Experimental Technique to Evolve CBR Values

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SYNOPSIS : With the increased traffic on Indian Highways, usually provided with flexible pavements, an acute need was felt to device a method which can evaluate the sufficient thickness of pavement in soils difficult to sample in natural state. Indian Road Congress (1970) recommended the "California Bearing Ratio (CBR) method of Design" to evaluate the thickness of the pavement in accordance with daily traffic volume. This method however incorporates many cumbersome and costly insitu tests which are time consuming.

In case of noncohesive soils undisturbed samples are not possible economically. An attempt to use cone test, commonly used as sounding tool, which is not only cheap but quick, to provide CBR values for such soils, is made in this paper. The test has specific advantage of covering more test points for given budget as compared to CBR test, thereby providing a stastically more dependable design parameter for long lengths of highway subsoils. A Nomograph gives the correlation of cone resistance with CBR value and pavement thickness recommended.

INTRODUCTION

All most all the entire network of the Indian Highway system consists of flexible pavements. Till the beginning of 1970 the thickness of pavements was decided at random without proper scientific justification and the same thickness was adopted along the entire highway irrespective of the soil type, climate and traffic condition. Hence, in 1970, the Indian Road Congress recognised the need for an unified national code for the design of pavements and brought out its "guidelines for the design of flexible pavements". These guidelines recommended the California Bearing Ratio Method of design. There are so many methods for the design of the flexible pavements available but they are not fully rational or based on theory hence they cannot be reliably used. But California Bearing Ratio method of design has been proved to be comparatively reliable than other methods. The field CBR test is combersome, time consuming and very costly. Laboratory determination requires undisturbed samples which are at present economically infeasible for noncohesive sub-soils.

The paper is just an attempt to arrive at an empirical test such as cone penetration to obtain CBR. The cone resistance has already been related to relative density, safe and allowable bearing capacity (Desai M.D., 1972). The same test has been extended to determine CBR value.

CBR AND ITS USE

CBR method of flexible pavement design is based on an arbitrary soil strength test which is a penetration test. It was originally devised in 1929 by the California Division of Highway (U.S.A.) and has since been developed and modified by the U.S. Corps of Engineers. It is a small scale penetration test developed to predict the behaviour of subgrade soils and of paving materials in terms of an empirical parameter known as the California Bearing Ratio, usually abreviated as CBR. It is defined as ratio of the load required to force a cylindrical plunger of 3 Sq in (19.35 cm^2) cross-section into a soil mass at the rate of 0.127 cm/min to the load required for corresponding penetration of the plunger into a standard sample of crushed stone, the latter load being known as the standard load. (Table). The ratio is expressed as a percentage (ISI, 1965).

$$CBR = \frac{\text{Test load}}{\text{standard load}} \times 100$$

TABLE

STANDARD LOAD CHART

Penetration depth in mm	Unit standard stress in kg/cm^2	Total standard load in kg
2.5	70	1370
5.0	105	2055
7.5	134	2630
10.0	162	3180
12.5	183	3600

The CBR is determined corresponding to both 2.5 mm and 5.0 mm penetration and the greater of the two values is reported as CBR.

CBR is used to determine the thickness of pavement and different component layers of the pavement like subgrade, sub base, base and wearing course.

METHODS OF DESIGN OF PAVEMENTS

Number of methods for pavement design have been proposed, which however, can be broadly classified into the following groups :

- Empirical methods based on soil classification and past experience of pavement performance e.g. The group index method (By Highway Research Board, U.S.A.).
- Empirical methods using soil strength along with past experience of pavement performance e.g. The CBR method and North Dakota cone method (Boyd, 1942 and Wise, 1955).
- Semitheoretical methods based partly on simplified theories of stress distribution and partly on experience, e.g. The Boussinesq analysis (Glossop, 1944) and the Westergaard analysis (Westergaard, 1926).
- Theoretical methods based on a mathematical analysis of stress and strain in the pavement and the subgrade e.g. the Burmister and its extensions (HMSO, 1952).

PENETRATION TEST

These tests involve the measurement of the resistance to penetration of sampling spoon, a cone or any other shaped tools under dynamic or static loadings. The resistance is empirically correlated with some of the engineering properties of soil such as density index, consistency, bearing capacity (Desai M.D., 1972). These tests are useful for general exploration of erratic soil profiles, for finding depth to bed rock or hard stratum and to have an approximate indication of the strength and other properties of soils, particularly the cohesionless soils, for which it is difficult to obtain undisturbed samples. Between the two penetration tests, standard and cone, the cone test is considered more useful in determining the bearing capacity of cohesionless soils as it is cheap, less time consuming and fairly reliable.

The Indian Standard (ISI, 1968) recommends the use of two different sizes of cones, 65 mm and 50 mm in diameter, having the same apex angle of 60°. The cones are attached to A-type drill rods (41.3 mm OD, 28.6 mm ID) and driven under the blows of a 65 kg hammer falling from a height of 75 cm. The number of blows for 30 cm penetration is termed the penetration resistance No.

Using the 50.8 mm cone, a fairly reliable chart, verified by case histories, has been developed (Desai and Singh, 1974) which can be used to estimate the density index and the allowable pressure (q_p) of cohesionless soils for a permissible settlement of 40 mm.

Local sand of grading shown in Fig. 1 having γ_d max = 1.93 g/cc, γ_d min = 1.60 g/cc has been used in the experiments. Its CBR value, in dry state, under surcharge of 0.056 kg/cm.sq. at different relative densities was determined in laboratory. The

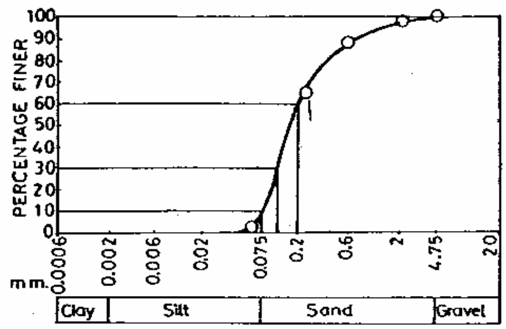

Fig. 1

Grain Size distribution Curve

results obtained are plotted against relative density in third quadrant of Fig. 2. The results of studies by Desai and Singh (1974) are reproduced in quadrant One and Two. The fourth quadrant is used to give recommended pavement thickness for a given CBR value (Singh A., 1975). For the known value of cone resistance and the effective overburden pressure, the nomograph can be read directly. For example Nc = 10 and surcharge of 0.56 0.056 kg/cm.sq., pavement thickness for various type of A, B, C, D, E, F and G vehicle loads works out to 8, 12, 15, 17, 19, 21, 23 cm respectively.

Work on determination of effect of variation of grading curves, degree of saturation, shapes of particles etc. is being continued.

CONCLUSION

1) The trend of results indicate a non linear increase of CBR for increasing values of relative density.

2) CBR can be related with Rd, bearing capacity, cone penetration for different surcharge value, so knowing the field value of penetration at no surcharge or with surcharge thickness of pavement can be designed. The usual costly and time consuming insitu CBR test could be replaced by a dynamic cone test for noncohesive subsoils.

3) Dynamic cone test used here being quick, cheap and unskilled, number of tests will be possible and thus provide better and economical design for highway projects. Due to cost and time taking processes of finding CBR in field normally limited tests could be planned and as a consequence adoption of higher factor of safety would result in increased cost of project.

ACKNOWLEDGEMENT

The authors are grateful to The Principal Dr. Y.N.V. Rao, for encouraging this work as research project of Department of Applied Mechanics, S.V. Regional College of Engineering and Technology, Surat. Experimental

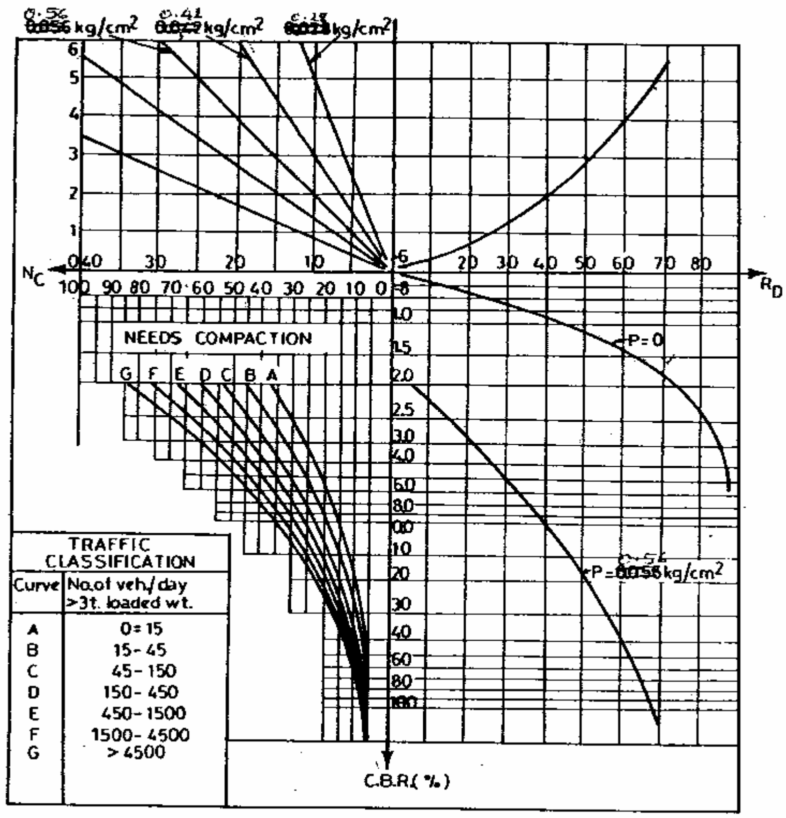

Fig. 2

Nomograph to obtain CBR from Nc Penetration Resistance

work, assisted by Shri D.C. Brahmbhatt, Chemist, is acknowledged.

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Free press Oct 11 Sunday

Seminar on geotechnical problems

-10-98 Free press

Our Staff Reporter

INDORE: A day-long national minar on 'geotechnical problems-se studies' was held at Jall auditoum on Saturday. Several research pars based on geotechnology were pre-nted in the seminar.

Professor in Indian Institute of Scice Dr A Sridharan who is president Indian geotechnical Society was ief guest. He inaugurated the semir by lighting the traditional lamp. e inaugural session was presided er by Dr P C Sharma. In his key te address Dr Sridharan pointed out e causes of tilting of tower of Pisa, gh lighting sub-soil conditions and medial measures. SGSITS director P C Sharma in his presidential dress highlighted the importance of se studies of distressed structure to lp in future construction.

M S Billore, advisor to MP govnment was special guest. In his address Billore emphasized on the importance of geotechnical aspects of civil engineering structures.

The inaugural function was followed by four technical sessions. In the first technical session Dr T S Nagraj presented case-studies of distressed structure due to inadequate subsirface exploration while Dr M R Madhav discussed in his lecture the building damage due to collapse settlement. Dr K S Rao from IIT Delhi discussed the stability of Himalayan landslides.

In the second session, professor N Som from Jadavpur University presented a case study regarding failure of a parabolic tunnel during construction in a coal mine area while Dr M D Desai from Surat delivered his lecture on the importance of geotechnical environment in roads-case study. Lecture on rehabilitation of Tillar Dam by special grunting technique was presented by M Bidasaria. In the third and fourth sessions, discussions were made on geotechnical problems of river valley structure, application functions and specifications of geofoam geosynthetic respectively.

IGS national president Dr A Sridharan inaugurating the seminar at Jall Auditorium on Saturday in the city.

At this occasion, gold medal and certificates for 1998 were presented by chief guest to Amit Thawani and engineer Pankaj for securing highest marks in geotechnical engineering and foundation engineering at BE and ME Examination, respectively. These awards have been instituted by M/s ferro concrete Construction Pvt Ltd and M/s Vastushilp Consultancy. Vote of thanks was proposed by Dr Abhay Gupta.

IMPORTANCE OF GEOTECHNICAL ENVIRONMENT IN ROADS
- CASE STUDY -

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INTRODUCTION

There are popular practices and codes (IRC etc) widely used in practice to design pavements. The interpretation and adoption of special recommendations vary widely for cohesive low to high expansive potential subsoils. The design is governed. by practice; cost, ignorance of traffic needs and other factors controlling final behaviour or recurring cost of maintenance. Poor roads and yearly repairs have been accepted as an unavoidable evils just like leaks through RCC roofs.

FACTORS TO BE CONSIDERED

The pavement designed must be modified to account for : (a) Overall drainage of area (b) Drainage for pavement in first phase (c) Ground Water table (d) Source of water in vicinity (e) Climetical variations: precipitation – evaporation and penetration of rain water in subsoil (f) State and changes likely in subsoil characteristics with time locations(g) Effect of thermal gradient – osmosis over the covered areas (h) Structure of subgrade at the time of exploration and execution (j) Disintegration of available materials – aggregates during compaction wetting and drying during cycles cyclical traffic loads (k) Design criterias to limit strain (l) Modern technologies of ground improvements – stabilization of subgrade providing Geofabrics – grids etc.

The overall time to construct, available materials, plants and capital / recurring cost analysis though not considered in past, can not be ignored in 21st century.

DESIGN

The most widely practised method assumed soaked CBR of compacted subgrade to MDD (at OMC) Its insitu compaction is rarely specified or executed in field.

According to John W.G. Kerr (1985)," the massive pavement system of US is falling apart because of unfounded belief that if road bases and subgrades are tested in saturated condition good drainage is not necessary". This is equally true for practice in this region of India.

The pavement in clayey silts under worst conditions could be standardised as soaked. CBR range is 2 to 4 in Gujarat and around. Unsoaked CBR varies from 5 to 15% Inadequate storm water drainage may submerge subgrade, sub base, base for few days every year. For aggregates apparent cohesion and friction are reduced, thus reducing effective CBR. The traffic induced cyclical pore pressures in blended sub base – base causes further loss and may cause shear failure. The result is strain, wearing surface displacements and punching of tyres of heavy vehicles. This will repeat every monsoon even if patch is filled back.(Fig. 1)

Literature shows that moist aggregates in base (fines 0.075 > 10% shows deflection of 1.0 mm at 5000 cycles of load. In submerged state, it will exhibit same deflection at 100th cycle. On 1000 cycle 12mm deflection was repeated. This forms ruts, failure of topping and a passage for rain water to subgrade.

The design of material grading for base and sub base, in field, has never been considered critical. The CBR value at MDD drops to 50% in case of fines (0.075 mm) less than 6% and more than 10%. The optimum fines 8% exhibited a CBR value of 40%.The field Engineer must understand consequences and need for quality control of grading.

The granular material for base, sub base is murrum disintegrated rock, angular metal: The compaction of such material may, in some cases, crush grains – corners to finally provid totally different grading of materials. This has a considerable influence on the performance.

Thus failure of pavements by rutting, punching or longitudinal wave displacements coul be primarily attributed to poor drainage., poor quality control of materials in sub base and bas and behaviour of sub grade in presence of water (swell / shrink / collapse).

LIMITING STRAINS

The thickness of granular material (modulus Eg) controls plastic strains in soil below (Es). Overall behaviour under cyclical loading requires prediction of foundation modulus (Er).

The top asphalt layer (Ea) may crack with plastic strains of 1%. For asphalt road with 57mm Asphalt and 225 mm crushed stone on silty clay subgrade exhibited 600 mm tensile strain below asphalt and above subgrade and 1100 μm vertical strains as critical to limit plastic strain in subgrade. Threshold deviator stress at surface is restricted to 0.7 x max stress ration (qult/p' – qult is UBC of subgrade, P' is effective overburden). This means theoretically subgrade shall have minimum shear resistance of Cu = 120 kpa in saturated state or ϕ = 20° with Cu = 40 kpa in wet state. For subsoil exhibiting lower resistance punching of aggregates into subgrade cause failure. A stabilized subgrade is thus obligatory. For Er = 20,000 kpa experimental records shows 20% axial strain for stone crusts after 1000 cycles of loading. This indicates the magnitudes of displacements involved.

Majority of cohesive low to high plasticity subsoil subjected to wetting, the subgrade must be improved by suitable technique. Only in dry confined, compact, subgrades the strength may be adequate.

CLIMATICAL FACTORS

In rainy days, precipitation, evaporation, state of subgrade and permeability of water, land drainage, ground water table have considerable influence on the depth of weathering in subsoil. This top crust subjected to swelling and shrinkage causes 3D cracks. The top material has structure with lumps of silt clays, in matrix of soft clay in wet cycle and air in dry spell. Degree of damage differs with soils having low to high expansive potential. The heavy strain in swelling increases from 4% for non expansive soil to more than 12% highly expansive subgrades.

The influence of location of ground water table as well as surcharge pressure on heave on wetting is illustrated in Fig.2. At surface hsw is 120 mm for zero depth to water table and is only 12 mm for GWT at low depth. This factor is ignored in design practice and scheduling construction of road. The state of subgrade in monsoon and summer at a time of construction hence behaviour in subsequent days, will be different.

The cracked top zone in part of road length could draw, water / waste of industries from side surface drain, ponds, canals (cyclical irrigating area), thermal diffusion (temperature gradient). The gradually increasing wetted depth increase swelling. Loss of moisture in long dry spell is by a formation or evaporation, from metal crust in construction phase could cause differential shrinkage, cracks and loosening of crust, The crust CBR drops and subsequent rains repeat this loss of metal crust into subgrade.

Heaving after construction causes cyclic reversal of strain every year. The subgrade in plastic or semiliquid state, under cyclical loads. is injected in to soling, voids of the base metal under pressure. This further reduces shear resistance of crust considerably.

Typical observations of heave of covered and uncovered areas subject to wetting cycles over years is illustrated in Fig. 3 The rains caused heaving followed by subsidence in summer at site. Over years cyclical changes shows overall heave of surface. The subsoil under surcharge of 3 M depth is not affected by wetting in both cases. The heave below covered areas is almost double that of the open ground. The moisture below covered area increased from 22 to

31% at 1 M depth and 27 to 33 % at 3 M depth in 45 months. Thermal diffusion from layers at high temperature to those at lower temperature in subgrade and diffusion of water vapour through free pores and cracks in soil must be accounted for in design as well as construction scheduling.

LATERAL SWELL

The soils in subgrade adjacent to the surface drains are subject to lateral swell pressure Fig. 4. Exhibits strain on swelling Vs days of wetting and latteral swelling pressure (Pg) for undisturbed and remoulded states. The latteral pressure is function of density and water contents. For Pd = 1.3 t/m^3, Pg = 4 t/m^2, Pd = 1.87 t/m^3, Pg = 22 t/m^2. For density 1.3 t/m^3, Pg varies from 17 t/m^2 to 12 t/m^2 for water content 8 to 25% (εsw = 0). The latteral strain is almost half for 10 days wetting in remoulded soil as compared to undisturbed state.

The lateral pressure is maximum at zero time and it decreases as wetting continues over days to study state value. This lateral pressure has been cause of failure of conventionally designed and executed drains. Sudden increase in Pg (Fig 4)on wetting is affected by remoulding. Steady state pressure is attained in 10-15 days in UD soil whereas it takes 30 days of wetting in remoulded (compacted) subgrade. On termination of wetting, the lateral pressure tends to zero at top and on rewetting cycle it regains original value. The rate of swelling varies from 0.05 to 0.7% per day. This establishes definite advantage of compacting subgrade to MDD (at OMC).

Depth (cm)	Pg max KPa	Pgc KPa	Heave (mm)	Pgc 30 days after stopping Wetting	Pgc 30 days after rewetting cycle
20	35	22	75	0	22
50	80	45	38	32	45
90	110	100	07	87	100

CHEMICAL FLUIDS

The industrial wastes containing sulfuric acid causes chemical swelling 2 to 7 times in excess of observed swelling during wetting with water. All corrosive substance have similar trends.

CONTRACTION OF SOIL

The volumetrics shrinkage depends on percentage colloids in soil (0.0005 mm) and rapidly increases with latter. Increase of colloids from 20% to 70% increases volumetric Shrinkage from 40% to 80% Shrinkage also depends on temperature of drying. It is rapid in range 300 to 600 C. liner shrinkage increased with initial moisture contents shrinkage decreases with increase in density.

During cyclical wetting and drying up to Sr = 65% the process of expansion and contraction is reversible, Drying to Sr < 65% led to a reducing swell moisture content in first two cycles. In remoulded samples with alternate drying and swelling cycles the difference between relative expansion and relative contraction remained same. The shrinkage moisture reached a constant value after 4th cycle. The shrinkage process was 3 to 4 times longer (duration) than the expansion process. This aspect has to be taken care in design of subgrade of all cohesive silts and clays

CASE STUDY

A typical design for road in Broach district developed by three consultants illustrates wide range of interpretations (Fig 5). The writer, considering factors listed above introduced a cut off to stop lateral seepage from drains. The introducing of insitu soil-lime-FA layers provided a vertical cut off for seepage of water through sand and material crust exposed to rains at end 1st phase of pavement. A road in industrial zone of Broach District was 5 km long. It is standard practice to leave base for a season before asphalt layer is provided. Major part was damaged in first monsoon of 1995. Repairs of making up the level by dumping metal could not make it servicible in 1996 monsoon. In both seasons surface blended base was completed before rains.

The analysis of poor performance with limited traffic is summarised below. The exploration observations of expansive clay, low OMC and reduction in CBR on soaking were inconsistent for use in analysis.

The report of samples taken during construction indicate soil as highly plastic expansive silt (Wl = 60, Ip=36 OMC 20% MDD 15 t/m^3 no CBR or tests for expansive potential are reported.)

The same batch of samples tested by another Lab. shows OH group SL = 16 to 24 (non critical), swelling pressure 6 T/M^2. Third testing lab indicate CH soil with SL=13 and classified soil as expansive.

The data collected shows 200-300 mm rainfall spread over 10-12 days every year. Groundwater is at depth 20 m. Even rainy days are hot windy. Overall shrinkage cracks extends to 1-0 m depth. Drainage is poor. Soil exhibits summer water content 10% (Pd = 1.45 g/cc), strain on swelling Esw = 20% water content after swelling Wsw = 49% (Pd 1.17 g/cc), cohesion after swelling 3 t/m^2 . For top 1 m layer subject to cyclic drying wetting heave of 200 mm at surface is likely with 10 days setting. The first phase of base imposed 520 surcharge is less than 1.0 t/m^2 against swell potential of 6.0 t/m^2. The wetted clay under pressure, mud flow grouts soling and base materials. Thus CBR of sub grade and base reduced shear resistance of sub grade leads to punching failure below wheel load.

For soaked CBR =2.5, 450 CV/day, designed pavement is 600 mm. For 525 mm Base-sub base to be safe, minimum CBR of sub grade will be 6% against actual CBR of 2%.

This road is in cutting. The excavated dump and trench for pipeline on either side had storage of water for weeks. The sub grade and pavement of 200 mm sand cushion, 130 mm quarry spouls, 170 mm hand broken metal and 750 mm black trap metal compacted to 525 mm is pervious in vertical and lateral directions. The top blindage 30% sand + 70% soft murrum had been washed out or dispersed in voids of crust by rains. Thus water stored in voids wetted sub grade. The process of wetting-gradual heaving-grouting of voids by semi liquid sub grade-loosening of sub base and base-loss of CBR – failure under type pressure - was the result. Thus loss of sub base and part of base into sub grade below load track was initiated. This is followed by spell of hot windy low humidity environment and a cycle of drying of sub grade. The air voids is crust permitted loss of vapour. The shrinkage created large pockets in sub base. This led to sinking and collapse of Base. With subsequent rains it is repeated.

The behaviour of sub grade is directly reflected on road surface. The oversafe design do not provide safety, speed and desired performance.

Over past few years a need to provide a insitu stabilized sub grade in top weathered zone of all cohesive soils, a impervious subbase, to keep Base over natural ground level, is adopted by author for some sites. (Fig.5) A puddled cut-off to minimise lateral seepage has been used for a site. The sub grade in any case is compacted to MDD at OMC. The provision of compacted impervious shoulders and apron between shoulders and drain is made.

The insitu 200-300 mm ploughing in summer by disc harrow, sprinkling water 24 hours before adding lime 3 to 5% and fly ash (4 times weight of lime), reploughing, adding water to OMC and compacting by sheepfoot or 6 t vibratory roller has been adopted for a municipal road in Udhna and Dahej. This treated sub base after 7 days of curing in moist state exhibited cube strength 3.5 kg/cm^2 and a min. CBR value of 10%. This layer is a water barrier for top and lateral seepage.

If construction is taken up just after rains (Sep), the sub grade is prewetted. It has less swell potential. The surface is compacted to proctor MDD. A layer of cohesive non swelling soil of 150 to 300 mm is laid over it covering sides, shoulders and width to drain at Proctor density.

The sub base is dropped. Base is covered by 20-30 mm grouted asphalt layer after blending of surface by FA/dust and fine sands. This topping protects sub grades from vertical seepage

For a road in Fertilizer plant area, specific part of 200 m exhibited repeated waves formation. The same road in rest of parts was unaffected. The exploration showed a RCC water resevior on right and green tall Eucalyptus plantation on left side. The monsoon wetting and wetting by seepage from reserviour during year with drying by the transpiration of green belt in Oct-Nov caused over-all differential heaves in zone. By Jan. shrinkage dominated performance.

For a site in Ankleshwar typical treatment of plot with stabilized fill for roads was evolved while providing over-all land development for drainage of large industrial plot.

CONCLUSIONS

Standard CBR based design can not guarantee performance of pavements. High recurring cost of maintenance have led to living with bad roads a habit.

The design has to consider local environmental factors like water table, schedule of constructions, sub grade behaviour with time and seasonal changes, drainage during construction etc. A stabilized sub grade with similar sub base to provide the road above the natural ground level has been successfully used on trial basis. The treated zone is extended beyond shoulders to surface drains. Provision of grouted watertight topping at end of completion of non asphaltic pavement is desirable.

The pavment along length will not have same design as is commonly practised. It has to consider envirnomental changes in sub grade and sources of troubles – canal, pond, kiln plantations etc. to provide for collapse, swell, shrinkage etc.in specific zone. Geogrids, filters, soil stabilisation, stabilising sub grade etc. could be used judicially for better performance.

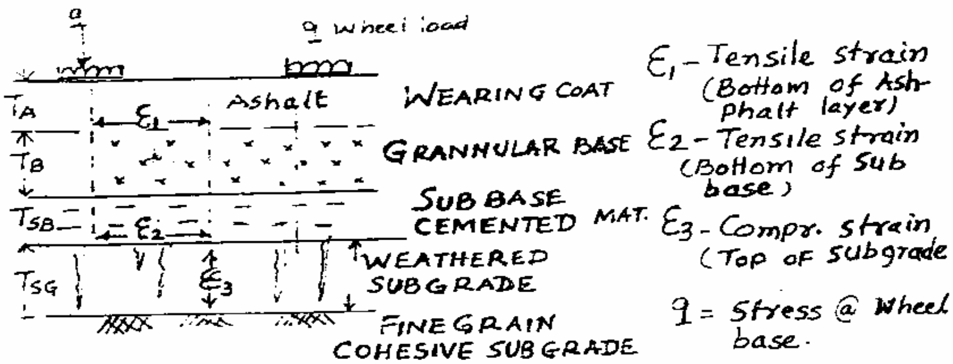

FIG.1 Model of pavement based on NAASRA ('87)

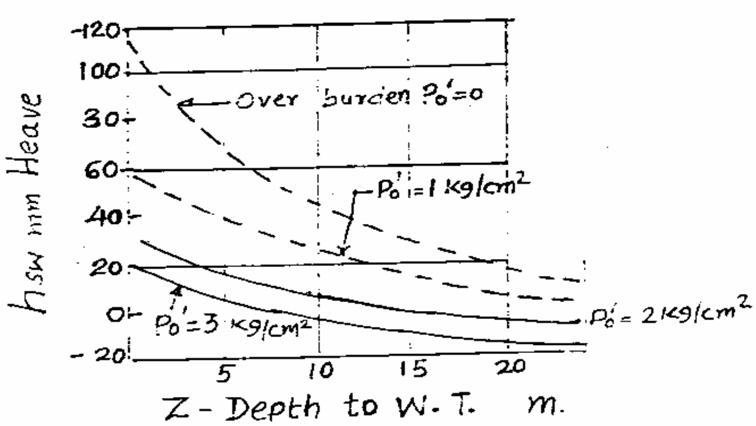

FIG:2. Correlation of heave, Location of W.T. and Surcharge pressure (E A Sorotchan)

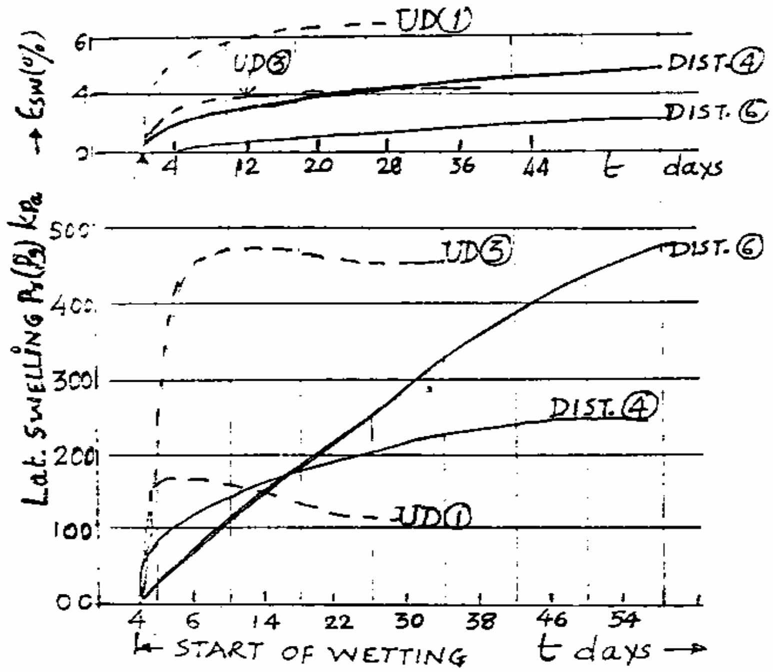

FIG 4. Variation of relative swelling & lat. pressure for disturbed & undist. soil with time. (E.A SOROTCHAN, P-33)

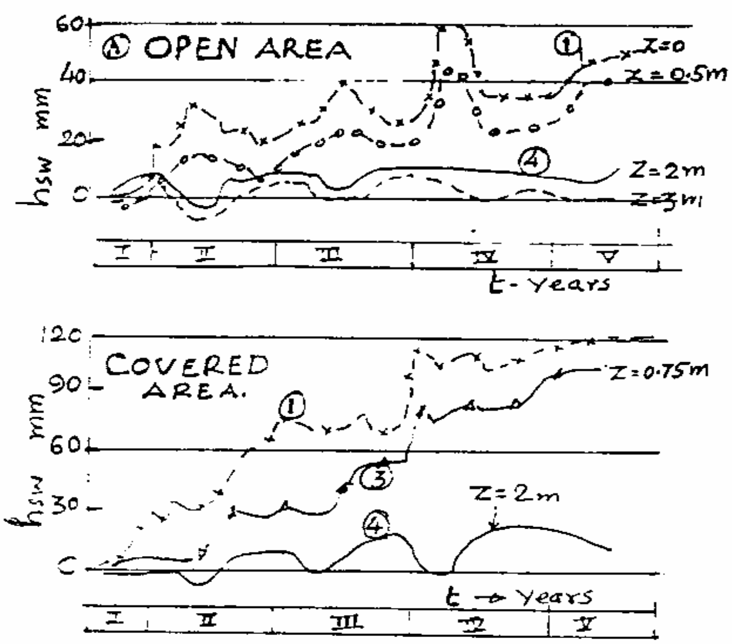

FIG 3: Dispacement of Soil over years (Sorotchan)

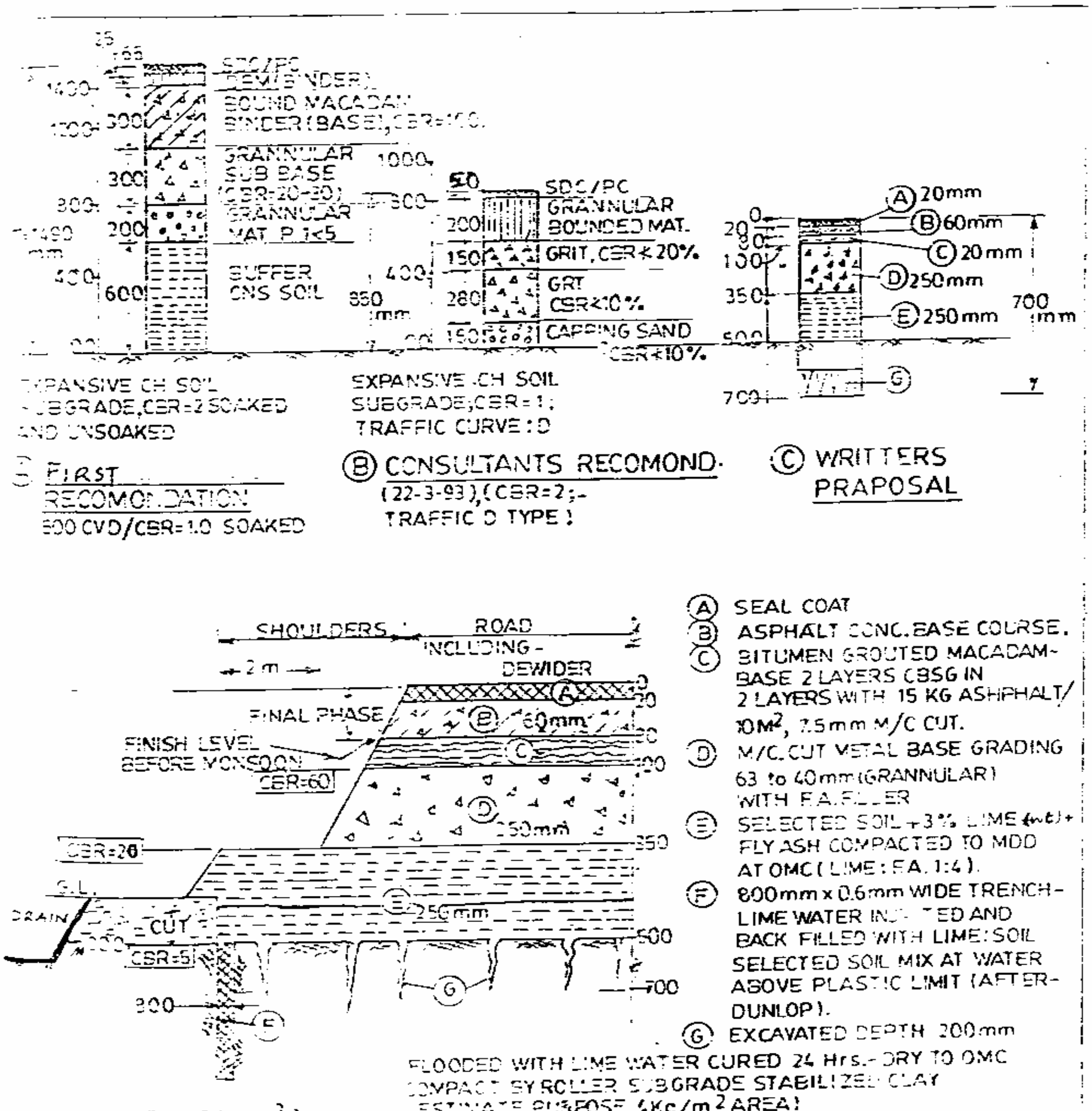

FIG.5 : PAVEMENT DESIGN FOR RD., BROACH (WRITTERS DESIGN)

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IMPORTANCE OF GEOTECHNICAL ENVIRONMENT IN ROADS – CASE STUDY

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SYNOPSIS

The concept and deep-rooted practice of providing a pavement thickness for worst CBR (Soaked) is all-best a designer can do. It presumes that all other factors – drainage, subgrade over years surrounding environment of storage, canal, temperature, humidity etc. are taken care of to best possible extent. If performance is unsatisfactory, it is attributed to subsoil characteristics beyond the knowledge and control of a Civil Engineer. The experience of investigating failures and experimenting simple modifications with cohesive subgrades based on judgement are presented in this paper.

INTRODUCTION

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FACTORS TO BE CONSIDERED

The pavement designed must be modified to account for : (a) overall drainage of area (b) Drainage for pavement in first phase (c) Ground Water table (d) Source of water in vicinity (e) Climetical variations: precipitation – evaporation and penetration of rain water in subsoil (f) State and changes likely in subsoil characteristics with time locations(g) Effect of thermal gradient – osmosis over the covered areas (h) Structure of subgrade at the time of exploration and execution (I) Disintegration of available materials – aggregates during compaction wetting and drying during cycles cyclical traffic loads (j) Design criterias to limit strain (k) Modern technologies of ground improvements – stabilization of subgrade providing Geofabrics – grids etc.

The overall time to construct, available materials, plants and capital / recurring cost analysis though not considered in past, can not be ignored in 21st century.

DESIGN

The most widely practised method assumed soaked CBR of compacted subgrade to MDD (at OMC) Its insitu compaction is rarely specified or executed in field.

According to John W.G. Kerr (1985)," the massive pavement system of US is falling apart because of unfounded belief that if road bases and subgrades are tested in saturated condition good drainage is not necessary". This is equally true for practice in this region of India.

The pavement in clayey silts under worst conditions could be standardised as soaked. CBR range is 2 to 4 in Gujarat and around. Unsoaked CBR varies from 5 to 15% Inadequate storm water drainage may submerge subgrade, sub base, base for few days every year. For aggregates apparent cohesion and friction are reduced, thus reducing effective CBR. The traffic induced cyclical pore pressures in blended sub base – base causes further loss and may cause shear failure. The result is strain, wearing surface displacements and punching of tyres of heavy vehicles. This will repeat every monsoon even if patch is filled back.(Fig. 1)

Literature shows that moist aggregates in base (fines 0.075 > 10% shows deflection of 1.0 mm at 5000 cycles of load. In submerged state, it will exhibit same deflection at 100^{th} cycle. On 1000 cycle 12mm deflection was repeated. This forms ruts, failure of topping and a passage for rain water to subgrade.

The design of material grading for base and sub base, in field, has never been considered critical. The CBR value at MDD drops to 50% in case of fines (0.075 mm) less than 6% and more than 10%. The optimum fines 8% exhibited a CBR value of 40%.The field Engineer must understand consequences and need for quality control of grading.

The granular material for base, sub base is murrum disintegrated rock, angular metals. . The compaction of such material may, in some cases, crush grains – corners to finally provide totally different grading of materials. This has a considerable influence on the performance.

Thus failure of pavements by rutting, punching or longitudinal wave displacements could be primarily attributed to poor drainage., poor quality control of materials in sub base and base and behaviour of sub grade in presence of water (swell / shrink / collapse).

LIMITING STRAINS

The thickness of granular material (modulus Eg) controls plastic strains in soil below (Es). Overall behaviour under cyclical loading requires prediction of foundation modulus (Er)

The top asphalt layer (Ea) may crack with plastic strains of 1%. For asphalt road with 57mm Asphalt and 225 mm crushed stone on silty clay subgrade exhibited 600 µm tensile strain below asphalt and above subgrade and 1100 µm vertical strains as critical to limit plastic strain in subgrade, Threshold deviator stress at surface is restricted to 0.7 x max stress ration (qult/p' – qult is UBC of subgrade, P' is effective overburden). This means theoretically subgrade shall have minimum shear resistance of Cu = 120 kpa in saturated state or $\phi = 20^{0}$ with Cu = 40 kpa in wet state. For subsoil exhibiting lower resistance punching of aggregates into subgrade cause failure. A stabilized subgrade is thus obligatory. For Er = 20,000 kpa experimental records shows 20% axial strain for stone crusts after 1000 cycles of loading. This indicates the magnitudes of displacements involved.

Majority of cohesive low to high plasticity subsoil subjected to wetting, the subgrade must be improved by suitable technique. Only in dry confined, compact, subgrades the strength may be adequate.

CLIMATICAL FACTORS

In rainy days, precipitation, evaporation, state of subgrade and permeability of water, land drainage, ground water table have considerable influence on the depth of weathering in subsoil. This top crust subjected to swelling and shrinkage causes 3D cracks. The top material has structure with lumps of silt clays, in matrix of soft clay in wet cycle and air in dry spell. Degree of damage differs with soils having low to high expansive potential. The heavy strain in swelling increases from 4% for non expansive soil to more than 12% highly expansive subgrades.

The influence of location of ground water table as well as surcharge pressure on heave on wetting is illustrated in Fig.2. At surface hsw is 120 mm for zero depth to water table and is only 12 mm for GWT at low depth. This factor is ignored in design practice and scheduling construction of road. The state of subgrade in monsoon and summer at a time of construction hence behaviour in subsequent days, will be different.

The cracked top zone in part of road length could draw, water / waste of industries from side surface drain, ponds, canals (cyclical irrigating area), thermal diffusion (temperature gradient). The gradually increasing wetted depth increase swelling. Loss of moisture in long dry spell is by a formation or evaporation, from metal crust in construction phase could cause differential shrinkage, cracks and loosening of crust, The crust CBR drops and subsequent rains repeat this loss of metal crust into subgrade.

Heaving after construction causes cyclic reversal of strain every year. The subgrade in plastic or semiliquid state, under cyclical loads. is injected in to soling, voids of the base metal under pressure. This further reduces shear resistance of crust considerably.

Typical observations of heave of covered and uncovered areas subject to wetting cycles over years is illustrated in Fig. 3 The rains caused heaving followed by subsidence in summer at site. Over years cyclical changes shows overall heave of surface. The subsoil under surcharge of 3 M depth is not affected by wetting in both cases. The heave below covered areas is almost double that of the open ground. The moisture below covered area increased from 22 to 31% at 1 M depth and 27 to 33 % at 3 M depth in 45 months. Thermal diffusion from layers at high temperature to those at lower temperature in subgrade and diffusion of water vapour through free pores and cracks in soil must be accounted for in design as well as construction scheduling.

LATTERAL SWELL

The soils in subgrade adjacent to the. surface drains are subject to lateral swell pressure Fig. 4. Exhibits strain on swelling Vs days of wetting and latteral swelling pressure (Pg) for undisturbed and remoulded states. The latteral pressure is function of density and water contents. For Pd = 1.3 t/m^3, Pg = 4 t/m^2, Pd = 1.87 t/m^3, Pg = 22 t/m^2. For density 1.3 t/m^3, Pg varies from 17 t/m^2 to 12 t/m^2 for water content 8 to 25% (Esw = 0). The latteral strain is almost half for 10 days wetting in remoulded soil as compared to undisturbed state.

The lateral pressure is maximum at zero time and it decreases as wetting continues over days to study state value. This lateral pressure has been cause of failure of conventionally designed and executed drains. Sudden increase in Pg (Fig 4)on wetting is affected by remoulding. Steady state pressure is attained in 10-15 [illegible] UD soil whereas it takes 30 days of wetting in remoulded (compacted) subgrade. On termination of wetting, the lateral pressure tends to zero at top and on rewetting cycle it regains original value. The rate of swelling varies from .05 to .7 % per day. This establishes definite advantage of compacting subgrade to MDD (at OMC).

DEPTH CM	Pg max KPa	Kpa Pgc	Heave Mmm	Pgc 30 days after stopping Wetting	Pgc 30 days after rewetting cycle
20	35	22	75	0	22
50	80	45	38	32	45
90	110	100	07	97	100

CHEMICAL FLUIDS

The industrial wastes containing sulfuric acid causes chemical swelling 2 to 7 times in excess of observed swelling during wetting with water. All corrosive substance have similar trends.

CONTRACTION OF SOIL

The volumetrics shrinkage depends on % colloids in soil (0.0005 mm) and rapidly increases with latter. Increase of colloids from 20% to 70% increases volumetric Shrinkage from 40% to 80% Shrinkage also depends on temperature of drying. It is rapid in range 30^0 to 60^0 C. liner shrinkage increased with initial moisture contents shrinkage decreases with increase in density.

During cyclical wetting and drying up to Sr = 65% the process of expansion and contraction is reversible, Drying to Sr < 65% led to a reducing swell moisture content in first two cycles. In remoulded samples with alternate drying and swelling cycles the difference between relative expansion and relative contraction remained same. The shrinkage moisture reached a constant value after 4^{th} cycle. The shrinkage process was 3 to 4 times longer (duration) than the expansion process. This aspect has to be taken care in design of subgrade of all cohesive silts and clays

CASE STUDY

A typical design for road in Broach district developed by three consultants illustrates wide range of interpretations (Fig 5). The writer, considering factors listed above introduced a cut off to stop lateral seepage from drains. The introducing of insitu soil-lime-FA layers provided a vertical cut off for seepage of water through sand and material crust exposed to rains at end 1^{st} phase of pavement. A road in industrial zone of Broach District was 5 km long.. It is standard practice to leave base for a season before asphalt layer is provided. Major part was damaged in first monsoon of 1995. Repairs of making up the level by dumping metal could not make it servicible in 1996 monsoon. In both seasons surface blended base was completed before rains.

The analysis of poor performance with limited traffic is summarised below. The exploration observations of expansive clay, low OMC and reduction in CBR on soaking were inconsistent for use in analysis.

The report of samples taken during construction indicate soil as highly plastic expansive silt (Wl = 60, Ip=36 OMC 20% MDD 15 t/m^3 no CBR or tests for expansive potential are reported.)

The same batch of samples tested by another Lab. shows OH group SL = 16 to 24 (non critical), swelling pressure 6 T/M^2 Third testing lab indicate CH soil with SL=13 and classified soil as expansive.

The data collected shows 200-300 mm rainfall spread over 10-12 days every year. Groundwater is at depth 20 m. Even rainy days are hot windy. Overall shrinkage cracks extends to 1-0 m depth. Drainage is poor. Soil exhibits summer water content 10% (Pd = 1.45 gkc), strain on swelling Esw = 20% water content after swelling Wsw = 49% (Pd 1.17 glcc), cohesion after swelling 3 t/m^2 . For top 1 m layer subject to cyclic drying wetting heave of 200 mm at surface is likely with 10 days setting. The first phase of base imposed 520 surcharge is less than 1.0 t/m^2 against swell potential of 6.0 t/m^2. The wetted clay under pressure mud flow grouting soling and base materials. Thus CBR of sub grade and base reduced shear resistance of sub grade leads to punching failure below wheel load.

For soaked CBR =2.5, 450 CV/day, designed pavement is 600 mm. For 525 mm Base-sub base to be safe, minimum CBR of sub grade will be 6% against actual CBR of 2%.

This road is in cutting. The excavated dump and trench for pipeline on either side had storage of water for weeks. The sub grade and pavement of 200 mm sand cushion, 130 mm quarry spouls, 170 mm hand broken metal and 750 mm black trap metal compacted to 525 mm is pervious in vertical and lateral directions. The top blindage 30% sand + 70% soft murrum had been washed out or dispersed in voids of crust by rains. Thus water stored in voids wetted sub grade. The process of wetting-gradual heaving-grouting of voids by semi liquid sub grade-loosening of sub base and base-loss of CBR – failure under type pressure-was the results. Thus loss of sub base and part of base into sub grade below load track was initiated. This is followed by spell of hot windy low humidity environment and a cycle of drying of sub grade. The air voids is crust permitted loss of vapour. The shrinkage created large pockets in sub base. This led to sinking and collapse of Base. With subsequent rains it is repeated.

The behaviour of sub grade is directly reflected on road surface. The oversafe design do not provide safety, speed and desired performance.

Over past few years a need to provide a insitu stabilized sub grade in top weathered zone of all cohesive soils, a impervious subbase, to keep Base over natural ground level, is adopted by author for some sites. (Fig.5) A puddled cut-off to minimise lateral seepage has been used for a site. The sub grade in any case is compacted to MDD at OMC. The provision of compacted impervious shoulders and apron between shoulders and drain is made.

The insitu 200-300 mm ploughing in summer by disc harrow, sprinkling water 24 hours before adding lime 3 to 5% and fly ash (4 times weight of lime), reploughing, adding water to OMC and compacting by sheepfoot or 6 t vibratory roller has been adopted for a municipal road in Udhna and Dahej. This treated sub base after 7 days of curing in moist state exhibited cube strength 3.5 kg/cm^2 and a min. CBR value of [redacted].10% . This layer is a water barrier for top & lateral seepage.

If construction is taken up just after rains (Sept.), the sub grade is prewetted. It has less swell potential. The surface is compacted to proctor MDD. A layer of cohesive non swelling soil of 150 to 300 mm is laid over it covering sides, shoulders and width to drain at Proctor density. The sub base is dropped. Base is covered by 20-30 mm grouted asphalt layer after blending of surface by FA/dust and fine sands. This topping protects sub grades from vertical seepage.

For a road in Fertilizer plant area, specific part of 200 m exhibited repeated waves formation. The same road in rest of parts was unaffected. The exploration showed a RCC water resevior on right and green tall Eucalyptus plantation on left side. The monsoon wetting and wetting by seepage from reserviour during year with drying by the transpiration of green belt in Oct-Nov caused over-all differential heaves in zone. By Jan. shrinkage dominated performance.

For a site in Ankleshwar typical treatment of plot with stabilized fill for roads was evolved while providing over-all land development for drainage of large industrial plot.

CONCLUSION

Standard CBR based design can not guarantee performance of pavements. High recurring cost of maintenance have led to living with bad roads a habit.

The design has to consider local environmental factors like water table, schedule of constructions, sub grade behaviour with time and seasonal changes, drainage during construction etc. A stabilized sub grade with similar sub base to provide the road above the natural ground level has been successfully used on trial basis. The treated zone is extended beyond shoulders to surface drains. Provision of grouted watertight topping at end of completion of non asphaltic pavement is desirable.

The pavment along length will not have same design as is commonly practised. It has to consider envirnomental changes in sub grade and sources of troubles – canal, pond, kiln plantations etc. to provide for collapse, swell, shrinkage etc.in specific zone. Geogrids, filters, soil stabilisation, stabilising sub grade etc. could be used judicially for better performance.

ACKNOWLEDGMENT

The work of typing was executed by M/s Rohit Traders, Surat.

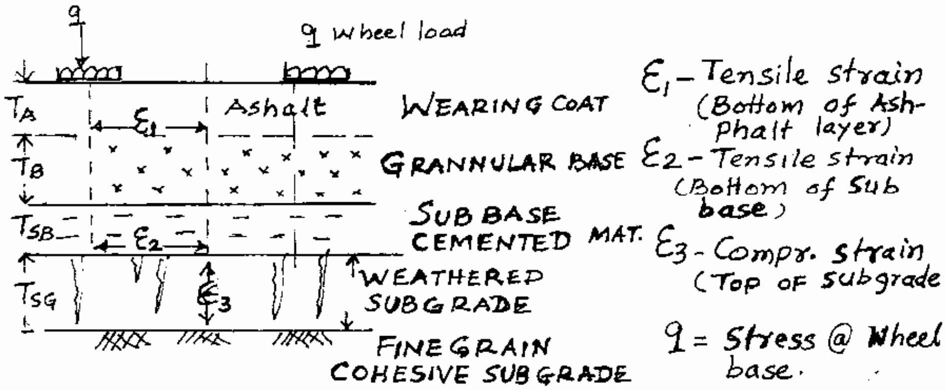

FIG.1 Model of pavement based on NAASRA ('87)

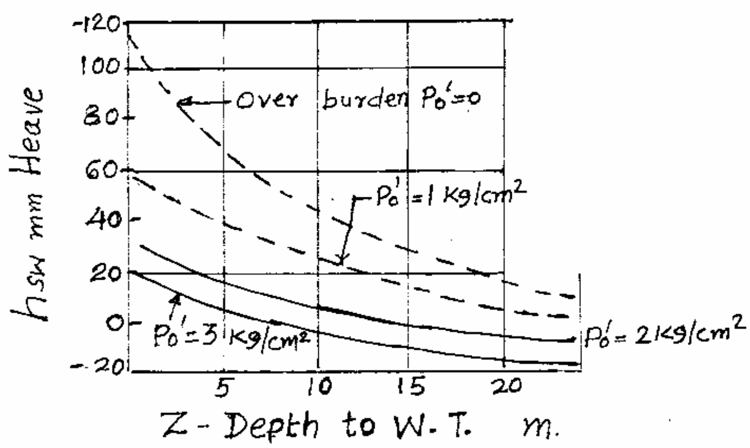

FIG:2. Correlation of heave, Location of W.T. and Surcharge pressure (E.A Sorotchan)

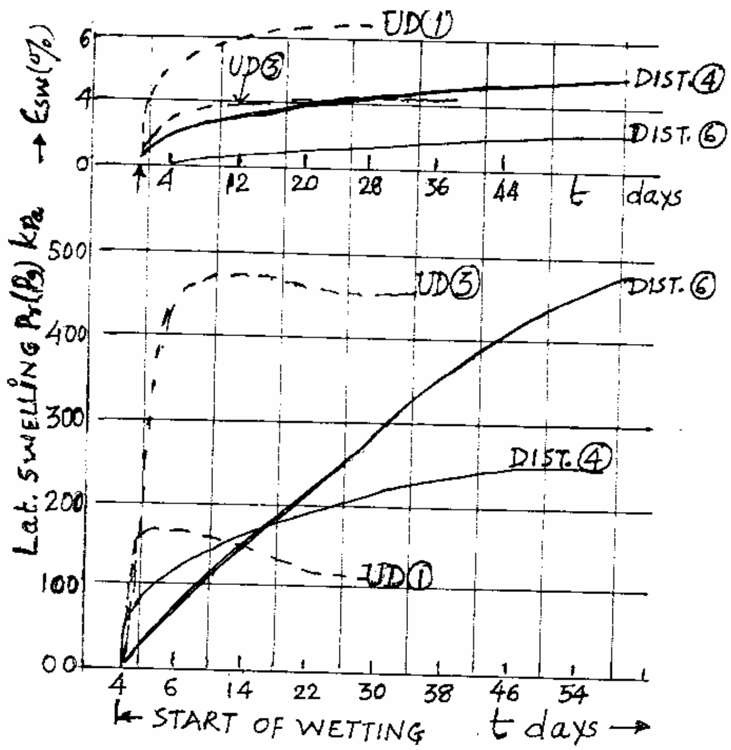

FIG 4: Variation of relative swelling & lat. pressure for disturbed & undist. soil with time. (E.A SOROTCHAN, P-33)

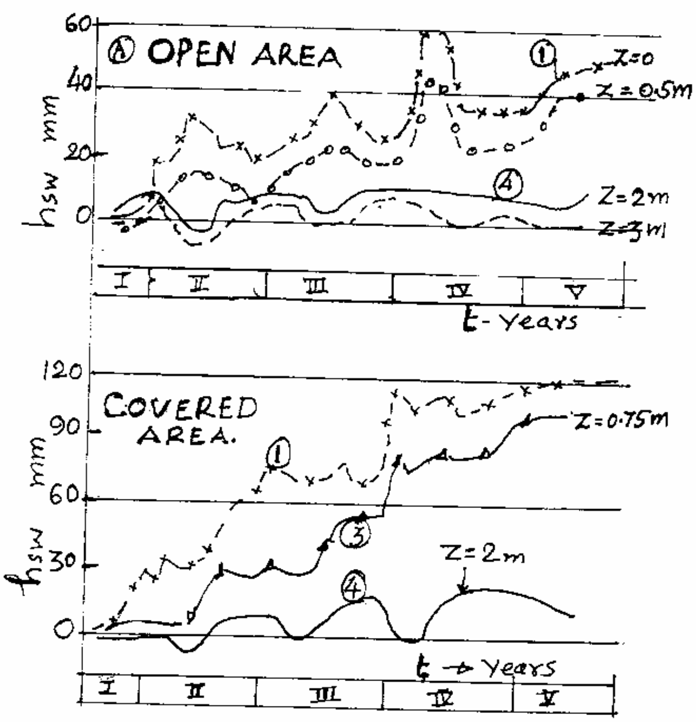

FIG3: Dispacement of soil over years (Sorotchan)

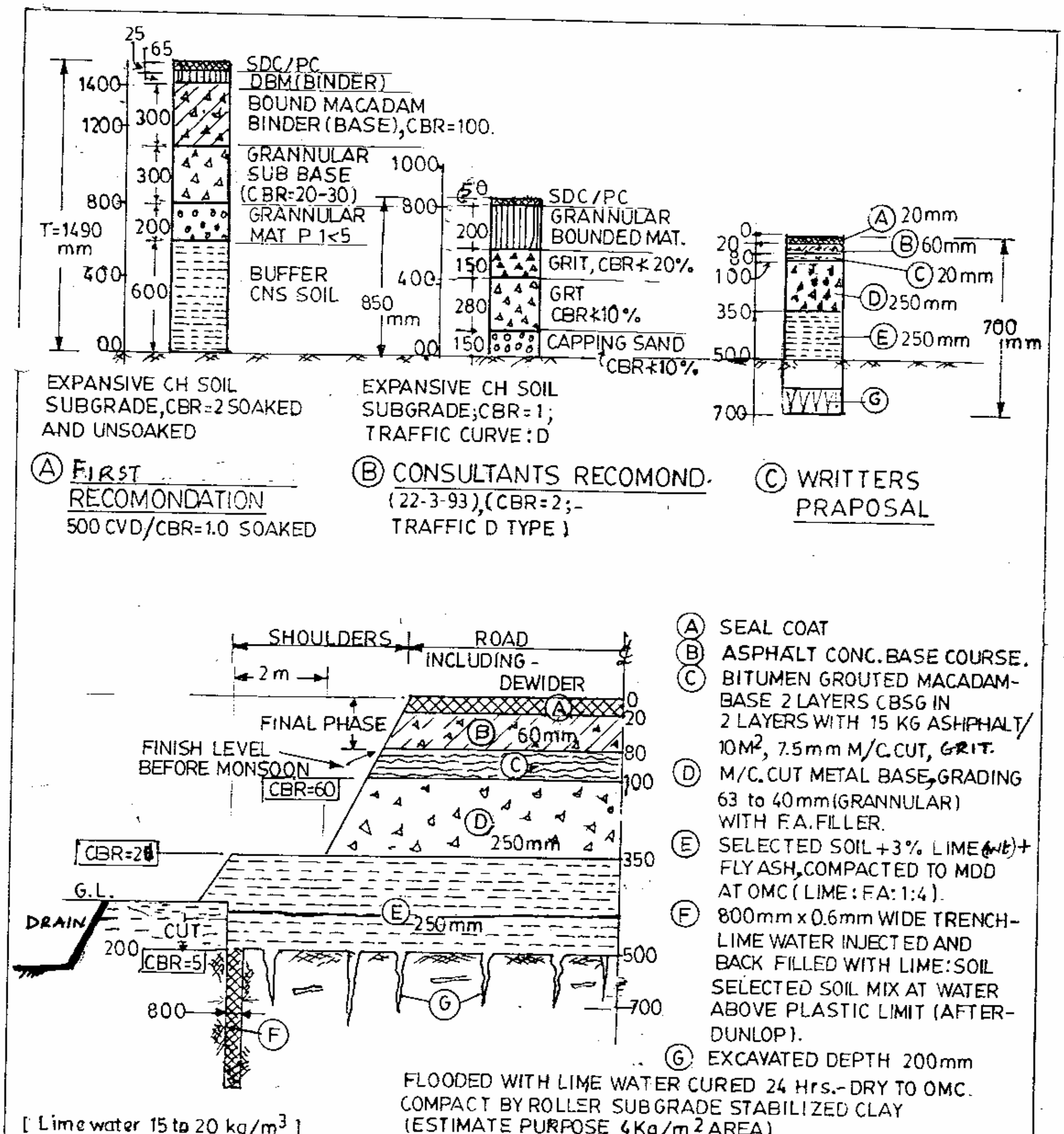

FIG.5: PAVEMENT DESIGN FOR ... RD., BROACH (WRITTERS DESIGN)

SOIL STABILIZATION FOR ROAD CONSTRUCTION

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1. INTRODUCTION :

The roads-both flexible pavement and rigid pavement are designed on virgin soil (sub grade).
This sub grade soil is important Parameter Controlling Economical design, performance and long term recurring maintenance aspects. All sub grades therefore must be loosened and recomputed at OMC to Max dry density. The work shall not be done in rains. The best time to start work is Nov.-Dec. even for expansive days. This expansive clays in summer will have deep cracks more than 2 m at times and in monsoon it results in flooding – heaving and squeezing in roads/latteral squeezing.

In silty soils dilatancy could be critical. The loose sand at surface needs to be compacted. So all the sub grades shall be cleared of plants, surface debris and loosened up to dry or welted to OMC. The compacted soil sub grade, now not done , has been major trouble shooter for pavements.

2. EXPANSIVE SUBGRADES :

For vast areas of state and country conversed with expansive clays a test to determine degree of expansiveness (low, medium, high) is a must, there are collapsible soils which collapses on wetting, flooding. The road subgrade wetted by rains, looses excess moisture with time & in summer, thus creating large voids, below pavements. If water finds way from sides (drain/excavation) it swells with swelling pressure 2 to 8 T/m^2 in next monsoon. This cycle of wetting drying the regimes of Gujarat have caused failure of sub grade & hence pavements. Thus stabilization forms a part of designing and investigation for roads.

3. CBR VALUE :

The rocked CBR for worst case in expansive soils of Gujarat has never been more than 2.0. The design is based on CBR and traffic intensity (Msa). In spite of satisfying design requirements failures are numerous.

The design for roaked CBR in and regimes with annual rain fall 500 mm or less, with deep water table, gives uneconomical designs.

4. METHODS OF SOIL STABILIZATION :

The soils in sub grade and embankment must be therefore stabilized using principles of Soil mechanics. The methods are

1.0 Compaction.
2.0 Lime Stabilization.
3.0 Cement Stabilization.
4.0 Resin Stabilization.
5.0 Adopting alternative construction, materials and available by products.

a. Fly ash
b. Slag from steel mill
c. Gypsum
d. Harmless heavy metal.
Wastes (like Birla copper)

The use of improved/.new materials in sub grade, sub base to obtain high CBR of sub grade 10 to 20 has been attemted successfully. The standard textbook and laboratory tools will be providing the details of chemicals stabilization.

I wish to report to you the use of waste – fly ash from GIPCL/Ukai for subgrade / sub base of flexible and rigid pavements in case study-1.

Also the studies on use of slag and design of road adopting it, has been shown in case study 2.

The usual will show to you a lime-FA stabilized sub grade technology adopted by SMC since 6 years to replace ineffective rouble soling in expansive soils around heavy rain fall area.

5. JUSTIFICATION FOR STABILIZATION :

The impact of stabilized subgrade on pavement thickness is explained by data for Road design for traffic 10 Msa.

(IRC37/2001)

~~Granular Sub base~~	Granular Subbase	Binder Course	Granular base
A) CBR2	460	100 DBM	250
B) Improve CBR 5	300	70 DBM	250
C) ImproveCBR10	200	50 DBM	250

The decreased DBM and thickness granular sub base is noteworthy for optimized economical and time constrained execution jobs.

6. DRAINAGE :

Non-provision of real working drainage along and across road can not give good drive surface even if low CBR design is adopted. The drainage is important in expansive cohesive soils as the flooding below sub grade through deep summer cracks can cause;

a) Loss of strength of sub soil & lateral movement due to swelling.
b) High swelling pressure on the pavement at edges of road (Cracks parallel to center line.
c) In summer shrinkage of sub grade subsoil, sinking of sub grade sub soil sinking of rouble along line load (rut), fatigue failure of base unsupported from bottom.

7. CASE STUDY – 1 :

- Material EAF & Slag ESSAR steel.
- Slag samples as supplied and after compaction are compared in table-1.
- Physical tests carried out for use the sub base, base of road are tabulated in table –2.
- Field crushing in compaction by bulldozer is shown in table-3.
- Summarized test results as per relevant code are tabulated in table-4. Limiting value by MDRT&H specifications are shown in this table.
- Water absorption of slag 4mm to 40mm size samples is less than 1% Fraction on finer than 4mm shows 10% absorption
- Typical road section designed for CBR 2 sub grade adopting slag is illustrated for 2 lane single carriageway on expansive soil is shown in fig. 1.
- About 2300 tons of slag is required for 100mm 2 lane single carriageway.
- By scientific study use of slag environmental problems waste is solved. The slag was analyzed by NEERI (1.3.02), GERI to conclude that it is inert, non toxic non hazardous, safe in construction applications.

8. CASE STUDY – 2 :

Millions of tons of Power Plant end product fly ash, bottom ash and pond ash poses are serious threat of ground, water, air pollution for the next generations.

This light weight fine material from Ukai, GIPCL, Birla Cellulosic and Atul were studied for specific uses as fill material in pavement.

After R&D and analysis of sinking of bounders or grouting of boulders by swollen expansive clay in voids, it was decided to drop it. It is replaced by insitu stabilized expansive soil by

a) Plouging to break the clods of soil.
b) Mixing 40 Kg. to 50 Kg. Fly ash + 10 to 15 Kg. of lime per Sq. Meter insitu of subgrade by use of tractor instu.
c) Watering the mixed soil lome FA mixture to saturation.
d) Allow 24 - 48 hours crusing.
e) Compacting about 150 to 180mm layer by 8T vibratory roller for a width 2m wider than road width. This layer after 90 days will have CBR=10 or more, a water tight separation layer between expansive clay and sub base of quarry spoils.

Exclusive use of line fly ash 1:8 to 1:10 with 5 to 10% clay of GIPCL invested shows CBR more than 50 (Nehal Desai).

The use of FA as fill material in embankment is shown in Fig.2

9. CONCLUSION :

It is our humble attempts to stress the needs to ;

1) Provide along and across a rain water drain which not allow wetting below the sub grade.
2) Insitu compaction with or without stabilization of the subsoil at OMC to MDD for in extra width on either side if road width.
3) Selection of economical appropriate sub stabilizer in sub grade. Sub base to achieve 4 to 10 CBR value by geotechnical experts.
4) Adoption of CBR value unsoaked or partially soaked wherever site condition permit
5) Adoption of available end products of steel – slag, power plant fly ash for formation fill, sub grade and sub base to help national improvement of environment.
6) Cost optimized design must keep in mind. It is never possible to adding strengthen sub grade by adding surface layers of metal /DBM.

10. ACKNOWLEDGEMENT :

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All these associated with waste to wealth" project in past 3 decades at App. Mech. Deptt. at SVR College gratefully acknowledged.

Table-1: Test Results: Sieve Analysis of EAF Slag before & after Modified Compaction (as analysed by M/s Unique Engg., Surat on 18th Aug. 2002 & M/s Dodsal Construction Pvt. Ltd, Chikhli on 25th Aug. 2002)

Sieve Size (mm)	% Passing (before Compaction) (By M/s Unique)	% Passing (After modified Compaction) (By M/s Unique)	% Passing (By M/s Dodsal Pvt. Ltd.) (Appx.)
40	82.04	100	-
31.5	76.56	100	75
25	63.72	100	66
20	56.16	100	63
16	52.0	98.03	53
12.5	45.9	94.97	48
10	40.68	88.39	45
8	29.36	75.86	35
6.3	17.04	54.51	-
4.75	5.96	29.04	25
2.36	1.92	15.96	20
1.18	0.68	6.47	-
600 μ	0.44	3.61	12
300 μ	0.28	1.99	-
150 μ	0.18	0.88	-
75 μ	0.10	0.41	00
Pan	0.00	0.00	-

Table-2: Physical Test Results by M/s Unique Engg. (dt. 18th August 2002)

Sr. No	Test	Results
1	Los Angles Abrasion	26.68%
2	Liquid Limit (LL)	20.00%
3	Plastic Limit (PL)	NP
4	Specific Gravity	3.84
5	Compaction (Std.)	2.39 gm/cc
6	Compaction (Modified)	2.41 gm/cc
7	CBR (soaked)/ (Un-soaked) (modified compaction)	25/55
8	Permeability	0.00722 cm/sec

Table 3: Test Results, carried out by M/s ESSAR Steel Ltd. – sieve analysis of crushed slag after crushing on 300 mm thick Slag bed (laid on WBM road) by 10 T Buldozer for 25 runs (till no further crushing is observed).

Sieve Size	% passing (after rolling)	
(A') 50-40 mm Slag sample		Mean Curve
40	100	100
20	89	80-100
10	50	40-90
06	21.42	18-50
04	12.5	12-24
< 4	7.14	-
2		0-12
0.15		0-02
(B') 40-20 mm Slag sample		Note :
40	100	Fro range-A'B'
20	71	(50-20mm)
10	30	Max : 20 mm
06	17	Min : 1.0 mm
04	10	D_{50} mean : 6 mm
< 4	10	

Table 4 : Summary of Tests data on EAF Slag Samples of ESSAR Steel Ltd. by different Agencies

Sr. No.	Test	IS Code of the Test Method	Limiting Values as per MORT&H for Base / Subbase	EAF Steel Slag
1	Grading, Nominal size/ D50/ Range (mm)	IS 2386 (Part 1)	See Figures-1, 2, 3 attached	(E) 20/09/20-0.2 (U) 20/06/20-4
2	Aggregate Impact Value (%)	IS 5640	Max 30	(D) 9.6 (G) 12-22
3	Aggregate Crushing Value (%)	IS 2386 (Part 4)	Max 45	(Lit) 15-18
4	Loss Angeles Abrassion Value (%)	IS 2386 (Part 1)	Max 40	(Lit) 9-10 (U) 26.68
5	Flakiness & Elongation Index (%)	IS 2386 (Part 1)	Max 30	(D) 11.75 (G) 11.0
6	Water Absorption (%)	IS 2386 (Part 3)	Max 2	(D) 1.5 (E) See Table-6 (G) 1.4
7	Soundness (% Passing) 40-20 mm 20-10 mm 10-4.75mm	IS 2386 (Part 5) *Table 500-14	12% (Na_2SO_4)	(Lit) 0.67 (Lit) 1.00 (Lit) 2.30
8	Stripping	IS 6241 *Table 500-14	Minimum coating retain 95%	(G) < 5%
9	10% Fineness Value (KN) Soaked Unsoaked	- -	- -	(D) 165.21 (D) 200.23

10	Standard Compaction OMC (%) /MDD (kg/m^3)	IS 2720 (Part 7)	3m high Embankment 1520-1600	(U) - / 2390
	Modified Standard Compaction OMC (%) /MDD (kg/m^3)	IS 2720 (Part 8)	> 3 m high Sub Grade 1750	(D) - / 2630 (U) - / 2410
11	Grading of Compacted Sample	-	-	See Figure-2
12	CBR value @ OMC MDD Soaked Unsoaked	IS 2720 (Part 16)	 Min 20 -	 (D) 77, (U) 25 (D) - , (U) 55
13	Specific Gravity	IS 2720 (Part 3)	-	(U) 3.84 (G) 3.4 (Lit) 3.5
14	Permeability (cm / sec)	IS 2720 (Part 17)	-	(U) 7.22 x 10^{-3}
15	LL / PL / PI of fines passing 2 mm		25/19/ 6	(U) 20/NP/-
16	Chemical Composition:			(E)
	a) CaO %	-	-	20-30
	b) MgO%	-	-	8-12
	c) SiO_2 %	-	-	10-15
	d) Fe_2O_3%	-	-	30-40
	e) Al_2O_3 %	-	-	03-06
	f) MnO%	-	-	01-04
	g) pH	-	-	8.6 before & 9.9 after cooling slag
	h) $\frac{CaO+MgO}{SiO_2}$	-	-	2 to 3
17	Density of rolled fill at site (kg/m^3)	-	Min 1750	(E) 1760 (40-50mm sized slag bed) (E) 1940 (20-40 mm sized slag bed)

** Reference: Tables: 400-13 & 500-14, MORT&H, Publication of IRC, 2001.*

Expansive Soil, CBR < 2, Traffic = 6-10 msa, for Crushed Run Macadam (CRM).

Layer No	Thickness (mm)	Material	Characteristics: PI	Characteristics: CBR at the Top of layer	Characteristics: Minimum Density (kg / cum)	Specification: IRC 37, 2001	Specification: MORT & H	Reference
F	f = 40	Bituminous Concrete	-	NA	-	-	-	
E	e = 100	Dense Bituminous Macadam	-	NA	-	-	-	Construction as per IRC & MORT & H Specifications.
D	d2 = 130	Natural Aggregate	NP	NS	1900 OMC - 0-3%	4.2.2	400.12	Slag in base application only for CRM Road. Top 130mm layer must be constructed with natural aggregate & bottom 120 mm with Slag
	d1 = 120	Slag						
C	c3 = 160	Slag	NP	30	1900 OMC - 0-3%	4.2.1.1	400.1 Grade-III	Slag is used in Sub base, there is no change in thickness. Construction must be done in 3 different layers of 150,150,160mm thickness after compaction.
	c2 = 150	Slag						
	c1 = 150	Slag						
Y	y = 150	CNS or Lime Stabilized Slag/	NP	10	-	4.2.1.5	-	Thickness of blanket zone and capping is reduced upto 800 mm if Slag is used in both the cases. (Save 350 mm)
X	x = 1000	any material having minimum 10 CBR	NP	NS	1750	Annexure4	-	
S	s = 100	After Stripping 100 mm Compacted at OMC	-	3	1750	-	-	Excavated 100 mm at GL and Compacted at OMC
Total Thickness T = 850								

R = Top of Road Level With Respect to Ground Level.

NP - Non Plastic

NA - Not Applicable

NS - Not Specified

CNS - Cohesive Nonswelling Soil.

MSA - Million Standard Axles.

A Swelling pr ↓ harrier slab - lesser thickness

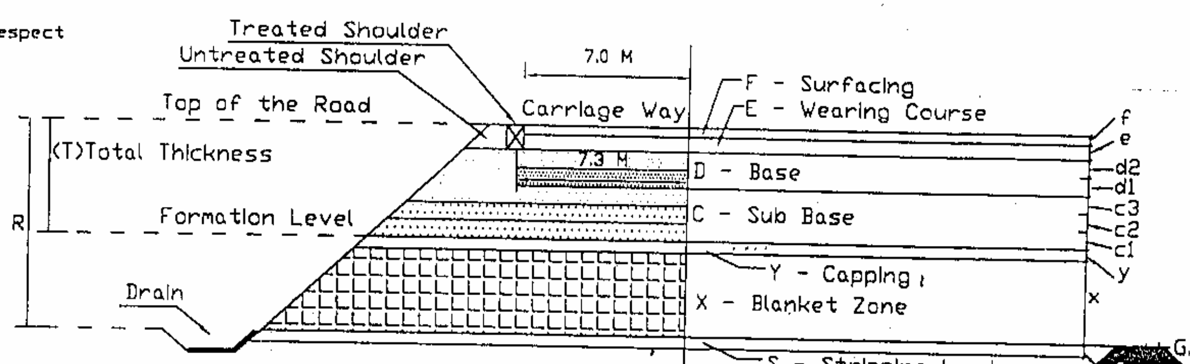
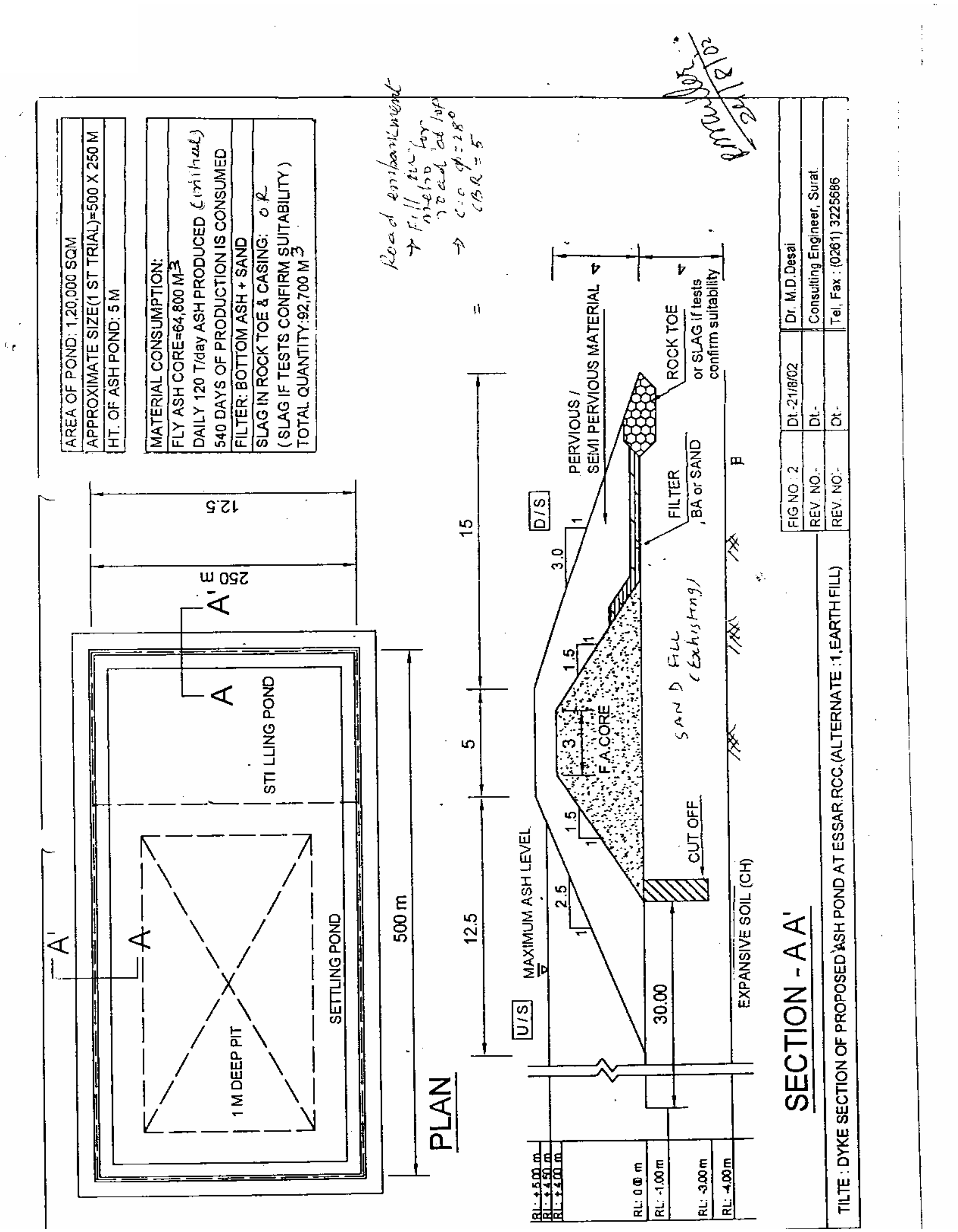


FIG 1 SECTION OF TYPICAL ROAD PROFILE IN EMBANKMENT

FIGURE 1 : TYPICAL ROAD SECTION IN EMBANKMENT (TWO LANE SINGLE CARRIAGE WAY)EXPANSIVE SUB SOIL (CBR<2)		FIG NO : 1	Dt.-6/9/02	Er. R.M.Tailor
		REV. NO:	Dt.-	Consulting Engineer,
	SCALE: NTS	REV. NO:	Dt.-	Surat.

RM Tailor


AREA OF POND: 1,20,000 SQM
APPROXIMATE SIZE(1 ST TRIAL)=500 X 250 M
HT. OF ASH POND: 5 M
MATERIAL CONSUMPTION:
FLY ASH CORE=64,800 M3
DAILY 120 T/day ASH PRODUCED
540 DAYS OF PRODUCTION IS CONSUMED
FILTER: BOTTOM ASH + SAND
SLAG IN ROCK TOE & CASING:
(SLAG IF TESTS CONFIRM SUITABILITY)
TOTAL QUANTITY:92,700 M3
12.5
250 m
A'
A
STILLING POND
SETTLING POND
1 M DEEP PIT
500 m
PLAN
12.5
5
15
U/S
D/S
MAXIMUM ASH LEVEL
2.5
1
1.5
3
F.A.CORE
3.0
PERVIOUS /
SEMI PERVIOUS MATERIAL
ROCK TOE
or SLAG if tests
confirm suitability
FILTER
BA or SAND
CUT OFF
30.00
EXPANSIVE SOIL (CH)
RL: +5.00 m
RL: +4.50 m
RL: +4.00 m
RL: 0.00 m
RL: -1.00 m
RL: -3.00 m
RL: -4.00 m
SECTION - A A'
FIG NO : 2
REV. NO.-
REV. NO:-
Dt.-21/8/02
Dt.-
Dt.-
Dr. M.D.Desai
Consulting Engineer, Surat.
Tel, Fax : (0261) 3225686
TILTE : DYKE SECTION OF PROPOSED ASH POND AT ESSAR RCC.(ALTERNATE :1,EARTH FILL)

MDD / 405 / 18

Design of Flexible Pavement Using Paver Blocks

For Petrol Pumps, Parking Lots and Industrial / Rural Streets.

BY:
Dr. M.D.Desai,
EFGE Consultant, Surat.

CLIENT:
M/s Vyara Tiles Pvt. Ltd.,
SURAT.

17th October 2004

Design of Flexible Pavement Using Paver Blocks

For Petrol Pumps, Parking Lots and Industrial / Rural Streets.

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Soil Exploration, Earthwork, Foundation, Ground Improvements and other Civil Engineering Problems

Ref. No. : MDD / 405 / 18 - 01

Date : Dt: 17 / 10 / 04

Design of Flexible Pavement Using Paver Blocks
For Petrol Pumps, Parking Lots and Industrial / Rural Streets:

1.0 SCOPE:

As a consultancy services to client M/s Vyara Tiles, working design has been on references ICPI Tech. Spec. No: 4, BS 7533 – 1992, Draft IS CED 5(6064), published records etc. The aim is to provide a basic minimum design parameters to start formulating

(a) Proposal for site,
(b) Examine economic alternatives using Aggregate Base, Asphalt Base, Cement treated Base(Lime Fly Ash Concrete IRC: SP 20 - 2002) for a site
(c) Finalize design quality control criteria for a site.

2.0 STRUCTURE:

The structure and terminology for Flexible Paver Pavement are shown in Fig: 1 & Fig: 2.

- Site requiring earth filling, the N.G.L. will be also treated as Sub-grade for the fill. (Fig: 2)
- For N.G.L. & filled up level ground the excavated level for placing Sub-base is treated as Sub-grade.
- Protection, if site dictates a Geo-textile filter as separator
- Sub-base
- Base
- Geo-filter
- Sand bedding
- Pavers (60 or 80 mm Height)
- Pre-cast curbing founded over the Sub-base
- Compound cum Toe retaining wall, Cut off for seepage and drain to divert surrounding water flow.

3.0 WHY PAVERS?

- Advantage of concrete materials with flexibility of asphalt pavements.
- High strength concrete pavers have higher resistance to weathering, high abrasion and excellent skid resistance.
- The surface is not damaged by petroleum products, concentrated loads and high rainfall / temperature variations.
- No waiting time for curing.
- Immediate use by traffic.
- No stress cracking and surface degradation because of joints.
- Aggregate base absorbs minor settlements without surface cracking.

- Speedy construction feasible with aggregate base and access to under ground services.
- Mechanical installation of pavers save cost / time.
- Pavers can be reinstalled by using same paver units easily.
- Provides a pilot system on the problematic Sub-grade - Expansive, Collapsible, Shrinkable and Sensitive till equilibrium is attained.
- Permits temporary flooring and traffic movements in areas where ultimate underground duct planning is phased over years.

4.0 PRINCIPLES OF INTERLOCK:

System must ensure control of vertical (shear), horizontal (displacement) as well as Rotational (torsion) stresses due to breaking, turning, accelerating heavy vehicles on pavements.

The best results are reported by using 45° Herringbone structure illustrated in Fig: 3. The side friction is controlled by friction between block and sluiced sand. The horizontal movements are checked by friction of bedding sand & end restrains by Curb wall.

Rotation is controlled by thickness of Block, Pattern of laying & Curb wall. Provision of slight crown to drain rain-water will ensure better durability.

The performance of Paver depends on

(a) Compressibility of fill,
(b) Water table fluctuations,
(c) Drainage around fill / dyke,
(d) CBR of Sub-grade & its state of wetting / drying cyclically,
(e) Selection of materials and its compaction in base, sub-base, sub-grade,
(f) Traffic intensity,
(g) Restraints provided by curb wall,
(h) Change in friction / grain size in bedding sand and sub-base over years.

5.0 DESIGN:

5.1 <u>Flexible Pavement for Petrol Pumps, Parking Lots and Industrial / Rural Streets on Moderately Expansive Soils:</u>

5.1.1 Type of Sub-grade soil:

Moderate Expansive CH Group Clay.

5.1.2 Other Environmental / Surrounding Conditions:

(a) Ground Water Table: Not available up to 3.0 m below G.L.
(b) Rain Water Drainage: Excellent
(c) Environmental Problems of near by Plots: Negligible
(d) Cyclical Flooding & Drying: No
(e) Sub-grade: Compacted N.G.L. or Well Compacted Fill of CNS soil.

5.1.3 Sub-grade Preparation:

Step: 1 – Plough the Dry surface,
Step: 2 – Wetted to OMC + 4 %,
Step: 3 – Cured for 24 hours,
Step: 4 – Rolled to MDD by Sheep or Pneumatic rollers.

Thickness of compacted Sub-grade: Minimum 100mm, Desirable 150mm (CBR of compacted samples in Lab > 8, under appropriate condition of wetting) Field CBR of compacted Sub-grade > 5, under appropriate condition of wetting.

5.1.4 Pavement Structure:

The pavement is designed for 20 years life.

(I) <u>Urban Roads (Traffic Intensity: 8.3 million EALs):</u>

<u>Note:</u>

In IS draft, Very Heavy Traffic of more than 1500 commercial vehicles exceeding 30 KN laden weight or more than 5 million standard axles for life of 20 years.

The Pavement is designed for 3 different categories of Base structure.

A1 – Flexible Pavement with Unbound Aggregate Base

A2 – Flexible Pavement with Asphalt – Treated Base
With Marshall Stability 8000 N

A3 – Flexible Pavement with Cement – Treated Base
With 7 days compressive strength 4.5 Mpa
(M10 or more Grade of Concrete)
(Refer IRC: 60 for Lime-Fly ash concrete as Pavement Base/Sub-Base & IRC: 74 for Lean Cement Concrete and Lean Cement–Fly Ash Concrete as Pavement base or Sub-base.)

The Pavement thicknesses for above 3 categories are as given in Table: 1.

Table: 1 – Pavement Thickness for Urban Roads

Category	Sub Grade * Thickness **(z) in mm**	Sub- Base ** Thickness **(x) in mm**	Base Thickness **(y) in mm**	Paver *** (80 mm) & Cushion (30 mm) Thickness **(p) in mm**	Total Thickness **(T) in mm**
A1	150	600	100	110	810
A2	150	150	100	110	360
A3	150	360	100	110	570

*- Minimum Sub-grade CBR > 5, under appropriate drainage condition

** - Minimum Sub-base CBR > 15, under appropriate drainage condition

*** - For sharp turning Corners Pavers with 100 mm or more height are advisable.

<u>Check As per IRC: 37</u>

CBR = 5%, Design Traffic = 8.3 msa,

Total Pavement Thickness desirable as per IRC: 37 is 660 mm + capping 600 - 1000 mm

(Sub-base, Base and Top Asphalt bound Layer, No Pavers)

<u>Divided Thickness</u>

Compacted Sub-grade thickness = 150 mm + capping

Sub Base thickness =.300 mm

Base thickness = 250 mm

DBM thickness = 70 mm

Surfacing thickness = 40 mm

Total thickness = **660 mm** < Designed Thickness **810 mm** (As per A1) **<u>OK</u>**

(II) Rural Roads (Traffic Intensity: 3.5 million EALs):
(600 Commercial Vehicles per Day):

The Pavement is designed for same 3 different categories of Base structure as in Para – 5.1.4 (I)

The Pavement thickness for Rural Road is as given in Table: 2.

Table: 2 – Pavement Thickness for Rural Roads

Category	Sub Grade* Thickness **(z) in mm**	Sub- Base** Thickness **(x) in mm**	Base Thickness **(y) in mm**	Paver *** (80 mm) & Cushion (30 mm) Thickness **(p) in mm**	Total Thickness **(T) in mm**
A1	150	390	100	110	600
A2	150	100	100	110	310
A3	150	220	100	110	430

*- Minimum Sub-grade CBR > 5, under appropriate drainage condition
** - Minimum Sub-base CBR > 15, under appropriate drainage condition
*** - For sharp turning Corners Pavers with 100 mm or more height are advisable.

Check As per IRC: 37
CBR = 5%, Design Traffic = 3.5 msa,
Total Pavement Thickness desirable as per IRC: 37 is 550 mm
(Sub-base, Base and Top Asphalt bound Layer, No Pavers)
Divided Thickness
Compacted Sub-grade thickness = 150 mm + capping
Sub Base thickness = 240 mm
Base thickness = 250 mm
Surfacing thickness = 70 mm
Total thickness = **560 mm** < Designed Thickness **600 mm** (As per A1) **OK**

Notes:
1) Curbstone to rest on compacted Sub-grade, L - shaped with height = (y + p)
For A1 Height of Curbstone = 100 + 110 = 210 mm
For A2 Height of Curbstone = 100 + 110 = 210 mm
For A3 Height of Curbstone = 100 + 110 = 210 mm
2) For case where Formation Level requires higher Total Pavement thickness, Layer of compacted CNS soil, Compacted to MDD at OMC in each layer of 200 mm is used as Make up / Capping zone at Sub-base. The fill is protected from lateral flow by Toe wall and seepage of water by Cut-off at G.L. above re-compacted natural soil. (Fig: 2)

5.1.5 Specifications:
Tentative specifications considered in designing, subject to modification for betterment, are shown in Annexure – I. This will provide starter for designer.

The most of the specifications are based on IRC, MORT&H and Ministry of transport specifications. The specifications in detail should aim at minimizing field quality control at remote sites and economize cost.

5.2 Flexible Pavement for Petrol Pumps, Parking Lots and Industrial / Rural Streets on Poor Sub Soil:

5.2.1 The design of pavements using pavers on poor, weathering sub-grade subjected to cyclical wetting, flooding drying is covered here. The subsoil is poor expansive or collapsible clay or soft marine clay; the soil CBR under worst appropriate condition of wetting is < 2.

5.2.2 The approach, to avoid recurring maintenance, would be to replace soil if the layer is thin.

It must be noted that colour is no indication of soil behaviour.

If layer is not economically replaceable, it is advisable to get geo-technical consultancy for the soil stabilization or ground improvement.

5.2.3 Recent decade have brought out feasibility of using stabilized Fly ash as lightweight fill material with better performance than local non-CNS mixes soils. The compacted Fly ash fill shall be planned at 98 % density and OMC.(Heavy Compaction) The field CBR in such case is adopted as 5. The design given earlier could be adopted on this fill as a sub-grade.

5.2.4 The thickness of improved sub-grade shall be minimum 300 mm in relatively better sub soils. It has to be 600 to 1000 mm if subsoil is sensitive to weathering & environmental water content changes. There are options to improve deeper insitu strata by use of Lime Piles, Sand Drains, Geo-reinforcements such as Mattress, Gabbions etc.

5.2.5 The performance of pavers will be poor if subsoil and fill on which it rests will be poor. A geo-textile reinforced fill with flexible Asphalt bonded base could offer better economical solution for some sites.

5.2.6 To conclude the objective is to improve the sub-grade such that CBR of 5 will achieve, hence design of Article 5.1 can be adopted.

6.0 ECONOMICS:

The cost aspect is governed by Cost of materials – Aggregates, Bitumen, Cement, Fly ash, Sand, etc. The design given is a typical pattern.

For site where good stone is easily available the unbound aggregate base and sub-base could be planned using economical thickness of each by proper designing. of base.

The modern material byproducts like Fly ash could prove economical & eco-friendly as explained ~~latter~~. earlier.

7.0 SCOPE FOR NEW MATERIALS:

Fly ash properly confined, drained and compacted to 98 % of modified proctor could provide sub-base with CBR = 15% (Pg – 247, IRC: SP -20)

The future trend and development which will be adopted in design will be, use of Fly ash with some 10 to 20 % clay as fill materials for the embankment or raising of existing low level ground to road formation level for petrol pumps. (Refer IRC: SP – 20, 2002, Pg – 222). The layers of properly mixed fly ash, 15 – 20 % soil and water in pan mixture, placed and rolled in

200 mm layers may provide CBR value more than 50. It will provide cheapest, Eco-friendly material for sub-base and base for some sites.

Similarly lime - fly ash stabilized soil could provide attractive alternative materials for sub-base and base in some locations. Use of waste fly ash and prevention of exploitation of topsoil layer would be demand of environment in future. The laboratory design mix of fly ash – soil - lime for strength will be specified with OMC and density (Heavy Compaction Test). Both mix in place or control-mixing techniques can be adopted (Refer IRC: SP– 20, 2002, Pg – 232).

Lime – Fly ash bound macadam (LFBM, Article 9.5 of IRC – 20), Lime fly ash concrete (suitable as replacement of WBM in heavy rainfall expansive soil areas (Article 9.6 of IRC – 20), Roller compacted Fly ash concrete pavement are alternative materials for Sub-base.

Base course of Dry Lean Fly ash concrete (cement binder) designed for zero slump can be an alternative. Dry lean fly ash concrete replacing 50 % sand (by wt) in conventional mix b equal absolute volume of fly ash (e.g. 1 Cement : 2.5 Sand : 2.5 n Fly ash : 10 Aggregates, n = ratio of sp. Gravity of ash / sp. Gravity of sand) may give 28 days strength 13 MPa. This may replace the sub-base economically at some of the sites where aggregates are not good in quality.

Acknowledgement:
The design was assisted by Er. Ravin Tailor and Mr. Mehul Jain of Vyra tiles provided considerable data of field of problems.

(Dr. M.D.Desai)

Annexure – I

Specifications:

1) **Grading for Aggregates in Base & Sub-base Layer:**

I. S. Sieve	Base Layer		Sub-base Layer	
	% Passing	Adoptable Variation	% Passing	Adoptable Variation
50.00 mm	100	(- 2)	100	(- 3)
37.50 mm	95 – 100	(± 5)	90 – 100	(± 5)
19.00 mm	70 – 89	(± 8)	-	-
9.50 mm	50 – 75	(± 8)	-	-
4.75 mm	35 – 55	(± 8)	30 – 60	(± 10)
600 µ	12 – 55	(± 5)	-	-
75 µ	0 – 8	(± 3)	1-12	(± 5)

2) **Grading for Bedding Sand for Pavers:**

I. S. Sieve	% Passing
10.00 mm	100
4.75 mm	95 – 100
2.36 mm	80 – 100
1.18 mm	50 – 85
600 µ	25 – 60
150 µ	2 – 10

3) **Geo-filter:**
UV Stable, Polypropylene Woven Geo-filter

Type: 1, For Soil CBR > 5
Having 90 ± 2 g / m^2 wt, k = 25 or more liter / m^2 / sec

Type: 2, For Soil CBR < 5
Having 140 ± 2 g / m^2 wt, k = 25 or more liter / m^2 / sec

4) **Cement Treated Base:**
With 7 days compressive strength 4.5 Mpa
M10 or more grade of Concrete (As per IRC: 60 for Lime-Fly ash concrete as Pavement Base/Sub-Base & IRC: 74 for Lean Cement Concrete and Lean Cement–Fly Ash Concrete as Pavement base or Sub-base.)

5) **Asphalt Treated Base:**
Having Marshall Stability 8000 N

6) Wet Mix Macadam for Sub-base.

The Aggregate for Wet Mix Macadam shall confirm to following Physical Properties & Grading

Physical requirements of Coarse Aggregates for Wet Mix Macadam for Sub-base / Base Course:

1. Los Angeles Abrasion Value: 40 % (Maximum) or
   Aggregate Impact Value: 30 % (Maximum)
2. Combined Flakiness and Elongation Indices (Total): 30 % (Maximum)

Grading Requirements of Aggregates For Wet Mix Macadam:

I. S. Sieve Designation	% By Wt. Passing the I. S. Sieve
53.00 mm	100
45.00 mm	95-100
26.50 mm	-
22.40 mm	60-80
11.20 mm	40-60
4.75 mm	25-40
2.36 mm	15-30
600.00 micron	8-22
75.00 micron	0-8

The material finer than 425 micron shall have Plasticity Index (PI) not exceeding 6.

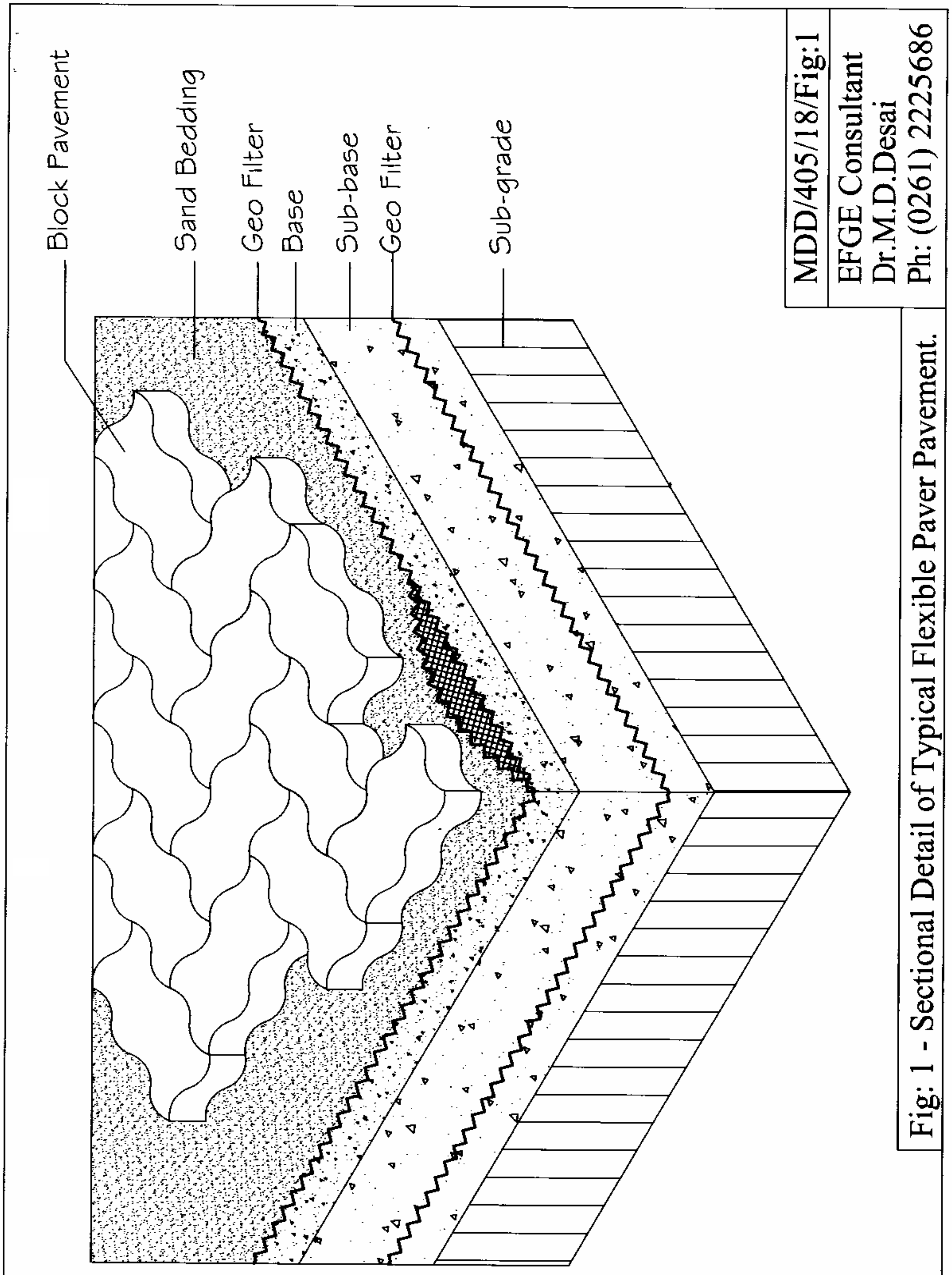


Fig: 1 - Sectional Detail of Typical Flexible Paver Pavement.

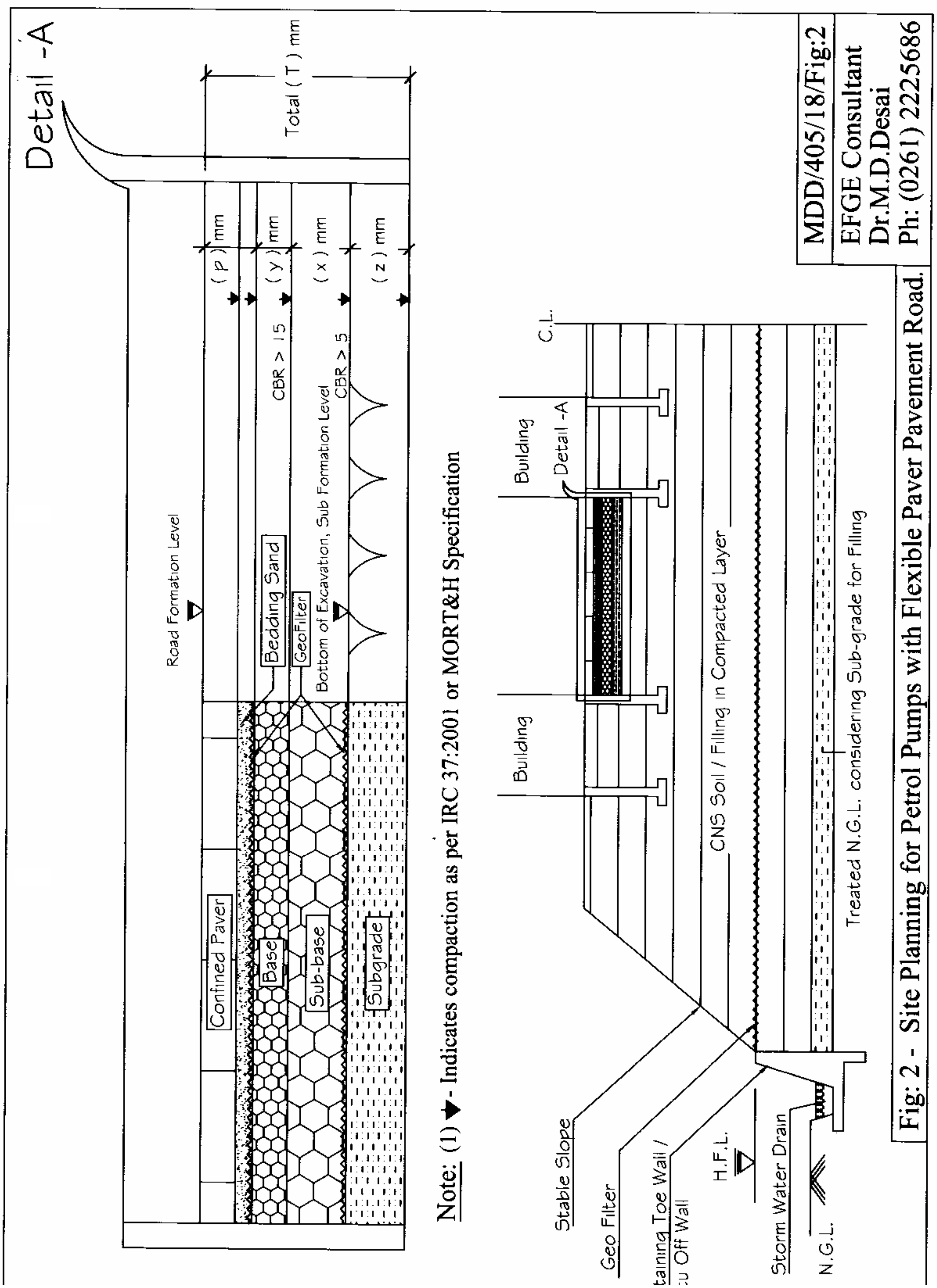
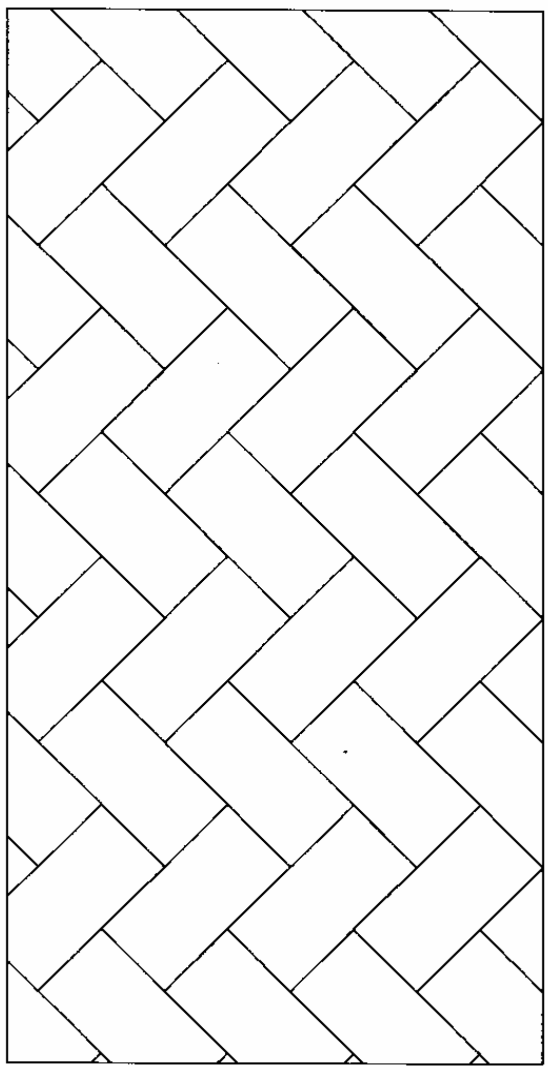


Fig: 2 - Site Planning for Petrol Pumps with Flexible Paver Pavement Road.



45° Herringbone Pattern

Fig: 3 - Paver Laying Pattern for Flexible Paver Pavement Road.

MDD/405/18/Fig:3

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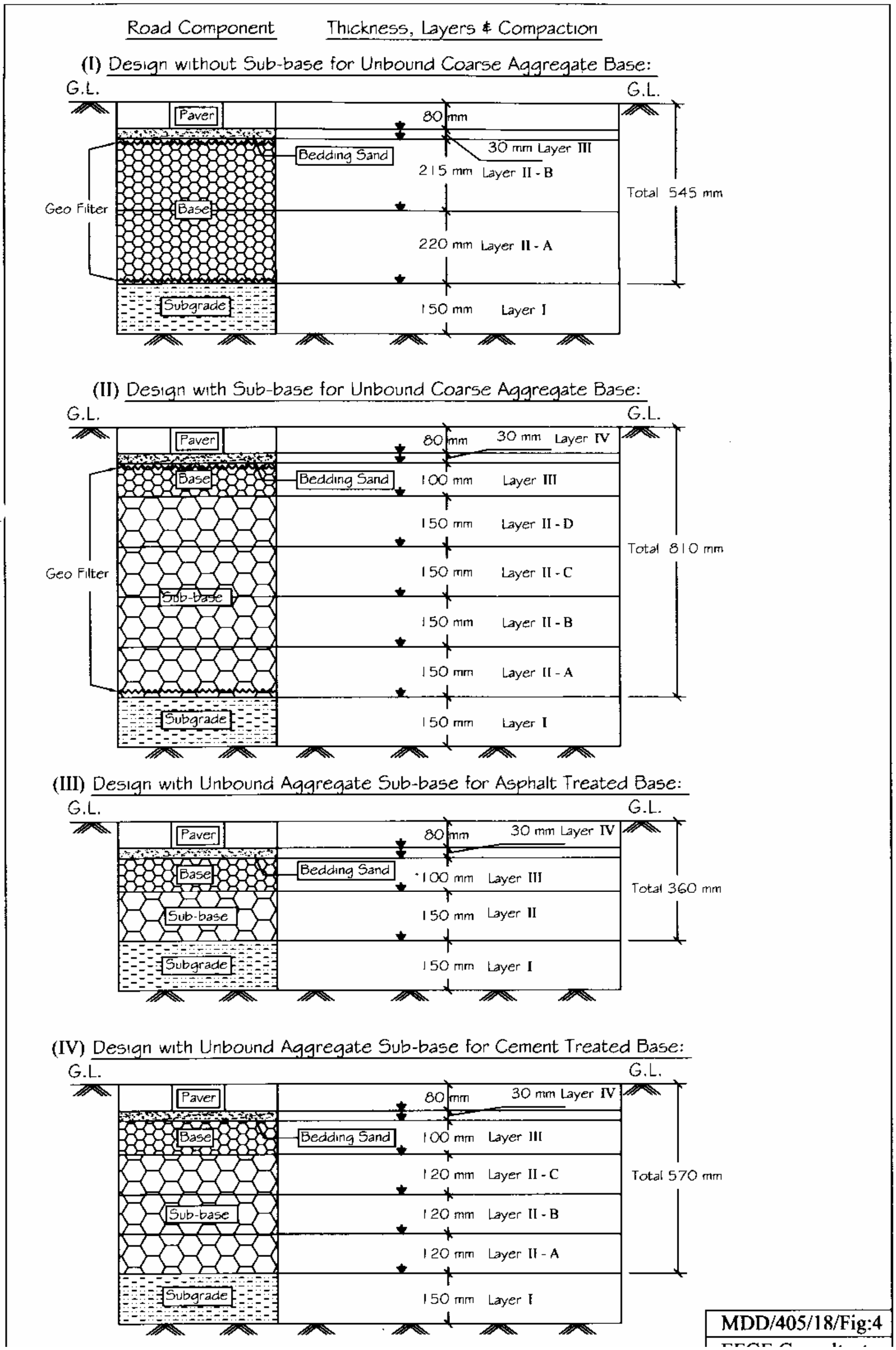
Road Component
Thickness, Layers & Compaction
(I) Design without Sub-base for Unbound Coarse Aggregate Base:
G.L.
G.L.
Paver
80 mm
30 mm Layer III
Bedding Sand
215 mm Layer II - B
Geo Filter
Base
Total 545 mm
220 mm Layer II - A
Subgrade
150 mm Layer I
(II) Design with Sub-base for Unbound Coarse Aggregate Base:
G.L.
G.L.
Paver
80 mm
30 mm Layer IV
Base
Bedding Sand
100 mm Layer III
150 mm Layer II - D
Geo Filter
Total 810 mm
150 mm Layer II - C
Sub-base
150 mm Layer II - B
150 mm Layer II - A
Subgrade
150 mm Layer I
(III) Design with Unbound Aggregate Sub-base for Asphalt Treated Base:
G.L.
G.L.
Paver
80 mm
30 mm Layer IV
Base
Bedding Sand
100 mm Layer III
Total 360 mm
Sub-base
150 mm Layer II
Subgrade
150 mm Layer I
(IV) Design with Unbound Aggregate Sub-base for Cement Treated Base:
G.L.
G.L.
Paver
80 mm
30 mm Layer IV
Base
Bedding Sand
100 mm Layer III
120 mm Layer II - C
Total 570 mm
Sub-base
120 mm Layer II - B
120 mm Layer II - A
Subgrade
150 mm Layer I
MDD/405/18/Fig:4