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GEOTECHNICAL ASSESSMENT OF NATURAL SLOPES AND VALLEYS BASED ON REAL TIME RAINFALL DATA

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ABSTRACT: In monsoon periods, slope failures are major natural disasters in western Japan where most of the areas are covered with weathered Granite (Masado) or weathered Rhyolite. The present hazard assessment system available to predict such catastrophic events in the region is not adequate for early evacuation purposes as it is not based on any reliable geotechnical assessments. To incorporate geotechnical inputs and to enhance present risk assessment, this study was proposed and initially a method to determine shear strength parameters was established. Authors have been already established a sound methodology and relationships to determine shear strength parameters for weathered Granite and the present study focuses to develop the same for weathered Rhyolite and to propose a method to evaluate real time factor of safety based on real time rainfall data. A series of laboratory model tests based on lightweight dynamic cone penetrometer and direct shear tests were conducted on weathered Rhyolite collected at Shobara city, Japan. Good graphical relationships were developed with cone resistance, q_d and void ratio e, void ratio e and apparent cohesion c_{d}, and void ratio e and internal friction angle ϕ_{d}. Stability analyses were conducted considering shear strength parameters and the water table estimated based on real time rainfall data. It was found that the time of occurring slope failures are well evaluated from the proposed method logy from this study. This study can be extended to evaluate slope failures in forecasted rainfall conditions.

Keywords: Cone Resistance, Shear Strength, Rainfall, Slope Stability

1. INTRODUCTION

One of the primary factors influencing the instability of slopes is water. Heavy rainfall trigger the majority of slope failures due to infiltration of water into the soils which results in reduction of shears strength and development of water table. Over the years, it has been reported that a large number of slopes failures occurred under heavy rainfall conditions elsewhere in the world. In last few decades, the magnitude of the areas affected due to slopes failures in western Japan was not proportional to amount of rainfall or rainfall patterns. Recently, a series of calamitous events was occurred on 20th August 2014, on locations about 15 km away from Hiroshima city, which led to a loss of 74 human lives, and fully destroy 133 houses. Due to this event, a large number of people was injured and lot of properties were partially damaged. Geologically, most of these areas covered with weathered remnants of Granite or Rhyolite. A part of the map extracted from GeomapNavi [1], is shown in Fig. 1 and it illustrates the distribution of geological formations in and around Hiroshima prefecture.

At present, about 21,943 natural slopes, and 9964 valley slopes are found to be susceptible to landslide disasters in Hiroshima Prefecture, the highest number of any prefecture in Japan [2]. It is

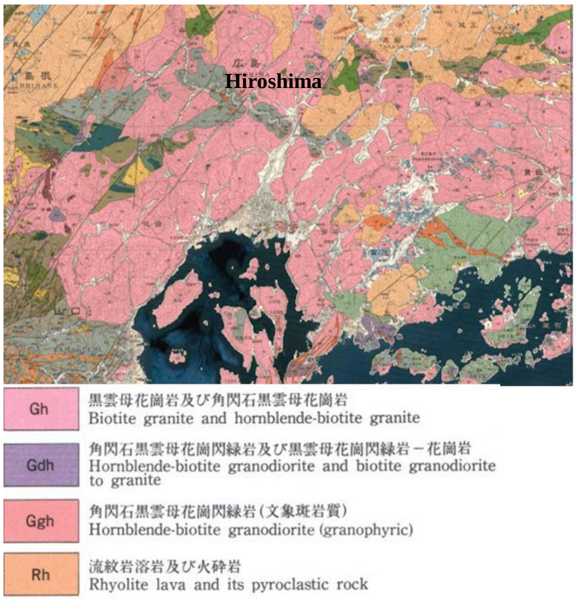


Fig.1 Geology in and around Hiroshima prefecture, Japan[1]

6.2% of the hazardous zones identified at 47 prefectures. The correct understanding of the mechanism of a landslide is important for disaster mitigation. Knowing such event prior to occurrence is much useful for protecting human lives and the environment as a whole. In order to forecast slope failures in the region, Hiroshima prefectural government has already developed a risk assessment system based on rainfall data and past records of failures. However, well documented past literatures reveal that slope failures occur during heavy rainfall conditions are due to weakening of shear strength of soils [3],[4]. Also, slope failures do not occur in direct proportion to precipitation quantity, and in some years a given quantity produces many landslides while in other years the same quantity produces a few failures [5]. Due to these facts, it was understood that the existing risk assessment system is not adequate for correct prediction of failures in the region. Moreover, the present system was found to be developed without carrying out any reliable slope stability analyses. Therefore, it is extremely important to enhance the present risk assessment system in terms of proper geotechnical inputs for better understanding of slopes vulnerability to failure in the region. Therefore, the researches of Geotechnical Engineering laboratory of Hiroshima University, Japan had initiated to examine the geotechnical properties of soils in Hiroshima prefecture for the purpose of use it for the enhancement of hazard assessment system. As a result of that, a sound method to investigate natural slopes covered with weathered Granite was established. Further, relationships were established to determine the shear strength parameters in terms of cone resistance obtained from Lightweight Dynamic Cone Penetration Tests (LWDCPTs). The detailed information of the procedure and outcomes are available in the literature published by Tsuchida et al. [6], [7], and Athapaththu et. al [8], [9].

In this research, same methodology was adopted to examine and to develop relationships for shear strength parameters for weathered Rhyolite, which covers considerable area in Hiroshima prefecture as shown in Fig 1. The experiment setup includes, a series of laboratory experiments to determine physical and shear strength properties, calibration tests based on LWDCPT, and a number of insitu LWDCPTs. Further, a comparison was carried out to examine the similarities and the differences of the two soils derived from Granite and Rhyolite. The research was then extended to analysis a few slopes covered with both soils for real time rainfall data. With the outcomes of this research, it is possible to take necessary steps in enhancing the present hazard system for better prediction of slope failures in Hiroshima, Japan. Proceeding sections explain the basic engineering properties of both soils and shear strength parameters under different void ratios and degrees of saturation.

2. ENGINEERING PROPERTIES OF SOILS

2.1 Sample Preparation and Basic Physical Properties

Soils derived from Rhyolite which were collected from Shobara, Japan were used for laboratory tests. Around 300 kg of soils were transported to the laboratory and soils passing through 2mm sieve were used for laboratory tests. The soil composes variety of particles ranging from cobbles to fine silts. Fig. 2 shows the grain size distribution curve for the particle size less than 2 mm. In the same plot, grading curve for weathered Granite (Masado) is shown for the easy comparison of two soils. It can be seen that both soils follow the same shape nearly up to 20% of percentage passing. According to the Japanese Geotechnical Society (JGS) standards [10], the composition of sand, silt and clay are 70%, 20%, 10% and 70%, 30%, 0% for weathered Granite and weathered Rhyolite, respectively. Rhyolite soils do not compose of clay particles where as weathered Granite composes of about 10% even though the coarse and fine contents are same for both soils. Table 1(a) presents some of the physical properties of both soils. Table 1b shows some of the properties that determined from undisturbed soil samples collected from time to time in various locations inside Hiroshima, Japan .

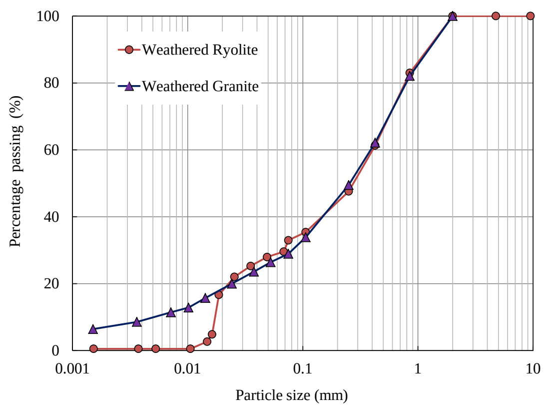


Fig. 2 Grading curve for weathered Rhyolite and weathered Granite for particles less than 2 mm

Table 1a Properties of soils having maximum particle size less than 2 mm

Property	Weathered Rhyolite	Weathered Granite
Specific Gravity, G_s	2.71	2.58
Max. void ratio, e_{max}	1.63	1.36
Min. void ratio, e_{min}	1.03	0.78

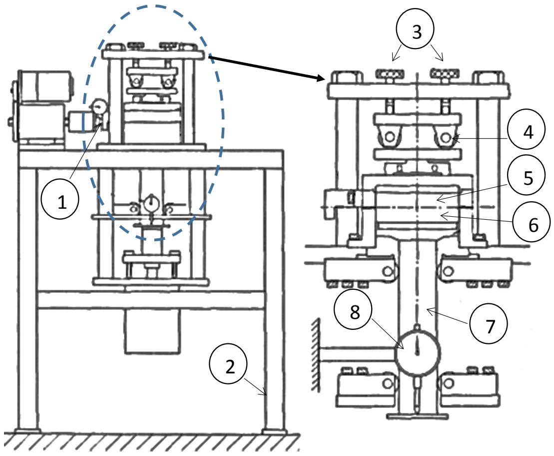
Table 1b Some properties of natural soils (undisturbed samples)

Property	Weathered Rhyolite	Weathered Granite
Void ratio, e	0.95-1.26	0.71-1.14
Hyd. conductivity, k, (cm/s) *10^{-3}	2.3-5.0	1.3-4.7

2.2 Laboratory Direct Shear Tests and Shear Strength Properties of Soils

As mentioned in section 2.1, soils collected from natural Rhyolite slopes were passed through 2 mm sieve for the laboratory direct shear and tri-axial tests. A series of consolidated drained tests was conducted on samples prepared for unsaturated weathered Rhyolite using a direct shear tests apparatus which was loading from bottom of the specimen. The apparatus is shown in Fig. 3. Remolded specimens were prepared for different void ratio ranging from 0.8 to 1.0 in 0.1 increments. The degree of saturation varied from 40% to 80% in 10% intervals for all cases in void ratios. Soils were mixed with required amount of water and light compaction was given in preparation of samples inside the direct shear mold. The specimen size was 6 cm in diameter and 2 cm in thickness. The upper half of the shear ring box was moved by the motor connected to it in constant shear rate of 0.2 mm/ min as per the JGS standards [11]. The lower part of the shear ring box was loaded through a loading ram and vertical stress with vertical displacement were measured through transducers connected to the data logger. Also shear stress and shear displacement were measured in similar manner in data logger. At least 3 samples were prepared for a particular void ratio, and degree of saturation and tests were carried out under the normal stresses 9.8 kPa, 19.6 kPa and 39.2 kPa until the specimen fails. However, in some cases additional tests were conducted under 60.0 kPa or 78.4 kPa normal stresses or repeated above stresses when 3 points are not enough to determine the failure envelope. After each test, degree of saturation and void ratio of the sheared samples were measured.

Fig. 4 shows some plots of shear stress vs shear displacement for different void ratios and degrees of saturation for weathered Rhyolite. As seen in the plots, shear stress decreases with increase of void ratios. Selected failure envelopes for some cases are shown in Fig. 5. The rest of the cases also analyzed and presented graphically in similar manner. The shear strength parameters, cohesion and friction angle of all cases were then analyzed according to the void ratios and degree of saturations.



1.Horizontal loading cell 2. Frame 3. Spacer screws
4. Bearings 5.Upper half-shear box 6. Lower half shear box
7. Vertical loading cell 8.Loading ram (from down to up)

Fig.3 Direct shear test apparatus

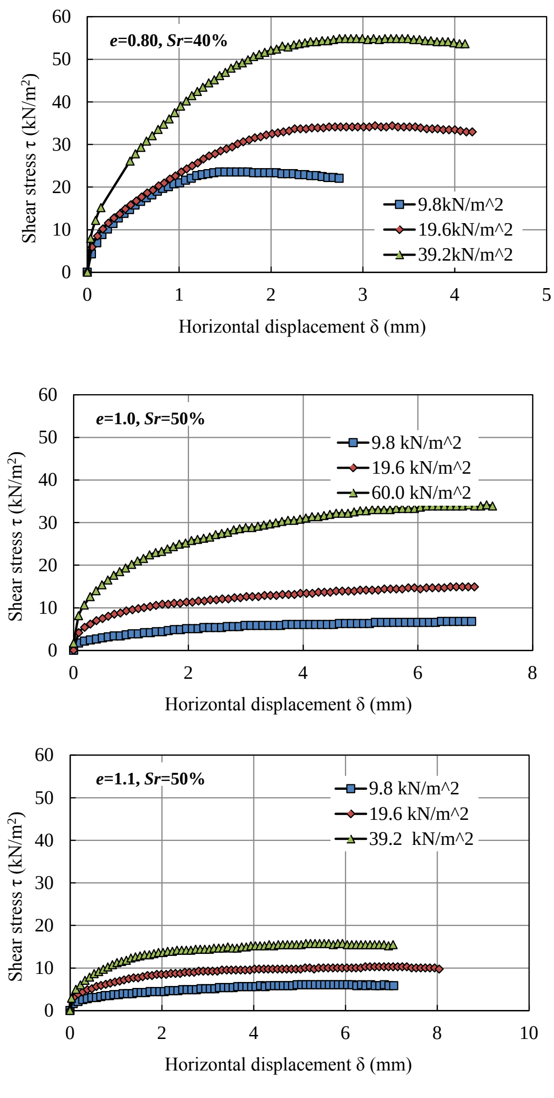


Fig. 4 shear stress vs shear displacement

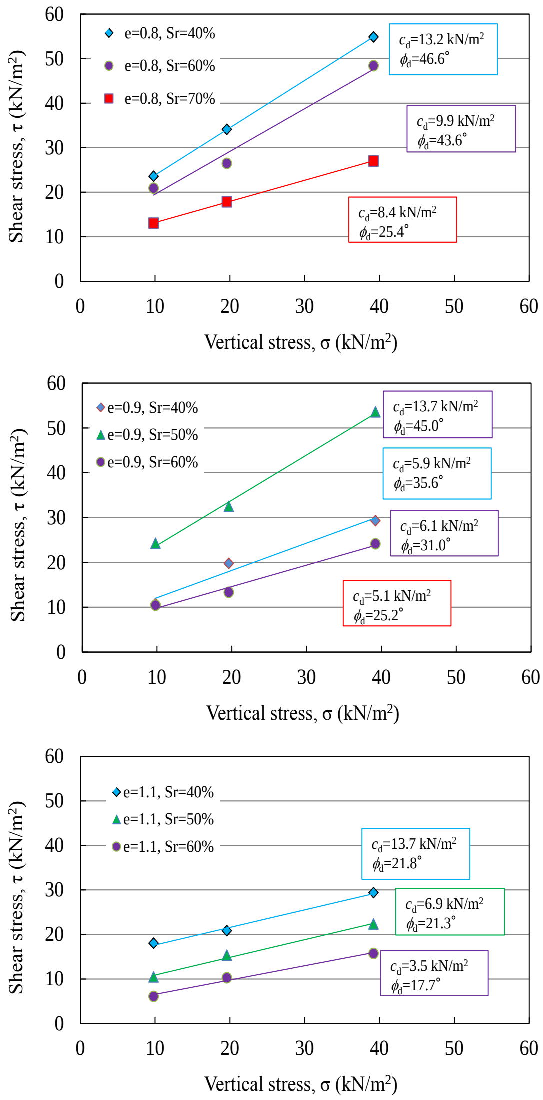


Fig. 5 Failure envelops for some cases

2.2.1 Shear strength parameters varying with void ratios and degrees of saturation (weathered Rhyolite)

Fig. 6(a) illustrates the relationship of apparent cohesion with void ratios for different degrees of saturation. Results of tri-axail experiments, i.e for 100% degree of saturation also plotted in Fig 6(a). The apparent cohesion shows nonlinear increment with the decrease of void ratios and the decrease of degree of saturation. Considering the trend of data points in the plot, curves were drawn manually for each degree of saturation as shown in the Fig. 6(a). Higher degree of saturation and higher void ratios show almost negligible values for apparent cohesion. However, more data are required to establish a better relationship of apparent cohesion with void ratios and degree of saturations. Fig. 6(b) illustrates the relationship of friction angle with void ratios for different degrees of saturation for weathered Rhyolite. Tri axial data also plotted in the same plot for 100% degree of saturation. The friction angle seems not much varies with the degree of saturation but with void ratios. The lines were drawn according to the trend of variation of friction angle with void ratios. Continues line is to show the average variation and discontinuous lines are to show the minimum and maximum variations of friction angle with void ratios. Eqs. (1a) and (1b) show the formulas for average friction angle developed for void ratios less than 0.8 and more than 0.8, respectively.

$$\phi_d = 20e + 22 \qquad (e < 0.8) \qquad (1a)$$

$$\phi_d = -43e + 72 \qquad (e > 0.8) \qquad (1b)$$

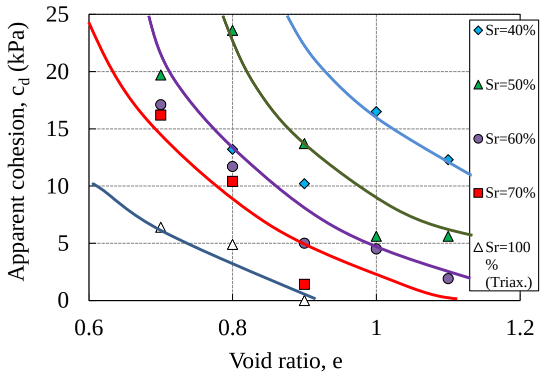


Fig. 6(a) Apparent cohesion varies with void ratios for different degrees of saturation -Rhyolite

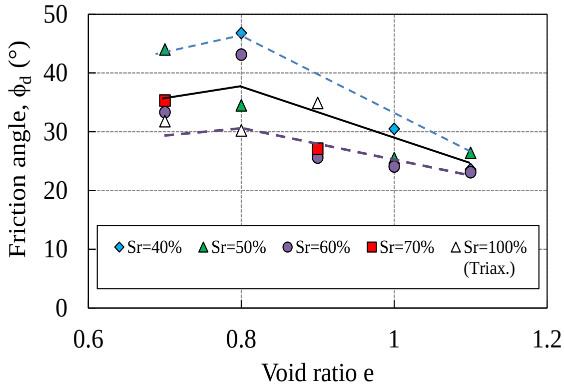


Fig. 6(b) Friction angle varies with void ratios for different degrees of saturation-Rhyolite

It is interesting to note that the friction angle shows maximum value at 0.8 void ratio and then descends. This may be due to the crushing of particle and rearrangement when preparing samples at less void ratios in direct shear molds. Also, so far, the void ratios measured from the field samples were ranging from 0.95 to 1.26. Therefore, there might not be the instances where weathered Rhyolite shows void ratio 0.7 in natural slopes.

2.2.2 Comparison of shear strength parameters of weathered Rhyolite and weathered Granite

In order to compare the differences and similarities of shear strength parameters of weathered Rhyolite with weathered Granite, relationships already developed by Tsuchida et al. for weathered Granite is presented in Figs. 7(a) and (b) [6]. The formulas to determine shear strength parameters, friction angle (ϕ_d), and apparent cohesion (c_d), for weathered Granite are shown in Eqs. (2a) and 2(b), respectively [8].

$$\phi_d = 52.7 - 19.2e \tag{2a}$$

$$c_d = 27.5 - 0.146 S_r - 14.2e \tag{2b}$$

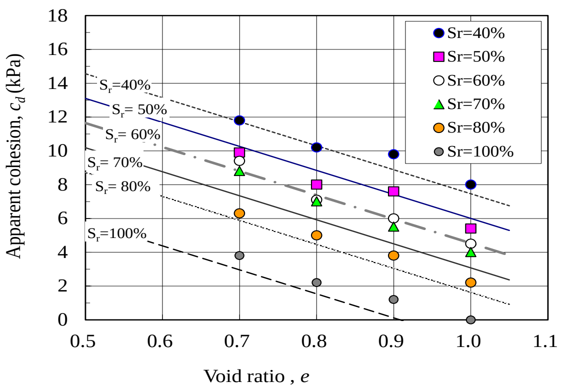


Fig. 7(a) Apparent cohesion varies with void ratios for different degrees of saturation-Granite [6]

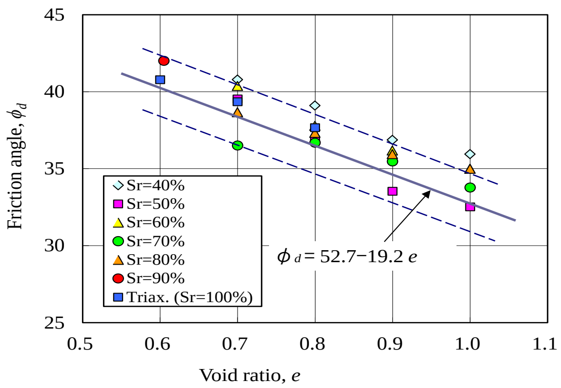
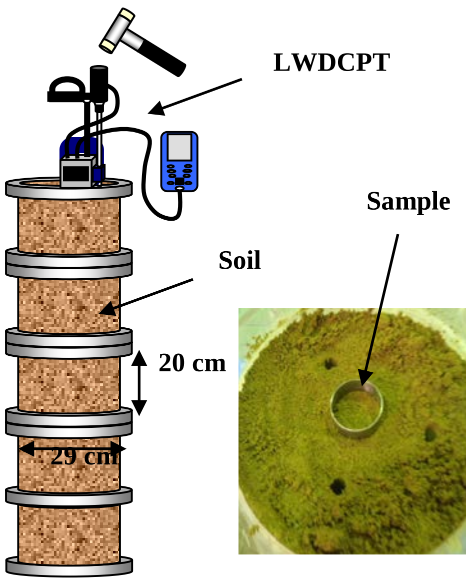


Fig. 7(b) Friction angle varies with void ratios for different degrees of saturation-Granite [6]

Both soils show different variation of shear strength parameters with void ratios and degrees of saturation. When comparing the friction angles, weathered Granite shows slightly higher values than weathered Rhyolite for same void ratios. Also, weathered Granite shows linear variation with void ratios and that for weathered Rhyolite is different as explained earlier. The variation of apparent cohesion is totally different in both soils, as one show linear (weathered Granite) and the other (weathered Rhyolite) shows non-linear behavior. For low void ratios, the apparent cohesion of weathered Granite is lesser than those of weathered Rhyolite. However, higher void ratios both soils show apparently same apparent cohesion, and when close to near saturation conditions, both soils show negligible values of apparent cohesion. Therefore, the major difference in shear strength parameters at high void ratios and near saturation conditions are the friction angle.

3. LABORATORY CALIBRATION TESTS

A series of laboratory calibration tests were conducted in order to develop a relationship between shear strength parameters and cone resistance obtained from LWDCPT. A same methodology which was adopted for weathered Granite (ex. [6], [8]) was used when carrying out the tests for weathered Rhyolite. Calibration tests were conducted for the void ratios ranging from 0.7 to 1.1 in 0.1 intervals and degrees of saturation ranging from 40% to 80% in 10% increments. The configuration of laboratory calibration test is shown in Fig. 8(a). Each column is 20 cm in height and 29 cm in diameter. Two test cases were conducted for a particular preparation of the calibration test; i.e is first two columns were prepared for particular void ratio and degree of saturation, and the rest of two columns were prepared for different void ratio, and degree of saturation. The base column . i.e the column at the bottom, was filled with dry soils. No samples were taken from that column.



(a) (b)

Fig. 8 Calibration test arrangement and sampling

Three LWDCPTs were conducted for each preparation and samples were collected at 10 cm intervals to measure the void ratio and degree of saturation after the tests. Fig. 8(b) shows the sampling location in the column. All the data were then analyzed according to the void ratio, degree of saturation with respect to cone resistance, q_d. Fig. 9 illustrates a typical sounding recorded at calibration test and respective void ratios and degrees of saturation measured after the tests. The initial void ratio and degree of saturation of this particular arrangement was 0.8,80% for first two columns and 0.9, 70% for the bottom two columns , respectively. Slight variations on void ratios and degrees of saturation were noticed and those variations were considered in analyzing the data.

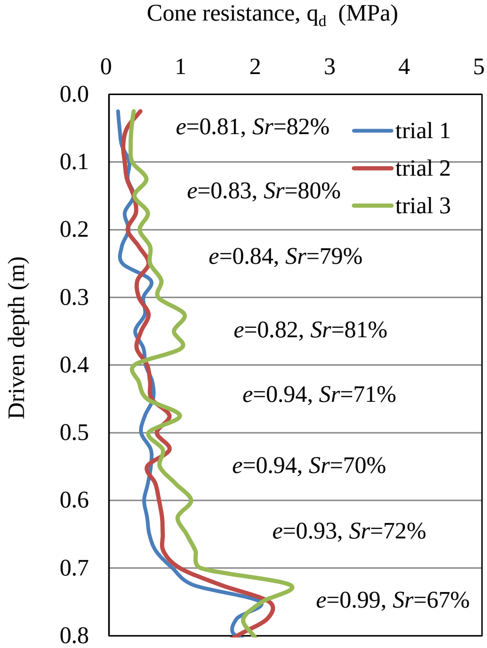


Fig. 9 Calibration test data for e=0.8 and 0.9

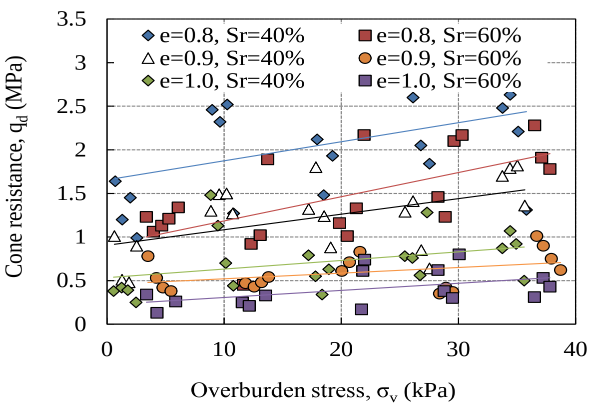


Fig. 10 Effect of overburden stress on cone resistance

3.1 Effect of Overburden stress on Cone resistance

It is well known that the overburden stress has effect on cone resistance. In order to account for overburden stress, a series of tests was conducted with number of weights placing on top of the column. Number of weights placing on the top of the column was changed in order to get the range of data varying overburden stress from 0 to 39.6 kPa. The results are demonstrated in Fig. 10. The trend of variation of cone resistance with respect to overburden stress was found to be linear and lines were drawn considering the average of all cases as shown in Fig. 10. The average gradient is 0.015 and hence the cone resistance for overburden stress 5 kPa, q_{d5} can be determined from Eq.(3) as follows.

$$q_{d5} = q_d - 0.015 \text{ X } (\gamma_t z - 5) \tag{3}$$

Where q_d and q_{d5} are the cone resistances in MPa ,γ_t is the unit weight of the soil in kN/m^3, and z is the depth in m. This equation is valid for the weathered Rhyolite in determining of the cone resistance.

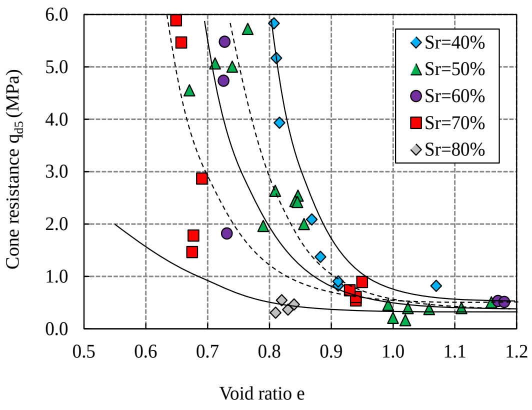


Fig. 11 Cone resistance against void ratio-Rhyolite

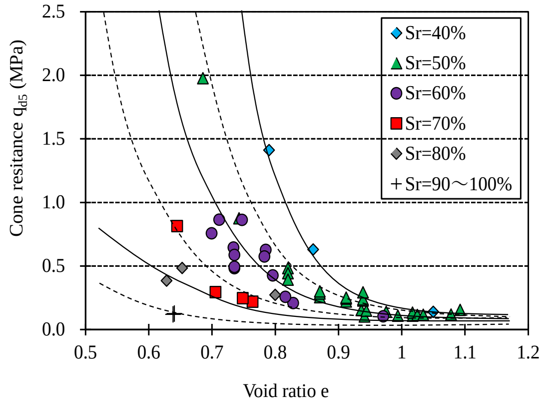


Fig. 12 Cone resistance against void ratio-Granite

However, the formula developed for weathered Granite is slightly different than Eq. (3) which was established for weathered Rhyolite. Eq. (4) was developed for weathered Granite by Tsuchida et. al [6].

$q_{d5}= q_d - 0.01 \text{ X } (\gamma_t z\text{-}5)$ (4)

Where the notation are similar to that for Eq. (3).

Based on Figs. 7 and 12, Tsuchida et.al developed relationships to evaluate friction angle, and apparent cohesion for weathered Granite in terms of cone resistance and degree of saturation as shown in Eqs. (5) and (6), respectively. More information on the derivation of the equations can be found from Tsuchida et. al [6].

$$\phi_d = 29.9 + 1.61\ln(q_{d5}) + 0.142 S_r \qquad (5)$$

$$c_d = 10.6 + 1.19\ln(q_{d5}) - 0.041 S_r \qquad (6)$$

Where ϕ_d is the friction angle in degrees (°) , c_d is the apparent cohesion in kPa, q_{d5} is the cone resistance for 5 kPa overburden stress in MPa, and S_r is the degree of saturation in percentage (%).

3.2 Relationship of cone resistance with void ratios for different degree of saturation

Considering the effect of overburden stress, cone resistance was corrected for all calibration tests data for weathered Rhyolite. Fig. 11 shows the relationship between corrected cone resistance, q_{d5} with void ratios for different degrees of saturation. The variation was found to be non-linear, and curves were drawn manually on the Fig. 11 for different degrees of saturation. It should be noted that, these curves can be better presented with more data. Fig.12 shows the same relationship that was obtained for weathered Granite. Close review of both plots reveal that weathered Rhyolite shows low void ratios for high cone resistance than that of weathered Granite. These two plots can be used to determine the void ratios from the cone resistance data and thereby determine the shear strength parameters, apparent cohesion and friction angle from Figs. 6 and 7, respectively. For a known degree of saturation, it is possible to determine the apparent cohesion and the friction angle for natural slopes found on weathered Granite or weathered Rhyolite based on cone resistance data.

4. APPLICABILITY OF LABORATORY GEOTECHNICAL FINDINGS AND NATURAL SLOPE INVESTIGATION AND ANALYSIS

In order to examine the validity of the established relationships developed based on the laboratory data, and also to investigate and analysis natural slopes, a series of field investigations was conducted at selected site locations in Hiroshima prefecture. All site locations selected for this study were the natural slopes covered with weathering remnants of Granite or Rhyolite.

4.1 Procedure for Site Investigation and Carryout LWDCPTs

A typical procedure for the investigation of natural slope was established based on the previous studies carried out and is available the literature published by Athapaththu et al. [8]. The procedure with slight modifications is as follows.

1) Gather geological and topographical maps of the area of interest (susceptible valley or planar slope of interest).
2) The in-situ testing points are determined at both sides of the center of the valley from top to the foot at 20 m intervals. In case of a planar slope, tests are to be conducted along the middle portion of the slope. The first point of the insitu test has to be closed to the top most point for planar slope and for valley , it is near to the starting point of the valley.
3) At each testing point, the gradient of slope need to be measured and the LWDCPTs are to be carried out 2 to 3 times until the penetration depth equal or close for a particular point. The termination depth has to be determined when cone resistance is equal or exceeds to 10 Mpa in three consecutive blows. The soil samples will be taken at 30 cm depth for permeability tests, and other laboratory tests. A few case studies is described in proceeding sections.

4.2 Site Exploration of Weathered Rhyolite Slopes

One of the site location selected for this study was at Kazahaya (Akitsu, Higashi-Hiroshima) in Hiroshima prefecture. This is one of the slopes which identified as one of the 32,000 slopes vulnerable to failure in the region by Hiroshima prefectural government, Japan. Geologically, this slope covers with weathering remnants of Rhyolite. A series of LWDCPTs was conducted on the slope as shown in Fig. 13.

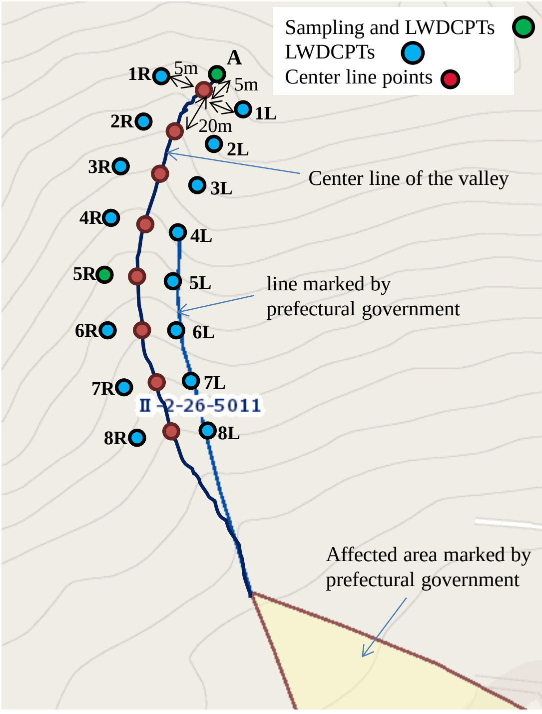
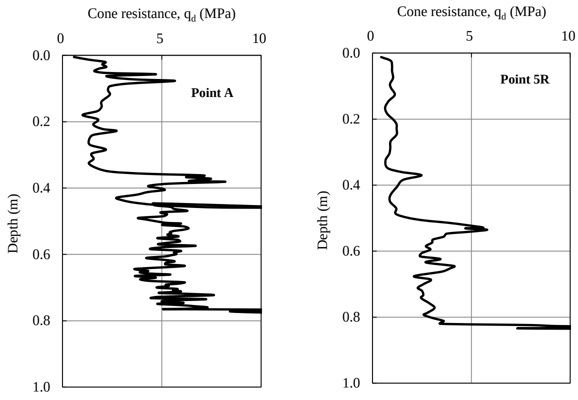


Fig. 13 Sampling and LWDCPTs locations – Kazahaya (part of map extracted from [12])

The first point which is marked as in green colour was selected at the uphill area near to the initiation of valley. The rest of the in-situ test locations were selected either side of the center of the valley as shown in Fig. 13. LWDCPTs were conducted up to the hard stratum (cone resistance exceeds to 10 MPa) at all locations 2 or 3 times until same or close reading recorded for depth of penetration. Fig. 14 shows typical sounding obtained at top most point (A) and 5R where sampling was carried out.

Soil samples were taken to carry out laboratory tests to determine the void ratios and shear strength parameters at the points A, and 5R as shown in Fig.13. Table 2 shows some of physical the properties determined at the laboratory. Both points show higher void ratios and comparatively higher coefficient of permeability. Void ratios were obtained from cone resistance, q_{d5} from Fig. 11. Table 3 shows the comparison of void ratios determined from the laboratory tests and that obtained from Fig. 11. Since, the data for degree of saturation 30% were not available, extrapolation was carried out to determine the void ratio from cone resistance data. The predicted and laboratory measured void ratios are too close. Therefore, the developed relationship as shown in Fig.11 can be fairly applied to determine the void ratio from the cone resistance data. However, more data are required to establish more sound relationships between cone resistance and void ratios for weathered Rhyolite.



(a) Point A (b) Point 5R

Fig. 14 Soundings at some locations-Kazahaya

Table 2 Some physical properties

Location	Point A		Point 5R
Sample depth, cm	0-10	10-20	20-30
Void ratio (laboratory)	0.95	1.26	1.18
Void ratio (Fig. 10)	0.94	1.21	1.17

Table 3 Comparison of void ratios

Location	Point A		Point 5R
Depth, cm	0-10	10-20	20-30
Void ratio	0.95	1.26	1.18
Deg. of saturation, %	27	30	49
Specific gravity	2.61	2.61	2.6
Coefficient of permeability, cm/s	-	-	$5x10^{-3}$

Also, a series of direct shear tests was conducted from the samples collected at those locations under 30% and 80% degree of saturation. Table 4 shows the shear strength parameters, cohesion and friction angle obtained for points A and 5R. In the same Table, shear strength parameters determined from the established relationships as shown in Figs. 6 are given. Both laboratory and predicted values of shear strength parameters are close. Therefore, the developed relationships can be successfully used to determine the shear strength parameters in terms of cone resistance data. As a whole, void ratio can be determined from cone resistance data for known degree of saturation. Upon determination of void ratio, shear strength parameters, apparent cohesion and friction angle can be determined for known degree of saturation. Therefore, shear strength parameters for weathered Rhyolite can be determined based on cone resistance data. With the availability of cone resistance data, it is possible to evaluate shear strength parameters spatially for a particular slope of interest.

Table 4 Comparison of shear strength parameters (void ratio =1.0)

Location	*Sr* (%)	Laboratory		Predicted	
		ϕ_d (°)	*c* (kPa)	ϕ_d (°)	*c* (kPa)
A	30	29.7	19.3	29±3	20.0
	80	24.0	1.1		0.5
5R	30	32.6	22.8	29±3	20.0
	80	25.5	0.54		0.5

4.3 Site investigation of weathered Granite slopes

In order to carry out stability analyses based on the shear strength parameters obtained by cone resistance data, a series of LWDCPTs were conducted at Kabe, in Hiroshima, Japan. This was one of the failure sites due to calamitous events occurred on 20th August 2014 near city of Hiroshima. A part of map of the area and the in-situ tests locations were shown in Fig. 15. The in-situ tests locations were selected close to the failure boundary. In similar manner, 2-3 tests were conducted at each point until the values of depth of penetration is close. The spacing of the in-situ tests from points A, 1L,1R, 2L, and 2R is 5 m and the rest is 20 m . Sampling was carried out at points A, 2R, and 3R to determine the basic soil parameters such as void ratios, degree of saturation, and coefficient of permeability. Table 5 shows the results of laboratory tests. A typical soundings obtained from LWDCPTs are shown in Fig. 16.

Table 5 Some physical properties-Kabe

Location	Point A	Point 2R	Point 3R
Sample depth, cm	20-30	20-30	20-30
Void ratio	1.3	0.77	1.16
Deg. of saturation, %	42	46	37
Specific gravity	2.56	2.63	2.61
Coef. of permeability, x 10^{-2} cm/s	4.4	5.36	

5. STABILITY ANALYSES BASED ON SHEAR STRENGTH PARAMETERS AND REAL TIME RAINFALL DATA

In order carry out stability analyses, the geometry of slopes, the shear strength parameters and the water table are required. Geometry and shear strength parameters can be determined based on site investigation and LWDCPTs data. Height of water table fluctuates depending on the rainfall conditions. In this study, to determine the water table under different rainfall conditions, a model proposed by Thi ha [13] was used. Following section describes the determination of ground water table.

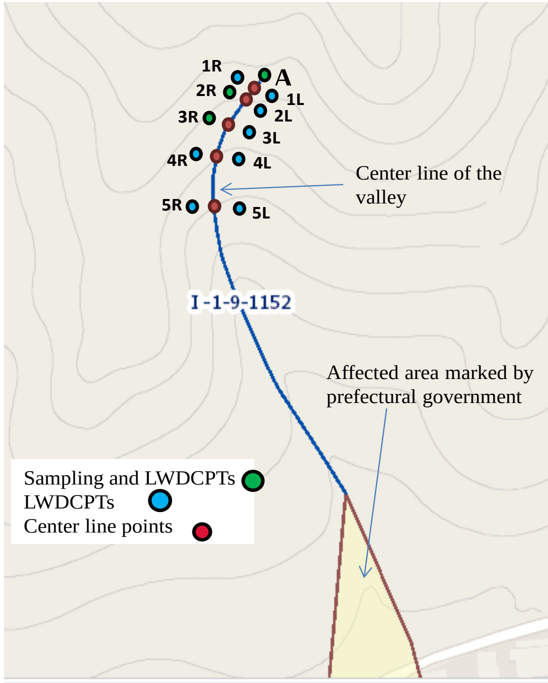
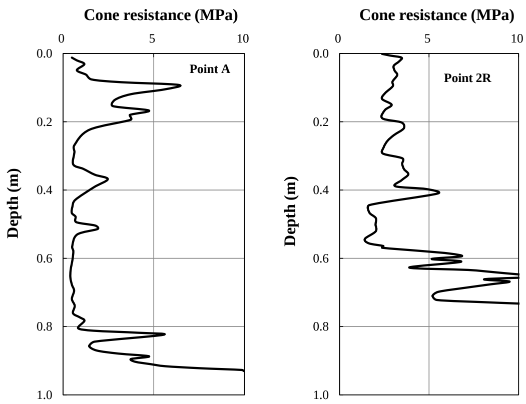


Fig. 15 Sampling and LWDCPTs locations - Kabe (part of map extracted from [12]

5.1 Determination of height of water table for different rainfall conditions



(a) Point A (b) Point 2R

Fig. 16 Soundings at some locations-Kabe

This ha [13] proposed Eq. (7) to determine the height of water table based on several factors.

$$h_w = \frac{(R - H_l(0.36 - \theta_o))S}{1/2S\cos\beta + k\sin\beta\cos\beta t} \qquad (7)$$

Where *R* is the cumulative rainfall, *S* is the slope length, H_l is height of the soil profile,β is the slope angle, *t* is the time, and *k* is the coefficient of permeability. θ_0 is the initial volumetric water content and the value 0.36 refers to the volumetric water content that requires for initiation of water table. This value. 0.36 was obtained under a series of site data collected from tensitometers and rainfall gauges instrumented at weathered Granitic slopes over 2-3 years at a site close to Hiroshima University, Japan . However, data are not available to determine the threshold value of volumetric water content for weathered Rhyolite. This is future concern of authors long time research plan. Therefore, stability analysis for real time rainfall data was conducted only for Kabe, Hiroshima where the slope covers with weathered Granite.

5.2 Rainfall and other data for stability analyses

The rainfall data recorded at rainfall gauging station near Kabe, Hiroshima is shown in Fig. 17. The peak rainfall recorded was 120 mm/hr from 3.00 to 4.00 am and the slope failure was believed to be occurred around 3.50 am as per the information received from the community living in the area.

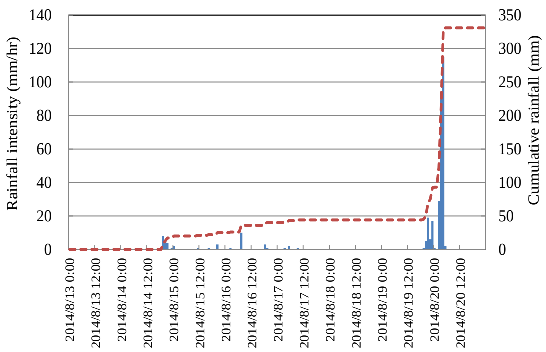


Fig. 17 Rainfall data near Kabe, Hiroshima, Japan

The other parameters required for the stability analyses are listed in Table 6. The thickness of the soil layer was determined based on LWDCPTs data and the average value was considered for stability calculations. The shear strength parameters were determined based on the cone resistance data. For 2D stability analysis, the minimum cone resistance recorded from all soundings was considered.

Table 6 Parameters for stability analysis

Parameter		Value
Thickness of soil layer (average)/ (m)		0.94
Slope angle (average) /(˚)		35
Length of the slope/ (m)		61
Unit weight/ (kN/m^3)	saturated	17.8
	unsaturated	14.7
Cohesion (kPa)	Sr= 40%	7.9
	Sr= 80-100%	0

However, for detailed 3D slope stability analyses, spatial variation of shear strength parameters can be evaluated from the cone resistance data. In this study, two dimensional translational slope analysis was conducted based on Eq. (8) as follows.

$$F_s = \frac{c_d + \{\gamma_t (H - h_w) + h_w(\gamma_{sat} - \gamma_w)\}\cos\beta\tan\phi_d}{\{\gamma_t (H - h_w) + \gamma_{sat} h_w\}\sin\beta} \quad (8)$$

Where, c_d, is the cohesion , ϕ_d is the friction angle , H is the thickness of the soil layer, h_w is the height of water table, γ_{sat} is the saturated unit weight , γ_t is the unit weight of water, and β is the slope angle.

The results of stability analyses are presented in Fig. 18. It can be seen that sudden decrease of FOS (Factor of safety) around 20th Aug. The real incident, i.e slope failure also happened around 3.50 am on 20th Aug, as for the information received by the community in the area. Therefore, the results obtained from this study is well explained the actual incident took place at Kabe , Hiroshima. The same methodology can be applied to individual slopes in the area to predict FOS with forecasted rainfall data. This type of hazard assessment system, which based on geotechnical and rainfall data, may provide better prediction than the existing system which only based on past records of failures and rainfall data.

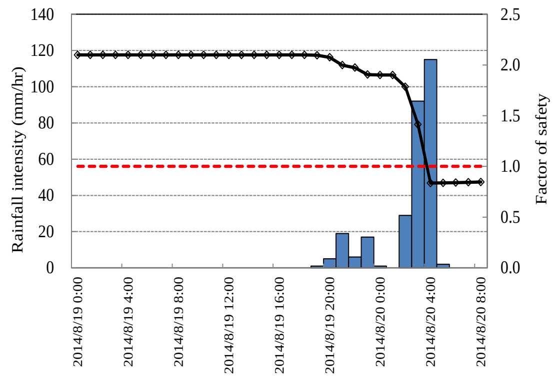


Fig. 18 FOS varies with rainfall

6. CONCLUDING REMARKS

This research was focused to evaluate shear strength parameters of weathered Rhyolite based on Lightweight dynamic cone penetration data. Also, a method was proposed to evaluate the factor of safety of natural slopes covered with weathered Rhyolite and weathered Granite based on geotechnical and real time rainfall data. On the basis of the analyses and results, following conclusions were drawn.

1. Based on laboratory calibration tests, a graphical relationship was proposed to determine the void ratio from cone resistance

data obtained from lightweight dynamic cone penetrometer for known degree of saturation for weathered Rhyolite. The established relationship was verified with some field data.

2. Graphical relationships were developed to evaluate shear strength parameters, cohesion and friction angle, from void ratios for known degree of saturation for weathered Rhyolite. The formulas to determine the friction angles are as follows.

   $\phi_d = 20e + 22$ for $e < 0.8$

   $\phi_d = -43e + 72$ for $e > 0.8$

   The same developed for weathered Granite is as follows.

   $\phi_d = 52.7 - 19.2e$

   $c_d = 27.5 - 0.146S_r - 14.2e$

3. Combing the graphical relationships described in above conclusions (1) and (2), shear strength parameters can be evaluated by means of cone resistance data obtained from LWDCPTs for known degree of saturation. The formulas developed for weathered Granite are given below.

   $\phi_d = 29.9 + 1.61\ln(q_{d5}) + 0.142S_r$

   $c_d = 10.6 + 1.19\ln(q_{d5}) - 0.041S_r$

4. A simple method was proposed to assess the stability of slopes based on geotechnical and geometrical data (shear strength parameters and slope depth, length and angles) with real time rainfall data. One of the examples was carried out to see the validity of the proposed method and found that the results well agreed with the actual occurrence of the failure.

The proposed method can be used to predict the individual slopes in the region with forecasted rainfall data which facilitates authorities to issue early evacuation orders. More data and analyses may need to enhance the proposed method for hazard assessment system for better forecasting of slope failures.

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