LATERAL LOAD CARRYING CAPACITY OF ROCK SOCKETS

A DISSERTATION

Submitted in partial fulfillment of the requirements for the award of the degree

of

MASTER OF TECHNOLOGY

in

CIVIL ENGINEERING

(With Specialization in Geotechnical Engineering)

By

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CANDIDATE'S DECLARATION

I hereby declare that the work which being presented in this dissertation entitled "**LATERAL LOAD CARRYING CAPACITY OF ROCK SOCKETS**" in partial fulfilment of the requirement for the award of degree of **"MASTER OF TECHNOLOGY"** in **"CIVIL ENGINEERING"**, with specialization in **"Geotechnical Engineering"**, submitted in the Department of Civil Engineering, Indian Institute of Technology Roorkee, Roorkee, is an authentic record of own work carried out from October 2011 to June 2012 under the guidance of **Dr. Mahendra Singh** and **Dr. Vishwas Sawant**, Department of civil engineering, IIT Roorkee, Roorkee.

The matter embodied in this dissertation has not been submitted by me for the award of any degree or diploma.

Date: 13/06/12

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CERTIFICATE

This is to certify that Mr. V. SUJITH KUMAR (En. No.-10521016), student of M. Tech IInd year (Geotechnical Engineering), Department of Civil Engineering, IIT Roorkee, Roorkee, has worked for the preparation of this dissertation report under our supervision and guidance and the above statement made by the candidate is true to the best of our knowledge and belief.

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ABSTRACT

Rock sockets are a type of deep foundation that are capable of supporting very large vertical and lateral loads. Rock sockets are widely used as foundations for bridges and other important structures. They are also used to stabilize a landslide. The sockets are constructed by drilling holes from the ground surface to the target depth of rock formation and filling the hole with reinforcing steel and concrete. The main loads applied on the socket are axial compressive or uplift loads as well as lateral loads with accompanying moments.

Rock encountered in the foundations are invariably jointed and the load carrying capacity of socket therefore depends not only on the quality of the intact rock but also on the characteristics of rock joints. The most important characteristics of the rock joints which will affect the load carrying capacity of socket are surface characteristics of the joints, frequency of joints and orientation of joints with respect to loading direction. In present study the effect of rock joints on the lateral load carrying capacity of rock sockets has been studied by using equivalent continuum material modelling approach.

In this approach the discontinues rock mass is converted into a equivalent continuum which has same properties as those of discontinuum. The most important index properties of the rock mass is the uniaxial compressive strength. A computer program has been developed which initially estimates the UCS of rock mass by different methods available in literature and based on this UCS value it calculates the shear strength parameters of rock mass. Finally considering the shear strength parameters, the lateral load carrying capacity of rock sockets is estimated using Carter and Kulhawy (1992) method.

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Chapter 1

INTRODUCTION

1.1 Statement of the problem

Drilled shafts socketed into rock are widely used as foundations for bridges and other important structures. Rock-socketed drilled shafts are also used to stabilize slopes. The main loads applied to the drilled shafts are axial compressive or uplift loads as well as lateral loads with accompanying moments. For axially loaded drilled shafts socketed into rock, numerous research efforts have been conducted in the past, especially for the determinations of side shear resistance.

However, for laterally loaded drilled shafts, there is a lack of validated, rational analysis and design methods. It has been a customary practice to adopt the techniques developed for laterally loaded piles in soil to solve the problem of rock-socketed drilled shafts under lateral loading (Gabr, 1993). This practice has created erroneous designs and often leads to excessive socket length. Although there exist several analysis and design methods specially for rock-socketed drilled shafts under lateral loading, including those by Carter and Kulhawy (1992), Reese (1997), Zhang et al. (2000), and Gabr et al. (2002); however, these methods were developed with limited validations against field lateral load test data.

Several researchers (such as DiGioia and Rojas-Gonzalez, 1993; Dykeman and Valsangkar, 1996; and Cho et al., 2001) have evaluated the methods of Carter and Kulhawy (1992) and Reese (1997) using their lateral load test data. They concluded that the two methods provided very unconservative results. Zhang et al. (2000)'s analysis method has not been evaluated by others due to the complexity of the method and the needs of a computer program. The assumption of an elastic-perfectly plastic model for rock masses by Zhang et al. (2000) prohibited its wide application, especially for weak rock masses which cannot be exactly characterized by elastic-perfectly plastic model. In addition to the need to develop analysis methods for predicting lateral shaft deflections, there is also a need for the development of methods for estimating lateral capacity of shafts. Very few methods (Carter and Kulhawy, 1992; and To et al., 2003) have been developed for estimating lateral capacity of piles in rock. Additionally, several other researchers, such as Reese (1997) and Zhang et al. (2000), have proposed methods to predict the ultimate lateral resistance of rock per unit shaft length. However, Carter and Kulhawy's (1992) method does not consider the effect of secondary structures of rock mass; and the method of To et al. (2003) is only suitable for two sets of parallel joints and rigid piles.

The lack of validated design methods stimulates the need to develop a more rational design approach for laterally loaded drilled shafts in rock. Additionally, a method for predicting ultimate capacity of drilled shafts in rock mass needs to be developed.

1.2 Objective of the study

The present study has been conducted with following objectives

- To critically review the existing methodology to asses lateral load carrying capacity of rock sockets.
- To understand the mechanism of failure of sockets embedded in jointed rocks.
- To incorporate the effect of the joint characteristics in analysis of lateral load carrying capacity of socket.

1.3 Organization of the report

The chapter 1 deals with the introduction ro rock sockets and statement of problem which is followed by chapter 2 which presents the literature review on analysis methods of laterally loaded rock sockets. Various methods to estimate the UCS of rock mass are presented in chapter 3.Chapter 4 presents the methodology adopted to calculate the lateral load. Results and discussion of the study are presented in chapter 5 followed by concluding remarks in chapter 6. Finally computer programs developed to calculate the lateral load is presented in appendix.

Chapter 2

LITERATURE REVIEW

2.1 General

To date, there are few published analysis methods for the lateral response of rock-socketed drilled shafts. It has been a customary practice to adopt the p-y analysis with p-y criterion developed for soils to solve the problem of rock-socketed drilled shafts (Gabr, 1993). Currently, two categories of analysis methods for laterally loaded rock-socketed drilled shafts have been developed. One category treats rock as a continuum mass (Carter and Kulhawy 1992; and Zhang et al 2000), the other one discretizes the rock mass into a set of non-linear springs (Reese 1997; and Gabr et al. 2002).

Carter and Kulhawy (1992) proposed a method that treats rock mass as a homogeneous elastic continuum. Parametric solutions for the load-displacement relationships were generated by using the finite element technique. However, elastic continuum model is only good for small loads. Zhang et al. (2000), therefore, developed a nonlinear continuum approach. The approach adopts and extends the basic idea of Sun (1994) on laterally loaded piles in soil. The elasto-plastic soil/rock response under lateral loads and the linearly variation of deformation modulus of soil/rock along depth were assumed. To consider the yielding, a method based on Hoek-Brown criterion (Hoek and Brown 1980, 1988) was proposed to calculate the ultimate resistance of rock masses. In practice, however, the rock masses, especially weak rock, show nonlinear and non-homogeneous properties which cannot be fully captured by an elasto-plastic model.

The second category of analysis method, such as p-y method, discretizes the rock masses into a series of nonlinear springs. The p-y method was extended to the analysis of single rock-socketed drilled shaft under lateral loading by Reese (1997). An interim p-y criterion for weak rock was proposed. Thereafter, Gabr et al. (2002) proposed a p-y criterion for weak rock based on their field test data.

In addition to the above mentioned analysis methods for solving load-deflection relationship at the drilled shaft head, methods for estimating ultimate rock reaction have also been proposed. Carter and Kulhawy (1992) presented a method to determine the rock capacity by using cohesion and friction angle of rock. This method was based on a theory of expansion of a long cylindrical cavity in an elasto-plastic, dilatant material. The method requires input of Poisson's ratio, shear modulus and dilation angle. By assuming distribution of ultimate rock resistance along the depth of a shaft, the ultimate lateral capacity of a rock-socketed drilled shaft was obtained by summing the capacity of compressive resistance and shear resistance between shaft and rock. Cater and Kulhawy (1992)'s method on rock resistance, treats rock mass as a homogeneous and elasto-plastic material, without considering secondary structures of rock mass, such as cracks and fractures.

Reese (1997) considered the secondary structure of rock mass by using a rock strength reduction factor which can be determined from Rock Quality Designation (RQD). However, Reese (1997)'s method for estimating ultimate rock reaction per unit length ignored the contribution of shear resistance between shaft and rock. Additionally, RQD cannot fully represent all the secondary rock structures, such as spacing and condition of discontinuities.

Zhang et al. (2000) proposed a method to estimate the ultimate reaction of rock masses per unit shaft length using Hoek-Brown rock strength criterion (Hoek and Brown 1988), in which RQD and other secondary rock structures were included. However, simple rock resistance distribution along the shaft circumference under lateral loads was assumed (Carter and Kulhawy 1992). It seems that Zhang et al. (2000)'s method for estimation of lateral capacity of rock socketed drilled shaft considered most of characteristics of rock mass; however, the authors did not investigate possible failure modes of rock mass, especially possible sliding failures along pre-existing joints. Regarding the sliding failure on joints, To et al. (2003) proposed a method to estimate the lateral load capacity of drilled shafts in join ted rock. The block theory (Goodman and Shi 1985) was used to identify the failure block, and the static limit equilibrium was used to obtain the ultimate capacity. The Coulomb failure criterion was utilized to model the sliding failure on joints.

2.2 Brief description of the existing methods

2.2.1 Reese (1997)

The p-y method for the analysis of drilled shafts in soils under lateral loading was extended to the analysis of rock-socketed drilled shafts by Reese (1997). In order to characterize the rock response under lateral loading, an interim p-y criterion for weak rock was suggested. Due to the lack of adequate test data, the term "interim" was applied to this p-y criterion. The ultimate reaction is given as

$$P_u = \alpha_r \sigma_{ci} D \left(1 + \frac{1.4\, z_r}{D} \right) \qquad 0 \leq z_r \leq 3D \qquad (2.1)$$

$$P_u = 5.2\, \alpha_r \sigma_{ci} D \qquad Z_r \geq 3D \qquad (2.2)$$

Where σ_{ci} = uniaxial compressive strength of intact rock; α_r = strength reduction factor, which is used to account for fracturing of rock mass; D = diameter of the drilled shaft; and z_r = depth below rock surface. The value of α_r is assumed to be 1/3 for RQD of 100% and it increases linearly to unity at a RQD of zero.

The slope of initial portion of p-y curves was given by

$$K_{ir} = k_{ir} E_m \qquad (2.3)$$

Where K_{ir} = initial tangent to p-y curve; E_m = deformation modulus of rock masses, which may be obtained from a pressure meter or dilatometer test; and k_{ir} = dimensionless constant. The expressions for k_{ir}, derived by correlation with experimental data, are as follows.

$$K_{ir} = \left(100 + \frac{400\, z_r}{3D}\right) \qquad 0 \leq z_r \leq 3D \qquad (2.4)$$

$$K_{ir} = 500 \qquad Z_r \geq 3D \qquad (2.5)$$

A complete description of the interim p-y criterion may be summarized as follows.

First segment: $p = K_{ir} y; y \leq y_A$ (2.6)

Second segment: $p = \frac{p_u}{2}(\frac{y}{y_{rm}})^{0.25}$ $y \geq y_A$ and $p \leq p_u$ (2.7)

Third segment: p_u; $p \geq p_u$ (2.8)

Where

$y_{rm} = K_{rm} D$ (2.9)

$y_A = (\frac{p_u}{2(y_{rm})^{.25} K_u})^{1.33}$ (2.10)

in which k_{rm} = strain at 50% of ultimate load, ranging from 0.0005 to 0.00005.

2.2.2 Zhang et al. (2000)

Zhang et al. (2000) proposed a nonlinear continuum method to predict the load-displacement response of rock-socketed drilled shafts under lateral loads by treating soil/rock as an elasto-plastic material. The approach was extended from the basic idea of Sun (1994) on laterally loaded piles in soil.

The model of rock-socketed drilled shafts under lateral loading is shown in Fig. 2-1. The deformation modulus of soils varies linearly from Es_1 to Es_2. Similarly, the deformation modulus of rock mass varies linearly from E_{m1} to E_{m2} at the tip of shaft and stays constant below

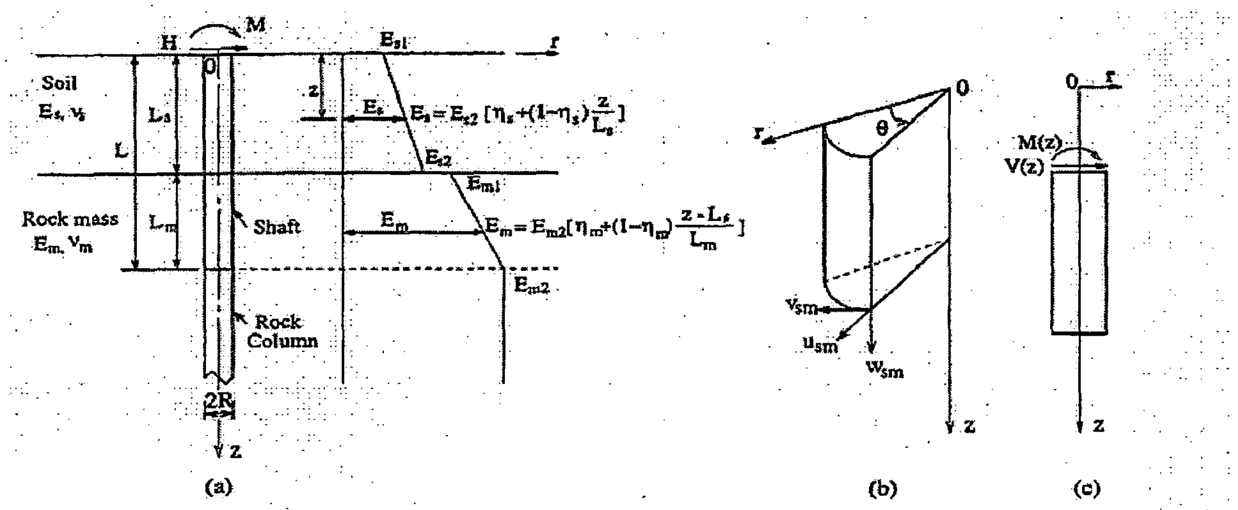

Fig.2-1 Rock-shaft model (a) shaft and soil/rock mass system; (b) coordinate system and displacement components; (c) shear force v(z) and moment m(z) acting on shaft at z(zhang et al. 2000)

the shaft tip. By minimizing the energy of rock-shaft system with respect to displacements, the following governing equations were obtained:

$$E_p I_p - \frac{d^4 u_s}{dz^4} - 2t_s \frac{d}{dz}\{[\eta_s + (1-\eta_s)] \frac{du_s}{dz}\} + k_s[\eta_s + (1-\eta_s)]u_s = 0 \;;(0 \leq z \leq L_s) \tag{2.11}$$

$$E_p I_p - \frac{d^4 u_m}{dz^4} - 2t_m \frac{d}{dz}\{[\eta_m + (1-\eta_m)] \frac{du_s}{dz}\} + k_m[\eta_m + (1-\eta_m)]u_m = 0 \;;(L_s \leq z \leq L) \tag{2.12}$$

where u_s and u_m = displacement components of the shaft in the soil and in the rock mass, respectively; E_pI_p = flexural rigidity of the shaft; z = depth starting from ground line; L_s =shaft length embedded in soils; L_m = shaft length embedded in rock masses; and

$$\eta_s = \frac{E_{s1}}{E_{s2}} \tag{2.13}$$

$$\eta_m = \frac{E_{m1}}{E_{m2}} \tag{2.14}$$

$$t_s = \frac{\pi E_{s2} m_1 R^2}{2(1+\nu_s)} \tag{2.15}$$

$$k_s = \frac{\pi(3-4\nu_s)(E_{s2} m_2)}{2(1+\nu_s)(1-2\nu_s)} \tag{2.16}$$

$$t_m = \frac{\pi E_{m2} m_1 R^2}{2(1+\nu_m)} \tag{2.17}$$

$$k_m = \frac{\pi(3-4\nu_m)(E_{m2} m_2)}{2(1+\nu_m)(1-2\nu_m)} \tag{2.18}$$

in which ν_s and ν_m = Poisson's ratio of soils and rock masses, respectively; m_1 and m_2 =parameters describing the behaviour of the elastic foundations.

The shear force V(z) acting on the shaft, shown in Fig. 2-1(c), can be obtained as

$$V(z) = E_p I_p \frac{d^3 u_s}{dz^3} - 2t_s [\eta_s + (1-\eta_s)] \frac{du_s}{dz} \ ;(0 \leq z \leq L_s) \tag{2.19}$$

$$V(z) = E_p I_p \frac{d^3 u_m}{dz^3} - 2t_m [\eta_m + (1-\eta_m)\frac{z-L_s}{L_m}] \frac{du_m}{dz} \ ;(L_s \leq z \leq L) \tag{2.20}$$

And the bending moment M(z) acting on the shaft is given by

$$M(z) = E_p I_p \frac{d^2 u_s}{dz^2} \ ;(0 \leq z \leq L_s) \tag{2.21}$$

$$M(z) = E_p I_p \frac{d^2 u_m}{dz^2} \ ;(L_s \leq z \leq L) \tag{2.22}$$

The governing differential equations and the shear force and bending moment can be solved using classical finite difference method and an iterative process. The above process considers the soil/rock to be elastic. Elastic-perfectly plastic stress-strain relationship, therefore, was proposed to consider the yielding of the soils or rock masses. The method for considering the yielding of soil or rock mass consists of several steps. Firstly, for the applied lateral load H and the moment M, the shaft is analyzed by using the above elastic solutions. Secondly, the lateral reaction force p at certain depth is computed and compared to the ultimate resistance p_u at that depth. If $p > p_u$, take the depth z as the yielding depth z_y. Thirdly, treat the unyielded portion of shaft as a new shaft, and analyze it by using the elastic solution while ignoring the effect of the yielded portion of shaft. Fourthly, repeat steps two and three until no further yielding of soil or rock occurs. Finally, the final results can be obtained by considering the two parts of the shaft separately. The portion of shaft in yielded soil and/or rock mass is analyzed as a beam with distributed load p_u acting on it. The other part of shaft in the unyielded soil and/or rock mass is analyzed by using the elastic solution.

To compute the ultimate resistance p_u of rock mass, Zhang et al. (2000) proposed to utilize the assumed resistance distribution (Carter and Kulhawy 1992), shown in Fig. 2-2,and Hoek-Brown rock strength criterion (Hoek and Brown 1988). The assumption for resistance distribution is that the total resistance of rock mass consists of two parts: the side resistance and the front normal resistance. The ultimate resistance p_u can be calculated by

$$Pu = (P_L + \tau_{max})D \quad (2\text{-}23)$$

where D = diameter of the drilled shaft; τ_{max} = maximum shearing resistance along the sides of the shaft; and P_L = normal limit resistance. τmax was assumed to be the same as the maximum side resistance under axial loading and can be given by 0.5

$$\tau_{max} = 0.20(\sigma_{ci})^{0.5} \text{ (MPa)} \qquad \text{for smooth socket} \quad (2\text{-}24)$$

$$\tau_{max} = 0.80(\sigma_{ci})^{0.5} \text{ (MPa)} \qquad \text{for rough socket} \quad (2\text{-}25)$$

where σ_{ci} = unconfined compressive strength of the intact rock (MPa).
The strength criterion for rock mass developed by Hoek and Brown (1980, 1988) was adopted to determine the normal limit stress P_L. The Hoek-Brown criterion, which is suitable for intact rock and rock mass, can be given by

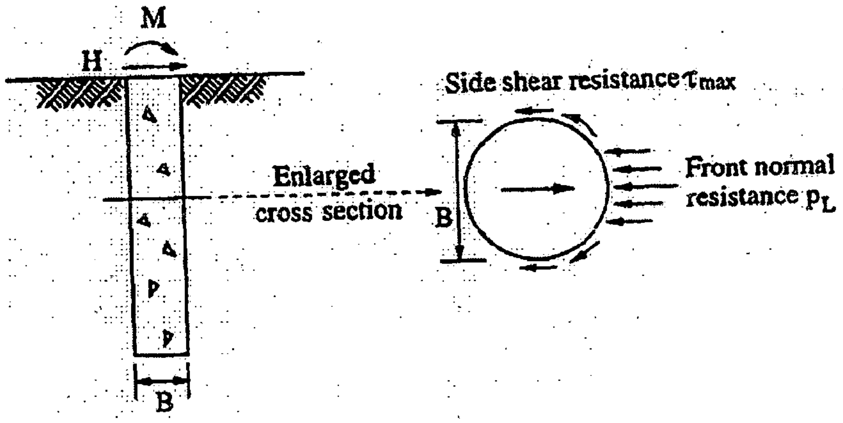

Fig. 2-2 Components of rock mass resistance (Zhang et al. 2000)

$$\sigma_1' = \sigma_3' + \sigma_{ci}(m_b \frac{\sigma_3'}{\sigma_{ci}} + s)^a \quad (2.26)$$

where σ_{ci} = uniaxial compressive strength of the intact rock; σ_1' and σ_3' = the major and minor effective principal stresses, respectively; and m_b, s, and a = constants depending on the characteristics of the rock.

For intact rock, $m_b = m_i$, a constant depending on rock type; s = 1; and a = 0.5. For rock mass, the values of m_b, s, and a can be estimated by correlations with Geological Strength Index (GSI) (Hoek, 1994). In addition to GSI, Rock Mass Rating (RMR) of Bieniawski can also be used to determine the constants m_b, s, and a (Hoek and Brown 1997). With Hoek-Brown's strength criterion, the normal limit stress P_L, which is the major principal effective stress σ_1', can be obtained by assuming that the minor principal effective stress is the effective overburden pressure $\gamma' z$.

2.2.3 Gabr et al. (2002)

Gabr et al. (2002) proposed a hyperbolic p-y criterion for weak rock based on field tests on small diameter drilled shafts socketed in weak rock. The following procedure can be used to construct a p-y curve according to Gabr et al. (2002).

Step 1: Calculation of coefficient of subgrade Reaction

The coefficient of subgrade reaction can be calculated as follows (Vesic, 1961):

$$n_h = \frac{0.65E_m}{D(1-\nu^2)}[\frac{E_m D^4}{E_p E_p}]^{\frac{1}{12}} \quad (2.27)$$

where D is the diameter of a drilled shaft, ν is Poisson's ratio of rock mass, and GSI is Geological Strength Index.

Step 2: Calculation of flexibility Factor

A flexibility factor, KR, is computed as follows

$$K_r = \frac{E_p E_p}{E_m L^4} \quad (2.28)$$

where, E_p is modulus of elasticity of shaft, I_p is the moment of inertia of shaft, L is the embedment length of shaft.

Step 3: Calculation of point of rotation

The following equation is used to define the turning point as a function of the embedded shaft length:

$$T_0 = (1+ 0.18\log K_R)L \quad (2\text{-}29)$$

where, T_0 is turning point.

Step 4: Calculation of I_T Number

$$I_T = -28 - 383 \log(T_0/L) \; IT \geq 1 \quad (2\text{-}30)$$

Step 5: Calculation of the Subgrade Reaction

$$k_h = n_h D \; (0 \leq z \leq T0) \quad (2\text{-}31)$$

$$k_h = I_T \, n_h D \; (T0 < z \leq L) \quad (2\text{-}33)$$

Step 6: Calculation of ultimate resistance of rock mass P_u

The Eq. (2-23) proposed by Zhang et al. (2000) was employed to calculate the ultimate resistance of rock. Smooth condition was assumed for all the cases when the side shear resistance is concerned.

Step 7: Construction of the P-y curve

$$p = \frac{y}{\frac{1}{k_h} + \frac{y}{p_u}} \quad (2.34)$$

2.2.4 To et al. (2003)

For the drilled shafts socketed into jointed rock, To et al. (2003) assumed a wedge type block failure and Coulomb failure criterion to obtain the lateral capacity of drilled shafts. Goodman and Shi (1985)'s block theory was used to determine the possible failure block for two sets jointed rock mass with the help of AutoCAD or Excel. Due to the complexity of the entire process to obtain the failure block, no details about the block theory will be described here. The assumed mechanisms of sliding failure along the joint plane and tensile failure on the rock mass, preventing the movement of the wedge, are depicted in Fig. 2-3.

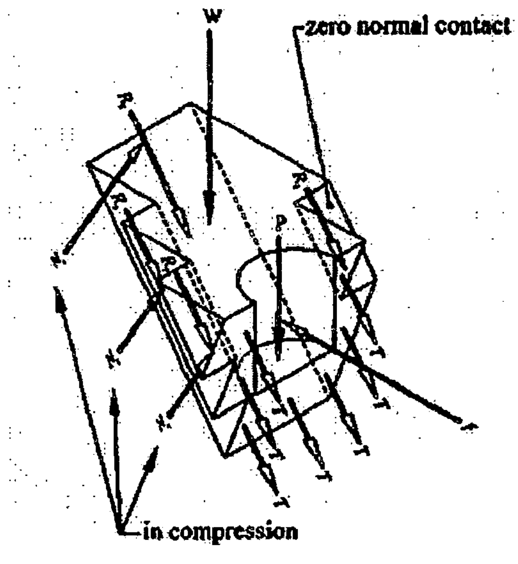

Fig. 2-3 Typical forces on wedge (To et al. (2003)

Where W = weight of the wedge; P = axial load of shaft; F = lateral force; T = tensile force due to the fracture of Category II blocks, which is defined as a block that is not removable, but becomes removable if it breaks due to the lateral force exerted by the pier; N1 = normal force on joint; and R1 = tangential force on joint. Normal force N and tangential force R can be related by the Coulomb failure criterion as follow:

$$\tau = c + \sigma \tan \varphi \tag{2-35}$$

where τ = shear stress; c = cohesion; σ = normal stress; φ = friction angle. Static limit equilibrium relation between the forces on the wedge (Fig. 2-3) was used to solve for ultimate lateral force F.

2.2.5 Carter and Kulhawy (1992)

Carter and Kulhawy (1992) performed a parametric study to obtain deflection u and rotation angle θ at the shaft head. The solutions for these two variables were expressed as a function of effective Young's modulus E_e and an equivalent shear modulus G^*, by using finite element technique. The drilled shaft is idealized as a cylindrical elastic inclusion with effective Young's modulus E_e, which is defined as

$$E_e = \frac{(EI)_c}{\frac{\Pi B^4}{64}} \tag{2.36}$$

in which, $(EI)_c$ = the actual bending rigidity of the shaft; D = diameter of the drilled shaft. The rock mass is assumed to be a homogeneous, isotropic elastic material. The equivalent shear modulus is given by

$$G^* = G_r(1 + \frac{3v_r}{4}) \tag{2.37}$$

Where

$$G_r = \frac{E_r}{2(1+v_r)} \tag{2.38}$$

in which E_r = Young's modulus of rock, and v_r = Poisson's ratio of rock

From the finite element analysis performed by Carter and Kulhawy (1992), it was found that u and θ are largely dependent on the ratio of E_e / G* and the ratio of the shaft socket length to the diameter L/D. Two categories of shafts, flexible and rigid, were classified by the authors. A flexible pile is one in which the following condition meets:

$$\frac{L}{D} \geq \left(\frac{E_e}{G^*}\right)^{\frac{2}{7}} \quad (2.39)$$

For a flexible drilled shaft, ground-line deflection u and rotation θ induced by the lateral load H and the moment M at shaft top are calculated from the following equations:

$$u = 0.50\left(\frac{H}{BG^*}\right)\left(\frac{E_e}{G^*}\right)^{\frac{-1}{7}} + 1.08\left(\frac{M}{G^*B^2}\right)\left(\frac{E_e}{G^*}\right)^{\frac{-3}{7}} \quad (2.40)$$

$$\theta = 1.08\left(\frac{H}{B^2G^*}\right)\left(\frac{E_e}{G^*}\right)^{\frac{-3}{7}} + 6.40\left(\frac{M}{G^*B^3}\right)\left(\frac{E_e}{G^*}\right)^{\frac{-5}{7}} \quad (2.41)$$

A drilled shaft is considered to be rigid when

$$\frac{L}{D} \leq 0.05\left(\frac{E_e}{G^*}\right)^{\frac{1}{27}} \quad (2.42)$$

For a rigid drilled shaft, ground-line deflection u and rotation θ are calculated from the following equations

$$u = 0.40\left(\frac{H}{BG^*}\right)\left(\frac{2D}{B}\right)^{\frac{-1}{3}} + 0.3\left(\frac{M}{G^*B^2}\right)\left(\frac{2D}{B}\right)^{\frac{-7}{8}} \quad (2.43)$$

$$\theta = 0.30\left(\frac{H}{B^2G^*}\right)\left(\frac{2D}{B}\right)^{\frac{-7}{8}} + 0.8\left(\frac{M}{G^*B^3}\right)\left(\frac{2D}{B}\right)^{\frac{-5}{3}} \quad (2.44)$$

For the drilled shafts having intermediate rigidity, the authors suggested that the displacements be taken as 1.25 times the larger displacements of those calculated values by treating the shaft as a flexible or a rigid shaft

For ultimate capacity of rock-socketed drilled shafts, Carter and Kulhawy (1992) proposed a solution in which they suggested that the lateral resistance were derived from side shear τ between shaft and rock, and frontal compressive strength of rock. The authors further suggested that the magnitude of this shear was equal to that produced in axial loading. The assumed distribution of ultimate resistance along the shaft is shown in Fig. 2-4, from which one can see that lateral resistance is equal to $\tau_{max}D$ at the surface of the rock and is increasing linearly with depth to a magnitude of $(P_L + \tau_{max})D$ at a depth of 3D.

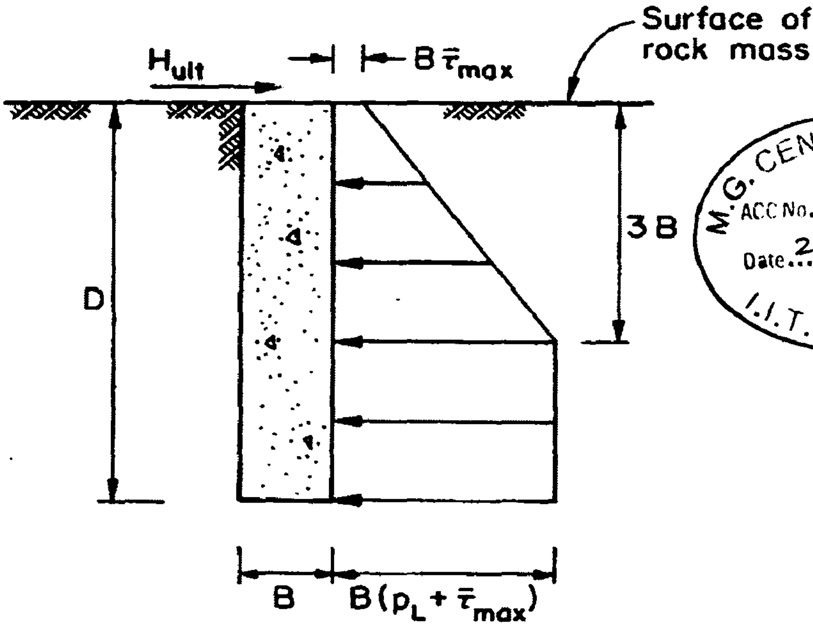

Fig. 2-4 Distribution of ultimate lateral force per unit length (after Carter and Kulhawy 1992)

Below this depth, the ultimate resistance remains constant with depth. P_L is the limit stress developed in rock, which can be calculated according to the expansion theory of a long cylindrical cavity in an elasto-plastic, cohesive-frictional, dilatant material (Carter et al. 1986). The following equations were derived by Carter and Kulhawy to determine the lateral capacity of rock-socketed drilled shafts, Hu:

$$H_u = (\frac{D\,P_L}{6} + B\,\tau_{max}) \qquad \text{for } D<3B \qquad (2.45)$$

$$H_u = (\frac{P_L}{2} + \tau_{max})3B^2 + (P_L + \tau_{max})(D-3B)B \qquad \text{for } D>3B \qquad (2.46)$$

where τ_{max} = maximum unit side resistance given as

$$\tau_{max} = 0.20(\sigma_{ci})^{0.5} \text{ (MPa)} \qquad \text{for smooth socket} \qquad (2\text{-}47a)$$

$$\tau max = 0.80(\sigma_{ci})^{0.5} \text{ (MPa)} \qquad \text{for rough socket} \qquad (2\text{-}47b)$$

B = diameter of the drilled shaft; and, D =length of drilled shafts embedded in rock.

2.3 Comments on the existing analysis methods

2.3.1 Zhang et al. (2000)

The yielding of rock mass was considered in Zhang et al. (2000) method, however, the elasto-perfectly plastic stress-strain relationship cannot fully represent the nonlinear behavior of rock masses. The actual nonlinear behavior may already appear before the stress reaches the peak. For weak rock mass, brittle post-failure may not occur. However, for good quality rock mass, which is likely to behave as a brittle material (Hoek and Brown , 1997), the stress will drop after it reaches the peak. Therefore, despite that yielding of rock was considered in this method, the behavior of different types of rock mass was not fully represented as an elasto-perfectly plastic model.

2.3.2 To et al. (2003)

The determination of removable wedge required by this method is a very tedious procedure which involves the use of AutoCAD and EXCEL. Furthermore, only two sets of joints can be considered and the joints in each set should be parallel. The failure mode is restricted to failure at the top portion of rock mass with a free surface at the ground-line.

2.3.3 Existing p-y criteria for rock

Reese (1997)'s interim rock p-y criterion was not well calibrated due to inadequate test data. The failure modes of rock mass were not well defined; and an estimation of ultimate lateral capacity did not include the effect of friction between rock mass and shaft. Determination of such parameters as constant k_{rm} appears to be empirical. Cho et al. (2001) conducted a lateral load test on rock-socketed drilled shafts. Two drilled shafts, 30 inch in diameter and 10 feet to 13 feet of socket were laterally loaded. Reese (1997)'s interim rock p-y criterion for weak rock was used to construct p-y curves. It was found that the interim p-y curves underestimated the deflections of shafts when comparing with the measured values.

Although the interim rock p-y criterion was not well established, the p-y method is still a promising method for rock-socketed drilled shaft under lateral loads. Gabr (1993) evaluated two field tests on rock-socketed drilled shafts under lateral loads presented by Carter and Kulhawy (1992) by using p-y analysis and the p-y criterion for stiff clay (Reese et al. 1975). The analysis results showed that p-y approach is analytically attractive because it can approximately model the nonlinearity in the load-displacement response. Gabr et al. (2002) p-y criterion is a most recently proposed p-y criterion for weak rock. However, it has not been further validated with other load tests. The equation for estimating modulus of subgrade reaction was based on Vesic (1961)'s equation for beam on elastic foundation. This is not same as the case of drilled shafts embedded in rock where shaft-rock interaction is more complicated.

2.3.4 Carter and Kulhawy (1992)

Carter and Kulhawy (1992) provide solutions for the lateral load-deflection relation at shaft head as well as shaft lateral capacity. For load-deflection prediction, Carter and Kulhawy (1992) assumed rock mass as an elastic material, which implies that the solution is only applicable to small loads. The solution for ultimate lateral capacity needs verification. One of the drawbacks of the solution is the requirement of numerous rock deformation and strength parameters, such as shear modulus, cohesion, friction angle, and dilation angle.

DiGioia and Rojas-Gonzalez (1993) evaluated this method by using their field lateral load test on drilled shafts socketed into rock mass. They found a reasonable agreement between the measured and predicted displacements for these foundations at low load levels (20-30% capacity). However, this method gave predictions that were stiffer than observed at higher load levels. Additionally, Dykeman and Valsangkar (1996) conducted a centrifuge test on eight model socketed shafts and used the test results to evaluate Carter and Kulhawy's (1992) method. The comparison showed that Carter and Kulhawy (1992)'s method tend to overestimate the ultimate lateral capacity by a factor of two, while it predicted smaller deflection at shaft head than measured deflection at a given load level. In addition to their own test data, Dykeman and Valsangkar (1996) evaluated Carter and Kulhawy's (1992) method by using Frantzen and Stratten (1987)'s field test data. Similar comparison results were found for the predicted deflection at shaft head.

ESTIMATION OF UCS OF ROCK MASS BY VARIOUS METHODS

One of the most important parameters on which the lateral load carrying capacity of rock socket depend is the UCS of the rock mass which is estimated by following methods.

3.1 Concept of joint factor (Singh et al., 2002)

The influence of the joint factor on the response of the intact rock was studied through a weakness coefficient factor called Joint Factor (Ramamurthy and Arora, 1994). This coefficient reveals the "weakness" brought in to the intact rock through jointing and takes into account the combined effect of frequency of joints, their inclination and roughness along the critical joints. The higher the joint factor, the greater is the "weakness". It is defined taking the three key factors controlling the response of the jointed mass into account.

$$J_f = \frac{J_n}{nr} \quad (3.1)$$

where, J_f = Joint Factor,

J_n = number of joints per meter depth in the direction of loading.

n = critical joint inclination parameter presented in Table 3.1. The parameter was derived by conducting experiments on specimens with inclined joints (Ramamurthy, 1993).

Table 3.1 Joint inclination parameters (joint orientatin β° w.r.t direction of loading) (Ramamurthy, 1993).

Orientation of joint, β°	Inclination parameter, n	Orientation of joint, β°	Inclination parameter, n
0	0.81	50	0.306
10	0.46	60	0.465

20	0.105	70	0.643
30	0.046	80	0.814
40	0.071	90	1

r = sliding joint strength parameter = $\tan\varphi_j$; where φ_j is friction angle along the critical joint at sufficiently low normal stress so that the initial roughness of the surface is reflected through this value. The value of 'r' is presented in Table 3.2

Table 3.2 Value of 'r' for UCS of intact rock (Ramamurthy,1994)

UCS of intact rock (MPa)	**< 50**	**50-100**	**>100**
Roughness Parameter r	0.8	0.9	1.0

Figure 3.1 shows a sample of rock mass in which

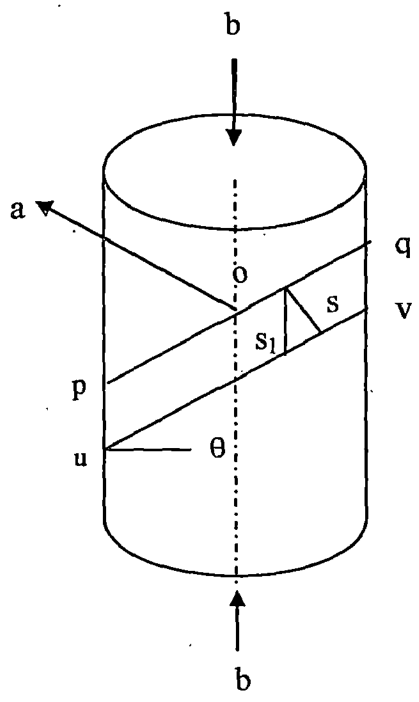

Fig 3.1 A typical rock with joints

s= spacing between the joint,

θ = inclination of joint,

pq and uv = joint planes,

oa = normal to joint plane,

bb = loadin direction,

s_1 = spacing between joints in the direction of loading = $\frac{s}{cos\theta}$, then

$$J_n = \frac{100}{S_1} = \frac{100\ Cos\ \theta}{S} \tag{3.2}$$

The modulus of jointed rock mass for different failure criteria is given as (Singh et.al.,2002).

$$E_j = E_i \exp(-0.020J_f) \tag{3.3a}$$

for splitting mode of failure and shearing mode of failure

$$E_j = E_i \exp(-0.040J_f) \tag{3.3b}$$

for rotational , mode of failure

$$E_j = E_i \exp(-0.035J_f) \tag{3.3c}$$

for sliding mode of failure.

Average value of all the failure modes is considered in present study which is

$$E_j = E_i \exp(-0.035J_f) \tag{3.3d}$$

$$G_r = \frac{E_j}{2(1+\nu)} \tag{3.4}$$

where, E_i and E_j are the Youngs modulus of intact and jointed rock respectively,

G_r = Shear modulus of rock mass,

ν = poissions ratio.

The acute angle 'δ' between normal to joint plane and direction of major principal stress (σ_1)shown in Fig 3.2 is given by the equation

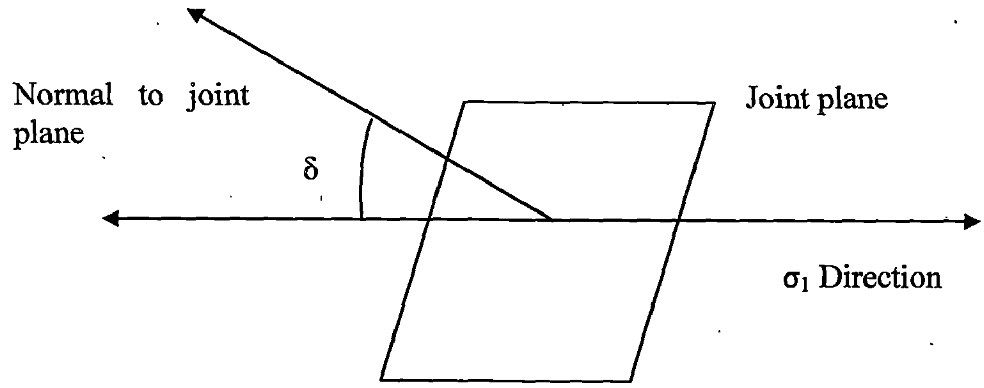

Fig 3.2 Angle between normal to joint plane and sample line

$$\cos\delta = (\cos\alpha_n - \cos\alpha_s)\cos\beta_n\ \cos\beta_s + \sin\beta_n\ \sin\beta_s \qquad (3.5)$$

where, α_s and β_s are the dip direction and dip of the sample line or direction of major principal stress, α_n and β_n are the dip direction and dip of the of the normal to the joint plane.
β^o = inclination of joint w.r.t direction of loading. From Fig 3.1, $\theta = \delta$ so,
$\beta^o = (90 - \delta^o)$ and

$$J_n = \frac{100}{S_1} = \frac{100 \cos\delta}{S} \qquad (3.6)$$

Based on the results of uniaxial and triaxial tests of intact and jointed specimens, Ramamurthy (1994) proposed the following empirical relation between unconfined compressive strength ratio σ_{cj}/σ_{ci} and joint factor J_f

$$\sigma_{cj} = \sigma_{ci} \exp(-0.008J_f) \qquad (3.7)$$

where, σ_{cj} = UCS of the jointed rock and,

σ_{ci} = UCS of the intact rock.

3.2 Singh et al. (1997)

The following expression for rock mass strength has been suggested by authors based on quality index Q

$$\sigma_{cj} = 7\gamma Q^{\frac{1}{3}} \text{ MPa} \tag{3.8}$$

Where,

σ_{cj} = rock mass strength in MPa,

γ = unit weight of rock (gm/cc),

Q = Barton's rock mass quality index.

Q is based on six parameters: RQD, number of joint or discontinuity sets (J_n), joint roughness (J_r), joint alteration (J_a), water flow (J_w) and a stress reduction factor (SRF). Because this system is mainly used for tunneling applications in Europe, the details to quantify these parameters will not be discussed here.

The rock mass quality (Q) is defined as:

$$Q = \left(\frac{RQD}{J_n}\right)\left(\frac{J_r}{J_a}\right)\left(\frac{J_W}{SRF}\right)$$

3.3. RMR (Ramamurthy et al. (1985)

(Ramamurthy et al. (1985) proposed a empirical relation between RMR (Rock Mass Rating) and UCS of the rock mass

$$\sigma_{cj} = \sigma_{ci}\, e^{\frac{(RMR-100)}{18.75}} \tag{3.9}$$

It should be noted that when a rock mass classification system is used for estimating rock mass strength (and deformation properties), only the inherent parameters of intact rock and discontinuities need be considered for evaluation of the classification index. Other parameters such as groundwater and in situ stress should not be considered to modify the classification index because they are considered in the analysis of rock structures. For example, when RMR is used for rock mass strength estimation, the rock mass should be assumed completely dry and a very favourable discontinuity orientation should be assumed.

Based on the joint factor J_f Ramamurthy (2010) gave approximate values for RMR and Q which are presented in Table 3.3

Table 3.3 Values of RMR and Q based on joint factor (Ramamurthy,2010)

Joint factor	RMR	Q
0	100	2154
100	80	100
200	60	4.64
300	40	0.215
400	20	0.010
500	0	0.000464

3.4 RQD (Zhang ,2010)

Since in many cases, rock quality designation (RQD) is the only information available for describing rock discontinuities, the following empirical relation was developed by Zhang (2010) for estimating rock mass strength based on RQD.

$$\sigma_{cj} = \sigma_{ci}\ 10^{(0.013RQD-1.34)} \qquad (3.10)$$

where, σ_{cj} = UCS of the jointed rock,

σ_{ci} = UCS of the intact rock.

Preist ans Hudson(1976) suggested following general equation between RQD and Joint number J_n

$$RQD = 110.4 - 3.68J_n \qquad (3.11)$$

3.4 Strength reduction factor (SRF) (Singh and Rao, 2005)

The most reliable results are available only from filed test. The best way to estimate the rock mass strength could have been to stress the rock mass up to failure. It is extremely difficult if not impossible to make the rock mass fail in the field especially during design stage. However the modulus of the rock mass may be obtained without loading the rock mass up to failure. Singh and Rao (2005) have shown that the reduction in modulus of rock due to presence of joints is strongly correlated with reduction in strength of rocks due to joints. Following expression was found to hold good

$$\text{SRF} = (\text{MRF})^{0.63} \qquad (3.12)$$

Where $\text{SRF} = \frac{\sigma_{cj}}{\sigma_{ci}}$

where, σ_{cj} = UCS of the jointed rock,

σ_{ci} = UCS of the intact rock,

MRF = modulus reduction factor $= \frac{E_e}{E_j}$, E_e = Modulus of the elasticity of rock mass in the field,

E_i = Modulus of the elasticity of intact rock in the field.

MRF values for various types of rocks given by Ramamurthy (2010) are presented in Table 3.4

Table 3.4 Values of MRF for various types of rock (Ramamurthy, 2010)

Serial No.	Rock Type	MRF
1	Sandstone	0.259-0.429
2	Claystone	-
3	Slate	0.049-0.039
4	Xenolith	0.2
5	Traprock	0.16-0.357
6	Shale	0.206-0.273

7	Limestone	0.047-0.407
8	Metabasic	0.209=0.317
9	Quartzite	0.035-0.289
10	Phyllite	0.109-0.584

METHODOLOGY TO CALCULATE LATERAL LOAD OF ROCK SOCKET

4.1 Equivalent material modeling approach

Figure 4.1 represents the cases where either rock is at ground surface or the lateral loading on the shaft at the level of the rock surface can be specified completely. The shaft is idealized as cylindrical elastic inclusion, with an effective young modulus (E_e), Poisson ratio (ν_c), depth (D), and diameter (B).It is assumed that elastic shaft is embedded in a homogenous, isotropic elastic rock mass, with properties E_r and ν_r. At the surface of the rock mass, it is subjected to known lateral (horizontal) force (H) and overturning moment.

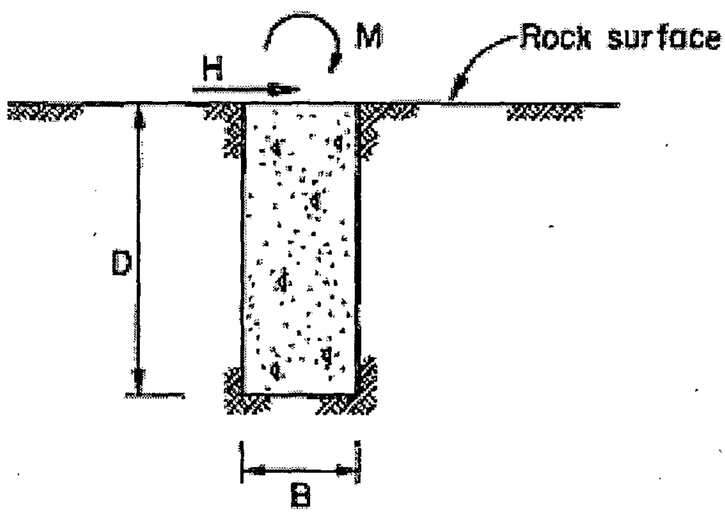

Fig. 4.1 Lateral loading of a rock-socketed shaft (Carter and Kulhawy 1992)

Carter and Kulhawy (1992) have presented an approximate theoretical approach for estimating the ultimate lateral capacity. It was assumed that the shaft section has sufficient moment and shear capacity to resist the applied loading, and the ultimate failure of the shaft occurs when the surrounding rock mass is not able to sustain any further lateral loading, similar to the so called short pile failure described by Broms(1964a,1964b).

When a lateral load is applied at the rock surface, the rock mass immediately in front of the shaft will undergo zero vertical stress, while horizontal stress is applied by the leading face of the shaft. Ultimately, the horizontal stress may reach uniaxial compressive strength of rock mass and, with further increase in lateral load, the horizontal stress may decreases as the rock mass softens during post peak deformation. Large lateral deformations may be required for the rock mass at the depth to exert a maximum reaction stress on the leading face of the shaft. Therefore, it is reasonable to assume that reaction stress at the rock mass surface, in the limiting case of loading of the shaft, is zero or very nearly zero as result of post peak softening. Along the sides of the shaft, some shearing resistance may be mobilized, and this is likely to be approximately the same as the maximum unit side resistance under axial compression, τ_{max}.

At greater depth, it is reasonable to assume that the stress in front of the shaft may increase from initial in situ horizontal stress level, σ_{hi} , to the limit stress P_L, reached during the expansion of a long cylindrical cavity i.e. the plane strain condition will apply. Behind the shaft the horizontal stresses will decrease, and after tensile rupture of the bond between the concrete and the rock mass, the horizontal stress will reduce to zero. At the sides of the shaft, some shearing resistance may also be mobilized. Therefore, at depth, the ultimate force per unit length resisting the lateral loading is likely to be about $B(P_L + \tau_{max})$. Where B, is the diameter of the shaft.

Closed form solutions have been found for the limit stresses developed during the expansion of a long cylindrical cavity in an elasto-plastic, cohesive-frictional, dilatent material. This limt stress P_L can be determined from the graph shown in Fig 4.2. The central vertical axis on each plot indicates the ratio of the plastic radius at limit condition (R) to the cavity radius (a).

4.2 Computer program to calculate the lateral load

A software program has been written using c language (refer appendix 1) which calculates the lateral load of rock socket (H). The steps involved in software program are presented below.

4.2.1 Calculation of UCS of rock mass

Based on the horizontal angle interval which is one of the input parameter the software initially calculates the acute angle δ between the normal to joint plane and loading line using equation 3.5 for one particular joint set. Then the spacing between the joint set in the direction of loading (s_l) and using s_l number of joints per meter depth (J_n) in the direction of loading will be calculated. Finally using all the above parameters joint factor (J_f) is calculated. In the similar way depending up on the horizontal angle interval (say 30^0) and the number of the joint sets (say 3) the program calculates J_f value in 12 different directions (i.e. 360/30 = 12) for each joint set so, for 3 joint sets there will be 36 different J_f values. As the J_f value indicates the weakness bought into intact rock due to presence of joint the higher value of J_f the higher will be weakness of the rock mass. So, in the calculation of the UCS of the rock mass maximum of all the calculated J_f will be used by the software. The UCS of the rock mass can also be calculated using other methods presented in previous chapter.

4.2.2 Calculation of strength parameters

The Mohr–Coulomb shear strength criterion is the most widely used criterion for jointed rocks. But, in its present form there are two major limitations of this criterion firstly it considers the strength response to be linear, and, secondly the effect of the intermediate principal stress on the strength behavior is ignored. Singh and Singh (2012) overcame these two major limitations and suggested the following modified non-linear form of Mohr–Coulomb strength criterion.

$$\text{Sin}\varphi_{jo} = \frac{(1-SRF)+\frac{sin\varphi_{io}}{1-sin\varphi_{io}}}{(2-SRF)+\frac{sin\varphi_{io}}{1-sin\varphi_{io}}} \quad (4.1)$$

where, SRF= Strength reduction factor calculated from equation (3.12), $\text{Sin}\varphi_{io}$ = friction angle of the rock, which can be calculated in laboratory from triaxial strength test and, $\text{Sin}\varphi_{jo}$ = friction angle of the rock mass, at very low confining pressure. ($\sigma_3 \sim 0$)

$$\sigma_1 - \sigma_3 = \sigma_{cj} + \frac{2\text{Sin}\varphi jo}{1-\text{Sin}\varphi jo}\sigma_3 - \frac{\sigma_3^2}{\sigma_{ci}}\frac{2\text{Sin}\varphi jo}{1-\text{Sin}\varphi jo} \qquad \text{for } 0 \leq \sigma_3 \leq \sigma_{ci} \quad (4.2)$$

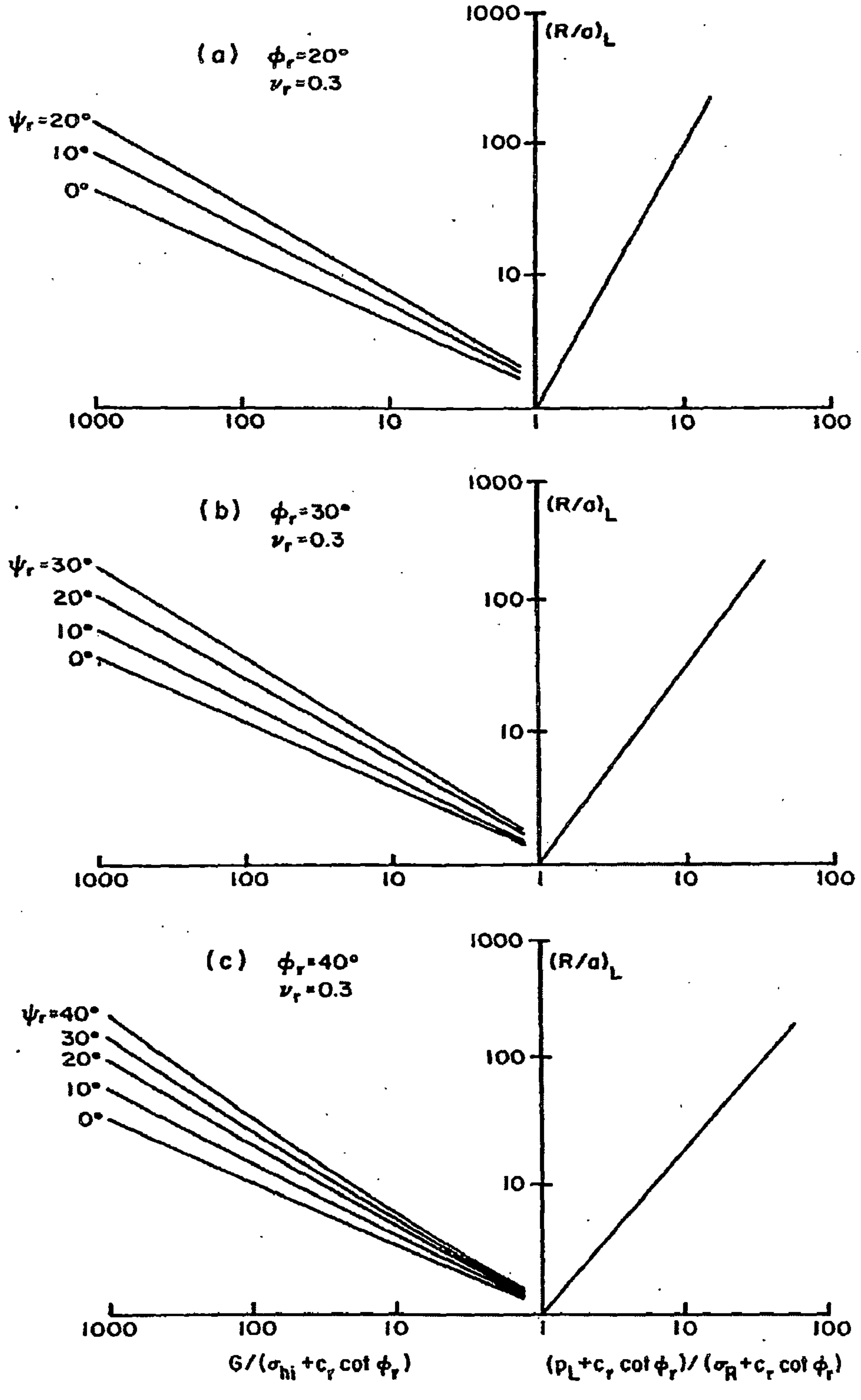

Fig.4.2 Limit solution for cylindrical cavity (Carter and Kulhawy 1992)

For calculating the shear strength parameters conventional Mohr-Coulomg criterion has been used. The program calculates 0.25 σ_{ci} and divides it into 8 equal parts to obtain different values of minimum principal stress σ_3. Then using the above equation corresponding values of major principal stress σ_1 will be calculated. Then the program calculates the shear strength parameters of rock mass by using conventional Mohr-Coulomb criteria.

Singh and Singh (2012) also suggested that the value of dilation angle (ψ_r) as

$$\Psi_r = \frac{(\Phi_{jo} - \Phi_{io})}{2} \tag{4.3}$$

The young's modulus and shear modulus of rock mass are calculated using equations 3.3d and 3.4 respectively.

4.2.3 Calculation of lateral load capacity

Using the values of the shear modulus (G_r), cohesion (c_r) and frictional angle of the rock mass (Φ_r) the program calculates the value of $G_r/(\sigma_{hi} + c_r \cot\Phi_r)$ for the corresponding curve of the dilation angle ψ_r shown in Fig 4.2. σ_{hi} is the in situ horizontal stress, in most of practical cases it will be small compared to cohesion so, for the simplicity its value is taken as zero. Then the program works in clockwise direction and finally calculates the value of the $(P_L + c_r \cot\varphi_r)/(\sigma_r + c_r \cot\varphi_r)$. From which the value of the limiting stress P_L can be calculated. And σ_r is a constant given the following equation.

$$\sigma_r = \left[\left(\frac{2N}{N+1}\right)(\sigma_{hi} + c_r cot\Phi_r)\right] - c_r cot\Phi_r \tag{4.4}$$

where, $N = \frac{1+sin\psi_r}{1-sin\psi_r}$ (4.5)

One final problem is to determine the stress at which limit stress is mobilized. Carter and Kulhawy (1992), suggested that, in a cohesive material, the depth at which limit stress is mobilized would be about 3 shaft diameters (3B).Therefore, the proposed distribution of ultimate force per unit length resisting the shaft is shown in Fig 2.4. Finally the lateral load carrying capacity of the rock socket is calculated by the program using the equations (2.53) and (2.54).

4.3 Input parameters

UCS of the intact rock in (σ_i) = 250 MPa,

Cohesion of the intact rock (C_i) = 10 MPa,

Type of the rock = Quartzite,

Friction angle of the intact rock (φ_i) = 30 degrees,

Youngs modulus of the intact rock (E_i) = 55 GPa

Following are the joint sets observed for the dam site (Table3.5) whose intact rock properties are mentioned above

Table 4.1 Joint set parameters

Joint set	Dip	Dip Direction	Spacing of joints (m)
J1	65	210	1
J2	45	180	0.60
J3	30	000	0.45

It is assumed in the analysis that load carrying capacity is governed by rock mass and the RCC has much higher strength and has not yielded or failed.

RESULTS AND DISCUSSION

5.1 Variation of UCS of rock mass with joint sets.

Figure 5.1,5.2 and 5.3 shows a radar graph which indicates the extent of anisotropy induced in the rock mass due to joint set J-1, J-1 and J-2, J-1, J-2 and J-3 respectively. The intact rock UCS value of 250 MPa has decreased to 116.37 MPa in rock mass with joint J-1 and the UCS value further decreased to 62.7 MPa and 45.69 MPa in rock mass with two joints(J-1 and J-2) and three joints (J-1,J-2 and J-3).

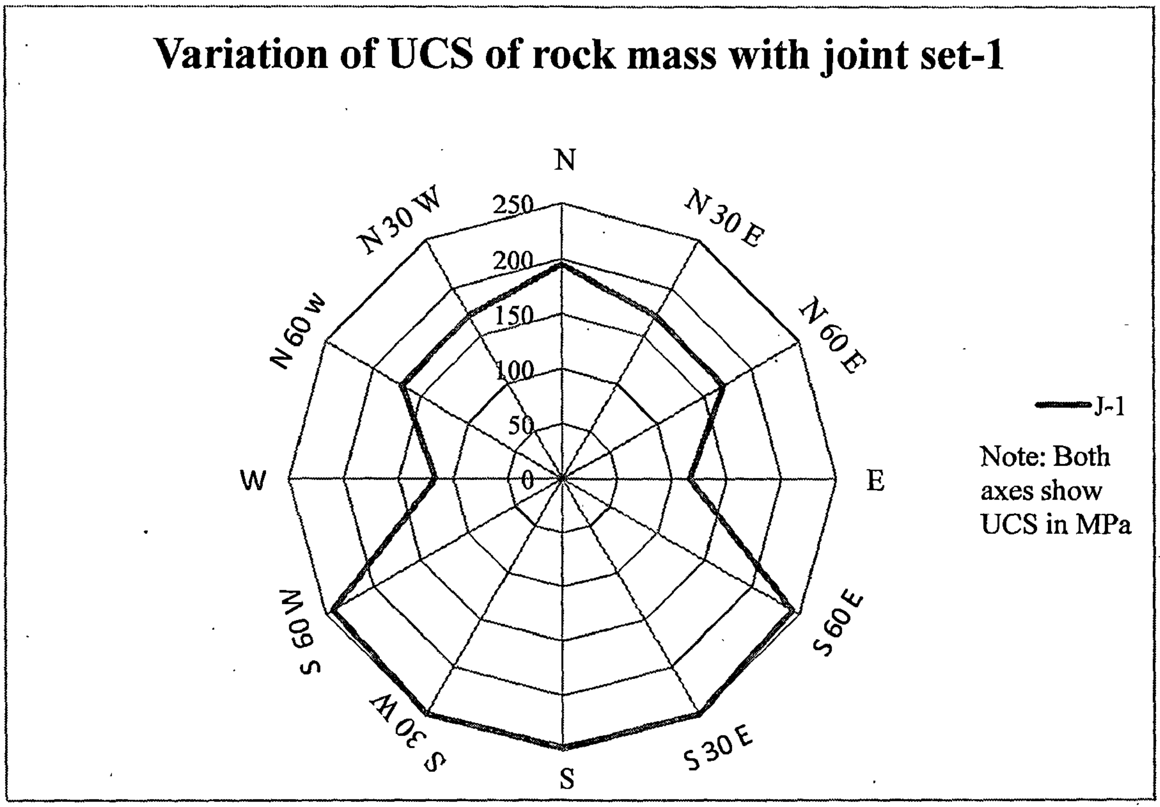

Fig.5.1 Variation of UCS of rock mass having joint Set J-1

5.2 Variation in lateral load capacity of rock socket with socket dimensions for given UCS

For a given UCS of intact rock (250MPa) shear strength parameters of jointed rock mass has been calculated by using modified Mohr-Coulomb criteria (Singh et al, 2012) . The cohesion, friction angle and dilation angle of the rock mass were found to be 12.89MPa, 37.560 and 3.780 respectively. Also elastic shear modulus which is indirectly related to UCS of jointed rock mass is found to be 10.06 GPa. Based on the above parameters and poission ratio of 0.3 the lateral load carrying capacity of rock socket (H) has been calculated for L/D (L=length of shaft, D=diameter of shaft) ratios of 2, 3 and 4 for diameter of socket equal 0.6, 0.9,1.2 and 1.5 meter. The values of H are presented in table 5.1. The variation in H with shaft dimensions shown in Fig 5.4, 5.5 and 5.6 for the given intact rock UCS value of 250, 150 and 50 MPa respectively.

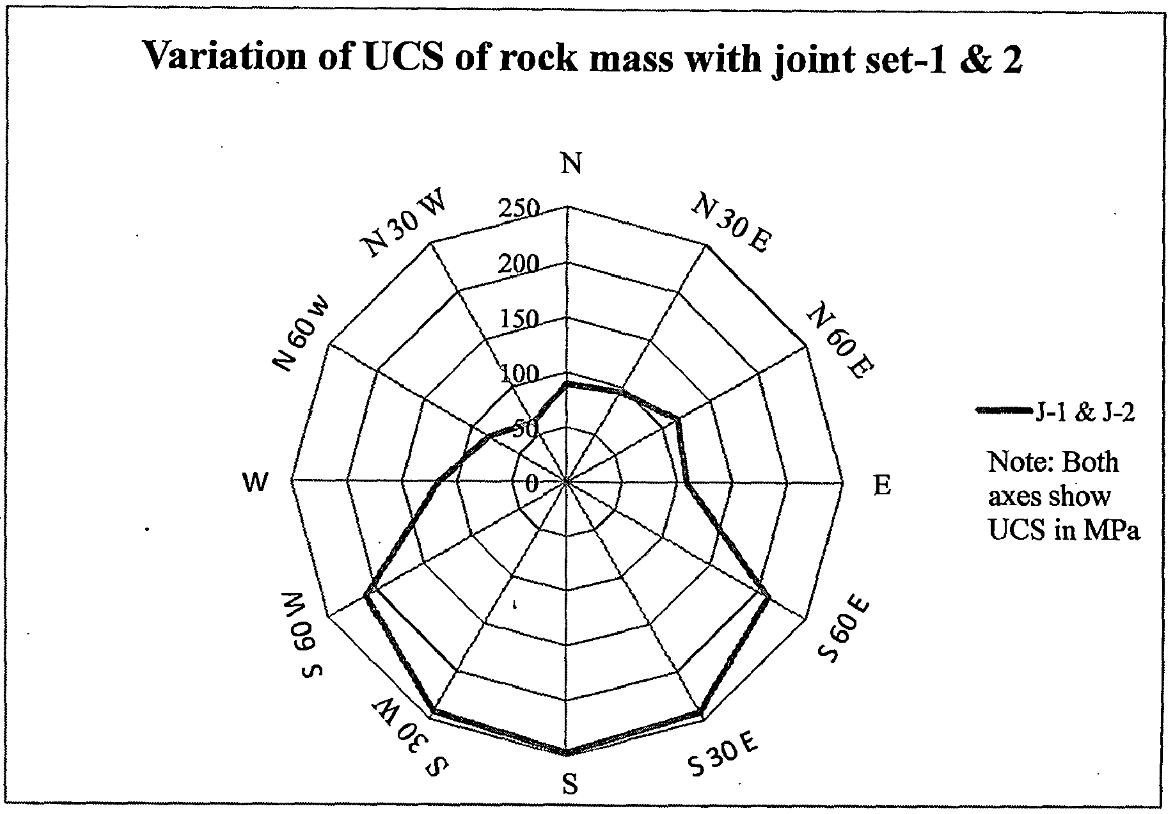

Fig.5.2 Variation of UCS of rock mass having joint set J-1 and J-2

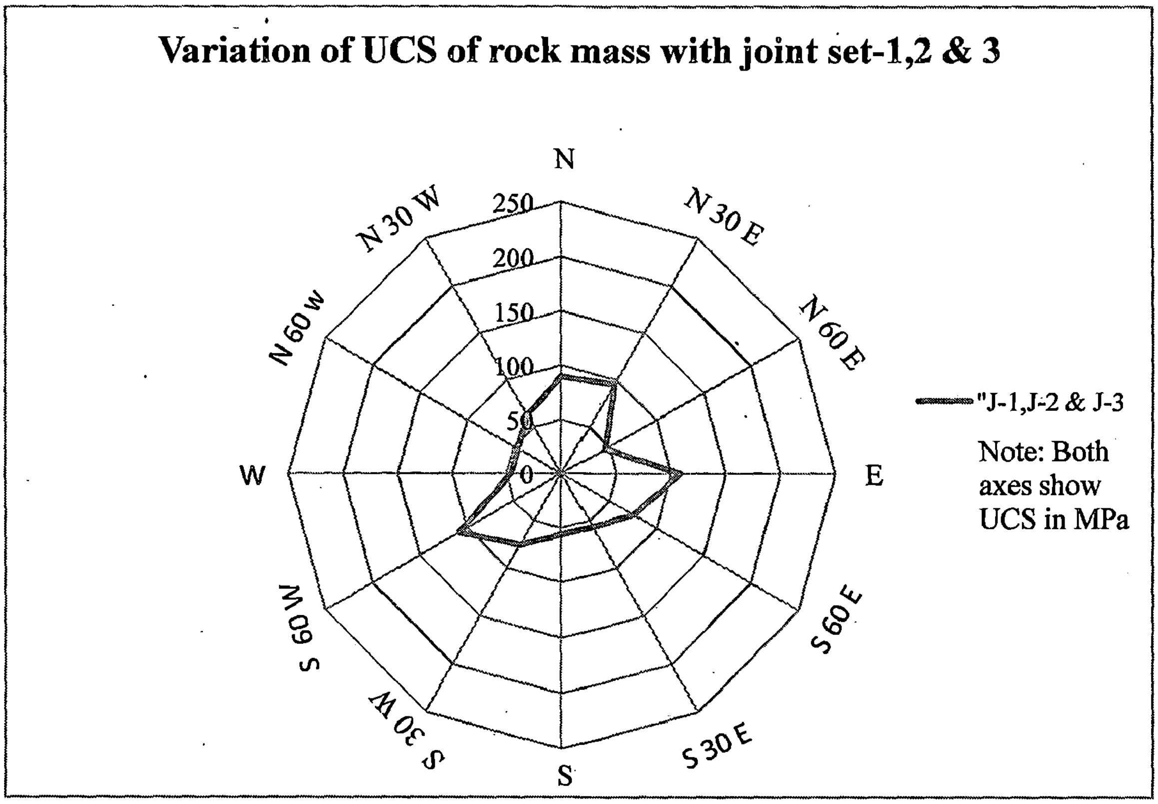

Fig.5.3 Variation of UCS of rock mass having joint set J-1,J-2 and J-3

Table 5.1 Lateral load Capacity of rock sockets for different diameters and L/D values

Diameter (D) m	L/D = 2	L/D =3	L/D =4
0.6	396	564	1670
0.9	565	1269	3161
1.2	1005	2256	5087
1.5	1570	3525	7451

5.3 Variation in lateral load capacity of rock socket with UCS at a given diameter.

The variation in H at a given diameter of socket equal to 0.6 m for L/D ratio of 2, 3 and 4 is shown in Fig 5.7. It has been found that the percentage increase in the H for change L/D from 2 to 3 when compared to change L/D from 3 to 4 is approximately equal at lower value of UCS. But at the higher value of UCS the increase in L/D is more when increased from 3 to 4 than when increased from 2 to 3.

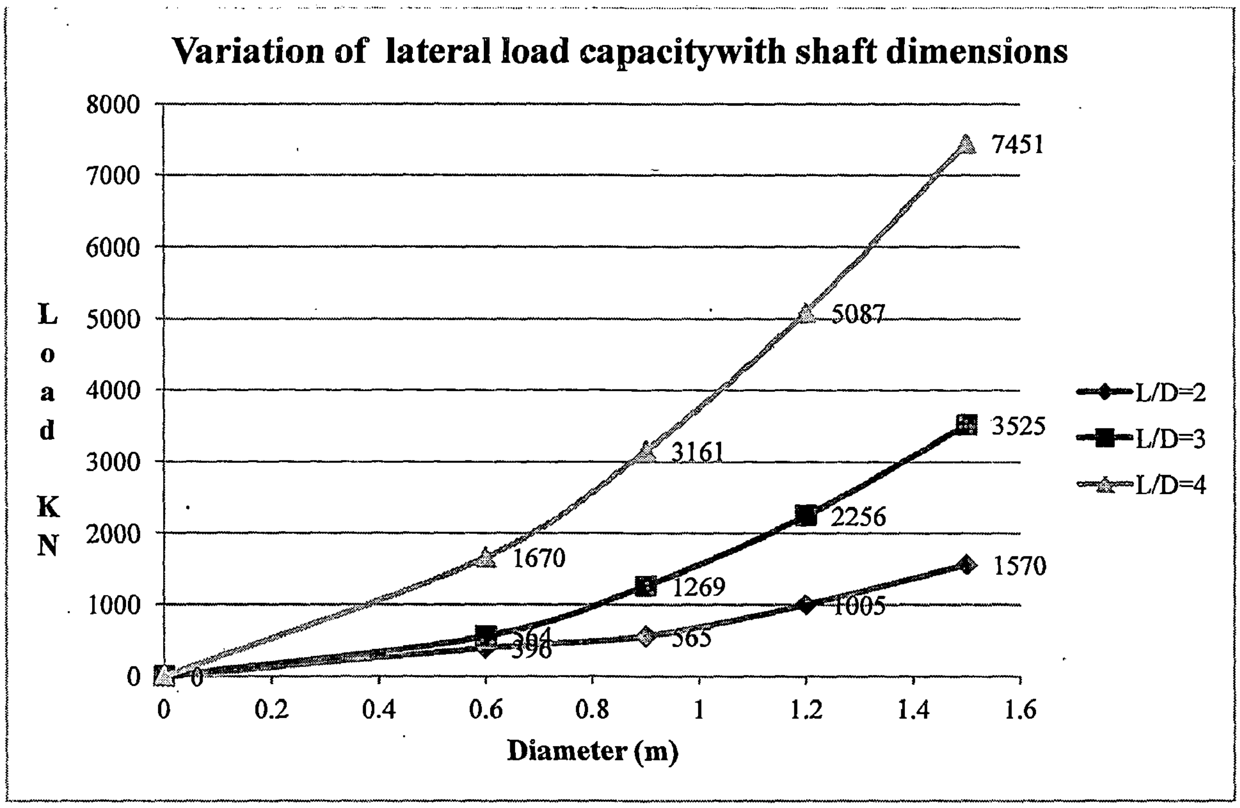

Fig 5.4 Variation of lateral load capacity with shaft dimensions for (σ_{ci} = 250 MPa)

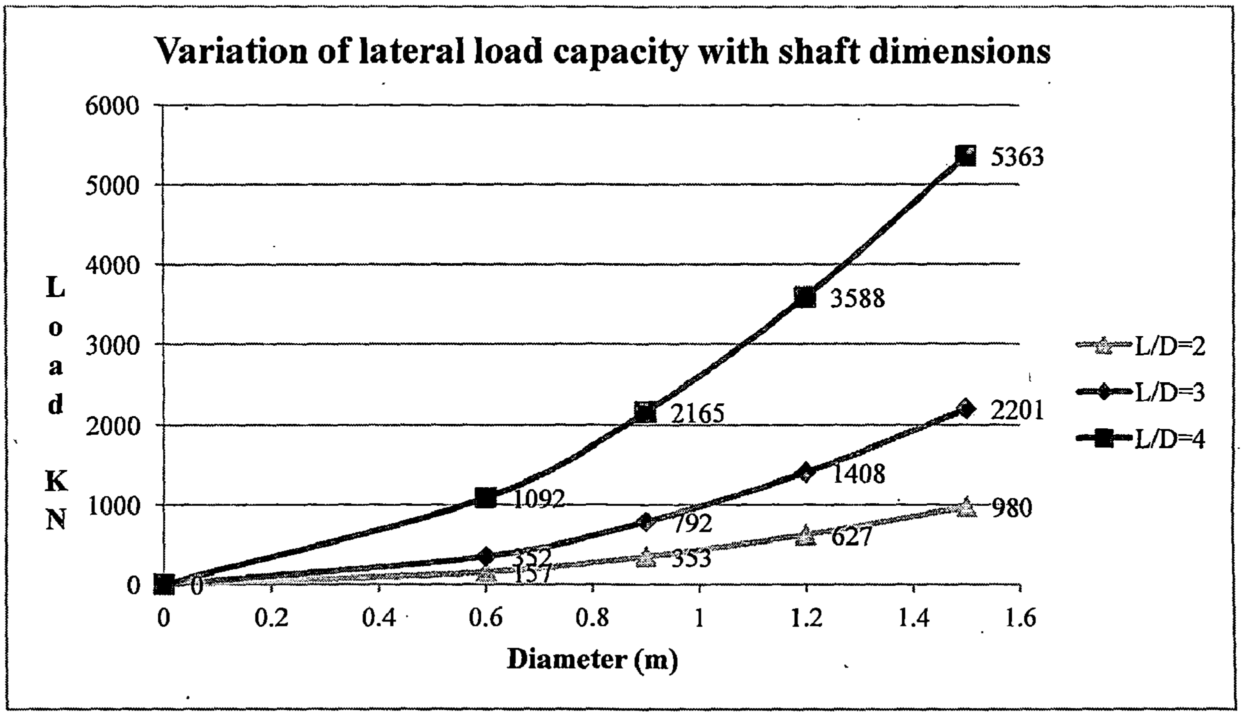

Fig 5.5 Variation of lateral load capacity with shaft dimensions for (σ_{ci} = 150 Mpa)

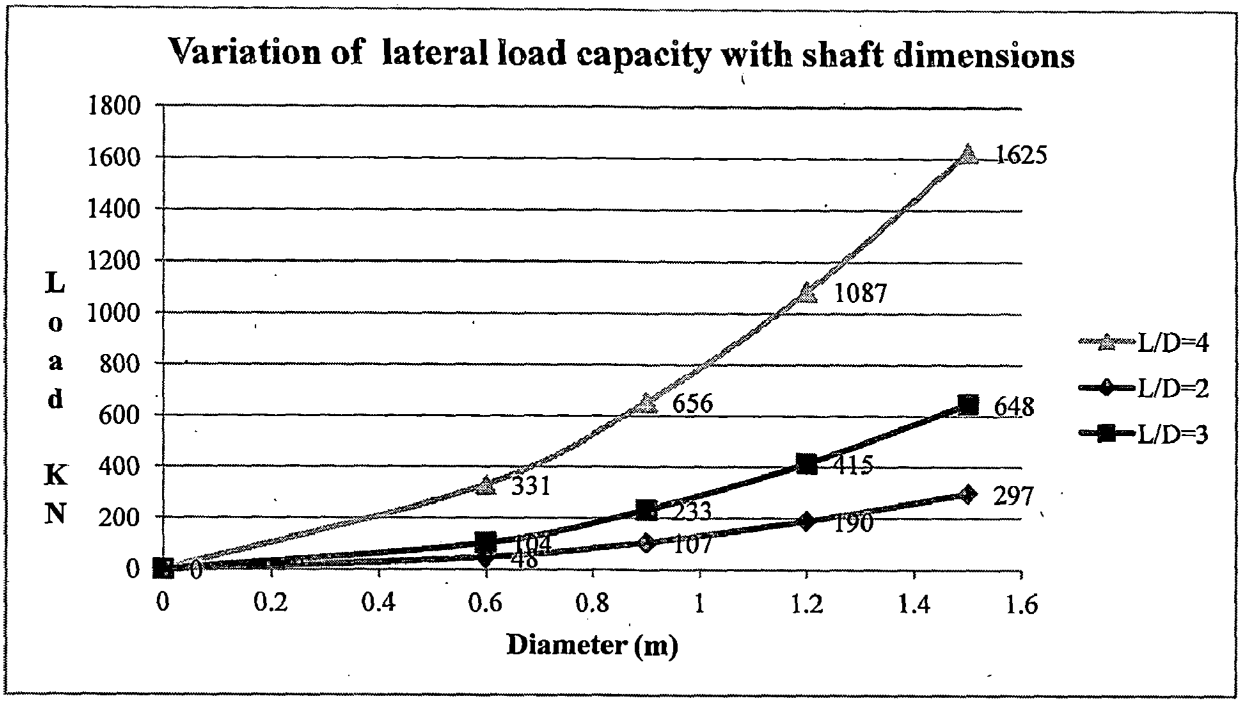

Fig 5.6 Variation of lateral load capacity with shaft dimensions for (σ_{ci} = 50 MPa)

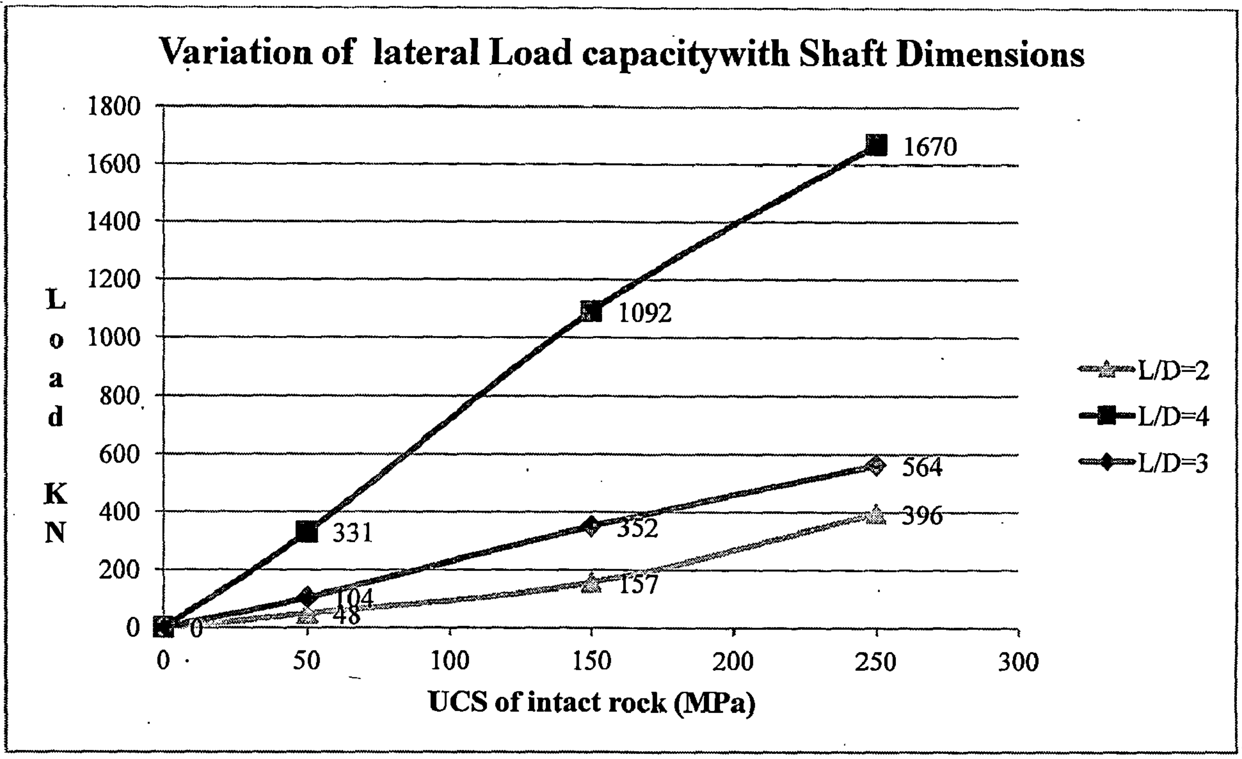

Fig 5.7 Variation in lateral load capacity with UCS

5.4 Variation in lateral load capacity with different methods.

Zhang (2000) has given an excellent discussion on assessing rock mass strength based on filed data generally available on the project sites. In actual filed condition there is always paucity of reliable estimates of design parameters and it is rightly stated that in filed it is sometimes only RQD which is available with the designer. Keeping in view that field testing is extremely expensive during design stage it is imperative for a designer to have idea about the outcome if various approaches are used while designing the structure. In the present study a parametric analysis has been carried out and lateral load carrying capacity of rock socket (H) has been computed using various approaches to have a better insight into the outcome of different approaches. H has been computed by employing Joint factor, Q system, RMR, SRF and RQD in estimation of rock mass strength. The results have been plotted in the form of H versus L/D ratio and the same are presented in Fig 5.8.

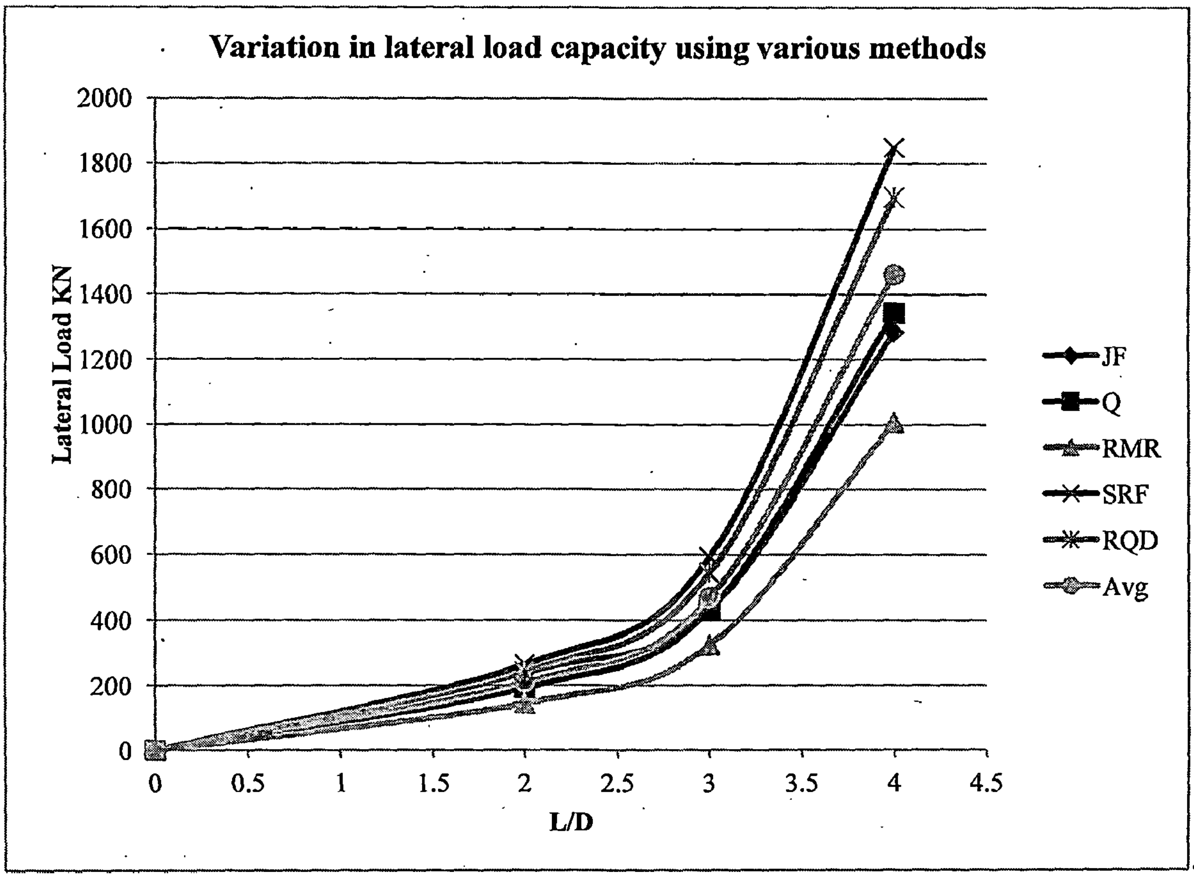

Fig 5.8 Variation in lateral load capacity with different methods

It has been found that upto L=3D the maximum value of H equal to 541 KN is obtained when SRF method is used and minimum value of 217 KN was obtained using RMR method is used where the average value calculated is 379 KN. The value of H obtained by SRF method is 42.74 % more than average and the value obtained by RMR is 25.34% less than the average. For L=4D it has been found that maximum value of H equal to 1680 KN is obtained when SRF method is used and minimum value of 642 KN was obtained using RMR method where average value calculated is 1176 KN. The value of H obtained by SRF method is 42.85 % more than average and the value obtained by RMR is 45.41 % less than the average. In both that cases the value of H calculated by Q system was found closest to average.

CONCLUDING REMARKS

Rocks encountered in foundations in field are invariably intercepted by discontinuities the most common being joints. Rock sockets are used to transmit the heavy loads to the rock foundation in vertical as well as lateral directions. Presence of joints make the rock weaker and the extent of weakness depends on the characteristics of the joint system. The present study has been conducted to evaluate the effect of jointing on lateral load carrying capacity of rock sockets. Equivalent continuum modeling approach has been used where in the rock mass which is actually discontinues is converted into a continuum with representative engineering properties. The various approaches i.e Joint factor, Q system, RMR, SRF and RQD have been used to assess the basic input parameter i.e UCS of the rock mass. This UCS has then been used in modified Mohr-Coulomb criteria (Singh & Singh 2012) to assess the shear strength parameters of the rock mass. These parameters are used in Carter and Kulhawy (1992) to get insight into lateral load carrying capacity of rock socket.

A computer program has been developed which incorporates all the methods and compares the lateral load of rock socket by considering the minimum out of all the dip directions. A parametric analysis has indicated UCS of the rock has decreased to 46.55% when joint set J-1 is incorporated, and it has further decreased to 25.08% when sets J-1 and J-2 are incorporated. When all the three joint sets are incorporated the UCS has further decreased to 18.23%. Also for a given UCS and diameter of 0.6 and 0.9 m, the percentage increase in the lateral load carrying capacity of rock socket (H) is more when the L/D is increased from 3 to 4, when compared to increase in L/D ratio from 2 to3. For the diameter of the shaft 1.2 m the percentage increase is approximately equal when the L/D ratio is increased from 2 to 3 and 3 to 4. But, for the diameter of shaft equal to 1.5 m the increase in load carrying capacity is more when the L/D is increased from 2 to 3 than when it is increased from 3 to 4.

A parametric analysis has also been carried out for given diameter with varying UCS. It has been found that the percentage increase in the lateral load capacity of rock socket for change in

L/D from 2 to 3 when compared to change L/D from 3 to 4 is approximately equal at lower value of UCS. But at the higher value of UCS the increase in L/D is more when increased from 3 to 4 than when increased from 2 to 3.

One more exercise was done to look into the performance of various approaches to compute UCS. It has been observed that most conservative results were obtained when RMR method is used. The value of lateral load capacity predicted by RMR method was found to be 25.34% and 45.41% lower than the average value for L=3D and L=4D respectively. The highest value were found when SRF approach was adopted and the values were 42.74% and 42.84% higher than the average load carrying capacity for L=3D and L=4D respectively. It was also observed that average values were close to those predicted by Q system.

REFERENCES

- Broms, B. B. (1964a). "Lateral resistance of piles in cohesive soils." J. Soil Mech.Found. Div., ASCE, 90(2), 27-63.
- Broms, B. B. (1964b). "Lateral resistance of piles in cohesionless soils." J. Soil Mech. Found. Div., ASCE, 90(3), 123-156.
- Carter, J. P., and Kulhawy, F. H. (1992). "Analysis of laterally loaded shafts in rock."Journal of Geotechnical Engineering, ASCE, 118(6), p. 839-855.
- Cho, K. H., Clark, S. C., Keaney, B. D., Gabr, M. A., and Borden, R. H. (2001)."Laterally loaded drilled shafts embedded in soft rock." Transportation Research Record.1772, p. 3-11.
- DiGioia, A. M., Jr., and Rojas-Gonzalez, L. F. (1993). "Discussion on 'Analysis of laterally loaded shafts in rock.'" Journal of Geotechnical Engineering, ASCE, 119(12), p.2014-2015.
- Dykeman, P., and Valsangkar, A. J. (1996). "Model studies of socketed caissons in soft rock." Canadian Geotechnical Journal, v. 33 issue 5, p. 747-759.
- Frantzen, J., and Stratten, W. F. (1987). "Final report: p-y curve data for laterally loaded piles in shale and sandstone." Rep. No. FHWA-KS-87-2, Kansas Department of Transportation, Topeka, Kansas.
- Gabr, M. A. (1993). "Discussion on 'Analysis of laterally loaded shafts in rock.'" Journal of Geotechnical Engineering, ASCE, 119(12), p. 2015-2018.
- Gabr, M.A., Borden, R.H., Cho, K.H., Clark, S.C., and Nixon, J.B. (2002). "P-y curves for laterally loaded drilled shafts embedded in weathered rock." FHWA/NC/2002-08, North Carolina Department of Transportation.
- Goodman, R., and Shi, G. (1985). Block theory and its application to rock engineering, Prentice-Hall, Englewood Cliffs, N.J.
- Kovari.K., and Fritz.P. (1984). "Recent development in analysis and monitoring of rock slopes". IVth Inteernational Symposium on landslides, Torento. p.1045-1058.
- Hoek, E. (1994). "Strength of rock and rock masses," ISRM News Journal, 2 (2), p. 4-16.

- Hoek, E., and Brown, E. T. (1980). “Empirical strength criterion for rock masses.”American Society of Civil Engineers, Journal of the Geotechnical Engineering Division,v. 106 issue 9, p. 1013-1035.
- Hoek, E., and Brown, E.T. (1988). “The Hoek-Brown criterion – a 1988 update.” 15th Canadian Rock Mechanics Symposium: rock engineering for underground excavations, University of Toronto, p. 31-38.
- Hoek, E., and Brown , E.T. (1997). “Practical estimates of rock mass strength.” Int.J. Rock Mech. Min. Sci., 34(8), p. 1165-1186.
- Priest.S.D., and Hudson.J.A. (1976). “Discontinuity Spacing in Rock”.International Journal of Rock Mechanics and Mining Scienc.13, p 134-153.
- Ramamurthy, T. (1993). ”Strength and modulus response of anisotropic rocks. In:Comprehensive rock engineering”. Pergamon Press, Oxford, vol. 1, p.313–329.
- Ramamurthy, T.(1994). ”Strength prediction for jointed rocks in confinedand unconfined states”. International Journal of Rock Mechanics and Mining Sciencs.13., p.9-22.
- Ramamurthy T (1996). “Stability of rock mass”.8th Indian Geotechnical Society Annual Lecture. Indian Geotech J 16:1–73
- Ramamurthy T, Rao GV, Rao KS (1985).” A strength criterion for rocks”. In: Proceedings of Indian geotechnical conference, vol 1, Roorkee, pp 59–64.
- Ramamurthy,.T. (1994). “Strength Criteria for Rocks with Tensile Strength”. Proc. Ind. Geotechnical Conference., Warangal,India.p. 411-414.
- Ramamurthy, T.(2010). “ Engineering in rocks for slopes,foundations and tunnels”. Rajkamal press, New Delhi, 2nd Edition p 139-147.
- Randolph, M.F., and Houlsby, G.T. (1984). “The limiting pressure on a circular pile loaded laterally I cohesive soils.” Geotechnique, 34 (4), 613-623.
- Reese, L. C. (1997). “Analysis of laterally loaded piles in weak rock.” J. Geotech. Envir. Engrg., ASCE, 123(11), p. 1010-1017.
- Reese, L. C., Cox, W. R., and Koop, F. D. (1975). “Field testing and analysis of laterally loaded piles in stiff clay.” 7th Offshore Technology Conference, p. 671-690.
- Singh B, Viladkar MN, Samadhiya NK, Mehrota VK (1997). “ Rock mass strength parameters mobilized in tunnels”. Tunn Underground Space Technol 12(1):47–54

- Singh M, and Singh B.(2012). "Modified Mohr–Coulomb criterion for non-linear triaxial and polyaxial strength of jointed rocks". Int J Rock Mech Min Sci ,48, p 45–55
- Singh, M., Rao, K.S. and Ramamurthy, T. (2002). "Strength and Deformational Behaviour of Jointed Rock Mass", Rock Mech. & Rock Engg, Vol 35(1), pp 45-64.
- Singh, M. and Rao, K.S (2005)."Empirical methods to estimate the strength of jointed rock masses." Engineering Geology, 77, p 127-137.
- Sun, K. (1994). "Laterally loaded piles in elastic media." Journal of Geotechnical Engineering, ASCE, 120(8), p. 1324-1344.
- To, A. C., Ernst, H., and Einstein, H. H. (2003). "Lateral load capacity of drilled shafts in jointed rock." Journal of Geotechnical and Geoenvironmental Engineering, Vol. 129, No. 8, p. 711-726.
- Vesic, A. S. (1961). "Beam on elastic subgrade and the Winkler hypothesis." Proc. 5th Int.Conf. Soil Mechanics and Foundation Engineering, Paris, Vol. 1, p. 845-850.
- Zhang, L., Ernst, H., and Einstein, H. H. (2000). "Nonlinear analysis of laterally loaded rock-socketed shafts", Journal of Geotechnical and Geoenvironmental Engineering.v. 126 issue 11, p. 955-968.
- Zhang, L. (2000). "Estimating the strength of jointed rock mass", Journal of Rock Mechanics and Rock Engineering. v. 43 issue 10, p. 391-402.

APPENDIX-I

PROGRAM TO CALCULATE LATERAL LOAD CARRYING CAPACITY OF ROCK SOCKETS

```
#include <stdio.h>
#include <math.h>
int main()
{
int loops, beta, temp=0,interval,i=0,noj,sig3count, cteta, sigmacj;
float srf,phii,dilAngle,sig1,sig3,ucs,r=0.9,jf,jn,change=0.017453, ,angle,n, ,min = 0.0,Er;
float loaddip=0;
float loaddd[360],con=57.2958;
float gamarock,RQD,RMR,SF,MRF,minsigmacj1,minsigmacj2,minsigmacj3,minsigmacj4;
loaddd[0]=0,Q;
float jointsets[10][3];
float Avalue,Bvalue,XYProductSum,XSquare,Cmass,PHImass;
printf("\n Enter number of joint sets:");
scanf("%d",&noj);
printf("Number entered %d:",noj);
printf("\n Enter ucs in MPa:");
scanf("%f",&ucs);
printf("\n Enter friction angle of intact rock:");
scanf("%f",&phii);
printf("ucs Number entered %f:",ucs);
printf("\n Enter the horizontal angle interval:");
scanf("%d",&interval);
loops=360/interval;
float sigmaarray[360][10]={};
float jfarray[360][10]={};
printf(" \n Total no of calculations: %d \n",loops);
```

```
for(i=0;i<noj;i++)
{
printf("\n Enter jointdip %d:",i+1);
scanf("%f",&jointsets[i][0]);
printf("Joint dip %f",jointsets[i][0]);
printf("\n Enter joint dip direction %d:",i+1);
scanf("%f",&jointsets[i][1]);
printf("\n Enter joint spacing in meters %d:",i+1);
scanf("%f",&jointsets[i][2]);
}
printf("Jointsets-->");
for(i=0;i<noj;i++)
printf("\t %d",i+1);
printf("\n\n Dip\t");
for(i=0;i<noj;i++)
printf("\t %.2f",jointsets[i][0]);
 printf("\n\n DiD\t");
for(i=0;i<noj;i++)
printf("\t %.2f",jointsets[i][1]);
printf("\n\n Spa\t");
  for(i=0;i<noj;i++)
printf("\t %.2f",jointsets[i][2]);
  for(i=1;i<loops;i++)
loaddd[i]=loaddd[i-1]+interval;
int h=0; int j=0;
  for(h=0;h<noj;h++)
{
for(i=0;i<loops;i++)
{
cteta=( cos(jointsets[h][1]*change-cos(loaddd[i]*change)* cos((loaddipjointsets[h][0])*change);
jn=100.0*cteta/jointsets[h][2];
```

```
angle=acosf(cteta);
angle=angle*con;
beta=(int)(90.0-angle);
if(beta<0)
beta=0-beta;
if((0<=beta)&&(beta<=10))
{
n=0.810-((10-beta)*(0.810-.460)/10);
}
else if((10<beta)&&(beta<=20))
{
n=0.460-((20-beta)*(0.460-.105)/10);
}
else if((20<beta)&&(beta<=30))
{
n=0.105-((30-beta)*(0.105-0.046)/10);
}
else if((30<beta)&&(beta<=40))
{
n=0.046+((40-beta)*(0.071-0.046)/10);
}
else if((40<beta)&&(beta<=50))
{
n=0.071+((50-beta)*(0.306-.071)/10);
}
else if((50<beta)&&(beta<=60))
{
n=0.306+((60-beta)*(0.465-0.306)/10);
}
else if((60<beta)&&(beta<=70))
{
```

```
n=0.465+((70-beta)*(0.634-0.465)/10);
}
else if((70<beta)&&(beta<=80))
{
n=0.634+((80-beta)*(0.814-0.634)/10);
}
 else if((80<beta)&&(beta<=90))
{
n=0.814+((90-beta)*(1.0-0.8140)/10);
}
 jf=jn/(n*r);
 jfarray[i][h]=jf;
printf("\n Joint Factor JF= %.2f",jf);
sigmacj=ucs*pow(2.71828,-0.008*jf);
sigmaarray[i][h]=sigmacj;
temp++;


}
}
float minsigcjarray[loops];
for(j=0;j<loops;j++)
{
min=sigmaarray[j][0];
for(i=0;i<noj;i++)
{
if(sigmaarray[j][i]<min)
min=sigmaarray[j][i];
}
minsigcjarray[j]=min;
 }
printf("\n\n DD//JS--> ");
```

```
for(i=0;i<noj;i++)
printf("\t %d",i+1);
printf("\n");
for(i=0;i<loops;i++)
{
printf("\n\n %.2f",loaddd[i]);
for(j=0;j<noj;j++)
{
printf("  \t %.2f",sigmaarray[i][j]);
}
printf("\t min= %.2f",minsigcjarray[i]);
printf("\n\n");
}
float minsigmacj=0.0;
for(i=0;i<loops;i++)
{
 if(i==0)
{
min = minsigcjarray[i];
}
if(minsigcjarray[i] <=min)
{
min=minsigcjarray[i];
}
}
printf("\n\n UCS of jointed rock in MPa = %.2f",min);
minsigmacj=min;
int option,option2;
printf("\n \n \n Enter 1 if you wish to continue \n:");
printf(" \n Enter 2 if you want to enter another UCS value \n :");
printf(" \n Enter 3 for other methods: ");
```

```
scanf("%d",&option);
switch(option)
{
case 2:   printf(" \n Enter the new UCS value:");
scanf("%f",&minsigmacj);
printf("sigma=%.2f",minsigmacj);
break;
case 3:   printf(" \n Enter 1 to calculate UCS of rock mass using Q System\n:");
printf(" \n Enter 2 to calculate UCS of rock mass using  RQD\n:");
printf(" \n Enter 3 to calculate UCS of rock mass using RMR\n:");
printf("\n Enter 4 to calculate UCS of rock mass Strength Reduction  Factor\n:");
printf(" \n Enter 5 to calculate UCS of rock mass using average of all the above systems\n:");
scanf("%d",&option2);
switch(option2)
{
case 1: printf("\n we are in Qsystem\n");
printf("\n Enter Rock Mass Qulity Index Q ");
scanf("%f",&Q);
printf("\n Enter unit weight of rock in gm/cm^3  ");
scanf("%f",&gamarock);
minsigmacj= 7*gamarock*pow(Q,0.333);
break;
case 2: printf("we are in RQD");
printf("\n Enter RQD  ");
scanf("%f",&RQD);
minsigmacj= ucs*pow(10,(0.013*RQD-1.34));
break;
case 3: printf("\n we are in RMR system");
printf("\n Enter RMR");
scanf("%f",&RMR);
minsigmacj= ucs*pow(2.7183,((RMR-100)/18.75));
```

```
break;
case 4: printf("\n we are in Strength Reduction Factor SF\n");
printf("\n Enter Modulus Reduction Factor MRF");
scanf("%f",&MRF);
SF= pow(MRF,0.63);
printf("Strength reduction factor SF=%.2f ",SF);
minsigmacj= ucs*SF;
break;
case 5: printf("we are in average");
printf("\n Enter Rock Mass Qulity Index Q \n");
scanf("%f",&Q);
printf("\n Enter unit weight of rock in gm/cm^3 \n  ");
scanf("%f",&gamarock);
minsigmacj1= 7*gamarock*pow(Q,0.333);
printf("\n Enter RQD\n  ");
scanf("%f",&RQD);
minsigmacj2= ucs*pow(10,(0.013*RQD-1.34));
printf("\nEnter RMR\n");
scanf("%f",&RMR);
minsigmacj3= ucs*pow(2.7183,((RMR-100)/18.75));

printf("\nEnter Modulus Reduction Factor MRF\n");
scanf("%f",&MRF);
SF= pow(MRF,0.63);
minsigmacj4= ucs*SF;
minsigmacj= (minsigmacj1+minsigmacj2+minsigmacj3+minsigmacj4)/4;

break;
}
break;
case 1: printf("The minsigmacj=%.2f ",minsigmacj);
```

```
break;
}
printf("Finally the minsigmacj=%.2f",minsigmacj);
srf=minsigmacj/ucs;
printf("\n\n SRF %f:",srf);
printf(" \nphii %.2f:",phii);
angle=((1-srf)+((sin(phii*change))/(1-sin(phii*change))))/((2-srf)+((sin(phii*change))/(1-
sin(phii*change))));
angle=asinf(angle);
angle=angle*con;
printf(" \n Joint friction angle %.2f:",angle);
float sig3Array[15];
sig3Array[0]=ucs*0.25/8;
sig3count=8;
int s;
for(s=1;s<=7;s++)
sig3Array[s]=sig3Array[s-1]+ucs*0.25/8;
float sig1Array[8];
for(i=0;i<sig3count;i++)
{
sig1Array[i]=sig3Array[i]+minsigmacj+2*((sin(angle*change))/(1-
sin(angle*change)))*sig3Array[i]-((sin(angle*change))/(1-
sin(angle*change)))*sig3Array[i]*sig3Array[i]/ucs;
printf(" \n\n sig3 %.2f \t sig1 %.2f   ",sig3Array[i],sig1Array[i]);
}
sig1=sig3+minsigmacj+2*((sin(angle*change))/(1-sin(angle*change)))*sig3-
((sin(angle*change))/(1-sin(angle*change)))*sig3*sig3/ucs;
scanf("%lf",&ucs);
for(i=0;i<sig3count;i++)
Avalue += sig3Array[i];
Bvalue += sig1Array[i];
```

```
printf(" \nSum of sig3Array Values %f",Avalue);
printf(" \nSum of sig1Array Values %f",Bvalue);
}
for(i=0;i<sig3count;i++)
{
XYProductSum += sig3Array[i]*sig1Array[i];
printf(" \nSum of XY %f:",XYProductSum);
}
for(i=0;i<sig3count;i++)
{
XSquare += sig3Array[i]*sig3Array[i];
printf(" \n X square %f:",XSquare);
}
float a,b;
b=(XYProductSum-(Avalue*Bvalue)/sig3count)/(XSquare-((Avalue)*(Avalue))/(sig3count));
 printf(" \n b %.2f:",b);
 a= (Bvalue-b*Avalue)/(sig3count);
 printf(" \n\n b,a = %.2f, %.2f,  ",b,a);
 PHImass=asinf((b-1)/(b+1));
 PHImass= PHImass*con;
 printf(" \n PHImass %f:",PHImass);
Cmass=a*(1-sin(PHImass*change))/(2*cos(PHImass*change));
printf(" \n\n PHImass,Cmass = %.2f, %.2f,  ",PHImass,Cmass);
Er=ucs*pow(2.71828,-0.0035*jf);
dilAngle= 0.5*(PHImass-phii);
printf(" \n\n Elastic shear modulus in MPa %.2f:",Er);
printf(" \n\n Rock mass friction angle %.2f:",PHImass);
printf(" \n\n Dilation Angle %.2f:",dilAngle);
printf(" \n\n Rock mass cohesion in Mpa %.2f:",Cmass);
scanf("%lf",&ucs);
{
```

```
double esm,hs=0.01,c,fa,pr,da,ucs,b,sm,l,change=0.017453;
double beta,alpha,beta1,beta2,sigmar,Pl,hr;
printf("\nEnter Elastic Shear Modulus in MPa:");
scanf("%lf",&esm);
esm=Er;
da=dilAngle;
c=Cmass;
fa=PHImass;
printf("\nEnter Horizontal Stress in Mpa:");
scanf("%lf",&hs);
printf("\nEnter Cohesion in MPa:");
scanf("%lf",&c);
printf("\nEnter Friction Angle of rock mass :");
scanf("%lf",&fa);
if((15<=fa)&&(fa<=45))
{
printf("\nEnter Poissions Ratio\n ");
scanf("%lf",&pr);
printf("\nEnter Dilation Angle ):");
scanf("%lf",&da);
printf("\nEnter UCS of rock mass MPa:");
scanf("%lf",&ucs);
printf("\nEnter Diameter of the Shaft in meters:");
scanf("%lf",&b);
printf("\nEnter Length of the Shaft in meters:");
scanf("%lf",&l);
sm=0.5*pow(ucs,0.5);
alpha=esm/(hs+(c/tan(fa*change)));
if((15<=fa)&&(fa<=25))
{
if((0<=da)&&(da<=5))
```

```
{
if(alpha==10)
beta=2.1;
else if(alpha==100)
beta=4;
else if(alpha==1000)
beta=7;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=2.1;
beta2=4;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=4;
beta2=7;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((5<da)&&(da<=15))
{
if(alpha==10)
beta=2.6;
else if(alpha==100)
beta=4.9;
else if(alpha==1000)
beta=9.8;
```

```
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=2.6;
beta2=4.9;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=4.9;
beta2=9.8;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((15<da)&&(da<=20))
{
if(alpha==10)
beta=3;
else if(alpha==100)
beta=5.9;
else if(alpha==1000)
beta=11;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=3;
beta2=5.9;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
```

```
}
if((100<alpha)&&(alpha<=1000))
{
beta1=5.9;
beta2=11;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
}
else if((26<=fa)&&(fa<=35))
{
if((0<=da)&&(da<=5))
{
if(alpha==10)
beta=2.1;
else if(alpha==100)
beta=5;
else if(alpha==1000)
beta=11;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=2.1;
beta2=5;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=5;
```

```
beta2=11;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((5<da)&&(da<=15))
{
if(alpha==10)
beta=3;
else if(alpha==100)
beta=6.5;
else if(alpha==1000)
beta=18;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=3;
beta2=6.5;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=6.5;
beta2=18;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((15<da)&&(da<=25))
{
```

```
if(alpha==10)
beta=3.2;
else if(alpha==100)
beta=9;
else if(alpha==1000)
beta=27;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=3.2;
beta2=9;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=9;
beta2=27;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((25<da)&&(da<=30))
{
if(alpha==10)
beta=4;
else if(alpha==100)
beta=11;
else if(alpha==1000)
beta=39;
else
```

```
{
if((10<=alpha)&&(alpha<=100))
{
beta1=4;
beta2=11;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=11;
beta2=39;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
}
else
{
if((0<=da)&&(da<=5))
{
if(alpha==10)
beta=2.1;
else if(alpha==100)
beta=5.1;
else if(alpha==1000)
beta=18;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=2.1;
```

```
beta2=5.1;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=5.1;
beta2=18;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((5<da)&&(da<=15))
{
if(alpha==10)
beta=3;
else if(alpha==100)
beta=8;
else if(alpha==1000)
beta=28;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=3;
beta2=8;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=8;
beta2=28;
```

```
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((15<da)&&(da<=25))
{
if(alpha==10)
beta=3.2;
else if(alpha==100)
beta=11;
else if(alpha==1000)
beta=40;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=3.2;
beta2=11;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=11;
beta2=40;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((25<da)&&(da<=35))
{
if(alpha==10)
```

```
beta=3.8;
else if(alpha==100)
beta=12;
else if(alpha==1000)
beta=50;
else
{
if((10<=alpha)&&(alpha<=100))
{
beta1=3.8;
beta2=12;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=12;
beta2=50;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
if((35<da)&&(da<=40))
{
if(alpha==10)
beta=4;
else if(alpha==100)
beta=18;
else if(alpha==1000)
beta=70;
else
{
```

```
if((10<=alpha)&&(alpha<=100))
{
beta1=4;
beta2=18;
beta=beta2-((beta2-beta1)*(100-alpha)/90);
}
if((100<alpha)&&(alpha<=1000))
{
beta1=18;
beta2=70;
beta=beta2-((beta2-beta1)*(1000-alpha)/900);
}
}
}
}
sigmar=((1+sin(fa*change))*(hs+(c/tan(fa*change))))-(c/tan(fa*change));
Pl=beta*(sigmar+(c/tan(fa*change)))-(c/tan(fa*change));
if(l<3*b)
{
hr=abs(((Pl*l/6)+sm*b)*l);
}
else
{
hr=abs((Pl/2+sm)*3*b*b+(Pl+sm)*(l-3*b)*b);
}

printf("\n Horizontal Reaction in KN: %.3lf",hr);
}
else
{
printf("wrong Input (Enter in between 20-40 only)");
```