SANDSTONE CHARACTERISATION THROUGH NON-DESTRUCTIVE TEST TECHNIQUES

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ABSTRACT: Rock characterization is one of the most important aspect in rock mechanics. The non-destructive tests are being increasingly used for easy, fast and economical characterization of rock mass. The elastic wave velocities and rebound hammer tests are commonly employed non-destructive tests. The sandstones are extensively encountered in India for number of infrastructure projects and mining works. Therefore, a study of its geotechnical characteristics is imperative in evaluating their response to various in-situ stress and environmental conditions. This paper presents assessment of strength and deformation behavior of sandstones of central India through non-destructive tests. The analysis of data indicates that non-destructive tests can be confidently employed for assessment of geotechnical response of rocks particularly for preliminary project planning and designing purpose.

1 INTRODUCTION

In fast developing country like India the need of large-scale structure resting on or in rocks like tunnels, dams, underground caverns, etc. is ever increasing. This emphasises the need to carry out rapid, economical and easy subsurface exploration and material characterization for analysis and design of such structures. There is growing awareness among geotechnical engineers to employ geophysical methods for site investigations duet to their rapidity, non-destructive nature and cost effectiveness.

The sedimentary rocks such as sandstone, limestone and shale occur prominently in India. They are extensively encountered during various mining, tunnelling and civil engineering construction activities. In terms of area of exposure, the sedimentary rocks constitute about 75% of all rocks. Further the three principal sedimentary rocks viz. sandstone, limestone and shale together constitute over 80% of all sedimentary rocks. In these rocks also sandstone has very high proportion. The sandstones exhibit several verities of structure and texture and are considered as problematic rocks due to large variations in their physical and geotechnical properties. The rational design and construction of structures in such rocks requires a proper evaluation of their properties and realistic assessment of their engineering behavior under different conditions. This requires planning and execution of elaborate, expensive and time consuming experimental programme.

Such elaborate, expensive and time consuming experimental programme may not be feasible at the planning and preliminary design stage of a project. Hence various attempts have been made and techniques developed to assess the geotechnical characteristics and response of rock mass through simple, less time consuming, easy to conduct and economical tests. The collection of sufficient rock samples is tedious, costly and time consuming particularly at preliminary project stage. To take care of these aspects various techniques are employed in which sample is not destroyed. Such tests are commonly referred as non-destructive tests.

The elastic wave velocities and Schmidt rebound number techniques are the non-destructive methods which can be employed in field for insitu measurements as well as in laboratory for measurements on rock specimens obtained from the field. The uniaxial compressive strength (σ_c) and modulus of elasticity (E) are two most significant engineering parameters; which are invariably required for any design of structure by the rock, in the rock and on the rock

However, in order to make use of these non-destructive tests in predicting σ_c and E it is imperative to develop rational, reliable and meaningful correlations among them. This paper presents is an significant attempt in this direction.

2 LITERATURE REVIEW

Various investigators have worked to develop interrelationship between strength parameters such as compressive strength (σ_c), tensile strength (σ_t), shear strength (τ) etc. and deformation parameters such as modulus of elasticity (E), Poisson’s ration (ν), shear modulus (G) etc. and non-destructive test parameters such as Compressional wave velocity (v_p), shear wave velocity (v_s), and rebound hammer index (R) etc. A brief review of these non-destructive tests and their available interrelationship with most significant engineering parameters i.e. uniaxial compressive strength (σ_c) and modulus of elasticity (E) is presented below.

2.1 Elastic Wave Velocities

There is awareness among geotechnical engineers to employ geophysical methods for site investigation due to non-destructive nature; rapidity and cost effectiveness of these methods. The seismic refraction is one such geophysical method which is nowadays extensively used for rock characterization. The field seismic refraction sounding data provide in-situ seismic wave velocities v_p and v_s of rock formations at different depths. Such data can further be interpreted to identify various rock strata and their depths so as to prepare subsurface profile. However, the potential of this method when used only for obtaining such subsurface profiles seems to be very much underutilized. The literature study does indicate that it may be possible to convert these in-situ seismic wave velocities into certain parameters required for analysis and design of engineering structures. These velocities can also be determined in laboratory and are commonly referred as elastic wave velocities.

Number of researchers reported increase in σ_c with increase in v_p. On the basis of 15 Eastern Indian Coalfields sandstones; Ramachandran and Narasimham (1987), reported following bilinear relationship σ_c with v_p as given in Equation 1.

$$\sigma_c = 7.21v_p - 6.43 \text{ (For } \sigma_c < 18 \text{ MPa)} \quad \text{and} \quad \sigma_c = 60.32v_p - 178.4 \text{ (For } \sigma_c > 18 \text{ MPa)} \tag{1}$$

On the basis of studies on 11 sedimentary rocks (majority sandstones); 10 metamorphic rocks and 3 igneous rocks Kate and Sthapak (1995) proposed the Equation 2 relating v_p and σ_c.

$$\sigma_c = 37.0v_p - 64.0 \text{ (For dry rocks)} \quad \text{and} \quad \sigma_c = 35.0v_p - 85.0 \text{ (For saturated rocks)} \tag{2}$$

On the basis of results of various sandstones, limestones, basalts and intrusive rocks Golodkovskya and Shaurnian (1970) proposed the following correlation between v_p and σ_c in non-dimensional form as given in Equation 3 in which σ_{cm} is the maximum uniaxial compressive strength (200 MPa) and V_{pm} is the maximum compressional wave velocity (6.5 krn/s).

$$(1 - \sigma_c/\sigma_{cm})^2 = 1-(v_p/v_{pm})^2 \tag{3}$$

The modulus of elasticity (E) is the most significant parameter characterizing the deformational behaviour of rocks. It can be obtained through conventional compression tests and is referred as static elastic constants E_s. It can also be obtained through theory of elasticity equations by knowing v_p, v_s and unit weight (γ) of the rock and is referred as dynamic elastic constant E_d.
Numerous studies indicate that E_d differs from E_s. Generally, E_d is observed to be higher than E_s. In some cases, the E_d/E_s ratio may go up to or beyond 3 or so. This is mainly because dynamic methods involve low stresses. However, since in design problems the E_s, is preferred over E_d being more representative of actual loading conditions; hence considerable attempts have been made to relate E_s with E_d or with v_p.
On the basis of 342 data sets for different rocks, Eissa and Kazi (1988), reported linear relationship between E_s and E_d. They further reported that the inclusion of rock unit weight (γ) improves the value of coefficient of correlation (r) and accordingly for above data they proposed the best fit power law as given in Equation 4.

$$E_s = 1.05[(\gamma/g)\ Ed]^{0.77} \qquad (r = 0.96) \tag{4}$$

Whiteley (1983) reported linear variation (a band) between E_d and E_s, on log-log plot again indicating power law relation. He further observed that E_d exhibits non-linear (power law) relationship with E_s because the E_d varies with v_p^2. He also reported plot of E_s with V_p on log-log scale indicating curvilinear increase (a band) in E_s with v_p, leading to approximate power relationship of E_s, with vp as presented in Equation 5 below; where a is a curve fitting parameter.

$$E_s = av_p^3 \tag{5}$$

2.2 Rebound Hammer Index

The Schmidt rebound hammer test is simple and quick non-destructive test originally developed for estimating compressive strength of concrete through surface hardness. Although it is being used in rock mechanics investigations, nevertheless its potential is not yet fully exploited for estimating/interpreting strength and deformation behaviour of rocks. The Schmidt rebound hammer test is increasingly becoming obvious choice for geotechnical engineers to estimate strength and deformation of rock with the help of Schmidt rebound number with fair degree of accuracy. A number of empirical equations have been proposed in literature for estimating σ_c and E_s on the basis of Schmidt rebound index (R). Some of the available correlationships are summarised in Table 1. From Table 1 it can be observed that majority of correlations of R with σ_c and E_s are nonlinear being exponential form or power law.

Table 1 Correlationship of Schmidt Rebound Hammer Index (R) with Uniaxial Compressive Strength (σ_c)

Sl. No.	Correlationships	Rock Type/Number	Reference
1	$\sigma_c = 6.9\text{x}10^{[0.16+0.0087(R\gamma)]}$ $E_s = 600.5(R\gamma)-20276$	Different rocks from 28 locations	Deere and Miller (1966) Reported by Haramy and DeMarco (1985)
2	$\sigma_c = 6.0\text{x}10^{[1.348\log(R\gamma)-1.325]}$ $E_s = 6.9\text{x}10^{[1.961\log(R\gamma)+1.861]}$	Different rocks from 168 locations	Aufmuth (1973)
3	$\sigma_c = 1.897\text{x}10^{-4}R^{3.45}$ $\log_e(E_s) = 0.092R+5.456$	Granite of different weathering grades	Irfan and Dearman (1978)
4	$\sigma_c = 12.74e^{0.0185(R\gamma)}$ $E_s = 192(R\gamma^2)-12710$	Different rocks from 48 locations	Beverly et. Al. (1979) Reported by Xu et.al. (1990)
5	$\sigma_c = 0.447e^{(0.045R+\gamma)}$	Different rock types	Kidybinski (1980) Reported by Haramy and DeMarco (1985)
6	$\sigma_c = 0.944R - 0.383$	Different coals from 10 locations	Haramy and DeMarco (1985)

7	$\log_e(\sigma_c) = 0.043R\gamma+1.2$	Different carbonate rocks from 6 locations	Cargill and Shakoor (1990)
8	$\sigma_c = e^{(aR+b)}$ $E_s = 10^{-3}e^{(cR+d)}$ a, b, c and d are function of rock type	Five rock types	Xu et.al. (1990)
9	$\log_{10}(\sigma_c) = 0.0054R\gamma + 1.25$	Different rocks from 24 locations	Kate and Sthapak (1995)

The critical examination of these correlationships indicated that there is considerable difference in predicted σ_c and E_s, by various equations. Similar observation was also reported by Xu et al. (1990). This indicates that the generalized correlationships of R with σ_c and E_s may lead to erroneous prediction for number of rock types. Hence it will be appropriate to develop correlationships between R and engineering parameters of rocks on the basis of some grouping. The rock lithology may be one of the appropriate basis for such grouping rocks. Some such relationships for specific rocks of the basis of lithology are available; however, these are based on limited data.

3 EXPERIMENTAL INVESTIGATION

In order to understand physical, geophysical and geotechnical behaviour of sandstones and to develop correlationships between strength and deformation characteristics and index and non-destructive test parameters elaborate experimental investigation programme was planned and conducted on 8 sandstones obtained from central India from states of Uttar Pradesh, Madhya Pradesh and Rajasthan .

The physical characteristics were assessed both at microscopic and macroscopic level. In order to identify the rock forming constituents, the studies pertaining to mineralogy, petrography and chemical analysis of the rocks are very much essential. Keeping this in view various tests/studies such as X- ray diffraction analysis, the studies of thin sections under microscope at magnification of x20, scanning electron microscopy and chemical analysis was conducted following conventional and standard procedure.

The macroscopic physical properties such as specific gravity, dry & saturated unit weights, effective & total porosities and water absorption have been determined in the laboratory for all the rock types following the procedure as per ISRM (1979). The physical properties tests (other than specific gravity) were conducted on rock specimens having diameter of 38 mm and length to diameter ratio (L/D) ratio of 0.5, 1.0, 1.4 and 2.0.

The non-destructive tests such as elastic wave velocities and rebound hammer index and simple to conduct index tests such as void index, slake durability index and water evaporation index tests were also conducted following standard procedure available in literature.

In order to assess geotechnical behaviour; various strength tests such as Brazilian tensile strength test, punch shear strength test, axial and diametral point load test and uniaxial compression test tests were conducted on dry and saturated rock specimens. In order to assess deformation behaviour of these rocks the axial and diametral strains were also measured during uniaxial compression using four strain gauges and values of modulus of elasticity and Poisson's ratio were also obtained.

In addition to data obtained from above elaborate experimentation the similar available data from published literature was also incorporated; in order to develop interrelation of strength and deformation characteristics with index and non-destructive test parameters.

4 RESULTS AND DISCUSSION

The typical experimental data are presented and analyzed along with similar data available in literature and the interrelations of geotechnical characteristics such as strength and modulus of elasticity with non-destructive test parameters such as seismic wave velocities, Schmidt rebound number and electrical resistivity are obtained and presented herein.

4.1 Location, Geology and Mineralogy

In present study is carried out on 8 sandstones, 3 sandstones each belongs to locations in Madhya Pradesh and Uttar Pradesh and 2 sandstones to Rajasthan. Geologically while 6 sandstones belong upper Proterozoic age and Vindhyan super group, 1 sandstone each belongs to middle Proterozoic age and Vindhyan super group and upper Proterozoic age and Gondwana sequence respectively. For brevity the sandstones are referred by 3 letter symbols. The first letter belongs to village, second to district and third letter S indicates it is Sandstone. For example, AKS stands for sandstone collected from Aroli village in Kota district and so on.

4.2 Physical Properties

The values of various physical properties viz. specific gravity (G), dry unit weight (γ_{dry}), saturated unit weight (γ_{sat}), total porosity (n_t), effective porosity (n_e) and water absorption (saturated moisture content, m_s) are reported in Table 2. The specific gravity for all 8 sandstones falls in narrow range of 2.62 to 2.78. The porosity for sandstones studied ranged from 5.03% to 14.25%

Table 2 Physical Properties of Rocks Studied

Type of Sandstone	G	γ_{dry}	γ_{sat}	n_t	n_e	m_s
		KN/m^3	KN/m^3	%	%	%
AKS	2.63	22.90	23.27	8.36	12.83	3.64
MBS	2.62	22.80	23.53	7.54	11.28	3.21
SAS	2.66	23.40	24.00	6.07	10.31	2.54
BMS	2.68	23.90	24.40	5.03	9.06	2.06
BLS	2.70	21.67	22.92	12.77	18.19	5.78
DSS	2.78	21.62	23.01	14.25	20.71	6.46
SDS	2.76	23.15	24.16	10.25	14.48	4.35
GGS	2.72	23.50	24.16	6.70	11.93	2.81

4.3 Non-Destructive Test Properties

The values Compressional wave velocity (v_p) and shear wave velocity (v_s) were evaluated in order to understand geophysical behaviour of sandstones. The values obtained are presented in Table 3. Also presented are values of Rebound number ® and values of dynamic elastic constants E_d and ν_d obtained on the basis of v_p and v_s. From this table it can be seen that the value of vp ranges from 1.66 km/s for DSS to 3.50 km/s for SAS. The value of R for the 8 sandstones studied varied from a low of 20 for DSS to a high of 55 for SAS. A close look at data presented in Table 3 indicate that while values of v_p are observed higher in saturated conditions; the v_s is lower in saturated condition.

Table 3 Non-Destructive Test Properties for Sandstones Studied

Rock Type	V_p, km/s		V_s, km/s		R		E_d, GPa		ν_d	
	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.
AKS	3.42	3.86	1.79	1.64	46	44	19.21	15.75	0.31	0.39
MBS	3.38	3.99	2.12	2.01	55	52	24.45	23.24	0.18	0.33
SAS	3.50	3.89	2.12	1.95	59	55	26.18	24.42	0.20	0.33
BMS	3.66	4.18	1.99	1.87	47	45	24.62	24.07	0.29	0.30
BLS	2.92	3.32	1.43	1.11	38	35	12.10	8.29	0.34	0.44
DSS	1.66	2.01	1.01	0.90	24	20	5.38	5.07	0.20	0.37
SDS	3.17	3.46	1.56	1.41	43	36	15.43	13.85	0.34	0.40
GGS	3.25	3.61	1.74	1.64	44	41	18.72	17.58	0.30	0.37

4.4 Strength and Deformation Properties

The values of, diametral point load strength (σ_{pd}), Brazilian tensile strength (σ_t), punch shear strength (τ_p), uniaxial compressive strength (σ_c), static modulus of elasticity (E_s) and static Poisson's ratio (ν_s) for the 8 sandstones studied are presented in Table 4 for both dry and saturated conditions.

Table 4 Strength and Deformation Properties of Rocks Studied

Rock Type	σ_{pd}, MPa		σ_{tb}, MPa		τ_p, MPa		σ_c, MPa		E_s, GPa		ν_s	
	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.	Dry	Sat.
AKS	4.94	2.94	8.01	5.14	17.84	13.27	78.49	54.14	11.80	6.94	0.21	0.29
MBS	7.50	5.40	10.25	6.80	23.77	16.21	101.98	6974	10.15	7.79	0.15	0.28
SAS	8.59	5.64	9.59	6.71	17.66	13.79	109.84	90.27	12.13	10.53	0.14	0.20
BMS	7.26	3.78	9.06	4.30	18.70	9.19	87.49	59.30	11.12	10.65	0.16	0.19
BLS	5.10	3.28	4.87	3.77	7.33	5.05	57.76	47.54	9.36	8.51	0.20	0.24
DSS	0.74	0.33	1.44	0.59	2.65	1.28	15.71	8.35	2.10	1.08	0.22	0.34
SDS	4.88	3.01	5.31	3.08	9.39	5.66	62.07	41.83	11.87	5.22	0.22	0.35
GGS	7.01	2.52	7.95	3.89	13.60	5.94	77.24	57.07	9.76	6.96	0.13	0.27

The data given in Table 4 indicates that σ_c for the sandstones studied ranges from a low of 15.71 MPa for DSS to a high of 109.84 MPa for SAS. All sandstones indicate significant reduction in σ_c upon saturation. The data in Table 4 also indicate that values of E_s are higher in dry condition while that of ν_s are higher in saturated condition.

5 CORRELATIONSHIP BETWEEN STRENGTH AND NON-DESTRUCTIVE TEST PROPERTIES

5.1 Elastic Wave Velocities

With a view to establish σ_c-v_p correlationship specifically for sandstones analysis of data obtained from present study and that available in literature (sandstone 58 data sets) was carried out. The plot of σ_c/σ_{cm} with v_p/v_{pm} for sandstone is presented in Figure 1. The values of σ_{cm} and v_{pm} and coefficient of correlation (r) are also indicated in the figures. The value of r = 0.77 is moderately significant. In order to develop equations for prediction of E_s on the basis of values of v_p and E_d, regression analysis of data available in literature and that obtained in present study was for sandstone and the results presented in Table 5.

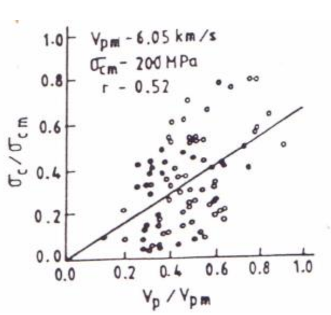

Fig. 1 Correlation of σ_c with v_p for Sandstones

Table 5 Correlationship of E_d with E_s and v_p for Sandstones

Form of Correlation Equation	n	A	B	r	n
$E_s = A+BE_d$	24	5.46	0.65	0.80	50
$E_s = A(E_d)^B$	24	2.16	0.71	0.79	50
$E_s = A(\rho E_d)^B$	19	1.42	0.68	0.80	29
$E_s = A(v^p)^B$	58	1.89	2.24	0.98	61

The data given in Table 5 shows the relationship between E_s and v_p is excellent with r greater than 0.98. clearly indicating that E_s can directly be predicted from v_p values.

5.2 Schmidt Rebound Number

In order to develop more meaningful correlations of Schmidt rebound number (R) with σ_c and E_s; analysis of present and available experimental data was carried out for various sedimentary rocks. The analysis was made for sandstone, limestone, shale and arenaceous; calcareous and argillaceous sedimentary rocks excluding sandstone, limestone, and shale respectively. As literature indicated that more meaningful relationships are obtained by incorporating rock unit weight (γ); same is adopted in present analysis. The obtained correlationships of product $R\gamma$ with (σ_c and E_s) are presented in Equation 6 and 7 respectively. The number of data points (N) and value of correlation coefficient (r) are also indicated in the equation. The correlationships given in Table 6 are equations of best-fit obtained through regression analysis. In the equations σ_c. is in MPa, E_s, in GPa and γ in KN/m^3.

$$\sigma_c = 0.100(R\gamma)^{0.974} \qquad (r = 0.88, N=50) \qquad (6)$$

$$E_s = 0.0329(R\gamma) - 9.309 \qquad (r = 077, N=40) \qquad (7)$$

For correlationships with σ_c value of r is above 0.88 indicating significant correlation between them. For E_s, correlationships also r is observed to be of 0.77.

6 CONCLUSIONS

(1) The paper presents the study on use of non-destructive tests for characterization of sandstones. The paper presents prediction of uniaxial compressive strength and Modulus of elasticity through non-destructive properties viz. Compressional wave velocity and rebound hammer number for sandstones.

(2) The non-destructive parameters show promising correlationships with strength and deformation parameters. The Schmidt rebound hammer index can be used to predict strength and deformation behaviour of rocks.

(3) The generalized correlationships of rebound hammer number with uniaxial compressive strength and modulus of elasticity for sandstones show significant correlationships.

(4) The compressional wave velocity shows very strong correlationships with both with uniaxial compressive strength and modulus of elasticity, clearly indicating that the compressional wave velocity is an excellent non-destructive parameter for prediction strength and deformation parameters for sandstone characterization.

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