

**EFFECT OF ADDITION OF MARBLE DUST IN SUBGRADE SOIL ON
THE FATIGUE AND RUTTING BEHAVIOUR OF FLEXIBLE
PAVEMENT**

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for the degree of

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IN
CIVIL INFRASTRUCTURE ENGINEERING**

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DECLARATION

I, Shubham Moudgil, hereby declare that this thesis entitled “**Effect of Addition of Marble Dust in Subgrade Soils on the Fatigue and Rutting Behaviour of Flexible Pavement**” is an authentic record of my study carried out as requirements for the award of degree of **Master of Engineering in Civil Infrastructure Engineering** in the Civil Engineering Department, Thapar University, Patiala, under the supervision of **Mr. Rajesh Pathak, Associate Professor and Mr. Tanuj Chopra, Assistant Professor**, Department of Civil Engineering, Thapar University, Patiala during July 2015 to July 2017. This matter embodied in this report has not been submitted in part or full to any other university or institute for the award of any degree.

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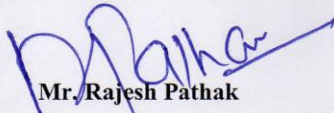


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CERTIFICATE

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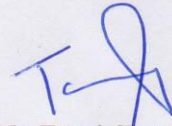


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ABSTRACT

Harnessing of industrial waste for improvement of soils is economical and efficient. It helps in enhancing the soil properties and overcoming the disposal problem. Therefore, it is essential to understand the properties of these wastes in order to understand their performance level.

In this study, waste marble dust was added to stabilize the soil which was collected from Shambu Kalan, Punjab. This soil was classified as CL based on Indian Standard Classification system (ISCS). Marble dust was added at 10%, 15%, 20% and 25% by weight of soil. Index properties (liquid limit, plastic limit, plasticity index) were determined at different percentages of marble dust specified above. Similarly, California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) was calculated at Optimum Moisture Content (OMC), which was determined by Proctor test for the modified and parent soil.

Observations based on tests specified above showed 20% of waste marble dust to be optimum for strengthening of the parent soil.

Also, the evaluation of observed CBR value with the predicted CBR value (which was calculated from the regression analysis using SPSS) was done.

Resilient Modulus was calculated for the subgrade soils using equations specified by Heukelom and Klomp, Thompson and Robnett, Transportation and Road Research Laboratory, Erdem Çöleri. Based on the results obtained, pavement thickness was determined for designing flexible pavement section as per IRC 37-2012 using IITPAVE.

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CHAPTER 1

INTRODUCTION

There is a great urgency nowadays for utilization of alternate materials which have better engineering properties than the traditional materials. One of the most important material which is used in the construction projects like embankment, subgrade, earth dams, earth canals, is soil. All of the resistive characteristics may be provided by soil in these cases. So, it is necessary to suggest methods which can improve soil characteristics.

Clay is a fine-grained soil which shows swelling upon its contact with water and shrinkage when water content decreases. In engineering terms, soil having particle size less than 2 microns is classified as clay. The behaviour of soil is highly dependent on the water content. Plasticity index is used to classify the fine-grained soil. Knowledge of clay mineralogy is important to know its behaviour. Illite, kaolinite and montmorillonite are mainly known clay minerals among which, montmorillonite shows high swelling. Also, clayey soils (with high moisture levels) generally show low California Bearing Ratio (CBR) values as compare to other type of soils. As pavement design is dependent upon the California Bearing Ratio (CBR) value of soil, this low California Bearing Ratio (CBR) value of clayey soil results into high pavement thickness and subsequently into higher construction cost. In India, clayey soils are mainly found in Tamil Nadu, Karnataka, Gujrat, Rajasthan, Andhra Pradesh, Madhya Pradesh and Maharashtra. It is also found in some areas of Punjab at a depth of 1.5-3 metres below surface level.

India is a developing country and its road network is expanding at a reasonable rate. Although, due to its narrow fiscal the expansion of road network is very challenging. So, there is a great urgency to suggest more convenient cost-effective methods, coping with the traffic needs. Construction cost can be substantially decreased by using regional materials which comprises of construction of granular layers of pavement by using soil available in that particular region. For this purpose, stabilizers can be used which can prove to be economical for pavement construction and maintenance. Major issue among the Indian industries is the demolition of waste products which are generated by them. Application of these waste materials for engineering purposes can cut down the disposal problems and environmental issues caused by their disposal.

A pavement section mainly consists of bituminous layers resting over base and/or sub-base which are together compacted over an advisable subgrade. Soil subgrade proves to be most essential element of a pavement section. There can be rutting in the granular base and subbase layers if the soil subgrade does not have appropriate bearing capacity. This rutting in the bottom layers can steadily cause fatigue cracking in the bituminous layers of the pavement section.

1.1 Soil Stabilization

Soil stabilization is generally defined as modification of one or more properties of soil such that desirable engineering properties are achieved. It is mainly done to

- increase the shear strength of an existing soil strata for strengthening its load bearing capacity
- to improve permeability and shrink/swell properties of soil.

Thus, soil stabilization results in improvement of subgrade (to be used for pavements and foundations). The application of soil stabilization can be found in roadways, airports, parking areas etc. where parent soil is not good enough for construction.

Soil stabilization is broadly classified into two categories i.e. mechanical stabilization and chemical stabilization.

Mechanical stabilization involves principle of friction. Strength is upgraded due to the friction between soil and added admixture.

Whereas, in case of chemical stabilization, properties are enhanced due to chemical reaction between soil minerals and added chemical.

As mentioned above, soil stabilization can be very useful but proper testing should be done so that it can aid to a good design. Also, it is very important to determine the optimum percentage of admixture required for stabilization.

1.1.1 Soil Stabilization using waste materials

As mentioned above, large amount of industrial waste is generated annually. Millions of tons of this generated waste not only causes disposal problems, but also increases environment contamination and health risks. Over the past few years, various studies are conducted for utilization of these industrial wastes for improving soil properties. Addition of waste materials like granulated blast furnace slag, rice husk ash, fly ash, plastic waste resulted in improvement of soil characteristics. Also, there are some studies on the improvement of soil using waste tire

rubber, egg shell powder, coir waste etc. which generally have low production in comparison to the industrial wastes. So, there is enough literature available which proves that waste materials can improve soil properties and can be used for various engineering purposes.

1.1.2 Soil Stabilization using waste marble dust

Marble production in India is very high. Large number of industries or marble cutting plants are there in India. Like most industries, waste generation in these marble units is also very high. Various studies are made in the past few years for utilization of waste produced by marble industries, for soil stabilization.

Marble is classified as ‘minor mineral’ as defined under clause of Section 3 of Mines and Minerals (Development & Regulation) Act, 1957. Geologically, marble is defined as metamorphosed limestone which is produced by re-crystallization of carbonate minerals under condition of thermal and regional metamorphism. In India, marble is found in following states: Rajasthan, Gujrat, Haryana, Andhra Pradesh, Madhya Pradesh, Jammu & Kashmir, Maharashtra, Sikkim, Uttar Pradesh and West Bengal. According to Indian Bureau of Mines, about 95% of the processing capacity is found to be in Rajasthan. Annual marble slab production in the state is found to be around 1,000 million sq. ft. whereas it is 3,000 million sq. ft. for polished tiles. Table 1.1 includes physical properties of marble blocks, slabs and tiles as mentioned in Indian Minerals Yearbook-2015.

Table 1.1 Physical properties of marble

S. No.	Characteristic	Requirement	Method of test
1	Moisture absorption after 24 hours immersion in cold water	0.4% max. by weight	IS: 1124-1974
2	Hardness	3 min.	Mohs’ scale
3	Specific gravity	2.5 min	IS: 1172-1974

Source: Indian Minerals Yearbook, 54th Edition (November, 2016)

The waste generated in the marble industries is around 40% of the total production per annum. The processing waste which is dumped into the river beds is quite threatening for the porosity of aquifer zones. So, problems like this implies that there is a need to develop a method to minimise and using this waste marble dust for enhancing soil properties can be helpful.

Recently, Rajasthan State Pollution Board asked National Highways Authority of India to use waste marble slurry for the construction of 120 Km stretch of four lane road from Gomati to Udaipur. So, it is important to study the effect of waste marble dust on properties and strength of soil and to identify its optimum percentage in the mix.

1.2 Resilient Modulus

The authenticity of pavement design depends on the accuracy by which material properties have been determined which are further required for the various prediction models. Resilient Modulus (M_R) is one of the most important factor in the mechanistic flexible pavement design. Resilient Modulus is the estimation or measurement of the elasticity of the material at a particular stress and temperature. Mathematically, it is defined as ratio of applied deviator stress to the recoverable strain.

$$M_R = \frac{\sigma_d}{\epsilon_r} \dots\dots\dots(1.1)$$

The laboratory test for determining resilient modulus has deviator stress applied repeatedly along with a steady cell pressure for the measurement of resilient axial strain. As the load cycles increases under repeated load tests, the secant modulus also increases. The value becomes steady after a number of load cycles and that constant value of modulus is known resilient modulus. This is a pricey and time-consuming test which requires large number of test samples for obtaining decent results. Due to intricacy and high cost of tests, various correlations have been developed to predict the resilient modulus. California Bearing Ratio (CBR) is mainly the parameter which is used to determine resilient modulus of subgrade. Even, Indian Road Congress has suggested correlations to determine resilient modulus based on the effective California Bearing Ratio (CBR) value. Although, various empirical relations have been developed by researchers showing that resilient modulus is not solely dependent on the California Bearing Ratio (CBR) value. These empirical relations showed that index properties of soil also play important part in the prediction of resilient modulus.

Relations specified by Heukelom and Klomp (1962), Thompson and Robnett (1979), Transportation and Road Research Laboratory (TRRL), Erdem Çöleri (2007) are used to calculate the resilient modulus and depending upon the values obtained, pavement design is done.

1.3 Regression Analysis using SPSS Statistics

SPSS (Statistical Package for the Social Sciences) statistics is an IBM developed software package of Java platform used for logical batched and non-batched statistical analysis.

Regression analysis is a statistical technique that is used to predict the variable of interest (known as dependent variable, criterion or target, the outcome) from a set of other variables (known as independent variables, regressor or explanatory variables, the predictor). Generally, a regression analysis having two or more independent variables is called as multiple regression analysis.

Regression analysis can be used for forecasting and prediction of various models. The results of regression analysis also help in depicting the independent variable which has major effect on the value of dependent variable. Checking R-squared value of the model is most common way of deciding its reliability. Also, the p -value obtained by the ANOVA table can be used for determining the significance of generated models.

Multiple linear regression is used for developing a correlation between California bearing ratio (CBR) and index properties of soil. Data to be used for the regression analysis, is to be obtained from the previous literature done on the stabilization of soil using marble dust/powder. The comparison of the CBR value calculated from the correlation is to be done with the CBR value obtained from the experimental study. Further, an equation is to be developed, using previous data and experimental data obtained.

CHAPTER 2

LITERATURE REVIEW

Over the past few years, various studies are conducted on the utilization of waste marble dust/powder/slurry, for the purpose of soil stabilization. Effect of marble waste on index properties and strength characteristics of soil was studied by various authors. Review of all the work done using marble waste is explained in section 2.1.

2.1 Literature review on soil stabilization using waste marble dust

Misra A K *et al.* (2009) added waste marble slurry dust to the soil obtained from construction site at Sirola to Kuncholi road, Rajasthan. Soil was classified as sand associated with silt of low compressibility (SM). Marble slurry dust was obtained from Moonlight marbles, Rajasthan and was mixture of clayey and silty size particles with low compressibility (CL-ML). Waste marble slurry dust was added in four different percentages i.e. 10%,20%,25% and 30% by weight of soil. Addition of waste marble slurry dust to the soil showed decrease in liquid limit and increase in the plastic limit values. Plasticity index of soil increased upon addition of waste marble slurry dust. Proctor test showed slight increase in the optimum moisture content and decrease in the maximum dry density, after addition of marble slurry dust. CBR values obtained after 10%, 20%, 25% and 30% addition of marble slurry dust to the soil were 11.6, 11.9, 12.1 and 12.1 respectively. Untreated soil i.e. soil with no marble slurry dust, had CBR value of 11. Therefore, there was increase in the CBR value when marble slurry dust was added. Similarly, there was continuous increase in the unconfined compressive strength. The unconfined compressive strength changed from 1.3 kg/cm² to 1.4 kg/cm², 1.4 kg/cm², 1.5 kg/cm² and 1.75 kg/cm² upon addition of marble dust by 10%,20%,25% and 30% by weight of soil.

Viswakarma A *et al.* (2013) added waste marble slurry to the black cotton soil at 40%, 50%, 60% and 70% by weight of soil. Marble dust samples were collected from Udaipur and the nearby marble cutting units. Based on different proportions of soil and marble, various experiments were conducted. Proctor test showed that there was decrease in optimum moisture content (OMC) and increase in maximum dry density (MDD) on addition of waste marble slurry. Optimum moisture content (OMC) value was increased by approx. 22% on addition of 70% marble slurry to the soil and the value of maximum dry density (MDD) was increased by

approx. 13%. The value of liquid limit decreased upon addition of waste marble slurry. The values of liquid limit obtained for untreated soil and soil with 70% of waste marble slurry were 46.90 and 33.80 respectively. There was no change in the plastic limit values on addition of waste marble slurry. Specific gravity test was conducted for untreated soil, waste marble slurry and soil with 50% of waste marble slurry and the values obtained were 2.596, 2.680 and 2.459, respectively. There was increase in the CBR value up to marble content of 50% then the value started decreasing. CBR values obtained for soil, soil with 40% marble, soil with 50% marble, soil with 60% marble and soil with 70% marble were 0.75, 2.26, 4.16, 0.82 and 0.88 respectively. The author concluded that marble slurry can be utilized for improving soil properties which will help in reducing its disposal problems up to some extent.

Gandhi K S (2013) carried out study on stabilization of expansive soil of Surat region using rice husk ash & marble dust. Marble dust was added 0%, 10%, 20% and 30% by weight of soil. Soil was tested with marble dust to determine its index properties, swelling characteristics and strength characteristics. There was decrease in liquid limit of soil by about 30% with the addition of 20% of marble dust. The calculated values of liquid limit were 62, 44, 39, 39 for 0%, 10%, 20%, 30% addition of marble dust. Also, plastic limit and shrinkage limit of the expansive soil were decreased by approx. 18% and 23% respectively with the addition of 30% marble dust. The calculated values of plastic limit were 25, 23, 21, 20 for 0%, 10%, 20%, 30% addition of marble dust. Free swell index was decreased by 80% by addition of marble dust. The value of CBR increased constantly with the increased percentage of admixture. There was 40% increase in the CBR value on addition of marble dust. The CBR values were 3.8, 4.04, 4.16 and 4.2 for 0%, 10%, 20% and 30% addition of marble dust. Also, there was decrease in swelling pressure by about 30.5% on 30% addition of marble dust. The author observed that marble dust was more efficient as stabilizing agent than rice husk ash. Author also mentioned that mixing of marble dust with wet soil is much easier as compared to the rice husk ash. As both rice husk ash and marble dust are waste products and are available easily, hence they can be used for stabilizing the soil, author concluded.

Parte S S et al. (2014) investigated the effect of marble dust on properties of black-cotton soil. Marble dust was added from 0 to 40% at an interval of 10% by weight of black cotton soil. Soil used in this study was brought from Tewar, Jabalpur, Madhya Pradesh (India). The soil was classified as clay of high plasticity having specific gravity 2.58 and 95% fines. The marble dust used was collected from a marble cutting/polishing industry situated at Adharatal, Madhya

Pradesh (India). Liquid limit and plastic limit of the soil decreased upon addition of marble dust. Liquid limit values for 0%, 10%, 20%, 30% and 40% addition of marble dust in the soil were 57.67, 52.43, 42.51, 39.21 and 33.90, respectively. And plastic limit values for 10%, 20%, 30% and 40% addition of marble dust in the soil were 29.32, 28.06, 23.50, 21.61 and 17.23, respectively. Compaction tests showed decrease in the optimum moisture content (OMC) and increase in the maximum dry density (MDD). Optimum moisture content (OMC) of soil changed from 20.5% for the parent soil to 17.6, 17, 15.1 and 14.2 for 10%, 20%, 30% and 40% addition of marble dust. Whereas, maximum dry density (MDD) of soil changed from 1.72 g/cc for the parent soil to 1.78, 1.79, 1.83 and 1.86 g/cc for 10%, 20%, 30% and 40% addition of marble dust. The shrinkage limit of the black-cotton soil increased by adding marble dust. The shrinkage limit increased from 8.06% to 18.39%. California bearing ratio (CBR) of soil also increased upon addition of marble dust. The values of CBR for 0%, 10%, 20%, 30% and 40% addition of marble dust in the soil were 1.81, 2.30, 3.57, 3.72 and 4.17, respectively. Also, the values of unconfined compressive strength (UCS) increased upon addition of marble dust. The values of UCS for 0%, 10%, 20%, 30% and 40% addition of marble dust in the soil were 110.86, 120.56, 144.76, 156.03 and 175.46, respectively. Based on the results obtained, the author concluded that marble dust can modify the behaviour of black cotton soil and make it reliable for many engineering applications.

Abdulla R S *et al.* (2014) studied the effect of waste marble powder on the physical properties of soil. Two types of soil were used for the study named as Bastora soil (classified as CH) and Erbil airport soil (classified as CL). Waste marble powder used for the study was obtained from Penjwen, Said Sadiq and Pirmam. Addition of waste marble powder obtained from Penjwen to the Bastora soil, decreased the value of liquid limit of soil. The value decreased from 51.5 for natural soil to 39.9, 37.2, 35.2 for 10 %, 20 % and 30% addition of marble powder. Whereas, addition of waste marble powder obtained from Said Sadiq to the Bastora soil decreased liquid limit value from 51.5 for natural soil to 44, 40.55, 37.6. Addition of waste marble powder obtained from Pirmam to the Bastora soil decreased liquid limit value from 51.5 for natural soil to 45.8, 43.78, 40. Similarly, addition of waste marble powder to the Erbil airport soil decreased the liquid limit value. Addition of marble powder obtained from Penjwen decreased liquid limit value from 44.2 for natural soil to 35, 34.25 and 30.78. Also, addition of marble powder obtained from Said Sadiq decreased liquid limit value from 44.2 for natural soil to 35.16, 33.2 and 30.8. Addition of marble powder obtained from Pirmam decreased liquid limit value from 44.2 for natural soil to 35, 34.25 and 30.78. Addition of waste marble powder

obtained from these 3 sources also showed reduction in value of plastic limit of both types of soil. Maximum reduction in plastic limit for Bastora soil was from 28.44 to 22.8 by 30% addition of waste marble powder obtained from Penjwen. While for Erbil airport soil, maximum reduction was from 24.20 to 17.25 with 30% addition of marble powder obtained from Said Sadiq.

Bhavsar S N *et al.* (2014) added marble powder to the black cotton soil. Marble powder was added 30%, 40% and 50% by weight of soil. Effect of marble powder on engineering properties of soil was studied. Liquid limit of soil decreased from 43 to 27.45% with 50% addition of marble powder. Whereas, plastic limit of soil decreased from 16.89% to 9.3% with 50% addition of marble powder. Value of linear shrinkage also decreased by 83.12% with 50% addition of marble powder. Addition of marble powder showed positive impact on black cotton soil, author concluded.

Kumar M M *et al.* (2015) added marble powder to the expansive soil as 5%, 10% 15%, 20%, 25% by weight and study of compaction characteristics and strength characteristics was done. Soil was collected from Authoor, near Tiruchendur, Tamilnadu (India). The marble powder was collected from marble cutting industry at Tirunelveli, Tamilnadu (India). Soil was classified as clay with high plasticity (CH). The value of liquid limit decreased from 77% to 55% on addition of marble powder. Whereas, the plastic limit was increased approximately by 50% at 25% addition of marble powder. Optimum Moisture Content of clay went on increasing with the addition of marble powder. The value changed from 18% to 24%. Maximum dry unit weight of clay increased with 10% addition of marble powder. Further addition of marble dust resulted in reduction of maximum dry unit weight. The value of unconfined compressive strength (UCS) increased up to 15% addition of marble powder. The maximum unconfined compressive strength of the clay was 215 kN/m² at 15% addition of marble powder. Author concluded that utilization of waste marble powder for soil stabilization can reduce the disposal problems and preserve the ecological system.

Tarkeshwar P *et al.* (2016) In this paper, sandy clayey soil was stabilised using the combination of marble dust and GGBS in different proportion (i.e. 0%+0%, 5%+5%, 10%+10%, 5%+15% & 20%+20%) and the characteristic behaviour of modified soil in the laboratory was studied. The sandy clayey soil was obtained from BIT campus, Sindri, Dhanbad (India). GGBS was obtained from blast furnaces of cement (ACC plant, Sindri). Marble dust was obtained from a local marble stone shop. According to Unified Soil Classification system,

the soil was classified as clayey sand with low plasticity (CL). Addition of marble dust and GGBS to the soil showed increase in optimum moisture content (OMC) and decrease in the maximum dry density (MDD) values. Unsoaked and soaked CBR value obtained for untreated soil was 2.27 and 2.06 respectively. The values of unsoaked CBR changed to 5.36, 5.98, 6.7 and 6.28 on addition of marble dust and GGBS in proportion of 5%+5%, 10%+10%, 5%+15% & 20%+20%, respectively. While, soaked CBR changed to 3.4, 3.61, 4.12 and 4.02 on addition of marble dust and GGBS in proportion of 5%+5%, 10%+10%, 5%+15% & 20%+20%, respectively. The unconfined compressive strength (UCS) values obtained for 0%+0%, 5%+5%, 10%+10%, 5%+15% & 20%+20% addition of marble dust and GGBS were as follows: 1.96, 2.48, 2.66, 2.89 and 3.1 kg/m². Also, the coefficient of permeability of the soil sample decreased as the percentage marble dust + GGBS increased. The coefficient of permeability changed from 9.86×10^{-6} cm/s to 2.98×10^{-6} cm/s after 20%+20% addition of marble dust and GGBS. This project work concluded that Marble dust and GGBS are potentially useful in stabilization of soil. However, author also mentioned that the stabilizing effect is primary a function of the chemical composition, fineness, and addition level of the Marble dust and GGBS as well as the type of parent soil.

Bansal H *et al.* (2016) studied the influence of waste marble powder on characteristics of clayey soil. Clay which was used for the research work was brought from a dry pond in Lehri village, about 20km from Talwandi Sabo, Bathinda, Punjab (India). The waste marble powder was brought from the Makrana, Rajasthan through a marble dealer of Bhuchomandi, Punjab (India). The marble is known as Makrana marble which is a calcite type of marble. The marble waste was added in the soil as 10%, 20% and 30% by weight and comparison of the properties of parent soil with the stabilized soil was done. Liquid limit of the stabilized soil decreased from 31.70% for parent soil to 28.10%, 26.40% and 25.00% with partial replacement of soil with marble powder as 10%, 20% and 30% respectively. Whereas, plastic limit of the stabilized soil increased from 17.69% for parent soil to 18.10%, 18.78% and 19.26% with partial replacement of soil with marble powder as 10%, 20% and 30% respectively. Optimum moisture content (OMC) of the stabilized soil decreased from 18.00% for parent soil to 17.30%, 16.80%, and 14.10% with partial replacement of soil with marble powder as 10%, 20% and 30% respectively. Maximum dry density (MDD) of the stabilized soil increased from 1.735 gm/cc for parent soil to 1.795 gm/cc, 1.805 gm/cc, and 1.884 gm/cc with partial replacement of soil with marble powder as 10%, 20% and 30% respectively. California bearing ratio (CBR) of the stabilized soil increased from 2.46 for parent soil to 4.78, 5.53 and 6.07 with partial replacement

of soil with marble powder as 10%, 20% and 30% respectively. The reason mentioned behind the increase in CBR value was presence of coarser particles and lime (which behaves as cementitious material). Addition of marble powder in the soil (10% by weight) showed increase in the coefficient of permeability. Further addition of waste marble powder showed decrease in the coefficient of permeability, as compared to the parent soil. The values of coefficient of permeability obtained for 0%, 10%, 20% and 30% addition of waste marble powder were 3×10^{-4} mm/s, 4×10^{-4} mm/s, 2×10^{-4} mm/s and 2×10^{-4} mm/s. Unconfined compressive strength (UCS) value of the stabilized soil increased from .603 kg/cm² for parent soil to 2.525 kg/cm² and 3.053kg/ cm² with partial replacement of soil with marble powder as 10% and 20% respectively with optimum value of replacement as 20%. There was fall in the unconfined compressive strength (UCS) value at 30% addition of marble dust. The value obtained was 2.543 kg/cm². Based on this study, author concluded that inclusion of waste marble in soil helps in improving its index and engineering properties.

Verma A et al. (2017) added marble dust to the soil along with fly ash. Soil used for this study was classified as CI. Soil showed maximum unconfined compressive strength (UCS) at 20% addition of fly ash. So, author added marble dust along with 20% of fly ash, to the soil. Marble dust was added at 5%, 10%, 15% and 20% by weight of soil. Results of triaxial test showed cohesion values of 23 kN/m², 22 kN/m², 18 kN/m² and 16 kN/m² for 5%, 10%, 15% and 20% addition of marble dust with optimum percentage of fly ash. Also, the angle of internal friction (in degrees) obtained after 5%, 10%, 15% and 20% addition of marble dust had values of 32.51, 35.08, 35.69 and 36.05. The values of angle of internal friction and cohesion for the parent soil were 26° and 10 kN/m², respectively. Author described 10% addition of marble dust as optimum along with 20% of fly ash.

2.2 Gap in Study

Previous work doesn't include use of marble dust for the pavement design purposes. Effect of marble dust observed on subgrade soil characteristics wasn't further studied to determine fatigue and rutting characteristics of a flexible pavement. Also, it is important to determine optimum percentage of marble dust in the soil, so that it can be used for subgrade modification.

2.3 Objective of Thesis

Objective of thesis work is mentioned in the points below:

- Determination of change in index properties and strength characteristics of soil, upon addition of waste marble dust.
- Change in the soil characteristics is to be used for determination of subgrade resilient modulus. Various empirical equations are to be used for calculating subgrade resilient modulus.
- Based on the subgrade resilient modulus, flexible pavement design is to be done. Design is to be done for each percentage of marble dust in the soil. Also, the difference in resilient modulus values and pavement thickness determined using different empirical equations is to be noted.
- Based on the experimental and analytical results, optimum percentage of waste marble dust in the soil is determined.
- For future prediction of California Bearing Ratio (CBR) of soil with addition of waste marble dust, an equation is to be developed relating California Bearing Ratio (CBR) with index properties and unconfined compressive strength (UCS) of soil. Difference in predicted California Bearing Ratio (CBR) value and observed California Bearing Ratio (CBR) values is also to be examined.

2.4 Outline of Thesis

The thesis work has been divided into following seven chapters:

- 1st chapter includes general introduction to soil stabilization, marble dust, resilient modulus and regression analysis.
- 2nd chapter includes the literature review of the work done on the soil stabilization using marble dust/powder/slurry.
- 3rd chapter deals with experimental programme which includes materials, tests conducted and procedure adopted for the experimental study.
- 4th chapter has all the results and findings of the experimental programme.

- 5th chapter includes prediction of resilient modulus of subgrade using various empirical relationships and the results obtained. It also includes design and methodology of flexible pavement done using marble dust.
- 6th chapter includes correlation developed for prediction of CBR value when soil is stabilized using marble dust. Further, the comparison of predicted and observed values are also included in this.
- 7th chapter deals with the conclusion of experimental and analytical study.

CHAPTER 3

EXPERIMENTAL PROGRAMME

Materials and experiments conducted using them are discussed in this chapter. Following sections discuss materials and their properties, experiments conducted and their procedure adopted, in details.

3.1 Materials

Soil and waste marble dust were used for the experimental study. Following sections show the source and gradation of soil and waste marble dust used, along with some of their properties.

3.1.1 Soil

Soil used for the experimental study was collected from Shambu Kalan, Punjab (India). Based upon grain size distribution (88.3% of the soil passed through 0.075mm sieve) and liquid limit value obtained, soil was classified as CL (silts and clays of low compressibility) as per Indian Standards. Grain size distribution of soil sample is shown in Figure 3.1. Table 3.1 shows some of the geotechnical properties of the soil obtained.

Table 3.1 Geotechnical properties of collected soil sample

S.No.	Properties	Test Result	Relevant IS Code
1	Particle Size < 75 μ , %	88.3	IS: 2720 (Part IV)-1985
2	Liquid Limit (w_L), %	28.7	IS: 2720 (Part V)-1985
3	Plastic Limit (w_P), %	16.67	IS: 2720 (Part V) -1985
4	Plasticity Index (I_p), %	12.03	IS: 2720 (Part V) -1985
5	Flow Index (I_f)	23.7	IS: 2720 (Part V) -1985
6	Toughness Index (I_T)	0.507	IS: 2720 (Part V) -1985
7	Optimum Moisture Content, %	15.29	IS: 2720 (Part VII)- 1980
8	Maximum Dry Density ($\gamma_{d\ max}$), kN/m ³	18.76	IS: 2720 (Part VII)- 1980
9	Unconfined Compressive Strength, kPa	62.34	IS: 2720 (Part X)-1991
10	California Bearing Ratio (CBR), %	2.96	IS: 2720 (Part XVI)-1987
11	Soil Classification	CL	IS: 1498-1970

3.1.2 Marble Dust

Waste marble dust was collected from a marble cutting store located at Sirhind road, Patiala (India). Grain size distribution of marble dust is also shown in Figure 3.1.

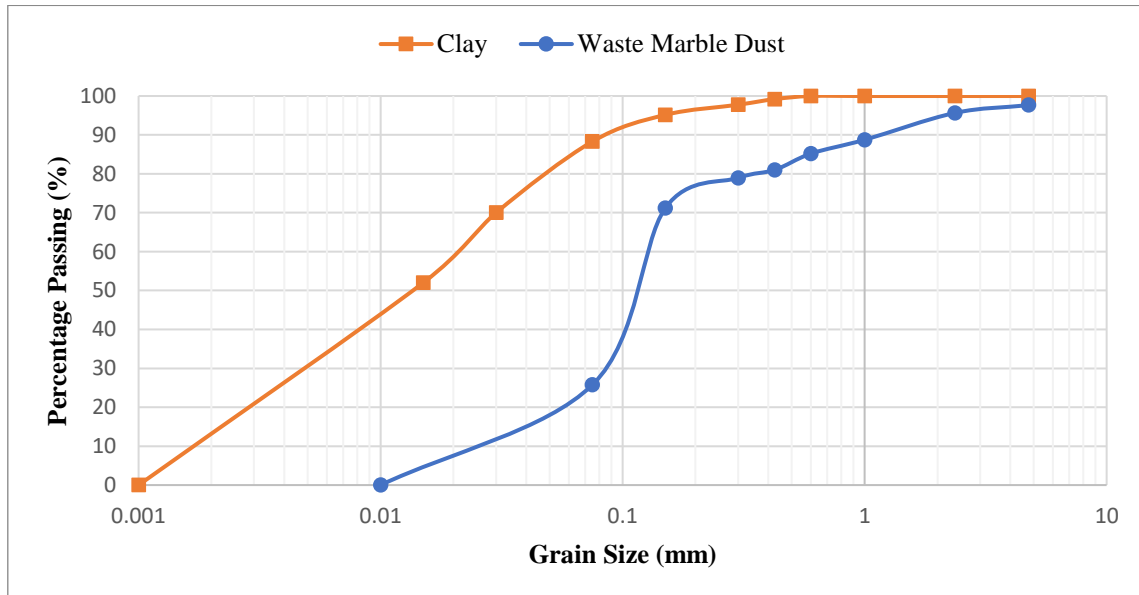


Figure 3.1: Grain size distribution of clay and waste marble dust

As compared to soil (clay), waste marble dust was much coarser. 97.6% of marble dust passed through 4.75mm sieve and only 25.75% of marble dust passed through 0.075mm sieve. Some of the physical properties of waste marble dust are mentioned in Table 3.2.

Table 3.2: Physical properties of waste marble dust

S.No.	Properties	Description
1	Particle Size < 75 μ , %	25.75
2	Specific Gravity	2.62
3	Colour	Grey
4	Odour	Odourless



Figure 3.2: Waste marble dust used for the experimental study

Marble dust was added at 10%, 15%, 20% and 25% by weight of soil. Clay and marble dust mixtures are denoted by CM0, CM10, CM15, CM20 and CM25 when they are present in ratio of 100:0, 90:10, 85:15, 80:20 and 75:25 respectively.

3.2 Atterberg's Limit Test

This test is done for determining liquid limit, plastic limit and plasticity index of soil. The test was conducted as per procedure specified in IS: 2720 (Part V)-1985.

Liquid limit value decreased on addition of marble dust to the soil. On the other hand, there was increase in the plastic limit value upon addition of marble dust to the soil. Also, Plasticity index increased upon addition of marble dust to the soil.

Table 3.3 Effect of marble dust on Atterberg's Limits

Soil ID	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %
CM0	28.7	16.67	12.03
CM1	24.8	17.64	7.16
CM15	24.3	18.34	5.96
CM20	22.8	18.65	4.15
CM25	22.1	19.04	3.06

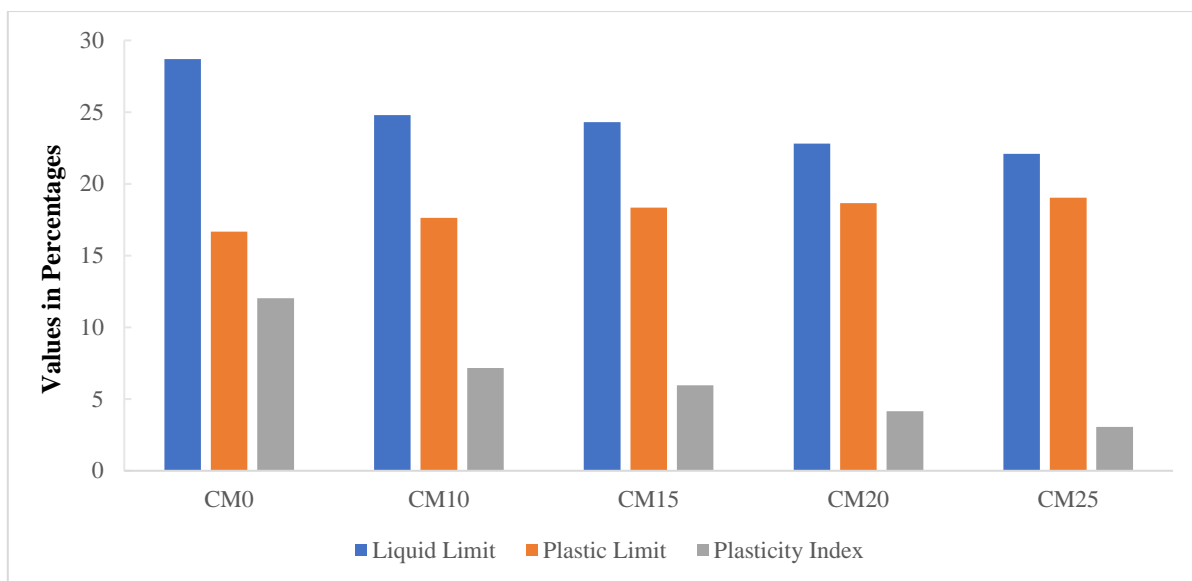


Figure 3.3: Variation of liquid limit, plastic limit, plasticity index with addition of marble dust

3.3 Proctor Compaction Test

Light compaction test, also known as standard Proctor test, was performed for each of the clay and marble dust mixtures for determining optimum moisture content (OMC) and maximum dry density (MDD), as per procedure specified in IS:2720 (Part VII):1980.

Compaction is defined as densification of soil mass by the reduction of air voids. Dry density is the parameter used for determining the degree of compaction. Soil attains the maximum dry density value at a particular water content (known as optimum moisture content) for a given compaction energy.

Compaction was done with the help of an automatic compactor (shown in Figure 3.7) having a light weight hammer weighing 2.6 kg. Soil mixed with water, was filled in 3 layers in a mould having volume of 1000 cc. Height of fall of hammer was 31 cm and each layer of soil was given 25 blows.

For clay and marble dust mixtures, named CM10, CM15 and CM20, there was decrease in the optimum moisture content (OMC) upon addition of marble dust as compared to parent soil, named CM0. Afterwards, for CM25 there was increase in the optimum moisture content (OMC) value in comparison to that of CM20.

Corresponding to optimum moisture content (OMC), there was increase in the maximum dry density (MDD) for clay and marble dust mixtures, named CM1, CM15 and CM20, to that of

parent soil, named CM0. Maximum dry density (MDD) value decreased on further addition of marble dust.

The values of optimum moisture content (OMC) and maximum dry density (MDD) obtained for every clay and marble dust mix is shown in Table 3.4. Figure 3.4 shows compaction curves for each of the clay and marble dust mix.

Table 3.4: Effect of marble dust on compaction characteristics

Soil ID	Optimum Moisture Content, %	Maximum Dry Density, kN/m ³
CM0	15.29	18.76
CM10	13.72	18.9
CM15	13.48	19.16
CM20	13.1	19.46
CM25	13.98	19.05

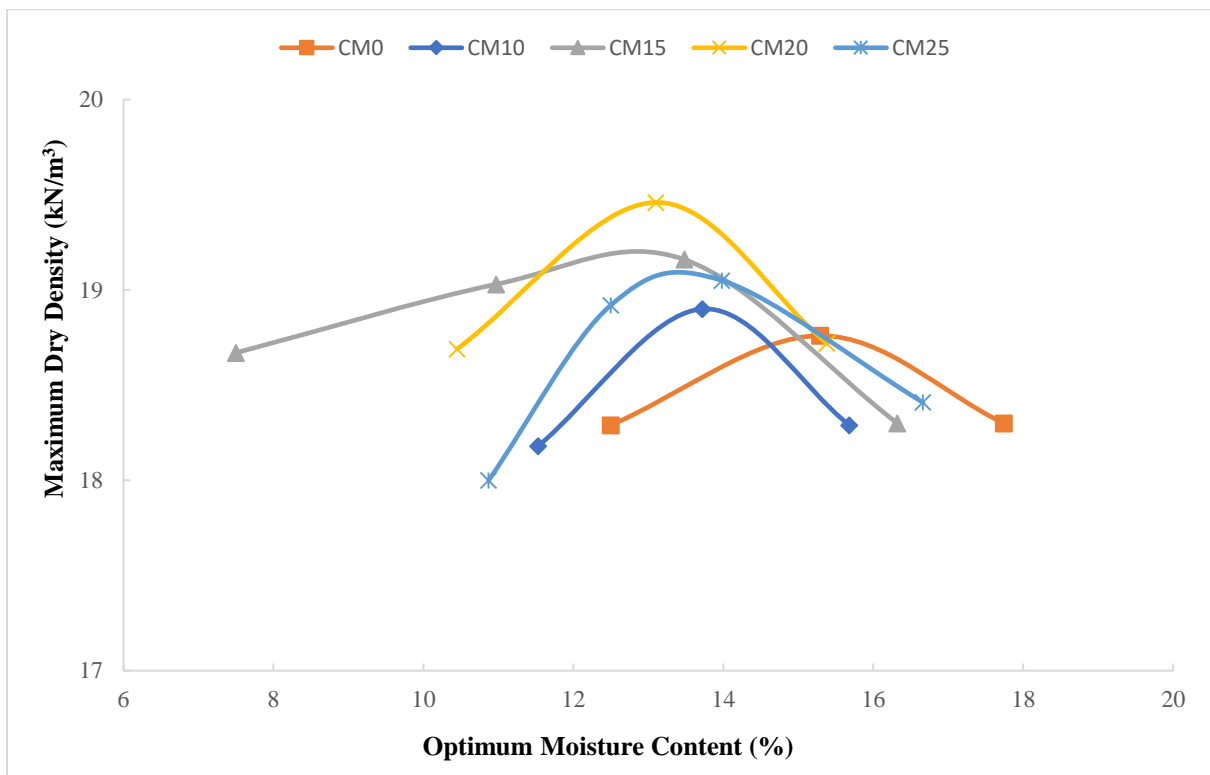


Figure 3.4: Compaction curves at various percentages of clay and marble dust

Figure 3.5 and Figure 3.6, shows variation of the optimum moisture content (OMC) and maximum dry density (MDD) at varying percentage of waste marble dust.

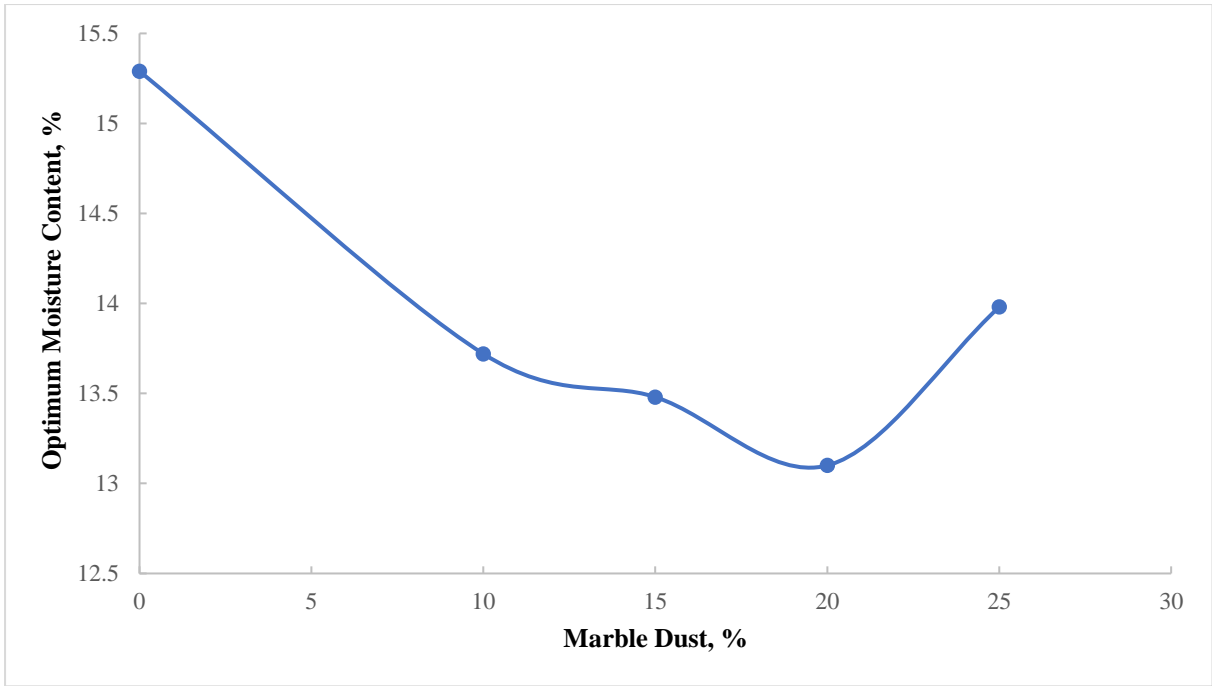


Figure 3.5: Variation of optimum moisture content with different percentages of marble dust

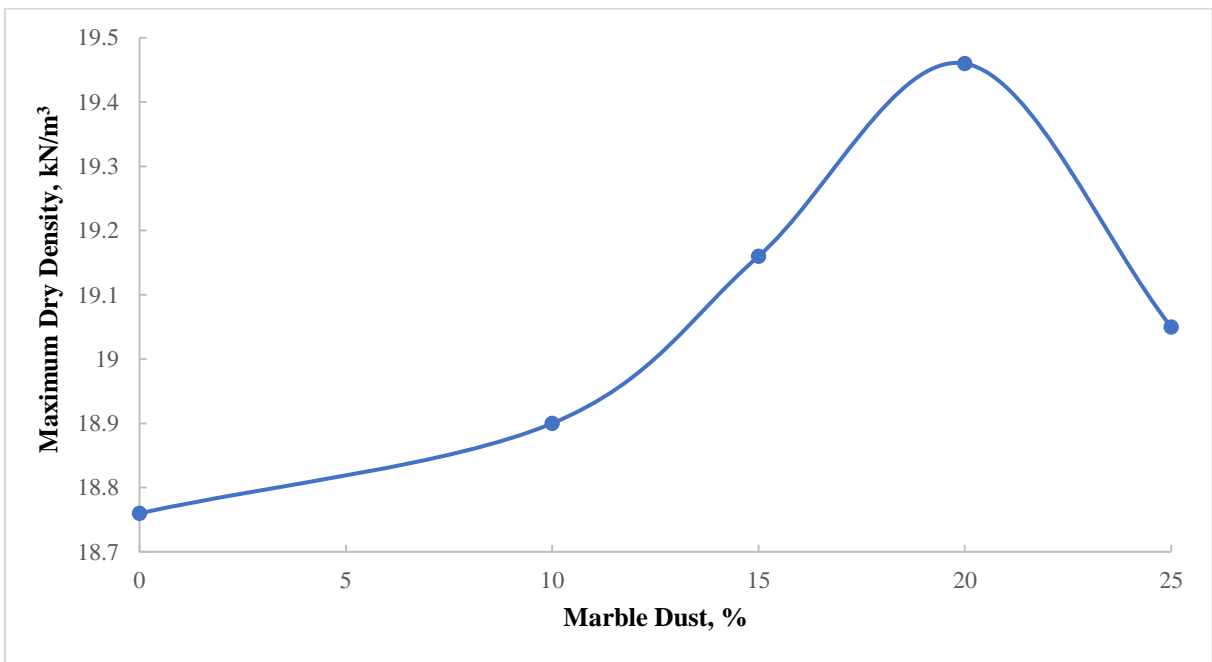


Figure 3.6: Variation of maximum dry density with different percentages of marble dust



Figure 3.7: Automatic compactor used for proctor compaction test

3.4 California Bearing Ratio Test

California Bearing Ratio (CBR) test is a penetration test designed to evaluate the strength of subgrade for road and pavements. The resulted CBR value is used for determining the thickness of pavement section. Sample preparation and testing for determining CBR value was done as per IS:2720 (Part XVI):1987.

The CBR tests for clay and marble dust mixtures were conducted in soaked conditions. Remoulded samples were prepared at optimum moisture content (OMC) and maximum dry density (MDD), determined using compaction tests. Prepared samples were kept in soaked conditions for a period of 96 hours. Further, the prepared mould was placed in CBR testing machine (shown in Figure 3.8) which had circular piston, penetrating the given soil mass at a rate of 1.25mm/min.

California Bearing Ratio (CBR) value is generally determined at 2.5 mm and 5 mm penetration. The California Bearing Ratio (CBR) value for a specimen is determined using equation, shown below:

$$CBR (\%) = \frac{\text{Test load at 2.5 mm or 5mm penetration depth}}{\text{Standard load at 2.5 mm or 5mm penetration}} \times 100 \quad \dots\dots\dots (3.1)$$

Where,

1370 kgf and 2055 kgf are the standard load at 2.5 mm and 5 mm penetration depth, respectively.

The greater among the CBR value obtained at 2.5 mm and 5 mm penetration depth is taken as CBR value of that particular sample. Generally, CBR value is maximum at 2.5 mm penetration depth. In case, the CBR value at 5 mm penetration is greater than that at 2.5 mm penetration, the test is to be repeated. If the identical results are obtained, value at 5 mm penetration is to be taken for design.



Figure 3.8: Testing of the sample after 96 hours in CBR test machine

3.4 Unconfined Compression Test

Unconfined Compression Test, also known as uniaxial compression test, is a simplified form of triaxial test, having no confining pressure.

A cylindrical sample is prepared at optimum moisture content (OMC) and is tested for failure in compression. Failure load per unit area gives the value of strength, known as unconfined compressive strength (UCS) of soil.

Determination of unconfined compressive strength (UCS) is helpful when it is difficult to determine in-situ strength of soil. Also, to choose the best material for the embankment, one has to conduct strength tests on the samples selected.

Various empirical equations are also available which can be used for determining resilient modulus of subgrade soil, based on its unconfined compressive strength (UCS).

For conducting unconfined compression test, soil was compacted at its optimum moisture content. Cylindrical samples of diameter 38 mm and height 76 mm (as shown in Figure 3.9) were extracted from the compacted mould of soil. Prepared samples were placed in the unconfined compression machine (as shown in Figure 3.10) where axial compressive load was applied at a rate of 1.25 mm/min.

Load and deformation values were noted for the sample up to the point when sample fails in compression (as shown in Figure 3.11). Compression or failure load divided by deformed area of the sample is known as unconfined compressive strength of that particular sample.

Three samples were prepared for determining unconfined compressive strength (UCS) for a particular percentage of marble dust. Average of these three is to be considered as unconfined compressive strength (UCS) of that particular mix.

Sample preparation and testing for determining unconfined compressive strength (UCS) of soil was done as per IS: 2720 (Part X)-1991.



Figure 3.9: Extracted samples



Figure 3.10: Testing of samples

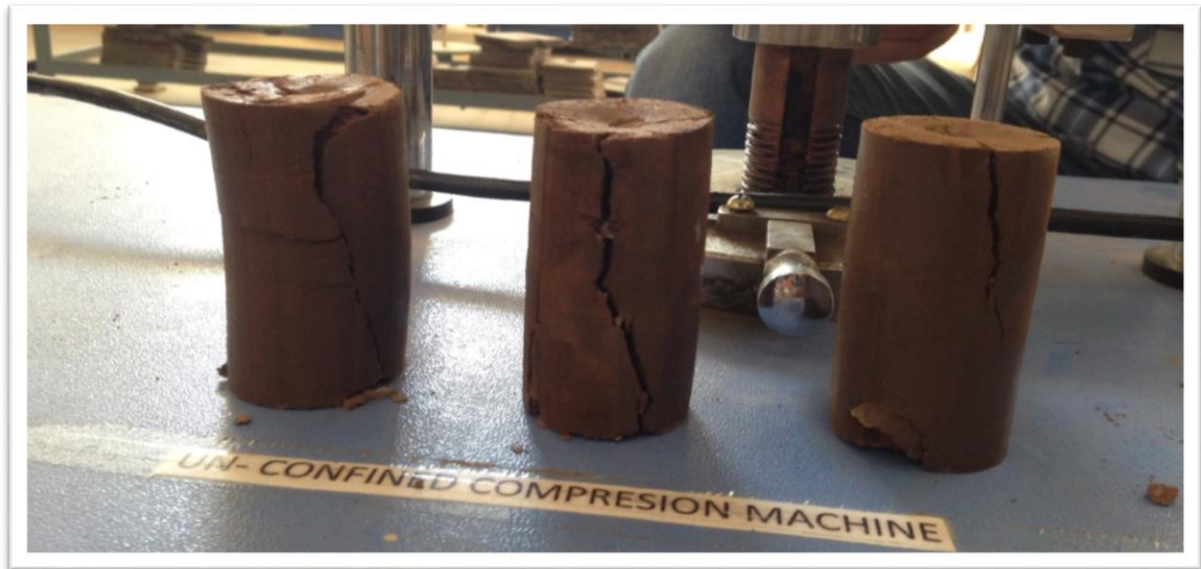


Figure 3.11: Failed samples

Results obtained in the unconfined compressive strength after 10%, 15%, 20% and 25% addition of waste marble dust are explained in details in the upcoming chapter

CHAPTER 4

EXPERIMENTAL RESULTS

Results obtained from the addition of marble dust to the soil are discussed here. California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) are two important parameters used to define strength characteristics of a soil. Effect of marble dust addition on California Bearing Ratio (CBR) and Unconfined Compressive Strength (UCS) of soil is explained in details in section 4.1 and section 4.2, respectively.

4.1 Effect of waste marble dust on California Bearing Ratio (CBR) of soil

As most of the pavement design models are based on the soaked CBR value, the samples were soaked for 96 hours before testing. The results obtained on addition of waste marble dust showed greater penetration resistance as compared to untreated soil. As a result, there was increase in the CBR value of the soil. This increase in the CBR value can be due to better cementation in the soil matrix on addition of waste marble dust, which made it resistant against penetration.

As mentioned in Chapter 3, waste marble dust was added in 5%, 10%, 15%, 20% and 25% by weight of soil. Further addition of waste marble dust was not done due to decrease in the CBR value. The results obtained at varying percentage of soil and waste marble dust, are shown in Table 4.1.

The observations obtained for parent soil i.e. clay with no dosage of marble dust (CM0), are represented in a Load Vs Penetration graph, shown below (Figure 4.1). The CBR value of 2.96% and 2.82% was obtained at 2.5 mm and 5mm penetration depth, respectively. Higher of the two i.e. 2.96%, is to be considered for the design purposes.

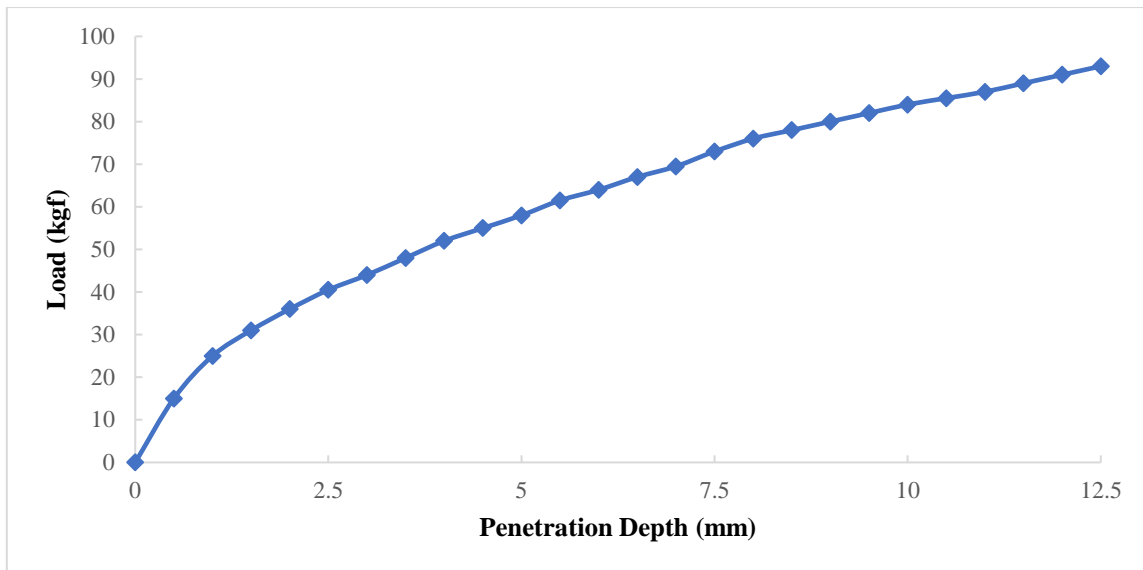


Figure 4.1: Load Vs Penetration graph at 0% marble dust dosage

The observations obtained for soil-marble dust mixture i.e. clay with 10% of marble dust (CM10), are represented in a Load Vs Penetration graph, shown below (Figure 4.2). The CBR value of 3.89% and 3.51% was obtained at 2.5 mm and 5mm penetration depth, respectively. Higher of the two i.e. 3.89%, is to be considered for the design purposes.

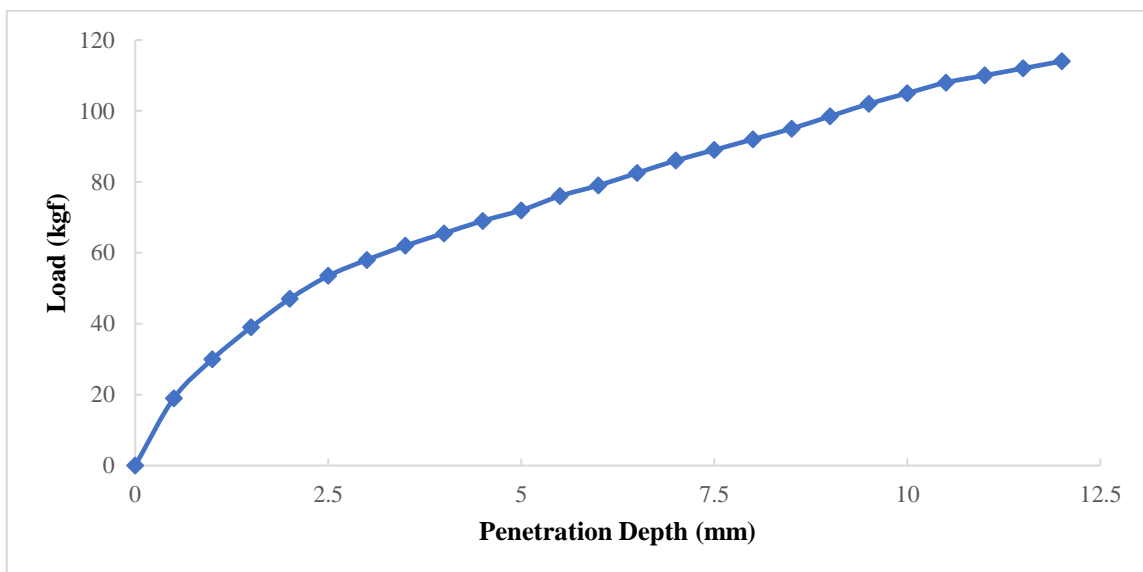


Figure 4.2: Load Vs Penetration graph at 10% marble dust dosage

The observations obtained for soil-marble dust mixture i.e. clay with 15% of marble dust (CM15), are represented in a Load Vs Penetration graph, shown below (Figure 4.3). The CBR value of 4.67% and 4.57% was obtained at 2.5 mm and 5mm penetration depth, respectively. Higher of the two i.e. 4.67%, is to be considered for the design purposes.

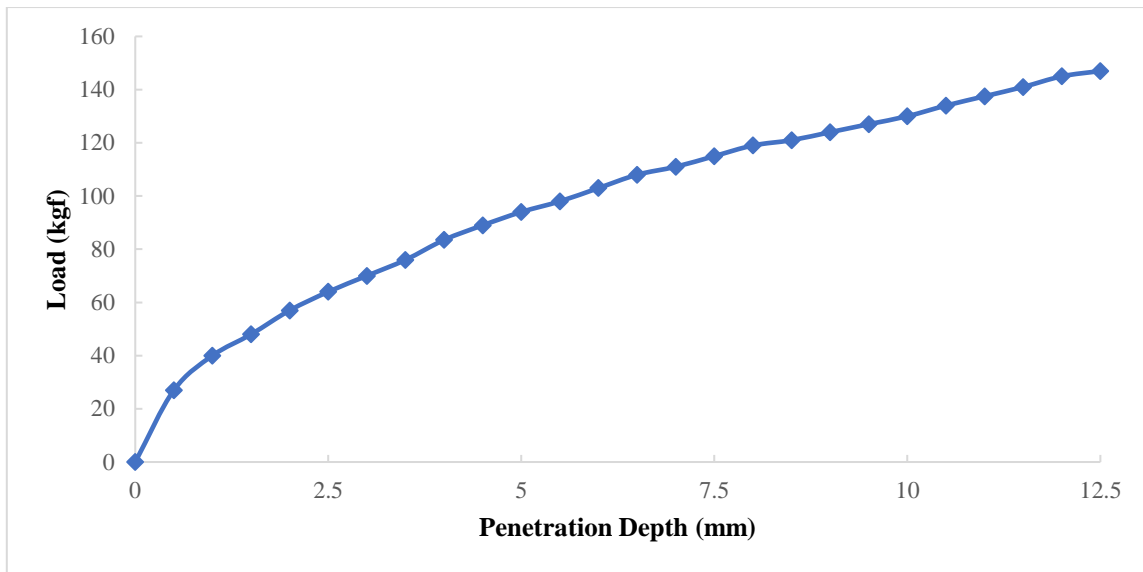


Figure 4.3: Load Vs Penetration graph at 15% marble dust dosage

The observations obtained for soil-marble dust mixture i.e. clay with 20% of marble dust (CM20), are represented in a Load Vs Penetration graph, shown below (Figure 4.4). The CBR value of 6.08% and 5.60% was obtained at 2.5 mm and 5mm penetration depth, respectively. Higher of the two i.e. 6.08%, is to be considered for the design purposes.

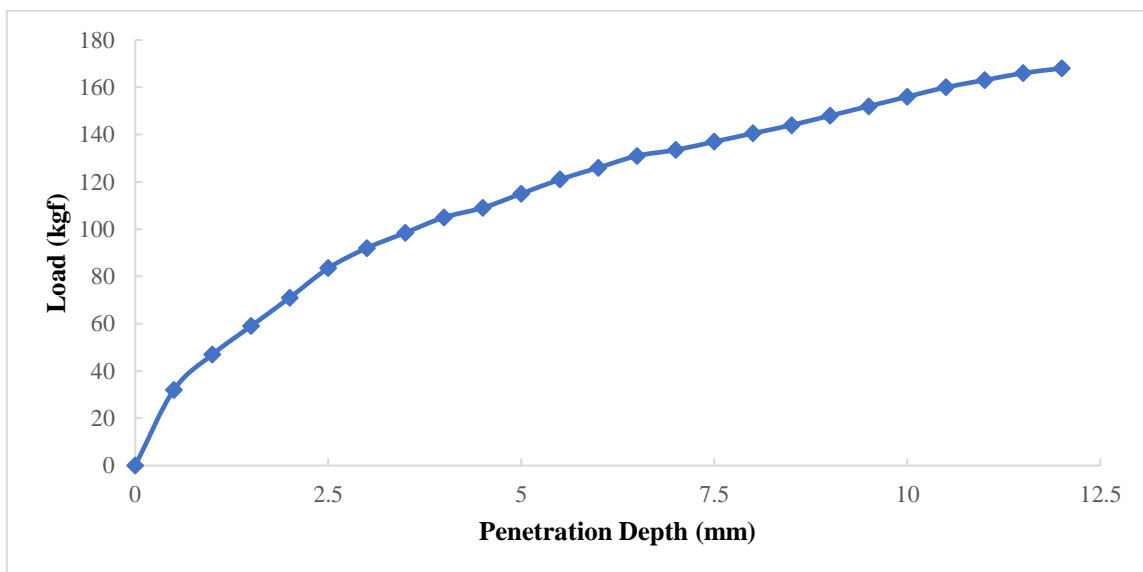


Figure 4.4: Load Vs Penetration graph at 20% marble dust dosage

The observations obtained for soil-marble dust mixture i.e. clay with 25% of marble dust (CM25), are represented in a Load Vs Penetration graph, shown below (Figure 4.5). The CBR value of 6.03% and 5.49% was obtained at 2.5 mm and 5mm penetration depth, respectively. Higher of the two i.e. 6.03%, is to be considered for the design purposes.

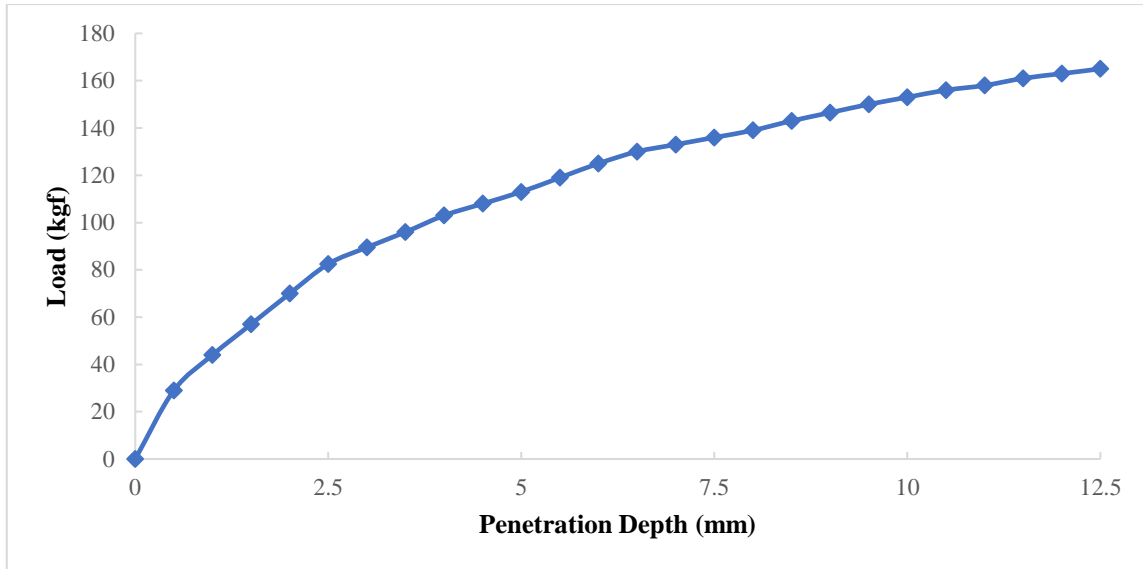


Figure 4.5: Load Vs Penetration graph at 25% marble dust dosage

Table 4.1 shows the CBR values and percentage increase in the CBR values for different clay and marble dust mixes. Graphical representation of the CBR values at different percentages of waste marble dust is shown in Figure 4.6.

Table 4.1: California Bearing Ratio value at various percentages of marble dust

Soil ID	Optimum Moisture Content (OMC), %	California Bearing Ratio (CBR), %	% Increase in California Bearing Ratio (CBR)
CM0	15.29	2.96	-
CM10	13.72	3.89	31.42
CM15	13.48	4.67	57.77
CM20	13.1	6.08	105.40
CM25	13.98	6.03	103.71

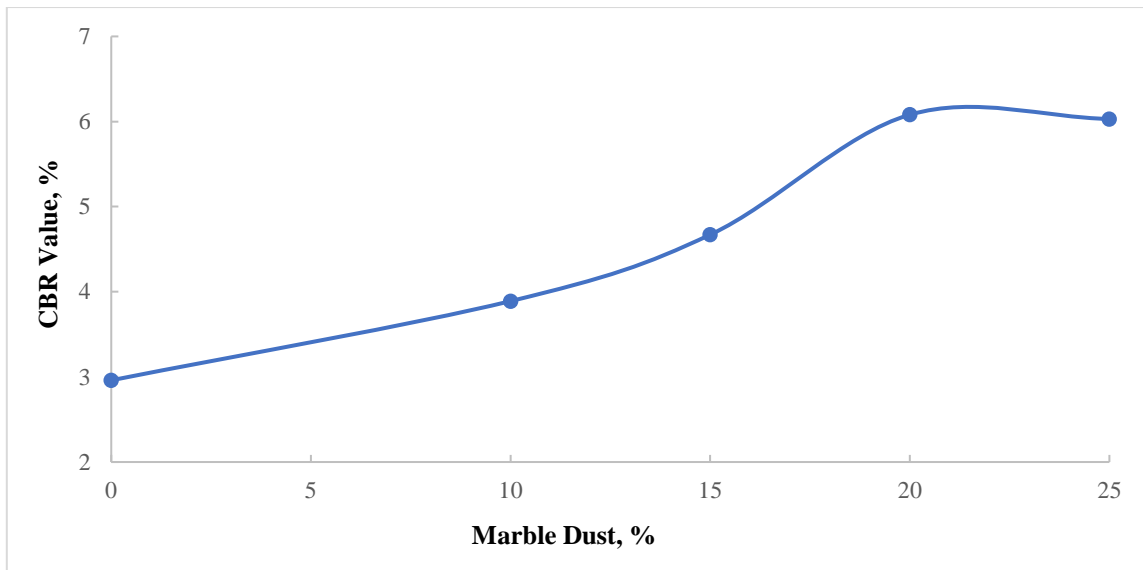


Figure 4.6: California Bearing Ratio at different percentages of marble dust

4.2 Effect of waste marble dust on Unconfined Compressive Strength (UCS) of soil

For conducting unconfined compression test, cylindrical samples of diameter 38 mm and height 76 mm were prepared. 3 samples were prepared for determining unconfined compressive strength (UCS) for a particular percentage of marble dust. Average of these three is to be considered as unconfined compressive strength (UCS) of that particular mix.

Results obtained by unconfined compression test showed similar behaviour to that of CBR, i.e. there was increase in the UCS value up to 20% addition of marble dust and 25% marble dust in the soil showed slight decrease in UCS value. The stress-strain curves obtained at different percentages of soil and marble dust, are shown in sections below.

Stress Vs Strain graph for CM0 i.e. clay with 0% of marble dust is shown in Figure 4.7. Unconfined compressive strength (UCS) values obtained from three test samples are 64.14 kPa, 58.06 kPa and 64.70 kPa. Average of these three i.e. 62.34 kPa is to be considered as unconfined compressive strength (UCS) value.

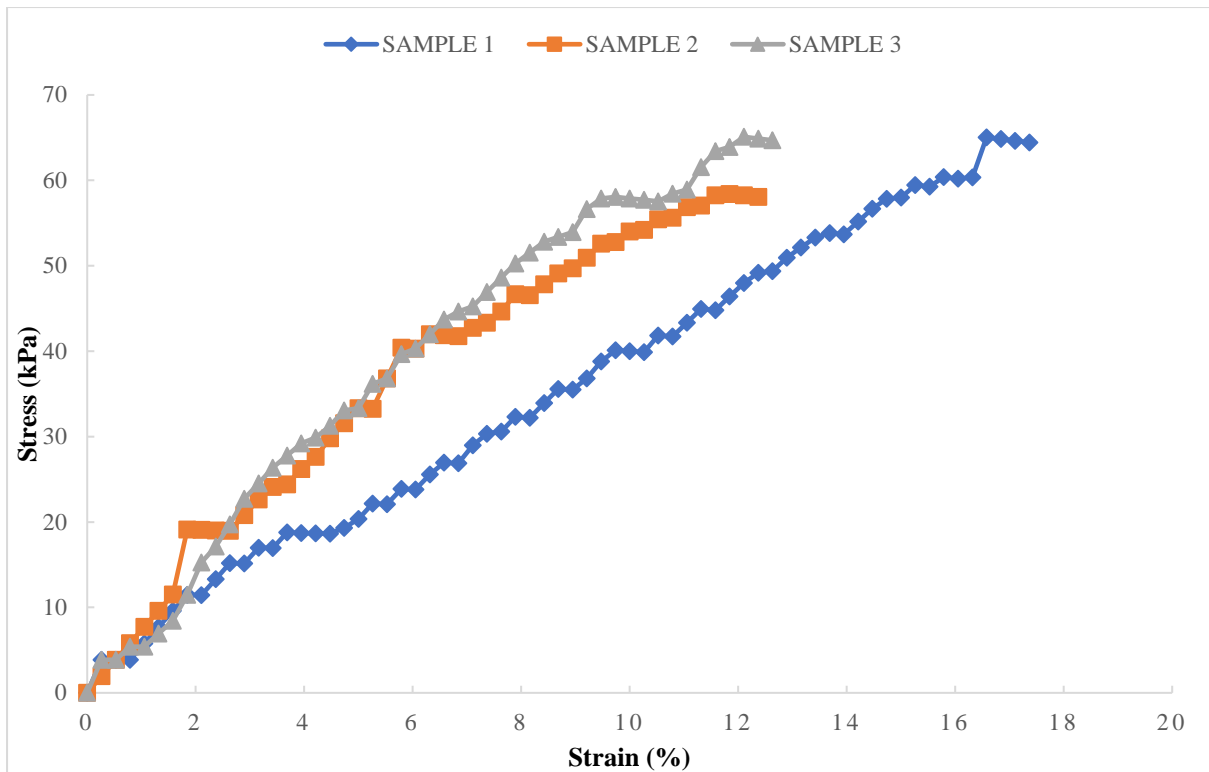


Figure 4.7: Stress Vs Strain graph at 0% marble dust dosage

Stress Vs Strain graph for CM1 i.e. clay with 10% of marble dust is shown in Figure 4.8. Unconfined compressive strength (UCS) values obtained from three test samples are 110.97 kPa, 96.48 kPa and 108.31 kPa. Average of these three i.e. 105.25 kPa is to be considered as unconfined compressive strength (UCS) value.

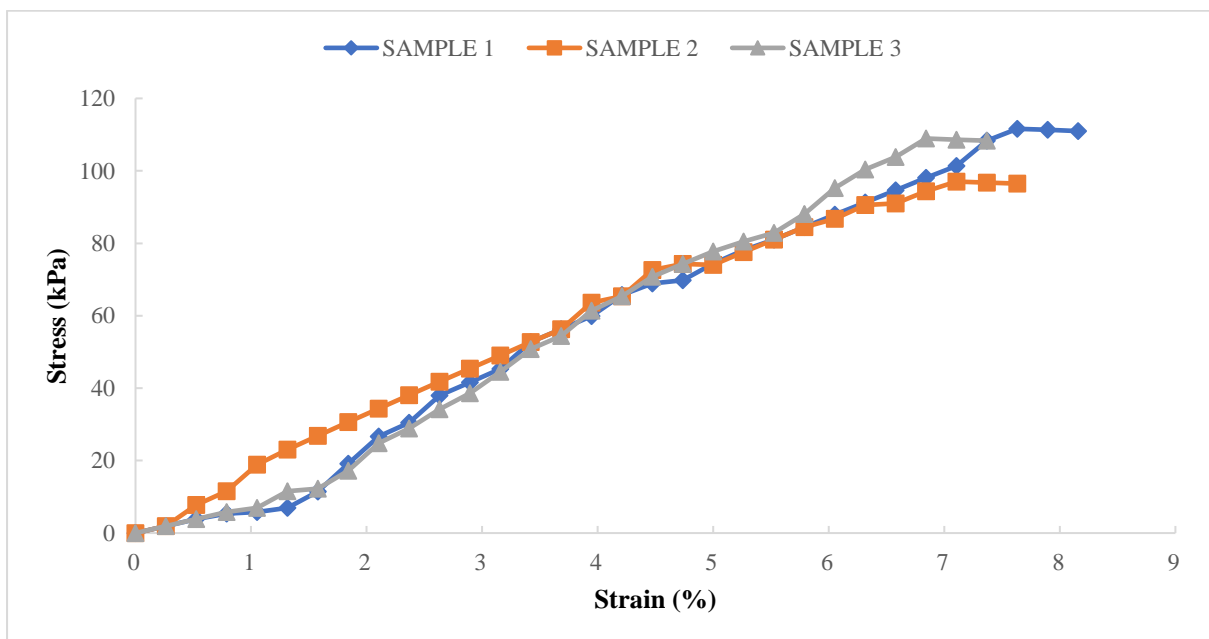


Figure 4.8: Stress Vs Strain graph at 10% marble dust dosage

Stress Vs Strain graph for CM15 i.e. clay with 15% of marble dust is shown in Figure 4.9. Unconfined compressive strength (UCS) values obtained from three test samples are 104.93 kPa, 109.18 kPa and 119.92 kPa. Average of these three i.e. 111.34 kPa is to be considered as unconfined compressive strength (UCS) value.

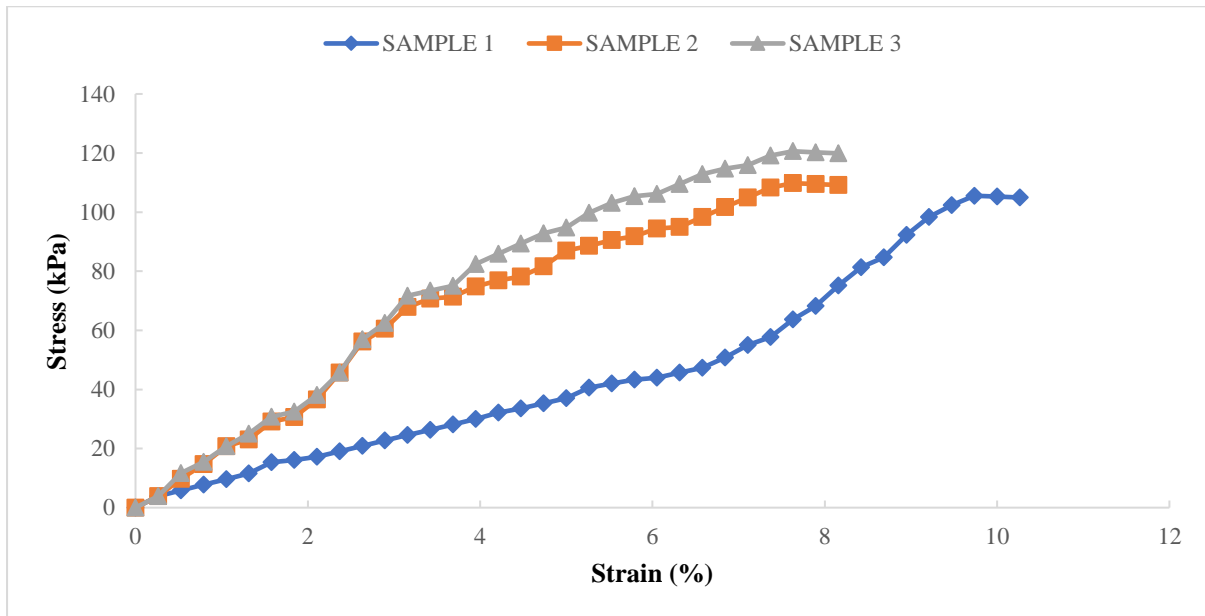


Figure 4.9: Stress Vs Strain graph at 15% marble dust dosage

Stress Vs Strain graph for CM20 i.e. clay with 20% of marble dust is shown in Figure 4.10. Unconfined compressive strength (UCS) values obtained from three test samples are 136.80 kPa, 135.60 kPa and 137.85 kPa. Average of these three i.e. 136.75 kPa is to be considered as unconfined compressive strength (UCS) value.

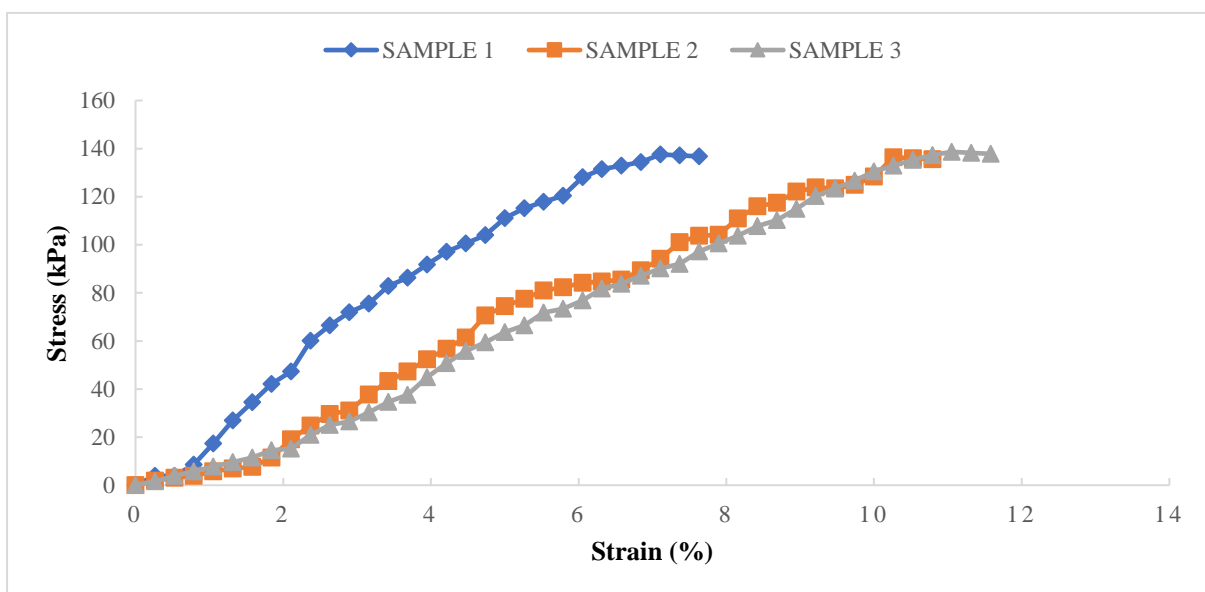


Figure 4.10: Stress Vs Strain graph at 20% marble dust dosage

Stress Vs Strain graph for CM20 i.e. clay with 20% of marble dust is shown in Figure 4.11. Unconfined compressive strength (UCS) values obtained from three test samples are 130.77 kPa, 129.03 kPa and 135.50 kPa. Average of these three i.e. 131.77 kPa is to be considered as unconfined compressive strength (UCS) value.

Table 4.2 shows the unconfined compressive strength (UCS) values for various clay and marble dust mixes. Graphical representation of the UCS values at different percentages of waste marble dust is shown in Figure 4.12.

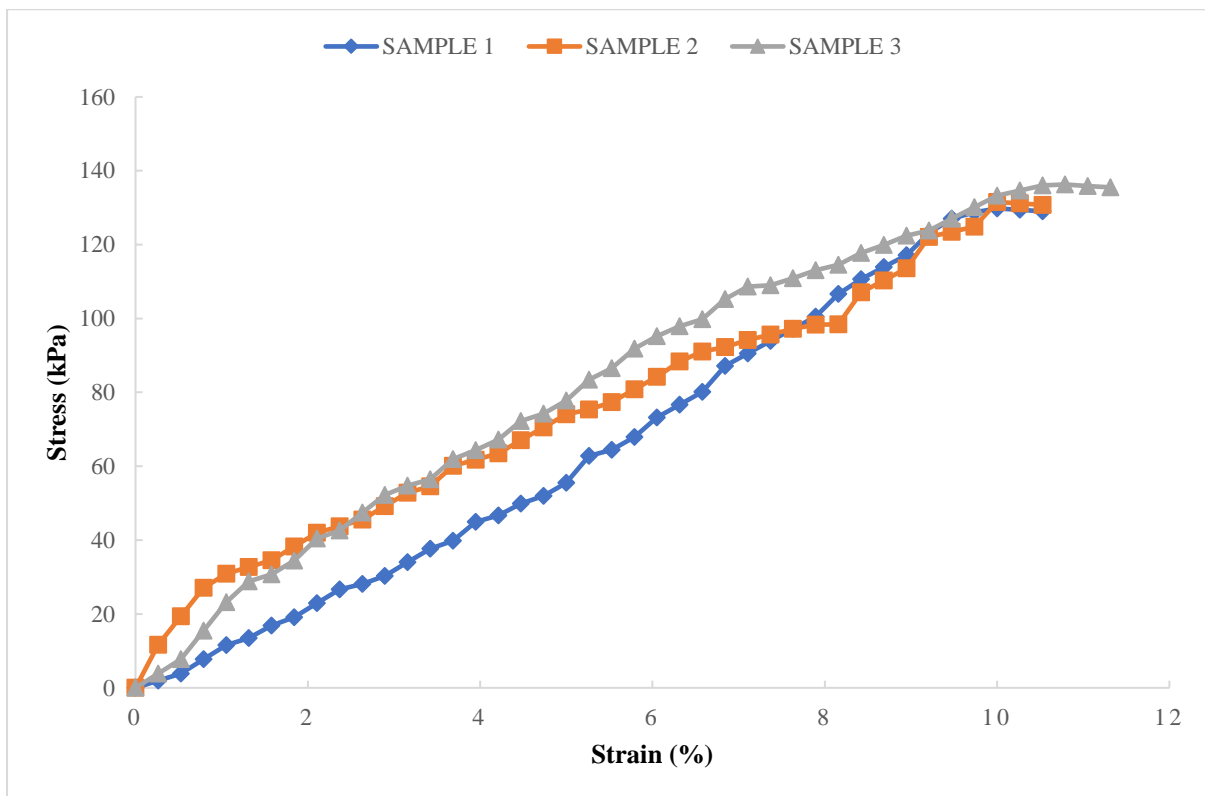


Figure 4.11: Stress Vs Strain graph at 25% marble dust dosage

Table 4.2: Unconfined compressive strength values for different marble dust percentages

Soil ID	Unconfined Compressive Strength (UCS), kPa	% Increase in Unconfined Compressive Strength
CM0	62.34	-
CM10	105.25	68.83
CM15	111.34	78.6
CM20	136.75	119.36
CM25	131.77	111.37

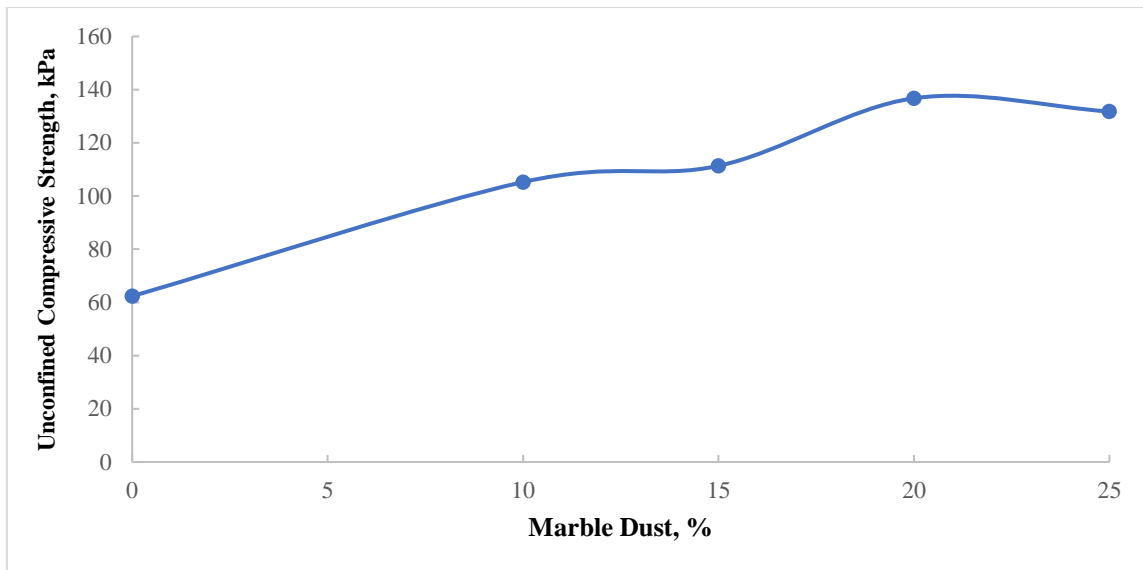


Figure 4.12: Unconfined compressive strength at different percentages of marble dust

Figure 4.12 shows increase in unconfined compressive strength of soil upon addition of waste marble dust. As seen from the graph, there was slight decrease in unconfined compressive strength when marble dust was added 25% by weight of soil.

DESIGN OF FLEXIBLE PAVEMENT USING MARBLE DUST

Results obtained from the experimental analysis are to be used for designing a flexible pavement section. Variation in the thickness of pavement with addition of marble dust, will help in understanding its application for subgrade modification. Following section explains design considerations, methodology and results obtained for different marble dust and soil mixes.

5.1 Design considerations

Following parameters/assumptions are to be considered for designing the flexible pavement section:

- Pavement design is to be done for a traffic of 50 msa.
- Number of layers of the pavement section are taken as 3 i.e. subgrade, granular layer and bituminous layer.
- The value of poisson's ratio for all the layers is to be taken as 0.35.
- The value of tyre pressure to be used for design is 0.56 MPa.
- Single wheel load is taken as 20,000 N.
- Dual wheel configuration is to be used for analysis of fatigue and rutting values.

5.2 Determination of Resilient Modulus (M_R)

Determination of resilient modulus for subgrade was done as per empirical equations suggested by Heukelom and Klomp (1962), Thompson and Robnett (1979), Transportation and Road Research Laboratory, Erdem Çöleri (2007).

Heukelom and Klomp (1962) determined an empirical equation showing relation between soaked CBR and resilient modulus (M_R), after dynamic testing on various soils and road constructions. The equation was said to be valid for fine grained soils having CBR value less than 10. The equation is as follows:

$$M_R(MPa) = 10 \times CBR \quad \dots\dots\dots (5.1)$$

Thompson and Robnett (1979) established a correlation between unconfined compressive strength (UCS) and resilient modulus (M_R), based on evaluation of 50 typical Illinois fine-grained soils. The relationship is as follows:

$$M_R(ksi) = 0.86 + 0.307(q_u) \quad \dots\dots\dots (5.2)$$

Where, q_u is unconfined compressive strength of soil in psi.

Transportation and Road Research Laboratory (TRRL) also determined an empirical relation between CBR and resilient modulus (M_R). The equation is as follows:

$$M_R(MPa) = 17.6 \times CBR^{0.64} \quad \dots\dots\dots (5.3)$$

Erdem Çöleri (2007) conducted 155 resilient modulus tests, 132 California bearing ratio (CBR) tests and 232 light falling weight deflectometer (LFWD) tests on 32 different types of soil to study the effect of different parameters on the resilient modulus (M_R) value. The following equation having R^2 of 0.7089 was established:

$$M_R(MPa) = 228376.7946 - 1479.8978 \cdot LL - 12381.4217W_{opt} + 689.5002 \cdot CBR + 152.9164 \cdot LL \cdot W_{opt} \quad \dots\dots\dots (5.4)$$

Where, LL is liquid limit, W_{opt} is optimum moisture content.

Resilient modulus values calculated using above empirical equations are mentioned in Table 5.1.

Table 5.1: Subgrade resilient modulus (M_R) values for clay and marble dust mixtures

Soil ID	Resilient Modulus (MPa)			
	Heukelom and Klomp (1962)	Thompson and Robnett (1979)	TRRL	Erdem Çöleri (2007)
CM0	29.6	25.06	35.24	65.74
CM10	38.9	38.23	41.98	76.52
CM15	46.7	40.099	47.19	78.82
CM20	60.8	47.87	55.87	82.30
CM25	60.3	46.36	55.58	73.98

Depending on the resilient modulus (M_R) of subgrade, the resilient modulus values of granular layers were determined as per the equation (shown below) 7.1 of IRC37:2012.

$$M_{R_granular} = 0.2h^{0.45} M_{R_subgrade} \dots\dots\dots (5.5)$$

Where, h is total thickness of granular layer.

5.3 Determination of Allowable Strains

Pavement section has two types of strains produced under the application of wheel load on the top surface. They are, horizontal tensile strain (ϵ_t) and vertical compressive strain (ϵ_z). Tensile strain (ϵ_t) is located at the bottom of bituminous layer/layers, whereas, vertical strain (ϵ_z) is located at the top of subgrade. If horizontal tensile strain (ϵ_t) exceeds the limiting strain value, there will be cracking on the top surface of bituminous layer and the pavement may distress due to fatigue. If vertical compressive strain (ϵ_z) exceeds the limiting strain value, there will be permanent deformation (rutting) on the pavement surface.

Two equations specified in IRC 37:2012 for computing horizontal tensile strain (ϵ_t) are shown below:

$$N_f = 2.21 \times 10^{-4} \times \left[\frac{1}{\epsilon_t} \right]^{3.89} \times [1/M_R]^{0.854} \dots\dots\dots (5.6)$$

$$N_f = 0.711 \times 10^{-4} \times \left[\frac{1}{\epsilon_t} \right]^{3.89} \times [1/M_R]^{0.854} \dots\dots\dots (5.7)$$

Where,

N_f = fatigue life in number of standard axles

ϵ_t = maximum tensile strain at the bottom of bituminous layers

M_R = resilient modulus of the bituminous layers

Equation 5.1 is to be used for traffic lesser than 30 msa having normal bituminous mixes with VG 30 grade of bitumen. Whereas, equation 5.2 is recommended for traffic greater than 30 msa and having bituminous mixes with VG 40 grade of bitumen.

Also, equations specified in IRC 37:2012 for computing vertical compressive strain (ϵ_z) are shown below:

$$N = 4.1656 \times 10^{-8} \times \left[\frac{1}{\epsilon_v} \right]^{4.5337} \dots\dots\dots (5.8)$$

$$N = 1.41 \times 10^{-8} \times \left[\frac{1}{\epsilon_v} \right]^{4.5337} \dots\dots\dots (5.9)$$

Where,

N = number of cumulative standard axles

ϵ_v = maximum vertical strain at the top of subgrade

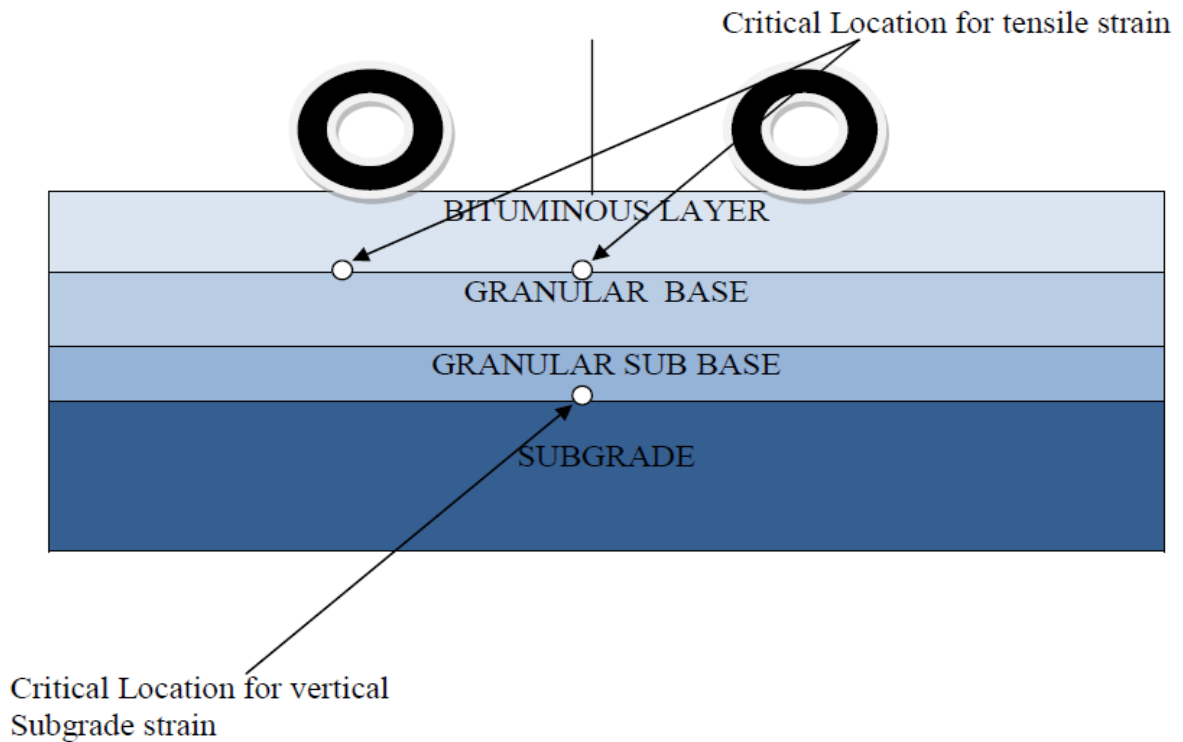


Figure 5.1: Critical locations of flexible pavement

Equations 5.2 and 5.4 are used for determining the fatigue and rutting values, for cumulative standard axles of 50×10^6 . Temperature of 35°C and VG 40 grade of bitumen is taken for determining resilient modulus of bitumen mixes. The value of resilient modulus is taken as 3000 MPa from Table 7.1 of IRC 37:2012.

The values of maximum horizontal tensile strain (ϵ_t) and maximum vertical strain (ϵ_v) calculated using equations 5.7 and 5.9 are 155.27 $\mu\epsilon$ and 371.7 $\mu\epsilon$, respectively.

5.4 Design of Flexible Pavement using IITPAVE

A flexible pavement is considered as multi-layer layer elastic structure. IITPAVE is used for determining stress and strain values at critical locations of pavement section. Design of flexible pavement is to be done for all of the clay and marble dust mixtures. Resilient modulus (M_R)

calculated using various empirical relationships, as shown in section 5.2, is used for determining pavement thickness. Detailed design done using various methods is shown in sections below:

5.4.1. Flexible pavement design for resilient modulus calculated using Erdem Çöleri method

Values in IITPAVE are inputted as per design considerations (mentioned in section 5.1). Resilient modulus values for subgrade are taken as calculated by using equation 5.4. Results obtained are shown in Table 5.2.

Table 5.2: Pavement thickness and strain values for various clay and marble dust mixes

Item	Depth and Radius (mm)	CM0	CM10	CM15	CM20	CM25
M_R Subgrade (MPa)		65.74	76.52	78.82	82.30	73.98
M_R Granular Layer (MPa)		249.08	257.47	260.66	264.84	257.21
M_R Bituminous Layer (MPa)		3000	3000	3000	3000	3000
Horizontal Tensile Strain (ϵ_t), $\mu\epsilon$	Depth- 160,160,160,160,160 mm Radius- 0 mm	151.6	151.4	151	150.4	150.8
	Depth- 160,160,160,160,160 mm Radius- 155 mm	153.4	153.2	152.7	152.1	152.6
Vertical Compressive Strain (ϵ_z), $\mu\epsilon$	Depth- 850,710, 670, 640, 730 mm Radius- 0 mm	163.1	201.5	205.7	212.7	189.7
	Depth- 850,710, 670, 640, 730 mm Radius- 155 mm	170.7	213.8	218.8	227.2	200.5
Thickness (mm)	Bituminous Layer	160	160	160	160	160
	Granular Layer	690	550	510	480	570

It can be seen from the output that there is reduction in total pavement thickness or granular layer thickness with addition of marble dust. The value of thickness increased when marble dust was added 25% by weight.

Total crust thickness obtained for each of the soil and marble dust mix, is shown in Figure 5.2. Also, variation in the thickness of granular layers is shown in Figure 5.3.

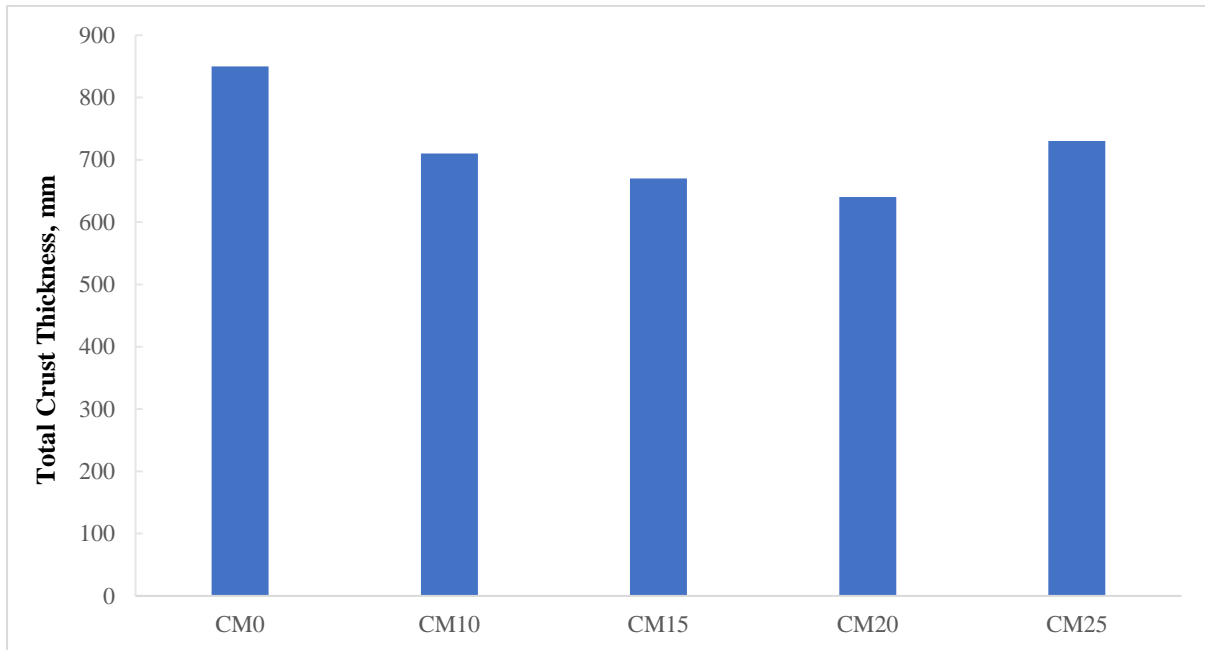


Figure 5.2: Variation in the total crust thickness for various percentages of marble dust

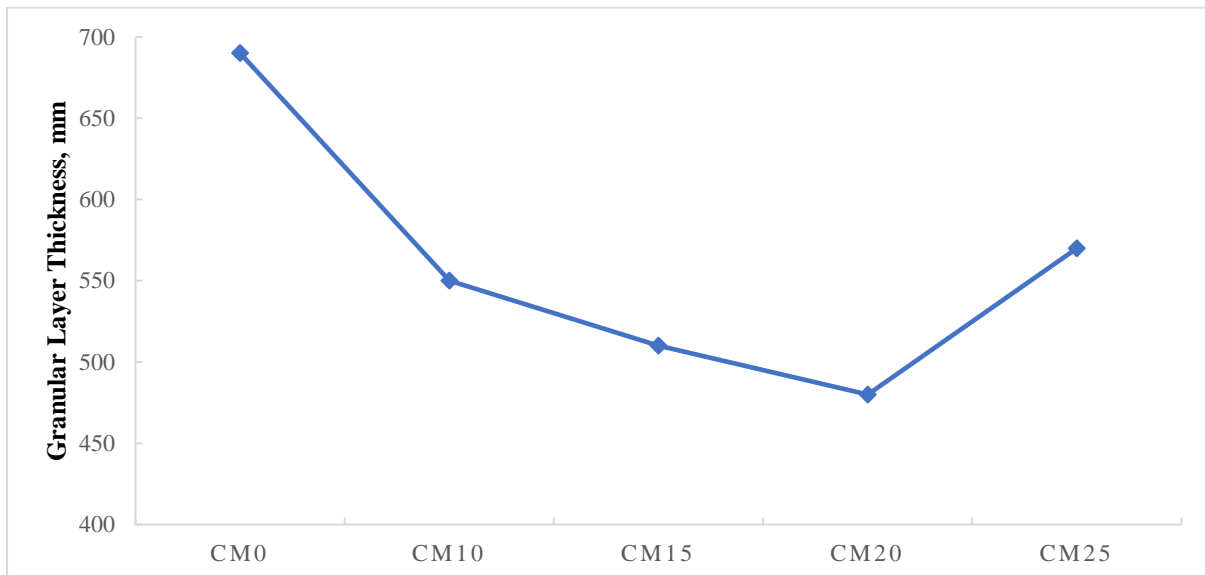


Figure 5.3: Variation in the granular layer thickness for various percentages of marble dust

5.4.2. Flexible pavement design for subgrade resilient modulus calculated using Heukelom and Klomp method

Values in IITPAVE are inputted as per design considerations (mentioned in section 5.1). Resilient modulus values for subgrade are taken as calculated by using equation 5.1. Results obtained are shown in Table 5.3.

Table 5.3: Pavement thickness and strain values for various clay and marble dust mixes

Item	Depth and Radius (mm)	CM0	CM10	CM15	CM20	CM25
M_R Subgrade (MPa)		29.6	38.9	46.7	60.8	60.3
M_R Granular Layer (MPa)		125.12	161.02	169.84	184.24	183.70
M_R Bituminous Layer (MPa)		3000	3000	3000	3000	3000
Horizontal Tensile Strain (ϵ_t), $\mu\epsilon$	Depth- 205,190,190,190,190 mm Radius- 0 mm	147.4	147.9	147.5	147.1	147.1
	Depth- 205,190,190,190,190 mm Radius- 155 mm	153.9	153.7	153.1	152.4	152.5
Vertical Compressive Strain (ϵ_z), $\mu\epsilon$	Depth- 1085,1030, 820, 610, 615 mm Radius- 0 mm	193.7	178.7	215.4	266.3	289.3
	Depth- 1085,1030, 820, 610, 615 mm Radius- 155 mm	198.2	178.7	228.3	281.6	294.8
Thickness (mm)	Bituminous Layer	205	190	190	190	190
	Granular Layer	880	840	630	420	425

It can be seen from the output that there is reduction in total pavement thickness or granular layer thickness with addition of marble dust. The value of thickness increased when marble dust was added 25% by weight. Also, for parent soil i.e. CM0, the pavement section is found

to be safe against fatigue for bituminous layer thickness of 205 mm. Rest of the cases had bituminous layer thickness of 190 mm.

Total crust thickness obtained for each of the soil and marble dust mix, is shown in Figure 5.4. Also, variation in the thickness of granular layers is shown in Figure 5.5.

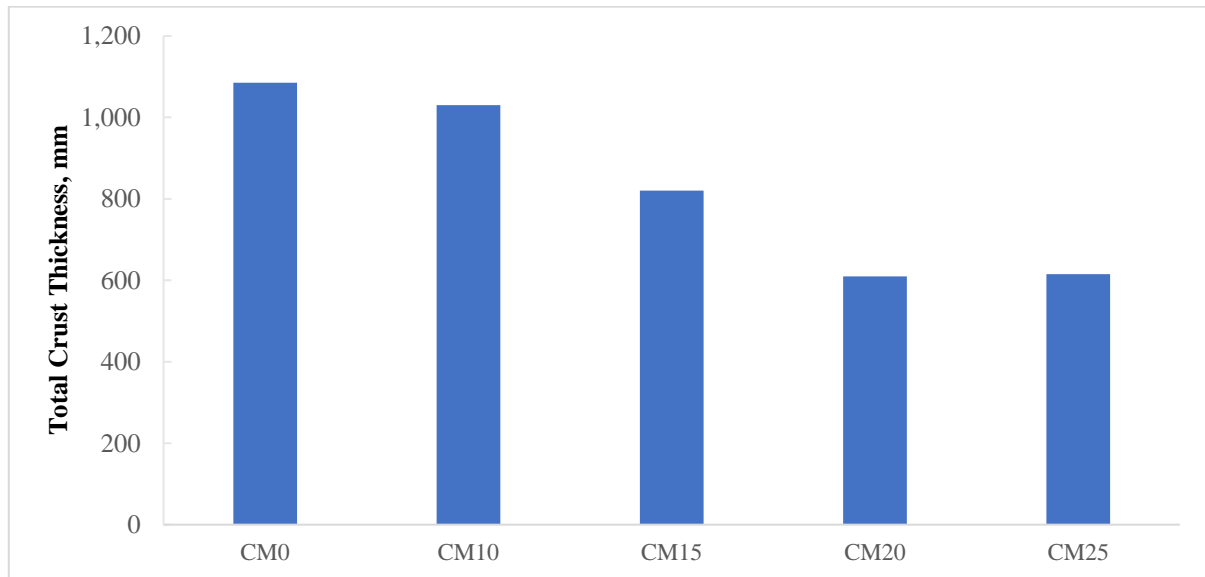


Figure 5.4: Variation in the total crust thickness for various percentages of marble dust

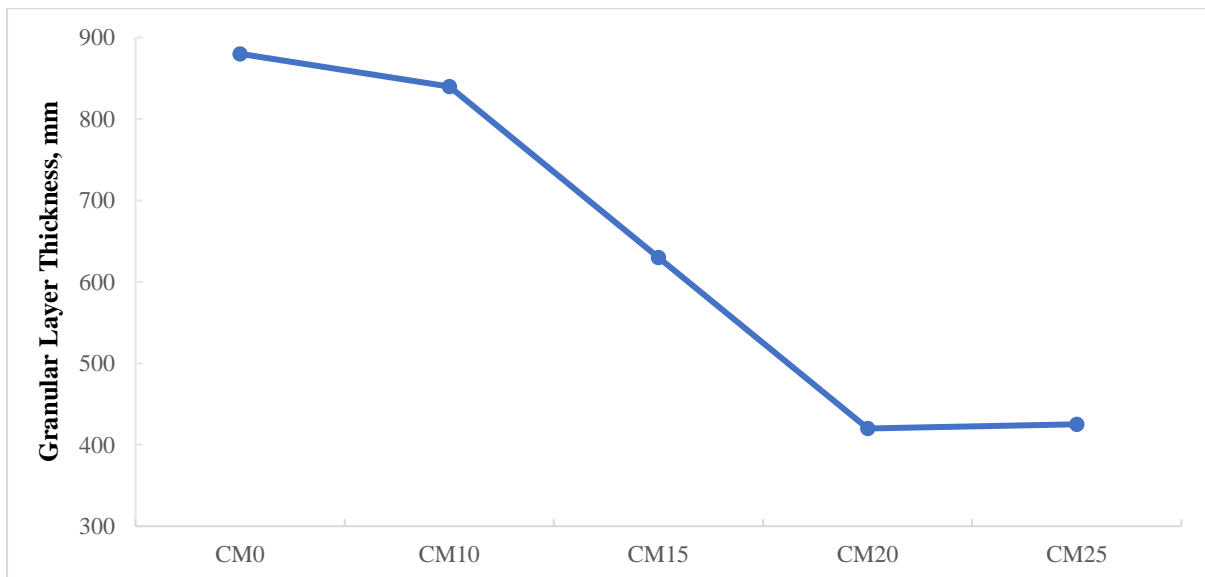


Figure 5.5: Variation in the granular layer thickness for various percentages of marble dust

5.4.3. Flexible pavement design for subgrade resilient modulus calculated using TRRL method

Values in IITPAVE are inputted as per design considerations (mentioned in section 5.1). Resilient modulus values for subgrade are taken as calculated by using equation 5.3. Results obtained are shown in Table 5.4.

Table 5.4: Pavement thickness and strain values for various clay and marble dust mixes

Item	Depth and Radius (mm)	CM0	CM10	CM15	CM20	CM25
M_R Subgrade (MPa)		35.24	41.98	47.19	55.87	55.58
M_R Granular Layer (MPa)		148.20	164.14	170.39	179.79	178.85
M_R Bituminous Layer (MPa)		3000	3000	3000	3000	3000
Horizontal Tensile Strain (ϵ_t), $\mu\epsilon$	Depth- 195,190,190,190,190 mm Radius- 0 mm	147.9	148.3	147.6	147.1	147.5
	Depth- 195,190,190,190,190 mm Radius- 155 mm	153.7	153.6	153	152.5	152.8
Vertical Compressive Strain (ϵ_z), $\mu\epsilon$	Depth- 1060,930, 810, 670, 670 mm Radius- 0 mm	176.5	193	217.6	250	250.9
	Depth- 1060,930, 810, 670, 670 mm Radius- 155 mm	181.3	199.1	226.8	263.7	264.7
Thickness (mm)	Bituminous Layer	195	190	190	190	190
	Granular Layer	870	740	620	480	480

It can be seen from the output that there is reduction in total pavement thickness or granular layer thickness with addition of marble dust. The value of thickness increased when marble

dust was added 25% by weight. Also, for parent soil i.e. CM0, the pavement section is found to be safe against fatigue for bituminous layer thickness of 195 mm. Rest of the cases had bituminous layer thickness of 190 mm.

Total crust thickness obtained for each of the soil and marble dust mix, is shown in Figure 5.6. Also, variation in the thickness of granular layers is shown in Figure 5.7.

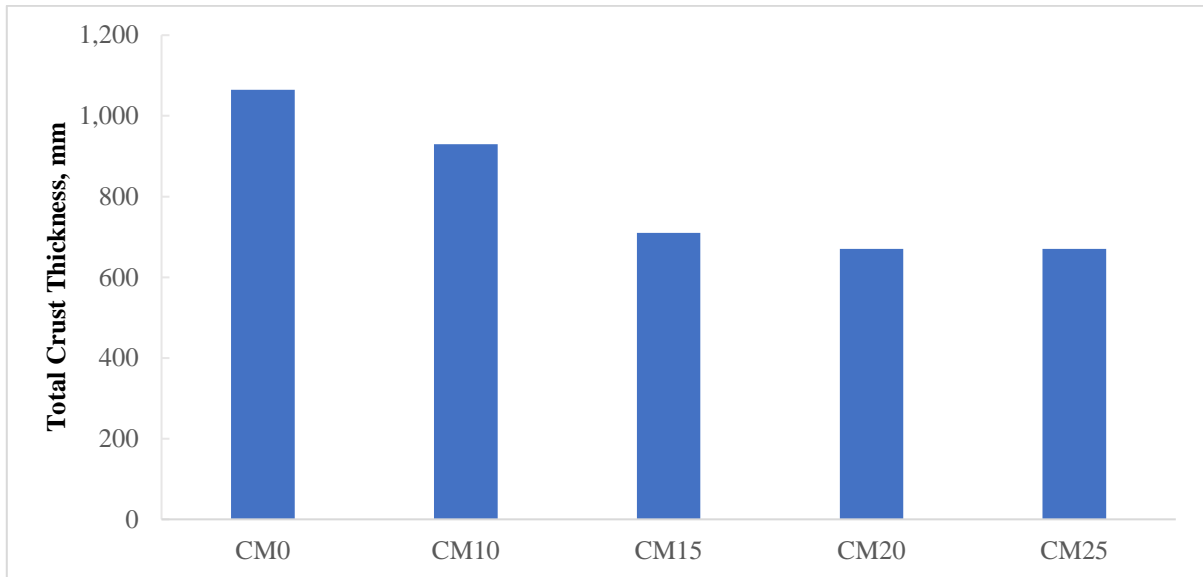


Figure 5.6: Variation in the total crust thickness for various percentages of marble dust

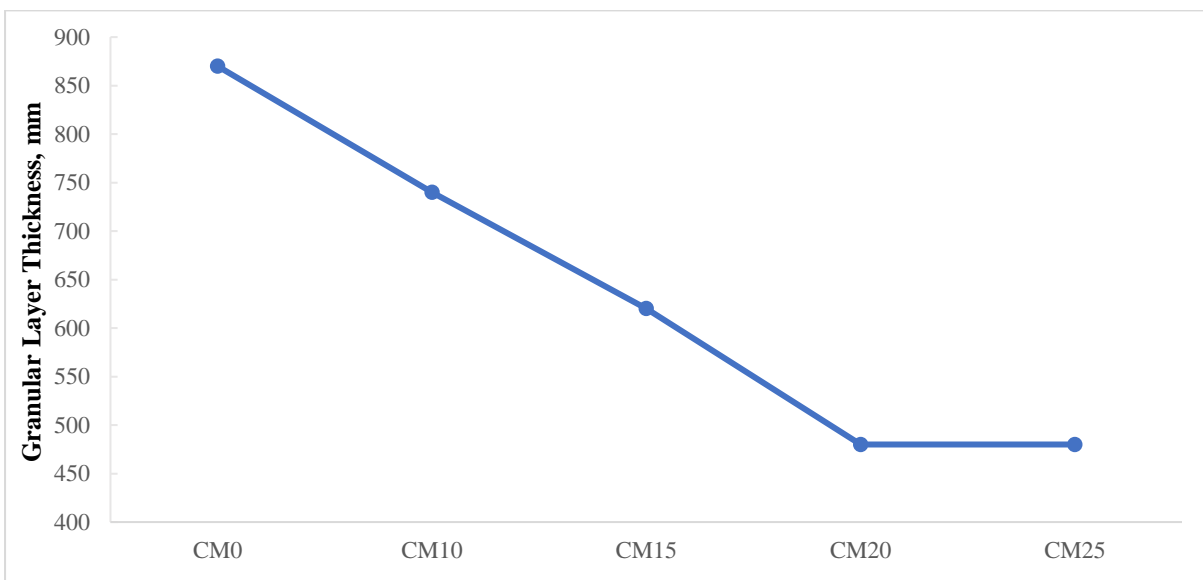


Figure 5.7: Variation in the granular layer thickness for various percentages of marble dust

5.4.4. Flexible pavement design for resilient modulus calculated using Thompson and Robnett method

Values in IITPAVE are inputted as per design considerations (mentioned in section 5.1). Resilient modulus values for subgrade are taken as calculated by using equation 5.2. Results obtained are shown in Table 5.5.

Table 5.5: Pavement thickness and strain values for various clay and marble dust mixes

Item	Depth and Radius (mm)	CM0	CM10	CM15	CM20	CM25
M_R Subgrade (MPa)		25.06	38.23	40.09	47.87	46.36
M_R Granular Layers (MPa)		105.38	160.35	162.35	170.72	168.60
M_R Bituminous Layer (MPa)		3000	3000	3000	3000	3000
Horizontal Tensile Strain (ϵ_t), $\mu\epsilon$	Depth- 215,190,190,190,190 mm Radius- 0 mm	146.9	148.2	148	148	148.1
	Depth- 215,190,190,190,190 mm Radius- 155 mm	153.8	153.7	153.6	153.4	153.5
Vertical Compressive Strain (ϵ_z), $\mu\epsilon$	Depth- 1060,930, 810, 670, 670 mm Radius- 0 mm	215.9	169.3	181	223.3	228.6
	Depth- 1060,930, 810, 670, 670 mm Radius- 155 mm	220.8	174	186.4	232.9	292.4
Thickness (mm)	Bituminous Layer	215	190	190	190	190
	Granular Layer	870	865	800	600	630

It can be seen from the output that there is reduction in total pavement thickness or granular layer thickness with addition of marble dust. The value of thickness increased when marble dust was added 25% by weight. Also, for parent soil i.e. CM0, the pavement section is found to be safe against fatigue for bituminous layer thickness of 215 mm. Rest of the cases had bituminous layer thickness of 190 mm.

Total crust thickness obtained for each of the soil and marble dust mix, is shown in Figure 5.8. Also, variation in the thickness of granular layers is shown in Figure 5.9.

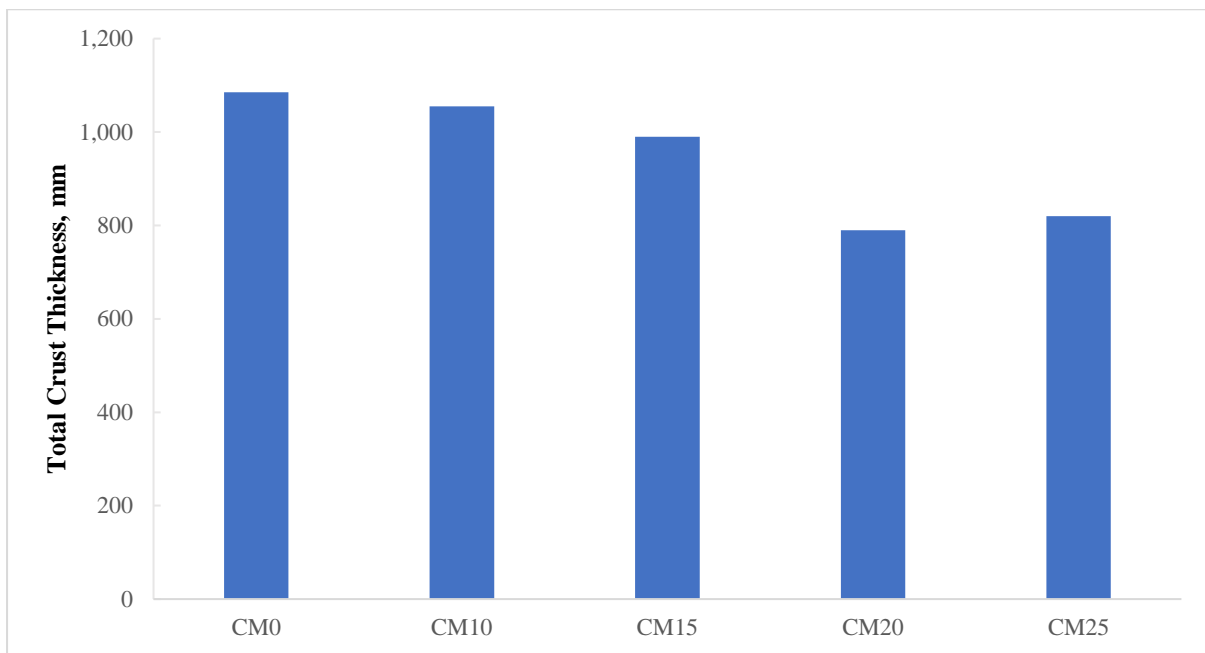


Figure 5.8: Variation in the total crust thickness for various percentages of marble dust

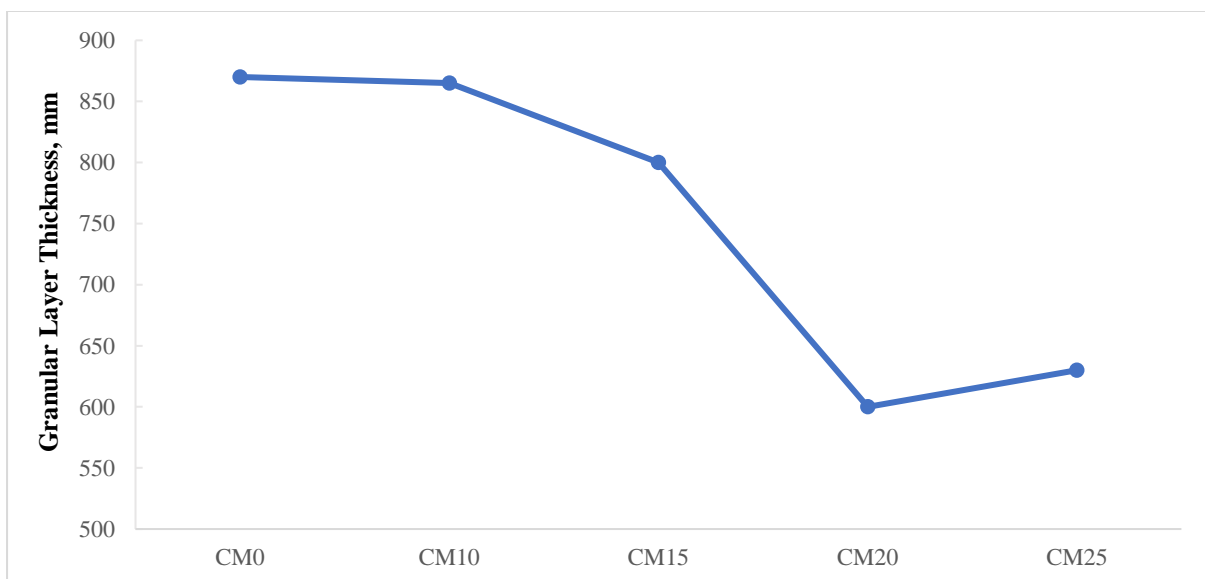


Figure 5.9: Variation in the granular layer thickness for various percentages of marble dust

Total crust thickness and granular layer thickness calculated using all of the methods is shown in Figure 5.10 and Figure 5.11, respectively. The thickness of bituminous layers was 160 mm for Erdem Çöleri Method. For Heukelom Klomp method, the bituminous layer thickness was 205 mm for CM0 and 190 mm for rest of the cases. Also, for TRRL method, the bituminous layer thickness was 195 mm for CM0 and 190 mm for rest of the cases. Thompson and Robnett method had bituminous layer thickness of 215 mm for CM0 and 190mm for the remaining cases.

All of these methods used for determining the subgrade resilient modulus showed decrease in the total crust thickness and granular layer thickness, with addition of waste marble dust.

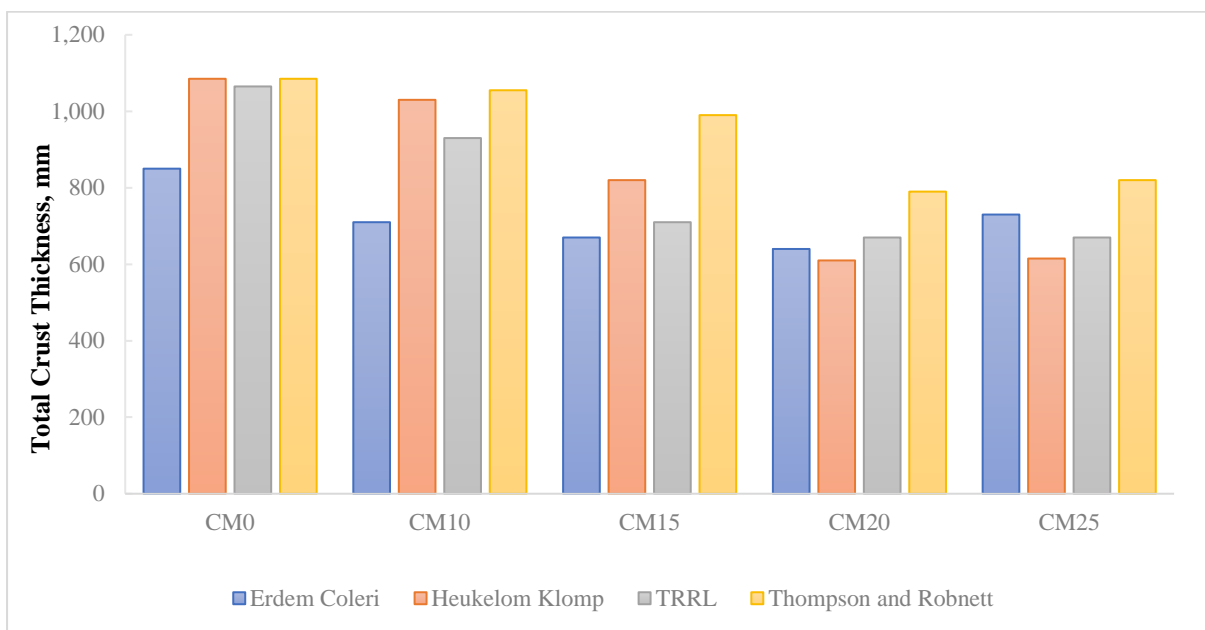


Figure 5.10: Total crust thickness calculated for every clay and marble dust mix using various methods

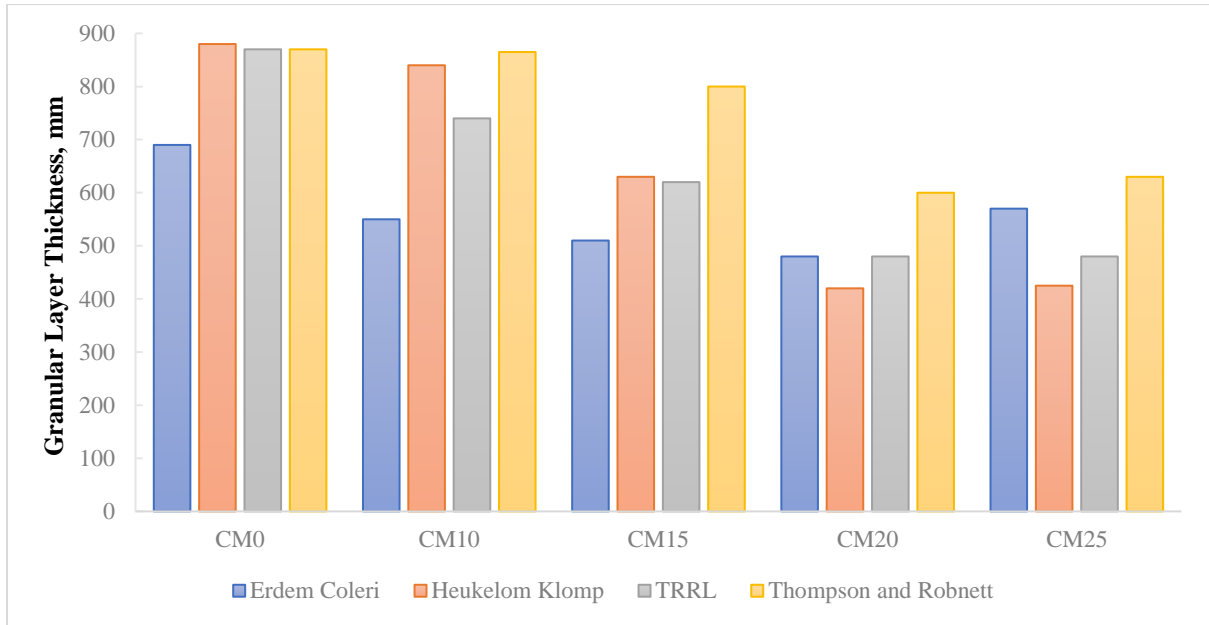


Figure 5.11: Granular layer thickness calculated for every clay and marble dust mix using various methods

5.5 Change in the Fatigue Life and Rutting Life with change in Granular Layer thickness

In this section, granular layer thickness is to be varied and its effect on fatigue life and rutting life is to be noted. Also, design granular layer thickness i.e. thickness at which actual strain values are less than limiting/allowable strain values, is to be determined for each marble dust percentage. The values of horizontal tensile strain and vertical compressive strain are obtained using IITPAVE. Design is to be done using following considerations:

- Design is to be done for the traffic of 50 msa.
- Thickness of bituminous layers is taken as 175 mm.
- The value of resilient modulus is taken as 3000 MPa for bituminous mixes.
- Thickness of granular layer is taken as 300 mm, 350 mm, 400 mm, 450 mm and 500 mm.
- Resilient modulus of subgrade used for the analysis was calculated from Erdem Çöleri method (equation 5.4), for different percentages of marble dust in the soil.
- Computation of fatigue life and rutting life is to be done using equation 5.7 and equation 5.9 respectively.

- Allowable strain values determined for safety against fatigue and rutting are 155.27 $\mu\epsilon$ and 371.7 $\mu\epsilon$, respectively.

Results obtained for different percentages of marble dust in the soil, are shown in Table 5.6.

Table 5.6: Variation in fatigue and rutting life with change in granular layer thickness

Granular Layer Thickness (mm)	Design Parameters	CM0	CM10	CM15	CM20	CM25
300	ϵ_t ($\mu\epsilon$)	176	164.9	162.7	159.6	167.3
	ϵ_z ($\mu\epsilon$)	379.6	342.6	336.1	326.4	350.8
	Fatigue Life (msa)	30.71	39.58	41.69	44.94	37.42
	Rutting Life (msa)	45.44	72.06	78.91	90.12	64.99
350	ϵ_t ($\mu\epsilon$)	169.1	158.2	156	153	160.6
	ϵ_z ($\mu\epsilon$)	338.3	304.2	297.9	288.8	311.5
	Fatigue Life (msa)	35.89	46.50	49.10	52.96	43.85
	Rutting Life (msa)	76.61	124.02	136.38	156.96	111.38
400	ϵ_t ($\mu\epsilon$)	163.2	152.4	150.3	147.2	154.7
	ϵ_z ($\mu\epsilon$)	301.5	270	264.2	255.9	276.8
	Fatigue Life (msa)	41.20	53.77	56.76	61.55	50.73
	Rutting Life (msa)	129.14	212.98	235.01	271.62	190.27
450	ϵ_t ($\mu\epsilon$)	158	147.3	145.3	142.3	149.7
	ϵ_z ($\mu\epsilon$)	269.1	240.1	234.8	227.2	246.3

	Fatigue Life (msa)	46.74	61.39	64.75	70.22	57.65
	Rutting Life (msa)	216.23	362.60	401.22	465.76	323.02
500						
	ϵ_t ($\mu\epsilon$)	153.5	143	141	138.1	145.3
	ϵ_z ($\mu\epsilon$)	240.7	214	209.1	202.3	219.7
	Fatigue Life (msa)	52.29	68.89	72.76	78.89	64.74
	Rutting Life (msa)	358.52	610.97	678.62	788.36	542.33

There was increase in the fatigue and rutting life with increase in the granular layer thickness. Different marble dust and clay mixtures had actual strain values lesser than limiting strain values at different thickness of granular layer. The pavement composition for each marble dust percentage in the soil is shown in the table below.

Table 5.7: Minimum granular layer thickness for each mix

Soil ID	Allowable Horizontal Tensile Strain ($\mu\epsilon$)	Allowable Vertical Compressive Strain ($\mu\epsilon$)	Minimum Granular Layer Thickness (mm)
CM0	155.27	371.7	480
CM10	155.27	371.7	375
CM15	155.27	371.7	356
CM20	155.27	371.7	333
CM25	155.27	371.7	395

So, it can be concluded from the results that addition of waste marble dust at optimum percentage to the soil, can reduce granular layer thickness up to 147 mm.

Also, there was increase in fatigue life and rutting life upon addition of waste marble dust to the soil. The variation in fatigue life and rutting life is shown in Figure 5.12 and Figure 5.13, respectively.

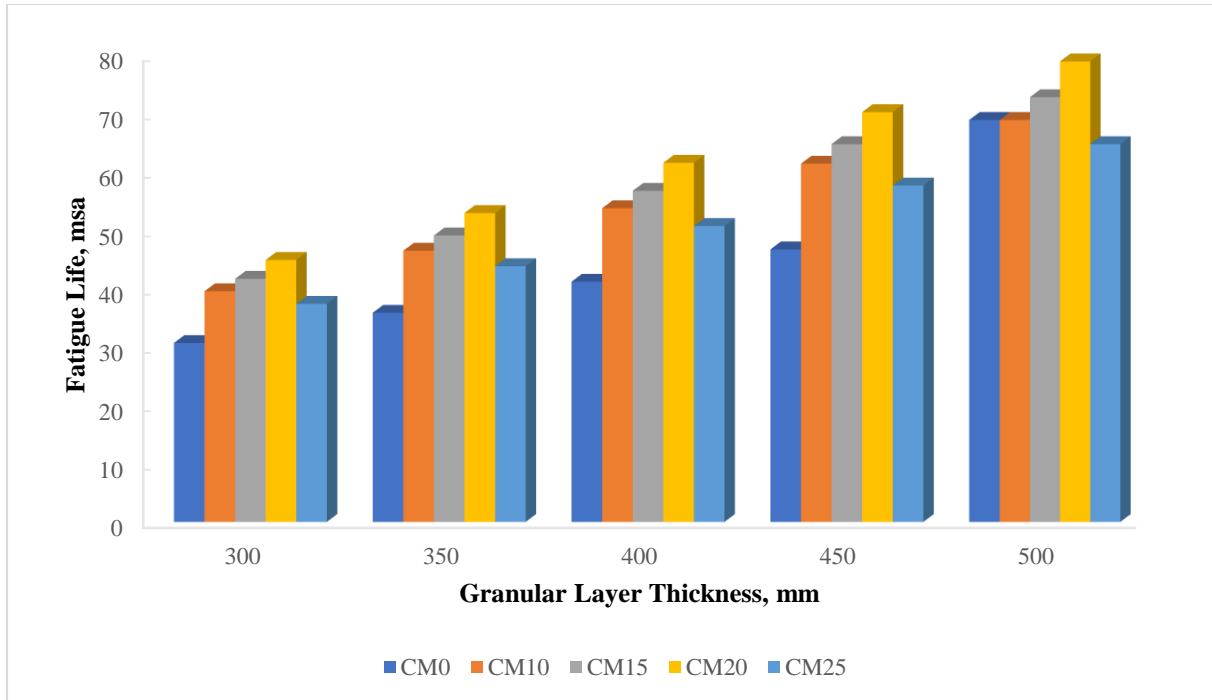


Figure 5.12: Variation in fatigue life with change in granular layer thickness

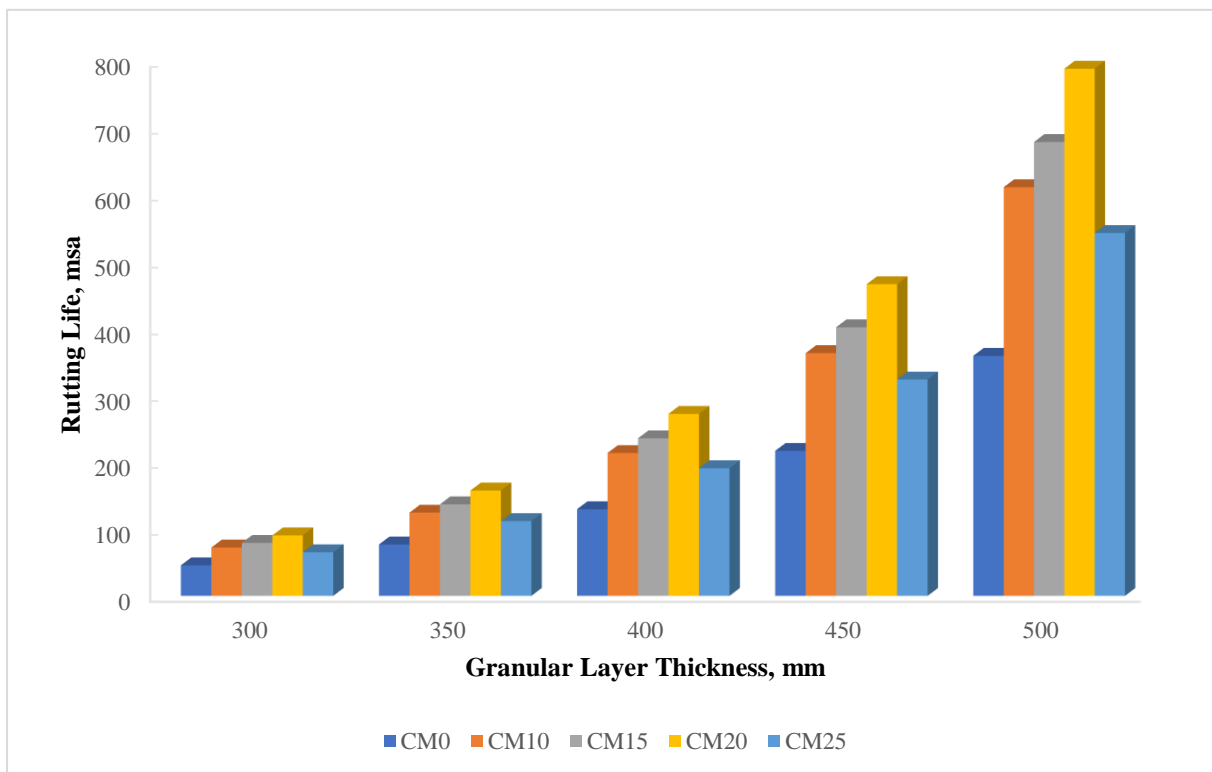


Figure 5.17: Variation in rutting life with change in granular layer thickness

CHAPTER 6

REGRESSION ANALYSIS USING SPSS

Regression analysis is done for predicting the value of California Bearing Ratio (CBR) on the basis of index properties and unconfined compressive strength of soil. Equation established from regression analysis is then used for determining the California Bearing Ratio (CBR) of soil. Difference in the predicted values and experimental values is to be noted.

6.1 Data used for the Regression Analysis

Data used for the regression analysis is taken from the study done by Bansal H *et al.* (2016) and Parte S S *et al.* (2014). Authors used waste marble dust/powder for stabilizing the soil and found improvement in the strength of soil upon its addition. Data used for the analysis includes index properties like liquid limit, plastic limit, plasticity index, optimum moisture content, maximum dry density, and unconfined compressive strength. Values are mentioned in the Table 6.1.

Table 6.1: Data used for the regression analysis using SPSS

Marble Dust, %	Liquid Limit, %	Plastic Limit, %	Plasticity Index, %	Optimum Moisture Content, %	Maximum Dry Density, kN/m³	California Bearing Ratio, %	Unconfined Compressive Strength, kPa
0	57.67	29.32	28.35	20.5	17.2	1.81	110.86
10	52.43	28.06	24.37	17.6	17.8	2.3	120.56
20	42.51	23.5	19.01	17	17.9	3.57	144.76
30	39.21	21.61	17.6	15.1	18.3	3.72	156.03
40	33.9	17.23	16.67	14.2	18.6	4.17	175.46
0	31.7	17.69	14	18	17.38	2.46	62.3
10	28.1	18.1	10	17.2	17.95	4.78	252.5
20	26.4	18.78	7.62	16.8	18.05	5.53	305.3
30	25	19.26	5.74	14.1	18.84	6.07	254.3

6.2 Output of the Regression Analysis

Output of the regression analysis showed R square value of 0.997. The value of standard error of estimate was 0.17200. Results of SPSS regression analysis, like Model Summary, ANOVA table, coefficient table are shown below.

Table 6.2: Descriptive Statistics

	Mean	Std. Deviation	N
CBR	3.8233	1.47088	9
MD	17.7778	13.94433	9
LL	37.4356	11.57660	9
PL	21.5056	4.54041	9
OMC	16.7222	2.02656	9
MDD	18.0022	.52855	9
UCS	175.7856	79.31011	9

Table 6.3: Correlations

		CBR	MD	LL	PL	OMC	DD	UCS
Pearson Correlation	CBR	1.000	.619	-.838	-.664	-.703	.786	.911
	MD	.619	1.000	-.406	-.431	-.918	.922	.445
	LL	-.838	-.406	1.000	.936	.623	-.577	-.654
	PL	-.664	-.431	.936	1.000	.620	-.510	-.448
	OMC	-.703	-.918	.623	.620	1.000	-.962	-.446
	MDD	.786	.922	-.577	-.510	-.962	1.000	.593
	UCS	.911	.445	-.654	-.448	-.446	.593	1.000
Sig. (1-tailed)	CBR	.	.038	.002	.026	.017	.006	.000
	MD	.038	.	.139	.123	.000	.000	.115
	LL	.002	.139	.	.000	.036	.052	.028
	PL	.026	.123	.000	.	.038	.081	.114

	OMC	.017	.000	.036	.038	.	.000	.114
	MDD	.006	.000	.052	.081	.000	.	.046
	UCS	.000	.115	.028	.114	.114	.046	.
N	CBR	9	9	9	9	9	9	9
	MD	9	9	9	9	9	9	9
	LL	9	9	9	9	9	9	9
	PL	9	9	9	9	9	9	9
	OMC	9	9	9	9	9	9	9
	MDD	9	9	9	9	9	9	9
	UCS	9	9	9	9	9	9	9

R square is generally defined as proportion of variance in the dependent variable (CBR) that can be explained by independent variables (LL, PL, OMC, MDD, UCS). Adjusted R square is the adjustment in R square value due to addition of some other predictors.

Std. error of estimate, also known as root mean squared error, is the standard deviation of the error term, and square root of the mean square for the residuals in the ANOVA table.

Table 6.4: Model Summary^b

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Change Statistics				
					R Square Change	F Change	df ₁	df ₂	Sig. F Change
1	.998 ^a	.997	.986	.17200	.997	97.149	6	2	.010

a. Predictors: (Constant), UCS, MD, PL, MDD, LL, OMC

b. Dependent Variable: CBR

ANOVA table shows us the effect the *p*-value (sig. or significance) of the predictor's effect on the available criterion. F-statistics is calculated by dividing the mean square (Regression) by mean square (Residual).

df known as degree of freedom, is defined as total number of coefficients in the regression analysis subtracted by 1.

Table 6.5: ANOVA^a

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	17.244	6	2.874	97.149	.010 ^b
	Residual	.059	2	.030		
	Total	17.303	8			

a. Dependent Variable: CBR

b. Predictors: (Constant), UCS, MD, PL, MDD, LL, OMC

Coefficient tables shows us the predictor variables. First term, known as constant, represents the Y-intercept. Y intercept is the height of regression line when it crosses the Y axis. Std. error shows us the standard error associated with the variables. Standardised coefficients are the coefficients which are obtained if all the variables in regression analysis i.e. dependent and independent, are standardised

t statistics and their associated 2-tailed p-values used in testing which shows whether a given coefficient is significantly different from zero.

Table 6.6: Coefficients^a

Model		Unstandardized Coefficients		Standardized Coefficients	t	Sig.
		B	Std. Error	Beta		
1	(Constant)	-32.932	19.592		-1.681	.235
	MD	.046	.022	.435	2.130	.167
	LL	-.155	.038	-1.217	-4.053	.056
	PL	.189	.071	.584	2.648	.118
	OMC	.517	.279	.713	1.856	.205
	MDD	1.582	.905	.569	1.747	.223
	UCS	.003	.003	.164	1.150	.369

Coefficients table shows us the beta coefficients for the actual regression equation. Standardised coefficients are based on the re-scaling of the model so that y-intercept is zero. So, the equation obtained using unstandardized coefficients for predicting the California Bearing Ratio (CBR) value on the basis of various soil properties is shown below:

$$CBR = -32.932 + 0.517 OMC + 1.582 MDD + 0.046 MD + 0.189 PL - 0.155 LL + 0.003 UCS \dots\dots (6.1)$$

Where,

CBR = California Bearing Ratio in %

OMC = Optimum Moisture Content in %

MDD = Maximum Dry Density in kN/m³

MD = Percentage of Marble Dust

PL = Plastic Limit in %

LL = Liquid Limit in %

UCS = Unconfined Compressive Strength in kPa

6.3 Comparison between predicted value and calculated value

Equation 6.1 is to be used for determining the California Bearing Ratio (CBR) of the soil based on the parameters obtained experimentally. The value of liquid limit, plastic limit, plasticity index, optimum moisture content, maximum dry density and unconfined compressive strength obtained experimentally, for different percentages of marble dust in the soil is mentioned in the Table 6.7.

Table 6.7: Experimental results for different percentages of marble dust

Marble Dust, %	Liquid Limit, %	Plastic Limit, %	Optimum Moisture Content, %	Maximum Dry Density, kN/m ³	Unconfined Compressive Strength, kPa	California Bearing Ratio, %
0	28.7	16.67	15.29	18.76	62.34	2.96
10	24.8	17.64	13.72	18.9	105.25	3.89
15	24.3	18.34	13.48	19.16	111.34	4.67
20	22.8	18.65	13.1	19.46	136.75	6.08
25	22.1	19.04	13.98	19.05	131.77	6.03

The value of California Bearing Ratio (CBR) obtained by inputting experimental results in the equation 6.1 is shown in Table 6.8.

Table 6.8: California Bearing Ratio determined using equation

Soil ID	Predicted California Bearing Ratio, %	Observed California Bearing Ratio, %
CM0	3.54	2.96
CM10	4.32	3.89
CM15	5.07	4.67
CM20	5.94	6.08
CM25	6.15	6.03

The variation between observed value i.e. CBR value determined experimentally, and predicted value i.e. CBR value determined using equation 6.1, is shown in Figure 6.1.

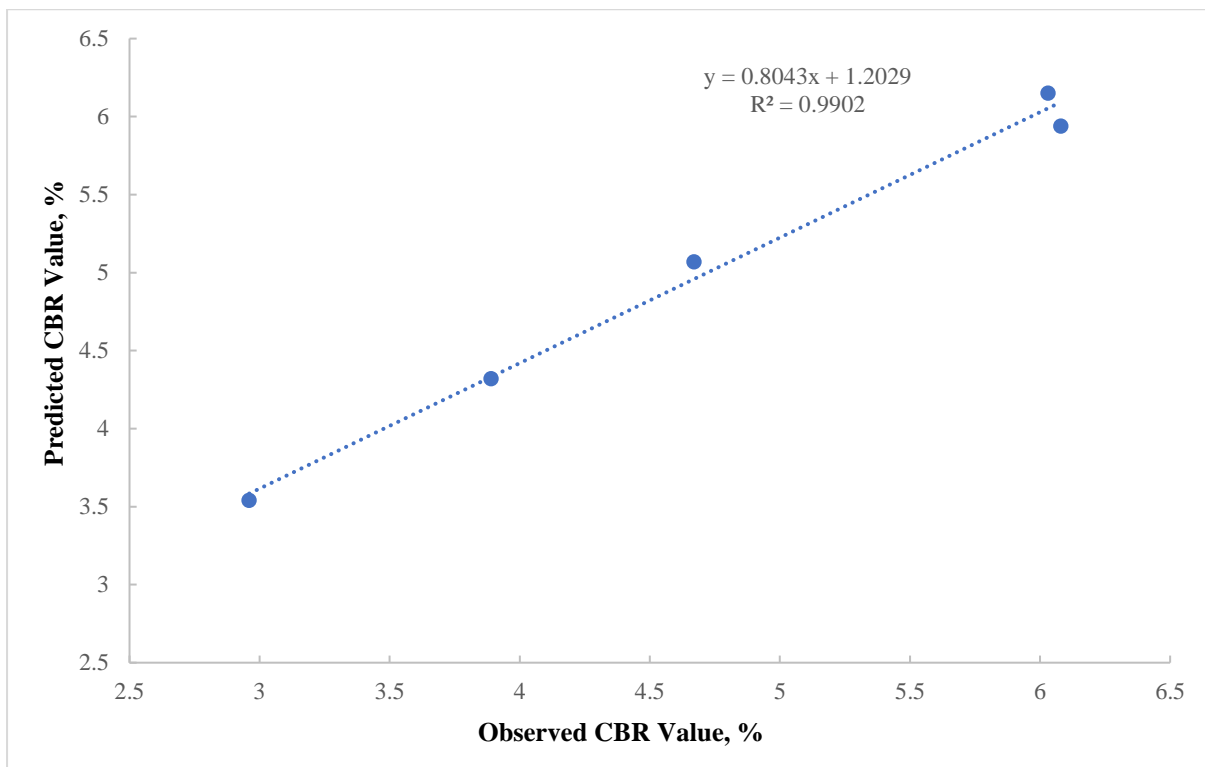


Figure 6.1: Variation between observed CBR value and predicted CBR value

6.4 Regression analysis using literature data and experimental data

As seen in the figure 6.1, there isn't much difference between the observed CBR value and predicted CBR value. So, the experimented data can also be used for predicting the CBR value of the soil based on these different parameters. Regression analysis is now to be done using previous literature data (Table 6.1) and experimentally determined data (Table 6.7). Results obtained are mentioned in the tables below.

Table 6.9: Descriptive Statistics

	Mean	Std. Deviation	N
CBR	4.1457	1.44852	14
OMC	15.7193	2.16600	14
MDD	18.3821	.68824	14
MD	16.4286	12.31456	14
PL	20.2779	3.98459	14
LL	32.8300	11.20808	14
UCS	152.1086	72.28783	14

Table 6.10: Model Summary^b

Model	R	R Square	Adjusted R Square	Std. Error of the Estimate	Change Statistics				
					R Square Change	F Change	df1	df2	Sig. F Change
1	.995 ^a	.990	.981	.19901	.990	113.617	6	7	.000

a. Predictors: (Constant), UCS, MDD, PL, MD, LL, OMC

b. Dependent Variable: CBR

Table 6.11: ANOVA^a

Model		Sum of Squares	df	Mean Square	F	Sig.
1	Regression	26.999	6	4.500	113.617	.000 ^b
	Residual	.277	7	.040		
	Total	27.277	13			

a. Dependent Variable: CBR

b. Predictors: (Constant), UCS, MDD, PL, MD, LL, OMC

Table 6.12: Coefficients^a

Model	Unstandardized Coefficients		Standardized Coefficients	t	Sig.	
	B	Std. Error	Beta			
1	(Constant)	-33.187	9.273		-3.579	.009
	OMC	.610	.159	.912	3.823	.007
	MDD	1.491	.393	.708	3.796	.007
	MD	.063	.011	.537	5.911	.001
	PL	.231	.042	.635	5.536	.001
	LL	-.176	.019	-1.358	-9.011	.000
	UCS	.003	.001	.127	1.824	.111

So, the equation obtained using unstandardized coefficients for predicting the California Bearing Ratio (CBR) value on the basis of various soil properties is shown below:

$$CBR = -33.187 + 0.610 OMC + 1.491 MDD + 0.063 MD + 0.231 PL - 0.176 LL + 0.003 UCS$$

..... (6.2)

Where,

CBR = California Bearing Ratio in %

OMC = Optimum Moisture Content in %

DD = Maximum Dry Density in kN/m³

MD = Percentage of Marble Dust

PL = Plastic Limit in %

LL = Liquid Limit in %

UCS = Unconfined Compressive Strength in kPa

So, California Bearing ratio (CBR) of soil can be predicted using above equation, when waste marble dust is added to it.

CHAPTER 7

CONCLUSIONS

Based on the experimental and analytical results, following conclusions can be drawn:

1. From the Atterberg's limit test, it was found that liquid limit decreased constantly with the addition of marble dust, whereas, plastic limit increased constantly with the addition of marble dust. Corresponding to liquid limit and plastic limit values, plasticity index decreased constantly with addition of waste marble dust.
2. Proctor compaction test results showed that there was decrease in optimum moisture content (OMC) and increase in maximum dry density (MDD) up to 20% addition of waste marble dust. The value of optimum moisture content (OMC) increased and maximum dry density (MDD) decreased when marble dust was added 25% by weight of soil.
3. California Bearing Ratio (CBR) test concluded that there was increase in the load bearing capacity of the soil, upon addition of waste marble dust. The value of California Bearing Ratio (CBR) increased by 105.40% with 20% addition of waste marble dust.
4. Results of the Unconfined Compressive Strength (UCS) tests also shows that increase in the compressive strength of the soil, upon addition of waste marble dust to it. The value of Unconfined Compressive Strength (UCS) increased by 119.36% with 20% addition of waste marble dust.
5. Resilient modulus of subgrade was determined using 4 methods, taking different soil parameters into considerations. Pavement design done using all of these methods showed decrease in the pavement thickness, with addition of waste marble dust. Decrease in the total crust thickness was maximum when subgrade elastic modulus was determined using Heukelom and Klomp method. Total crust thickness calculated by Heukelom and Klomp method was decreased by 43.77% upon 20% addition of waste marble dust.
6. Pavement thickness was minimum for each addition of waste marble dust to the soil when subgrade elastic modulus was determined using Erdem Çöleri method. This method takes soil properties like liquid limit, optimum moisture content (OMC) along with California Bearing Ratio (CBR) into consideration. So, it can be said that if properties like liquid limit and optimum moisture content (OMC) plays an important part in determination of subgrade resilient modulus, we're overestimating pavement thickness using methods which only

takes California Bearing Ratio (CBR) into account, for determining the subgrade resilient modulus.

7. Analysis done using Erdem Çöleri method showed that fatigue life and rutting life also increased upon addition of waste marble dust to the soil. For a constant thickness of granular layer, addition of marble dust showed continuous increase in fatigue and rutting life. Each granular layer thickness had maximum fatigue and rutting life at 20% addition of waste marble dust to the soil.
8. California Bearing Ratio (CBR) values obtained from the equation developed using regression analysis (from literature data), showed similar trend and there was only slight change in the California Bearing Ratio (CBR) values determined experimentally. So, prediction of California Bearing Ratio (CBR) value can be done using the equation developed using previous data and experimental data if soil type and marble dust type are same as that used for analysis.

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