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Prediction of Resilient Modulus of Subgrade Loamy Soils Through Wave Propagation Technique

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Abstract: The strength and performance of the subgrade are evaluated by its resilient modulus (MR) for the design of flexible pavement. The MR is routinely assessed using the cyclic triaxial test by conducting as per the American Association of State Highway and Transportation Officials (AASHTO). Since the triaxial test facility is not widely available and expensive, the proposed study intends to develop an MR relationship with CBR. For this purpose, eight disturbed soil samples were gathered from the Potohar region of Pakistan. The non-destructive test for MR measurement utilizing a new sonic viewer was performed before and after carrying out the CBR. The travel times of the compression (Vc) and shear (Vs) waves were also measured to calculate MR before and after each soaking period. A new empirical correlation between MR and CBR was developed using the ultrasonic pulse velocity (UPV) approach. This correlation was then evaluated by comparing it to past MR and CBR relationships, resulting in a strong agreement. Moreover, another excellent correlation was found between MR and compression wave velocity (Vc). It was also observed that larger compaction effort (blows/layer) influenced the linear increase in MR, Vc, and Vs values. Finally, UPV for predicting the MR of loamy soils for pavement design was more cost-effective and accurate than the conventional techniques which are complex and time taking.

Keywords: Resilient modulus, Compaction effort, Ultrasonic pulse velocity, California bearing ratio, Loamy soils, pavement design.

I. INTRODUCTION

The characterization of subgrade soil is a crucial step in flexible pavement design, according to the American Association of State Highway and Transportation Officials (AASHTO) [1]. The design approaches examine how pavement materials normally react to vehicular loads. The California bearing ratio (CBR) test has been employed for decades to assess subgrade soil. But there are some limitations in the CBR, which does not simulate the subgrade soil pragmatic behavior under moving traffic volumes. Due to modern pavement design, subgrade soil design philosophies have been developed to consider the resilient modulus (MR) to illustrate the resiliency of the subgrade [2]. Initially, MR was introduced by AASHTO to forecast the temporary soil deformation induced by traffic loads.

The MR simulates the actual traffic load conditions due to the load applied repeatedly on the test sample. The load-carrying capacity of the subgrade is a ratio of axial repeated deviator stress σd (cyclic stress in excess of confining pressure) to the recoverable or elastic axial strain (εr) (in the vertical direction) given by MR = σd/εr = (σ1 -σ3)/εr. Here, σd = σ1 -σ3, where σ1 is the major principal or total axial stress, and σ3 is the minor principal or total radial stress [3]. The granular materials are categorized by their nonlinear elastoplastic behavior, and MR is the characteristic modulus of this behavior [4]. The MR evaluates the strength of fine and granular subgrade soil and can be found using a triaxial test. The repeated load triaxial test has been used in highway laboratories for MR prediction by conducting tests as per AASHTO T 294-94 [5]. In this test, the cylindrical specimen is subjected to cyclic load under continuous confinement pressure (σ3 or σc), and the recoverable axial strain (εr) is measured. Establishing approximate methods for estimating MR is preferable due to the testing complexity and equipment needed for the repeated load triaxial tests.

First, Kirwan and Snaith [6] suggested a structural design procedure requiring MR to design flexible pavement. The MR is the key parameter for the mechanical design of flexible pavement. It evaluates the performance of pavement construction material under various stress-strain conditions induced by the moving vehicle load and different environmental conditions [7]. Recently, the MR has been used for pavement design to describe the mechanical characteristics of subgrade soil [1,8-9], and it depends on several parameters, including particle state (e.g., particle shape and degree of compaction), moisture state (e.g., pore water pressure and moisture content), and stress state (e.g., load and penetration).

In pavement design, the subgrade is compacted to achieve the maximum dry density (MDD) at an optimum moisture content (OMC). As moisture content lowers the particle's internal friction, which identifies the significant relationship with subgrade MR. Currently, several methods have been used to predict the MR values for various subgrade soil types, e.g., CBR tests [10-14], soil index properties [15, 16], unconfined compaction tests [17, 18], falling weight deflectometer [19], and repeated load triaxial test [20, 21].

Table 1. Correlation of MR with soil index properties

Proposed models	Source
MR (MPa) = 10.345 (CBR)	[10]
MR (MPa) = 16.82 (CBR) 0.64	[13]
MR (MPa) = 6.89160 (CBR) 0.7662	[14]
MR (MPa) = 10.342 (CBR)	[11]
MR (MPa) = 17.6 (CBR) 0.64	[12]
MR (MPa) = 5.00535 (CBR) + 2.95173	[15]
MR (MPa) = 147.46 (CBR) 0.34	[16]
CBRL/CBR45=0.00494S+0.78 where S = surcharge load in N	[13]
CBRL/CBR45=0.0173T+0.78 where T = pavement thickness	[13]
MR =4283+143×Qu, where Qu=ultimate compressive strength (psi)	[17]
MR = 6113.0 + 95.1 × (Qu) + 173.7 × PI – 27.8 × P200	[17]
MR = 29.370× σd/qu + 0.632× qu – 0.546× σd - 26.360	[18]
MR = 418.25x0.3889, R2 = 0.5761 where x= elastic modulus (psi)	[19]

The previous studies show that MR prediction is modeled from other soil geotechnical properties. The MR prediction models were made from unconfined compressive strength, plastic limit, and clay content [22-24]. In another study, where MR was aimed to predict the design of the foundation, the plastic index and moisture content were utilized to develop the MR prediction model based on several stress conditions [25]. To avoid the complexity of performing triaxial tests and expansive equipment, many researchers across the globe employed the ultrasonic pulse velocity (UPV) method to calculate the MR of soil [9,13-15, 26-29]. The UPV technique is the non-destructive method used in geotechnical engineering to determine dynamic strength parameters like elastic modulus, shear modulus, and poison ratio due to a convenient and effective testing approach. This method generates an ultrasonic pulse of 50-58 KHz from an electroacoustic transducer. The pulse or vibrational energy is passed from one surface of the CBR mold to the other. The travel time for a wave to pass from one end to another depends upon the density of the material. The fundamental principle is to measure an ultrasonic wave's propagation across a medium by measuring its amplitude and transit time. It has been applied in both in-situ and laboratory testing for establishing the mechanical properties of soil, rock, and concrete [29].

In this study, MR of subgrade was predicted from soaked CBR for pavement design. The UPV technique was utilized instead of repeated load triaxial tests to reduce testing time and cost. The eight soil samples were collected and analyzed for laboratory testing. It was found that more than 35% of soil passed the No. 200 (0.075mm) sieve, i.e., silt and clayey soils, according to the unified soil classification system (USCS). The travel times for Vc and Vs were calculated from soaked subgrade samples. It was the cheapest way to improve the geotechnical properties of soil subjected to poor strength under soaked conditions. Hence, the study would provide an alternate method to complex laboratory testing for predicting MR of loamy soils in pavement design.

II. METHODS

A total of eight disturbed soil samples were collected from the Potohar region of Pakistan, as shown in Fig 1. The soil samples were taken at about 1m depth from the ground surface. Four districts of the Potohar region were under observation, including Rawalpindi (RWP), Chakwal (CHK), Attack (ATG), and Jhelum (JMR). Two samples were collected from each district. The wet sieve analysis and Atterberg's Limits, such as liquid and plastic limit tests as per ASTM D 4318 [30] were conducted to determine soil index properties.

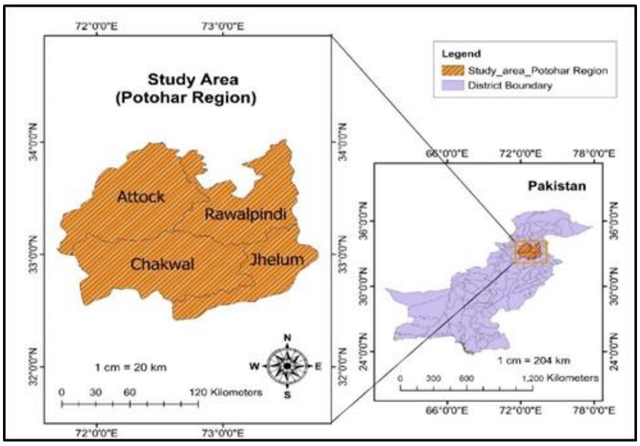

Fig. 1. Study Area.

The soil was classified according to AASHTO soil classification. Further, standard modified proctor compaction (MPT) was performed on each soil sample to find the OMC and

MDD. CBR test samples were compacted at 95% MDD. The non-destructive test for measuring MR using the UPV was conducted before and after soaking each CBR sample. The MR was measured from travel times of compression (Tc) and shear wave (Ts). Afterward, regression analysis was used to empirically correlate MR with CBR and shear waves.

Fig. 2. Experimental program

III. Results and Discussion

(a) 3.1 Soil classification

The fundamental geotechnical properties of soil were determined through a variety of experiments as shown in Fig 2. These include plastic limit (PL), wet sieve analysis, liquid limit (LL), and specific gravity test. Furthermore, the (MDD), and (OMC) were obtained from a modified proctor compaction test (MPCT), soaked CBR test, and MR obtained from UPV test were conducted for eight different soils samples (RWP-1, RWP-2, CHK-3, CHK-4, ATK-5, ATK-6, JHR-7, and JHR-8). In this designation, the first three letters (i.e., RWP-1) represent the city name, and the numeric digit depicts the soil sample number. Fig 3 shows the plot of the gradation curve for each soil sample, taking particle size on the x-axis and percentage finer on the y-axis.

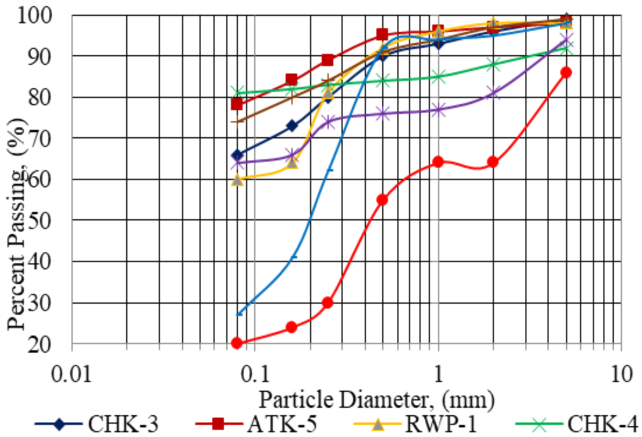

Fig. 3. Soil gradation curve.

The soil samples were classified as per the AASHTO and USCS based on the various index property tests. Overall, four types of soil, namely A-4, A-6, A-3, and A-2-4, generally called loamy soil, were classified as per AASHTO and CL-ML, CL, SP, and SC-SM (see Notation for details) according to USCS, and their index properties are given below in Table. 2.

Table 1: Summary of the test results

Sample ID	AASHTO	USCS	LL (%)	PL (%)	PI (%)	OMC (%)	γ_{dmax}(lb/ft^3)	Soaked CBR (%)	M_R (MPa)
RWP-1	A-4	CL-ML	29.96	19.40	10.60	13	121.3	4.26	45.23
RWP-2	A-4	CL-ML	28.74	19.85	8.89	11.8	122.3	4.37	37.73
CHK-3	A-4	CL	27.35	19.84	7.51	10	117.75	8.48	75.38
CHK-4	A-6	CL	33.96	21.20	12.76	14.95	119.66	4.15	33.35
ATK-5	A-4	CL	30.22	19.92	10.30	12.50	119.94	6.92	55.06
ATK-6	A-3	SP	NP	NP	NP	9.30	134.68	9.30	108.22
JMR-7	A-6	CL	31.51	20.51	11	9	121.89	3.18	26.12
JMR-8	A-2-4	SC-SM	NP	NP	NP	10.3	118.54	10.78	102.96

(b) 3.2 MPCT

The MPCT is performed per AASHTO T 180-86 (i.e., the standard test procedure for soil moisture-density relationship) [31]. The compaction effort (CE) can be found using the relation given by Das [32] as follows.

$$CE = \frac{(N_L \times N_b \times h \times W)}{V} \quad (1)$$

Here CE = compaction effort, NL = number of layers, Nb = number of blows per layer, W = weight of the hammer, h = drop of the hammer, and V = volume of soil.

The multiple stages of CE are required to evaluate its impact on the CBR and MR of the soil under examination. This can be achieved by varying the number of blows/layers as defined by the AASHTO compaction test. The number of blows per layer used in this work is 13, 30, and 65 using a 4.45 kg (10 lb.) rammer and a 457 mm (18 in.) drop.

The compaction curves for each soil sample are drawn as shown in Fig. 4. In this curve, the dry density is plotted on the y-axis while moisture content (MC) is taken on the x-axis. The peak of the curve represents the point where the soil has attained the MDD, and the MC at that point is known as OMC. It can be seen that ATK-6 has an MDD of 134.68 lb/ft3 at OMC of 9.30%, while CHK-3 has a minimum value of 117.75 lb/ft3 MDD at 10% of OMC. The proctor test results are considered from the author's previous study [33].

(c) 3.3 CBR test.

The soaked CBR tests were performed on soil samples per AASHTO T 193-99 [34]. Based on the MDD acquired from MPCT for each soil sample, the specimens for the soaking CBR test were prepared at 95% relative compaction. The results of the soaked CBR test are given in Table 2, and Fig. 5 shows the load penetration curve for the CBR test.

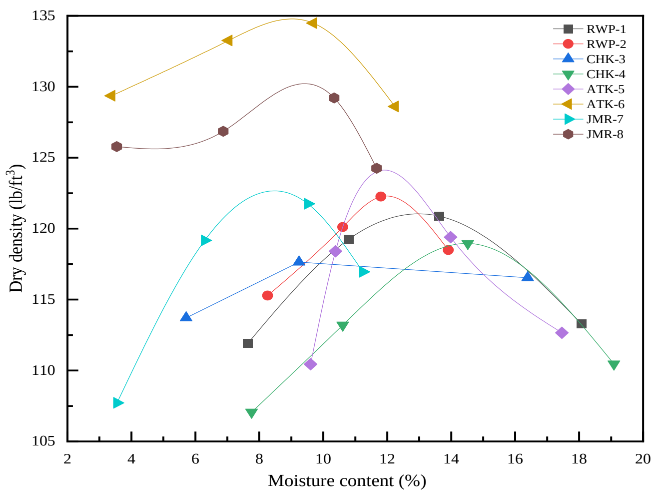

Fig. 4. Moisture density relation of tested soils.

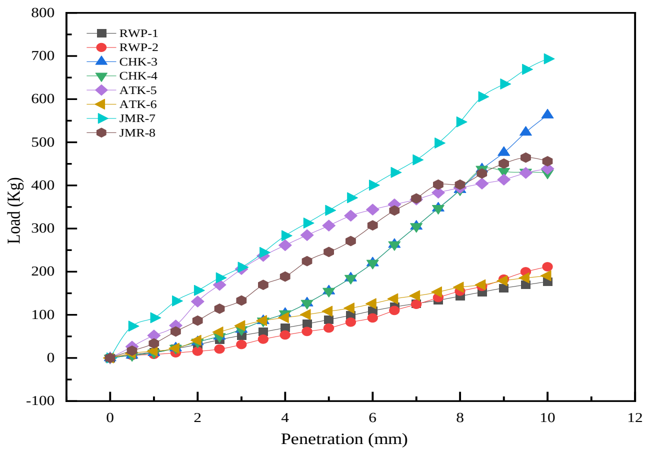

Fig. 5. Load versus penetration curves for soaked CBR

(d) 3.4 Summary of test results

All eight soil samples were tested for the correlation establishment of MR and CBR for indigenous subgrade soils. Table 2 gives an overview of the experimental findings for the examined soil samples. Atterberg limits revealed that the soil LL was 27-33.96%, the PL was 9-21.51%, and the PI was 10-12.76 %. Modified proctor test results of soil samples showed that OMC ranged from 9-14.95% and MDD 118-134.68 lb/ft3. Soaked CBR test results indicated that overall CBR values ranged from 2-11%. MR values ranged from 25-118 MPa. Soil samples were classified as A-4, A-3, A-6, and A-2-4 based on the AASHTO soil classification system.

IV. MR PREDICTION USING UPV

Stephenson [35] claimed that UPV could be measured through the soil, and the testing method used to determine wave velocity can be utilized to characterize the desired dynamic properties. The values of MR were numerically obtained from the UPV test. The same CBR molds were used for the UPV test, as shown in Fig. 2 (k). The VC and VS were passed through CBR molds for 10, 30, and 65 blows per layer. The MR and Poisson's ratio (μ) can be estimated by measuring the time for wave propagation, according to ASTM D 2845-83 [36]. The MR value at 95 % MDD was calculated using Eq. (2), and the results were computed in Table 3.

Table 3: UPV test results for M_R

Sample ID	CE	ρ (Kg/m³)	L (m)	T_C (sec)	T_S (sec)	V_C (m/s)	V_S (m/s)	M_R (MPa)	M_R at 95 % MDD
RWP-1	10	1657.19	0.11643	0.00107324	0.0016999	108.4845	68.4922	45.58	45.23
	30	1778.83	0.11643	0.00103450	0.0015068	112.5471	77.2697	47.51	
	65	1963.39	0.11643	0.00101205	0.0014029	115.0437	82.9923	49.76	
RWP-2	10	1696.79	0.11643	0.00126566	0.0017959	91.9987	64.8310	34.12	37.73
	30	1818.77	0.11643	0.00118821	0.0017675	97.9877	65.8697	41.88	
	65	1984.82	0.11643	0.00115421	0.0016956	100.8739	68.6659	43.15	
CHK-3	10	1786.69	0.11643	0.00104543	0.0085494	111.3704	13.6191	75.86	75.38
	30	1940.112	0.11643	0.00098789	0.0076758	117.8573	15.1684	80.39	
	65	1985.634	0.11643	0.00097432	0.0066785	119.4983	17.4336	81.80	
CHK-4	10	1683.88	0.11643	0.00143029	0.0035644	81.4035	32.6638	33.99	33.35
	30	1747.94	0.11643	0.00136872	0.0031234	85.0649	37.2766	34.94	
	65	1834.50	0.11643	0.00131687	0.0025438	88.4142	45.7688	36.41	
ATK-5	10	1680.42	0.11643	0.00114368	0.0019548	101.8029	59.5586	56.29	55.06
	30	1808.89	0.11643	0.00105856	0.0017234	109.989	67.5583	58.43	
	65	1860.08	0.11643	0.00101687	0.0015945	114.4984	73.0170	59.18	
ATK-6	10	2128.44	0.11643	0.00090972	0.0045644	127.9844	25.5077	105.49	108.22
	30	2369.01	0.11643	0.00089865	0.0031234	129.5610	37.2766	115.71	
	65	2376.44	0.11643	0.00089234	0.0023434	130.4771	49.6827	120.55	
JMR-7	10	2055.53	0.11643	0.00184399	0.0065644	63.1402	17.7363	25.87	26.12
	30	2135.87	0.11643	0.00175568	0.0059912	66.3162	19.4334	27.55	
	65	2186.95	0.11643	0.00170012	0.0046734	68.4834	24.9131	29.09	
JMR-8	10	2207.91	0.11643	0.00093998	0.0056644	123.8643	20.5543	104.28	102.96
	30	2279.51	0.11643	0.00091998	0.0052123	126.5566	22.3373	108.36	
	65	2329.72	0.11643	0.00090245	0.0043234	129.0151	26.9300	112.50	

$$M_R = \frac{[(3V_C^2 - 4V_S^2)\rho V_S^2]}{(V_C^2 - V_S^2)} \quad (2)$$

$$\mu = \frac{[(V_C^2 - 2V_S^2)]}{(V_C^2 - V_S^2)\times 2} \quad (3)$$

Where;

$$V_C = \frac{L}{T_C} \text{ and } V_S = \frac{L}{T_S} \quad (4)$$

Here, L is the pulse-travel distance (m), TC and TS are the pulse-travel times (s) to be evaluated in the laboratory for waves propagation, ρ is the mass density in (kg m-3), and VS and VC are in (ms-1) for shear and compression waves, respectively.

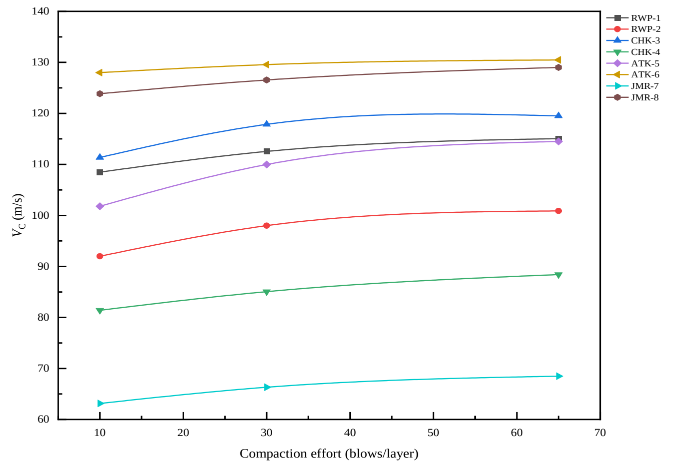

Fig. 6. Effect of compaction effort on V_C of soil CBR samples

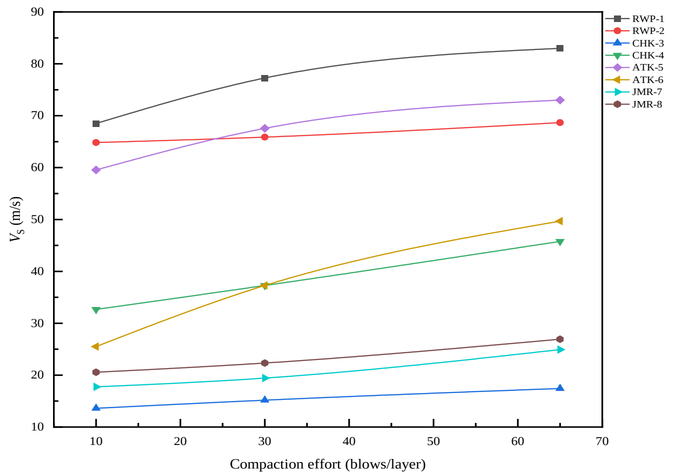

Fig. 7. Effect of compaction effort on V_S of soil CBR samples.

The ultrasonic pulse velocity equipment has two transducers acting as transmitters and receivers, one for P for compression wave and one S for shear wave. The top of the CBR sample was in contact with one transducer and the bottom with the other. The P or S wave time was measured to propagate through the sample using a new sonic viewer before and after soaking the sample, as shown in Fig. 2 (k). It is important to note that once the samples were taken out from the water tank after being soaked, the swell plate was removed from the CBR samples. After that transcoder was placed, and then VS and VC were passed through soil samples that can be calculated from Eq. (4). It was observed that the VC and VS values were linearly increasing with the increase in CE, as shown in Fig. 6 and 7. This is because the soil stiffness increases with the increase in CE. This effect can be more prominent for a longer duration of soaking the samples.

V. Correlation between MR and VC

The nonlinear relationship between the MR and VC is expressed in Fig. 8. The exponential increase is observed in the curve, and the slope increases with the increase in VC. It begins as a horizontal line rises gradually, then the growth becomes rapid. The VC in CBR mold has a high impact on soil samples, identifying the exponential increase in MR due to VC. Evidently, there exists a strong correlation between MR and VC with a coefficient of determination R2 = 0.9391, and the obtained Eq. (5) is given below:

$$M_R(\text{MPa}) = 0.036V_c^2 - 5.8388V_c + 258.72 \quad (5)$$

The assessment of the prediction model is made from the criteria defined by Witckaz [37] given below in Table 4.

Table 4. Criteria for the goodness of fit statistical parameters [39]

Sr. No	Criteria	Coefficient of determination (R^2)
i.	Excellent	>0.9
ii.	Good	0.7-0.89
iii.	Fair	0.4-0.69
iv.	Poor	0.2-0.39
v.	Very Poor	<0.2

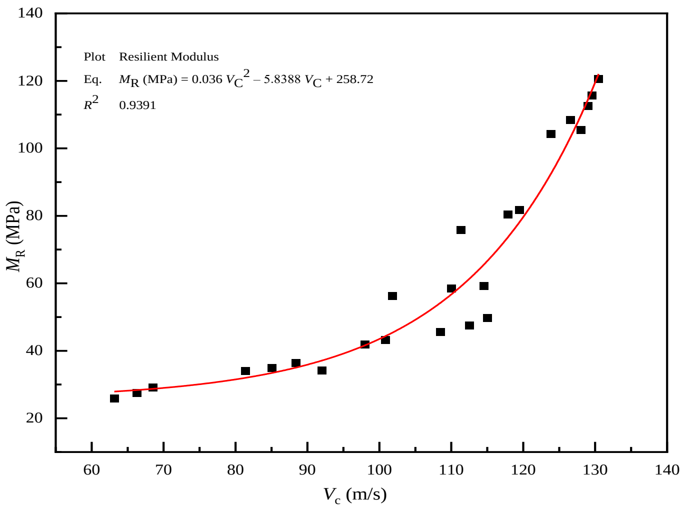

Fig. 8. Relationships between M_R and V_c.

VI. Correlation between MR and Soaked CBR

The need for simplified methods for MR prediction became essential because of the time-consuming and expansive nature of conventional laboratory methods. Salter [38] found that CBR can be used to find MR, however, it is difficult to find the MR value directly. According to the results deduced by Heukelom and Klomp [10], it was possible to correlate CBR with MR, and their proposed relation is given by Eq. (6).

$$M_R(\text{MPa}) = 10.3454 \times CBR \quad (6)$$

The AASHTO stated that Eq. (6) is suitable for fine-grained soils [1]. Later, S.S.Razouki et al. [13] predicted MR from CBR and presented the relation expressed as:

$$M_R(\text{MPa}) = 16.82 \times (CBR)^{0.64} \quad (7)$$

Similarly, Razouki and Kuttah [15] and Powell et al. [12] employed the UPV approach to find the in-situ subgrade resilience by the following Eqs. (8) and (9) respectively.

$$M_R(\text{MPa}) = 5.00535 \times (CBR) + 2.95173 \quad (8)$$

$$M_R(\text{MPa}) = 17.6 \times (CBR)^{0.64} \quad (9)$$

Garber and Hoel [11] and Xinman et al. [14] tried to substitute the extensive and time-consuming triaxial test with the CBR model to estimate the subgrade's MR and came across the following Eqs. (10) and (11) respectively.

$$M_R(\text{MPa}) = 10.342 \times (CBR) \quad (10)$$

$$M_R(\text{MPa}) = 6.89610 \times (CBR)^{0.7662} \quad (11)$$

For this reason, the CBR value was determined for all the prepared soil CBR samples to correlate MR with CBR in this work. Fig. 5(k) illustrates the use of the UPV approach on a CBR sample to obtain the TC and TS values, as shown in Table 3. According to Fig. 9, the MR obtained from Eq. (2) is correlated with CBR. The single linear regression analysis obtained the best-fit line for the MR-CBR relation. The regression model obtained from the method of least squares is given by Eq (12).

$$M_R(\text{MPa}) = 10.711 \times (CBR) - 8.3633 \quad \text{for CBR} \geq 1 \quad (12)$$

The coefficient of correlation R = 0.9605 (R2 = 0.9227) indicates an excellent correlation between MR and CBR, as per the criteria defined in Table 4 [39]. The value of R indicates a strong correlation after Anderson and Sclove [40]. Furthermore, the influence of CE on MR is shown in Fig. 10. It demonstrates that the MR is improved as CE increases, which agrees with previous studies that more compaction increases strength [9,15]. Additionally, by extending the time that CBR samples are permitted to soak, increasing their MR many times is possible by increasing the amount of CE.

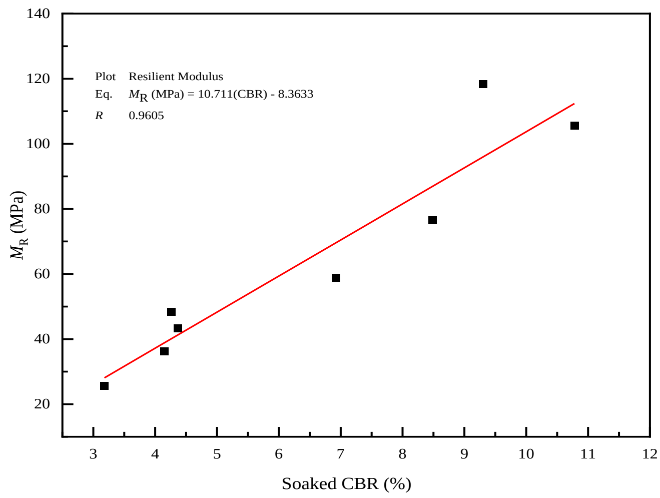

Fig. 9. Relationships between M_R and CBR.

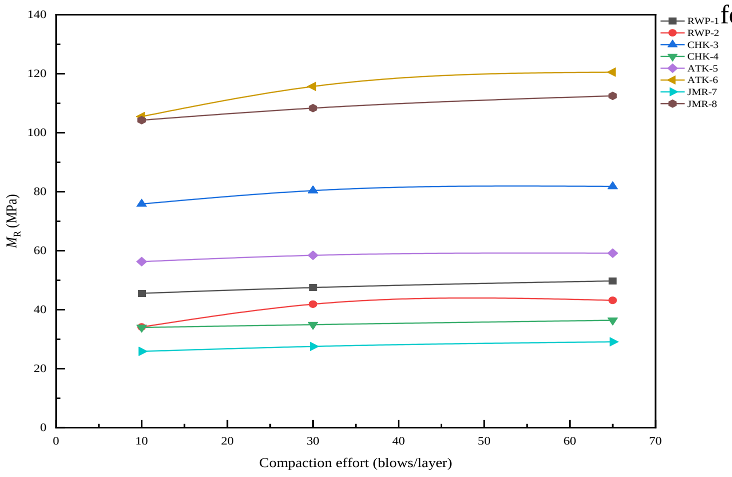

Fig. 10. Effect of CE on M_R

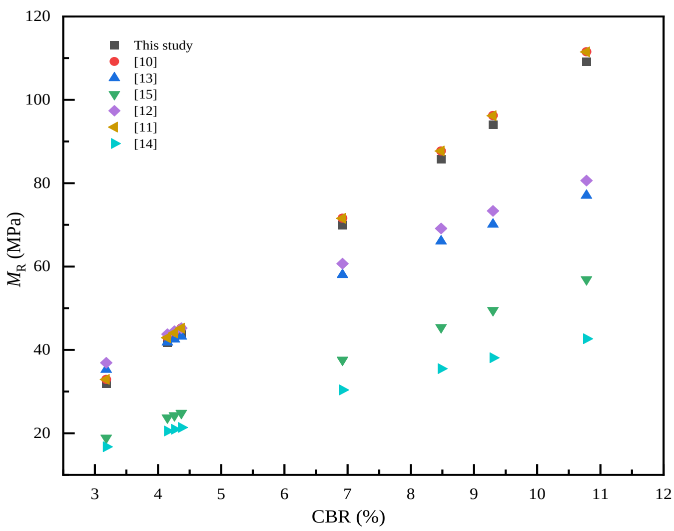

Fig. 11. Comparison of developed M_R - CBR relationship with previous studies.

In order to validate the proposed MR-CBR relation, it is correlated with the previous six models, i.e., Eqs. (6-11) as shown in Fig. 11. It can be seen that the obtained results are almost in exact correspondence with Heukelom and Garber [10-11], which demonstrates the effectiveness of the developed model. However, the results proposed by Powell [12], S.S. Razouki [13], Xinman [14], and Razouki [15] start linearly increasing and correlating with the constructed model, then increased slowly, and finally, a slow decay was noticed after that, slightly departing from the presented model due to change in soil composition.

CONCLUSIONS

Eight different subgrade samples were tested to evaluate the effectiveness of the ultrasonic pulse velocity technique as cost-effective and easily available compared to repeated load triaxial test to calculate resilient modulus. From the Atterberg's limit test, wet sieve analysis, MPCT, CBR, and UPV tests, the following key conclusions can be drawn:

1. The resilient modulus of compacted loamy soil samples in the CBR molds increases with the compaction effort as the soil becomes denser. It is best to use maximum compaction effort that results in at least 100% relative compaction.

2. The compression and shear wave velocities of subgrade in the CBR molds increase linearly as a function of compaction effort before and after soaking the soil samples. However, there exists a strong nonlinear correlation between the resilient modulus and compression wave velocities, which could predict the M_R at various compression wave velocities for diverse soil types at different moisture contents.

3. The study demonstrates the prediction model based on the soaked subgrade CBR using the UPV technique for M_R. The model could accurately predict the M_R at various CBR values which would replace the repeated load triaxial test.

4. The comparative analysis of developed M_R and CBR relations with previous research showed a strong agreement, showing the effectiveness of the proposed model. It is more conservative than the relation developed by Garber & Hoel (2015) which can only be used for gypsum sand.

5. Due to sufficient accuracy, the proposed method can be used for the preliminary prediction of M_R for locally available soils in the Potohar region, Pakistan. It would provide guidance to assess M_R for future pavement design.

Based on the current study, the developed linear MR and CBR relation applies to loamy soils. It includes a range of Potohar soils with limited plasticity indices. This study contained eight soils and did not consider all possible conditions in the unsaturated soils. To broaden the applicability of the research, further experiments can be performed on a larger scale.

Abbreviations			
AASHTO	American Association of State Highway and Transportation Officials	UPV	Ultrasonic pulse velocity
ASTM	American Society for Testing and Materials	V_c	Compression wave velocity (m/s)
USCS	Unified Soil Classification System	V_s	Shear wave velocity (m/s)
M_R	Resilient modulus	MPCT	Modified proctor compaction test
σ_d	Axial repeated deviator stress	OMC	Optimum moisture content
ε_r	Recoverable or elastic axial strain	MDD	Maximum dry density
σ	Stress	PL	Plastic limit
σ_1	Major principal or total axial stress	LL	Liquid limit
σ_3	Minor principal or total radial stress	PI	Plastic index
CL-ML	Silty clay	L	Pulse-travel distance (m)
CL	Lean clay	ρ	Density (kg/m^3)
SP	Poorly graded sand	μ	Poisson's ratio
SC-SM	Silty clayey sand	T_C	Pulse-travel time (s) for compression wave
CE	Compaction effort (number of blows/layer)	T_S	Pulse-travel time (s) for shear wave
CBR	California bearing ratio	R	Coefficient of determination
IIT	Pavement design software		

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