CHAPTER 2
LITRATURE REVIEW
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2. LITERATURE REVIEW

2.1 General

Axisymmetric excavations are quite common in the construction industry. Manholes, inspection or access chamber, service entrances, and excavation for bored piles or piers are mainly axisymmetric. The understanding of the mechanics of the behavior of axisymmetric excavation should also throw light on the complex three-dimensional behavior of trench headings and open pit excavations. The advancing face of a rectangular trench could be approximated as semicircular in plan, or in a further idealization, condition of axisymmetry could be assumed. Rectangular or square open pit excavations may be idealized as axisymmetric excavations as a first approximation.

2.2 Axisymmetric Excavations of Small Diameters

Axisymmetric unsupported excavations of small diameters are limited to exploratory works in civil engineering. In petroleum industry it is more prevalent and mostly their diameters are less than 500mm. These small diameter boreholes pass through soil and rock mass. The stability of such small diameter unsupported vertical openings has been studied by many investigators using different methodologies as detailed below.
2.2.1 Analytical and Numerical Modeling

Quantification of the risk of failure, either progressive or catastrophic, open hole or cased, during drilling or production, requires the calculation of stress and deformation fields around the unsupported vertical opening and a comparison with accepted criteria of failure. These calculations can be performed with varying degrees of sophistication and with varying predictive limitations.

Two major cases have been considered. Supported excavation and unsupported excavation. The unsupported case which is the subject matter of investigation, often referred to as the open hole situation in civil engineering applications, will be discussed.

2.2.2 Unsupported Openings of Small Diameters

Most of the existing work on borehole stability is based on perfectly elastic/plastic models that are calibrated with test data taken from conventional triaxial compression experiments. Rupture is assumed to occur when the stresses reach the elastic limit that is usually set at the peak of the experimental stress-strain curve.

2.2.3 Elastic Analysis

The solution for the stresses and displacements due to an infinite circular hole in a homogeneous, isotropic, linear elastic medium is given by the superposition of Kirsch's solution (the antiplane solution), and the solution for an internally pressurized hole of arbitrary orientation (Fairhurst, 1968; Deily and Owens, 1969; Daneshy, 1973a; Bradley, 1979; Richardson, 1981; Hsiao, 1987; Yew and Li, 1987).
Stress transformations and assessment in a local coordinate system are a routine mathematical exercise. The solutions at the borehole wall can be simplified, with little loss of accuracy, by assuming Poisson's ratio to be equal to zero (Li, 1986). The major difference between a borehole (horizontal or otherwise) drilled parallel to a principal axis and one non-collinear is that, one of the shear components remains finite at the borehole wall. Its magnitude affects the overall integrity of the borehole, and the principal stress tensor will be rotated in the neighborhood of the circular opening. Knowing the stress field associated with a particular wellbore configuration, various failure mechanisms can be assessed.

2.2.4 Tensile Failure
This criterion states that tensile failure will occur when the minor principal stress reaches the tensile strength of the medium. Two generic kinds of tensile failure can be envisioned. The first occurs at or close to the borehole wall (Santarelli et al., 1986; Detournay and Cheng, 1988) and the fracture trace can theoretically be at an angle to the borehole axis; this has been experimentally observed by Kuriyagawa et al. (1988). The second kind does not intersect the borehole, is concentric, and by itself may be kinematically stable. The principal stresses at the wall of the borehole and the theoretical trace of the fracture on the borehole wall are given by Daneshy (1973b).

2.2.5 Shear Failure
This mechanism can occur in an active mode (inward movement) during production or in a passive mode during pressurization. If wedges of rock are produced and fall inward, there is little consequence in wells that are not highly deviated from the vertical
during drilling unless severe bridging has occurred. In wells deviated at least in excess of 60 to 75 degrees from the vertical, this debris is a potential source of future problems such as bridging and sticking (Tomren et al., 1983; Okrajni and Azar, 1985; Wilson and Willis, 1986). As with tensile failure, there are a number of criteria available for determination of the potential for shear failure. A Drucker-Prager criterion incorporating frictional characteristics, taking into account all stresses acting on the medium (Bradley, 1979), is commonly used for oilfield stability assessment.

2.2.6 Matrix Collapse

This is a common description associated with volume reduction and densification of the medium. This can occur in poorly consolidated formations and in rocks that have an unusually large porosity (Ruddy et al., 1988). This failure is controlled by the hydrostatic compressive strength of the medium. When this parameter is exceeded by the mean stress, the porous matrix collapses. It has been noted that, although there are many published accounts of such elastic analyses being used, the predictions are conservative. They cannot account for pseudo-stable states associated with progressive failure and redistribution of stresses due to nonelastic material properties.

2.3 Inelastic Analysis

Despite the fact that the elastic approach is widely used in the design of circular openings, it is viewed as being insufficient to predict quantitatively failure and to describe various failure modes observed in underground excavations. The assumption of linear elastic behavior up to failure leads to excessive stress concentrations at the borehole wall. Cases of abnormal stability as compared to the theoretical stress
concentration given by linear elasticity are frequently observed in situ or in hollow cylinder experiments. This point of view is emphasized in Guenot's (1987) review of experimental results from hollow-cylinder testing for a great variety of materials.

A major drawback in the above classical procedures is the inadequacy to describe some surface rupture modes usually referred to as axial cleavage fractures or extension ruptures (Maury 1987). This deficiency is related to the ad hoc assumption that failure is an intrinsic property of the material that should naturally be associated with the elastic-plastic limit. An alternative way to describe rupture phenomena in rocks is based on the bifurcation theory, together with more realistic constitutive modeling. This approach to differentiates between the rheological behavior of the material and the various rupture phenomena. Furthermore, the bifurcation theory can be used to describe and predict the occurrence of the various observed failure modes.

2.4 Simplified Bifurcation Analysis

Vardoulakis and Papanastasiou (1988) considered borehole stability in a deep rock layer under a uniform stress field at infinity, as illustrated in Figure 2.1. Furthermore, they considered a deep section of the borehole. The creation of the hole was simulated by a gradual reduction of the support pressure at the borehole wall. The most bifurcation of a monotonous deformation process indicates that at some critical state, the deformation process does not follow its straight-ahead continuation but turns to be an entirely different mode. Typical examples of bifurcation phenomena are buckling, barreling, necking, shear-banding and axial splitting observed in rock specimens.
2.4.1 Cylindrical Convergency of the Borehole

As illustrated in Figure 2.2, this case corresponds to a uniform reduction of the borehole radius. This case can be reproduced mathematically with an isotropic, linear elastic model for the surrounding rock, and is known as the Lamé’s solution (Timoshenko, 1934). Other solutions can be easily constructed for more complicated constitutive models.

2.4.2 Warping of the Borehole Wall

This deformation mode is sketched in Figure 2.3. It should be emphasized that the majority of the existing solutions to surface warping is for the half-space problem and not to the borehole problem (Vardoulakis, 1984). The warping mode must be accounted for by the formation of shear-bands, as shown in Figure 2.3, or exfoliation at the borehole wall due to activation and subsequent unstable propagation of pre-existing cracks, as shown in figure 2.5
Figure 2.1. Borehole stability in a deep rock layer under uniform stress at infinity

Figure 2.2. Cylindrical convergency of a borehole.

Figure 2.3. Warping of a borehole wall.

Figure 2.4. Formation of shear bands.

Figure 2.5 Exfoliation at the borehole wall.
Vardoulakis and Papanastasiou (1988) analyzed borehole stability using the finite element method, determining the lowest stress at infinity that causes warping of the borehole. This theory suggests that the critical bifurcation stress corresponds to the short wavelength limit that is affecting a vanishingly narrow ring of material in the vicinity of the borehole wall. Consequently, one can restrict the analysis to a small domain in the neighborhood of the borehole wall and neglect the stress-gradient. For the same reason, this domain can be replaced by a half-space of material loaded by the boundary stresses that may cause plane-strain surface instabilities. If surface instabilities are not possible, then the dominant failure mode is shear-band formation in the vicinity of the borehole wall.

### 2.4.3 Isotropic Far-Field Stresses

Vardoulakis and Papanastasiou (1988), Vardoulakis et al. (1988), and Sulem and Vardoulakis (1988) analyzed borehole stability for the case of isotropic far-field stress within the frame of a deformation theory for rigid-plastic, pressure-sensitive materials. Papanastasiou and Vardoulakis (1989) and Papanastasiou et al. (1989) studied also the scale effect (i.e., the dependency of the borehole stability on the radius of the borehole) using a deformation theory for rigid-plastic material with microstructure (Cosserat material). These theoretical predictions have been validated experimentally by Ewy and Cook (1989) and Haimson and Herrick (1989).

### 2.4.4 Numerical Analysis of Localization Phenomena in Deep Boreholes

The Isotropic far-field stress results so obtained encouraged Papanastasiou (1990) and Papanastasiou and Vardoulakis (1991) to
pursue the task of treating the borehole problem within a 2D, nonlinear finite element analysis to model the evolution of the rupture zone and the progressive failure of the structure. The rock was modeled by the elasto-plastic constitutive equations of flow theory of plasticity for cohesive frictional/hardening softening, dilatant material. These constitutive equations were fitted on true stress-strain data from experiments on Carboniferous sandstone. The ill-posed boundary value problem of borehole stability in strain-softening rock was regularized by adopting a continuum with the Cosserat microstructure in which the individual grains possess, in addition to the translational degrees of freedom, a rotational degree of freedom as well. Grain rotation and its gradient give rise to a nonsymmetrical stress tensor and couple stresses that introduce a length scale into the problem, which improves the computational stability and allows for robust post-localization computations. The results (Figure 2.6) clearly show a progressive failure mechanism; the computed failure modes are in a good qualitative agreement with the laboratory and field observations. Also, the existence of an internal length in the constitutive equations enables one to model the existence of the scale effect in the problem; thus, small holes fail at higher external stresses than large holes.

2.4.5 Nonlinear Effects

The introduction of nonlinear effects, such as stress-dependent elastic constants, adds further complications. Indeed, for certain parameter combinations, failure can initiate inside the material (Santarelli et al., 1986; Santarelli and Brown, 1987 Guenot and Santarelli, 1988) at a finite distance from the borehole wall. Specifically, Santarelli and Brown (1987) used a numerical model incorporating an increase in stiffness with confining pressure. The observations made by Daemen
and Fairhurst (1971) who showed that there were no indications of fracturing or loosening around hydrostatically (external) pressurized thick-walled cylinders, even when the tangential stress was four times the uniaxial compressive strength (Hobbs, 1966; Hoskins, 1969; Haimson and Edl, 1972), as well as observations (e.g., Kaiser et al., 1985) that linear elasticity inadequately predicted tunnel convergence near failure. Using a power law variation for Young's modulus as a function of the confining stress for a vertical wellbore in an elastic medium with isotropic horizontal stresses, a generalized representation of Hooke's law was presented. The observations were as follows:

1) The maximum tangential stress concentration occurs within the rock and not at the borehole wall for specific loading conditions.

2) The formulations do not take into account prepeak yield. The consequence is an overestimate of the tangential stress at the well and an underestimate of strains.

3) The representation could be improved by incorporating moduli relationships, which do not imply that the modulus approaches zero as the confining stress approaches zero.
Figure 2.6 (a) Global picture of the plastic and elastic domains before localization. Detail of the plastic and elastic domains at (b) first step after localization and (c) after completion of localization. Detail of (d) isolines of accumulated plastic shear strain (e) incremental displacement field and (f) deformed mesh.
Improved recognition of the pre-yield behavior recognizes the occurrence of damage and the influence of microstructure. Mühlhaus (1987) described the microstructure using the generalized Cosserat continuum theory with micro rotations and coupled stresses. Damage may be viewed as a radial variation of the shear modulus governed by the minor principal stress acting on an element. Using these concepts, surface instabilities can be delineated, where surface buckling of small finite columns occurs. Presuming surface instability of a half-plane on a local scale, the location where the maximum tensile radial stress occurs can be delineated. Such stresses occurring in the post-buckling range can cause latent cracks parallel to the surface to open, potentially promoting unstable crack growth and consequent spalling.

In a manner analogous to surface instabilities, other localized effects could be active. Localization of deformation can occur in shear bands. These can be understood as instability in the macroscopic constitutive description of the rock, corresponding to weak discontinuities of the displacement field. In terms of the Cosserat theory, the thickness of the shear bands is defined by certain material parameters depending on a length dimension, such as grain diameter. The concept of localization in shear bands diverges from standard plasticity concepts, which presume that all slip surfaces are active and that there is pervasive failure.

2.5 Time-Dependent Effects

The integrity of a wellbore regime can be influenced by time-dependent material response (i.e., viscoelastic or viscoplastic behavior). It can be further affected by temporal variation of the stress field due, for example, to poroelastic effects. Fully coupled analyses mathematically acknowledge that transient changes in the pore
pressure field around the wellbore—due to fluid permeation—modify the in-situ stress regime. The transience may lead to failure away from the borehole wall. Such a limited equilibrium state does not necessarily appear during drilling but may possibly occur later in the life of the well, as delayed instabilities (Paslay and Cheatham, 1963; Haimson, 1968; Detournay and Cheng, 1988; Guenot and Santarelli, 1988)

2.6 Scale Effects

The scale effects in the determination of rock mass strength and deformability have been extensively discussed by Heuze (1980). The scale effect, as far as boreholes are concerned, was studied experimentally by Haimson and Herrick (1989) and numerically by Papanastasiou et al. (1989a, 1989b).

In the aforementioned bifurcation analysis of deep boreholes, rock was described by a classical continuum theory of plasticity for rigid-plastic, cohesive/frictional material. It was found under these constitutive assumptions, the critical diffuse bifurcation mode is surface instability. This finding was explained by the high stress gradient at the borehole wall. Surface instability corresponds to the infinitesimally small wavelength limit with respect to the borehole radius, i.e., to a bifurcation solution that is affecting only an infinitesimal ring of material close to the borehole wall. The short wavelength limit is an accumulation point of bifurcations. Consequently, almost all wavelengths of the warping mode are possible, and there is no influence of the borehole radius.

The pressure-sensitive character of the material behavior gives rise to solutions that can be used to explain the experimentally observed high
resilience of model boreholes to external pressure (Guenot, 1987). However, experiments suggest also that small boreholes fail at higher external stresses than large holes (Haimson and Herrick, 1986, 1989). This means that to achieve an accurate statement about borehole stability based on laboratory model tests (the dependency of the borehole stability on the borehole radius), scale effect must be also modeled.

2.7 Axisymmetric Excavations of Medium Diameters

The unsupported vertical circular openings of medium diameter are commonly of the range between 750 to 1500mm in diameter. These are normally seen as open wells for domestic drinking water, access chambers and excavations of exploratory in nature. For long term stability these openings must be supported. Between excavation and supporting the excavation would be unsupported for a period of 6 to 7 days. During this window period the stability is very critical.

In the past, some work has been done on axisymmetric excavations in clays by Bjerrum and Eide(1956), Prater(1977), Pastor and Turgeman(1979), Pastor (1981), Britto, Kusakabe, and Schofield (1981), and Britto and Kusakabe(1983). In all the aforementioned studies, the soil was assumed to possess uniform undrained shear strength. It is often the case, however, that the shear strength increases with depth, and thus must be taken into consideration when calculating the stability of the excavation. For the plane-strain situation, Gibson and Morgenstern (1962), and Hunter and Schuster (1968) analyzed the stability of cut slopes in clay with untrained shear strength, $Cu$, varying linearly with depth. Snitbhan, et. al (1975);
Chen, et al (1975), and Reddy and Srinivasan (1967), have applied the upper bound theorem of limit analysis to plane-strain problems for layered soils and non homogeneous, anisotropic C-\(\Phi\) soils.

The previously published works on stability of axisymmetric excavations will be summarized, and the critical failure mode will be examined. The un-drained shear strength of the soil will be either a zero or a non zero value at the ground surface, and increases linearly with depth. The soil was also assumed to satisfy the Tresca yield criterion (\(\Phi = 0\)). Therefore, the results presented are only relevant to un-drained clays.

2.8 Critical Failure Mode for \(C_u = \text{Constant Case}\)

Figure 2.7 shows the idealization of the problem where the excavation is of depth \(D\) and radius \(r_0\), the soil is assume to be saturated; to have the uniform shear strength \(C_u\), and to obey Tresca yield criterion. There are four ways of obtaining stability solution for this problem:

- Using the bearing capacity formula
- The limit equilibrium method
- Bound theorem of plasticity, (limit analysis)
- Finite element analysis

The results of various researchers are presented in figure 2.8, in which the dimensionless stability number \(N\) defined by

\[
N = \frac{\gamma D}{C_u} \text{..............................................................}(1)
\]
In which $\gamma = \text{Bulk unit weight of soil}$, is plotted against the aspect ratio of the excavation i.e.; $D/r_0$.

Figure 2.7 -- Idealization of problem – Shaft Excavation

Curve A is Skempton’s (1957) bearing capacity factor, $N_c$, for deep circular footing. Bjerrum and Eide (1956) analyzed a number of field-strutted excavations using Skempton’s bearing capacity formula. They stated that the base failure of an excavation (due to unloading) is the reverse of a failure by loading of a deep footing. Good agreement was obtained between the analyses based on the bearing capacity formula and field data including some axisymmetric cases.
Curves B and C are upper bound solutions by Prater (1977) for wall failure and base failure, respectively. Extending the simple triangular failure mechanism of a vertical cut under plane strain conditions, a conical failure surface was considered for the axis symmetric situation. Assuming that the excavation was laterally supported, he derived the expression of the load on the wall. By minimizing this expression with respect to the cone angle and then equating the load on the wall to zero, he finally obtained the expression for the critical height of the excavation, $D_{cr}$. This result is plotted in terms of $N$ as the curve B in fig 2.8. The base failure considered by Prater is again an extension of the plane strain situation (Terzaghi, 1943). Making use of Skempton’s bearing capacity factor but using $N_{cr}=6$ to obtain the smallest value of the critical height of the excavation, the curve C was derived.
and Turgeman (1979), using an upper bound calculation, obtained \( N = 5.298 \) for \( D/\rho_0 = 1 \). Pastor (1979) also presented a lower bound solution of \( N = 3.464 \) for \( D/\rho_0 = 1 \). Britto and Kusakabe (1983) who also used an upper bound calculation, considers a number of wall and base failure mechanisms. The most critical failure mechanism among them was the wall failure mechanism of curve D for \( D/\rho_0 > 2 \) and of curve E for \( D/\rho_0 < 2 \). The wall failure mechanisms used for curve D is well supported by the experimental work of Brito, Kusakabe and Schofield (1981).

Meyerhof (1972) analyzed the stability of slurry supported axisymmetric, rectangular, and square excavation using Rankine’s pressure theory. For the axisymmetric excavations assigning zero value for the slurry density and expression for the critical depth of unsupported excavation is derived

\[
D_0 = 4 \frac{C_u}{\gamma} \left[ \ln \left( \frac{D}{\rho_0} - 1 \right) + 1 \right]
\]

And this is shown as curve F in fig 8. Sloan (1981) analyzed the problem using the finite element method and found collapse conditions at \( N = 7.7 \) for \( D/\rho_0 = 4 \). Finite element analysis carried out by Kusakube et al gave \( N = 6.6 \) for \( D/\rho_0 = 7 \) and 9. In these calculations the soil was considered to behave as a Tresca material with a ratio of Young’s modulus, \( E_u \), to the undrained shear strength \( C_u \), of 100 and a Poisson’s ratio of 0.49. These results are close to the curves D and E of the wall failure mechanism, for \( D/\rho_0 < 7 \).

From these results, it can be said that the most critical failure mode of the axisymmetric unsupported excavation is the wall failure mode, and
the best upper bound solution to date is that of curve D for D/ro>2 and of curve E for D/ro<2. However, Skempton’s Nc line gives a lower value for D/ro>7, which implies that it is possible for unsupported deep excavations to experience a large and unacceptable base heave probably due to local failure. But this mode of base failure is only possible if the walls of the excavation are laterally supported. Therefore, the finite element results (for D/ro >=7) confirm that, for unsupported excavations, wall failure is the most critical.

All these results are based on the assumptions that the soil possesses a uniform undrained shear strength profile. When the shear strength increases with depth; base failure is unlikely to occur because failure has to develop in a region of the soil where the shear strength is greater than the soil above it. Therefore it is probably justifiable to say that the wall failure mode is the most critical for the situation where the soil has an undrained shear strength profile with strength increase with depth. Therefore the previous study by Britto and Kusakube was confined to wall failure in the region of 0<D/ro<7.

Upper bound solutions give stability solutions which are known to be on the unsafe side of the true solution, and there is a dearth of information on good lower bound calculations which can bracket the exact solution of the problem considered here. It raises the question of the validity of using this upper bound solution for assessment of the safety of axisymmetric excavations.
2.9 Analytical Solution

The soil is assumed to be saturated to the ground surface. The variation of undrained shear strength is shown in fig. 2.9. The origin of the axes is chosen as the center of the base of the excavation; therefore, $C_u$ is given by

$$C_u (z) = C_{u0} k (D-z)$$

(3)

In which $C_{u0}$= the shear strength at surface; $k$= the rate of increase in undrained shear strength with depth; and $Z$= the depth of the excavation.

The undrained shear strength of normally consolidated clay is directly proportional to the consolidation pressure. Skempton presented the empirical relationship between the plasticity index (PI) and the ratio $C_u/\sigma'$.

$$C_u/\sigma' = 0.11 - 0.0037(PI)$$

(4)

![Fig 2.9 Variation of Undrained](image)
In which $\sigma'$ is the consolidation pressure. In situations where the ground is saturated to the ground surface, $\sigma'$ is directly proportional to the depth. Therefore, Equation (3) is applicable to normally consolidated soils.

The upper bound mechanism and the admissible velocity field is shown in fig 2.10(a) and is the same as the mechanism giving the result shown by curve D in fig 2.8. Region 1 moves vertically downward as a rigid block, region 2 is a shear zone, and region 3 is stationary. The boundaries ac, bc, and dc are lines of discontinuity. The velocity field satisfies the incompressibility condition, which means that the failure takes place under undrained conditions.

In fig 2.10 (a), $r_o$ is the radius of the excavation. The geometric parameter, $\alpha$, $\beta$ and $H$, are also shown. By considering the volume flow across the boundaries, the expressions for the velocity components are derived. Two variations of the previously mentioned mechanism are shown in fig 2.10 (b) and 2.10 (c). The relevance of these two mechanism will be displaying the condition that normal velocity component across a discontinuity must be continuous. Details of these calculations are given by Britto and Kusakabe.

The energy dissipated for a Tresca material is given by Shield and Drucker (1953)

$$E = 2\int_{\text{vol}} C_u |\varepsilon_{\max}| \; dV + \int_{\text{s}} C_u |\Delta V| \; dS$$

In which $C_u =$ undrained shear strength; $|\varepsilon_{\max}| =$ max value of principle strain rate component; $\text{VOL} =$ volume domain; $\Delta V =$ velocity
jump across discontinuity; and $S$ = surface area of discontinuity. The first term is the energy dissipated in the shearing zone and the second dissipated along discontinuity. (Fig 2.10)

The total work done

$$E = E_{bcz} + E_{bc} + E_{ac} + E_{dc}$$  \hspace{1cm} (6)

And the total work done

$$\omega = \omega_{abc} + \omega_{bcd}$$ \hspace{1cm} (7)\hspace{1cm}

by equating the total energy dissipated and the total work done, an expression for the stability number was derived.

Hunter and Schuster(1968), who analyzed the slope stability problem under the same shear strength condition, introduced a parameter $M$, which is defined by $M = \frac{C_{uo}}{kD}$, and gave the expression for the safety factor, $F_s$, as $F_s = KNs/Y$ in which $Ns$ = their stability no. which is a function of $M$; and $K$ = the rate of increase in shear strength with depth.

$$Ns = \frac{\gamma D}{(C_{uo} + KD)}$$ \hspace{1cm} (8)

$$M = \frac{\text{Undrained shear strength at the ground surface}}{\text{Undrained shear strength at the level of the excavation base}}$$ \hspace{1cm} (9)

And the critical path, $D_{cr}$, is given by $NsC_{uo}/\gamma M$. The reason for defining $Ns$ and $M$ in this manner is that Hunter and Schuster’s definition $M$ does not include the case of uniform shear strength, in a satisfactory fashion (where $k = 0$, $M = \infty$). For a soil with uniform undrained shear strength, $Ns$ reduce to $\gamma D/Cu$, and $M$ becomes unity.
Fig. 2.10 Upper Bound Mechanisms For wall Failure
When the shear strength at the surface is zero, as for the slope stability problem analyzed by Gibson and Morgenstern, $N_s$ reduces to $\gamma/k$ and $M$ becomes zero. Therefore, an expression of $N_s$ covering the case when the shear strength linearly increases with depth as well as the uniform shear strength case was established by Britto and Kusakabe. According to them, for axisymmetric unsupported excavations in soils having uniform shear strength wall collapse mode is most suitable for a depth to radius ratio of 0 to 7. Also they have presented stability charts for the case of soil whose undrained shear strength increases linearly. Also they have established plastic zone, which thought to be an indication of the influence zone was approximately 0.4 times the depth of excavation. When the excavation is laterally supported the base failure by base heave will become more significant.

2.10 Axisymmetric Excavations of Large Diameters.

The unsupported vertical circular openings of large diameters are shafts, open wells for agricultural purpose, access and ventilation chambers of tunnels etc. The distribution of stress is two dimensional. These types of openings required to be supported during the phase of excavation also. The stability of such openings may be treated as trench openings and two dimensional stability analyses may be adopted.
2.11 Summary and Conclusion

From the previous works it is evident that the stability analysis mostly concerned for saturated clays. The base failure is assumed for most of the analysis. For few cases a wall failure mechanism is assumed. As the depth increases, by their findings the width of plastic zone also more and this may not extend laterally beyond a certain width. The failure mechanism is thought to be a function of shear parameters and moisture content. Apart from this, in the urban environment, there are situations where the ground surface near the openings may be loaded which may influence the mode of failure. Also if it fails, what would be the limit to which it can be loaded? Keeping these in view the scope is drawn.

2.12 Scope of Investigation

From the Literature review to the best knowledge of the present investigator, it is evident that the stability analysis of vertical circular unsupported openings has not been attempted for general soils. This analysis would be helpful in the field for safety check of unsupported circular vertical openings before and after excavation. Hence the scope of the investigation is to find most suitable failure mechanism, to estimate the width of plastic zone beyond which the soil on the surface can be loaded with out risking stability of the vertical circular opening (safe width), and to estimate the distance beyond which an excavation can be attempted without causing any disturbance to the existing structure.
2.13 Objectives of the Investigation

The objectives of the investigation of stability of vertical unsupported circular openings, proposed to be investigated are summarized below.

1. To identify the most suitable failure mechanism for unsupported vertical circular openings for general soil.

2. To study the effect of diameter of unsupported vertical circular opening on stability of general soil.

3. To study the influence of factors, such as physical properties of soil on stability of unsupported vertical circular openings.

4. To study the effect of area of loading on stability of unsupported vertical circular openings.

5. To study the influence of position of loading on stability of vertical unsupported circular openings (Distance from axis of opening).