					
Hyposludge	5.306					
Cement	18.557	**30%**	**0.55**	**1.0**	**1 : 1.5 : 3**	**97**
Water	114.574					
Sand	39.747					
20mm	39.747					
10mm	39.747					
Hyposludge	7.952					
Cement	15.900	**40%**	**0.67**	**1.0**	**1 : 1.5 : 3**	**100**
Water	17.507					
Sand	39.747					
20mm	39.747					
10mm	39.747					
Hyposludge	10.598					

Figure 4: Cylindrical Specimen being tested in compression testing machine after being capped using artite powder

VII. OBSERVATION TABLE

Table 4A: 7 day Cube Strength

S.No.	Cement Replacement Percentage (%)	Sample	Load (kN)	Mean Maximum stress (MPa)
(1)	0	M20-0%	435.94	19.30
(2)	10	M20-10%	499.30	22.16
(3)	20	M20-20%	433.00	19.24
(4)	30	M20-30%	334.47	14.86
(5)	40	M20-40%	167.40	7.45

Table 4B: 28day Cube Strength

S.No.	Cement Replacement Percentage (%)	Sample	Load (kN)	Mean Maximum stress (MPa)
(1)	0	M20-0%	434.22	19.37
(2)	10	M20-10%	586.3	25.52
(3)	20	M20-20%	528.03	23.45
(4)	30	M20-30%	350.87	15.59
(5)	40	M20-40%	220.17	9.71

Table 4C: 7 **day** Cylinder Strength

S.No.	Cement Replacement Percentage (%)	Sample	Load (kN)	Mean Maximum stress (MPa)
(1)	0	M20-0%	270.17	15.28
(2)	10	M20-10%	389.40	22.04
(3)	20	M20-20%	274.03	15.48
(4)	30	M20-30%	218.60	12.36
(5)	40	M20-40%	101.30	5.73

Table 4D: 28 day Cylinder Strength

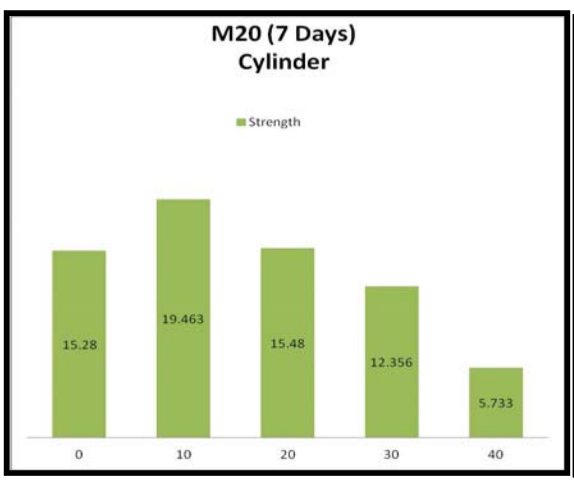
S.No.	Cement Replacement Percentage (%)	Sample	Load (kN)	Mean Maximum stress (MPa)	Mean Tangent Modulus (MPa)
(1)	**0**	M20-0%	**338.90**	**19.18**	$2.3950*10^4$
(2)	**10**	M20-10%	**344.13**	**19.46**	$2.4150*10^4$
(3)	**20**	M20-20%	**367.10**	**19.48**	$2.4121*10^4$
(4)	**30**	M20-30%	**251.30**	**14.20**	1.9040*104
(5)	**40**	M20-40%	**138.20**	**7.82**	2.4150*104

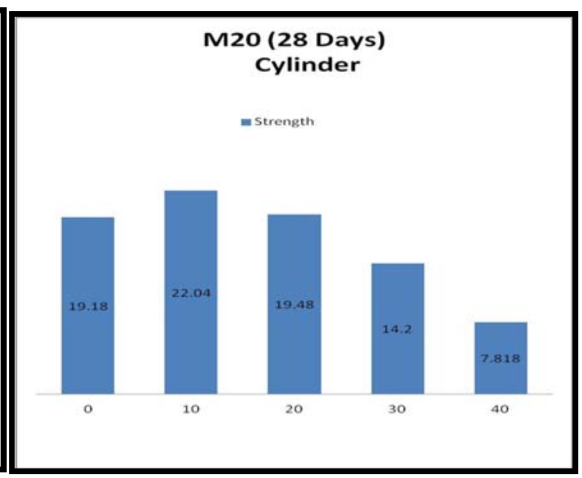
VIII. RESULTS:

Table 5: Results of the experimental study conducted is summarized in table below

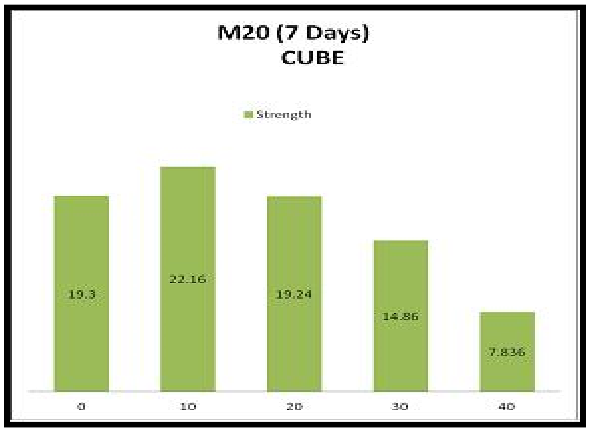
AGE OF SPECIMEN	**STRENGTH (MPa)**				
	0 %	**10%**	**20%**	**30%**	**40%**
7 Day Cube	**20**	**22.2**	**19.24**	**15**	**8**
28 Day Cube	**23.5**	**26.05**	**23.46**	**15.6**	**9.8**
7 Day Cylinder	**15.28**	**19.463**	**15.48**	**12.356**	**5.733**
28 Day Cylinder	**19.18**	**22.04**	**19.48**	**14.2**	**7.818**
PLASTICIZER	**0**	**0.8%**	**0.9%**	**1%**	**1%**

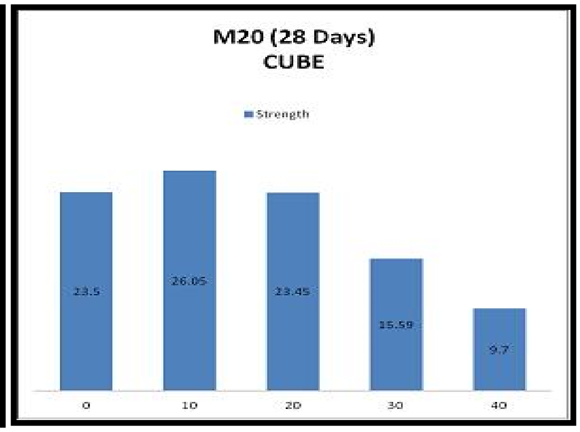
- **Comparison of Cylinder Strength (M 20)**





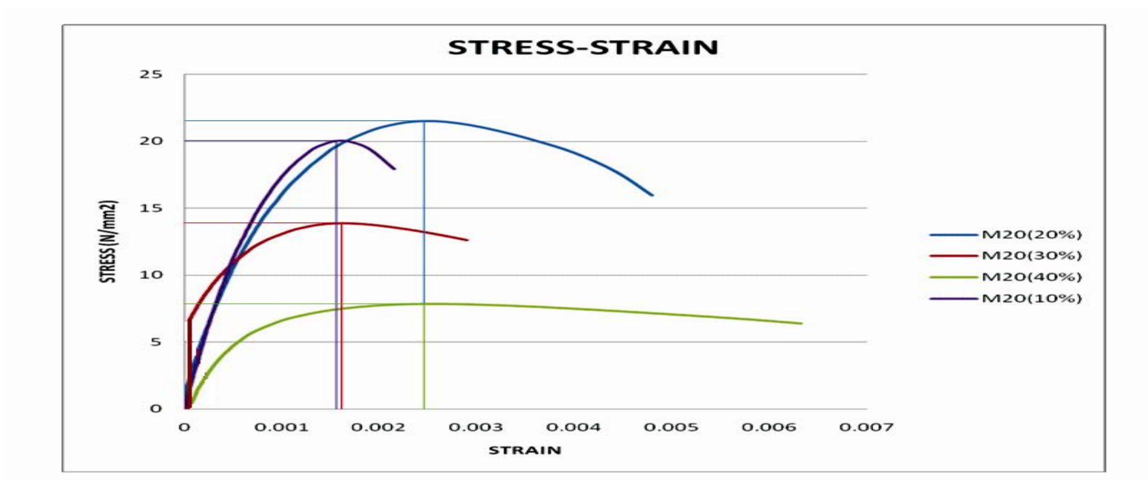
- **Comparison of Cube Strength (M 20)**





- The comparison of cube and cylinder strengths shown above for different cement replacement levels by hyposludge are having units **MPa** or **N/mm²**

- **Comparison of Stress-Strain Curves for Cylinder (M 20)**



- **Cost Analysis (M20 (1:1.5:3))**

  Cost of one bag of cement (50kg) = 300 Rs.

  One cubic meter of concrete contains = 436 kg of cement

  Percentage of hypo-sludge will be used = 20%

  Quantity of hypo-sludge will be used in one cubic meter of concrete = 87.2 kg

  Since the cost of hypo-sludge is negligible, so it can widely be used in areas near by paper mill.

  Amount of money saved in one cubic meter of concrete = Rs 523

IX. CONCLUSIONS: For M20 (nominal mix) concrete

- **Workability:**
  - Experiments reveal that as the percentage of hypo-sludge in the mix increases the slump decreases.

- **28 day Cube strength:**
  - 20% replacement of cement by hypo-sludge gives as much strength as pure cement concrete.
  - 10% replacement of cement by hypo-sludge gives about 11% higher strength than pure concrete.
- **Hypo-sludge** which is available in abundance in the vicinity of every paper mill is not a waste but a useful material which can be used in concrete manufacturing. Its application in concrete manufacturing:
  - Helps to some extent in preserving the environment as its application reduces the requirement of cement's raw material.
  - Solves its problem of disposal.
  - Economises the cost of concrete.

X. BIBLIOGRAPHY

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Fig 3 Variation in CBR for clayey soil with different proportion of lime

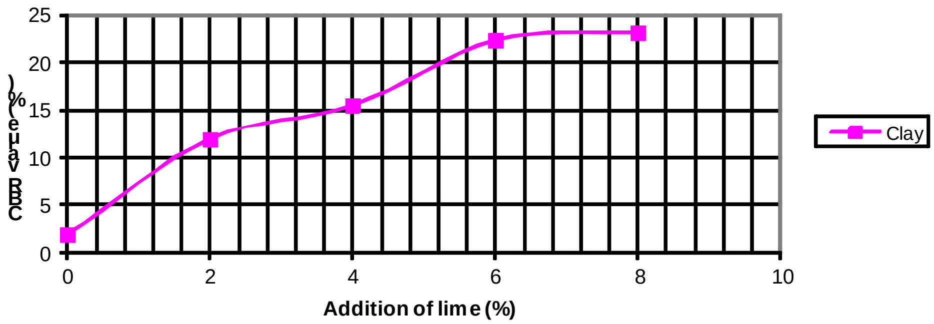


Fig.4 Variation in CBR value of clayey soil with different proportion of pond ash

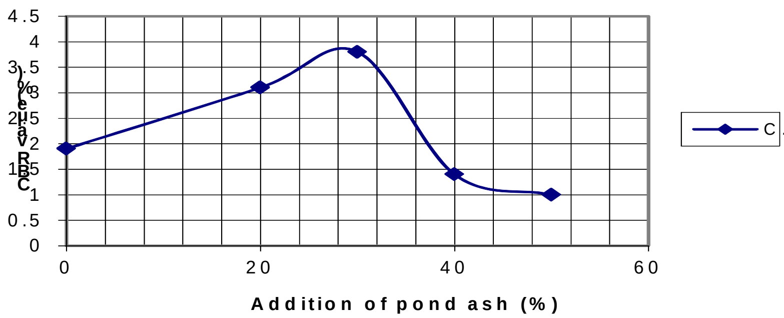
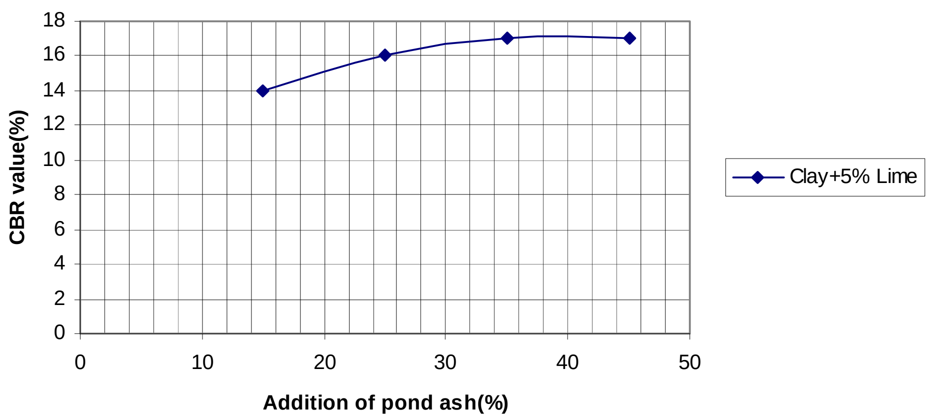


Fig.5 Variation In CBR value of clayey soil mixed with fixed percentage of lime against varying proportion of pond ash



EXPERIMENTAL STUDIES ON CONCRETE REPLACING FINE AGGREGATE WITH QUARRY DUST WASTES

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Abstract
Concrete is the most undisputable and indispensable material being used in infrastructure development throughout the world. Many investigations were done to produce various varieties of concrete by reusing waste materials. Fine Aggregate (FA) plays a major role in making of concrete and the availability of fine aggregate is dwindling due to it's over explosion throughout the world. In this context an experimental studies was done based on Quarry Dust Wastes (QDW) for replacement of fine aggregate in concrete. Compressive strength tests were carried out with different proportions of replacements of FA with QDW. It was observed that on 75% substitution of FA with QDW the compressive strength of concrete was within permissible limit for use in construction industry.

***Key words**: - Pozzolana Portland Cement, Quarry Dust Waste, Concrete, Compressive strength.*

I. Introduction

Explosion in population coupled with rapid industrialization and subsequent demand for infrastructure facilities has ushered a need to provide the necessary civil engineering structures. As the resources available for construction are limited, there is a need to go for some alternatives and use of industrial wastes appears to be an attractive option. Several waste materials such as spent fire bricks and recycled aggregate were used as replacements to concrete with varying compressive strength values

S. Keerthinarayana et.al.,[2] had studied the strength and durability properties by partial replacement of fine aggregate with crushed spent fire bricks and have reported an increase in the compressive strength with the partial replacement of CFBS. Malek Batayneh et.al.,[3] have successfully demonstrated the application of demolished concrete, glass and plastics as partial substitutes to concrete. J. Selwyn Babu et.al.,[9] had studied the physical and mechanical properties of concrete, replacing fine aggregate with GGBFS and BFS.

Quarry Dust Waste (QDW) has the same phycial characteristics of fine aggregate, as its size and properties are very to sand. In this investigation it is proposed to utilize Quarry Dust Waste (QDW) as replacement in the fine aggregate in different proportions.

II. Materials and Methods

A. Cement

In this present investigation Pozzolana Portland Cement (PPC) was used.

B. Fine Aggregate (FA)

The sand used for the experimental procedure was locally procured from a river and confirmed to Indian Standard Specifications [4]. It was passed through a 4.75 mm sieve, washed to remove any dust and then used as it was for further investigations.

C. Coarse Aggregate (CA)

Broken granite stones are generally used as a Coarse Aggregates (CA). The nature of work decides the maximum size of the CA. Locally available CA having the maximum size of 20 mm was used in our work. The aggregates were washed to remove any dust and were dried. The aggregates were tested as per Indian Standard Specifications [4].

D. Water

Water is an important ingredient of concrete s it actually participates in the chemical reaction with cement. Since it helps to form the strength giving cement gel, the quantity and quality of water required is to be looked into very carefully. In practice, very often great control on properties of cement and aggregate is exercised but the control on the quality of water is often neglected. So quality of water is checked to its purity.

E. Quarry Dust Wastes (QDW)

The QDW is the by product which is formed in the processing of the granite stones. QDW has the same physical characteristics to sand. It was made to pass through a 4.75 mm sieve [4], washed and used for further studies.

F. Methodology

Cement used for the study was tested for the parameters, Fineness, Consistency, Initial & Final Setting times and Specific Gravity [6]. Aggregates were tested for Fineness Modulus, Specific gravity, Water Absorption [5], Bulk density and Moisture content as per IS codes. Concrete was tested for Compressive strength under five cases as per M 25 mix design. In case-A, conventionally used Cement, Fine Aggregate, Coarse Aggregate and Water were mixed and analyzed for strength parameters. In case-B, fine aggregates was completely replaced by QDW and the other ingredients were the same as in case-A. In case-C, 50% fine aggregates and 50% QDW were used and the other ingredients were the same as in Case-A. In case-D, 75% fine aggregates and 25% QDW were used and the other ingredients were the same as in case-A. In case-E, 25% fine aggregates and 75% QDW were used and the other ingredients were the same as in case-A.

III. Mix design

As per the code IS: 10262 –1979 [8], the mix design is found and the amount of materials is calculated. According to the mix ratio, the amount of materials is given below, in Table I.

TABLE I. MIX PROPORTION

Water	Cement	Fine Aggregate	Coarse Aggregate
150	300	200	265
0.5	1	1.77	2.88

IV. Results and Discussion

The various results of tests done for cement are presented in Table II. All the parameters were observed to be within the permissible limits, though the initial setting time was found to be at a slightly upper level.

TABLE II. RESULTS OF TESTS DONE ON CEMENT

Type of tests	Results
Fineness	0.9%
Consistency	30.5%
Initial Setting Time	30 min
Final Setting Time	350 min
Specific Gravity	2.64

The results of tests done on aggregates are presented in Table III, and all the parameters were within the permissible limits.

TABLE III. RESULTS OF TESTS DONE ON AGGREGATES

Type of Tests	CA	FA	QDW
Specific Gravity (%)	2.72	2.65	2.56
Water Absorption (%)	0.5	1.0	.5
Bulk Density (kg/m^3)	1469.8	1460	1765
Fineness Modulus (%)	4.51	3.54	3.81
Moisture content (%)	1.90	1.50	Nil

The results of Compressive strength test on four cases are presented in Table IV**.** It was observed that concrete of all the five cases exhibited good compressive strength.

TABLE IV. RESULTS OF COMPRESSIVE STRENGTH ON CONCRETE

Type of Concrete	Compressive strength of cubes, MPa	
	7 days	28 days
Case-A	28.50	36.05
Case-B	27.32	37.98
Case-C	24.16	33.80
Case-D	26.81	26.04
Case-E	22.52	34.10

A graphical comparison of 7 day compressive strengths of all the five cases of mix designs is presented in figure 1. It can be observed that case-B exhibited a compressive strength of 27.32 MPa, which is close to the conventionally used case-A mix design. This is far better than the replacement of fine aggregate with 15% Ground Granulated Blast Furnace Slag which gave a 7 day compressive strength of 22.82 MPa [9]. Case-E obtained a compressive strength of 22.52 MPa which is similar to 10% replacement of Quarry dust [10] for 7days.

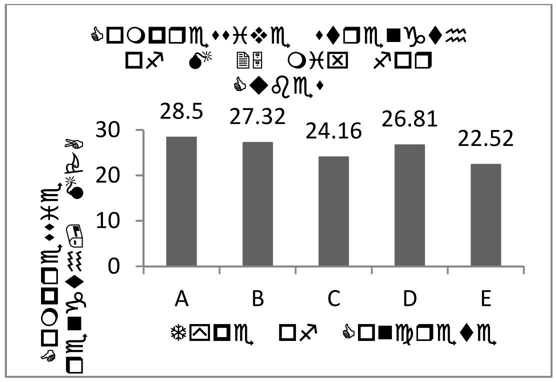

Figure 1. Compressive strength of 7 days for cube

Figure 2. depicts the 28 day compressive strengths of five cases of mix designs. In case-B which was replaced by QDW the 28 day compressive strength was observed to be 37.98 MPa which is quite a satisfactory value. This is very close to the replacement of fine aggregate with 20% Blast Furnace Slag which gave a 28 day compressive strength of 36.47 MPa [9]. Case-E obtained a compressive strength of 34.10 MPa for 28 days which is similar to 10% replacement of Quarry dust [10] for 27 days.

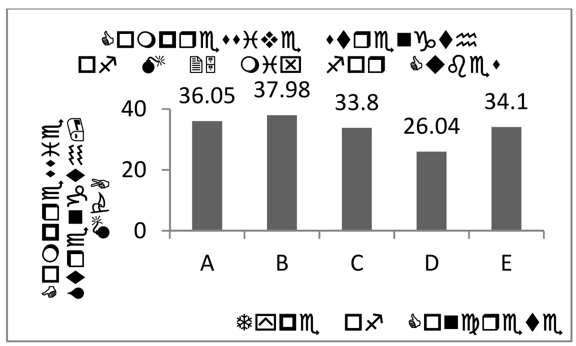

Figure 2. Compressive strength of 28 days for cube

V. Conclusion

The following conclusions were drawn from the experimental investigation.

1. A maximum compressive strength of 37.98 MPa was obtained with 100% replacement of QDW for 28 days.

2. It was observed that 100% replacement of QDW gave a satisfactory result when compared with conventional concrete for 7 days.

3. From this test, replacement of fine aggregate with QDW gave a compressive strength of 50% and 75% replacement respectively.

4. Partial replacement of FA with QDW showed encouraging results in terms of compressive strength.

5. Environmental wastes which pose a difficult problem in its disposal can be efficiently addressed through the results of this research.

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IMPORTANCE OF ROADSIDE VEGETATION

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ABSTRACT

Roads are the integral part of transportation system. It plays a significant role in achieving national development and with the help of road side vegetation and by selecting right species of plant at right area we can reduce the maintenance needs and cost of road, provides safety for vehicles, improves the overall driving experience of roads, reduce soil erosion.

Enhance the drainage aspect of roads as vegetation increase the water infiltration capacity of soil, improves the shear strength of embankments by controlling the moisture content and increase the life of shoulder. Beside all these factors vegetation also cover the environmental aspect such as control noise pollution, air pollution and maintains the ecological balance and aesthetic view.

1. INTRODUCTION

As with growing time government of every country wants the best & economic technique should be adopted in each part of the country & an attempt is made by the engineers to find the alternatives of each technique. The Roadside vegetation or bio engineering is a technique through which the life of road can be increased by controlling the moisture content of soil, by improving shear strength of soil, by improving infiltration capacity of soil & by controlling soil erosion. Through this technique we can also reduce the cost of construction, maintenance cost of roads.

The road side vegetation technique or bio engineering technique requires assessment of existing road condition determination of type of roadside environment desired according to increased public demand and customer expectations. There are various factors on which vegetation techniques depend:

Soil conditions; Traffic Composition;
Location of road; Topography; Adjacent

Land Use; The Priority of Road; Aesthetic appearance

It is a rapidly growing field subject to innovations & changing design specifications. Due to increased environmental awareness this technique is beneficial than traditional approaches.

2. BENEFITS OF VEGETATION

2.1 ECONOMIC ASPECTS

- Improved Road side conditions enhance the visitor visit.
- Reduce cost of construction activities due to less requirement of improved technology.
- Also improves life of pavement. (This technique can be used in soil stabilization situations)
- It also reduces maintenance cost and needs.
- It improves water infiltration capacity of soil & reduces run off.
- The roots, stems & associated woods that we obtained from cutting are used to build the structures.
- Traditional method of controlling stream flow & wave induced erosion on embankment have relied on structural practice like retaining wall & sheet piles which are expensive, ineffective whereas Bio engineering technique as one of best economic alternative approach.

2.2 ENVIRONMENTAL ASPECTS

- It improves air quality by absorbing carbon monoxide, and carbon dioxide.
- It also stabilizes the ground surface to prevent soil erosion as with time the strength of root system increases which increase the soil stability and the soil is less prone to soil erosion.
- Provides habitats for wildlife.
- Control weeds on roadside conditions.
- Increased biodiversity (variation of species)

2.3 SAFETY ASPECTS

- Vegetation proves an effective tool for slope protection in road projects.
- It minimizes effect of rain, snow and ice formation.
- It also minimizes hazardous conditions for maintenance staff.
- It reduces the slippery on the roads and provides safety for vehicles.

3. BENEFITS OF VEGETATION ON EMBANKMENTS

In Embankment design slope stability is the major consideration or element on which design of embankments depends and there is complex relationship between vegetation and slope stability. Vegetation enhance slope stability in following ways:-

By Removing water from soil

(i) Due to shading of trees, the soil becomes dry which increases the infiltration capacity of the soil and allows deep penetration of the rain water.

(ii)Due to capillary action of plants the r is drawn up from the roots or soil to the leaves which is then removed through process of transpiration it also by controlling moisture content of soil.

Mechanical Reinforcement

i. Roots increase the shear strength of the soil by binding the particles along the potential failure plant.

ii. Due to root elongation across slip plane there is development of root tensile force which is transferred to soil.

4. EFFECT OF VEGETATION ON SHOULDER

A shoulder is a portion of roadway that is continuous with the travelled way and is provided for lateral support of base and surface course. Due to lack of funds most common types of shoulder prevail in India are earthen shoulders that are compacted in different layers. Due to earthen shoulders maintenance requirement is an essential component and vegetation plays a vital role in maintaining shoulders during rainy season as it prevent the rain cuts, reduce slipperiness of the shoulder. Improves the water Infiltration capacity of the shoulder and also avoid soil erosion during rainy season by firmly biding the soil particles. Vegetation is also the one of way of keeping the earthen shoulder in proper shape of profile. With the help of vegetation dust nuisance is also minimized and load carrying capacity of shoulder is also increased.

5. IMPORTANCE OF VEGETATION ON HILL ROADS

As hill ranges are very young due to which a minor disturbances can cause slips, subsidence and Land-slides. Landslides are basic problem on all hill roads. There are many factors which contribute the land slide whereas deforestation, grazing of animals is also a major contributing factor. As trees or vegetation on roadside not only increase shear strength along the failure plane but also improves the load carrying capacity of soil along the failure plane, provides lateral support by preventing soil erosion. As a preventive measure to avoid landslides afforestation & fencing should be done so that grazing of animals should be stopped.

6. AESTHETIC ASPECT OF VEGETATION

- Roadside vegetation protects from unsightly views such as slums, Junk Yards, Storage depots etc.
- Trees provide shade, colour if they are of flowering variety and also yields fruits.

7. ENVIRONMENTAL ASPECT OF VEGETATION

Noise Pollution: Noise is an unwanted sound on the road &it is mainly caused by breaks, horn, and engine of vehicles. So for highway engineering it is also a better opportunity to control noise pollution by just planting the trees and shrubs on the roadside.

Air Pollution: As lot of poisonous fumes and smell are caused by the engines of vehicles which are hazardous to environment and driver. All types of pollutant like lead particles, oxides of nitrogen, Carbon monoxide, Oxides of nitrogen can be easily controlled by the roadside vegetation.

8. LIMITATIONS OF VEGETATION

- Vegetation doesn't stabilize instable slopes as due to higher planting difficulties and a higher erosion hazard produce by greater runoff velocity.
- Improper drainage and poor consolidation of roads are less stabilized by vegetation.
- The availability of easily adapted plants may be limited.

- Labour needs are intensive & skilled experience labour may not be available. So pre-requisite training is required.
- The planting season of plant or vegetation may be limited.
- If trees are planted at top of slope extra 10% factor of safety should be required as tree of 30-50m height generally applies loading of 150km/m^2.

9. SPECIES SELECTION

It should be beneficial to select native species instead of non-native species as these can easily compete with the prevailing climatic conditions and one should try to select those species of vegetation that can roughly match with the environmental conditions of road and special attention should be given in following cases:-

- Select those species with that are comfortable with soil movement at project sites.
- The deep and widespread root system should be adopted where deep earth movements are there. E.g.: Popular, Eucalypts Acacia.
- Special attention should be given in shady regions as most of plants material will grow poorly and their life is also short.

10. CONCLUSION

Although roadside vegetation has certain limitations like limited plantation season of trees but keeping in view all the above benefits of roadside vegetation, considering its economic, environmental, safety aspect etc. ; it should be given due importance.

Since, roadside vegetation has varied benefits on hilly roads, embankments, to improve soil strength, improving infiltration capacity of soil, reduction in soil erosion. So considering the benefits of roadside vegetation, this paper has been attempted to promote roadside vegetation as an important aspect in Highway Engineering.

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AN INVESTIGATION OF STRENGTH CHARACTERISTICS OF CONCRETE CONTAINING RECYCLED AGGREGATES OF MARBLE AND GRANITE WASTE.

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Abstract— The results obtained from the present investigation on strength characteristics of concrete containing natural aggregates and natural aggregates with partial replacement by marble and granite waste aggregates in different percentages have been presented. In the series of test conducted when natural aggregates were replaced by marble waste and granite waste aggregates used in equal proportions with replacement of natural aggregates by 20% (10% marble +10% granite) , 30% (15% marble +15% granite),40% (20% marble +20% granite) were cast. The compressive strength of specimens were tested for mixes containing marble and granite waste as recycled aggregates increased for replacement 20% and 30%. However for the 40% replacement of marble and granite waste aggregate with natural aggregate a marginal decrease in compressive strength is recorded. Therefore it can be concluded that the production of concrete of normal strength is feasible and viable by replacing the natural aggregates by the waste marble and granite aggregates without compromising the strength characteristics.

Index Terms—Specific Gravity, water Absorption , Fineness Modulus and Compressive Strength.

I. INTRODUCTION

Recycling is the act of processing the used material for use in creating new product. Stone waste i.e. Marble and Granite waste has been commonly used as building materials. Today industry's disposal of stone waste is one of the environmental problems around the world .Stones are cut into smaller blocks in order to give them the desired shape and size. During the process of cutting, the original stone mass is lost by 30%. The waste is dumped in nearby pits and vacant spaces. This leads to serious environmental pollution an occupation of vast area of land. So it poses a severe threat on the environment, eco-system and the health of the people. The Quarrying and Trimming waste also poses a serious environmental damages.

So it is necessary to use this stone waste in construction industry. Recycled aggregate of Marble and Granite waste are comprised of crushed, graded inorganic particles processed from the materials that have been considered as a waste material. In the present study an effort has been made to explore the possibility of using these materials as part replacement of natural aggregates for making concrete.

Terzi and Karasahin (2003) investigated the use of marble dust in asphalt mixtures as a filler material for optimum filler/bitumen and filler ratio. They have concluded that marble wastes in the dust form could be used in such cases.

Abkulut and Cahit (2007) studied the use of marble quarry waste in asphalt pavements with bitumen. They reported that waste materials can potentially be used as aggregates in light to medium trafficked asphalt pavement binder layers.

Binici etal. (2008) studied durability of concrete containing granite and marble as coarse aggregates. The result indicated that marble, granite and ground blast furnace slag replacement provide a good durable concrete.

Wattanasiriwech et al. (2009) investigated the use of waste mud from ceramic tile production in paving blocks and determined compressive strengths of these blocks. They observed that the blocks containing cement 20 weight% gave satisfactory strength values.

Pereira et al. (2009) performed an experimental study using a number of coarse aggregates from different geological sources including granite, basalt, limestone and marble. They produced concretes in specific mix proportions and laboratory controlled conditions. They explored that concrete durability properties were not affected by aggregates mineralogy, but in turn were significantly affected by the aggregate size and its water content.

Padmini et al. (2009) investigated the properties of recycled aggregates from parent concrete (PC) of three strengths, each of them made with three maximum sizes of aggregates. They produced recycled aggregate concrete (RAC) using these recycled aggregates. They found that RAC required relatively lower water-cement ratio as compared to PC to achieve a particular compressive strength. They also determined that the difference in strength between PC and RAC increased with strength of concrete.

Martínez-Barrera and Brostow (2010) studied effects of gamma irradiation and the marble particle size on compressive properties and the dynamic elastic modulus of polymer concretes. One of the conclusions was that both compressive properties and the dynamic elastic modulus values depend on the combination of the marble particle sizes and the applied radiation dose. Higher numbers of dispersed particles per unit volume provide more resistance to crack propagation. Medium size marble particles provide better compression modulus.

II EXPERIMENTAL PROGRAMME

The test programme consisted of the testing of the constituent materials i.e. cement, fine aggregate, coarse aggregate as per relevant Indian Standard Codes of Practice and testing of specimens containing Natural aggregates and with recycled aggregates of marble for compression and split tensile strength. The physical properties of cement used in the present study are given in Table 1.The physical properties of fine and coarse natural and granite aggregates used in investigation are presented in Tables 2, 3, 4 and 5.

Table 1: Physical Properties of Cement

Sr. No.		Property	Experimental value
1		Consistency of Cement	30%
2		Specific Gravity	3.14
3		Initial Setting Time	92 minutes
4		Final Setting Time	298 minutes
5	i) ii) iii)	Comp. Strength (N/mm^2) 3 days 7days 28 days	24.67 35.04 47.28
6		Fineness (Dry Sieving)	2.5 %

Table 2: Physical Properties of Fine Aggregates

Characteristics	Results Obtained
Grading	Grading Zone II (IS: 383-1970)
Fineness Modulus	2.55
Specific Gravity	2.62
Water Absorption (%)	0.48%
Free Moisture Content (%)	Nil

Table 3: Physical Properties of Coarse Natural Aggregates

Characteristics	Results Obtained
Fineness Modulus	6.6

Specific Gravity	2.66
Water Absorption (%)	0.50%
Moisture Content (%)	Nil

Table 4: Physical Properties of Coarse Marble Aggregates

Characteristics	Results Obtained
Fineness Modulus	6.51
Specific Gravity	2.68
Water Absorption (%)	0.32
Moisture Content (%)	Nil

Table 5: Physical Properties of Coarse Granite Aggregates

Characteristics	Results Obtained
Fineness Modulus	6.51
Specific Gravity	2.70
Water Absorption (%)	0.49
Moisture Content (%)	Nil

Sieve analysis of fine aggregates, coarse natural, marble and granite waste aggregates is carried out and the results are presented in Tables 6, 7, 8 and 9. The details of mixes with and without marble and granite waste aggregates are given in Table 10.

Table 6: Sieve Analysis of Fine Aggregates

IS Sieve Designation	Wt. Retained on Sieve (gm)	Cumulative Weight Retained (gm)	Cumulative Percentage Weight Retained (gm)	%age Passing
10mm	0.00	0.00	0.00	100.00
4.75 mm	15.10	15.1	1.51	98.49
2.36 mm	25.20	40.30	4.03	95.97
1.18 mm	250.10	290.40	29.04	70.96
600 µ	160.00	450.40	45.04	54.96
300 µ	320.10	770.50	77.05	22.95
150 µ	217.10	987.60	98.76	1.24
Pan	12.40	1000	-	-

Table 7: Fineness Modulus of Proportioned Coarse Aggregates

I S Sieve Designation	Wt. Retained on Sieve (10 mm Agg) (gm)	Wt. Retained on Sieve (20mm Agg) (gm)	Average Weight Retained (gm)	Cumulative Wt. Retained (gm)	Cumulative %age Wt Retained (gm)	%age Passing
80mm	0.00	0.00	0.00	0.00	0.00	100.00
40 mm	0.00	0.00	0.00	0.00	0.00	100.00
20 mm	0.00	335	167.5	167.5	3.35	96.65
10 mm	1225	4565	2895	3062.5	61.25	38.75
4.75 mm	3625	90	1857.5	4920	98.40	1.6
Pan	-	-	-	-	-	-

Table 8: Fineness Modulus of Proportioned Coarse Marble Aggregates

I S Sieve Designation	Wt. Retained on Sieve (10mm Agg) (gm)	Wt. Retained on Sieve (20mm Agg) (gm)	Average Weight Retained (gm)	Cumulative Wt. Retained (gm)	Cumulative %age Wt Retained (gm)	%age Passing
80mm	0.00	0.00	0.00	0.00	0.00	100.00
40 mm	0.00	0.00	0.00	0.00	0.00	100.00
20 mm	0.00	275	137.5	137.5	2.75	97.25
10 mm	170	4570	2370	2507.5	50.15	49.85
4.75 mm	4715	145	2430	4937.5	98.75	1.25
Pan	-	-	-	-	-	-

Table 9: Fineness Modulus of Proportioned Coarse Granite Aggregates

I S Sieve Designation	Wt. Retained on Sieve (10mm Agg) (gm)	Wt. Retained on Sieve (20mm Agg) (gm)	Average Weight Retained (gm)	Cumulative Wt. Retained (gm)	Cumulative %age Wt Retained (gm)	%age Passing
80 mm	0.00	0.00	0.00	0.00	0.00	100.00
40 mm	0.00	0.00	0.00	0.00	0.00	100.00
20 mm	0.00	225	112.5	112.5	2.25	97.75
10 mm	2432.5	4190	3311.25	3423.75	68.47	31.53
4.75 mm	1916.75	505	1210.87	4634.62	92.69	7.31
Pan	-	-	-	-	-	-

Cumulative percentage wt. retained = 163.41 + 500

= 663.41

Fineness Modulus (F.M.) = 663.41/100=6.63

Table 10: Detailed Mix Proportions for Natural and Recycled Aggregates of Marble and Granite

Mix Desi-gnation	Cement (kg/m3)	Fine aggregates (kg/m3)	Natural Coarse Aggregates (kg/m3)	Marble Coarse Aggregates (kg/m3)	Granite Coarse Aggregates (kg/m3)	Water (kg/m3)	w/c ratio
M	3	64	1157	---		19	0.48
M2	399	643	926.0	115.76	115.76	191.5	0.48
M3	399	643	810.3	173.64	173.64	191.5	0.48
M4	399	643	694.5	231.52	231.52	191.5	0.48

From the tables the fineness modulus of fine aggregates, coarse natural aggregates, marble and granite waste aggregates are 2.55, 6.6, 6.51 and 6.63.

III RESULTS AND DISCUSSION

COMPRESSIVE STRENGTH

To study the effect of replacement of natural aggregates by marble waste and granite waste aggregates used in equal proportions, cubical specimens with replacement of natural aggregates by 20% (10% marble +10% granite) , 30% (15% marble +15% granite),40% (20% marble +20% granite) were cast and tested. The results obtained for the specimen tested for compressive strength at 7 days and 28 days are reported in Table 11 and 12 respectively. The comparison of compressive strength at 7 days and 28 days for specimens with natural aggregates and the specimens containing marble and granite waste aggregates in different percentages is shown in Figure 1 and 2.

Table 11: Test Results of Compressive Strength of Specimens at 7 Days

Mix Designation	%age Replacement	Load (kN)	Compressive Strength	Average Compr
M1	0	473.70	21.05 26.19	22.87
M2	20	589.27	26.19 25.90	25.95
M3	30	558.00	24.80 24.33	25.22
M4	40	515.70	22.92 24.65	24.39

Table 12: Test Results of Compressive Strength of Specimens at 28 Days

Mix Designation	%age Repl	Load (kN)	Compressive Strength	Average Compr
M1	0	670.50	29.78 28.47	29.53
M2	20	771.75	34.30 27.56	32.12
M3	30	745.65	33.14 31.52	31.15
M4	40	657.67	29.23 29.38	29.20

It can be seen from Tables 11, 12 and Figures 1, 2 that the compressive strength of mix M2 at 7 days and 28 days increased with replacement of natural aggregates by marble and granite waste aggregates by 20% when compared to the control mix M1. For M3 30% replacement of natural aggregates by marble and granite waste aggregates further increase in compressive strength was recorded as compared to control mix M1. For the mix M4 containing 40% replacement of natural aggregates by marble waste aggregates the decrease in compressive strength is recorded. The increase in compressive strength of concrete with replacement of natural aggregate by marble and granite waste aggregates can be attributed to improved microstructure of concrete containing marble and granite waste aggregates which may be due to higher specific surface area of marble and granite aggregates and thus improving bond in between mortar and aggregates. The decrease in percentage improvement in compressive strength at higher replacement levels may be attributed to the grading effect. Figure 3 represents the typical mode of failure of cubical specimens.

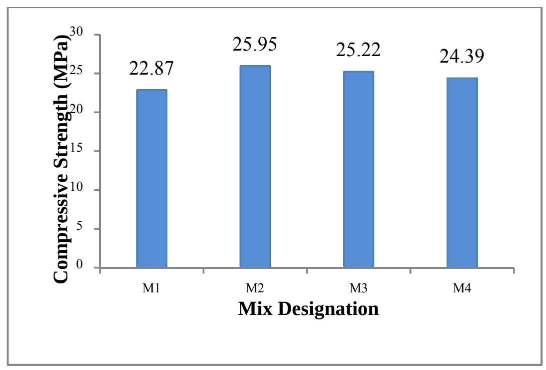

Fig. 1: Comparison of Compressive Strength of Specimens at 7 Days

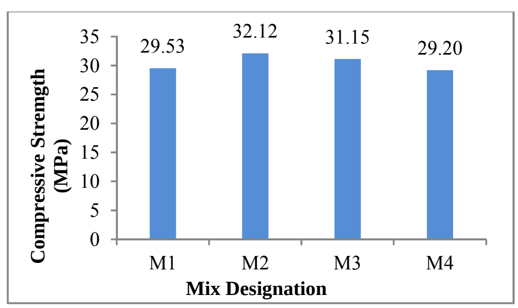

Fig. 2: Comparison of Compressive Strength of Specimens at 28 Days

Fig.3:Typical Mode of Failure for Cubical Specimens.

SPLIT TENSILE STRENGTH

To study the effect of replacement of natural aggregates by marble waste and granite waste aggregates used in equal proportions, cylindrical specimens (Series-2) with replacement of natural aggregates by 20% (10% marble+10%granite),30%(15%marble +15% granite),40% (20% marble +20% granite) were caste and tested. The results obtained for the specimen at 7 days and at 28 days are reported in Table 13 and 14.

Table 13: Test Results of Split Tensile Strength of Specimens at 7 Days

Mix Designation	%age Replacement	Split Tensile Strength (Tonnes)	Split Tensile Strength (N/mm^2)	Average strength (N/mm^2)
M1	0	14 14	1.98 1.98	2.07
M2	20	16 16	2.26 2.26	2.21
M3	30	16 14 16	2.26 1.98 2.26	2.17
M4	40	14 14	1.98 1.98	1.98

Table 14: Test Results of Split Tensile Strength of Specimens at 28 Days

Mix Designation	Split Tensile Strength (Tonnes)	Split Tensile Strength (N/mm^2)	Average strength (N/mm^2)
M1	20 22	2.83 3.11	2.92
M2	24 24	3.39 3.39	3.29
M3	24 20	3.39 2.83	3.10
M4	20 20	2.83 2.83	2.73

The comparison of compressive strength at 7 days and 28 days for specimens with natural aggregates and the specimens containing marble and granite waste aggregates in different percentages is shown in Figures 4 and 5.

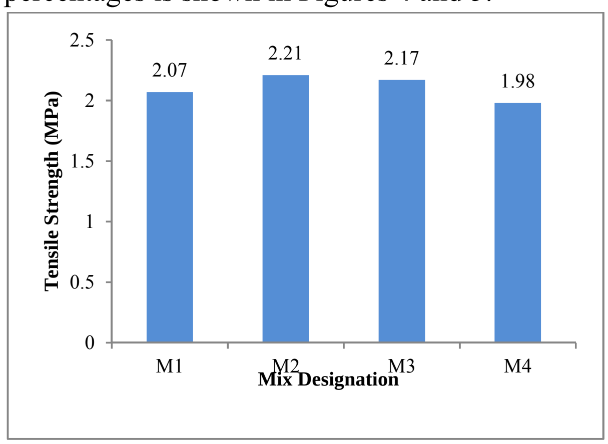

Fig. 4: Comparison of Split -Tensile Strength of Specimens at 7 Day

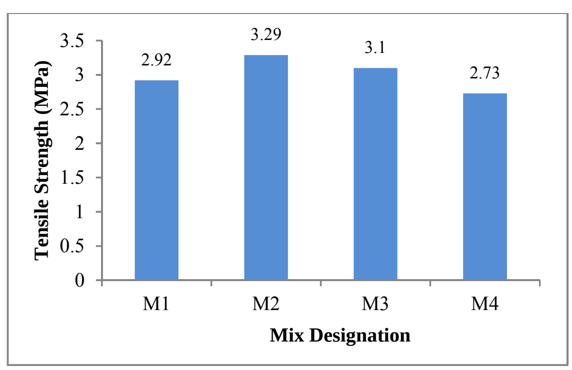

Fig. 5: Comparison of Split -Tensile Strength of Specimens at 28 Days

It can be seen from above Tables 13 and 14 and Figures 4 and 5 that in line with the results obtained for compressive strength for both the series, the similar trends were obtained for split tensile strength also which correlate the beneficiary effect of replacing natural aggregates by marble waste and granite waste aggregates mixed in equal proportions.

FLEXURAL STRENGTH

To study the effect of replacement of natural aggregates by marble waste and granite waste aggregates used in equal proportions, beam specimens with replacement of natural aggregates by 20% (10% marble +10% granite), 30% (15% marble +15% granite),40% (20% marble +20% granite) were cast and tested. The results obtained for the specimen are reported in Tables 15 and 16.

Table 15: Test Result of Flexural Strength of Specimens at 7 days

Mix Designation	%age Replacement	Flexural Strength (Tonnes)	Flexural Strength (N/mm^2)	Average strength (N/mm^2)
M1	-	1.0 1.2	2.02 2.43	2.29
M2	2 (	1.4 1.3	2.83 2.63	2.76
M3	3 (	1.4 1.4	2.83 2.63	2.69
M4	4 (	1.2 1.2	2.43 2.43	2.49

Table 16: Test Result of Flexural Strength of Specimens at 28 days

Mix Designation	%age Replacement	Flexural Strength (Tonnes)	Flexural Strength (N/mm^2)	Average strength (N/mm^2)
M1	-	1.8 1.9	3.64 3.84	3.70
M2	2	2.1 2.1	4.25 4.25	4.18
M3	3	1.8 2.0	3.64 4.05	3.98
M4	4	1.8 1.8	3.64 3.64	3.64

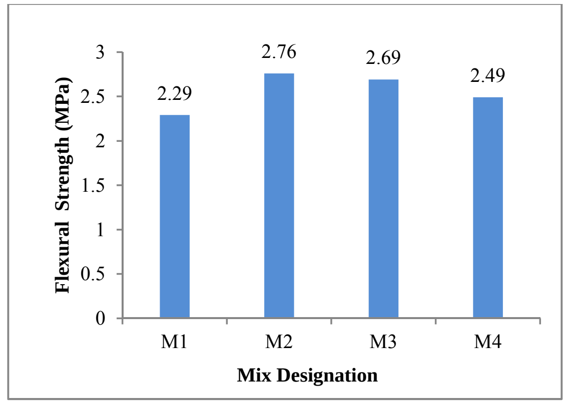

Fig. 6: Comparison of Flexural Strength of Specimens at 7 Days

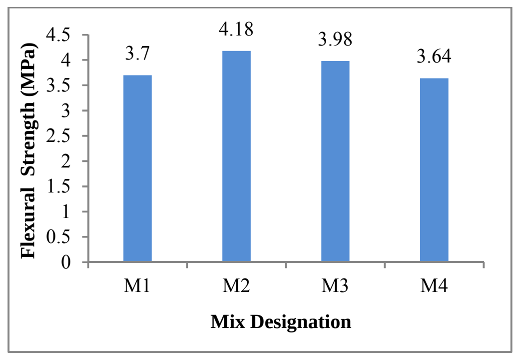

Fig. 7: Comparison of Flexural Strength of Specimens at 28 Days

It can be seen from above Tables 17 and 18 and Figures 6 and 7 that in line with the results obtained for compressive strength for both the series, the similar trends were obtained for flexural strength also which correlate the beneficiary effect of replacing natural aggregates by marble waste and granite waste aggregates mixed in equal proportions.

CONCLUSIONS

1. The compressive strength, split-tensile strength and

flexural strength of specimens tested in mixes containing marble and granite waste as recycled aggregates increased for replacement of 20% and 30%. However for the 40% replacement of marble and granite waste aggregates with natural aggregates marginal decrease in compressive strength is recorded. For mix containing 20% and 30% waste marble aggregates the compressive strength at 28 days was increased by 8.7 % and 5.5% when compared to the control mix.

2. The split tensile strength of specimens tested for mixes containing marble and granite waste as recycled aggregates increased for replacement of 20% and 30%. However for the 40% replacement of marble and granite waste aggregate with natural aggregate a marginal decrease in compressive strength is recorded. For mix containing 20% and 30% waste marble aggregates the split tensile strength is increased by 12.0% and 6% when compared to the control mix.

3. The flexural strength of specimens tested for mixes containing marble and granite waste as recycled aggregates increased for replacement of 20% and 30%. However for the 40% replacement of marble and granite waste aggregate with natural aggregate a marginal decrease in compressive strength is recorded. For mix containing 20% and 30% waste marble aggregates the flexural strength is increased by 12.9% and 7.5% when compared to the control mix.

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DESIGN, FABRICATION, DEVELOPMENT AND CONSTRUCTION OF LOW COST LAVATORY

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Abstract— It is evident that, in India, approximately 30% rural people live below the poverty line. In this context, assuring basic hygiene for one and all is a major task. Further, poor sanitation affects the health of the people and also the development of the nation. Also, it is observed that, in India, rural community lives housing without toilets due to their poverty. In this paper, an attempt is made to show the solution for sanitation of rural people by providing them with a low–cost lavatory, developed in such a way that, it can be affordable by a common man. The main aim of this work is to promote better human health and improve quality of life among people living in rural areas through improved sanitation measures.

Keywords-Rural India; Sanitation; Low–Cost Lavatory.

I. Introduction

In India, 80% of the population resides in 6, 00, 000 villages spread across the country. In rural areas, people live in adverse conditions of sanitation without having adequate facilities for defecation, due to their poverty and ignorance. People require safe and hygienic facilities for excretion . In this connection, it is important to consider several technological aspects such as affordability, space, cultural habits, availability of water and labour for construction etc., to maintain sanitation for rural people. The present paper deals with design, fabrication development and construction of Low-Cost Lavatory for Rural India.

II. Need of the Study

In India, many villagers go barefoot for open air defecation due to which ailments such as dysentery, diarrhea and cholera spread over.

Despite several programs have been taken up by government on sanitation, adverse effects of insanitation and its impact on human health, rural people are still neglecting safe mode of defecation system. Generally, rural people prefer low–cost, location specific and acceptable design and technologies depending on their socio–economic status. Hence, there is a dire necessity to have a lavatory system which is cost effective.

III. Fabrication and construction of Low Cost Lavatory

For the construction of a model design of Low–Cost lavatory, the site nearby workshop area of S. R. Engineering College, Warangal was chosen. The site is cleared from debris and leveled. A soak pit (5' depth and 3.5' diameter) was excavated and cement rings were dropped slowly into the pit with the help of rope to stop collapse sides of pit (Fig. 2).

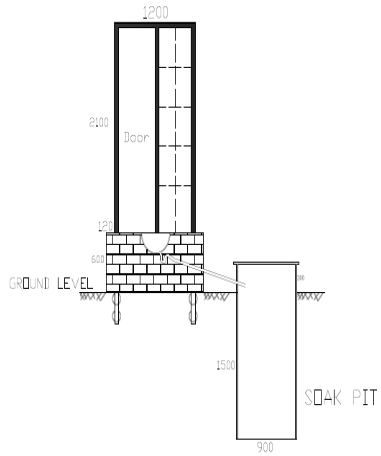

Fig. 1. Front view of Low–Cost Lavatory.

For the construction of lavatory, the prefabricated steel cage was used. The legs of the cage were driven into the pits (2' depth and 4' apart) and concrete was poured into pits for firm gripping of the legs. A basement with cement bricks is constructed and the set up for Indian water closet was arranged. This arrangement is connected to soak pit to facilitate the passage of digested waste (Fig. 1 and 2).

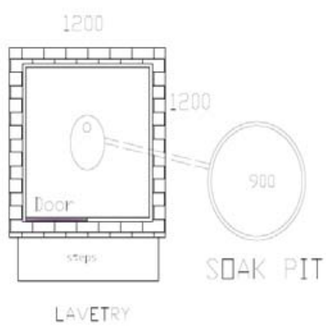

Fig. 2. Top view of Low–Cost Lavatory.

Plate 1. Students at work

The cage was enclosed with the Fiber Reinforced Plastic (FRP) sheets for privacy. This sheet is fixed to steel anglers with nut and bolt system to get the final shape of the lavatory (Plate 1). The surrounding ground is leveled for accessibility.

IV. Salient features of Low-cost Lavatory

- Less time of construction.
- Superstructure is made from steel giving rise to high strength.
- Low cost of the construction **(less than INR 10,000)** (Table 1).
- Suitable for local festival places/people gathering.
- Trouble-free to empty the pit when it fills.
- Apt for construction where limited space is available.

v. Advantages of Low–Cost Lavatory

- Prevents spreading of infections and diseases (diarrhea, dysentery, dehydration and cholera) due to open air defecation.
- Reduces the rate of dropping out school going girl children.
- Hygiene of surroundings can be maintained.
- Contamination of soil is also avoided.
- Decomposed material in soak pit can be utilized as manure (natural fertilizer) for growth of crops.

vI. Cost Estimate

Table 1. Estimation of the cost incurred in the construction of Low–Cost Lavatory

S.No.	Description of Item/Work	Quantity	Unit Cost (INR)	Total Cost (INR)
1.	Earth work Excavation for pit (5' depth, 3' diameter)	L.S.	---	650
2.	Cement Concrete Rings (3'	6	180	1080

DRAINAGE ON ROADS

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ABSTRACT:

It has been seen many times that water in pavements is one of the major causes of premature pavement failure. Water may enter the pavement due to various reasons which may be stagnation of water on the surface or faulty construction of the roads leading to seepage of water into the pavement and thus causing damage to the same. Water in the pavement system can lead to moisture damage, modulus reduction and loss of strength. In order to prevent such damages to the pavement, it is essential to provide proper drainage to the roads. The presence of water in a highway layer reduces the bearing capacity of the road, and in doing so it also reduces the structure's lifetime. Highway drainage is used to clear surface water from the highway. . Roads need to be well drained to stop flooding, even surface water can cause problems with ice in the winter. Water left standing on roads can also cause maintenance problems, as it can soften the ground under a road making the road surface break up.

1. INTRODUCTION:

Highway drainage is the process of removing and controlling excess surface and sub-surface water within the right way. This includes interception and diversion of water from the road surface and sub-grade. The installation of suitable surface and sub-surface drainage system is an essential part of highway design and construction.

During rain, part of the rain water flows on surface and part of it percolates through the soil mass as gravitational water until it reaches the ground water below the water table. Removal and diversion of surface water from the roadway and adjoining land is termed as surface drainage, while the removal of excess soil-water from the sub-grade is termed as sub-surface water.

2. NECESSITY OF HIGHWAY DRAINAGE

Highway drainage is important from various view points:

- Excess moisture in soil sub-grade causes instability under the road surface. The pavement may fail due to sub-grade failure. In some clayey soil variation in moisture content causes considerable variation in volume of sub-grade. This sometimes contributes to pavement failure.
- The waves and corrugations formed in case of flexible pavements also play an important role in pavement failure.
- Sustained contact of water with bituminous pavements causes failure due stripping bitumen from the aggregates like loosening of some of the bituminous pavement layer and formation of pot holes.
- The prime cause of failures in rigid pavements by mud pumping is due to the presence of water in fine sub-grade soil.
- Excess water on shoulders and pavement edge causes considerable damage.
- Excess moisture causes increase in weight and thus increase in stress and simultaneous reduction in strength in soil mass. This is one of the main reasons of failure of earth slope and embankment foundations.
- In place where freezing temperatures are prevalent in winter, the presence of water in sub-grade and a continuous supply of water from the ground water can cause considerable damage to the pavement due to in frost action.
- Erosion of soil from top of un-surface roads and slopes of embankment, cut and hill side is also due to surface water.
- Failure due to hydraulic pressure and failure due to binder stripping can be avoided with the help of proper drainage on roads.

3. ROAD DRAINAGE

Well designed and well maintained road drainage is important in order to:

- Minimize the environmental impact of road runoff on the receiving water environment.
- Ensure the speedy removal of surface water to enhance safety and minimize disruption to road users.
- Maximize the longevity of the road surface and associated infrastructures.

There are many different types of drainage systems with different design features and attributes that can be used to manage flows and treat water quality. Drainage which is needed on the Highways Agency network depends not just on any flood risks and pollution risks identified but the characteristics of the natural water catchment area in which the network is based. The size, shape, gradient and geology of a catchment area are all factors which can influence the type of drainage methods used.

4. SURFACE DRAINAGE

The surface water is to be collected and then disposed off. The water on the surface is first collected in longitudinal drains, generally in side drains and then the water is disposed off at the nearest stream, valley or water course. For the preparation of surface drainage, we should keep in mind various things like

COLLECTION OF SURFACE WATER

Seeing the amount of rainfall and slope a suitable camber is to be provided for collection of surface water. The shoulders of rural roads are constructed with suitable cross slopes so that the water is drained across the shoulders to the side drains. These side drains of rural roads are generally Open (kutcha) drains of trapezoidal shape, cut to suitable cross-section and longitudinal slopes. These sides are provided parallel to the road alignment and hence these are also known as longitudinal drains. In embankments the longitudinal drains are provided on one or both sides beyond the toe; in cutting, drains are installed on either side of the formation.

In urban roads because of the limitation of land width and also due to the presence of

footpath, diving island and other road facilities, it is necessary to provide underground longitudinal drains. Water drained from the pavement surface can be carried forward in the longitudinal direction between the kerb and the pavement for short distances which may be collected in catch pits at suitable intervals and lead through underground pipes.

Drainage of surface water is all the more important in hill roads. In hill roads disposal of water is also very important. Certain maintenance problems may arise due to faulty hill road construction.

5. CROSS DRAINAGE

For streams crossing the runways, drainage needs to be provided. Also often the water from the side drain is taken across by these cross drains in order to divert the water away from the road, to a water course or valley in the form of culverts or bridges. When a small stream crosses a road with linear water way less than amount six meter, the cross drainage structure provided is called culvert; for higher value of linear waterway, the structure is called bridge.

6. SUB-SURFACE DRAIN

Change in moisture content of sub-grade are caused by fluctuations in ground water table seepage flow, percolation of rain water and movement of capillary water and even water vapour. Although sub-surface drainage helps in removal of gravitational water, it is designed to keep minimum moisture in sub-grade.

LOWERING OF WATER TABLE

The highest level of water table should be fairly below the level of sub grade, in order that the sub grade and pavements layers are not subjected to excessive moisture. From practical considerations it is suggested that the water table should be kept at least 1.0 to 1.2 meter below the sub grade. In place where water table is high (almost at ground level at times) the best remedy is to take the road formation on embankment of height not less than 1.0 to 1.2 meter. When the formation is to be at or below the general ground level, it would be necessary to lower the water table.

If the soil is relatively permeable, it may be possible to lower the high water table merely construction of longitudinal drainage trenches with drain pipe and filter sand. If the soil is relatively less permeable, the lowering of ground water level may not be adequate at the center of the pavement or in between the two longitudinal drainage trenches. Hence in addition, transverse drainage may have to provide in order to effectively drain off the water and thus lower the water table up to the level of transverse drains.

7. PREVENTIVE MEASURES

- **CONTROL OF SEEPAGE FLOW**

When the general ground and impervious strata below are slopping, seepage flow is likely to exist. If the seepage zone is at depth less than 0.6 to 0.9 meter from the sub grade level, longitudinal pipe drain in trench filled with filler material and clay seal may be constructed to intercept the seepage flow.

- **CONTROL OF CAPILLARY RISE**

If the water reaches the sub grade due to capillary rise is likely to be detrimental, it is possible to solve the problem by arresting the capillary rise instead of lowering the water table. The capillary rise may be checked either by capillary cut-off of any one of the following two types:-

a) A layer of granular material of suitable thickness is provided during the construction of embankment, between the sub grade and the highest level of sub surface water table.

The thickness of the granular capillary cut-off layer should be sufficiently higher than the anticipated capillary rise with in the granular layer so that the capillary water cannot rise above the cutoff layer.

b) Another method of providing capillary cut-off is by inserting an impermeable or Bituminous layer in the place of granular blanket.

8. CONCLUSION

Seeing the above properties of drainage and keeping in view the necessity of drainage at surface as well as sub-surface level, drainage plays an important role in highway engineering. As drainage helps in avoiding various types of failures as may be caused by stagnant water on the road surface or its seepage beneath the pavement, it is important to provide drainage facility while construction of roads. Thus to increase the life of the road and to reduce the maintenance cost drainage of roads must be properly provided. Considering the above factors, this paper has been attempted in lieu of highway engineering.

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COMPARATIVE STUDY OF THERMAL PERFORMANCE OF INSULATED LIGHT ROOFS IN TROPICAL CLIMATE

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Abstract: Thermal performance of a building depends on many factors. It is associated with solar insolation, the type of roof, type of wall, orientation, colour of paint given to the walls, and the motion of Earth around the sun. The building elements, roof and wall play an important role in the thermal reception from the solar insolation. Residential buildings consume more energy for heating and cooling. A residential building should not be a mere shadow maker. It should be a less energy consumer. It should provide a good thermal experience. To reach this aim the indoor environment should be feasible. The indoor environment depends on the indoor temperature and the indoor relative humidity. If the indoor temperature increases then the relative humidity will be decreased. Now the occupants feel thermal discomfort. So control over the indoor temperature is a must. The quantity of heat entering the inside the building can be modified through Roof and wall insulation. In this study, to get a better thermal performance roof insulation is alone considered. Two side by side modules of the same size and orientation were constructed with different insulated roofs. The construction of walls and floor are having the same dimension. Polyurethane readymade panels are used as roof material in one of the modules and named as PUD module. The polyurethane panel is an industrial product. The other module is provided with a newly designed roof called Double Decker (DOD). Both the roofs are having intermediate insulation. Thermal performance of the newly designed roof is better than the industrial product in the summer peak hours. The cost of the DOD is relatively less. The study has been carried out from September 2013 to August 2014, in the tropical climate.

Key words: DOD, Indoor temperature, PUD, Relative humidity, Thermal comfort, Thermal performance.

I. INTRODUCTION

Energy consumption in buildings is also increasing at an unprecedented rate, as more and more buildings are designed in lightweight materials like Galvanized sheets, Asbestos sheets, glass and aluminium giving little importance to the passive methods of heat control and human adaptation to comfort. It is well known now that buildings with poor adaptive opportunities often produce intolerable indoor conditions within, and consume high energy.

In the residential buildings, indoor thermal discomfort has been very challenging and it depends on, one or more of the materials used either as ceiling or wall or making doors or roofing support or combination of all. The ceiling materials are made by different materials with different thermal conductivity, thermal absorptivity, thermal diffusivity and thermal resistivity. Heat propagated into indoor space is partly through ceiling and partly through walls by the process of conduction, convection and radiation. Shortage of conventional energy source and enhancing energy cost has caused the re-examination of the general design practice Hence, the major focus of researchers, policy makers, environmentalists and building architects has been on the conservation of energy in buildings. Energy efficiency in buildings can be achieved through a multipronged approach involving adoption of bioclimatic architectural principles responsive to the climate. Use of materials with low embodied energy and effective utilization of renewable energy sources should be carried out to power the building. Studies of various countries have shown that buildings with wooden structures require less energy and emit less CO2 during their life cycle than buildings with other type of structures.

The energy efficient roofs so far designed costs too high, and they do not reach the common. Most of the heat developed by the solar radiation in the roof and wall is transferred into the occupant zone. The sun is the primary source of thermal energy. Roof receives the solar radiation directly from the sun light and it becomes a secondary source of heat. The east wall receives heat directly up to the noon and the west wall receives heat directly after the noon. The two walls also becomes the secondary source of heat. The floor of a building does not heated directly by the sun light in most of the buildings. The roof and walls become secondary heat sources. As soon as the roof and walls emit heat radiations into the building, the heat is transferred to the indoor air. Regarding light roofs the indoor temperature of a building closely follows the outdoor temperature.

II. BACK GROUND OF THE STUDY

The Tropics is regarded as a region where the human evolved and comfort has been often taken for granted, built environments are increasingly becoming issues of public concern. The tropical outdoor environment has been regarded as important as indoors in the life of the populace. This tendency has put increased demand on the comfort requirements in the design of buildings. Comfortable outdoor spaces have a significant bearing on the comfort perception of the indoor ambience. The demand for comfort conditions in buildings are significantly increased as a result of exposure to uncomfortable outdoors [1]. Generally the tropical zone is defined as the area of land and water between the Tropic of Cancer (latitude 23.5^{o} N) and the Tropic of Capricorn (latitude 23.5^{o}S).Occupying approximately 40% of the land surface of the earth, the tropics are the home to almost half of the world's population. There are variations in climate within the tropic. However 90% of the tropical zones embody hot and humid climatic regions, whether permanent or seasonal. The remaining 10% is desert like, and characterized as hot and dry climate [2].

The higher thermal resistance systems containing bulk insulation within the timber frame, the measured result for a typical installation was as low as 50% of the thermal resistance determined considering two dimensional thermal bridging using the parallel path method. This result was attributed to three dimensional heat flow and insulation installation defects, resulting from the design and construction method used. Translating these results to a typical house with a 200 m2 floor area, the overall thermal resistance of the roof was at least 23% lower than the overall calculated thermal resistance including two dimensional thermal bridging [3]. Providing insulation for walls and roof in a building increases their thermal resistance and limits conductive heat flow through the building envelope. The building envelope insulation is a main component because it plays a major function in the energy consumption. The building's roof, windows, walls and floors lead the flow of energy between the indoor and the outdoor of the building. The envelope insulation is very important, and it is the best solution in order to have an efficient and less consuming energy building. [4].

Thermal insulation has a dual nature. It decreases daytime the extra heat that come to a building, but prevents the building from cooling down at night. Based on their study, this dual nature makes insulation inappropriate for buildings with natural climate control. Perhaps the solution is to first define the cooling load at the design phase and then making decision whether this cooling load would be decreased by applying thermal insulation in the building or by using passive means of control [5]. Many studies have also quantified the energy savings from improved insulation. Retrofitting exterior masonry wall insulation from R-3 to R-13, energy consumption reduces by 9 -15% in Arizona [6].

Requirements for energy efficiency in a building envelope surrounding the heated and cooled parts of the building is generally set based on resistance or contribution to heat transparency through a unit of the construction, respectively R-value or U-value. [7]. A study of a typical un-insulated masonry house in the hot and humid climate of Bangkok, Thailand indicated 3-4% annual energy savings from light-weight walls with R-11 batt insulation and from cement tile roof with R-11 batt insulation [8]. A study of a house in Bangkok showed 8% of total energy reduction from light-weight concrete block walls with R-10 exterior insulation, and 9% reduction from similar wall construction with R-10 interior insulation [9]. Wall insulation does not significantly affect reducing heating and cooling load in buildings. He stated that adding 50 mm of polystyrene as wall insulation only causes in 1.7 % 33 [10]. Structural control of a modern building are its main parts such as walls, roof, floors and glazed materials (glass) in openings. The ability of a building enclosing elements to conduct heat from one side of the wall to the other is the thermal transmittance denoted as U value for the element [11].

Mineral wool, also known as rock wool, is an insulation material produced from steel slag. The slag, a by-product of steel manufacturing containing of dirt and limestone, is combined with other chemicals, heated and turned into a fibrous material that is a good insulator. It defined as a permanent insulation because it does not rot; burn or melt, and it does not absorb moisture, and does not maintain mould or mildew. It is available in batts or as a loose-fill product that can be blown into walls and ceilings. It can also be installed between wall studs by using a mesh screen across one side of the studs, letting floor to ceiling filling with a technique virtually the same as with blown-in cellulose. Because of its greater density and water resistant properties, mineral wool performs as a vapour barrier and, unlike fiberglass, does not need an additional vapour barrier to be effective (ORNL 2002) [12].

The major importance of good insulation of the roof in tropical climate is thickness and colour of insulation. In general, 5cm insulation is being used for red and blue tiled roofs, which is inadequate. Therefore, insulation thickness needs to be at least 8cm (the value for medium colours) and to use polystyrene as insulation rather than mineral wool. Mineral wool is fairly cheap but not very well adapted to tropical climates: it loses its thermal properties when it absorbs ambient humidity. In another experiment more than 3˚C have been observed between a dwelling with a well- insulated roof and with no insulation [13].

III. RESEARCH DESCRIPTION

The earlier research efforts have investigated the thermal performance of various roofing systems. In this study an attempt has been made to quantify the influence of insulation on indoor ambient temperature. The two modules have same floor, wall area and orientation. The size of the module is 3mx 3m x 3m. The galvanized sheets used in the modules have the same thickness of 0.21 mm. The walls have a thickness of 230 mm made up of brick and cement mortar. Two angles are used as purlins. It is a low sloped roof and is maintained to be 2˚. Walls of the modules are white washed and the flooring is done with cement mortar.

First Module (PUD):

The first module was constructed with Polyurethane panels of length 3660 mm and breadth 1000 mm is used as roof, which is an industrial product. The thickness of the Poly Urethane Decker is 35 mm and the thickness of the sheets is 0.35 mm. The white painted panels reflects the solar radiation on one hand and on the other hand polyurethane prevents heat entering the inside of the building.

Second Module (DOD):

The roof of the second module was newly designed. The design was carried out in four steps. In the first step, first roof was made using galvanised sheets. In the second step wooden reapers of size 3000 mm X 50 mm X 25 mm were arranged over the roof. The spacing between the reapers is 200 mm. In the third step packed mineral wool roll was spread. Thickness of the mineral wool is 50 mm. In the fourth step galvanized sheets were set over it as second roof. The two roofs are separated by 100 mm to 122 mm. Since light roofing system have two light roofs enclosing the wooden reaper and mineral wool, it was named as Double Decker. Since the sheets are trapezoidal, air gap of 11 mm above and below the mineral wool pack and wooden reapers is formed. The air vents created are the passage for the air and takes away the heat produced between the galvanized sheet and the mineral wool bed. Likewise the air vents created between the lower roof sheet, the wooden reaper and the mineral wool bed is also drains away the heat produced by convection. The mineral wool, wooden reapers and the air enclosed in the gaps are serving as insulators. This assembly possesses three insulators wooden reapers, mineral wool and air gap. Mineral wool has a low thermal conductivity among the building materials used (K= 0.04 W/m K).

Fig.1 Mineral Wool

Fig.2 Double Decker

Fig.3 Polyurethane Decker

IV. EXPERIMENTAL PROCEDURE

The experiments were carried out in Chidambaram, Tamil Nadu, 11˚24´N latitude and longitude 79˚44´E.The location is characterized by hot and humid weather. The modules, used in this study are exactly identical in terms of their geometry, orientation, area and climate conditions. DOD and PUD are reflective roof material. All the modules are fully instrumented. To measure the Indoor

Ambient Temperature and Relative Humidity Single channel data logger is used. In six hours interval (6, 12, 18hr) the roof, wall and floor temperatures are measured by means of Infra-Red Thermometer. Roof, wall, floor and indoor and outdoor ambient temperature and relative humidity field data have been catalogued for eleven months for two different insulated roofing systems exposed to weathering on an indoor and outdoor test facility. The data are plotted for the time period between September 2013 and August.2014.

V. RESULT AND ANALYSIS

Fig.4 shows the Mean Monthly indoor ambient temperature by 6 hours for the observation period. This figure shows that the indoor ambient temperature of the DOD module is higher than the PUD module from September to March 2014. After March to July the indoor ambient temperature of DOD module is lesser than the PUD module. But the variation of indoor temperature between the two modules during the months of September to March is 0.44.the variation during the months between March to July is 0.2 to 0.4˚C. The indoor temperature decreases from September to December.2014 and then increases to a maximum during the summer months of June and July.2014.

Fig.5 shows the Mean Monthly indoor ambient temperature by 12 hours for the observation period. This figure shows that the indoor ambient temperature of the DOD module is lesser than the PUD module from September to July 2014. But the variation of indoor temperature between the two modules during the months of September to July is from 0.4 to 1˚C.

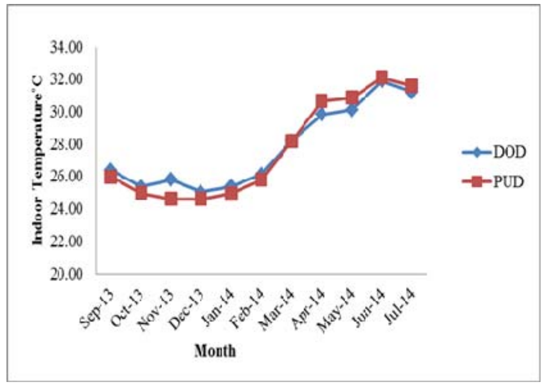

Fig.4 Mean Monthly indoor ambient temperature by 6 hours

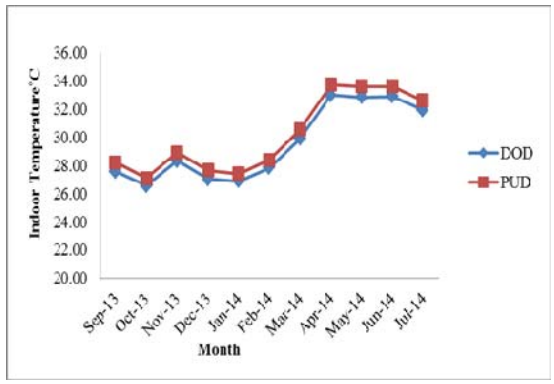

Fig.5 Mean Monthly indoor ambient temperature by 12 hours

Fig.6. shows the indoor ambient temperature of the modules by 18 hours. The variation of the temperature is alike as the 6 hours temperature performance.

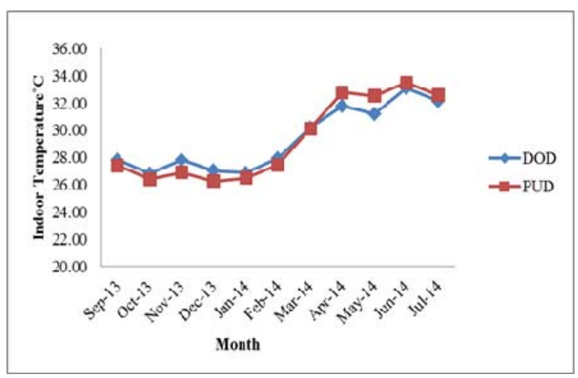

Fig.6. Mean Monthly indoor ambient temperature by18 hours

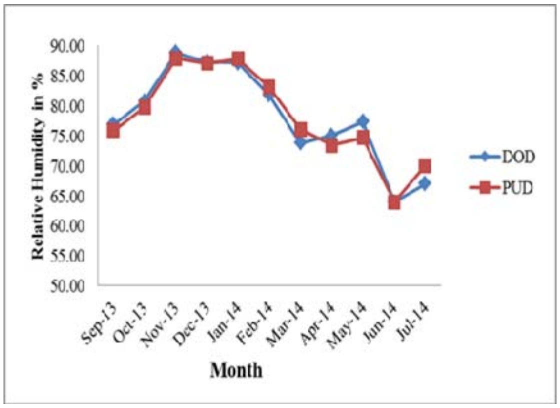

Fig.7 Indoor Relative humidity of the two modules by 6 hours

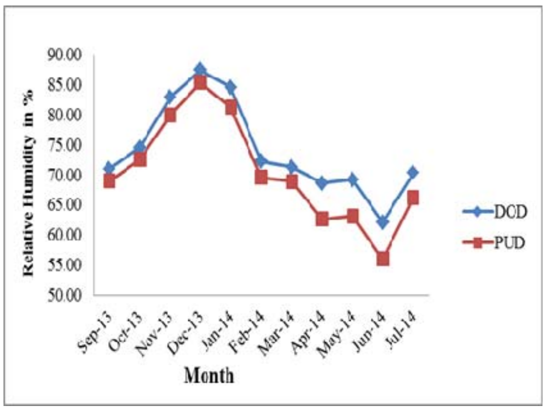

Fig.8 Indoor Relative humidity of the two modules by 12 hours

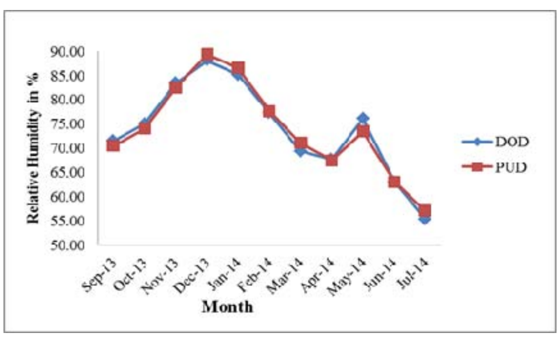

Fig.9. Indoor Relative humidity of the two modules by 18 hours

Fig.7 & 9 shows the Indoor Relative humidity of the two modules by 6 hour and 18 hour during the monitoring period. Relative humidity during these periods is showing a little difference. Measurement during 12 hours Fig.8 shows an obvious difference. The relative humidity increases from September and reaches a maximum in December and then deceases up to June. From June to July there is an increase. During the winter months DOD module attains a maximum of 87.5% and the PUD module attains a maximum of 85.5%. During the summer months the PUD module reaches minimum of 56% and the DOD reaches a minimum of 62%. The variation in the relative humidity is appreciable.

VI. CONCLUSION

In this Study two insulated roofs have been engaged. Insulated roofs are highly effective in reducing the indoor temperature. The PUD and DOD module performs well. The DOD performs in a better way than the PUD module during the summer months. The indoor temperature of DOD is 7 - 9 ˚C lesser than the outdoor temperature. The indoor relative humidity is appreciable during the summer months in the insulated roofs. The DOD has a higher relative humidity of 6% than PUD during summer. The cost of PUD roof per square meter is Rs.1500/- Whereas the cost of DOD roof per square meter is Rs. 1000/- Hence the newly designed roof is comparatively less cost. Regarding the thermal performance and cost effectiveness the DOD roof is superior to the other one.

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	diameter and 1' height)			
3.	Cover Slab for Pit	1	410	410
4.	Cement	3	325	975
5.	Mason Charges	L.S.	---	1000
6.	L-Angulars, Steel Strips	60 kg	45	2700
7.	Welding Charges	L.S.	--	300
8.	Indian Water Closet	1	400	300
9.	Plumbing Items (PVC pipe, P-Trap)	L.S.	---	250
10.	FRP Sheet	112 sft	10/sft	1120
11.	Cement Bricks, Sand and Aggregate	L. S.	---	550
Total Cost				**9335**
(Rupees Nine Thousand Three Hundred and Thirty Five only)				

vII. Conclusions

Low–Cost Lavatories promote better human health and also improve the quality of life of rural people through improved sanitation measures. As these lavatories can be constructed to serve for various purposes such as house hold, public places, institutions, at the places of religious festivals etc., the hygiene could be maintained. *Further, they serve the society and support in order to have a* ***cleaner India*** *to achieve* ***Swach Bharath*** *to fulfill the dream of our* ***Honorable Prime Minister, Mr. Narendra Modi.***

Acknowledgment

The authors are thankful to Mr. A. Madhukar Reddy, Secretary of S. R. Engineering College, Warangal for his motivation and encouragement to carry out 'Socially Relevant Projects' by applying engineering knowledge. Further, the authors are grateful to him for providing financial assistance to accomplish the project work. Also, authors are indeed thankful to all the staff members of the institution who are directly or indirectly helped to complete this task.

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UNDRAINED SHEAR STRENGTH OF COHESIVE SOILS AT CONSISTENCY LIMITS

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Abstract—Shear strength of cohesive soils at consistency limits is considered to be constant and various apparatus are designed based on this, for easy determination of consistency limits. But there is a significant difference in the shear strength values quoted by different researchers. This study mainly deals with the systematic approach of determining the undrained shear strength of cohesive soil at consistency limits. The density and shear strength variation of the soil with variation of water content between consistency limits has been evaluated. Based on which it is under stood that the shear strength and density decreases with increase in water content of a soil-water mixture. The values of shear strength at consistency limits are observed to be within the range suggested in literature.

Key words— Consistency limits, Undrained shear strength, Vane shear test, Dry density

I. Introduction

Consistency limits plays an important role in defining the behavior of cohesive soil. Efforts are made by various researchers to propose correlations between consistency limits and other index properties of soil [1, 2]. Researchers on other hand worked to device various instruments for easy and accurate determination of consistency limits. These instruments works based on the shear strength of the soil at consistency limits. The undrained shear strength of soils at consistency limits plays a significant role in devising the mechanism of these instruments. But literature suggests a range of shear strengths 0.5-5.6 kPa and 20-320 kPa for soils at liquid limit and plastic limit respectively [3, 4, 5, 6]. An attempt is made in this study to determine exactly the shear strength of soil between consistency limits in a systematic manner.

II. Materials and Methods

Two locally available cohesive soils having plasticity index 20 and 33 were selected for this study. The various geotechnical properties of selected soils are summarized in table 1 and the grain size distribution of the soils are depicted in figure 1. The undrained shear strength of these soil are determined using laboratory vane shear test as specified in [7]. A range of water contents is selected between the consistency limits (Liquid and Plastic limits) of the soil and is placed in a cylindrical mould of 38mm Ø x 76mm, with a small 1mm hole at its bottom.

TABLE 1. Various Geotechnical properties of selected soils.

Properties	Red soil	Black soil
Specific Gravity (G)	2.69	2.635
Hygroscopic Water Content (%)	5	12.5
Liquid limit (LL)	40	66
Plastic limit (PL)	20	33
Plasticity Index (%)	20	33

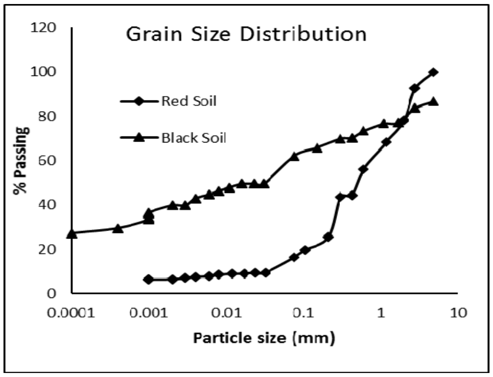

Figure 1. Grain size distribution of selected soils.

The soil mixed at selected water content is placed in three layers and every time it is tamped 25 times to remove air voids present in the soil, which creates a slight compactive effort that densifies the soil to an extent. This density mainly depends on the type of soil and has its effect on the strength characteristics determined using vane shear. Hereafter the variation in dry density is also considered significant in this study.

III. Results and Discussion

Figures 2 and 3 depict the variation of shear strength and bulk density of red soil respectively with variation of water content between consistency limits. Corresponding values are summarized in table 2. From figures, it can be observed that the shear strength and the bulk density decreases with increase in water content of the soil sample. Similar variations are observed in case of black soil as depicted in figures 4 and 5 and are summarized in table 3. Decrease in shear strength is obvious as the increased water has no resistance to shear force. And the decrease in density is a result of low specific gravity of high water content soil mixture. Similar results are observed in case of [8].

It can be observed that the shear strength of soils considered in this study, at liquid and plastic limit fall in the range of 0.5 – 5.6 kPa and from 20 – 320 kPa respectively as reported by [3, 4, 5, 6]. The variation of shear strength observed in figures 2 and 4 follows an exponential manner and the fitting the trend result in equation as shown in (1). Using this equation the strength of soil can be predicted at any water content between consistency limits. The values of shear strength, the constants of exponential equation and accuracy of exponential fitting R2 are reported in table 4.

$$S_u = ce^{-mw} \quad (1)$$

where, Su= undrained shear strength

c & m = constants depending on soil type

w = water content between consistency limits

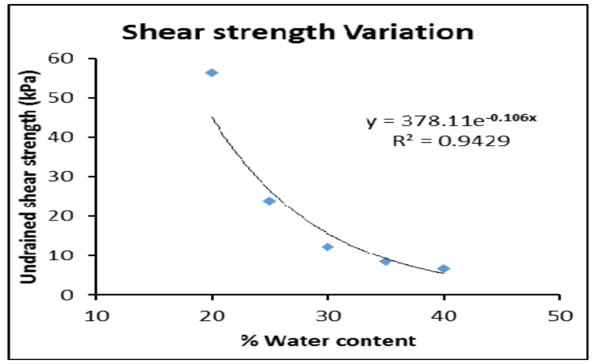

Figure 2. Shear strength variation of Red soil.

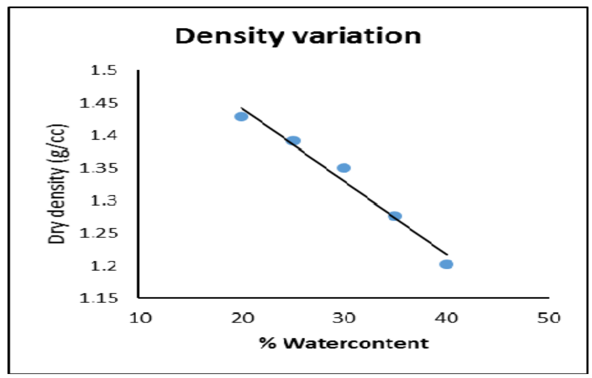

Figure 3. Density variation of Red soil.

TABLE 2. Summary of shear strength and density results of Red soil.

% Water content	Shear strength (kPa)	Dry density (g/cc)
20	56.291	1.429
25	23.932	1.391
30	12.152	1.351
35	8.477	1.276
40	6.6346	1.203

ANALYSIS OF SPEED CHARACTERISTICS ON VIDYA PATH CHANDIGARH-A CASE STUDY

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Abstract- Speed is considered as a quality measurement of travel as the drivers and passengers will be concerned more about the speed of the journey than the design aspects of the traffic. It is an important parameter in transportation as it relates to safety, time, comfort, convenience, and economics. Spot speed studies are done to estimate the distribution of speeds of vehicles in a stream of traffic at a particular location. The data collected from the spot speed is to be used for assessing general speed trends and for setting speed limits. This paper presents the analysis of spot speed of the vehicles travelling on the Vidya Path, Chandigarh.

Index terms: spot speed, speed percentiles

I. INTRODUCTION

Speed defines the distance travelled by user in a given time, and this is a vibrant in every traffic movement. The actual speed of traffic flow over a given route may fluctuated widely, as because at each time the volume of traffic varies. A typical unit of speed is kilometers per hour (Kmph). Speed is considered as a quality measurement of travel as the drivers and passengers will be concerned more about the speed of the journey than the design aspects of the traffic. . It is defined as the rate of motion in distance per unit of time. Mathematically speed or velocity v is given by, $v = d/t$ where, v is the speed of the vehicle in m/s, d is distance traveled in m in time t seconds. Speed of different vehicles will vary with respect to time and space.

Spot speed is the instantaneous speed of a vehicle at a specified location. Spot speed can be used to design the geometry of road like horizontal and vertical curves, super elevation etc. Location and size of signs, design of signals, safe speed, and speed zone determination, require the spot speed data. Accident analysis, road maintenance, and congestion are the modern fields of traffic engineer, which uses spot speed data as the basic input. Spot speed can be measured using an endoscope, pressure contact tubes or direct timing procedure or radar speedometer or by time-lapse photographic methods. **(Source: Evaluation of Traffic Characteristics: A Case Study by Arash Moahadkhani Roshandeh)**

A. Study area

Area of study is taken along the Vidya Path in Chandigarh as shown in Fig. 1. The condition of the road is average and there are many potholes and cracks which are clearly visible along the length of the road due to which there is wide fluctuation in the speed adopted by the different vehicles.

Fig. 1

B. Objective of the study

The objective of the study is to analyse the speed characteristics along the study stretch and to determine the speed percentiles, which are useful in designing and regulating the traffic.

II. DATA COLLETION

Various methods of data collection for spot speed are: (1) stopwatch method, (2) radar meter method, or (3) pneumatic road tube method. The data gathered in spot speed studies are used to determine vehicle speed percentiles, which are useful in making many speed-related decisions. Spot speed data have a number of applications which are as:

i. Determining existing traffic operations and evaluation of traffic control devices
   a) Evaluating and determining proper speed limits
   b) Determining the 50th and 85th speed percentiles
   c) Evaluating and determining proper advisory speeds
   d) Establishing the limits of no-passing zones
   e) Determining the proper placements of traffic control signs and markings
   f) Setting appropriate traffic signal timing

ii. Establishing roadway design elements
   a) Evaluating and determining proper intersection sight distance
   b) Evaluating and determining proper passing sight distance
   c) Evaluating and determining proper stopping sight distance

iii. Assessing roadway safety questions
   a) Evaluating and verifying speeding problems
   b) Assessing speed as a contributor to vehicle crashes
   c) Investigating input from the public or other officials

iv. Monitoring traffic speed trends by systematic ongoing speed studies

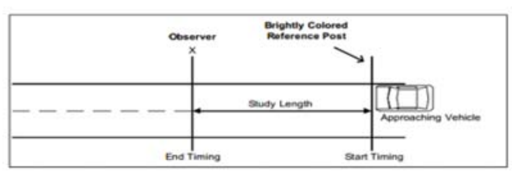
v. Measuring effectiveness of traffic control devices or traffic programs, including signs and

vi. Markings, traffic operational changes, and speed enforcement programs.

Data was collected manually on the study stretch for 2- wheelers and 4-wheelers using stopwatch method as shown in Table 1 and Table 2. 100 m stretch was taken on the Vidya Path.

Calculation of spot speed by stopwatch method

The stopwatch method can be used to successfully complete a spot speed study using a small sample size taken over a relatively short period of time. The stopwatch method is a quick and inexpensive method for collecting speed data.



A stopwatch spot speed study includes five key steps:

a) Obtain appropriate study length.
b) Select proper location and layout.
c) Record observations on stopwatch spot speed study data form.
d) Calculate vehicle speeds.
e) Generate frequency distribution table and determine speed percentiles.

III. DATA ANALYSIS

The analysis of the study is very important to achieve the key objectives. After Data Collection, analysis is done. Collected data was compiled in tabular form and the following steps have been taken to analyse the data. Analysis is done in order to find the key parameters which may include Mean speed, 85th Percentile Speed, 98th Percentile Speed, 50th Percentile Speed, Median, Mode, Speed Variance etc. Some values are directly obtained from the data and some can be drawn from the graphs. 50th percentile speed represents the average speed of the traffic stream. The 85th percentile is the speed at which 85% of the observed vehicles are travelling at or below the particular speed. This percentile is used in evaluating/recommending posted speed limits based on the assumption that 85% of the drivers are travelling at a speed they perceive to be safe. The 98th percentile speed is the speed at which 98% of observed vehicles are travelling at or below that particular speed. The 98th percentile is the design speed.

Table 1: Spot speed study for 2-wheelers on Vidya Path Chandigarh

WEATHER:GOOD DATE: 12-NOV-2014 ROAD SURFACE:AVERAGE TIME: 3.00-4.00pm				
SPEED CLASS LIMITS (K.P.H)	MID POINT SPEED (K.P.H)	NO. OF VEHICLES	FREQUENCY %	CUMULATIVE FREQUENCY %
18-23	20.5	3	7.5	7.5
23-28	25.5	4	10	17.5
28-33	30.5	8	20	37.5
33-38	35.5	11	27.5	65
38-43	40.5	0	0	65
43-48	45.5	9	22.5	87.5
48-53	50.5	0	0	87.5
53-58	55.5	0	0	87.5
58-63	60.5	5	12.5	100

Table 2: Spot speed study for 4-wheelers on Vidya Path Chandigarh

WEATHER:GOOD DATE: 12-NOV-2014 ROAD SURFACE:AVERAGE TIME: 3.00-4.00pm				
SPEED CLASS LIMITS (K.P.H)	MID POINT SPEED (K.P.H)	NO. OF VEHICLES	FREQUENCY %	CUMULATIVE FREQUENCY %
18-23	20.5	10	25	25
23-28	25.5	5	12.5	37.5
28-33	30.5	9	22.5	60
33-38	35.5	9	22.5	82.5
38-43	40.5	0	0	82.5
43-48	45.5	4	10	92.5
48-53	50.5	0	0	92.5
53-58	55.5	0	0	92.5
58-63	60.5	3	7.5	100

A. Graphical analysis

The Cumulative percentages calculated for 2-wheelers and 4-wheelers in Table-1 and Table-2 respectively is plotted against the upper limit of the various speed groups as shown in Fig. 2 and Fig. 3. A smooth S-shape curve is obtained which is called the cumulative speed curve.

The vertical axis of the curve indicates the percentage of the number of vehicles travelling at or below the indicated speed.

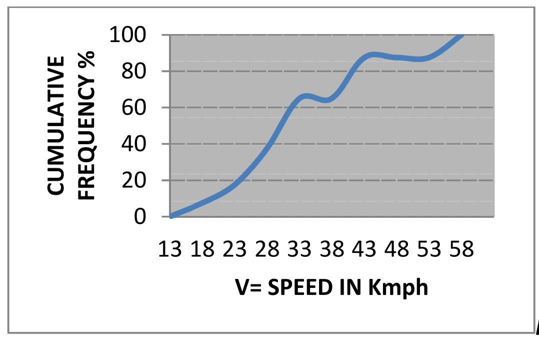


Fig. 2 Cumulative Speed Distribution for 2-wheelers

From graphical analysis, the following results have been obtained for 2-wheelers:

- **15th percentile speed- 22 Kmph**
- **50th percentile speed- 31 Kmph**
- **85th percentile speed-42 Kmph**
- **98th percentile speed-58 Kmph**

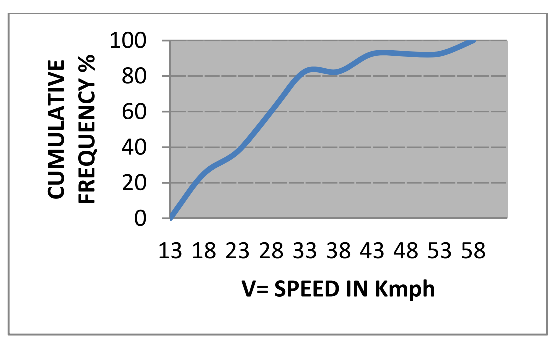


Fig. 3 Cumulative Speed Distribution for 4-wheelers

From above graph, the following results were obtained for 4- wheelers:

- **15th percentile speed- 16 Kmph**
- **50th percentile speed- 26 Kmph**
- **85th percentile speed- 40 Kmph**
- **98th percentile speed- 58 Kmph**

IV. CONCLUSIONS

The following conclusions have been drawn from the study:

1) The maximum speed limit on the road is equal to 58 Kmph for 2-wheelers as well as for 4-wheelers.
2) All the 2-wheelers and 4-wheelers plying on the road moved with a speed ranging between 13-58 Kmph.
3) Maximum 2-wheelers moved with the average speed of 31 Kmph and maximum number of 4-wheelrs moved with an average speed of 26 Kmph.
4) The 85th percentile speed i.e. the critical speed for 2-wheelers is 42 Kmph and for 4-wheelers is 40 Kmph.
5) The 15th percentile speed i.e the minimum speed for 2 wheelers is 22 kmph and for 4- wheelers is 16 Kmph.

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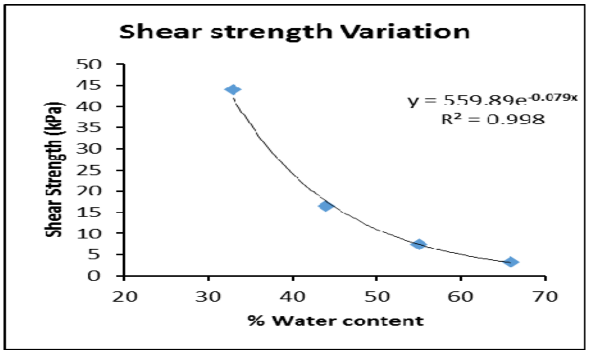

Figure 4. Shear strength variation of Black soil

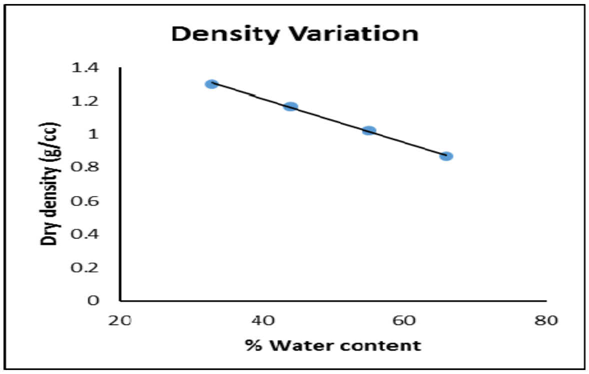

Figure 5. Shear strength variation of Black soil

TABLE 3. Summary of shear strength and density results of Black soil.

% Water content	Shear strength (kPa)	Dry density (g/cc)
33	43.97	1.3
44	16.5	1.17
55	7.46	1.022
66	3.22	0.867

TABLE 4. Summary of exponential fitting for shear strength variation

Soil type	S_u Liquid limit (kPa)	S_u Plastic limit (kPa)	constant 'c'	constant 'm'	R^2 value
Red soil	6.635	56.291	378.11	0.106	0.943
Black soil	3.220	43.970	559.89	0.079	0.998

Conclusions

Within the limited work done in this study, the following conclusions are drawn:

- The shear strength of a given soil between consistency limits depends on soil type and lies in the limits as suggested in literature.
- The variation of shear strength of soil between consistency limits follow an exponential trend and corresponding fitting equation helps in estimating strength at any corresponding water content.

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WIND INDUCED INTERFERENCE EFFECTS ON NATURAL DRAUGHT COOLING TOWER

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Abstract— The wind load is always the dominant load in the design of the cooling tower due to its large size, complex geometry and thin wall. In a series of wind tunnel tests, the wind-induced stresses in cooling towers situated in an arrangement of typical power plant buildings, are investigated and compared to the stresses in an isolated tower. Interference factors are developed to quantify the stress increase due to the group effect. The design wind pressure at various level of tower measured from gust factor method and peak wind method. The variation of the flow-induced forces produced on each tower by the other one is referred to as interference. Using the registered pressures, numerical linear and nonlinear analyses were performed to calculate the structural responses of the isolated and grouped towers. The net coefficient of pressure distribution was plotted for various angle of wind incidence. From the study, it was found that Meridional stress is 8.86% more and circumferential stress is about 9.43% more in present study compared to existing NDCT model. Also, the highest net pressure coefficient is obtained as 1.436, when the wind incidence angle is about 0°. The value approaches to a minimum value of about -0.934, when the wind incidence angle is about 330° and occurring at about 105° angle. The results of present study are in close agreement with the existing structure. Thus, the numerical model is validated.

Index Terms— Wind interference, Aero-elastic wind tunnel tests, Stress responses, cooling tower.

I. INTRODUCTION

Natural Draught Cooling Towers are Hyperbolic Reinforced Concrete (RC) shell structures used in thermal and nuclear power plants as cooling devices. In the last decade, Natural Draught Cooling Towers became even more inevitable means for the economic generation of electricity under environmental aspects.

The hyperboloid of revolution can be generated by rotating a hyperbola about its directrix. Shells of this type are built throughout the world as cooling towers to lower the temperature of coolants (water) used in electricity generating plants and chemical plants. This type of shell has proven to be efficient for use in Reinforced Concrete Natural Draught Cooling Towers for the conservation and reuse of the coolant.

In the present study, the sizing of cooling tower is taken based on the thermal design report and capacity of cooling tower. In this study 500MW capacity of Natural Draught Cooling Tower for Thermal Power Plant is taken. The tower is analyzed using the commercially available Staad Pro v8i software.

The wind load is calculated as per IS 11504 and IS 875 for the analysis of isolated case of cooling tower. For Interference case of cooling tower based on the wind tunnel study report pressure co-efficient is considered and it is multiplied with the dynamic wind pressure and corresponding surface area. Modal analysis is done for dynamic seismic load as per IS 1893:2002. In this study the cooling tower is analyzed for both wind and seismic loads.

DESCRIPTION OF THE COOLING TOWER

General Arrangement

Cooling tower consists of RCC shell, which is hyperbolic, shaped except for the portion at bottom, which is conical. The shell is supported on 44 pairs of diagonal columns in RCC, which are raked tangential to the Meridional profile of the shell at its bottom; the open system of columns also provides the air inlet opening. The diagonal columns rest on RCC pedestals, which are in the same inclined plane. The RCC pedestals are an integral part of the pond wall in RCC, which retains the re-cooled water. Pond wall spanning between the pedestals will be considered. At bottom, a ring shaped horizontal RCC ring foundation below the pond wall and pedestal is provided. The soil bearing capacity for ring foundation is considered 50t/m2 at depth of 5.0m from FGL.

RCC platform 1.2m wide all around the tower at top shall be provided, which is accessed by two M.S. cage ladders. These ladders spring from the top of an RCC staircase. The ladders are on outside up to throat level and then on the inside up to the top, with inter connection through a landing platform and access door at the throat level. The RCC staircase leads from ground level up to the level of water distribution system. Internal walkways in RCC are provided on periphery of tower cantilevering from the shell at the hot water distribution level and on the hot water distribution duct inside the tower.

Functional Requirements of the Cooling Tower

Duty and Capacity

a. Quantity of circulating water per tower : 60000 cum/hr
b. Type of tower : Natural draught (hyperbolic)
c. Period of operation : 24 hrs continuous
d. Hot water inlet temperature : 43.0 degree C
e. Re-cooled water outlet temperature : 32.5 degree C
f. Design relative humidity : 50%
g. Design ambient wet bulb temperature : 27 degree C
h. Design wind speed : 39 m/sec

Important Dimensions

a) **Elevations (in meters)**

i) Pond sill +0.00
ii) Ground level -0.30
iii) Basin floor at periphery -2.30
iv) Working level of water -0.30
v) Top of the tower +160.00
vi) Throat of the tower +129.00
vii) Bottom of ring beam +8.00
viii) Top of fill +14.00
ix) Bottom of fill +8.00
x) Bottom of drift eliminators +15.55

Table 5.1 Elevation details of Natural Draught Cooling Tower

b) Internal Diameters of the Tower (in metres)

i) Diameter at sill level +122.00
ii) Diameter at throat level +67.10
iii) Diameter at top of tower +68.50

Accordingly, the profiles of the towers are as shown in fig. 1 all the details i.e. height of tower above ground level, height from throat to top of the tower, height of air vent, Diameter at sill level, Diameter at throat level, Diameter at top of tower indicated in the following fig.1, are in meters.

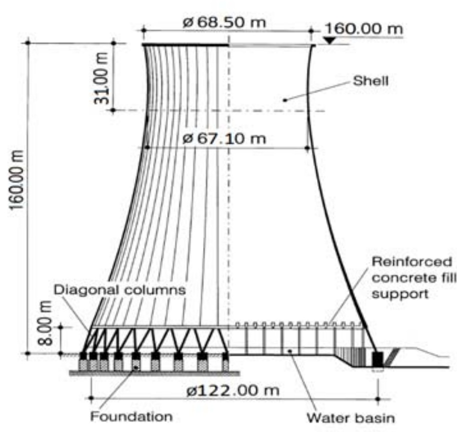


Fig: 1. Profile of the cooling tower

In this case, wind load is calculated by the following two methods and the results are tabulated in table 1.

a. Gust factor method
b. Peak wind method

Table 1 Design wind pressure at various levels of cooling tower

	Peak Wind Method				Gust Factor Method			
Level	K2 Table 2	Vz	PzN/m²	Pz KN/m²	K2 Table33	Vz	Pz =0.6Vz²	Pz G*Coeff.
8.836	0.9300	52.452	2360.54	2.361	0.6700	37.7	2140.0	2.140
10	0.93	52.452	2360.54	2.361	0.67	37.7	2140.0	2.140
11.589	0.9427	53.168	2425.50	2.425	0.6859	38.6	2242.7	2.243
15	0.97	54.708	2567.96	2.568	0.72	40.6	2471.3	2.471
15.575	0.9734	54.902	2586.25	2.586	0.7234	40.8	2495.1	2.495
19.715	0.9983	56.303	2719.92	2.720	0.7483	42.2	2669.3	2.669
20	1	56.4	2729.26	2.729	0.75	42.3	2681.6	2.682
23.858	1.0154	57.270	2814.14	2.814	0.7654	43.1	2793.0	2.793
28.004	1.0320	58.2057	2906.82	2.907	0.7820	44.1	2915.4	2.915
30	1.04	58.656	2951.97	2.952	0.79	44.5	2975.2	2.975
32.154	1.0465	59.0204	2988.77	2.989	0.7965	44.9	3024.1	3.024
36.308	1.0589	59.7233	3060.37	3.060	0.8089	45.6	3119.5	3.120
40.4658	1.0714	60.4268	3132.90	3.133	0.8214	46.3	3216.4	3.216
44.6285	1.0839	61.1311	3206.36	3.206	0.8339	47.0	3315.0	3.315
48.7962	1.0964	61.8363	3280.76	3.281	0.8464	47.7	3415.1	3.415
50	1.1	62.04	3302.41	3.302	0.85	47.9	3444.3	3.444
52.9693	1.1042	62.2745	3327.42	3.327	0.8542	48.1	3478.1	3.478
57.1484	1.1100	62.6044	3362.77	3.363	0.8600	48.5	3525.9	3.526
61.3338	1.1159	62.9349	3398.37	3.398	0.8659	48.8	3574.1	3.574
65.5262	1.1217	63.2660	3434.21	3.434	0.8717	49.1	3622.8	3.623
69.7261	1.1276	63.5976	3470.31	3.470	0.8776	49.4	3671.8	3.672
73.9341	1.1335	63.9298	3506.67	3.507	0.8835	49.8	3721.3	3.721
78.1507	1.1394	64.2628	3543.29	3.543	0.8894	50.1	3771.2	3.771
82.3765	1.1453	64.5965	3580.18	3.580	0.8953	50.4	3821.5	3.822
86.6120	1.1513	64.9309	3617.35	3.617	0.9013	50.8	3872.3	3.872
90.8577	1.1572	65.2661	3654.79	3.655	0.9072	51.1	3923.5	3.924
95.1139	1.1632	65.6022	3692.53	3.693	0.9132	51.5	3975.2	3.975
99.3806	1.1691	65.9391	3730.55	3.731	0.9191	51.8	4027.4	4.027
100	1.17	65.988	3736.09	3.736	0.92	51.8	4035.0	4.035
103.658	1.1729	66.1530	3754.80	3.755	0.9229	52.0	4060.7	4.061
107.945	1.1764	66.3465	3776.79	3.777	0.9264	52.2	4091.0	4.091
112.241	1.1798	66.5403	3798.89	3.799	0.9298	52.4	4121.4	4.121
116.546	1.1832	66.7345	3821.10	3.821	0.9332	52.6	4152.0	4.152
120.856	1.1867	66.9290	3843.41	3.843	0.9367	52.8	4182.7	4.183
125.171	1.1901	67.1237	3865.80	3.866	0.9401	53.0	4213.6	4.214
129.269	1.1934	67.3086	3887.13	3.887	0.9434	53.2	4243.0	4.243
133.370	1.1967	67.4936	3908.53	3.909	0.9467	53.3	4272.6	4.273
137.690	1.2002	67.6886	3931.14	3.931	0.9502	53.5	4303.8	4.304
142.009	1.2036	67.8834	3953.80	3.954	0.9536	53.7	4335.2	4.335
146.328	1.2071	68.0783	3976.53	3.977	0.9571	53.9	4366.7	4.367
150	1.21	68.244	3995.91	3.996	0.96	54.1	4393.5	4.394
150.646	1.2104	68.2658	3998.48	3.998	0.9605	54.1	4398.2	4.398
154.965	1.2130	68.4120	4015.61	4.016	0.9639	54.3	4429.9	4.430
158.565	1.2151	68.5338	4029.93	4.030	0.9668	54.5	4456.5	4.457
160	1.24	69.936	4196.52	4.197	1	56.4	4767.3	4.767

Wind interference case:

a. Surrounding Structures

The plan view of the proposed Bellary thermal power station is shown in figure 3 and 4, the figure shows the two cooling tower, two chimneys, and other structures such as ESP, Boilers, and power house. For simulation of vicinity terrain around the proposed cooling towers, all the adjoining structures as mentioned above are to be included.

b. Site Location

The site of Bellary thermal power plant stage – II expansion is located at Bellary district in the State of Karnataka, India. The general terrain around the TPS location is in category 2 with open terrain with well scattered obstructions having heights generally between 1.5 to 10m

c. Wind Speed

The basic wind speed (Vb), from figure 1 of IS: 875 (Part 3) – 1987, is 39m/sec at Bellary. Basic wind speed is based on peak gust velocity averaged over a short time interval of about 3 seconds and corresponds to mean heights 10 m above ground level in an open terrain (Category 2) for a 50 year return period. The basic wind speed is modified to include the following effects to get design wind velocity at a height (Vz) for the structure:

Fig.2 Picture of instrumented NDCT along with other nearby plant structures for interference study in the wind tunnel (typical orientation).

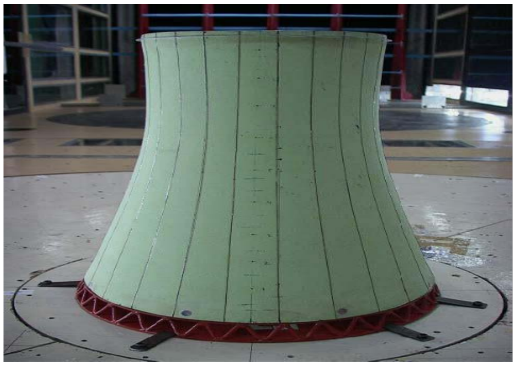


Fig 3 Isolated case of cooling tower

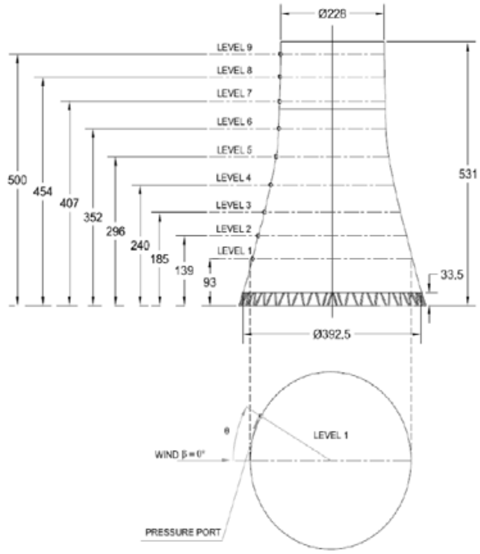
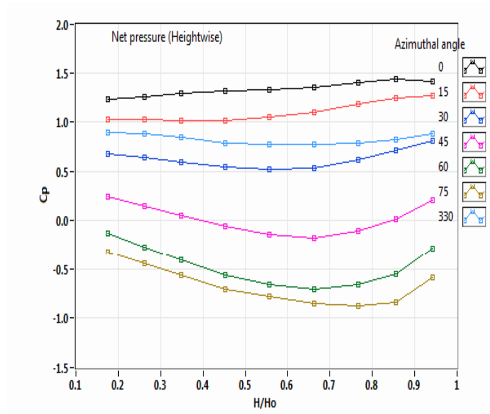


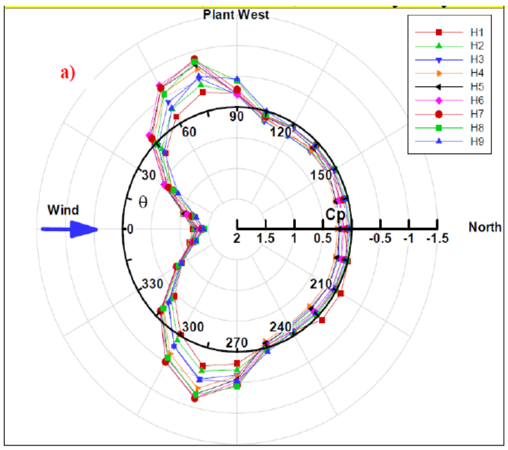
Fig.4 Sectional elevation of the pressure model of NDCT

A NDCT model of 1:300 scales was tested under simulated flow conditions for interference configurations. The mean pressure data has been obtained at nine different heights all around the periphery of the model in 15o interval.

The highest net pressure coefficient is obtained as 1.436, when the wind incidence angle is about 0°.

The minimum value of Cp is about -0.934, when the wind incidence angle is about 330° and occurring at about 105° angle in azimuth with respect to wind.





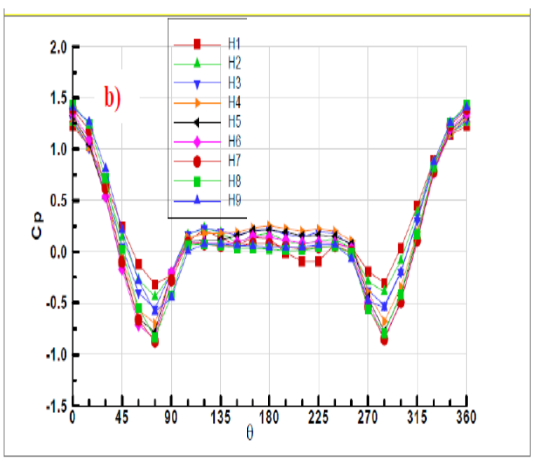


Fig.5 Interference case, wind incidence angle = 0 degree

a) Cp distribution along the periphery in polar plot

b) Cp distribution along the periphery in X-Y plot

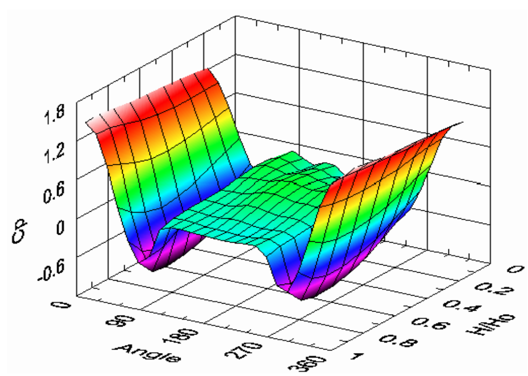
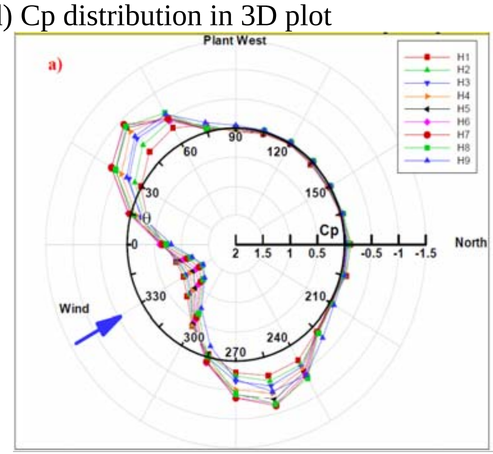


Fig.6Net pressure coefficient distribution on the NDCT for interference case, wind incidence angle = 0 degree

c) Cp distribution along the height

d) Cp distribution in 3D plot



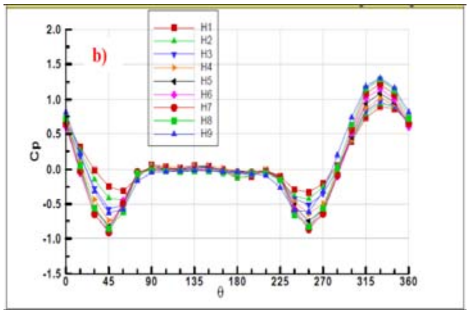
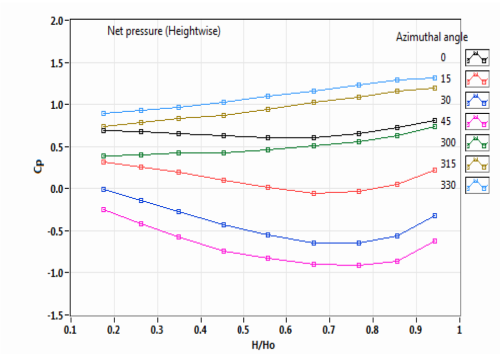


Fig.7 Interference case, wind incidence angle = 30 degree

a) Cp distribution along the periphery in polar plot

b) Cp distribution along the periphery in X-Y plot



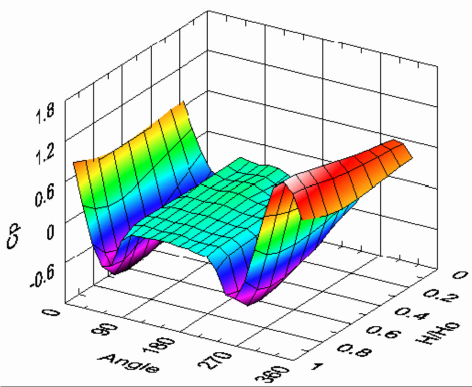
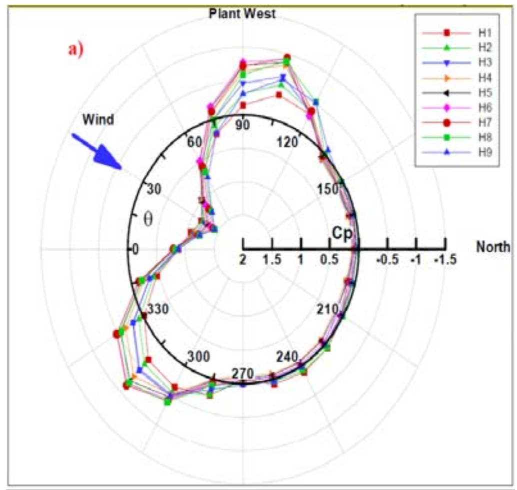


Fig.8 Net pressure coefficient distribution on the NDCT for interference case, wind incidence angle = 30 degree

c) Cp distribution along the height

d) Cp distribution in 3D plot



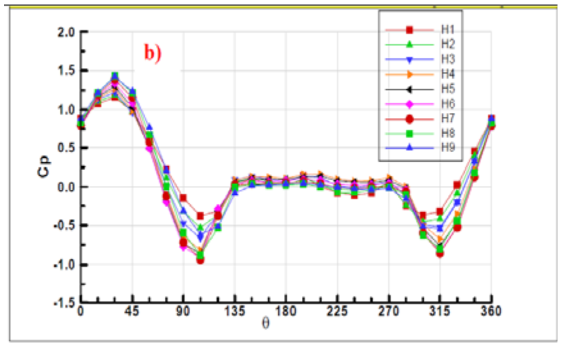
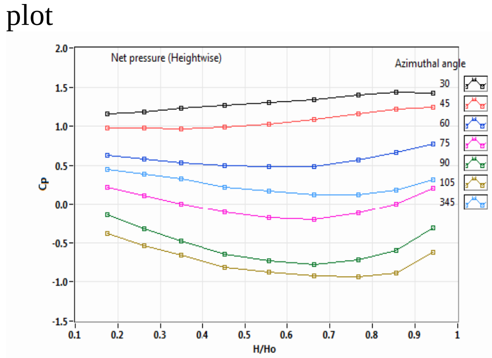


Fig.9 Interference case, wind incidence angle = 330 degree

a) Cp distribution along the periphery in polar plot

b) Cp distribution along the periphery in X-Y plot



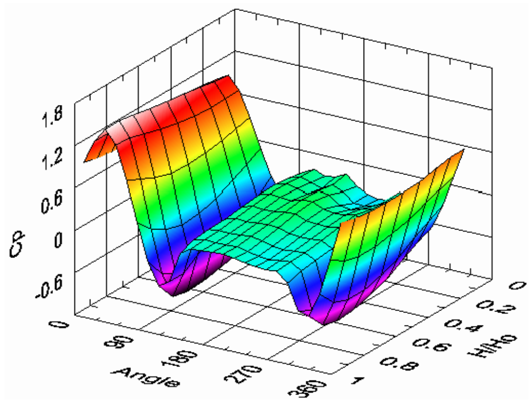


Fig. 10 Net pressure coefficient distribution on the NDCT for interference case, wind incidence angle = 330 degree

c) Cp distribution along the height

d) Cp distribution in 3D plot

Modeling and Meshing

The structure is modeled using beam and plate elements available in Staad Pro. v8i. The shells are meshed using quadratic 4 node plate element, raker column is modeled using 3D beam element and pedestal, pond wall is modeled by 4 node quadrilateral elements. The ring beam at the the base of the shell which is modeled by using 3 nodded triangular elements. The cooling tower shell is supported by diagonal columns called raker columns which are fixed at the base.

Finite element model of the problem generated using Staad Pro is shown in Fig. 11. Therefore the total number of the nodes and elements used in the entire model is 2948 and 2684 respectively. Node to node connection is used to join the elements and 88 numbers of 3D beam members are used to model the raker columns.

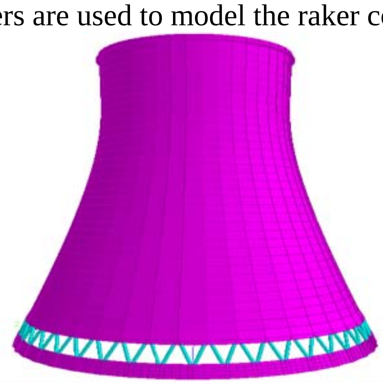


Fig. 11 Finite Element Model of cooling tower

Validation of the Model

Results of the numerical simulation are compared with that obtained by the existing cooling tower is given in Table 2. It can be seen that the deflection of the shell and Raker column predicted by present study is more by about 19.4% and 24% respectively.

The Meridional Stress distribution along the length and circumferential stress distribution at the ring beam level are shown in Fig. 12 & 13. It is observed that stresses obtained by the present study are more compared to the existing Natural Draught Cooling Tower. It can be observed that 8.86% more Meridional stress in present study compared to existing structure and in circumferential stress is about 9.43% more compared to existing structure.

It can be observed that the results of present study are in close agreement with the existing structure. Thus, the numerical model is validated.

Table 2 Validation Of The Numerical Model By Considering Displacement Due To Wind

Displacement in m due to wind load at extreme top level	Present study	Existing NDCT
shell	0.048	0.042
Raker column	0.0031	0.0025

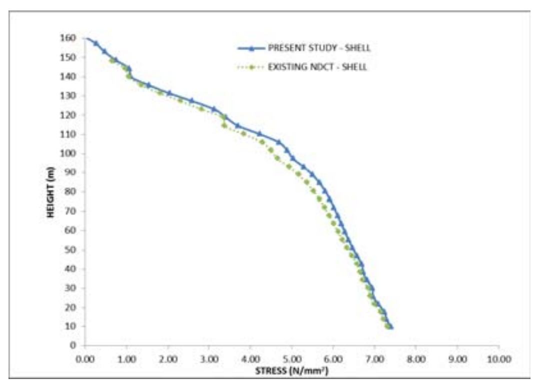


Fig.12 Meridional stress distribution

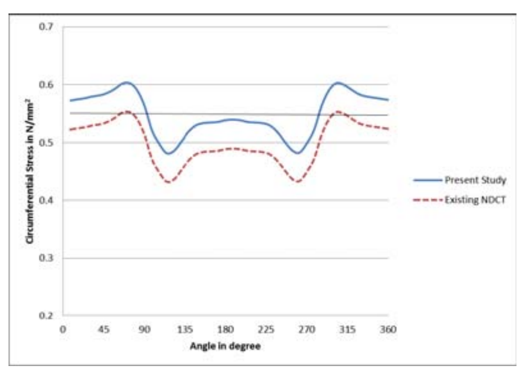


Fig.13 Circumferential stress distribution at ring beam level

Conclusions

Based on the present numerical investigation which includes circumferential pressure variation along the periphery as well as deflection control along the height of the tower for various wind incidence angle, the following conclusions are drawn:

The highest net pressure coefficient is obtained as 1.436, when the wind incidence angle is about 0°. The value approaches to a minimum value of about -0.934, when the wind incidence angle is about 330° and occurring at about 105° angle.

The deflection of the shell and Raker column predicted by present study is more by about 19.4% and 24% respectively compared to existing structure.

It can be observed that, Meridional stress is 8.86% more in present study compared to existing structure and circumferential stress is about 9.43% more compared to existing structure.

The results of present study are in close agreement with the existing NDCT. Thus, the numerical model is validated.

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