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Baghdad Subgrade Resilient Modulus and liquefaction Evaluation for Pavement Design using Load Cyclic Triaxial Strength

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Abstract

Pavements fail for different reasons; poor design, poor materials and poor construction methods are the most common. The pavement foundation (subgrade) represents one of the key elements in the pavement design. The American Association of State Highway and Transportation officials (AASHTO) published the AASHTO Guide for Design of Pavement Structures (AASHTO, 1986) in which the use of Resilient Modulus (M_r) was adopted as the principal soil property contributing to the design of flexible pavements. It can consider that resilient modulus (M_r) is a key value in pavement design.

The present study uses the standard laboratory test for load cyclic Triaxial strength to evaluate the resilient modulus and liquefaction condition of some Baghdad soils ,as well as using the neural network approach to develop a model that can be used to predict resilient modulus values for Baghdad soils . The model uses the results of routine laboratory tests like specific gravity, water content, Atterberg limits, soil classification and unconfined compressive strength to predict M_r.

It is well-known that the Performance of resilient modulus tests are difficult, expensive and time consuming and hence there has been an interest in adopting the Ohio State University mathematical model (OSU Model) introduced by Kim 2004 and confirmed by Rodgers 2006 that satisfactorily predicts resilient modulus values without the necessity of a laboratory test. It is very important for a mathematical model to accommodate new data as it becomes available.

It is concluded that soil brought from Baghdad City exhibited the resilient modulus (M_r) of pavement subgrade soils which has been adopted by the American Association of State Highway and Transportation Officials (AASHTO) for the purpose of designing flexible roadway pavement systems, values ranging from 40 MPa to about 100MPa. Based on ASTM subgrade resilient modulus criterion, the A-7-5 and A-6 untreated subgrade soil would be classified as fair to poor (unacceptable as a competent subgrade).

To prove the capability of the network, M_r predicted values for Baghdad soil were compared with its corresponding M_r measured. It is concluded that Baghdad soils need to be provided with new network and model with some modification needed to be done on the OSU models to provide a good estimation of M_r for the Baghdad soils.

The results of cyclic load test carried out in laboratory to conduct Liquefaction indicate that for a given initial water content and specific dry density with initial effective stress, it is concluded that generally all samples didn't exhibit significant gain in liquefaction condition and didn't show conflict values due to the reduction in the rate of pore water pressure generation and shear strain of all samples subjected to cyclic loading. they shows withstanding against liquefaction by reaching high value of Normalized principal Stress when reaching to critical built up of Pore water pressure which lead to the fact that a liquefied condition could not possibly develop in those soils.

Keywords: Resilient Modulus, C.B.R, Subgrade Compaction, Pavement Design

1.Introduction

Pavements fail for different reasons; poor design, poor materials and poor construction methods are the most common. The pavement foundation (subgrade) represents one of the key elements in the pavement design; its behavior will influence the overall pavement performance.Subgrade soils are subjected to repeated loads due to heavy traffic, which can cause deformations and distress of the overlying structures. To improve and standardize design procedures, The American Association of State Highway and Transportation officials (AASHTO) published the AASHTO Guide for Design of Pavement Structures (AASHTO, 1986) in which the use of Resilient Modulus (M_r) was adopted as the principal soil property contributing to the design of flexible pavements.

Resilient Modulus (M_r) is a key value in pavement design. Performance of resilient modulus tests is difficult, expensive and time consuming and hence many researchers were developing a mathematical model that satisfactorily predicts resilient modulus values without the necessity of a laboratory test. It is very important for a mathematical model to accommodate new data as it becomes available.

Resilient Modulus is the failure of a flexible pavement structure supported on a subgrade soil and subjected to repeated traffic loading, can occur through two primary mechanisms - collapse of the pavement structure or cracking of the surface of the pavement. A collapse of the pavement structure can occur due to large plastic (permanent) deformations in the subgrade soils. However, even when the loads on the pavement are not excessive but nominal, the pavement surface can crack due to fatigue, caused by the reversal of elastic strains at any location in the pavement system. As a result of repeated loads such as those caused by moving traffic, cohesive soils in the subgrade incur repeated elastic deformations. When these deformations exceed a threshold value, premature fatigue failure of the flexible pavement through cracking of the pavement surface occurs.

Kim 2004 studied the suitability of existing regression models and, if necessary, develops an improved model for predicting M_r of cohesive soils without conducting expensive and time-consuming M_r tests. Additional tests were performed on samples compacted to optimum conditions but allowed to fully saturate. M_r predicted from six existing models studied showed wide scatter and poor correlation with the measured M_r. An improved constitutive model was developed to account for the effects on M_r of the stress state of the soil and its engineering properties obtained from simple laboratory tests.

George 2004 used an existing models to study significantly overestimated the M_r of a cohesive soil, the proposed model predictions are close to the experimental values and are in most cases a slight underestimation. This implies that M_r Values predicted by the proposed model are generally slightly conservative, and can be safely used in the design of flexible pavements to be built on cohesive soils. The proposed model can be a useful and reliable tool for estimating M_r of cohesive subgrade soils using basic soil properties and the stress state of the soil.

Rodgers 2006 studied the improvement of the OSU regression method used to estimate the resilient modulus from commonly performed tests, expand the model data set and evaluate the model's performance with additional data. She uses the neural network approach to develop a model that can be used to predict resilient modulus values for Ohio Soils.

Proper determination of the resilient modulus to be used in pavement design has been studied by a large number of researchers (e.g., Seed, et al. (1962), Fredlund et al. (1977), Drumm et al. (1990), Li and Selig (1994), Pezo and Hudson (1994), Lee et al. (1995), Guan et al. (1998), Mohammad et al. (1999), Kim (1999), Li and Qubain, (2003), and Butalia et al. (2003)) and several different methods have been developed for evaluating the appropriate value of M_r to use in design. Some of those methods use laboratory test results from reconstituted or undisturbed samples to create regression models, relating static soil properties and, usually the stress state to determine M_r.

Liquefaction denotes a condition where, during the course of cyclic stress applications, the residual pore water pressure on completion of any full stress cycle become equal to the applied confining pressure, it was seen many times that failure occurs in Subgrade clayey layer due to the rapid acceleration and build up of pore water pressure which leads to initial liquefaction [Seed, et al.1975]. The materials used in soil stabilization required to lead to maintain in the stress ration required to cause liquefaction to prevent this phenomenon from occurs. An alternative explanation is that during any period of cyclic straining, there is a progressive change in the soil structure with the result that the volume change occurring in any one cycles decrease progressively with increasing numbers of cycle so precautions should be taken in selecting any additive to stabilized soil against cyclic loading [Raad,et al.1990;Little,1987]. Liquefaction of Subgrade soil can cause severe damage to roads and bridges and earth structures during severe cyclic loading, dynamic forces or earthquake (Rodriguez et al. 2008)

2. Purpose of the Study

The main purpose of this research is to find real and accurate direct values of the Resilient Modulus carried out using cyclic loading available in the laboratories of soil mechanics in the Department of Civil Engineering at the Ohio State University, the United States to assist highways designer in Iraq to put this parameter into consideration for city of Baghdad as a parameter in the design of roads ,highways and airports, as well as to find out whether these types of soil affected by liquefaction condition at selected relative densities ,confining pressure and cyclic stress ratio.

3. Testing Procedure

The resilient modulus and liquefaction test is a cyclic triaxial test usually performed on undisturbed cohesive soils.

Since AASHTO first proposed T274-82 as the testing procedure for determining M_r of soils, three additional modifications, AASHTO T292-91, and T294-94, and T307-99, have been introduced. The basic differences among the four testing procedures, AASHTO T274-82, T292-91, T294-94, and T307-99, are the applied waveform and sequence, sample conditioning before testing, number of loading cycles, and introduction of a linear variable differential transformer (LVDT) to measure axial displacements. Table 1 summarizes the dynamic waveform, load and cycle duration for each of the testing procedure, and Table 2 lists the confining stress, deviator stress, and number of loading cycles. After the 1986 adoption of M_r of soil for the design of pavement structures, the severe sample conditioning before testing often resulted in disturbance to the soil sample, and sometimes sample failure was experienced during testing. In 1991, AASHTO T292-91 modified T274-82. The sequence of applying the confining pressure and deviator stress to the specimens in the AASHTO T292-91 testing procedure has raised some concerns. As shown in Table 1, the AASHTO T274-82 and T292-91 testing procedures allow various waveform and loading frequencies, permitting the tester to choose among the various options. This may lead to different M_r Values for the same specimen. In 1994, AASHTO introduced T294-94 based upon the SHRP protocol P-46 as suggested by Claros et al. (1990). It has been reported that the AASHTO T294-94 testing procedure yields more consistent results than the other two testing procedures (Claros, et al. (1990), and Cosentino, et al. (1991)). Mohammad, et al. (1994) reported that the AASHTO T294-94 testing procedure yields higher M_r than those obtained by using the AASHTO T292-91 testing procedure.

As shown in Table 1, the AASHTO T274-82 and T292-91 testing procedures allow various waveform and loading frequencies. Permitting the tester to choose among the various options may lead to different results for the same specimen. In 1992, AASHTO introduced T294-92. This procedure is based upon the SHRP protocol P-46 as suggested by Claros et al. (1990). AASHTO formally adopted this testing procedure for measurement of M_r in 1994, and designated this testing procedure as AASHTO T294-94. It has been reported that the AASHTO T294-94 testing procedure yields more consistent results than the other two testing procedures (Claros, et al., 1990; Cosentino, et al., 1991). Mohammad, et al. (1994) has reported that the AASHTO T294-94 testing procedure yields higher M_r Values than those obtained by using the AASHTO T292-91 testing procedure.

Table 1 Comparison of resilient modulus test procedures(after Kim2004)

Testing Procedure	Wave Type	Load Duration (Sec.)	Cyclic Duration (Sec.)	σ_d (kPa)	σ_3 (kPa)	Number of Cycles
T274-82	Sine, Haversine, Rectangular Triangular	0.1	1.0 to 3.0	7	41, 21, 0	200
				14	41, 21, 0	200
				28	41, 21, 0	200
				55	41, 21, 0	200
				69	41, 21, 0	200
T292-91	Rectangular Triangular	0.1 to 1.0	1.0 to 3.0	21, 34, 48, 69, 103	21	50
T294-94	Haversine	0.1	1.0	14, 28, 41, 55, 69	41	100
				14, 28, 41, 55, 69	21	100
				14, 28, 41, 55, 69	0	100
T307-99	Haversine	0.1	1.0 to 3.0	14, 28, 41, 55, 69	41	100
				14, 28, 41, 55, 69	28	100
				14, 28, 41, 55, 69	14	100

The current AASHTO protocol for determination of resilient modulus of soils and aggregate material (T307-99) is based largely on Long Term Pavement Performance (LTPP) Protocol P46 (AASHTO T294-94). Similarities and differences between LTPP Protocol P46 and AASHTO T307 include the loading system, load cell location, deformation measurement, load and cycle duration, number and type of linear variable differential transformers (LVDTs) to measured axial displacement, specimen size, and compaction procedures are discussed by Groeger et al (2003). Table 2 compares the two standard specification T294-94 (SHRP Protocol P46) and T307. The two procedures have similar load control (closed loop), load cell (external), deformation measurement (external), confining fluid (air), load pulse shape (haversine), specimen L/D ratio (>= 2:1), and the number of LVDTS used. T307 also allows the use of a pneumatic loading system beside the hydraulic one.

Table 2 Comparison of P46 and T307 (after Groeger et al, 2003)

Protocol specification	P46	T307
Type of Loading System	Hydraulic	Hydraulic/Pneumatic
Load control	uses 200 points and not 500 as in P46, and its cycle can have duration of up to 3 seconds; in addition, kneading compaction can also be use as an alternative compaction method. Closed Loop	Closed Loop
Load Cell Location	External	External
Deformation Measurement	External	External
Confining Fluid	Air	Air
Load Pulse Shape	Haversine	Haversine
Load duration	0.1 s	0.1 s
Cycle Duration	1.0 s	1.0 s to 3.0 s
Number of LVDTs	2	2
# of pts per cycle	500	200
Specimen L/D Ratio	>= 2:1	>=2:1
Type of compaction	Static/Vibratory	Static/Vibratory/Kneading

4. Parameters Affecting Resilient Modulus of Fine Grained Soils

M_r is numerically equal to the ratio of the deviator stress to the resilient or recoverable strain after large number of load cycles $M_r = \sigma_d / \varepsilon_r$. The resilient modulus value can be estimated directly from laboratory testing, indirectly through correlations with other laboratory/field tests, or back calculated from deflection measurements the resilient response of a soil has been studied and documented by several researchers over the past 50 years. These studies evaluated the characteristics of M_r for cohesive soils in association with the stress state and engineering properties, and developed procedures for estimating M_r. The results of these studies show that M_r of cohesive soils depends on deviator stress, confining stress, water content, and degree of saturation, plasticity index, unconfined compressive strength, freeze-thaw action, and pore water pressure.

M_r of cohesive soils at constant confining stress decreased nonlinearly with increasing deviator stress (Seed, et al. (1962), Fredlund, et al. (1977), Woolstrum (1990), Drumm, et al. (1990), Li and Selig (1994), Pezo and Hudson (1994), Lee et al. (1995), Mohammad, et al. (1999), Kim (1999), Huang (2001), and Masada and Sargand (2002)). M_r for cohesive soils steeply decreases with an increase in the amplitude of the cyclic load up to a deviator stress, called the 'breakpoint' suggested by Thomson and Robnett (1976). Then with increasing deviator stress, M_r may gradually increase, decrease, or remain constant. M_r of cohesive soils at constant deviator stress increased as the confining stress increased (Pezo and Hudson (1994), Lee et al. (1994), Mohammad, et al. (1999), and Kim (1999)). Kim (1999), and Butalia, et al. (2003) showed that the effect of effective confining stress on M_r of cohesive soils gradually decreases with an increase in the moisture content. However, other researchers have suggested that the confining stress around cohesive soils has no significant effect on the M_r (Fredlund, et al. (1977), Muhanna, et al. (1999), and Masada and Sargand (2002)). The effect of the number of repeated stresses (Seed, et al. (1962) and Raad and Zeid (1990)) appeared to be negligible.

Guan, et al. (1998) suggested a pavement design weight factor that can be calculated on the basis of seasonal changes in M_r obtained from laboratory tests or nondestructive in situ tests. Lee, et al. (1995, 1997) proposed that the unconfined compressive stress at 1% axial strain was a good predictor of M_r for cohesive soils. M_r for some cohesive soils was reported to increase with increasing soil plasticity index (Woolstrum (1990), Pezo and Hudson (1994), and Kim (1999)).

The relationship between M_r and soil engineering properties as well as the stress state of cohesive soils became the foundation for the development of models to estimate M_r of cohesive soils.

Huang (2001) and Butalia et al. (2003) tested fully saturated cohesive soils for resilient modulus characteristics to determine the degradation of resilient modulus due to high pore water pressure buildup. It was observed that the pore water pressure buildup significantly reduced the resilient modulus of saturated cohesive soils

In general, M_r of cohesive soils is nonlinear with respect to deviator stress. The Hyperbolic, GDOT, and UCS models include nonlinear modeling. However, USDA, TDOT, and ODOT models predict linear behavior. Although confining stress can affect M_r of cohesive soils, the effect of confining stress is not considered in Hyperbolic, GDOT, and ODOT models. Also, the ODOT model does not include the effect of deviator stress.

However, most of these models were not developed on the basis of results obtained from M_r testing of a wide variety of cohesive soils. Kim (2006) showed that M_r predicted using three of these regression models, USDA, Hyperbolic, and GDOT models, did not compare well with measured M_r Values for A-4 and A-6 soil samples. In this study, soils from four sites in Baghdad-Iraq are investigated as elaborated in Table 3.

Table 3 Summary of Existing M_r Regression Models in Common Use (after Kim 2004)

Existing Model	Input Parameters	Advantages	Limitations
USDA Model (Carmichael & Stuart, 1986)	USCS soil type, PI, *w*, % passing No. 200 sieve, σ_3, σ_d	Includes effect of: - σ_3 - PI - *w*	- Linear model - Soil type
Hyperbolic Model (Drumm, et al., 1990)	q_u, % of *clay*, PI, γ, S, % passing No. 200 sieve, Hyperbolic parameter *a*, LL, σ_d	- Nonlinear model Includes effect of: - q_u - PI - S	- σ_3 not considered
GDOT Model (Santha, 1994)	w, w_{opt}, γ_d, $\gamma_{d,max}$, % of silt, % of clay,% swell, % passing #40 sieve, S, % shrinkage, LL, PI, σ_d, P_a	- Nonlinear model Includes effect of: - w and w_{opt} - S and PI - P_a	- σ_3 not considered - Complex model - Many tests required
TDOT Model (Pezo & Hudson, 1994)	w, γ_d, $\gamma_{d,max}$, PI, Sample age, σ_3, σ_d	Includes effect of: - *w* - PI - Sample age - σ_3	- Linear model - Input parameters have narrow range
UCS Model (Lee, et al., 1995)	$S_{u\text{ at 1.0\% of axial strain}}$, σ_3, σ_d	- Nonlinear model - Simplicity of Model	- σ_3 at 0, 20.7, 41.4 kPa - 13 kPa < σ_d < 60 kPa
ODOT Model (ODOT, 1999)	GI (% passing No. 200 sieve, LL, PI), CBR	- Simplicity of model	- Linear model - σ_3 and σ_d not considered
OSU Model 2006	q_u, % of *clay*, PI, γ, S, % passing No. 200 sieve, Hyperbolic parameter *a*, LL, σ_d, w, γ_d, $\gamma_{d,max}$, PI, Sample age	Includes effect of: - σ_3 - PI - *w*	- Linear model - σ_3 and σ_d t considered

5. Sample Collection

Representative Cohesive soil samples that are used in pavement subgrade from four sites distributed throughout Baghdad City in Republic of Iraq were collected from a depth of about (0.50to1.5) m. from ground surface elevation to represent Al.Baladiat Site (BB1), Zaiona (BZ1), Al.Kazalia (BK1) and Al.Mansour (BM1).

Laboratory tests were performed on the samples to determine their basic engineering properties. M_r and liquefaction Tests were conducted on soil samples at three different moisture contents which are dry of optimum(DOP), optimum(OPT), and wet of optimum(WOP).

6. Basic Engineering Properties of Used Soil

Laboratory tests were conducted on the four soil samples to determine their basic engineering properties. Laboratory tests conducted were Atterberg limits, sieve analysis, hydrometer, Standard Proctor compaction, unconfined compressive strength, and UU tests. All soil samples collected were transported to the Soil Mechanics Laboratory at The Ohio State University's Department of Civil, Environmental and Geodetic Engineering. The samples were oven-dried at 60 °C, for 24 hours and then air-dried in the laboratory over a two-week period. All dried soil samples were thoroughly pulverized.

According to Unified Soil Classification system in ASTM D2487-93 and AASHTO Soil Classification system in AASHTO M145-91, the soil type for each soil sample was identified on the basis of the results of Atterberg limit, and particle size distribution tests (see Table 4). In the Unified Soil Classification system, as shown in table 4 were found to be classified as CL (low plasticity clay) for BB1, BZ1, BM1 and Bk1.

Atterberg limit tests were performed in accordance with AASHTO T89-96, and T90-96 testing procedures. As shown in Table 4, the liquid limit of A-6 location ranged about 38, and that of A-7-5 locations were much higher (40 to 49). The plasticity index of A-6 group ranged about 17 while it shows higher for A-7-5 which was above 20.

Sieve analyses and hydrometer tests were conducted in accordance with AASHTO T88-97. As shown in Table 4, all soil of A-7-5 had approximately highest percent of Clay (generally ranging from 40% to 50%). The A-6 soil had Clay ranging between 25% and 30%. The A-7-5 soil had the lowest amount of sand.

Table 4 Classification and Engineering Properties of each location

Soil Location	Soil Type AASHTO	Soil Type USCS	Gs	Liquid Limit LL	Plastic Limit PL	PI	Passing #200 Finer	Sand %	Silt %	Clay %	O.M.C %	Max. Dry Density kN/m^3	TSS %
BB1	**A-6**	**CL**	**2.67**	**38.32**	**20.38**	**17.94**	**78.92**	**24**	**49**	**27**	**16.96**	**16.81**	**11.2**
BZ1	**A-7-5**	**CL**	**2.69**	**44.46**	**21.15**	**23.31**	**82.17**	**17**	**37**	**46**	**17.45**	**16.67**	**9.95**
BM1	**A-7-5**	**CL**	**2.68**	**46.41**	**21,04**	**25.37**	**84.26**	**21**	**38**	**41**	**17.21**	**16.23**	**8.51**
BK1	**A-7-5**	**CL**	**2.70**	**45.78**	**18.52**	**26.89**	**88.49**	**19**	**39**	**42**	**17.76**	**15.78**	**10.8**

Standard Proctor compaction tests were conducted on each soil sample in accordance with procedure A in AASHTO T99-97 testing methods as shown in figure 1. Table 4 summarizes the optimum moisture content, maximum dry density, sample moisture content, sample dry density, and unconfined compressive strength for the soil samples for each location.

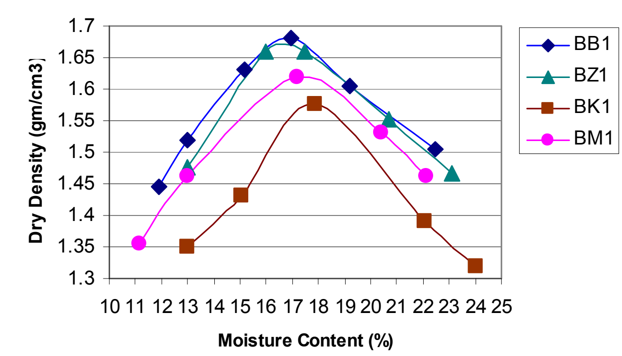


Fig.(1) Moisture Content Vs. Dry Density For each location (BB1,BZ1,BK1 & BM1)

Unconfined compressive strength tests were conducted immediately after sample compaction in accordance with AASHTO T208-96 testing procedures. The unconfined compressive strength tests were conducted on each soil sample at three different moisture contents. As shown in Table 5, the three different moisture contents were dry of optimum moisture content (DOP), optimum moisture content (OPT), and wet of optimum moisture content (WOP).

As shown in Table 5, the unconfined compressive strength for A-7-5 group were found to higher at dry of optimum moisture content, than values obtained from OPT and WOP.In general, the dry of optimum samples exhibited the highest unconfined compressive strength values. The measured strength values typically decreased with increasing sample moisture content.

Table 5Compaction and Unconfined Compressive Strength Test Results

Soil Type	BB1			BZ1			BM1			BK1		
Soil Condition	DOP	OPT	WOP	DOP	OPT	WOP	DOP	OPT	WOP	DOP	OPT	WOP
Unconfined Compression Strength (kPa)	156	139	126	192	176	138	189	169	135	176	162	132

Soil sample for unconfined compression tests was compacted at desired dry, optimum and wet density and moisture content (-2, 0, +2 from optimum) % respectively. it is quite obvious that A-7-5 soil shows good ability to withstand higher stress before failure than A-6 soil. Clearly, saturation adversely affects the unconfined compressive strength of soils compacted at optimum moisture content

7. Evaluation of Resilient Modulus (Testing Procedure)

The major components of M_r testing as performed in the Soil Mechanics Laboratory at The Ohio State University are shown in Figure 2. The specified load was applied by a loading system manufactured by MTS.The Triaxial pressure chamber (see Figure 3) was modified to include a load cell to measure axial load, an LVDT to measure axial displacement. The LVDT was mounted on the external steel rod in the top cover of the Triaxial pressure chamber.

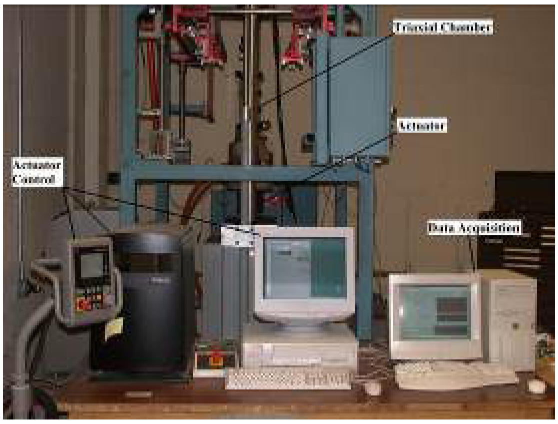


Figure 2 M_r Testing System

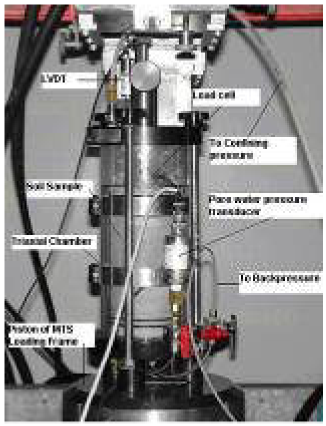


Figure 3 Triaxial Cells for M_r Test

Table 6 M_r Testing Sequences for Unsaturated Samples

Sequence No.	Confining Pressure (kPa)	Deviator Stress (kPa)	Number of load applications
0	41	28	1000
1	41	14	100(95 + 5)
2	41	28	100(95 + 5)
3	41	41	100(95 + 5)
4	41	55	100(95 + 5)
5	41	69	100(95 + 5)
6	21	14	100(95 + 5)
7	21	28	100(95 + 5)
8	21	41	100(95 + 5)
9	21	55	100(95 + 5)
10	21	69	100(95 + 5)
11	0	14	100(95 + 5)
12	0	28	100(95 + 5)
13	0	41	100(95 + 5)
14	0	55	100(95 + 5)
15	0	69	100(95 + 5)

Figures 4, 5,6and 16 show typical results of M_r test on BB1, BZ1, BM1 and BK1 at DOP,OPT and WOP for whole samples. Figures 17, 18 and 19 illustrate the effects of varying deviator stresses and Resilient Modulus Values at different moisture contents.

As shown in Figures 4, 5, 6, and 19, M_r at constant confining stress gradually decreased with an increase in deviator stress. In many cases, the decreasing rate at the low deviator stress was more pronounced than that at high deviator stress. This nonlinear trend of M_r to deviator stress is similar to observations of other researchers (Seed, et al. (1962), Fredlund, et al. (1977), Woolstrum (1990), Drumm, et al. (1990), Li and Selig (1994), Pezo and Hudson (1994), Lee et al. (1995), Mohammad, et al. (1999), Kim (1999), Huang (2001), and Masada and Sargand (2002)). M_r increased with an increase in confining stress. As mentioned previously, it is noted that M_r is closely related to the moisture content in soils. M_r of the soil samples decreased with an increase in moisture content. Kim 2004 and Rodgers 2006 confirmed the same results.

8. Model Verification

The present study uses the neural network approach to develop a model that can be used to predict resilient modulus values for Baghdad Soils and can easily accommodate new data as this becomes available. The model uses the results of commonly performed laboratory tests like water content, Atterberg limits, soil classification and unconfined compressive strength to predict M_r. The network was trained using all laboratory test results performed in the Soil Mechanics Laboratory of The Ohio State University for A-6 and A-7-5

Baghdad soils and the Neural Network Math Works Toolbox.

It is believed that M_r of a cohesive soil is dependent upon its moisture content. To study this phenomenon for the proposed constitutive model, the predicted and measured M_r at various moisture contents (dry of optimum, optimum, and wet of optimum) were investigated. Figures 19, 20, and 21 show comparison of the measured M_r with the predicted M_r for BB1, BZ1, BM1 and BK1 soils, respectively. To prove the capability of the network, M_r predicted values for Baghdad soils were compared with its corresponding M_r measured as illustrated and explained in Figures 19, 20 and 21. It can be observed that as the sample moisture content increases, M_r predicted by the model reduces significantly and is generally close to the experimentally measured M_r, irrespective of the sample moisture content. It can be observed that as the sample moisture content increases, M_r predicted by the model reduces significantly and is generally close to the experimentally measured M_r, irrespective of the sample moisture content. this model was performed previously by Kim (2004) and Rodgers (2006).It is obvious that conducting the Mr test in laboratory on subgrade soil is the best way to get accurate results.

It is concluded that existing M_r prediction models investigated in this study significantly overestimate M_r and show a large scatter of data when compared with experimental observations. The proposed model is generally slightly conservative in its estimation of M_r and hence can be safely used in the design of flexible pavements supported on cohesive soils.

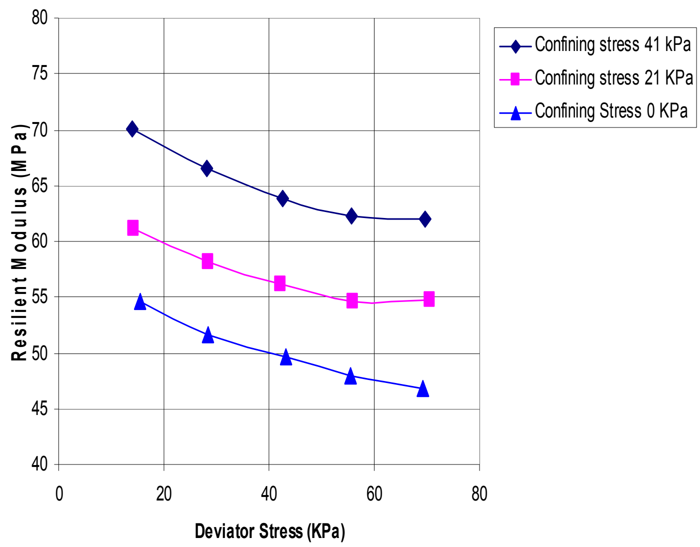


Fig. (4) Resilient Modulus From Mr laboratory test For BB1 Location (DOP)

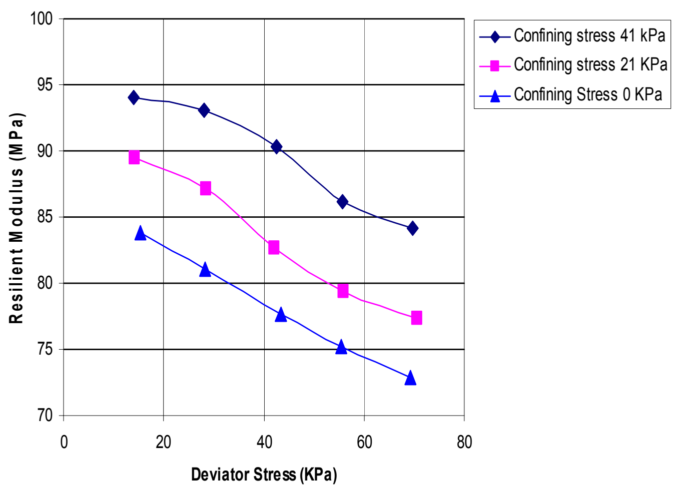


Fig. (5) Resilient Modulus From Mr laboratory test For BZ1 Location (DOP)

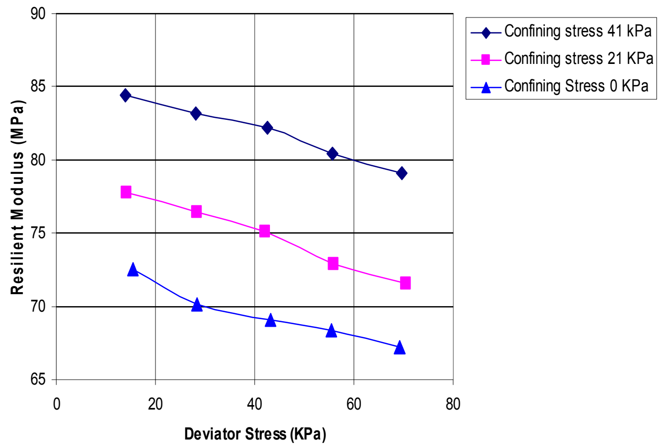


Fig. (6) Resilient Modulus From Mr laboratory test For BM1 Location (DOP)

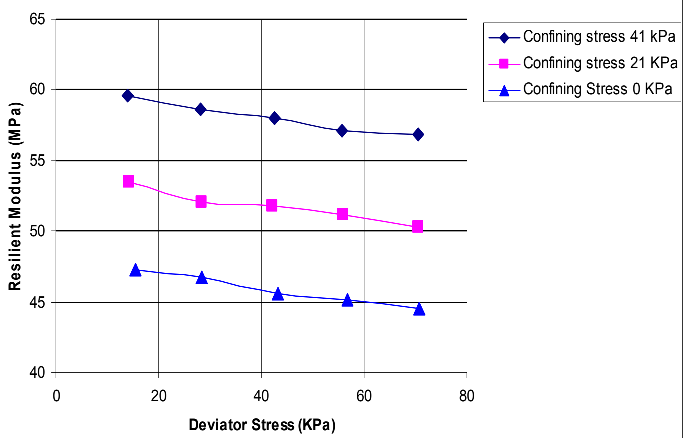


Fig. (7) Resilient Modulus From Mr laboratory test For BK1 Location (DOP)

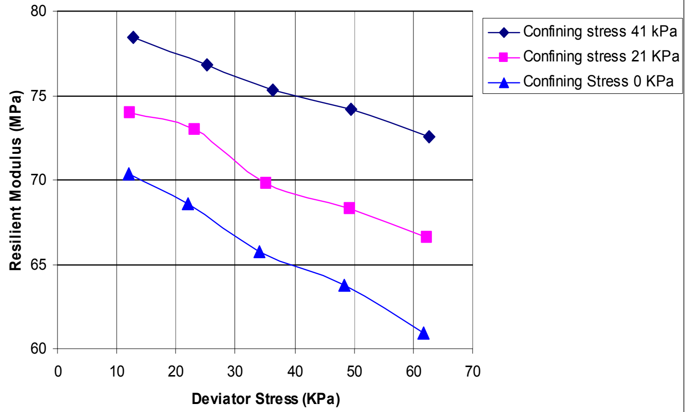


Fig. (8) Resilient Modulus From Mr laboratory test For BZ1 Location (OPT)

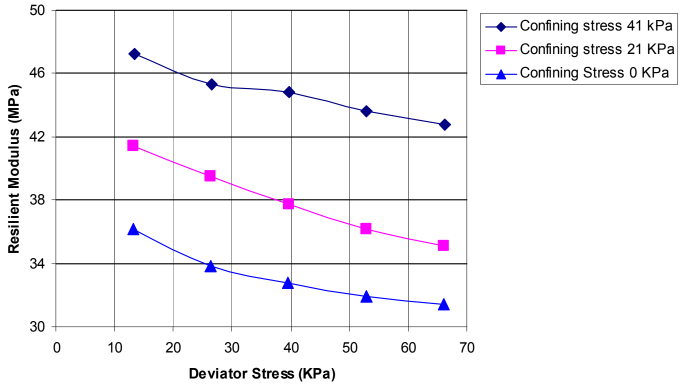


Fig. (9) Resilient Modulus From Mr laboratory test For BB1 Location (OPT)

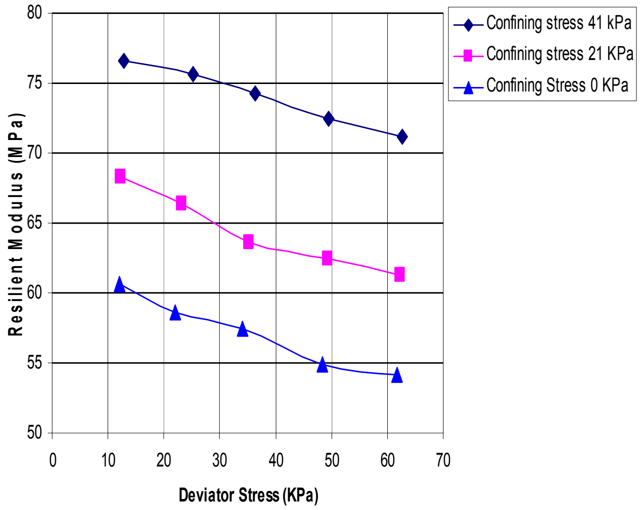


Fig. (10) Resilient Modulus From Mr laboratory test For BM1 Location (OPT)

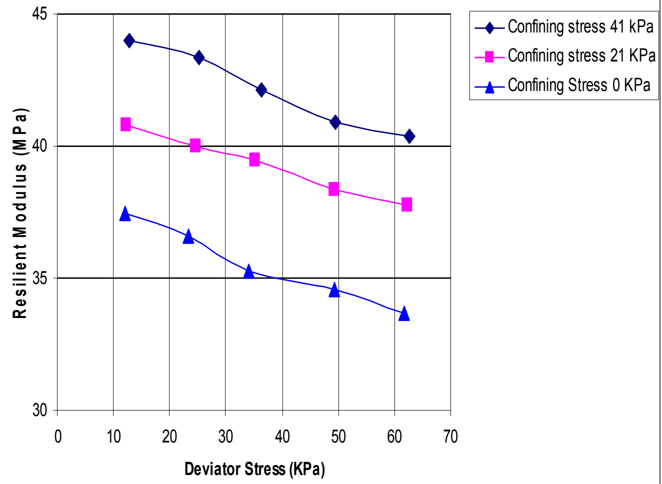


Fig. (11) Resilient Modulus From Mr laboratory test For BK1 Location (OPT)

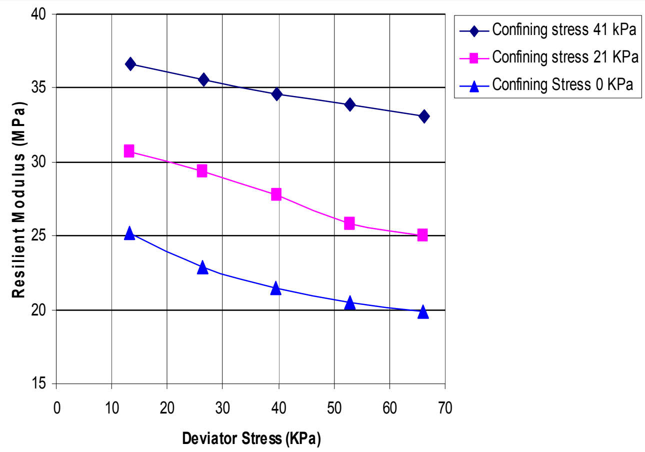


Fig. (12) Resilient Modulus From Mr laboratory test For BB1 Location (WOP)

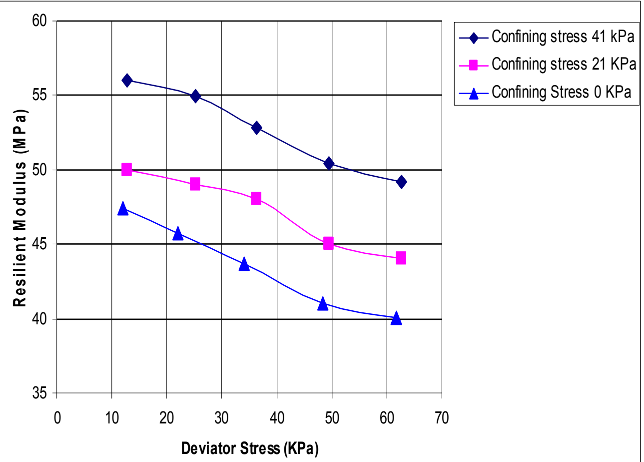


Fig. (13) Resilient Modulus From Mr laboratory test For BM1 Location (WOP)

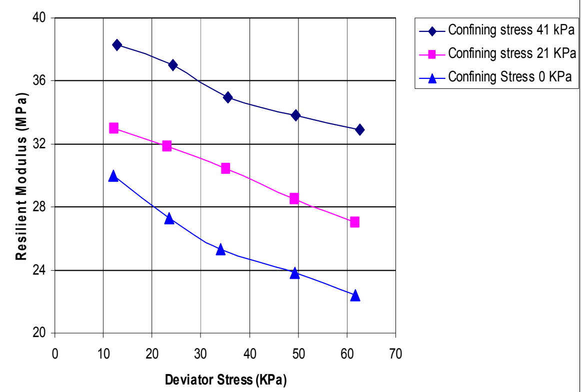


Fig. (14) Resilient Modulus From Mr laboratory test For BK1 Location (WOP)

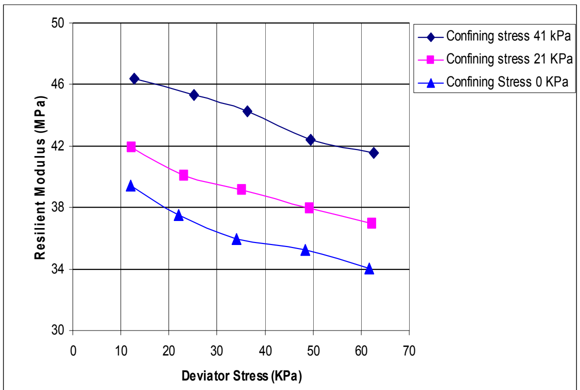


Fig. (15) Resilient Modulus From Mr laboratory test For BZ1 Location (WOP)

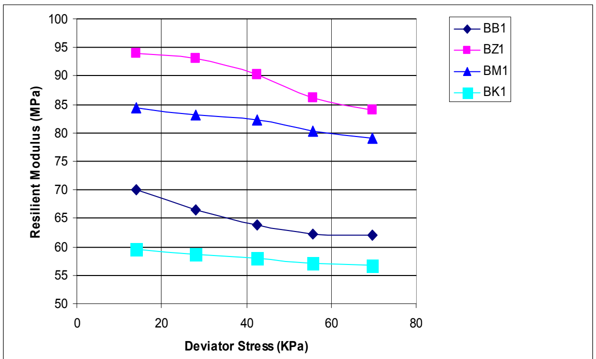


Fig. (16) Resilient Modulus From Mr laboratory test For BB1,BZ1, BM1 Location (DOP) at Confining Pressure 41kPa

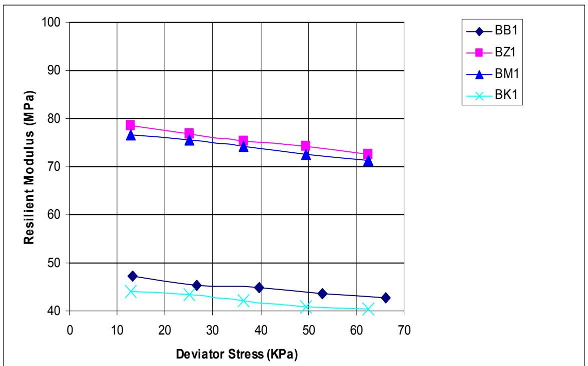


Fig. (17) Resilient Modulus From Mr laboratory test For BB1,BZ1, BM1 Location (OPT) at Confining Pressure 41kPa

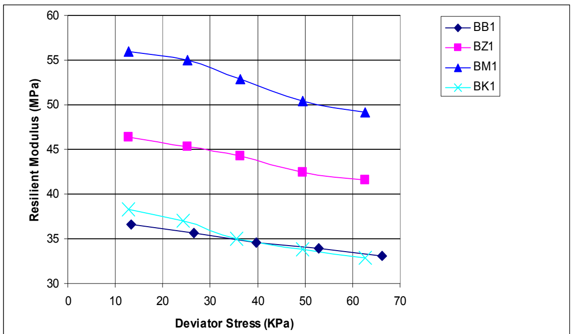


Fig. (18) Resilient Modulus From Mr laboratory test For BB1,BZ1, BM1 Location (WOP) at Confining Pressure 41kPa

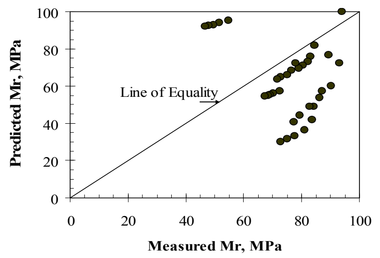


Fig.(19) Measured and predicted Resilient Modolus for all soils at DOP

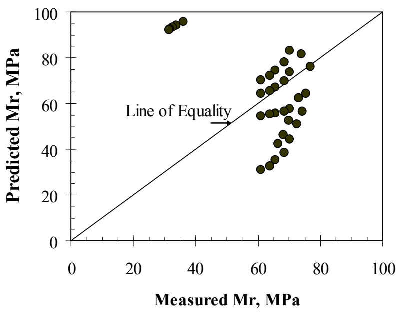


Fig.(20) Measured and predicted Resilient Modolus for all soils at OPT

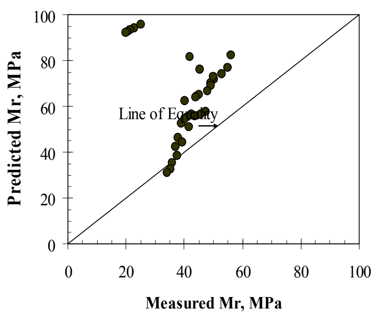


Fig.(21) Measured and predicted Resilient Modolus for all soils at WOP

9. Liquefaction Potenenial of Baghdad Soil (Testing and Results)

Cyclic Triaxial tests were performed to evaluate the liquefaction potential and measured with guidance from the standard test method for load controlled cyclic Triaxial strength of soil (ASTM D 5311) (see Fig.2). The test was carried out on each soil at wet of optimum which considered the most worst condition if there than DOP and OPT conditions. All samples should have be saturated before starting the test, the B – Value of about 0.90 was required to perform a cyclic test. However, if the specimen took longer than 10 days to reach required B-Value, the specimen was tested due to time constraints. The liquefaction test results are presented in table 7. After reaching required level of saturation. To develop cyclic strength curves, confining pressure ranged between 115kPa to 280kPa and cyclic stress ratios between 0.100 to 0.40.The cyclic stress ratio (CSR) is a non dimensional measure of the induced cyclic stress (Kramer,1996).

$CSR = \tau_{cycl.} / \sigma_0$

Table 7 Summary of liquefaction test results on soil samples at WOP

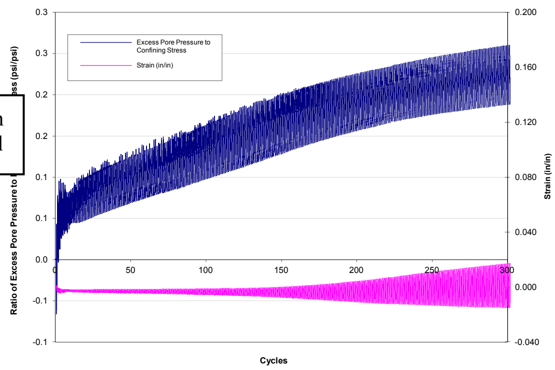
SOIL TYPE	Cyclic Stress Amplitude(psi)	Confining Pressure (psi)	CSR	Cycles to Liquefaction
BB1	7.2	20	0.18	243
BZ1	10.4	20	0.26	DNL
BM1	10.8	20	0.27	DNL
BK1	11.6	20	0.29	DNL

DNL = Did Not Liquefy within 400 cycles

Figures 22, 23, 24 and 25 shows the liquefaction tests results on samples BB1, BK1, BZ1 and BM1.

It could be concluded from test results that there is no precautions for cohesive subgrade should be taken concerning liquefaction.

Fig.(22) Liquefaction test results of A7-5 soil



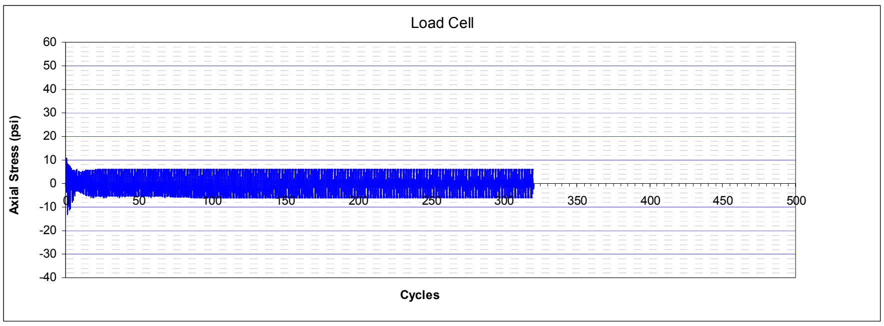


Fig.(23) Liquefaction test results of A6 soil

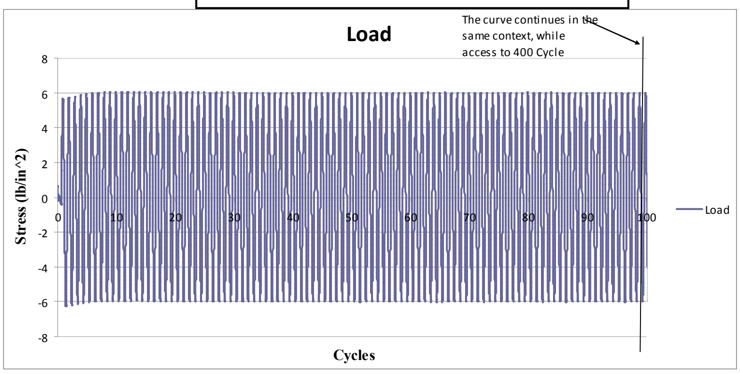


Fig.(24) Liquefaction test results of A7-5 soil

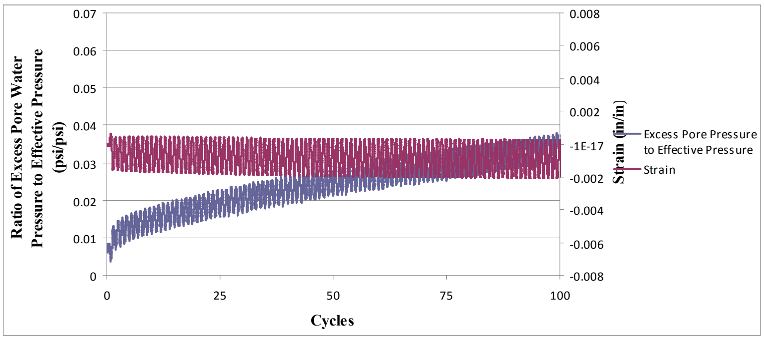


Fig.(25) Liquefaction test results of A7-5 soil

8. Conclusions and Recommendations

Evaluation of Baghdad Soil brought from four locations was well studied to evaluate the resilient modulus and the following conclusions were drawn:

1. The results of all experimental programs show the real need in evaluating the resilient modulus by adopting laboratory methodology.
2. A total collapse of the pavement structure can occur due to large plastic deformations arising in the subgrade soil due to extremely heavy traffic loads.
3. Resilient modulus (M_r) of pavement subgrade soils has been adopted by the American Association of State Highway and Transportation Officials (AASHTO) for the purpose of designing flexible roadway pavement systems for Baghdad City.
4. For natural soils of Baghdad city, all samples exhibited resilient modulus values ranging from 40 MPa to about 100MPa. Based on ASTM subgrade resilient modulus criterion, the A-7-5 and A-6 untreated subgrade soil would be classified as fair to poor (unacceptable as a competent subgrade) (from a resilient modulus criterion perspective).
5. A comparison of the resilient modulus predictions using the OSU model (originally developed for untreated cohesive soils and laboratory measured resilient modulus values shows that most of the predicted resilient modulus values were within the allowable percent error of around ±30 %. For samples prepared at dry of optimum. In particular, all the soil samples were in the allowable range if some Mr Values were ruled out and excluded, the results of predicted Mr Value were very close to the measured value. This validates the applicability of the OSU model to stabilized cohesive soils.
6. Liquefaction condition didn't show conflict values and could be not recommended to conduct this test in study the possibility of acceptance of clay subgrade in site.
7. It is recommended to make some modifications on OSU model to be used and predict all values of resilient modulus for all location in Baghdad City which lead to find out the most reliable formulas to depend on in evaluating Mr.

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