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"Affect and improvement of flexible pavement subgrade in frost affected areas"

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ABSTRACT
A frost heave distress on a roadway is caused by the upward movement of the pavement resulting from expansion of trapped water beneath the roadway. A significant heave can result in permanent damage of deformations and cracking with varying severities. These distresses can greatly affect the ride quality and leave motorists with an uncomfortable and unsafe ride.
Pavement distresses caused by frost heave action are usually unpredictable and can be very costly to repair. In order to mitigate frost heave action, designers must understand the different conditions in which water can become trapped within the frost depth. The presence of highly frost susceptible materials and improper drainage are key contributors to frost heaves, but other conditions such as the existing terrain and constructed cut-to fill transitions can also lead to potential frost heaves problems.
The objective of this paper is to investigate the causes of frost heaves and how to mitigate these effects during design and construction stages to avoid costly repairs down the road. It will also review the techniques for a frost heave field investigation and what rehabilitation techniques can be used to improve the frost heave protection.

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I. INTRODUCTION

GENERAL
LOCATION AREA:

The Sela Pass (more appropriately called Se La, as La means Pass) is a high-altitude mountain pass located in Tawang District of Arunachal Pradesh, India. It has an elevation of 4170m (13,800 ft). It connects Tawang Town to Tezpur and Guwahati and is the main road connecting Tawang with the rest of India. Tawang is situated at a distance of 78 km from Sela Pass while Guwahati is at a distance of 340 km. It is the main route to access Tawang town, and given the proximity of Tawang District with China,

Border Roads Organization (BRO) of India works hard to keep the pass open throughout the year. The pass has hardly any vegetation and is usually has long winter, the snow remains on the ground from November until the end of April and offers excellent and heavenly views all- round the year. During winter temperature in the pass can go down to −10°C.It is usually open throughout the year unless landslides or snow require the pass to be shut down temporarily. This part of the Eastern Himalayan range is pretty special for the Buddhists as it is believed that about 101 lakes exist in and around Sela pass and each of these lakes has a huge religious significance for the Buddhist community.

RESEARCH OBJECTIVES AND SCOPE
The specific objectives of this study are as follows:
1. To review literature
2. To investigate the physical and geotechnical properties of the soil sample collected from frost susceptible area (Sela Pass Tawang District).
3. To determine whether stabilisation or improvements is required in the soil sample collected from the selected road stretch.
4. To evaluate which stabilisation method/techniques most appropriate to improve the frost susceptible subgrade soils.

PAVEMENT DEFINITION

The structure of a road is actually called the "pavement." The more normal everyday use of this word, the surface next to the road we walk on, is a hangover from the Victorians. Until Victorian days, footways (which are the correct term) were mud tracks next to the road surface, or pavement. Once they started using the same rigid materials to construct the footway, people started to use the same word, pavement, to refer to it.

TYPES OF ROAD

There are two main types of pavement (road structure), flexible and rigid. Rigid roads are more complex to build, requiring more specialised equipment. The current preference within the road building industry is to flexible and composite roads (see later). Rigid roads consist of a thick concrete top surface. Where voids appear below the surface, in a flexible road, the surface will sink. This will not (normally) happen with a rigid or composite road. The danger here is that in exceptional circumstances where a large void has appeared, eventually the unsupported concrete will collapse. Composite pavements (road structure) are where a flexible layer has been added on top of the surface of a rigid road, or where a concrete layer exists below a bitumen top surface.

FLEXIBLE CARRIAGEWAY CONSTRUCTION

When a road is built, the surface is dug-out down to the designed depth of the intended road. Preparation is carried out on the ground now exposed below (such as compaction). The road itself will then be built up above, usually consisting of four layers: -

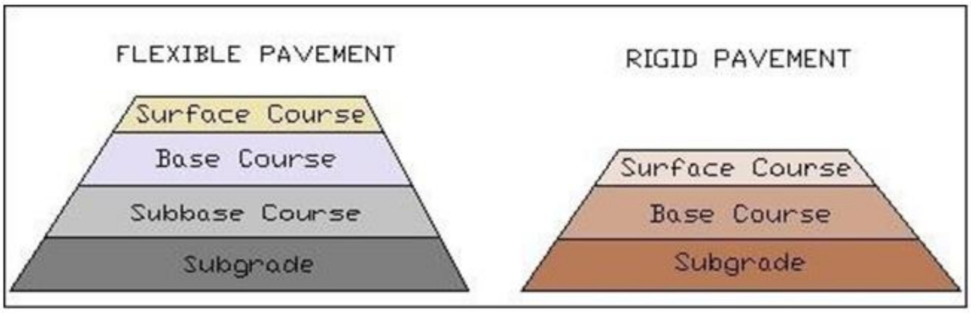

Fig.1. Cross Section of Pavement

The sub-grade is the ground below the road layers which is exposed once the ground has been dug out ready to build the road. The top level of this is termed the formation. The capping is a layer added above the sub-grade to protect it in new constructions. In this case, the top layer of the capping will constitute the formation. The four (typically) layers of the road above are termed (bottom to top) sub-base, base (formerly known as road base), base (formerly known as road base), binder course (formerly known as base course) and surface course (formerly known as wearing course).As the stress transmitted through the road structure from the vehicles above spreads and lessens with depth, stronger and more expensive materials are needed in the upper levels.

Additionally, the nearer the surface, the flatter the profile must be. This is obviously because an uneven surface will be uncomfortable for vehicle occupants and will wear more quickly (each time a vehicle hits a bump, it is in effect hammering the surface). These factors are the main reasons for the layered construction of the road.

MAJOR PARTS OF THE CROSS SECTION OF THE ROAD

THE SUB-BASE

The sub-base should be laid as soon as possible after final stripping to formation level, to prevent damage from rain or sun baking which could cause surface cracks. The fact that this is required when roads are constructed, emphasizes the importance of backfilling excavations quickly and properly and preventing ingress of moisture when roads have been excavated for utility works. The most commonly used material for use in sub-bases is termed Type 1. This is an unbound material made from crushed rock, crushed slag, crushed concrete, recycled aggregates or well burnt non-plastic shale. It contains particles of various sizes, the percentage of each size being within a defined range. Up to 10% may be natural sand. The predefined and calculated range of material sizes contained means that once compacted; it will resist further movement within its structure. In other words, it tends not to sink with time (though it will sink if not compacted properly when laid).

Other materials used for the construction of sub-bases include bituminous-bound materials and concrete and cement-bound materials, including wet-lean concrete.

SUB-BASE AND BASE MATERIALS

Again, Type 1 is most commonly used. Other materials include Type 2 and Type 3. Slag bound material used to be known as Wet Mix. It is a plant manufactured granular aggregate. It must be laid and compacted quickly, as this must take place within 6 hours of the GBS and activator components. Various other

materials are less commonly used. All materials on arrival from the plant must be protected from the weather, as drying or wetting changes the composition. They must be spread evenly. They are laid in layers of 110mm - 225mm compacted thickness, the thickness of the layers being gauged by various means including pegs and lines, sight rails and a guide wire. In initial build and reinstatement, the thickness of the layers depends on the compaction plant being used.
Bituminous base materials are either dense base macadam or rolled asphalt. Various concrete and cement - bound materials are used, the specifications for these being different to those applying for sub-base materials.

SURFACING

Both the surface course and binder course are included in the part of the road structure termed the surfacing. Occasionally the surfacing is laid as a single course. Normally, it is laid as two course binder and surface.

The binder course helps distribute the load of traffic above onto the base course, which is usually a weaker material. It also provides a flat surface onto which the normally thinner surface course is laid. In new construction, typical thickness is between 45mm and 105mm. Thickness may vary considerably where a new binder course is laid to an existing road structure for strengthening purposes. Stone sizes used are 20, 28 or 40mm.The thicker the binder course, the larger the stone size.
Materials used include open graded macadam, dense coated macadam and rolled asphalt. Surface courses are laid in a wide range of bituminous materials, ranging in thickness from 20 to 40mm. The material selected is dependent on the anticipated traffic intensity. The two most commonly used surface materials in the UK are HRA and SMA. Hot rolled asphalt is made with high fines and asphaltic cement content with crushed rock, slag or gravel added. Normal thickness is 40mm with 20mm coated chippings rolled into the surface providing better skid resistance.
Stone mastic asphalt is not as susceptible to rutting as other surfaces and reduces surface noise. Normal layer thickness is between 20mm and 40mm.

FROST HEAVES, ICE LENSES AND ITS EFFECTS
DEFINITION
Frost heaving (or a frost heave) is an upwards swelling of soil during freezing conditions caused by an increasing presence of ice as it grows towards the surface, upwards from the depth in the soil where freezing temperatures have penetrated into the soil (the freezing front or freezing boundary).

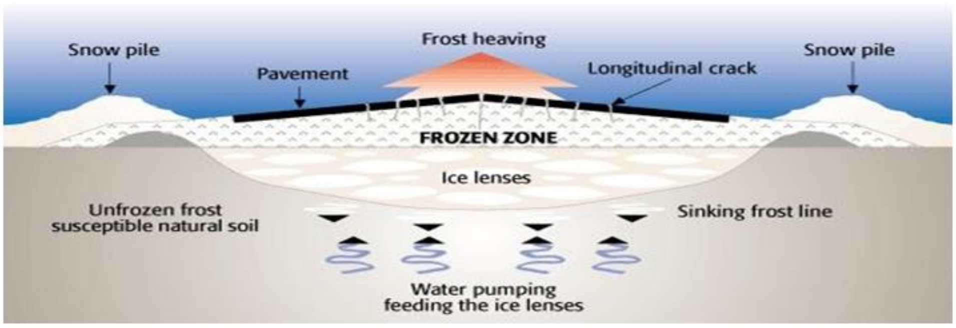

Fig.2. Effect of frost heave on pavement

ICE LENSE

An ice lens is formed when moisture, diffused within soil or rock, accumulates in a localized zone. The ice initially accumulates within small collocated pores or pre-existing crack, and, as long as the conditions remain favorable, continues to collect in the ice layer or ice lens, wedging the soil or rock apart. Ice lenses grow parallel to the surface and several centimeters to several decimeters (inches to feet) deep in the soil or rock.

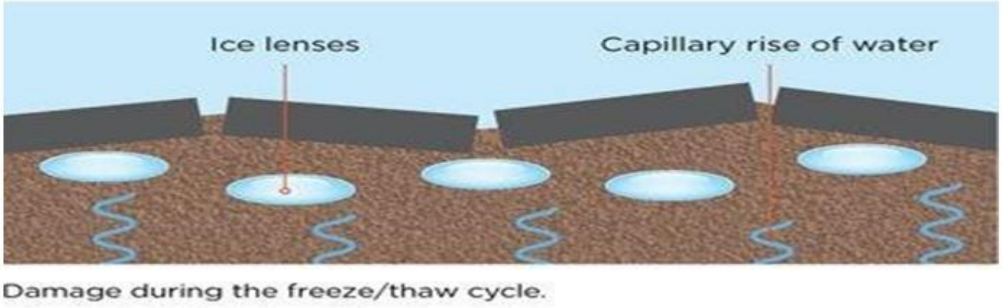

Fig.3. Effect of ice lens on pavement

EFFECTS

Frost heave affects soils greatly. Small lateral differences in snow cover, soil texture, vegetation, and topography can lead to differences in the amount of heave experienced by regions in the soil. Differential heaving causes layers to be displaced varying distances, leading either to the formation of wavy boundaries, or, in extreme cases, to the destruction of horizon boundaries altogether.

At the surface, differential heaving often forms a pattern of circular bulges with depressions between them. These small bulges are better-drained than the depressions, and they thus retain their heat longer during cold spells. Frost heave then begins in the depressions first, causing lateral pressure towards the centres of the bulges. This pressure displaces more soil and pushes the bulges higher, forming hummocks, circular mounds roughly 1-2 meters in diameter and up to 0.5 meters high. Hummocks are the most common type of ground pattern caused by frost heave, and they are a tell-tale marker of a soil prone to heaving Frost heave can also lead to the movement of clasts within the soil. When a freezing front encounters a clast in the soil, it bonds the clast to the soil.

As heaving progresses in the soil around the clast, a cavity is opened above the clast. The ad freeze strength of the bond between the frozen soil and the clast becomes great enough to lift the clast when the front is between 30 and 50% of the way down the clast. At this point, the clast begins to be heaved with the soil above it. A cavity opens in the unfrozen soil beneath the clast and is partially in filled with unfrozen soil that is perturbed by the upward movement of the clast.

When the soil is thawed, the clast drops down, but does not return to its original position because of the partial infilling of the cavity beneath it. This process is repeated in every freeze-thaw cycle and leads to the progressive upward migration of clasts through the soil. The amount of vertical motion depends upon the heaving strain in the soil (i.e. the amount of vertical expansion experienced by the soil), the length of the clast not yet ad frozen to the soil when clast heave begins (which in turn depends upon clast size, the strength of the ad freeze bond, and the shear strength of the unfrozen soil beneath the clast), and the amount of infilling of the cavity beneath the heaved clast .

Experiments by have found that in the course of seven freeze-thaw cycles, a 9x4x4 cm clast can be heaved over 12 cm. Though these experiments were under idealized conditions, used a very large clast size, and had some errors, they do serve as a indication of the order of magnitude of up freezing that can be expected. Up freezing can cause many problems for farmers, in that it brings rocks to the surface of the soil and often separates plants from their roots.

It can also help to explain the presence of gravel-sized clasts in the middle of loess-derived soils. Archaeologists have also begun to study up freezing, fearing that it might cause artefacts to be stratigraphically displaced in the soil.

The final major effect of frost heave occurs during seasonal thawing. A great deal of water accumulates in the upper soil horizons when ice lenses form. During thawing, the upper portion of the soil melts first. Because the bottom layers are still frozen at this point, the melt water cannot drain.

The soil becomes saturated and loses most of its strength. When soils supporting roads, fence posts, foundations, and other structures lose strength in this manner, the roads develop potholes while the fence posts and foundations can often become skewed. Thawing areas on slopes are also susceptible to land sliding.

Fig.4. Pot holes due to the frost heaves

SOIL STABILISERS

GEOSYNTHETICS

Geo-synthetics are synthetic products used to stabilize terrain. They are generally polymeric products used to solve civil engineering problems. This includes eight main product categories: geo-textiles, geo-grids, geo-nets, geo-membranes, geo-synthetic clay liners, geo- foam, geo-cells and geo-composites.

FLY ASH

Fly ash, also known as "pulverized fuel ash" in the United Kingdom, is one of the residues generated by coal combustion, and is composed of the fine particles that are driven out of the boiler with the flue gases. Ash that falls in the bottom of the boiler is called bottom ash.

CEMENT

Cement is a binder, a substance that sets and hardens and can bind other materials together. The word "cement" can be traced back to the Roman term opus caementicium, used to describe masonry resembling modern concrete that was made from crushed rock with burnt lime as binder.

LIME STABILISATION

Stabilization of soils with hydrated lime is applicable to far heavier clayey soils and is less suitable for granular materials and second it is used more widely as a construction expedient that is to prepare a soil for further treatment or to render a sufficient improvement to support construction traffic. As a temporary measure such modification or stabilization need not necessarily affected to the standards required for permanent construction. Quick lime or lime slurries may also be used for excessively wet or dry conditions respectively. It is therefore a very versatile stabilizer. In roads lime stabilization is widely used for sub-base construction or sub grade improvement.

EFFECTS ON ROAD PAVEMENT

Over the performance life period of pavement structure, it is vulnerable to different kinds of distresses. Permanent deformation (rutting) is one of the serious distresses in which pavement structure may be involved. A lot of research has been conducted so as to prevent diminishing pavements by rutting phenomenon. Both traditional and modern methods have been taken as measures to deal with such distress. One of the latter methods is related to reinforcing pavement structures by means of geo-synthetics. Using geo-synthetic materials as a reinforcing means in pavement structure mostly in road base and embankment is well investigated, and many researches on reinforcement of asphalt concrete are involved in prevention of reflection cracking.

II. LITERATURE REVIEW

GENERAL

Due to the inadequate drainage and frost heave action the pavement gets damaged which results in high maintenance cost and decreases the functioning of the pavement. Referring to this, we have studied these following papers:-

Yuanming Lai et al (2012) stated that 'Frost boiling and frost heave are the main factors that cause road damage in cold regions'. They brought into account a new kind of embankment structure, which consists of geo textile, crushed-rock layer and geo membrane. The key of the new structure is the porous crushed-rock layer which has smaller thermal conductivity with function of drainage and blocking of moisture migration induced by freezing. Laboratory and field results showed that the frost penetration and thawing depths of the new structural embankment are much smaller than those of the coarse-grained soil embankment. Also, the new embankment structure has lower water content in the upper layer and smaller frost heave and thawing settlement than the latter does. In addition, it has good drainage effect. Water coming from the road surface can be drained away from the embankment through the porous crushed-rock layer. All these states that the new structure consisting of the porous crushed rock layer is superior in frost damage mitigation to the normal structure used in cold regions.

Khan, A. and Mrawira, D. (2010) investigated an alternative approach for reducing frost penetration into the pavement layers by engineering a less conductive asphalt surface layer. The new approach involved replacement of conventional aggregate in the asphalt mix with lightweight aggregate (LWA). Because of its superior insulating behaviour, LWA-asphalt mix can reduce frost penetration into the underlying pavement layers. It also reduced the maintenance cost associated with frost damage.

TH Wang et al (2005) created a simulation model for investigating the changing progress of climate features; the thermal regime of subgrade in permafrost region in each month is got. The variations of thermal regime with time indicate the variation so frosts oil body. Then, on the basis of the thermal elastic theory in which the tensile failure is taken into account, the deformation features of subgrade in the permafrost region with consideration of body force and frozen-heave force are attained. Furthermore, the cause of formation for the longitudinal cracks in subgrade is explored. And it is exposed that the longitudinal cracks primarily arise in the central section of the sub grade and in the position near road shoulder.

This is identical to the situation in site. It is indicated that the building sub grade with low frost heaving soil, example for sandy gravel, is beneficial to prevent longitudinal cracks developing.

Janoo, V. et al (2002) investigated the potential for geo synthetic capillary barriers to reduce frost heave in soils by freezing upright, cylindrical soil specimens with horizontal disks of geo synthetics placed in them. During freezing, water was freely available at 25 mm above the base of 150 mm high specimens.

The geo synthetics were located 5 mm above the water supply. Geo composites comprising needle-punched polypropylene geo textiles sandwiching a drainage net, prepared in the same way as the moistened geo textiles containing soil fines, reduced frost heave when the soil water suction head in the overlying soil was 1800 mm or more.

However, the geo composites did not significantly reduce heave when the soil water suction head in the overlying soil was 800 mm or less. This is probably due to water migration between the two layers of soil, through the geo textiles and along the net of the geo composite.

Jean-Marie Konrad, Marius Roy (2000) enlightened the behavior of flexible pavements in cold regions. They stated that basic geotechnical engineering principles can be extremely useful for the analysis of pavement structures subjected to freezing and thawing. The frost susceptibility of subgrade soils should be analyzed for three distinct zones: (i) unsaturated zone, (ii) zone saturated by capillary action, and (iii) zone below the water table. Basic geotechnical engineering principles can be extremely useful for the analysis of pavement structures subjected to freezing and thawing. The frost susceptibility of sub grade soils should be analyzed for three distinct zones: (i) unsaturated zone, (ii) zone saturated by capillary action, and (iii) zone below the water table.

Christopher, B. R. et al (2000) evaluated the use of a special geo composite drainage net as a drainage layer and capillary barrier (to mitigate frost heave) on a section of road plagued with weak, frost-susceptible sub grade soils and poor pavement performance. The special geo composite drainage net that is being used has a higher flow capacity than conventional geo nets.

Jean-Marie Konrad, Norbert R. Morgenstern (1980) revealed that a freezing soil can be characterized by two parameters, the segregation-freezing temperature Ts and the overall permeability of the frozen fringe. During unsteady heat flow, the variation of these parameters with temperature produces rhythmic ice banding in fine-grained soils. At the onset of steady-state conditions, freezing tests conducted at a fixed warm end temperature showed that Ts was independent of the cold side step temperature.

ANALYSIS OF FROST HEAVES ON SUBGRADE IN PERMAFROST REGION
(China Journal of Highway and Transport, Vol. 18 No.2, April 2005)

By simulating the changing progress of climate features, the thermal regime of subgrade in permafrost region in each month is got. The variations of thermal regime with time indicate the variation so frosts oil body. Then, on the basis of the thermal elastic theory in which the tensile failure is taken into account, authors attain the deformation features of subgrade in the permafrost region with consideration of body force and frozen-heave force. Furthermore, the cause of formation for the longitudinal cracks in subgrade is explored. And it is exposed that the longitudinal cracks primarily arise in the central section of the sub grade and in the position near road shoulder. This is identical to the situation in site. It is indicated that the building sub grade with low frost heaving soil, example for sandy gravel, is beneficial to prevent longitudinal cracks developing.

Summary on literature review:-

Inadequate drainage of surface and subsurface water has significant impact on the pavement behavior which may cause long-term maintenance costs.

. Moisture damage causes seepage from culverts, perched water tables, and weathered zones in sedimentary rock, permeable glacial deposits, and ponded water.

. In the winter, the pavement structure modulus increases because of ice segregation in the unbound base or sub grade, and because of the influence of temperature on the viscosity of the asphalt or concrete.

. During spring thaw, the pavement foundation can become saturated with water from the thawing ice lenses, thus reducing the structural adequacy of the base or sub grade.

. Damage to the pavement structure will reveal itself on the surface in the form of fatigue cracking and rutting, owing to deformation in the base or sub grade.

. Pavement strength can be determined using nondestructive testing, such as a Falling Weight Deflect meter (FWD), or existing reduction factors.

III. METHODOLOGY

Site conditions and Visual Identification

The preliminary investigation shall involve collecting all relevant information pertaining to the proposed roadway.

i. Information on the pavement surface condition.

ii. Geotechnical data, to understand site condition like soil types, groundwater condition, water table depth, etc.

iii. Climate data, to identify precipitation/ rainfall type and intensity, minimum and maximum temperature, duration of snowfall, freeze-thaw, etc.

iv. Collection of Soil samples

Laboratory Test to be performed on the collected soil samples

1. Optimum moisture content
2. Maximum dry density
3. Sieve Analysis
4. Gradation
5. Load bearing test: - California Bearing Ratio (CBR) test (soaked and unsoaked)

IV. CALCULATIONS

To find the Optimum Moisture Content (OMC) and Dry Density(DD)

Table.1 OMC & DD

S.No	Mould		Container (For water content)		
	Empty	Full	Empty	Full	After Drying
1.	2.76	4.51	27.62	40.17	39.36
2.	2.76	4.540	16.30	44.30	41.77
3.	2.76	4.63	24.67	60.30	56.41
4.	2.76	4.690	25.38	76.31	69.81
5.	2.76	4.790	33.08	94.40	83.37
6.	2.76	4.730	35.85	82.77	73.65

Calculation:-

Step1:- Find the water content

1) 40.17−39.36 = 0.81gm

2) 44.30−41.77 = 2.53 gm

3) 60.30 – 56.41 = 3.89 gm

4) 76.31 −69.81 = 6.5 gm

5) 94.40 − 83.37 = 11.03 gm

6) 82.77− 73.65 = 9.12 gm

Step 2:- Finding water content ratio (in percentage) $\{\frac{W_w}{W_s} \times 100\}$

1) $[\frac{0.81}{(40.17-27.62)}]\times 100 = 6.45$

2) 9.04%

3)10.92%

4)12.76%

5)18%

6)19.43%

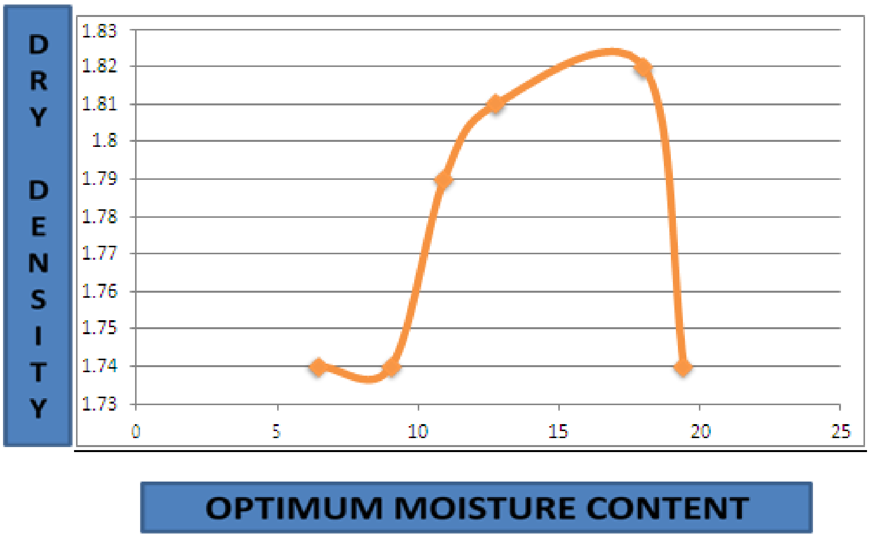

Fig.5. OMC v/s DD graph

Step 3:- Mass of wet compacted specimen (M)

1) 4.51– 2.76 = 1.75 kg => 1750 gm

Similarly

2) 1780 gm

3)1870 gm

4) 1930 gm

5) 2030 gm

6) 1970 gm

Step 4:- Find the bulk density P = $\frac{M}{v}$

Mould specification:- Circular Cylinder

Dia: 150 mm, Height: 175 mm Volume: $\pi \times \frac{d^2}{4} \times h$

$$= \pi \times \frac{15^2}{4} \times 17$$

$$= 3092.50 \text{ cm}^3$$

∴ Bulk Density (g/cm^3)

1) 1750/945 = 1.85 gm/cm^3

2) 1.88 g/cm^3

3) 1.98 g/cm^3

4) 2.04 g/cm^3

5) 2.15 g/cm^3

6) 2.08 g/cm^3

Then Dry Density:- $P_d = \frac{P}{1+w}$ (g/cm^3)

1) $\frac{1.85}{(1+\frac{6.45}{100})} = 1.74$ g/cm^3

2) 1.73 g/cm^3

3) 1.79 g/cm^3

4) 1.81 g/cm^3

5) 1.82 g/cm^3

6) 1.764 g/cm^3

After that a graph is plotted and the OMC = 18% and Dry Density = 1.82 g/cm^3

∴ Weight of soil = Volume intake of mould × Optimum Dry Density

3092.50 cm^3 × 1.82 g/cm^3

5628.35 g or 5.63 kg (wt. of soil)

Water required = 5628.35 g × 0.18= 1013.10 gm =1.013 kg

Table.2: Test results of CBR using soil only

Dial Gauge			Proving Ring	
S.No	**Penetration (cm)**	**Standard Loads**	**Load kg(f)× $\frac{10}{1009}$**	**Load kg(f)× $\frac{10}{1009}$**
1.	0.5		26 = 0.26	28.2 = 0.279
2.	1.0		28 = 0.227	29.4 = 0.291
3.	1.5		33 = 0.327	36.1 = 0.357
4.	2.0		37 = 0.336	38.5 = 0.381
5.	2.5	1370	42.6 = 0.442	44.1 = 0.437
6.	3.0		45.2 = 0.447	47.3 = 0.468
7.	4.0		51.8 = 0.513	54.6 = 0.541
8.	5.0	2055	56.4 = 0.558	55.2 = 0.547
9.	7.5	2630	64.3 = 0.637	67.5 = 0.668
10.	10	3180	78 = 0.773	78.2 = 0.775
11.	12.5	3600	84 = 0.832	83.3 = 0.825

Upward

For, 2.5 mm = $\frac{0.422 \times 101.97 \times 100}{1370}$ = 3.14 %

For, 5.0 mm = $\frac{0.558 \times 101.97 \times 100}{2055}$ = 2.77 %

Downward

2.5 mm = 3.25 %

5.0 mm = 2.71 %

In CBR bearing ratio ranges from 3.25 – 3.14 %, average = 3.195 % ≈ 3.2 %

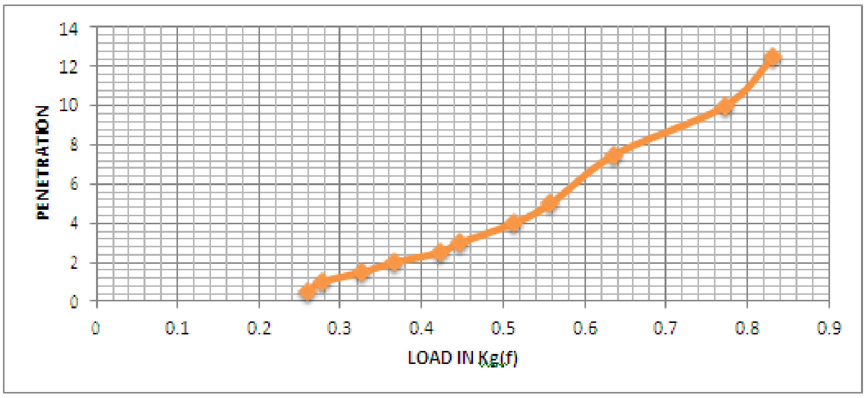

Fig. 6 CBR using soil only

Table .3: CBR results using 5 % fly ash

Dial Gauge			Proving Ring 0.002≈2 mm	
S.No	Penetration (cm)	Standard Loads	Load kg(f)× $\frac{10}{1009}$	Load kg(f)× $\frac{10}{1009}$
1.	0.5		0 = 0	6 = 0.0594
2.	1.0		1 = 0.01	10 = 0.0991
3.	1.5		1.5 = 0.015	16 = 0.159
4.	2.0		2 = 0.02	24 = 0.238
5.	2.5	1370	4 = 0.0396	33 = 0.327
6.	3.0		6 = 0.0594	41 = 0.406
7.	4.0		9.5 = 0.094	63 = 0.624
8.	5.0	2055	17 = 0.168	88 = 0.872
9.	7.5	2630	49.5 = 0.491	152 = 1.506
10.	10	3180	101.5 = 1.0059	214 = 2.121
11.	12.5	3600	158 = 1.566	273 = 2.71

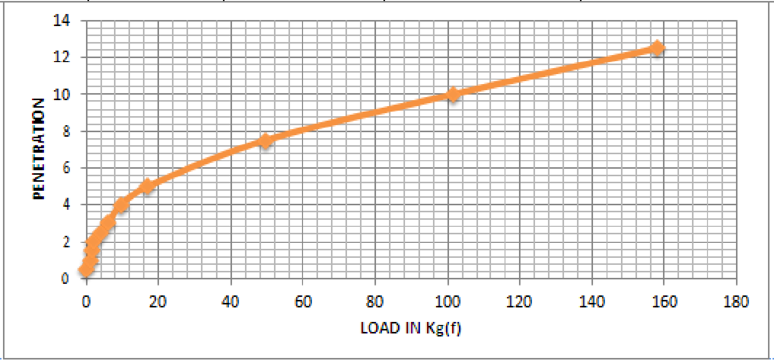

Fig.7 CBR using 5 % fly ash

Upward

For, 2.5 mm = $\frac{0.0396 \times 101.97 \times 100}{1370}$

= 0.29 %

For, 5 mm = $\frac{0.168 \times 101.97 \times 100}{2055}$

= 0.833 %

Downward

For, 2.5 mm = $\frac{0.327 \times 101.97 \times 100}{1370}$

= 2.43 %

For, 5 mm = $\frac{0.872 \times 101.97 \times 100}{2055}$

= 4.32%

Both cases CBR ratio for 5 mm > 2.5 mm penetration

∴ CBR ration will be ≈ 4.32 %

Table.4: CBR results using 10 % fly ash

Dial Gauge			Proving Ring 0.002≈2 mm	
S.No	Penetration (cm)	Standard Loads	Load kg(f)× $\frac{10}{1009}$	Load kg(f)× $\frac{10}{1009}$
1.	0.5		4 = 0.039	3 = 0.029
2.	1.0		9 = 0.089	7 = 0.069
3.	1.5		14 = 0.139	11 = 0.109
4.	2.0		22 = 0.218	20 = 0.198
5.	2.5	1370	30 = 0.297	28 = 0.278
6.	3.0		40 = 0.396	37 = 0.367
7.	4.0		66 = 0.654	58 = 0.575
8.	5.0	2055	95 = 0.942	92 = 0.912
9.	7.5	2630	154 = 1.526	149 = 1.476
10.	10	3180	237 = 2.349	232 = 2.3
11.	12.5	3600	304 = 3.012	297 = 2.943

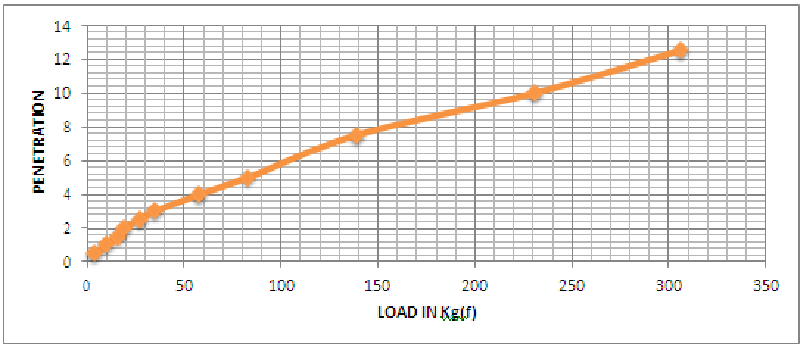

Fig.8 CBR using 10 % fly ash

Upward

For, 2.5 mm = $\frac{0.297 \times 101.97 \times 100}{1370}$

= 2.21 %

For, 5 mm = $\frac{0.942 \times 101.97 \times 100}{2055}$

= 4.67%

Downward

For, 2.5 mm = $\frac{0.278 \times 101.97 \times 100}{1370}$

= 2.07 %

For, 5 mm = $\frac{0.912 \times 101.97 \times 100}{2055}$

= 4.53%

CBR for 10 % fly ash ranges from 4.67 % to 4.53 %. On an average it is taken as 4.6 %

Table.5: CBR results using 15% fly ash

Dial Gauge			Proving Ring 0.002≈2 mm	
S.No	Penetration (cm)	Standard Loads	Load kg(f)× $\frac{10}{1009}$	Load kg(f)× $\frac{10}{1009}$
1.	0.5		4 = 0.039	6 = 0.059
2.	1.0		10 = 0.099	16 = 0.159
3.	1.5		16 = 0.159	24 = 0.238
4.	2.0		19 = 0.188	34 = 0.337
5.	2.5	1370	27 = 0.267	44 = 0.436
6.	3.0		35 = 0.347	56 = 0.555
7.	4.0		58 = 0.575	84 = 0.833
8.	5.0	2055	83 = 0.822	111= 1.1
9.	7.5	2630	139 = 1.378	169 = 1.675
10.	10	3180	231 = 2.29	253 = 2.51
11.	12.5	3600	306 = 3.033	336 = 3.33

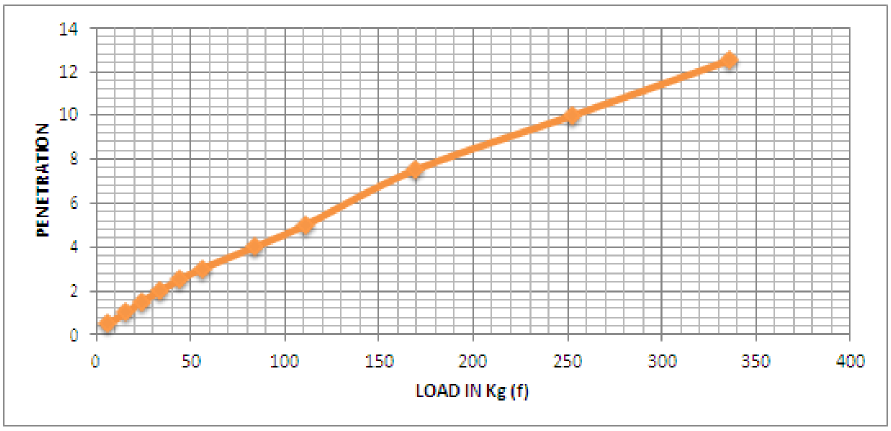

Fig.9 CBR using 15 % fly ash

Upward

For. 2.5 mm = $\frac{0.267\times101.97\times100}{1370}$

= 1.98 %

For. 5 mm = $\frac{0.822\times101.97\times100}{2055}$

= 4.07%

Downward

For. 2.5 mm = $\frac{0.436\times101.97\times100}{1370}$

= 3.24 %

For. 5 mm = $\frac{1.1\times101.97\times100}{2055}$

= 5.46%

CBR for 15 % fly ash ranges from 4.07% – 5.46 %. On an average it is taken as 4.77 %

Up to CBR with 5 %, 10 %, 15 % fly ash we can summarise thefollowing points:

1. As the percentage of fly ash increases the Bearing Ratio of soil sample also increases.

2. When we done the CBR test without fly ash we get the Bearing Ration as 3.2 %.

3. With 5 % fly ash CBR result is 4.32 %

4. With 10% fly ash CBR result is 4.36 %

5. With 15% fly ash CBR result is 4.77 %

6. Notable fact is, without fly ash we get the CBR value from 2.5 penetration i.e. CBR valueof 2.5 mm > CBR value of 5.0 mm. But the same in case of CBR values with fly ash we get CBR value of 2.5 mm penetration < CBR value of 5.0 mm

7. Proper Drainage factor is dually checked and an interval of around 10 minutes is given tothe 4 days soaked soil sample prior to CBR test.

Table.6: Fine aggregate taken => 1590 gm (Reducing the 410 g of CA from 2 kg)

Sl. No.	IS sieve size (mm)	Weight Retained (gm)	Cumulative Weight Retained (gm)	Cumulative % retained	Cumulative % passing
1	10	210	210	13.21 %	86.79 %
2	4.75	193	403	25.34 %	74.66 %
3	2.36	120.9	523.9	32.95 %	67.05 %
4	1.18	155.24	679.14	42.71 %	57.29 %
5	600	112.71	791.85	49.80 %	50.2 %
6	300	210.69	1002.54	63.05 %	36.95 %
7	150	274.83	1277.37	80.33 %	19.67 %
8	75	225.79	1503.16	94.54 %	5.46 %
			Residue 86.84 gm	401.93, Fineness Modulus 4.02 unsuitable for making concrete	

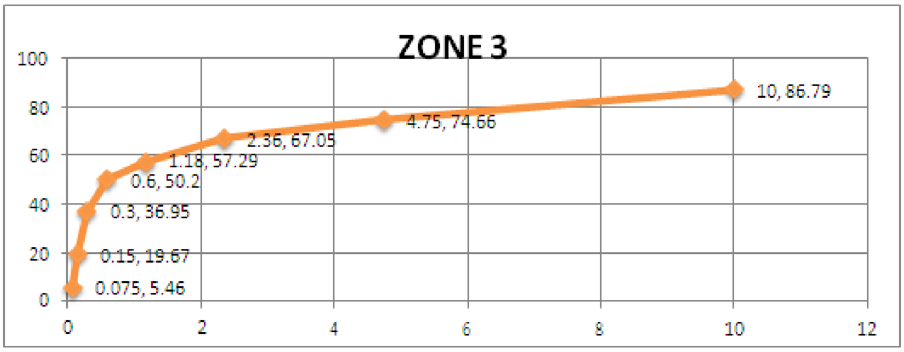

Fig.10 Soil Zone

Computation of Design

TrafficCBR value of our

subgrade soil = 3.2 % **Available**

Data:

1. Design of CBR of subgrade soil: 3 % (say) taking lower value for safeside
2. Design life of pavement: 15 years (say)
3. Annual growth rate: 7.5 %
4. Distribution of commercial vehicle for Single lane:- Double lane
5. Computation of Design Traffic for the end of Design Life:- 0.75

Vehicle Damage Factor => 1.5 (For hilly areas)

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

where, N = the cumulative number of standard axles to be catered for in the design in terms of msa

A = Initial traffic in the year of completion of construction in terms of the number of commercial vehicles per day (CVPD)

D = lane distribution factor

F = vehicle damage factor

n = Design life of pavement in years

r = annual growth rate of commercial vehicles

$A = P(1+r)^x$

where, P = number of commercial vehicles as per last count

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

Design calculation of pavement thickness

1. Commercial vehicle "p" per day = Approximately 270 CV/day

2. r = 7.5 %

3. x = 1 year

4. $A = P(1+r)^x$

$= 270\ [1+(\frac{7.5}{100})] = 290.25$

5. D = 1

6. F = 1.5

7. $N = \frac{365\times[(1+0.075)^{15}-1]}{0.075} \times 290.25 \times 1 \times 1.5$

= 4150518.307 standard axles

≅ 4.15 msa (million standard axles)

say 4.2 msa.

Now from IRC 37-2001 we get

For 3 % CBR we have to interpolate

so, according to chart	Total pavement thickness
For 3 msa	645 mm
For 5 msa	690 mm
∴ For 4.2 mm	say y mm

∴ By formulae $y = 645 + \frac{(4.2-3)+(690-645)}{(5-3)}$

= 672 mm pavement thickness (excluding geotextile)

Where,

Granular Sub base thickness (in mm) = 335 mm

Granular Base thickness (in mm) = 250 mm

Binder course (in mm) = 60 Bituminous Macadam

Wearing Course (in mm) = 27 mm Semi dense Bituminous concrete.

Simulation and real model photos and data acquired

Fig.11 Construction of model

Fig.12 Pavement Model

Fig.13 Freezing Cycle

Fig.13 Freezing Cycle 2

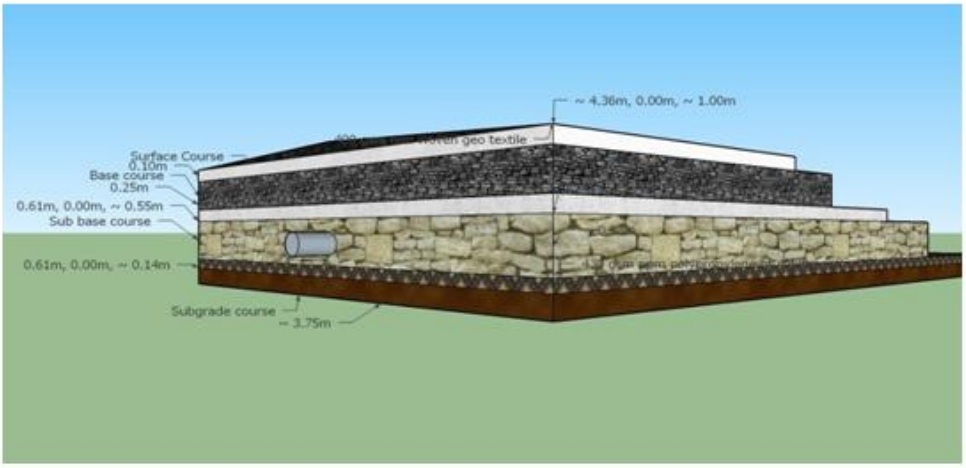

Fig.14 Simulated Model

Pavement model have been created and 2 cycles of freezing and thawing are completed notable facts are enlisted in summary.

V. SUMMARY AND CONCLUSION

Summary

1. The OMC of the soil sample is at 18% and Dry density is 1.82 gram/cm^3
2. After sieve analysis we found that the soil is a fine soil with fineness modulus of 4.02 (unsuitable for making concrete).
3. As the % of fly ash increases the bearing ratio of soil sample also increases.
4. When we done the CBR test without fly ash we get the bearing ratio as 3.2%.
5. With 5%, 10% & 15% fly ash CBR result is 4.32%, 4.6% & 4.77% respectively.
6. Notable fact is, without fly ash we get the CBR value from 2.5 mm penetration i.e. CBR value of 2.5mm> CBR value of 5mm. But the same, in case of CBR values with fly ash we get CBR value of 2.5mm penetration < CBR value of 5mm.
7. The pavement thickness is 672 mm (excluding geo textile material).
8. We created our model having ***400 GSM Non woven Geo textile*** for base course both top and bottom part and ***430 GSM PPM polypropylene multifilament*** for sub-base course.

Conclusion

Geo textile material is required both for protection as well as drainage of the road pavement so as to drain off the rain water and snow after melting conveniently .We basically aim at reducing the formation of potholes and transverse cracks due to frost action. So, we have created a model which has already been shown in the figure givenbelow after 2 cycles of freezing and thawing it has been found that the model can drain enough water without swelling as well as maintaining its load bearing capacity.

Fig.15 Practical Model.

Fig.16 Cross section of the Model

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